Appendix IS-2

Geotechnical Reports

Appendix IS-2.1

Geotechnical Investigation

GEOTECHNICAL INVESTIGATION

PROPOSED HIGH-RISE DEVELOPMENT
1708-1732 NORTH CAHUENGA BOULEVARD
6381-6385 WEST HOLLYWOOD BOULEVARD
LOS ANGELES, CALIFORNIA

TRACT: HOLLYWOOD, BLOCK: 20, LOTS 8-12

GEOCON
WEST, INC.

GEOTECHNICAL
ENVIRONMENTAL
MATERIALS

PREPARED FOR

ONNI CONTRACTING (CALIFORNIA) INC.
LOS ANGELES, CALIFORNIA

PROJECT NO. W1063-06-01

OCTOBER 17, 2019

GEOCON
WEST, INC.
GEOTECHNICAL ■ ENVIRONMENTAL ■ MATERIALS

Project No. W1063-06-01
October 17, 2019

Mr. Canyon Law
ONNI Contracting (California) Inc.
315 West 9th Street, Suite 801
Los Angeles, California 90015

Subject: GEOTECHNICAL INVESTIGATION
PROPOSED HIGH-RISE DEVELOPMENT
1708-1732 NORTH CAHUENGA BOULEVARD
6381-6385 WEST HOLLYWOOD BOULEVARD
LOS ANGELES, CALIFORNIA
TRACT: HOLLYWOOD, BLOCK: 20, LOTS 8-12

Dear Mr. Law:

In accordance with your authorization of our revised proposal dated September 6, 2019, we have performed a geotechnical investigation for the proposed high-rise development located at 1708-1732 North Cahuenga Boulevard and 6381-6385 West Hollywood Boulevard in the City of Los Angeles, 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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GEOTECHNICAL INVESTIGATION

1. PURPOSE AND SCOPE

This report presents the results of a geotechnical investigation for the proposed high-rise development located at 1708-1732 North Cahuenga Boulevard and 6381-6385 West Hollywood Boulevard in the City of Los Angeles, 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. A surface fault rupture hazard investigation has been previously performed for the site by ENGEO; the results of the investigation are presented in a report dated January 23, 2015 (ENGEO, 2015). The results of the fault investigation indicate that the potential for Holocene-active faults to impact the proposed structures is considered low and no building setbacks were recommended. The surface fault rupture hazard investigation report was reviewed and approved by the City of Los Angeles Department of Building and Safety, Grading Division under Log Number 87442 dated March 26, 2015.

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 September 14, 2019, by drilling one 8-inch-diameter boring to a depth of 150½ feet below the existing ground surface utilizing a truck-mounted hollow-stem auger drilling machine. The location of the exploratory boring is depicted on the Site Plan (see Figure 2). A detailed discussion of the field investigation, including excavation 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 site is comprised of several rectangular-shaped parcels and is currently developed with a seven-story historical structure with a basement, a single-story commercial structure, and a 2-story mixed-use structure, along with adjacent asphalt-paved parking lots and small planter areas. The site is bounded by asphalt parking to the north, by North Cahuenga Boulevard to the west, by West Hollywood Boulevard to the south, and by single- to 3-story commercial structures to the east. The site is gently sloping to the south, with a 2- to 2½-foot-high retaining wall along the northern property line. Surface water drainage at the site appears to be by sheet flow along the existing ground contours to the city street, or to area drains along the existing buildings that lead to the street. Vegetation onsite consists of trees and grasses within small planter areas.

Based on the information provided by the Client, it is our understanding that the proposed development will include the demolition of the single-story and 2-story structures for the construction of a high-rise structure up to 180 feet in height. The proposed structure will be constructed either on-grade or underlain by up to seven levels of subterranean parking. Formal plans for the proposed structure are not available. The property limits and boring locations at the site are shown on the Site Plan (see Figure 2).

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 may be up to 3,500 kips, and that a bearing pressure of 7,500 psf may be required for support of the proposed tower.

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. BACKGROUND

A prior geotechnical investigation was performed by MACTEC Engineering and Consulting, Inc. (MACTEC) in 2007 for the purpose of retrofitting the existing historical structure at 6381 West Hollywood Boulevard, as well as the construction of an adjacent parking structure at 1716-1720 North Cahuenga Boulevard (MACTEC, 2007):

- *Report of Geotechnical Investigation, Proposed Building Conversion and Parking Structure, 6381 Hollywood Boulevard and 1716-1720 North Cahuenga Boulevard, Hollywood District of Los Angeles, California*, prepared by MACTEC Engineering and Consulting, Inc., dated June 21, 2007.

The 2007 geotechnical report included two (2) borings drilled to depths of 50 and 50½ feet below the existing surface grade. Laboratory testing of selected soil samples was performed.

As previously indicated, the site is located within a state-designated Alquist-Priolo Earthquake Fault Zone for surface fault rupture hazards associated with the nearby Hollywood Fault Zone. A site-specific surface fault rupture hazard investigation has been previously performed for the site by ENGEO (2015):

- *Fault Rupture Hazard Exploration, Security Pacific Bank Building, 6381 Hollywood Boulevard and 1716-1726 North Cahuenga Boulevard, Los Angeles, California*, prepared ENGEO Inc., dated January 23, 2015.

The 2015 fault rupture hazard evaluation consisted of twelve (12) Cone Penetrometer Tests (CPTs) and four (4) continuous-core hollow stem auger borings, advanced along a roughly north-south transect, extending from the alley between 6381 Hollywood Boulevard and 1716 North Cahuenga Boulevard, to the parking lot north of 1726 North Cahuenga Boulevard. The report by ENGEO also contains three boring logs from a 1984 investigation performed by Converse Consultants for an adjacent Metro Rail project. The report concludes that the site is not impacted by Holocene-active faults. The ENGEO report was reviewed and approved by the City of Los Angeles LADBS Grading Division and supplemental exploration to evaluate the potential for surface fault rupture at the site is not required. The surface fault rupture hazard investigation report was reviewed and approved by the City of Los Angeles Department of Building and Safety, Grading Division under Log Number 87442 dated March 26, 2015.

Geocon West, Inc. has reviewed the referenced reports and the recommendations presented herein are based on our analysis of the subsurface and laboratory data obtained as part of this investigation and the prior investigation at the site by MACTEC (2007) and ENGEO (2015). Furthermore, we assume responsibility for the utilization of the exploration and laboratory data presented within the geotechnical report by MACTEC and ENGEO. Geocon West, Inc. is the Geotechnical Consultant of Record and will be providing all necessary geotechnical consultation, plan review, design recommendations, inspection and testing services for this project. Where differing, the recommendations presented herein supersede all previous recommendations.

Copies of the prior reports for this site are included herein as Appendix C (CD only). A complete list of the documents reviewed as part of this study is presented in the *List of References* section of this report.

4. GEOLOGIC SETTING

The site is located on an alluvial fan surface near to the southern flank of the Santa Monica Mountains. The Santa Monica Mountains, formed during regional uplift, trend east-west on the north side of the Los Angeles Basin. The Santa Monica Mountains are a broad west-plunging anticline, the south flank of which is truncated by the Hollywood-Santa Monica Fault Zone. In the site vicinity, the Hollywood Fault Zone forms a structural separation between the mountain range and the alluviated Los Angeles Basin to the south. Rock types exposed in the Santa Monica Mountain, in the site vicinity, consist of Tertiary age sedimentary rocks and locally Tertiary age basaltic intrusions.

Regionally, the site is located in the northern portion of the Peninsular Ranges geomorphic province, near the boundary of the Transverse Ranges geomorphic province. The nearby Hollywood Fault, inferred approximately 230 feet north of the site, is part of the Santa Monica-Hollywood-Raymond fault system and acts as the boundary between the Peninsular Ranges geomorphic province to the south and the Transverse Ranges geomorphic province to the north. The Transverse Ranges province is characterized by east-west trending geologic structures and physiographic features in contrast to the Peninsular Ranges province that is characterized by northwest-trending geologic structures and physiographic features.

5. SOIL AND GEOLOGIC CONDITIONS

The geologic units encountered at the site consist of artificial fill and Quaternary age alluvium (CGS, 2012). The geologic units encountered at the site are summarized below and described in detail on the boring logs presented in Appendix A.

5.1 Artificial Fill

Artificial fill was encountered in our boring to a maximum depth of 3 feet below existing ground surface. Prior explorations at the site encountered fill up to approximately 8 feet beneath the existing ground surface. The artificial fill generally consists of reddish brown, fine- to medium-grained sand with silt. The artificial fill is characterized as moist and very loose, with concrete and brick fragments present. The fill is likely the result of past grading, construction or landscaping activities at the site. Deeper fill may exist between excavations and in other portions of the site that were not directly explored.

5.2 Alluvial Fan Deposits

Quaternary age alluvial fan deposits were observed below the fill to the maximum depth explored. The alluvium generally consists of varying shades of reddish brown, to brown, interbedded poorly graded and well-graded sand, silty sand, and sand with silt with some localized silt beds. Grain size varies from very fine to coarse, with trace to some gravel and cobbles. The soils are characterized as dry to saturated and very loose to very dense or firm. Although only minor silt interbeds were encountered in our boring, the prior borings and CPTs performed at the site indicate that fine-grained layers up to 5 feet thick are present below the site.

6. GROUNDWATER

Based on a review of the Seismic Hazard Zone Report for the Hollywood Quadrangle (California Division of Mines and Geology [CDMG], 1998), the historically highest groundwater level in the area is approximately 80 feet beneath the ground surface. Groundwater information presented in this document is generated from data collected in the early 1900’s to the late 1990s. Based on current groundwater basin management practices, it is unlikely that groundwater levels will ever exceed the historic high levels.

Groundwater was encountered in our boring at a depth of 102.7 feet, stabilizing at a depth of 106 feet below existing surface grade. Considering the historic high groundwater level, and the depth to groundwater encountered in our borings, and the depth of the proposed construction, it is unlikely that groundwater will be encountered during construction. However, 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 8.25).

7. GEOLOGIC HAZARDS

7.1 Surface Fault Rupture

The numerous faults in Southern California include active, potentially active, 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 (APEFZ) Program (CGS, 2018). By definition, an active fault is one that has had surface displacement within Holocene time (about the last 11,700 years). A potentially active 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 property is located within the boundaries of an APEFZ established for the Hollywood Fault (California Geological Survey, 2014). As previously indicated, a site-specific hazard rupture hazard investigation to evaluate the potential for surface fault rupture to impact the proposed structure was previously performed (ENGEO, 2015). Based on the results of the site-specific investigation, active or potentially active faults do not traverse the area of the proposed structure. Therefore, the potential for surface rupture due to faulting occurring beneath the proposed structure 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 3, Regional Fault Map.

The closest surface trace of an active fault to the site is the Hollywood Fault located approximately 230 feet to the north (CGS, 2014). Other nearby active faults are the Raymond Fault, Newport-Inglewood Fault Zone, and the Santa Monica Fault located approximately 4.6 miles east-northeast, 5.4 miles west-southwest, and 5.5 miles west of the site, respectively (Ziony and Jones, 1989). The active San Andreas Fault Zone is located approximately 32 miles northeast of the site.

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 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.

7.2 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 4, 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 table on the following page.

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	62	E
Long Beach	March 10, 1933	6.4	39	SE
Tehachapi	July 21, 1952	7.5	73	NW
San Fernando	February 9, 1971	6.6	22	NNW
Whittier Narrows	October 1, 1987	5.9	15	E
Sierra Madre	June 28, 1991	5.8	22	ENE
Landers	June 28, 1992	7.3	108	E
Big Bear	June 28, 1992	6.4	86	E
Northridge	January 17, 1994	6.7	14	WNW
Hector Mine	October 16, 1999	7.1	122	ENE
Ridgecrest	July 5, 2019	7.1	122	NNE

The site is located in a seismically active area. The main trace of Hollywood Fault has been inferred less than several hundred feet north of the site and a future earthquake originating on this fault could produce very strong near-field ground motions at the site. However, 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.

7.3 Seismic Design Criteria

The table on the following page summarizes summarizes site-specific design criteria obtained from the 2016 California Building Code (CBC; Based on the 2015 International Building Code [IBC] and ASCE 7-10), Chapter 16 Structural Design, Section 1613 Earthquake Loads. The data was calculated using the computer program *U.S. Seismic Design Maps*, provided by the USGS. The short spectral response uses a period of 0.2 second. We evaluated the Site Class based on the discussion in Section 1613.3.2 of the 2016 CBC and Table 20.3-1 of ASCE 7-10. The values presented below are for the risk-targeted maximum considered earthquake (MCE_R).

2016 CBC SEISMIC DESIGN PARAMETERS

Parameter	Value	2016 CBC Reference
Site Class	D	Section 1613.3.2
MCE_R Ground Motion Spectral Response Acceleration – Class B (short), S_S	2.543g	Figure 1613.3.1(1)
MCE_R Ground Motion Spectral Response Acceleration – Class B (1 sec), S_1	0.946g	Figure 1613.3.1(2)
Site Coefficient, F_A	1.0	Table 1613.3.3(1)
Site Coefficient, F_V	1.5	Table 1613.3.3(2)
Site Class Modified MCE_R Spectral Response Acceleration (short), S_{MS}	2.543g	Section 1613.3.3 (Eqn 16-37)
Site Class Modified MCE_R Spectral Response Acceleration – (1 sec), S_{M1}	1.419g	Section 1613.3.3 (Eqn 16-38)
5% Damped Design Spectral Response Acceleration (short), S_{DS}	1.695g	Section 1613.3.4 (Eqn 16-39)
5% Damped Design Spectral Response Acceleration (1 sec), S_{D1}	0.946g	Section 1613.3.4 (Eqn 16-40)

The table below presents the mapped maximum considered geometric mean (MCE_G) seismic design parameters for projects located in Seismic Design Categories of D through F in accordance with ASCE 7-10.

