Appendix F

Geotechnical Reports

Appendix F-1

Geotechnical Investigation

UPDATED GEOTECHNICAL INVESTIGATION

PROPOSED HOTEL AND CONDOMINIUM DEVELOPMENT
8850 SUNSET BOULEVARD
WEST HOLLYWOOD, CALIFORNIA
APNS 4339-017- 001 THROUGH -008

GEOCON
WEST, INC.

GEOTECHNICAL
ENVIRONMENTAL
MATERIALS

PREPARED FOR

8850 SUNSET BLVD, LLC
C/O PLUS DEVELOPMENT GROUP
WEST HOLLYWOOD, CALIFORNIA

PROJECT NO. A9899-06-01

FEBRUARY 17, 2021

GEOCON
WEST, INC.
GEOTECHNICAL ■ ENVIRONMENTAL ■ MATERIALS

Project No. A9899-06-01
February 17, 2021

8850 Sunset Blvd. LLC
c/o Mike Unwin
Plus Development Group
8920 West Sunset Boulevard, #200A
West Hollywood, California 90069

Subject: UPDATED GEOTECHNICAL INVESTIGATION
PROPOSED HOTEL AND CONDOMINIUM DEVELOPMENT
8850 SUNSET BOULEVARD, WEST HOLLYWOOD, CALIFORNIA
APNS 4339-017- 001 THROUGH -008

Dear Mr. Unwin:

In accordance with your authorization of our proposal dated December 9, 2020, we have prepared an updated geotechnical investigation for the proposed hotel and condominium development located at 8850 Sunset Boulevard in the City of West Hollywood, California. The accompanying report presents the findings of our study, and our conclusions and recommendations pertaining to the geotechnical aspects of proposed design and construction. Based on the results of our investigation, it is our opinion that the site can be developed as proposed, provided the recommendations of this report are followed and implemented during design and construction.

If you have any questions regarding this report, or if we may be of further service, please contact the undersigned.

Very truly yours,

GEOCON WEST, INC.

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TABLE OF CONTENTS

TABLE OF CONTENTS (Continued)

UPDATED GEOTECHNICAL INVESTIGATION

1. PURPOSE AND SCOPE

This report presents the results of an updated geotechnical investigation for the proposed hotel and condominium development located at 8850 Sunset Boulevard in the City of West Hollywood, California (see Vicinity Map, Figure 1). The purpose of the investigation was to evaluate subsurface soil and geologic conditions underlying the site and, based on conditions encountered, to provide conclusions and recommendations pertaining to the geotechnical aspects of design and construction.

The site is located within a state-designated Alquist-Priolo Earthquake Fault Zone (APEFZ) for surface fault rupture hazards associated with the Hollywood Fault Zone. The site is also located within a City of West Hollywood Fault Precaution Zone FP-1 and a site-specific fault rupture hazard investigation is required for proposed structures. A site-specific fault rupture hazard investigation was performed for the proposed development by John Helms, CEG (Helms, 2018). The results of the fault investigation indicate that the potential for active faults to impact the proposed structures is low. The report was reviewed and approved by the City of West Hollywood. Detailed results of the fault investigation are presented in the following report:

> *Fault Rupture Hazard Investigation, 8850 to 8878 Sunset Boulevard, West Hollywood, California*, by John Helms, CEG, Prepared for Silver Creek Commercial Development, LLC, dated September 19, 2018.

The scope of this investigation included a site reconnaissance, field exploration, laboratory testing, engineering analysis, and the preparation of this report. The site was explored on November 20 and November 21, 2018 by excavating three 8-inch-diameter borings using a truck-mounted hollow-stem auger drilling machine. The borings were excavated to depths of approximately 60½ and 80½ feet below the existing ground surface. The approximate locations of the exploratory borings are depicted on the Topographic Plan and Site Plan (see Figures 2A and 2B). A detailed discussion of the field investigation, including boring logs, is presented in Appendix A.

Laboratory tests were performed on selected soil samples obtained during the investigation to determine pertinent physical and chemical soil properties. Appendix B presents a summary of the laboratory test results.

The recommendations presented herein are based on analysis of the data obtained during the investigation and our experience with similar soil and geologic conditions. References reviewed to prepare this report are provided in the *List of References* section.

If project details vary significantly from those described herein, Geocon should be contacted to determine the necessity for review and possible revision of this report.

2. SITE AND PROJECT DESCRIPTION

The subject site is located at 8850 Sunset Boulevard in the City of West Hollywood, California. The site is bounded by Sunset Boulevard to the north, by Larrabee Street to the east, by San Vicente Boulevard to the west, and by the London Hotel to the south. The site is a rectangular-shaped parcel and is currently occupied by commercial structures and a paved surface parking lot. Topographic relief across the site is approximately 19 feet in the north-south direction. Surface water drainage at the site appears to be by sheet flow along the existing ground contours to the city streets. Vegetation onsite consists of shrubs and trees located in isolated planter areas.

Based on the information provided by the Client, it is our understanding that the proposed development will consist of up to a 14-story hotel tower and a 15-story residential tower underlain by five levels of subterranean parking. Based on the sloped nature of the site, it is our understanding that the proposed first floor and all parking levels will be entirely below grade along Sunset Boulevard, with up to four levels of subterranean parking below grade at the south property line. The depth to the lowest FFE along Sunset Boulevard is anticipated to be approximately 68 feet below existing grade. The depth to the lowest FFE along the south property line is anticipated to be approximately 40 feet below existing grade. Including an assumed foundation thickness, the maximum depth of excavation is anticipated to be 74 feet from existing grade. The existing site conditions are depicted on the Topographic Plan (see Figure 2A) and the proposed structure is shown on the Site Plan and Cross-Sections (see Figures 2B through 3B).

Based on the preliminary nature of the design at this time, wall and column loads were not available. It is anticipated that column loads for the proposed structure will be up to 1,800 kips, and wall loads will be up to 18 kips per linear foot. If a mat foundation is used to support the tower core, it is anticipated that a bearing pressure on the order of 6,500 pounds per square foot (psf) may be required.

Once the design phase and foundation loading configuration proceeds to a more finalized plan, the recommendations within this report should be reviewed and revised, if necessary. Any changes in the design, location or elevation of any structure, as outlined in this report, should be reviewed by this office. Geocon should be contacted to determine the necessity for review and possible revision of this report.

3. GEOLOGIC SETTING

The property is located within the northern portion of the Los Angeles Basin on a southerly sloping alluvial fan at the base of the southern flank of the Santa Monica Mountains. The Los Angeles Basin is a coastal plain between the Santa Monica Mountains to the north, the Puente Hills and Whittier Fault to the east, the Palos Verdes Peninsula and Pacific Ocean to the west and south, and the Santa Ana Mountains and San Joaquin Hills to the southeast. The basin is underlain by a deep structural depression which has been filled by both marine and continental sedimentary deposits that are underlain by a basement complex of igneous and metamorphic composition.

Regionally, the site is located in the southern portion of the Transverse Ranges geomorphic province, near the boundary of the Peninsular Ranges geomorphic province to the south. The Transverse Ranges is characterized by east-west trending geologic structures in contrast to the Peninsular Ranges that is characterized by northwest-trending geologic structures. The boundary between the two geomorphic provinces is a system of faults that include the Malibu Coast, Santa Monica, Hollywood, Raymond, and Sierra Madre Fault zones. The State of California Earthquake Fault Zone Maps (CGS, 2018b) and City documents (City of West Hollywood, 2011; City of West Hollywood, 2010) indicate that traces of the Hollywood Fault may be located approximately 100 feet to 200 feet south of the property.

4. SOIL AND GEOLOGIC CONDITIONS

Based on our field investigation, the site-specific fault investigation report by Helms (2018), and published geologic maps of the area (California Geological Survey, 2012; City of West Hollywood, 2011), the geologic materials at the site consist of artificial fill that is underlain by a thin veneer of Holocene age alluvium that is in turn underlain by a thick section of Pleistocene age alluvial fan deposits consisting of sand, silt, clay and gravel (Helms, 2018). The geologic conditions at the site and in the immediate area are shown on Figure 4, Geology Map and discussed in detail by Helms (2018). Detailed stratigraphic profiles of the geologic materials encountered at the site are provided on the boring logs in Appendix A. The City of West Hollywood Geotechnical Report Guide Checklist Items 13 and 15 are addressed in detail in the site-specific fault rupture hazard report for the proposed development (Helms, 2018).

4.1 Artificial Fill

Artificial fill was encountered in our field explorations to a maximum depth of 3 feet below existing ground surface. The artificial fill generally consists of yellowish brown to brown sand with silt and well-graded sand with various amounts of gravel. The artificial fill is characterized as dry to slightly moist and loose. The fill is likely the result of past grading or construction activities at the site. Deeper fill may exist between excavations and in other portions of the site that were not directly explored.

4.2 Alluvium

Holocene age alluvial fan deposits were encountered beneath the fill. The Holocene age alluvium is interpreted to be approximately 6 to 9 feet thick and underlain by Pleistocene age alluvial fan deposits (Helms, 2018). The alluvium generally consists of brown to reddish brown silty sand, clayey sand, poorly graded sand, sand with silt, sandy silt, silt, and sandy clay with various amounts of clay and gravel. The alluvial soils are primarily fine- to medium-grained, dry to wet and very loose to very dense or firm to hard.

5. GROUNDWATER

Based on a review of the Seismic Hazard Evaluation Report for the Beverly Hills 7.5 Minute Quadrangle (California Division of Mines and Geology [CDMG], 1998) and the City of West Hollywood General Plan (2011), the historic high groundwater level beneath the site is approximately 20 feet below the existing ground surface. Groundwater information presented in the CDMG publication is based on data collected from the early 1900's to the late 1990's.

Groundwater was encountered in borings B1, B2 and B3 at depths of approximately 40, 40½, and 42 feet below the existing ground surface, respectively. The geotechnical borings were not left open a sufficient length of time to measure true static groundwater levels. However, borings and cone penetration tests performed as part of the site-specific fault rupture hazard investigation documented static groundwater levels ranging from approximately 19 feet to 26 feet (Helms, 2018). Considering the historic high groundwater level, the depth to groundwater encountered in our borings, and the static groundwater levels encountered during the fault investigation (Helms, 2018), groundwater will likely be encountered during construction. Also, it is not uncommon for groundwater levels to vary seasonally or for groundwater seepage conditions to develop where none previously existed, especially in impermeable fine-grained soils which are heavily irrigated or after seasonal rainfall. In addition, recent requirements for stormwater infiltration could result in shallower seepage conditions in the immediate site vicinity. Proper surface drainage of irrigation and precipitation will be critical for future performance of the project. Recommendations for drainage are provided in the S*urface Drainage* section of this report (see Section 7.24).

6. GEOLOGIC HAZARDS

6.1 Surface Fault Rupture

The numerous faults in Southern California include Holocene-active, pre-Holocene, and inactive faults. The criteria for these major groups are based on criteria developed by the California Geological Survey (CGS, formerly known as CDMG) for the Alquist-Priolo Earthquake Fault Zone Program (CGS, 2018a). By definition, a Holocene-active fault is one that has had surface displacement within Holocene time (about the last 11,700 years). A pre-Holocene fault has demonstrated surface displacement during Quaternary time (approximately the last 1.6 million years) but has had no known Holocene movement. Faults that have not moved in the last 1.6 million years are considered inactive.

The site is located within a state-designated Alquist-Priolo Earthquake Fault Zone (CGS, 2018b; CGS, 2021a; CGS, 2021b). The site is also located within a city-designated Fault Precaution Zone FP-1 (City of West Hollywood, 2019) for surface fault rupture hazards. The location of the site relative to the State of California and the City of West Hollywood seismic hazard zones are shown of Figure 5A, City Seismic Hazard Zone Map and Figure 5B, City Fault Location and Precaution Zone Map.

Similar to the State of California's Alquist-Priolo Earthquake Fault Zones, the City of West Hollywood's Fault Precautionary Zones are specifically designed to mitigate the hazard of surface fault rupture and requires that a site-specific fault rupture hazard investigation be performed prior to construction of human-occupied structures to rule out the potential for surface fault rupture hazards to impact the proposed structure. As previously indicated, a site-specific fault rupture hazard investigation was performed for the proposed development by John Helms, CEG (Helms, 2018). Based on the results of the site-specific investigation Holocene-active or pre-Holocene faults do not traverse the site or are located within 50 feet of the southern site boundary. Therefore, the potential for surface rupture due to faulting occurring beneath the site during the design life of the proposed structures is considered low. However, the site is located in the seismically active Southern California region and could be subjected to moderate to strong ground shaking in the event of an earthquake on one of the many active Southern California faults. The faults in the vicinity of the site are shown in Figure 6, Regional Fault Map.

The closest surface trace of an active fault to the site is the Hollywood Fault located approximately 100 to 200 feet to the south (Helms, 2018). Other nearby active faults include the Santa Monica Fault, the Newport-Inglewood Fault Zone, the Raymond Fault, and the Verdugo Fault located approximately 1.4 miles southwest, 2.9 miles south, 9.2 miles northeast, and 9.8 miles northeast of the site, respectively (USGS, 2006; Ziony and Jones, 1989). The active San Andreas Fault Zone is located approximately 35 miles northeast of the site (USGS, 2006).

Several buried thrust faults, commonly referred to as blind thrusts, underlie the Los Angeles Basin at depth. These faults are not exposed at the ground surface and are typically identified at depths greater than 3.0 kilometers. The October 1, 1987, M_w 5.9 Whittier Narrows earthquake and the January 17, 1994, M_w 6.7 Northridge earthquake were a result of movement on the Puente Hills Blind Thrust and the Northridge Thrust, respectively. These thrust faults and others in the greater Los Angeles area are not exposed at the surface and do not present a potential surface fault rupture hazard at the site; however, these deep thrust faults are considered active features capable of generating future earthquakes that could result in moderate to significant ground shaking at the site.

6.2 Seismicity

As with all of Southern California, the site has experienced historic earthquakes from various regional faults. The seismicity of the region surrounding the site was formulated based on research of an electronic database of earthquake data. The epicenters of recorded earthquakes with magnitudes equal to or greater than 5.0 in the site vicinity are depicted on Figure 7, Regional Seismicity Map. A partial list of moderate to major magnitude earthquakes that have occurred in the Southern California area within the last 100 years is included in the following table.

LIST OF HISTORIC EARTHQUAKES

Earthquake (Oldest to Youngest)	Date of Earthquake	Magnitude	Distance to Epicenter (Miles)	Direction to Epicenter
Near Redlands	July 23, 1923	6.3	65	E
Long Beach	March 10, 1933	6.4	40	SE
Tehachapi	July 21, 1952	7.5	72	NW
San Fernando	February 9, 1971	6.6	22	N
Whittier Narrows	October 1, 1987	5.9	18	E
Sierra Madre	June 28, 1991	5.8	25	ENE
Landers	June 28, 1992	7.3	111	E
Big Bear	June 28, 1992	6.4	89	E
Northridge	January 17, 1994	6.7	12	NW
Hector Mine	October 16, 1999	7.1	126	ENE
Ridgecrest	July 5, 2019	7.1	124	NNE

The site could be subjected to strong ground shaking in the event of an earthquake, particularly an earthquake originating along the nearby Hollywood Fault Zone. This hazard is common in Southern California and the effects of ground shaking can be mitigated if the proposed structures are designed and constructed in conformance with current building codes and engineering practices. The proposed structure should be designed for the potential strong ground motions that may result from future earthquakes along the Hollywood Fault Zone.

6.3 Site-Specific Ground Motion Hazard Analysis

A site-specific ground motion hazard analyses was performed in accordance with ASCE 7-16 Chapter 21 and Section 1613 of the 2019 CBC using the online applications developed by USGS.

6.3.1 Site-Specific Shear Wave Velocity

On January 4, 2021, GeoVision, Inc. performed a multi-channel analysis of surface waves (MASW) survey at the site. MASW surveys are accepted as an accurate method of determining the soil shear wave velocity. The methodologies used by GeoVision for the data acquisition and analysis are presented in their report dated January 21, 2021 report. A copy of the report is provided as Appendix C.

Based on the results of the MASW survey, the site-specific soil shear wave velocity for the soils underlying the proposed subterranean levels, 70 to 170 feet, ($V_{s70\text{-}170ft}$) is estimated as 1,514 feet/second. In accordance with Section 1613.3.2 of the 2019 California Building Code and Table 20.3-1 of ASCE 7-16, the estimated soil shear wave velocity falls within the boundaries of a Site Class "C".

6.3.2 Probabilistic Seismic Hazard Analysis

The risk-targeted Maximum Considered Earthquake (MCE_R) probabilistic response spectrum consists of the spectral response accelerations which are expected to achieve a 1 percent probability of collapse within a 50-year period, evaluated at 5 percent damping.

The mean spectral response accelerations having a 2 percent chance of exceedance in 50 years were evaluated at 5 percent damping using the USGS Unified Hazard Tool (UHT). The Dynamic U.S. 2014 (v4.2.0) edition was used within the analysis, which is based on the UCERF-3 fault model. The soil underlying the site was modeled as a Site Class "C" with a corresponding average shear wave velocity (V_S30) of 560 meters per second.

The web application uses the ground motion prediction equations (GMPEs) from the NGA-West 2 project: Abrahamson-et al. (2014) NGA West 2, Boore et al. (2014) NGA West 2, Campbell-Bozorgnia (2014) NGA West 2, and Chiou-Youngs (2014) NGA West 2. Each GMPE was assigned an equal weight and the mean value of the four GMPEs was evaluated. The mean spectral accelerations were rotated to maximum direction using the period specific ratios from Shahi et al. (2013 & 2014).

The GMPE of Campbell and Borzorgnia requires that the depth to where the shear wave velocity reaches 2.5 kilometers per second (Z2.5) be defined. Additionally, the GMPEs of Abrahamson-et al., Boore et al. and Chiou-Youngs require that the depth to where the shear wave velocity reaches 1 kilometer per second (Z1.0) be defined. The values of Z2.5 and Z1.0 are internally calculated by the Uniform Hazard Tool.

The MCE uniform hazard response spectra was adjusted to risk-targeted spectral accelerations corresponding to a 1 percent chance of collapse in 50 years by using the USGS Risk-Targeted Ground Motion Calculator and following ASCE 7-16 Section 21.2.1.2 Method 2.

The risk-targeted Maximum Considered Earthquake (MCE_R) probabilistic response spectrum is provided on Figure 6.

6.3.3 Deterministic Seismic Hazard Analysis

In order to define the deterministic scenario events, deaggregation of the uniform hazard probabilistic response spectrum was performed using the USGS Uniform Hazard Tool. The inversion approach used by UCERF-3 allows for a large number of variations for each source scenario, including multi-fault ruptures. Therefore, deaggregation of UCERF-3 consists of the contributions from multi-fault ruptures rather than individual source contributions. To address this, the USGS Unified Hazard Tool aggregates the contributions on a per-fault-section basis, with rupture contributions only ever counted once. The Unified Hazard Tool deaggregation contributor list shows the fault sections which contribute most to hazard at a site and report a mean earthquake magnitude for each section identified by a 'parent' fault name and section index. Based on the deaggregation, we have considered scenario events with the greatest contribution to the deterministic ground motions.

The earthquake magnitude of the deterministic scenario events were based on the BSSC 2014 Scenario Event which includes the parent fault identified in the deaggregation and which has the largest earthquake magnitude. The closest distance (Rrup) from the fault to the site was taken from the Uniform Hazard Tool deaggregation results. Other fault source parameters were defined by the values in the BSSC2014 Scenario Catalog. The values of Z2.5 and Z1.0 were estimated using data from the Community Velocity Model (CVM) Version 4 developed by Southern California Earthquake Data Center (SCEDC) accessed by the OpenSHA Site Data Application (v1.4.0).

Two deterministic scenario events were considered for this analysis and consisted of a magnitude 6.7 event occurring on the Hollywood fault at distance (Rx) of 0.03 km and a magnitude 6.8 event occurring on the Santa Monica alt 2 fault at distance (Rx) of 2.25 km.

The deterministic median and standard deviation (sigma) for the scenario events were evaluated using the USGS NSHMP-HAZ-WS Response Spectra online application. The deterministic analysis used the same four GMPEs, equally weighted, to generate the median and standard deviation of the ground motion which were then used to calculate the 84th percentile at 5% damping. The geometric median spectral accelerations were rotated to maximum direction using the period specific ratios from Shahi et al. (2013 & 2014).

The deterministic scenarios were compared, and the event occurring on the Santa Monica alt 2 fault is considered the controlling deterministic event.

The 84th percentile maximum rotated component deterministic response spectra is provided on Figure 7.

6.3.4 Site-Specific Response Spectrum

The lesser of the probabilistic and deterministic MCE_R response spectrums is the Site-Specific MCE_R. Two thirds of the Site-Specific MCE_R is the Design Earthquake (DE) Response Spectrum, provided the results are not less than 80 percent of the modified General Design Response Spectrum determined by ASCE 7-16 Section 11.4.6 with Fa and Fv determined as specified in Section 21.3.

Graphical representations of the analyses are presented on Figures 6 and 7. The Site-Specific Design Earthquake response spectrum at 5 percent damping is presented on Figure 7 and in tabular form on Figure 8.

6.3.5 Mapped Acceleration Parameters

The following table summarizes the mapped acceleration parameters obtained from the 2019 California Building Code (CBC; Based on the 2018 International Building Code [IBC] and ASCE 7-16), Chapter 16 Structural Design, Section 1613A Earthquake Loads. The data was calculated using the online application *Seismic Design Maps*, provided by OSHPD. The short spectral response uses a period of 0.2 second.

MAPPED SPECTRAL ACCELERATIONS

Parameter	Value	2019 CBC Reference
MCE_R Ground Motion Spectral Response Acceleration – Class B (short), S_S	2.122g	Figure 1613.3.1(1)
MCE_R Ground Motion Spectral Response Acceleration – Class B (1 sec), S_1	0.76g	Figure 1613.3.1(2)

6.3.6 Site-Specific Seismic Design Criteria

Based the site-specific ground motion hazard analysis performed, and in accordance with the ASCE 7-16 Section 21.4, site-specific design acceleration parameters shall be derived using the results of the site-specific ground motion hazard analysis.

The parameter S_{DS} shall be taken as equal to 90 percent of the maximum spectral acceleration obtained from the site-specific analysis at any period within the range from 0.2 to 5 seconds, inclusive. The parameter S_{D1} shall be taken as the maximum value of the product of the spectral acceleration and period for periods from 1 to 2 seconds, inclusive. The values of S_{MS} and S_{M1} shall be taken as 1.5 times the site-specific values of S_{DS} and S_{D1}. The site-specific design acceleration parameters shall not be less than 80 percent of the general seismic design values determined by ASCE 7-16 Section 11.4.

The following table presents the site-specific seismic design parameters based on the site-specific ground motion hazard analysis.

SITE-SPECIFIC DESIGN ACCELERATION PARAMETERS

Parameter	Value
Site Class Modified MCE_R Spectral Response Acceleration (short), S_{MS}	2.5g
Site Class Modified MCE_R Spectral Response Acceleration – (1 sec), S_{M1}	1.076g
5% Damped Design Spectral Response Acceleration (short), S_{DS}	1.666g
5% Damped Design Spectral Response Acceleration (1 sec), S_{D1}	0.717g

6.3.7 Site-Specific Peak Ground Acceleration

The site-specific Maximum Considered Earthquake (MCE_G) geometric mean peak ground acceleration was evaluated in accordance with ASCE 7-16 Section 21.5.

The probabilistic geometric mean peak ground acceleration and the deterministic 84th percentile geometric mean peak ground acceleration were analyzed using the same approaches as described above. The analysis used the same Site Class and scenario earthquake.

The deterministic MCE_G shall not be less than $0.5F_{PGA}$, where F_{PGA} is determined from ASCE 7-16 Table 11.8-1 with the value of PGA taken as 0.5g. The site-specific MCE_G peak ground acceleration is taken as the lesser of the probabilistic and deterministic MCE_G, provided the value is not less than 80 percent of the value of PGA_M as determined by ASCE 7-16 Equation 11.8.1.

ASCE 7-16 SITE-SPECIFIC PEAK GROUND ACCELERATION

Parameter	Value	ASCE 7-16 Reference
Site-Specific MCE_G Peak Ground Acceleration, PGA_M	1.002g	Section 21.5

The Maximum Considered Earthquake Ground Motion (MCE) is the level of ground motion that has a 2 percent chance of exceedance in 50 years, with a statistical return period of 2,475 years. According to the 2019 California Building Code and ASCE 7-16, the MCE is to be utilized for the evaluation of liquefaction, lateral spreading, seismic settlements, and it is our understanding that the intent of the Building code is to maintain “Life Safety” during a MCE event. The Design Earthquake Ground Motion (DE) is the level of ground motion that has a 10 percent chance of exceedance in 50 years, with a statistical return period of 475 years.

Deaggregation of the MCE peak ground acceleration was performed using the USGS online Unified Hazard Tool, 2014 Conterminous U.S. Dynamic edition (v4.2.0). The result of the deaggregation analysis indicates that the predominant earthquake contributing to the MCE peak ground acceleration is characterized as a 6.86 magnitude event occurring at a hypocentral distance of 5.91 kilometers from the site.

Deaggregation was also performed for the Design Earthquake (DE) peak ground acceleration, and the result of the analysis indicates that the predominant earthquake contributing to the DE peak ground acceleration is characterized as a 6.72 magnitude occurring at a hypocentral distance of 9.45 kilometers from the site.

Conformance to the criteria in the above tables for seismic design does not constitute any kind of guarantee or assurance that significant structural damage or ground failure will not occur if a large earthquake occurs. The primary goal of seismic design is to protect life, not to avoid all damage, since such design may be economically prohibitive.

6.4 Liquefaction Potential

Liquefaction is a phenomenon in which loose, saturated, relatively cohesionless soil deposits lose shear strength during strong ground motions. Primary factors controlling liquefaction include intensity and duration of ground motion, gradation characteristics of the subsurface soils, in-situ stress conditions, and the depth to groundwater. Liquefaction is typified by a loss of shear strength in the liquefied layers due to rapid increases in pore water pressure generated by earthquake accelerations.

The current standard of practice, as outlined in the “Recommended Procedures for Implementation of DMG Special Publication 117, Guidelines for Analyzing and Mitigating Liquefaction in California” and “Special Publication 117A, Guidelines for Evaluating and Mitigating Seismic Hazards in California” requires liquefaction analysis to a depth of 50 feet below the lowest portion of the proposed structure. Liquefaction typically occurs in areas where the soils below the water table are composed of poorly consolidated, fine to medium-grained, primarily sandy soil. In addition to the requisite soil conditions, the ground acceleration and duration of the earthquake must also be of a sufficient level to induce liquefaction.

The State of California Seismic Hazard Zone Map for the Beverly Hills Quadrangle (CGS, 2018b) indicates that the southeastern portion of the site is located within an area designated as having a potential for liquefaction. In addition, the City of West Hollywood (City of West Hollywood, 2010 and 2011) indicates that the southeastern portion of the site is located within an area identified as having a potential for liquefaction.

Liquefaction analysis of the soils underlying the site was performed using an updated version of the spreadsheet template LIQ2_30.WQ1 developed by Thomas F. Blake (1996). This program utilizes the 1996 NCEER method of analysis. This semi-empirical method is based on a correlation between values of Standard Penetration Test (SPT) resistance.

Screening criteria presented by Bray and Sancio (2006) was used to evaluate the liquefaction susceptibility of the fine-grained soils encountered in the boring. Based on these screening criteria, fine-grained soils with a plasticity index of greater than 18 or a saturated water content of less than 85 percent of the liquid limit are considered not susceptible to liquefaction. Laboratory test results used for the screening criteria are presented as Figures B7 and B8.

The liquefaction analysis was performed for a Design Earthquake level by using a historic high groundwater table of 20 feet below the ground surface, a magnitude 6.72 earthquake, and a peak horizontal acceleration of 0.668g (⅔PGA_M). The enclosed liquefaction analyses, included herein for boring B2, indicates that the alluvial soils below the proposed subterranean structure could be susceptible to approximately 0.5 inches of settlement during Design Earthquake ground motion (see enclosed calculation sheets, Figures 9 and 10).

It is our understanding that the intent of the Building Code is to maintain “Life Safety” during Maximum Considered Earthquake level events. Therefore, additional analysis was performed to evaluate the potential for liquefaction during a MCE event. The structural engineer should evaluate the proposed structure for the anticipated MCE liquefaction induced settlements and verify that anticipated deformations would not cause the foundation system to lose the ability to support the gravity loads and/or cause collapse of the structure.

The liquefaction analysis was also performed for Maximum Considered Earthquake levels by using a historic high groundwater table of 20 feet below the ground surface, a magnitude 6.86 earthquake, and a peak horizontal acceleration of 1.002g (PGA_M). The enclosed liquefaction analyses, included herein for boring B2, indicates that the alluvial soils below the proposed subterranean structure could be susceptible to approximately 0.5 inches of settlement during Maximum Considered Earthquake ground motion (see enclosed calculation sheets, Figures 11 and 12).

6.5 Slope Stability

Topographic relief across the site is approximately 19 feet and topography at the site and in the site vicinity slopes to the south at a gradient of approximately 7:1 (H:V). A review of the City of West Hollywood General Plan (2011) and the Los Angeles County Safety Element (Leighton, 1990) indicates the site is not located within a designated “hillside area” or an area identified as having a potential for slope instability. Additionally, the site is not within an area identified as having a potential for seismic slope instability (CDMG, 1999; City of West Hollywood, 2011; CGS, 2018b). There are no known landslides near the site, nor is the site in the path of any known or potential landslides. Therefore, the potential for slope stability hazards to adversely affect the proposed development is considered low.

6.6 Earthquake-Induced Flooding

Earthquake-induced flooding is inundation caused by failure of dams or other water-retaining structures due to earthquakes. The City of West Hollywood Safety Element (2011) indicates that the site is not located within a potential inundation area. Therefore, the potential for inundation at the site as a result of an earthquake-induced dam failure is considered low.

6.7 Tsunamis, Seiches, and Flooding

The site is not located within a coastal area. Therefore, tsunamis are not considered a significant hazard at the site.

Seiches are large waves generated in enclosed bodies of water in response to ground shaking. No major water-retaining structures are located immediately up gradient from the project site. Therefore, flooding from a seismic-induced seiche is considered unlikely.

The site is within an area of minimal flooding (Zone X) as defined by the Federal Emergency Management Agency (FEMA, 2021; LACDPW, 2021b). Therefore, flooding is not anticipated to adversely impact the site.

6.8 Oil Fields & Methane Potential

Based on a review of the California Geologic Energy Management Division (CalGEM) Well Finder Website, the site is not located within an oil field and no active or inactive oil or gas wells are located within the immediate vicinity of the site (CalGEM, 2021). Due to the voluntary nature of record reporting by the oil well drilling companies, wells may be improperly located or not shown on the location map and undocumented wells could be encountered during construction. Any wells encountered will need to be properly abandoned in accordance with the current requirements of the CalGEM.

Since the site is not located within the boundaries of a known oil field, the potential for the presence of methane or other volatile gases at the site is considered low. However, should it be determined that a methane study is required for the proposed development it is recommended that a qualified methane consultant be retained to perform the study and provide mitigation measures as necessary.

6.9 Subsidence

Subsidence occurs when a large portion of land is displaced vertically, usually due to the withdrawal of groundwater, oil, or natural gas or as a result of decomposition of natural organic materials. Soils that are particularly subject to subsidence include those with high silt or clay content and/or high organic content. The site is located outside the boundary of the area identified in the City of West Hollywood Safety Element as a former marsh (City of West Hollywood, 2011). In addition, organic materials were not encountered in our borings at the site. Therefore, the potential for subsidence related to decomposition of organic materials at the site is considered low. Also, the potential for subsidence at the site related to fluid or gas withdrawal is considered low. Oil or gas extraction is not currently occurring in the nearby abandoned Sherman Oilfield. Also, only marginal activity is currently ongoing in the Salt Lake Oilfield located approximately 1-mile south of the site. Water injection and flooding operations as part of secondary recovery is believed to have largely mitigated hazards related to fluid or gas withdrawal in the area (City of West Hollywood, 2011).

7. CONCLUSIONS AND RECOMMENDATIONS

7.1 General

7.1.1 It is our opinion that neither soil nor geologic conditions were encountered during this investigation that would preclude the construction of the proposed development provided the recommendations presented herein are followed and implemented during design and construction.

7.1.2 Up to 3 feet of existing artificial fill was encountered during site exploration. The existing fill encountered is believed to be the result of past grading and construction activities at the site. Deeper fill may exist in other areas of the site that were not directly explored. Demolition of the existing structures which occupy the site are anticipated to disturb the upper few feet of soil below the existing subterranean levels. The existing fill and site soils are suitable for re-use as engineered fill provided the recommendations in the *Grading* section of this report are followed (see Section 7.5).

7.1.3 Potentially liquefiable soils were identified in boring B2 between depths of 59 and 62 feet. Soils with a similar description and blow counts were not encountered in borings B1 or B3, indicating that the site is not uniformly underlain by this layer of potentially liquefiable soils. Based on the anticipated depth of excavations and the use of a mat foundation, we anticipate that where the potentially liquefiable soils are present near B2, they will be greater than 10 feet below the bottom of the proposed foundation and therefore do not pose a risk to loss of bearing capacity. The enclosed liquefaction settlement analyses indicate that the site soils could be susceptible to approximately ½ inches of total settlement as a result of the Design Earthquake peak ground acceleration (⅔PGA_M). Differential settlement at the foundation level is anticipated to be less than ¼ inch over a distance of 20 feet. The foundation design recommendations presented herein are intended to mitigate the effects of settlement on proposed improvements.

7.1.4 It is anticipated that the proposed subterranean parking levels will extend to depths of approximately 68 feet below grade along Sunset Boulevard and approximately 40 feet below grade along the south property line, not including foundation depths. Including the foundations, the maximum depth of excavation is anticipated to be 74 feet from existing grade. Excavation of the proposed subterranean levels is anticipated to penetrate through the existing artificial fill and alluvial soils throughout the excavation bottom.

7.1.5 Groundwater has been encountered at depths ranging from 19 to 42 feet below the ground surface, and the historic high groundwater level is at a depth of 20 feet below the ground surface. Considering the sloped nature of the site, the depth to groundwater encountered during site exploration, and the historic high groundwater depth, it is recommended that the proposed structure be designed for a groundwater table corresponding to elevation 339 feet MSL at the northern extent of the property and 315 feet MSL at the southern extent of the property. These elevations are based on a historic high groundwater depth of 20 feet as measured from the highest ground surface elevation at the northern and southern extents of the site.

7.1.6 Excavation for construction of the proposed subterranean levels is anticipated to extend to depths of approximately 74 feet below the ground surface, including foundation excavations. It is anticipated that groundwater will be encountered during construction and the contractor should be prepared for these conditions. Due to the depth of the proposed excavation and the potential for seasonal fluctuation in the groundwater level, temporary dewatering measures will likely be required to mitigate groundwater during excavation and construction. Recommendations for temporary dewatering are discussed in Section 7.2 of this report.

7.1.7 The structure should be designed for hydrostatic pressure based on a groundwater elevation of 339 feet MSL at the northern extent of the property and 315 feet MSL at the southern extent of the property. The hydrostatic design will result in uplift forces on the structure that must be resisted by counterweight or structural design measures. The recommended floor slab uplift pressure to be used in design would be 62.4(H) in units of pounds per square foot, where “H” is the height of the water above the bottom of the foundation in feet. If the proposed structure does not provide sufficient dead load to resist the buoyant forces, then recommendations for uplift resistance will be required. Recommendations for uplift resistance are provided in Section 7.8 of this report.

7.1.8 Based on the depth of proposed construction and the potential for hydrostatic pressures, it is recommended that the proposed structure be supported on a reinforced concrete mat foundation system. A mat foundation system is anticipated to be a very cost-effective foundation system for this project since the pad can remain relatively flat which allows for more efficient construction of waterproofing, saving a significant amount of time and labor. In order to minimize differential settlement between the ramp, ramp walls, and basement level, it is recommended that the ramp and ramp walls for the subterranean parking garage be structurally supported on the mat foundation. Recommendations for the design of a mat foundation system are provided in Section 7.7 of this report.

7.1.9 Once proposed building loads become available and subterranean elevations are established, additional analyses will be required to evaluate the anticipated total and differential settlements between the foundation elements. Updated foundation design recommendations will be provided as necessary in an addendum report.

7.1.10 Excavation for construction of the proposed subterranean levels is anticipated to extend to depths of 74 feet, including foundation excavations. Due to the depth of the excavation and the proximity to the property lines, city streets and adjacent offsite structures and improvements, excavation of the proposed subterranean level will require sloping and/or shoring in order to provide a stable excavation. Where shoring is required it is recommended that a soldier pile shoring system be utilized. In addition, where the proposed excavation will be deeper than and adjacent to an offsite structure, the proposed shoring should be designed to resist the surcharge imposed by the adjacent offsite structure. Recommendations for shoring are provided in Section 7.18 of this report.

7.1.11 Due to the nature of the proposed design and intent for a subterranean level, waterproofing of subterranean walls and slabs is suggested. Particular care should be taken in the design and installation of waterproofing to avoid moisture problems, or actual water seepage into the structure through any normal shrinkage cracks which may develop in the concrete walls, floor slab, foundations and/or construction joints. The design and inspection of the waterproofing is not the responsibility of the geotechnical engineer. A waterproofing consultant should be retained in order to recommend a product or method, which would provide protection to subterranean walls, floor slabs and foundations.

7.1.12 Based on the historic and current groundwater levels, as well as the footprint and depth of proposed construction, infiltration of stormwater is not recommended for this project. It is suggested that stormwater be retained, filtered and discharged in accordance with the requirements of the local governing agency.

7.1.13 This statement is made in accordance with the County of Los Angeles, Section 111. It is the opinion of this office that, provided our recommendations are followed and properly maintained, (1) the proposed grading and proposed structures will be safe for its intended use against hazard from landslide, settlement or slippage and (2) the proposed grading and proposed structures will have no adverse effect on the stability of the site or adjoining properties.

7.1.14 Once the design and foundation loading configuration for the proposed structure proceeds to a more finalized plan, the recommendations within this report should be reviewed and revised, if necessary. If the final foundation loading configurations are greater than the assumed loading conditions, the potential for settlement should be reevaluated by this office.

7.1.15 Any changes in the design, location or elevation, as outlined in this report, should be reviewed by this office. Geocon should be contacted to determine the necessity for review and possible revision of this report.

7.2 Temporary Dewatering

7.2.1 Groundwater was encountered during current and prior site exploration at depths ranging from 19 to 42 feet below existing ground surface. Based on the conditions encountered at the time of exploration, it is anticipated that groundwater will be encountered during construction activities. The depth to groundwater at the time of construction can be further verified during initial dewatering well or shoring pile installation. If groundwater is present above the depth of the proposed foundation excavation bottom, temporary dewatering will be necessary to maintain a safe working environment during excavation and construction activities.

7.2.2 It is recommended that a qualified dewatering consultant be retained to design the dewatering system and determine the design flow rates for dewatering. The dewatering consultant should also provide the minimum depth that the temporary dewatering be effective to, and also the potential effects of temporary dewatering on adjacent structures and the public right of way. Temporary dewatering may consist of perimeter wells with interior well points as well as gravel filled trenches (French drains) placed adjacent to the shoring system and interior of the site. The number and locations of the wells or French drains can be adjusted during excavation activities as necessary to collect and control any encountered seepage. The French drains will then direct the collected seepage to a sump where it will be pumped out of the excavation.

7.2.3 The embedment of perimeter shoring piles should be deepened as necessary to take into account any required excavations necessary to place an adjacent French drain system, or sub-slab drainage system, should it be deemed necessary. It is not anticipated that a perimeter French drain will be more than 24 inches in depth below the proposed excavation bottom. If a French drain is to remain on a permanent basis, it must be lined with filter fabric to prevent soil migration into the gravel.

7.3 Soil and Excavation Characteristics

7.3.1 The in-situ soils can be excavated with moderate effort using conventional excavation equipment. Some caving should be anticipated in unshored excavations, especially where granular and/or saturated soils are encountered.

7.3.2 It is the responsibility of the contractor to ensure that all excavations and trenches are properly shored and maintained in accordance with applicable OSHA rules and regulations to maintain safety and maintain the stability of adjacent existing improvements. The soils underlying the site may be classified as a CalOsha Type B soil.

7.3.3 All onsite excavations must be conducted in such a manner that potential surcharges from existing structures, construction equipment, and vehicle loads are resisted. The surcharge area may be defined by a 1:1 projection down and away from the bottom of an existing foundation or vehicle load. Penetrations below this 1:1 projection will require special excavation measures such as sloping or shoring. Excavation recommendations are provided in the *Temporary Excavations* section of this report (see Section 7.17).

7.3.4 The upper 5 feet of existing site soils encountered during this investigation are considered to have a “very low” expansive potential (EI = 0); and are classified as “non-expansive” based on the 2019 California Building Code (CBC) Section 1803.5.3. The recommendations presented herein assume that foundations and slabs at the ground surface will derive support in these materials. Furthermore, based on depth of the proposed subterranean levels, the proposed structure would not be prone to the effects of expansive soils.

7.4 Minimum Resistivity, pH, and Water-Soluble Sulfate

7.4.1 Potential of Hydrogen (pH) and resistivity testing as well as chloride content testing were performed on representative samples of soil to generally evaluate the corrosion potential to surface utilities. The tests were performed in accordance with California Test Method Nos. 643 and 422 and indicate that the soils are considered “corrosive” with respect to corrosion of buried ferrous metals on site. The results are presented in Appendix B (Figure B12) and should be considered for design of underground structures. Due to the corrosive potential of the soils, it is recommended that corrosion-resistant ABS pipes (or equivalent) be utilized in lieu of cast-iron for subdrains and retaining wall drains beneath the structure.

7.4.2 Laboratory tests were performed on representative samples of the site materials to measure the percentage of water-soluble sulfate content. Results from the laboratory water-soluble sulfate tests are presented in Appendix B (Figure B12) and indicate that the on-site materials possess a sulfate exposure class of “S0” to concrete structures as defined by 2019 CBC Section 1904 and ACI 318-14 Table 19.3.1.1.

7.4.3 Geocon West, Inc. does not practice in the field of corrosion engineering and mitigation. If corrosion sensitive improvements are planned, it is recommended that a corrosion engineer be retained to evaluate corrosion test results and incorporate the necessary precautions to avoid premature corrosion of buried metal pipes and concrete structures in direct contact with the soils.

7.5 Grading

7.5.1 Grading is anticipated to include excavation of site soils for the subterranean level, foundations, and utility trenches, as well as placement of backfill for walls, ramps, and trenches.

7.5.2 A preconstruction conference should be held at the site prior to the beginning of grading operations with the owner, contractor, civil engineer, geotechnical engineer, and building official in attendance. Special soil handling requirements can be discussed at that time.

7.5.3 Earthwork should be observed, and compacted fill tested by representatives of Geocon West, Inc. The existing fill and alluvial soils encountered during exploration are suitable for re-use as an engineered fill, provided any encountered oversize material (greater than 6 inches) and any encountered deleterious debris is removed.

7.5.4 Grading should commence with the removal of all existing vegetation and existing improvements from the area to be graded. Once a clean excavation bottom has been established it must be observed and approved in writing by the Geotechnical Engineer (a representative of Geocon West, Inc. Deleterious debris such as wood and root structures should be exported from the site and should not be mixed with the fill soils. Asphalt and concrete should not be mixed with the fill soils unless approved in writing by the Geotechnical Engineer. All existing underground improvements planned for removal should be completely excavated and the resulting depressions properly backfilled in accordance with the procedures described herein.

7.5.5 Based on the depth of proposed construction and potential hydrostatic pressures, it is recommended that proposed structure be supported on a reinforced concrete mat foundation system deriving support in competent alluvial soils found at and below the proposed excavation bottom. Foundations should be deepened as necessary to extend into satisfactory soils and must be observed and approved by the Geotechnical Engineer (a representative of Geocon), prior to placing steel or concrete.

7.5.6 Due to the potential for high-moisture content soils at the excavation bottom, or if construction is performed during the rainy season and the excavation bottom becomes saturated, stabilization measures may have to be implemented to prevent excessive disturbance the excavation bottom. Should this condition exist, rubber tire equipment should not be allowed in the excavation bottom until it is stabilized or extensive soil disturbance could result. Track mounted equipment should be considered to minimize disturbance to the soils.

7.5.7 Subgrade stabilization would consist of introducing a thin lift of 3 to 6-inch diameter crushed angular rock into the soft excavation bottom. The use of crushed concrete will also be acceptable. The crushed rock should be spread thinly across the excavation bottom and pressed into the soils by track rolling or wheel rolling with heavy equipment. It is very important that voids between the rock fragments are not created so the rock must be thoroughly pressed or blended into the soils. All subgrade soils must be properly compacted and proof-rolled in the presence of the Geotechnical Engineer (a representative of Geocon West, Inc.).

7.5.8 All fill and backfill soils should be placed in horizontal loose layers approximately 6 to 8 inches thick, moisture conditioned to at least optimum moisture content, and properly compacted to a minimum 95 percent of the maximum dry density in accordance with ASTM D 1557 (latest edition).

7.5.9 Although not anticipated for this project, all imported fill shall be observed, tested, and approved by Geocon West, Inc. prior to bringing soil to the site. Rocks larger than 6 inches in diameter shall not be used in the fill. If necessary, import soils used as structural fill should have an expansion index less than 20 and soil corrosivity properties that are equally or less detrimental to that of the existing onsite soils (see Figure B12).

7.5.10 Utility trenches should be properly backfilled in accordance with the requirements of the Green Book (latest edition). The pipe should be bedded with clean sands (Sand Equivalent greater than 30) to a depth of at least 1 foot over the pipe, and the bedding material must be inspected and approved in writing by the Geotechnical Engineer (a representative of Geocon). The use of gravel is not acceptable unless used in conjunction with filter fabric to prevent the gravel from having direct contact with soil. The remainder of the trench backfill may be derived from onsite soil or approved import soil, compacted as necessary, until the required compaction is obtained. The use of minimum 2-sack slurry as backfill is also acceptable. Prior to placing any bedding materials or pipes, the excavation bottom must be observed and approved in writing by the Geotechnical Engineer (a representative of Geocon).

7.5.11 All trench and foundation excavation bottoms must be observed and approved in writing by the Geotechnical Engineer (a representative of Geocon), prior to placing bedding material, fill, steel, gravel or concrete.

7.6 Mat Foundation Design

7.6.1 Based on the depth of proposed construction and potential hydrostatic pressures, it is recommended that proposed structure be supported on a reinforced concrete mat foundation system. A mat foundation system is anticipated to be a very cost-effective foundation system for this project since the pad can remain relatively flat which allows for more efficient construction of waterproofing, saving a significant amount of time and labor.

7.6.2 Foundations should be deepened as necessary to extend into satisfactory soils and must be observed and approved in writing by the Geotechnical Engineer (a representative of Geocon West, Inc.).

7.6.3 It is anticipated that the mat foundation will impart an average pressure of less than 3,000 psf, with locally higher pressures up to 6,500 psf. The use of a maximum allowable bearing pressure of 6,500 psf is feasible and anticipated differential settlements should be further evaluated once the bearing pressure distribution beneath the mat foundation is available. The allowable bearing pressure may be increased by up to one-third for transient loads due to wind or seismic forces.

7.6.4 It is recommended that a modulus of subgrade reaction of 100 pounds per cubic inch (pci) be utilized for the design of the mat foundation bearing in the undisturbed alluvial soils found at the excavation bottom. If the subgrade is stabilized in accordance with the recommendations in the *Grading* section of this report, a modulus of subgrade reaction of 200 pci may be utilized. This value is a unit value for use with a 1-foot square footing. The modulus should be reduced in accordance with the following equation when used with larger foundations:

$$K_R = K\left[\frac{B+1}{2B}\right]^2$$

where: K_R = reduced subgrade modulus
K = unit subgrade modulus
B = foundation width (in feet)

7.6.5 The thickness of and reinforcement for the mat foundation should be designed by the project structural engineer.

7.6.6 If the proposed structure is to be designed for full hydrostatic pressure, the recommended floor slab uplift pressure to be used in design would be 62.4(H) in units of pounds per square foot, where “H” is the height of the water above the bottom of the mat foundation in feet. For design purposes the water table may be assumed at an elevation of 339 feet MSL at the northern extent of the property and at an elevation of 315 feet MSL at the southern extent of the property.

7.6.7 For seismic design purposes, an allowable coefficient of friction of 0.4 may be utilized between the concrete mat and alluvium or engineered fill without a moisture barrier; 0.45 may be utilized between the concrete mat and stabilized subgrade without a moisture barrier; and 0.15 for slabs underlain by a moisture barrier.

7.6.8 No special subgrade presaturation is required prior to placement of concrete. However, the slab and foundation subgrade should be sprinkled as necessary; to maintain a moist condition as would be expected in any concrete placement.

7.6.9 Waterproofing of subterranean walls and slabs is recommended for this project for any portions of the structure that will be constructed below the groundwater table. Particular care should be taken in the design and installation of waterproofing to avoid moisture problems, or actual water seepage into the structure through any normal shrinkage cracks which may develop in the concrete walls, floor slab, foundations and/or construction joints. The design and inspection of the waterproofing is not the responsibility of the geotechnical engineer. A waterproofing consultant should be retained in order to recommend a product or method, which would provide protection to subterranean walls, floor slabs and foundations.

7.6.10 Foundation excavations should be observed and approved in writing by the Geotechnical Engineer (a representative of Geocon West, Inc.), prior to the placement of the methane system, reinforcing steel and concrete to verify that the excavations and exposed soil conditions are consistent with those anticipated. If unanticipated soil conditions are encountered, foundation modifications may be required.

7.6.11 This office should be provided a copy of the final construction plans so that the excavation recommendations presented herein could be properly reviewed and revised if necessary.

7.7 Foundation Settlement

7.7.1 The enclosed liquefaction and seismically-induced settlement analyses indicate that the site soils could be susceptible to approximately 0.5 inches of total settlement as a result of the Design Earthquake peak ground acceleration (⅔PGA_M). The differential settlement at the foundation level is anticipated to be less than ¼ inch over a distance of 20 feet. These settlements are in addition to the static settlements indicated below and must be considered in the structural design.

7.7.2 The maximum anticipated settlement for a reinforced concrete mat foundation designed with an average allowable bearing value of 4,750 psf and deriving support in the recommended bearing materials is expected to be less than 1 inch and occur below the heaviest loaded structural element. A majority of the settlement of the foundation system is expected to occur on initial application of loading; however, minor additional settlements are expected within the first twelve months. Differential settlement between the center and corner of the mat is expected to be less than ¾ inch.

7.7.3 Based on seismic considerations, the proposed structure supported on a reinforced concrete mat foundation should be designed for a combined static and seismically induced total settlement of approximately 1½ inches, and a combined static and seismically induced differential settlement of 1 inch between the center and corner of the mat.

7.7.4 Once the design and foundation loading configurations for the proposed structures proceeds to a more finalized plan, the estimated settlements presented in this report should be reviewed and revised, if necessary. Foundation dimensions and allowable bearing pressures may require adjustment to minimize potential differential settlements. The potential for settlement should be reevaluated by this office.

7.8 Uplift Resistance

7.8.1 Foundation uplift may be resisted by the weight of structure as well as friction along the sides of foundations. If additional uplift resistance is required, the perimeter shoring piles may be utilized provided the toes of the piles are poured with structural concrete and are designed as permanent piles. Recommendations for the design of shoring are provided in Section 7.18.

7.8.2 If the structural design will rely on uplift resistance along the sides of the foundations and waterproofing is present, the waterproofing manufacturer should specify the allowable coefficient of friction based on their product’s material properties.

7.8.3 Uplift resistance may also be generated by additional piles constructed within the interior of the structure. It is recommended that post-grouted friction piles be utilized. The uplift capacity may be determined using a frictional resistance of 250 psf (⅔ the downward capacity).

7.8.4 Post-grouted friction piles should be a minimum of 12 inches in diameter and uniformly spaced at least 3 times the diameter on-center. If so spaced, no reduction of the axial capacity for group effects will be necessary. The allowable uplift capacity may be increased by one-third when considering transient wind or seismic loads.

7.8.5 Pile testing should be considered and performed as required by the building official to verify the uplift resistance prior to finalizing pile lengths or commencement of permanent pile installation.

7.9 Lateral Design

7.9.1 Resistance to lateral loading may be provided by friction acting at the base of foundations, slabs and by passive earth pressure. An allowable coefficient of friction of 0.4 may be used with the dead load forces in the competent alluvial soils or in properly compacted engineered fill.

7.9.2 Passive earth pressure for the sides of foundations poured against undisturbed alluvium may be computed as an equivalent fluid having a density of 130 pounds per cubic foot (pcf) with a maximum earth pressure of 1,300 psf (these values have been adjusted for buoyant forces). For soils above the groundwater table, passive pressure may be computed as 300 pcf with a maximum earth pressure of 3,000 pcf. When combining passive and friction for lateral resistance, the passive component should be reduced by one-third.

7.10 Exterior Concrete Slabs-on-Grade

7.10.1 Exterior slabs, not subject to traffic loads, should be at least 4 inches thick and reinforced with No. 3 steel reinforcing bars placed 18 inches on center in both horizontal directions, positioned near the slab midpoint. Prior to construction of slabs, the upper 12 inches of subgrade should be moistened to at least optimum moisture content and properly compacted to at least 95 percent relative compaction, as determined by ASTM Test Method D 1557 (latest edition). Crack control joints should be spaced at intervals not greater than 12 feet and should be constructed using saw-cuts or other methods as soon as practical following concrete placement. Crack control joints should extend a minimum depth of one-fourth the slab thickness. The project structural engineer should design construction joints as necessary.

7.10.2 The moisture content of the slab subgrade should be maintained and sprinkled as necessary to maintain a moist condition as would be expected in any concrete placement.

7.10.3 The recommendations of this report are intended to reduce the potential for cracking of slabs due to settlement. However, even with the incorporation of the recommendations presented herein, foundations, stucco walls, and slabs-on-grade may exhibit some cracking due to minor soil movement and/or concrete shrinkage. The occurrence of concrete shrinkage cracks is independent of the supporting soil characteristics. Their occurrence may be reduced and/or controlled by limiting the slump of the concrete, proper concrete placement and curing, and by the placement of crack control joints at periodic intervals, in particular, where re-entrant slab corners occur.

7.11 Preliminary Pavement Recommendations

7.11.1 Where new paving is to be placed, it is recommended that all existing fill and soft or unsuitable alluvial materials be excavated and properly compacted for paving support. The client should be aware that excavation and compaction of all existing artificial fill and soft alluvium in the area of new paving is not required; however, paving constructed over existing unsuitable material may experience increased settlement and/or cracking, and may therefore have a shorter design life and increased maintenance costs. As a minimum, the upper 12 inches of paving subgrade should be scarified, moisture conditioned to at least optimum moisture content, and properly compacted to at least 95 percent relative compaction, as determined by ASTM Test Method D 1557 (latest edition).

7.11.2 The following pavement sections are based on an assumed R-Value of 20. Once site grading activities are complete an R-Value should be obtained by laboratory testing to confirm the properties of the soils serving as paving subgrade, prior to placing pavement.

7.11.3 The Traffic Indices listed below are estimates. Geocon does not practice in the field of traffic engineering. The actual Traffic Index for each area should be determined by the project civil engineer. If pavement sections for Traffic Indices other than those listed below are required, Geocon should be contacted to provide additional recommendations. Pavement thicknesses were determined following procedures outlined in the *California Highway Design Manual* (Caltrans). It is anticipated that the majority of traffic will consist of automobile and large truck traffic.

PRELIMINARY PAVEMENT DESIGN SECTIONS

Location	Estimated Traffic Index (TI)	Asphalt Concrete (inches)	Class 2 Aggregate Base (inches)
Automobile Parking And Driveways	4.0	3.0	4.0
Trash Truck & Fire Lanes	7.0	4.0	12.0

7.11.4 Asphalt concrete should conform to Section 203-6 of the "*Standard Specifications for Public Works Construction*" (Green Book). Class 2 aggregate base materials should conform to Section 26-1.02A of the "*Standard Specifications of the State of California, Department of Transportation*" (Caltrans). The use of Crushed Miscellaneous Base in lieu of Class 2 aggregate base is acceptable. Crushed Miscellaneous Base should conform to Section 200-2.4 of the "*Standard Specifications for Public Works Construction*" (Green Book).

7.11.5 Unless specifically designed and evaluated by the project structural engineer, where exterior concrete paving will be utilized for support of vehicles, it is recommended that the concrete be a minimum of 6 inches of concrete reinforced with No. 3 steel reinforcing bars placed 18 inches on center in both horizontal directions. Concrete paving supporting vehicular traffic should be underlain by a minimum of 4 inches of aggregate base and a properly compacted subgrade. The subgrade and base material should be compacted to 95 percent relative compaction as determined by ASTM Test Method D 1557 (latest edition).

7.11.6 The performance of pavements is highly dependent upon providing positive surface drainage away from the edge of pavements. Ponding of water on or adjacent to the pavement will likely result in saturation of the subgrade materials and subsequent cracking, subsidence and pavement distress. If planters are planned adjacent to paving, it is recommended that the perimeter curb be extended at least 12 inches below the bottom of the aggregate base to minimize the introduction of water beneath the paving.

7.12 Retaining Wall Design

7.12.1 The recommendations presented below are generally applicable to the design of rigid concrete or masonry retaining walls having a maximum height of 70 feet. In the event that walls significantly higher than 70 feet are planned, Geocon should be contacted for additional recommendations.

7.12.2 Retaining wall foundations may be designed in accordance with the recommendations provided in the *Mat Foundation Design* section of this report (see Section 7.6).

7.12.3 Retaining walls with a level backfill surface that are not restrained at the top should be designed utilizing a triangular distribution of pressure (active pressure). Restrained walls are those that are not allowed to rotate more than 0.001H (where H equals the height of the retaining portion of the wall in feet) at the top of the wall. Where walls are restrained from movement at the top, walls may be designed utilizing a triangular distribution of pressure (at-rest pressure). The table on the following page presents recommended pressures to be used in retaining wall design, assuming that proper drainage will be maintained.

RETAINING WALL WITH LEVEL BACKFILL SURFACE

HEIGHT OF RETAINING WALL (Feet)	ACTIVE PRESSURE EQUIVALENT FLUID PRESSURE (Pounds Per Cubic Foot)	AT-REST PRESSURE EQUIVALENT FLUID PRESSURE (Pounds Per Cubic Foot)
Up to 25	41	57
26 to 35	44	57
36 to 50	47	57
51 to 70	49	57

7.12.4 The wall pressures provided above assume that the retaining wall will be properly drained preventing the buildup of hydrostatic pressure. If retaining wall drainage is not implemented, the equivalent fluid pressure to be used in design of undrained walls is 90 pcf. The value includes hydrostatic pressures plus buoyant lateral earth pressures.

7.12.5 Additional active pressure should be added for a surcharge condition due to sloping ground, vehicular traffic, or adjacent structures. Recommendations for the incorporation of surcharges are provided in section 7.23 of this report. Once the design becomes more finalized, an addendum letter can be prepared revising recommendations and addressing specific surcharge conditions throughout the project, if necessary.

7.15.8 In addition to the recommended earth pressure, the upper 10 feet of the subterranean wall adjacent to the street and parking lot should be designed to resist a uniform lateral pressure of 100 psf, acting as a result of an assumed 300 psf surcharge behind the walls due to normal street traffic. If the traffic is kept back at least 10 feet from the subterranean walls or a distance from the subterranean walls equal to at least half the wall height, whichever is greater, the traffic surcharge may be neglected.

7.12.6 Seismic lateral forces should be incorporated into the design as necessary, and recommendations for seismic lateral forces are presented below.

7.13 Dynamic (Seismic) Lateral Forces

7.13.1 The structural engineer should determine the seismic design category for the project in accordance with Section 1613 of the CBC. If the project possesses a seismic design category of D, E, or F, proposed retaining walls in excess of 6 feet in height should be designed with seismic lateral pressure (Section 1803.5.12 of the 2019 CBC).

7.13.2 A seismic load of 21 and 47 pcf should be used for design of displacing and non-displacing walls, respectively, which support more than 6 feet of backfill in accordance with Section 1803.5.12 of the 2019 CBC. The seismic load is applied as an equivalent fluid pressure along the height of the wall and the calculated loads result in a maximum load exerted at the base of the wall and zero at the top of the wall. This seismic load should be applied in addition to the static earth pressure. The earth pressure is based on a free field PGA of $S_{DS}/2.5$ and with a mean dynamic earth pressure coefficient of 0.25 and 0.57 for displacing and non-displacing walls, respectively (Mikola, 2013).

7.14 Retaining Wall Drainage

7.14.1 Retaining walls not designed for hydrostatic pressures should be provided with a drainage system. At the base of the drain system, a subdrain covered with a minimum of 12 inches of gravel should be installed, and a compacted fill blanket or other seal placed at the surface (see Figure 15). The clean bottom and subdrain pipe, behind a retaining wall, should be observed by the Geotechnical Engineer (a representative of Geocon), prior to placement of gravel or compacting backfill.

7.14.2 As an alternative, a plastic drainage composite such as Miradrain or equivalent may be installed in continuous, 4-foot wide columns along the entire back face of the wall, at 8 feet on center. The top of these drainage composite columns should terminate approximately 18 inches below the ground surface, where either hardscape or a minimum of 18 inches of relatively cohesive material should be placed as a cap (see Figure 16). These vertical columns of drainage material would then be connected at the bottom of the wall to a collection panel or a 1-cubic-foot rock pocket drained by a 4-inch subdrain pipe.

7.14.3 Subdrainage pipes at the base of the retaining wall drainage system should outlet to an acceptable location via controlled drainage structures. Drainage should not be allowed to flow uncontrolled over descending slopes.

7.14.4 Moisture affecting below grade walls is one of the most common post-construction complaints. Poorly applied or omitted waterproofing can lead to efflorescence or standing water. Particular care should be taken in the design and installation of waterproofing to avoid moisture problems, or actual water seepage into the structure through any normal shrinkage cracks which may develop in the concrete walls, floor slab, foundations and/or construction joints. The design and inspection of the waterproofing is not the responsibility of the geotechnical engineer. A waterproofing consultant should be retained in order to recommend a product or method, which would provide protection to subterranean walls, floor slabs and foundations.

7.15 Elevator Pit Design

7.15.1 The elevator pit slab and retaining wall should be designed by the project structural engineer. Elevator pit walls may be designed in accordance with the recommendations in the *Mat Foundation Design* and *Retaining Wall Design* sections of this report (see Sections 7.6 and 7.12).

7.15.2 Additional active pressure should be added for a surcharge condition due to sloping ground, vehicular traffic or adjacent foundations and should be designed for each condition as the project progresses.

7.15.3 If retaining wall drainage is to be provided, the drainage system should be designed in accordance with the *Retaining Wall Drainage* section of this report (see Section 7.14).

7.15.4 It is suggested that the exterior walls and slab be waterproofed to prevent excessive moisture inside of the elevator pit. Waterproofing design and installation is not the responsibility of the geotechnical engineer.

7.16 Elevator Piston

7.16.1 If a plunger-type elevator piston is installed for this project, a deep drilled excavation will be required. It is important to verify that the drilled excavation is not situated immediately adjacent to a foundation or shoring pile, or the drilled excavation could compromise the existing foundation or pile support, especially if the drilling is performed subsequent to the foundation or pile construction.

7.16.2 Casing will likely be required in the drilled excavation. The contractor should be prepared to use casing and should have it readily available at the commencement of drilling activities. The contractor should also be prepared to mitigate buoyant forces during installation of the piston casing. Continuous observation of the drilling and installation of the elevator piston by the Geotechnical Engineer (a representative of Geocon West, Inc.) is required.

7.16.3 The annular space between the piston casing and drilled excavation wall should be filled with a minimum of 1½-sack slurry pumped from the bottom up. As an alternative, pea gravel may be utilized. The use of soil to backfill the annular space is not acceptable.

7.17 Temporary Excavations

7.17.1 Excavations on the order of 74 feet in height may be required for excavation and construction of the proposed subterranean levels. The excavations are expected to expose artificial fill and alluvial soils, which may be subject to caving where granular or saturated soils are exposed. Vertical excavations up to 5 feet in height may be attempted where not surcharged by adjacent traffic or structures.

7.17.2 Vertical excavations greater than 5 feet or where surcharged by existing structures will require sloping or shoring measures in order to provide a stable excavation. Where sufficient space is available, temporary unsurcharged embankments could be sloped back at a uniform 1:1 slope gradient or flatter up to a maximum of 10 feet in height. A uniform slope does not have a vertical portion. Where space is limited, shoring measures will be required. *Shoring* data is provided in Section 7.18 of this report.

7.17.3 Where temporary construction slopes are utilized, the top of the slope should be barricaded to prevent vehicles and storage loads at the top of the slope within a horizontal distance equal to the height of the slope. If the temporary slopes 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. Geocon personnel should inspect the soils exposed in the cut slopes during excavation so that modifications of the slopes can be made if variations in the soil conditions occur. All excavations should be stabilized within 30 days of initial excavation.

7.18 Shoring – Soldier Pile Design and Installation

7.18.1 The following information on the design and installation of shoring is preliminary. Review of the final shoring plans and specifications should be made by this office prior to bidding or negotiating with a shoring contractor.

7.18.2 One method of shoring would consist of steel soldier piles, placed in drilled holes and backfilled with concrete. Where maximum excavation heights are less than 12 feet the soldier piles are typically designed as cantilevers. Where excavations exceed 12 feet or are surcharged, soldier piles may require lateral bracing utilizing drilled tie-back anchors or raker braces to maintain an economical steel beam size and prevent excessive deflection. The size of the steel beam, the need for lateral bracing, and the acceptable shoring deflection should be determined by the project shoring engineer.

7.18.3 The design embedment of the shoring pile toes must be maintained during excavation activities. The toes of the perimeter shoring piles should be deepened to take into account any required excavations necessary for foundations, subgrade stabilization, and/or adjacent drainage systems.

7.18.4 The proposed soldier piles may be utilized to provide a component of uplift resistance. If required to provide uplift resistance, the shoring piles must be designed as permanent piles. The uplift capacity may be taken as ⅔ of the downward frictional capacity.

7.18.5 All piles utilized for shoring can also be incorporated into a permanent retaining wall system (shotcrete wall) and should be designed in accordance with the earth pressure provided in the *Retaining Wall Design* section of this report (see Section 7.12).

7.18.6 Drilled cast-in-place soldier piles should be placed no closer than 3 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 soil. For design purposes, an allowable passive value for the soils below the bottom plane of excavation may be assumed to be 130 psf per foot (value has been reduced for buoyant forces). The allowable passive value may be doubled for isolated piles spaced a minimum of three times the pile diameter. To develop the full lateral value, provisions should be implemented to assure firm contact between the soldier piles and the undisturbed soils. Increases in passive pressure may be available at greater depths and Geocon should be contacted to provide an updated value once a preliminary shoring design is available.

7.18.7 Groundwater was encountered during exploration and the contractor should be prepared for groundwater during pile installation. Piles placed below the water level require the use of a tremie to place the concrete into the bottom of the hole. A tremie should consist of a rigid, water-tight tube having a diameter of not less than 6 inches with a hopper at the top. The tube should 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 should 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 should be closed at the start of the work to prevent water entering the tube and should be entirely sealed at all times, except when the concrete is being placed. The tremie tube should be kept full of concrete. The flow should be continuous until the work is completed and the resulting concrete seal should be monolithic and homogeneous. The tip of the tremie tube should always be kept about 5 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.

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

7.18.9 Casing may be required if caving may occur in the saturated soils. 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.

7.18.10 The frictional resistance between the soldier piles and retained soil may be used to resist the vertical component of the load. The coefficient of friction may be taken as 0.4 based on uniform contact between the steel beam and lean-mix concrete and alluvium. 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 400 psf (value has been reduced for buoyant forces). Increases in frictional resistance may be available at greater depths and Geocon should be contacted to provide updated values once a preliminary shoring design is available.

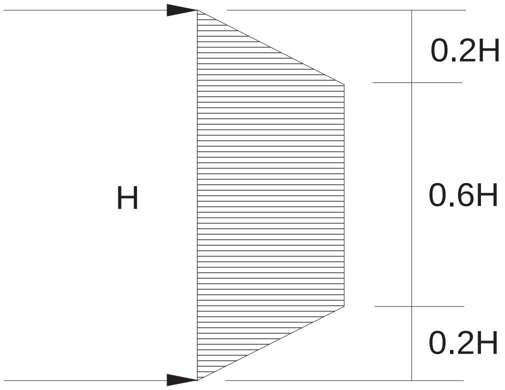
7.18.11 Due to the nature of the site soils, it is expected that continuous lagging between soldier piles will be required. However, it is recommended that the exposed soils be observed by the Geotechnical Engineer (a representative of Geocon West, Inc.), to verify the presence of any cohesive soils and the areas where lagging may be omitted.

7.18.12 The time between lagging excavation and lagging placement should be as short as possible. Soldier piles should be designed for the full-anticipated pressures. Due to arching in the soils, the pressure on the lagging will be less. It is recommended that the lagging be designed for the full design pressure but be limited to a maximum of 400 psf due to earth pressures. Additional active pressure should be added for a surcharge condition due to slopes, vehicular traffic or adjacent structures and should be designed for each condition. The surcharge pressure should be evaluated in accordance with the recommendations in Section 7.23 of this report. The proper strength lagging board should be selected and incorporated into the shoring design by a qualified shoring engineer.

7.18.13 For the design of shoring, it is recommended that an equivalent fluid pressure based on the following table, be utilized for design. A trapezoidal distribution of lateral earth pressure may be used where shoring will be restrained by bracing or tie backs. The recommended active and trapezoidal pressure are provided in the following table. A diagram depicting the trapezoidal pressure distribution of lateral earth pressure is provided below the table.

HEIGHT OF SHORING (FEET)	**EQUIVALENT FLUID PRESSURE (Pounds Per Cubic Foot) (ACTIVE PRESSURE)**	**EQUIVALENT FLUID PRESSURE (Pounds Per Square Foot per Foot) TRAPEZOIDAL (Where H is the height of the shoring in feet)**
Up to 30	34	21H
31 to 35	36	23H
36 to 55	39	25H
56 to 74	41	26H

Trapezoidal Distribution of Pressure



7.18.14 Where a combination of sloped embankment and shoring is utilized, the pressure will be greater and must be determined for each combination. Additional active pressure should be added for a surcharge condition due to slopes, vehicular traffic or adjacent structures and should be designed for each condition. The surcharge pressure should be evaluated in accordance with the recommendations in Section 7.23 of this report.

7.21.22 In addition to the recommended earth pressure, the upper 10 feet of the shoring adjacent to the street or driveway areas should be designed to resist a uniform lateral pressure of 100 psf, acting as a result of an assumed 300 psf surcharge behind the shoring due to normal street traffic. If the traffic is kept back at least 10 feet from the shoring or a distance from the shoring equal to at least half the shoring height, whichever is greater, the traffic surcharge may be neglected.

7.18.15 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 recommended that the deflection be minimized to prevent damage to existing structures and adjacent improvements. Where public right-of-ways are present or adjacent offsite structures do not surcharge the shoring excavation, the shoring deflection should be limited to less than 1 inch at the top of the shored embankment. Where offsite structures are within the shoring surcharge area it is recommended that the beam deflection be limited to less than ½ inch at the elevation of the adjacent offsite foundation, and no deflection at all if deflections will damage existing structures. The allowable deflection is dependent on many factors, such as the presence of structures and utilities near the top of the embankment, and will be assessed and designed by the project shoring engineer.

7.18.16 Because of the depth of the excavation, some means 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.

7.18.17 Due to the depth of the excavation and proximity to adjacent structures, it is suggested that prior to excavation the existing improvements be inspected to document the present condition. For documentation purposes, photographs should be taken of preconstruction distress conditions and level surveys of adjacent grade and pavement should be considered. During excavation activities, the adjacent structures and pavement should be periodically inspected for signs of distress. In the event that distress or settlement is noted, an investigation should be performed and corrective measures taken so that continued or worsened distress or settlement is mitigated. Documentation and monitoring of the offsite structures and improvements is not the responsibility of the geotechnical engineer.

7.19 Temporary Tie-Back Anchors

7.19.1 Temporary tie-back anchors may be used to resist lateral loads. The owner is responsible for obtaining agreements for installation of temporary tie-backs which extend beyond the property lines. Post-grouted 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 and to greater lengths if necessary to develop the desired capacities. The locations and depths of all offsite utilities should be thoroughly checked and incorporated into the drilling angle design for the tie-back anchors.

7.19.2 The capacities of the anchors should be determined by testing of the initial anchors as outlined in a following section. Only the frictional resistance developed beyond the active wedge would be effective in resisting lateral loads. Anchors should be placed at least 6 feet on center to be considered isolated. For preliminary design purposes, it is estimated that drilled friction anchors constructed without utilizing post-grouting techniques will develop average skin frictions as follows:

- 5 feet below the top of the excavation – 1,300 pounds per square foot
- 15 feet below the top of the excavation – 1,000 pounds per square foot*
- 25 feet below the top of the excavation – 1,250 pounds per square foot*
- 35 feet below the top of the excavation – 1,500 pounds per square foot*
- 45 feet below the top of the excavation – 1,750 pounds per square foot*
- 55 feet below the top of the excavation – 2,100 pounds per square foot*
- 65 feet below the top of the excavation – 2,450 pounds per square foot*

* values have been reduced for buoyant forces

7.19.3 Depending on the techniques utilized, and the experience of the contractor performing the installation, a maximum allowable friction capacity of 4.0 kips per linear foot for post-grouted anchors (for a minimum 20-foot length beyond the active wedge) may be assumed for design purposes. Only the frictional resistance developed beyond the active wedge should be utilized in resisting lateral loads. Higher capacity assumptions may be acceptable, but must be verified by testing.

7.20 Anchor Installation

7.20.1 Tied-back anchors are typically installed between 20 and 40 degrees below the horizontal; however, occasionally alternative angles are necessary to avoid existing improvements and utilities. The locations and depths of all offsite utilities should be thoroughly checked prior to design and installation of the tie-back anchors. Caving of the anchor shafts, particularly within sand and gravel deposits or seepage zones, should be anticipated during installation and provisions should be implemented in order to minimize such caving. It is suggested that hollow-stem auger drilling equipment be used to install the anchors. 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.

7.21 Anchor Testing

7.21.1 All of the 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.

7.21.2 At least 10 percent of the anchors should be selected for "quick" 200 percent tests and three additional anchors should be selected for 24-hour 200 percent tests. The purpose of the 200 percent tests is to verify the friction value assumed in design. The anchors should be tested to develop twice the assumed friction value. These tests should be performed prior to installation of additional tiebacks. Where satisfactory tests are not achieved on the initial anchors, the anchor diameter and/or length should be increased until satisfactory test results are obtained.

7.21.3 The total deflection during the 24-hour 200 percent test should not exceed 12 inches. During the 24-hour tests, the anchor deflection should not exceed 0.75 inches measured after the 200 percent test load is applied.

7.21.4 For the "quick" 200 percent tests, the 200 percent test load should be maintained for 30 minutes. The total deflection of the anchor during the 200 percent quick tests should not exceed 12 inches; the deflection after the 200 percent load has been applied should not exceed 0.25 inch during the 30-minute period.

7.21.5 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. A representative of this firm should observe the installation and testing of the anchors.

7.22 Internal Bracing

7.22.1 Rakers may be utilized to brace the soldier piles in lieu of tieback anchors. The raker bracing could be supported laterally by temporary concrete footings (deadmen) or by the permanent, interior footings. For design of such temporary footings or deadmen, poured with the bearing surface normal to rakers inclined at 45 degrees, a bearing value of 2,000 psf in competent alluvial soil, provided the shallowest point of the footing is at least 1 foot below the lowest adjacent grade. The client should be aware that the utilization of rakers could significantly impact the construction schedule due to their intrusion into the construction site and potential interference with equipment. The structural engineer should review the shoring plan to determine if the raker footings conflict with the structural foundation system.

7.23 Surcharge from Adjacent Structures and Improvements

7.23.1 Additional pressure should be added for a surcharge condition due to sloping ground, vehicular traffic or adjacent structures and should be designed for each condition as the project progresses.

7.23.2 It is recommended that line-load surcharges from adjacent wall footings, use horizontal pressures generated from NAV-FAC DM 7.2. The governing equations are:

$$For\ {}^{x}/_{H} \leq 0.4$$

$$\sigma_H(z) = \frac{0.20 \times \left(\frac{z}{H}\right)}{\left[0.16 + \left(\frac{z}{H}\right)^2\right]^2} \times \frac{Q_L}{H}$$

and

$$For\ {}^{x}/_{H} > 0.4$$

$$\sigma_H(z) = \frac{1.28 \times \left(\frac{x}{H}\right)^2 \times \left(\frac{z}{H}\right)}{\left[\left(\frac{x}{H}\right)^2 + \left(\frac{z}{H}\right)^2\right]^2} \times \frac{Q_L}{H}$$

where x is the distance from the face of the excavation or wall to the vertical line-load, H is the distance from the bottom of the footing to the bottom of excavation or wall, z is the depth at which the horizontal pressure is desired, Q_L is the vertical line-load and $\sigma_H(z)$ is the horizontal pressure at depth z.

7.23.3 It is recommended that vertical point-loads, from construction equipment outriggers or adjacent building columns use horizontal pressures generated from NAV-FAC DM 7.2. The governing equations are:

$$For\ {}^{x}/_{H} \leq 0.4$$

$$\sigma_H(z) = \frac{0.28 \times \left(\frac{z}{H}\right)^2}{\left[0.16 + \left(\frac{z}{H}\right)^2\right]^3} \times \frac{Q_P}{H^2}$$

and

$$For\ {}^{x}/_{H} > 0.4$$

$$\sigma_H(z) = \frac{1.77 \times \left(\frac{x}{H}\right)^2 \times \left(\frac{z}{H}\right)^2}{\left[\left(\frac{x}{H}\right)^2 + \left(\frac{z}{H}\right)^2\right]^3} \times \frac{Q_P}{H^2}$$

then

$$\sigma'_H\ (z) =\ \sigma_H(z)cos^2(1.1\theta)$$

where x is the distance from the face of the excavation/wall to the vertical point-load, H is distance from the outrigger/bottom of column footing to the bottom of excavation, z is the depth at which the horizontal pressure is desired, Q_P is the vertical point-load, $\sigma_H(z)$ is the horizontal pressure at depth z, θ is the angle between a line perpendicular to the excavation/wall and a line from the point-load to location on the excavation/wall where the surcharge is being evaluated, and $\sigma_H(z)$ is the horizontal pressure at depth z.

7.24 Surface Drainage

7.24.1 Proper surface drainage is critical to the future performance of the project. Uncontrolled infiltration of irrigation excess and storm runoff into the soils can adversely affect the performance of the planned improvements. Saturation of a soil can cause it to lose internal shear strength and increase its compressibility, resulting in a change in the original designed engineering properties. Proper drainage should be maintained at all times.

7.24.2 All site drainage should be collected and controlled 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. The site should be graded and maintained such that surface drainage is directed away from structures in accordance with 2019 CBC 1804.4 or other applicable standards. In addition, drainage should not be allowed to flow uncontrolled over any descending slope. Discharge from downspouts, roof drains and scuppers are not recommended onto unprotected soils within 5 feet of the building perimeter. Planters which are located adjacent to foundations should be sealed to prevent moisture intrusion into the soils providing foundation support. Landscape irrigation is not recommended within 5 feet of the building perimeter footings except when enclosed in protected planters.

7.24.3 Positive site drainage should be provided away from structures, pavement, and the tops of slopes to swales or other controlled drainage structures. The building pad and pavement areas should be fine graded such that water is not allowed to pond.

7.24.4 Landscaping planters immediately adjacent to paved areas are not recommended due to the potential for surface or irrigation water to infiltrate the pavement's subgrade and base course. Either a subdrain, which collects excess irrigation water and transmits it to drainage structures, or an impervious above-grade planter boxes should be used. In addition, where landscaping is planned adjacent to the pavement, it is recommended that consideration be given to providing a cutoff wall along the edge of the pavement that extends at least 12 inches below the base material.

7.25 Plan Review

7.25.1 Grading, foundation, and shoring plans should be reviewed by the Geotechnical Engineer (a representative of Geocon West, Inc.), prior to finalization to verify that the plans have been prepared in substantial conformance with the recommendations of this report and to provide additional analyses or recommendations.

LIMITATIONS AND UNIFORMITY OF CONDITIONS

1. The recommendations of this report pertain only to the site investigated and are based upon the assumption that the soil conditions do not deviate from those disclosed in the investigation. If any variations or undesirable conditions are encountered during construction, or if the proposed construction will differ from that anticipated herein, Geocon West, Inc. should be notified so that supplemental recommendations can be given. The evaluation or identification of the potential presence of hazardous or corrosive materials was not part of the scope of services provided by Geocon West, Inc.

2. This report is issued with the understanding that it is the responsibility of the owner, or of his representative, to ensure that the information and recommendations contained herein are brought to the attention of the architect and engineer for the project and incorporated into the plans, and the necessary steps are taken to see that the contractor and subcontractors carry out such recommendations in the field.

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

4. The firm that performed the geotechnical investigation for the project should be retained to provide testing and observation services during construction to provide continuity of geotechnical interpretation and to check that the recommendations presented for geotechnical aspects of site development are incorporated during site grading, construction of improvements, and excavation of foundations. If another geotechnical firm is selected to perform the testing and observation services during construction operations, that firm should prepare a letter indicating their intent to assume the responsibilities of project geotechnical engineer of record. A copy of the letter should be provided to the regulatory agency for their records. In addition, that firm should provide revised recommendations concerning the geotechnical aspects of the proposed development, or a written acknowledgement of their concurrence with the recommendations presented in our report. They should also perform additional analyses deemed necessary to assume the role of Geotechnical Engineer of Record.

LIST OF REFERENCES

Abrahamson, N.A, Silva, W.J, and Kamai, R., 2014, *Summary of the ASK14 Ground Motion Relation for Active Crustal Regions*, Earthquake Spectra, Volume 30, No. 3, pages 1025-1055, August 2014.

Boore, D.M., Stewart, J.P., Seyhan, E., and Atkinson, G.M., 2014, *NGA-West2 Equations for Predicting PGA, PGV, and 5% Damped PSA for Shallow Crustal Earthquakes*, Earthquake Spectra, Volume 30, No. 3, pages 1057-1085, August 2014.

California Department of Water Resources, 2021, *Groundwater Level Data by Township, Range, and Section*, Web Site Address: http://www.water.ca.gov/waterdatalibrary/groundwater/hydrographs/index_trs.cfm.

California Department of Water Resources, 1961, *Planned Utilization of Groundwater Basins of the Coastal Plain of Los Angeles County*, Bulletin 104, Appendix A.

California Division of Mines and Geology, 1999; *State of California Seismic Hazard Zones, Beverly Hills Quadrangle*, Official Map Released March 25, 1999.

California Division of Mines and Geology, 1998, *Seismic Hazard Evaluation of the Beverly Hills 7.5-Minute Quadrangle, Los Angeles County, California*, Open File Report 98-14.

California Division of Mines and Geology, 1986, *State of California Special Studies Zones*, *Beverly Hills Quadrangle, Revised Official Map Effective July 1, 1986.*

California Geologic Energy Management Division, 2021, Geologic Energy Management Division Well Finder, http://maps.conservation.ca.gov.doggr/index.html#close.

California Geological Survey, 2021a, CGS Information Warehouse, Regulatory Map Portal, http://maps.conservation.ca.gov/cgs/informationwarehouse/index.html?map=regulatorymaps.

California Geological Survey, 2021b, Earthquake Zones of Required Investigation, https://maps.conservation.ca.gov/cgs/EQZApp/app/.

California Geological Survey, 2018a *Earthquake Fault Zones, A Guide for Government Agencies, Property Owners/Developers, and Geoscience Practitioners for Assessing Fault Rupture Hazards in California*, Special Publication 42, Revised 2018.

California Geological Survey, 2018b, *Zones of Required Investigation, Beverly Hills Quadrangle*, Released January 11, 2018.

California Geological Survey, 2018c, *Fault Evaluation Report FER 259, The Hollywood, Santa-Monica and Newport-Inglewood Faults in the Beverly Hills and Topanga 7.5' Quadrangles, Los Angeles County, California*, by Brian P. E. Olson, Engineering Geologist, dated January 5, 2018 (revised).

California Geological Survey, 2012, *Geologic Compilation of Quaternary Surficial Deposits in Southern California, Los Angeles 30' X 60' Quadrangle*, A Project for the Department of Water Resources by the California Geological Survey, Compiled from existing sources by Trinda L. Bedrossian, CEG and Peter D. Roffers, CGS Special Report 217, Plate 9, Scale 1:100,000.

LIST OF REFERENCES (continued)

Campbell, K.W. and Bozorgnia, Y., 2014, *NGA-West2 Ground Motion Model for the Average Horizontal Components of PGA, PGV, and 5% Damped Linear Acceleration Response Spectra*, Earthquake Spectra, Volume 30, No. 3, pages 1087-1115, August 2014.

Chiou, B. S.-J., and Youngs, R.R., 2014, *Update of the Chiou and Youngs NGA Model for the Average Horizontal Component of Peak Ground Motion and Response Spectra*, Earthquake Spectra, Volume 30, No. 3, pages 1117-1153, August 2014.

Dibblee, T. W., Jr., 1991, *Geologic Map of the Beverly Hills and Van Nuys (South ½) Quadrangles, Los Angeles County, California,* Dibblee Foundation Map # DF-31.

FEMA, 2021, Online Flood Hazard Maps, http://www.esri.com/hazards/index.html.

Helms, John, 2018, *Fault Rupture Hazard Investigation, 8850 to 8878 Sunset Boulevard, West Hollywood, California*, Prepared for Silver Creek Commercial Development, LLC, dated September 19, 2018.

Jennings, C. W. and Bryant, W. A., 2010, *Fault Activity Map of California*, California Geological Survey Geologic Data Map No. 6.

KFM GeoScience, 2010, *Geologic and Seismic Technical Background Report, City of West Hollywood General Plan Update, West Hollywood, Los Angeles County, California*, Prepared for the City of West Hollywood, dated March 10, 2010.

Leighton and Associates, Inc., 1990, *Technical Appendix to the Safety Element of the Los Angeles County General Plan, Hazard Reduction in Los Angeles County*.

Los Angeles County Department of Public Works, 2021a, Ground Water Wells Website, http://dpw2.co.la.ca.us/website/wells/viewer.asp.

Los Angeles County Department of Public Works, 2021b, Flood Zone Determination Website, http://dpw.lacounty.gov/apps/wmd/floodzone/map.htm.

OpenSha, Site Data Application, Version 1.5.0, http://opensha.org/apps, accessed January 2021.

OSHPD Seismic Design Maps Web Application, https://seismicmaps.org/, accessed January 2021.

Shahi, S.K., Baker, J.W., 2013, *NGA-West2 Models for Ground-Motion Directionality*, Pacific Earthquake Engineering Research Center, PEER 2013/10.

Shahi, S.K., Baker, J.W., 2014, *NGA-West2 Models for Ground-Motion Directionality,* Earthquake Spectra, Volume 30, o.3, paged 1285-1300, August 2014.

Toppozada, T., Branum, D., Petersen, M, Hallstrom, C., and Reichle, M., 2000, *Epicenters and Areas Damaged by M> 5 California Earthquakes, 1800 – 1999*, California Geological Survey, Map Sheet 49.

LIST OF REFERENCES (continued)

USGS, 2006, Quaternary Fault and Fold Database of the United States, https://earthquake.usgs.gov/hazards/qfaults.

USGS, 1972, *Beverly Hills 7.5-Minute Topographic Map*.

USGS, BSSC2014 (Scenario Catalog), https://earthquake.usgs.gov/scenarios/catalog/bssc2014/, accessed January 2021.

USGS, NSHMP_HAZ Response Spectral Application, https://earthquake.usgs.gov/nshmp-haz-ws/apps/spectra-plot.html/, accessed January 2021.