ASCE 7-10 PEAK GROUND ACCELERATION

Parameter	Value	ASCE 7-10 Reference
Mapped MCE_G Peak Ground Acceleration, PGA	0.991g	Figure 22-7
Site Coefficient, F_{PGA}	1.0	Table 11.8-1
Site Class Modified MCE_G Peak Ground Acceleration, PGA_M	0.991g	Section 11.8.3 (Eqn 11.8-1)

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 2016 California Building Code and ASCE 7-10, 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, 2008 Conterminous U.S. Dynamic edition. The result of the deaggregation analysis indicates that the predominant earthquake contributing to the MCE peak ground acceleration is characterized as a 6.67 magnitude event occurring at a hypocentral distance of 4.17 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 6.66 magnitude occurring at a hypocentral distance of 8.33 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.

7.4 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 Hollywood Quadrangle (CGS, 2014) indicates that the site is not located within an area designated as having a potential for liquefaction. The historic high groundwater beneath the site is estimated to be approximately 80 feet beneath the site, and the soils encountered at and below the depth are generally dense to very dense. Based on these considerations, it is our opinion that the potential for liquefaction and associated ground deformations beneath the site is very low.

7.5 Slope Stability

The topography at the site is relatively level and the topography in the immediate vicinity slopes gently to the south. The property is located within a City of Los Angeles Hillside Grading Area but is not located within a City of Los Angeles Hillside Ordinance Area (City of Los Angeles, 2019). The County of Los Angeles Safety Element (Leighton, 1990), indicates the site is not within a "Hillside Area" or an area identified as having a potential for slope instability. Also, the site is not within an area identified as having a potential for seismic slope instability (CGS, 2014). 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.

7.6 Earthquake-Induced Flooding

Earthquake-induced flooding is inundation caused by failure of dams or other water-retaining structures due to earthquakes. The Los Angeles County Safety Element (Leighton, 1990) indicates that the site is located within the Mulholland Dam inundation area. However, this reservoir, as well as others in California, are continually monitored by various governmental agencies (such as the State of California Division of Safety of Dams and the U.S. Army Corps of Engineers) to guard against the threat of dam failure. Current design, construction practices, and ongoing programs of review, modification, or total reconstruction of existing dams are intended to ensure that all dams are capable of withstanding the maximum considered earthquake (MCE) for the site. Therefore, the potential for inundation at the site as a result of an earthquake-induced dam failure is considered low.

7.7 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 resulting from a seismically-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, 2019; LACDPW, 2019).

7.8 Oil Fields & Methane Potential

Based on a review of the California Division of Oil, Gas and Geothermal Resources (DOGGR) Well Finder Website, the site is not located within the limits of an oilfield and oil or gas wells are not located in the immediate site vicinity (DOGGR, 2019). However, 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 during construction will need to be properly abandoned in accordance with the current requirements of the DOGGR.

The site is not located within the boundaries of a city-designated Methane Zone or Methane Buffer Zone (City of Los Angeles, 2019). 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.

7.9 Subsidence

Subsidence occurs when a large portion of land is displaced vertically, usually due to the withdrawal of groundwater, oil, or natural gas. Soils that are particularly subject to subsidence include those with high silt or clay content. The site is not located within an area of known ground subsidence. No large-scale extraction of groundwater, gas, oil, or geothermal energy is occurring or planned at the site or in the general site vicinity. There appears to be little or no potential for ground subsidence due to withdrawal of fluids or gases at the site.

8. CONCLUSIONS AND RECOMMENDATIONS

8.1 General

8.1.1 It is our opinion that neither soil nor geologic conditions were encountered during the investigation that would preclude the construction of the proposed development provided the recommendations presented herein are followed and implemented during design and construction. The geotechnical design parameters presented herein should be reviewed and updated once subterranean elevations and structural loads are established.

8.1.2 Up to 8 feet of existing artificial fill was encountered during the current and prior site investigations. 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. Future demolition of the existing structures which occupy the site will likely disturb the upper few feet of soil below those existing improvements. It is our opinion that the existing fill, in its present condition, are not suitable for direct support of proposed foundations or slabs. 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 8.4).

8.1.3 Due to the preliminary nature of the project at this time, development plans depicting the proposed structure, including the extents of the subterranean levels, are not available. We understand that the proposed tower will be up to 180 feet in height and will be constructed either on-grade or underlain by subterranean parking. The levels of subterranean parking may vary from none to seven levels. Once proposed building loads become available and elevations are established, additional analyses will be required to evaluate the anticipated total and differential settlements between the foundation elements for verification that the settlements are in conformance with the City of Los Angeles policy. Updated foundation design recommendations will be provided as necessary in an addendum report.

8.1.4 The soils underlying the site generally consist of granular alluvial deposits; some silt and clay interbeds up to 5 feet thick were observed. The alluvial soils to a depth of approximately 20 feet in depth are described as very loose and highly to moderately compressible. Although the alluvial soils increase in density and decrease in compressibility with depth, the alluvial soils generally remain medium dense and moderately to slightly compressible to the total depth explored (approximately 150 feet).

8.1.5 Based on these considerations and the loads anticipated for a 180-foot tower, it is recommended that where the lowest level of the proposed structure is located between the ground surface and a depth of 50 feet, the proposed structure be supported on a deepened foundation system deriving support in the competent alluvial soils. Recommendations for the design and construction of drilled cast-in-place friction piles are provided in Section 8.7.

8.1.6 Where the proposed project incorporates subterranean levels which will be embedded more than 50 feet below the ground surface, it is recommended that a reinforced concrete mat foundation system be used for support of the proposed structure. Recommendations for the design of a mat foundation are provided in Section 8.9.

8.1.7 The historic high groundwater level is reported at a depth of 80 feet below the ground surface. Based on the depth of proposed construction, groundwater is neither expected to be encountered during construction, nor have a detrimental effect on the project.

8.1.8 Due to the depth of the excavation and the proximity to the property lines, city streets and adjacent offsite structures, excavations will require sloping and/or shoring measures 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 *Temporary Excavations* are provided in Section 8.17 of this report.

8.1.9 Due to the nature of the proposed design and intent for subterranean levels, waterproofing of subterranean walls and slabs is recommended. 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.

8.1.10 Based on the results of the percolation testing performed at the site, a stormwater infiltration system is considered feasible for this project. A discussion of the test results is provided in the *Stormwater Infiltration* section of this report (see Section 8.24).

8.1.11 Any changes in the design, location or elevation, as outlined in this report, should be reviewed by this office. Once the foundation loading configuration and design elevations for the existing and proposed structures proceeds to a more finalized plan, the recommendations within this report should be reviewed and revised, as necessary. Based on the final foundation loading configurations and building elevations, the potential for settlement should be reevaluated by this office.

8.2 Soil and Excavation Characteristics

8.2.1 The in-situ soils can be excavated with moderate effort using conventional excavation equipment. Caving should be anticipated in unshored excavations, especially where granular soils are encountered.

8.2.2 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.

8.2.3 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 and shoring. Excavation recommendations are provided in the *Temporary Excavations* section of this report (see Section 8.17).

8.2.4 Based on depth of the proposed subterranean levels, the proposed structure would not be prone to the effects of expansive soils.

8.3 Minimum Resistivity, pH, and Water-Soluble Sulfate

8.3.1 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 “moderately corrosive” with respect to corrosion of buried ferrous metals on site. The results are presented in Appendix B (Figure B15) and should be considered for design of underground structures.

8.3.2 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 B15) and indicate that the on-site materials possess “negligible” sulfate exposure to concrete structures as defined by 2016 CBC Section 1904 and ACI 318-11 Sections 4.2 and 4.3.

8.3.3 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.

8.4 Grading

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

8.4.2 Earthwork should be observed, and compacted fill tested by representatives of Geocon West, Inc. The existing fill and alluvial soil 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 are removed.

8.4.3 Grading should commence with the removal of all existing vegetation and existing improvements from the area to be graded. 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 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. 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.) and the City of Los Angeles Inspector.

8.4.4 Where the lowest level of the proposed structure is located between the ground surface and a depth of 50 feet, it is recommended that the proposed structure be supported on a deepened foundation system deriving support in the competent alluvial soils. Recommendations for the design and construction of drilled cast-in-place friction piles are provided in Sections 8.7 and 8.8.

8.4.5 Provided that all existing artificial fill is properly excavated and compacted, a conventional slab-on-grade may be used. The client and contractor should be aware that excavations on the order of 8 feet in depth may be required to completely remove and replace all existing artificial fill. Excavations should be conducted as necessary to remove any encountered fill or soft soil as necessary at the direction of the Geotechnical Engineer (a representative of Geocon). The limits of existing fill and/or soft soil removal will be verified by the Geocon representative during site grading activities.

8.4.6 Alternatively, the proposed slab may be designed as a structural slab deriving all support from the deepened foundation system and eliminating reliance on the underlying soil. As a minimum, it is recommended that the upper 12 inches of slab subgrade be compacted to provide a suitable temporary surface upon which concrete can be poured and placed. Any soils unintentionally disturbed should be properly compacted prior to slab construction.

8.4.7 Where the proposed project incorporates subterranean levels which will be embedded more than 50 feet below the ground surface, it is recommended that a reinforced concrete mat foundation system be used for support of the proposed structure. Recommendations for the design of a mat foundation are provided in Section 8.9.

8.4.8 The City of Los Angeles Department of Building and Safety requires a minimum compactive effort of 95 percent of the laboratory maximum dry density in accordance with ASTM D 1557 (latest edition) where the soils to be utilized in the fill have less than 15 percent finer than 0.005 millimeters. Soils with more than 15 percent finer than 0.005 millimeters may be compacted to 90 percent of the laboratory maximum dry density in accordance with ASTM D 1557 (latest edition). It is anticipated that the soils encountered by this firm would require the minimum 95 percent compaction requirement; however additional laboratory testing can be performed during construction to verify the compaction requirement. All fill and backfill soils should be placed in horizontal loose layers approximately 6 to 8 inches thick, moisture conditioned to optimum moisture content, and properly compacted to the required degree of compaction in accordance with ASTM D 1557 (latest edition).

8.4.9 Prior to construction of exterior slabs, the upper 12 inches of the subgrade should be moisture conditioned to optimum moisture content and properly compacted to at least 95 percent relative compaction, as determined by ASTM Test Method D1557 (latest edition).

8.4.10 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 corrosivity properties that are equally or less detrimental to that of the existing onsite soils (see Figure B15).

8.4.11 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 is also acceptable as backfill (see Section 8.5). 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).

8.4.12 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.

8.5 Controlled Low Strength Material (CLSM)

8.5.1 Controlled Low Strength Material (CLSM) may be utilized in lieu of compacted soil as engineered fill where approved in writing by the Geotechnical Engineer. Where utilized within the City of Los Angeles use of CLSM is subject to the following requirements:

Standard Requirements

1. CLSM shall be ready-mixed by a City of Los Angeles approved batch plant;
2. CLSM shall not be placed on uncertified fill, on incompetent natural soil, nor below water;
3. CLSM shall not be placed on a sloping surface with a gradient steeper than 5:1 (horizontal to vertical);
4. Placement of the CLSM shall be under the continuous inspection of a concrete deputy inspector;
5. The excavation bottom shall be accepted by the soil engineer and the City Inspector prior to placing CLSM.

Requirements for CLSM that will be used for support of footings

1. The cement content of the CLSM shall not be less than 188 pounds per cubic yard (min. 2 sacks);
2. The excavation bottom must be level, cleaned of loose soils and approved in writing by Geocon prior to placement of the CLSM;
3. The ultimate compressive strength of the CLSM shall be no less than 100 pounds per square inch (psi) when tested on the 28th-day per ASTM D4832 (latest edition), Standard Test Method for Preparation and Testing of Controlled Low Strength Material Test Cylinders. Compression testing will be performed in accordance with ASTM C39 and City of Los Angeles requirements;
4. Samples of the CLSM will be collected during placement, a minimum of one test (two cylinders) for each 50 cubic yards or fraction thereof;
5. Overexcavation for CLSM placement shall extend laterally beyond the footprint of any proposed footings as required for placement of compacted fill, unless justified otherwise by the soil engineer that footings will have adequate vertical and horizontal bearing capacity.

8.6 Foundation Design

8.6.1 Where the lowest level of the proposed structure is located between the ground surface and a depth of 50 feet, it is recommended that the proposed structure be supported on a deepened foundation system deriving support in the competent alluvial soils. Recommendations for the design and construction of drilled cast-in-place friction piles are provided in Sections 8.7 and 8.8.

8.6.2 Where the proposed project incorporates subterranean levels which will be embedded more than 50 feet below the ground surface, it is recommended that a reinforced concrete mat foundation system be used for support of the proposed structure. Recommendations for the design of a mat foundation are provided in Section 8.9.

8.6.3 Once proposed foundation depths and building loads are available, additional analyses will be required to evaluate the anticipated total and differential settlements between the foundation elements for verification that the settlements are in conformance with the City of Los Angeles policy. Updated foundation design recommendations will be provided as necessary in an addendum report.

8.6.4 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.

8.6.5 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.

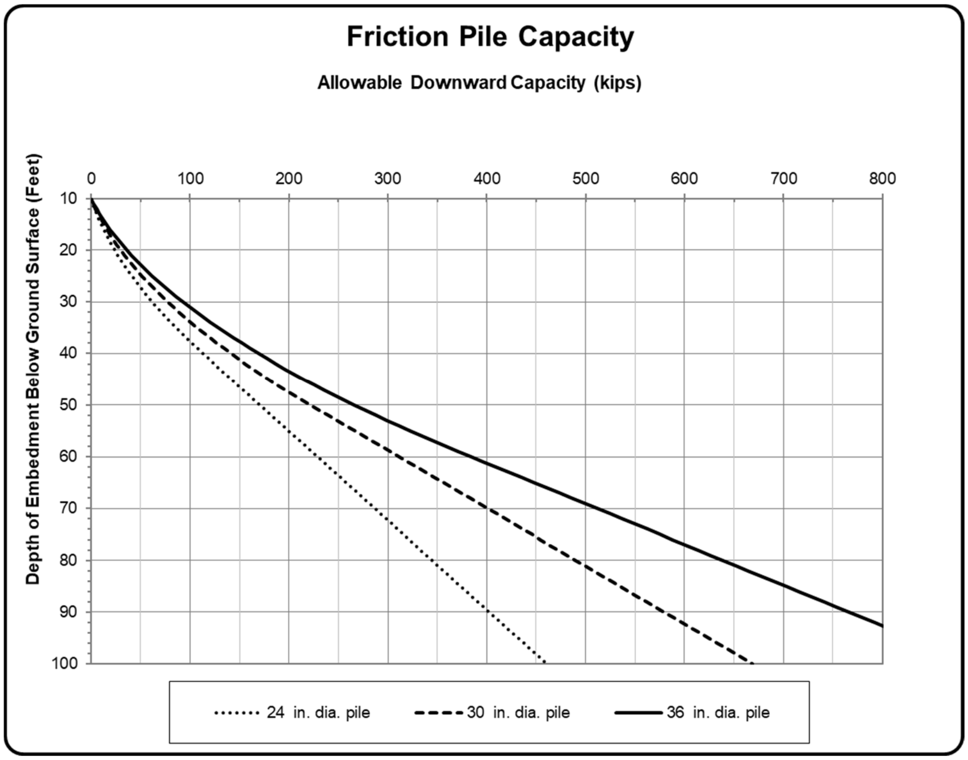
8.6.6 Foundation excavations should be observed and approved in writing by the Geotechnical Engineer (a representative of Geocon West, Inc.), prior to the placement of the methane system, reinforcing steel and concrete to verify that the excavations and exposed soil conditions are consistent with those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

8.6.7 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.

8.7 Friction Pile Design

8.7.1 For preliminary design purposes 24-, 30-, and 36-inch diameter drilled cast-in-place friction piles have been evaluated. Friction piles should be embedded a minimum of 15 feet into the alluvial soils found at and below a depth of 10 feet. The allowable axial capacities for pile embedment into the competent alluvial soils are provided in the charts below. The axial capacities are based on skin friction; end-bearing capacity is not being considered. Although not required, pile load testing can be considered to confirm the allowable pile capacities. Additional recommendations regarding a pile-load testing program can be provided under separate cover.

8.7.2 Friction piles supporting the proposed structure may use the capacities presented in the chart below.



8.7.3 All drilled pile excavations should be continuously observed by personnel of this firm to verify adequate penetration into the recommended bearing materials. The capacity presented is based on the strength of the soils. The compressive and tensile strength of the pile sections should be checked to verify the structural capacity of the piles.

8.7.4 Single pile uplift capacity can be taken as 60 percent of the allowable downward capacity.

8.7.5 The allowable downward capacity and allowable uplift capacity may be increased by one-third when considering transient wind or seismic loads.

8.7.6 The maximum expected static settlement for the structure supported on friction piles is estimated to be less than ½ inch. Differential settlement between adjacent pile foundations is not expected to exceed ¼ inch. The majority of the foundation settlement is expected to occur on initial application of loading and during construction.

8.7.7 For increased resistance to differential foundation movement and lateral drift, the pile tops should be interconnected in two horizontal directions with grade beams or tied with a structural slab. The project structural engineer should provide slab and grade beam design, reinforcement and spacing dependent on anticipated loading. However, for grade beams we recommend a minimum embedment depth below lowest adjacent pad grade of 24 inches and a minimum width of 12 inches. In addition, minimum reinforcement should consist of four No. 4 steel reinforcing bars; two placed near the top of the grade beam and two near the bottom.

8.7.8 If pile spacing is at least three times the maximum dimension of the pile, no reduction in axial capacity is considered necessary for group effects. If pile spacing is closer than three pile diameters, an evaluation for group effects including appropriate reductions should be performed by Geocon based on pile dimension and spacing.

8.8 Deepened Foundation Installation

8.8.1 Casing may be required if caving occurs in the granular soil layers during deep drilled excavation. The contractor should have casing available and should be prepared to use it. 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. Continuous observation of the drilling and pouring of the piles by the Geotechnical Engineer (a representative of Geocon West, Inc.), is required.

8.8.2 Friction piles do not require the complete removal of all loose earth materials from the bottom of the excavation since the end-bearing capacity is not being considered for design. However, a cleanout of the excavation bottom will be required. Foundation excavations should be observed and approved in writing by the Geotechnical Engineer (a representative of Geocon West, Inc.), prior to the placement of reinforcing steel and concrete.

8.8.3 Groundwater was encountered at a depth of approximately 106 feet below existing ground surface; therefore, it is not anticipated that groundwater will be encountered 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.

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

8.8.5 Closely spaced piles should be drilled and filled alternately, with the concrete permitted to set at least eight hours before drilling an adjacent hole. Pile excavations should be filled with concrete as soon after drilling and inspection as possible; the holes should not be left open overnight.

8.9 Mat Foundation Design

8.9.1 It is anticipated that the mat foundation constructed for support of the tower will impart an average pressure of approximately 7,500 psf psf. The recommended maximum allowable bearing value is 7,500 psf. The allowable bearing pressure may be increased by up to one-third for transient loads due to wind or seismic forces.

8.9.2 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 newly placed engineered fill. 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)

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

8.9.4 For seismic design purposes, a coefficient of friction of 0.45 may be utilized between the concrete mat and alluvium without a moisture barrier, and 0.15 for slabs underlain by a moisture barrier.

8.9.5 The maximum expected static settlement for a mat foundation deriving support in competent alluvial soils and utilizing a uniformly distributed maximum allowable bearing pressure of 7,500 psf is estimated to be less than 4 inches and occur below the central portion of the mat. The differential settlement between the center and corner of the mat is estimated to be less than 2 inches.

8.9.6 Once the design and foundation loading configuration proceeds to a more finalized plan, the recommendations within this report should be reviewed and revised, if necessary. Updated estimates of settlement should be anticipated based on the relationship between the foundation depth and bearing pressure distribution. Based on the final foundation loading configuration, the potential for settlement should be reevaluated by this office.

8.10 Lateral Design

8.10.1 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 0.45 may be used with the dead load forces in the competent alluvial soils or newly placed engineered fill.

8.10.2 Passive earth pressure for the sides of foundations and slabs poured against alluvial soils or newly placed engineered fill may be computed as an equivalent fluid having a density of 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.

8.10.3 Ultimate lateral capacities for ¼ inch deflection of fixed and free-head drilled cast-in place piles are presented in the table below. No factors of safety have been applied to the lateral load values calculated to induce ¼-inch lateral deflection. Lateral capacities provided are for 24-, 30-, and 36-inch diameter drilled cast-in-place concrete piles, penetrating the earth materials encountered during the course of this investigation. Assumed as part of these lateral capacity calculations are a concrete modulus of elasticity of at least 3,000,000 psi.

LATERAL LOAD CAPACITIES OF DRILLED CAST-IN-PLACE PILES

FIXED HEAD (NO HEAD ROTATION)

PILE NUMBER	PILE DIAMETER (INCHES)	Lateral Load Capacity "P" (KIPS)	Maximum Positive Moment "Mp" (LAT FORCE =P)	Maximum Negative Moment "Mp" (LAT FORCE =P)	Depth to Max Pos. Moment (Feet)	Depth to Zero Moment (Feet)	Depth to Inflection Point (Feet)	MINIMUM PILE LENGTH FOR APPLICABILITY OF LATERAL DESIGN DATA (FEET)
1	24	**38**	**1.5** P	**-5.3** P	**13**	**26**	**6.6**	**26**
2	30	**55**	**1.7** P	**-6.4** P	**15**	**31**	**7.9**	**31**
3	36	**73**	**2.0** P	**-7.3** P	**18**	**34**	**9.1**	**34**

FREE HEAD (HINGED)

PILE NUMBER	PILE DIAMETER (INCHES)	Lateral Load Capacity "P" (KIPS)	Maximum Moment "Mp" (LAT FORCE =P)	Depth to Zero Moment (Feet)	Depth to Maximum Moment (Feet)
1	24	**16**	**4.5** P	**23**	**7**
2	30	**22**	**5.4** P	**30**	**9**
3	36	**30**	**6.2** P	**34**	**10**

Lateral capacities are based on 1/4-inch deflection.
Moment magnitudes are presented as a function of the applied lateral load “P”.
"P" is entered in units of kips and the moment magnitude will be in units of kip-feet.
The maximum negative moment is at the rigid, pile to pile cap or grade beam connection at the top of the pile.

8.10.4 Once the project design proceeds to a more finalized state and the foundation system has been selected, an LPile analysis of lateral pile capacity can be performed, if necessary. If piles are spaced at least at least 8 diameters on-center when loaded in-line and at least 3 diameters on-center when loaded in parallel, no reduction in lateral capacity is considered necessary for group effects. If pile spacing is closer, an evaluation for group effects including appropriate reductions should be incorporated into the pile design based on pile dimension, spacing, and the direction of loading.

8.11 Concrete Slabs-on-Grade

8.11.1 The project structural engineer may determine and design the necessary slab thickness and reinforcing for this structure. Unless specifically analyzed and designed by the project structural engineer, the slab-on-grade should be a minimum of 5 inches concrete reinforced with No. 4 steel reinforcing bars placed 16 inches on center in both horizontal directions and positioned vertically near the slab midpoint. The use of a conventional slab-on-grade is contingent upon the excavation and recompaction of all existing artificial fill from below the slab. The client and contractor should be aware that excavations on the order of 8 feet in depth may be required to completely remove and replace all existing artificial fill.

8.11.2 Alternatively, the concrete slab-on-grade for a pile supported structure be designed as a structural slab deriving all support from the deepened foundation system. The thickness and reinforcing of the structural slab should be designed by the project structural engineer. It is recommended that the upper 12 inches of slab subgrade be compacted to provide a suitable surface upon which concrete can be placed. Any soils unintentionally disturbed should be properly compacted prior to slab construction.

8.11.3 Slabs-on-grade at the ground surface that may receive moisture-sensitive floor coverings or may be used to store moisture-sensitive materials should be underlain by a vapor retarder placed directly beneath the slab. The vapor retarder and acceptable permeance should be specified by the project architect or developer based on the type of floor covering that will be installed. The vapor retarder design should be consistent with the guidelines presented in Section 9.3 of the American Concrete Institute's (ACI) Guide for Concrete Slabs that Receive Moisture-Sensitive Flooring Materials (ACI 302.2R-06) and should be installed in general conformance with ASTM E 1643 (latest edition) and the manufacturer's recommendations. A minimum thickness of 15 mils extruded polyolefin plastic is recommended; recycled content or woven materials are not recommended. The material should have a permeance of less than 0.01 perms demonstrated by testing before and after mandatory conditioning. The vapor retarder should be installed in direct contact with the concrete slab with proper perimeter seal. If the Los Angeles Green Building Code requirements apply to this project, the vapor retarder should be underlain by 4 inches of clean aggregate. It is important that the vapor retarder be puncture resistant since it will be in direct contact with angular gravel. As an alternative to the clean aggregate suggested in the Los Angeles Green Building Code, it is our opinion that the concrete slab-on-grade may be underlain by a vapor retarder over 4 inches of clean sand (sand equivalent greater than 30), since the sand will serve a capillary break and will minimize the potential for punctures and damage to the vapor barrier.

8.11.4 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.

8.11.5 For seismic design purposes, a coefficient of friction of 0.45 may be utilized between concrete slabs and soil without a moisture barrier and 0.15 for slabs underlain by a vapor retarder or methane barrier.

8.11.6 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 moisture conditioned to 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 10 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.

8.11.7 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 or concrete shrinkage. The occurrence of concrete shrinkage cracks is independent of the supporting soil characteristics. Their occurrence may be reduced 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.

8.12 Retaining Wall Design

8.12.1 The recommendations presented below are generally applicable to the design of rigid concrete or masonry retaining walls having a maximum height of 75 feet. In the event that walls significantly higher than 75 feet are planned, Geocon should be contacted for additional recommendations.

8.12.2 Retaining wall foundations may be designed in accordance with the recommendations provided in the *Foundation Design* sections of this report (see Sections 8.6 through 8.9).

8.12.3 Retaining walls with a level backfill surface and 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 below presents recommended pressures to be used in retaining wall design, assuming that proper drainage will be maintained. Calculation of the recommended earth pressures is provided as Figures 5 through 8.

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 10	35	54
Up to 20	43	54
Up to 40	47	54
Up to 75	48	54

8.12.4 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 cantilever and restrained undrained walls is 90 pcf. The value includes hydrostatic pressures plus buoyant lateral earth pressures.

8.12.5 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 8.23 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.

8.12.6 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 pounds per square foot, 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, the traffic surcharge may be neglected.

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

8.13 Dynamic (Seismic) Lateral Forces

8.13.1 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 2016 CBC).

8.13.2 A seismic load of 10 pcf should be used for design of walls that support more than 6 feet of backfill in accordance with Section 1803.5.12 of the 2016 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 active earth pressure. The earth pressure is based on half of two thirds of PGA_M calculated from ASCE 7-10 Section 11.8.3.

8.14 Retaining Wall Drainage

8.14.1 Retaining walls 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 9). 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.

8.14.2 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 10). 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.

8.14.3 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.

8.14.4 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.

8.15 Elevator Pit Design

8.15.1 The elevator pit slab and retaining wall should be designed by the project structural engineer. As a minimum the slab-on-grade for the elevator pit bottom should be at least 4 inches thick and reinforced with No. 4 steel reinforcing bars placed 16 inches on center in both horizontal directions, positioned near the slab midpoint. Elevator pit walls may be designed in accordance with the recommendations in the *Foundation Design* and *Retaining Wall Design* sections of this report (see Sections 8.6 through 8.9 and 8.12).

8.15.2 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.

8.15.3 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 8.14).

8.15.4 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.

8.16 Elevator Piston

8.16.1 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.

8.16.2 Due to the preliminary nature of the project at this time, it is unknown if a plunger-type elevator piston will be included for this project. If in the future it is determined that a plunger-type elevator piston will be constructed, the location of the proposed elevator should be reviewed by the Geotechnical Engineer to evaluate the setback from foundations and shoring piles. Additional recommendations will be provided as necessary.

8.16.3 Casing may 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. Continuous observation of the drilling and installation of the elevator piston by the Geotechnical Engineer (a representative of Geocon West, Inc.) is required.

8.16.4 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.

8.17 Temporary Excavations

8.17.1 Excavations on the order of 75 feet in height are anticipated for excavation and construction of the proposed structure level, including foundation excavations. The excavations are expected to expose alluvial soils, which are suitable for vertical excavations up to 5 feet where loose soils or caving sands are not present or where not surcharged by adjacent traffic or structures.