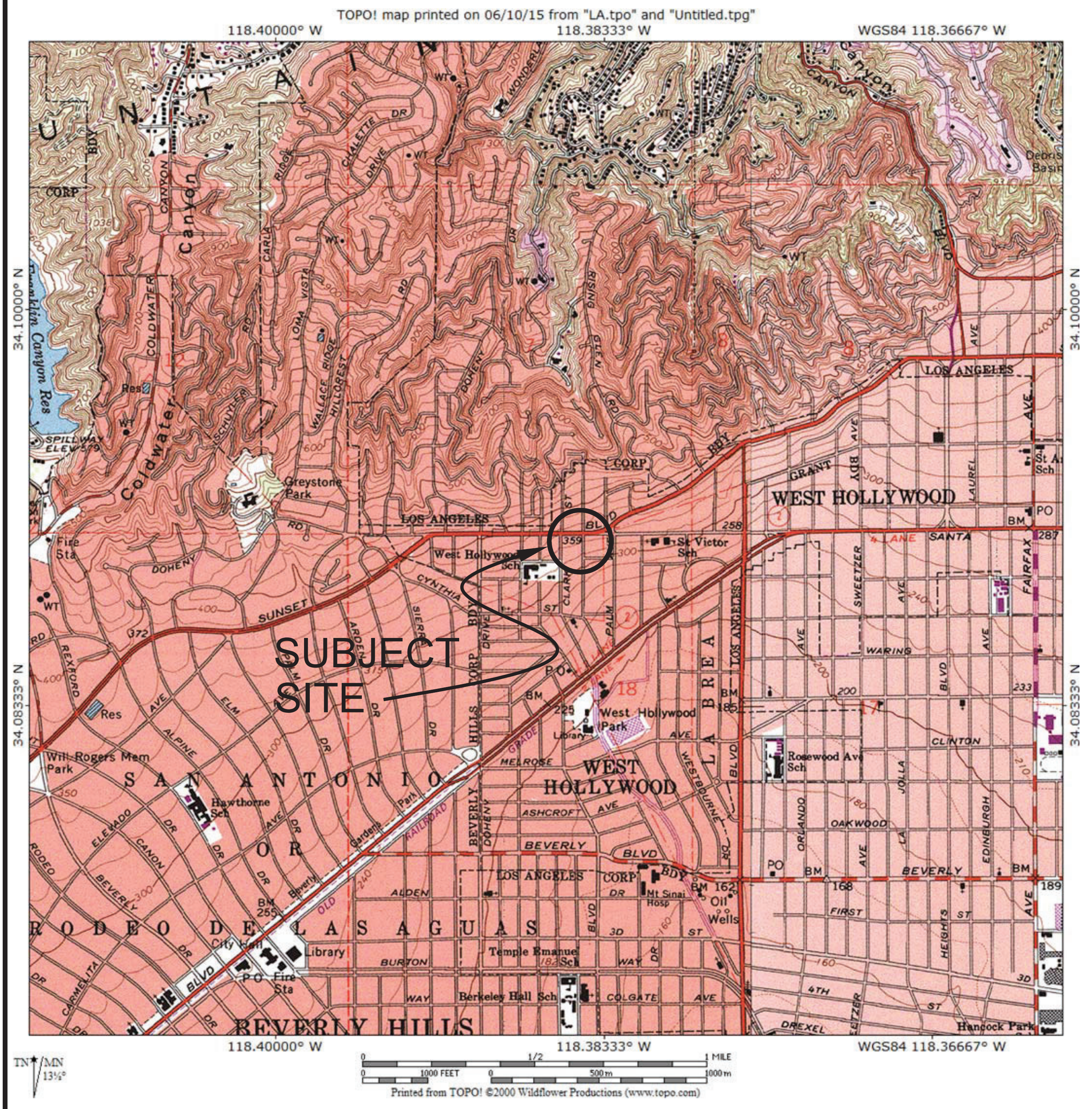
USGS, Unified Hazard Tool, https://earthquake.usgs.gov/hazards/interactive/, accessed January 2021.

West Hollywood, City of, 2019, City Fault Location and Precaution Zone Map, by TetraTech, Figure 3.

West Hollywood, City of, 2011, *West Hollywood General Plan 2035*, Chapter 10, Safety and Noise.

West Hollywood, City of, 2010, *Geologic and Seismic Technical Background Report*, *City of West Hollywood General Plan Update*, *West Hollywood*, *Los Angeles County*, *California*, Prepared for the City of West Hollywood by KFM GeoScience, report dated March 10, 2010.

Ziony, J. I., and Jones, L. M., 1989, *Map Showing Late Quaternary Faults and 1978–1984 Seismicity of the Los Angeles Region, California*, U.S. Geological Survey Miscellaneous Field Studies Map MF-1964.



REFERENCE:U.S.G.S. TOPOGRAPHIC MAPS, 7.5 MINUTE SERIES, BEVERLY HILLS AND HOLLYWOOD, CA QUADRANGLES

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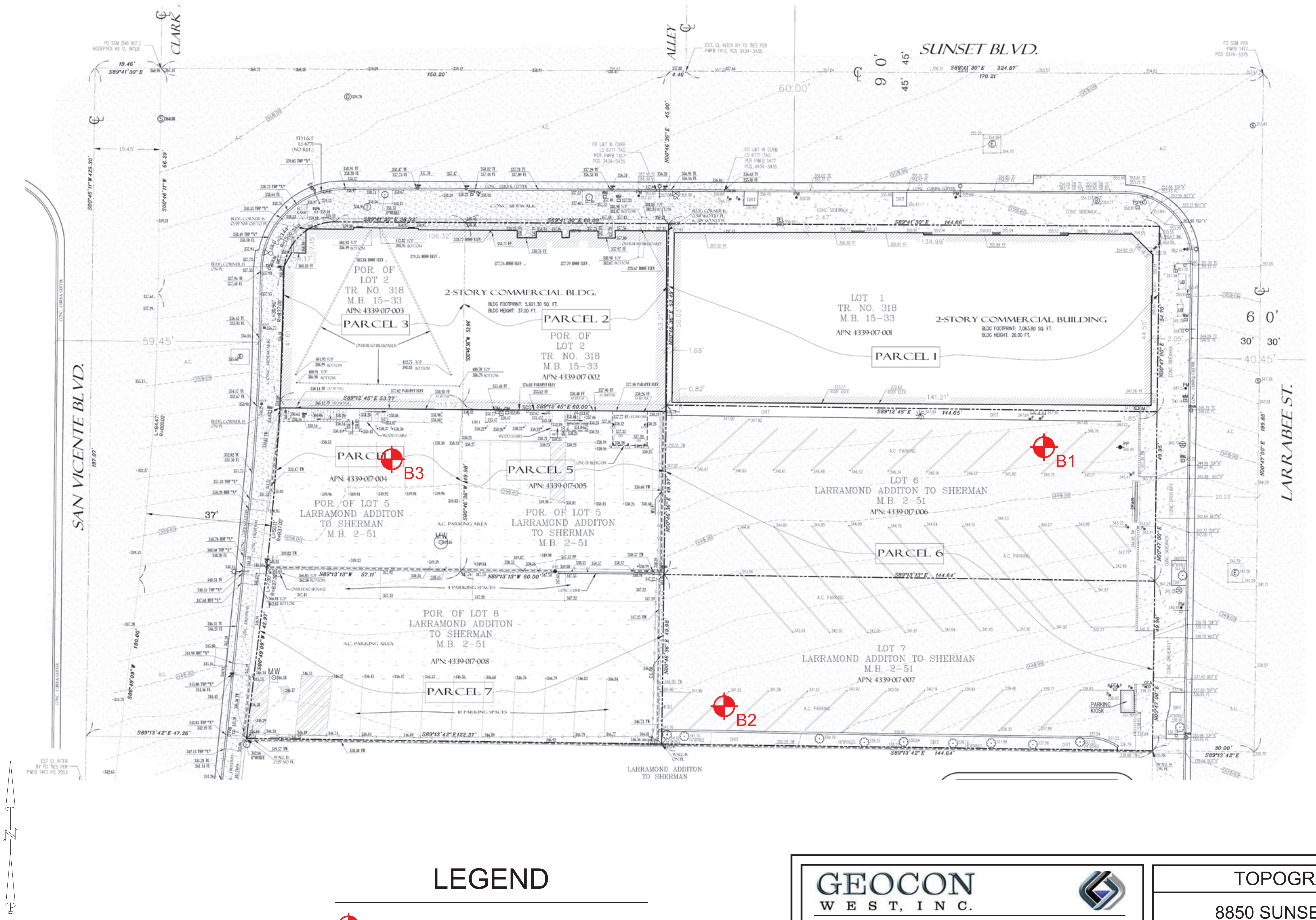
DRAFTED BY: RA | CHECKED BY: SFK

VICINITY MAP

8850 SUNSET BOULEVARD
WEST HOLLYWOOD, CALIFORNIA

FEB. 2021 | PROJECT NO. A9899-06-01 | FIG. 1



0 30' 60'

LEGEND

⊕ B3 Boring Location and Number

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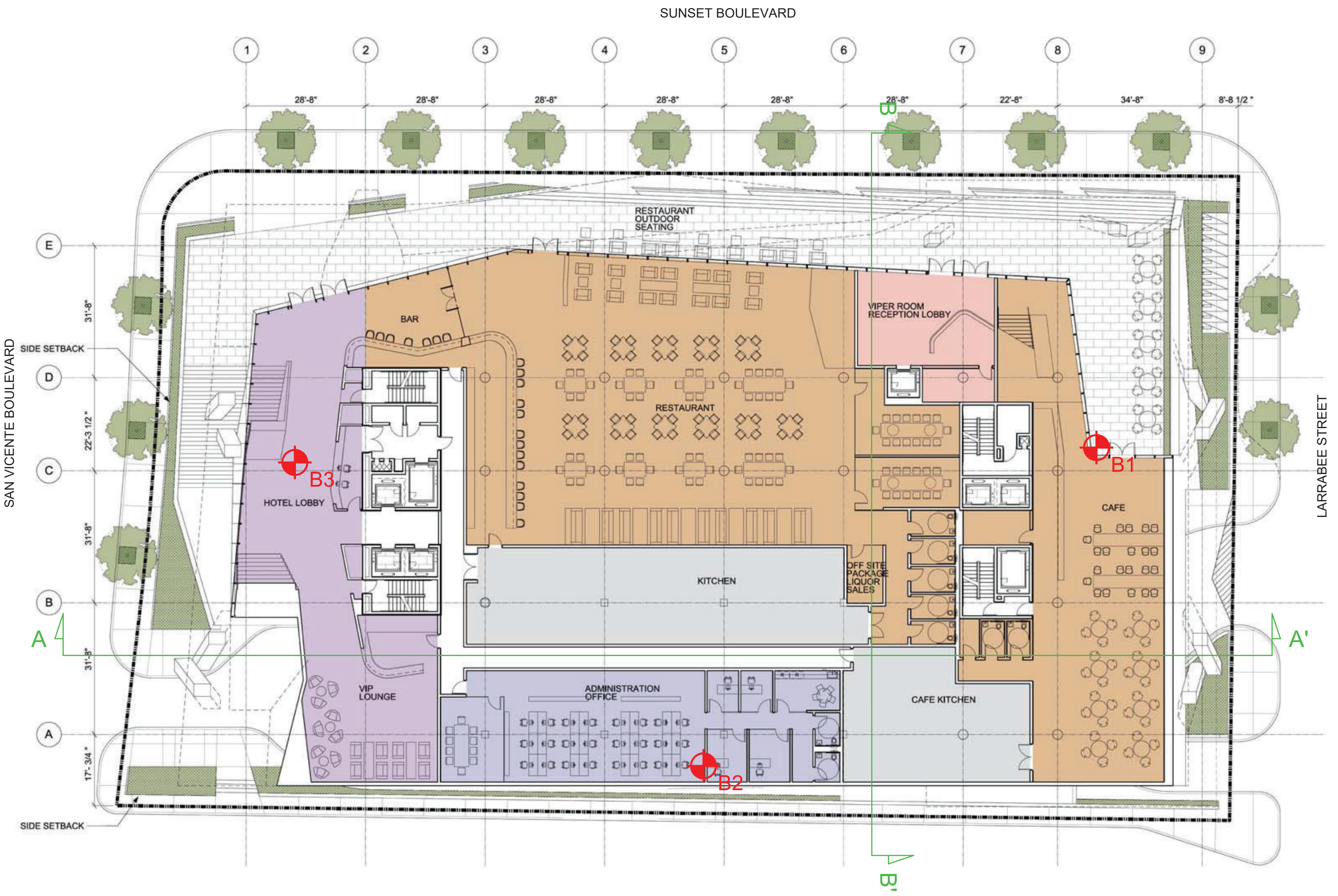
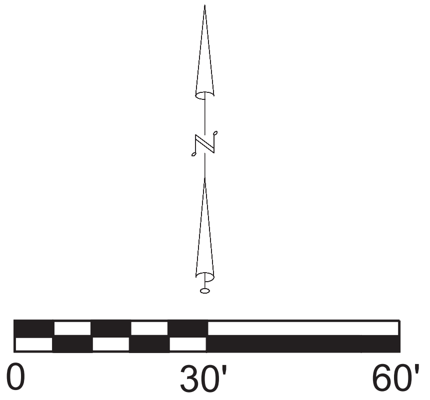
TOPOGRAPHIC PLAN

8850 SUNSET BOULEVARD
WEST HOLLYWOOD, CALIFORNIA

FEB. 2021
PROJECT NO. A9899-06-01
FIG. 2A





LEGEND

B3 Boring Location and Number

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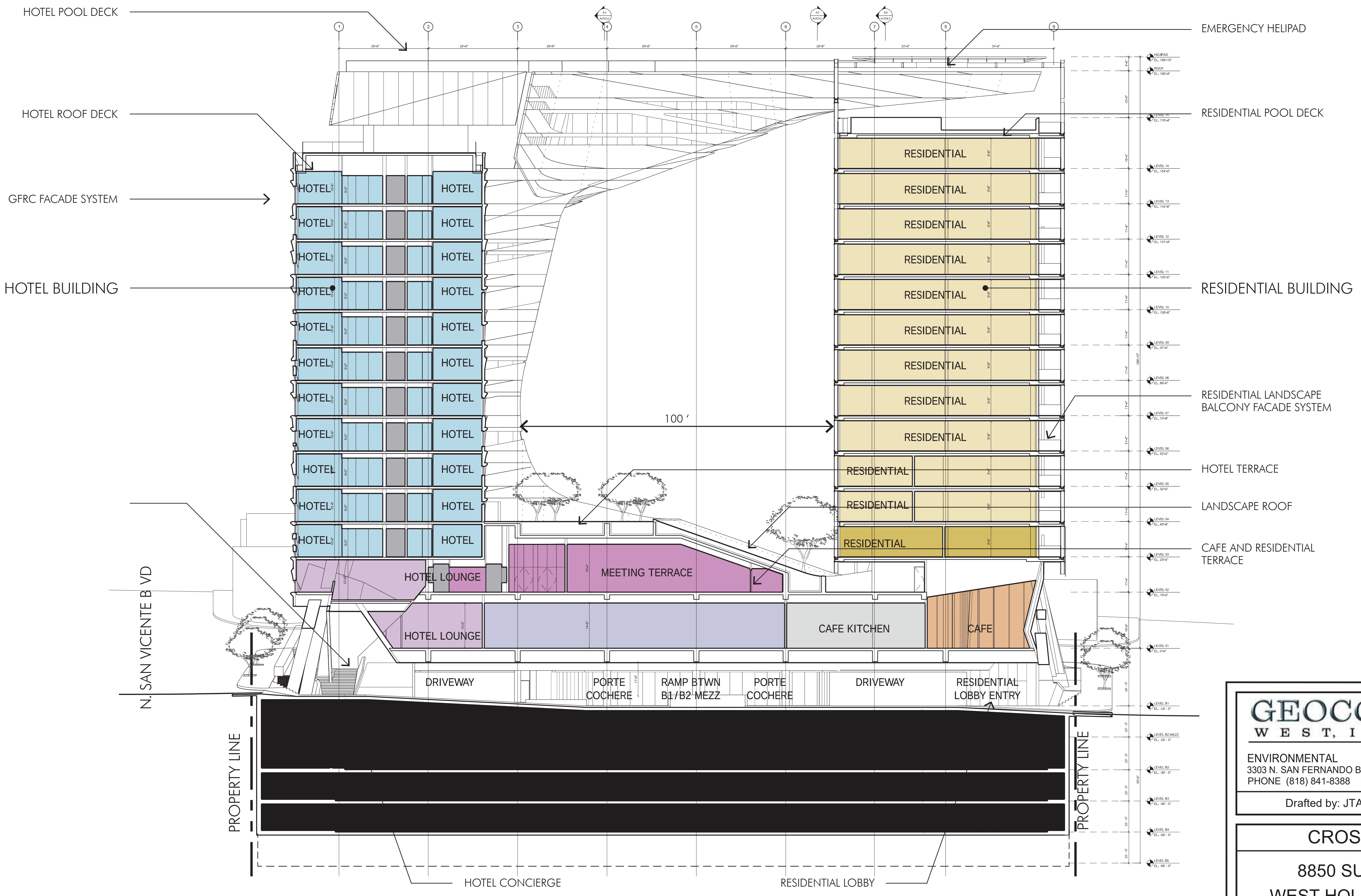
SITE PLAN

8850 SUNSET BOULEVARD
WEST HOLLYWOOD, CALIFORNIA

FEB. 2021 | PROJECT NO. A9899-06-01 | FIG. 2B



Scale: 1" = 30' (H&V)

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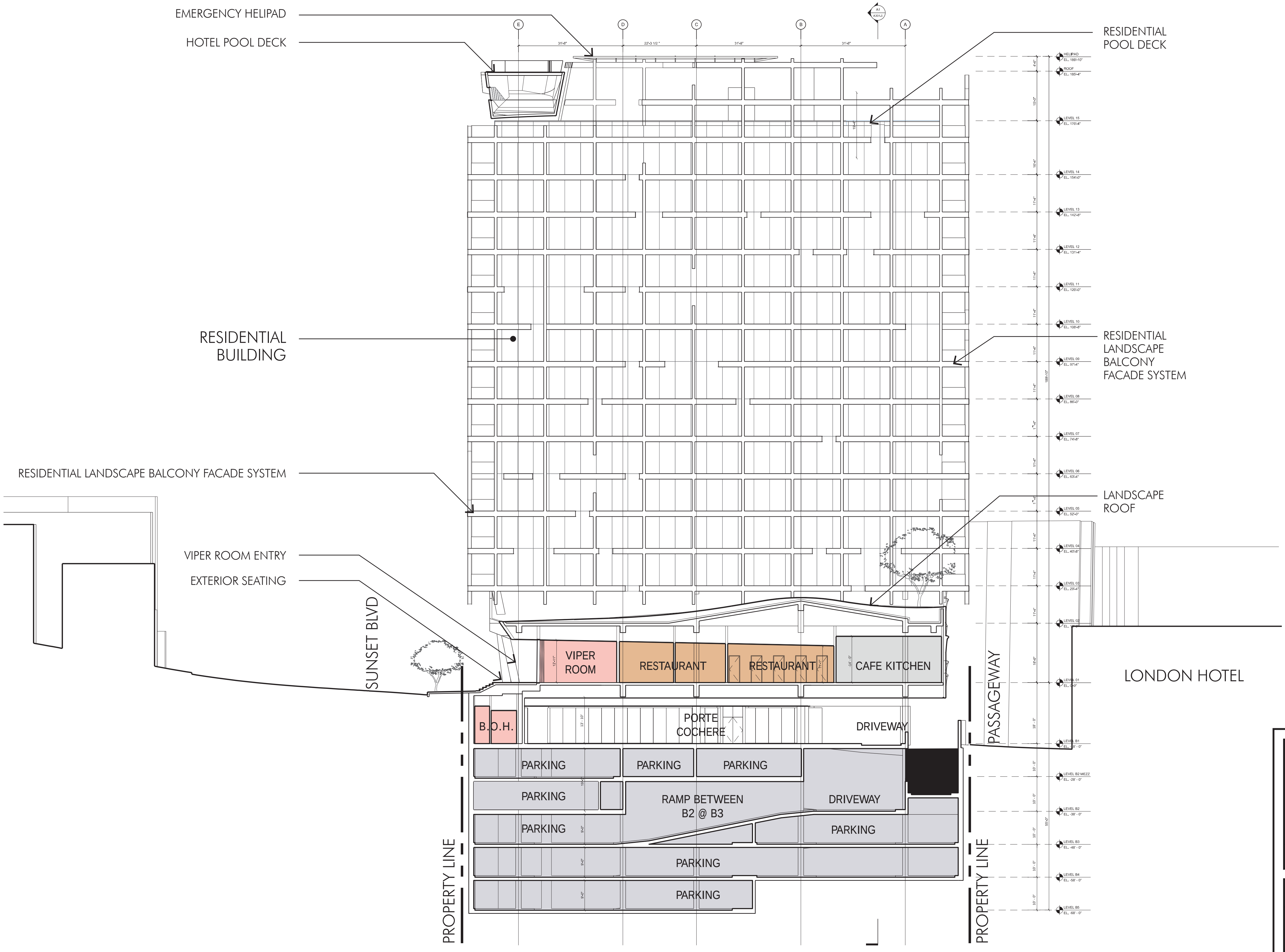
CROSS-SECTION A-A'

8850 SUNSET BOULEVARD
WEST HOLLYWOOD, CALIFORNIA

FEB. 2021	PROJECT NO. A9899-06-01	FIG. 3A



Scale: 1" = 30' (H&V)

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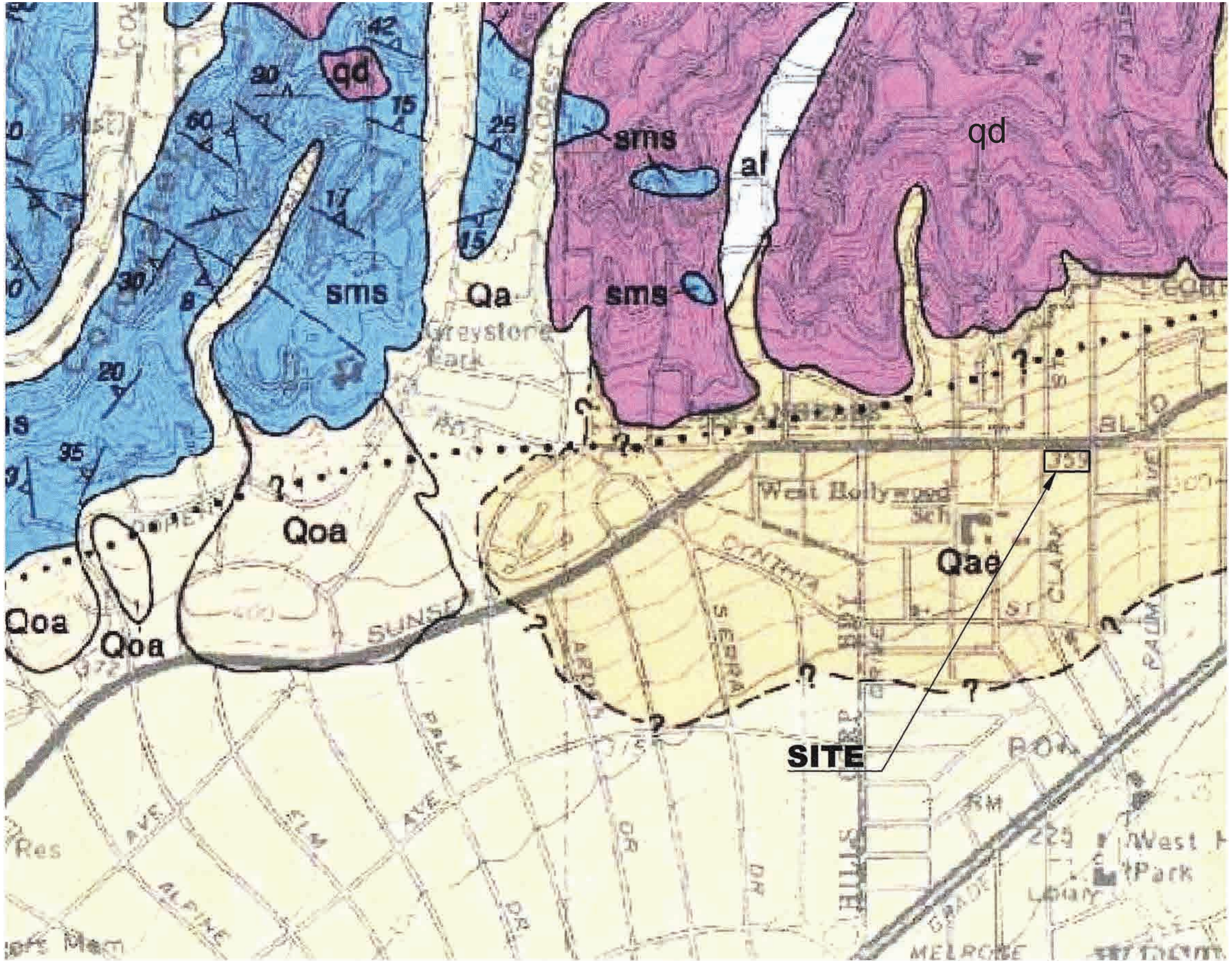
CROSS-SECTION B-B'

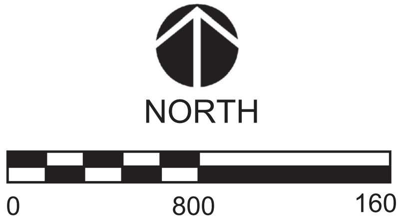
8850 SUNSET BOULEVARD
WEST HOLLYWOOD, CALIFORNIA

FEB. 2021	PROJECT NO. A9899-06-01	FIG. 3B



LEGEND

af - Artificial Fill
Qa - Holocene Alluvium
Qae - Late Holocene to Pleistocene Alluvium
Qoa - Pleistocene Alluvial Fan Deposits
sms - Santa Monica Slate
qd - Quartz Diorite



MAP REFERENCE: HELMS, J., 2018, FIG. 3
DIBBLEE, T.W. AND EHRENSPECK, H.E., ED:
GEOLOGIC MAP OF THE BEVERLY HILLS AND
VAN NUYS (SOUTH 1/2) QUADRANGLES,
LOS ANGELES COUNTY, CALIFORNIA, 1991
DF-31, SCALE 1:24,000

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GEOLOGY MAP

8850 SUNSET BOULEVARD
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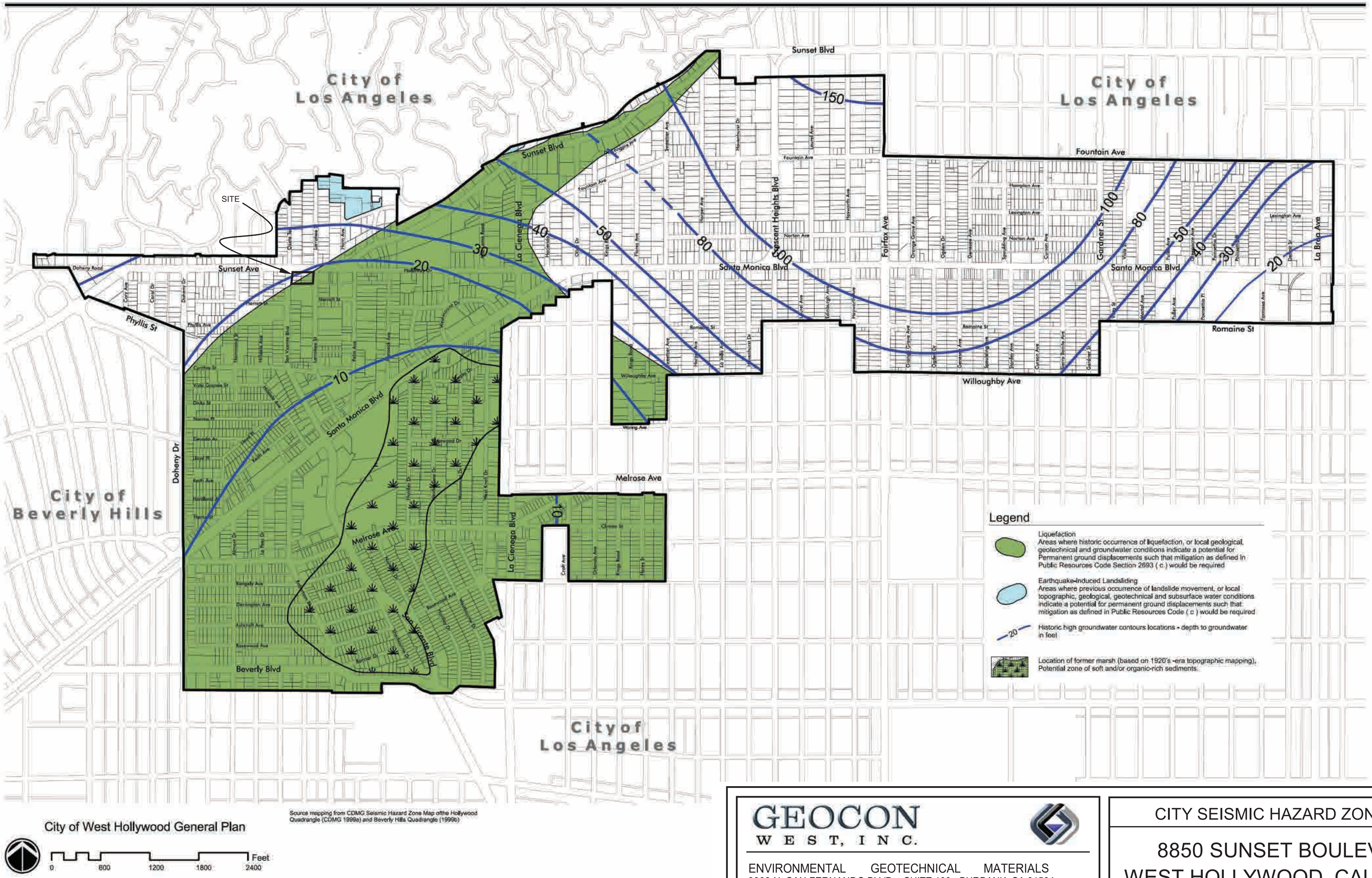

Source mapping from CDMG Seismic Hazard Zone Map ofthe Hollywood Quadrangle (CDMG 1999a) and Beverly Hills Quadrangle (1999b)

MAP REFERENCE: KFM, 2010 (FIG. 3)

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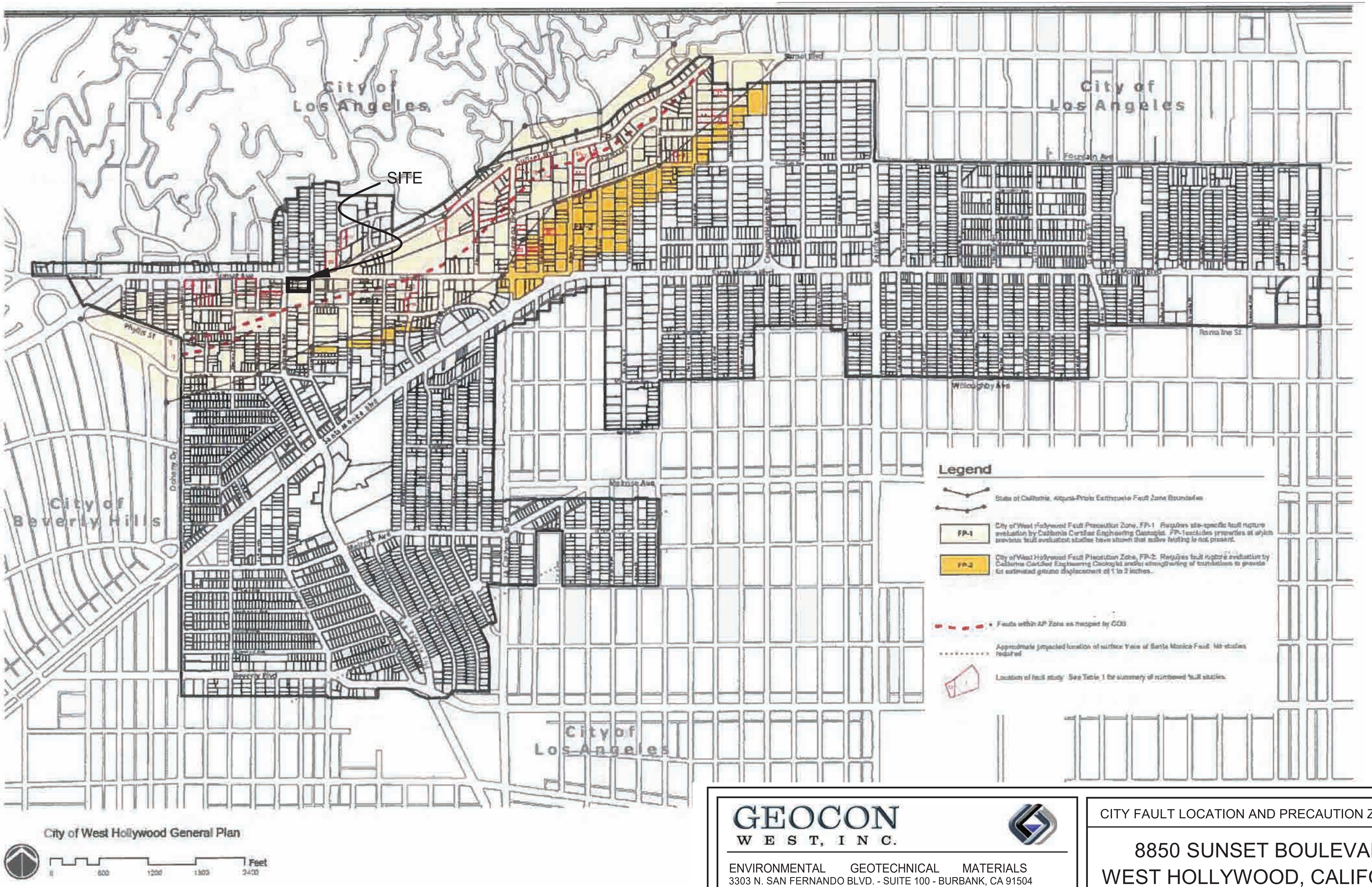
CITY SEISMIC HAZARD ZONE MAP

8850 SUNSET BOULEVARD
WEST HOLLYWOOD, CALIFORNIA

FEB. 2021	PROJECT NO. A9899-06-01	FIG. 5A



MAP REFERENCE: TETRA TECH, 2010 (UPDATED 2019)

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CITY FAULT LOCATION AND PRECAUTION ZONE MAP

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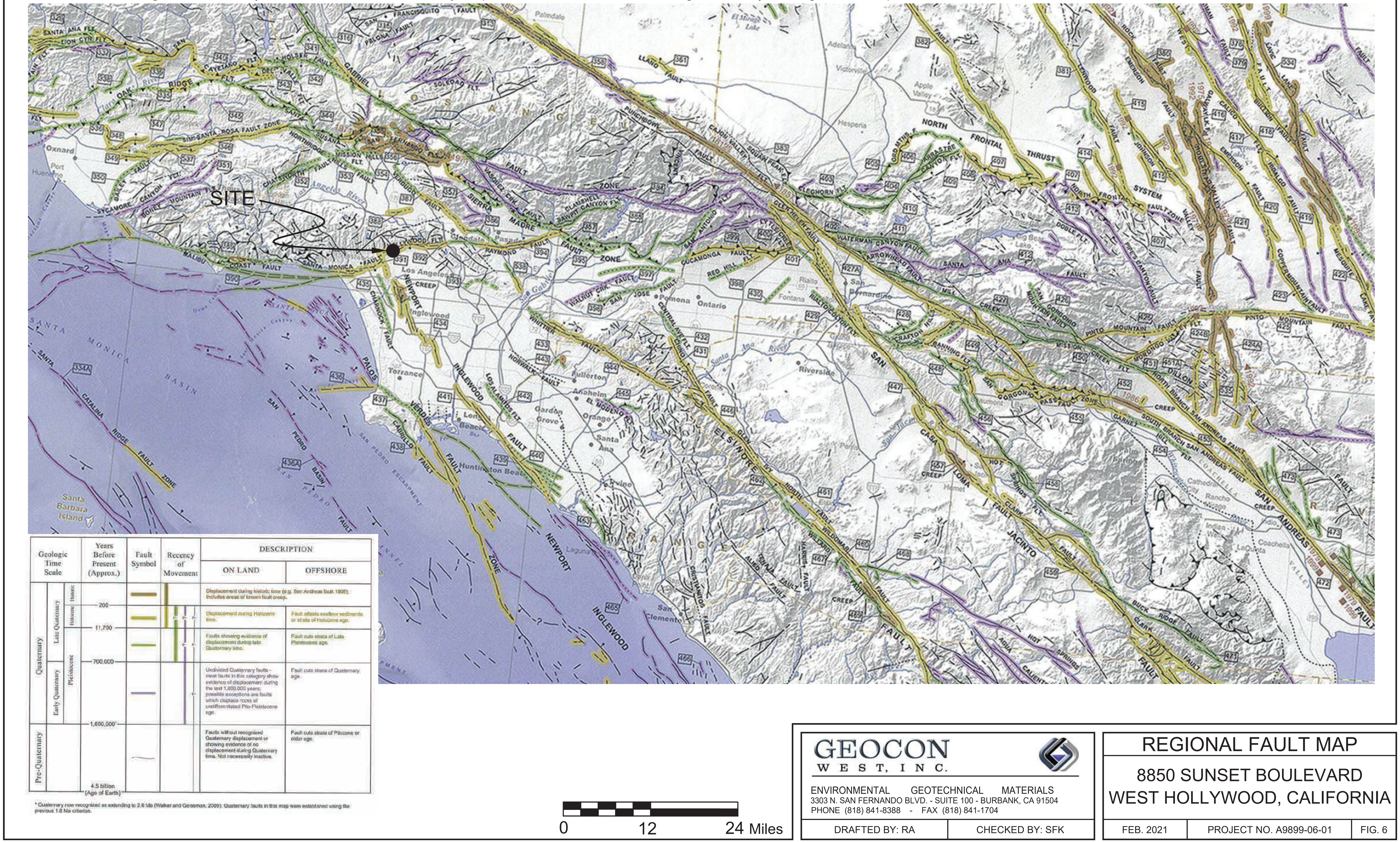
Reference: Jennings, C.W. and Bryant, W. A., 2010, Fault Activity Map of California, California Geological Survey Geologic Data Map No. 6.



Geologic Time Scale			Years Before Present (Approx.)	Fault Symbol	Recency of Movement	DESCRIPTION	
						ON LAND	OFFSHORE
Quaternary	Late Quaternary	Historic	200			Displacement during historic time (e.g. San Andreas fault 1906). Includes areas of known fault creep.	
		Holocene	11,700			Displacement during Holocene time.	Fault offsets seafloor sediments or strata of Holocene age.
		Pleistocene	700,000			Faults showing evidence of displacement during late Quaternary time.	Fault cuts strata of Late Pleistocene age.
	Early Quaternary		1,600,000*			Undivided Quaternary faults - most faults in this category show evidence of displacement during the last 1,600,000 years; possible exceptions are faults which displace rocks of undifferentiated Plio-Pleistocene age.	Fault cuts strata of Quaternary age.
Pre-Quaternary			4.5 billion (Age of Earth)			Faults without recognized Quaternary displacement or showing evidence of no displacement during Quaternary time. Not necessarily inactive.	Fault cuts strata of Pliocene or older age.

* Quaternary now recognized as extending to 2.6 Ma (Walker and Geissman, 2009). Quaternary faults in this map were established using the previous 1.6 Ma criterion.

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REGIONAL FAULT MAP

8850 SUNSET BOULEVARD
WEST HOLLYWOOD, CALIFORNIA

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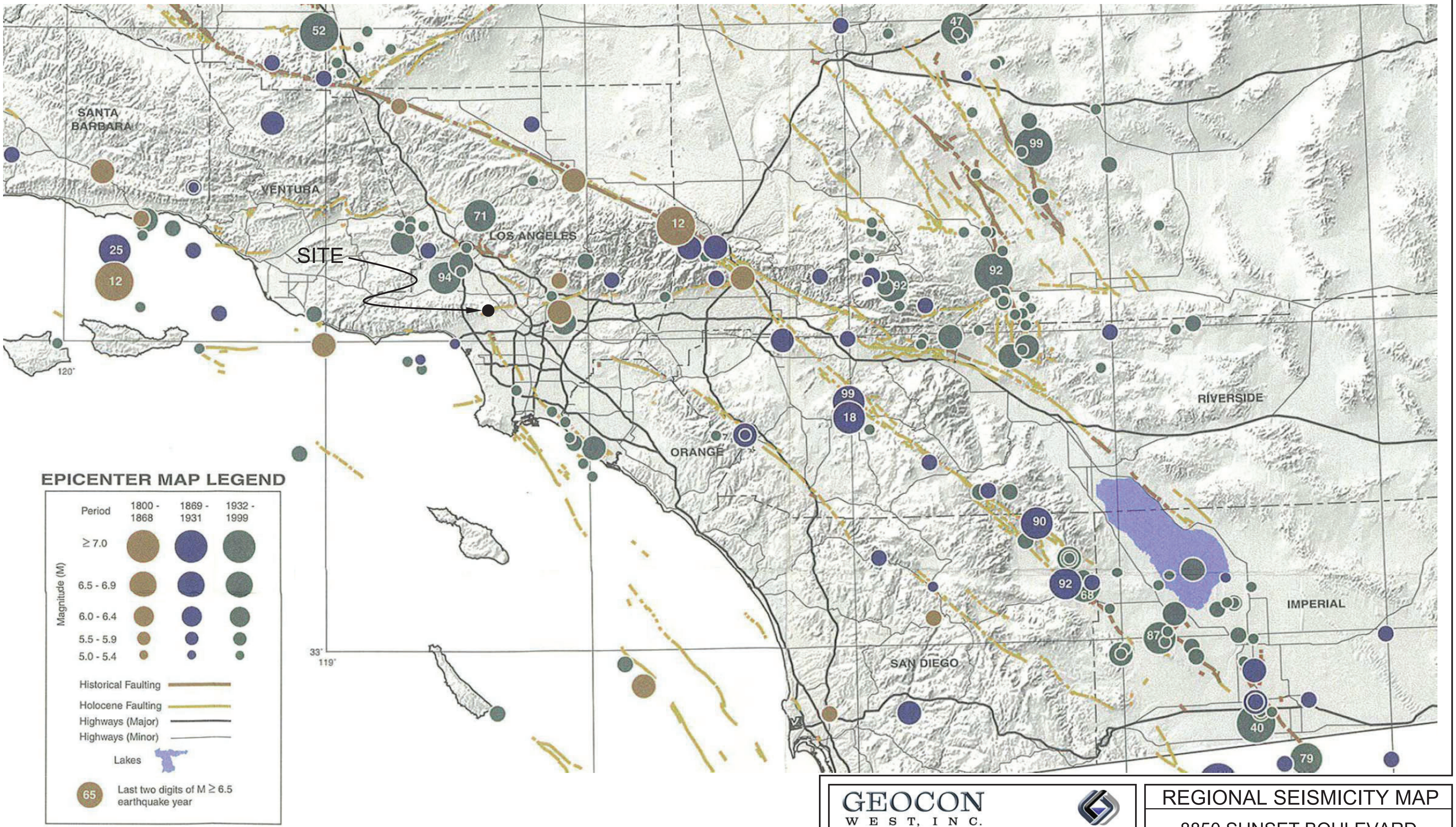
PROJECT NO. A9899-06-01
FIG. 6



Reference: Toppozada, T., Branum, D., Petersen, M., Hallstrom, C., Cramer, C., and Reichle, M., 2000, Epicenters and Areas Damaged by M≥5 California Earthquakes, 1800 - 1999, California Geological Survey, Map Sheet 49.

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REGIONAL SEISMICITY MAP

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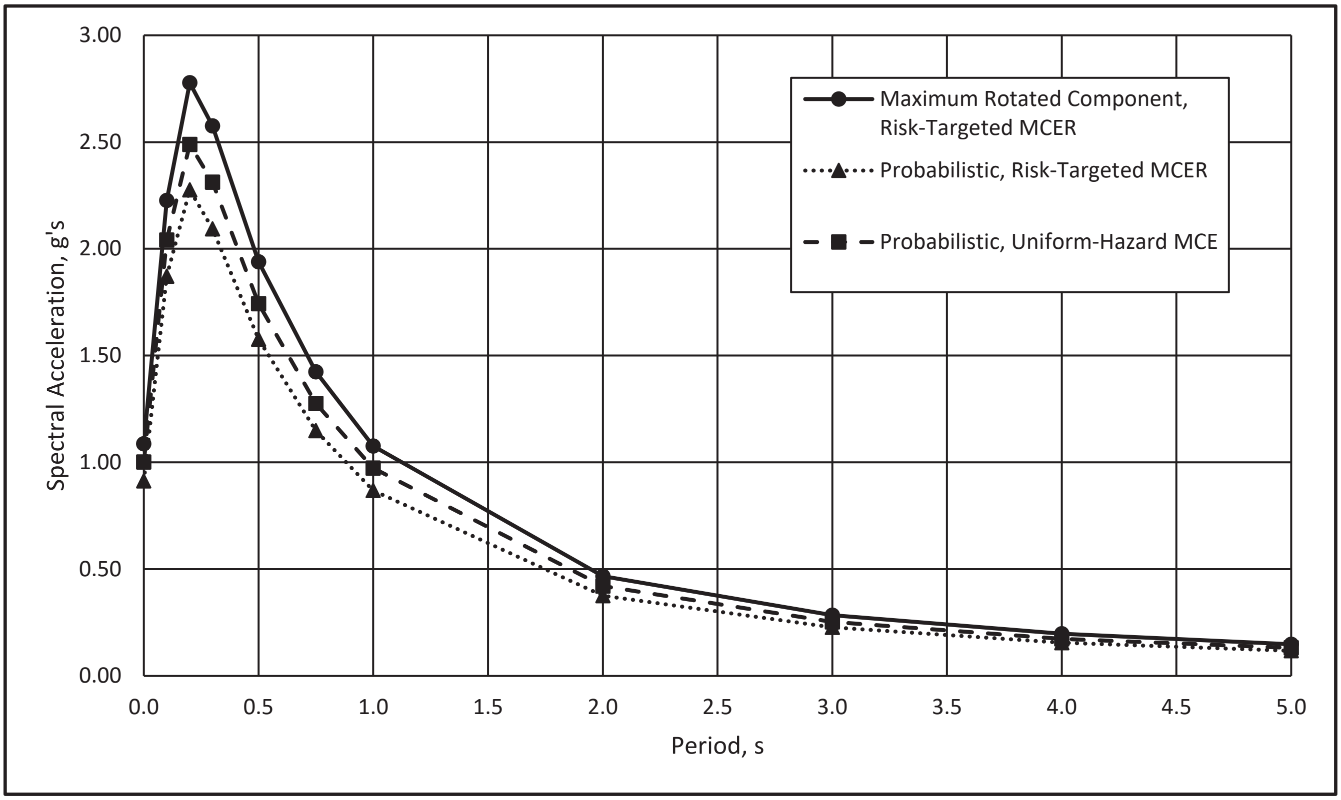
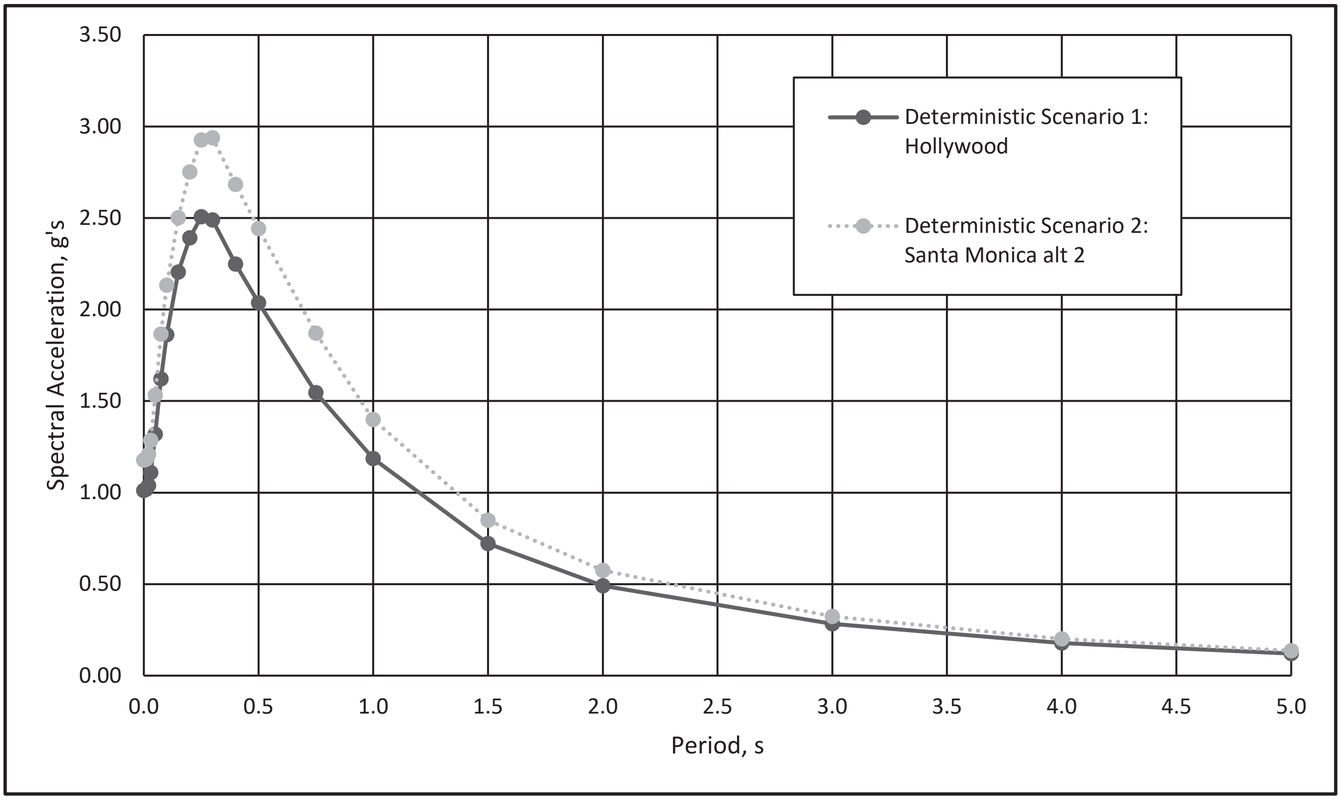
WEST HOLLYWOOD, CALIFORNIA

FEB. 2021
PROJECT NO. A9899-06-01
FIG. 7





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DESIGN RESPONSE SPECTRUM

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Project No.: A9899-06-01

8850 SUNSET BOULEVARD
WEST HOLLYWOOD, CALIFORNIA

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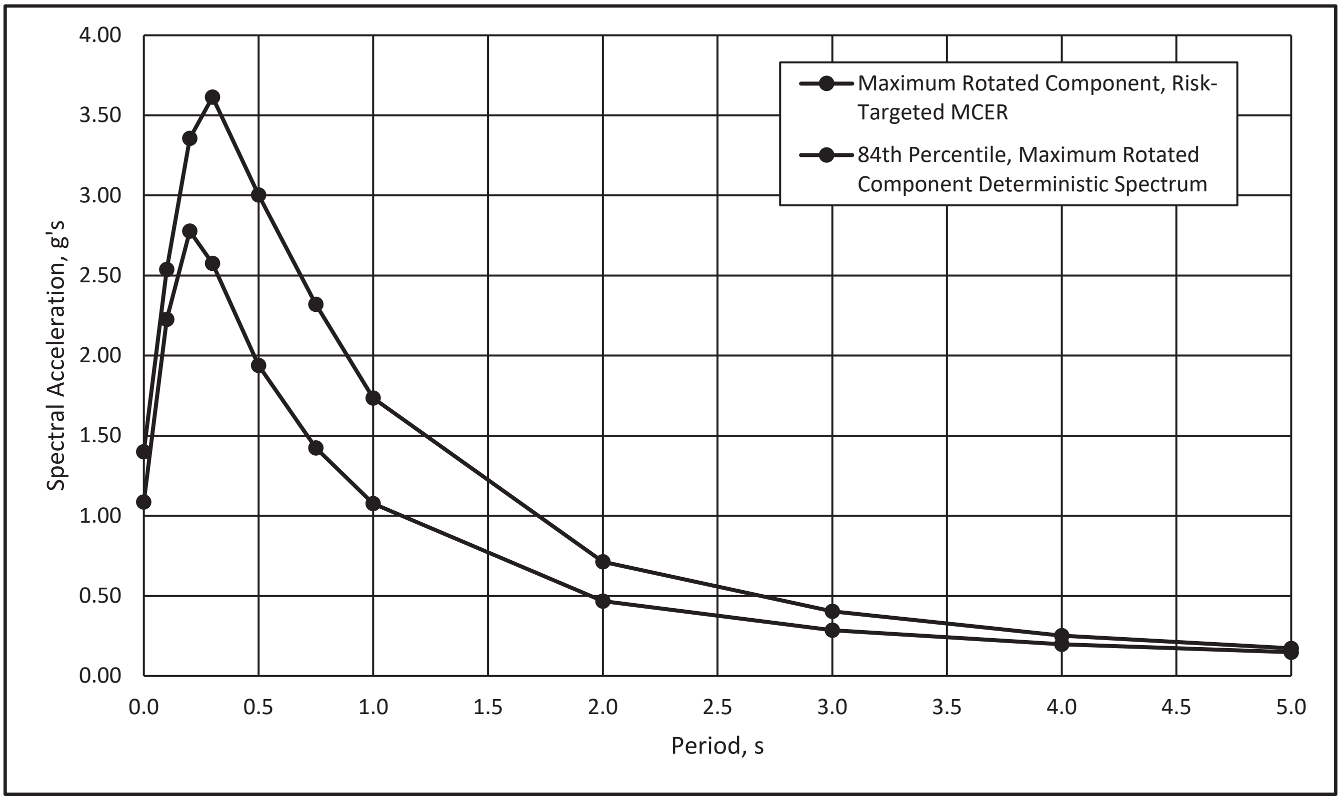
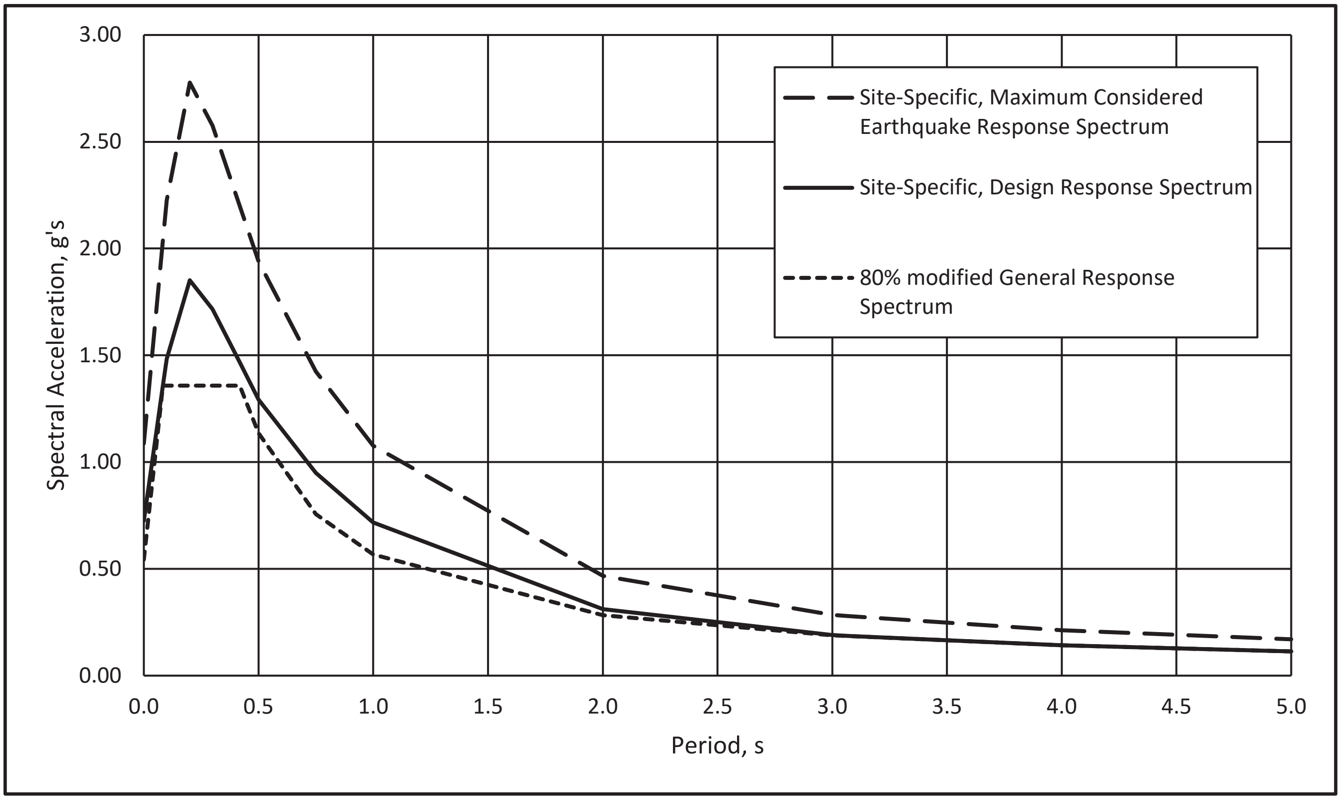
Figure 8





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DESIGN RESPONSE SPECTRUM

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WEST HOLLYWOOD, CALIFORNIA

FEB. 2021

Figure 9

Spectral Period (seconds)	Probabilistic Uniform-Hazard	Risk-Targeted, Probabilistic	Risk Factor, Cr	Maximum-Rotated Componet Scale Factor	MRC, Risk-Targeted Probablistic	84th Percentile, Deterministic	Site-Specific Design Earthquake	80% Modifed General Response Spectrum	Site-Specific Maximum Considered Earthquake
0.00	1.002	0.914	0.912	1.190	1.087	1.400	0.725	0.543	1.087
0.08	--	--	--	--	--	--	1.360	1.358	2.040
0.10	2.041	1.871	0.917	1.190	2.227	2.538	1.485	1.358	2.227
0.20	2.489	2.276	0.914	1.220	2.777	3.356	1.851	1.358	2.777
0.30	2.312	2.094	0.906	1.230	2.575	3.615	1.717	1.358	2.575
0.42	--	--	--	--	--	--	1.467	1.358	2.200
0.50	1.742	1.577	0.905	1.230	1.939	3.003	1.293	1.135	1.939
0.75	1.275	1.148	0.900	1.240	1.424	2.320	0.949	0.757	1.424
1.00	0.973	0.868	0.892	1.240	1.076	1.735	0.717	0.567	1.076
2.00	0.421	0.377	0.895	1.240	0.467	0.713	0.311	0.284	0.467
3.00	0.253	0.228	0.900	1.250	0.285	0.404	0.190	0.189	0.285
3.08	--	--	--	--	--	--	0.186	0.186	0.278
4.00	0.174	0.156	0.900	1.260	0.197	0.252	0.142	0.142	0.213
5.00	0.131	0.118	0.896	1.260	0.148	0.172	0.113	0.113	0.170

SM_S = 2.500 g
SM_1 = 1.076 g

SD_S = 1.666 g
SD_1 = 0.717 g

Reference: ASCE 7-16 21.4 DESIGN ACCELERATION PARAMETERS

Where the site-specific procedure is used to determine the design ground motion in accordance with Section 21.3, the parameter S_{DS} shall be taken as 90% of the maximum spectral acceleration, S_a, obtained from the site-specific spectrum, at any period within the range from 0.2 to 5 s, inclusive. The parameter S_{D1} shall be taken as the maximum value of the product, TS_a, for periods from 1 to 2 s for sites with $v_{s,30}$ > 1,200 ft/s ($v_{s,30}$ > 365.76 m/s) and for periods from 1 to 5 s for sites with $v_{s,30}$ ≤ 1,200 ft/=s ($v_{s,30}$ ≤ 365.76 m/s). The parameters S_{MS} and S_{M1} shall be taken as 1.5 times S_{DS} and S_{D1}, respectively. The values so obtained shall not be less than 80% of the values determined in accordance with Section 11.4.3 for S_{MS} and S_{M1} and Section 11.4.5 for S_{DS} and S_{D1}.

"--" Indicates that spectral period was not used at that calculation step

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DESIGN RESPONSE SPECTRUM

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Project No.: A9899-06-01

8850 SUNSET BOULEVARD
WEST HOLLYWOOD, CALIFORNIA

FEB. 2021

Figure 10

GEOCON

Client : Plus Development
File No. : A9899-06-01
Boring : 2

EMPIRICAL ESTIMATION OF LIQUEFACTION POTENTIAL
DESIGN EARTHQUAKE

NCEER (1996) METHOD

EARTHQUAKE INFORMATION:

Earthquake Magnitude:	6.72
Peak Horiz. Acceleration PGA_M (g):	1.002
2/3 PGA_M (g):	0.668
Calculated Mag.Wtg.Factor:	0.759
Historic High Groundwater:	20.0
Groundwater Depth During Exploration:	40.5

By Thomas F. Blake (1994-1996)

ENERGY & ROD CORRECTIONS:

Energy Correction (CE) for N60:	1.25
Rod Len.Corr.(CR)(0-no or 1-yes):	1.0
Bore Dia. Corr. (CB):	1.00
Sampler Corr. (CS):	1.20
Use Ksigma (0 or 1):	1.0

LIQUEFACTION CALCULATIONS:

Unit Wt. Water (pcf): 62.4

Depth to Base (ft)	Total Unit Wt. (pcf)	Water (0 or 1)	FIELD SPT (N)	Depth of SPT (ft)	Liq.Sus. (0 or 1)	-200 (%)	Est. Dr (%)	CN Factor	Corrected (N1)60	Eff. Unit Wt. (psf)	Resist. CRR	rd Factor	Induced CSR	Liquefac. Safe.Fact.
1.0	120.0	0	6.0	1.0	1		59	1.700	11.5	120.0	0.125	0.998	0.329	--
2.0	120.0	0	6.0	2.0	1		58	1.700	11.5	120.0	0.125	0.993	0.327	--
3.0	120.0	0	6.0	3.0	1		56	1.700	11.5	120.0	0.125	0.989	0.326	--
4.0	120.0	0	2.0	4.0	1		32	1.700	3.8	120.0	0.060	0.984	0.324	--
5.0	120.0	0	2.0	5.0	1		31	1.700	3.8	120.0	0.060	0.979	0.323	--
6.0	120.0	0	2.0	6.0	1		30	1.700	3.8	120.0	0.060	0.975	0.321	--
7.0	120.0	0	2.0	7.0	1		29	1.636	3.7	120.0	0.059	0.970	0.320	--
8.0	120.0	0	4.0	8.0	1		41	1.523	6.9	120.0	0.080	0.966	0.318	--
9.0	120.0	0	4.0	9.0	1		40	1.431	6.4	120.0	0.076	0.961	0.317	--
10.0	120.0	0	4.0	10.0	1		39	1.353	6.1	120.0	0.074	0.957	0.315	--
11.0	120.0	0	4.0	10.0	1		39	1.287	5.8	120.0	0.071	0.952	0.314	--
12.0	120.0	0	9.0	15.0	1		54	1.230	13.4	120.0	0.146	0.947	0.312	--
13.0	120.0	0	9.0	15.0	1		54	1.180	12.8	120.0	0.140	0.943	0.311	--
14.0	120.0	0	9.0	15.0	1		54	1.135	12.4	120.0	0.135	0.938	0.309	--
15.0	120.0	0	9.0	15.0	1		54	1.095	11.9	120.0	0.130	0.934	0.308	--
16.0	120.0	0	9.0	15.0	1		54	1.060	11.5	120.0	0.126	0.929	0.306	--
17.0	120.0	0	9.0	15.0	1		54	1.027	11.2	120.0	0.122	0.925	0.305	--
18.0	120.0	0	31.0	20.0	1		93	0.997	41.5	120.0	Infin.	0.920	0.303	--
19.0	120.0	0	31.0	20.0	1		93	0.970	40.4	120.0	Infin.	0.915	0.302	--
20.0	120.0	0	31.0	20.0	1		93	0.945	39.3	120.0	Infin.	0.911	0.300	--
21.0	120.0	1	31.0	20.0	1		93	0.921	38.3	57.6	Infin.	0.906	0.303	Non-Liq.
22.0	120.0	1	31.0	20.0	1		93	0.900	37.4	57.6	Infin.	0.902	0.308	Non-Liq.
23.0	120.0	1	31.0	20.0	1		93	0.879	36.6	57.6	Infin.	0.897	0.314	Non-Liq.
24.0	120.0	1	35.0	25.0	1		95	0.860	43.2	57.6	Infin.	0.893	0.319	Non-Liq.
25.0	120.0	1	35.0	25.0	1		95	0.843	42.3	57.6	Infin.	0.888	0.324	Non-Liq.
26.0	120.0	1	35.0	25.0	1		95	0.826	41.4	57.6	Infin.	0.883	0.328	Non-Liq.
27.0	120.0	1	35.0	25.0	1		95	0.810	40.6	57.6	Infin.	0.879	0.332	Non-Liq.
28.0	120.0	1	35.0	25.0	1		95	0.795	39.9	57.6	Infin.	0.874	0.336	Non-Liq.
29.0	120.0	1	35.0	25.0	1		95	0.781	39.2	57.6	Infin.	0.870	0.339	Non-Liq.
30.0	120.0	1	25.0	30.0	1	36	78	0.768	35.8	57.6	Infin.	0.865	0.342	Non-Liq.
31.0	120.0	1	25.0	30.0	1	36	78	0.755	35.3	57.6	Infin.	0.861	0.345	Non-Liq.
32.0	120.0	1	25.0	30.0	1	36	78	0.743	34.9	57.6	Infin.	0.856	0.348	Non-Liq.
33.0	120.0	1	25.0	30.0	1	36	78	0.732	34.4	57.6	Infin.	0.851	0.351	Non-Liq.
34.0	120.0	1	25.0	30.0	1	36	78	0.721	34.0	57.6	Infin.	0.847	0.353	Non-Liq.
35.0	120.0	1	18.0	35.0	0	39		0.710	26.2	57.6	~	0.842	0.355	~
36.0	120.0	1	18.0	35.0	0	39		0.700	25.9	57.6	~	0.838	0.357	~
37.0	120.0	1	18.0	35.0	0	39		0.690	25.6	57.6	~	0.833	0.359	~
38.0	120.0	1	18.0	35.0	0	39		0.681	25.4	57.6	~	0.829	0.361	~
39.0	120.0	1	18.0	35.0	0	39		0.672	25.2	57.6	~	0.824	0.362	~
40.0	120.0	1	15.0	40.0	0	39		0.664	21.9	57.6	~	0.819	0.363	~
41.0	120.0	1	15.0	40.0	0	39		0.658	21.8	57.6	~	0.815	0.364	~
42.0	120.0	1	15.0	40.0	0	39		0.654	21.7	57.6	~	0.810	0.365	~
43.0	120.0	1	25.0	45.0	1	42	72	0.650	31.4	57.6	Infin.	0.806	0.366	Non-Liq.
44.0	120.0	1	25.0	45.0	1	42	72	0.646	31.2	57.6	Infin.	0.801	0.367	Non-Liq.
45.0	120.0	1	25.0	45.0	1	42	72	0.642	31.1	57.6	Infin.	0.797	0.368	Non-Liq.
46.0	120.0	1	25.0	45.0	1	42	72	0.639	31.0	57.6	Infin.	0.792	0.368	Non-Liq.
47.0	120.0	1	25.0	45.0	1	42	72	0.635	30.8	57.6	Infin.	0.787	0.369	Non-Liq.
48.0	120.0	1	25.0	45.0	1	42	72	0.632	30.7	57.6	Infin.	0.783	0.369	Non-Liq.
49.0	120.0	1	25.0	45.0	1	42	72	0.628	30.6	57.6	Infin.	0.778	0.369	Non-Liq.
50.0	120.0	1	20.0	50.0	0			0.625	18.7	57.6	~	0.774	0.369	~
51.0	120.0	1	20.0	50.0	0			0.622	18.6	57.6	~	0.769	0.370	~
52.0	120.0	1	20.0	50.0	0			0.618	18.5	57.6	~	0.765	0.369	~
53.0	120.0	1	20.0	50.0	0			0.615	18.5	57.6	~	0.760	0.369	~
54.0	120.0	1	20.0	50.0	0			0.612	18.4	57.6	~	0.755	0.369	~
55.0	120.0	1	26.0	55.0	1	39	70	0.609	30.7	57.6	Infin.	0.751	0.369	Non-Liq.
56.0	120.0	1	26.0	55.0	1	39	70	0.606	30.6	57.6	Infin.	0.746	0.369	Non-Liq.
57.0	120.0	1	26.0	55.0	1	39	70	0.603	30.5	57.6	Infin.	0.742	0.368	Non-Liq.
58.0	120.0	1	26.0	55.0	1	39	70	0.600	30.4	57.6	Infin.	0.737	0.368	Non-Liq.
59.0	120.0	1	26.0	55.0	1	39	70	0.597	30.3	57.6	Infin.	0.733	0.367	Non-Liq.
60.0	120.0	1	19.0	60.0	1	28	59	0.594	22.3	57.6	0.214	0.728	0.366	0.58
61.0	120.0	1	19.0	60.0	1	28	59	0.591	22.2	57.6	0.213	0.723	0.366	0.58
62.0	120.0	1	19.0	60.0	1	28	59	0.588	22.1	57.6	0.212	0.719	0.365	0.58
63.0	120.0	1	24.0	65.0	0	25		0.585	25.8	57.6	~	0.714	0.364	~
64.0	120.0	1	24.0	65.0	0	25		0.583	25.7	57.6	~	0.710	0.363	~
65.0	120.0	1	24.0	65.0	0	25		0.580	25.6	57.6	~	0.705	0.362	~
66.0	120.0	1	24.0	65.0	0	25		0.577	25.5	57.6	~	0.701	0.361	~
67.0	120.0	1	47.0	70.0	1		88	0.574	40.5	57.6	Infin.	0.696	0.360	Non-Liq.
68.0	120.0	1	47.0	70.0	1		88	0.572	40.3	57.6	Infin.	0.691	0.359	Non-Liq.
69.0	120.0	1	47.0	70.0	1		88	0.569	40.1	57.6	Infin.	0.687	0.358	Non-Liq.
70.0	120.0	1	47.0	70.0	1		88	0.567	40.0	57.6	Infin.	0.682	0.357	Non-Liq.
71.0	120.0	1	47.0	70.0	1		88	0.564	39.8	57.6	Infin.	0.678	0.356	Non-Liq.
72.0	120.0	1	26.0	75.0	0	30		0.562	27.7	57.6	~	0.673	0.355	~
73.0	120.0	1	26.0	75.0	0	30		0.559	27.6	57.6	~	0.669	0.353	~
74.0	120.0	1	26.0	75.0	0	30		0.557	27.5	57.6	~	0.664	0.352	~
75.0	120.0	1	26.0	75.0	0	30		0.555	27.4	57.6	~	0.659	0.351	~
76.0	120.0	1	26.0	75.0	0	30		0.552	27.3	57.6	~	0.655	0.349	~
77.0	120.0	1	26.0	75.0	0	30		0.550	27.2	57.6	~	0.650	0.348	~
78.0	120.0	1	26.0	75.0	0	30		0.548	27.1	57.6	~	0.646	0.346	~
79.0	120.0	1	26.0	75.0	0	30		0.545	27.0	57.6	~	0.641	0.345	~
80.0	120.0	1	33.0	80.0	0	44		0.543	33.9	57.6	~	0.637	0.343	~
81.0	120.0	1	33.0	80.0	0	44		0.541	33.8	57.6	~	0.632	0.342	~

Figure 11

GEOCON

Client : Plus Development
File No. : A9899-06-01
Boring : 2

LIQUEFACTION SETTLEMENT ANALYSIS
DESIGN EARTHQUAKE

(SATURATED SAND AT INITIAL LIQUEFACTION CONDITION)

NCEER (1996) METHOD
EARTHQUAKE INFORMATION:

Earthquake Magnitude:	6.72
PGAM (g):	1.002
2/3 PGAM (g):	0.67
Calculated Mag.Wtg.Factor:	0.759
Historic High Groundwater:	20.0
Groundwater @ Exploration:	40.5

DEPTH TO BASE	BLOW COUNT N	WET DENSITY (PCF)	TOTAL STRESS O (TSF)	EFFECT STRESS O' (TSF)	REL. DEN. Dr (%)	ADJUST BLOWS (N1)60	Tav/σ'_o	LIQUEFACTION SAFETY FACTOR	Volumetric Strain $[e_{15}\}$ (%)	EQ. SETTLE. Pe (in.)
1	6	120	0.030	0.030	59	11	0.434	--	0.00	0.00
2	6	120	0.090	0.090	58	11	0.434	--	0.00	0.00
3	6	120	0.150	0.150	56	11	0.434	--	0.00	0.00
4	2	120	0.210	0.210	32	4	0.434	--	0.00	0.00
5	2	120	0.270	0.270	31	4	0.434	--	0.00	0.00
6	2	120	0.330	0.330	30	4	0.434	--	0.00	0.00
7	2	120	0.390	0.390	29	4	0.434	--	0.00	0.00
8	4	120	0.450	0.450	41	7	0.434	--	0.00	0.00
9	4	120	0.510	0.510	40	6	0.434	--	0.00	0.00
10	4	120	0.570	0.570	39	6	0.434	--	0.00	0.00
11	4	120	0.630	0.630	39	6	0.434	--	0.00	0.00
12	9	120	0.690	0.690	54	13	0.434	--	0.00	0.00
13	9	120	0.750	0.750	54	13	0.434	--	0.00	0.00
14	9	120	0.810	0.810	54	12	0.434	--	0.00	0.00
15	9	120	0.870	0.870	54	12	0.434	--	0.00	0.00
16	9	120	0.930	0.930	54	12	0.434	--	0.00	0.00
17	9	120	0.990	0.990	54	11	0.434	--	0.00	0.00
18	31	120	1.050	1.050	93	41	0.434	--	0.00	0.00
19	31	120	1.110	1.110	93	40	0.434	--	0.00	0.00
20	31	120	1.170	1.170	93	39	0.434	--	0.00	0.00
21	31	120	1.230	1.214	93	38	0.440	Non-Liq.	0.00	0.00
22	31	120	1.290	1.243	93	37	0.451	Non-Liq.	0.00	0.00
23	31	120	1.350	1.272	93	37	0.461	Non-Liq.	0.00	0.00
24	35	120	1.410	1.301	95	43	0.471	Non-Liq.	0.00	0.00
25	35	120	1.470	1.330	95	42	0.480	Non-Liq.	0.00	0.00
26	35	120	1.530	1.358	95	41	0.489	Non-Liq.	0.00	0.00
27	35	120	1.590	1.387	95	41	0.498	Non-Liq.	0.00	0.00
28	35	120	1.650	1.416	95	40	0.506	Non-Liq.	0.00	0.00
29	35	120	1.710	1.445	95	39	0.514	Non-Liq.	0.00	0.00
30	25	120	1.770	1.474	78	36	0.522	Non-Liq.	0.00	0.00
31	25	120	1.830	1.502	78	35	0.529	Non-Liq.	0.00	0.00
32	25	120	1.890	1.531	78	35	0.536	Non-Liq.	0.00	0.00
33	25	120	1.950	1.560	78	34	0.543	Non-Liq.	0.00	0.00
34	25	120	2.010	1.589	78	34	0.550	Non-Liq.	0.00	0.00
35	18	120	2.070	1.618		26	0.556	~	0.00	0.00
36	18	120	2.130	1.646		26	0.562	~	0.00	0.00
37	18	120	2.190	1.675		26	0.568	~	0.00	0.00
38	18	120	2.250	1.704		25	0.574	~	0.00	0.00
39	18	120	2.310	1.733		25	0.579	~	0.00	0.00
40	15	120	2.370	1.762		22	0.584	~	0.00	0.00
41	15	120	2.430	1.790		22	0.590	~	0.00	0.00
42	15	120	2.490	1.819		22	0.595	~	0.00	0.00
43	25	120	2.550	1.848	72	31	0.599	Non-Liq.	0.00	0.00
44	25	120	2.610	1.877	72	31	0.604	Non-Liq.	0.00	0.00
45	25	120	2.670	1.906	72	31	0.609	Non-Liq.	0.00	0.00
46	25	120	2.730	1.934	72	31	0.613	Non-Liq.	0.00	0.00
47	25	120	2.790	1.963	72	31	0.617	Non-Liq.	0.00	0.00
48	25	120	2.850	1.992	72	31	0.622	Non-Liq.	0.00	0.00
49	25	120	2.910	2.021	72	31	0.626	Non-Liq.	0.00	0.00
50	20	120	2.970	2.050		19	0.629	~	0.00	0.00
51	20	120	3.030	2.078		19	0.633	~	0.00	0.00
52	20	120	3.090	2.107		19	0.637	~	0.00	0.00
53	20	120	3.150	2.136		18	0.641	~	0.00	0.00
54	20	120	3.210	2.165		18	0.644	~	0.00	0.00
55	26	120	3.270	2.194	70	31	0.648	Non-Liq.	0.00	0.00
56	26	120	3.330	2.222	70	31	0.651	Non-Liq.	0.00	0.00
57	26	120	3.390	2.251	70	31	0.654	Non-Liq.	0.00	0.00
58	26	120	3.450	2.280	70	30	0.657	Non-Liq.	0.00	0.00
59	26	120	3.510	2.309	70	30	0.660	Non-Liq.	0.00	0.00
60	19	120	3.570	2.338	59	22	0.663	0.58	1.40	0.17
61	19	120	3.630	2.366	59	22	0.666	0.58	1.40	0.17
62	19	120	3.690	2.395	59	22	0.669	0.58	1.40	0.17
63	24	120	3.750	2.424		26	0.672	~	0.00	0.00
64	24	120	3.810	2.453		26	0.675	~	0.00	0.00
65	24	120	3.870	2.482		26	0.677	~	0.00	0.00
66	24	120	3.930	2.510		26	0.680	~	0.00	0.00
67	47	120	3.990	2.539	88	41	0.683	Non-Liq.	0.00	0.00
68	47	120	4.050	2.568	88	40	0.685	Non-Liq.	0.00	0.00
69	47	120	4.110	2.597	88	40	0.688	Non-Liq.	0.00	0.00
70	47	120	4.170	2.626	88	40	0.690	Non-Liq.	0.00	0.00
71	47	120	4.230	2.654	88	40	0.692	Non-Liq.	0.00	0.00
72	26	120	4.290	2.683		28	0.695	~	0.00	0.00
73	26	120	4.350	2.712		28	0.697	~	0.00	0.00
74	26	120	4.410	2.741		27	0.699	~	0.00	0.00
75	26	120	4.470	2.770		27	0.701	~	0.00	0.00
76	26	120	4.530	2.798		27	0.703	~	0.00	0.00
77	26	120	4.590	2.827		27	0.705	~	0.00	0.00
78	26	120	4.650	2.856		27	0.707	~	0.00	0.00
79	26	120	4.710	2.885		27	0.709	~	0.00	0.00
80	33	120	4.770	2.914		34	0.711	~	0.00	0.00

TOTAL SETTLEMENT = 0.5 INCHES

Figure 12

Client : Plus Development
File No. : A9899-06-01
Boring : 2

EMPIRICAL ESTIMATION OF LIQUEFACTION POTENTIAL
MAXIMUM CONSIDERED EARTHQUAKE

NCEER (1996) METHOD

By Thomas F. Blake (1994-1996)

EARTHQUAKE INFORMATION:

Earthquake Magnitude:	6.86
Peak Horiz. Acceleration PGA$_M$ (g):	1.002
Calculated Mag.Wtg.Factor:	0.800
Historic High Groundwater:	20.0
Groundwater Depth During Exploration:	40.5

ENERGY & ROD CORRECTIONS:

Energy Correction (CE) for N60:	1.25
Rod Len.Corr.(CR)(0-no or 1-yes):	1.0
Bore Dia. Corr. (CB):	1.00
Sampler Corr. (CS):	1.20
Use Ksigma (0 or 1):	1.0

LIQUEFACTION CALCULATIONS:

Unit Wt. Water (pcf):	62.4

Depth to Base (ft)	Total Unit Wt. (pcf)	Water (0 or 1)	FIELD SPT (N)	Depth of SPT (ft)	Liq.Sus. (0 or 1)	-200 (%)	Est. Dr (%)	CN Factor	Corrected (N1)60	Eff. Unit Wt. (psf)	Resist. CRR	rd Factor	Induced CSR	Liquefac. Safe.Fact.
1.0	120.0	0	6.0	1.0	1		59	1.700	11.5	120.0	0.125	0.998	0.520	--
2.0	120.0	0	6.0	2.0	1		58	1.700	11.5	120.0	0.125	0.993	0.517	--
3.0	120.0	0	6.0	3.0	1		56	1.700	11.5	120.0	0.125	0.989	0.515	--
4.0	120.0	0	2.0	4.0	1		32	1.700	3.8	120.0	0.060	0.984	0.513	--
5.0	120.0	0	2.0	5.0	1		31	1.700	3.8	120.0	0.060	0.979	0.510	--
6.0	120.0	0	2.0	6.0	1		30	1.700	3.8	120.0	0.060	0.975	0.508	--
7.0	120.0	0	2.0	7.0	1		29	1.636	3.7	120.0	0.059	0.970	0.505	--
8.0	120.0	0	4.0	8.0	1		41	1.523	6.9	120.0	0.080	0.966	0.503	--
9.0	120.0	0	4.0	9.0	1		40	1.431	6.4	120.0	0.076	0.961	0.501	--
10.0	120.0	0	4.0	10.0	1		39	1.353	6.1	120.0	0.074	0.957	0.498	--
11.0	120.0	0	4.0	10.0	1		39	1.287	5.8	120.0	0.071	0.952	0.496	--
12.0	120.0	0	9.0	15.0	1		54	1.230	13.4	120.0	0.146	0.947	0.493	--
13.0	120.0	0	9.0	15.0	1		54	1.180	12.8	120.0	0.140	0.943	0.491	--
14.0	120.0	0	9.0	15.0	1		54	1.135	12.4	120.0	0.135	0.938	0.489	--
15.0	120.0	0	9.0	15.0	1		54	1.095	11.9	120.0	0.130	0.934	0.486	--
16.0	120.0	0	9.0	15.0	1		54	1.060	11.5	120.0	0.126	0.929	0.484	--
17.0	120.0	0	9.0	15.0	1		54	1.027	11.2	120.0	0.122	0.925	0.482	--
18.0	120.0	0	31.0	20.0	1		93	0.997	41.5	120.0	Infin.	0.920	0.479	--
19.0	120.0	0	31.0	20.0	1		93	0.970	40.4	120.0	Infin.	0.915	0.477	--
20.0	120.0	0	31.0	20.0	1		93	0.945	39.3	120.0	Infin.	0.911	0.474	--
21.0	120.0	1	31.0	20.0	1		93	0.921	38.3	57.6	Infin.	0.906	0.478	Non-Liq.
22.0	120.0	1	31.0	20.0	1		93	0.900	37.4	57.6	Infin.	0.902	0.487	Non-Liq.
23.0	120.0	1	31.0	20.0	1		93	0.879	36.6	57.6	Infin.	0.897	0.496	Non-Liq.
24.0	120.0	1	35.0	25.0	1		95	0.860	43.2	57.6	Infin.	0.893	0.504	Non-Liq.
25.0	120.0	1	35.0	25.0	1		95	0.843	42.3	57.6	Infin.	0.888	0.511	Non-Liq.
26.0	120.0	1	35.0	25.0	1		95	0.826	41.4	57.6	Infin.	0.883	0.518	Non-Liq.
27.0	120.0	1	35.0	25.0	1		95	0.810	40.6	57.6	Infin.	0.879	0.525	Non-Liq.
28.0	120.0	1	35.0	25.0	1		95	0.795	39.9	57.6	Infin.	0.874	0.531	Non-Liq.
29.0	120.0	1	35.0	25.0	1		95	0.781	39.2	57.6	Infin.	0.870	0.536	Non-Liq.
30.0	120.0	1	25.0	30.0	1	36	78	0.768	35.8	57.6	Infin.	0.865	0.541	Non-Liq.
31.0	120.0	1	25.0	30.0	1	36	78	0.755	35.3	57.6	Infin.	0.861	0.546	Non-Liq.
32.0	120.0	1	25.0	30.0	1	36	78	0.743	34.9	57.6	Infin.	0.856	0.550	Non-Liq.
33.0	120.0	1	25.0	30.0	1	36	78	0.732	34.4	57.6	Infin.	0.851	0.554	Non-Liq.
34.0	120.0	1	25.0	30.0	1	36	78	0.721	34.0	57.6	Infin.	0.847	0.558	Non-Liq.
35.0	120.0	1	18.0	35.0	0	39		0.710	26.2	57.6	~	0.842	0.561	~
36.0	120.0	1	18.0	35.0	0	39		0.700	25.9	57.6	~	0.838	0.564	~
37.0	120.0	1	18.0	35.0	0	39		0.690	25.6	57.6	~	0.833	0.567	~
38.0	120.0	1	18.0	35.0	0	39		0.681	25.4	57.6	~	0.829	0.570	~
39.0	120.0	1	18.0	35.0	0	39		0.672	25.2	57.6	~	0.824	0.572	~
40.0	120.0	1	15.0	40.0	0	39		0.664	21.9	57.6	~	0.819	0.574	~
41.0	120.0	1	15.0	40.0	0	39		0.658	21.8	57.6	~	0.815	0.576	~
42.0	120.0	1	15.0	40.0	0	39		0.654	21.7	57.6	~	0.810	0.578	~
43.0	120.0	1	25.0	45.0	1	42	72	0.650	31.4	57.6	Infin.	0.806	0.579	Non-Liq.
44.0	120.0	1	25.0	45.0	1	42	72	0.646	31.2	57.6	Infin.	0.801	0.580	Non-Liq.
45.0	120.0	1	25.0	45.0	1	42	72	0.642	31.1	57.6	Infin.	0.797	0.581	Non-Liq.
46.0	120.0	1	25.0	45.0	1	42	72	0.639	31.0	57.6	Infin.	0.792	0.582	Non-Liq.
47.0	120.0	1	25.0	45.0	1	42	72	0.635	30.8	57.6	Infin.	0.787	0.583	Non-Liq.
48.0	120.0	1	25.0	45.0	1	42	72	0.632	30.7	57.6	Infin.	0.783	0.583	Non-Liq.
49.0	120.0	1	25.0	45.0	1	42	72	0.628	30.6	57.6	Infin.	0.778	0.584	Non-Liq.
50.0	120.0	1	20.0	50.0	0			0.625	18.7	57.6	~	0.774	0.584	~
51.0	120.0	1	20.0	50.0	0			0.622	18.6	57.6	~	0.769	0.584	~
52.0	120.0	1	20.0	50.0	0			0.618	18.5	57.6	~	0.765	0.584	~
53.0	120.0	1	20.0	50.0	0			0.615	18.5	57.6	~	0.760	0.584	~
54.0	120.0	1	20.0	50.0	0			0.612	18.4	57.6	~	0.755	0.583	~
55.0	120.0	1	26.0	55.0	1	39	70	0.609	30.7	57.6	Infin.	0.751	0.583	Non-Liq.
56.0	120.0	1	26.0	55.0	1	39	70	0.606	30.6	57.6	Infin.	0.746	0.582	Non-Liq.
57.0	120.0	1	26.0	55.0	1	39	70	0.603	30.5	57.6	Infin.	0.742	0.582	Non-Liq.
58.0	120.0	1	26.0	55.0	1	39	70	0.600	30.4	57.6	Infin.	0.737	0.581	Non-Liq.
59.0	120.0	1	26.0	55.0	1	39	70	0.597	30.3	57.6	Infin.	0.733	0.580	Non-Liq.
60.0	120.0	1	19.0	60.0	1	28	59	0.594	22.3	57.6	0.214	0.728	0.579	0.37
61.0	120.0	1	19.0	60.0	1	28	59	0.591	22.2	57.6	0.213	0.723	0.578	0.37
62.0	120.0	1	19.0	60.0	1	28	59	0.588	22.1	57.6	0.212	0.719	0.577	0.37
63.0	120.0	1	24.0	65.0	0	25		0.585	25.8	57.6	~	0.714	0.576	~
64.0	120.0	1	24.0	65.0	0	25		0.583	25.7	57.6	~	0.710	0.574	~
65.0	120.0	1	24.0	65.0	0	25		0.580	25.6	57.6	~	0.705	0.573	~
66.0	120.0	1	24.0	65.0	0	25		0.577	25.5	57.6	~	0.701	0.571	~
67.0	120.0	1	47.0	70.0	1		88	0.574	40.5	57.6	Infin.	0.696	0.570	Non-Liq.
68.0	120.0	1	47.0	70.0	1		88	0.572	40.3	57.6	Infin.	0.691	0.568	Non-Liq.
69.0	120.0	1	47.0	70.0	1		88	0.569	40.1	57.6	Infin.	0.687	0.566	Non-Liq.
70.0	120.0	1	47.0	70.0	1		88	0.567	40.0	57.6	Infin.	0.682	0.564	Non-Liq.
71.0	120.0	1	47.0	70.0	1		88	0.564	39.8	57.6	Infin.	0.678	0.563	Non-Liq.
72.0	120.0	1	26.0	75.0	0	30		0.562	27.7	57.6	~	0.673	0.561	~
73.0	120.0	1	26.0	75.0	0	30		0.559	27.6	57.6	~	0.669	0.559	~
74.0	120.0	1	26.0	75.0	0	30		0.557	27.5	57.6	~	0.664	0.556	~
75.0	120.0	1	26.0	75.0	0	30		0.555	27.4	57.6	~	0.659	0.554	~
76.0	120.0	1	26.0	75.0	0	30		0.552	27.3	57.6	~	0.655	0.552	~
77.0	120.0	1	26.0	75.0	0	30		0.550	27.2	57.6	~	0.650	0.550	~
78.0	120.0	1	26.0	75.0	0	30		0.548	27.1	57.6	~	0.646	0.548	~
79.0	120.0	1	26.0	75.0	0	30		0.545	27.0	57.6	~	0.641	0.545	~
80.0	120.0	1	33.0	80.0	0	44		0.543	33.9	57.6	~	0.637	0.543	~
81.0	120.0	1	33.0	80.0	0	44		0.541	33.8	57.6	~	0.632	0.540	~

Figure 13

GEOCON

Client : Plus Development
File No. : A9899-06-01
Boring : 2

LIQUEFACTION SETTLEMENT ANALYSIS
MAXIMUM CONSIDERED EARTHQUAKE

(SATURATED SAND AT INITIAL LIQUEFACTION CONDITION)

6.86

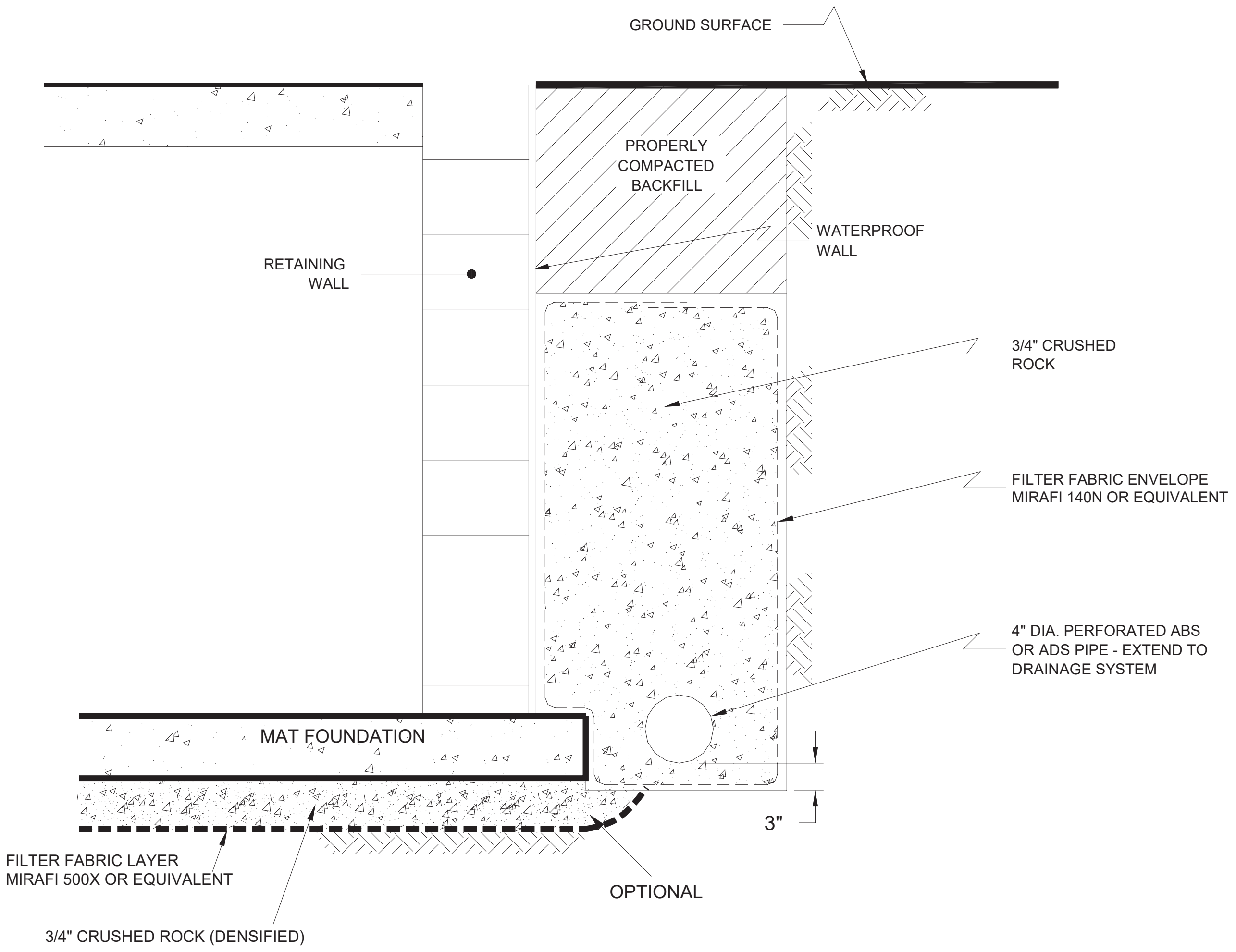
NCEER (1996) METHOD 1.002

EARTHQUAKE INFORMATION:

Earthquake Magnitude:	6.86
PGA_M (g):	1.002
Calculated Mag.Wtg.Factor:	0.800
Historic High Groundwater:	20.0
Groundwater @ Exploration:	40.5

DEPTH TO BASE	BLOW COUNT N	WET DENSITY (PCF)	TOTAL STRESS O (TSF)	EFFECT STRESS O' (TSF)	REL. DEN. Dr (%)	ADJUST BLOWS (N1)60	Tav/σ'_o	LIQUEFACTION SAFETY FACTOR	Volumetric Strain [e_{15}} (%)	EQ. SETTLE. Pe (in.)
1	6	120	0.030	0.030	59	11	0.651	--	0.00	0.00
2	6	120	0.090	0.090	58	11	0.651	--	0.00	0.00
3	6	120	0.150	0.150	56	11	0.651	--	0.00	0.00
4	2	120	0.210	0.210	32	4	0.651	--	0.00	0.00
5	2	120	0.270	0.270	31	4	0.651	--	0.00	0.00
6	2	120	0.330	0.330	30	4	0.651	--	0.00	0.00
7	2	120	0.390	0.390	29	4	0.651	--	0.00	0.00
8	4	120	0.450	0.450	41	7	0.651	--	0.00	0.00
9	4	120	0.510	0.510	40	6	0.651	--	0.00	0.00
10	4	120	0.570	0.570	39	6	0.651	--	0.00	0.00
11	4	120	0.630	0.630	39	6	0.651	--	0.00	0.00
12	9	120	0.690	0.690	54	13	0.651	--	0.00	0.00
13	9	120	0.750	0.750	54	13	0.651	--	0.00	0.00
14	9	120	0.810	0.810	54	12	0.651	--	0.00	0.00
15	9	120	0.870	0.870	54	12	0.651	--	0.00	0.00
16	9	120	0.930	0.930	54	12	0.651	--	0.00	0.00
17	9	120	0.990	0.990	54	11	0.651	--	0.00	0.00
18	31	120	1.050	1.050	93	41	0.651	--	0.00	0.00
19	31	120	1.110	1.110	93	40	0.651	--	0.00	0.00
20	31	120	1.170	1.170	93	39	0.651	--	0.00	0.00
21	31	120	1.230	1.214	93	38	0.660	Non-Liq.	0.00	0.00
22	31	120	1.290	1.243	93	37	0.676	Non-Liq.	0.00	0.00
23	31	120	1.350	1.272	93	37	0.691	Non-Liq.	0.00	0.00
24	35	120	1.410	1.301	95	43	0.706	Non-Liq.	0.00	0.00
25	35	120	1.470	1.330	95	42	0.720	Non-Liq.	0.00	0.00
26	35	120	1.530	1.358	95	41	0.734	Non-Liq.	0.00	0.00
27	35	120	1.590	1.387	95	41	0.747	Non-Liq.	0.00	0.00
28	35	120	1.650	1.416	95	40	0.759	Non-Liq.	0.00	0.00
29	35	120	1.710	1.445	95	39	0.771	Non-Liq.	0.00	0.00
30	25	120	1.770	1.474	78	36	0.782	Non-Liq.	0.00	0.00
31	25	120	1.830	1.502	78	35	0.793	Non-Liq.	0.00	0.00
32	25	120	1.890	1.531	78	35	0.804	Non-Liq.	0.00	0.00
33	25	120	1.950	1.560	78	34	0.814	Non-Liq.	0.00	0.00
34	25	120	2.010	1.589	78	34	0.824	Non-Liq.	0.00	0.00
35	18	120	2.070	1.618		26	0.833	~	0.00	0.00
36	18	120	2.130	1.646		26	0.843	~	0.00	0.00
37	18	120	2.190	1.675		26	0.851	~	0.00	0.00
38	18	120	2.250	1.704		25	0.860	~	0.00	0.00
39	18	120	2.310	1.733		25	0.868	~	0.00	0.00
40	15	120	2.370	1.762		22	0.876	~	0.00	0.00
41	15	120	2.430	1.790		22	0.884	~	0.00	0.00
42	15	120	2.490	1.819		22	0.891	~	0.00	0.00
43	25	120	2.550	1.848	72	31	0.899	Non-Liq.	0.00	0.00
44	25	120	2.610	1.877	72	31	0.906	Non-Liq.	0.00	0.00
45	25	120	2.670	1.906	72	31	0.913	Non-Liq.	0.00	0.00
46	25	120	2.730	1.934	72	31	0.919	Non-Liq.	0.00	0.00
47	25	120	2.790	1.963	72	31	0.926	Non-Liq.	0.00	0.00
48	25	120	2.850	1.992	72	31	0.932	Non-Liq.	0.00	0.00
49	25	120	2.910	2.021	72	31	0.938	Non-Liq.	0.00	0.00
50	20	120	2.970	2.050		19	0.944	~	0.00	0.00
51	20	120	3.030	2.078		19	0.949	~	0.00	0.00
52	20	120	3.090	2.107		19	0.955	~	0.00	0.00
53	20	120	3.150	2.136		18	0.960	~	0.00	0.00
54	20	120	3.210	2.165		18	0.966	~	0.00	0.00
55	26	120	3.270	2.194	70	31	0.971	Non-Liq.	0.00	0.00
56	26	120	3.330	2.222	70	31	0.976	Non-Liq.	0.00	0.00
57	26	120	3.390	2.251	70	31	0.981	Non-Liq.	0.00	0.00
58	26	120	3.450	2.280	70	30	0.986	Non-Liq.	0.00	0.00
59	26	120	3.510	2.309	70	30	0.990	Non-Liq.	0.00	0.00
60	19	120	3.570	2.338	59	22	0.995	0.37	1.40	0.17
61	19	120	3.630	2.366	59	22	0.999	0.37	1.40	0.17
62	19	120	3.690	2.395	59	22	1.003	0.37	1.40	0.17
63	24	120	3.750	2.424		26	1.008	~	0.00	0.00
64	24	120	3.810	2.453		26	1.012	~	0.00	0.00
65	24	120	3.870	2.482		26	1.016	~	0.00	0.00
66	24	120	3.930	2.510		26	1.020	~	0.00	0.00
67	47	120	3.990	2.539	88	41	1.023	Non-Liq.	0.00	0.00
68	47	120	4.050	2.568	88	40	1.027	Non-Liq.	0.00	0.00
69	47	120	4.110	2.597	88	40	1.031	Non-Liq.	0.00	0.00
70	47	120	4.170	2.626	88	40	1.034	Non-Liq.	0.00	0.00
71	47	120	4.230	2.654	88	40	1.038	Non-Liq.	0.00	0.00
72	26	120	4.290	2.683		28	1.041	~	0.00	0.00
73	26	120	4.350	2.712		28	1.045	~	0.00	0.00
74	26	120	4.410	2.741		27	1.048	~	0.00	0.00
75	26	120	4.470	2.770		27	1.051	~	0.00	0.00
76	26	120	4.530	2.798		27	1.054	~	0.00	0.00
77	26	120	4.590	2.827		27	1.057	~	0.00	0.00
78	26	120	4.650	2.856		27	1.060	~	0.00	0.00
79	26	120	4.710	2.885		27	1.063	~	0.00	0.00
80	33	120	4.770	2.914		34	1.066	~	0.00	0.00
81	33	120	4.830	2.942		34	1.069	~	0.00	0.00

TOTAL SETTLEMENT = 0.5 INCHES



NO SCALE

GEOCON
W E S T, I N C.