8.17.2 Vertical excavations greater than 5 feet will require sloping and/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 8 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 8.18 of this report.

8.17.3 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.

8.18 Shoring – Soldier Pile Design and Installation

8.18.1 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.

8.18.2 One method of shoring would consist of steel soldier piles, placed in drilled holes and backfilled with concrete. The steel soldier piles may also be installed utilizing high frequency vibration. 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.

8.18.3 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 grading activities, foundations, and/or adjacent drainage systems.

8.18.4 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 8.12).

8.18.5 Drilled cast-in-place soldier piles should be placed no closer than three 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 300 psf per foot. Where piles are installed by vibration techniques, the passive pressure may be assumed to mobilize across a width equal to the two times the dimension of the beam flange. The allowable passive value may be doubled for isolated piles, spaced a minimum of two 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 alluvium.

8.18.6 Groundwater was encountered at a depth of approximately 106 feet below existing ground surface; therefore, it is not anticipated that groundwater will be encountered during pile installation. Should groundwater or local seepage be encountered during pile installation, the contractor should be prepared. 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.

8.18.7 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.

8.18.8 Casing may be required if caving is experienced, and the contractor should have casing available prior to commencement of pile excavation. When 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.

8.18.9 As an alternative, piles may be vibrated into place; however, there is always a risk that excessive vibrations in sandy soils could induce settlements and distress to adjacent offsite improvements. Continuous observation of the drilling and pouring of the piles by the Geotechnical Engineer (a representative of Geocon West, Inc.), is required.

8.18.10 If a vibratory method of solider pile installation is utilized, predrilling may be performed prior to installation of the steel beams. If predrilling is performed, the bore diameter should be no greater than 75 percent of the largest dimension of the pile to prevent excessive loss in the frictional component of the pile capacity. Predrilling should not be conducted below the proposed excavation bottom, and the auger should be backspun out of the pilot holes, leaving the soil in place.

8.18.11 If a vibratory method is utilized, the owner should be aware of the potential risks associated with vibratory efforts, which typically involve inducing settlement within the vicinity of the pile which could result in a potential for damage to existing improvements in the area.

8.18.12 The level of vibration that results from the installation of the piles should not exceed a threshold where occupants of nearby structures are disturbed, despite higher vibration tolerances that a building may endure without deformation or damage. The main parameter used for vibration assessment is peak particle velocity in units of inch per second (in/sec). The acceptable range of peak particle velocity should be evaluated based on the age and condition of adjacent structures, as well as the tolerance of human response to vibration.

8.18.13 Based on Table 19 of the *Transportation and Construction Induced Vibration Guidance Manual* (Caltrans 2013), a continuous source of vibrations (ex. vibratory pile driving) which generates a maximum peak particle velocity of 0.5 in/sec is considered tolerable for modern industrial/commercial buildings and new residential structures. The Client should be aware that a lower value may be necessary if older or fragile structures are in the immediate vicinity of the site.

8.18.14 Vibrations should be monitored and record with seismographs during pile installation to detect the magnitude of vibration and oscillation experienced by adjacent structures. If the vibrations exceed the acceptable range during installation, the shoring contractor should modify the installation procedure to reduce the values to within the acceptable range. Vibration monitoring is not the responsibility of the Geotechnical Engineer.

8.18.15 Geocon does not practice in the field of vibration monitoring. If construction techniques will be implemented, it is recommended that qualified consultant be retained to provide site specific recommendations for vibration thresholds and monitoring.

8.18.16 The frictional resistance between the soldier piles and retained soil may be used to resist the vertical component of the anchor load. The coefficient of friction may be taken as 0.45 based on uniform contact between the steel beam and lean-mix concrete and retained earth. The portion of soldier piles below the plane of excavation may also be employed to resist the downward loads. The downward capacity may be determined using a frictional resistance of 300 psf per foot. 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.

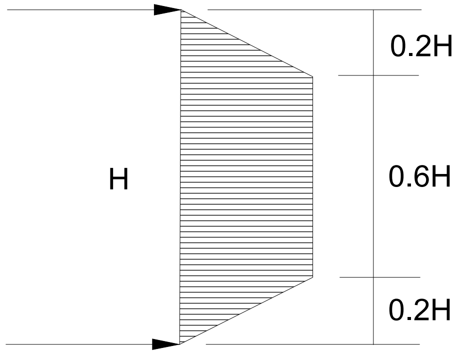
8.18.17 Due to the nature of the site soils, it is expected that continuous lagging between soldier piles will be required. However, it is recommended that the exposed soils be observed by the Geotechnical Engineer (a representative of Geocon West, Inc.), to verify the presence of any competent, cohesive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

8.18.18 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.

8.18.19 For the design of unbraced shoring, it is recommended that an equivalent fluid pressure be utilized for design. A trapezoidal distribution of lateral earth pressure may be used where shoring will be restrained by bracing or tiebacks. 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. Calculation of the recommended shoring pressures is provided as Figures 11 through 14.

HEIGHT OF SHORING (FEET)	**EQUIVALENT FLUID PRESSURE (Pounds Per Cubic Foot) (ACTIVE PRESSURE)**	**EQUIVALENT FLUID PRESSURE Trapezoidal (Where H is the height of the shoring in feet)**
Up to 15	31	19H
Up to 25	36	23H
Up to 45	39	24H
Up to 80	41	26H

Trapezoidal Distribution of Pressure



8.18.20 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 8.23 of this report.

8.18.21 In addition to the recommended earth pressure, the upper ten 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 ten feet from the shoring, the traffic surcharge may be neglected.

8.18.22 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.

8.18.23 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.

8.18.24 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.

8.19 Temporary Tie-Back Anchors

8.19.1 Temporary tie-back anchors may be used with the solider pile wall system to resist lateral loads. 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.

8.19.2 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 – 750 pounds per square foot
- 15 feet below the top of the excavation – 1,500 pounds per square foot
- 35 feet below the top of the excavation – 3,000 pounds per square foot
- 60 feet below the top of the excavation – 4,800 pounds per square foot

8.19.3 Depending on the techniques utilized, and the experience of the contractor performing the installation, a maximum allowable friction capacity of 5 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.

8.20 Anchor Installation

8.20.1 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.

8.21 Anchor Testing

8.21.1 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.

8.21.2 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.

8.21.3 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.

8.21.4 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.

8.21.5 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.

8.22 Internal Bracing

8.22.1 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 1,500 psf may be used, provided the shallowest point of the footing is at least one foot below the lowest adjacent grade. The structural engineer should review the shoring plans to determine if raker footings conflict with the structural foundation system. The client should be aware that the utilization of rakers could significantly impact the construction schedule do to their intrusion into the construction site and potential interference with equipment.

8.23 Surcharge from Adjacent Structures and Improvements

8.23.1 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.

8.23.2 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.

8.23.3 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.

8.24 Stormwater Infiltration

8.24.1 During the September 14, 2019 site exploration, boring B1 was utilized to perform percolation testing. Subsequent to excavation of the boring to a depth of 150½ feet, the boring was backfilled to a depth of approximately 90 feet using soil cuttings and a 2-foot thick bentonite seal. Slotted casing was placed in the boring, and the annular space between the casing and excavation was filled with gravel. The boring was then filled with water to pre-saturate the soils. On September 15, 2019, the casing was refilled with water and percolation test readings were performed after repeated flooding of the cased excavation. The percolation test field data is presented on Figure 15. The engineer in responsible charge of the stormwater infiltration system design should determine the appropriate percolation rate using the field test data, as well as any necessary factors of safety or reduction factors based on the type of infiltration system proposed and any applicable design guidelines.

Boring	Soil Type	Test Depth (ft)
B1	Sand with Silt (SP-SM)	20-25

8.24.2 The results of the percolation testing indicate that the soils at depths in the above table are conductive to infiltration. It is our opinion that the soil zone encountered at the depth and location as listed in the table above are suitable for infiltration of stormwater and will not induce excessive hydro-consolidation, will not create a perched groundwater condition, will not affect soil structure interaction of existing or proposed foundations due to expansive soils, will not saturate soils supported by existing or proposed retaining walls, and will not increase the potential for liquefaction. Resulting settlements are anticipated to be less than ¼ inch, if any.

8.24.3 The infiltration system must be located such that the closest distance between an adjacent foundation is at least 10 feet in all directions from the zone of saturation. The zone of saturation may be assumed to project downward from the discharge of the infiltration facility at a gradient of 1:1. Additional property line or foundation setbacks may be required by the governing jurisdiction and should be incorporated into the stormwater infiltration system design as necessary.

8.24.4 Where the 10-foot horizontal setback cannot be maintained between the infiltration system and an adjacent footing, and the infiltration system penetrates below the foundation influence line, the proposed stormwater infiltration system must be designed to resist the surcharge from the adjacent foundation. The foundation surcharge line may be assumed to project down away from the bottom of the foundation at a 1:1 gradient. The stormwater infiltration system must still be sufficiently deep to maintain the 10-foot vertical offset between the bottom of the footing and the zone of saturation.

8.24.5 Subsequent to the placement of the infiltration system, it is acceptable to backfill the resulting void space between the excavation sidewalls and the infiltration system with minimum 2-sack slurry provided the slurry is not placed in the infiltration zone. It is recommended that gravel, approved by the project civil engineer, be utilized adjacent to the infiltration zone so communication of water to the soil is not hindered.

8.24.6 Due to the preliminary nature of the project at this time, the type of stormwater infiltration system and location of the stormwater infiltration systems has not yet been determined. The design drawings should be reviewed and approved by the Geotechnical Engineer. The installation of the stormwater infiltration system should be observed and approved by the Geotechnical Engineer (a representative of Geocon).

8.25 Surface Drainage

8.25.1 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.

8.25.2 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 2016 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 five 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.

8.25.3 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.

8.25.4 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.

8.26 Plan Review

8.26.1 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.

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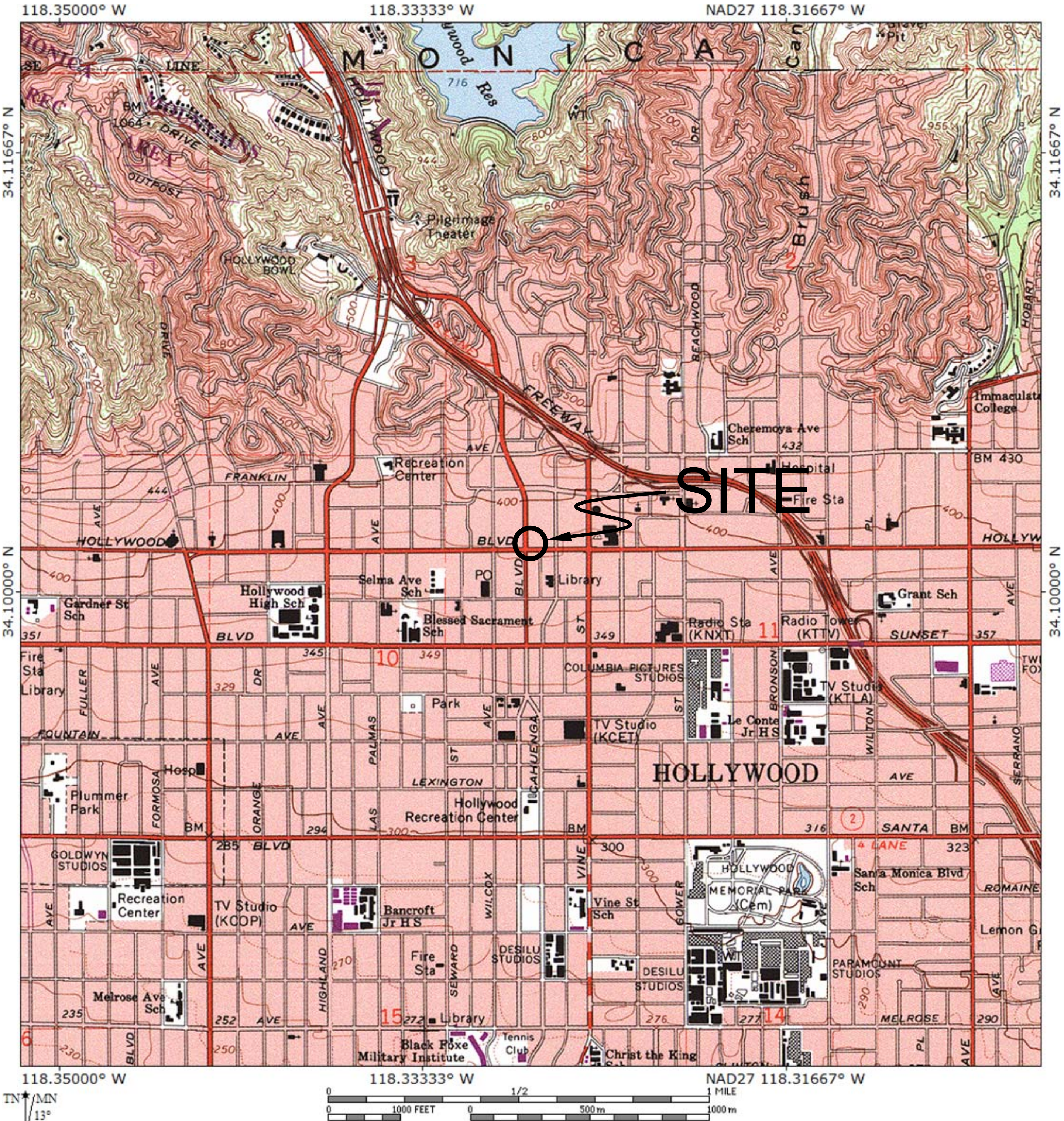

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U.S.G.S. TOPOGRAPHIC MAPS, 7.5 MINUTE SERIES, HOLLYWOOD, CA QUADRANGLE

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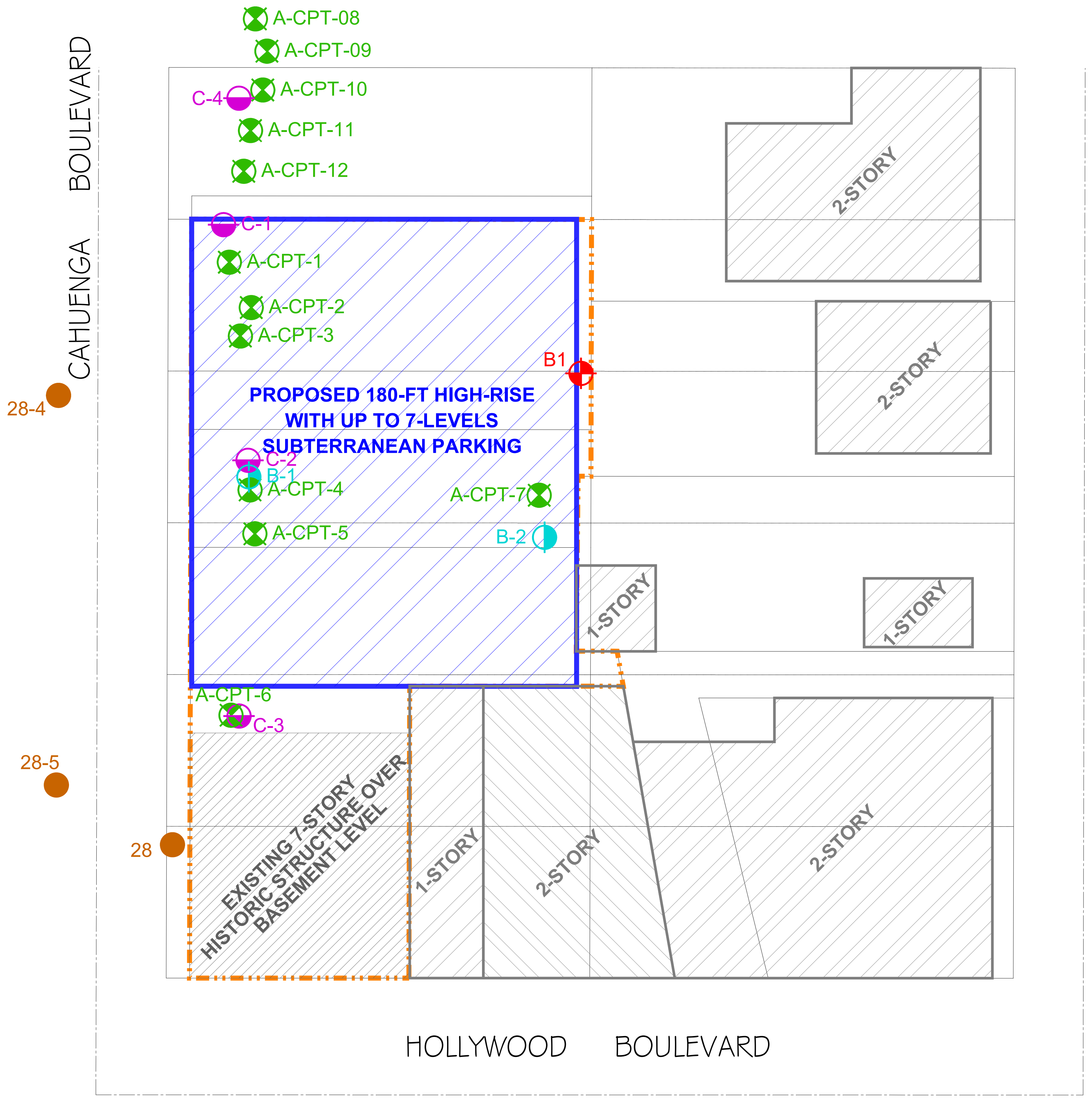
VICINITY MAP

1708-1732 NORTH CAHUENGA BOULEVARD
6381-6385 WEST HOLLYWOOD BOULEVARD
LOS ANGELES, CALIFORNIA

DRAFTED BY: CB	CHECKED BY: SFK	OCT 2019	PROJECT NO. W1063-06-01	FIG. 1



LEGEND

B1	Approximate Location of Boring (Geocon)
A-CPT-12	Approximate Location of CPT (EnGeo, 2014)
C-4	Approximate Location of Boring (EnGeo, 2014)
B-2	Approximate Location of Boring (Mactec, 2007)
28	Approximate Location of Boring (Converse, 1984)
▨	Existing Offsite Structure

0 50' 100'

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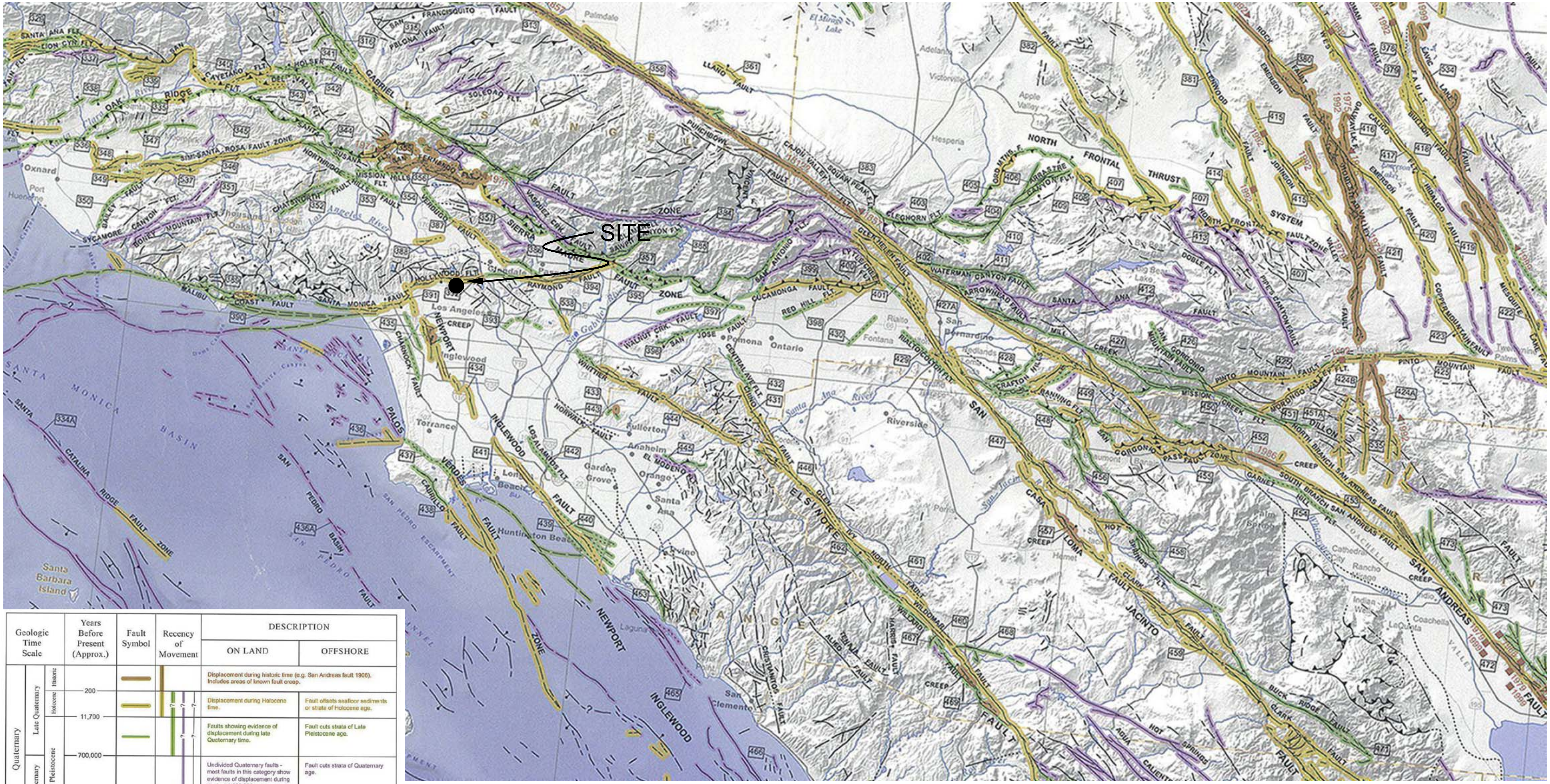
Drafted by: JTA
Checked by: NDB

SITE PLAN

1708-1732 NORTH CAHUENGA BOULEVARD
6381-6385 WEST HOLLYWOOD BOULEVARD
LOS ANGELES, CALIFORNIA

OCT 2019
PROJECT NO. W1063-06-01
FIG. 2

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	DESCRIPTION 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.

0 12 24 Miles

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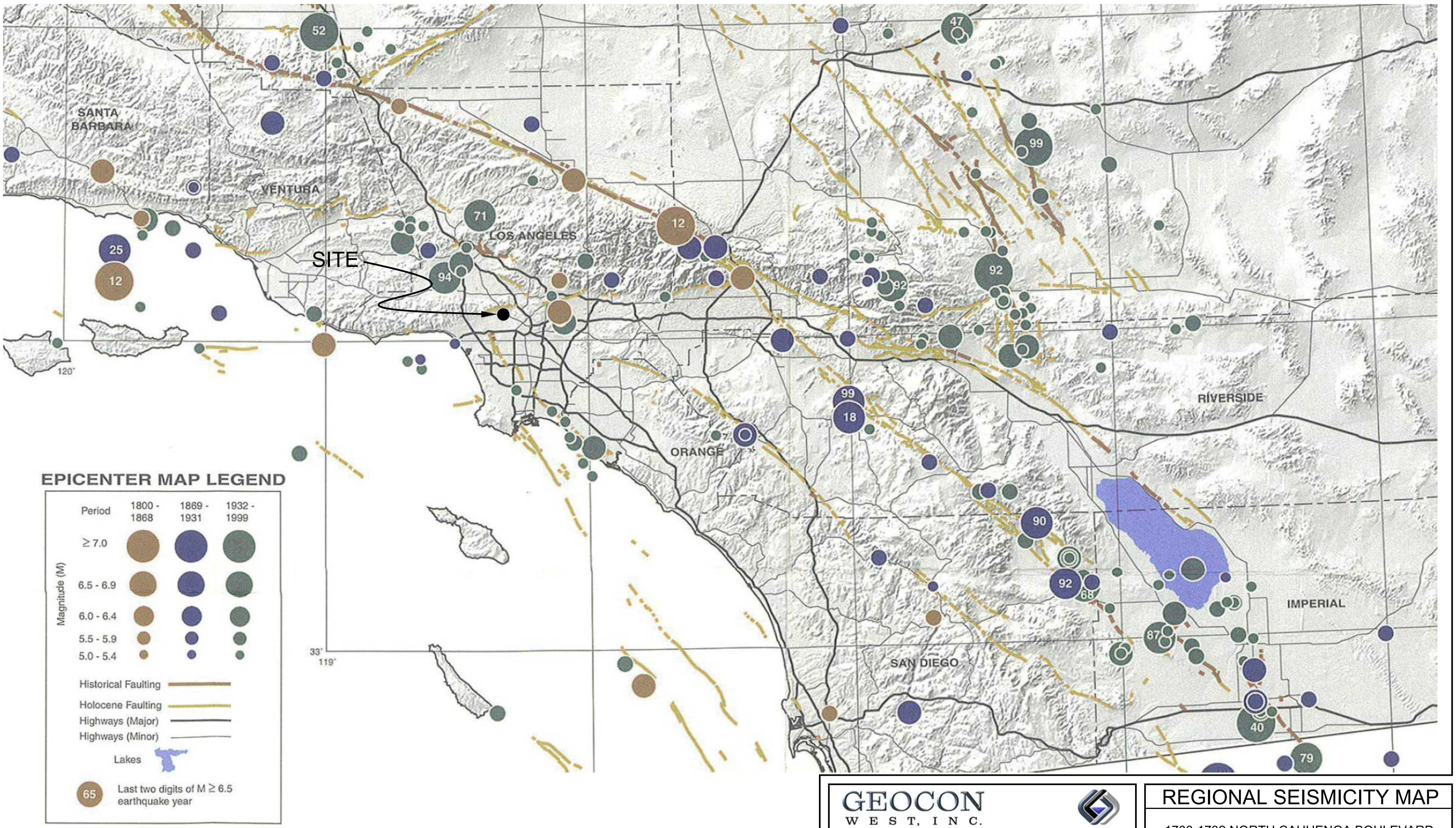
REGIONAL FAULT MAP

1708-1732 NORTH CAHUENGA BOULEVARD
6381-6385 WEST HOLLYWOOD BOULEVARD
LOS ANGELES, CALIFORNIA

OCT 2019 | PROJECT NO. W1063-06-01 | FIG. 3



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.