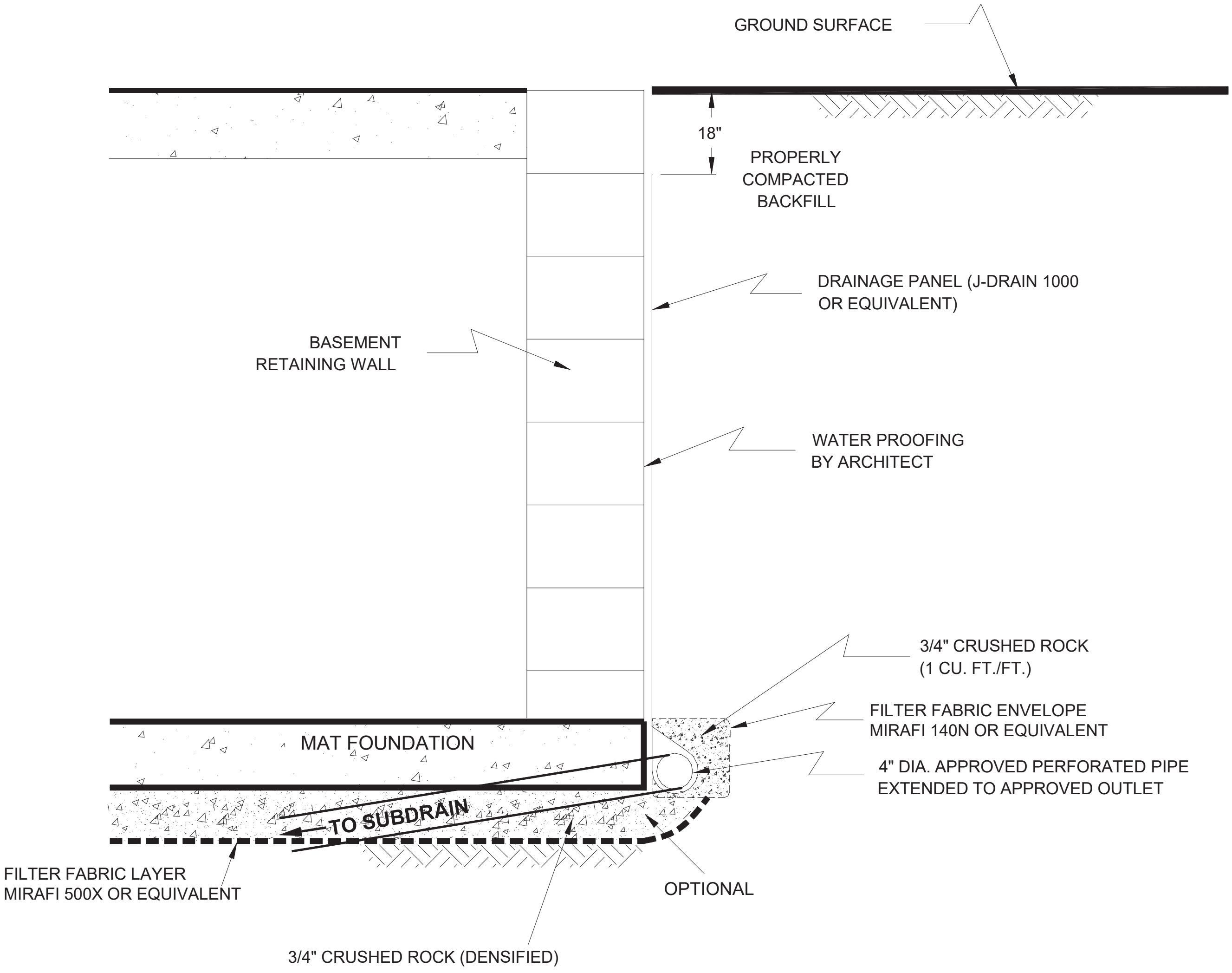
ENVIRONMENTAL GEOTECHNICAL MATERIALS
3303 N. SAN FERNANDO BLVD. - SUITE 100 - BURBANK, CA 91504
PHONE (818) 841-8388 - FAX (818) 841-1704

Drafted by: JTA | Checked by: NDB

RETAINING WALL DRAIN DETAIL

8850 SUNSET BOULEVARD
WEST HOLLYWOOD, CALIFORNIA

FEB. 2021 | PROJECT NO. A9899-06-01 | FIG. 15



NOTE: TOP OF DRAINAGE PANEL NOT MORE THAN
18 INCHES FROM GROUND SURFACE

NO SCALE

GEOCON
WEST, INC.

ENVIRONMENTAL GEOTECHNICAL MATERIALS
3303 N. SAN FERNANDO BLVD. - SUITE 100 - BURBANK, CA 91504
PHONE (818) 841-8388 - FAX (818) 841-1704

Drafted by: JTA | Checked by: NDB

RETAINING WALL DRAIN DETAIL

8850 SUNSET BOULEVARD
WEST HOLLYWOOD, CALIFORNIA

FEB. 2021 | PROJECT NO. A9899-06-01 | FIG. 16

APPENDIX A

APPENDIX A

FIELD INVESTIGATION

The site was explored on November 20 and November 21, 2018 by excavating three 8-inch-diameter borings using a truck-mounted hollow-stem auger drilling machine. The borings were excavated to depths of approximately 60½ and 80½ feet below the existing ground surface. Representative and relatively undisturbed samples were obtained by driving a 3 inch, O. D., California Modified Sampler into the “undisturbed” soil mass with blows from a 140-pound auto-hammer falling 30 inches. The California Modified Sampler was equipped with 1-inch high by 2 $^{3}/_{8}$-inch diameter brass sampler rings to facilitate soil removal and testing. Bulk samples were collected and Standard Penetration Tests (SPTs) were also performed.

The soil conditions encountered in the borings were visually examined, classified and logged in general accordance with the Unified Soil Classification System (USCS). The logs of the borings are presented on Figures A1 through A3. The logs depict the soil and geologic conditions encountered and the depths at which samples were obtained. The logs also include our interpretation of the conditions between sampling intervals. Therefore, the logs contain both observed and interpreted data. We determined the lines designating the interface between soil materials on the logs using visual observations, penetration rates, excavation characteristics and other factors. The transition between materials may be abrupt or gradual. Where applicable, the boring logs were revised based on subsequent laboratory testing. The locations of the borings are shown on Figures 2A and 2B.

PROJECT NO. A9899-06-01

BORING 1

ELEV. (MSL.) 345 DATE COMPLETED 11/20/18

EQUIPMENT HOLLOW STEM AUGER BY: CB

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0	BULK 0-5'				**AC: 5" BASE: 3.5"** **ARTIFICIAL FILL** Sand, well-graded, loose, slightly moist, yellowish brown, fine- to coarse-grained, some fine gravel.			
2	B1@2.5'					12	107.4	6.1
4					**ALLUVIUM** Silty Sand, loose, slightly moist, brown, fine- to medium-grained, trace calcium carbonate nodules, trace fine gravel.			
6	B1@5'				- reddish brown	9	96.0	9.2
8								
10	B1@10'			SM	- increase in silt	17	111.3	8.6
12								
14								
16	B1@15'				- medium dense	22	107.7	11.0
18								
20	B1@20'				Silty Sand with Clay, medium dense, slightly moist, reddish brown, fine- to medium-grained, trace calcium carbonate nodules, trace fine gravel.	44	125.5	13.6
22								
24				SM				
26	B1@25'				- brown, increase in clay content	51	113.8	16.0
28				SP	Sand with Clay, poorly graded, dense, slightly moist, reddish brown, trace fine gravel, trace silt.			

Figure A1,
Log of Boring 1, Page 1 of 3

SAMPLE SYMBOLS

☐ ... SAMPLING UNSUCCESSFUL
◧ ... STANDARD PENETRATION TEST
■ ... DRIVE SAMPLE (UNDISTURBED)
☒ ... DISTURBED OR BAG SAMPLE
◪ ... CHUNK SAMPLE
▼ ... WATER TABLE OR SEEPAGE

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

PROJECT NO. A9899-06-01

BORING 1

ELEV. (MSL.) **345** DATE COMPLETED **11/20/18**

EQUIPMENT **HOLLOW STEM AUGER** BY: **CB**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
30	B1@30'					77	125.8	13.4
32								
34				SP				
	B1@35'				- loose	12	111.8	19.0
36								
38	BULK 38-43'							
40	B1@40'		▼	SC	Clayey Sand, poorly-graded, loose, wet, reddish brown, very fine- to fine-grained, some silt, trace gravel.	17	112.9	18.6
					Silty Sand, loose, saturated, reddish brown, fine- to coarse-grained.			
42	B1@42'					16	115.1	18.8
44								
	B1@45'					16	117.6	18.6
46				SM				
48								
50	B1@50'				- medium dense, increase in fine-grained	22	120.4	16.0
52								
					Sand with Clay, poorly graded, loose wet, reddish brown, fine- to medium-grianed, some silt, trace fine gravel.			
54								
	B1@55'					30	117.7	17.5
56				SP				
58								

A9899-06-01 BORING LOGS.GPJ

Figure A1,
Log of Boring 1, Page 2 of 3

SAMPLE SYMBOLS
☐ ... SAMPLING UNSUCCESSFUL
◧ ... STANDARD PENETRATION TEST
■ ... DRIVE SAMPLE (UNDISTURBED)
☒ ... DISTURBED OR BAG SAMPLE
◪ ... CHUNK SAMPLE
▼ ... WATER TABLE OR SEEPAGE

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

BORING 1

ELEV. (MSL.) **345** DATE COMPLETED **11/20/18**

EQUIPMENT **HOLLOW STEM AUGER** BY: **CB**

DEPTH IN FEET	SAMPLE NO.		LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
60	B1@60'	■			ML	Sandy Silt with Clay, stiff, wet, reddish brown, fine-to medium-grained, trace gravel.	26	115.5	16.9

Total depth of boring: 60.5 feet
Fill to 3 feet.
Groundwater encountered at 40 feet.
Backfilled with soil cuttings and tamped.
Concrete patched.

*Penetration resistance for 140-pound hammer falling 30 inches by auto-hammer.

Figure A1,
Log of Boring 1, Page 3 of 3

SAMPLE SYMBOLS			
	☐ ... SAMPLING UNSUCCESSFUL	◧ ... STANDARD PENETRATION TEST	■ ... DRIVE SAMPLE (UNDISTURBED)
	☒ ... DISTURBED OR BAG SAMPLE	◪ ... CHUNK SAMPLE	▼ ... WATER TABLE OR SEEPAGE

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

PROJECT NO. A9899-06-01

BORING 2

ELEV. (MSL.) **341** DATE COMPLETED **11/20/18**

EQUIPMENT **HOLLOW STEM AUGER** BY: **CB**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0					**AC: 5" BASE: 3.5"** **ARTIFICIAL FILL** Gravelly Sand, well-graded, loose, dry, brown, fine- to coarse-grained.			
2	B2@2'					14	114.8	7.5
4					**ALLUVIUM** Silty Sand, very loose, dry, brown, fine- to coarse-grained, trace calcium carbonate nodules, trace fine gravel.			
	B2@4.5'			SM		2		
6								
8	B2@7.5'				Silty Sand with Clay, loose, slightly moist, reddish brown, fine- to coarse-grained, trace calcium carbonate nodules, trace fine gravel.	12	112.8	10.4
				SM				
10	B2@9.5'					4		
12				SP-SM	Sand with Silt, poorly graded, loose, slightly moist, reddish brown, fine- to medium-grained, trace calcium carbonate nodules, trace fine gravel.			
	B2@12.5'					17	120.5	10.3
14	B2@14.5'				Silty Sand with Clay, loose, dry, brown, fine- to medium-grained, some calcium carbonate nodules.	9		
16				SM				
18	B2@17.5'			ML	Sandy Silt with Clay, hard, slightly moist, reddish brown, fine- to coarse-grained, trace calcium carbonate nodules.	49	121.5	14.3
20	B2@19.5'				Silty Sand with Clay, medium dense, slightly moist, reddish brown, fine- to coarse-grained, trace gravel.	31		
22				SM				
	B2@22.5'				- no recovery, sandy	46		
24	B2@24.5'				Silty Clay with Sand, hard, slightly moist, dark brown, fine- to medium-grained, trace calcium carbonate and gravel.	35		
26				CL				
28	B2@27.5'				- reddish brown	48	120.1	14.3
				SM	Silty Sand with Clay, medium dense, slightly moist, reddish brown, fine- to			

A9899-06-01 BORING LOGS.GPJ

Figure A2,
Log of Boring 2, Page 1 of 3

SAMPLE SYMBOLS
☐ ... SAMPLING UNSUCCESSFUL
◧ ... STANDARD PENETRATION TEST
■ ... DRIVE SAMPLE (UNDISTURBED)
⊠ ... DISTURBED OR BAG SAMPLE
◪ ... CHUNK SAMPLE
▼ ... WATER TABLE OR SEEPAGE

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

BORING 2

ELEV. (MSL.) **341** DATE COMPLETED **11/20/18**

EQUIPMENT **HOLLOW STEM AUGER** BY: **CB**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
30	B2@29.5'			SM	medium-grained, trace gravel.	25		
32	B2@32.5'					43	127.4	14.2
34	B2@34.5'					18		
36				CL	Sandy Clay with Silt, stiff, moist, reddish brown, fine- to coarse-grained, trace gravel.			
38	B2@37.5'					42	120.3	17.1
40	B2@39.5'		▼		- increase in silt content	15		
42	B2@42.5'			SC	Clayey Sand with Silt, medium dense, wet, reddish brown, fine- to medium-grained, trace gravel.	31	122.5	16.1
44	B2@44.5'					25		
46								
48	B2@47.5'			ML	Sandy Silt with Clay, stiff, wet, reddish brown, fine- to medium-grained, trace gravel.	36	117.0	17.9
50	B2@49.5'					20		
52				SM	Silty Sand, medium dense, wet, reddish brown, fine- to coarse-grained, trace gravel.			
	B2@52.5'			ML	Sandy Silt with Clay, hard, wet, reddish brown, fine- to coarse-grained.	47	118.9	16.3
54	B2@54.5'					26		
56				SM	Silty Sand, medium dense, wet, reddish brown, fine- to coarse-grained, some clay, trace gravel.			
58	B2@57.5'					40	119.8	16.3
				SC	Clayey Sand with Silt, medium dense, wet, reddish brown, fine- to			

Figure A2,
Log of Boring 2, Page 2 of 3

SAMPLE SYMBOLS

☐ ... SAMPLING UNSUCCESSFUL
◧ ... STANDARD PENETRATION TEST
■ ... DRIVE SAMPLE (UNDISTURBED)
☒ ... DISTURBED OR BAG SAMPLE
◪ ... CHUNK SAMPLE
▼ ... WATER TABLE OR SEEPAGE

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

PROJECT NO. A9899-06-01

BORING 2

ELEV. (MSL.) 341 DATE COMPLETED 11/20/18

EQUIPMENT HOLLOW STEM AUGER BY: CB

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
60	B2@59.5' ■			SC	medium-grained, trace gravel.	19		
62	B2@62.5' ■			SM	Silty Sand, medium dense, wet, reddish brown with greenish gray mottles, fine- to medium-grained, some fine gravel.	49	120.7	15.9
64	B2@64.5' ■					24		
66				SC	Clayey Sand, medium dense, saturated, reddish brown, fine- to medium-grained, some fine gravel.			
68	B2@67.5' ■					45	117.3	17.0
70	B2@69.5' ■				- dense	47		
72	B2@72.5' ■			SM	Silty Sand, dense, wet, reddish brown, fine-grained, some fine gravel.	66	126.7	14.5
74	B2@74.5' ■				- medium dense	26		
76								
78	B2@77.5' ■				- very dense	50 (5")	125.6	15.5
80	B2@79.5' ■				- dense	33		

Total depth of boring: 80.5 feet
Fill to 3 feet.
Groundwater encountered at 40.5 feet.
Backfilled with soil cuttings and tamped.
Concrete patched.

*Penetration resistance for 140-pound hammer falling 30 inches by auto-hammer.

A9899-06-01 BORING LOGS.GPJ

Figure A2,
Log of Boring 2, Page 3 of 3

SAMPLE SYMBOLS

☐ ... SAMPLING UNSUCCESSFUL
◧ ... STANDARD PENETRATION TEST
■ ... DRIVE SAMPLE (UNDISTURBED)
⊠ ... DISTURBED OR BAG SAMPLE
◪ ... CHUNK SAMPLE
▼ ... WATER TABLE OR SEEPAGE

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PROJECT NO. A9899-06-01

BORING 3

ELEV. (MSL.) **350** DATE COMPLETED **11/21/18**

EQUIPMENT **HOLLOW STEM AUGER** BY: **CB**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0					**AC: 6" BASE: NONE** **ARTIFICIAL FILL** Sand with Silt, loose, slightly moist brown with gray mottles, fine- to medium-grained, some clay and gravel, trace carbonate nodules.			
2	B3@2.5'			SW-SM	**ALLUVIUM** Sand with Silt, well-graded, loose, dry, fine- to coarse-grianed, brown, trace carbonate nodules.	10	114.9	6.4
4	B3@5'				- some clay	9	107.1	9.1
6								
8								
10	B3@10'			SM	Silty Sand with Clay, loose, slightly moist, fine- to coarse-grained, reddish brown.	11	114.3	11.1
12								
14	B3@15'				- increase in clay	12	117.2	12.9
16								
18				ML	Sandy Silt with Clay, hard, slightly moist, fine- to medium-grained, reddish brown, trace carbonate nodules.			
20	B3@20'					50	124.0	13.1
22								
24	B3@25'				- trace gravel, increase in clay	44	118.2	16.1
26								
28				ML	Silt with Sand, dense, slightly moist, reddish brown with grayish brown mottles, fine- to medium-grained, some clay, trace gravel.			

Figure A3,
Log of Boring 3, Page 1 of 3

SAMPLE SYMBOLS			
	☐ ... SAMPLING UNSUCCESSFUL	◧ ... STANDARD PENETRATION TEST	■ ... DRIVE SAMPLE (UNDISTURBED)
	☒ ... DISTURBED OR BAG SAMPLE	◪ ... CHUNK SAMPLE	▼ ... WATER TABLE OR SEEPAGE

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PROJECT NO. A9899-06-01

BORING 3

ELEV. (MSL.) **350** DATE COMPLETED **11/21/18**

EQUIPMENT **HOLLOW STEM AUGER** BY: **CB**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
30	B3@30' ■					70	119.4	15.4
32								
34								
	B3@35' ■				- no gravel	56	116.8	16.4
36				ML				
38								
	B3@39' ■				- medium dense, moist	40	119.6	15.7
40								
42	B3@42' ■		▼	ML	Sandy Silt with Clay, stiff, wet, reddish brown, fine- to medium-grained.	30	112.0	18.8
44					Silt with Clay, firm, wet, reddish brown, fine- to medium-grained, trace sand.			
	B3@45' ■			ML		18	100.0	21.1
46					Sandy Silt with Clay, stiff, wet, reddish brown, fine- to medium-grained.			
48	B3@48' ■			ML		41	111.7	17.8
50	B3@50' ■				Silty Sand, medium dense, wet, reddish brown, fine- to medium-grained, trace clay.	46	106.4	18.1
52				SM				
54								
	B3@55' ■				Silty Sand with Clay, medium dense, wet, reddish brown, fine- to medium-grained.	42	116.4	17.3
56								
58				SM				

Figure A3,
Log of Boring 3, Page 2 of 3

A9899-06-01 BORING LOGS.GPJ

SAMPLE SYMBOLS	☐ ... SAMPLING UNSUCCESSFUL	◧ ... STANDARD PENETRATION TEST	■ ... DRIVE SAMPLE (UNDISTURBED)
	☒ ... DISTURBED OR BAG SAMPLE	◪ ... CHUNK SAMPLE	▼ ... WATER TABLE OR SEEPAGE

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PROJECT NO. A9899-06-01

BORING 3

ELEV. (MSL.) **350** DATE COMPLETED **11/21/18**

EQUIPMENT **HOLLOW STEM AUGER** BY: **CB**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
60	B3@60' ■			SM	- no recovery	33		

Total depth of boring: 60.5 feet
Fill to 2 feet.
Groundwater encountered at 42 feet.
Backfilled with soil cuttings and tamped.
Concrete patched.

*Penetration resistance for 140-pound hammer falling 30 inches by auto-hammer.

A9899-06-01 BORING LOGS.GPJ

Figure A3,
Log of Boring 3, Page 3 of 3

SAMPLE SYMBOLS			
	☐ ... SAMPLING UNSUCCESSFUL	◧ ... STANDARD PENETRATION TEST	■ ... DRIVE SAMPLE (UNDISTURBED)
	⊠ ... DISTURBED OR BAG SAMPLE	◪ ... CHUNK SAMPLE	▼ ... WATER TABLE OR SEEPAGE

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

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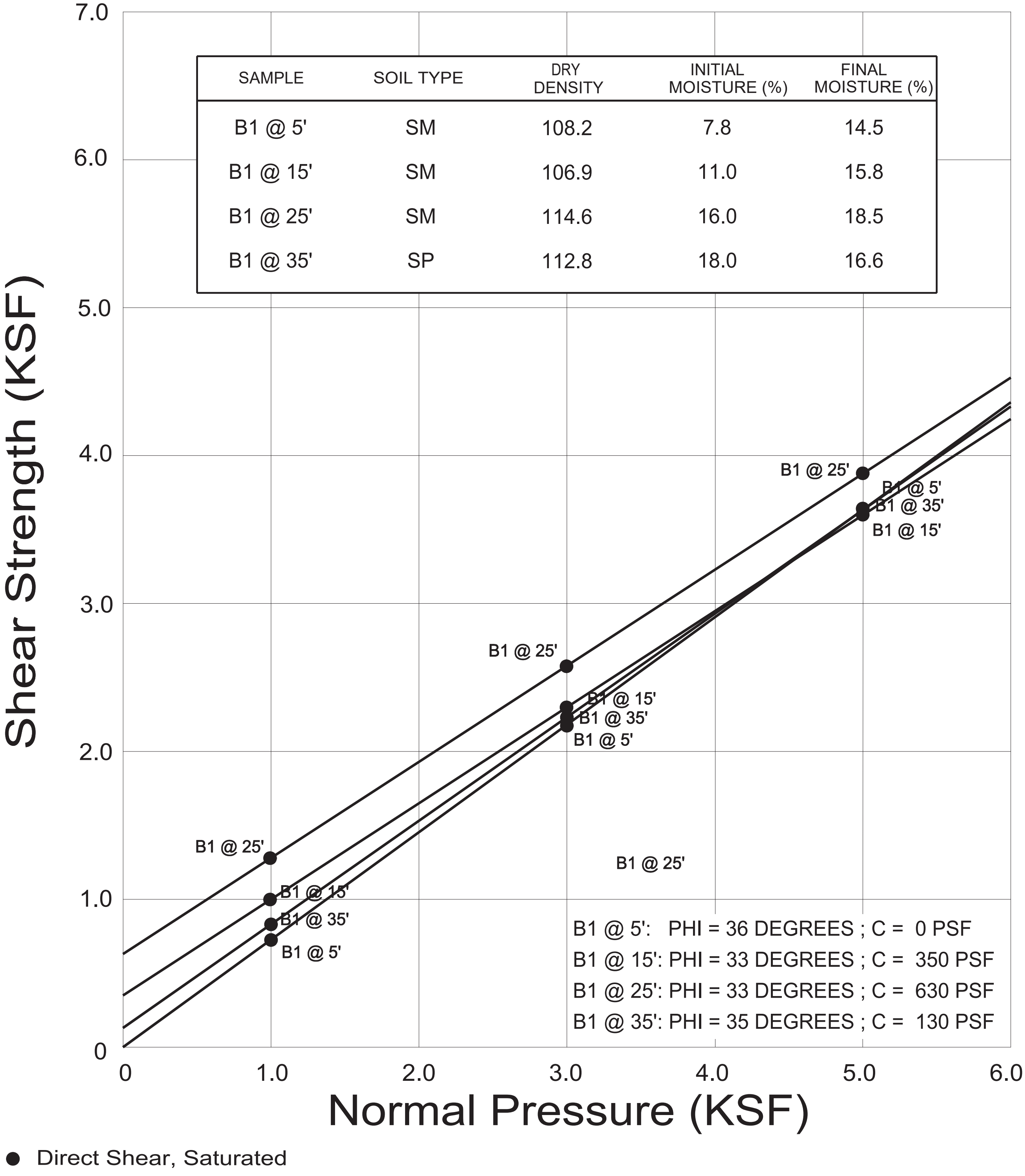
APPENDIX B

APPENDIX B

LABORATORY TESTING

Laboratory tests were performed in accordance with generally accepted test methods of the "American Society for Testing and Materials (ASTM)", or other suggested procedures. Selected samples were tested for direct shear strength, consolidation and expansion characteristics, plasticity indices, grain size distribution, water-soluble sulfates, and in-place dry density and moisture content. The results of the laboratory tests are summarized in Figures B1 through B12. The in-place dry density and moisture content of the samples tested are presented on the boring logs, Appendix A.



SAMPLE	SOIL TYPE	DRY DENSITY	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B1 @ 5'	SM	108.2	7.8	14.5
B1 @ 15'	SM	106.9	11.0	15.8
B1 @ 25'	SM	114.6	16.0	18.5
B1 @ 35'	SP	112.8	18.0	16.6

B1 @ 5': PHI = 36 DEGREES ; C = 0 PSF
B1 @ 15': PHI = 33 DEGREES ; C = 350 PSF
B1 @ 25': PHI = 33 DEGREES ; C = 630 PSF
B1 @ 35': PHI = 35 DEGREES ; C = 130 PSF

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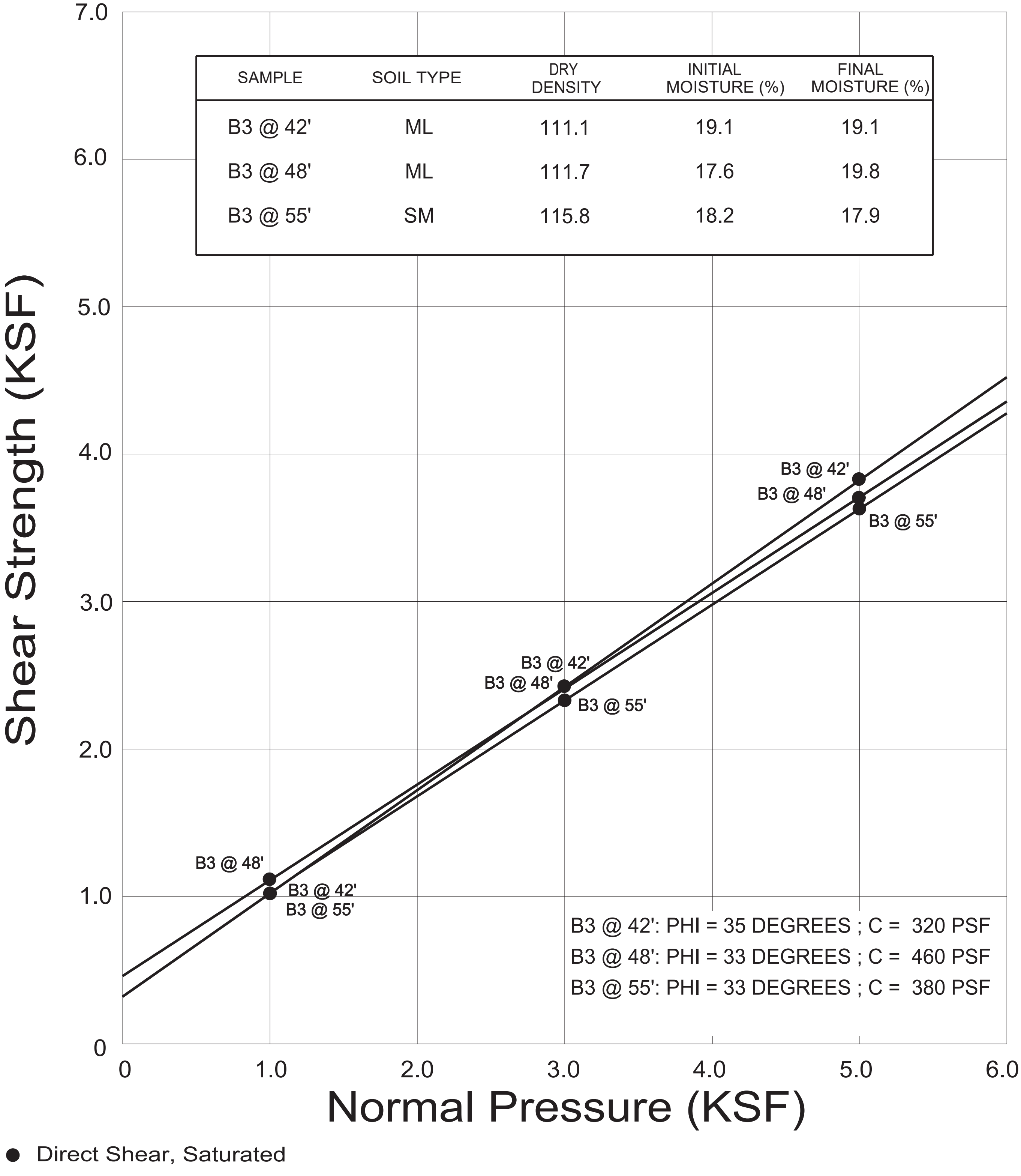
DIRECT SHEAR TEST RESULTS

8850 SUNSET BOULEVARD
WEST HOLLYWOOD, CALIFORNIA

DRAFTED BY: JS	CHECKED BY: JTA	FEB. 2021	PROJECT NO. A9899-06-01	FIG. B1



SAMPLE	SOIL TYPE	DRY DENSITY	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B3 @ 42'	ML	111.1	19.1	19.1
B3 @ 48'	ML	111.7	17.6	19.8
B3 @ 55'	SM	115.8	18.2	17.9

B3 @ 42': PHI = 35 DEGREES ; C = 320 PSF
B3 @ 48': PHI = 33 DEGREES ; C = 460 PSF
B3 @ 55': PHI = 33 DEGREES ; C = 380 PSF

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DIRECT SHEAR TEST RESULTS

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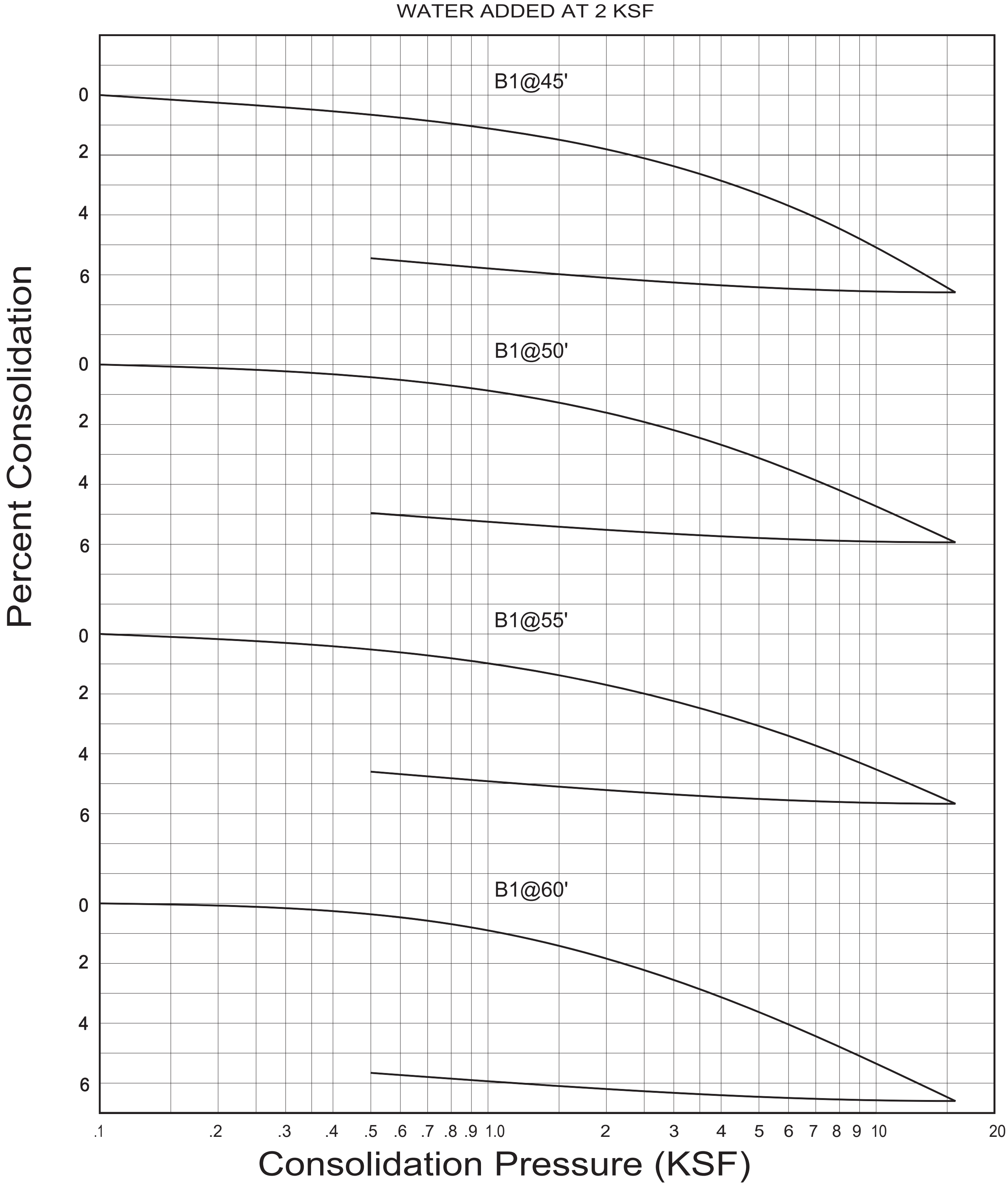
DRAFTED BY: JS	CHECKED BY: JTA	FEB. 2021	PROJECT NO. A9899-06-01	FIG. B2



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CONSOLIDATION TEST RESULTS

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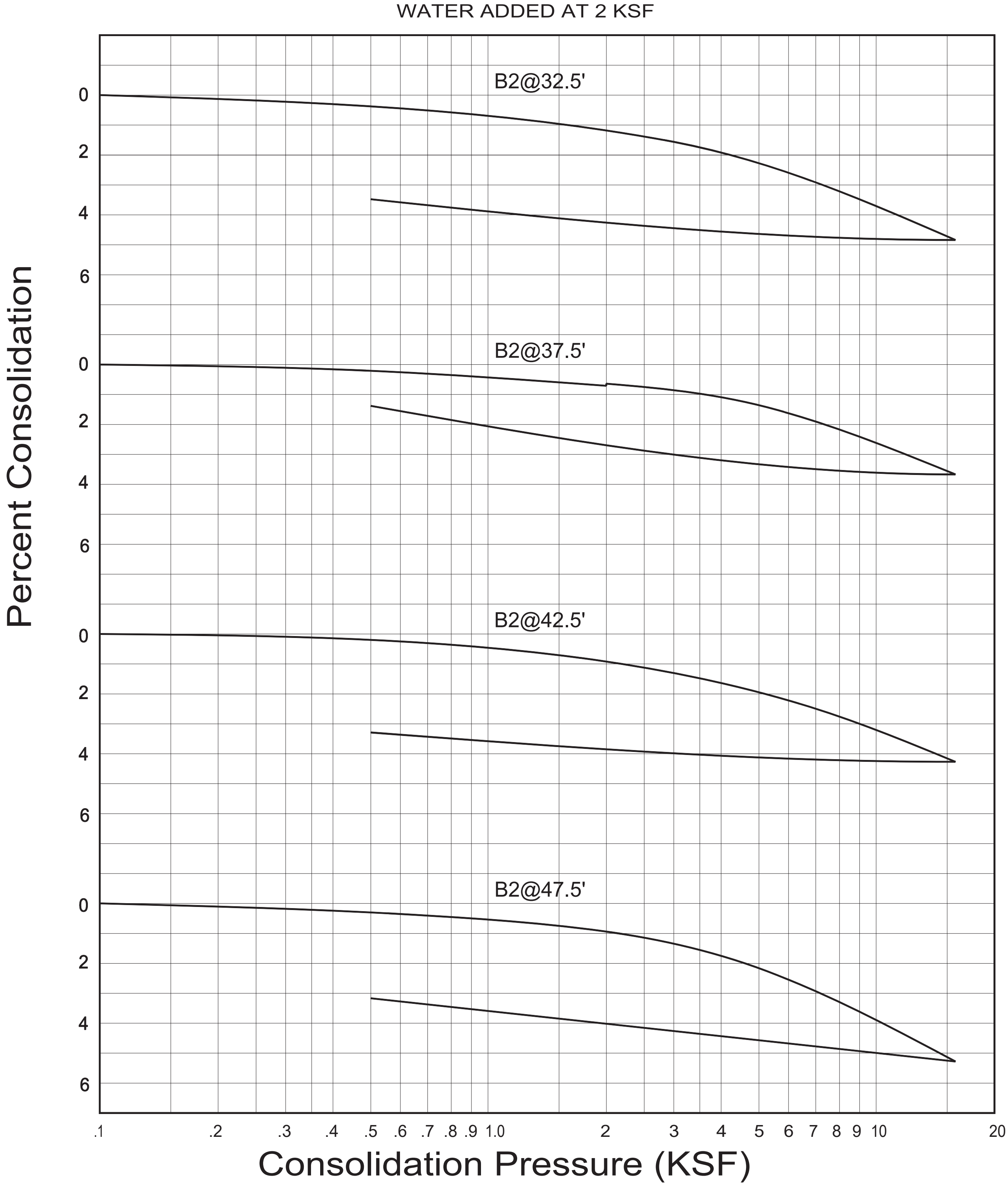

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CONSOLIDATION TEST RESULTS

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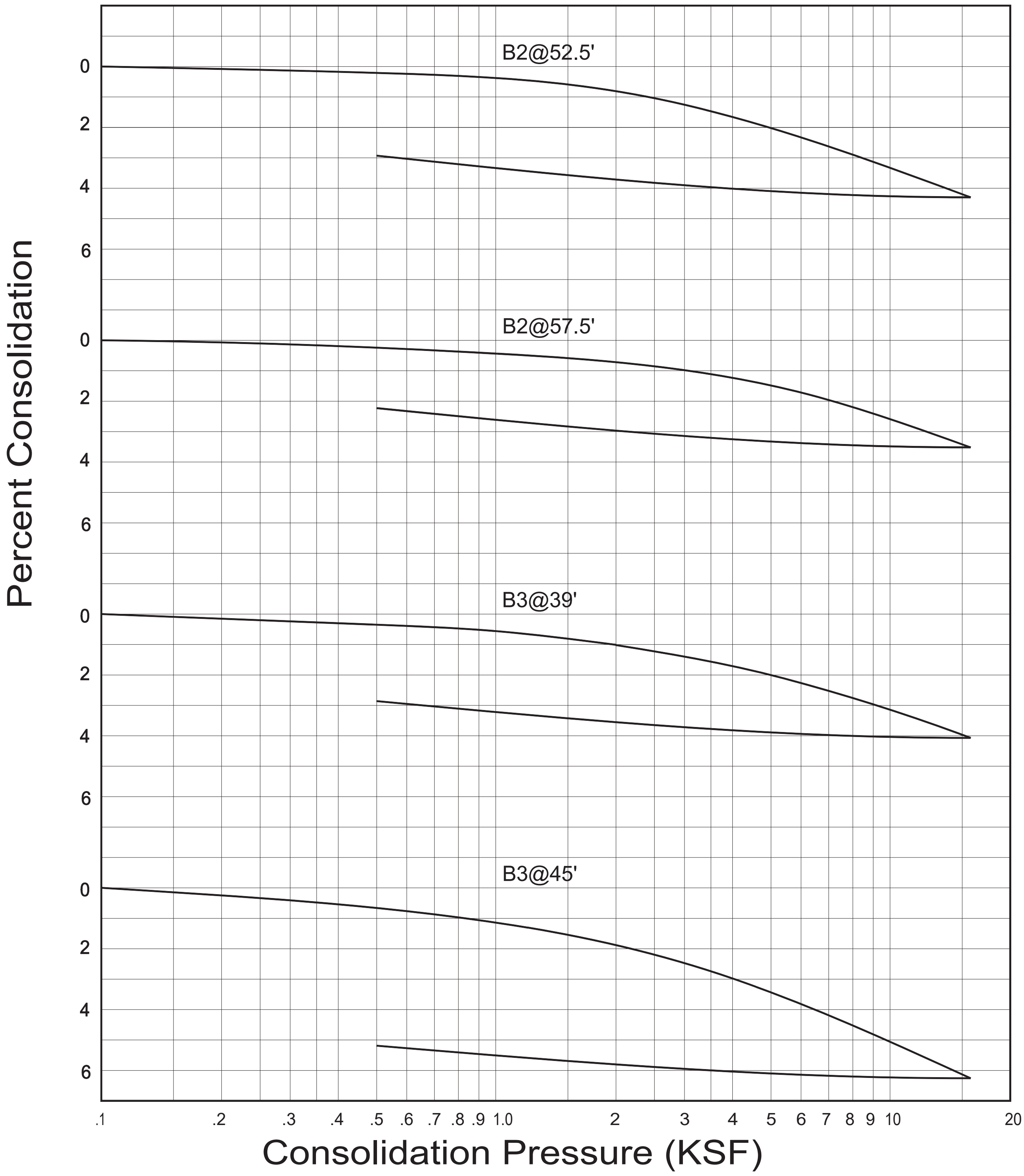
WATER ADDED AT 2 KSF



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CONSOLIDATION TEST RESULTS

8850 SUNSET BOULEVARD
WEST HOLLYWOOD, CALIFORNIA

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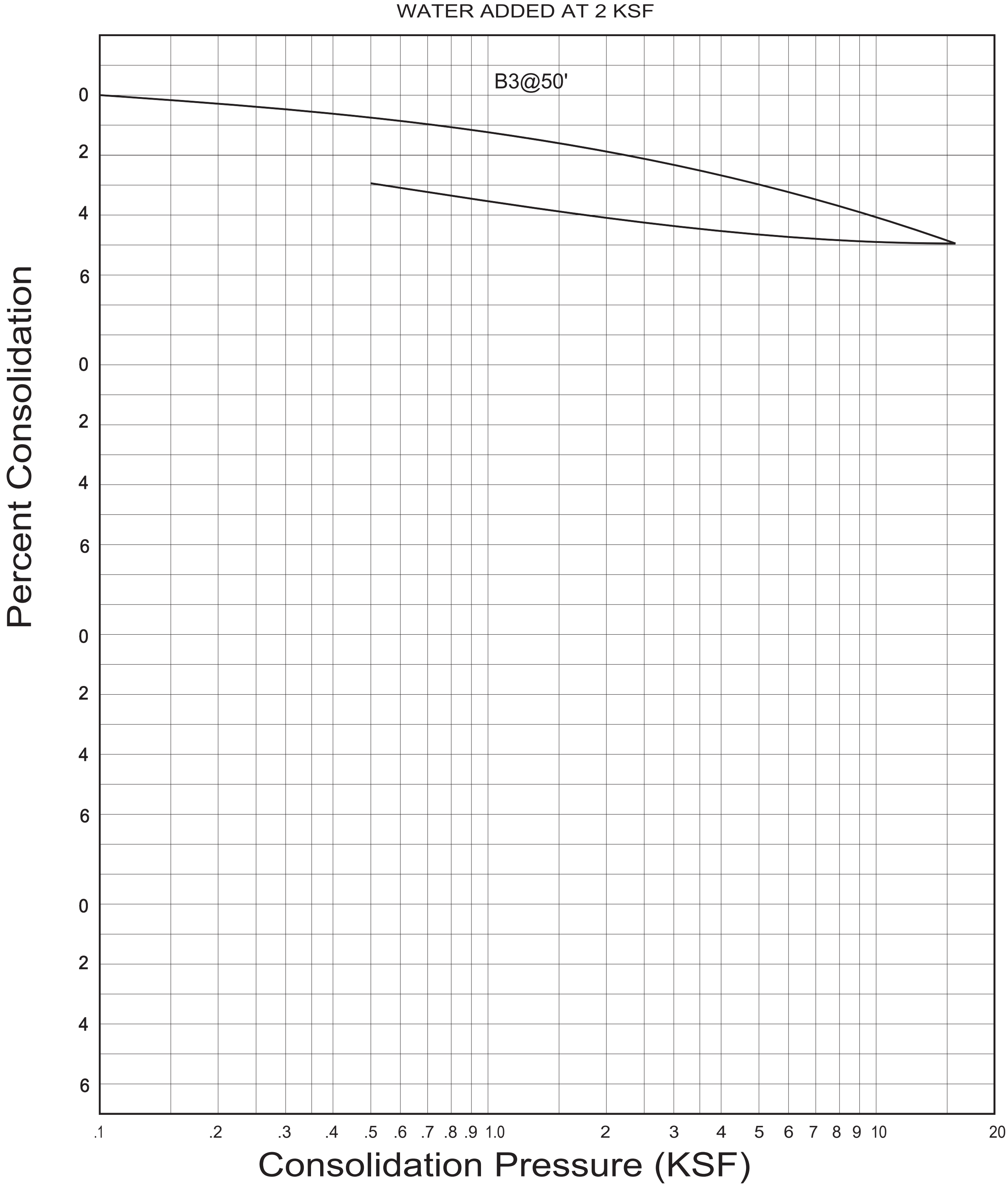

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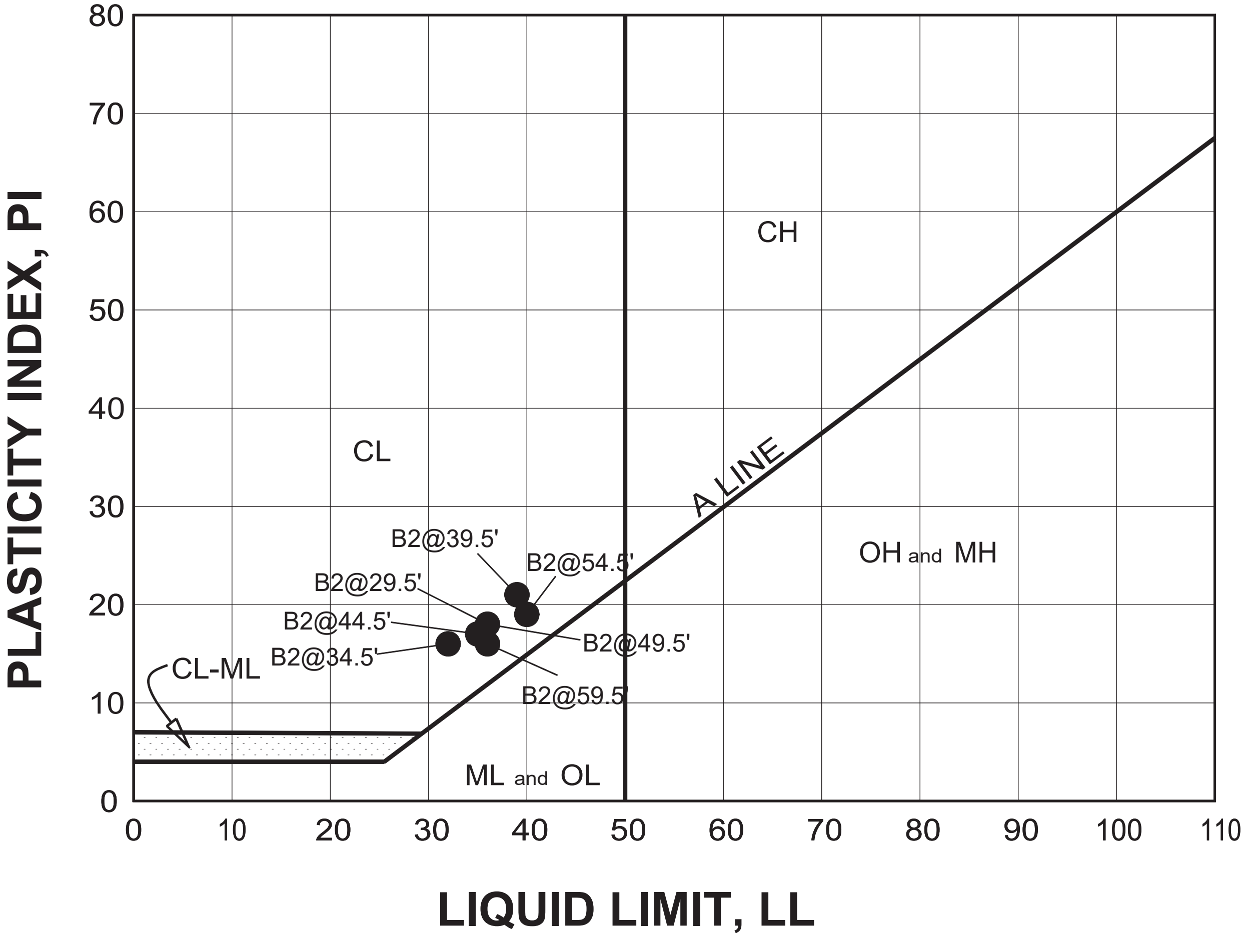
CONSOLIDATION TEST RESULTS

8850 SUNSET BOULEVARD
WEST HOLLYWOOD, CALIFORNIA

FEB. 2021 | PROJECT NO. A9899-06-01 | FIG. B6



BORING NUMBER	DEPTH (FEET)	LL	PL	PI	MOISTURE CONTENT AT SATURATION	SOIL BEHAVIOR
B2	**29.5**	**36**	**18**	**18**	**- -**	**CL**
B2	**34.5**	**32**	**16**	**16**	**- -**	**CL**
B2	**39.5**	**39**	**18**	**21**	**- -**	**CL**
B2	**44.5**	**35**	**18**	**17**	**- -**	**CL**
B2	**49.5**	**36**	**18**	**18**	**- -**	**CL**
B2	**54.5**	**40**	**21**	**19**	**- -**	**CL**
B2	**59.5**	**36**	**20**	**16**	**- -**	**CL**

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ATTERBERG LIMITS

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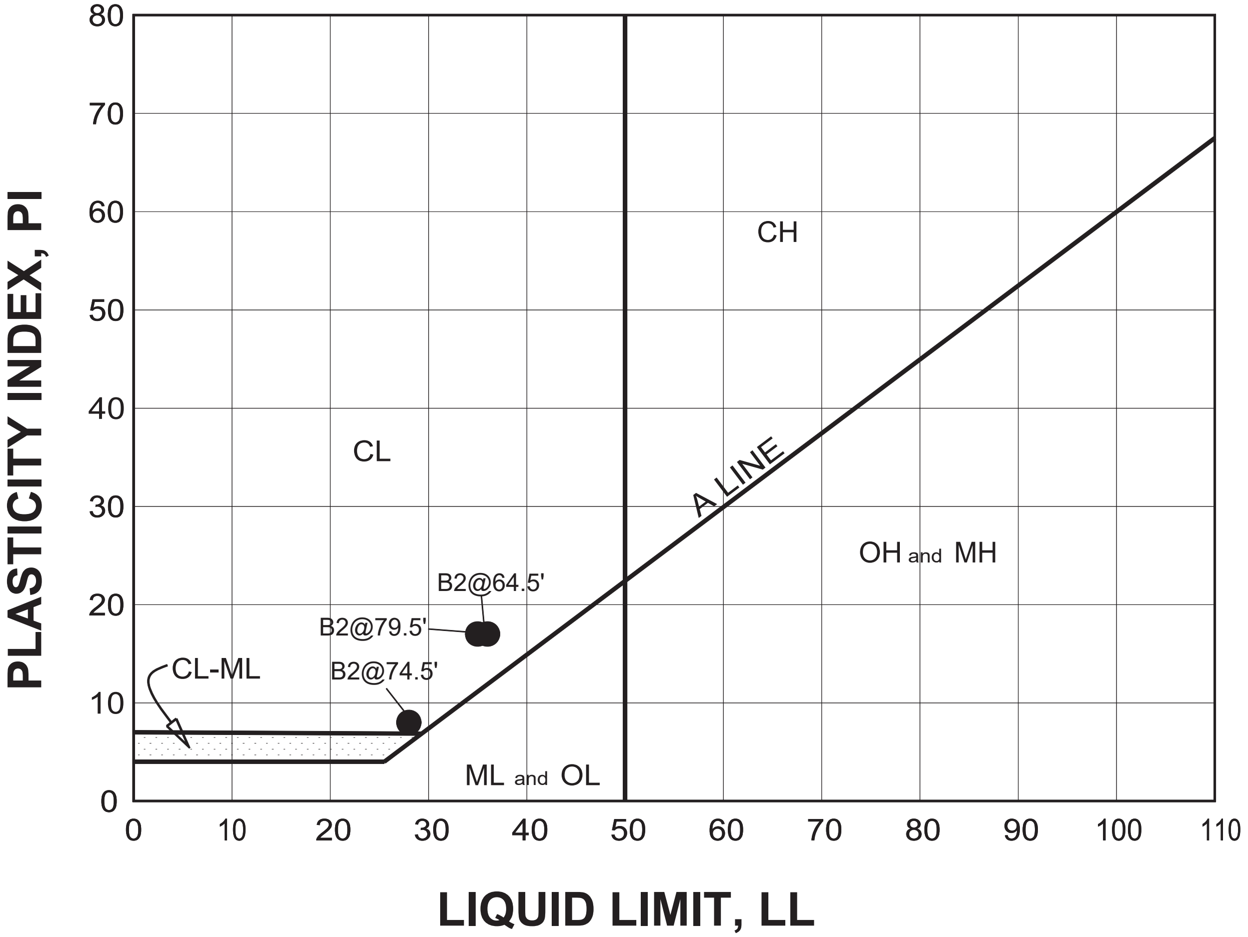
FIG. B7



BORING NUMBER	DEPTH (FEET)	LL	PL	PI	MOISTURE CONTENT AT SATURATION	SOIL BEHAVIOR
B2	**64.5**	**36**	**19**	**17**	**- -**	**CL**
B2	**74.5**	**28**	**20**	**8**	**- -**	**CL**
B2	**79.5**	**35**	**18**	**17**	**- -**	**CL**

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ATTERBERG LIMITS

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WEST HOLLYWOOD, CALIFORNIA

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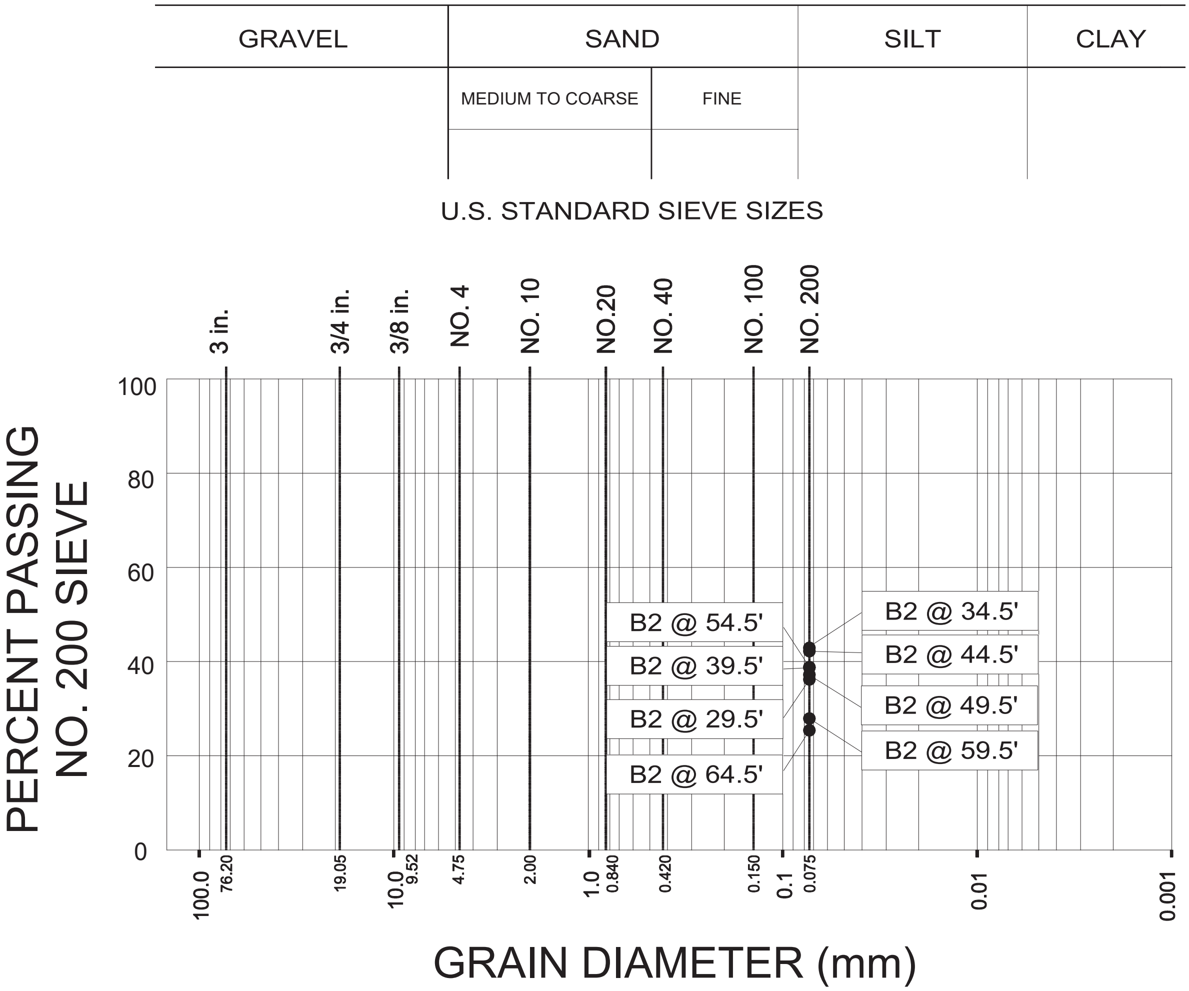
PROJECT NO. A9899-06-01
FIG. B8



SAMPLE	PERCENT PASSING NO. 200 SIEVE
B2 @ 29.5'	36.2
B2 @ 34.5'	42.9
B2 @ 39.5'	38.7
B2 @ 44.5'	42.2
B2 @ 49.5'	37.2
B2 @ 54.5'	38.8
B2 @ 59.5'	27.9
B2 @ 64.5'	25.4

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GRAIN SIZE ANALYSIS

8850 SUNSET BOULEVARD
WEST HOLLYWOOD, CALIFORNIA

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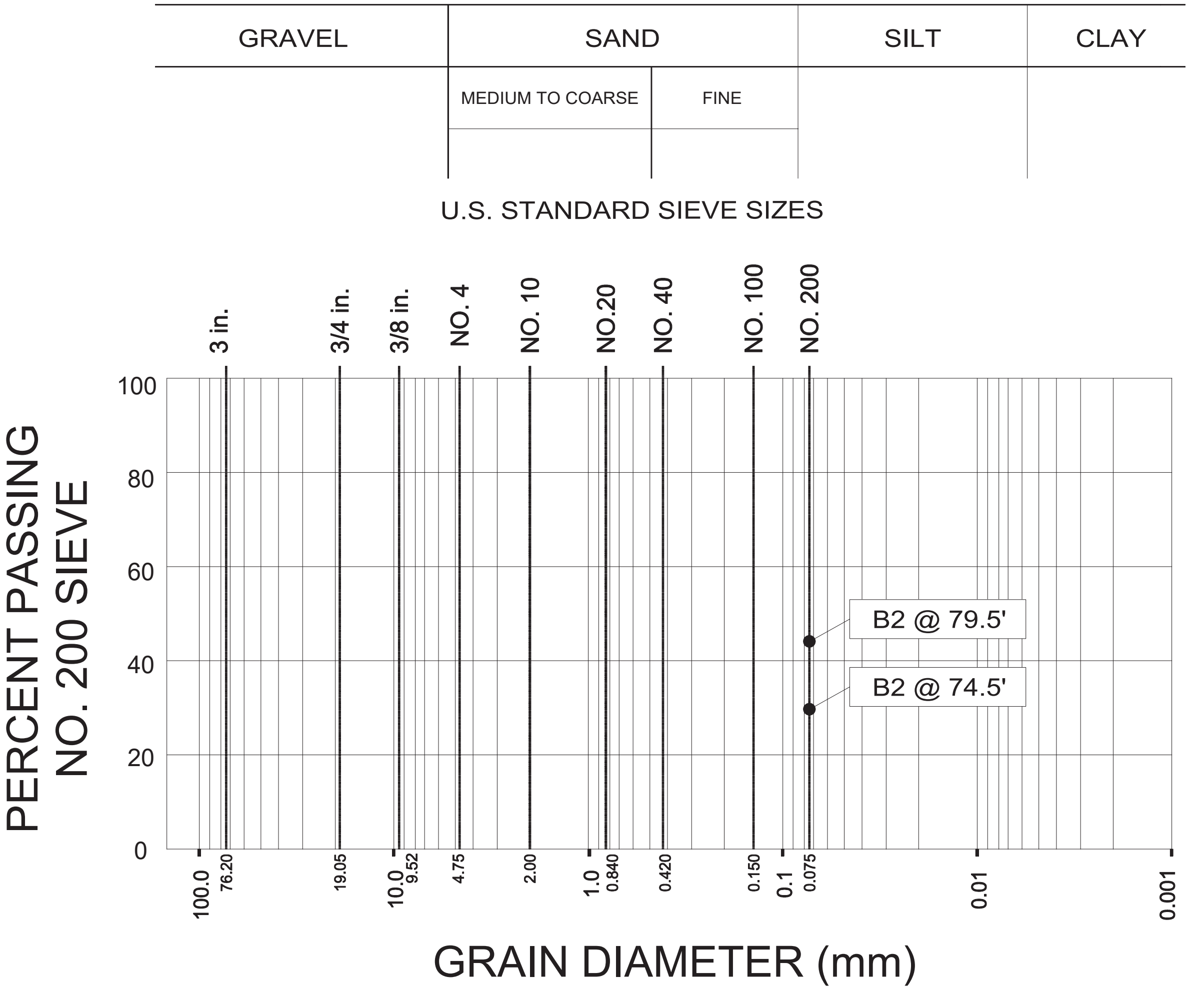

SAMPLE	PERCENT PASSING NO. 200 SIEVE
B2 @ 74.5'	29.7
B2 @ 79.5'	44.1

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GRAIN SIZE ANALYSIS

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WEST HOLLYWOOD, CALIFORNIA

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PROJECT NO. A9899-06-01
FIG. B10

SUMMARY OF LABORATORY EXPANSION INDEX TEST RESULTS
ASTM D 4829-11

Sample No.	Moisture Content (%)		Dry Density (pcf)	Expansion Index	*UBC Classification	**CBC Classification
	Before	After				
B3 @ 0-5'	8.6	11.7	115.3	0	Very Low	Non-Expansive

* Reference: 1997 Uniform Building Code, Table 18-I-B.

** Reference: 2019 California Building Code, Section 1803.5.3

SUMMARY OF LABORATORY MAXIMUM DENSITY AND AND OPTIMUM MOISTURE CONTENT TEST RESULTS
ASTM D 1557-12

Sample No.	Soil Description	Maximum Dry Density (pcf)	Optimum Moisture (%)
B3 @ 0-5'	Brown Silty Sand	134.5	8.0

SUMMARY OF LABORATORY POTENTIAL OF HYDROGEN (pH) AND RESISTIVITY TEST RESULTS
CALIFORNIA TEST NO. 643

Sample No.	**pH**	**Resistivity (ohm centimeters)**
B1 @ 38-43'	7.9	1700 (Corrosive)

SUMMARY OF LABORATORY CHLORIDE CONTENT TEST RESULTS
EPA NO. 325.3

Sample No.	Chloride Ion Content (%)
B1 @ 38-43'	0.005

SUMMARY OF LABORATORY WATER SOLUBLE SULFATE TEST RESULTS
CALIFORNIA TEST NO. 417

Sample No.	Water Soluble Sulfate (% SO_4)	Sulfate Exposure*
B1 @ 38-43'	0.003	S0

* Reference: 2019 California Building Code, Section 1904 and 318-14 Table 19.3.1.1

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CORROSIVITY TEST RESULTS

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APPENDIX C

REPORT

SURFACE WAVE MEASUREMENTS

8850 SUNSET BLVD
WEST HOLLYWOOD, CALIFORNIA

GEO*Vision* Project No. 20430

Prepared for

Geocon West, Inc.
15520 Rockfield Blvd, Suite J
Irvine, California 92618

Prepared by

GEO*Vision*, Inc.
1124 Olympic Drive
Corona, California 92881
(951) 549-1234

Report 20430-01 Rev 1

January 21, 2021

TABLE OF CONTENTS

LIST OF TABLES

LIST OF FIGURES

1 INTRODUCTION

In-situ seismic measurements using active- and passive-source surface wave techniques were performed at the property located at 8850 Sunset Blvd, West Hollywood, California on January 4, 2021. The purpose of this investigation was to provide a shear (S) wave velocity profile to a depth of 30 m (100 ft), or greater, and estimate the average S-wave velocity of the upper 30 m (V_{S30}) or 100 ft (V_{S100ft}). The active-source surface wave technique utilized during this investigation consisted of the multi-channel analysis of surface waves (MASW) method. The passive-source surface wave technique consisted of the array microtremor method. The locations of the active- and passive-source surface wave testing locations are shown on Figure 1. Array microtremor measurements were made using a L-shaped array (Array 1) and MASW measurements made in the parking lot of the structure (Array 2).

V_{S30} is used in the NEHRP provisions and the Uniform Building Code (UBC) to separate sites into classes for earthquake engineering design (BSSC, 2009). V_{S100ft} is used in the International Building Code (IBC) for site classification. These site classes are as follows:

Class A – hard rock – $V_{S30} > 1500$ m/s (UBC) or $V_{S100ft} > 5,000$ ft/s (IBC)
Class B – rock – $760 < V_{S30} \leq 1500$ m/s (UBC) or $2,500 < V_{S100ft} \leq 5,000$ ft/s (IBC)
Class C – very dense soil and soft rock – $360 < V_{S30} \leq 760$ m/s (UBC)
or $1,200 < V_{S100ft} \leq 2,500$ ft/s (IBC)
Class D – stiff soil – $180 < V_{S30} \leq 360$ m/s (UBC) or $600 < V_{S100ft} \leq 1,200$ ft/s (IBC)
Class E – soft soil – $V_{S30} < 180$ m/s (UBC) or $V_{S100ft} < 600$ ft/s (IBC)
Class F – soils requiring site-specific evaluation

At many sites, active surface wave techniques (MASW) with the utilization of portable energy sources, such as hammers and weight drops, are sufficient to obtain S-wave velocity sounding to 30 m (100 ft) depth. At sites with high ambient noise levels and/or very soft soils, these energy sources may not be sufficient to image to this depth and a larger energy source, such as a bulldozer, is necessary. Alternatively, passive surface wave techniques, such as the array microtremor technique can be used to extend the depth of investigation at sites that have adequate ambient noise conditions. It should be noted that two-dimensional passive-source surface wave arrays (e.g., triangular, circular, or L-shaped arrays) are expected to perform better than linear arrays.

This report contains the results of the active and passive surface wave measurements conducted at the site. An overview of the surface wave methods is given in Section 2. Field and data reduction procedures are discussed in Sections 3 and 4, respectively. Data modeling is presented in Section 5 and interpretation and results are presented in Section 6. References and our professional certification are presented in Sections 7 and 8, respectively.

— Active Surface Wave Array (MASW)

— Passive Surface Wave Array (L-Array)

NOTES:
1. Coordinate System: California State Plane, NAD83, Zone V (0405), US Survey Feet
2. Base map source: Esri and the Los Angeles Regional Imagery Consortium (LAR-IAC), winter 2010/2011

GEOVision geophysical services

Date:	1/11/2021
GV Project:	20430
Developed by:	C Martinez
Drawn by:	T Rodriguez
Approved by:	V Gonzalez
File Name:	20430-1.MXD

FIGURE 1
SITE MAP

SITE LOCATED AT
8850 SUNSET BLVD
WEST HOLLYWOOD, CALIFORNIA

PREPARED FOR
GEOCON WEST, INC.

2 OVERVIEW OF SURFACE WAVE TECHNIQUES

2.1 Introduction

Active- and passive-source (ambient vibration) surface wave techniques are routinely utilized for site characterization. Active surface wave techniques include the spectral analysis of surface waves (SASW) and multi-channel array surface wave (MASW) methods. Passive surface wave techniques include the horizontal over vertical spectral ratio (HVSR) technique and the array and refraction microtremor methods.

The basis of surface wave methods is the dispersive characteristic of Rayleigh and Love waves when propagating in a layered medium. Surface waves of different wavelengths (λ) or frequencies (f) sample different depth. As a result of the variance in the shear stiffness of the distinct layers, waves with different wavelengths propagate at different phase velocities; hence, dispersion. A surface wave dispersion curve is the variation of V_R or V_L with λ or f. The Rayleigh wave phase velocity (V_R) depends primarily on the material properties (V_S, mass density, and Poisson's ratio or compression wave velocity) over a depth of approximately one wavelength. The Love wave phase velocity (V_L) depends primarily on V_S and mass density. Rayleigh and Love wave propagation are also affected by damping or seismic quality factor (Q). Rayleigh wave techniques are utilized to measure vertically polarized S-waves (S_V-wave); whereas Love wave techniques are utilized to measure horizontally polarized S-waves (S_H-wave).

2.2 Surface Wave Techniques

The MASW and array microtremor techniques were utilized during this investigation and are discussed below.

2.2.1 MASW Technique

A description of the MASW method is given by Park, 1999a and 1999b and Foti, 2000. Ground motions are typically recorded by 24, or more, geophones typically spaced 1 to 3 m apart along a linear array and connected to a seismograph. Energy sources for shallow investigations include various sized hammers and vehicle mounted weight drops. When applying the MASW technique to develop a one-dimensional (1-D) V_S model, it is preferable to use multiple-source offsets from both ends of the array. The most commonly applied MASW technique is the Rayleigh-wave based MASW method, which we refer to as MAS_RW to distinguish from Love-wave based MASW (MAS_LW). MAS_RW and MAS_LW acquisition can easily be combined with P- and S-wave seismic refraction acquisition, respectively. MAS_RW data are generally recorded using a vertical source and vertical geophone but may also be recorded using a horizontal geophone with radial (in-line) orientation. MAS_LW data are recorded using transversely orientated horizontal source and transverse horizontal geophone.

A wavefield transform is applied to the time-history data to convert the seismic record from time-offset space to frequency-wavenumber (f-k) space in which the fundamental or higher surface-wave modes can be easily identified as energy maxima and picked. Frequency and/or wavenumber can easily be mapped to phase velocity, slowness, or wavelength using the following properties: $k = 2\pi/\lambda$, $\lambda = v/f$. Common wave-field transforms include: the f-k transform (a 2D fast Fourier transform), slant-stack transform (also referred to as interceptslowness or τ-p transform and equivalent to linear Radon transform), frequency domain beamformer, and phase-shift transform. The minimum wavelength that can be recovered from MASW data set without spatial aliasing is equal to the minimum receiver spacing. Occasionally, SASW analysis procedures are used to extract surface wave dispersion data, from fixed receiver pairs, at smaller wavelengths than can be recovered by wavefield transformation. Construction of a dispersion curve over the wide frequency/wavelength range necessary to develop a robust V_S model while also limiting the maximum wavelength based on an established near-field criterion (e.g. Yoon and Rix, 2009; Li and Rosenblad, 2011), generally requires multiple source offsets.

Although the clear majority of MASW surveys record Rayleigh waves, it has been shown that Love wave techniques can be more effective in some environments, particularly shallow rock sites and sites with a highly attenuative, low velocity surface layer (Xia, et al., 2012; **GEO***Vision*, 2012; Yong, et al., 2013; Martin, et al., 2014). Rayleigh wave techniques, however, are generally more effective at sites where velocity gradually increases with depth because larger energy sources are readily available for the generation of Rayleigh waves. Rayleigh wave techniques are also more applicable to sites with high velocity layers and/or velocity inversions because the presence of such structures is more apparent in the Rayleigh wave dispersion curves than in Love wave dispersion curves. Rayleigh wave techniques are preferable at sites with a high velocity surface layer because Love waves do not theoretically exist in such environments. Occasionally, the horizontal radial component of a Rayleigh wave may yield higher quality dispersion data than the vertical component because different modes of propagation may have more energy in one component than the other. Recording both the vertical and horizontal components of the Rayleigh wave is particularly useful at sites with complex modes of propagation or when attempting to recover multiple Rayleigh wave modes for multi-mode modeling as demonstrated in Dal Moro, et al, 2015. Joint inversion of Rayleigh and Love wave data may yield more accurate V_S models and also offer a means to investigate anisotropy, where S_V- and S_H-wave velocity are not equal, as shown in Dal Moro and Ferigo, 2011.

2.2.2 Array Microtremor Technique

A detailed discussion of the array microtremor method can be found in Okada, 2003. Unlike active source techniques which use an active energy source (i.e. hammer), the array microtremor technique (also referred to as passive surface wave or array ambient vibration method) records background noise (ambient vibrations) emanating from ocean wave activity, wind noise, traffic, industrial activity, construction, etc. The technique uses 4, or more, receivers aligned in a 2-dimensional array. Triangle, circle, semi-circle, and “L” shaped arrays are commonly used, although any 2-dimensional arrangement of receivers can be used. For investigations of the upper 100 m, receivers typically consist of 1 to 4.5 Hz geophones. For deeper investigations, 5 to 120 s seismometers are generally utilized. The nested triangle array, which consists of several embedded equilateral triangles, is popular as it provides accurate dispersion curves with a relatively small number of geophones. The “L” array is useful at sites located at the corner of intersecting streets. The maximum receiver separation in an array should be at a minimum equal to the desired depth of investigation. Typically, 15 to 60 minutes of ambient vibration data is recorded depending on the size of the array, desired depth of investigation, and noise conditions. Investigations to depths on the order of 1 km may require that ambient vibrations are recorded for a much longer duration. The surface wave dispersion curve is typically estimated from array microtremor data using various f-k methods such as beamforming (Lacoss, et al., 1969), and maximum-likelihood (Capon, 1969), and the spatial-autocorrelation (SPAC) method. The beam-forming and maximum-likelihood methods are generally referred to as the frequency wavenumber (FK) and high-resolution frequency wavenumber (HRFK or HFK) methods. The SPAC method was originally based on work by Aki, 1957 and has since been extended and modified (Ling and Okada, 1993 and Ohori *et al.*, 2002) to permit the use of noncircular arrays, and is now collectively referred to as extended spatial autocorrelation (ESPAC or ESAC). Further modifications to the SPAC method permit the use of irregular or random arrays (Bettig *et al.*, 2001). Although it is common to apply SPAC methods to obtain a surface wave dispersion curve for modeling, other approaches involve direct modeling of the coherency data, also referred to as SPAC coefficients (Asten, 2006 and Asten, *et al.*, 2015). The beam-forming and maximum-likelihood methods are generally referred to as the frequency wavenumber (FK) and high-resolution frequency wavenumber (HRFK or HFK) methods, respectively. More recently, a Rayleigh wave three-component beamforming method (RTBF) has been developed (Wathelet, et al., 2018) and appears to offer significant resolution enhancements over other methods.

FK, HRFK and RTBF methods are generally expected to perform better when ambient vibration sources are not azimuthally well-distributed (e.g. rural area where the primary noise source is a large industrial facility). SPAC methods are expected to perform better when noise sources are azimuthally well-distributed (e.g. in a large urbanized area).