0 20 40 Miles

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REGIONAL SEISMICITY MAP

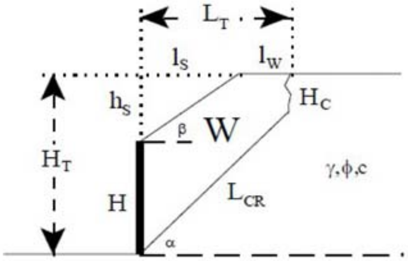
1708-1732 NORTH CAHUENGA BOULEVARD
6381-6385 WEST HOLLYWOOD BOULEVARD
LOS ANGELES, CALIFORNIA

OCT 2019 | PROJECT NO. W1063-06-01 | FIG.4

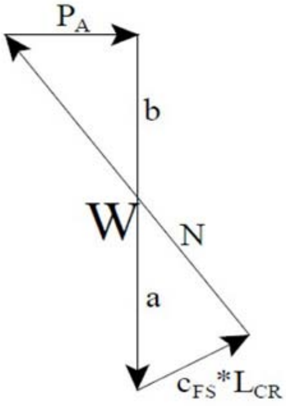
Retaining Wall Design with Transitioned Backfill
(Vector Analysis)

Input:

Retaining Wall Height	(H)	10.00	feet
Slope Angle of Backfill	(b)	0.0	degrees
Height of Slope above Wall	(h_s)	0.0	feet
Horizontal Length of Slope	(l_s)	0.0	feet
Total Height (Wall + Slope)	(H_T)	10.0	feet
Unit Weight of Retained Soils	(g)	125.0	pcf
Friction Angle of Retained Soils	(f)	35.0	degrees
Cohesion of Retained Soils	(c)	100.0	psf
Factor of Safety	(FS)	1.50	
Factored Parameters:	(f_{FS})	25.0	degrees
	(c_{FS})	66.7	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	2.0	48	5999.9	11.3	2000.4	3999.5	1453.8
46	1.9	46	5807.6	11.2	1890.0	3917.6	1502.0
47	1.9	45	5619.3	11.1	1789.2	3830.0	1545.6
48	1.9	43	5434.9	11.0	1697.1	3737.8	1584.8
49	1.8	42	5254.5	10.8	1612.7	3641.8	1619.7
50	1.8	41	5078.1	10.7	1535.0	3543.1	1650.4
51	1.8	39	4905.6	10.6	1463.5	3442.0	1677.1
52	1.7	38	4736.8	10.5	1397.5	3339.3	1699.7
53	1.7	37	4571.7	10.4	1336.4	3235.3	1718.5
54	1.7	35	4410.1	10.3	1279.7	3130.4	1733.5
55	1.7	34	4251.9	10.1	1227.1	3024.8	1744.7
56	1.7	33	4096.8	10.0	1178.0	2918.8	1752.2
57	1.7	32	3944.8	9.9	1132.2	2812.6	1755.9
58	1.7	30	3795.8	9.8	1089.4	2706.4	1756.0
59	1.7	29	3649.5	9.7	1049.3	2600.2	1752.3
60	1.7	28	3505.9	9.6	1011.7	2494.2	1744.9
61	1.7	27	3364.7	9.5	976.2	2388.5	1733.8
62	1.7	26	3225.9	9.4	942.8	2283.0	1718.9
63	1.7	25	3089.2	9.3	911.2	2178.0	1700.2
64	1.8	24	2954.7	9.2	881.3	2073.4	1677.6
65	1.8	23	2822.1	9.1	852.8	1969.3	1651.1
66	1.8	22	2691.3	9.0	825.7	1865.6	1620.4
67	1.8	20	2562.2	8.9	799.8	1762.5	1585.6
68	1.9	19	2434.7	8.7	774.9	1659.9	1546.6
69	1.9	18	2308.7	8.6	750.9	1557.8	1503.1
70	2.0	17	2183.9	8.5	727.7	1456.2	1455.0



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

$P_{A,\ max}$ 1756.0 lbs/lineal foot

Equivalent Fluid Pressure (per lineal foot of wall)

$EFP = 2*P_A/H^2$

EFP 35.1 pcf 53.3 pcf

Design Wall for an Equivalent Fluid Pressure: 35 pcf 54 pcf

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Drafted by: JTA | Checked by: NDB

RETAINING WALL PRESSURE CALCULATION

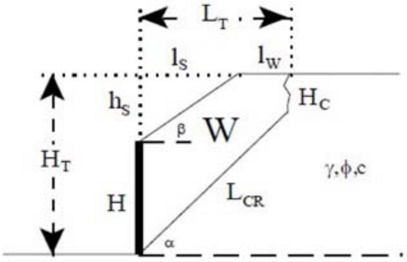
1708-1732 NORTH CAHUENGA BOULEVARD
6381-6385 WEST HOLLYWOOD BOULEVARD
LOS ANGELES, CALIFORNIA

OCT 2019 | PROJECT NO. W1063-06-01 | FIG. 5

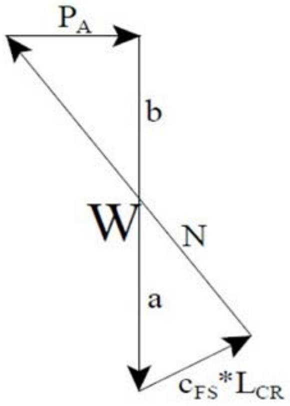
Retaining Wall Design with Transitioned Backfill
(Vector Analysis)

Input:

Retaining Wall Height	(H)	20.00	feet
Slope Angle of Backfill	(b)	0.0	degrees
Height of Slope above Wall	(h_s)	0.0	feet
Horizontal Length of Slope	(l_s)	0.0	feet
Total Height (Wall + Slope)	(H_T)	20.0	feet
Unit Weight of Retained Soils	(g)	125.0	pcf
Friction Angle of Retained Soils	(f)	35.0	degrees
Cohesion of Retained Soils	(c)	100.0	psf
Factor of Safety	(FS)	1.50	
Factored Parameters:	(f_{FS})	25.0	degrees
	(c_{FS})	66.7	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	2.0	198	24749.9	25.5	4501.0	20248.8	7360.6
46	1.9	191	23914.3	25.1	4235.8	19678.5	7544.6
47	1.9	185	23103.9	24.8	3996.4	19107.5	7710.8
48	1.9	179	22317.5	24.4	3779.5	18537.9	7859.9
49	1.8	172	21553.6	24.1	3582.4	17971.2	7992.5
50	1.8	166	20811.2	23.8	3402.6	17408.6	8109.1
51	1.8	161	20089.0	23.5	3238.2	16850.8	8210.2
52	1.7	155	19385.9	23.2	3087.4	16298.5	8296.1
53	1.7	150	18700.8	22.9	2948.8	15752.0	8367.2
54	1.7	144	18032.8	22.6	2821.1	15211.7	8423.9
55	1.7	139	17380.7	22.4	2703.0	14677.7	8466.2
56	1.7	134	16743.9	22.1	2593.8	14150.1	8494.4
57	1.7	129	16121.2	21.8	2492.4	13628.9	8508.5
58	1.7	124	15512.1	21.6	2398.2	13113.9	8508.7
59	1.7	119	14915.6	21.4	2310.4	12605.3	8494.9
60	1.7	115	14331.2	21.1	2228.5	12102.7	8467.0
61	1.7	110	13758.0	20.9	2152.0	11606.0	8425.0
62	1.7	106	13195.4	20.7	2080.3	11115.1	8368.7
63	1.7	101	12642.8	20.5	2013.0	10629.8	8297.9
64	1.8	97	12099.7	20.3	1949.8	10149.9	8212.4
65	1.8	93	11565.4	20.1	1890.3	9675.1	8111.6
66	1.8	88	11039.4	19.9	1834.1	9205.3	7995.4
67	1.8	84	10521.1	19.7	1781.0	8740.2	7863.2
68	1.9	80	10010.2	19.5	1730.6	8279.6	7714.5
69	1.9	76	9506.1	19.3	1682.8	7823.3	7548.7
70	2.0	72	9008.3	19.2	1637.2	7371.1	7365.1



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

$P_{A,\ max}$ 8508.7 lbs/lineal foot

Equivalent Fluid Pressure (per lineal foot of wall)

$EFP = 2^*P_A/H^2$

EFP 42.5 pcf 53.3 pcf

Design Wall for an Equivalent Fluid Pressure: 43 pcf 54 pcf

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RETAINING WALL PRESSURE CALCULATION

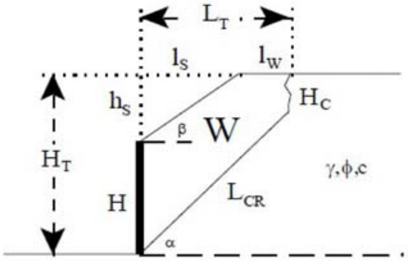
1708-1732 NORTH CAHUENGA BOULEVARD
6381-6385 WEST HOLLYWOOD BOULEVARD
LOS ANGELES, CALIFORNIA

OCT 2019 | PROJECT NO. W1063-06-01 | FIG. 6

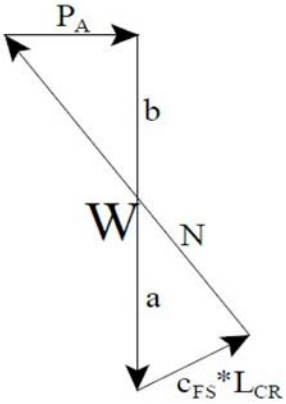
Retaining Wall Design with Transitioned Backfill
(Vector Analysis)

Input:

Retaining Wall Height	(H)	40.00	feet
Slope Angle of Backfill	(b)	0.0	degrees
Height of Slope above Wall	(h_s)	0.0	feet
Horizontal Length of Slope	(l_s)	0.0	feet
Total Height (Wall + Slope)	(H_T)	40.0	feet
Unit Weight of Retained Soils	(g)	125.0	pcf
Friction Angle of Retained Soils	(f)	35.0	degrees
Cohesion of Retained Soils	(c)	100.0	psf
Factor of Safety	(FS)	1.50	
Factored Parameters:	(f_{FS})	25.0	degrees
	(c_{FS})	66.7	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	2.0	798	99749.9	53.7	9502.3	90247.5	32805.7
46	1.9	771	96340.9	52.9	8927.5	87413.4	33513.9
47	1.9	744	93042.5	52.1	8410.8	84631.7	34153.2
48	1.9	719	89847.8	51.3	7944.4	81903.4	34726.4
49	1.8	694	86750.1	50.6	7521.9	79228.3	35235.9
50	1.8	670	83743.7	49.9	7137.8	76605.9	35683.8
51	1.8	647	80822.8	49.2	6787.6	74035.2	36072.0
52	1.7	624	77982.3	48.6	6467.3	71515.0	36401.9
53	1.7	602	75217.4	47.9	6173.7	69043.7	36675.0
54	1.7	580	72523.5	47.3	5903.7	66619.8	36892.4
55	1.7	559	69896.3	46.8	5654.9	64241.4	37054.8
56	1.7	539	67332.0	46.2	5425.2	61906.8	37162.9
57	1.7	519	64826.8	45.7	5212.7	59614.1	37217.2
58	1.7	499	62377.3	45.2	5015.6	57361.7	37217.8
59	1.7	480	59980.2	44.7	4832.5	55147.7	37164.8
60	1.7	461	57632.4	44.2	4662.2	52970.3	37057.9
61	1.7	443	55331.2	43.8	4503.4	50827.7	36896.8
62	1.7	425	53073.6	43.4	4355.2	48718.4	36680.8
63	1.7	407	50857.2	43.0	4216.6	46640.6	36409.0
64	1.8	389	48679.6	42.6	4086.9	44592.8	36080.4
65	1.8	372	46538.4	42.2	3965.2	42573.3	35693.6
66	1.8	355	44431.5	41.8	3850.9	40580.6	35247.1
67	1.8	339	42356.8	41.4	3743.4	38613.4	34739.1
68	1.9	322	40312.2	41.1	3642.1	36670.1	34167.4
69	1.9	306	38295.9	40.8	3546.6	34749.4	33529.6
70	2.0	290	36306.1	40.4	3456.2	32849.9	32823.1



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

$P_{A,\ max}$ 37217.8 lbs/lineal foot

Equivalent Fluid Pressure (per lineal foot of wall)

$EFP = 2*P_A/H^2$

EFP 46.5 pcf 53.3 pcf

Design Wall for an Equivalent Fluid Pressure: 47 pcf 54 pcf

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Drafted by: JTA | Checked by: NDB

RETAINING WALL PRESSURE CALCULATION

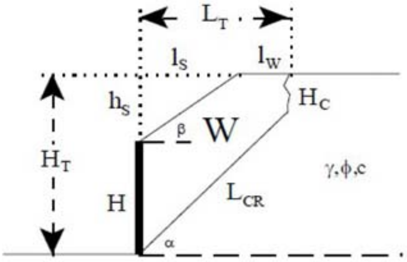
1708-1732 NORTH CAHUENGA BOULEVARD
6381-6385 WEST HOLLYWOOD BOULEVARD
LOS ANGELES, CALIFORNIA

OCT 2019 | PROJECT NO. W1063-06-01 | FIG. 7

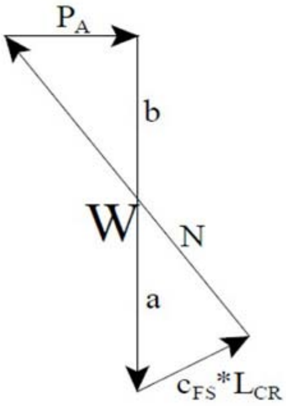
Retaining Wall Design with Transitioned Backfill
(Vector Analysis)

Input:

Retaining Wall Height	(H)	75.00	feet
Slope Angle of Backfill	(b)	0.0	degrees
Height of Slope above Wall	(h_s)	0.0	feet
Horizontal Length of Slope	(l_s)	0.0	feet
Total Height (Wall + Slope)	(H_T)	75.0	feet
Unit Weight of Retained Soils	(g)	125.0	pcf
Friction Angle of Retained Soils	(f)	35.0	degrees
Cohesion of Retained Soils	(c)	100.0	psf
Factor of Safety	(FS)	1.50	
Factored Parameters:	(f_{FS})	25.0	degrees
	(c_{FS})	66.7	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	2.0	2810	351312.4	103.2	18254.6	333057.8	121069.1
46	1.9	2714	339272.0	101.6	17138.0	322134.0	123504.7
47	1.9	2621	327628.4	100.0	16135.9	311492.4	125703.1
48	1.9	2531	316355.7	98.4	15232.9	301122.8	127673.9
49	1.8	2443	305430.1	97.0	14415.9	291014.2	129425.4
50	1.8	2359	294829.7	95.6	13674.3	281155.4	130965.1
51	1.8	2276	284534.1	94.3	12999.0	271535.1	132299.3
52	1.7	2196	274524.5	93.0	12382.1	262142.4	133433.4
53	1.7	2118	264783.3	91.8	11817.1	252966.2	134372.0
54	1.7	2042	255294.3	90.6	11298.3	243996.0	135118.9
55	1.7	1968	246042.3	89.5	10820.8	235221.5	135677.1
56	1.7	1896	237013.0	88.4	10380.3	226632.7	136048.7
57	1.7	1826	228193.4	87.4	9973.2	218220.2	136235.2
58	1.7	1757	219571.0	86.5	9596.1	209974.9	136237.4
59	1.7	1689	211134.2	85.5	9246.3	201887.9	136055.2
60	1.7	1623	202872.1	84.7	8921.1	193951.0	135687.9
61	1.7	1558	194774.5	83.8	8618.5	186156.0	135134.1
62	1.7	1495	186831.8	83.0	8336.3	178495.4	134391.7
63	1.7	1432	179034.7	82.2	8073.0	170961.8	133457.6
64	1.8	1371	171374.8	81.5	7826.7	163548.1	132328.1
65	1.8	1311	163844.0	80.8	7596.3	156247.7	130998.7
66	1.8	1251	156434.4	80.1	7380.3	149054.1	129463.9
67	1.8	1193	149138.7	79.5	7177.6	141961.1	127717.4
68	1.9	1136	141950.0	78.8	6987.2	134962.8	125751.8
69	1.9	1079	134861.7	78.3	6808.2	128053.6	123558.8
70	2.0	1023	127867.4	77.7	6639.5	121227.9	121128.9