The minimum wavelength surface wave that can be extracted from an array microtremor dataset acquired utilizing a symmetric array is typically set equal to the minimum receiver spacing. The maximum wavelength is often set equal to twice the maximum receiver separation for SPAC analysis and the maximum receiver spacing for FK analysis.

2.3 Surface Wave Dispersion Curve Modeling

The dispersion curves generated from the active and passive surface wave soundings are generally combined and modeled using iterative forward and inverse modeling routines. The final model profile is assumed to represent actual site conditions. The theoretical model used to interpret the dispersion curve assumes horizontally layered, laterally invariant, homogeneous-isotropic material. Although these conditions are seldom strictly met at a site, the results of active and/or passive surface wave testing provide a good “global” estimate of the material properties along the array. The results may be more representative of the site than a borehole “point” estimate.

The surface wave forward problem is typically solved using the Thomson-Haskell transfer-matrix (Thomson, 1950; Haskell, 1953) later modified by Dunkin (1965) and Knopoff (1964), dynamic stiffness matrix (Kausel and Roësset, 1981), or reflection and transmission coefficient (Kennett, 1974) methods. All of these methods can determine fundamental- and higher-mode phase velocities, which correspond to plane waves in 2-D space. The transfer-matrix method is often used in MASW and passive surface-wave software packages, whereas the dynamic stiffness matrix is utilized in many SASW software packages. $\mathrm{MAS_RW}$ and/or passive surface-wave modeling may involve modeling of the fundamental mode, some form of effective mode, or multiple individual modes (multi-mode). As outlined in Roësset et al. (1991), several options exist for forward modeling of Rayleigh wave SASW data. One formulation takes into account only fundamental mode plane Rayleigh-wave motion (called the 2-D solution), whereas another includes all stress waves (e.g. body, fundamental, and higher mode surface waves) and incorporates a generalized receiver geometry (3-D global solution) or actual receiver geometry (3-D array solution).

The fundamental mode assumption is generally applicable to modeling Rayleigh-wave dispersion data collected at normally dispersive sites, providing there are not abrupt increases in velocity or steep velocity gradients. Effective-mode or multi-mode approaches are often required for irregularly dispersive sites and sites with steep velocity gradients at shallow depth. If active and passive surface wave data are combined or MAS_RW data are combined from multiple seismic records with different source offsets and receiver gathers, then effective-mode computations are limited to algorithms that assume far-field plane Rayleigh wave propagation. Local search (e.g. linearized matrix inversion methods) or global search methods (e.g., Monte Carlo approaches such as simulated annealing, generic algorithms and neighborhood algorithm) are typically used to solve the inverse problem.

The maximum wavelength (λ_{max}) recovered from a surface wave data set is typically used to estimate depth of investigation although a sensitivity analysis of the V_S models would be a more robust means to estimate depth of investigation. For normally dispersive velocity profiles with a gradual increase in V_S with depth, the maximum depth of investigation is on the order of $\lambda_{max}/2$ for both Rayleigh and Love wave dispersion data. For velocity profiles with an abrupt increase in V_S at depth, the maximum depth of investigation is on the order of $\lambda_{max}/3$ for Rayleigh wave dispersion data but less than $\lambda_{max}/3$ for Love wave dispersion data. The depth of investigation can be highly variable for sites with complex velocity structure (e.g. high velocity layers).

As with all surface geophysical methods, the inversion of surface wave dispersion data does not yield a unique V_S model and multiple possible solutions may equally fit the experimental data. Based on experience at other sites, the shear wave velocity models (V_S and layer thicknesses) determined by surface wave testing are within 20% of the velocities and layer thicknesses that would be determined by other seismic methods (Brown, 1998). The average velocity of the upper 30 m, however, is much more accurate, often to better than 5%, because it is not sensitive to the layering in the model. Because V_{S30} does not appear to be sensitive non-uniqueness inherent in V_S models derived from surface wave dispersion curves (Martin et al., 2006, Comina et al., 2011), it can be accurately estimated from a single V_S model.

It may not always be possible to develop a coherent, fundamental mode dispersion curve over sufficient frequency range for modeling due to dominant higher modes with the higher modes not clearly identifiable for multi-mode modeling. It may, however, be possible to identify the Rayleigh wave phase velocity of the fundamental mode at 40 m wavelength (V_{R40}) in which case V_{S30} can at least be estimated using the Brown et al., 2000 relationship:

$$V_{S30} = 1.045V_{R40}$$

This relationship was established based on a statistical analysis of a large number of surface wave data sets from sites with control by velocities measured in nearby boreholes and has been further evaluated by Martin and Diehl, 2004, and Albarello and Gargani, 2010. Further investigation of this approach has revealed that V_{S30} is generally between V_{R40} and V_{R45} with V_{R40} often being most appropriate for shallow groundwater sites and V_{R45} for deep ground water sites. A detailed study of such an approach for Love wave dispersion data has not been conducted; however, preliminary analysis demonstrates that V_{S30} is generally between V_{L50} and

V_{L55}. Although we do not recommend that these empirical V_{S30} estimates replace modeling of surface wave dispersion data, they do offer a means of cost effectively evaluating V_{S30} over a large area. V_{R40} or V_{L55} can also be used to quantify error in V_{S30} by evaluating the scatter in the dispersion data at these wavelengths.

3 FIELD PROCEDURES

The active- and passive-source surface wave sounding locations at the site were established by **GEO***Vision* personnel and are shown in Figure 1. Two types of surface wave data were acquired at the site: an active-source surface wave array to characterize near-surface velocity structure and a passive-source surface wave array to characterize deeper velocity structure. Passive surface wave data were acquired along Array 1 using the array microtremor method. Active surface wave data were acquired along Array 2 using the MASW technique.

The passive surface wave equipment consisted of two Geometrics Geode signal enhancement seismographs, 4.5 Hz vertical geophones, and seismic cables. The L-shaped Array 1 consisted of 44, 4.5 Hz geophones spaced 4 m (13.1 ft) apart with lengths of the legs of the array being 72 and 100 m (236 and 328 ft), respectively. Ambient noise measurements were made along the passive array for about 30 minutes (63, 30 second seismic records) with a 2 ms sample rate. All passive surface wave data were stored on a laptop computer for later processing. The field geometry and associated files names were documented in field notes.

MASW equipment used during this investigation consisted of a Geometrics Geode signal enhancement seismographs 4.5 Hz vertical geophones, seismic cable, a 4 lb hammer, a 10 lb sledgehammer, and a 240 lb accelerated weight drop (AWD). MASW data were acquired along a linear array of 24 geophones spaced 1.5 m (4.9 ft) apart. Shot points were located between 1.5 and 9 m (4.9 and 29.5 ft) from the end geophone locations and at 9 m (29.5 ft) intervals in the interior of the array. The 4 lb hammer and/or 10 lb sledgehammer were used for the near offset source locations and interior source locations. The AWD was used for source locations offset from the ends of the array. Data from the transient impacts (hammers) were generally averaged 5 to 10 times to improve the signal-to-noise ratio. All field data were saved to hard disk and documented on field data acquisition forms.

4 DATA REDUCTION

The MASW data were reduced using the software Seismic Pro Surface V9.0 developed by Geogiga and multiple in-house scripts for various data extraction and formatting tasks, with all data reduction documented in a Microsoft Excel spreadsheet.

The following steps were used for data reduction:

- Input seismic records to be used for analysis into software package.
- Check and correct source and receiver geometry as necessary.
- Select offset range used for analysis (multiple offset ranges utilized for each seismic record as discussed below) and document in spreadsheet.
- Apply phase shift transform to seismic record to convert the data from time – offset to frequency – phase velocity space.
- Identify, pick, save, and document dispersion curve.
- Change the receiver offset range and repeat process.
- Repeat process for all seismic records.
- Use in-house script to apply near-field criteria with maximum wavelength set equal to 1.0 times the source to midpoint of receiver array distance.
- Use in-house script to merge multiple dispersion curves extracted from the MASW data collected along each seismic line for a specific source type (different source locations, different receiver offset ranges, etc.).
- Edit dispersion data, as necessary (e.g. delete poor quality curves and outliers).
- Calculate a representative dispersion curve at equal log-frequency or log-wavelength spacing for the MASW dispersion data using a moving average, polynomial curve fitting routine.

This unique data reduction strategy, which can involve combination of over 50 dispersion curves for a 1D sounding, is designed for characterizing sites with complex velocity structure that do not yield surface wave dispersion data over a wide frequency range from a single source type or source location. The data reduction strategy ensures that the dispersion curve selected for modeling is representative of average conditions beneath the array and spans as broad a frequency/wavelength range as possible while considering near field effects.

The array microtremor data were reduced using the Seisimager software package developed by Oyo Corporation/Geometrics, Inc. and the following steps:

The processing sequence for implementation of the ESAC method in the SeisImager software package is as follows:

- Input all seismic records for a dataset into software.
- Load receiver geometry (x and y positions) for each channel in seismic record.
- Calculate the SPAC coefficients for each seismic record and average.
- Optionally, select a subset of receiver offset ranges for analysis (e.g. only select receiver pairs with multiple azimuths).
- For each frequency calculate the RMS error between the SPAC coefficients and a Bessel function of the first kind and order zero over a user defined phase velocity range and velocity step.

- Plot an image of RMS error as a function for frequency (f) and phase velocity (v).
- Identify and pick the dispersion curve as the continuous trend on the f-v image with the lowest RMS error.
- Repeat the process for all arrays and time blocks.
- Use an in-house script to convert dispersion curves to appropriate format for editing.
- Edit dispersion data, as necessary, and use in-house script to combine all dispersion data after setting maximum wavelength to about 2 to 3 times the maximum receiver spacing.
- Calculate a representative dispersion curve for the passive dispersion data from each array using a moving average polynomial curve fitting routine.

The representative dispersion curves from the active and passive surface wave data were combined and the moving average polynomial curve fitting routine in WinSASW V3 was used to generate a composite representative dispersion curve for modeling. During this process, the active and passive surface wave dispersion data were given equal weights. An equal logarithm wavelength sample rate was used for the representative dispersion curve to reflect the gradual loss in model resolution with depth.

5 DATA MODELING

Surface wave data were modeled using the fundamental mode routine in WinSASW V3 software package. During this process, an initial velocity model was generated based on general characteristics of the dispersion curve and the inverse modeling routine utilized to adjust the layer V_S until an acceptable agreement with the observed data was obtained. Layer thicknesses were adjusted, and the inversion process repeated until a V_S model was developed with low RMS error between the observed and calculated dispersion curves. In many cases, once an acceptable V_S model is developed, layer thicknesses are again adjusted, and the inversion process repeated to develop an ensemble of V_S models with similar RMS error to quantify non-uniqueness. The primary purpose of this investigation was to estimate V_{S30} and, therefore, it was not considered necessary to develop multiple V_S models. Data inputs into the modeling software include layer thickness, S-wave velocity, P-wave velocity or Poisson's ratio, and mass density. P-wave velocity and mass density only have a very small influence (i.e. less than 10%) on the S-wave velocity model generated from a surface wave dispersion curve. However, realistic assumptions for P-wave velocity, which is significantly impacted by the location of the saturated zone, and mass density will slightly improve the accuracy of the S-wave velocity model.

Constant mass density values of 1.81 to 2.06 gm/cm^3 (113 to 129 lb/ft^3) were used in the velocity profiles for subsurface soils/rock depending on P- and S-wave velocity. Within the normal range encountered in geotechnical engineering, variation in mass density has a negligible (±2%) effect on the estimated V_S from surface wave dispersion data. During modeling of Rayleigh wave dispersion data, the compression wave velocity, V_P, for unsaturated sediments was estimated using a Poisson's ratio, v, of 0.3 and the relationship:

$$V_P = V_S\,[(2(1-v))/(1-2v)]^{0.5}$$

Poisson's ratio has a larger effect than density on the estimated V_S from Rayleigh wave dispersion data. Achenbach (1973) provides approximate relationship between Rayleigh wave velocity (V_R), V_S and v:

$$V_R = V_S\,[(0.862 + 1.14\,v)/(1+ v)]$$

Using this relationship, it can be shown that V_S derived from V_R only varies by about 10% over possible 0 to 0.5 range for Poisson's ratio where:

$$V_S = 1.16V_R \text{ for } v = 0$$
$$V_S = 1.05V_R \text{ for } v = 0.5$$

The realistic range of the Poisson's ratio for typical unsaturated sediments is about 0.25 to 0.35. Over this range, V_S derived from modeling of Rayleigh wave dispersion data will vary by about 5%. An intermediate Poisson's ratio of 0.3 was selected for modeling to minimize any error associated with the assumed Poisson's ratio.

High Poisson's ratio saturated sediments with $V_P > 1{,}500$ m/s (4,921 ft/s) were constrained at an approximate depth of 6 m (19.7 ft) based on interactive analysis of seismic refraction first arrival data.

6 INTERPRETATION AND RESULTS

The fit of the calculated fundamental mode dispersion curve to the experimental data collected along Arrays 1 and 2 and the modeled V_S profile for the surface wave sounding are presented as Figure 2. The resolution decreases gradually with depth due to the loss of sensitivity of the dispersion curve to changes in V_S at greater depth. The V_S profile used to match the field data is provided in tabular form in both metric and Imperial units as Tables 1 and 2.

The Rayleigh wave phase velocities from the passive surface wave array are in acceptable agreement with those from the MASW data in the region of overlapping wavelength. Differences in dispersion data from the two methods are expected to be associated with lateral velocity variability beneath the respective arrays. Scatter in the dispersion data from each technique is expected to be primarily associated with lateral velocity variability beneath the array.

The estimated depth of investigation for the combined active and passive surface wave sounding is about 50 m (164 ft). The V_S model indicates that V_S gradually increases with depth from about 190 m/s (630 ft/s) immediately below the surface to about 600 m/s (1,970 ft/s) at a depth of about 43 m (141 ft).

The average shear wave velocity to a depth of 30 m (V_{S30}) is 308 m/s for the Arrays 1 and 2 V_S model. The average shear wave velocity to a depth of 100 ft (V_{S100ft}) is 1,017 ft/s for the V_S model. Therefore, according to the NEHRP provisions of the Uniform Building Code, the area in the vicinity of Arrays 1 and 2 is classified as Site Class D, stiff soil. The proposed structure at the site is expected to have multiple underground levels and, therefore, the average S-wave velocity was also computed between 35 and 135 ft ($V_{S35\text{-}135ft}$) and 70 and 170 ft ($V_{S70\text{-}170ft}$). $V_{S35\text{-}135ft}$ is 1,229 ft/s and $V_{S70\text{-}170ft}$ is 1,514 ft/s.

Table 1 Arrays 1 and 2 V_S Model (Metric Units)

Depth to Top of Layer (m)	Layer Thickness (m)	S-Wave Velocity (m/s)	Inferred P-Wave Velocity (m/s)	Inferred Poisson's Ratio	Assumed Density (g/cm^3)
0	1	192	360	0.300	1.81
1	2	234	438	0.300	1.87
3	3	301	563	0.300	1.93
6	10	305	1750	0.484	1.93
16	12	327	1850	0.484	1.94
28	15	484	2000	0.469	2.02
43	Half Space	601	2100	0.455	2.06

Table 2 Arrays 1 and 2 V_S Model (Imperial Units)

Depth to Top of Layer (ft)	Layer Thickness (ft)	S-Wave Velocity (ft/s)	Inferred P-Wave Velocity (ft/s)	Inferred Poisson's Ratio	Assumed Density (lb/ft^3)
0.0	3.3	631	1181	0.300	113
3.3	6.6	768	1437	0.300	117
9.8	9.8	987	1846	0.300	120
19.7	32.8	1001	5741	0.484	120
52.5	39.4	1073	6070	0.484	121
91.9	49.2	1589	6562	0.469	126
141.1	Half Space	1970	6890	0.455	129

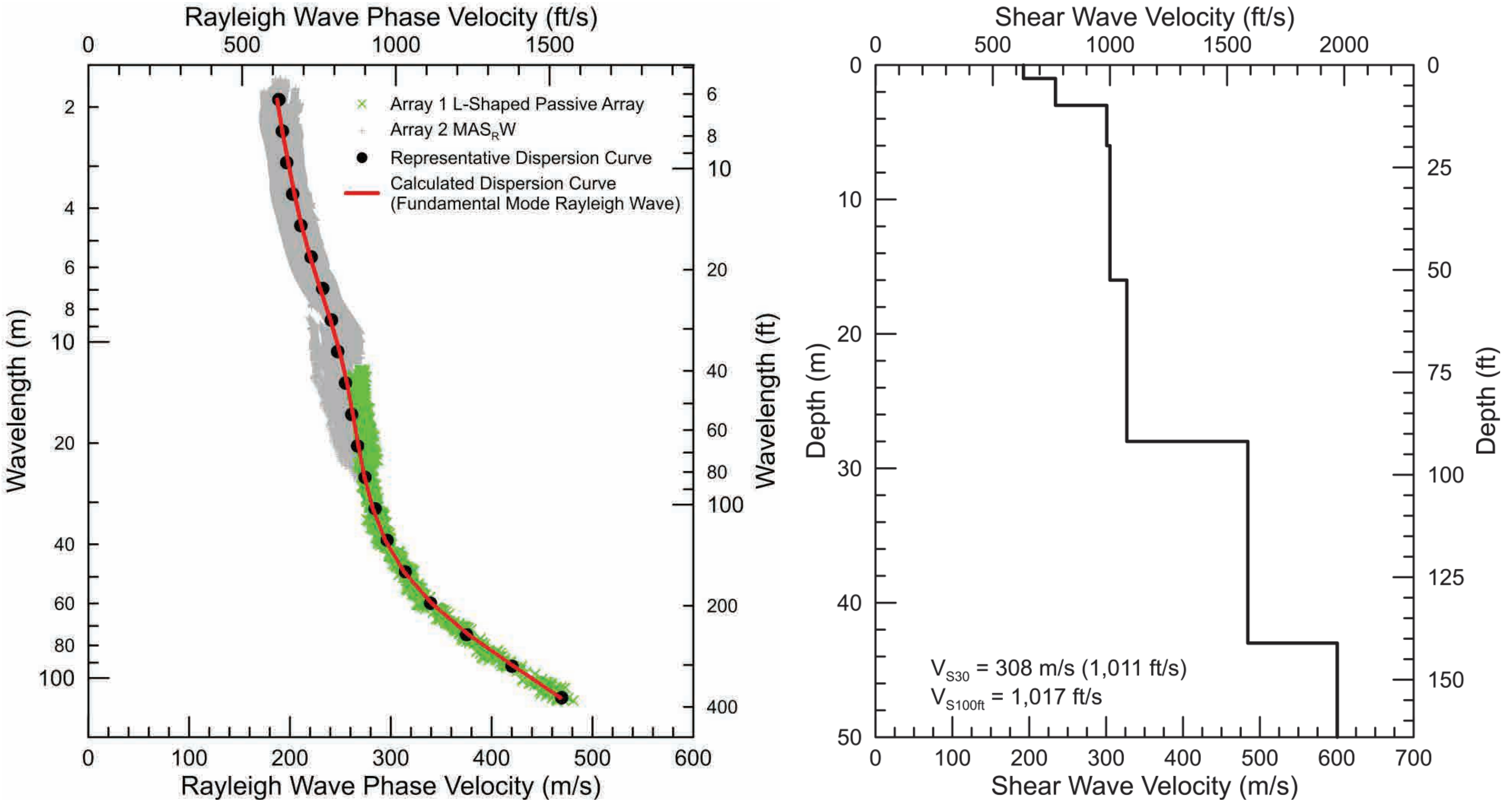

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FIGURE 2
SURFACE WAVE MODEL - ARRAYS 1 AND 2

Project No: 20430
Date: JAN 20, 2021
Drawn By: A MARTIN
Approved By: Antony J Martin

8850 SUNSET BLVD
WEST HOLLYWOOD, CALIFORNIA

PREPARED FOR
GEOCON WEST, INC.

R:\GVProjects\2020\20430 GEOCON WEST\Report\Figure 2.cdr

7 REFERENCES

Achenbach J, 1973, Wave Propagation in Elastic Solids, Elsevier, Amsterdam, Netherlands.

Aki K, 1957, Space and time spectra of stationary stochastic waves, with special reference to microtremors. *Bull. Earthq. Inst. Univ. Tokyo*, Vol. 35, p 415–457.

Albarello D and Gargani G, 2010, "Providing NEHRP soil classification from the direct interpretation of effective Rayleigh-wave dispersion curves", *Bulletin of the Seismological Society of America*, Vol. 100, No. 6, p 3284-3294.

Asten M, 2006, Site shear velocity profile interpretation from microtremor array data by direct fitting of SPAC curves, in *Proceedings of the Third Int. Symp. Effects of Surface Geol. Seismic Motion*, Vol. 2, LCPC Edition, Grenoble, France, August 30–September 01, Paper No. 99, p 1069-1080.

Asten M, Stephenson WJ and Hartzell, S, 2015, The use of wavenumber normalization in computing spatially averaged coherencies (krSPAC) of microtremor data from asymmetric arrays, *Proceedings 6th International Conference on Earthquake Geotechnical Engineering*, Christchurch, New Zealand, 9 p.

Bettig B, Bard PY, Scherbaum F, Riepl J, Cotton F, Cornou C and Hatzfeld D, 2001, Analysis of dense array noise measurements using the modified spatial auto-correlation method (SPAC): application to the Grenoble area, *Bollettino di Geofisica Teoricaed Applicata*, Vol 42, p 281–304.

Brown L, 1998, Comparison of V_S profiles from SASW and borehole measurements at strong motion sites in Southern California, Master's thesis, University of Texas, Austin.

Brown L, Diehl J and Nigbor R, 2000, A simplified method to measure average shear-wave velocity in the top 30 m (V_s30), *Proc. 6th International Conference on Seismic Zonation*, p 1–6.

BSSC, 2009, *NEHRP recommended provisions for seismic regulations for new buildings and other structure (FEMA 450), Part I: Provisions*, Building Seismic Safety Council, Federal Emergency Management Agency, Washington D.C.

Capon J, 1969, High-resolution frequency-wavenumber spectrum analysis, *Proc. Institute of Electrical and Electronics Engineers (IEEE)*, Vol. 57, no. 8, p 1408–1418.

Comina C, Foti S, Boiero D, and Socco LV, 2011, Reliability of $V_{S,30}$ evaluation from surface-wave tests, *J. Geotech. Geoenviron. Eng.*, p 579–586.

Dal Moro G, Moura R, Moustafa A, 2015, Multi-component joint analysis of surface waves, *Journal of Applied Geophysics*, Vol. 119, p 128-138.

Dal Moro G and Ferigo F, 2011, Joint Analysis of Rayleigh- and Love-wave dispersion curves: Issues, criteria and improvements, *Journal of Applied Geophysics*, Vol. 75, p 573-589.

Dunkin J W, 1965, Computation of modal solutions in layered, elastic media at high frequencies, *Bulletin of the Seismological Society of America*, Vol. 55, pp. 335–358.

Foti S, 2000, Multistation Methods for Geotechnical Characterization using Surface Waves, Ph.D. Dissertation, Politecnico di Torino, Italy.

GEOVision, Inc (2012): EPRI (2004, 2006) ground-motion model (GMM) review project: Shear wave velocity measurements at seismic recording stations, http://www.epri.com/abstracts/Pages/ProductAbstract. aspx?ProductId=000000003002000719.

Haskell N A, 1953, The dispersion of surface waves on multilayered media, *Bull. Seismol. Soc. Am.*, v. 43, pp. 17–34.

García-Jerez A, Piña-Flores J, Sánchez-Sesma F, Luzón, F, Perton, M, 2016, A computer code for forward computation and inversion of the H/V spectral ratio under the diffuse field assumption, Computers & Geosciences, In Press.

Kausel, E. and J. M. Röesset (1981). Stiffness matrices for layered soils, *Bulletin of the Seismological Society of America*, v. 71, no. 6, pp. 1743–1761.

Kennett, B.L.N (1974). Reflections, rays and reverberations, *Bulletin of the Seismological Society of America*, v. 64, no. 6, pp. 1685–1696.

Knopoff L. (1964). “A matrix method for elastic wave problems.” *Bulletin of the Seismological Society of America*, Vol. 54, pp. 431–438.

Lacoss R, Kelly E and Toksöz M, 1969, Estimation of seismic noise structure using arrays, *Geophysics*, v. 34, pp. 21-38.

Li J and Rosenblad B, 2011, Experimental study of near-field effects in multichannel array-based surface wave velocity measurements, *Near Surface Geophysics*, Vol. 9, 357-366.

Ling S and Okada H, 1993, An extended use of the spatial correlation technique for the estimation of geological structure using microtremors, *Proc. the 89th Conference Society of Exploration Geophysicists Japan (SEGJ)*, pp. 44–48 (in Japanese).

Martin A and Diehl J, 2004, Practical experience using a simplified procedure to measure average shear-wave velocity to a depth of 30 meters (V_S30), *Proceedings of the 13th World Conference on Earthquake Engineering*, Vancouver, B.C., Canada, August 1-6, 2004, Paper No. 952.

Martin A, Shawver J and Diehl J, 2006, Combined use of Active and Passive Surface Wave Techniques for Cost Effective UBC/IBC Site Classification, *Proceedings of the 8th National Conference on Earthquake Engineering*, San Francisco, California, Paper No. 1013.

Martin A, Yong A and Salomone L, 2014, Advantages of active Love wave techniques in geophysical characterization of seismographic station sites – case studies in California and the Central and Eastern United States, Proceedings of the Tenth U.S. National Conference on Earthquake Engineering, Frontiers of Earthquake Engineering, July 21-25, Anchorage, Alaska.

Ohori M, Nobata A, and Wakamatsu K, 2002, A comparison of ESAC and FK methods of estimating phase velocity using arbitrarily shaped microtremor arrays, *Bull. Seismol. Soc. Am.*, v. 92, no. 6, p. 2323–2332.

Okada H, 2003, The Microtremor Survey Method, *Society of Exploration Geophysics Geophysical Monograph Series*, Number 12, 135p.

Park C, Miller R and Xia J, 1999a, Multimodal analysis of high frequency surface waves, *Proceedings of the Symposium on the Application of Geophysics to Engineering and Environmental Problems ’99*, 115-121.

Park C, Miller R and Xia, J, 1999b, Multichannel analysis of surface waves, *Geophysics*, Vol 64, No. 3, 800-808.

Roesset J, Chang D and Stokoe K, 1991, Comparison of 2-D and 3-D Models for Analysis of Surface Wave Tests, *Proceedings*, 5th *International Conference on Soil Dynamics and Earthquake Engineering*, Karlsruhe, Germany.

Thomson W T, 1950, Transmission of elastic waves through a stratified solid medium, *J. Applied Physics*, v. 21, no. 2, pp. 89–93.

Wathelet M, Guillier B, Roux P, Cornou C and Ohrnberger M, 2018, Rayleigh wave three-component beamforming: signed ellipticity assessment from high-resolution frequency-wavenumber processing of ambient vibration arrays, *Geophysical Journal International*, Vol 215, p 507-523.

Xia J, Xu Y, Luo Y, Miller R, Cakir R and Zeng C, 2012, Advantages of using Multichannel analysis of Love waves (MALW) to estimate near-surface shear-wave velocity, *Surveys in Geophysics*, 841–860.

Yong A, Martin A, Stokoe K and Diehl J, 2013, ARRA-funded V_{S30} measurements using multi-technique method approach at strong-motion stations in California and Central-Eastern United States, Open-File Report 2013-1102, United States Geological Survey, http://pubs.usgs.gov/of/2013/1102/.

Yoon S and Rix G, 2009, Near-field effects on Array-based surface wave methods with active sources, *Journal of Geotechnical and Geoenvironmental Engineering*, Vol. 135, no. 3, 399-406.

8 CERTIFICATION

All geophysical data, analysis, interpretations, conclusions, and recommendations in this document have been prepared under the supervision of and reviewed by a **GEO*Vision*** California Professional Geophysicist.

Reviewed and approved by,

Antony J Martin

01/21/2021

Antony J. Martin
California Professional Geophysicist, P. Gp.
GEO*Vision* Geophysical Services

Date

ANTONY JEFFREY MARTIN
GP 989

* This geophysical investigation was conducted under the supervision of a California Professional Geophysicist using industry standard methods and equipment. A high degree of professionalism was maintained during all aspects of the project from the field investigation and data acquisition, through data processing interpretation and reporting. All original field data files, field notes and observations, and other pertinent information are maintained in the project files and are available for the client to review for a period of at least one year.

A professional geophysicist's certification of interpreted geophysical conditions comprises a declaration of his/her professional judgment. It does not constitute a warranty or guarantee, expressed or implied, nor does it relieve any other party of its responsibility to abide by contract documents, applicable codes, standards, regulations or ordinances.

GEOCON
WEST, INC.
GEOTECHNICAL ■ ENVIRONMENTAL ■ MATERIALS

Project No. A9899-06-01
May 25, 2021

8850 Sunset Blvd. LLC
c/o Mike Unwin
Plus Development Group
8920 West Sunset Boulevard, #200A
West Hollywood, California 90069

Subject: RESPONSE TO GEOTECHNICAL, GEOLOGY, AND SEISMIC REVIEW SHEET
PROPOSED HOTEL AND CONDOMINIUM DEVELOPMENT
8850 SUNSET BOULEVARD, WEST HOLLYWOOD, CALIFORNIA
APNS 4339-017- 001 THROUGH -008

References: *Geotechnical Investigation* prepared by Geocon West, Inc., dated February 17, 2021;

City of West Hollywood Geotechnical, Geology, and Seismic Review Sheet, Review No. 1, dated May 4, 2021.

Dear Mr. Unwin:

This letter has been prepared in response to the referenced Geotechnical, Geology, and Seismic Review Sheet prepared by the City of West Hollywood, consisting of six comments, dated May 4, 2021. A copy of the review letter is appended herein.

Comment 1: Site-Specific Ground Motion Hazard Analysis -- page 6

a) *Please explain why site-specific ground motion analysis is performed for this project.*

b) *Section 6.3.2: The text states that Vs,30 of 560 m/s was utilized in the UHT tool. It appears that the tool offers only 537 m/s option. Please explain or correct.*

c) *Section 6.3.2: UHT was utilized to develop the probabilistic RotD050 ARS. Please confirm that appropriate ratios associated with RotD050 accelerations from Shahi et al. were used to obtain maximum rotated direction ARS.*

d) *Section 6.3.3: USGS NSHMP-HAZ-WS Response Spectra application was used for the deterministic analysis. This application yields RotD050 accelerations for the 2014 NGA West 2 models. However, the text states that geometric mean accelerations were rotated to the maximum direction. Please clarify or correct.*

e) *Section 6.3.5: Please provide 2019 CBC ground motion parameters to develop 2019 CBC ARS. It is not clear why parameters for Site Class B are utilized in the provided table?*

f) *Please include comparison of the code and site-specific parameters and ARS (in a table and on Figure 9)*

g) *Section 6.3.7: Please verify that the* PGA_M *provided in the table is indeed geometric mean and not RotD050.*

h) *Figure numbers 6, 7 and 8 referenced in the text are incorrect. Figure 8 appears to have an incorrect title.*

Response 1: a) There is not a condition on this project which requires a site-specific ground motion analysis. The Building Code does allow a site-specific ground motion analysis to be used for any project, and the project elected to perform this study.

b) Acknowledged. The 560 m/s is a typographical error. A shear wave velocity **(**Vs30) of 537 m/s (Site Class C) was utilized in the UHT tool.

c) Confirmed, the rotation factors developed by Shahi and Baker (2013, 2014) were used to convert the RotD50 spectral accelerations to RotD100 (i.e. maximum rotated component).

d) Acknowledged, based on the use of the NGA-West2 GMPEs, the deterministic spectral accelerations are RotD50 values.

e) We assume that the reviewer is referring to the table provided under Section 6.3.5. These values are provided for reference only, and only the Class B parameters are provided because the values of S_{MS}, S_{M1}, S_{DS}, S_{D1} are superseded by the site-specific values.

At the reviewer's request, the following table summarizes site-specific design criteria obtained from the 2019 California Building Code (CBC; Based on the 2018 International Building Code [IBC] and ASCE 7-16), Chapter 16 Structural Design, Section 1613 Earthquake Loads. The data was calculated using the online application *Seismic Design Maps*, provided by OSHPD.

2019 CBC SEISMIC DESIGN PARAMETERS

Parameter	Value	2019 CBC Reference
Site Class	C	Section 1613.2.2
MCE$_R$ Ground Motion Spectral Response Acceleration – Class B (short), S_S	2.122g	Figure 1613.2.1(1)
MCE$_R$ Ground Motion Spectral Response Acceleration – Class B (1 sec), S_1	0.76g	Figure 1613.2.1(2)
Site Coefficient, F_A	1.2	Table 1613.2.3(1)
Site Coefficient, F_V	1.4	Table 1613.2.3(2)
Site Class Modified MCE$_R$ Spectral Response Acceleration (short), S_{MS}	2.546g	Section 1613.2.3 (Eqn 16-36)
Site Class Modified MCE$_R$ Spectral Response Acceleration – (1 sec), S_{M1}	1.064g	Section 1613.2.3 (Eqn 16-37)
5% Damped Design Spectral Response Acceleration (short), S_{DS}	1.697g	Section 1613.2.4 (Eqn 16-38)
5% Damped Design Spectral Response Acceleration (1 sec), S_{D1}	0.709g	Section 1613.2.4 (Eqn 16-39)

f) At the reviewer's request, we have plotted the MCE RotD100, MCE RotD50, DE RotD100, and General Code MCE response spectra on the same plot as Figure 1. We have also provided the values of these curves in tabular format on Figure 2.

g) Acknowledged. The use of the NGA-West 2 Ground Motion Prediction Equations (GMPEs) is considered to be the current standard of practice and they provide RotD50 spectral acceleration values. The prior set of GMPEs (the NGA-West set) were developed to provide geometric mean values of ground acceleration. It is the understanding of the undersigned engineer that the state of practice within the ground motion community is to use the RotD50 ground motions and that the reference to geometric mean ground motions within ASCE 7-16 is an artifact of the NGA-West GMPEs. The most important characteristic of ASCE 7-16 Section 21.5 is that the peak ground acceleration to be used in geotechnical evaluations should be calculated *without the risk coefficient*. The difference between geometric mean and RotD50 ground motions is very small, as discussed in Boore et al 2006 and Boore 2010. It is the opinion of the undersigned engineer that the PGA_M value presented in the referenced Geotechnical Report meets the intent of the Building Code.

h) Acknowledged. The referenced figure numbers 6, 7, and 8 should be revised to Figures 8, 9, and 10, respectively.

Comment 2: *Liquefaction Potential -- page 11* *a) It is notable that the Consultant utilizes 1996 method of analysis as there has been appreciable advance in the field. Specifically, Cetin (2004) and Idriss & Boulanger (2008, 2015) methods should be considered in modern analyses. Although analysis update is not requested, please comment.*

b) It is noted that the CBC 2019 §1803.5.12 clearly requires evaluation of liquefaction for MCE ground motions. The Consultant provides analyses also based on "Design Earthquake" and 2/3▪PGAM. It is understood that this approach is adopted by the City of Los Angeles, but it is not necessary in the city of West Hollywood. Although analysis update is not requested, please acknowledge.

Response 2: a) We have used the 1996 spreadsheet by Thomas Blake as a starting point and have updated it based on recent industry findings. It is the opinion of the undersigned engineer that the liquefaction analysis as presented in the referenced Geotechnical Investigation report dated February 17, 2021 may be relied upon.

b) Acknowledged.

Comment 3: Mat Foundation Design -- page 22

Please show determination of the provided reference subgrade modulus of 100 pci and 200 pci for the undisturbed and prepared mat subgrade respectively, considering the saturated conditions at the subgrade.

Response 3: The subgrade modulus values were provided as a starting point that should be reduced in accordance with the equation provided in the referenced report dated February 17, 2021. It is our understanding that the reduced subgrade modulus will then be revised using an iterative design process, which analyzes various subgrade moduli and the resulting settlements. The final subgrade modulus will be coordinated and verified with the project structural engineer.

Comment 4: Foundation Settlement -- page 24

Please provide calculations justifying the reported 1 inch of settlement considering the recommended allowable bearing pressure, the provided compressibility test results, presence of groundwater, and the large mat footprint.

Response 4: We have prepared an analysis of the anticipated foundation settlement using an excel spreadsheet based on Boussinesq theory for stress distribution. The settlement analysis is evaluated by modeling the mat foundation with 10-foot by 10-foot grid elements. The spreadsheet calculates the anticipated settlement at each grid element based on the Boussinesq theory for stress distribution, for each depth layer that has been specified, and includes consideration of the influence of the loading of all adjacent grid elements.

The largest anticipated mat foundation layout is assumed to be 240 feet by 140 feet in footprint. The project is not yet at a point where a structural engineer is available to provide input regarding the bearing pressure distribution. We do not anticipate a mat which is uniformly loaded to the maximum allowable bearing pressure; we anticipate that there will be increases in the bearing pressures around wall and column loads. Until a foundation layout becomes available, assumptions will have to be made. Therefore, we have modeled the mat foundation with a uniform load of 4.75 ksf, which is the anticipated average pressure. This settlement analysis should be updated once the structural engineer can provide the bearing pressure distribution diagram. We have also assumed that the pre-excavation pressures will not relax during excavation, and that the pressures will remain locked-in.

The values of the soil compression index (Cc) and recompression index (Cr) were evaluated based on the consolidation testing presented in the referenced Geotechnical Investigation report dated February 17, 2021. Groundwater was modeled at a depth of 20 feet below the ground surface.

The analysis was extended to a depth at which the increase in soil pressure from the mat foundation is less than 20 percent of the original soil pressure. We have also assumed a foundation rigidity factor of 0.8; this value will be discussed and confirmed by the structural engineer. The analysis of the anticipated settlements at the center of the uniformly loaded mat foundation is provided as Figure 3.

The maximum expected settlement for a mat foundation system with a uniform bearing pressure of 4.75 ksf and deriving support in the recommended bearing materials is estimated to be 1 inch. A majority of the settlement of the foundation system is expected to occur on initial application of loading; however, minor additional settlements are expected within the first twelve months. Differential settlement between the center and corner of the mat is expected to be less than ¾ inch.

Based on seismic considerations, the proposed structure supported on a reinforced concrete mat foundation should be designed for a combined static and seismically induced total settlement of approximately 1½ inches, and a combined static and seismically induced differential settlement of 1 inch between the center and corner of the mat.

Once the design and foundation loading configurations for the proposed structure proceeds to a more finalized plan, the estimated settlements presented in this report will require updating and the potential for settlement should be reevaluated by this office.

Comment 5: *Shoring -- Soldier Pile Design and Installation -- page 34*

Impacts on adjacent lots is an ongoing concern with infill construction in the City of West Hollywood when basement excavations extend to property lines and create a zero-lot-line condition. Consequently, the excavation design, shoring design, and eventually the basement wall design need to recognize this condition and implement measures that will protect the adjacent lots (buildings, flatwork, fence walls, etc.) from any distress due to excavation-induced ground movements. Specifically, selection of the loading diagrams and surcharge considerations must reflect the need to minimize movements behind the retaining structure. In practical terms this means designing for lateral pressures in excess of active pressures, e.g., at-rest pressures, or use of restrained shoring systems and prescribing minimal design shoring deflections. The Consultant needs to clearly reflect this approach in their design recommendations and highlight this critical construction issue to the Owner, the Design Team, and eventually the Contractor. Specifically, active pressure should not be recommended without qualifications that they may not be used for excavations extending near/to the property lines.

a) Please provide calculations justifying the provided trapezoidal loading diagram (21H ~ 26H). The provided values appear rather high.

b) Please clarify that loading diagrams based on active pressure are acceptable only within the site sufficiently away from the property lines.

Response 5: a) Calculation of the recommended shoring pressures are provided as Figures 4 through 7 herein. The trapezoidal pressure is based on the active earth pressure. If the shoring engineer determines that at-rest earth pressure are needed for this project, they will be provided under separate cover.

b) Acknowledged. We will rely on the shoring engineer to make a determination of the anticipated shoring deflections. The shoring engineer, design team, and Owner should be aware that the use of active earth pressures may result in deflections since movement in the shoring system would be required in order to achieve active pressures. As indicated in the review comment, the proximity of the shoring system to the property line and offsite improvements and any anticipated shoring deflections (which could manifest as settlement behind the shoring) should be considered in the shoring design.

Comment 6: *Anchor Testing -- page 37*

The provided tieback testing protocol is incomplete. It is missing a loading increment schedule, hold times, reading frequency, alternatives and courses of action if criteria are not met, etc. It is suggested the testing protocol should reference a sanctioned testing procedure, e.g., PTI DC35.1-14, rather than providing incomplete testing requirements.

Response 6: Acknowledged. Preliminary tieback testing protocols are provided in the referenced Geotechnical Investigation report. We also concur the sanctioned testing procedures outlined in PTI DC35.1-14 should be considered by the tieback specialty contractor to verify the suitability of installed anchors.

If you have any questions regarding this letter, or if we may be of further service, please contact the undersigned.

Very truly yours,

GEOCON WEST, INC.

Petrina Zen

REGISTERED PROFESSIONAL ENGINEER
PETRINA ZEN
C87489
CIVIL
STATE OF CALIFORNIA

REGISTERED PROFESSIONAL ENGINEER
JELISA THOMAS ADAMS
GE3092
GEOTECHNICAL
STATE OF CALIFORNIA

Petrina Zen
PE 87489

Jelisa Thomas Adams
GE 3092

Enclosures: Copy of Geotechnical, Geology, and Seismic Review Sheet
Figures 1 and 2, Design Response Spectra
Figure 3, Mat Foundation Settlement Calculation
Figures 4 through 7, Shoring Pressure Calculation

References:

Boore, D. M., J. Watson-Lamprey, and N. A. Abrahamson, 2006, *Orientation-independent measures of ground motion*, Bull. Seismol. Soc. Am. 96, 1502–1511.

Boore, D. M., 2010, *Orientation-Independent, Nongeometric-Mean Measures of Seismic Intensity from Two Horizontal Components of Motion*, Bull. Seismol. Soc. Am. 100, 11830-1835.

Shahi, S.K., Baker, J.W., 2013, *NGA-West2 Models for Ground-Motion Directionality*, Pacific Earthquake Engineering Research Center, PEER 2013/10.

Shahi, S.K., Baker, J.W., 2014, *NGA-West2 Models for Ground-Motion Directionality*, Earthquake Spectra, Volume 30, o.3, paged 1285-1300, August 2014.

Review No.1

City of West Hollywood
California 1984

CITY OF
WEST HOLLYWOOD

PLANNING AND DEVELOPMENT SERVICES DEPARTMENT
CURRENT AND HISTORIC PRESERVATION DIVISION

8300 Santa Monica Boulevard
West Hollywood, CA
90069-4313
Tel: (323) 848-6475
Fax: (323) 848-6569

GEOTECHNICAL, GEOLOGY, AND SEISMIC REVIEW SHEET

May 4, 2021

Tetra Tech BAS Project No.: cWH 18-56E
Site Address: 8850 Sunset Blvd., West Hollywood
Lot/Block/Tract: APNS 4339-017-001 through 008
Owner: 8850 Sunset Blvd, LLC
c/o Plus Development Group, West Hollywood, California

Project Type: New 14-story hotel and 15-story residential building over 5 levels of zero-lot-line subterranean parking

Geotechnical Engineer: Geocon West, Inc. (Jelisa T. Adams, GE 3092)
Engineering Geologist: Geocon West, Inc. (Susan F. Kirkgard, CEG 1754)

Reviewed Report: **February 17, 2021**, Proposed Hotel and Condominium Development, Project No.A9899-06-01

Previous Reviewed Report: none

<u>ACTION:</u>

_____ **APPROVAL recommended**

__X__ **RESPONSE TO COMMENTS required prior to approval**

The response to this review needs to specifically address each of the review comments. Please include this Review Sheet with the response. In order to expedite the review, an unlocked, legible, searchable pdf file format is recommended. If an addendum to a report is provided to support the response to the review comments, a cover sheet needs to be included which relates each review response to a specific section of the addendum.

1. Site-Specific Ground Motion Hazard Analysis -- page 6

a) Please explain why site-specific ground motion analysis is performed for this project.

b) Section 6.3.2: The text states that Vs,30 of 560 m/s was utilized in the UHT tool. It appears that the tool offers only 537 m/s option. Please explain or correct.

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e) Section 6.3.5: Please provide 2019 CBC ground motion parameters to develop 2019 CBC ARS. It is not clear why parameters for Site Class B are utilized in the provided table?

f) Please include comparison of the code and site-specific parameters and ARS (in a table and on Figure 9)

g) Section 6.3.7: Please verify that the PGA_M provided in the table is indeed geometric mean and not RotD050.

h) Figure numbers 6, 7 and 8 referenced in the text are incorrect. Figure 8 appears to have an incorrect title.

2. Liquefaction Potential -- page 11

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3. Mat Foundation Design -- page 22

Please show determination of the provided reference subgrade modulus of 100 pci and 200 pci for the undisturbed and prepared mat subgrade respectively, considering the saturated conditions at the subgrade.

4. Foundation Settlement -- page 24

Please provide calculations justifying the reported 1 inch of settlement considering the recommended allowable bearing pressure, the provided compressibility test results, presence of groundwater, and the large mat footprint.

5. Shoring -- Soldier Pile Design and Installation -- page 34

Impacts on adjacent lots is an ongoing concern with infill construction in the City of West Hollywood when basement excavations extend to property lines and create a zero-lot-line condition. Consequently, the excavation design, shoring design, and eventually the basement wall design need to recognize this condition and implement measures that will protect the adjacent lots (buildings, flatwork, fence walls, etc.) from any distress due to excavation-induced ground movements. Specifically, selection of the loading diagrams and surcharge considerations must reflect the need to minimize movements behind the retaining structure. In practical terms this means designing for lateral pressures in excess of active pressures, e.g., at-rest pressures, or use of restrained shoring systems and prescribing minimal design shoring deflections. The Consultant needs to clearly reflect this approach in their design recommendations and highlight this critical construction issue to the Owner, the Design Team, and eventually the Contractor. Specifically, active pressure should not be recommended without qualifications that they may not be used for excavations extending near/to the property lines.

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b) Please clarify that loading diagrams based on active pressure are acceptable only within the site sufficiently away from the property lines.

6. Anchor Testing -- page 37

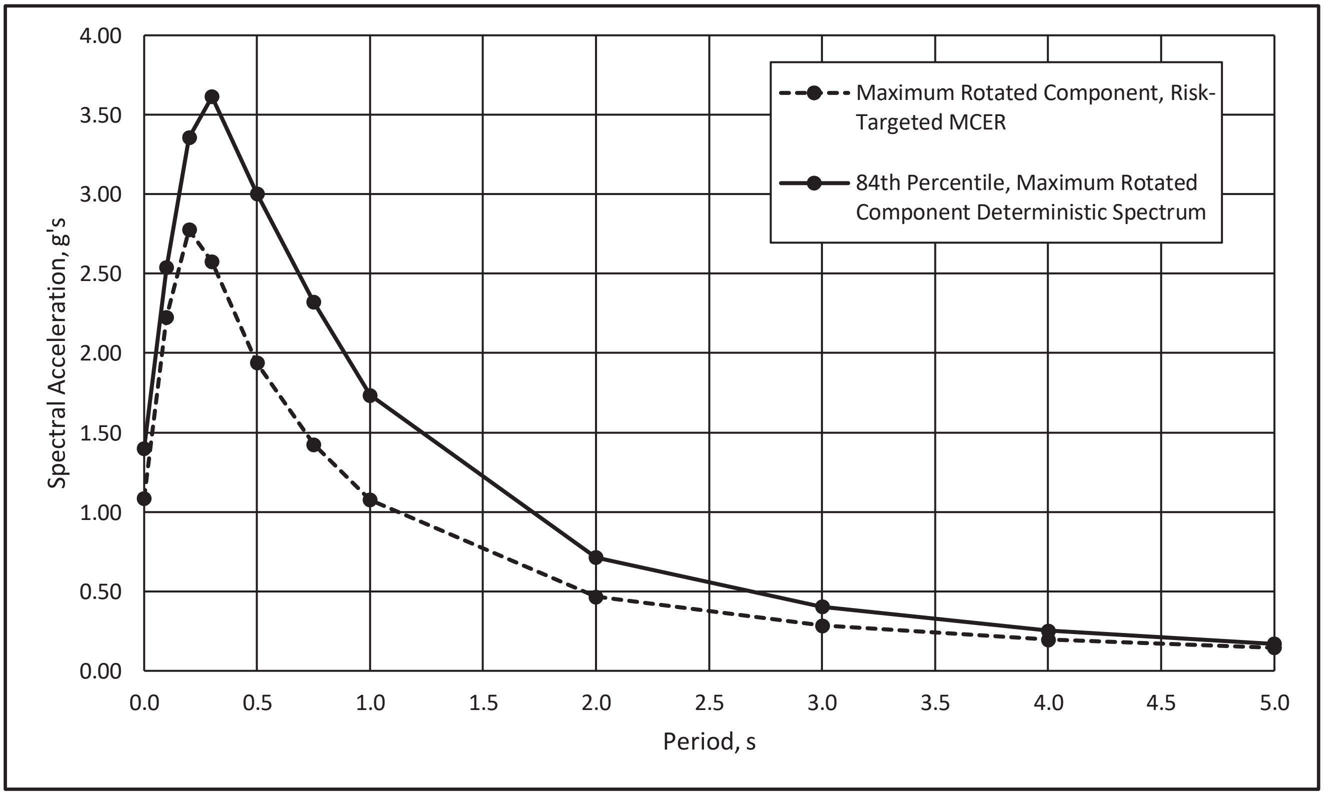
The provided tieback testing protocol is incomplete. It is missing a loading increment schedule, hold times, reading frequency, alternatives and courses of action if criteria are not met, etc. It is suggested the testing protocol should reference a sanctioned testing procedure, e.g., PTI DC35.1-14, rather than providing incomplete testing requirements.

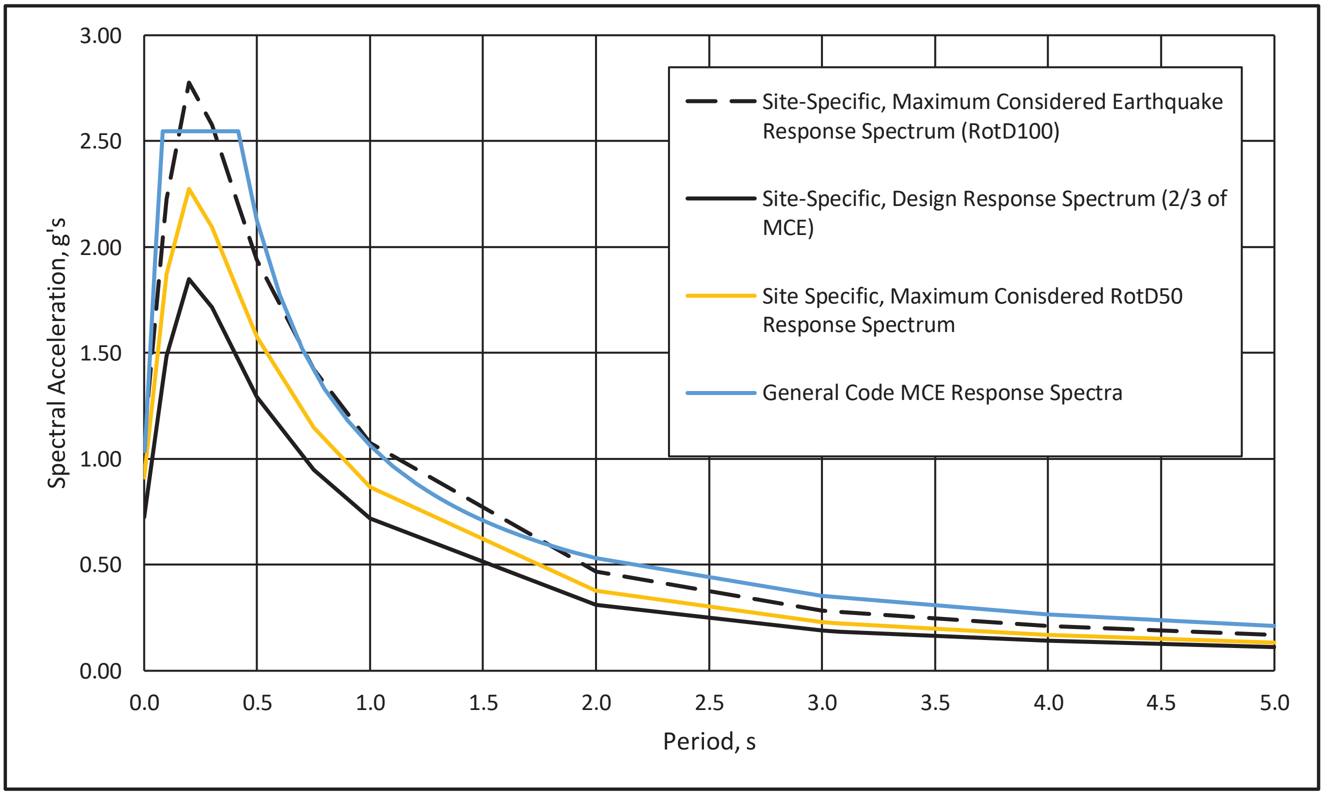
CLOSURE:

This document and all associated written comments and responses are an integral part of the geotechnical design documentation for this project. It is the responsibility of the Geotechnical Engineer of Record and the Owner to provide all herein referenced documents and the associated review comments to the appropriate parties for implementation into the design plans and specifications and for construction.

Reviewed by: ______________________ Date: May 4, 2021
Peter Skopek, PhD, GE2635
Geotechnical Engineering Reviewer

Filename: 2021-05-04 -- 8850 Sunset Blvd. -- 1st REV.docx





GEOCON

DESIGN RESPONSE SPECTRUM

Checked by: PZ

Project No.: A9899-06-01

8850 SUNSET BOULEVARD
WEST HOLLYWOOD, CALIFORNIA

MAY 2021

Figure 1

Spectral Period (seconds)	Probabilistic Uniform-Hazard	Risk-Targeted, Probabilistic	Risk Factor, Cr	Maximum-Rotated Componet Scale Factor	MRC, Risk-Targeted Probablistic	84th Percentile, Deterministic	Site-Specific Design Earthquake	80% Modifed General Response Spectrum	Site-Specific Maximum Considered Earthquake (RotD100)	Site-Specific Average Earthquake (RotD50)	General Code MCE Response Spectra
0.00	1.002	0.914	0.912	1.190	1.087	1.400	0.725	0.543	1.087	0.914	1.037
0.08	--	--	--	--	--	--	1.360	1.358	2.040	1.714	2.546
0.10	2.041	1.871	0.917	1.190	2.227	2.538	1.485	1.358	2.227	1.871	2.546
0.20	2.489	2.276	0.914	1.220	2.777	3.356	1.851	1.358	2.777	2.276	2.546
0.30	2.312	2.094	0.906	1.230	2.575	3.615	1.717	1.358	2.575	2.094	2.546
0.42	--	--	--	--	--	--	1.467	1.358	2.200	1.789	2.546
0.50	1.742	1.577	0.905	1.230	1.939	3.003	1.293	1.135	1.939	1.577	2.128
0.75	1.275	1.148	0.900	1.240	1.424	2.320	0.949	0.757	1.424	1.148	1.419
1.00	0.973	0.868	0.892	1.240	1.076	1.735	0.717	0.567	1.076	0.868	1.064
2.00	0.421	0.377	0.895	1.240	0.467	0.713	0.311	0.284	0.467	0.377	0.532
3.00	0.253	0.228	0.900	1.250	0.285	0.404	0.190	0.189	0.285	0.228	0.355
3.08	--	--	--	--	--	--	0.186	0.186	0.278	0.223	0.346
4.00	0.174	0.156	0.900	1.260	0.197	0.252	0.142	0.142	0.213	0.169	0.266
5.00	0.131	0.118	0.896	1.260	0.148	0.172	0.113	0.113	0.170	0.135	0.213

SM_S = 2.500 g
SM_1 = 1.076 g

SD_S = 1.666 g
SD_1 = 0.717 g

Reference: ASCE 7-16 21.4 DESIGN ACCELERATION PARAMETERS

Where the site-specific procedure is used to determine the design ground motion in accordance with Section 21.3, the parameter S_{DS} shall be taken as 90% of the maximum spectral acceleration, S_a, obtained from the site-specific spectrum, at any period within the range from 0.2 to 5 s, inclusive. The parameter S_{D1} shall be taken as the maximum value of the product, TS_a, for periods from 1 to 2 s for sites with $v_{s,30} > 1{,}200$ ft/s ($v_{s,30} > 365.76$ m/s) and for periods from 1 to 5 s for sites with $v_{s,30} \leq 1{,}200$ ft/=s ($v_{s,30} \leq 365.76$ m/s). The parameters S_{MS} and S_{M1} shall be taken as 1.5 times S_{DS} and S_{D1}, respectively. The values so obtained shall not be less than 80% of the values determined in accordance with Section 11.4.3 for S_{MS} and S_{M1} and Section 11.4.5 for S_{DS} and S_{D1}.

"--" Indicates that spectral period was not used at that calculation step

GEOCON

DESIGN RESPONSE SPECTRUM

Checked by: JTA

Project No.: A9899-06-01

8850 SUNSET BOULEVARD
WEST HOLLYWOOD, CALIFORNIA

MAY 2021

Figure 2

MAT FOUNDATION SETTLEMENT CALCULATION

Depth Offset 40
Rigidity Factor 0.8
Settlement Cuttoff 20 %
Location
x 120 (ft)
y 70 (ft)
Soil Unit Weight, γ (pcf): 120

Total 0.99 in

Depth from Datus (ft)	**Depth from GS (ft)**	**Existing Pressure, σ'ex (ksf)**	**Past Pressure, σ'_{VP} (psf)**	**Past Pressure, σ'_{VP} (ksf)**	**C_{ce}**	**C_{re}**	**Δq_v (ksf)**	**Total Pressure**	**ΔH (in)**	**Cumulative Settlement (in)**
0	40	3.552	3552	3.6	0.0339	0.0090	1.20	4.75		
2	42	3.667	3667	3.7	0.0339	0.0090	1.20	4.87	0.10	0.10
4	44	3.782	3782	3.8	0.0339	0.0090	1.20	4.98	0.10	0.20
6	46	3.898	3898	3.9	0.0214	0.0152	1.20	5.10	0.06	0.26
8	48	4.013	4013	4.0	0.0214	0.0152	1.20	5.21	0.06	0.32
10	50	4.128	4128	4.1	0.0214	0.0152	1.20	5.32	0.06	0.37
12	52	4.243	4243	4.2	0.0306	0.0065	1.20	5.44	0.08	0.45
14	54	4.358	4358	4.4	0.0306	0.0065	1.19	5.55	0.08	0.53
16	56	4.474	4474	4.5	0.0306	0.0065	1.19	5.67	0.08	0.60
18	58	4.589	4589	4.6	0.0390	0.0140	1.19	5.78	0.09	0.70
20	60	4.704	4704	4.7	0.0390	0.0140	1.19	5.89	0.09	0.79
22	62	4.819	4819	4.8	0.0331	0.0091	1.18	6.00	0.08	0.86
24	64	4.934	4934	4.9	0.0331	0.0091	1.18	6.11	0.07	0.94
26	66	5.050	5050	5.0	0.0246	0.0086	1.17	6.22	0.05	0.99
28	68	5.165	5165	5.2	0.0246	0.0086	1.17	6.33	0.05	1.04
30	70	5.280	5280	5.3	0.0246	0.0086	1.16	6.44	0.05	1.09
32	72	5.395	5395	5.4	0.0246	0.0086	1.15	6.55	0.05	1.14
34	74	5.510	5510	5.5	0.0246	0.0086	1.15	6.66	0.05	1.19
36	76	5.626	5626	5.6	0.0246	0.0086	1.14	6.76	0.05	1.24

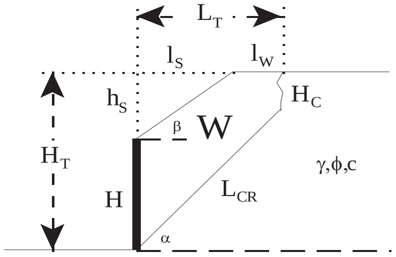
GEOCON

Project Name: 8850 Sunset
Project No.: A9899-06-01

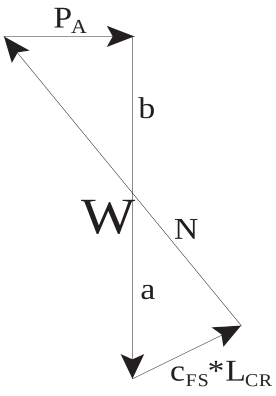
Shoring Design with Transitioned Backfill
(Vector Analysis)

Input:

Shoring Height	(H)	30.00	feet
Slope Angle of Backfill	(b)	0.0	degrees
Height of Slope above Shoring	(h_s)	0.0	feet
Horizontal Length of Slope	(l_s)	0.0	feet
Total Height (Shoring + Slope)	(H_T)	30.0	feet
Unit Weight of Retained Soils	(g)	125.0	pcf
Friction Angle of Retained Soils	(f)	33.0	degrees
Cohesion of Retained Soils	(c)	200.0	psf
Factor of Safety	(FS)	1.25	
Factored Parameters:	(f_{FS})	27.5	degrees
	(c_{FS})	160.0	psf



Failure Angle (a) degrees	Height of Tension Crack (H_C) feet	Area of Wedge (A) feet^2	Weight of Wedge (W) lbs/lineal foot	Length of Failure Plane (L_{CR}) feet	a lbs/lineal foot	b lbs/lineal foot	Active Pressure (P_A) lbs/lineal foot
45	5.3	436	54475.7	34.9	16431.7	38044.0	12029.5
46	5.1	422	52725.0	34.6	15426.0	37299.1	12514.1
47	5.0	408	51009.8	34.2	14518.9	36490.9	12955.8
48	4.8	395	49331.3	33.9	13697.9	35633.4	13356.1
49	4.7	382	47690.0	33.5	12952.5	34737.5	13716.3
50	4.6	369	46085.5	33.1	12273.6	33812.0	14037.8
51	4.5	356	44517.3	32.8	11653.3	32864.0	14321.7
52	4.4	344	42984.3	32.4	11085.1	31899.2	14568.9
53	4.4	332	41485.3	32.1	10563.1	30922.2	14780.2
54	4.3	320	40019.1	31.7	10082.5	29936.6	14956.5
55	4.3	309	38584.3	31.4	9638.7	28945.6	15098.3
56	4.3	297	37179.5	31.1	9228.1	27951.4	15206.0
57	4.2	286	35803.2	30.7	8847.2	26956.1	15280.2
58	4.2	276	34454.3	30.4	8493.1	25961.1	15320.9
59	4.2	265	33131.2	30.1	8163.3	24967.8	15328.5
60	4.2	255	31832.6	29.8	7855.4	23977.2	15302.8
61	4.2	244	30557.2	29.5	7567.3	22989.9	15243.8
62	4.3	234	29303.7	29.1	7297.1	22006.6	15151.3
63	4.3	225	28071.0	28.8	7043.3	21027.8	15025.0
64	4.4	215	26857.8	28.5	6804.1	20053.7	14864.4
65	4.4	205	25662.9	28.2	6578.3	19084.6	14669.0
66	4.5	196	24485.3	27.9	6364.5	18120.8	14438.2
67	4.6	187	23323.7	27.6	6161.5	17162.2	14171.1
68	4.7	177	22177.1	27.3	5968.2	16209.0	13866.7
69	4.8	168	21044.4	27.0	5783.4	15261.1	13524.2
70	4.9	159	19924.6	26.7	5606.0	14318.6	13142.2



Design Equations (Vector Analysis):

$a = c_{FS}*L_{CR}*\sin(90+f_{FS})/\sin(a-f_{FS})$

$b = W-a$

$P_A = b*\tan(a-f_{FS})$

$EFP = 2*P_A/H^2$

Maximum Active Pressure Resultant

$P_{A,\ max}$ 15328.5 lbs/lineal foot

Equivalent Fluid Pressure (per lineal foot of shoring)

$EFP = 2*P_A/H^2$

EFP 34.1 pcf

Design Shoring for an Equivalent Fluid Pressure: 34 pcf 21H

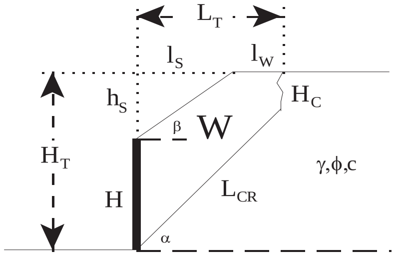
GEOCON

Project Name: 8850 Sunset
Project No.: A9899-06-01

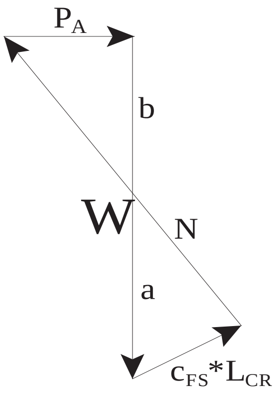
Shoring Design with Transitioned Backfill
(Vector Analysis)

Input:

Shoring Height	(H)	35.00	feet
Slope Angle of Backfill	(b)	0.0	degrees
Height of Slope above Shoring	(h_S)	0.0	feet
Horizontal Length of Slope	(l_s)	0.0	feet
Total Height (Shoring + Slope)	(H_T)	35.0	feet
Unit Weight of Retained Soils	(g)	125.0	pcf
Friction Angle of Retained Soils	(f)	33.0	degrees
Cohesion of Retained Soils	(c)	200.0	psf
Factor of Safety	(FS)	1.25	
Factored Parameters:	(f_{FS})	27.5	degrees
	(c_{FS})	160.0	psf



Failure Angle (a) degrees	Height of Tension Crack (H_C) feet	Area of Wedge (A) $feet^2$	Weight of Wedge (W) lbs/lineal foot	Length of Failure Plane (L_{CR}) feet	a lbs/lineal foot	b lbs/lineal foot	Active Pressure (P_A) lbs/lineal foot
45	5.3	598	74788.2	42.0	19761.8	55026.5	17399.4
46	5.1	579	72340.6	41.5	18528.6	53812.0	18054.3
47	5.0	560	69951.5	41.1	17420.1	52531.5	18650.8
48	4.8	541	67620.8	40.6	16419.7	51201.1	19191.1
49	4.7	523	65347.4	40.1	15513.7	49833.6	19677.2
50	4.6	505	63129.8	39.7	14690.4	48439.3	20110.7
51	4.5	488	60966.0	39.2	13939.9	47026.2	20493.4
52	4.4	471	58854.2	38.8	13253.6	45600.5	20826.5
53	4.4	454	56791.9	38.3	12624.4	44167.5	21111.3
54	4.3	438	54777.0	37.9	12045.9	42731.1	21348.7
55	4.3	422	52807.2	37.5	11512.6	41294.6	21539.6
56	4.3	407	50880.4	37.1	11019.9	39860.5	21684.8
57	4.2	392	48994.3	36.7	10563.7	38430.7	21784.6
58	4.2	377	47146.9	36.3	10140.2	37006.7	21839.5
59	4.2	363	45336.1	35.9	9746.3	35589.8	21849.6
60	4.2	348	43560.0	35.5	9379.1	34180.9	21815.0
61	4.2	335	41816.6	35.2	9036.1	32780.5	21735.6
62	4.3	321	40104.1	34.8	8715.0	31389.1	21611.0
63	4.3	307	38420.8	34.5	8413.7	30007.0	21440.9
64	4.4	294	36764.9	34.1	8130.5	28634.4	21224.6
65	4.4	281	35134.8	33.8	7863.6	27271.2	20961.4
66	4.5	268	33529.0	33.4	7611.5	25917.4	20650.4
67	4.6	256	31945.8	33.1	7372.8	24573.1	20290.3
68	4.7	243	30383.9	32.7	7146.0	23237.9	19880.0
69	4.8	231	28841.7	32.4	6929.9	21911.8	19417.9
70	4.9	219	27317.8	32.0	6723.3	20594.5	18902.5



Design Equations (Vector Analysis):

$a = c_{FS}*L_{CR}*\sin(90+f_{FS})/\sin(a-f_{FS})$

$b = W-a$

$P_A = b*\tan(a-f_{FS})$

$EFP = 2*P_A/H^2$

Maximum Active Pressure Resultant

$P_{A,\ max}$ 21849.6 lbs/lineal foot

Equivalent Fluid Pressure (per lineal foot of shoring)

$EFP = 2*P_A/H^2$

EFP 35.7 pcf

Design Shoring for an Equivalent Fluid Pressure: 36 pcf 23H

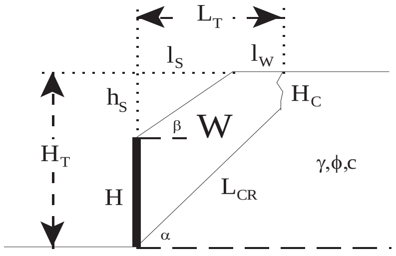
GEOCON

Project Name: 8850 Sunset
Project No.: A9899-06-01

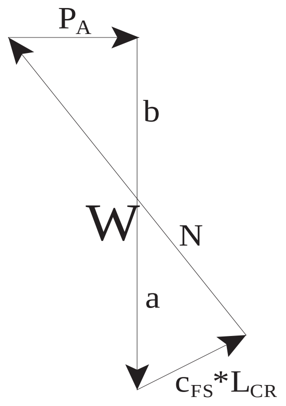
Shoring Design with Transitioned Backfill (Vector Analysis)

Input:

Shoring Height	(H)	55.00	feet
Slope Angle of Backfill	(b)	0.0	degrees
Height of Slope above Shoring	(h_S)	0.0	feet
Horizontal Length of Slope	(l_s)	0.0	feet
Total Height (Shoring + Slope)	(H_T)	55.0	feet
Unit Weight of Retained Soils	(g)	125.0	pcf
Friction Angle of Retained Soils	(f)	33.0	degrees
Cohesion of Retained Soils	(c)	200.0	psf
Factor of Safety	(FS)	1.25	
Factored Parameters:	(f_{FS})	27.5	degrees
	(c_{FS})	160.0	psf



Failure Angle (a) degrees	Height of Tension Crack (H_C) feet	Area of Wedge (A) $feet^2$	Weight of Wedge (W) lbs/lineal foot	Length of Failure Plane (L_{CR}) feet	a lbs/lineal foot	b lbs/lineal foot	Active Pressure (P_A) lbs/lineal foot
45	5.3	1498	187288.2	70.2	33082.0	154206.3	48760.0
46	5.1	1448	180980.6	69.3	30939.1	150041.4	50339.9
47	5.0	1399	174859.5	68.4	29024.9	145834.6	51777.3
48	4.8	1351	168916.2	67.5	27306.9	141609.4	53077.9
49	4.7	1305	163142.1	66.6	25758.6	137383.5	54246.9
50	4.6	1260	157528.5	65.8	24357.9	133170.6	55288.9
51	4.5	1217	152066.7	65.0	23086.2	128980.6	56208.1
52	4.4	1174	146748.8	64.2	21927.7	124821.0	57007.8
53	4.4	1133	141566.7	63.4	20869.3	120697.4	57691.2
54	4.3	1092	136513.0	62.6	19899.4	116613.6	58260.8
55	4.3	1053	131580.6	61.9	19008.3	112572.3	58718.7
56	4.3	1014	126762.6	61.2	18187.5	108575.1	59066.7
57	4.2	976	122052.7	60.5	17429.7	104623.0	59306.1
58	4.2	940	117444.7	59.9	16728.5	100716.3	59437.6
59	4.2	903	112933.0	59.2	16078.2	96854.8	59461.9
60	4.2	868	108511.9	58.6	15473.9	93038.0	59379.0
61	4.2	833	104176.3	58.0	14911.2	89265.1	59188.5
62	4.3	799	99921.4	57.5	14386.3	85535.1	58889.9
63	4.3	766	95742.4	56.9	13895.6	81846.7	58482.0
64	4.4	733	91634.8	56.4	13436.1	78198.7	57963.2
65	4.4	701	87594.4	55.8	13005.0	74589.4	57331.6
66	4.5	669	83617.2	55.3	12599.7	71017.6	56585.0
67	4.6	638	79699.3	54.8	12217.8	67481.5	55720.4
68	4.7	607	75836.8	54.3	11857.2	63979.6	54734.5
69	4.8	576	72026.4	53.8	11516.0	60510.4	53623.5
70	4.9	546	68264.4	53.3	11192.2	57072.2	52383.1



Design Equations (Vector Analysis):
$a = c_{FS}*L_{CR}*\sin(90+f_{FS})/\sin(a-f_{FS})$
$b = W-a$
$P_A = b*\tan(a-f_{FS})$
$EFP = 2*P_A/H^2$

Maximum Active Pressure Resultant

$P_{A,\ max}$ 59461.9 lbs/lineal foot

Equivalent Fluid Pressure (per lineal foot of shoring)

$EFP = 2*P_A/H^2$

EFP 39.3 pcf

Design Shoring for an Equivalent Fluid Pressure: 39 pcf 25H

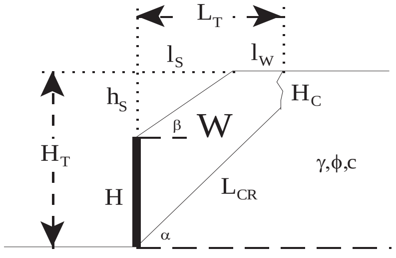
GEOCON

Project Name: 8850 Sunset
Project No.: A9899-06-01

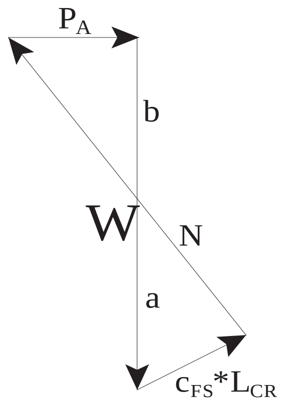
Shoring Design with Transitioned Backfill (Vector Analysis)

Input:

Shoring Height	(H)	74.00	feet
Slope Angle of Backfill	(b)	0.0	degrees
Height of Slope above Shoring	(h_S)	0.0	feet
Horizontal Length of Slope	(l_S)	0.0	feet
Total Height (Shoring + Slope)	(H_T)	74.0	feet
Unit Weight of Retained Soils	(g)	125.0	pcf
Friction Angle of Retained Soils	(f)	33.0	degrees
Cohesion of Retained Soils	(c)	200.0	psf
Factor of Safety	(FS)	1.25	
Factored Parameters:	(f_{FS})	27.5	degrees
	(c_{FS})	160.0	psf



Failure Angle (a) degrees	Height of Tension Crack (H_C) feet	Area of Wedge (A) $feet^2$	Weight of Wedge (W) lbs/lineal foot	Length of Failure Plane (L_{CR}) feet	a lbs/lineal foot	b lbs/lineal foot	Active Pressure (P_A) lbs/lineal foot
45	5.3	2724	340475.7	97.1	45736.1	294739.6	93196.6
46	5.1	2631	328912.0	95.7	42729.2	286182.9	96016.3
47	5.0	2542	317709.1	94.4	40049.4	277659.7	98580.6
48	4.8	2455	306846.9	93.1	37649.7	269197.2	100900.2
49	4.7	2370	296306.0	91.8	35491.2	260814.8	102984.6
50	4.6	2289	286068.0	90.6	33542.0	252526.1	104842.2
51	4.5	2209	276115.5	89.4	31775.1	244340.4	106480.4
52	4.4	2131	266432.0	88.3	30168.1	236263.8	107905.5
53	4.4	2056	257001.8	87.2	28701.9	228299.8	109123.1
54	4.3	1982	247810.3	86.1	27360.3	220450.0	110137.9
55	4.3	1911	238843.6	85.1	26129.2	212714.4	110953.7
56	4.3	1841	230088.9	84.1	24996.7	205092.2	111573.7
57	4.2	1772	221533.8	83.2	23952.4	197581.4	112000.0
58	4.2	1705	213166.9	82.3	22987.3	190179.6	112234.3
59	4.2	1640	204977.3	81.4	22093.5	182883.8	112277.6
60	4.2	1576	196954.7	80.6	21263.9	175690.8	112129.9
61	4.2	1513	189089.6	79.8	20492.6	168597.0	111790.7
62	4.3	1451	181372.6	79.0	19774.0	161598.6	111258.7
63	4.3	1390	173795.3	78.2	19103.4	154691.9	110532.0
64	4.4	1331	166349.3	77.5	18476.5	147872.9	109607.8
65	4.4	1272	159026.9	76.8	17889.3	141137.6	108482.6
66	4.5	1215	151820.7	76.1	17338.4	134482.3	107152.1
67	4.6	1158	144723.5	75.4	16820.6	127902.9	105611.2
68	4.7	1102	137728.6	74.8	16332.9	121395.7	103853.8
69	4.8	1047	130829.5	74.1	15872.8	114956.7	101873.2
70	4.9	992	124020.1	73.5	15437.7	108582.4	99661.2



Design Equations (Vector Analysis):

$a = c_{FS}*L_{CR}*\sin(90+f_{FS})/\sin(a-f_{FS})$

$b = W-a$

$P_A = b*\tan(a-f_{FS})$

$EFP = 2*P_A/H^2$

Maximum Active Pressure Resultant

$P_{A,\ max}$ 112277.6 lbs/lineal foot

Equivalent Fluid Pressure (per lineal foot of shoring)

$EFP = 2*P_A/H^2$

EFP 41.0 pcf

Design Shoring for an Equivalent Fluid Pressure: 41 pcf 26H

GEOCON
WEST, INC.
GEOTECHNICAL ■ ENVIRONMENTAL ■ MATERIALS

Project No. A9899-06-01
July 28, 2021

8850 Sunset Blvd. LLC
c/o Mike Unwin
Plus Development Group
8920 West Sunset Boulevard, #200A
West Hollywood, California 90069

Subject: RESPONSE TO GEOTECHNICAL, GEOLOGY, AND SEISMIC REVIEW SHEET
PROPOSED HOTEL AND CONDOMINIUM DEVELOPMENT
8850 SUNSET BOULEVARD, WEST HOLLYWOOD, CALIFORNIA
APNS 4339-017- 001 THROUGH -008

References: *Geotechnical Investigation* prepared by Geocon West, Inc., dated February 17, 2021;

City of West Hollywood Geotechnical, Geology, and Seismic Review Sheet, Review No. 1, dated May 4, 2021;

Response Letter prepared by Geocon West, Inc., dated May 25, 2021;

City of West Hollywood Geotechnical, Geology, and Seismic Review Sheet, Review No. 2, dated July 2, 2021.

Dear Mr. Unwin:

This letter has been prepared in response to the referenced Geotechnical, Geology, and Seismic Review Sheet prepared by the City of West Hollywood, consisting of five comments, dated July 2, 2021. A copy of the review letter is appended herein. The Geotechnical Report has been prepared in support of the Environmental Impact Report and, therefore, the analyses presented herein may require updating once the project design progresses.

Comment 1: Site-Specific Ground Motion Hazard Analysis

The responses are satisfactory. However, the following comments provided to be further addressed+:

a) It is estimated that the buildings will have natural period of about 1.3 to 1.7 seconds. It is noted that the site-specific design ARS yields higher spectral accelerations than the Code ARS for longer periods. In our experience this is not unusual, but it is a consideration for the structural designer. Please plot the design Code ARS and the site-specific design ARS on one plot.

We are not sure which reference in ASCE 7-16 mentioned in Item g is meant in the statement "… the reference to geometric mean ground motions within ASCE 7-16 is an artifact of the NGA-West GMPEs." However, we note that the use of geometric mean for liquefaction assessment is due to the historical context and how the liquefaction assessment methods were developed. No response to this item is requested.

Response 1: Acknowledged, the requested plot is provided on the lower half of Figure 1.

Comment 2: *Liquefaction Potential*

The analyses results are acceptable. However, please specify, what methods or approaches from the "recent industry findings" were utilized in the analyses.

Response 2: We have updated the 1996 spreadsheet by Thomas Blake by revising the Magnitude Scaling Factor (MSF) equation as well as calculating the overburden Correction Factor based on the insitu water and not historic high water.

Comment 3: Mat Foundation Design

The requested response was not provided. Please show the determination of the provided reference subgrade moduli of 100 pci and 200 pci for the undisturbed and prepared mat subgrade, respectively, considering the saturated conditions at the subgrade.

Response 3: Using the equation for subgrade modulus of:

$$k_s = {}^{q}/_{s}$$

Where ks = coefficient of subgrade reaction
q = applied bearing pressure at the ground surface
s = settlement

Using the maximum allowable bearing pressure of 6,500 psf and the maximum anticipated settlement (see response 4 below), the modulus of subgrade reaction for undisturbed soil subgrade may be taken as approximately 15 pci. This value is already adjusted for the approximate foundation dimensions analyzed in the updated settlement analyses. Since our settlement analyses did not consider the addition of a stabilization layer, the modulus of subgrade reaction for the stabilized condition should be evaluated at a future date.

The provided modulus of subgrade reaction is a preliminary value and should be further evaluated once the structural engineer can provide a bearing pressure plot. The final subgrade modulus will be coordinated and verified with the project structural engineer and submitted under an addendum report.

Comment 4: Foundation Settlement -- page 24

This is the key review comment. The provided settlement analyses could not be reviewed because of lack of information, nor could they be independently verified since significantly larger settlement magnitudes were calculated.

a) Please provide a reviewable settlement calculation with all relevant assumptions. Please consider:

The stated contact pressure is 4,750 psf. It seems that only surcharge in excess of the pre-excavation geostatic pressure is considered and so only 1,200 psf surcharge is considered. However, the settlement will take place under the full 4,750 psf surcharge. There will be a rebound due to excavation which will decrease the in-situ stresses followed by recompression settlement due to surcharge. This is not reflected in the presented analysis which seems to indicate that no settlement will take place until the last 1,200 psf of the total 4,750 psf will be installed.

- *It is recognized that the report states "A majority of the settlement of the foundation system is expected to occur on initial application of loading" but that aspect is not demonstrated by the analyses nor by the lab testing. Consider, that the subgrade will be fully saturated, and the groundwater will rise once the construction dewatering is completed.*
- *The calculated pre-excavation pressures are correct, but this stress condition is not rational as the initial pressure for settlement calculation because of the large excavation and the associated pressure relief.*
- *The presented calculation is significantly (and unconservatively) affected by the 20 percent cut-off. Whereas the concept is sound, the fact that it uses the "high" pre-excavation pressure and only "low" surcharge (1,200 psf, i.e., low) results in calculation depth for a 240 ft by 140 ft loaded area of only 36 feet. This is unconservative and unsubstantiated simply by inspection. This is also noted by observing that the settlement calculation is terminated when the surcharge increment dropped only by 5 percent (from 1,200 psf to 1,140 psf).*
- *The utilized compression indices are acceptable.*

b) Please confirm that the considered loaded area is 240 ft by 140 ft.

c) Please explain the statement: "The settlement analysis is evaluated by modeling the mat foundation with 10-foot by 10-foot grid elements." It was expected that closed-form solution for a Boussinesq's stress distribution for a given loading area size was utilized in the calculation spreadsheet instead of superposition of what appears to be 24 x 14 = 336 10x10-foot grid elements

Response 4: Acknowledged. Our response is provided below. As a part of the preparation of this letter, we had a conference call with the City of July 13, 2021 to discuss the review comments.

Settlement Analysis

Geocon previously prepared and submitted our settlement analyses that were based on consolidation theory and the Boussinesq theory of stress distribution and calculated using an in-house excel spreadsheet. In order to address the review comments, we have updated the settlement analyses to use the computer program Settle3 by Rocscience.

Our updated analysis considers the following assumptions:

- Existing grade ranges from ~358 to ~336 feet MSL
  - Up to 22 feet of relief onsite
  - Representative cross section drawn at roughly center of site in east west direction
- Subterranean will extend 68 feet below Sunset Blvd. and 40 feet below the south property line
- Assumed 6-foot-thick mat foundation
- Groundwater at 339 ft at north PL and 315 ft at south PL
  - Based on HHGW of 20 feet below grade
- Groundwater will be temporarily lowered to bottom of mat foundation
- Maximum recommended allowable bearing pressure of 6,500 psf
- Assumed 240 by 140 foot mat foundation

The soil properties used within Settle3D are indicated in the following table:

Layer No.	Layer Name	Layer Depth (Elevation)	Cce	Ccr	Consol Curve Reference	Total Unit Weight (pcf)
1	Upper Alluvium	47 – 65 (294 – 275)	0.038	0.009	AVG see discussion below	135
2	Lower Alluvium	66 - 86 (275 – 255)	0.055	0.005	B2@70’ (A9286-06-01)	135
3	Quartz Diorite	87+ (255+)	0.001	0.001	Incompressible	140

Geocon drilled three borings at the subject site ranging in depth from 60½ to 80½ feet. Review of the boring logs suggests that the soils are not sufficiently uniform for modeling of discreet soil layers. For the alluvial soils below the foundation level and to a depth of approximately 66 feet, we grouped the consolidation testing together and modeled this soil as soil layer 1, “Upper Alluvium”. We calculated the compression indices for all samples between depths of 45 and 66 feet. Based on our review of this data, we felt that taking an average of the compression indices would be appropriate for the settlement analysis. The following table presents the compression indices for each sample considered, as well as the average values.

Boring	Sample Depth (ft)	Ccε	Crε
B1	45	0.044	0.008
B1	50	0.045	0.007
B1	55	0.039	0.007
B1	60	0.049	0.006
B2	47.5	0.039	0.013
B2	52.5	0.033	0.009
B2	57.5	0.025	0.009
B3	45	0.044	0.007
B3	50	0.025	0.013
	Average	**0.038**	**0.009**

In boring B2, drilled to 80½, the blow counts below a depth of 66 feet generally increase and, therefore, we felt it would be appropriate to model this as a different soil layer. Since we did not perform consolidation testing at this depth, to supplement our data, we reviewed the boring logs and laboratory results from 8920 Sunset Boulevard (Geocon Project No. A9286-06-01). Based on our review, we feel that the soil sample from B2@70’ is similar in soil type and density to the material encountered at the subject site at the depth range of 62 to 81 feet. This sample was used for the basis of soil layer 2, “Lower Alluvium”. Although the blow counts were slightly higher, the compression index for the Lower Alluvium layer was higher than the Upper Alluvium and the recompression index is lower. Copies of the boring log and laboratory test results are provided in Appendix B.

We also used the boring logs referenced within the A9286-06-01 report for the property located at 8950 Sunset to estimate the depth to granitic bedrock. The borings which encountered bedrock were borings B4 and B5 by Kovacs Byer and boring B10 by GeoDesign. Copies of the boring logs are also provided in Appendix B. Based on the descriptions and blow counts from these boring logs, the quartz diorite is considered very dense and relatively incompressible at the bedrock contact. We modeled the Quartz Diorite as soil layer 3. Since the depth to bedrock is projected from borings performed to the east, additional borings should be considered in the future to confirm the depth to bedrock underlying the site and the settlement analysis updated as needed.

Within the Settle3 model, we modeled a non-horizontal ground surface and non-horizontal soil layers (see Figure C-1). We have assumed that the soil layering generally follows a similar slope to that of the underlying bedrock contact (i.e. the soil layers do not have a uniform elevation across the site). A site plan and the representative cross-section used for the settlement analysis are provided herein as Figures 2 and 3.

The settlement analysis was modeled using the following stages:

- Existing conditions
- GW lowered for construction (dewatering)
- Excavation
- Foundation Loading
- End of dewatering system

The dewatering is based on a preliminary assumption that the groundwater will be temporarily lowered to an elevation of 284 feet MSL, corresponding to the assumed lowest bottom of foundation. For the purpose of evaluating the anticipated settlement below the mat foundation, we consider this an acceptable assumption. Additional analyses will be required in the future to update the settlement based on the proposed dewatering system and associated drawdown curves. As a condition of approval, the future analyses will be necessary to show that dewatering will not result in a significant amount of settlement below the surrounding properties and right-of-way.

A uniform load of 6,500 pounds per square foot was applied across the foundation footprint. This is a preliminary assumption and was used to estimate the maximum total settlement. However, as a condition of approval, additional analysis will be required in the future once a structural engineer can provide a bearing pressure contour plot. Depending on the distribution of bearing pressures, the magnitude of differential settlement may be higher than those estimated using the analyses presented herein.

During the July 13, 2021 conference call, the City requested that we consider how construction staging may affect the foundation loading. Based on subsequent discussions with our Client, the proposed towers will be constructed simultaneously, therefore, staging of the foundation loads is not considered necessary.

The output plots of total settlement are provided at the lowest ground surface for Stages 1 and 2 (Existing Conditions & Dewatering, Elev 340) and at each foundation depth (294 and 284 feet MSL) for stages 3 through 5 (Excavation, Foundation Loading, End of Dewatering). Note, for the 294 ft MSL output, settlements are not shown in the upper half of the plot since this soil is “excavated” and, therefore, there are no settlements to display. An output report is also provided. Our analysis also uses query points to display the calculated settlement at specific locations. The output plots and report are provided as Figures C-2 through C-10.

Based on our updated settlement analyses, the maximum anticipated settlement for a mat foundation loaded with a bearing pressure of 6,500 psf is approximately 3⅓ inches. Differential settlement across the stepped transition between the two foundation levels is estimated to be approximately ¾ inch.

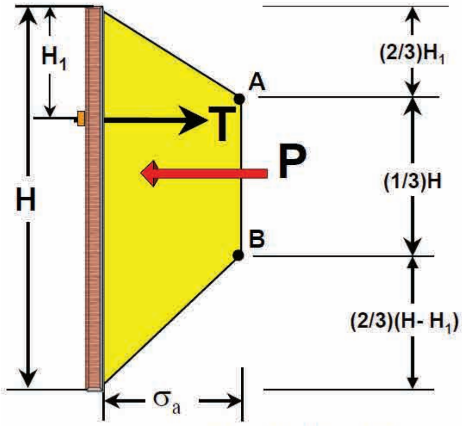
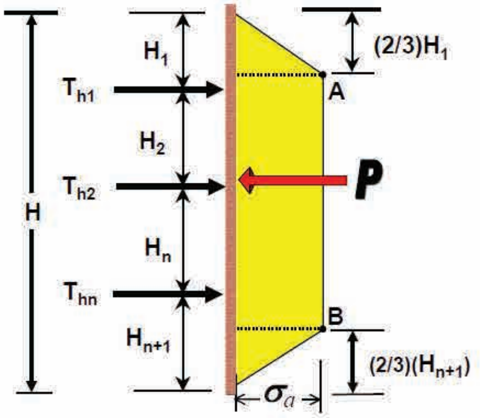
The anticipated settlements are preliminary and should be verified once the project structural engineer can provide a final diagram of the anticipated mat foundation bearing pressures.

Comment 5: *Shoring -- Soldier Pile Design and Installation*

The response is not satisfactory and is not consistent with the practice in the City of West Hollywood. Whereas design loading diagrams based on active pressure are acceptable where the shoring is outside the zone of influence to a property line, design of shoring along property lines needs to be based on either at-rest or restrained loading diagrams. All loading diagrams developed from calculations shown of Figure 4 through 6 are based simply on redistributed active pressure. It is suggested that loading diagrams based on redistributed at -rest pressure (very conservative) or FHWA or Caltrans methodology be utilized for development of the restrained loading diagrams.

Re. a) *The statement "If the shoring engineer determines that at-rest earth pressure are needed for this project, they will be provided under separate cover." is not appropriate. The determination of the design earth pressure is up to the Geotechnical Engineer. Please provide the appropriate design lateral earth pressure diagrams based on active and/or at rest/restrained pressures with clear guidance where the respective diagrams are to be used (i.e., at property lines v. away from property lines).*

Response 5: Acknowledged. A trapezoidal distribution of lateral earth pressure may be used where shoring will be restrained by bracing or temporary tie-backs. It is recommended that shoring be designed in accordance the equations provided below, which have been derived from the *State of California Department of Transportation Trenching and Shoring Manual, dated August 2011* for cohesionless soils. We reviewed the equations for both cohesionless and cohesive soils with respect to the soil properties underlying the site, and it is our opinion that the equations for cohesionless soil may be used. Diagrams depicting the trapezoidal pressure distribution of lateral earth pressure are provided on the following page.

For shoring with a single level of anchors or braces:	*For shoring with multilevel anchors:*
$\sigma_a = \frac{1.3P}{\frac{2}{3}H}$	$\sigma_a = \frac{1.3P}{[H - {}^{1}/_{3}(H_1 + H_{n+1})]}$
 Figure 7-2. Pressure Diagram for Single Anchored/Braced Wall	 Figure 7-3. Pressure Diagram for Multi Anchored/Braced Wall

where: $P = 18.4H^2$
H = total height of shoring (in feet)
H_1 = distance from top of shoring to uppermost anchor (in feet)
H_{n+1} = distance from bottom of shoring to lowermost anchor (in feet)
n = number of anchors

If you have any questions regarding this letter, or if we may be of further service, please contact the undersigned.

Very truly yours,

GEOCON WEST, INC.

C87489

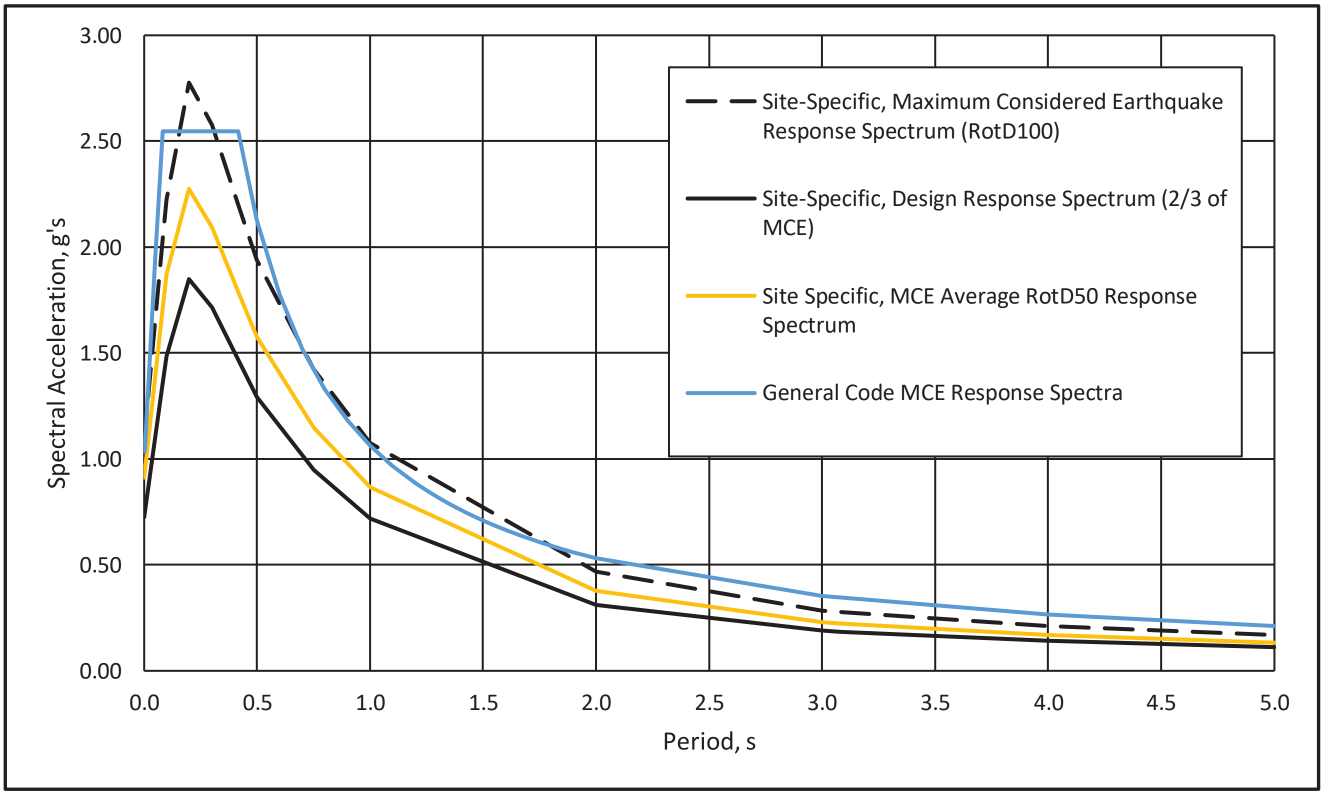
Petrina Zen

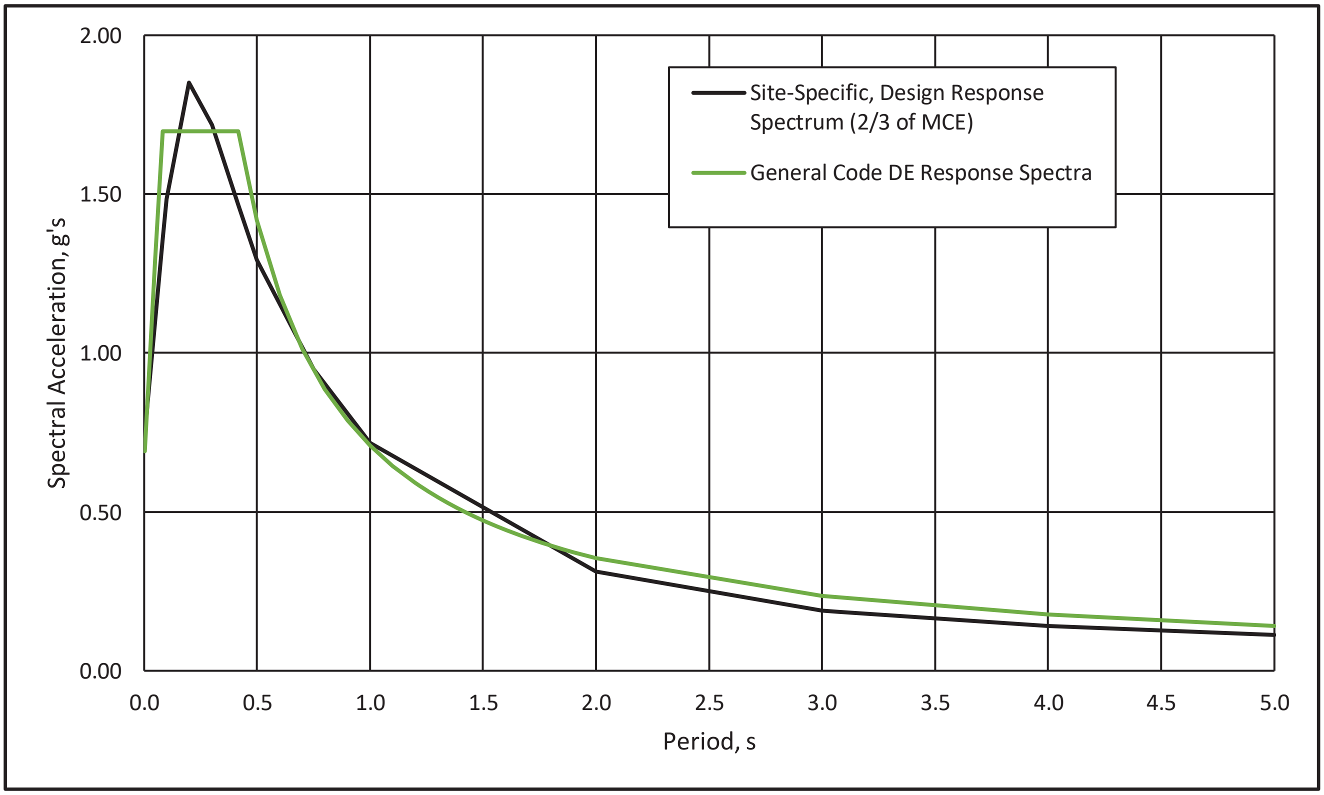
GE3092

Petrina Zen
PE 87489

Jelisa Thomas Adams
GE 3092

Enclosures: Copy of Geotechnical, Geology, and Seismic Review Sheet
Figure 1, Design Code ARS vs Site-Specific Design ARS
Figure 2, Site Plan
Figure 3, Cross-Section
Appendix A, Settle3 Analysis
Appendix B, Referenced Data – 8920 Sunset Blvd.





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DESIGN RESPONSE SPECTRUM

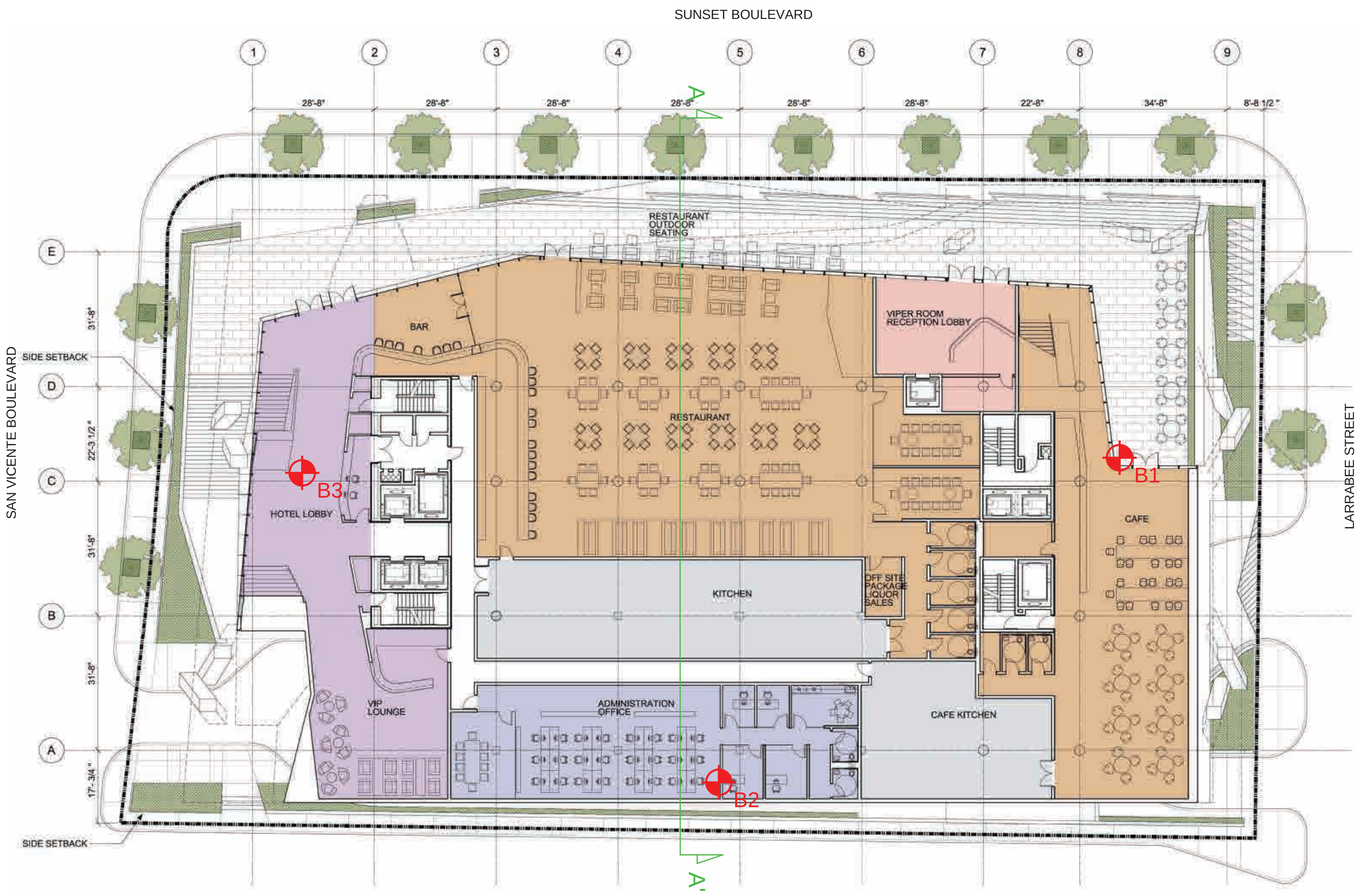
Checked by: JTA

Project No.: A9899-06-01

8850 SUNSET BOULEVARD
WEST HOLLYWOOD, CALIFORNIA

Jul 21

Figure 1



LEGEND

⊕ B3 Boring Location and Number

0 30' 60'

GEOCON
WEST, INC.

ENVIRONMENTAL GEOTECHNICAL MATERIALS
3303 N. SAN FERNANDO BLVD. - SUITE 100 - BURBANK, CA 91504
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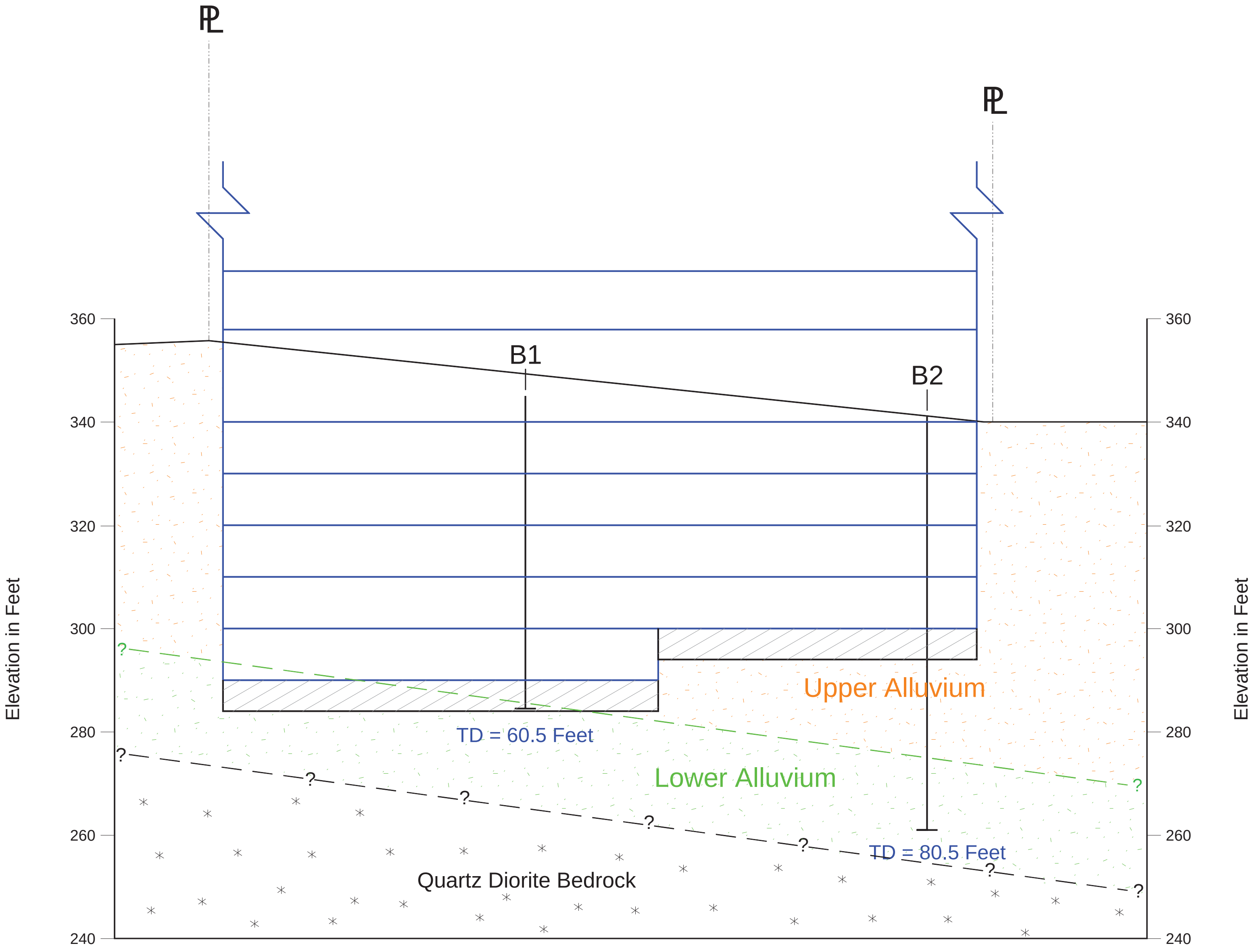
Drafted by: JTA
Checked by: NDB

SITE PLAN

8850 SUNSET BOULEVARD
WEST HOLLYWOOD, CALIFORNIA

JULY 2021	PROJECT NO. A9899-06-01	FIG. 2



Note: Section is for illustration purposes only; data is interpolated and extrapolated.
See report for additional description.

Scale: 1" = 30' (H&V)

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Drafted by: JTA	Checked by: NDB

CROSS-SECTION

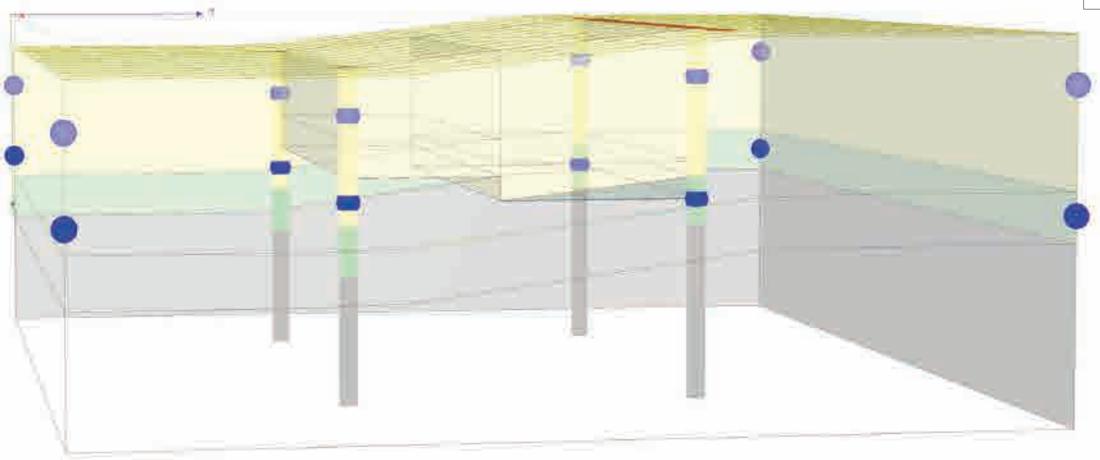
8850 SUNSET BOULEVARD
WEST HOLLYWOOD, CALIFORNIA

JULY 2021	PROJECT NO. A9899-06-01	FIG. 3

APPENDIX A

APPENDIX A

SETTLE3 ANALYSIS



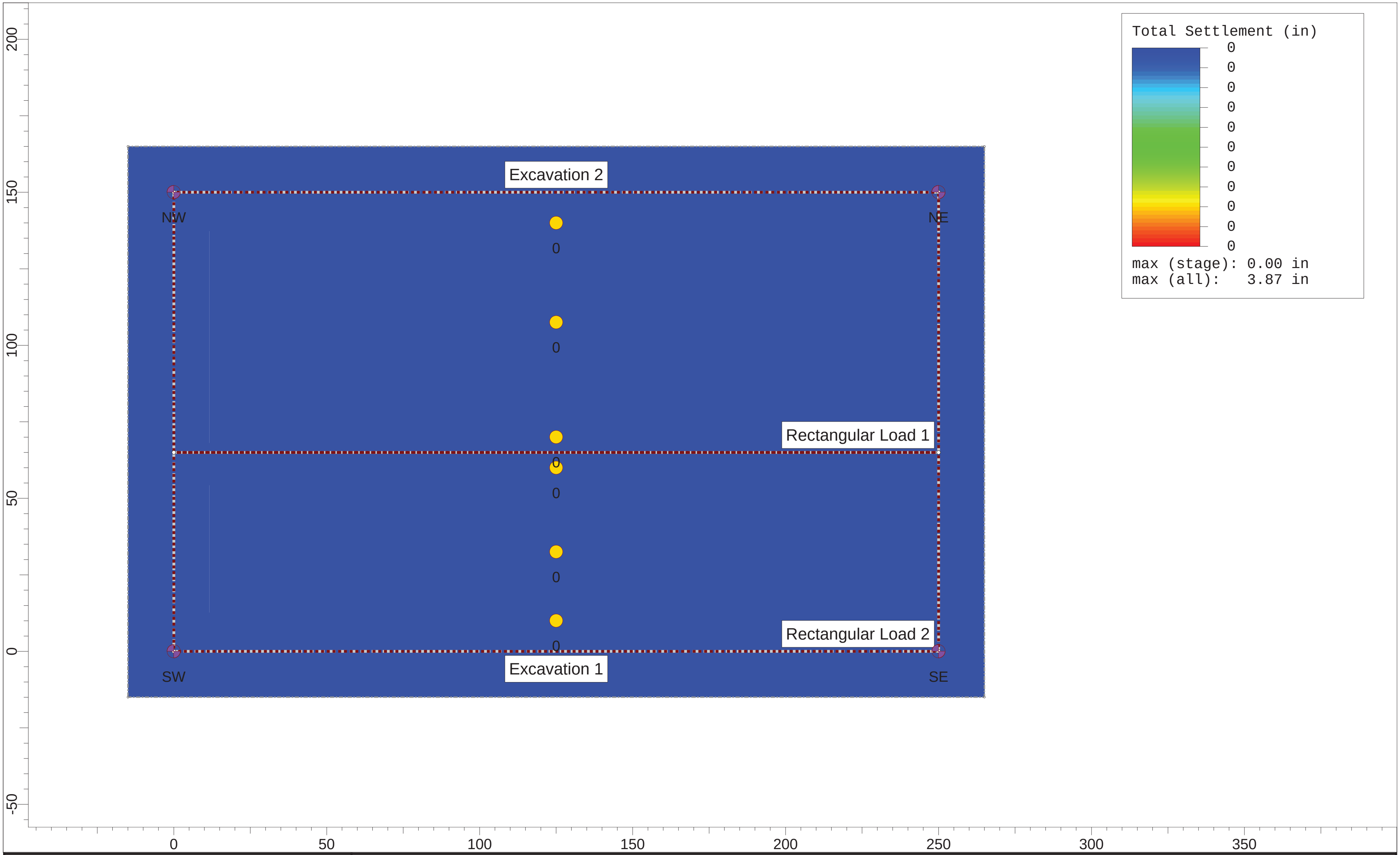
Project	8850 Sunset
Analysis Description	Settlement below Mat Foundation
Drawn By	JTA
Company	Geocon
Date	7/27/2021, 12:49:25 PM
File Name	Settle - 4 Boreholes GW.s3z

SETTLE3 5.011

GEOCON

Figure C-1

Stage: Existing - 340' msl



Project	8850 Sunset
Analysis Description	Settlement below Mat Foundation

Drawn By	JTA	*Company*	Geocon
Date	7/27/2021, 12:49:25 PM	*File Name*	Settle - 4 Boreholes GW.s3z

SETTLE3 5.011

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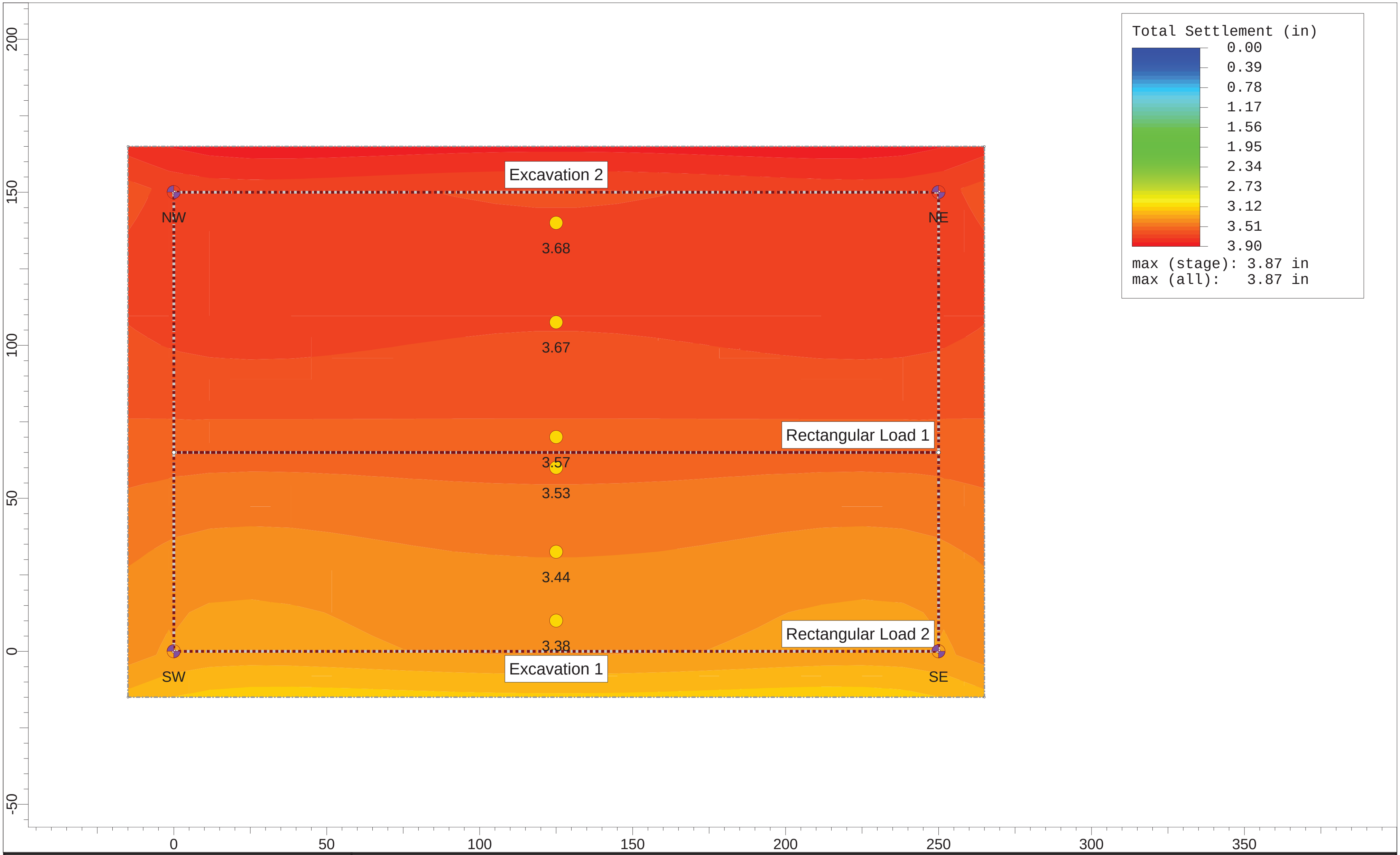
Figure C-2

Stage: Dewatering - 340' msl



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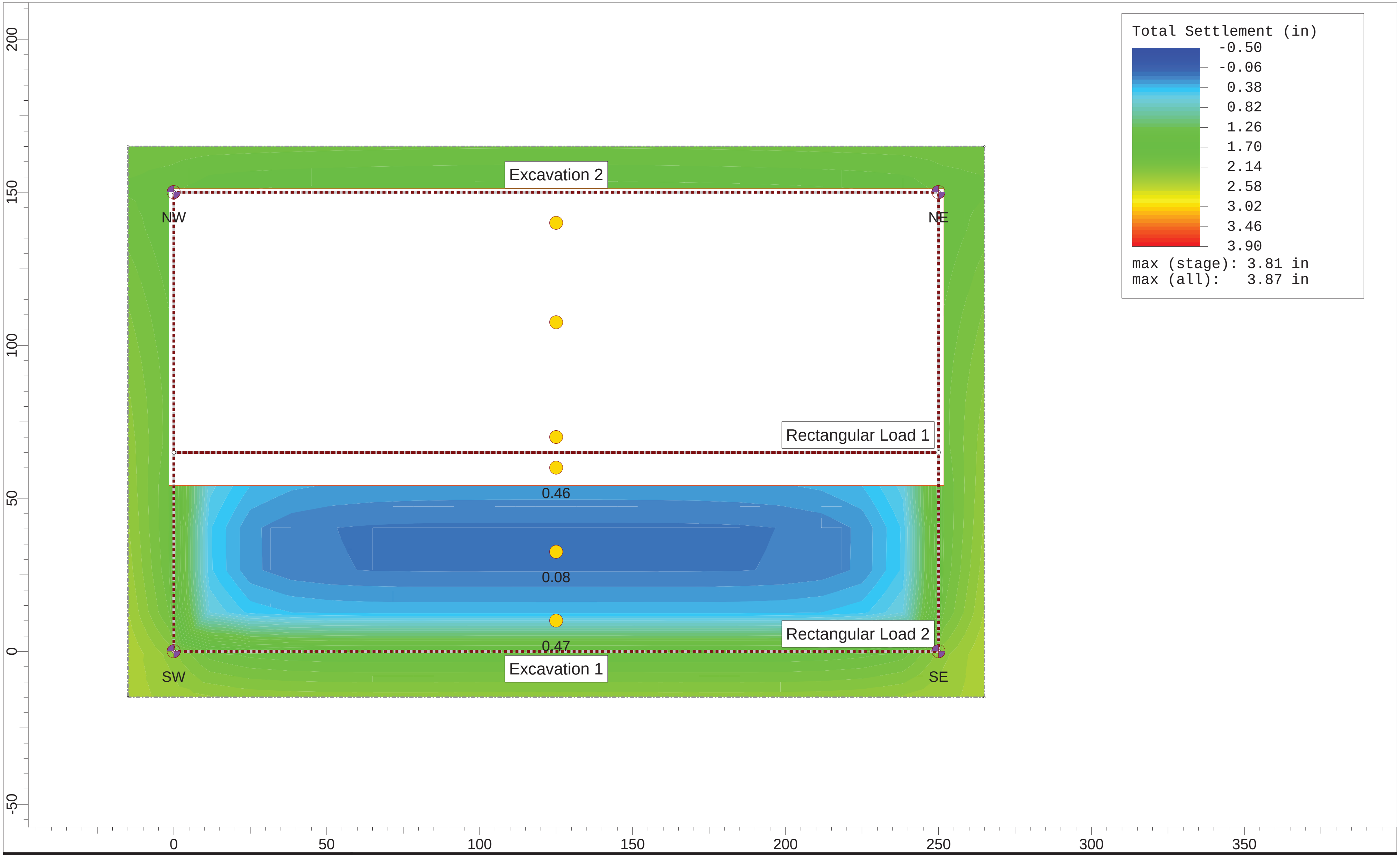
SETTLE3 5.011

Project	8850 Sunset
Analysis Description	Settlement below Mat Foundation

Drawn By	JTA	*Company*	Geocon
Date	7/27/2021, 12:49:25 PM	*File Name*	Settle - 4 Boreholes GW.s3z

Figure C-3

Stage: Excavation - 294' msl



SETTLE3 5.011

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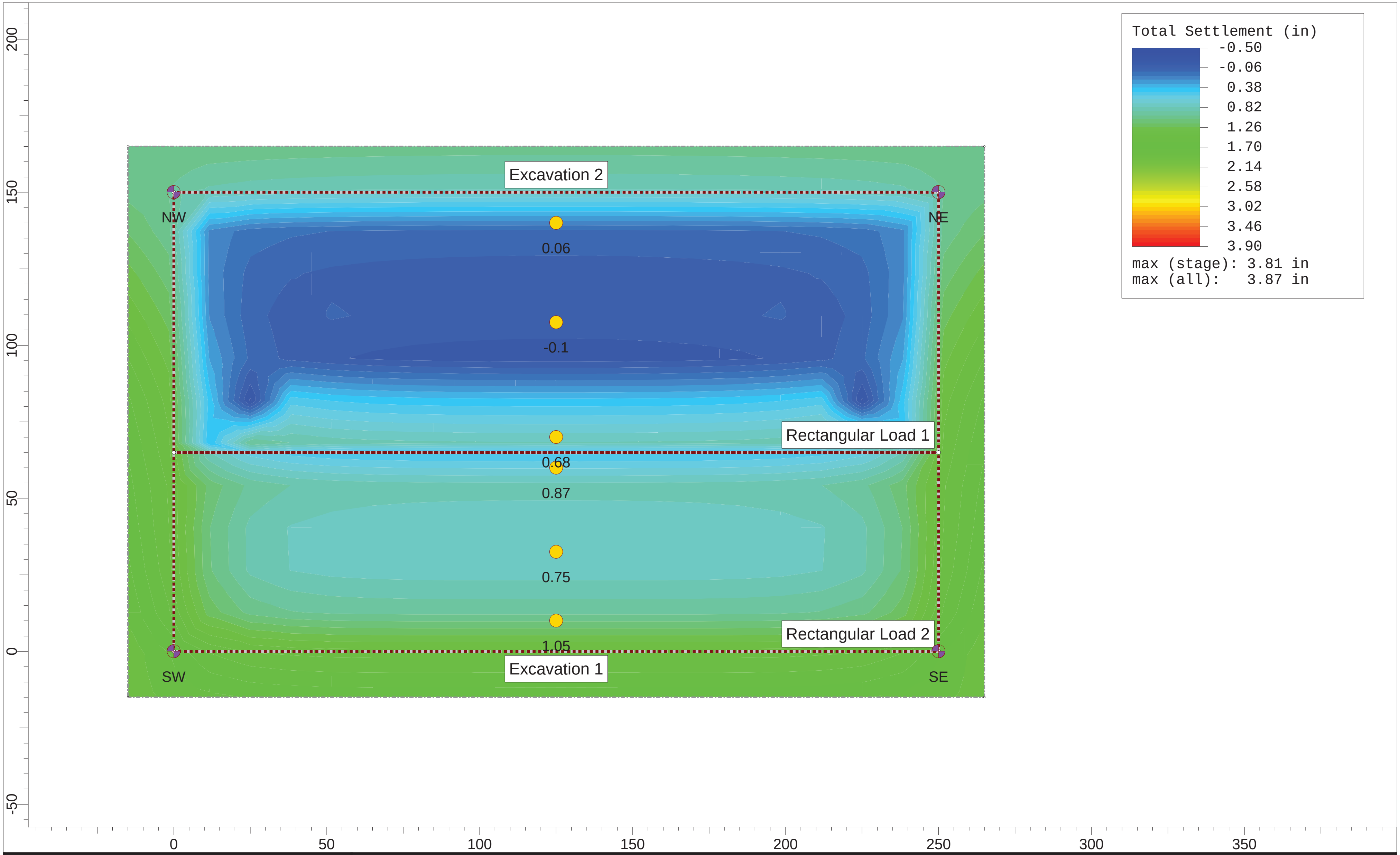
Project	8850 Sunset		
Analysis Description	Settlement below Mat Foundation		
Drawn By	JTA	*Company*	Geocon
Date	7/27/2021, 12:49:25 PM	*File Name*	Settle - 4 Boreholes GW.s3z

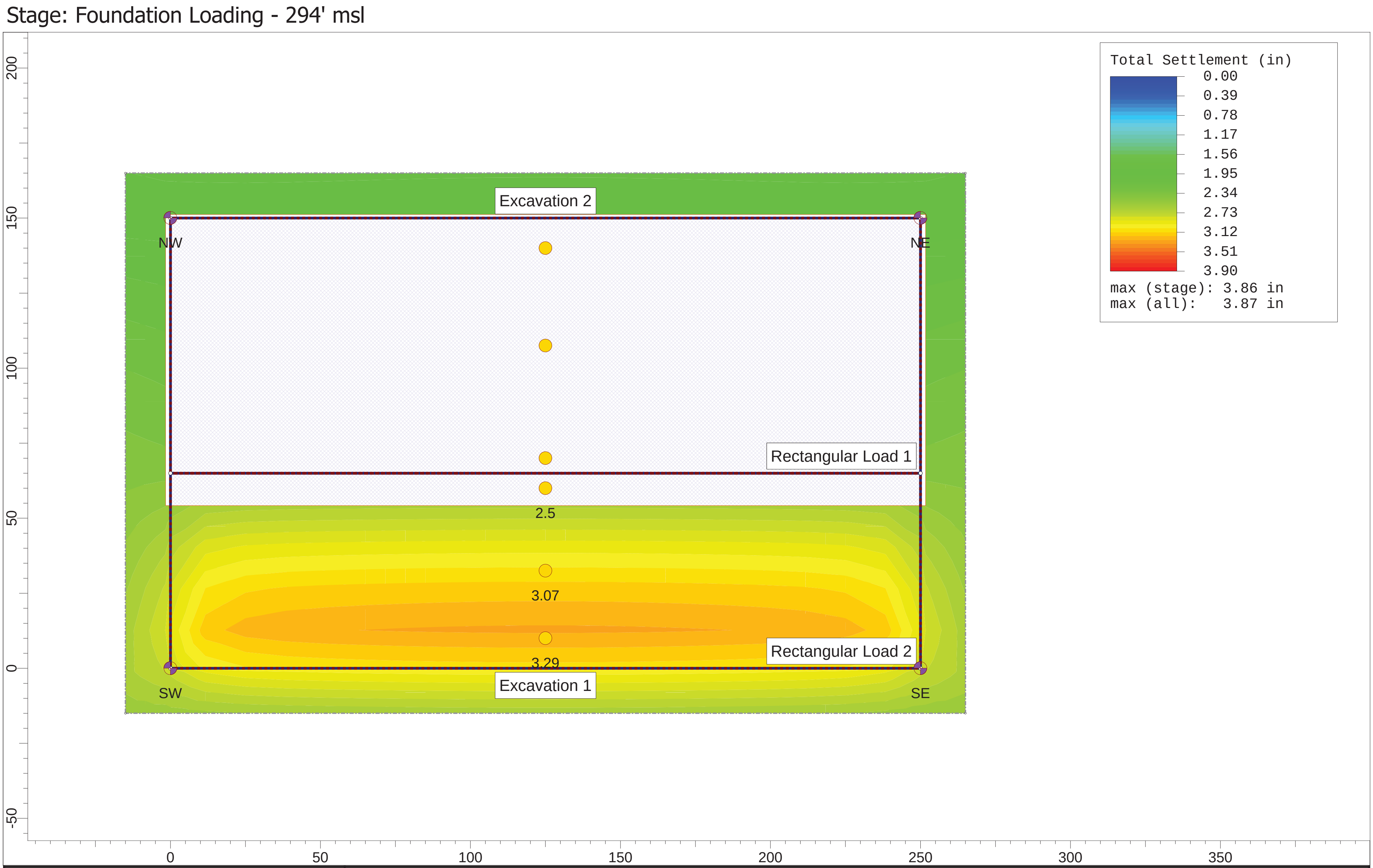
Figure C-4

Stage: Excavation - 284' msl



GEOCON	*Project* 8850 Sunset		
	Analysis Description Settlement below Mat Foundation		
	Drawn By JTA	*Company* Geocon	
SETTLE3 5.011	*Date* 7/27/2021, 12:49:25 PM	*File Name* Settle - 4 Boreholes GW.s3z	

Figure C-5



Project	8850 Sunset
Analysis Description	Settlement below Mat Foundation
Drawn By	JTA
Company	Geocon
Date	7/27/2021, 12:49:25 PM
File Name	Settle - 4 Boreholes GW.s3z

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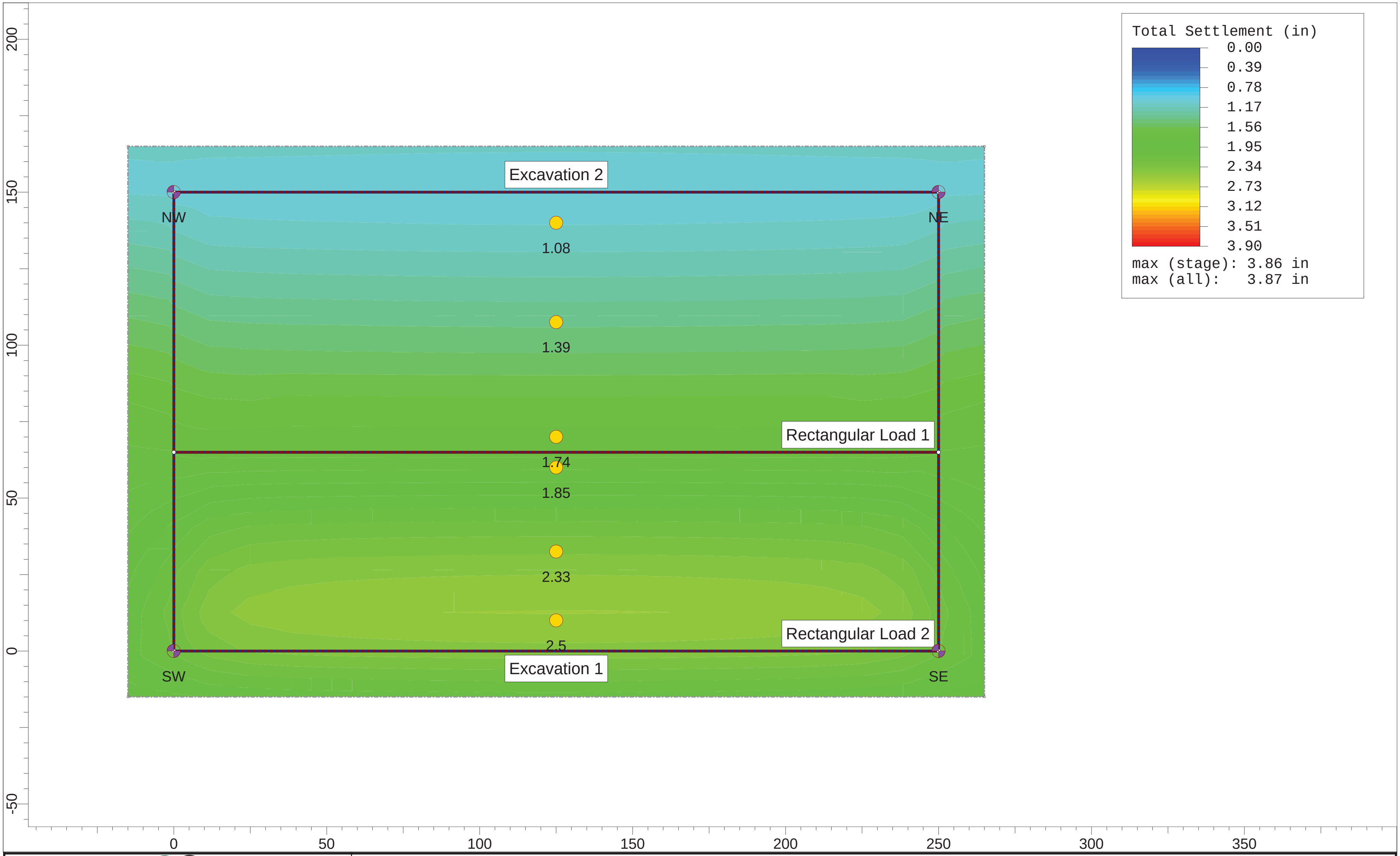
SETTLE3 5.011

Figure C-6

Stage: Foundation Loading - 284' msl



SETTLE3 5.011

Project 8850 Sunset

Analysis Description Settlement below Mat Foundation

Drawn By JTA

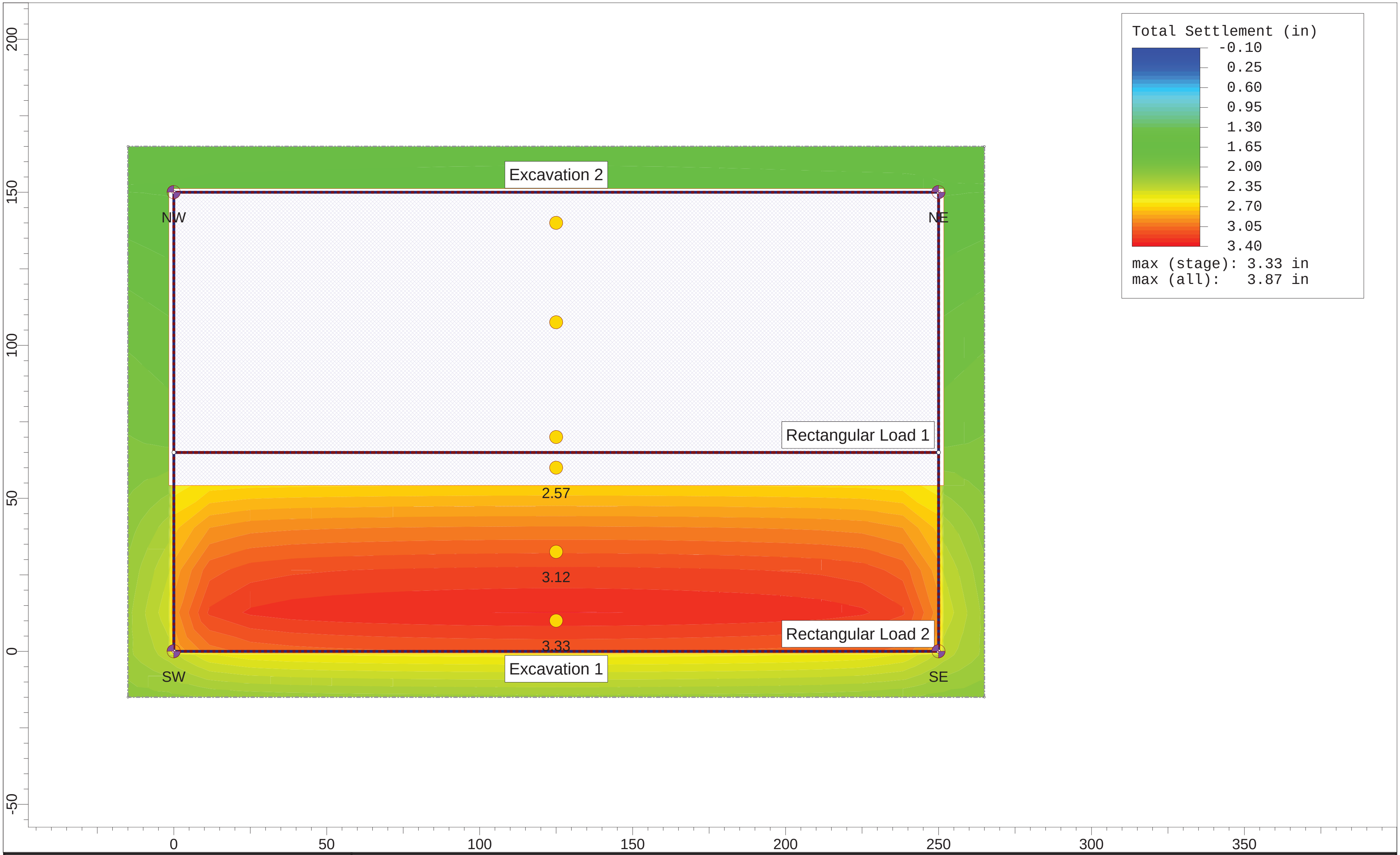
Company Geocon

Date 7/27/2021, 12:49:25 PM

File Name Settle - 4 Boreholes GW.s3z

Figure C-7

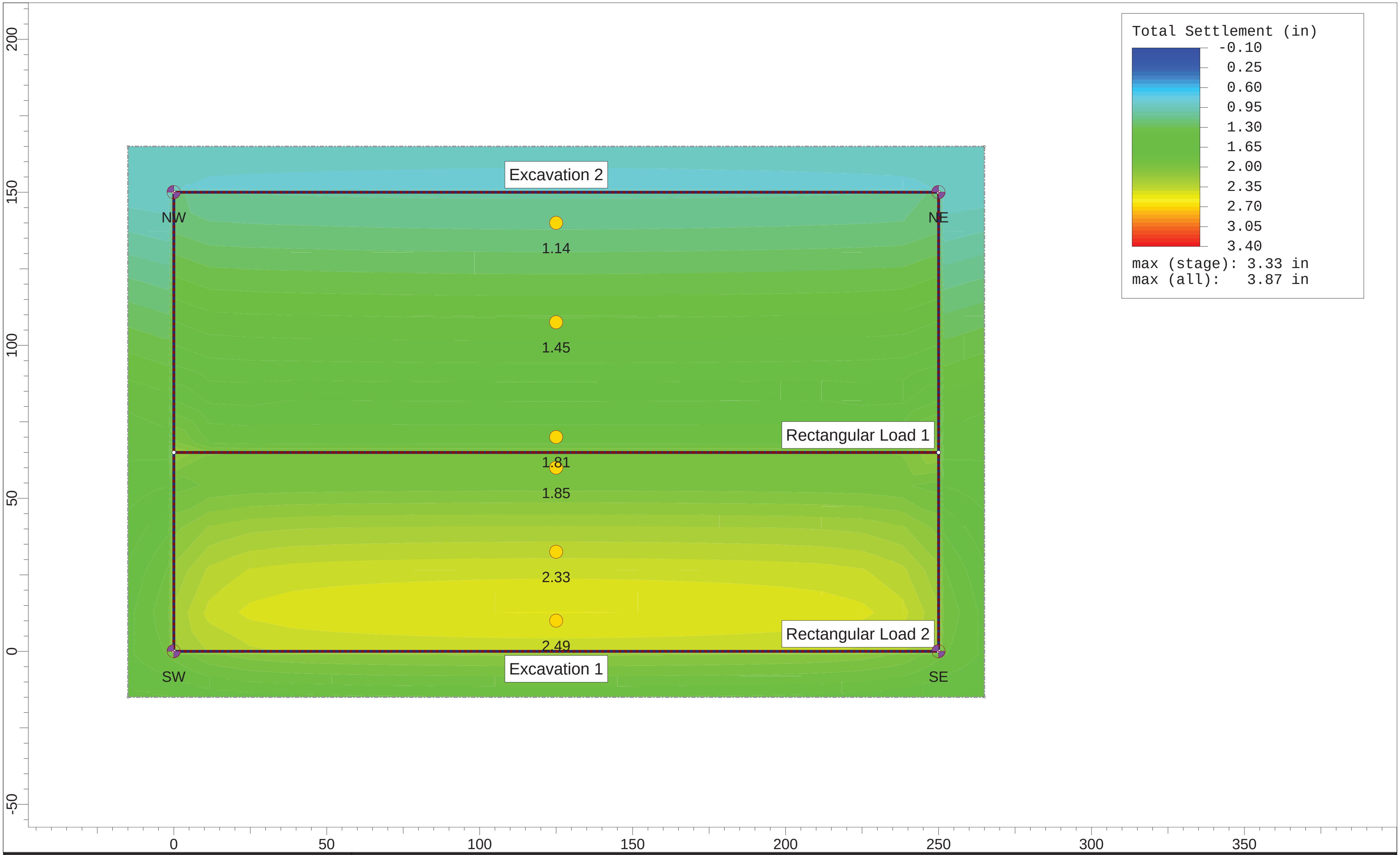
Stage: End of Dewatering - 294' msl



SETTLE3 5.011	GEOCON
Project	8850 Sunset
Analysis Description	Settlement below Mat Foundation
Drawn By	JTA
Company	Geocon
Date	7/27/2021, 12:49:25 PM
File Name	Settle - 4 Boreholes GW.s3z

Figure C-8

Stage: End of Dewatering - 284' msl



GEOCON

SETTLE3 5.011

Project	8850 Sunset
Analysis Description	Settlement below Mat Foundation

Drawn By	JTA	*Company*	Geocon
Date	7/27/2021, 12:49:25 PM	*File Name*	Settle - 4 Boreholes GW.s3z

Figure C-9

Figure C-10

Settle3 Analysis Information

Settle - 4 Boreholes GW

Project Settings

Document Name	Settle - 4 Boreholes GW.s3z
Date Created	7/27/2021, 12:49:25 PM
Stress Computation Method	Boussinesq
Minimum settlement ratio for subgrade modulus	0.9
Use average properties to calculate layered stresses	
Improve consolidation accuracy	
Ignore negative effective stresses in settlement calculations	

Stage Settings

Stage #	Name
1	Existing
2	Dewatering
3	Excavation
4	Foundation Loading
5	Termination of Dewatering

Results

Time taken to compute: 0 seconds

Stage: Existing

Data Type	Minimum	Maximum
Total Settlement [in]	0	0
Total Consolidation Settlement [in]	0	0
Virgin Consolidation Settlement [in]	0	0
Recompression Consolidation Settlement [in]	0	0
Immediate Settlement [in]	0	0
Loading Stress ZZ [ksf]	0	0
Loading Stress XX [ksf]	0	0
Loading Stress YY [ksf]	0	0
Effective Stress ZZ [ksf]	0	12.5196
Effective Stress XX [ksf]	0	12.5196
Effective Stress YY [ksf]	0	12.5196
Total Stress ZZ [ksf]	0	20.9117
Total Stress XX [ksf]	0	20.9117
Total Stress YY [ksf]	0	20.9117
Modulus of Subgrade Reaction (Total) [ksf/ft]	0	0
Modulus of Subgrade Reaction (Immediate) [ksf/ft]	0	0
Modulus of Subgrade Reaction (Consolidation) [ksf/ft]	0	0
Total Strain	0	0
Pore Water Pressure [ksf]	0	8.39205
Degree of Consolidation [%]	0	0
Pre-consolidation Stress [ksf]	0.023785	12.514
Over-consolidation Ratio	1	1
Void Ratio	1.1	1.1
Hydroconsolidation Settlement [in]	0	0
Undrained Shear Strength	0	0

Stage: Dewatering

Figure C-10

Data Type	Minimum	Maximum
Total Settlement [in]	0	3.67688
Total Consolidation Settlement [in]	0	3.67688
Virgin Consolidation Settlement [in]	0	3.67688
Recompression Consolidation Settlement [in]	0	0
Immediate Settlement [in]	0	0
Loading Stress ZZ [ksf]	0	0
Loading Stress XX [ksf]	0	0
Loading Stress YY [ksf]	0	0
Effective Stress ZZ [ksf]	0	14.9127
Effective Stress XX [ksf]	0	14.9127
Effective Stress YY [ksf]	0	14.9127
Total Stress ZZ [ksf]	0	20.1543
Total Stress XX [ksf]	0	20.1543
Total Stress YY [ksf]	0	20.1543
Modulus of Subgrade Reaction (Total) [ksf/ft]	0	0
Modulus of Subgrade Reaction (Immediate) [ksf/ft]	0	0
Modulus of Subgrade Reaction (Consolidation) [ksf/ft]	0	0
Total Strain	0	0.00791108
Pore Water Pressure [ksf]	0	5.2416
Degree of Consolidation [%]	0	100
Pre-consolidation Stress [ksf]	0.023785	14.9072
Over-consolidation Ratio	1	1
Void Ratio	1.08339	1.1
Hydroconsolidation Settlement [in]	0	0
Undrained Shear Strength	0	0

Stage: Excavation

Figure C-10

Data Type	Minimum	Maximum
Total Settlement [in]	-0.26436	1.05872
Total Consolidation Settlement [in]	-0.26436	1.05872
Virgin Consolidation Settlement [in]	0	2.52892
Recompression Consolidation Settlement [in]	-2.39006	0
Immediate Settlement [in]	0	0
Loading Stress ZZ [ksf]	-9.12696	-3.03639
Loading Stress XX [ksf]	-5.51234	-0.119374
Loading Stress YY [ksf]	-8.61867	0.459525
Effective Stress ZZ [ksf]	-1.43482	10.8511
Effective Stress XX [ksf]	2.07562	13.9942
Effective Stress YY [ksf]	-1.03071	13.6736
Total Stress ZZ [ksf]	-1.40301	16.0926
Total Stress XX [ksf]	2.07562	19.2358
Total Stress YY [ksf]	-1.03071	18.9152
Modulus of Subgrade Reaction (Total) [ksf/ft]	0	0
Modulus of Subgrade Reaction (Immediate) [ksf/ft]	0	0
Modulus of Subgrade Reaction (Consolidation) [ksf/ft]	0	0
Total Strain	-0.0135668	0.00777769
Pore Water Pressure [ksf]	0	5.2416
Degree of Consolidation [%]	0	100
Pre-consolidation Stress [ksf]	5.65775	14.9072
Over-consolidation Ratio	1	219.397
Void Ratio	1.08446	1.12398
Hydroconsolidation Settlement [in]	0	0
Undrained Shear Strength	0	0

Stage: Foundation Loading

Figure C-10

Data Type	Minimum	Maximum
Total Settlement [in]	0	3.29449
Total Consolidation Settlement [in]	0	3.29449
Virgin Consolidation Settlement [in]	0	3.29449
Recompression Consolidation Settlement [in]	-0.119983	0
Immediate Settlement [in]	0	0
Loading Stress ZZ [ksf]	-1.95584	0.855994
Loading Stress XX [ksf]	-0.960207	0.397958
Loading Stress YY [ksf]	-1.87365	1.00679
Effective Stress ZZ [ksf]	6.49994	14.2194
Effective Stress XX [ksf]	6.04192	14.4722
Effective Stress YY [ksf]	6.44388	14.2208
Total Stress ZZ [ksf]	6.49994	19.461
Total Stress XX [ksf]	6.04192	19.7138
Total Stress YY [ksf]	6.44388	19.4624
Modulus of Subgrade Reaction (Total) [ksf/ft]	0	0
Modulus of Subgrade Reaction (Immediate) [ksf/ft]	0	0
Modulus of Subgrade Reaction (Consolidation) [ksf/ft]	0	0
Total Strain	4.28947e-05	0.00839564
Pore Water Pressure [ksf]	0	5.2416
Degree of Consolidation [%]	0	100
Pre-consolidation Stress [ksf]	6.29969	14.9072
Over-consolidation Ratio	1	1.29641
Void Ratio	1.08237	1.09991
Hydroconsolidation Settlement [in]	0	0
Undrained Shear Strength	-0.000680844	0.0318593

Stage: Termination of Dewatering

Figure C-10

Data Type	Minimum	Maximum
Total Settlement [in]	0	3.3336
Total Consolidation Settlement [in]	0	3.3336
Virgin Consolidation Settlement [in]	0	3.34356
Recompression Consolidation Settlement [in]	-0.0565588	0.018563
Immediate Settlement [in]	0	0
Loading Stress ZZ [ksf]	-1.95584	0.855994
Loading Stress XX [ksf]	-0.960207	0.397958
Loading Stress YY [ksf]	-1.87365	1.00679
Effective Stress ZZ [ksf]	6.90513	14.9768
Effective Stress XX [ksf]	6.44709	15.2295
Effective Stress YY [ksf]	6.84957	14.6783
Total Stress ZZ [ksf]	6.90513	20.2184
Total Stress XX [ksf]	6.44709	20.4711
Total Stress YY [ksf]	6.84957	20.0176
Modulus of Subgrade Reaction (Total) [ksf/ft]	0	0
Modulus of Subgrade Reaction (Immediate) [ksf/ft]	0	0
Modulus of Subgrade Reaction (Consolidation) [ksf/ft]	0	0
Total Strain	5.71707e-05	0.00838591
Pore Water Pressure [ksf]	0	5.8656
Degree of Consolidation [%]	0	100
Pre-consolidation Stress [ksf]	6.49043	14.9707
Over-consolidation Ratio	1	1.21598
Void Ratio	1.08239	1.09988
Hydroconsolidation Settlement [in]	0	0
Undrained Shear Strength	-0.0305847	0.0318593

Loads

1. Rectangular Load: "Rectangular Load 1"

Length	250 ft
Width	85 ft
Rotation angle	0 degrees
Load Type	Flexible
Area of Load	21250 ft2
Load	6.5 ksf
Elevation	284 ft
Installation Stage	Foundation Loading

Coordinates

X [ft]	Y [ft]
7.10543e-14	65
250	65
250	150
7.10543e-14	150

2. Rectangular Load: "Rectangular Load 2"

Length	250 ft
Width	65 ft
Rotation angle	0 degrees
Load Type	Flexible
Area of Load	16250 ft2
Load	6.5 ksf
Elevation	294 ft
Installation Stage	Foundation Loading

Coordinates

X [ft]	Y [ft]
3.69482e-13	4.26326e-14
250	4.26326e-14
250	65
3.69482e-13	65

Excavations

1. Excavation: "Excavation 1"

Bottom Elevation	294 ft
Installation Stage	Excavation

Coordinates

X [ft]	Y [ft]
0	0
250	0
250	65
0	65

2. Excavation: "Excavation 2"

Bottom Elevation	284 ft
Installation Stage	Excavation

Coordinates

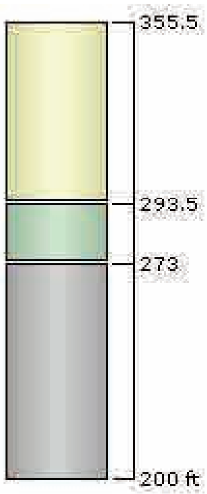
X [ft]	Y [ft]
0	65
250	65
250	150
0	150

Soil Layers

NW

XY Location: NW: (0, 150)

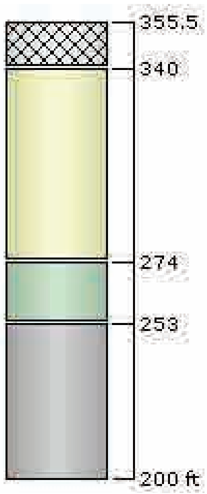
Layer #	Type	Thickness [ft]	Elevation [ft]
1	Upper Alluvium	62	355.5
2	Lower Alluvium	20.5	293.5
3	Quartz Diorite	73	273



SW

XY Location: SW: (0, 0)

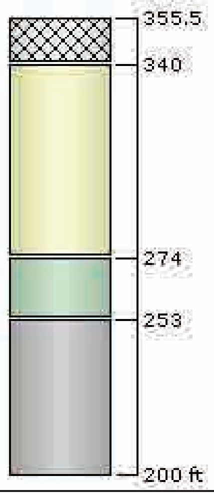
Layer #	Type	Thickness [ft]	Elevation [ft]
1	Upper Alluvium	66	340
2	Lower Alluvium	21	274
3	Quartz Diorite	53	253



SE

Figure C-10

XY Location:		SE: (250, 0)	
Layer #	**Type**	**Thickness [ft]**	**Elevation [ft]**
1	Upper Alluvium	66	340
2	Lower Alluvium	21	274
3	Quartz Diorite	53	253



NE

XY Location:		NE: (250, 150)	
Layer #	**Type**	**Thickness [ft]**	**Elevation [ft]**
1	Upper Alluvium	62	355.5
2	Lower Alluvium	20.5	293.5
3	Quartz Diorite	73	273

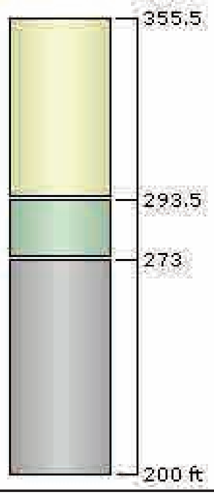


Figure C-10

Soil Properties

Property	Upper Alluvium	Lower Alluvium	Quartz Diorite
Color	🟨	🟩	◼
Unit Weight [kips/ft3]	0.12	0.12	0.14
Saturated Unit Weight [kips/ft3]	0.135	0.135	0.14
K0	1	1	1
Primary Consolidation	Enabled	Enabled	Enabled
Material Type	Non-Linear	Non-Linear	Non-Linear
Cce	0.038	0.055	0.001
Cre	0.009	0.005	0.001
e0	1.1	1.1	1.1
OCR	1	1	1
Undrained Su A [kips/ft2]	0	0	0
Undrained Su S	0.2	0.2	0.2
Undrained Su m	0.8	0.8	0.8
Grid Name	Staged	Staged	Staged

Groundwater

Groundwater method	Grids
Water Unit Weight	0.0624 kips/ft3

Groundwater Grid: Initial

X	Y	Elevation (ft)
0	150	335.5
0	0	320
250	0	320
250	150	335.5
375	-125	320
-125	-125	320
-125	275	335.5
375	275	335.5

Groundwater Grid: Dewatered

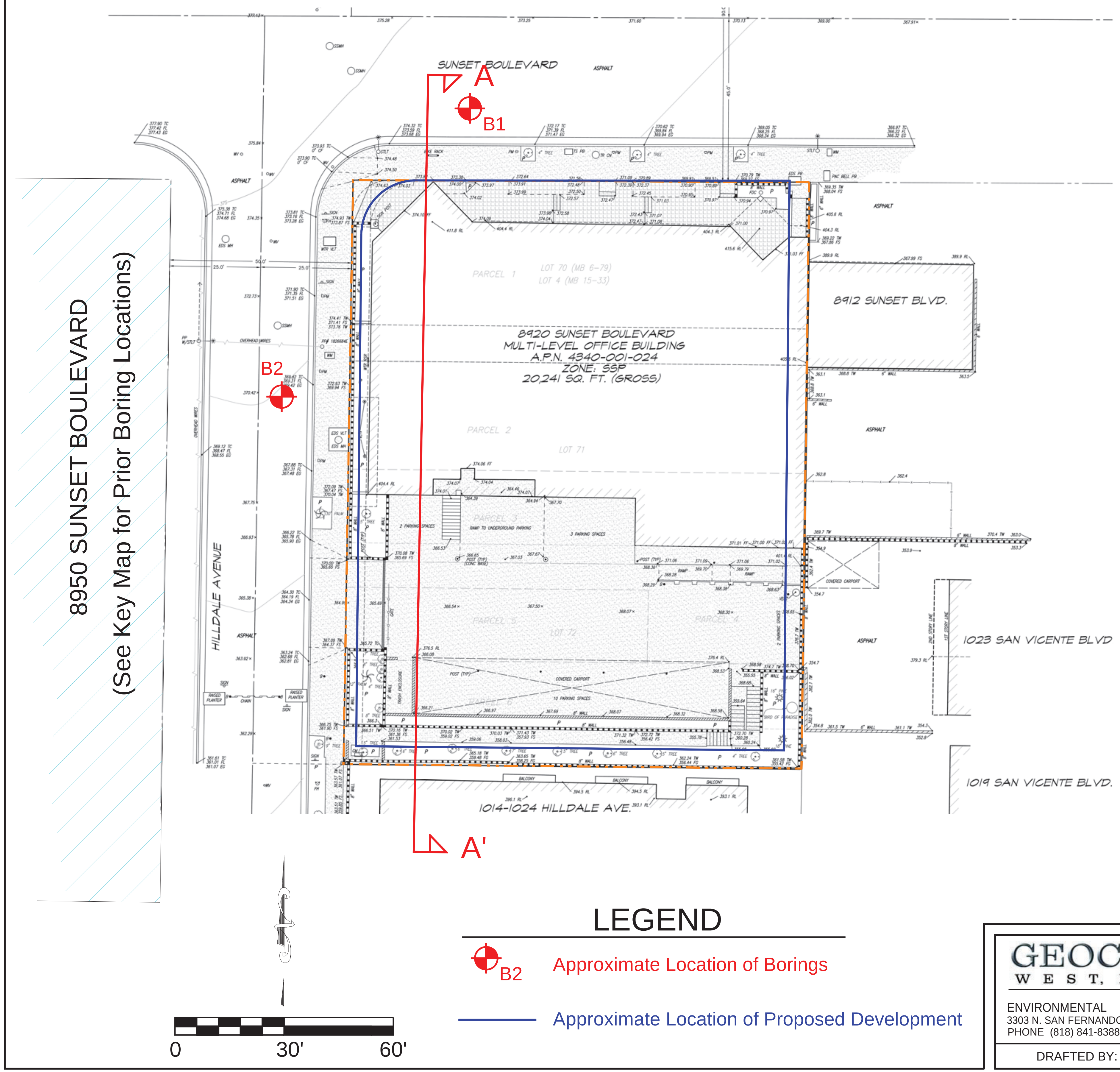
X	Y	Elevation (ft)
0	150	284
0	0	284
250	0	284
250	150	284
375	-125	284
-125	-125	284
-125	275	284
375	275	284

APPENDIX B

APPENDIX B

REFERENCED DATA – 8920 SUNSET BLVD

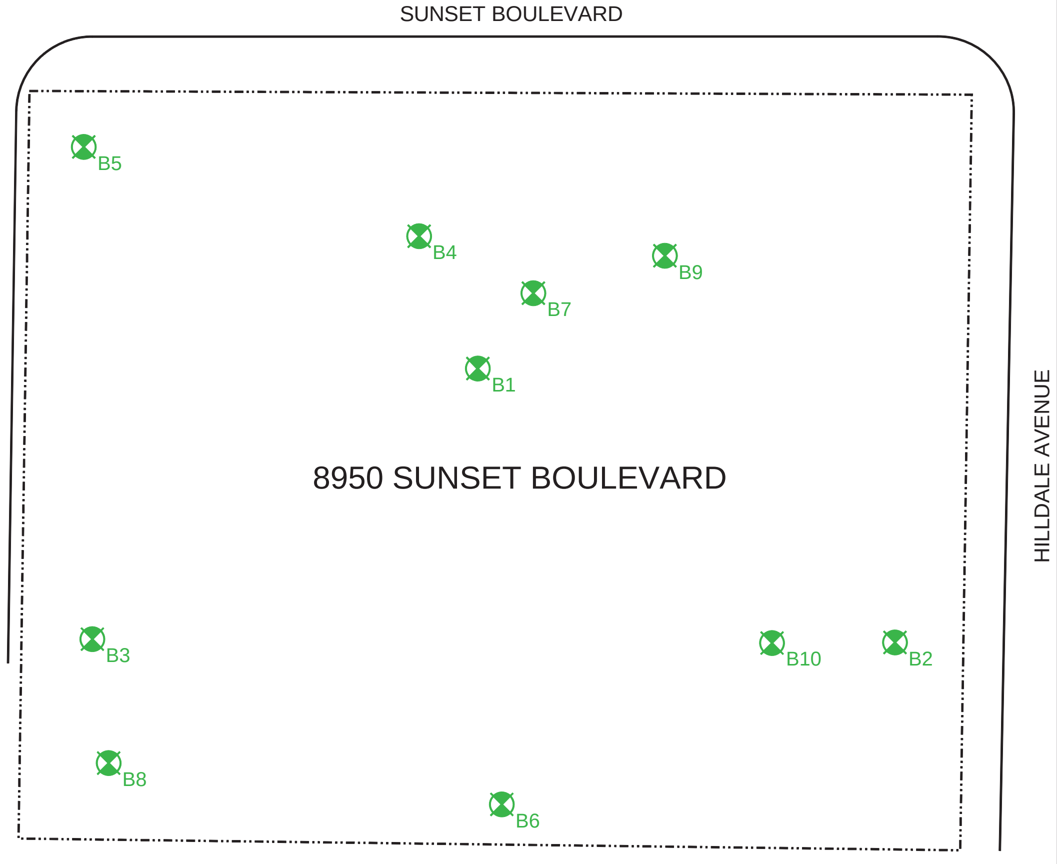
Geotechnical Investigation, Proposed Mixed-Use Development 8920 Sunset Boulevard, West Hollywood, prepared by Geocon West, Inc., dated July 2, 2015, Project No. A9286-06-01.



LEGEND

B2 Approximate Location of Borings

Approximate Location of Proposed Development



PRIOR BORING LOCATIONS MAP

SCALE: 1" = 50'

GEOCON
WEST, INC.

ENVIRONMENTAL GEOTECHNICAL MATERIALS
3303 N. SAN FERNANDO BLVD. - SUITE 100 - BURBANK, CA 91504
PHONE (818) 841-8388 - FAX (818) 841-1704

DRAFTED BY: JMT
CHECKED BY: SKF

SITE PLAN

8920 SUNSET BOULEVARD LLC
8920 SUNSET BOULEVARD
WEST HOLLYWOOD, CALIFORNIA

JULY 2015
PROJECT NO. A9286-06-01
FIG. 2

PROJECT NO. A9286-06-01

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	**BORING 1** ELEV. (MSL.) ______ DATE COMPLETED **5/30/15** EQUIPMENT **HOLLOW STEM AUGER** BY: **RA**	PENETRATION RESISTANCE (BLOWS/FT)*	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
					MATERIAL DESCRIPTION			
0 2	BULK 0-3' ▨				**AC: 3" BASE: 6"** **ARTIFICIAL FILL** Silty Sand, medium dense, slightly moist, yellowish brown, fine- to medium-grained.			
					Total depth of boring: 3 feet Fill to 3 feet. Groundwater not encountered. Patched with concrete containing black dye. Boring attempted at 3 locations. Underground utilities encountered at each location.			

A9286-06-01 BORING LOGS.GPJ

Figure A1,
Log of Boring 1, Page 1 of 1

SAMPLE SYMBOLS			
	☐ ... SAMPLING UNSUCCESSFUL	◧ ... STANDARD PENETRATION TEST	■ ... DRIVE SAMPLE (UNDISTURBED)
	☒ ... DISTURBED OR BAG SAMPLE	◩ ... CHUNK SAMPLE	▼ ... WATER TABLE OR SEEPAGE

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

GEOCON

PROJECT NO. A9286-06-01

BORING 2

ELEV. (MSL.) _______ DATE COMPLETED **5/30/15**

EQUIPMENT **HOLLOW STEM AUGER** BY: **RA**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT)*	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0	BULK 0-3'				**AC: 3" BASE: 3"** **ARTIFICIAL FILL** Silty Sand, medium dense, slightly moist, yellowish brown, fine- to medium-grained, trace rootlets, trace concrete fragments.			
2								
4								
6	B2@5'					19	--	--
8								
10	B2@10'			SP	**ALLUVIUM** Sand, poorly graded, loose, slightly moist, yellowish brown, fine- to medium-grained, trace coarse-grained, trace silt. - grades coarser	14	95.4	3.1
12				ML	Sandy Silt with Clay, firm, slightly moist, dark yellowish brown, very fine- to medium-grained.			
14								
16	B2@15'			SC	Clayey Sand, poorly graded, dense, slightly moist, dark reddish brown, fine- to medium-grained, moderate plasticity.	55	--	--
18								
20	B2@20'				- increase in sand content, dark yellowish brown	81	122.4	10.3
22				SP	Sand with Clay, poorly graded, dense, dry, yellowish brown, some silt.			
24								
26	B2@25'			SC	Clayey Sand, poorly graded, dense, slightly moist, dark reddish brown, fine- to medium-grained, moderate plasticity, some silt.	55	--	--
28					- increase in clay content			

A9286-06-01 BORING LOGS.GPJ

Figure A2,
Log of Boring 2, Page 1 of 3

SAMPLE SYMBOLS			
	☐ ... SAMPLING UNSUCCESSFUL	◧ ... STANDARD PENETRATION TEST	■ ... DRIVE SAMPLE (UNDISTURBED)
	⊠ ... DISTURBED OR BAG SAMPLE	◪ ... CHUNK SAMPLE	▼ ... WATER TABLE OR SEEPAGE

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

GEOCON

PROJECT NO. A9286-06-01

BORING 2

ELEV. (MSL.) _______ DATE COMPLETED **5/30/15**

EQUIPMENT **HOLLOW STEM AUGER** BY: **RA**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT)*	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
30	B2@30'			SC	- moist	54	117.0	13.6
32								
34	B2@35'			CL	Clay with Sand, hard, slightly moist, dark reddish brown, fine- to medium-grained, some silt, medium plasticity.	50 (5")	--	--
36								
38			▼					
40	B2@40'			SC	Clayey Sand, poorly graded, medium dense, wet, dark yellowish brown, fine- to medium-grained, some silt, some coarse-grained, moderate plasticity.	22	--	--
42								
44				SP	Sand with Clay, poorly graded, medium dense, wet, yellowish brown, fine- to medium-grained.			
	B2@45' BULK 45-50'			SC	Clayey Sand, poorly graded, medium dense, wet, dark yellowish brown, fine- to medium-grained, some coarse-grained, some silt, moderate plasticity.	22	119.1	16.0
46								
48								
50	B2@50'				- decrease in clay content	27	--	--
52				SP	Sand with Clay, poorly graded, medium dense, wet, yellowish brown, fine- to medium-grained, some silt.			
	B2@52.5'			SC	Clayey Sand, poorly graded, medium dense, yellowish brown, fine- to medium-grained, some silt.	23	124.8	14.1
54				SP	Sand with Clay, poorly graded, medium dense, yellowish brown, fine- to medium-grained, some silt.			
	B2@55'			SC	Clayey Sand, poorly graded, medium dense, wet, yellowish brown, fine- to medium-grained, some very fine-grained, some silt.	23	--	--
56								
	B2@57.5'			SP	Sand with Clay, poorly graded, dense, wet, yellowish brown, fine- to medium-grained, some silt, red mottling.	61	120.7	14.5
58								
					- dense			

A9286-06-01 BORING LOGS.GPJ

Figure A2, Log of Boring 2, Page 2 of 3

SAMPLE SYMBOLS			
	☐ ... SAMPLING UNSUCCESSFUL	◧ ... STANDARD PENETRATION TEST	■ ... DRIVE SAMPLE (UNDISTURBED)
	▩ ... DISTURBED OR BAG SAMPLE	◪ ... CHUNK SAMPLE	▼ ... WATER TABLE OR SEEPAGE

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

PROJECT NO. A9286-06-01

BORING 2

ELEV. (MSL.) ________ DATE COMPLETED **5/30/15**

EQUIPMENT **HOLLOW STEM AUGER** BY: **RA**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT)*	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
60	B2@60' ■					61	--	--
62								
64					- very dense, some pockets of clay			
	B2@65' ■			SP		50 (6")	116.3	16.1
66								
68								
70	B2@70' ■				Clayey Sand, poorly graded, medium dense, wet, yellowish brown, fine- to medium-grained, some very fine-grained, some silt.	48	118.9	19.9
72				SC				
74					- increase in clay content, dense			
	B2@75' ■					50 (4")	100.0	21.1

Total depth of boring: 75.5 feet
Fill to 9 feet.
Groundwater encountered at 38 feet.
Backfilled with grout and bentonite.
Patched with concrete containing black dye.

*Penetration resistance for 140-pound hammer falling 30 inches by auto hammer.

A9286-06-01 BORING LOGS.GPJ

Figure A2,
Log of Boring 2, Page 3 of 3

SAMPLE SYMBOLS

☐ ... SAMPLING UNSUCCESSFUL	◧ ... STANDARD PENETRATION TEST	■ ... DRIVE SAMPLE (UNDISTURBED)
▨ ... DISTURBED OR BAG SAMPLE	◩ ... CHUNK SAMPLE	▼ ... WATER TABLE OR SEEPAGE

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

GEOCON

BORING LOG NUMBER 1

Drilling Date August 13, 1986

Elevation 375'

Project KB 14298-S Communication Art House

Sample Depth ft.	Blows per ft.	Moisture Content %	Dry Unit Weight p.c.f.	Depth in feet	Graphic Log	Description Surface Conditions 1½ inch A.C.
				0		FILL: Silty Sand, dark reddish-brown, slightly moist, loose, fine to medium grained, clay binder, trace gravel
2	1/Tap	9.8	105.5	2		medium to coarse grained, tile chip
				4		
5	Push/ Tap	10.9	104.6			
				6	SC/SM	CLAYEY SAND TO SILTY SAND, dark reddish-brown, moist, medium dense, medium to coarse grained, slightly plastic, porous, trace gravel
				8		
10	3	15.8	112.5	10		
				12		
				14		dense
15	7	15.1	118.7			
				16	SP/SM	SAND TO CLAYEY SILTY SAND, reddish-brown, moist, dense, medium to coarse grained, poorly graded, some gravel
				18		
20	5	9.9	119.5	20	SM/SC	SILTY SAND TO CLAYEY SAND, reddish-brown, moist, dense, medium to coarse grained, some gravel
				22		
				24		

BORING LOG NUMBER 1 - Continued

Drilling Date August 13, 1986 **Elevation**

Project KB 14298-S Communication Art House

Sample Depth ft.	Blows per ft.	Moisture Content %	Dry Unit Weight p.c.f.	Depth in feet	Graphic Log	Description Surface Conditions 1½ inch A.C.
25	9	12.8	113.0	25		SILTY SAND TO CLAYEY SAND - continued
				27		
				29		
30	14	12.5	120.9			
				31		
						very moist
				33		
35	4	15.8	112.8	35	SC/SM	CLAYEY SAND TO SILTY SAND, reddish-brown,wet,medium dense, medium to coarse grained, some gravel
				37		
						free water
				39		
40	1/Tap	22.3	107.5			
				41	SP/SC	SAND TO CLAYEY SAND, reddish-brown, saturated,medium dense,coarse grained, some gravel
				43		
45	7	16.4	121.4	45		dense
				47		
				49		

BORING LOG NUMBER 1 - Continued

Drilling Date August 13, 1986 **Elevation**

Project KB 14298-S Communication Art House

Sample Depth ft.	Blows per ft.	Moisture Content %	Dry Unit Weight p.c.f.	Depth in feet	Graphic Log	Description Surface Conditions 1½ inch A.C.
50	27	16.4	118.3	50		SAND TO CLAYEY SAND - continued
				52		
				54		
				56		
				58		
60	40 for 11"	21.6	112.6	60		END at 60 feet; No Caving; Fill to 6 feet; Free Water at 38 feet.

BORING LOG NUMBER 2

Drilling Date August 13, 1986

Elevation 364'

Project KB 14298-S Communication Art House

Surface Conditions 1½ inch A.C.

Sample Depth ft.	Blows per ft.	Moisture Content %	Dry Unit Weight p.c.f.	Depth in feet	Graphic Log	Description
				0		FILL: Silty Sand brown, slightly moist, loose, medium to coarse grained, trace gravel
2	Push	7.0	106.1	2		roots to 1/8 inch, root hairs
				4		light brown
5	1/Tap	5.7	107.8			SAND TO SILTY SAND, light brown, slightly moist, medium dense, coarse grained
				6		
					SP/SM	SAND TO SILTY SAND, light reddish-brown, slightly moist, medium dense, some gravel
				8		
10	2	6.6	105.0	10		porous
				12		
				14		
15	5	14.7	113.8		SM/SC	SILTY SAND TO CLAYEY SAND, reddish-brown, slightly moist, medium dense, medium to coarse grained, some gravel
				16		
						light reddish-brown, sandier
				18		
20	9	13.1	122.8	20		dense
					SC/SM	CLAYEY SAND TO SILTY SAND, reddish-brown, moist, dense, medium to coarse grained, some gravel
				22		
				24		

BORING LOG NUMBER 2 - Continued

Drilling Date August 13, 1986 **Elevation**

Project KB 14298-S Communication Art House

Sample Depth ft.	Blows per ft.	Moisture Content %	Dry Unit Weight p.c.f.	Depth in feet	Graphic Log	**Description** Surface Conditions 1½ inch A.C.
						CLAYEY SAND TO SILTY SAND - continued
25	7	13.1	121.1	25		
				27		very moist
				29		wet free water
				31		
				33		
35	5	17.5	112.0	35		
						END at 35 feet; No Caving; Fill to 7 Feet; Free Water at 29 Feet.

BORING LOG NUMBER 3

Drilling Date August 14, 1986 **Elevation** 371'

Project KB 14298-S Communication Art House

Sample Depth ft.	Blows per ft.	Moisture Content %	Dry Unit Weight p.c.f.	Depth in feet	Graphic Log	Description Surface Conditions Weeds and Loose Soil
				0		FILL: Silty Sand, light brown, slightly moist, loose, roots, trace gravel, wood chips, glass fragments
2	Push	8.8	103.6	2		light reddish-brown, some gravel, clay binder
				4	SM	SILTY SAND, reddish-brown, slightly moist, medium dense, some gravel, clay binder, porous, medium to coarse grained
5	1	9.2	106.5			
				6		
				8	SM/SP	SILTY SAND TO SAND, reddish-brown, moist, medium dense, some gravel, clay binder, medium to coarse grained
10	2	14.1	111.0	10		
					SM/SC	SILTY SAND TO CLAYEY SAND, dark reddish-brown, moist, medium dense, some gravel, medium to coarse grained, roots
				12		
				14		
15	7	11.8	122.6			dense
				16		
				18		
20	4	11.2	122.7	20		
					SC/SM	CLAYEY SAND TO SILTY SAND, dark reddish-brown, moist, dense, trace gravel, medium to coarse grained
				22		
						light reddish-brown
				24		

BORING LOG NUMBER 3 - Continued

Drilling Date August 14, 1986
Elevation

Project KB 14298-S Communication Art House

Sample Depth ft.	Blows per ft.	Moisture Content %	Dry Unit Weight p.c.f.	Depth in feet	Graphic Log	Description Surface Conditions Weeds and Loose Soil
25	9	10.5	113.5	25		CLAYEY SAND TO SILTY SAND - Continued
				27		
					SM	SILTY SAND, light orange-brown, moist, dense, trace gravel, medium to coarse grained, clay binder
				29		
30	26	10.4	124.4			
				31		reddish-brown
				33		
35	7	15.6	116.6	35		
					SM/SC	SILTY SAND TO CLAYEY SAND, reddish-brown, very moist, dense, wet free water
				37		
				39		
40	7	20.2	112.4			
				41		END at 40 feet; No Caving; Fill to 4 feet; Free Water at 36 feet. NOTE: On Plates A-1 and A-7 The stratification lines represent the approximate boundary between earth types; the transition may be gradual. NOTE on Plates A-1 and A-7: Borings 1 through 3 excavated with 18" bucket-auger

BORING LOG NUMBER 4

Drilling Date April 19, 1991

Elevation 381'

Project KB 14298-S Communication Art House

Sample Depth ft.	Blows per ft.	Moisture Content %	Dry Unit Weight p.c.f.	Depth in feet	Graphic Log	Description
						Surface Conditions A/C Driveway 2" thick
				1		FILL: Silty Sand, brown, moist, medium dense, fine grained, glass fragments
2	20	7.9	109.1		SM	Silty Sand, brown, moist, dense, fine to coarse grained, some clay binder, porous
5	10	7.5	102.6	5		
						sandier, coarser grained
						light orange-brown, slightly moist to moist
10	15	9.4	108.7	10		
				15		moist
					SC	Clayey Sand, orange-red, moist, dense, fine to coarse grained
20	54	14.2	119.2	20	CL	Sandy Clay, orange-brown, moist, stiff, medium grained sand in clay matrix
					SC	Clayey Sand, reddish-brown, moist, dense, fine to coarse grained
				25		clayier

KOVACS-BYER and ASSOCIATES INC.

PLATE A-8

BORING LOG NUMBER 4 (continued)

Drilling Date

Elevation

Project KB 14298-S Communication Art House

Sample Depth ft.	Blows per ft.	Moisture Content %	Dry Unit Weight p.c.f.	Depth in feet	Graphic Log	Description
						Surface Conditions A/C Driveway 2" thick
				26		
						fine to medium grained, some coarse grains
30	40	11.2	121.2	30		
				35		
						fine grained, some medium grains
40	43	13.8	116.5	40		
				45		
						orange-brown, fine to coarse grained
50	20	15.5	120.1	50		

BORING LOG NUMBER 4 (continued)

Drilling Date

Elevation

Project KB 14298-S Communication Art House

Sample Depth ft.	Blows per ft.	Moisture Content %	Dry Unit Weight p.c.f.	Depth in feet	Graphic Log	Description Surface Conditions A/C Driveway 2" thick
				51		
				55		
						very moist, dense, fine to coarse grained
60	25	16.5	117.2	60		
				65		
						reddish brown, dense, fine to medium grained
70	39	16.8	119.3	70		
				75		sandier

BORING LOG NUMBER 4 (continued)

Drilling Date ______________ **Elevation** ______________

Project KB 14298-S Communication Art House

Sample Depth ft.	Blows per ft.	Moisture Content %	Dry Unit Weight p.c.f.	Depth in feet	Graphic Log	Description Surface Conditions A/C Driveway, 2" thick
				76		
80	47	17.7	117.3	80		mottled yellow, tan and brown, moist, dense, fine to coarse grained
85	75	13.3	123.6	85		
						BEDROCK: Granite, orange-brown with black staining, very moist, moderately hard to hard, massive, moderately weathered
90	90	11.9	124.5	90		
				95		
100	116	9.8	122.4	100		End at 100 feet; Water at 46 feet; No caving; Fill to 1½ feet NOTE: The stratification depth represents the approximate boundary between earth types; the transition may be gradual. 8" Hollow Stem 140 lb. Hammer

BORING LOG NUMBER 5

Drilling Date: 11/10/99 **Elevation: 384.0'**

Project: File No. 17507-S **Sunset Heights, LLC**

Sample Depth Ft	Blows per ft	Moisture content %	Dry Unit Weight p.c.f.	Depth in feet	USCS Class.	Description Surface Conditions: 1-in. asphalt, fair condition, 3-in. sand base
				0		FILL: Silty Sand, brown, slightly moist, dense, fine grained, trace gravel
1	15	4.4	109.5	1		abundant gravel
				2		
3	13	8.6	108.2	3	SM	Silty Sand, medium brown, moist, medium dense, fine to medium grained, some coarse grained, trace gravel, slightly porous
				4		
5	15	8.3	109.9	5		
				6		
7	13	7.3	104.9	7		orange-brown
				8		
				9		
10	19	8.0	111.7	10		
				11		
				12		
				13		
				14		
15	25	8.7	114.7	15		reddish brown, dense
				16		
				17		
				18		
				19		
20	72	12.0	126.4	20		clay binder, very dense
				21		
				22		
				23		
				24		
25	54	9.3	127.4	25		trace gravel
				26		
				27		
				28		
				29		
30	62	13.0	122.4	30	SC	Clayey Sand, reddish brown, moist, very dense, fine to medium grained, trace gravel

BORING LOG NUMBER 5 (continued)

Project: File No. 17507-S **Sunset Heights, LLC**

Sample Depth Ft	Blows per ft	Moisture content %	Dry Unit Weight p.c.f.	Depth in feet	USCS Class.	Description Surface Conditions
				31		
				32		
				33		
				34		
35	49	9.7	120.4	35	SM	Silty Sand, orange-brown, moist, very dense, fine to medium grained, trace coarse grained and gravel
				36		
				37		
				38		
				39		
40	90	14.4	118.2	40		reddish brown, clay binder
				41		
				42		
				43		
				44		
45	56	13.2	120.2	45		fine to coarse grained, very moist
				46		
				47		
				48		
				49		
50	67	15.1	119.4	50		orange-brown
				51		
				52		
				53		water
				54		
55	31 50/4"	20.1	111.9	55	SC	Clayey Sand, orange-brown, very moist to wet, very dense, fine to coarse grained, occasional large decomposed granite fragments
				56		
				57		
				58		
				59		
60	53	20.6	SPT	60	SM	Silty Sand, orange-brown, wet, very dense, fine to coarse grained

BORING LOG NUMBER 5 (continued)

Project: File No. 17507-S **Sunset Heights, LLC**

Sample Depth Ft	Blows per ft	Moisture content %	Dry Unit Weight p.c.f.	Depth in feet	USCS Class.	Description
						Surface Conditions
				61		
				62		
62.5	63	17.2	116.0			
				63		
				64		
65	49	21.5	SPT	65		
				66	SC	Clayey Sand, orange-brown, very moist to wet, very dense, fine to coarse grained, abundant gravel
				67		
67.5	78	18.0	113.0			
				68	SM	Silty Sand, reddish brown, wet, very dense, fine grained
				69		
70	33 50/5"	11.8	SPT	70		
				71		grades sandier, gray-brown, wet, very dense, fine to coarse grained, abundant gravel and slate fragments
				72		
72.5	72	15.0	118.3			
				73		fine grained, orange-brown
				74		
75	28 50/5"	16.3	SPT	75		
				76		BEDROCK: Quartz Diorite, highly weathered, orange-brown, wet, hard, massive
				77		
77.5	12 50/5"	15.9	118.9			
				78		
				79		
80	28 50/4"	12.8	SPT	80		
				81		
				82		
82.5	50/5" 50/1"	13.4	disturbed			moderately weathered
				83		
				84		
85	50/5" 50/2"	15.4	SPT	85		
				86		
				87		
87.5	50/4" 50/1"	8.1	132.8			gray
				88		
				89		
90	50/4" 50/2"	5.1	SPT	90		

BORING LOG NUMBER 5 (continued)

Project: File No. 17507-S **Sunset Heights, LLC**

Sample Depth Ft	Blows per ft	Moisture content %	Dry Unit Weight p.c.f.	Depth in feet	USCS Class.	Description Surface Conditions
				91		
				92		
92.5	40 50/2"	17.3	122.7	-		
				93		
				94		
95	50/5"	13.0	SPT	95		
				96		
				97		
97.5	50/4" 50/1"	5.5	158.6	-		
				98		
				99		
100	100/3"	11.3	SPT	100		
				101		Total depth: 100 feet Water at 53 feet Fill to 3 feet
				102		
				103		
				104		
				105		
				106		
				107		
				108		
				109		
				110		
				111		
				112		
				113		
				114		
				115		
				116		
				117		
				118		
				119		
				120		

BORING LOG NUMBER 6

Drilling Date: 11/11/99
Project: File No. 17507-S

Elevation: 365.0'
Sunset Heights, LLC

Sample Depth Ft	Blows per ft	Moisture content %	Dry Unit Weight p.c.f.	Depth in feet	USCS Class.	Description Surface Conditions: Dead grass and scattered asphalt fragments
				0		FILL: Silty Sand, dark brown, moist, dense, fine grained, trace gravel
1	15	6.4	103.9	1	SM	Silty Sand, dark brown, moist, medium dense, fine grained, trace gravel
				2		
3	18	7.4	99.9	3		medium brown, moist, medium dense, fine to medium grained, some clay binder
				4		
5	21	7.5	106.6	5		
				6		
7	28	7.2	109.4	7		orange-brown
				8		
				9		
10	36	7.7	112.6	10		reddish brown, slightly moist, dense, fine to medium grained, some coarse grained and gravel
				11		
				12		
				13		
				14		
15	63	17.2	113.0	15		Clayey Sand, dark reddish brown, slightly moist, very dense, fine to coarse grained
				16		
				17		
				18		
				19		
20	53	11.6	122.2	20	SM	Silty Sand, reddish brown, moist, very dense, fine to medium grained, some roots, gravel, clay binder
				21		
				22		
				23		
				24		
25	62	12.3	123.0	25		moist to very moist
				26		
				27		
				28		
				29		
30	54	21.4	108.2	30	SC	Clayey Sand, dark reddish brown, very moist, very dense, fine to medium grained

BORING LOG NUMBER 6 (continued)

Project: File No. 17507-S **Sunset Heights, LLC**

Sample Depth Ft	Blows per ft	Moisture content %	Dry Unit Weight p.c.f.	Depth in feet	USCS Class.	Description Surface Conditions
				31		
				32		
				33		
				34		
				-		water
35	64	17.8	112.7	35		
				-		grades less clayey
				36		
				37		
				38		
				39		
40	48	15.3	SPT	40		
				-		orange-brown, wet, some gravel
41	51	11.8	SPT	41		
				-	SM	Silty Sand, reddish brown, wet, very dense, fine grained
				42		
42.5	66	15.4	119.4	-		
				43		orange-brown, fine to coarse grained
				44		
45	57	14.0	SPT	45		
				-		medium brown, fine to medium grained, occasional slightly clayey lenses
				46		
				47		
47.5	73	15.8	116.0	-		
				48	SW	Sand, tan, wet, very dense, fine to coarse grained
				49		
50	55	14.4	SPT	50		
				-	SM	Silty Sand, orange-brown, wet, very dense, fine to medium grained
				51		
				52		
52.5	35 50/3"	17.4	113.5	-		
				53		grades sandier, mostly fine grained
				54		
55	62	16.7	SPT	55		
				-		orange-brown with light brown lenses, clay binder
				56		
				57		
57.5	83	14.8	122.4	-		
				58		reddish brown, fine to medium grained
				59		
60	50	15.0	SPT	60		

BORING LOG NUMBER 6 (continued)

Project: File No. 17507-S **Sunset Heights, LLC**

Sample Depth Ft	Blows per ft	Moisture content %	Dry Unit Weight p.c.f.	Depth in feet	USCS Class.	Description Surface Conditions
				61 --		
				62 --		
62.5	36 50/4"	14.6	122.0			
				63 --		
				64 --		
65	65	13.6	SPT	65 --		
						grades sandier
				66 --		
				67 --		
67.5	77	16.1	119.3			
				68 --		clay binder
				69 --		
70	53	17.2	SPT	70 --		
				71 --		
				72 --		
72.5	20 50/5"	14.2	121.9			grades sandier, fine grained
				73 --		
				74 --		
75	69	14.8	SPT	75 --		
						orange-brown
				76 --		
				77 --		
77.5	30 50/5"	17.3	114.9			
				78 --		dark reddish brown, fine to medium grained, clay binder
				79 --		
80	72	17.7	SPT	80 --		
					SC	Clayey Sand, reddish brown, very moist, very dense, fine to medium grained
				81 --		
				82 --		
82.5	79	16.3	116.7			
				83 --		
				84 --		
85	57	15.0	SPT	85 --		
					SM	Silty Sand, orange-brown, very moist, very dense, fine to medium grained
				86 --		
				87 --		
				88 --		
				89 --		
90	74	15.9	SPT	90 --		

BORING LOG NUMBER 6 (continued)

Project: File No. 17507-S **Sunset Heights, LLC**

Sample Depth Ft	Blows per ft	Moisture content %	Dry Unit Weight p.c.f.	Depth in feet	USCS Class.	Description Surface Conditions
				91 --		
				92 --		
				93 --		
				94 --		
95	37 50/5"	19.4	111.6	95 --		
				-	SC	Clayey Sand, reddish brown, very moist, very dense, fine to medium grained
				96 --		
				97 --		
				98 --		
				99 --		
100	90	12.4	SPT	100 --		
				101 --		Total depth: 100 feet Water at 34 feet Fill to 1 foot
				102 --		
				103 --		
				104 --		
				105 --		
				106 --		
				107 --		
				108 --		
				109 --		
				110 --		
				111 --		
				112 --		
				113 --		
				114 --		
				115 --		
				116 --		
				117 --		
				118 --		
				119 --		
				120 --		

BORING LOG NUMBER 7

Drilling Date: 06/03/05 **Elevation: 380'**

Project: File No. 18931 **James Hotels**

km

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description Surface Conditions: 2-inch Asphalt - Very Poor Condition, No Base
				0 --		FILL: Silty Sand, brown, moist, medium dense, fine grained, minor gravel
				-		
				1 --		
				-		
				2 --		
2½	51	6.7	120.5	-		
				3 --		some gravel, concrete fragments
				-		
				4 --		
				-		
5	8	6.7	SPT	5 --		
				-	SM	Silty Sand, brown, moist, medium dense, fine grained, minor gravel
				6 --		
				-		
				7 --		
7½	17	7.2	114.2	-		
				8 --		orange-brown
				-		
				9 --		
				-		
10	25	8.2	SPT	10 --		
				-		
				11 --		
				-		
				12 --		
12½	30	6.5	110.0	-		
				13 --		
				-		
				14 --		
				-		
15	31	7.8	SPT	15 --		
				-		grades sandier, fine to medium grained
				16 --		
				-		
				17 --		
17½	19 50/6"	15.5	117.1	-		
				18 --	SC	Clayey Sand, orange-brown, moist, very dense, fine grained, minor gravel
				-		
				19 --		
				-		
20	46	12.5	SPT	20 --		
				-		
				21 --		
				-		
				22 --		
22½	75	9.7	128.6	-		
				23 --	SM	Silty Sand, orange-brown, moist, very dense, fine grained, some gravel
				-		
				24 --		
				-		
25	46	9.6	SPT	25 --		
				-		
				26 --		
				-		
				27 --		
27½	78	10.1	128.5	-		
				28 --	SC	Clayey Sand, orange-brown, moist, very dense, fine grained, minor gravel
				-		
				29 --		
				-		
30	42	15.3	SPT	30 --		
				-		very moist, dense, water seepage

BORING LOG NUMBER 7

Project: File No. 18931 **James Hotels**

km

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description
				-		
				31 --		
				-		
				32 --		
32½	73	14.1	118.0	-		
				33 --		
				-		
				34 --		
				-		
35	61	16.7	SPT	35 --		
				-		
				36 --		
				-		
				37 --		
37½	70	14.2	121.5	-		
				38 --	SM	Silty Sand, orange-brown, very moist, very dense, fine grained, minor gravel
				-		
				39 --		
				-		
40	53	16.6	SPT	40 --		
				-		
				41 --		
				-		
				42 --		
42½	60	No Recovery		-		
				43 --		
				-		
				44 --		
				-		
45	57	14.3	SPT	45 --		
				-	SC	Clayey Sand, orange-brown, very moist, dense, fine grained, minor gravel
				46 --		
				-		
				47 --		
47½	52	14.9	121.4	-		
				48 --		
				-		
				49 --		
				-		
50	59	16.4	SPT	50 --		
				-		very moist to wet
				51 --		
				-		
				52 --		
52½	65	13.6	123.6	-		
				53 --		
				-		
				54 --		
				-		
55	46	16.8	SPT	55 --		
				-		
				56 --		
				-		
				57 --		
57½	48	17.6	117.2	-		
				58 --		orange-brown to gray-brown, moist
				-		
				59 --		
				-		
60	50	20.6	SPT	60 --		
				-		orange-brown

BORING LOG NUMBER 7

Project: File No. 18931 **James Hotels**

km

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description
				-		
				61 --		
				-		
				62 --		
62½	35	18.0	116.2	-		
	50/4"			63 --		very dense to hard
				-		
				64 --		
				-		
65	80	18.5	SPT	65 --		
				-		
				66 --		
				-		
				67 --		
67½	48	18.1	116.7	-		
				68 --		
				-		
				69 --		
				-		
70	73	17.3	SPT	70 --		
				-		
				71 --		
				-		
				72 --		
72½	72	17.0	119.3	-		
				73 --		
				-		
				74 --		
				-		
75	30	14.5	SPT	75 --		
	50/4"			-	SM	Silty Sand, brown-gray, very moist to wet, very dense, medium grained
				76 --		
				-		
				77 --		
77½	19	16.5	120.7	-		
	50/5"			78 --		fine grained
				-		
				79 --		
				-		
80	50/3"	16.6	SPT	80 --		
				-		
				81 --		
				-		
				82 --		
82½	50/4"	18.4	117.6	-		
				83 --	SC	Clayey Sand, olive-gray, very moist, very dense, fine grained, minor gravel
				-		
				84 --		
				-		
85	50/3"	12.5	SPT	85 --		
				-		
				86 --		
				-		
				87 --		
87½	50/4"	18.8	115.4	-		
				88 --		
				-		
				89 --		
				-		
90	50/3"	14.8	SPT	90 --		
				-	SM	Silty Sand, olive-gray, very moist, very dense, fine to coarse grained, some gravel

BORING LOG NUMBER 7

Project: File No. 18931 **James Hotels**

km

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description
				-		
				91 --		
				-		
				92 --		
92½	50/6"	19.2	118.6	-		
				93 --	SC	Clayey Sand, olive-gray, very moist, hard, fine grained
				-		
				94 --		
				-		
95	35 50/3"	15.8	SPT	95 --		
				-		
				96 --		
				-		
				97 --		
97½	19 50/5"	No Recovery		-		
				98 --		
				-		
				99 --		
				-		
100	35 50/3"	18.0	SPT	100 --		
				-		Total depth: 100 feet
				101 --		Water at 30 feet
				-		Fill to 5 feet
				102 --		
				-		
				103 --		NOTE: The stratification lines represent the approximate
				-		boundary between earth types; the transition may be gradual
				104 --		Used 8-inch diameter Hollow-Stem Auger
				-		140-lb. Slide Hammer, 30-inch drop
				105 --		Modified California Sampler used unless otherwise noted
				-		
				106 --		SPT=Standard Penetration Test
				-		
				107 --		
				-		
				108 --		
				-		
				109 --		
				-		
				110 --		
				-		
				111 --		
				-		
				112 --		
				-		
				113 --		
				-		
				114 --		
				-		
				115 --		
				-		
				116 --		
				-		
				117 --		
				-		
				118 --		
				-		
				119 --		
				-		
				120 --		
				-		

BORING LOG NUMBER 8

Drilling Date: 06/06/05 **Elevation: 366'**

Project: File No. 18931 **James Hotels**

km

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description Surface Conditions: Grassy Area
				0 --		FILL: Silty Sand, brown-gray, moist, medium dense, fine grained
				-		
1	28	8.7	111.4	1 --		
				-		some gravel, wood chips
				2 --		
				-		
3	14	9.9	107.2	3 --		
				-	SM	Silty Sand, brown-gray, moist, medium dense, fine grained, some gravel, some rootlets
				4 --		
				-		
5	16	9.4	111.6	5 --		
				-		brown, minor gravel, no rootlets
				6 --		
				-		
7	15	10.3	108.0	7 --		
				-		orange-brown
				8 --		
				-		
				9 --		
				-		
10	21	14.0	119.6	10 --		
				-		brown
				11 --		
				-		
				12 --		
				-		
				13 --		
				-		
				14 --		
				-		
15	37	21.9	115.2	15 --		
				-	SC	Clayey Sand, brown, moist, dense, fine grained, minor gravel, some rootlets
				16 --		
				-		
				17 --		
				-		
				18 --		
				-		
				19 --		
				-		
20	41	16.9	116.8	20 --		
				-		
				21 --		
				-		
				22 --		
				-		
				23 --		water seepage
				-		
				24 --		
				-		
25	90	16.1	115.9	25 --		
				-		very dense to hard, some roots
				26 --		
				-		
				27 --		
				-		
				28 --		
				-		
				29 --		
				-		
30	57	19.1	112.6	30 --		Silty Sand, orange-brown, moist to very moist, dense, fine grained, minor gravel
				-	SM	

GEOTECHNOLOGIES, INC.

Plate A-8a

BORING LOG NUMBER 8

Project: File No. 18931 **James Hotels**

km

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description
				-		
				31 --		
				-		
				32 --		
				-		
				33 --		
				-		
				34 --		
				-		
35	54	No Recovery		35 --		
				-		
				36 --		
				-		
				37 --		
				-		
				38 --		
				-		
				39 --		
				-		
40	33	16.9	119.7	40 --		
	50/2"			-		very moist, very dense
				41 --		
				-		
				42 --		
				-		
				43 --		
				-		
				44 --		
				-		
45	50	19.6	114.9	45 --		
				-	SC	Clayey Sand, orange-brown, moist, dense, fine grained, minor gravel
				46 --		
				-		
				47 --		
				-		
				48 --		
				-		
				49 --		
				-		
50	51	14.9	115.3	50 --		
				-	SM	Silty Sand, orange-brown, very moist to wet, dense, fine grained, minor gravel
				51 --		
				-		
				52 --		
				-		
				53 --		
				-		
				54 --		
				-		
55	61	16.7	119.0	55 --		
				-		
				56 --		
				-		
				57 --		
				-		
				58 --		
				-		
				59 --		
				-		
60	59	18.6	115.6	60 --		
				-		

BORING LOG NUMBER 8

Project: File No. 18931 **James Hotels**

km

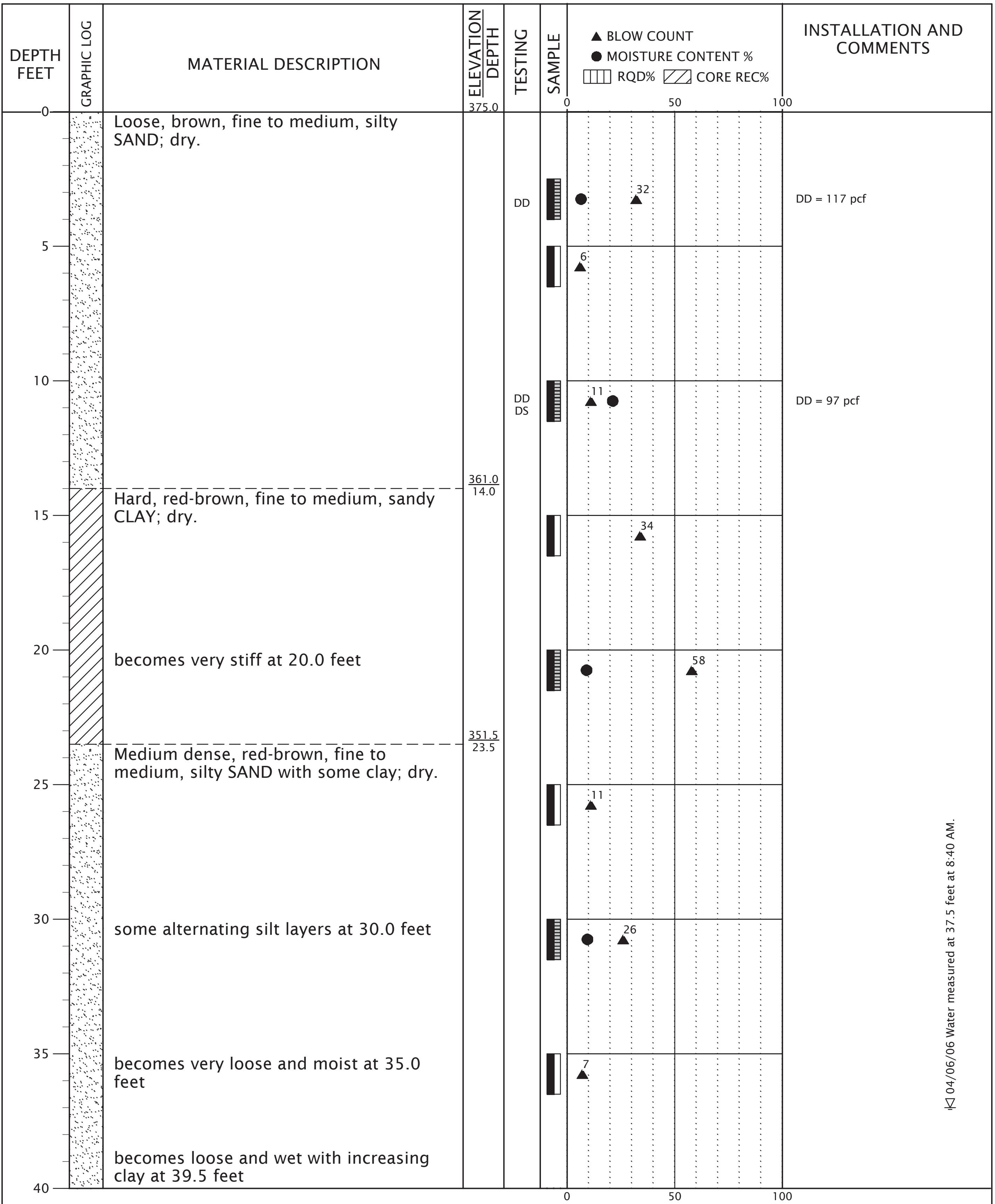
Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description
				61 --		
				62 --		
				63 --		
				64 --		
65	90	15.3	120.4	65 --	SC	Clayey Sand, orange-brown, moist, very dense, fine grained, minor gravel
				66 --		
				67 --		
				68 --		
				69 --		
70	69	15.6	120.3	70 --	SM	Silty Sand, orange-brown, wet, dense, fine grained, minor gravel
				71 --		
				72 --		
				73 --		
				74 --		
75	62	No Recovery		75 --		
				76 --		
				77 --		
				78 --		
				79 --		
80	64	20.7	114.5	80 --		
				81 --		
				82 --		
				83 --		
				84 --		
85	82	18.5	114.3	85 --	SC	Clayey Sand, orange-brown, moist, very dense, fine grained, minor gravel
				86 --		
				87 --		
				88 --		
				89 --		
90	19 50/4''	19.8	111.8	90 --		

BORING LOG NUMBER 8

Project: File No. 18931 **James Hotels**

km

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description
				-		
				91 --		
				-		
				92 --		
				-		
				93 --		
				-		
				94 --		
				-		
95	38 50/5"	20.0	112.7	95 --		
				-		
				96 --		
				-		
				97 --		
				-		
				98 --		
				-		
				99 --		
				-		
100	69	20.2	113.1	100 --		
				-		Total depth: 100 feet Water at 22½ feet Fill to 3 feet
				101 --		
				-		
				102 --		
				-		
				103 --		
				-		
				104 --		
				-		
				105 --		
				-		
				106 --		
				-		
				107 --		
				-		
				108 --		
				-		
				109 --		
				-		
				110 --		
				-		
				111 --		
				-		
				112 --		
				-		
				113 --		
				-		
				114 --		
				-		
				115 --		
				-		
				116 --		
				-		
				117 --		
				-		
				118 --		
				-		
				119 --		
				-		
				120 --		
				-		



Loose, brown, fine to medium, silty SAND; dry.

Hard, red-brown, fine to medium, sandy CLAY; dry.

becomes very stiff at 20.0 feet

Medium dense, red-brown, fine to medium, silty SAND with some clay; dry.

some alternating silt layers at 30.0 feet

becomes very loose and moist at 35.0 feet

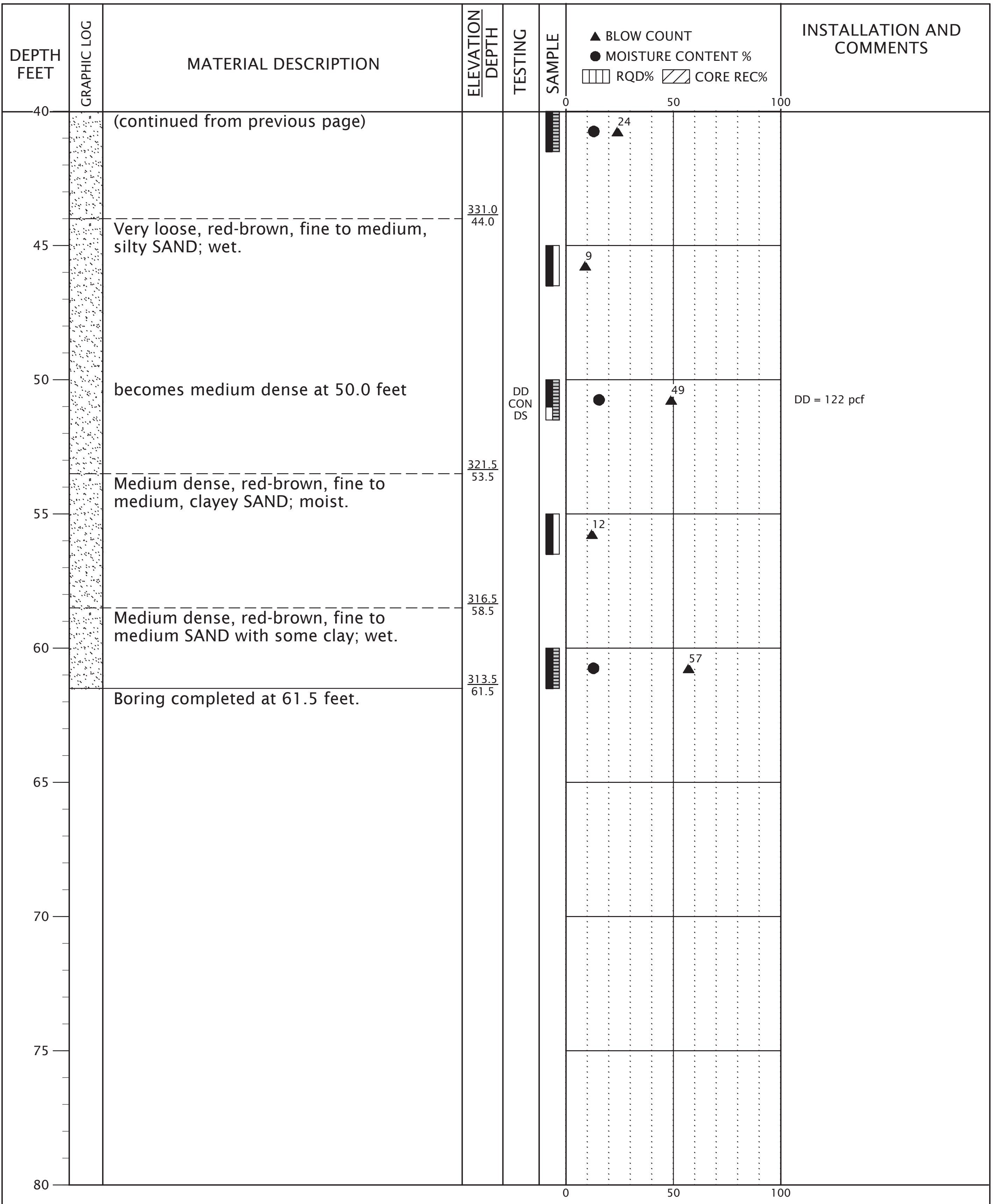
becomes loose and wet with increasing clay at 39.5 feet

DRILLED BY: 2-R Drilling, Inc.
LOGGED BY: LAS
COMPLETED: 04/06/06

BORING METHOD: hollow-stem auger (see report text)
BORING BIT DIAMETER: 8.0 in

GEODESIGN INC 2121 S Towne Centre Place - Suite 130 Anaheim CA 92806 Off 714.634.3701 Fax 714.634.3711	JAMESHOTEL-1-01	**BORING B-9**
	JUNE 2006	JAMES HOTEL WEST HOLLYWOOD, CA
		FIGURE A-1

DEPTH FEET	MATERIAL DESCRIPTION	ELEVATION DEPTH	TESTING	BLOW COUNT	INSTALLATION AND COMMENTS
40	(continued from previous page)			24	
	Very loose, red-brown, fine to medium, silty SAND; wet.	331.0 44.0			
45				9	
50	becomes medium dense at 50.0 feet		DD CON DS	49	DD = 122 pcf
	Medium dense, red-brown, fine to medium, clayey SAND; moist.	321.5 53.5			
55				12	
	Medium dense, red-brown, fine to medium SAND with some clay; wet.	316.5 58.5			
60				57	
	Boring completed at 61.5 feet.	313.5 61.5			



DRILLED BY: 2-R Drilling, Inc.
LOGGED BY: LAS
COMPLETED: 04/06/06

BORING METHOD: hollow-stem auger (see report text)
BORING BIT DIAMETER: 8.0 in

GEODESIGN INC
2121 S Towne Centre Place - Suite 130
Anaheim CA 92806
Off 714.634.3701 Fax 714.634.3711

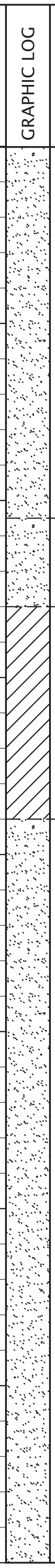
JAMESHOTEL-1-01

JUNE 2006

BORING B-9
(continued)

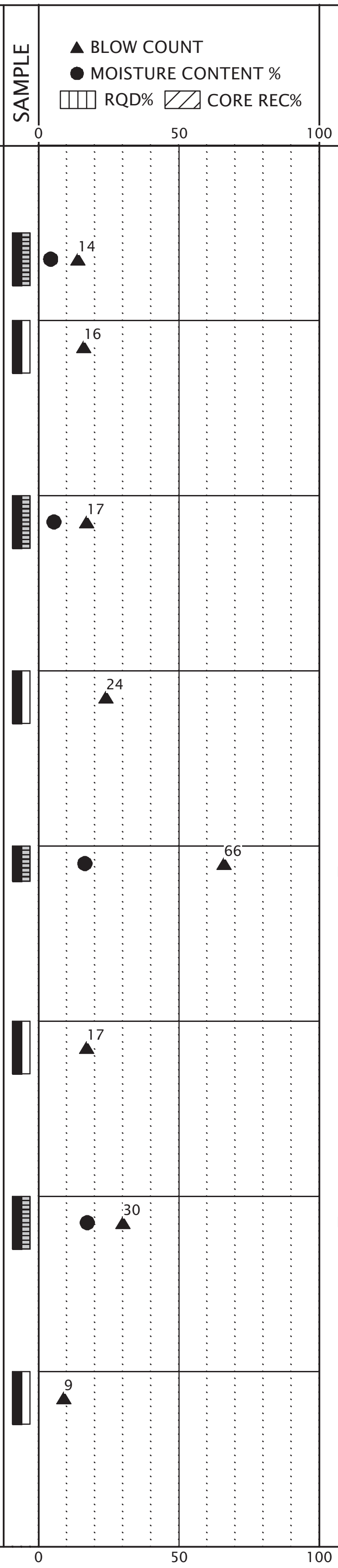
JAMES HOTEL
WEST HOLLYWOOD, CA

FIGURE A-1



DEPTH FEET	MATERIAL DESCRIPTION	ELEVATION / DEPTH	TESTING	INSTALLATION AND COMMENTS
0	Loose, brown, fine to medium, silty SAND; moist.	364.0		
5	becomes medium dense, light brown, and dry at 5.0 feet			
10				
	Loose, red-brown, fine to medium, silty SAND; moist.	353.5 / 10.5		
	Very stiff, dark red-brown, fine to medium, sandy CLAY; moist.	351.0 / 13.0		
15				
	Medium dense, dark red-brown, fine to medium, clayey SAND; moist.	345.0 / 19.0		
20			DD DS	DD = 116 pcf
25				
30			DD DS	DD = 115 pcf
35				
40	becomes loose at 39.5 feet			

▽ 04/06/06 Water measured at 27.0 feet at 10:20 AM.



DRILLED BY: 2-R Drilling, Inc.

LOGGED BY: LAS

COMPLETED: 04/06/06

BORING METHOD: hollow-stem auger (see report text)

BORING BIT DIAMETER: 8.0 in

GEODESIGN INC

2121 S Towne Centre Place - Suite 130
Anaheim CA 92806
Off 714.634.3701 Fax 714.634.3711

JAMESHOTEL-1-01

BORING B-10

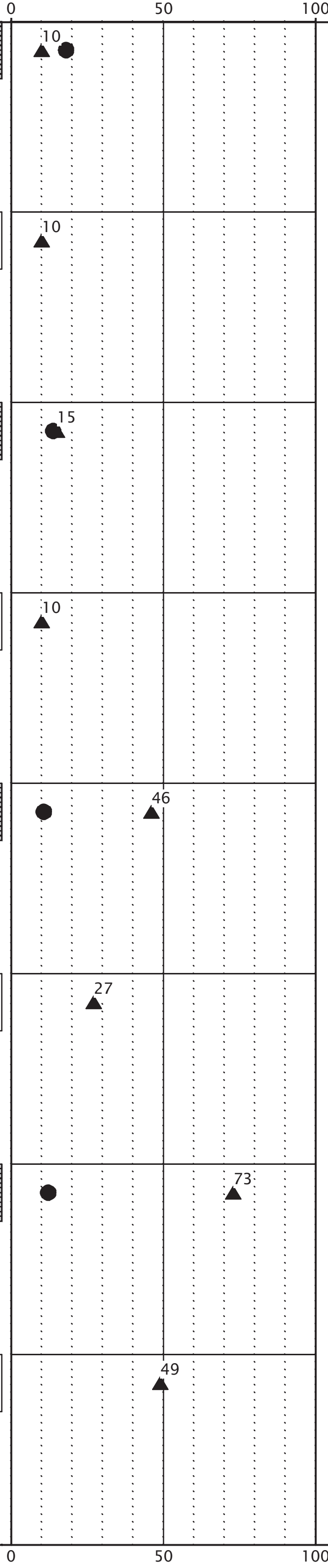
JUNE 2006

JAMES HOTEL
WEST HOLLYWOOD, CA

FIGURE A-2

▲ BLOW COUNT
● MOISTURE CONTENT %
▥ RQD% ▨ CORE REC%

DEPTH FEET	MATERIAL DESCRIPTION	ELEVATION / DEPTH	TESTING	▲ BLOW COUNT	INSTALLATION AND COMMENTS
40	(continued from previous page) Very loose, red-brown, fine to medium, clayey SAND; wet.	324.0 40.0	DD CON DS	10	DD = 114 pcf
45	becomes loose at 45.0 feet			10	
50				15	
	Very stiff, red-brown, fine to medium, sandy CLAY; moist.	311.0 53.0			
55				10	
	Medium dense, red-brown, fine to medium, clayey SAND; moist.	305.5 58.5			
60				46	
65			ATT	27	LL = 35% PL = 17%
70			DD DS	73	DD = 121 pcf
	Very stiff to hard, red-brown, fine to medium, sandy CLAY; moist to wet.	291.0 73.0			
75				49	
	Dense, red-brown, fine to medium, clayey SAND; moist.	286.0 78.0			
80					



DRILLED BY: 2-R Drilling, Inc.
LOGGED BY: LAS
COMPLETED: 04/06/06

BORING METHOD: hollow-stem auger (see report text)
BORING BIT DIAMETER: 8.0 in

GEODESIGN INC
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Anaheim CA 92806
Off 714.634.3701 Fax 714.634.3711

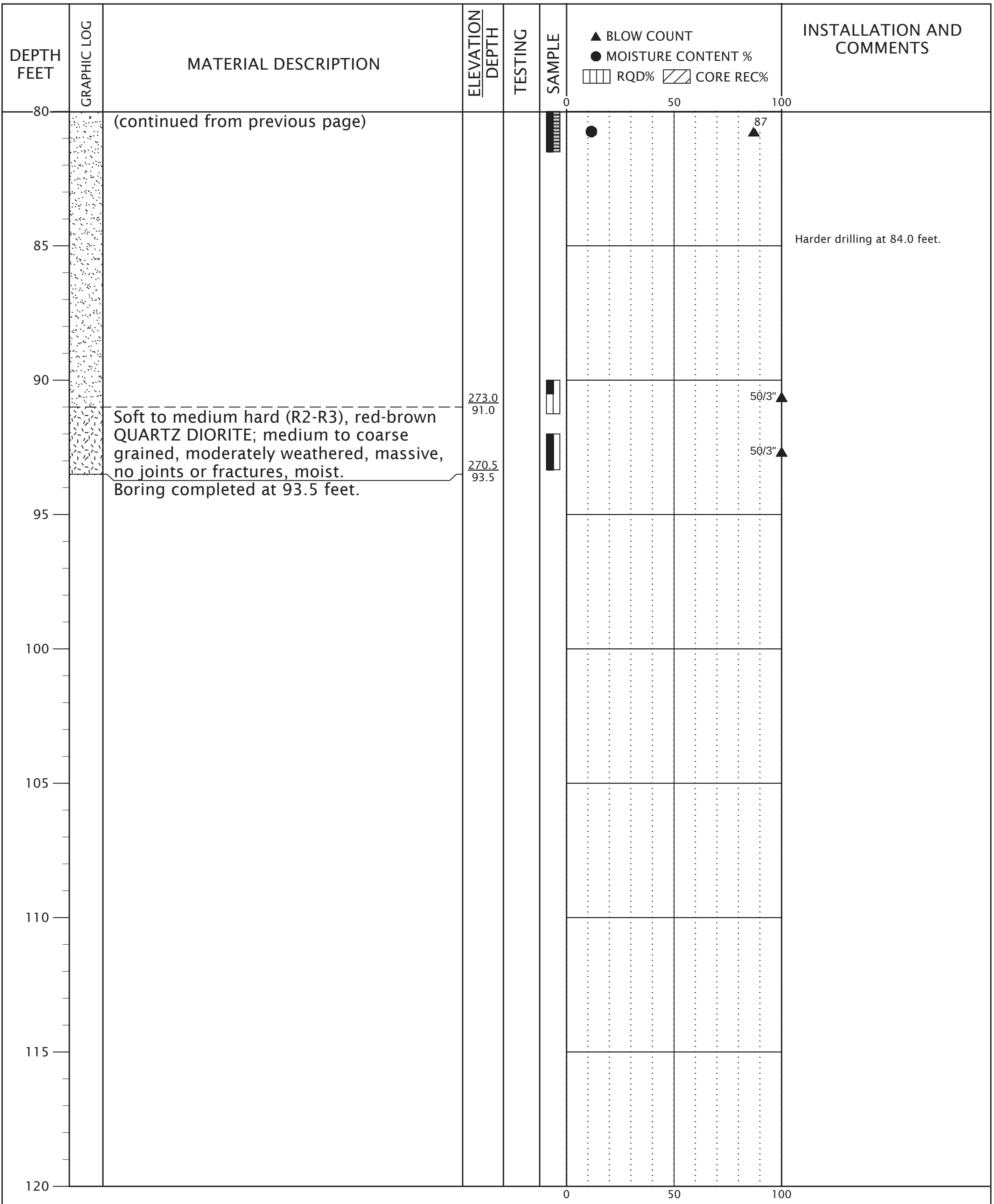
JAMESHOTEL-1-01

JUNE 2006

BORING B-10
(continued)

JAMES HOTEL
WEST HOLLYWOOD, CA

FIGURE A-2



(continued from previous page)

Soft to medium hard (R2-R3), red-brown QUARTZ DIORITE; medium to coarse grained, moderately weathered, massive, no joints or fractures, moist.

Boring completed at 93.5 feet.

Harder drilling at 84.0 feet.

DRILLED BY: 2-R Drilling, Inc.
LOGGED BY: LAS
COMPLETED: 04/06/06

BORING METHOD: hollow-stem auger (see report text)
BORING BIT DIAMETER: 8.0 in

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Anaheim CA 92806
Off 714.634.3701 Fax 714.634.3711

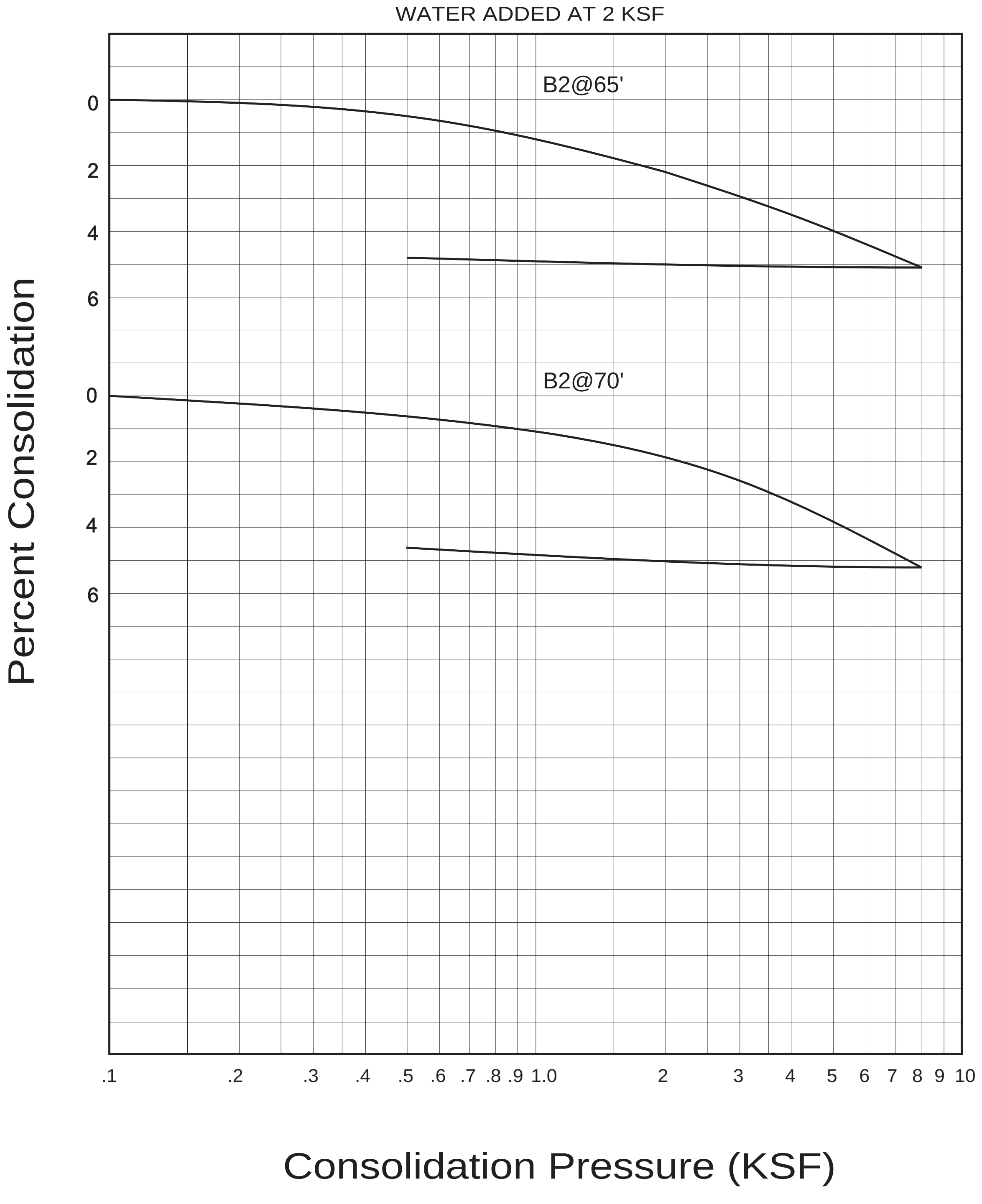
JAMESHOTEL-1-01

JUNE 2006

BORING B-10
(continued)

JAMES HOTEL
WEST HOLLYWOOD, CA

FIGURE A-2



GEOCON
WEST, INC.

ENVIRONMENTAL GEOTECHNICAL MATERIALS
3303 N. SAN FERNANDO BLVD. - SUITE 100 - BURBANK, CA 91504
PHONE (818) 841-8388 - FAX (818) 841-1704

CONSOLIDATION TEST RESULTS

8920 SUNSET BOULEVARD, LLC
8920 SUNSET BOULEVARD
WEST HOLLYWOOD, CALIFORNIA

DRAFTED BY: PZ	CHECKED BY: JMT	JULY 2015	PROJECT NO. A9286-06-01	FIG. B6

Appendix F-2

Fault Investigation

Fault Rupture Hazard Investigation
8850 to 8878 Sunset Boulevard
West Hollywood, California

Prepared by:

John Helms, CEG
40344 Wood Court
Palmdale, CA 93551
Voice (661) 609-0239
helmsceg@yahoo.com

Submitted to:

Mr. Jim Cooper
Silver Creek Commercial Development, LLC
16055 North Dial Boulevard, Suite 4
Scottsdale, CA 85260

September 19, 2018

John Helms, CEG
40344 Wood Court, Palmdale, CA 93551; (661)609-0239

Mr. Jim Cooper
Silver Creek Commercial Development, LLC
16055 North Dial Boulevard, Suite 4
Scottsdale, CA 85260

September 19, 2018

Subject: **Fault Rupture Hazard Investigation, 8850 to 8878 Sunset Boulevard, West Hollywood, California**

Dear Mr. Cooper:

I am pleased to present to you the findings of this fault rupture hazard investigation for your properties located at 8850 to 8878 Sunset Boulevard. The project site is located along the south side of Sunset Boulevard and spans the area in between San Vicente Boulevard to the east and Larrabee Street to the west, in the City of West Hollywood.

Our investigation shows that no faults are present beneath the site, or within 50 feet to the south of the property. Thus the 8850 to 8878 Sunset Boulevard project site is not exposed to the hazard of surface fault rupture, and the planned development of the site will not be impacted by any fault setback distances or 'no-build' zones.

Thank you for this opportunity to be of service. Should you have any questions or require additional information, please do not hesitate to contact me.

Sincerely,

John Helms, CEG 2272

JOHN G. HELMS
No. 2272
CERTIFIED ENGINEERING GEOLOGIST

EXECUTIVE SUMMARY

John Helms, CEG has performed a fault rupture hazard investigation for the properties located at 8850 to 8878 Sunset Boulevard, City of West Hollywood, California. The mapped trace of the Hollywood Fault Zone in the site vicinity, lies directly south of the project site. The purpose of this investigation was to assess the potential for surface fault rupture at the site and determine if the area of the planned development is suitable for the construction of human-occupied structures.

This study consisted of a review of published and unpublished data, geomorphic analysis, and subsurface exploration. The subsurface exploration program consisted of 5 cone penetration test (CPT) soundings totaling 300 feet and 5 hollow stem auger (HSA) borings, in which a total of 295 feet of borehole was drilled, logged and continuously sampled. This fault rupture evaluation of the parcel block has found no faults traversing the area. The combination of nearly continuous, unbroken Late Pleistocene soil horizons and stratigraphy, and the lack of any step or deflection in groundwater levels provide compelling evidence to demonstrate the absence of active faulting beneath the entire project site.

No faults were found to traverse the site, nor within 50 feet of the southern site boundary, and not along the northern limits of the Fault Precautionary Zone, thus the project site is not exposed to the hazard of surface fault rupture. Accordingly, no fault setback distances or “no-build” zones are delineated that would impact or limit any future development.

This investigation has shown that human-occupied structures will not be subjected to the hazard of surface fault rupture. However, the design of the structure should account for the potential of strong ground shaking that will result from future earthquakes that will occur on the nearby Hollywood, Santa Monica, New Port – Inglewood, Raymond Hill, Sierra Madre, Verdugo Mountain, and Northridge Fault zones.

TABLE OF CONTENTS

1.0 INTRODUCTION

This report presents the results of a fault rupture hazard investigation prepared by John Helms CEG for the properties located at 8850 to 8878 Sunset Boulevard in the City of West Hollywood, California (Figure 1). The Hollywood Fault Zone lies to the south of the project site.

The proposed development is located within the current State of California Earthquake Fault Zone for the Hollywood Fault Zone (Figure 5) and, therefore, the site is subject to the conditions of the Alquist-Priolo Special Studies Zone Act of 1972 (California Public Resources Code, Chapter 7.5, Division 2). The Act is designed specifically to mitigate the hazard of surface fault rupture in future earthquakes and defines a fault as active if it has demonstrated movement in Holocene time (past 11,000 years). The Alquist-Priolo Act mandates that sites located within “special studies zones”, which are delineated by the California Geologic Survey (CGS) along active faults, require detailed geologic investigation to preclude the construction of human-occupied structures astride active fault strands. The 1994 Seismic Hazards Mapping Act changed the name of the zones from Special Studies Zones to Earthquake Fault Zones (EFZ).

The proposed development is also located within the current City of West Hollywood’s Fault Precautionary Zone (Figure 2). Similar to the State of California’s Alquist-Priolo Act, the City of West Hollywood’s Fault Precautionary Zones are designed specifically to mitigate the hazard of surface fault rupture in future earthquakes and defines a fault as active if it has demonstrated movement in Holocene time (past 11,000 years).

The City of West Hollywood’s Fault Precaution Zone Map (KFM, 2010) delineates Fault Precautionary Zones 1 and 2 (FP-1 and FP-2) in which specialized, site-specific geologic investigations or special foundation designs are required to mitigate the potential hazard of surface fault rupture. Prior to any construction of human-occupied structures within the FP-1 zone, the City of West Hollywood requires a detailed geologic investigation to demonstrate the absence of active or potentially active fault strands within 50 feet of the proposed structures. The project site at 8850 to 8878 Sunset Boulevard is located along the northern boundary of the FP-1 zone (Figure 2). The purpose of this investigation, therefore, was to assess the potential for surface fault rupture at the site and determine if the area is suitable for the construction of human-occupied structures.

1.1 Site Description

The project site is located at 8850 to 8878 Sunset Boulevard, within an existing subdivision in the City of West Hollywood that lies in between San Vicente Boulevard to the East and Larrabee Street to the east (Figure 1). The project site is bound on the north by Sunset Boulevard, west by San Vicente Boulevard, and south by existing development, and to the east by Larrabee Street. The area of the proposed development is a relatively flat area that lies along a gently sloping south facing slope. Overall, the parcel consists of a several graded flat pads. The original structures on this parcel were constructed prior to the development of the Fault Precautionary Zones. Thus, this property had not previously been investigated for the hazard of surface fault rupture. The Hollywood Fault Zone is mapped to the south of the site (Figures 2, 3, and 5). In the vicinity of the project site area, the location of the Hollywood Fault Zone is poorly constrained and is mapped as being concealed or buried and approximately located.

1.2 Scope of Work

Typically, trenching is the preferred method for evaluating the presence or absence of faults because it offers a continuous, direct exposure of the fault zone or near surface stratigraphy. However, the Hollywood fault zone has been difficult to expose in trenches due to the dense urban cover and thick accumulation of young alluvium that has been deposited across the fault since the last rupture. Therefore, we proposed that this fault location study consist of a series of strategically placed CPT soundings and continuously cored HSA borings to resolve the issue of surface faulting hazard at the site.

The scope of work for this fault rupture hazard investigation at 8850 to 8878 Sunset Boulevard consisted of the following tasks:

- Review of published and unpublished geotechnical data in the site vicinity;
- Analysis of topographic maps of the site vicinity;
- Geologic reconnaissance of the site;
- Drill, continuously sample, and log a total of 295 linear feet of material in 5 hollow stem auger (HSA) borings across the site;
- Observe 300 linear feet of cone penetrometer testing (CPT) in 5 soundings across the project site area;
- Re-examine the HSA core samples to identify buried soil horizons and to visually compare and correlate alluvial units.
- Synthesize the boring log and CPT data to construct a geologic cross section that shadows the project site area;
- Preparation of this report.

John Helms, CEG, performed all tasks of this investigation. This study conforms to the provisions of the Alquist-Priolo Act and Title 24 of the California Code of Regulations.

2.0 REGIONAL AND LOCAL GEOLOGY

2.1 Geologic Setting

The project site is located in the northern Hollywood Basin, which makes up part of the Transverse Ranges Geomorphic province. The Hollywood Basin lies at the southern edge of the Transverse Ranges geomorphic province and near the northern boundary of the Peninsular Ranges geomorphic provinces (Yerkes et a1. 1965). The basin is bounded on the north by the Santa Monica Mountains and the Hollywood fault, on the east by the Elysian Hills, the west by the Newport-Inglewood Uplift and the south by the La Brea high, an area of shallow bedrock (DWR, 2004).

The most predominate structures in the project site area is the east-west trending Hollywood Fault Zone that separates older surficial deposits to the south from the bedrock units found in the Santa Monica Mountains to the north. In the project site area, alluvial fans have been created by sediments carried by water flowing out of area canyons. The adjacent area of the Santa Monica Mountains are composed primarily of Santa Monica slate and granitic rocks.

2.2 Hollywood Fault Zone

The ~15-km long Hollywood fault is expressed as a series of linear, ~N70°E to ~N78°E trending scarps and faceted south-facing ridges along the southern margin of the eastern Santa Monica Mountains. Active deposition of numerous small alluvial fans at the mountain front and a lack of fan incision suggest late Quaternary uplift of the Santa Monica Mountains along the Hollywood fault (Dolan and others, 1997; Dolan and Sieh, 1992; Crook and others, 1983). The fault dips steeply to the north and has juxtaposed pre-Tertiary granite, metamorphic, and Tertiary sedimentary rocks over young sedimentary deposits of the northern Los Angeles basin. The Hollywood fault has not produced any damaging earthquakes during the historical period and has had relatively minor micro seismic activity.

The linear trace of the Hollywood fault and steep dips found in exposures and borings (65 to 90 degrees) suggest that motion along the fault may be largely strike-slip (Dolan and Sieh, 1993). Other westerly trending faults in the Transverse Ranges exhibit a left-lateral component of slip such as the Santa Ynez, San Fernando, Raymond, and Malibu Coast faults. The orientation of the Hollywood fault suggests that the horizontal component of slip also would be left-lateral. If the entire 15-km-long Hollywood fault ruptured by itself, it could produce an Mw ~6.6 earthquake (Dolan and others, 1997). However, if the fault ruptures together with other faults to the west (Santa Monica, Malibu Coast) or to the east (Raymond), then earthquakes much larger than Mw ~6.6 could result. Assuming a minimum slip rate of 0.35 mm/yr for the Hollywood fault, Dolan and others (1997) estimate a recurrence interval of ~4,000 years for an Mw 6.6 event. Dolan and others, 2000, also documented an early to mid-Holocene earthquake on the Hollywood fault zone. The timing of the most recent earthquake is constrained between 6 and 11 ka.

The precise location of the Hollywood fault currently is poorly defined along much of its length. Large scale geomorphic features such as the southern margin of the Hollywood Hills and the over-steepened alluvial fans along this range front have provided the basis for identifying the fault’s approximate location. However, the precise locations of individual fault strands within the Hollywood fault zone have been documented only at a few sites. The Hollywood fault has been difficult to study due primarily to (1) the dense urbanization that covers nearly the entire fault trace; and (2) the accumulation of young alluvium at the base of the mountain front which locally buries the fault.

Because the city was developed primarily in the first quarter of this century before the widespread use of mechanized grading equipment, development was draped over the existing landscape with minimal modification to the natural ground surface (Dolan and others, 1997). Therefore, fault scarps and other topographic features are preserved locally beneath the pavement and can be observed along some streets of Hollywood, West Hollywood and Beverly Hills. Many of the scarps, however, are broad features of significant width (>50-200 ft) that preclude one from precisely locating a particular fault trace on geomorphic evidence alone.

2.3 Topographic Review

A combined review of previous geomorphic analyses (Dolan and others, 1997) with a review of historical US Geologic Survey topographic map (1894) and detailed City of West Hollywood topographic maps (1995) with a 2 foot contour interval was performed. The historical US Geologic Survey map is not a detailed contour interval, but shows the location of an adjacent marsh to the south of the project site. The topography on the City of West Hollywood's maps was examined for recognizable scarps or breaks in slope that may suggest the location of faulting.

No significant topographic features suggestive of surface faulting were found projecting towards or in the vicinity of the project site. A subtle break in slope is located in San Vicente Boulevard approximately 80 feet north of the project site area. This topographic feature, present on the City of West Hollywood's maps, can be observed in the field and may indicate the possible location of a strand of the Hollywood fault zone. Figure 4 is a geomorphic interpretation of the project site and surrounding area.

Significant topographic features suggestive of surface faulting were found projecting towards the vicinity of the project site. To the west, this possible strand of the Hollywood fault zone is also expressed geomorphically where it crosses Doheny Drive ~500 ft south of Sunset (Dolan and others, 1997). To the east, a strong break in slope is located across Holloway Drive. This topographic feature has been accentuated through past grading activities. A broader break in slope is present on the historical topographic maps extends south of the project site area, and can be observed in the field. These features may indicate the possible location of a northern strand of the Hollywood fault zone.

The historical US Geologic Survey map (Figure 4) shows the location of a marsh approximately 650 feet south of the project site area. This is substantiated by the presence of shallow ground water across the project site. Ground water was found to vary from 19 to 25 feet deep across the site.

2.4 Previous Investigations

A review of previous geotechnical and fault rupture hazard investigations that have been completed in the project site's vicinity were be reviewed for any information that may be pertinent to the project site area. The reports reviewed are summarized below.

Several previous fault rupture hazard investigations completed adjacent to the project site area were reviewed for information that may be pertinent to the project site area. To the east of the project site area, a fault study was reviewed at 1014 Larrabee Street by William Lettis and Associates (2006) (Figure 8). This study found study found similar stratigraphy to that encountered at 8850 to 8878 Sunset Boulevard. WLA (2006) did find an undulating groundwater levels across their site, but concluded that groundwater level changes were not systematic and did not represent a buried fault strand. This study found no faulting across the site.

To the west of the project site area, two fault studies were reviewed at 1019 San Vicente Boulevard by MACTEC (2004) and FUGRO West (2004) (Figure 9). These studies also found similar stratigraphy and groundwater levels to those encountered at the 8850 to 8878 Sunset Boulevard project site. The MACTEC (2004) and FUGRO

West (2004) studies found no faulting across the site. The transect from this study shadows the northwestern portion of the project site area (Figure 6).

Farther to the west of the project site area, a fault study was reviewed at 1016-1020 Hilldale Avenue by Advanced Geotechniques (1998). This study found study found a thicker section of Holocene aged alluvial stratigraphy with similar groundwater levels to those encountered at the 8850 to 8878 Sunset Boulevard project site. The Advanced Geotechniques (1998) study also found no faulting across the site.

To the southeast of the project site area, fault studies was reviewed at 8703 West Knoll Drive by Earth Consultants International (2003) and at 1006 Hancock Avenue by Subsurface Designs (2017). These studies found no faulting with a very thick section of Holocene aged alluvial stratigraphy and shallow groundwater conditions. ECI (2003) did find artesian groundwater conditions along the southern portion of their study area and concluded that a fault strand could be impeding the groundwater flow south of West Knoll Drive.

South of the project site area several fault studies at the 966 and 972 North San Vicente Boulevard properties were reviewed (Fugro, 2005, GeoConcepts, 2016, and Helms CEG, 2017). The Fugro (2005) preliminary study consisted of 3 three continuously cored borings widely spaced apart between the 966 and 972 North San Vicente Boulevard properties. Fugro (2005) found similar Pleistocene aged stratigraphy to that encountered in this study. Fugro (2005) also found similar undulating groundwater levels across their northern two borings. Fugro concluded that the observed groundwater level changes in this vicinity may represent a buried fault strand.

A second preliminary fault study by GeoConcepts, Inc. (2016) consisted of 4 four continuously cored borings evenly spaced apart west of the 966 and 972 North San Vicente Boulevard properties. This preliminary study found three discrete buried shear zones with corresponding groundwater level and stratigraphic changes. The GeoConcepts, Inc. (2016) preliminary fault study found stratigraphy and groundwater changes with shear zones along the west side of North San Vicente Boulevard transect. This enabled an estimate in the strike (or orientation) and structure of the fault zone in the vicinity of the project site area.

A final fault study was conducted only for the 972 North San Vicente Boulevard property (Helms CEG, 2017) consisted of six continuously cored borings and 6 CPT soundings along the east side of North San Vicente Boulevard. This study was able to better characterize three discrete buried shear zones with corresponding groundwater level and stratigraphic changes. The Helms, CEG (2017) fault study was able to age date the unfaulted stratigraphy and this enabled the fault zone in the vicinity of the project site area to be deemed in-active.

Farther south of the project site area at 935 North San Vicente Boulevard Advanced Geotechniques (2013). This study consisted of two continuously cored borings and 5 CPT soundings, and found a thick section of Pleistocene aged alluvial stratigraphy. Water levels are very shallow and consistent across this study area, and key stratigraphic markers at depth appear consistent across the project site area. The Advanced Geotechniques (2013) study found no faulting across the site. The shallow groundwater encountered at the southern end of 935 North San Vicente Boulevard (>5 feet depth) indicates that a fault strand could be impeding the groundwater flow south of the subject site.

Lastly, a newly released Fault Evaluation Reports by the California Geological Survey (2017 and 2018) were reviewed for any information that may be pertinent to the project site area (Figure 5a and 5). The CGS notes that the Hollywood Fault in the project site area is expressed by an over-steepening of the alluvial fans at the base of the mountains and several small scarps and subtle breaks in slope visible on the historic topographic maps. Some less-prominent scarps near the base of the hills west of West Hollywood, are thought to be related to a paleo-shoreline identified in the area (Figure 4). Additionally, the scarps noted along Sunset Boulevard in this same the area adjacent to the project site area are likely related to grading for the original road construction. South of the project site, the significant break in slope observed south of Santa Monica Boulevard likely marks the distal extent of the alluvial fan emanating from Rising Glen Road canyon, and is probably not related to faulting due to a lack of discernable scarps or slope breaks east or west of the feature. Figure 5a and 5 shows that south of the project site area there is a semi-continuous series of slope breaks extending over 1 km west from Larrabee Street, which is mapped as the main Hollywood Fault zone. The Hollywood Fault zone is mapped by the state as dying out or becoming in-active farther to the west.

3.0 METHODOLOGY

3.1 Approach

The subsurface investigation was designed to investigate across the entire subject parcel (Figure 6). Typically, trenching is the preferred method for evaluating the presence or absence of faults because it offers a continuous, direct exposure of the fault zone or near surface stratigraphy. However, the Hollywood fault zone has been difficult to expose in trenches due to the dense urban cover and thick accumulation of young alluvium that has been deposited across the fault since the last rupture. Therefore, we proposed that this fault location study consist of a series or transect of strategically placed, continuously cored hollow stem auger (HSA) borings and cone penetrometer testing (CPT) soundings.

The boreholes were located on a 1 inch = 20 foot scale base map (Figure 6). Our exploration program consisted of a single line of continuously cored hollow stem auger (HSA) borings and cone penetrometer testing (CPT) soundings. The transect was approximately 230 feet long and was designed to capture any east northeast striking fault strands of the Hollywood Fault zone that might traverse the site or run along the northern limits of the fault precautionary zone or within 50 feet from the southwest corner of the property (Figure 10). Due to site access limitations and current structures located on the property, the north-south trending HSA and CPT transect was located within the western parking lane of Larrabee Street.

3.2 Field Exploration

Prior to beginning the subsurface field exploration, a literature review, topographic analysis, and geologic reconnaissance of the site was performed. Following this general review, Underground Service Alert (USA) was notified to identify buried utilities in the vicinity of the proposed excavations, as required by law.

Subsurface conditions at the site were explored in two phases along a single transect along North San Vicente Boulevard (Figure 3). The first phase was performed on June 26 through July 2, 2017 and included drilling and sampling five hollow stem auger (HSA) borings. The continuously sampled HSA borings were drilled to depths that ranged between 45 and 65 feet beneath the existing ground surface for a total footage of 295 feet (Table 1). The second phase was performed on July 11, 2017 and included 5 cone penetrometer testing (CPT) soundings to depths 60 feet beneath the existing ground surface each for a total footage of 300 feet (Table 2). The boring logs are included in Appendix A and the CPT data are included in Appendix B.

The HSA borings were drilled by One Way Drilling using both a truck mounted and limited access 8-inch diameter hollow stem auger drilling equipment. Continuous sampling of all borings was performed using a CME continuous sampling system. The core samples were collected for the purpose of visual classification and to accurately determine the elevations of buried soils and primary alluvial stratigraphy. The core samples were boxed and stored to allow for further inspections and a more detailed examination of the buried soils.

4.0 RESULT*S*

This investigation shows that there are no active faults in the area explored. No lineaments or geomorphic features suggestive of active faulting traverse or project towards the project site.

4.1 Groundwater

An important indicator for the presence or absence of faulting is the depth to groundwater. Past studies have shown that both inactive and active fault strands along the Hollywood Fault zone act as groundwater barriers and produce abrupt steps in the groundwater surface.

Along the attached cross section (Figure 7), the groundwater levels were directly measured in both the open HSA borings and CPT soundings. These data define a gently southward sloping groundwater surface that drops only about 6 feet across the entire cross section with no evidence of an abrupt step in the groundwater surface. Depths to groundwater ranged from 19 to 26 feet below the ground surface (Tables 1 and 2). Adjacent sites to the west (Figure 8) along North San Vicente Boulevard and to the east (Figure 9) along Larrabee Street also show a similar gently southward sloping groundwater surface that shows no evidence of an abrupt step in the groundwater surface.

4.2 Alluvium and Soil Horizons

The continuity of soil horizons and primary stratigraphic contacts provides essential data to evaluate the presence or absence of faulting. Four continuous and conformable stratigraphic units within the alluvium were encountered in each of the HSA borings and CPT soundings along cross section A-A’ (Figure 7). The attached Table 3 (Stratigraphic Summary Table) compares and contrasts the stratigraphic units described below. The uppermost unit (Qal) in section A-A’ is interpreted as a Holocene aged alluvial deposit and appears continuous and unbroken across the northern half of the cross section. A thin veneer of artificial fill and the road pavement is shown along the ground surface across the entire cross section.

Unit Qal is the youngest unit encountered during the subsurface exploration and is interpreted to be a Holocene aged alluvial deposit. It consists of a highly truncated and weakly developed soil, and typically is slightly hard, slightly moist to dry, silty SAND that is coarse-grained with few fine gravel and little clay. This unit is characterized with moderately low CPT friction ratios and the basal contact of this unit with Qoa1 is characterized as a gradational increase in friction ratio (Figure 7).

Unit Qoa1 directly underlies unit Qal and is interpreted as latest Pleistocene alluvial fan deposits. This unit consists of a series of two stacked, truncated, and strongly developed argillic soil profiles, and are characterized as a hard, slightly moist to moist, clayey SAND. There is moderately strong pedogenic development within this unit in the form of significant clay film development, moderately strong oxidation, and a moist color hue of 7.5YR. The CPT friction ratios for this unit grade from low to high as the clay content gradationally decreases with depth. The basal contact of this unit with Qoa2 is characterized as stacked across the area investigated where the CPT signature shows no decrease in friction ratio (Figure 7).

Unit Qoa2 directly underlies unit Qoa1 and is also interpreted as late Pleistocene alluvial fan deposits. This unit consists of a series of two truncated and stacked moderately well-developed argillic soil profiles. The material is characterized as a slightly hard to hard, moist to wet, silty SAND with clay to clayey SAND with localized gravel. The moderately strong pedogenic development within this unit has been locally gleyed (or reduced) through prolonged exposure to anoxic groundwater conditions at this depth. The CPT friction ratios for this unit undulates from low to high as the clay content gradationally decreases and increases with depth and the basal contact of this unit with Qoa3 is characterized as an stacked and varied contact and generally shows an subtle decrease in friction ratio (Figure 7).

Unit Qoa3 directly underlies unit Qoa2 and is interpreted as Pleistocene alluvial fan deposits. This unit consists of two stacked thick argillic soil profiles. The material is characterized as a hard, wet, clayey SAND with localized gravel to sandy CLAY. The strong pedogenic development in the form of advanced clay film development within this unit is also locally gleyed (or reduced) in clay rich zones through prolonged exposure to anoxic groundwater conditions at depth. The CPT friction ratios for this unit are relatively low as the clay content is relatively high and the basal contact of this unit was not determined (Figure 7).

No features characteristic of faulting, such as shear zones or high angle contacts between units were observed within the five HSA borings. These several stratigraphic units described above and in Table 3 provide visually and texturally distinct, mappable contacts that are overlapping along the entire length of the transect.

5.0 EVIDENCE FOR THE ABSENCE OF FAULTING

Several subsurface geologic relationships at the 8850 to 8878 Sunset Boulevard project site provide direct evidence to preclude the presence of Holocene faulting. The topographic / geomorphic analysis also provides indirect evidence that the site is not traversed by active faults. When these relationships are considered together, there is compelling evidence for the absence of faulting beneath the subject site. The primary lines of evidence that support the interpretation that no faults traverse the site are:

- **Continuous, unfaulted soil horizons and primary stratigraphy across and adjacent to the site.** Cross section A-A’, B-B’, and C-C’ (Figures 7, 8, and 9) all exhibit multiple similar continuous stratigraphic horizons across each entire HSA and CPT transect. The conclusion that these units are not faulted is based on the assumption that any faults would exhibit a vertical slip component that, over repeated seismic events, would produce recognizable, vertical separations of the units. It would be more difficult to make this case for a pure strike-slip fault. However, even strike-slip faults would likely produce an apparent dip-slip component or truncation of units due to the juxtaposition of different Pleistocene strata or pedogenic horizons.

- **No faults were encountered in the subsurface exploration.** No features characteristic of faulting, such as shear zones or high angle contacts between units were observed within the five HSA borings. This line of evidence by itself is not considered compelling enough to preclude the presence of faulting, but it is consistent with and corroborates the other lines of evidence.

- **No abrupt steps in the groundwater surface.** Many of the faults encountered in West Hollywood act as groundwater barriers and produce abrupt steps in the groundwater surface. Cross sections A-A’, B-B’, and C-C’ (Figures 7, 8, and 9) all define a gently southward sloping groundwater surface that drops across the entire project site area with no evidence of an abrupt step in the groundwater surface. No steps in the groundwater surface were encountered in the area explored for this project. This line of evidence by itself is not considered compelling enough to preclude the presence of faulting, but it is consistent with and corroborates the other lines of evidence.

- **No irregularities or topographic features indicative of faulting were observed in the project site area.** A subtle break in slope is located in San Vicente Boulevard approximately 80 feet south of the project site area. This topographic feature may indicate the possible location of a strand of the Hollywood fault zone. However, this feature may have non-tectonic origin and may simply represent a segmentation of the alluvial fan surface through depositional or fluvial processes.

6.0 CONCLUSIONS AND RECOMMENDATIONS

This fault rupture evaluation at 8850 to 8878 Sunset Boulevard has found no faults traversing the subject property. The presence of multiple continuous Pleistocene stratigraphic horizons and a lack of a step in the groundwater surface provide compelling evidence to demonstrate the absence of active faulting beneath the site.

Because no faults were found to traverse the site across the northern limits of the Fault Precautionary Zone and more than 50 feet beyond the southern site boundaries, the project site is not exposed to the hazard of surface fault rupture. Therefore, we have not delineated any ‘no-build’ zones or fault setback distances that would impact or limit future development of the site.

The main trace of the Hollywood fault zone is likely located south of the project site. While the area explored in our study is not subject to the hazard of surface faulting, a future earthquake on the Hollywood or Santa Monica fault zones will likely produce very strong, near-field ground motions at the 8850 to 8878 Sunset Boulevard project site that could possibly exceed the provisions set forth in the current building codes.

7.0 LIMITATIONS

The conclusions and recommendations presented herein are the results of an inherently limited scope. Specifically, the scope of services consisted of an assessment of whether or not active faults are present within the area explored at the site. The conclusions and recommendations contained in this report are professional opinions derived in accordance with current standards of professional practice. No warranty is expressed or implied.

This report is intended to be used only in its entirety. No portion or section of the report, by itself, is designed to completely represent any aspect of the project described herein. If any reader requires additional information or has questions regarding this report, John Helms, CEG should be contacted.

Subsurface conditions were interpreted on the basis of our field explorations and past experience. Although, between exploratory excavations, subsurface earth materials may vary in type, strength and many other properties from those interpreted. The findings, conclusions and recommendations presented herein are for the soil conditions encountered in the specific locations. Earth materials and conditions immediately adjacent to, or beneath those observed may have different characteristics, such as, earth type, physical properties and strength. Other soil conditions due to non-uniformity of the soil conditions or manmade alterations may be revealed during construction. If subsurface conditions differ from those encountered in the described exploration, this office should be advised immediately so that further recommendations may be made if required. If it is desired to minimize the possibility of such changes, additional explorations and testing can/should be performed.

This Report has been prepared for the exclusive *use and benefit of, and may be relied upon by (i) CIT Bank, N.A., as Administrative Agent and for the benefit of the Lenders in connection with a financing of the subject property, (ii) any participating lenders in such financing, (iii) any successors and assigns of the foregoing and (iv) the respective affiliates, agents and advisors of the foregoing (the parties identified in clauses (i)-(iv) being referred to herein as the "Reliance Parties"). Each Reliance Party may use and rely on this Report in its entirety (whether in paper, digital, electronic, or any other form). In addition, the Reliance Parties may share the Report with their authorized reviewers, examiners, auditors and regulators.*

This report applies only to the proposed development located at 8850 to 8878 Sunset Boulevard in the City of West Hollywood, California. In the event that significant changes in the construction plans should occur, the conclusions and recommendations contained in this report shall not be considered valid unless the changes are reviewed by John Helms CEG, and the conclusions and recommendations of this report are verified in writing.

8.0 REFERENCES

Advanced Geotechniques, 1998, Geological Fault Study, Proposed Residential Buildings, 1016, 1018, and 1020 Hilldale Avenue, West Hollywood, CA. Consultant report prepared for Harvard Investment Group, Dated July 8, 1998. Job No. 98-403G, pp. 12.

Advanced Geotechniques, 2013, Geological Fault Study, Proposed Residential Development, 935 North San Vicente Boulevard, West Hollywood, CA. Consultant report prepared for Mrs. Dorianne Mir of the 5to9 Group, Dated July 8, 1998. Job No. 98-403G, pp. 12.

California Geological Survey, 2017 Preliminary Map of Alquist-Priolo Earthquake Fault Zones, Beverly Hills Quadrangle (1979), Scale 1:24,000.

California Geological Survey, 2017, Preliminary Seismic Hazard Evaluation Report for the Beverly Hills 7.5' Quadrangle, Seismic Hazard Zone Report 259, pp. 72.

California Geological Survey, 2018, Official Map of Alquist-Priolo Earthquake Fault Zones, Beverly Hills Quadrangle (1979), Scale 1:24,000.

California Geological Survey, 2018, Final Seismic Hazard Evaluation Report for the Beverly Hills 7.5' Quadrangle, Seismic Hazard Zone Report 259, pp. 72.

California Department of Water Resources (DWR), 2004. Bulletin 118 – Coastal Plain of Los Angeles Groundwater Basin, Hollywood Subbasin. Updated 2/27/04. Website: http://www.dpla2.water.ca.gov/publications/groundwater/bulletin118/basins/pdfs_desc/4 11.02.pdf

Crook, R., Allen, C., Kamb, B., Payne, C., and Proctor, R., 1987, Quaternary Geology and Seismic Hazards of the Sierra Madre and Associated Faults, Western San Gabriel Mountains, in Recent Reverse Faulting in the Transverse Ranges, California: US Geological Survey Professional Paper 1339:27-63, map scale 1:24,000.

Dibblee, T.W., Jr., 1991, Geologic Map of the Beverly Hills and Van Nuys (South ½) Quadrangles, Los Angeles County, California, The Dibblee Geological Foundation, DF-31.

Dolan, J. F., and Sieh, K. E., 1992, Structural style and tectonic geomorphology of the Northern Los Angeles Basin: Seismic hazards and kinematics of recent fault movement, Proceedings of the 35th Annual Meeting, Association of Engineering Geologists, P. 621-622.

Dolan, J., Sieh, K., Rockwell, T., Guptill, P., Miller, G., 1997, Active tectonics, paleoseismology and seismic hazards associated with the Hollywood fault, northern Los Angeles basin, California, Geological Society of America Bulletin, v. 109, no. 12, p. 1595-1616.

Earth Consultants International, 2003, Report, Study of the Potential for Surface Fault Rupture in the Southern Portion Plus 50 Feet South, of a Proposed Development at 8703 West Knoll Drive in the City of West Hollywood, Los Angeles County, CA. Consultant report prepared for Mr. Zed Shooshani, Dated June 4, 2003. Job No. 2206.01, pp. 26.

FUGRO West Inc., 2004, Report of Fault Rupture Hazard Study, 1019 San Vicente Boulevard, West Hollywood, CA. Consultant report prepared for Mr. Howard Fundenburg, Dated August 11, 2004. Job No. 3407.001, pp. 5.

GeoConcepts, Inc., 2015, Fault Rupture Hazard Investigation 948, 950, 954 North San Vicente Boulevard, West Hollywood, California. Consultant report prepared for San Vicente Limited, Dated August 6, 2015. Job No. 5004-1, pp. 10.

GeoConcepts, Inc., 2016, Preliminary Fault Rupture Hazard Investigation 966 and 972 North San Vicente Boulevard, West Hollywood, California. Consultant report prepared for escrow, Dated January 2016. Job No. 5038-1, pp. 3.

KFM Geoscience, 2010, Geologic and Seismic Technical Background Report City of West Hollywood General Plan Update, West Hollywood, Los Angeles County, California, pp. 18.

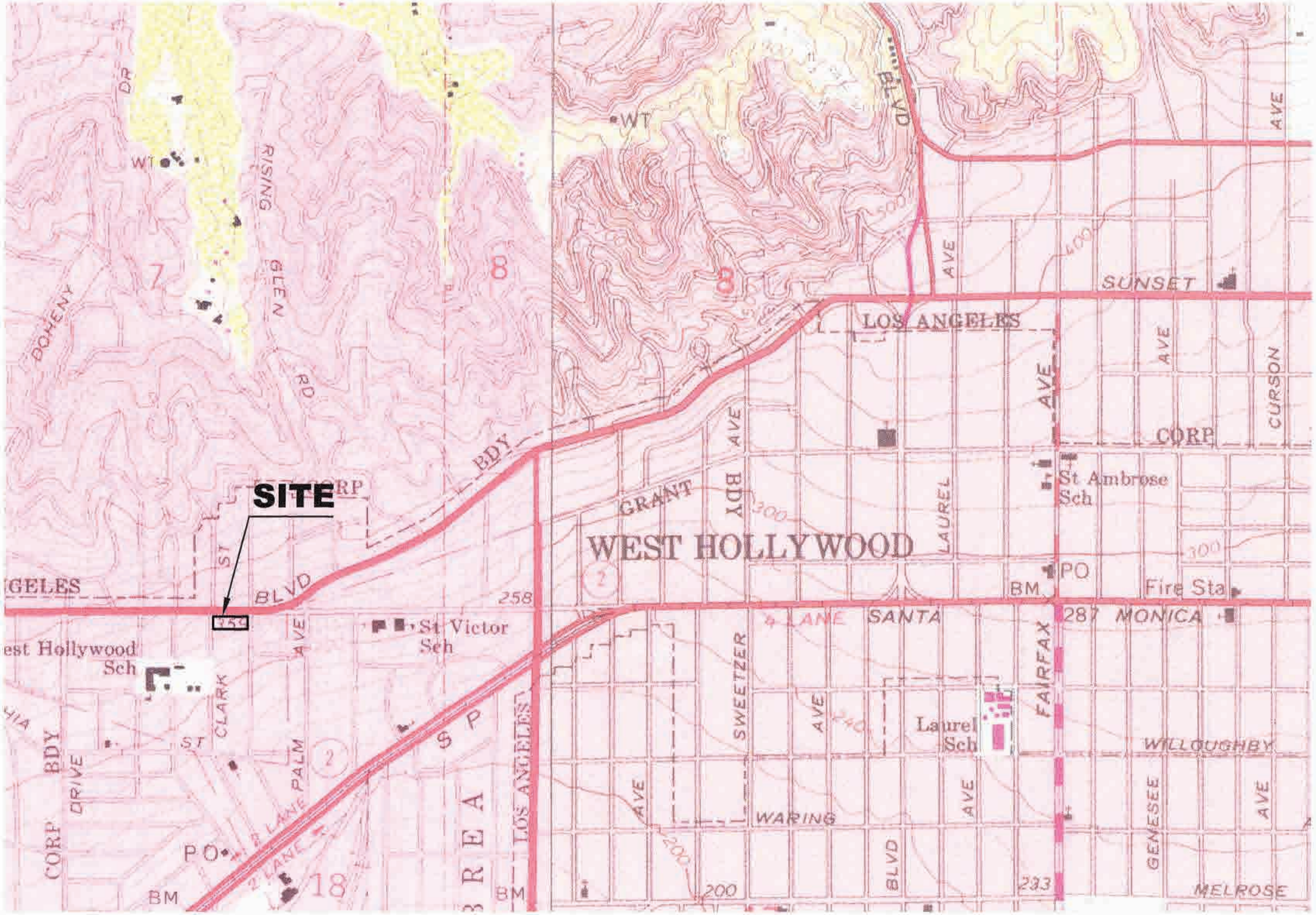
MACTEC, 2004, Report of Fault Rupture Hazard Study, Proposed Residential Development, 1019 San Vicente Boulevard, West Hollywood, CA. Consultant report prepared for San Vicente LLP, Inc., Dated December 7, 2004. Job No. 4953-04-3271, pp. 15.

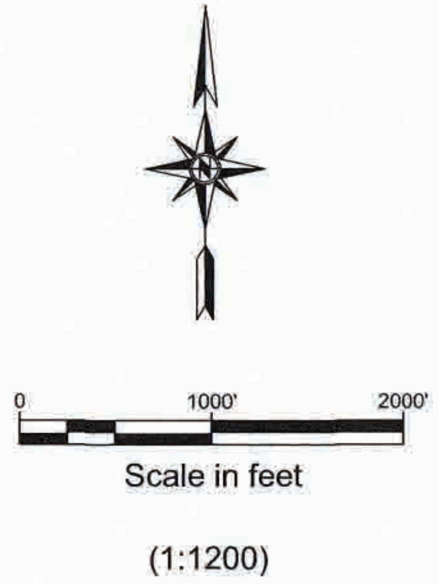
Subsurface Designs, Inc., 2017, Fault Rupture Hazard And Soils Engineering Investigation 1006 Hancock Street, West Hollywood, California. Consultant report prepared for MR. SIAMAK BAGHERI / LOC ARCHITECTS, Dated May 17, 2017. Job No. 6667, pp. 39.

William Lettis and Associates Inc., 2006, Fault Rupture Hazard Investigation, 1014 Larrabee Street, City of West Hollywood, CA. Consultant report prepared for Ms. Rima Jian Tabrizy, Dated January 17, 2006. Job No. 1776, pp. 11.

Yerkes, R.F., McCulloh, T.H., Schoellhamer, J.E., and Vedder, J.G., 1965, Geology of the Los Angeles Basin – An Introduction, United States Geological Survey Professional Paper 420-A, pp. 55.

US Geological Survey, 1894, Los Angeles Sheet 100,000 scale topographic map.





Title

Site Location Map

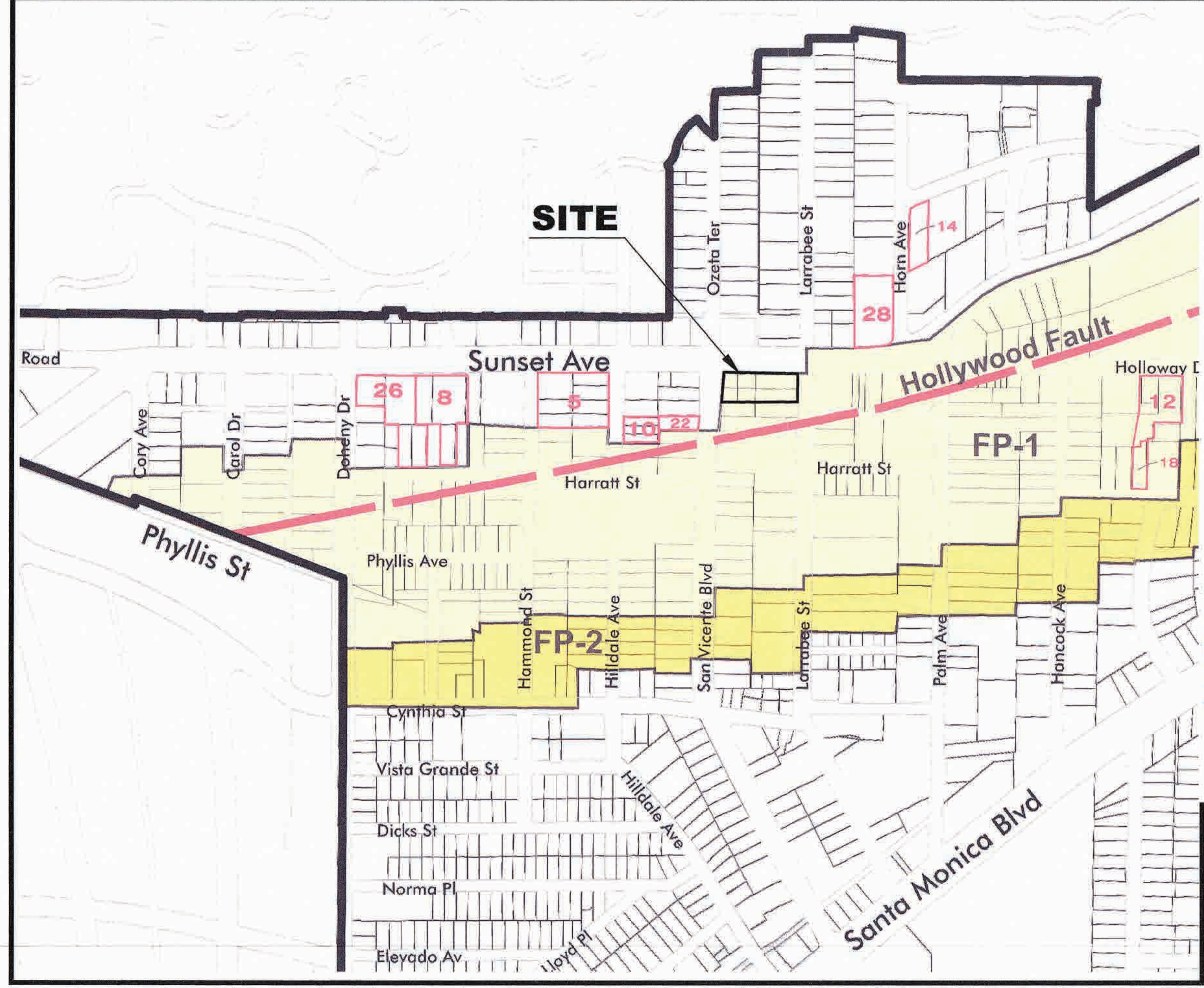
Project

8852 to 8874 Sunset Boulevard
West Hollywood, California

John Helms, CEG

FIGURE 1

Reference:
City of West Hollywood General Plan



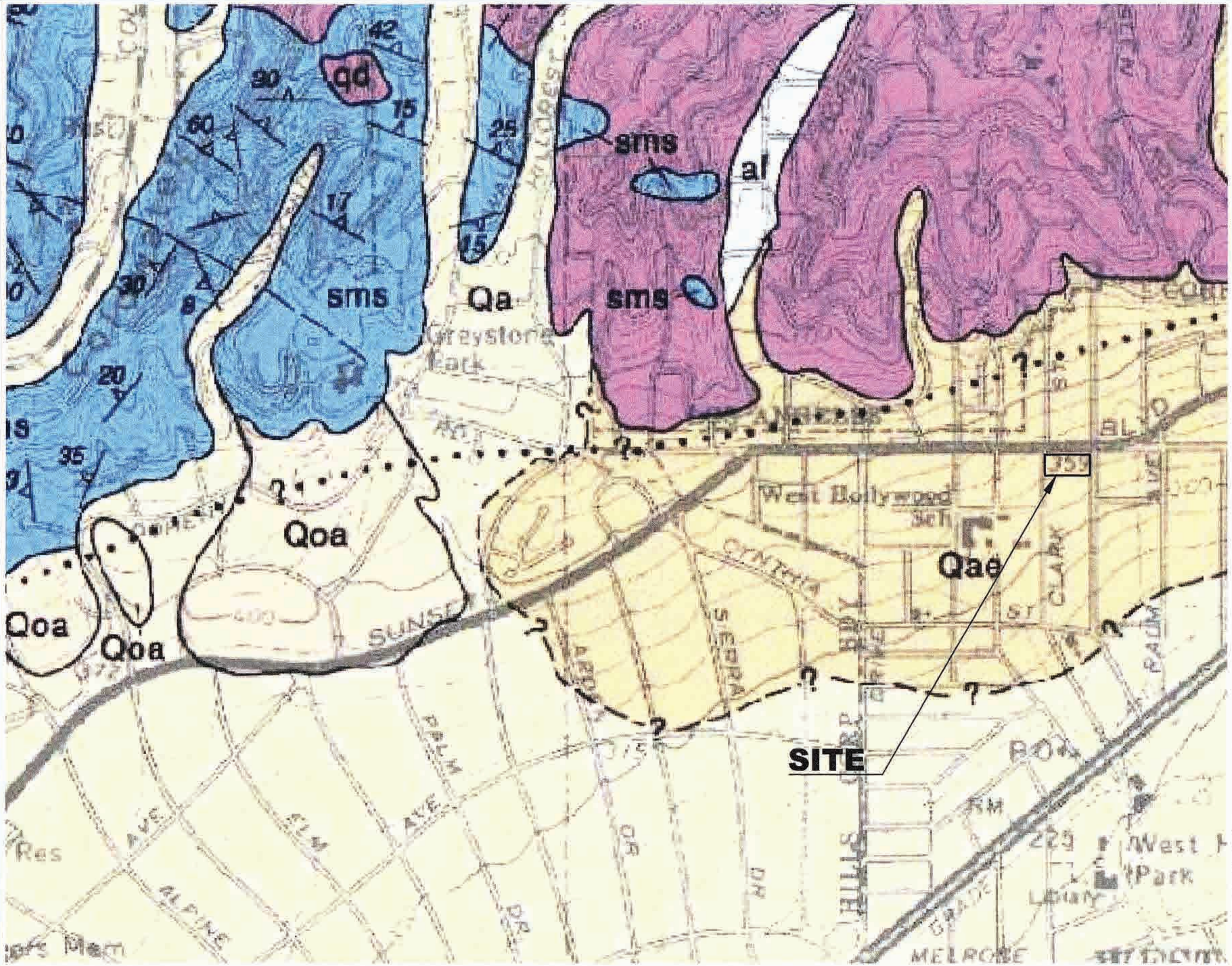
LEGEND

FP-1	City of West Hollywood Fault Precaution Zone, FP-1: Requires site-specific fault rupture evaluation by California Certified Engineering Geologist. FP-1excludes properties at which previous fault evaluation studies have shown that active faulting is not present.
FP-2	City of West Hollywood Fault Precaution Zone, FP-2: Requires fault rupture evaluation by California Certified Engineering Geologist and/or strengthening of foundations to provide for estimated ground displacement of 1 to 2 inches.
— — -	Approximate surface trace of the Hollywood Fault
—— - -	Approximate surface trace of active subsidiary splay of the Hollywood Fault
········	Approximate projected location of surface trace of Santa Monica Fault: No studies required.
⬠	Location of fault study. See Table 1 for summary of numbered fault studies.

Title
Fault Location Map

Project
8852 to 8874 Sunset Boulevard
West Hollywood, California

John Helms, CEG **FIGURE 2**

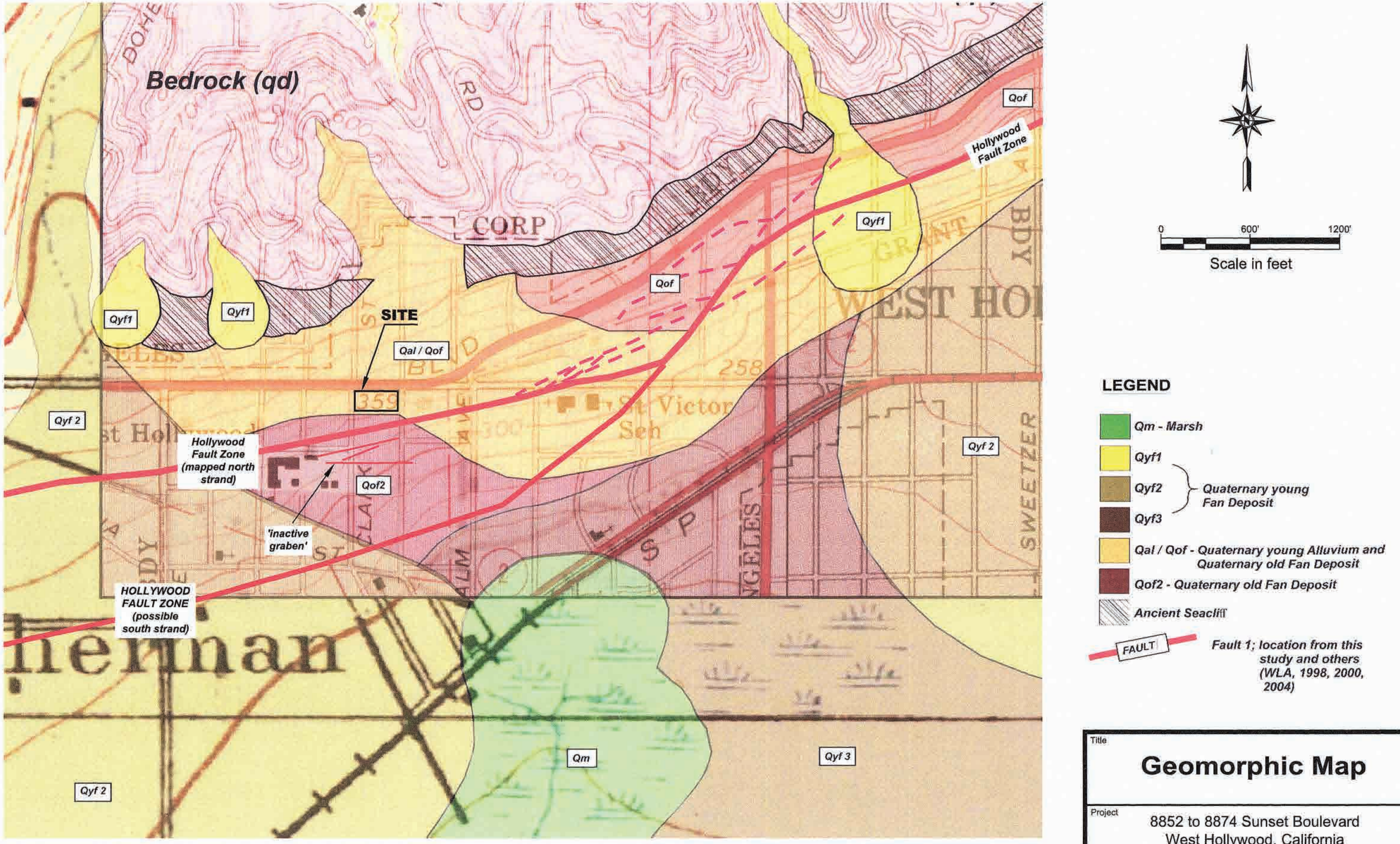


source maps: Dibblee, T.W. and Ehrenspeck, H.E., ed: Geologic Map of the Beverly Hills & Van Nuys (south 1/2) quadrangles, Los Angeles County, California, 1991, DF-31, scale 1:24,000

Title
Regional Geologic Map

Project
8852 to 8874 Sunset Boulevard
West Hollywood, California

John Helms, CEG **FIGURE 3**



source maps: 1966 7.5 Minute USGS Topographic Maps, Beverly Hills & Hollywood Quadrangles;
1897 USGS Topographic Map: "California-Los Angeles County", 1:62,500 scale

Title

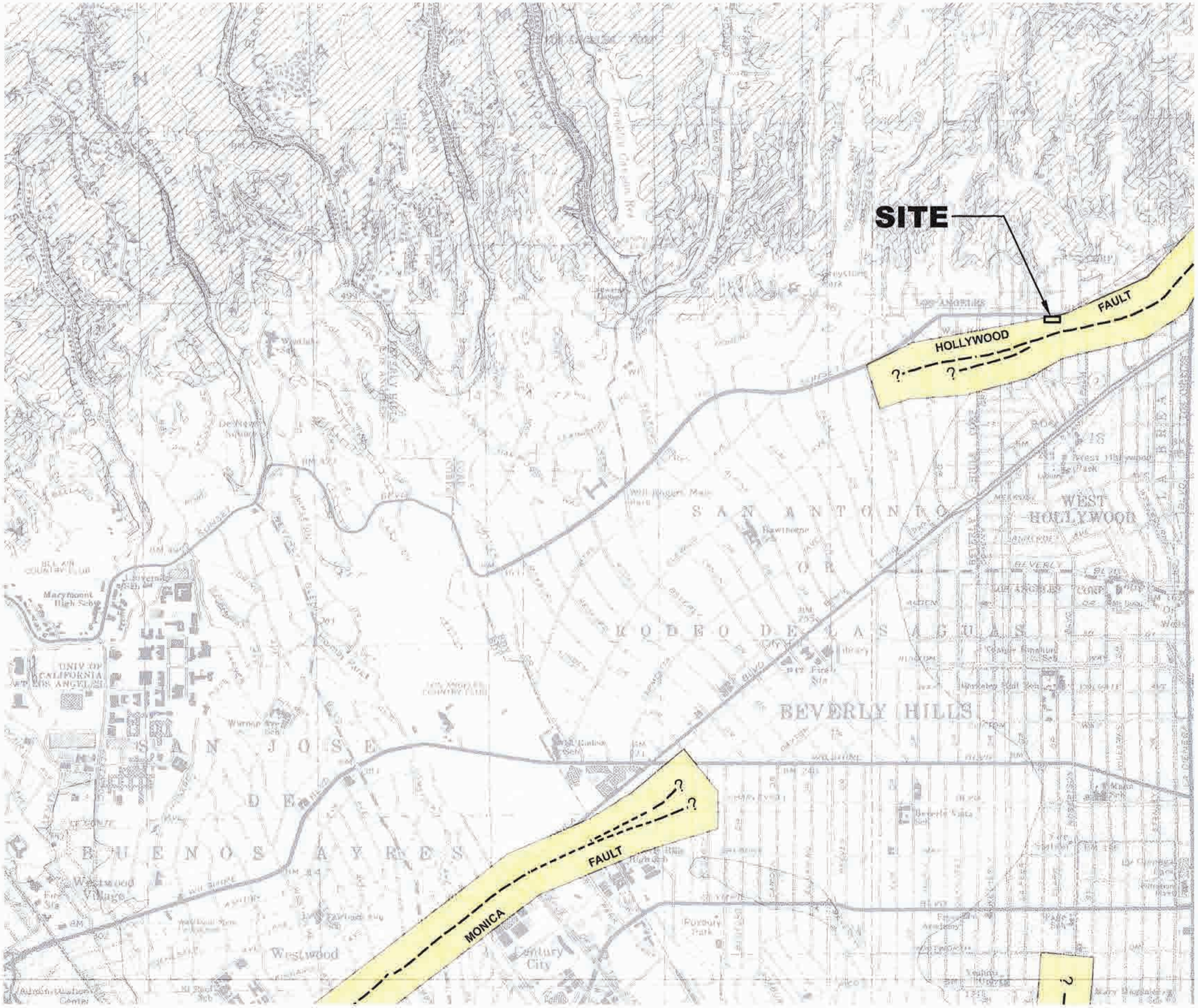
Geomorphic Map

Project

8852 to 8874 Sunset Boulevard
West Hollywood, California

John Helms, CEG

FIGURE 4



0 2000' 4000'

Scale in feet

EXPLANATION

ALQUIST-PRIOLO EARTHQUAKE FAULT ZONES

🟨 **Earthquake Fault Zones**
Zone boundaries are delineated by straight-line segments; the boundaries define the zone encompassing active faults that constitute a potential hazard to structures from surface faulting or fault creep such that avoidance as described in Public Resources Code Section 2621.5(a) would be required.

☐ **Earthquake Fault Zones**
(Not considered for this Preliminary Review)
Zone boundaries are delineated as straight-line segments; the boundaries define the zone encompassing active faults that constitute a potential hazard to structures from surface faulting or fault creep such that avoidance as described in Public Resources Code Section 2621.5(a) would be required.

1906 C ?

Active Fault Traces
Faults considered to have been active during Holocene time and to have potential for surface rupture: Solid Line in Black or Red where Accurately Located; Long Dash in Black or Solid Line in Purple where Approximately Located; Short Dash in Black or Solid Line in Orange where Inferred; Dotted Line in Black or Solid Line in Rose where Concealed; Query (?) indicates additional uncertainty. Evidence of historic offset indicated by year of earthquake-associated event or C for displacement caused by fault creep.

SEISMIC HAZARD ZONES

Liquefaction Zones
Areas where historical occurrence of liquefaction, or local geological, geotechnical and ground water conditions indicate a potential for permanent ground displacements such that mitigation as defined in Public Resources Code Section 2693(c) would be required.

Earthquake-Induced Landslide Zones
Areas where previous occurrence of landslide movement, or local topographic, geological, geotechnical and subsurface water conditions indicate a potential for permanent ground displacements such that mitigation as defined in Public Resources Code Section 2693(c) would be required.

Earthquake Zones of Required Investigation
Beverly Hills Quadrangle

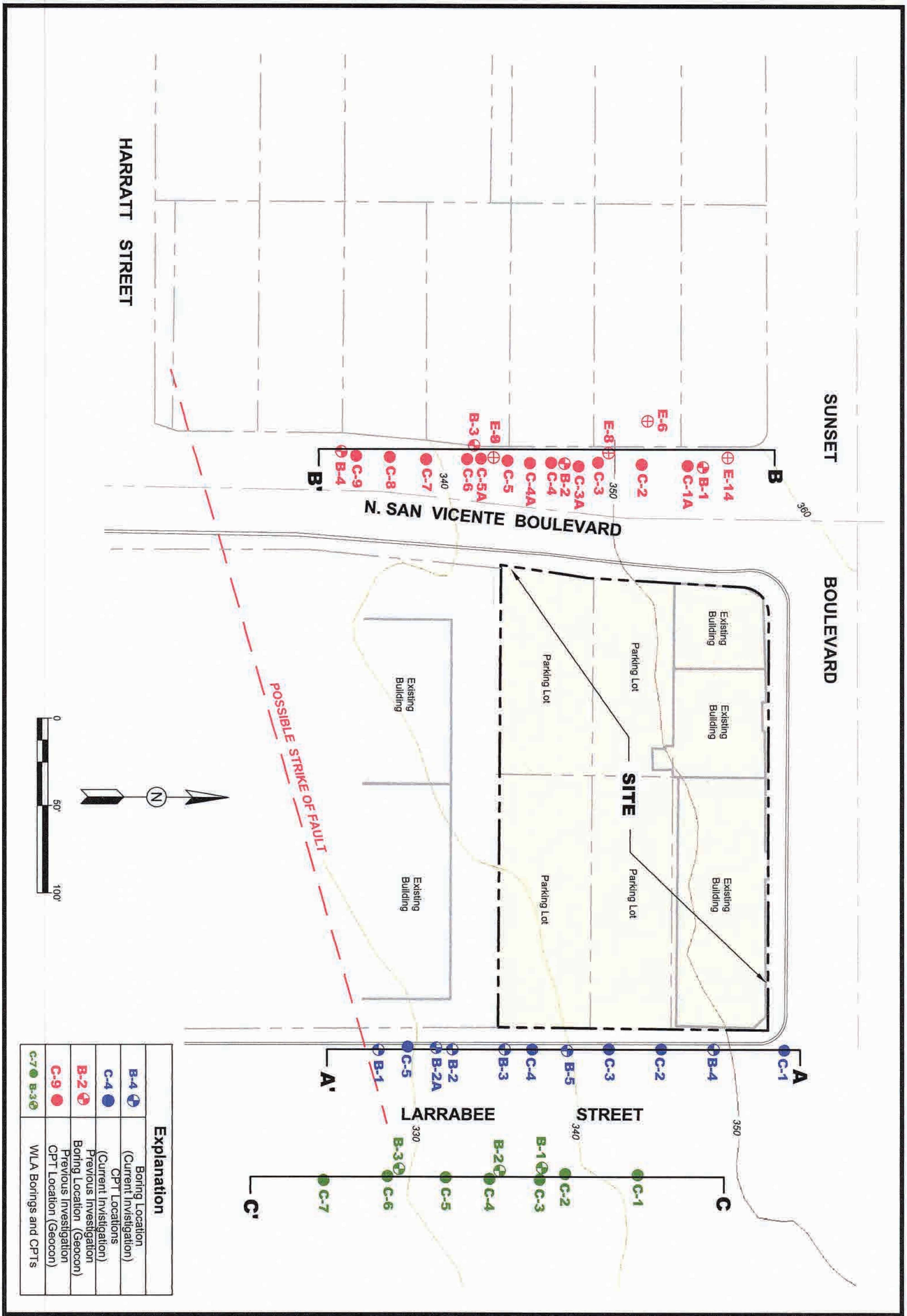
Title

Preliminary AP Zone Map

Project

8852 to 8874 Sunset Boulevard
West Hollywood, California

John Helms, CEG FIGURE 5



JOHN HELMS, CEG

Description:
Boring Location Map

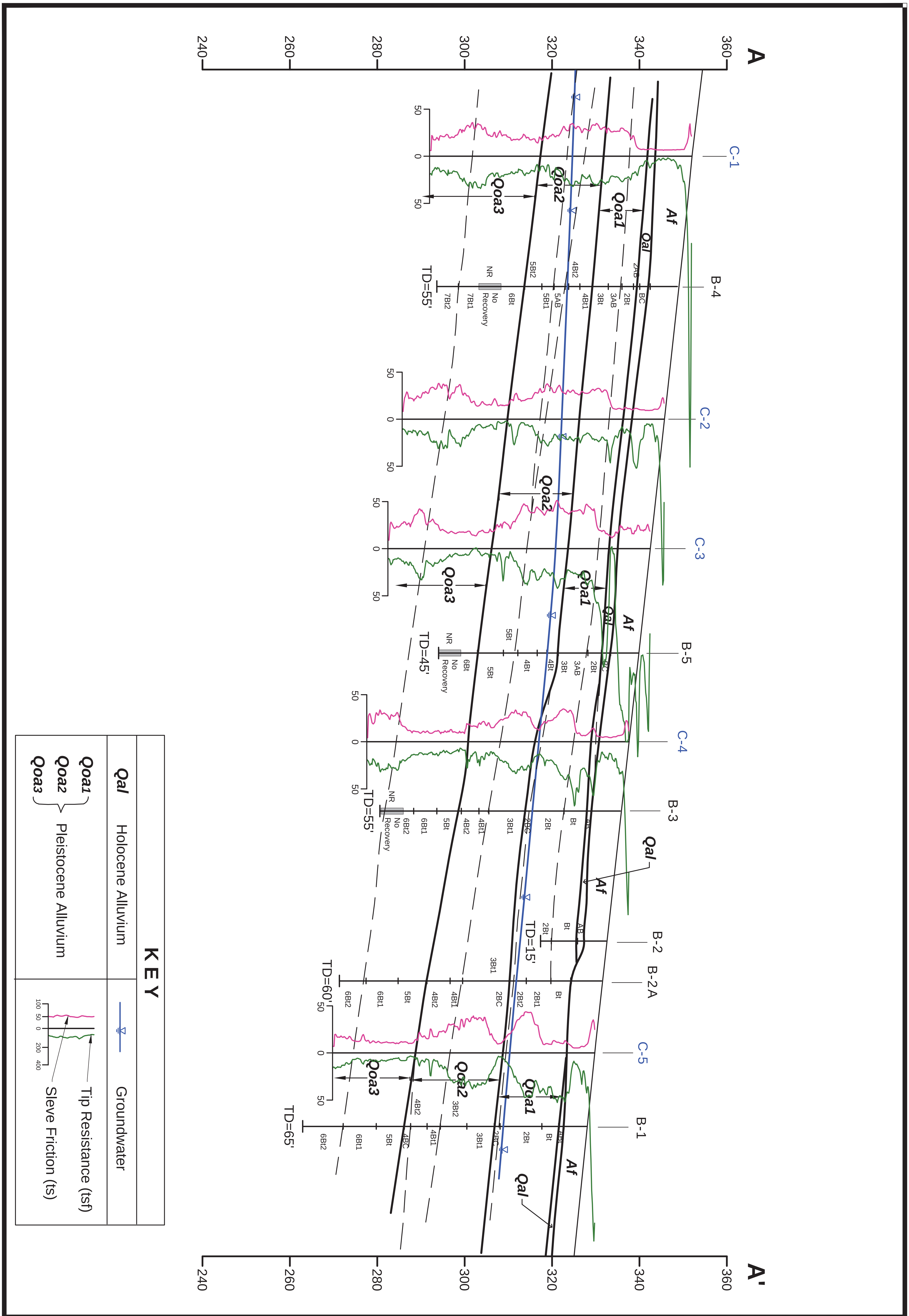
Base Map From
GIS-NET3 Public

Project Address:
8852 to 8874 Sunset Boulevard
West Hollywood, California

Date: Aug. 2017

Scale: 1" = 50'

Figure 6



JOHN HELMS, CEG

Description:

Geologic Cross Section A-A'

Project Address:

8850 to 8878 Sunset Boulevard
West Hollywood, California

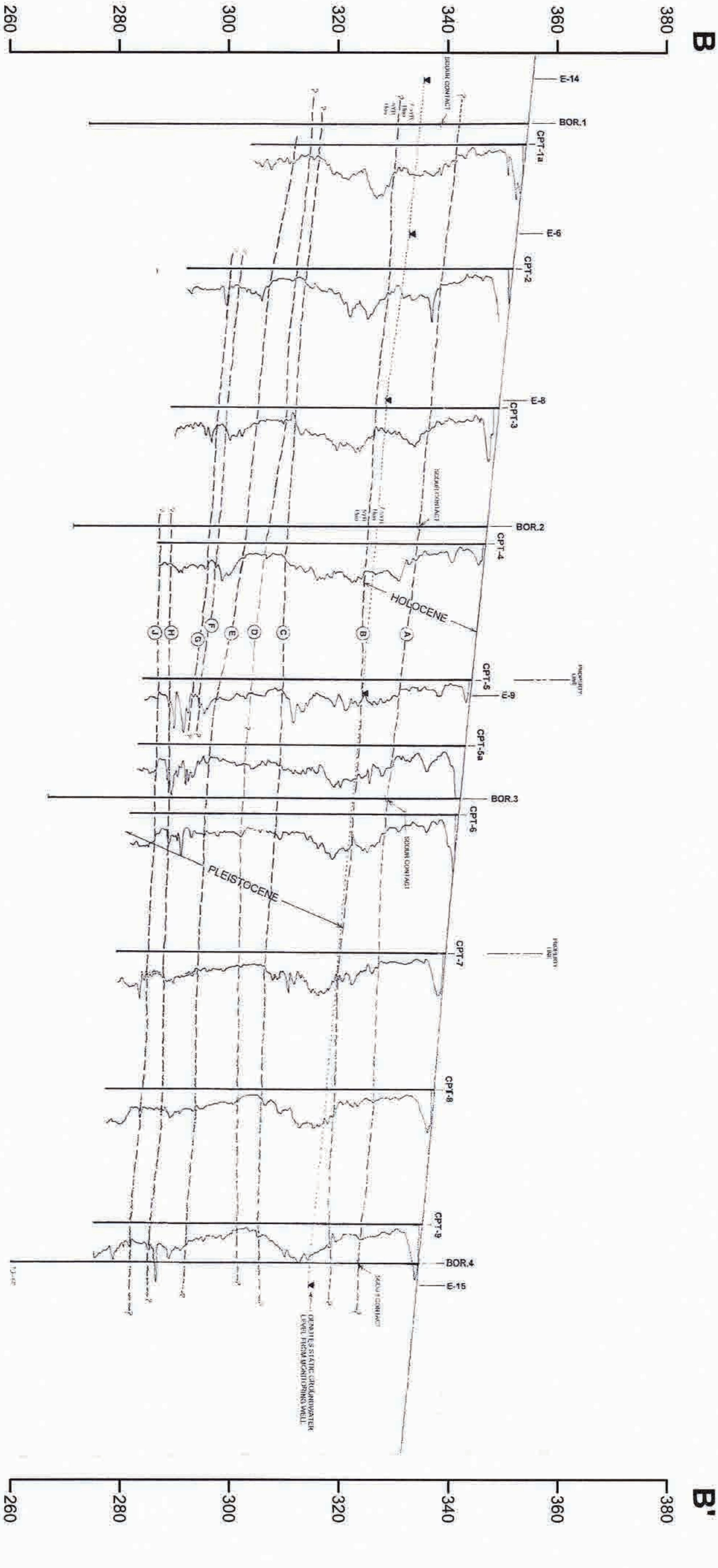
Date: Sept. 2018

Scale: 1" = 20'

Figure 7

MACTEC

MACTEC ENGINEERING AND CONSULTING, INC.
200 CITADEL DRIVE LOS ANGELES CALIFORNIA
(323) 889-5300 FAX (323) 721-6700



EXPLANATION:

BOR.4 LOCATION OF CURRENT CONTINUOUS-CORE BORING

CPT-9 LOCATION OF CPT BY FUGRO (2004)

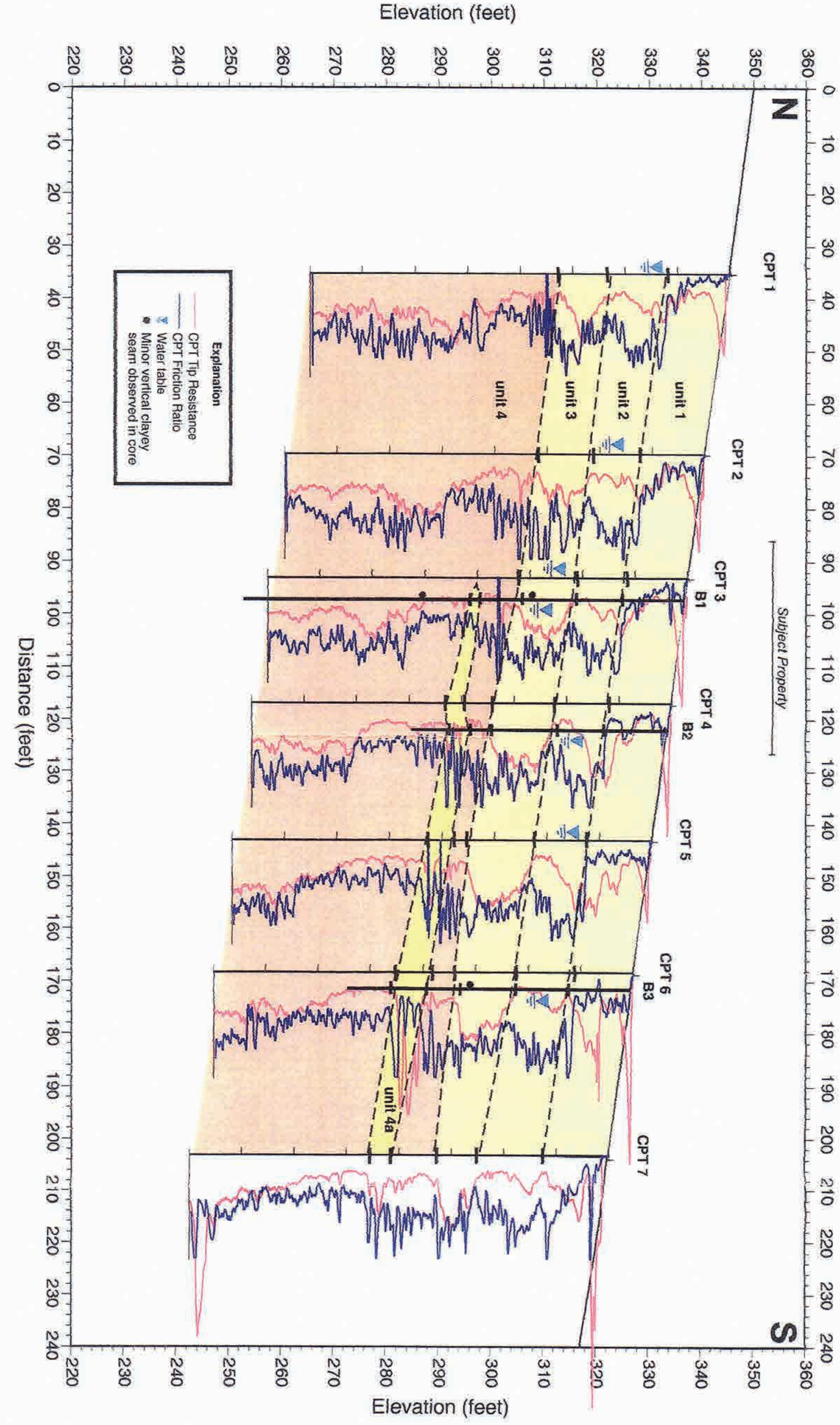
E-15 LOCATION OF MONITORING WELL

(J) LOCATION OF SUBSURFACE HORIZON AS REFERRED TO IN REPORT TEXT

LOCATION OF STATIC GROUND WATER SURFACE AS DETERMINED FROM MONITORING WELLS

JOHN HELMS, CEG	Description: **Geologic Cross Section B-B'**	Project Address: 8852 to 8874 Sunset Boulevard West Hollywood, California	Date: Aug. 2017 Scale: 1" = 20' Figure 8

WLA William Lettis & Associates, Inc.



JOHN HELMS, CEG

Description:

Geologic
Cross Section C-C'

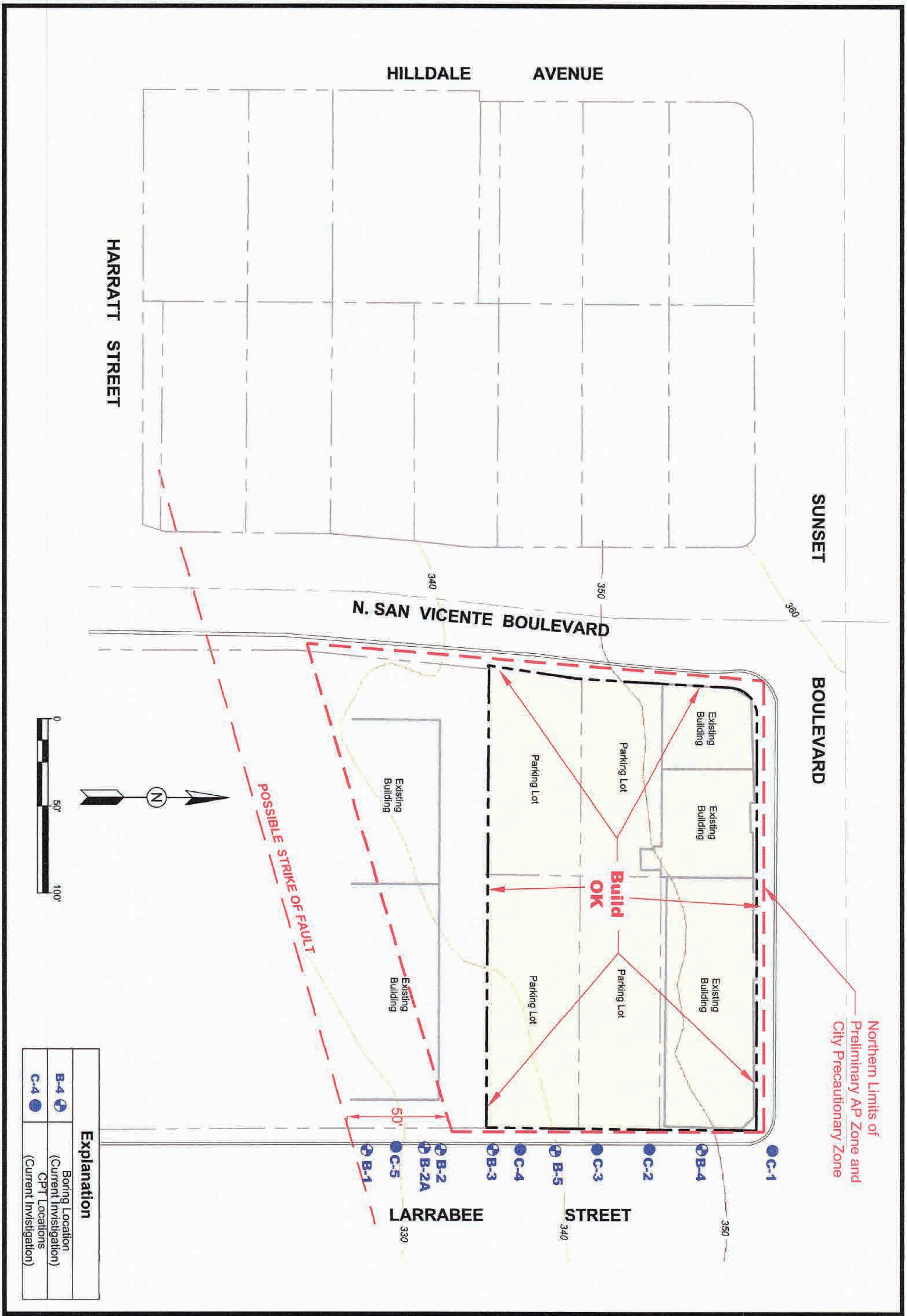
Project Address:

8852 to 8874 Sunset Boulevard
West Hollywood, California

Date: Aug. 2017

Scale: 1" = 20'

Figure7



JOHN HELMS, CEG

Description:
Setback Zone Map

Base Map From
GIS-NET3 Public

Project Address:
8852 to 8874 Sunset Boulevard
West Hollywood, California

Date: Aug. 2017

Scale: 1" = 50'

Figure 10

Table 1. Summary of Hollow Stem Auger Borings

					Groundwater	
Boring No.	Date Drilled	Total Depth (ft)	Boring G.S. Elevation (ft)	Boring Depth Elevation (ft)	Depth (ft)	Elevation (ft)
B-1	6/26/2017	65	328	263	19	309
B-2	6/28/2017	15	327.5	312.5	n e	--
B-2A	6/30/2017	60	329.5	269.5	19.5	310
B-3	7/1/2017	55	335.5	280.5	21	314.5
B-4	7/2/2017	55	349	294	26	323
B-5	7/2/2017	45	339.5	294.5	21	318.5

Table 2. Summary of CPT Soundings

					Groundwater	
CPT No.	Date	Total Depth (ft)	Boring G.S. Elevation (ft)	Boring Depth Elevation (ft)	Depth (ft)	Elevation (ft)
CPT-1	7/11/2017	60	352.5	292.5	26	326.5
CPT-2	7/11/2017	60	349.5	289.5	23	326.5
CPT-3	7/11/2017	60	342.5	282.5	20	322.5
CPT-4	7/11/2017	60	337.5	277.5	20	317.5
CPT-5	7/11/2017	60	329.5	269.5	19	310.5

Table 3

Stratigraphic Summary And Correlations

Major Unit	Approximate Age	Boring B-5		Boring B-4		Boring B-3		Boring B-2		Boring B-2A		Boring B-1		Unit Definition And Material Description
		Soil Units	**Unit Thickness**	**Soil Units**	**Unit Thickness**	**Soil Units**	**Unit Thickness**	**Soil Units**	**Unit Thickness**	**Soil Units**	**Unit Thickness**	**Soil Units**	**Unit Thickness**	
Qal	Middle Holocene	BC	2.3'	BC	2.5'	AB	1.5'	AB	1.0'	N.E.	- -	AB	1.3'	The youngest major sedimentological unit encountered on site. Qal material is found in two different deposits; sheet wash deposits (BC) and an organic-rich bioturbated top soil (AB). The Qal sheet wash (BC) deposits represent a Middle Holocene aged alluvial deposit observed across the northern portion of the site in borings B-4 and B-5. This material consists of unconsolidated and slightly oxidized Silty SAND and localized fine gravel that is massive to crudely stratified with weak soil development. The Qal top soil (AB) deposits represent a thin section of organic-rich and reworked or disturbed older alluvium across the southern portion of the project site area. This material consists of unconsolidated Silty SAND and localized fine gravel that is massive and organic-rich with weak soil development.
Qoa1	Latest Pleistocene	2Btb, 3ABb, 3Btb	9.9'	2Btb, 3ABb, 3Btb	10.5'	Bt, 2Btb, 2BCb	13.5'	Bt, 2Btb	9.3'	Bt, 2Btb1, 2Btb2, 2BCb	2.9+'	Bt, 2Btb, 2BCb	11.0'	The youngest of the Pleistocene aged major sedimentological units encountered on site, Qoa1, represents the Latest Pleistocene aged alluvium observed on site. The Qoa1 material consists of two alluvial fan deposits. Unit Qoa1 is buried across the entire project site area beneath the Holocene aged Qal unit. The major distinctions between the Qoa1 and the Qal alluvial deposits are their relative positions in the stratigraphic column (Qoa1 is lower and buried by Qal), degrees of consolidation (unit Qoa1 is more consolidated), and stronger soil development (more secondary clay in unit Qao1). The upper alluvial fan in unit Qoa1 is truncated across the northern portion of the transect in borings B-4 and 5 where overlain by the coarse grained BC horizon. The upper alluvial fan in unit Qoa1 is well preserved across the southern portion of the transect in borings B-1, 2, and 3 where overlain by the organic rich AB horizon. This material consists of hard and moderately well oxidized Silty SAND with Clay and common fine gravel that is massive with weak to moderately strong soil development.. The lower alluvial fan deposit in Qoa1 is a well preserved deposit in the northern portion of the site in borings B-4 and B-5 where this deposit is capped by the organic rich 3AB horizon.. This material consists of vey hard and moderately well oxidized Clayey SAND and few to common fine gravel that is massive with moderately strong soil development.
Qoa2	Late Pleistocene	4Btb, 5Btb1, 5Btb2	18.0'	4Btb1, 4Btb2, 5Btb1, 5Btb2	16.6'	3Btb, 4Btb1, 4Btb2	16.6'		19.5'	3Btb, 4Btb1, 4Btb2		3Btb, 4Btb, 4BCb,	21.0'	The middle member of the Pleistocene aged major sedimentological units, Qoa2 is found in a distinct alluvial deposit that displays moderately strong to strong soil (or pedogenic) development. The Qoa2 material consists of two stacked and truncated alluvial fan deposits. Unit Qoa2 is buried across the entire area investigated and the major distinctions between the Qoa1 and the Qoa2 alluvial deposits are their relative positions in the stratigraphic column (Qoa2 is lower and buried by Qoa1), degrees of consolidation (unit Qoa2 is more consolidated over all), and stronger soil development (stronger oxidation in Qao2). The upper alluvial fan in unit Qoa2 is well horizonated across the northern portion of the transect in borings B-4 and 5 where occupied by the 4Bt1b and 4Bt2b horizons. The upper alluvial fan in unit Qoa1 is truncated and massive across the remainder of the area investigated. This material consists of hard and moderately well oxidized Silty SAND with Clay to Clayey SAND with many fine gravel that is moderately well oxidized with moderately strong soil development. The lower alluvial fan deposit in Qoa2 is a well horizonated deposit across the entire area investigated. In the southern portion of the site the lower alluvial fan deposit in Qoa2 contains a basal sour or coarse-grained BC horizon in boring B-1.This material consists of vey hard and moderately well oxidized Clayey SAND with few fine gravel that is massive with moderately strong to strong soil development.
Qoa3	Pleistocene	6Btb	8.3'+	6Btb, 7Btb1, 7Btb2	19.0'+	5Btb, 6Btb1, 6Btb2	24.4'+		20.0'+	5Btb, 6Btb1, 6Btb2		5Btb, 6Btb1, 6Btb2	23.0'+	The lowest member Pleistocene aged major sedimentological unit Qoa3 represents the oldest Pleistocene aged alluvium observed on site. The Qoa3 material consists of two stacked alluvial fan deposits. Unit Qoa3 is buried across the entire project site area and the major distinctions between the Qoa2 and the Qoa3 alluvial deposits are their relative positions in the stratigraphic column (Qoa3 is lower and buried by Qoa2), degrees of consolidation (unit Qoa3 is more consolidated over all), and stronger soil development (stronger oxidation and clay film development in Qoa3). The upper alluvial fan in unit Qoa3 is truncated across the entire transect of borings. The upper alluvial fan in unit Qoa3 is thick and consists of hard and crudely stratified Silty SAND with Clay and common to many fine gravel with moderately strong soil development.. The lower alluvial fan deposit in Qoa3 is a well preserved and well horizonated deposit across the entire transect of borings. This material consists of vey hard and moderately well oxidized Sandy CLAY with few fine gravel that is massive with strong soil development.

Appendix A

Hollow Stem Auger Boring Logs

LOG OF BORING "B-1"

Page 1 of 4

Project 8852 - 8874 Sunset Blvd.	Job No.	Elevation @ Top of Hole 328'	Total Depth 65.0'	Boring Location See Boring Location Map
Type & Diameter of Boring 8" Hollow Stem Auger		Groundwater Depth 19.0'	Date Started/Completed 6/26/17	
Sampling Method CME Continuous Core sampler 2.5" ID, 3.5" OD CME 75		Sample Driving Hammer and Drop N / A		
Drilling Contractor and Rig One - Way Drilling, Rigo		Logged By John Helms		

Notes:
Started drilling at 9:30AM and finished drilling at 12:30PM.

Uncorrected Blows per 6-inch drive	Depth (feet)	Sample Run No.	Recovery (feet)	Sampler Type	Graphic Log	Description	Remarks
	0					Asphalt Roadway and Base Material	Start setup at 8:00 AM Hand augered to 5' depth Start drilling at 9:15 AM
	2					Hand augured to 5' depth	
	3					**Af, Artificial Fill** – Not described	AF
	5	1	3.6/5.0		5.0'	**Qal, Alluvium** Silty SAND, Dark yellowish brown, 10YR 3/4 (m), dry, massive, slight organics, soft, friable, slightly to moderately sticky, slightly plastic, coarse-grained poorly-sorted sand, slightly oxidized, gradational lower boundary to;	AB Holocene
	6.3				6.3'	**Qoa1, Old Alluvium** Silty SAND, Dark yellowish brown, 10YR 4/4 (m), slightly moist, massive, slightly hard, friable, slightly to moderately sticky, slightly plastic, coarse-grained poorly-sorted sand with few pea gravel, slightly oxidized, with few to common fine clay films, gradational lower boundary to;	Pleistocene Bt
	10	2	5.0/5.0		10.0'	Silty SAND with Clay, Brown, 7.5YR 4/4 (m), moist to wet, massive, hard, friable, slightly to moderately sticky, slightly plastic, coarse-grained poorly-sorted sand with common pea gravel, moderately well oxidized, few moderately thick clay films, gradational lower boundary to;	2Btb
	15						

▨ 3.0-inch O.D. split spoon sampler	☒ 2.5-inch O.D. split spoon sampler	▮ Standard Penetration Test (SPT) sampler

LOG OF BORING "B-1"

Page 2 of 4

Project	Job No.	Logged by:
8852 - 8874 Sunset Blvd.		John Helms

Uncorrected Blows per 6-inch drive	Depth (feet)	Sample Run No.	Recovery (feet)	Sampler Type	Graphic Log	Description	Remarks
	15					Same as above	2Btb
	16					Gradational increase in Clay with depth	
	15–20	3	5.0/5.0				
	20	4	4.0/5.0		20.0'	Silty SAND with Clay, Brown, 7.5YR 5/4 (m), wet, massive to crudely stratified, soft, friable, slightly sticky, slightly plastic, coarse-grained poorly-sorted sand with common to many pea gravel, slightly well oxidized, clear lower boundary to;	2BCb
	21				21.0'	**Qoa2, Old Alluvium** Clayey SAND, Brown, 7.5YR 3/4 (m), moist to wet, massive, very hard, friable, moderately to very sticky, very plastic, coarse-grained poorly-sorted sand with few pea gravel, moderately well oxidized, common moderately thick clay films, gradational lower boundary to; @23' –material becomes partially gleyed – 2.5Y 5/3 Light olive brown	3Btb1
	25	5	5.0/5.0				
	27.5				27.5'	Clayey SAND, Strong brown, 7.5YR 4/6(m), wet, massive, very hard, friable, very sticky, very plastic, coarse-grained poorly-sorted sand with common pea gravel, well oxidized, common to many moderately thick clay films, clear lower boundary to;	3Btb2
	30	6	5.0/5.0			Gradational loss of clay and increase in sand with depth	
	33.2				33.2'	Sandy CLAY, Brown, 7.5YR 4/4 (m), wet, massive, very hard, friable, very sticky, very plastic, coarse-grained poorly-sorted sand with few pea gravel, moderately well oxidized, few to common moderately thick clay films, locally gleyed light olive brown (2.5Y 5/3) in patches and random streaks, gradational lower boundary to;	4Btb1
	35						

LOG OF BORING"B-1"

Page 3 of 4

Project: 8852 - 8874 Sunset Blvd.

Job No.:

Logged by: John Helms

Uncorrected Blows per 6-inch drive	Depth (feet)	Sample Run No.	Recovery (feet)	Sampler Type	Graphic Log	Description	Remarks
	35					Same as above	4Btb1
	36	7	4.0/5.0		36.2'	Silty SAND with Clay, Strong brown, 7.5YR 4/6 (m) and Light grayish brown (2.5Y 6/2), wet, massive, strong redox, hard, friable, moderately sticky, very plastic, coarse-grained poorly-sorted sand with common pea gravel, few moderately thick and common thin clay films, well oxidized and well gleyed, clear lower boundary to;	4Btb2
	40	8	3.6/5.0		40.0'	Silty SAND with Clay, Strong brown, 7.5YR 4/6 (m) and Light grayish brown (2.5Y 6/2), wet, crudely stratified, moderate redox, slightly hard, friable, moderately sticky, slightly plastic, coarse-grained poorly-sorted sand with common to many pea gravel, few moderately thick and common thin clay films, well oxidized and moderately gleyed, clear lower boundary to;	4BCb
	42				42.0'	**Qoa3, Old Alluvium** Clayey SAND, Brown, 7.5YR 4/4 (m), wet, massive to crudely stratified, very hard, friable, very sticky, very plastic, coarse-grained poorly-sorted sand with few pea gravel, moderately well oxidized, common to many moderately thick clay films, locally gleyed light grayish brown (2.5Y 6/2) in patches, gradational lower boundary to;	5Btb
	45	9	5.0/5.0			Gradational loss of gleying with depth	
	48				48.0'	Sandy CLAY, Brown, 7.5YR 4/3 (m), wet, massive, very hard, friable, very sticky, very plastic, coarse-grained poorly-sorted sand with few pea gravel, moderately well oxidized, slight organics, common moderately thick clay films, locally gleyed light olive brown (2.5Y 5/3) in patches, gradational lower boundary to;	6Btb1
	50	10	4.8/5.0			Same as above	
	55						

LOG OF BORING"B-1"

Page 4 of 4

Project	Job No.	Logged by:
8852 - 8874 Sunset Blvd.		John Helms

Uncorrected Blows per 6-inch drive	Depth (feet)	Sample Run No.	Recovery (feet)	Sampler Type	Graphic Log	Description	Remarks
	55					Same as above	6Btb1
	55.8'					Clayey SAND, Strong Brown, 7.5YR 4/6 (m), wet, massive, very hard, friable, very sticky, very plastic, coarse-grained poorly-sorted sand with common pea gravel, well oxidized, few moderately thick and common thin clay films, undetermined lower boundary;	6Btb2
	57	7	5.0/5.0				
	60						
	61					Gradational loss of clay and increase in sand with depth	
	62	8	5.0/5.0				
	65					Stopped drilling at 12:00PM @65'. Started backfilling at 1:00PM. Backfilled with cuttings to 20', bentonite chips to 10', and grout to 0' (top dyed black). Water levels: During drilling at ~19'	
	70						
	75						

LOG OF BORING "B-2"

Page 1 of 3

Project 8852 - 8874 Sunset Blvd.	Job No.	Elevation @ Top of Hole 332'	Total Depth 15.0'	Boring Location See Boring Location Map
Type & Diameter of Boring 8" Hollow Stem Auger		Groundwater Depth N.E.	Date Started/Completed 6/27/17	
Sampling Method CME Continuous Core sampler 2.5" ID, 3.5" OD Limited Access Drill Rig		Sample Driving Hammer and Drop N / A		
Drilling Contractor and Rig One - Way Drilling, Rigo		Logged By John Helms		

Notes:
Started drilling at 9:30AM and finished drilling at 10:30AM.

Uncorrected Blows per 6-inch drive	Depth (feet)	Sample Run No.	Recovery (feet)	Sampler Type	Graphic Log	Description	Remarks
	0					Asphalt Roadway and Base Material	Start setup at 8:30 AM Hand augered to 5' depth Start drilling at 9:30 AM
	1					Hand augured to 5' depth	
	2					**Af, Artificial Fill** – Not described	
	3						AF
	5	1	3.6 / 5.0	▨	5.0'	**Qal, Alluvium** Silty SAND, Brown, 10YR 4/3 (m), dry, massive, slight organics, soft, friable, slightly to moderately sticky, slightly plastic, coarse-grained poorly-sorted sand, gradational lower boundary to;	AB
	6				6.0'		
	7					**Qoa1, Old Alluvium** Silty SAND, Dark yellowish brown, 10YR 4/4 (m), slightly moist, massive, slightly hard, friable, slightly sticky, slightly plastic, coarse-grained poorly-sorted sand with few to common pea gravel, slightly oxidized, with few fine clay films, gradational lower boundary to;	Bt
	10	2	5.0 / 5.0	▨			
	12				12.1'	Silty SAND with Clay, Brown, 7.5YR 4/4 (m), moist, massive, hard, friable, moderately sticky, slightly plastic, coarse-grained poorly-sorted sand with few pea gravel, moderately well oxidized, common thin moderately thick clay films, undetermined lower boundary.	2Btb
	14					Drill rig broke down. Bent head gear and Kelly bar.	
	15						

▒ 3.0-inch O.D. split spoon sampler ☒ 2.5-inch O.D. split spoon sampler ▌ Standard Penetration Test (SPT) sampler

LOG OF BORING "B-2"

Page 2 of 3

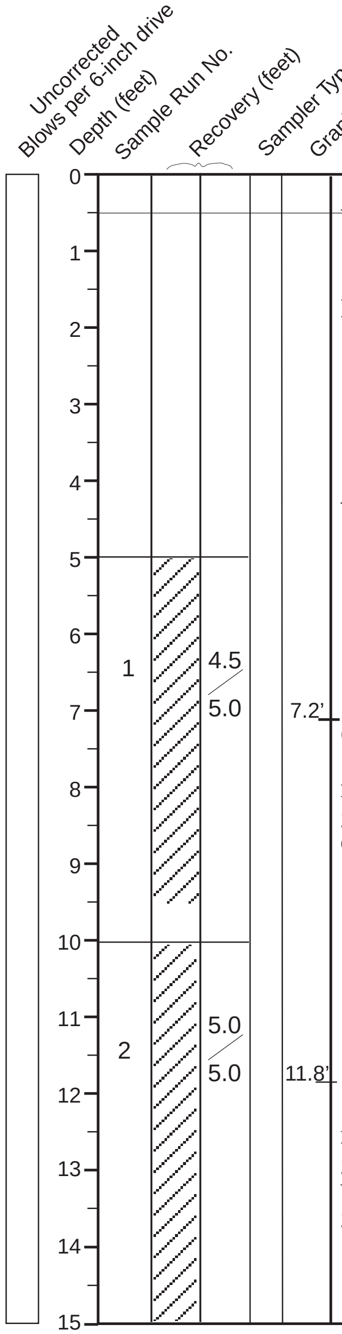
Project	Job No.	Logged by:
8852 - 8874 Sunset Blvd.		John Helms

Uncorrected Blows per 6-inch drive	Depth (feet)	Sample Run No.	Recovery (feet)	Sampler Type	Graphic Log	Description	Remarks
	15					Drill rig broke down. Bent head gear and Kelly bar. Stopped drilling at 10:00AM @15'. Started backfilling at 11:30AM. Backfilled with cuttings to 5', and grout to 0' (top dyed black). Water levels: Not encountered during drilling	
	16						
	17						
	18						
	19						
	20						
	21						
	22						
	23						
	24						
	25						
	26						
	27						
	28						
	29						
	30						
	31						
	32						
	33						
	34						
	35						

LOG OF BORING "B-2A"

Project: 8850 to 8878 Sunset Blvd.
Job No.:
Elevation @ Top of Hole: 329.5'
Total Depth: 60.0'
Boring Location: See Boring Location Map
Type & Diameter of Boring: 8" Hollow Stem Auger
Groundwater Depth: 19.5’
Date Started/Completed: 6/28/17
Sampling Method: CME Continuous Core sampler 2.5" ID, 3.5" OD Limited Access Drill Rig
Sample Driving Hammer and Drop: N / A
Drilling Contractor and Rig: One - Way Drilling, Rigo
Logged By: John Helms

Notes: Started drilling at 8:30AM and finished drilling at 1:30PM.



Description	Remarks
Asphalt Roadway and Base Material	Start setup at 8:00 AM
Hand augured to 5’ depth	Hand augered to 5’ depth Start drilling at 8:30 AM
Af, Artificial Fill – Not described	AF Holocene
Qoa1, Old Alluvium Silty SAND, Dark yellowish brown, 10YR 4/4 (m), slightly moist, massive, slightly hard, friable, slightly sticky, slightly plastic, coarse-grained poorly-sorted sand with common pea gravel, slightly well oxidized, with few fine clay films, gradational lower boundary to;	Pleistocene Bt
Silty SAND with Clay, Brown, 7.5YR 4/4 (m), slightly moist, massive, hard, friable, moderately sticky, slightly plastic, coarse-grained poorly-sorted sand with common pea gravel, moderately well oxidized, common thin and few moderately thick clay films, gradational lower boundary to;	2Btb1

▮ 3.0-inch O.D. split spoon sampler
☒ 2.5-inch O.D. split spoon sampler
◧ Standard Penetration Test (SPT) sampler

LOG OF BORING "B-2A"

Page 2 of 4

Project: 8850 to 8878 Sunset Blvd.

Job No.:

Logged by: John Helms

Uncorrected Blows per 6-inch drive	Depth (feet)	Sample Run No.	Recovery (feet)	Sampler Type	Graphic Log	Description	Remarks
	15 16 17					Same as above	2Btb1
	18 19	3	5.0/5.0		17.2'	Silty SAND with Clay, Brown, 7.5YR 3/4 (m), wet, massive, soft, friable, moderately sticky, moderately plastic, coarse-grained poorly-sorted sand with few pea gravel, slightly well oxidized, common thin and moderately thick clay films, clear lower boundary to;	2Btb2
	20				20.0'	Silty SAND with Clay, Brown, 7.5YR 4/4 (m), wet, massive, soft, friable, slightly sticky, slightly plastic, coarse-grained poorly-sorted sand with common pea gravel, moderately well oxidized, clear lower boundary to;	2BCb
	21 22 23 24	4	4.5/5.0		20.5'	**Qoa2, Old Alluvium** Clayey SAND, Brown, 7.5YR 3/4 (m), moist to wet, massive, very hard, friable, very sticky, moderately to very plastic, coarse-grained poorly-sorted sand with few pea gravel, slightly well oxidized, common moderately thick clay films, material locally gleyed – 2.5Y 5/3 Light olive brown, gradational lower boundary to;	3Btb
	25 26 27					Same as above	
	28 29 30 31	5	5.0/5.0			@28' –material becomes gleyed – 2.5Y 5/3 Light olive brown in random streaks	
	32 33	6	5.0/5.0		31.6'	Sandy CLAY, Brown, 7.5YR 4/4 (m), wet, massive, very hard, friable, very sticky, very plastic, coarse-grained poorly-sorted sand with few pea gravel, moderately well oxidized, common thick and moderately thick clay films, gradational lower boundary to;	4Btb1
	34 35				34.0'	See next page	4Btb2

LOG OF BORING"B-2A"

Page 3 of 4

Project: 8850 to 8878 Sunset Blvd.
Job No.:
Logged by: John Helms

Uncorrected Blows per 6-inch drive	Depth (feet)	Sample Run No.	Recovery (feet)	Sampler Type	Graphic Log	Description	Remarks
	35	7	4.5/5.0			Silty SAND with Clay, Strong brown, 7.5YR 4/6 (m), wet, crudely stratified, slightly hard, friable, moderately sticky, slightly plastic, coarse-grained poorly-sorted sand with few pea gravel, moderately well oxidized, common moderately thick clay films, clear lower boundary to;	4Btb2
	40	8	3.8/5.0		40.0	**Qoa3, Old Alluvium** Clayey SAND, Brown, 7.5YR 4/4 (m) and gleyed light grayish brown (2.5Y 6/2), wet, massive, very hard, friable, very sticky, very plastic, coarse-grained poorly-sorted sand with few pea gravel, moderately well oxidized, common to many moderately thick clay films, common MnO webbing, locally gleyed in patches and streaks, gradational lower boundary to;	5Btb
	45	9	4.8/5.0			Gradational loss of MnO and clay with depth	
					46.6	Sandy CLAY, Brown, 7.5YR 4/3 (m) and gleyed light grayish brown (2.5Y 6/2), wet, massive, very hard, friable, very sticky, very plastic, coarse-grained poorly-sorted sand with few pea gravel, moderately well oxidized, slight organics, common moderately thick clay films, locally gleyed in patches, gradational lower boundary to;	6Btb1
	50	10	5.0/5.0				
					53.4	Clayey SAND, Strong Brown, 7.5YR 4/4-6 (m), wet, massive, very hard, friable, very sticky, very plastic, coarse-grained poorly-sorted sand with few pea gravel, well oxidized, few moderately thick and common thin clay films, locally gleyed light grayish brown (2.5Y 6/2) in patches, undetermined lower boundary;	6Btb2
	55						

LOG OF BORING"B-2A"

Page 4 of 4

Project	Job No.	Logged by:
8850 to 8878 Sunset Blvd.		John Helms

Uncorrected Blows per 6-inch drive	Depth (feet)	Sample Run No.	Recovery (feet)	Sampler Type	Graphic Log	Description	Remarks
	55	▨				Same as above	6Btb2
	56	▨					
7	57	▨	4.0/5.0				
	58	▨					
	59	▨					
	60					Stopped drilling at 1:30PM @60'. Started backfilling at 2:00PM. Backfilled with cuttings to 20', bentonite chips to 10', and grout to 0' (top dyed black). Water levels: During drilling at ~19.5'	
	61						
	62						
	63						
	64						
	65						
	66						
	67						
	68						
	69						
	70						
	71						
	72						
	73						
	74						
	75						

LOG OF BORING "B-3"

Page 1 of 3

Project 8852 - 8874 Sunset Blvd.	Job No.	Elevation @ Top of Hole 335.5'	Total Depth 55.0'
Type & Diameter of Boring 8" Hollow Stem Auger		Groundwater Depth 21.0'	Date Started/Completed 6/30/17
Sampling Method CME Continuous Core sampler 2.5" ID, 3.5" OD Limited Access Drill Rig		Sample Driving Hammer and Drop N / A	
Drilling Contractor and Rig One - Way Drilling, Rigo		Logged By John Helms	

Boring Location: See Boring Location Map

Notes: Started drilling at 8:30AM and finished drilling at 12:30PM.

Uncorrected Blows per 6-inch drive	Depth (feet)	Sample Run No.	Recovery (feet)	Sampler Type	Graphic Log	Description	Remarks
	0					Asphalt Roadway and Base Material	Start setup at 8:00 AM Hand augered to 5' depth Start drilling at 8:30 AM
	1–5					Hand augured to 5' depth **Af, Artificial Fill** – Not described	AF
	5–10	1	▨ 4.0/5.0				
	7				7.0'	**Qal, Alluvium** Silty SAND, Brown, 10YR 4/3 (m), slightly moist, massive, slight organics, soft, friable, slightly sticky, slightly plastic, coarse-grained poorly-sorted sand, few fine clay films, gradational lower boundary to;	Holocene - AB
	7.5				7.5'		Pleistocene Bt
	9					**Qoa1, Old Alluvium** Silty SAND, Brown, 10YR 4/4 (m), slightly moist, massive, slightly hard, friable, slightly sticky, non-plastic, coarse-grained poorly-sorted sand with few pea gravel, slightly well oxidized, with common fine clay films, gradational lower boundary to;	
	10–15	2	▨ 5.0/5.0				
	13				13.0'	Silty SAND with Clay, Brown, 7.5YR 4/4 (m), slightly moist to moist, massive, hard, friable, slightly to moderately sticky, slightly plastic, coarse-grained poorly-sorted sand with common pea gravel, slightly well oxidized, common thin and few moderately thick clay films, gradational lower boundary to;	2Btb
	15						

▮ 3.0-inch O.D. split spoon sampler ☒ 2.5-inch O.D. split spoon sampler ▌ Standard Penetration Test (SPT) sampler

LOG OF BORING "B-3"

Project	Job No.	Logged by:
8852 - 8874 Sunset Blvd.		John Helms

Uncorrected Blows per 6-inch drive	Depth (feet)	Sample Run No.	Recovery (feet)	Sampler Type	Graphic Log	Description	Remarks
	15 16 17 18 19	3	5.0/5.0	▨		Same as above Gradational loss of clay and increase in sand with depth	2Btb
	20			▨	20.0'	Silty SAND, Brown, 7.5YR 4/4 (m), wet, massive, soft, friable, slightly sticky, non-plastic, coarse-grained poorly-sorted sand with common pea gravel, slightly well oxidized, clear lower boundary to;	2BCb
	21 22 23 24	4	5.0/5.0	▨	21.0'	**Qoa2, Old Alluvium** Clayey SAND, Brown, 7.5YR 3/4 (m), wet, massive, slight organics, very hard, friable, moderately sticky, moderately to very plastic, coarse-grained poorly-sorted sand with few pea gravel, moderately well oxidized, common moderately thick and many thin clay films, undetermined lower boundary to;	3Btb
	25 26 27 28 29	5	5.0/5.0	▨		Gradational increase in oxidation with depth (color changes to 7.5YR 4/4, Brown)	
	30 31 32	6	4.0/5.0	▨	30.0'	Clayey SAND, Dark Brown to Brown, 7.5YR 4/3 to 3/4 (m), wet, massive, very hard, friable, very sticky, very plastic, coarse-grained poorly-sorted sand, slightly well oxidized, slight organics, common to many moderately thick and few thick clay films, gradational lower boundary to;	4Btb1
	33 34 35				32.5'	Sandy CLAY, Brown, 7.5YR 4/6 (m), wet, massive, hard, friable, moderately to very sticky, very plastic, coarse-grained poorly-sorted sand with common pea gravel, well oxidized, common moderately thick clay films, gradational lower boundary to;	4Btb2

LOG OF BORING"B-3"

Page 3 of 3

Project: 8852 - 8874 Sunset Blvd.
Job No.:
Logged by: John Helms

Uncorrected Blows per 6-inch drive	Depth (feet)	Sample Run No.	Recovery (feet)	Sampler Type	Graphic Log	Description	Remarks
	35					@35' material is locally gleyed light grayish brown (2.5Y 6/2) in patches same as above	4Btb2
	36.6'	7	3.2/5.0			**Qoa3, Old Alluvium** Clayey SAND to Silty SAND with Clay, Strong brown, 7.5YR 4/6 (m), wet, crudely stratified, hard, friable, moderately sticky, moderately to very plastic, coarse-grained poorly-sorted sand with common and localized many pea gravel, well oxidized, common thin and few moderately thick clay films, locally gleyed light grayish brown (2.5Y 6/2) in streaks and patches; clear lower boundary to;	5Btb
	40	8	4.7/5.0				
	42.0'					Clayey SAND, Reddish brown, 5YR 4/4 (m), wet, massive, hard to very hard, friable, very sticky, very plastic, coarse-grained poorly-sorted sand with few pea gravel, well oxidized, common to many moderately thick and few thick clay films, gradational lower boundary to;	6Btb1
	45	9	4.3/5.0				
	47.0'					Sandy CLAY, Brown, 7.5YR 4/3 (m), wet, massive, very hard, friable, very sticky, very plastic, coarse-grained poorly-sorted sand, slightly well oxidized, many moderately thick and common thick clay films, undetermined lower boundary;	6Btb2
	50	10	0/5.0			Run 10 No Recovery - sample slid out of core barrel Stopped drilling at 12:30PM @55'. Started backfilling at 1:30PM. Backfilled with cuttings to 20', bentonite chips to 10', and grout to 0' (top dyed black). Water levels: During drilling at ~21'	
	55						

LOG OF BORING "B-4"

Page 1 of 3

Project 8850 to 8878 Sunset Blvd.	Job No.	Elevation @ Top of Hole 349'	Total Depth 55.0'
Type & Diameter of Boring 8" Hollow Stem Auger		Groundwater Depth 26.0'	Date Started/Completed 6/29/17
Sampling Method CME Continuous Core sampler 2.5" ID, 3.5" OD Limited Access Drill Rig		Sample Driving Hammer and Drop N / A	
Drilling Contractor and Rig One - Way Drilling, Rigo		Logged By John Helms	

Boring Location
See Boring Location Map

Notes:
Started drilling at 8:00AM and finished drilling at 12:00PM.

Uncorrected Blows per 6-inch drive	Depth (feet)	Sample Run No.	Recovery (feet)	Sampler Type	Graphic Log	Description	Remarks
	0					Asphalt Roadway and Base Material	Start setup at 7:30 AM Hand augered to 5' depth Start drilling at 8:00 AM
	1–2					Hand augured to 5' depth	
	3					**Af, Artificial Fill** – Not described	
	5	1	4.6/5.0				AF
	6.4				6.4'	**Qal, Alluvium** Silty SAND, Dark yellowish brown, 10YR 4/6 (m), dry to slightly moist, massive, soft, friable, non- to slightly sticky, non-plastic, coarse-grained poorly-sorted sand with few pea gravel, slightly well oxidized, gradational lower boundary to;	BC Holocene
	8.9				8.9'	**Qoa1, Old Alluvium** Silty SAND, Dark brown, 10YR 3/3 (m), slightly moist, massive, organic rich, slightly hard, friable, slightly sticky, non- to slightly plastic, coarse-grained poorly-sorted sand with few pea gravel, gradational lower boundary to;	Pleistocene 2ABb
	10.0	2	5.0/5.0		10.0'	Silty SAND, Brown, 7.5YR 4/4 (m), slightly moist, massive, slightly hard, friable, moderately sticky, slightly plastic, coarse-grained poorly-sorted sand with common pea gravel, moderately well oxidized, with few to common fine clay films, clear lower boundary to;	2Btb
	12.3				12.3'	Silty SAND with Clay, Brown, 7.5YR 3/4 (m), slightly moist to moist, massive, slight organics, hard, friable, moderately sticky, slightly to moderately plastic, coarse-grained poorly-sorted sand with few pea gravel, moderately well oxidized, common thin clay films, gradational lower boundary to;	3ABb
	15						

▨ 3.0-inch O.D. split spoon sampler ☒ 2.5-inch O.D. split spoon sampler ▮ Standard Penetration Test (SPT) sampler

LOG OF BORING "B-4"

Project	Job No.	Logged by:
8850 to 8878 Sunset Blvd.		John Helms

Uncorrected Blows per 6-inch drive	Depth (feet)	Sample Run No.	Recovery (feet)	Sampler Type	Graphic Log	Description	Remarks
	15					Same as above	3ABb
	16 17 18 19	3	5.0/5.0		15.5'	Clayey SAND, Brown, 7.5YR 4/4 (m), moist, massive, hard, friable, very sticky, very plastic, coarse-grained poorly-sorted sand with common pea gravel, moderately well oxidized, common moderately thick clay films, clear lower boundary to;	3Btb
	20 21				19.4'	**Qoa2, Old Alluvium** Clayey SAND to Sandy CLAY, Dark Brown, 7.5YR 4/3 (m), moist, massive, slight organics, very hard, friable, very sticky, very plastic, coarse-grained poorly-sorted sand with few pea gravel, slightly well oxidized, common moderately thick clay films, gradational lower boundary to;	4Btb1
	22 23 24	4	5.0/5.0		22.0'	Clayey SAND, Brown, 7.5YR 4/4-6 (m), moist to wet, massive, hard, friable, moderately sticky, slightly to moderately plastic, coarse-grained poorly-sorted sand with many pea gravel, moderately well oxidized, common thin and few moderately thick clay films, clear lower boundary to;	4Btb2
	25 26 27	5	5.0/5.0		24.8'	Sandy CLAY, Dark Brown, 7.5YR 4/3 (m), wet, massive, slight organics, very hard, friable, very sticky, very plastic, coarse-grained poorly-sorted sand with few pea gravel, slightly well oxidized, common to many moderately thick and few thick clay films, gradational lower boundary to;	5ABb
	28 29				27.3'	@27.3' – zone of localized light grayish brown (2.5Y 6/2) gleying in patches and streaks Clayey SAND, Brown, 7.5YR 4/4-6 (m), wet, massive, very hard, friable, very sticky, very plastic, coarse-grained poorly-sorted sand with many pea gravel, well oxidized, common to many moderately thick clay films, gleyed light grayish brown (2.5Y 6/2) in patches and streaks, clear lower boundary to;	5Btb1
	30 31 32 33 34 35	6	4.0/5.0		30.0'	Silty SAND with Clay, Reddish brown, 5YR 4/4 (m), wet, massive, very hard, friable, very sticky, very plastic, coarse-grained poorly-sorted sand with many pea gravel, well oxidized, common to many thin and few moderately thick clay films, locally gleyed light grayish brown (2.5Y 6/2) in patches and streaks, clear lower boundary to;	5Btb2

LOG OF BORING"B-4"

Page 3 of 3

Project	Job No.	Logged by:
8850 to 8878 Sunset Blvd.		John Helms

Uncorrected Blows per 6-inch drive	Depth (feet)	Sample Run No.	Recovery (feet)	Sampler Type	Graphic Log	Description	Remarks
	35					Same as above	5Btb2
	36	7	3.5/5.0		36.0'	**Qoa3, Old Alluvium** Sandy CLAY, Reddish brown, 5YR 4/4-6 (m), wet, massive, very hard, friable, very sticky, very plastic, coarse-grained poorly-sorted sand with few pea gravel, well oxidized, many moderately thick and common thick clay films, locally gleyed light grayish brown (2.5Y 6/2) in patches, clear lower boundary to;	6Btb
	40	8	0/5.0			Run 8 No Recovery – Sample slid out of core barrel	
	45	9	5.0/5.0		45.0'	Clayey SAND to Sandy CLAY, Brown, 7.5YR 4/4 (m), wet, massive, very hard, friable, very sticky, very plastic, coarse-grained poorly-sorted sand with few pea gravel, moderately well oxidized, common thick and many moderately thick clay films, locally gleyed light grayish brown (2.5Y 6/2) in streaks; gradational lower boundary to;	7Btb1
	49.5				49.5'	Clayey SAND, Strong brown, 7.5YR 4/6 (m), wet, massive, hard, friable, very sticky, very plastic, coarse-grained poorly-sorted sand with common pea gravel, moderately well oxidized, common to many moderately thick and common thick clay films, undetermined lower boundary; Stopped drilling at 12:00PM @55'. Started backfilling at 1:00PM. Backfilled with cuttings to 20', bentonite chips to 10', and grout to 0' (top dyed black).	7Btb2
	50	10	2.8/5.0				
	53					Water levels: During drilling at ~26'	
	55						

LOG OF BORING "B-5"

Page 1 of 3

Project 8850 to 8878 Sunset Blvd.	Job No.	Elevation @ Top of Hole 339.5'	Total Depth 45.0'
Type & Diameter of Boring 8" Hollow Stem Auger		Groundwater Depth 21.0’	Date Started/Completed 6/29/17
Sampling Method CME Continuous Core sampler 2.5" ID, 3.5" OD Limited Access Drill Rig		Sample Driving Hammer and Drop N / A	
Drilling Contractor and Rig One - Way Drilling, Rigo		Logged By John Helms	

Boring Location

See Boring Location Map

Notes:
Started drilling at 1:30PM and finished drilling at 4:00PM.

Uncorrected Blows per 6-inch drive	Depth (feet)	Sample Run No.	Recovery (feet)	Sampler Type	Graphic Log	Description	Remarks
	0					Asphalt Roadway and Base Material	Start setup at 1:00 PM
	1					Hand augured to 5’ depth	Hand augered to 5’ depth
	2					**Af, Artificial Fill** – Not described	Start drilling at 1:30 PM
	3						
	4						AF
	5						
	6	1	4.0/5.0		5.8’	**Qal, Alluvium** Silty SAND, Brown, 10YR 5/4 (m), dry to slightly moist, crudely stratified, soft, friable, non- to slightly sticky, non-plastic, coarse-grained poorly-sorted sand with few pea gravel, slightly oxidized, clear lower boundary to;	BC Holocene
	8				8.1’	**Qoa1, Old Alluvium** Silty SAND with Clay, Dark yellowish brown, 10YR 4/4 (m), slightly moist, massive, hard, friable, slightly sticky, non-plastic, coarse-grained poorly-sorted sand with common pea gravel, slightly well oxidized, with few to common fine clay films, clear lower boundary to;	Pleistocene 2Btb
	10						
	11	2	1.6/5.0		11.2’	Clayey SAND, Brown, 7.5YR 4/4 (m), moist, massive, hard, friable, moderately sticky, moderately plastic, coarse-grained poorly-sorted sand with few pea gravel, moderately well oxidized, common moderately thick clay films, clear lower boundary to;	3ABb
	12						
	13						
	14						
	15						

■ 3.0-inch O.D. split spoon sampler ☒ 2.5-inch O.D. split spoon sampler ◧ Standard Penetration Test (SPT) sampler

LOG OF BORING "B-5"

Project	Job No.	Logged by:
8850 to 8878 Sunset Blvd.		John Helms

Uncorrected Blows per 6-inch drive	Depth (feet)	Sample Run No.	Recovery (feet)	Sampler Type	Graphic Log	Description	Remarks
	15	3	5.0 / 5.0			Same as above	3Btb
	18.0'					**Qoa2, Old Alluvium** Clayey SAND to Sandy CLAY, Brown to Strong brown, 7.5YR 4/4-6 (m), moist to wet, massive, very hard, friable, moderately to very sticky, very plastic, coarse-grained poorly-sorted sand with few to common pea gravel, moderately well oxidized, common moderately thick and few thick clay films, gradational lower boundary to;	4Btb1
	20	4	5.0 / 5.0				
	23.8'					Clayey SAND, Strong brown, 7.5YR 4/6 (m), wet, massive, hard, friable, moderately to very sticky, very plastic, coarse-grained poorly-sorted sand with common to many pea gravel, moderately well oxidized, common moderately thick clay films, clear lower boundary to;	4Btb2
	25	5	3.3 / 5.0				
	27.5'					Sandy CLAY, Brown, 7.5YR 4/4 (m), wet, massive, very hard, friable, very sticky, very plastic, coarse-grained poorly-sorted sand with few pea gravel, moderately well oxidized, common to many moderately thick and few thick clay films, gradational lower boundary to;	5Btb1
	30	6	5.0 / 5.0				
	30.5'					Clayey SAND, Strong brown, 7.5YR 4/6 (m), wet, massive, hard, friable, moderately sticky, moderately plastic, coarse-grained poorly-sorted sand with few pea gravel, well oxidized, common moderately thick clay films, clear lower boundary to;	5Btb2
	35						

LOG OF BORING"B-5"

Page 3 of 3

Project	Job No.	Logged by:
8850 to 8878 Sunset Blvd.		John Helms

Uncorrected Blows per 6-inch drive	Depth (feet)	Sample Run No.	Recovery (feet)	Sampler Type	Graphic Log	Description	Remarks
	35	7	▨ 2.6/5.0			Same as above	5Btb2
	36.7				36.7	**Qoa3, Old Alluvium** Sandy CLAY, Reddish brown, 5YR 4/3 (m), wet, massive, slight organics, very hard, friable, very sticky, very plastic, coarse-grained poorly-sorted sand with few to common pea gravel, well oxidized, common to many moderately thick and common thick clay films, undetermined lower boundary to;	6Btb
	40	8	0/5.0			Run 8 No Recovery – Sample slid out of core barrel Stopped drilling at 4:00PM @45'. Started backfilling at 4:30PM. Backfilled with cuttings to 20', bentonite chips to 10', and grout to 0' (top dyed black). Water levels: During drilling at ~21'	
	45						

Depth scale: 35, 36, 37, 38, 39, 40, 41, 42, 43, 44, 45, 46, 47, 48, 49, 50, 51, 52, 53, 54, 55

Appendix B

Cone Penetrometer Testing Data

SUMMARY

OF

CONE PENETRATION TEST DATA

Project:

8852 & 8874 Sunset Blvd.
Los Angeles, CA
July 11, 2017

Prepared for:

Mr. John Helms
40344 Wood Court
Palmdale, CA 93551
Office (661) 206-5860 / Fax (661) 718-3646

Prepared by:

KEHOE TESTING & ENGINEERING
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www.kehoetesting.com

TABLE OF CONTENTS

SUMMARY
OF
CONE PENETRATION TEST DATA

1. INTRODUCTION

This report presents the results of a Cone Penetration Test (CPT) program carried out for the project located at 8852 & 8874 Sunset Blvd. in Los Angeles, California. The work was performed by Kehoe Testing & Engineering (KTE) on July 11, 2017. The scope of work was performed as directed by John Helms personnel.

2. SUMMARY OF FIELD WORK

The fieldwork consisted of performing CPT soundings at five locations to determine the soil lithology. Groundwater measurements and hole collapse depths provided in **TABLE 2.1** are for information only. The readings indicate the apparent depth to which the hole is open and the apparent water level (if encountered) in the CPT probe hole at the time of measurement upon completion of the CPT. KTE does not warranty the accuracy of the measurements and the reported water levels may not represent the true or stabilized groundwater levels.

LOCATION	DEPTH OF CPT (ft)	COMMENTS/NOTES:
CPT-1	60	Groundwater @ 26 ft
CPT-2	60	Groundwater @ 23 ft
CPT-3	60	Groundwater @ 20 ft
CPT-4	60	Groundwater @ 20 ft
CPT-5	60	Groundwater @ 19 ft

TABLE 2.1 - Summary of CPT Soundings

3. FIELD EQUIPMENT & PROCEDURES

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

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

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

4. CONE PENETRATION TEST DATA & INTERPRETATION

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

Tables of basic CPT output from the interpretation program CPeT-IT are provided for CPT data averaged over one foot intervals in the Appendix. We recommend a geotechnical engineer review the assumed input parameters and the calculated output from the CPeT-IT program. A summary of the equations used for the tabulated parameters is provided in the Appendix.

It should be noted that it is not always possible to clearly identify a soil type based on qc, fs and u. In these situations, experience, judgement and an assessment of the pore pressure data should be used to infer the soil behavior type.

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

Sincerely,

KEHOE TESTING & ENGINEERING

Richard W. Koester, Jr.
General Manager

07/19/17-kk-8386

APPENDIX

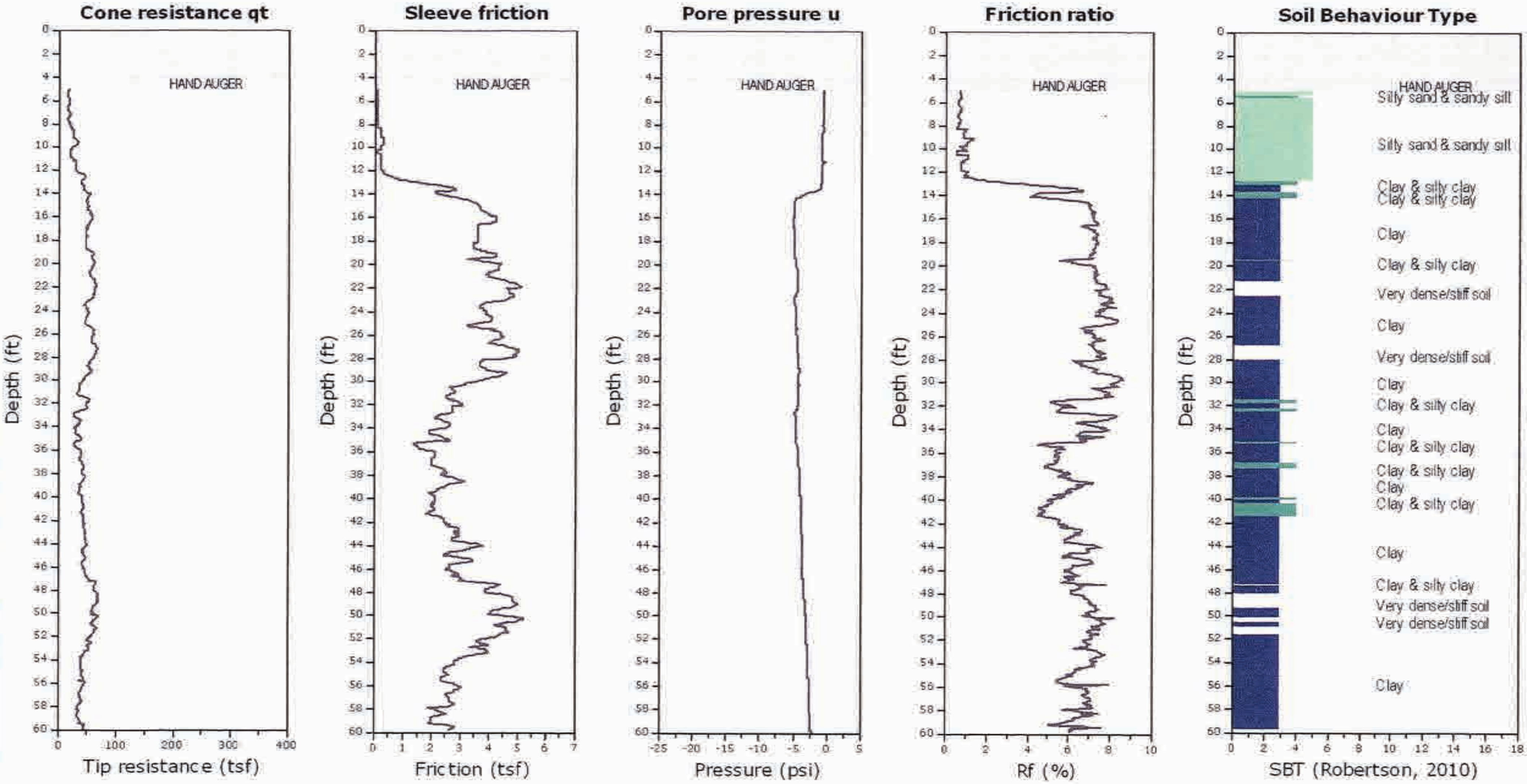
Kehoe Testing and Engineering
714-901-7270
rich@kehoetesting.com
www.kehoetesting.com

CPT-1

Project: **John Helms**

Location: **8852 & 8874 Sunset Blvd Los Angeles, CA**

Total depth: 60.12 ft, Date: 7/11/2017
Cone Type: Vertek



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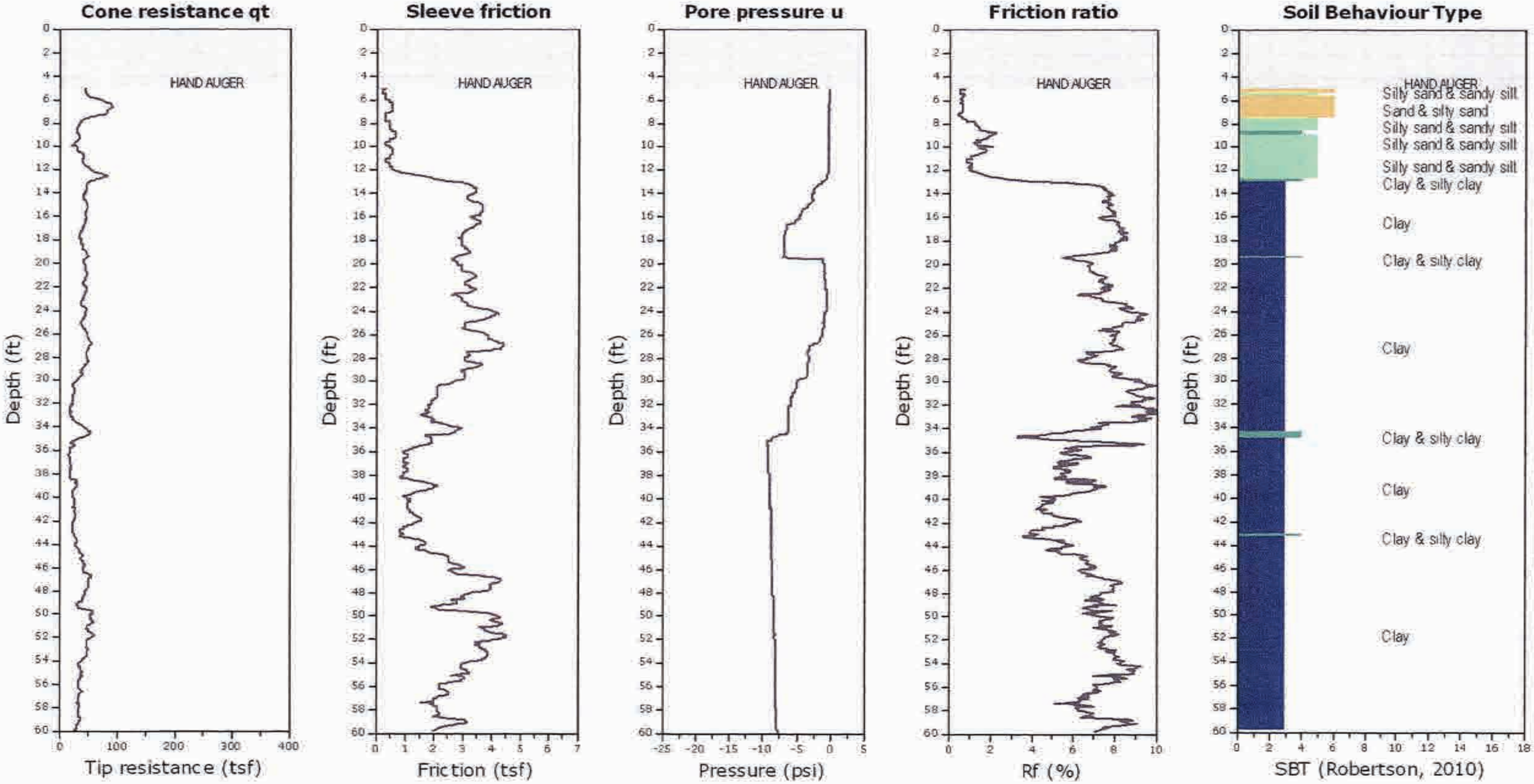
CPT-2

Project: **John Helms**

Total depth: 60.18 ft, Date: 7/11/2017

Location: **8852 & 8874 Sunset Blvd Los Angeles, CA**

Cone Type: Vertek



CPeT-IT v.2.0.1.55 - CPTU data presentation & interpretation software - Report created on: 7/12/2017, 10:07:12 AM
Project file: C:\HelmsLosAngeles7-17\Plot Data\Plots w-ha.cpt

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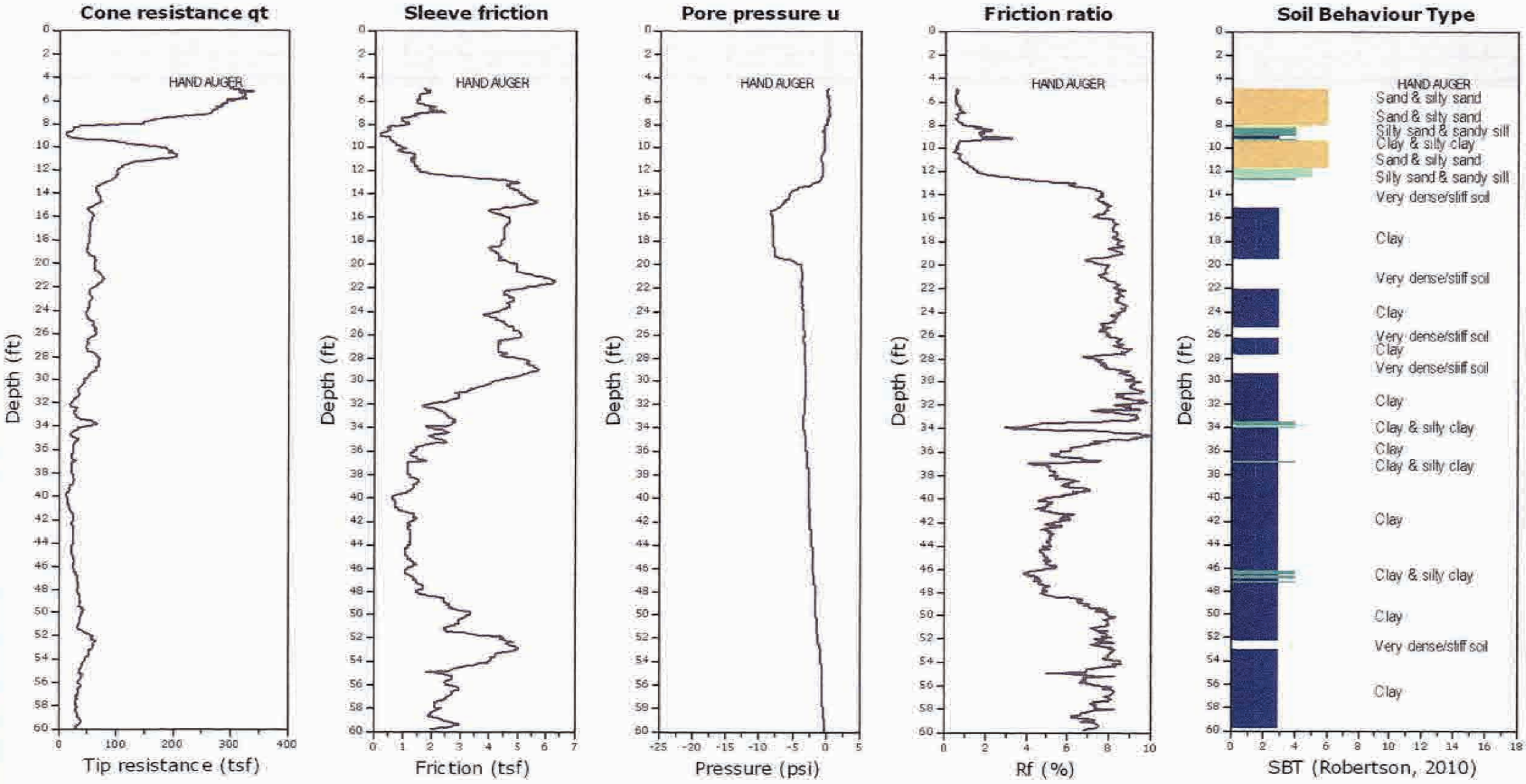
Project: **John Helms**
Location: **8852 & 8874 Sunset Blvd Los Angeles, CA**

CPT-3
Total depth: 60.18 ft, Date: 7/11/2017
Cone Type: Vertek



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714-901-7270
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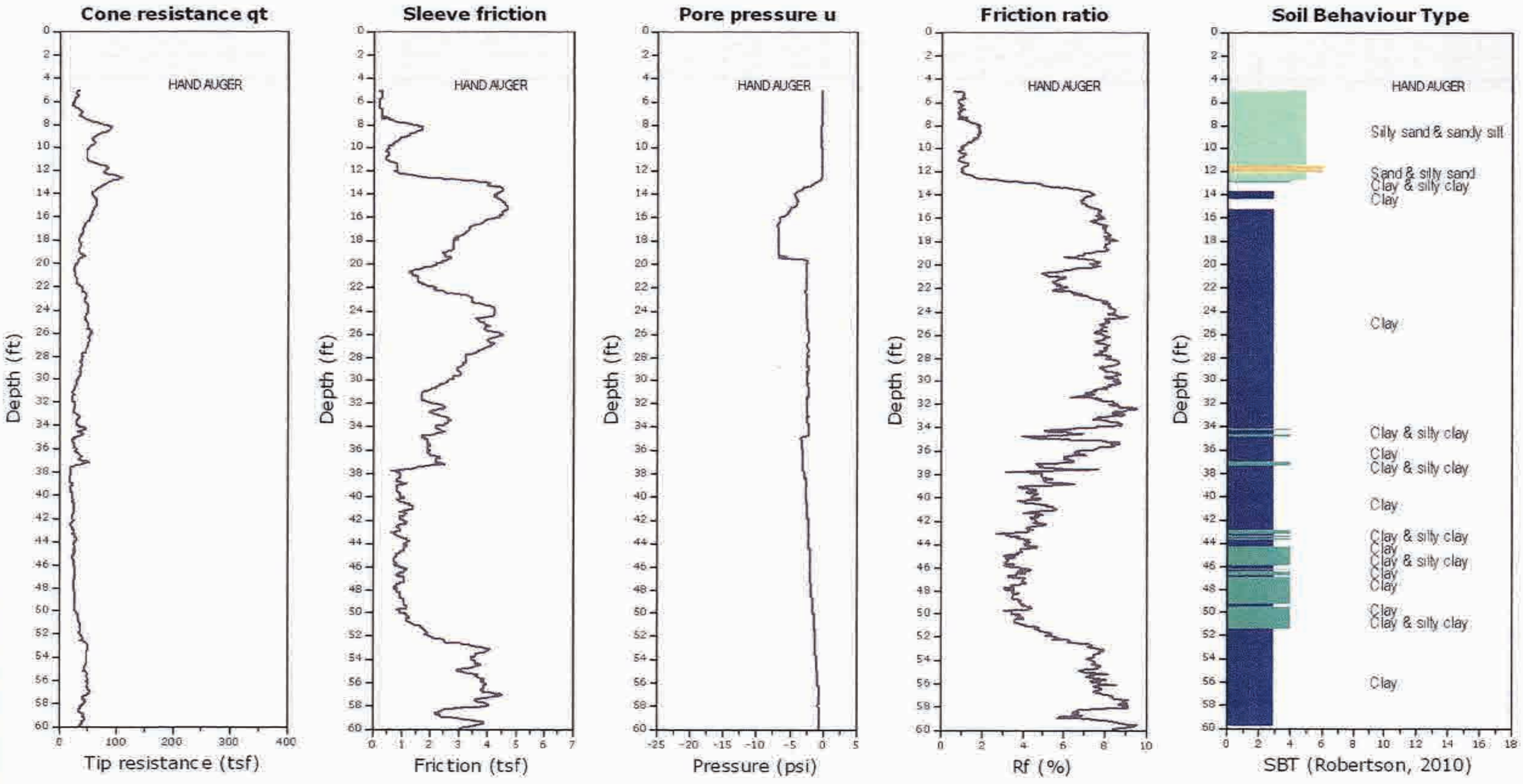
Project: **John Helms**
Location: **8852 & 8874 Sunset Blvd Los Angeles, CA**

CPT-4
Total depth: 60.24 ft, Date: 7/11/2017
Cone Type: Vertek



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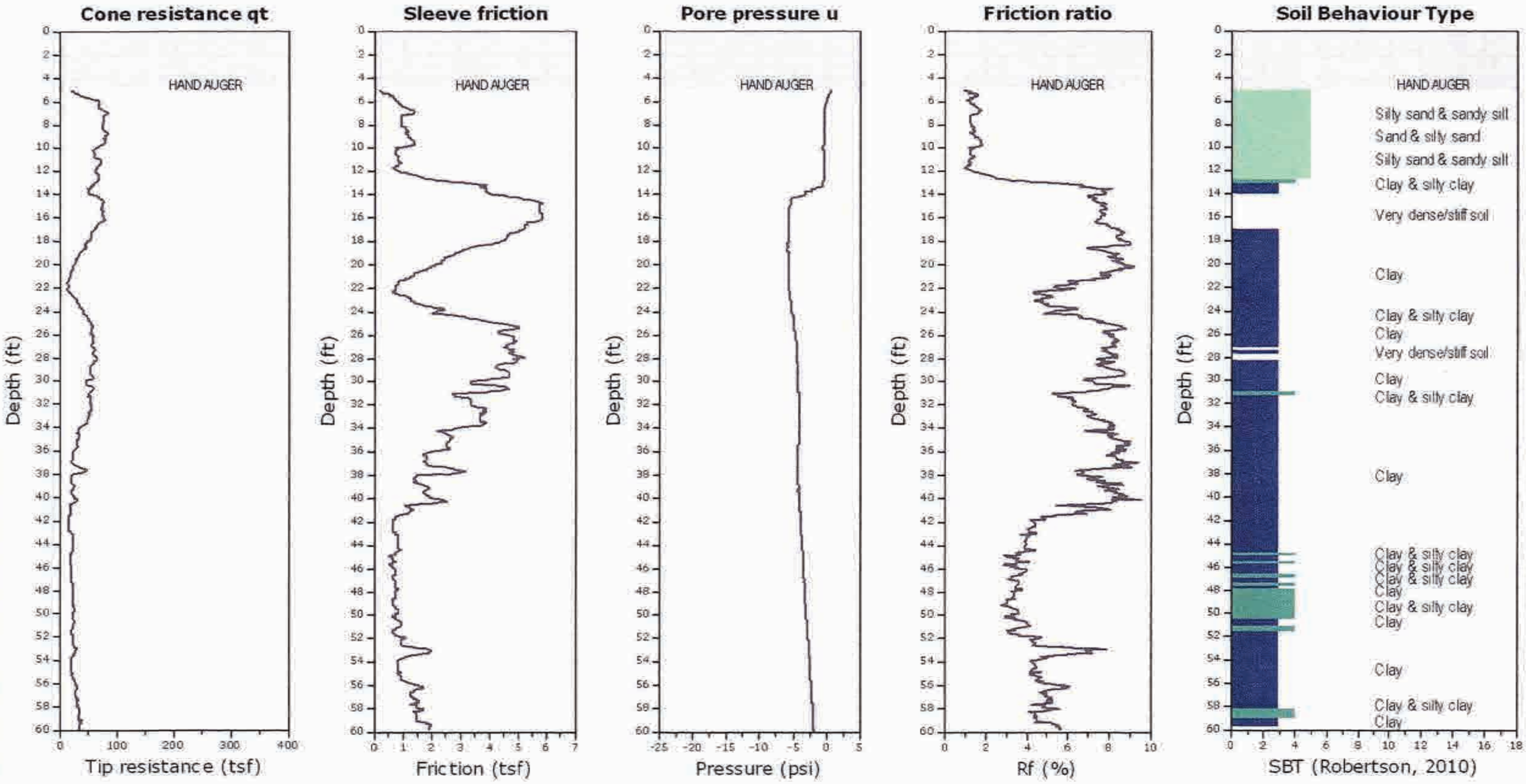
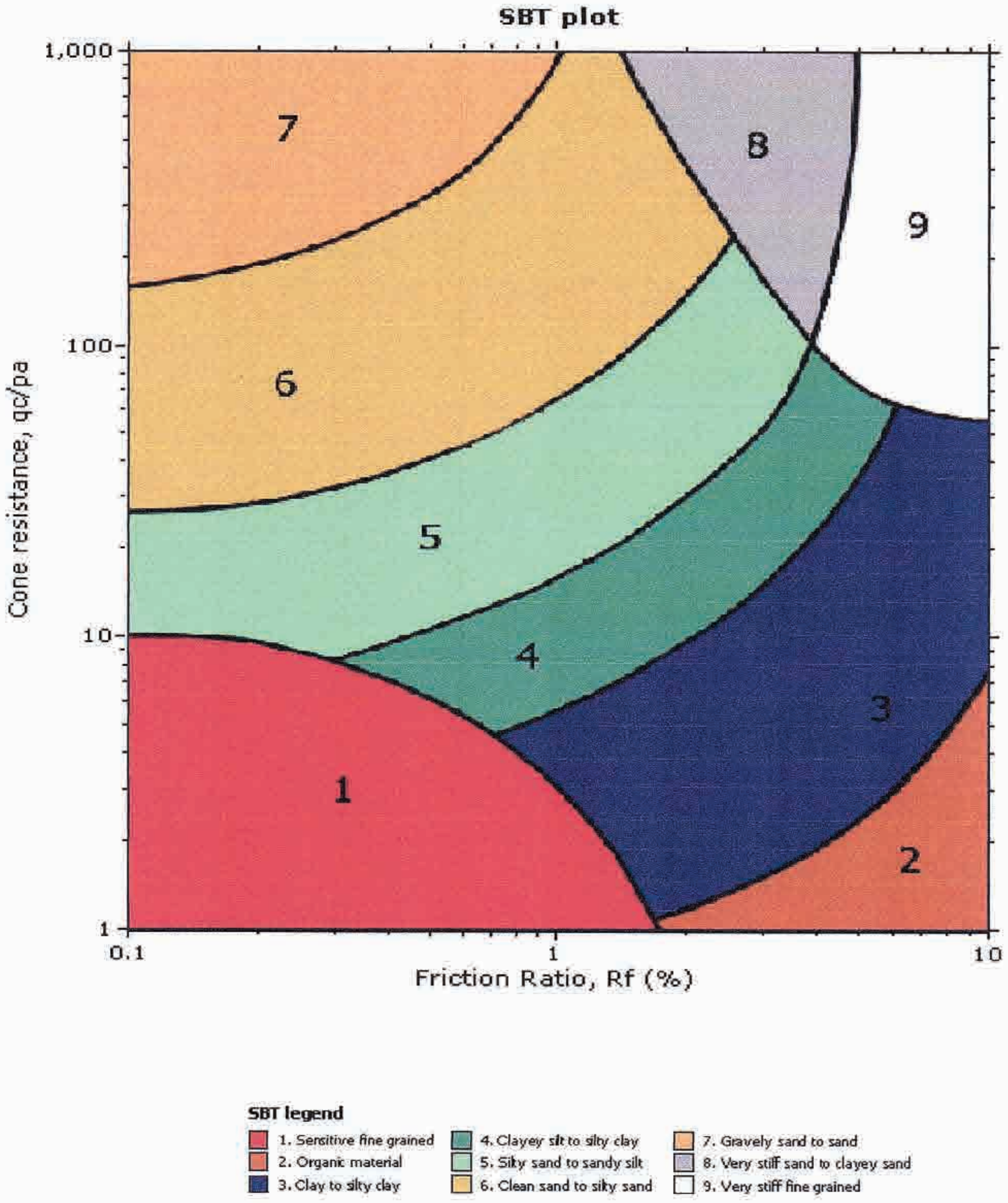
CPT-5

Project: **John Helms**
Location: **8852 & 8874 Sunset Blvd Los Angeles, CA**

Total depth: 60.11 ft, Date: 7/11/2017
Cone Type: Vertek



CPeT-IT v.2.0.1.55 - CPTU data presentation & interpretation software - Report created on: 7/12/2017, 10:08:06 AM
Project file: C:\HelmsLosAngeles7-17\Plot Data\Plots w-ha.cpt


SBT plot
1,000
100
10
1
Cone resistance, qc/pa
0.1
1
10
Friction Ratio, Rf (%)
7
8
9
6
5
4
3
1
2
SBT legend
1. Sensitive fine grained
2. Organic material
3. Clay to silty clay
4. Clayey silt to silty clay
5. Silty sand to sandy silt
6. Clean sand to silty sand
7. Gravely sand to sand
8. Very stiff sand to clayey sand
9. Very stiff fine grained

	CPT-1	In situ data									Basic output data												
Depth (ft)	qc (tsf)	fs (tsf)	u (psi)	Other	qt (tsf)	Rf(%)	SBT	Ic SBT	ã (pcf)	ó,v (tsf)	u0 (tsf)	ó',vo (tsf)	Qt1	Fr (%)	Bq	SBTn	n	Cn	Ic	Qtn	U2	I(B)	Mod. SBTn
1	168.75	3.03	-0.08	-2.96	168.75	1.79	6	1.94	130.89	0.07	0	0.07	2575.9	1.8	0	8	0.47	3.74	1.63	596.84	-0.09	53.16	7
2	57.54	0.31	-0.19	-3.15	57.54	0.54	6	1.98	111.66	0.12	0	0.12	473.12	0.55	0	6	0.47	2.76	1.61	149.52	-0.11	105.24	7
3	29.34	0.1	-0.29	-3.1	29.34	0.36	5	2.17	101.98	0.17	0	0.17	169.37	0.36	0	6	0.54	2.65	1.78	73.04	-0.12	86.36	7
4	19.74	0.1	-0.39	-3.04	19.73	0.53	5	2.39	101.01	0.22	0	0.22	87.58	0.54	0	6	0.63	2.67	2.02	49.18	-0.13	61.44	6
5	16.19	0.1	-0.48	-3.08	16.18	0.65	5	2.51	100.53	0.27	0	0.27	58.26	0.66	0	5	0.68	2.53	2.16	38.02	-0.13	50.57	6
6	17.23	0.1	-0.58	-3.35	17.22	0.61	5	2.47	100.68	0.32	0	0.32	52.25	0.62	0	5	0.69	2.26	2.16	36.17	-0.13	49.99	6
7	16.81	0.1	-0.68	-3.44	16.8	0.62	5	2.49	100.62	0.37	0	0.37	43.96	0.64	0	5	0.71	2.09	2.21	32.46	-0.13	46.85	6
8	17.96	0.1	-0.7	-3.43	17.95	0.58	5	2.45	100.78	0.42	0	0.42	41.34	0.6	0	5	0.71	1.91	2.21	31.7	-0.12	46.92	6
9	23.5	0.21	-0.77	-3.35	23.49	0.89	5	2.42	106.51	0.48	0	0.48	48.21	0.91	0	5	0.72	1.77	2.22	38.53	-0.12	46.23	6
10	24.54	0.31	-0.77	-3.49	24.53	1.28	5	2.49	109.58	0.53	0	0.53	45.1	1.31	0	5	0.76	1.68	2.31	38.14	-0.1	40.19	7
11	24.54	0.21	-0.76	-3.51	24.53	0.85	5	2.4	106.62	0.59	0	0.59	40.9	0.87	0	5	0.73	1.54	2.25	34.95	-0.09	44.73	6
12	30.28	0.42	-0.77	-3.39	30.27	1.38	5	2.43	112.2	0.64	0	0.64	46.18	1.41	0	5	0.76	1.46	2.31	40.95	-0.09	39.89	7
13	39.47	1.46	-0.97	-3.55	39.46	3.7	4	2.61	122.01	0.7	0	0.7	55.18	3.77	0	4	0.84	1.41	2.51	51.69	-0.1	23.28	5
14	54.41	3.03	-3.68	-3.64	54.36	5.57	4	2.64	128.12	0.77	0	0.77	69.91	5.65	0	9	0.86	1.32	2.57	66.93	-0.35	17.16	3
15	52.42	3.55	-5.03	-3.66	52.36	6.78	3	2.71	129.2	0.83	0	0.83	61.99	6.89	-0.01	9	0.9	1.24	2.66	60.57	-0.44	14.48	3
16	58.27	3.86	-5.13	-3.7	58.21	6.64	3	2.68	130.07	0.9	0	0.9	63.94	6.74	-0.01	9	0.9	1.16	2.64	62.89	-0.41	14.76	3
17	49.6	3.55	-4.96	-3.71	49.54	7.17	3	2.75	129.06	0.96	0	0.96	50.56	7.31	-0.01	3	0.94	1.09	2.73	50.25	-0.37	13.78	3
18	48.35	3.55	-4.93	-3.74	48.29	7.35	3	2.76	129	1.03	0	1.03	46.1	7.51	-0.01	3	0.95	1.03	2.77	46.03	-0.35	13.48	3
19	57.33	3.55	-4.84	-3.76	57.27	6.2	3	2.66	129.42	1.09	0	1.09	51.54	6.32	-0.01	3	0.92	0.97	2.68	51.66	-0.32	15.55	3
20	60.88	3.76	-4.64	-3.74	60.82	6.18	3	2.64	129.98	1.15	0	1.15	51.66	6.3	-0.01	3	0.92	0.92	2.67	52.01	-0.29	15.59	3
21	55.56	4.28	-4.55	-3.76	55.5	7.71	3	2.74	130.71	1.22	0	1.22	44.48	7.89	-0.01	3	0.97	0.87	2.79	44.66	-0.27	12.94	3
22	64.54	4.91	-4.55	-3.83	64.48	7.61	9	2.69	132.07	1.29	0	1.29	49.12	7.77	-0.01	3	0.96	0.83	2.76	49.49	-0.25	13.09	3
23	56.91	4.39	-4.93	-3.91	56.85	7.71	3	2.73	130.94	1.35	0	1.35	41.06	7.9	-0.01	3	0.99	0.79	2.82	41.19	-0.26	12.94	3
24	51.48	3.76	-4.64	-3.95	51.43	7.31	3	2.74	129.57	1.42	0	1.42	35.3	7.52	-0.01	3	1	0.75	2.84	35.3	-0.24	13.51	3
25	46.05	4.07	-4.64	-3.95	46	8.85	3	2.84	129.88	1.48	0	1.48	30.05	9.15	-0.01	3	1	0.71	2.95	30.05	-0.23	11.61	3
26	60.78	4.07	-4.55	-3.92	60.72	6.71	3	2.67	130.56	1.55	0	1.55	38.25	6.88	-0.01	3	0.99	0.69	2.79	38.45	-0.21	14.48	3
27	64.54	4.28	-4.45	-3.91	64.48	6.64	3	2.65	131.07	1.61	0	1.61	39.11	6.81	-0.01	3	0.99	0.66	2.78	39.35	-0.2	14.6	3
28	62.76	4.39	-4.35	-3.93	62.71	6.99	3	2.67	131.18	1.68	0.01	1.67	36.59	7.19	-0.01	3	1	0.63	2.82	36.59	-0.19	13.99	3
29	55.56	3.76	-4.16	-3.95	55.5	6.77	3	2.7	129.76	1.74	0.01	1.73	31.11	6.99	-0.01	3	1	0.61	2.86	31.11	-0.18	14.3	3
30	41.77	3.55	-4.26	-3.99	41.72	8.51	3	2.85	128.64	1.81	0.02	1.79	22.33	8.9	-0.01	3	1	0.59	3.03	22.33	-0.18	12.03	3
31	34.25	2.82	-4.26	-4.05	34.2	8.24	3	2.9	126.47	1.87	0.02	1.85	17.51	8.72	-0.01	3	1	0.57	3.1	17.51	-0.18	12.35	3
32	48.66	2.82	-4.45	-4.07	48.61	5.8	3	2.68	127.33	1.93	0.03	1.9	24.5	6.04	-0.01	3	1	0.56	2.89	24.5	-0.18	15.83	3
33	26.52	2.61	-4.74	-4.1	26.47	9.86	3	3.03	125.28	2	0.03	1.96	12.47	10.67	-0.02	3	1	0.54	3.27	12.47	-0.19	11.07	2
34	28.93	2.51	-4.73	-4.14	28.87	8.68	3	2.97	125.2	2.06	0.04	2.02	13.27	9.35	-0.01	3	1	0.52	3.21	13.27	-0.19	11.99	3
35	39.26	1.98	-4.64	-4.18	39.21	5.06	3	2.7	124.23	2.12	0.04	2.08	17.85	5.35	-0.01	3	1	0.51	2.95	17.85	-0.18	16.83	3
36	38.43	1.67	-4.35	-4.26	38.38	4.35	4	2.67	122.92	2.18	0.05	2.13	16.96	4.62	-0.01	3	1	0.5	2.93	16.96	-0.17	18.18	3
37	40	2.19	-4.26	-4.32	39.94	5.49	3	2.72	125.01	2.25	0.05	2.19	17.2	5.82	-0.01	3	1	0.48	2.99	17.2	-0.16	15.99	3
38	42.08	2.61	-4.06	-4.39	42.03	6.21	3	2.75	126.41	2.31	0.06	2.25	17.66	6.57	-0.01	3	1	0.47	3.02	17.66	-0.16	14.87	3
39	37.7	2.51	-4.06	-4.45	37.65	6.66	3	2.8	125.84	2.37	0.06	2.31	15.29	7.1	-0.01	3	1	0.46	3.08	15.29	-0.15	14.16	3
40	41.88	1.98	-3.97	-4.49	41.83	4.74	4	2.67	124.39	2.43	0.07	2.37	16.66	5.04	-0.01	3	1	0.45	2.96	16.66	-0.15	17.32	3
41	42.82	2.09	-3.87	-4.59	42.77	4.88	4	2.67	124.82	2.5	0.07	2.42	16.62	5.19	-0.01	3	1	0.44	2.97	16.62	-0.15	17.04	3
42	43.44	2.51	-3.87	-4.64	43.39	5.78	3	2.72	126.19	2.56	0.08	2.48	16.46	6.14	-0.01	3	1	0.43	3.02	16.46	-0.14	15.47	3
43	45.32	2.92	-3.77	-4.72	45.28	6.46	3	2.74	127.42	2.62	0.08	2.54	16.8	6.86	-0.01	3	1	0.42	3.04	16.8	-0.14	14.47	3
44	49.92	2.82	-3.77	-4.68	49.87	5.65	3	2.67	127.39	2.69	0.09	2.6	18.16	5.98	-0.01	3	1	0.41	2.98	18.16	-0.14	15.77	3
45	45.22	2.51	-3.68	-4.7	45.17	5.55	3	2.69	126.29	2.75	0.09	2.66	15.97	5.91	-0.01	3	1	0.4	3.02	15.97	-0.13	15.8	3
46	43.55	2.72	-3.68	-4.72	43.5	6.24	3	2.74	126.78	2.81	0.1	2.72	14.99	6.67	-0.01	3	1	0.39	3.07	14.99	-0.13	14.7	3
47	53.47	3.24	-3.58	-4.7	53.42	6.06	3	2.67	128.57	2.88	0.1	2.77	18.22	6.4	-0.01	3	1	0.38	3	18.22	-0.13	15.12	3
48	63.18	4.28	-3.39	-4.7	63.14	6.78	3	2.66	131.02	2.94	0.11	2.83	21.23	7.11	-0.01	3	1	0.37	2.98	21.23	-0.12	14.13	3
49	68.82	4.49	-3.29	-4.68	68.78	6.53	9	2.62	131.58	3.01	0.11	2.9	22.71	6.83	-0.01	3	1	0.37	2.95	22.71	-0.12	14.53	3
50	61.61	4.28	-3.11	-4.68	61.57	6.95	3	2.68	130.96	3.07	0.12	2.96	19.79	7.32	-0.01	3	1	0.36	3.01	19.79	-0.12	13.87	3
51	61.19	4.49	-3.1	-4.68	61.16	7.34	3	2.7	131.29	3.14	0.12	3.02	19.23	7.74	-0.01	3	1	0.35	3.04	19.23	-0.11	13.36	3
52	55.45	3.97	-3	-4.66	55.41	7.16	3	2.71	130.15	3.21	0.13	3.08	16.96	7.6	-0.01	3	1	0.34	3.07	16.96	-0.11	13.55	3
53	52.74	3.13	-2.9	-4.71	52.7	5.94	3	2.67	128.3	3.27	0.13	3.14	15.76	6.34	-0.01	3	1	0.34	3.04	15.76	-0.11	15.16	3
54	39.58	2.72	-2.81	-4.76	39.54	6.87	3	2.8	126.55	3.33	0.14	3.19	11.33	7.5	-0.01	3	1	0.33	3.2	11.33	-0.11	13.77	3
55	42.19	2.51	-2.81	-4.8	42.15	5.95	3	2.73	126.12	3.4	0.14	3.25	11.91	6.47	-0.01	3	1	0.33	3.14	11.91	-0.11	14.9	3
56	45.43	2.51	-2.71	-4.8	45.39	5.52	3	2.69	126.3	3.46	0.15	3.31	12.66	5.98	-0.01	3	1	0.32	3.1	12.66	-0.1	15.56	3
57	36.97	2.61	-2.61	-4.83	36.94	7.07	3	2.83	126.1	3.52	0.15	3.37	9.92	7.81	-0.01	3	1	0.31	3.25	9.92	-0.1	13.5	2
58	32.79	2.4	-2.61	-4.86	32.76	7.33	3	2.87	125.19	3.58	0.16	3.43	8.51	8.23	-0.01	3	1	0.31	3.32	8.51	-0.1	13.22	2
59	35.19	2.3	-2.42	-4.89	35.16	6.53	3	2.82	125.04	3.65	0.16	3.48	9.04	7.29	-0.01	3	1	0.3	3.26	9.04	-0.1	14.01	2
60	44.17	0	-2.32	-4.91	44.14	0	0	0	87.36	3.69	0.17	3.52	11.48	0	-0.01	0	1	0.3	4.06	11.48	-0.09	30.69	0

	CPT-2	In situ data									Basic output data												
Depth (ft)	qc (tsf)	fs (tsf)	u (psi)	Other	qt (tsf)	Rf(%)	SBT	Ic SBT	ā (pcf)	ó,v (tsf)	u0 (tsf)	ó',vo (tsf)	Qt1	Fr (%)	Bq	SBTn	n	Cn	Ic	Qtn	U2	I(B)	Mod. SBTn
1	119.46	1.46	-0.59	-0.69	119.46	1.22	6	1.93	124.72	0.06	0	0.06	1913.4	1.22	0	6	0.45	3.59	1.57	405.48	-0.68	73.34	7
2	43.02	0.31	-0.1	-0.63	43.02	0.73	5	2.15	110.95	0.12	0	0.12	363.87	0.73	0	6	0.52	3.12	1.74	126.68	-0.06	84.11	7
3	19.63	0.21	-0.1	-0.62	19.63	1.06	5	2.53	106.07	0.17	0	0.17	113.93	1.07	0	5	0.66	3.32	2.1	61.13	-0.04	52.45	7
4	20.15	0.21	-0.1	-0.78	20.15	1.04	5	2.51	106.14	0.22	0	0.22	89	1.05	0	5	0.68	2.85	2.14	53.76	-0.03	50.47	7
5	42.29	0.21	-0.19	-0.95	42.29	0.49	5	2.08	107.95	0.28	0	0.28	151.17	0.5	0	6	0.55	2.09	1.8	82.87	-0.05	83.52	7
6	67.98	0.42	-0.19	-1.37	67.98	0.61	6	1.94	114.17	0.34	0	0.34	201.9	0.62	0	6	0.52	1.83	1.73	116.88	-0.04	89.24	7
7	82.18	0.42	-0.29	-1.77	82.18	0.51	6	1.83	114.64	0.39	0	0.39	208.43	0.51	0	6	0.5	1.64	1.65	126.72	-0.05	101.48	7
8	34.57	0.42	-0.39	-2.42	34.56	1.21	5	2.35	112.52	0.45	0	0.45	76.06	1.22	0	5	0.69	1.81	2.15	58.25	-0.06	48.29	7
9	32.58	0.42	-0.39	-2.91	32.58	1.28	5	2.39	112.38	0.5	0	0.5	63.54	1.3	0	5	0.71	1.7	2.21	51.45	-0.06	44.85	7
10	26.94	0.42	-0.39	-74.93	26.94	1.55	5	2.5	111.92	0.56	0	0.56	47.04	1.58	0	5	0.77	1.63	2.34	40.6	-0.05	37.68	7
11	40	0.52	-0.39	-77.98	39.99	1.31	5	2.32	114.51	0.62	0	0.62	63.71	1.33	0	5	0.71	1.47	2.19	54.63	-0.05	45.37	7
12	54.93	0.94	-0.39	-77.92	54.92	1.71	5	2.28	119.59	0.68	0	0.68	80.02	1.73	0	5	0.71	1.37	2.18	70.43	-0.04	41.89	7
13	52.21	2.51	-1.49	-77.6	52.2	4.8	4	2.6	126.64	0.74	0	0.74	69.44	4.87	0	4	0.85	1.35	2.52	65.75	-0.14	19.41	3
14	42.82	3.45	-2.81	-75.75	42.78	8.06	3	2.83	128.49	0.81	0	0.81	52.12	8.21	0	3	0.94	1.29	2.77	51.31	-0.25	12.48	3
15	47.51	3.45	-3.44	-75.26	47.47	7.26	3	2.76	128.74	0.87	0	0.87	53.58	7.39	-0.01	3	0.93	1.2	2.72	52.84	-0.28	13.64	3
16	45.53	3.55	-4.74	-74.64	45.47	7.81	3	2.8	128.85	0.93	0	0.93	47.68	7.97	-0.01	3	0.95	1.13	2.78	47.4	-0.37	12.82	3
17	38.43	3.24	-6.67	-76.4	38.35	8.44	3	2.87	127.76	1	0	1	37.42	8.67	-0.01	3	0.99	1.06	2.87	37.4	-0.48	12.03	3
18	35.92	2.92	-7.06	-76.02	35.84	8.16	3	2.88	126.85	1.06	0	1.06	32.76	8.41	-0.01	3	1	1	2.9	32.76	-0.48	12.38	3
19	43.96	2.92	-6.96	-73.15	43.88	6.66	3	2.76	127.34	1.13	0	1.13	38	6.84	-0.01	3	0.97	0.94	2.79	38.08	-0.45	14.55	3
20	41.35	2.82	-1.06	-76.84	41.34	6.82	3	2.78	126.93	1.19	0	1.19	33.78	7.02	0	3	0.99	0.89	2.83	33.83	-0.06	14.25	3
21	46.16	3.13	-0.97	-76.73	46.14	6.79	3	2.75	127.97	1.25	0	1.25	35.84	6.98	0	3	0.98	0.85	2.81	35.95	-0.06	14.32	3
22	44.38	2.92	-0.77	-76.71	44.37	6.59	3	2.75	127.37	1.32	0	1.32	32.71	6.79	0	3	0.99	0.81	2.83	32.76	-0.04	14.62	3
23	40	3.03	-0.59	-76.68	39.99	7.57	3	2.83	127.38	1.38	0	1.38	27.98	7.84	0	3	1	0.77	2.93	27.98	-0.03	13.12	3
24	45.84	3.45	-0.77	-76.59	45.83	7.52	3	2.78	128.65	1.44	0	1.44	30.84	7.76	0	3	1	0.74	2.89	30.84	-0.04	13.2	3
25	36.65	3.55	-1.08	-76.72	36.64	9.69	3	2.93	128.33	1.51	0.01	1.5	23.44	10.11	0	3	1	0.71	3.06	23.44	-0.06	10.9	2
26	48.25	3.55	-1.26	-76.48	48.23	7.36	3	2.76	129	1.57	0.01	1.56	29.94	7.61	0	3	1	0.68	2.9	29.94	-0.07	13.41	3
27	53.47	3.86	-3.23	-76.52	53.43	7.23	3	2.73	129.86	1.64	0.02	1.62	32	7.46	0	3	1	0.65	2.87	32	-0.16	13.6	3
28	45.84	3.34	-3.24	-76.45	45.8	7.3	3	2.78	128.43	1.7	0.02	1.68	26.29	7.58	-0.01	3	1	0.63	2.93	26.29	-0.15	13.48	3
29	38.32	3.34	-3.48	-76.46	38.28	8.73	3	2.89	127.99	1.77	0.03	1.74	21.03	9.15	-0.01	3	1	0.61	3.06	21.03	-0.16	11.82	3
30	29.66	2.72	-5.04	-76.38	29.6	9.17	3	2.98	125.84	1.83	0.03	1.79	15.47	9.78	-0.01	3	1	0.59	3.18	15.47	-0.22	11.51	3
31	23.5	1.98	-5.81	-76.3	23.43	8.47	3	3.02	122.98	1.89	0.04	1.85	11.63	9.21	-0.02	3	1	0.57	3.25	11.63	-0.25	12.21	3
32	21.72	1.88	-6	-76.2	21.65	8.68	3	3.05	122.39	1.95	0.04	1.91	10.32	9.54	-0.02	3	1	0.55	3.3	10.32	-0.25	12.06	2
33	19.32	1.98	-6.29	-75.98	19.24	10.31	3	3.14	122.5	2.01	0.05	1.96	8.77	11.52	-0.03	3	1	0.54	3.4	8.77	-0.26	10.98	1
34	40.73	2.09	-6.29	-75.92	40.65	5.14	3	2.7	124.7	2.08	0.05	2.02	19.08	5.41	-0.01	3	1	0.52	2.93	19.08	-0.25	16.78	3
35	29.55	1.57	-8.91	-75.91	29.44	5.32	3	2.81	121.8	2.14	0.06	2.08	13.15	5.74	-0.03	3	1	0.51	3.07	13.15	-0.34	15.92	3
36	17.54	1.15	-9.29	-75.93	17.43	6.59	3	3.04	118.26	2.2	0.06	2.13	7.15	7.54	-0.05	3	1	0.5	3.35	7.15	-0.34	13.84	2
37	17.44	0.94	-9.19	-75.9	17.33	5.42	3	2.98	116.77	2.25	0.07	2.19	6.9	6.24	-0.05	3	1	0.48	3.31	6.9	-0.33	14.95	2
38	16.81	1.25	-9.09	-75.92	16.7	7.5	3	3.09	118.79	2.31	0.07	2.24	6.43	8.71	-0.05	3	1	0.47	3.43	6.43	-0.33	13.04	2
39	28.4	1.46	-9.09	-75.85	28.29	5.17	3	2.81	121.2	2.37	0.08	2.3	11.29	5.64	-0.03	3	1	0.46	3.12	11.29	-0.32	15.93	2
40	21.62	1.04	-8.99	-75.8	21.51	4.86	3	2.88	118.07	2.43	0.08	2.35	8.12	5.47	-0.04	3	1	0.45	3.22	8.12	-0.31	15.83	2
41	25.9	1.25	-8.93	-75.68	25.79	4.86	3	2.82	119.85	2.49	0.09	2.4	9.69	5.38	-0.03	3	1	0.44	3.16	9.69	-0.3	16.12	2
42	23.7	1.36	-8.9	-75.65	23.6	5.75	3	2.9	120.22	2.55	0.09	2.46	8.56	6.45	-0.03	3	1	0.43	3.25	8.56	-0.3	14.82	2
43	22.97	1.15	-8.8	-75.42	22.87	5.02	3	2.87	118.92	2.61	0.1	2.51	8.06	5.67	-0.04	3	1	0.42	3.24	8.06	-0.29	15.61	2
44	27.05	1.78	-8.8	-75.32	26.94	6.59	3	2.9	122.5	2.67	0.1	2.57	9.44	7.32	-0.03	3	1	0.41	3.25	9.44	-0.29	13.98	2
45	39.68	2.4	-8.8	-75.25	39.57	6.07	3	2.76	125.65	2.74	0.11	2.63	14.02	6.52	-0.02	3	1	0.4	3.09	14.02	-0.28	14.88	3
46	40.52	3.24	-8.7	-75.28	40.41	8.01	3	2.84	127.89	2.8	0.11	2.69	13.99	8.61	-0.02	3	1	0.39	3.17	13.99	-0.28	12.6	3
47	51.27	3.97	-8.61	-74.29	51.17	7.76	3	2.76	129.95	2.87	0.12	2.75	17.58	8.22	-0.02	3	1	0.39	3.08	17.58	-0.27	12.86	3
48	45.22	3.45	-8.61	-74.08	45.11	7.64	3	2.79	128.61	2.93	0.12	2.81	15.03	8.17	-0.02	3	1	0.38	3.13	15.03	-0.26	12.98	3
49	32.27	2.92	-8.51	-75.36	32.16	9.09	3	2.95	126.59	2.99	0.13	2.87	10.18	10.02	-0.03	3	1	0.37	3.32	10.18	-0.26	11.73	2
50	55.76	3.86	-8.44	-77.44	55.66	6.94	3	2.7	129.96	3.06	0.13	2.93	17.98	7.35	-0.01	3	1	0.36	3.04	17.98	-0.25	13.85	3
51	49.39	4.39	-8.41	-77.46	49.29	8.9	3	2.82	130.6	3.12	0.14	2.99	15.46	9.5	-0.02	3	1	0.35	3.17	15.46	-0.25	11.74	3
52	54.51	4.18	-8.32	-77.38	54.41	7.68	3	2.74	130.48	3.19	0.14	3.05	16.81	8.16	-0.01	3	1	0.35	3.1	16.81	-0.24	12.95	3
53	49.19	3.55	-8.27	-77.34	49.08	7.23	3	2.75	129.04	3.25	0.15	3.11	14.76	7.75	-0.02	3	1	0.34	3.12	14.76	-0.24	13.43	3
54	35.92	3.45	-8.32	-77.3	35.82	9.62	3	2.94	128.05	3.32	0.15	3.16	10.27	10.6	-0.02	3	1	0.33	3.33	10.27	-0.24	11.33	2
55	37.38	2.92	-8.32	-77.09	37.28	7.84	3	2.86	126.95	3.38	0.16	3.22	10.52	8.62	-0.02	3	1	0.33	3.26	10.52	-0.23	12.77	2
56	30.81	2.51	-8.22	-76.96	30.71	8.16	3	2.93	125.35	3.44	0.16	3.28	8.31	9.19	-0.03	3	1	0.32	3.36	8.31	-0.23	12.51	2
57	31.75	2.3	-8.22	-76.9	31.65	7.26	3	2.88	124.78	3.51	0.17	3.34	8.43	8.16	-0.03	3	1	0.32	3.32	8.43	-0.23	13.28	2
58	33	2.09	-8.12	-76.72	32.9	6.35	3	2.83	124.18	3.57	0.17	3.4	8.64	7.12	-0.03	3	1	0.31	3.27	8.64	-0.22	14.17	2
59	36.03	2.19	-8.07	-76.4	35.93	6.1	3	2.79	124.75	3.63	0.18	3.45	9.35	6.79	-0.02	3	1	0.31	3.23	9.35	-0.22	14.5	2
60	28.72	0	-7.93	-76.25	28.62	0	0	0	87.36	3.67	0.18	3.49	7.14	0	-0.03	0	1	0.3	4.06	7.14	-0.22	24.49	0

	CPT-3	In situ data									Basic output data												
Depth (ft)	qc (tsf)	fs (tsf)	u (psi)	Other	qt (tsf)	Rf(%)	SBT	Ic SBT	ã (pcf)	ó,v (tsf)	u0 (tsf)	ó',vo (tsf)	Qt1	Fr (%)	Bq	SBTn	n	Cn	Ic	Qtn	U2	I(B)	Mod. SBTn
1	251.77	1.88	0.75	-0.64	251.78	0.75	6	1.55	128.37	0.06	0	0.06	3919.1	0.75	0	7	0.34	2.62	1.28	623.49	0.84	118.28	7
2	185.67	1.57	0.58	-0.67	185.68	0.84	6	1.68	126.3	0.13	0	0.13	1456.2	0.84	0	6	0.4	2.34	1.43	410.9	0.33	100.96	7
3	342.42	1.78	1.06	-0.83	342.43	0.52	6	1.34	128.71	0.19	0	0.19	1786.2	0.52	0	7	0.31	1.7	1.19	550.61	0.4	157.65	7
4	211.78	1.46	1.16	-1.23	211.79	0.69	6	1.58	126.11	0.25	0	0.25	830.54	0.69	0	6	0.4	1.76	1.41	352.28	0.33	115.57	7
5	290.41	1.78	0.18	-0.21	290.41	0.61	6	1.44	128.3	0.32	0	0.32	909.69	0.61	0	7	0.37	1.55	1.31	424.86	0.04	131.78	7
6	298.24	1.67	0.29	-0.36	298.25	0.56	6	1.41	127.92	0.38	0	0.38	777.92	0.56	0	7	0.36	1.44	1.3	406.34	0.05	139.74	7
7	264.51	1.57	0.32	1.64	264.52	0.59	6	1.46	127.16	0.45	0	0.45	591.4	0.59	0	6	0.39	1.4	1.36	348.87	0.05	129.58	7
8	146.41	0.73	0	3.47	146.41	0.5	6	1.62	120.14	0.51	0	0.51	288.1	0.5	0	6	0.45	1.39	1.5	191.51	0	121.43	7
9	12.84	0.52	-0.3	4.29	12.84	4.07	3	3.01	111.74	0.56	0	0.56	21.83	4.25	0	3	0.96	1.83	2.83	21.23	-0.04	19.48	3
10	169.9	0.94	-0.39	0	169.9	0.55	6	1.59	122.34	0.62	0	0.62	271.47	0.56	0	6	0.45	1.27	1.51	203.44	-0.04	116.66	7
11	181.6	1.36	-0.77	0.24	181.59	0.75	6	1.65	125.2	0.69	0	0.69	263.63	0.75	0	6	0.49	1.23	1.58	211.04	-0.08	96.79	7
12	100.98	2.3	-0.58	0.35	100.97	2.28	5	2.17	127.61	0.75	0	0.75	133.63	2.29	0	5	0.69	1.27	2.11	120	-0.06	37.67	7
13	82.39	4.18	-1.92	0.46	82.37	5.07	9	2.49	131.49	0.82	0	0.82	99.99	5.12	0	9	0.82	1.24	2.44	95.36	-0.17	18.87	3
14	67.56	5.12	-5.53	0.38	67.5	7.58	9	2.68	132.49	0.88	0	0.88	75.54	7.68	-0.01	9	0.9	1.18	2.64	74.16	-0.45	13.16	3
15	63.91	5.12	-6.92	-1.31	63.82	8.02	9	2.71	132.35	0.95	0	0.95	66.32	8.14	-0.01	9	0.92	1.11	2.7	65.75	-0.53	12.52	3
16	60.46	4.39	-8.24	-0.24	60.36	7.27	3	2.7	131.09	1.01	0	1.01	58.55	7.39	-0.01	3	0.92	1.04	2.7	58.36	-0.59	13.64	3
17	55.56	4.59	-8.12	0.16	55.46	8.29	3	2.76	131.22	1.08	0	1.08	50.38	8.45	-0.01	3	0.96	0.98	2.78	50.42	-0.54	12.18	3
18	52.53	4.28	-8.03	1.26	52.43	8.17	3	2.77	130.57	1.14	0	1.14	44.81	8.35	-0.01	3	0.97	0.93	2.81	44.9	-0.51	12.34	3
19	48.77	4.18	-7.91	0.8	48.67	8.58	3	2.81	130.21	1.21	0	1.21	39.24	8.8	-0.01	3	1	0.87	2.86	39.24	-0.47	11.85	3
20	62.45	4.59	-4.26	-0.09	62.4	7.36	9	2.69	131.51	1.28	0	1.28	47.92	7.52	-0.01	3	0.96	0.84	2.75	48.29	-0.24	13.46	3
21	74.25	5.33	-3.77	-0.36	74.2	7.18	9	2.64	133.01	1.34	0	1.34	54.49	7.31	0	3	0.94	0.8	2.71	55.2	-0.21	13.77	3
22	63.81	5.53	-3.77	-0.29	63.76	8.68	9	2.74	132.93	1.41	0.01	1.4	44.58	8.88	0	3	0.99	0.76	2.83	44.64	-0.2	11.72	3
23	58.17	4.59	-3.68	0.11	58.12	7.91	3	2.73	131.34	1.47	0.01	1.46	38.82	8.11	0	3	1	0.73	2.84	38.82	-0.19	12.68	3
24	50.44	4.49	-3.58	-0.01	50.39	8.91	3	2.81	130.82	1.54	0.02	1.52	32.15	9.19	-0.01	3	1	0.7	2.94	32.15	-0.18	11.53	3
25	59	4.39	-3.39	0.52	58.96	7.44	3	2.71	131.03	1.6	0.02	1.58	36.29	7.65	0	3	1	0.67	2.84	36.29	-0.17	13.32	3
26	65.16	4.59	-3.29	0.29	65.12	7.06	9	2.67	131.62	1.67	0.03	1.64	38.66	7.24	0	3	1	0.65	2.8	38.72	-0.16	13.9	3
27	50.44	4.39	-3.29	0.09	50.4	8.7	3	2.81	130.65	1.74	0.03	1.7	28.59	9.01	-0.01	3	1	0.62	2.96	28.59	-0.16	11.78	3
28	67.04	4.91	-3.21	0.37	67	7.33	9	2.67	132.17	1.8	0.04	1.76	36.99	7.53	0	3	1	0.6	2.83	36.99	-0.15	13.49	3
29	64.64	5.22	-3.1	0.54	64.6	8.08	9	2.71	132.53	1.87	0.04	1.82	34.39	8.32	0	3	1	0.58	2.88	34.39	-0.15	12.46	3
30	44.07	4.28	-3.1	0.69	44.03	9.72	3	2.88	130.14	1.93	0.05	1.88	22.34	10.17	-0.01	3	1	0.56	3.08	22.34	-0.14	10.88	2
31	33.94	3.03	-3.1	1.02	33.9	8.93	3	2.93	126.97	2	0.05	1.94	16.42	9.49	-0.01	3	1	0.54	3.15	16.42	-0.14	11.7	3
32	20.68	2.51	-3.29	1.81	20.64	12.14	3	3.17	124.38	2.06	0.06	2	9.29	13.49	-0.02	3	1	0.53	3.43	9.29	-0.15	9.88	1
33	27.78	2.51	-3.39	5.38	27.74	9.04	3	2.99	125.1	2.12	0.06	2.06	12.45	9.78	-0.01	3	1	0.51	3.24	12.45	-0.15	11.7	3
34	48.56	2.4	-3.48	3.48	48.52	4.95	4	2.63	126.15	2.18	0.07	2.12	21.89	5.18	-0.01	3	1	0.5	2.88	21.89	-0.15	17.38	3
35	30.49	1.78	-3.1	4.52	30.45	5.83	3	2.83	122.8	2.25	0.07	2.17	12.98	6.29	-0.01	3	1	0.49	3.1	12.98	-0.14	15.15	3
36	24.12	1.57	-2.9	0.66	24.09	6.5	3	2.93	121.32	2.31	0.08	2.23	9.77	7.19	-0.01	3	1	0.47	3.23	9.77	-0.13	14.09	2
37	26.73	1.46	-2.9	0.52	26.7	5.48	3	2.85	121.06	2.37	0.08	2.28	10.65	6.01	-0.01	3	1	0.46	3.16	10.65	-0.13	15.41	2
38	22.77	1.25	-2.71	-0.13	22.73	5.51	3	2.9	119.54	2.43	0.09	2.34	8.68	6.17	-0.01	3	1	0.45	3.23	8.68	-0.12	15.12	2
39	21.93	1.25	-2.71	0.23	21.9	5.72	3	2.92	119.45	2.49	0.09	2.39	8.11	6.46	-0.01	3	1	0.44	3.27	8.11	-0.12	14.8	2
40	12.95	0.84	-2.71	0.94	12.92	6.47	3	3.13	115.2	2.54	0.1	2.45	4.24	8.06	-0.03	2	1	0.43	3.55	4.24	-0.12	13.67	2
41	18.07	0.94	-2.51	1.96	18.04	5.21	3	2.96	116.87	2.6	0.1	2.5	6.17	6.09	-0.02	3	1	0.42	3.35	6.17	-0.11	15.03	2
42	24.44	1.36	-2.42	1.71	24.41	5.56	3	2.88	120.3	2.66	0.11	2.56	8.51	6.24	-0.01	3	1	0.41	3.24	8.51	-0.11	15.03	2
43	23.81	1.25	-2.32	1.35	23.78	5.27	3	2.87	119.65	2.72	0.11	2.61	8.07	5.95	-0.01	3	1	0.41	3.25	8.07	-0.11	15.31	2
44	24.23	1.15	-2.18	1.7	24.2	4.75	3	2.84	119.06	2.78	0.12	2.66	8.04	5.36	-0.01	3	1	0.4	3.22	8.04	-0.1	15.95	2
45	23.08	1.15	-2.03	0.21	23.05	4.98	3	2.87	118.94	2.84	0.12	2.72	7.43	5.68	-0.01	3	1	0.39	3.26	7.43	-0.1	15.53	2
46	23.7	1.36	-1.97	0.48	23.68	5.73	3	2.9	120.23	2.9	0.13	2.77	7.49	6.53	-0.01	3	1	0.38	3.3	7.49	-0.1	14.71	2
47	31.01	1.36	-1.74	0.44	30.99	4.38	4	2.73	120.88	2.96	0.13	2.83	9.9	4.84	-0.01	3	1	0.37	3.12	9.9	-0.09	16.87	2
48	30.91	1.98	-1.64	0.94	30.89	6.42	3	2.85	123.65	3.02	0.14	2.89	9.65	7.12	-0.01	3	1	0.37	3.24	9.65	-0.09	14.17	2
49	37.07	2.61	-1.55	1.8	37.05	7.05	3	2.83	126.1	3.09	0.14	2.95	11.53	7.69	-0.01	3	1	0.36	3.2	11.53	-0.09	13.57	3
50	40.62	3.03	-1.55	0.34	40.6	7.46	3	2.82	127.41	3.15	0.15	3	12.47	8.09	-0.01	3	1	0.35	3.19	12.47	-0.09	13.15	3
51	34.46	3.13	-1.45	-9.29	34.44	9.1	3	2.93	127.26	3.21	0.15	3.06	10.2	10.03	-0.01	3	1	0.35	3.32	10.2	-0.08	11.72	2
52	59.63	3.97	-1.26	-9.87	59.61	6.66	3	2.67	130.33	3.28	0.16	3.12	18.04	7.04	0	3	1	0.34	3.03	18.04	-0.08	14.23	3
53	59.84	4.59	-0.87	-10.91	59.83	7.68	3	2.72	131.41	3.35	0.16	3.18	17.74	8.14	0	3	1	0.33	3.08	17.74	-0.07	12.94	3
54	46.37	3.55	-0.77	-11.54	46.36	7.66	3	2.79	128.9	3.41	0.17	3.24	13.24	8.27	-0.01	3	1	0.33	3.18	13.24	-0.07	12.95	3
55	35.19	2.72	-0.58	-11.67	35.18	7.72	3	2.87	126.26	3.47	0.17	3.3	9.61	8.56	-0.01	3	1	0.32	3.29	9.61	-0.06	12.88	2
56	34.46	2.61	-0.48	-11.71	34.46	7.58	3	2.87	125.93	3.54	0.18	3.36	9.2	8.44	-0.01	3	1	0.31	3.3	9.2	-0.06	13	2
57	32.06	2.51	-0.52	-11.68	32.05	7.82	3	2.9	125.45	3.6	0.18	3.42	8.33	8.81	-0.01	3	1	0.31	3.34	8.33	-0.06	12.79	2
58	28.09	1.98	-0.37	-11.87	28.09	7.06	3	2.91	123.42	3.66	0.19	3.47	7.03	8.12	-0.01	3	1	0.3	3.38	7.03	-0.06	13.4	2
59	36.24	2.09	-0.21	-11.85	36.23	5.76	3	2.77	124.42	3.72	0.19	3.53	9.21	6.42	-0.01	3	1	0.3	3.22	9.21	-0.06	14.87	2
60	28.93	0	-0.09	-12.12	28.93	0	0	0	87.36	3.77	0.2	3.57	7.05	0	-0.01	0	1	0.3	4.06	7.05	-0.06	24.35	0

	CPT-4	In situ data									Basic output data												
Depth (ft)	qc (tsf)	fs (tsf)	u (psi)	Other	qt (tsf)	Rf(%)	SBT	Ic SBT	ã (pcf)	ó,v (tsf)	u0 (tsf)	ó',vo (tsf)	Qt1	Fr (%)	Bq	SBTn	n	Cn	Ic	Qtn	U2	I(B)	Mod. SBTn
1	138.37	2.09	-0.39	-1.2	138.36	1.51	6	1.95	127.68	0.06	0	0.06	2164.8	1.51	0	6	0.47	3.69	1.6	482.93	-0.44	61.67	7
2	54.09	0.63	0	-1.27	54.09	1.16	5	2.18	116.58	0.12	0	0.12	441.61	1.16	0	6	0.54	3.22	1.8	164.3	0	66.85	7
3	33.83	0.42	0	-1.31	33.83	1.23	5	2.36	112.47	0.18	0	0.18	188.76	1.24	0	6	0.62	3	1.99	95.36	0	55.94	7
4	30.18	0.21	0	-1.29	30.18	0.69	5	2.28	107.12	0.23	0	0.23	129.14	0.7	0	6	0.6	2.49	1.94	70.46	0	67.53	7
5	33.31	0.21	-0.1	-1.29	33.31	0.63	5	2.22	107.36	0.29	0	0.29	115.63	0.63	0	6	0.6	2.19	1.93	68.35	-0.02	69.2	7
6	25.06	0.31	-0.1	-1.41	25.06	1.25	5	2.47	109.64	0.34	0	0.34	72.61	1.27	0	5	0.7	2.22	2.2	51.82	-0.02	45.56	7
7	37.49	0.42	-0.1	-1.55	37.49	1.11	5	2.3	112.72	0.4	0	0.4	93.46	1.13	0	5	0.66	1.91	2.08	66.95	-0.02	52.93	7
8	76.86	0.94	-0.25	-1.63	76.86	1.22	5	2.07	120.41	0.46	0	0.46	167.21	1.23	0	6	0.6	1.65	1.91	119.46	-0.04	59.67	7
9	63.07	1.15	-0.29	-1.7	63.07	1.82	5	2.25	121.39	0.52	0	0.52	120.84	1.84	0	5	0.68	1.62	2.1	95.79	-0.04	43.02	7
10	52.42	0.63	-0.19	-1.79	52.42	1.2	5	2.2	116.51	0.58	0	0.58	90.02	1.21	0	5	0.66	1.5	2.07	73.39	-0.02	52.55	7
11	49.5	0.63	-0.19	-1.75	49.5	1.27	5	2.23	116.37	0.63	0	0.63	77.05	1.28	0	5	0.69	1.42	2.12	65.66	-0.02	49.07	7
12	78.11	1.46	-0.19	-1.79	78.11	1.87	5	2.19	123.68	0.7	0	0.7	111.22	1.89	0	5	0.68	1.33	2.11	97.47	-0.02	42.3	7
13	84.59	3.34	-1.44	-1.93	84.57	3.95	4	2.4	129.92	0.76	0	0.76	110.15	3.99	0	9	0.78	1.29	2.33	102.29	-0.14	23.5	6
14	57.33	4.49	-4.25	-2.02	57.28	7.84	3	2.74	131.13	0.83	0	0.83	68.31	7.95	-0.01	9	0.91	1.25	2.68	66.83	-0.37	12.77	3
15	64.33	4.49	-4.6	-2.06	64.27	6.99	9	2.67	131.41	0.89	0	0.89	71.04	7.08	-0.01	9	0.9	1.16	2.63	69.78	-0.37	14.14	3
16	54.82	4.18	-6.57	-2.08	54.74	7.63	3	2.74	130.49	0.96	0	0.96	56.17	7.77	-0.01	3	0.93	1.1	2.72	55.8	-0.49	13.07	3
17	41.77	3.34	-6.86	-2.08	41.69	8.02	3	2.83	128.2	1.02	0	1.02	39.8	8.22	-0.01	3	0.98	1.03	2.84	39.78	-0.48	12.54	3
18	34.77	2.82	-6.86	-2.12	34.69	8.13	3	2.89	126.51	1.08	0	1.08	30.98	8.39	-0.01	3	1	0.98	2.92	30.98	-0.46	12.42	3
19	34.36	2.72	-6.77	-2.16	34.27	7.92	3	2.89	126.2	1.15	0	1.15	28.86	8.2	-0.01	3	1	0.92	2.93	28.86	-0.42	12.68	3
20	29.45	2.09	-2.66	-2.2	29.42	7.1	3	2.9	123.91	1.21	0	1.21	23.31	7.4	-0.01	3	1	0.87	2.96	23.31	-0.16	13.73	3
21	25.69	1.36	-2.71	-2.22	25.66	5.29	3	2.85	120.42	1.27	0	1.27	19.27	5.57	-0.01	3	1	0.84	2.94	19.27	-0.16	16.51	3
22	36.13	2.19	-2.61	-2.27	36.1	6.07	3	2.79	124.76	1.33	0.01	1.32	26.29	6.31	-0.01	3	1	0.8	2.88	26.29	-0.15	15.39	3
23	44.07	3.55	-2.71	-2.41	44.04	8.06	3	2.82	128.77	1.4	0.01	1.38	30.85	8.33	0	3	1	0.77	2.92	30.85	-0.15	12.5	3
24	49.5	3.97	-2.61	-2.5	49.47	8.02	3	2.78	129.87	1.46	0.02	1.44	33.29	8.27	0	3	1	0.73	2.89	33.29	-0.14	12.54	3
25	48.14	3.76	-2.46	-2.58	48.11	7.81	3	2.78	129.41	1.53	0.02	1.5	31.02	8.07	0	3	1	0.7	2.9	31.02	-0.13	12.81	3
26	56.91	3.97	-2.32	-2.69	56.88	6.98	3	2.7	130.21	1.59	0.03	1.56	35.39	7.18	0	3	1	0.68	2.83	35.39	-0.13	14.01	3
27	50.54	3.76	-2.42	-2.68	50.51	7.44	3	2.75	129.53	1.66	0.03	1.62	30.12	7.69	0	3	1	0.65	2.9	30.12	-0.13	13.3	3
28	39.89	3.34	-2.38	-3.91	39.86	8.38	3	2.86	128.09	1.72	0.04	1.68	22.69	8.76	-0.01	3	1	0.63	3.02	22.69	-0.13	12.16	3
29	36.55	3.03	-2.32	-2.64	36.52	8.29	3	2.88	127.15	1.78	0.04	1.74	19.97	8.72	-0.01	3	1	0.61	3.06	19.97	-0.12	12.28	3
30	30.39	2.51	-2.42	-2.68	30.36	8.26	3	2.94	125.32	1.85	0.05	1.8	15.86	8.79	-0.01	3	1	0.59	3.14	15.86	-0.12	12.35	3
31	23.6	1.98	-2.32	-2.82	23.57	8.42	3	3.02	122.99	1.91	0.05	1.85	11.68	9.16	-0.01	3	1	0.57	3.25	11.68	-0.12	12.25	3
32	27.15	1.78	-2.42	-2.7	27.12	6.55	3	2.9	122.52	1.97	0.06	1.91	13.16	7.06	-0.01	3	1	0.55	3.13	13.16	-0.12	14.22	3
33	27.78	2.19	-2.42	-2.74	27.75	7.9	3	2.95	124.12	2.03	0.06	1.97	13.07	8.53	-0.01	3	1	0.54	3.19	13.07	-0.12	12.71	3
34	31.95	2.4	-2.32	-2.8	31.93	7.52	3	2.89	125.13	2.09	0.07	2.03	14.73	8.05	-0.01	3	1	0.52	3.13	14.73	-0.12	13.11	3
35	26.52	1.98	-3.3	-2.87	26.48	7.49	3	2.95	123.28	2.16	0.07	2.08	11.68	8.16	-0.01	3	1	0.51	3.21	11.68	-0.15	13.12	3
36	28.93	1.98	-3.09	-96.34	28.89	6.87	3	2.89	123.49	2.22	0.08	2.14	12.47	7.44	-0.01	3	1	0.49	3.16	12.47	-0.14	13.81	3
37	41.04	1.88	-3	-104.3	41	4.58	4	2.66	123.95	2.28	0.08	2.2	17.63	4.85	-0.01	3	1	0.48	2.93	17.63	-0.14	17.76	3
38	19.32	1.15	-2.86	-109.7	19.28	5.96	3	2.98	118.5	2.34	0.09	2.25	7.53	6.78	-0.02	3	1	0.47	3.31	7.53	-0.13	14.48	2
39	21.93	0.84	-2.71	-111.3	21.9	3.82	3	2.81	116.48	2.4	0.09	2.3	8.46	4.28	-0.01	3	1	0.46	3.15	8.46	-0.13	17.38	2
40	21.41	1.04	-2.61	-111.6	21.38	4.89	3	2.89	118.06	2.46	0.1	2.36	8.02	5.52	-0.02	3	1	0.45	3.23	8.02	-0.12	15.77	2
41	24.54	1.15	-2.42	-111.8	24.51	4.69	3	2.83	119.09	2.52	0.1	2.41	9.12	5.22	-0.01	3	1	0.44	3.17	9.12	-0.11	16.25	2
42	21.83	0.94	-2.32	-111.8	21.8	4.31	3	2.84	117.33	2.57	0.11	2.47	7.79	4.89	-0.01	3	1	0.43	3.21	7.79	-0.11	16.46	2
43	23.08	0.94	-2.22	-111.9	23.05	4.08	3	2.81	117.47	2.63	0.11	2.52	8.1	4.6	-0.01	3	1	0.42	3.18	8.1	-0.11	16.87	2
44	26.11	1.04	-2.22	-111.9	26.08	4	4	2.76	118.54	2.69	0.12	2.57	9.08	4.47	-0.01	3	1	0.41	3.13	9.08	-0.11	17.26	2
45	25.06	0.94	-2.13	-112	25.04	3.75	4	2.76	117.67	2.75	0.12	2.63	8.48	4.22	-0.01	3	1	0.4	3.14	8.48	-0.1	17.47	2
46	24.12	0.84	-2.03	-112.1	24.1	3.47	4	2.75	116.72	2.81	0.13	2.68	7.94	3.92	-0.01	3	1	0.39	3.15	7.94	-0.1	17.73	2
47	25.9	0.84	-1.93	-112.2	25.87	3.23	4	2.71	116.89	2.87	0.13	2.74	8.41	3.63	-0.01	3	1	0.39	3.11	8.41	-0.1	18.31	2
48	23.6	0.84	-1.84	-112.2	23.58	3.54	4	2.76	116.66	2.93	0.14	2.79	7.41	4.05	-0.01	3	1	0.38	3.18	7.41	-0.1	17.41	2
49	26.84	0.94	-1.64	-112.5	26.82	3.5	4	2.72	117.84	2.98	0.14	2.84	8.38	3.94	-0.01	3	1	0.37	3.13	8.38	-0.09	17.84	2
50	28.61	0.94	-1.48	-112.9	28.59	3.29	4	2.68	118	3.04	0.15	2.9	8.82	3.68	-0.01	3	1	0.37	3.09	8.82	-0.09	18.37	2
51	34.88	1.36	-1.35	-113.2	34.86	3.89	4	2.66	121.17	3.1	0.15	2.95	10.76	4.27	-0.01	3	1	0.36	3.06	10.76	-0.08	17.9	2
52	35.71	2.3	-1.26	-113.4	35.7	6.44	3	2.81	125.08	3.17	0.16	3.01	10.81	7.06	-0.01	3	1	0.35	3.2	10.81	-0.08	14.22	2
53	51.17	3.34	-1.16	-113.5	51.16	6.53	3	2.71	128.7	3.23	0.16	3.07	15.61	6.97	-0.01	3	1	0.34	3.07	15.61	-0.08	14.32	3
54	47.1	3.55	-1	-113.6	47.08	7.54	3	2.78	128.94	3.3	0.17	3.13	13.99	8.11	-0.01	3	1	0.34	3.15	13.99	-0.08	13.08	3
55	42.71	3.55	-0.87	-113.6	42.7	8.32	3	2.84	128.7	3.36	0.17	3.19	12.34	9.03	-0.01	3	1	0.33	3.22	12.34	-0.07	12.32	3
56	48.66	3.86	-0.76	-113.7	48.65	7.94	3	2.79	129.64	3.43	0.18	3.25	13.92	8.54	-0.01	3	1	0.33	3.17	13.92	-0.07	12.66	3
57	54.2	3.76	-0.68	-113.7	54.19	6.94	3	2.71	129.7	3.49	0.18	3.31	15.32	7.42	0	3	1	0.32	3.1	15.32	-0.07	13.79	3
58	42.5	3.13	-0.77	-113.8	42.49	7.37	3	2.8	127.77	3.55	0.19	3.37	11.56	8.05	-0.01	3	1	0.31	3.21	11.56	-0.07	13.23	3
59	40.62	2.72	-0.57	-113.9	40.62	6.68	3	2.78	126.61	3.62	0.19	3.43	10.8	7.34	-0.01	3	1	0.31	3.21	10.8	-0.07	13.94	2
60	34.88	0	-0.48	-113.9	34.87	0	0	0	87.36	3.66	0.2	3.46	9.01	0	-0.01	0	1	0.31	4.06	9.01	-0.07	27.16	0

	CPT-5	In situ data									Basic output data												
Depth (ft)	qc (tsf)	fs (tsf)	u (psi)	Other	qt (tsf)	Rf(%)	SBT	Ic SBT	ã (pcf)	ó,v (tsf)	u0 (tsf)	ó',vo (tsf)	Qt1	Fr (%)	Bq	SBTn	n	Cn	Ic	Qtn	U2	I(B)	Mod. SBTn
1	172.51	3.34	0.39	-0.04	172.52	1.94	6	1.96	131.66	0.07	0	0.07	2617.9	1.94	0	8	0.48	3.82	1.65	622.89	0.42	49.56	7
2	69.13	0.94	0	-0.04	69.13	1.36	5	2.14	120.15	0.13	0	0.13	547.71	1.36	0	6	0.54	3.13	1.78	204.32	0	61.54	7
3	50.13	0.31	0	-0.06	50.13	0.63	6	2.06	111.33	0.18	0	0.18	275.16	0.63	0	6	0.52	2.5	1.73	117.9	0	88.85	7
4	26.94	0.21	-0.27	-0.06	26.94	0.78	5	2.34	106.85	0.23	0	0.23	113.65	0.78	0	6	0.62	2.56	2	64.5	-0.08	61.85	7
5	19.32	0.31	0.66	-0.06	19.33	1.62	4	2.63	109	0.29	0	0.29	65.76	1.65	0	5	0.74	2.61	2.3	46.9	0.17	38.66	7
6	67.67	0.84	0	-0.06	67.67	1.23	5	2.12	119.24	0.35	0	0.35	192.81	1.24	0	6	0.59	1.93	1.91	122.62	0	59.69	7
7	85	1.15	-0.29	-0.06	85	1.35	5	2.07	122.12	0.41	0	0.41	206.19	1.36	0	6	0.59	1.75	1.89	139.83	-0.05	57.65	7
8	74.46	0.94	-0.39	-0.06	74.45	1.26	5	2.09	120.33	0.47	0	0.47	157.32	1.27	0	6	0.61	1.64	1.93	114.57	-0.06	57.79	7
9	79.16	1.15	-0.39	-0.06	79.15	1.45	5	2.11	121.95	0.53	0	0.53	147.98	1.46	0	6	0.63	1.54	1.98	114.54	-0.05	52.47	7
10	64.54	1.15	-0.48	-0.08	64.53	1.78	5	2.24	121.45	0.59	0	0.59	108	1.8	0	5	0.68	1.49	2.12	89.88	-0.06	43.15	7
11	71.85	0.73	-0.29	-0.08	71.84	1.02	6	2.05	118.4	0.65	0	0.65	109.31	1.03	0	6	0.62	1.35	1.95	91.05	-0.03	61.81	7
12	62.97	1.15	-0.29	-0.08	62.97	1.82	5	2.25	121.39	0.71	0	0.71	87.43	1.85	0	5	0.71	1.32	2.17	77.93	-0.03	41.13	7
13	61.72	2.61	-0.48	-0.08	61.71	4.23	4	2.51	127.35	0.78	0	0.78	78.57	4.28	0	4	0.82	1.29	2.45	74.28	-0.04	21.71	3
14	57.33	4.49	-3.11	-0.1	57.29	7.84	3	2.74	131.13	0.84	0	0.84	67.11	7.95	0	9	0.91	1.23	2.69	65.8	-0.27	12.77	3
15	74.77	5.74	-5.51	-0.1	74.7	7.69	9	2.66	133.58	0.91	0	0.91	81.27	7.78	-0.01	9	0.89	1.15	2.63	79.97	-0.44	12.99	3
16	76.96	5.64	-5.7	-0.12	76.89	7.33	9	2.63	133.52	0.97	0	0.97	77.88	7.43	-0.01	9	0.9	1.08	2.62	77.22	-0.42	13.55	3
17	60.78	5.01	-5.71	-0.12	60.71	8.26	9	2.74	132.08	1.04	0	1.04	57.32	8.4	-0.01	9	0.94	1.02	2.74	57.27	-0.39	12.21	3
18	48.87	4.07	-5.8	-0.12	48.8	8.35	3	2.8	130.03	1.11	0	1.11	43.14	8.54	-0.01	3	0.98	0.96	2.83	43.17	-0.38	12.12	3
19	35.09	2.82	-5.65	-0.12	35.02	8.05	3	2.88	126.53	1.17	0	1.17	28.96	8.33	-0.01	3	1	0.91	2.94	28.96	-0.35	12.52	3
20	24.64	2.09	-5.7	-0.1	24.57	8.5	3	3.01	123.47	1.23	0	1.23	19.04	8.95	-0.02	3	1	0.86	3.08	19.04	-0.34	12.08	3
21	16.6	1.25	-5.7	-0.08	16.53	7.58	3	3.1	118.76	1.29	0.01	1.28	11.91	8.22	-0.03	3	1	0.83	3.21	11.91	-0.33	13.05	3
22	12.95	0.84	-5.7	-0.06	12.88	6.49	3	3.13	115.19	1.35	0.01	1.33	8.65	7.24	-0.04	3	1	0.79	3.28	8.65	-0.32	14.06	2
23	28.51	1.46	-5.47	-0.08	28.44	5.14	3	2.81	121.22	1.41	0.02	1.39	19.47	5.41	-0.02	3	1	0.76	2.93	19.47	-0.3	16.81	3
24	40.52	2.61	-5.32	-0.08	40.45	6.45	3	2.77	126.32	1.47	0.02	1.45	26.94	6.7	-0.01	3	1	0.73	2.89	26.94	-0.28	14.75	3
25	54.93	3.86	-4.93	-0.08	54.87	7.04	3	2.71	129.93	1.54	0.03	1.51	35.39	7.24	-0.01	3	1	0.7	2.83	35.39	-0.26	13.91	3
26	57.33	4.59	-4.68	-0.1	57.27	8.02	3	2.74	131.3	1.6	0.03	1.57	35.51	8.25	-0.01	3	1	0.67	2.87	35.51	-0.24	12.53	3
27	56.91	4.8	-4.54	-0.12	56.86	8.45	3	2.76	131.61	1.67	0.04	1.63	33.88	8.7	-0.01	3	1	0.65	2.9	33.88	-0.22	12.03	3
28	64.64	4.49	-4.45	-0.12	64.59	6.95	9	2.66	131.43	1.73	0.04	1.69	37.2	7.14	-0.01	3	1	0.63	2.81	37.2	-0.22	14.06	3
29	52.94	4.28	-4.35	-0.15	52.89	8.09	3	2.77	130.59	1.8	0.05	1.75	29.2	8.38	-0.01	3	1	0.6	2.93	29.2	-0.21	12.46	3
30	46.68	4.49	-4.25	-0.15	46.63	9.63	3	2.86	130.63	1.86	0.05	1.81	24.73	10.03	-0.01	3	1	0.58	3.04	24.73	-0.2	10.92	3
31	52.42	4.18	-4.16	-0.13	52.37	7.98	3	2.77	130.39	1.93	0.06	1.87	26.97	8.28	-0.01	3	1	0.57	2.95	26.97	-0.19	12.6	3
32	54.41	3.45	-4.06	-0.12	54.36	6.34	3	2.68	129.07	1.99	0.06	1.93	27.13	6.58	-0.01	3	1	0.55	2.88	27.13	-0.18	14.94	3
33	52	3.65	-4.16	-0.12	51.95	7.04	3	2.73	129.39	2.06	0.07	1.99	25.07	7.33	-0.01	3	1	0.53	2.94	25.07	-0.18	13.83	3
34	34.25	3.34	-4.16	-0.12	34.2	9.77	3	2.95	127.71	2.12	0.07	2.05	15.66	10.42	-0.01	3	1	0.52	3.19	15.66	-0.18	11.01	2
35	30.39	2.72	-4.16	-0.1	30.34	8.95	3	2.96	125.9	2.19	0.08	2.11	13.36	9.64	-0.01	3	1	0.5	3.22	13.36	-0.18	11.75	3
36	23.08	2.3	-4.25	-0.08	23.03	9.98	3	3.08	124.01	2.25	0.08	2.16	9.6	11.06	-0.02	3	1	0.49	3.36	9.6	-0.18	11.13	1
37	19.84	2.19	-4.25	-0.08	19.79	11.08	3	3.16	123.3	2.31	0.09	2.22	7.87	12.55	-0.02	2	1	0.48	3.46	7.87	-0.18	10.59	1
38	24.33	2.51	-4.35	-0.08	24.28	10.32	3	3.07	124.77	2.37	0.09	2.28	9.62	11.44	-0.02	3	1	0.46	3.37	9.62	-0.18	10.9	1
39	23.81	1.57	-4.16	-0.08	23.76	6.59	3	2.94	121.28	2.43	0.1	2.33	9.14	7.34	-0.02	3	1	0.45	3.26	9.14	-0.17	13.96	2
40	24.54	1.46	-4.16	-0.09	24.49	5.97	3	2.9	120.85	2.49	0.1	2.39	9.21	6.65	-0.02	3	1	0.44	3.23	9.21	-0.17	14.64	2
41	18.8	0.94	-4.01	-0.1	18.75	5.01	3	2.94	116.97	2.55	0.11	2.44	6.63	5.8	-0.02	3	1	0.43	3.31	6.63	-0.16	15.33	2
42	14.52	0.63	-3.87	-0.12	14.47	4.33	3	2.98	113.37	2.61	0.11	2.5	4.75	5.28	-0.03	3	1	0.42	3.4	4.75	-0.16	15.51	2
43	17.13	0.63	-3.82	-0.14	17.08	3.67	3	2.88	113.77	2.66	0.12	2.55	5.66	4.35	-0.03	3	1	0.42	3.29	5.66	-0.15	16.55	2
44	21.51	0.84	-3.68	-0.17	21.47	3.89	3	2.82	116.43	2.72	0.12	2.6	7.21	4.46	-0.02	3	1	0.41	3.21	7.21	-0.15	16.85	2
45	17.13	0.63	-3.58	-0.17	17.08	3.67	3	2.88	113.77	2.78	0.13	2.65	5.39	4.38	-0.03	3	1	0.4	3.31	5.39	-0.15	16.44	2
46	18.27	0.63	-3.51	-0.18	18.23	3.44	3	2.84	113.93	2.84	0.13	2.7	5.69	4.07	-0.03	3	1	0.39	3.27	5.69	-0.14	16.84	2
47	20.47	0.73	-3.35	-0.24	20.43	3.58	3	2.81	115.34	2.89	0.14	2.76	6.36	4.17	-0.02	3	1	0.38	3.24	6.36	-0.14	16.95	2
48	22.03	0.73	-3.29	-0.25	21.99	3.32	4	2.77	115.52	2.95	0.14	2.81	6.78	3.84	-0.02	3	1	0.38	3.2	6.78	-0.13	17.47	2
49	21.2	0.73	-3.19	-0.27	21.16	3.45	4	2.79	115.42	3.01	0.15	2.86	6.34	4.03	-0.02	3	1	0.37	3.23	6.34	-0.13	17.1	2
50	21.83	0.84	-3	-0.29	21.79	3.83	3	2.81	116.47	3.07	0.15	2.92	6.42	4.46	-0.02	3	1	0.36	3.25	6.42	-0.13	16.64	2
51	22.24	0.84	-2.9	-0.31	22.21	3.76	3	2.8	116.52	3.13	0.16	2.97	6.43	4.38	-0.02	3	1	0.36	3.25	6.43	-0.12	16.74	2
52	22.87	0.84	-2.77	-0.31	22.84	3.66	4	2.78	116.59	3.18	0.16	3.02	6.5	4.25	-0.02	3	1	0.35	3.24	6.5	-0.12	16.9	2
53	27.88	1.04	-2.66	-0.33	27.85	3.75	4	2.72	118.7	3.24	0.17	3.08	8	4.24	-0.01	3	1	0.34	3.16	8	-0.12	17.31	2
54	19.53	0.94	-2.51	-0.35	19.5	4.82	3	2.91	117.06	3.3	0.17	3.13	5.17	5.8	-0.02	3	1	0.34	3.4	5.17	-0.11	15.17	2
55	20.15	0.94	-2.42	-0.35	20.12	4.67	3	2.89	117.14	3.36	0.18	3.18	5.26	5.61	-0.02	3	1	0.33	3.38	5.26	-0.11	15.34	2
56	26.94	1.15	-2.32	-0.37	26.91	4.27	3	2.77	119.32	3.42	0.18	3.24	7.25	4.89	-0.01	3	1	0.33	3.23	7.25	-0.11	16.36	2
57	26	1.25	-2.22	-0.39	25.98	4.82	3	2.82	119.87	3.48	0.19	3.29	6.83	5.57	-0.02	3	1	0.32	3.29	6.83	-0.11	15.58	2
58	31.01	1.36	-2.13	-0.42	30.99	4.38	4	2.73	120.88	3.54	0.19	3.35	8.19	4.95	-0.01	3	1	0.32	3.19	8.19	-0.1	16.46	2
59	32.37	1.57	-1.93	-0.44	32.35	4.84	3	2.75	122.03	3.6	0.2	3.41	8.44	5.45	-0.01	3	1	0.31	3.21	8.44	-0.1	15.9	2
60	34.36	0	-1.84	-0.44	34.33	0	0	0	87.36	3.65	0.2	3.44	8.91	0	-0.01	0	1	0.31	4.06	8.91	-0.1	27.01	0

Presented below is a list of formulas used for the estimation of various soil properties. The formulas are presented in SI unit system and assume that all components are expressed in the same units.

:: Unit Weight, g (kN/m³) ::

$$g = g_w \cdot \left(0.27 \cdot \log(R_f) + 0.36 \cdot \log(\frac{q_t}{p_a}) + 1.236 \right)$$

where g_w = water unit weight

:: Permeability, k (m/s) ::

$I_c < 3.27$ and $I_c > 1.00$ then $k = 10^{0.952-3.04 \cdot I_c}$

$I_c \le 4.00$ and $I_c > 3.27$ then $k = 10^{-4.52-1.37 \cdot I_c}$

:: N$_{SPT}$ (blows per 30 cm) ::

$$N_{60} = \left(\frac{q_c}{P_a} \right) \cdot \frac{1}{10^{1.1268-0.2817 \cdot I_c}}$$

$$N_{1(60)} = Q_{tn} \cdot \frac{1}{10^{1.1268-0.2817 \cdot I_c}}$$

:: Young's Modulus, Es (MPa) ::

$$(q_t - \sigma_v) \cdot 0.015 \cdot 10^{0.55 \cdot I_c + 1.68}$$

(applicable only to $I_c < I_{c\_cutoff}$)

:: Relative Density, Dr (%) ::

$$100 \cdot \sqrt{\frac{Q_{tn}}{k_{DR}}}$$

(applicable only to SBT$_n$: 5, 6, 7 and 8 or $I_c < I_{c\_cutoff}$)

:: State Parameter, ψ ::

$$\psi = 0.56 - 0.33 \cdot \log(Q_{tn,cs})$$

:: Peak drained friction angle, φ (°) ::

$$\varphi = 17.60 + 11 \cdot \log(Q_{tn})$$

(applicable only to SBT$_n$: 5, 6, 7 and 8)

:: 1-D constrained modulus, M (MPa) ::

If $I_c > 2.20$

$\alpha = 14$ for $Q_{tn} > 14$

$\alpha = Q_{tn}$ for $Q_{tn} \le 14$

$M_{CPT} = \alpha \cdot (q_t - \sigma_v)$

If $I_c \le 2.20$

$M_{CPT} = (q_t - \sigma_v) \cdot 0.0188 \cdot 10^{0.55 \cdot I_c - 1.68}$

:: Small strain shear Modulus, Go (MPa) ::

$$G_0 = (q_t - \sigma_v) \cdot 0.0188 \cdot 10^{0.55 \cdot I_c - 1.68}$$

:: Shear Wave Velocity, Vs (m/s) ::

$$V_s = \left(\frac{G_0}{\rho} \right)^{0.50}$$

:: Undrained peak shear strength, Su (kPa) ::

$N_{kt} = 10.50 + 7 \cdot \log(F_r)$ or user defined

$$S_u = \frac{(q_t - \sigma_v)}{N_{kt}}$$

(applicable only to SBT$_n$: 1, 2, 3, 4 and 9 or $I_c > I_{c\_cutoff}$)

:: Remolded undrained shear strength, Su(rem) (kPa) ::

$S_{u(rem)} = f_s$ (applicable only to SBT$_n$: 1, 2, 3, 4 and 9 or $I_c > I_{c\_cutoff}$)

:: Overconsolidation Ratio, OCR ::

$$k_{OCR} = \left[\frac{Q_{tn}^{0.20}}{0.25 \cdot (10.50 + 7 \cdot \log(F_r))} \right]^{1.25}$$ or user defined

$OCR = k_{OCR} \cdot Q_{tn}$

(applicable only to SBT$_n$: 1, 2, 3, 4 and 9 or $I_c > I_{c\_cutoff}$)

:: In situ Stress Ratio, Ko ::

$$K_0 = (1 - \sin\varphi') \cdot OCR^{\sin\varphi'}$$

(applicable only to SBT$_n$: 1, 2, 3, 4 and 9 or $I_c > I_{c\_cutoff}$)

:: Soil Sensitivity, S$_t$ ::

$$S_t = \frac{N_S}{F_r}$$

(applicable only to SBT$_n$: 1, 2, 3, 4 and 9 or $I_c > I_{c\_cutoff}$)

:: Effective Stress Friction Angle, φ' (°) ::

$$\varphi' = 29.5° \cdot B_q^{0.121} \cdot \left(0.256 + 0.336 \cdot B_q + \log Q_t\right)$$

(applicable for $0.10 < B_q < 1.00$)

References

- Robertson, P.K., Cabal K.L., Guide to Cone Penetration Testing for Geotechnical Engineering, Gregg Drilling & Testing, Inc., 5th Edition, November 2012
- Robertson, P.K., Interpretation of Cone Penetration Tests - a unified approach., Can. Geotech. J. 46(11): 1337–1355 (2009)