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

$P_{A,\ max}$ 136237.4 lbs/lineal foot

Equivalent Fluid Pressure (per lineal foot of wall)

$EFP = 2*P_A/H^2$

EFP 48.4 pcf 53.3 pcf

Design Wall for an Equivalent Fluid Pressure: 48 pcf 54 pcf

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RETAINING WALL PRESSURE CALCULATION

1708-1732 NORTH CAHUENGA BOULEVARD
6381-6385 WEST HOLLYWOOD BOULEVARD
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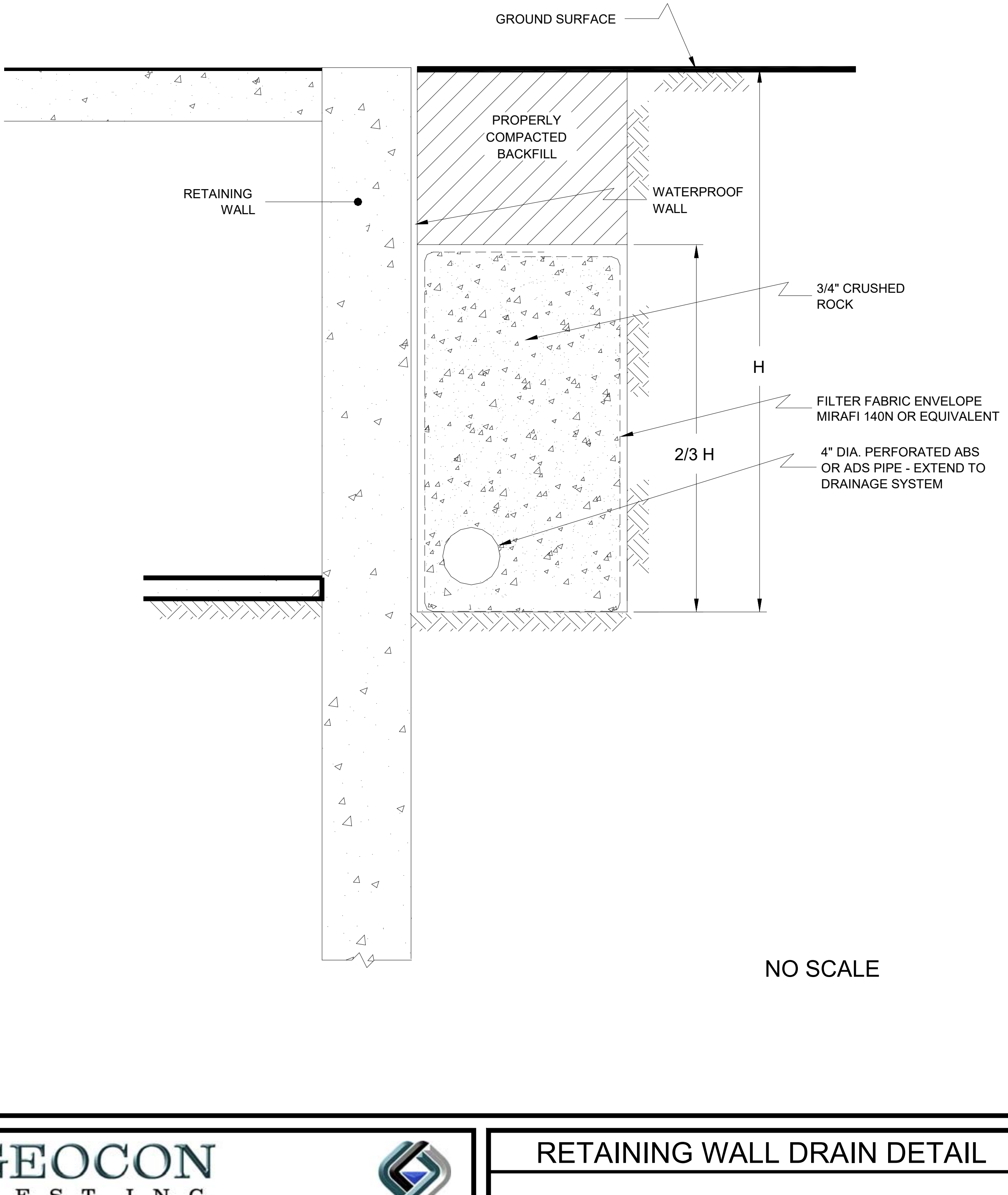
OCT 2019 | PROJECT NO. W1063-06-01 | FIG. 8



NO SCALE

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RETAINING WALL DRAIN DETAIL

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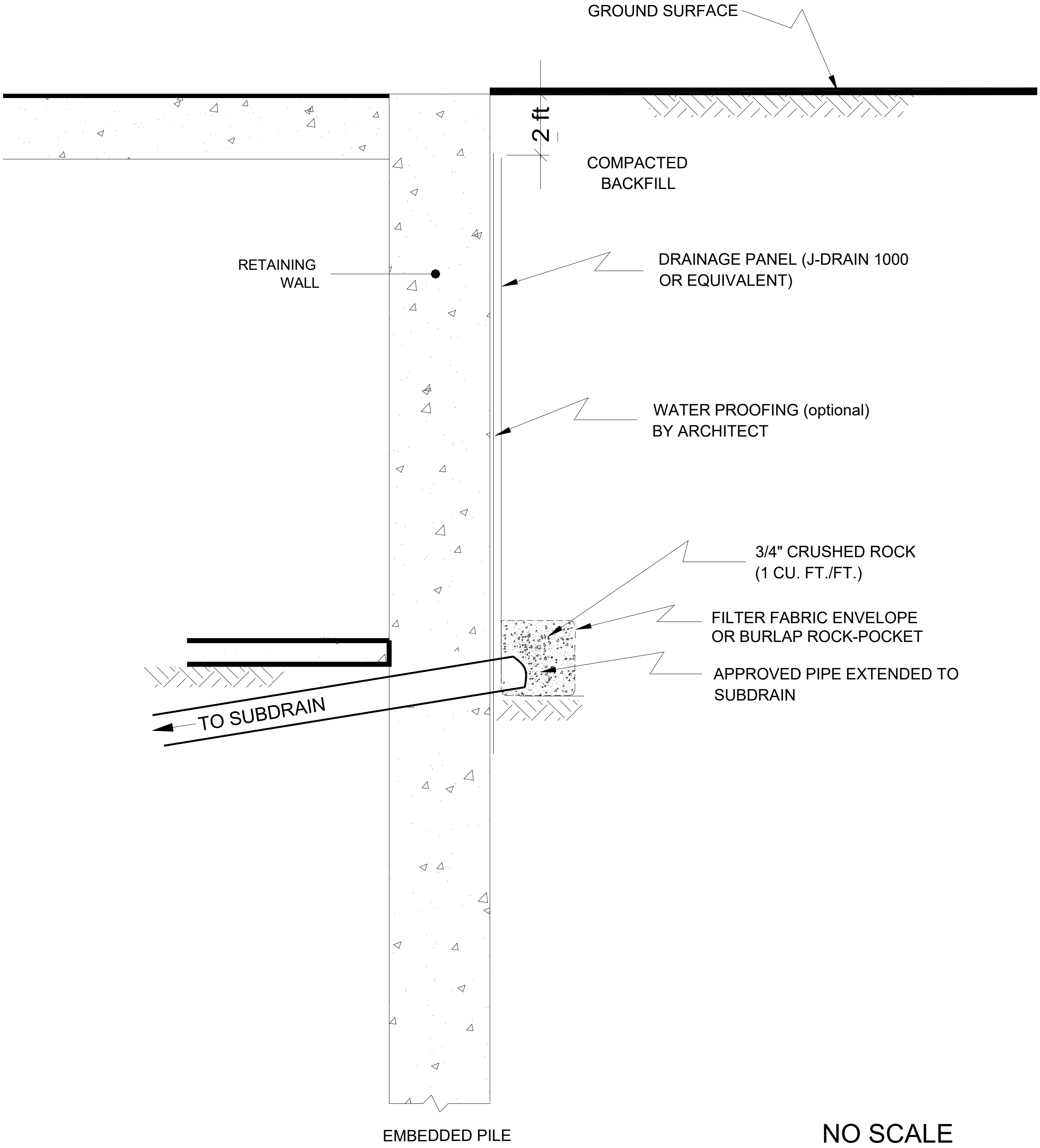
OCT 2019 | PROJECT NO. W1063-06-01 | FIG. 9



NO SCALE

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RETAINING WALL DRAIN DETAIL

1708-1732 NORTH CAHUENGA BOULEVARD
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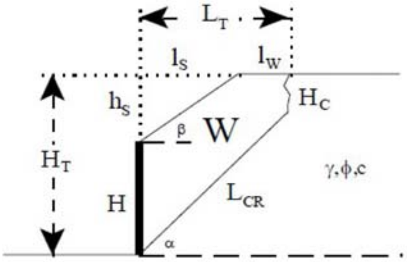
OCT 2019 | PROJECT NO. W1063-06-01 | FIG. 10

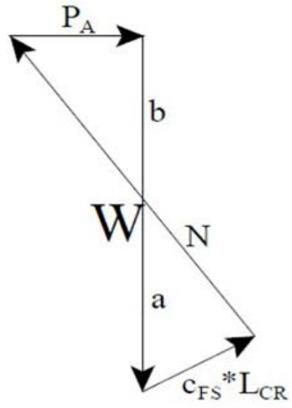
Shoring Design with Transitioned Backfill
(Vector Analysis)

Input:

Shoring Height	(H)	15.00	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)	15.0	feet
Unit Weight of Retained Soils	(g)	125.0	pcf
Friction Angle of Retained Soils	(f)	35.0	degrees
Cohesion of Retained Soils	(c)	100.0	psf
Factor of Safety	(FS)	1.25	
Factored Parameters:	(f_{FS})	29.3	degrees
	(c_{FS})	80.0	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	2.9	108	13533.2	17.1	4398.0	9135.2	2575.4
46	2.8	105	13110.2	17.0	4112.2	8998.0	2707.0
47	2.7	102	12692.9	16.8	3855.9	8837.0	2827.7
48	2.6	98	12282.4	16.7	3625.1	8657.3	2937.7
49	2.5	95	11879.5	16.5	3416.7	8462.8	3037.5
50	2.5	92	11484.4	16.4	3227.7	8256.7	3127.2
51	2.4	89	11097.3	16.2	3055.8	8041.5	3207.2
52	2.3	86	10718.1	16.1	2899.0	7819.1	3277.8
53	2.3	83	10346.8	15.9	2755.6	7591.2	3339.3
54	2.3	80	9983.1	15.7	2623.9	7359.2	3391.7
55	2.2	77	9626.8	15.6	2502.8	7124.0	3435.3
56	2.2	74	9277.7	15.4	2391.1	6886.6	3470.2
57	2.2	71	8935.4	15.3	2287.9	6647.6	3496.6
58	2.2	69	8599.7	15.1	2192.1	6407.6	3514.4
59	2.2	66	8270.3	15.0	2103.2	6167.1	3523.9
60	2.2	64	7946.8	14.8	2020.4	5926.4	3525.0
61	2.2	61	7629.0	14.6	1943.1	5685.8	3517.7
62	2.2	59	7316.5	14.5	1870.8	5445.7	3501.9
63	2.2	56	7009.1	14.3	1803.0	5206.0	3477.8
64	2.2	54	6706.5	14.2	1739.3	4967.1	3445.0
65	2.3	51	6408.4	14.1	1679.3	4729.1	3403.7
66	2.3	49	6114.5	13.9	1622.6	4491.9	3353.5
67	2.3	47	5824.6	13.8	1568.8	4255.8	3294.4
68	2.4	44	5538.4	13.6	1517.8	4020.6	3226.2
69	2.4	42	5255.6	13.5	1469.0	3786.6	3148.6
70	2.5	40	4976.0	13.3	1422.4	3553.7	3061.4



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

$P_{A,\ max}$ 3525.0 lbs/lineal foot

Equivalent Fluid Pressure (per lineal foot of shoring)

$EFP = 2*P_A/H^2$

EFP 31.3 pcf

Design Shoring for an Equivalent Fluid Pressure: 31 pcf

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SHORING PRESSURE CALCULATION

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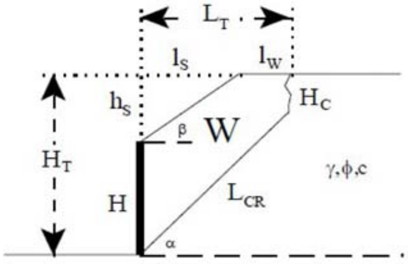
OCT 2019 | PROJECT NO. W1063-06-01 | FIG. 11

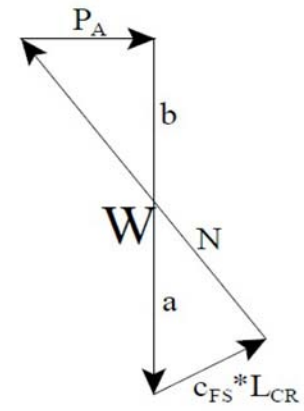
Shoring Design with Transitioned Backfill
(Vector Analysis)

Input:

Shoring Height	(H)	45.00 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)	45.0 feet
Unit Weight of Retained Soils	(g)	125.0 pcf
Friction Angle of Retained Soils	(f)	35.0 degrees
Cohesion of Retained Soils	(c)	100.0 psf
Factor of Safety	(FS)	1.25
Factored Parameters:	(f_{FS})	29.3 degrees
	(c_{FS})	80.0 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	2.9	1008	126033.2	59.5	15311.2	110722.0	31214.2
46	2.8	974	121750.2	58.7	14215.9	107534.3	32351.7
47	2.7	941	117600.8	57.9	13250.0	104350.8	33390.8
48	2.6	909	113577.9	57.1	12393.4	101184.5	34335.6
49	2.5	877	109674.3	56.3	11629.4	98044.9	35190.0
50	2.5	847	105883.1	55.5	10944.8	94938.3	35957.4
51	2.4	818	102198.0	54.8	10328.7	91869.3	36640.9
52	2.3	789	98612.7	54.1	9771.9	88840.8	37243.0
53	2.3	761	95121.6	53.5	9266.9	85854.7	37766.1
54	2.3	734	91719.1	52.8	8807.4	82911.8	38212.2
55	2.2	707	88400.2	52.2	8387.8	80012.4	38582.9
56	2.2	681	85159.9	51.6	8003.7	77156.3	38879.7
57	2.2	656	81993.8	51.0	7651.0	74342.8	39103.6
58	2.2	631	78897.5	50.5	7326.4	71571.1	39255.4
59	2.2	607	75867.1	49.9	7026.9	68840.2	39335.7
60	2.2	583	72898.7	49.4	6750.0	66148.7	39344.8
61	2.2	560	69988.7	48.9	6493.4	63495.3	39282.8
62	2.2	537	67133.8	48.5	6255.2	60878.6	39149.3
63	2.2	515	64330.7	48.0	6033.6	58297.1	38943.9
64	2.2	493	61576.4	47.6	5827.1	55749.3	38665.9
65	2.3	471	58868.0	47.2	5634.2	53233.8	38314.3
66	2.3	450	56202.7	46.7	5453.9	50748.9	37887.6
67	2.3	429	53578.0	46.4	5284.8	48293.2	37384.4
68	2.4	408	50991.3	46.0	5126.2	45865.1	36802.7
69	2.4	388	48440.3	45.6	4977.0	43463.3	36140.3
70	2.5	367	45922.7	45.2	4836.3	41086.3	35394.6



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

$P_{A,\ max}$ 39344.8 lbs/lineal foot

Equivalent Fluid Pressure (per lineal foot of shoring)

$EFP = 2*P_A/H^2$

EFP 38.9 pcf

Design Shoring for an Equivalent Fluid Pressure: 39 pcf

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SHORING PRESSURE CALCULATION

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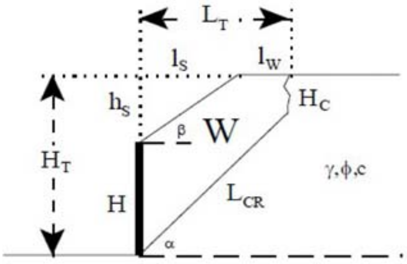
LOS ANGELES, CALIFORNIA

OCT 2019 | PROJECT NO. W1063-06-01 | FIG. 12

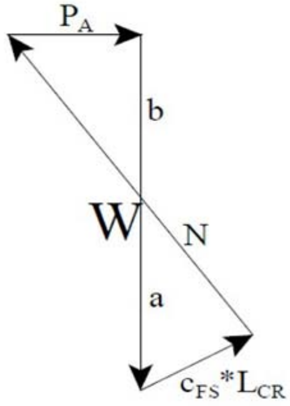
Shoring Design with Transitioned Backfill
(Vector Analysis)

Input:

Shoring Height	(H)	80.00 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)	80.0 feet
Unit Weight of Retained Soils	(g)	125.0 pcf
Friction Angle of Retained Soils	(f)	35.0 degrees
Cohesion of Retained Soils	(c)	100.0 psf
Factor of Safety	(FS)	1.25
Factored Parameters:	(f_{FS})	29.3 degrees
	(c_{FS})	80.0 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	2.9	3196	399470.7	109.0	28043.2	371427.4	104710.9
46	2.8	3086	385805.7	107.3	26003.5	359802.2	108246.6
47	2.7	2981	372585.4	105.7	24209.9	348375.5	111475.2
48	2.6	2878	359782.1	104.2	22623.0	337159.2	114410.3
49	2.5	2779	347369.8	102.7	21210.9	326158.9	117064.0
50	2.5	2683	335324.4	101.2	19948.2	315376.3	119447.2
51	2.4	2589	323623.3	99.9	18813.7	304809.6	121569.3
52	2.3	2498	312245.5	98.5	17790.3	294455.2	123438.7
53	2.3	2409	301171.6	97.3	16863.5	284308.0	125062.6
54	2.3	2323	290383.1	96.1	16021.4	274361.7	126447.3
55	2.2	2239	279863.2	94.9	15253.6	264609.6	127598.0
56	2.2	2157	269595.8	93.8	14551.6	255044.2	128519.0
57	2.2	2077	259566.2	92.8	13908.0	245658.2	129213.8
58	2.2	1998	249760.2	91.8	13316.4	236443.9	129685.0
59	2.2	1921	240164.9	90.8	12771.3	227393.7	129934.2
60	2.2	1846	230767.9	89.9	12267.9	218500.0	129962.5
61	2.2	1772	221557.6	89.0	11802.1	209755.5	129769.9
62	2.2	1700	212523.1	88.1	11370.3	201152.8	129355.7
63	2.2	1629	203654.1	87.3	10969.3	192684.8	128718.4
64	2.2	1560	194940.8	86.5	10596.1	184344.7	127855.5
65	2.3	1491	186374.0	85.8	10248.3	176125.7	126764.0
66	2.3	1424	177944.9	85.1	9923.7	168021.3	125439.7
67	2.3	1357	169645.3	84.4	9620.2	160025.2	123877.6
68	2.4	1292	161467.3	83.7	9336.0	152131.2	122071.7
69	2.4	1227	153403.1	83.1	9069.6	144333.6	120015.1
70	2.5	1164	145445.8	82.5	8819.3	136626.4	117699.7



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

$P_{A,\ max}$ 129962.5 lbs/lineal foot

Equivalent Fluid Pressure (per lineal foot of shoring)

$EFP = 2*P_A/H^2$

EFP 40.6 pcf

Design Shoring for an Equivalent Fluid Pressure: 41 pcf

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OCT 2019
PROJECT NO. W1063-06-01
FIG. 14

BORING PERCOLATION TEST FIELD LOG

Date: 9/15/2019

Project Number: W1063-06-01

Project Location: Cahuenga & Hollywood

Earth Description: SW

Tested By: JOA/JA

Liquid Description: Clear Clean Tap Water

Measurement Method: Sounder

Boring/Test Number: Boring B1

Diameter of Boring: 8 inches

Diameter of Casing: 2 inches

Depth of Boring: 90.18 feet

Depth to Invert of BMP: 79.15 feet

Depth to Water Table: 106 feet

Depth to Initial Water Depth (d_1): 949.8 inches

Start Time for Pre-Soak: 2:30 PM

Start Time for Standard: 6:50 AM

Water Remaining in Boring (Y/N): Yes

Standard Time Interval Between Readings: 30 min

Reading Number	Time Start (hh:mm)	Time End (hh:mm)	Elapsed Time Δtime (min)	Water Drop During Standard Time Interval, Δd (in)	Water Drop Per Hour (in)	Soil Description Notes Comments
1	6:55 AM	7:25 AM	30	0.6	1.2	
2	7:26 AM	7:56 AM	30	0.8	1.68	
3	7:56 AM	8:26 AM	30	4.7	9.36	
4	8:27 AM	8:57 AM	30	3.8	7.68	
5	8:57 AM	9:27 AM	30	4.1	8.16	
6	9:28 AM	9:58 AM	30	4.2	8.4	
7	9:58 AM	10:28 AM	30	3.5	6.96	
8	10:29 AM	10:59 AM	30	3.7	7.44	

FIGURE 15

APPENDIX A

APPENDIX A

FIELD INVESTIGATION

The site was explored on September 14, 2019, by excavating one 8-inch-diameter boring to a depth of approximately 150½ feet below the existing ground surface utilizing a truck-mounted hollow-stem auger drilling machine. 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 (auto-hammer). The California Modified Sampler was equipped with 1-inch high by 2⅜-inch diameter brass sampler rings to facilitate soil removal and testing. Bulk samples were also obtained.

The soil conditions encountered in the boring were visually examined, classified and logged in general accordance with the Unified Soil Classification System (USCS). The log of the boring is presented on Figure A1. The log depicts the soil and geologic conditions encountered and the depth at which samples were obtained. The log also includes our interpretation of the conditions between sampling intervals. Therefore, the log contains both observed and interpreted data. We determined the lines designating the interface between soil materials on the log using visual observations, penetration rates, excavation characteristics and other factors. The transition between materials may be abrupt or gradual. Where applicable, the boring log was revised based on subsequent laboratory testing. The location of the boring is shown on Figure 2.

PROJECT NO. W1063-06-01

BORING 1

ELEV. (MSL.) **--** DATE COMPLETED **9/14/19**

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: 3" BASE: NONE** **ARTIFICIAL FILL** Sand with Silt, well-graded, very loose, moist, reddish brown trace brick fragments., concrete slab at 19" approximately 20" thick.			
4				SM	**ALLUVIUM** Silty Sand, poorly graded, very loose, slightly moist, brown, fine- to medium-grained, trace fine gravel, trace pockets of light grayish brown well-graded sand.			
6	B1@5'					6	105.1	10.1
8	B1@7'			SW	Sand, well-graded, very loose, slightly moist, brown, trace silt and fine gravel, thin layer of silt at 7', soft, brown.	6	96.8	14.4
10	B1@10' BULK 10-15'			SM	Silty Sand, poorly graded, very loose, slightly moist, reddish brown, fine- to medium-grained, trace coarse-grained.	6	102.1	13.1
16	B1@15'				- loose	14	109.1	11.7
20	B1@20' BULK 20-25'			SP	Sand, poorly graded, medium dense, slightly moist, orangish brown, trace silt, fine-grained with trace medium-grained.	35	96.1	6.9
22					- loose			
26	B1@25'				- thin layers of interbedded silt	13	113.9	14.0
28					- trace fine gravel			

W1063-06-01 BORING LOGS.GPJ

Figure A1,
Log of Boring 1, Page 1 of 6

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 1

ELEV. (MSL.) -- DATE COMPLETED 9/14/19

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' ■			SP		14	103.3	9.7
32								
34								
	B1@35' ■			ML	Silt, trace fine sand, orangish brown, semi-plastic, slightly moist, firm, medium dense.	21	109.0	16.7
36				SP-SM	Sand with Silt, poorly graded, medium dense, dry, orangish brown, fine-grained, trace medium-grained.			
38								
40	B1@40' ■					37	110.0	4.6
42								
44				SM	Silty Sand, poorly graded, medium dense, dry, orangish brown, fine-grained, trace medium-grained.			
	B1@45' ■					25	113.0	10.6
46								
48					- gradually increasing grain size			
50	B1@50' ■			SP	Sand, poorly graded, medium dense, slightly moist, orangish brown, fine- to medium-grained, trace coarse-grained, trace fine gravel	29	117.1	11.7
52								
54								
				SM	Silty Sand, poorly graded, medium dense, dry, orangish brown, fine-grained, trace medium-grained.			
56								
58					- gradually increasing grain size			

Figure A1,
Log of Boring 1, Page 2 of 6

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. W1063-06-01

BORING 1

ELEV. (MSL.) -- DATE COMPLETED 9/14/19

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' ■				Sand, poorly graded, medium dense, dry, orangish brown, fine- to medium-grained, trace coarse-grained, trace silt.	42	105.4	9.3
62					- gradually increasing grain size			
64				SP				
66								
68								
70	B1@70' ■				Sand, well-graded, dense, dry, orangish brown, trace fine gravel	50 (5")	105.2	10.3
72				SW				
74								
76					Silty Sand, poorly graded, dense, slightly moist, brown, fine- to medium-grained, trace coarse-grained, trace clay.			
78								
80	B1@80' ■					61	124.2	11.8
82				SM	- gravel layer (3' thick)			
84								
86								
88								

Figure A1,
Log of Boring 1, Page 3 of 6

W1063-06-01 BORING LOGS.GPJ

SAMPLE SYMBOLS

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

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. W1063-06-01

BORING 1

ELEV. (MSL.) --　DATE COMPLETED **9/14/19**

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 (%)
90	B1@89.5'			SM	- very dense	50 (2")	103.2	12.5
92								
					- added water			
				SP-SM	Sand with Silt, poorly graded, medium dense, slightly moist, orangish brown, fine- to medium-grained.			
94								
	B1@95'	■				40	116.5	15.5
96								
					- thin interbeds of sandy silt, firm, slightly moist, orangish brown, plastic			
98								
100	B1@100'	■				21	118.7	15.1
102								
104								
				SW	Sand, well-graded, medium dense, saturated, orangish brown.			
			▼		- groundwater			
106								
108								
110	B1@110'	■				38	110.8	15.6
112								
114								
116								
118								

Figure A1,
Log of Boring 1, Page 4 of 6

SAMPLE SYMBOLS

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

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BORING 1

ELEV. (MSL.) -- DATE COMPLETED 9/14/19

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 (%)
120	B1@120' ☐				- very dense (cobble in sampler), no recovery	50 (3")		
122				SW				
124								
126								
128					Sand with Silt, poorly graded, very dense, wet, orangish brown, fine- to medium-grained, trace fine gravel.			
130	B1@130' ■					50 (4")	123.5	12.4
132								
134								
136				SP-SM				
138								
140	B1@140' ☐				- no recovery, cobble in sampler	50 (6")		
142								
144					Sand, well-graded, very dense, saturated, orangish brown, trace silt and gravel.			
146				SW				
148								

Figure A1,
Log of Boring 1, Page 5 of 6

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. W1063-06-01

BORING 1

ELEV. (MSL.) -- DATE COMPLETED 9/14/19

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 (%)
150	B1@150'	■	░		SW		50 (2")	116.2	15.2
						Total depth of boring: 150.5 feet Fill to 3 feet. Groundwater encountered at 102.7 feet; stabilized at 106 feet after 12 hours. Perc well set / presoaked 09/14/19.			

Figure A1,
Log of Boring 1, Page 6 of 6

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

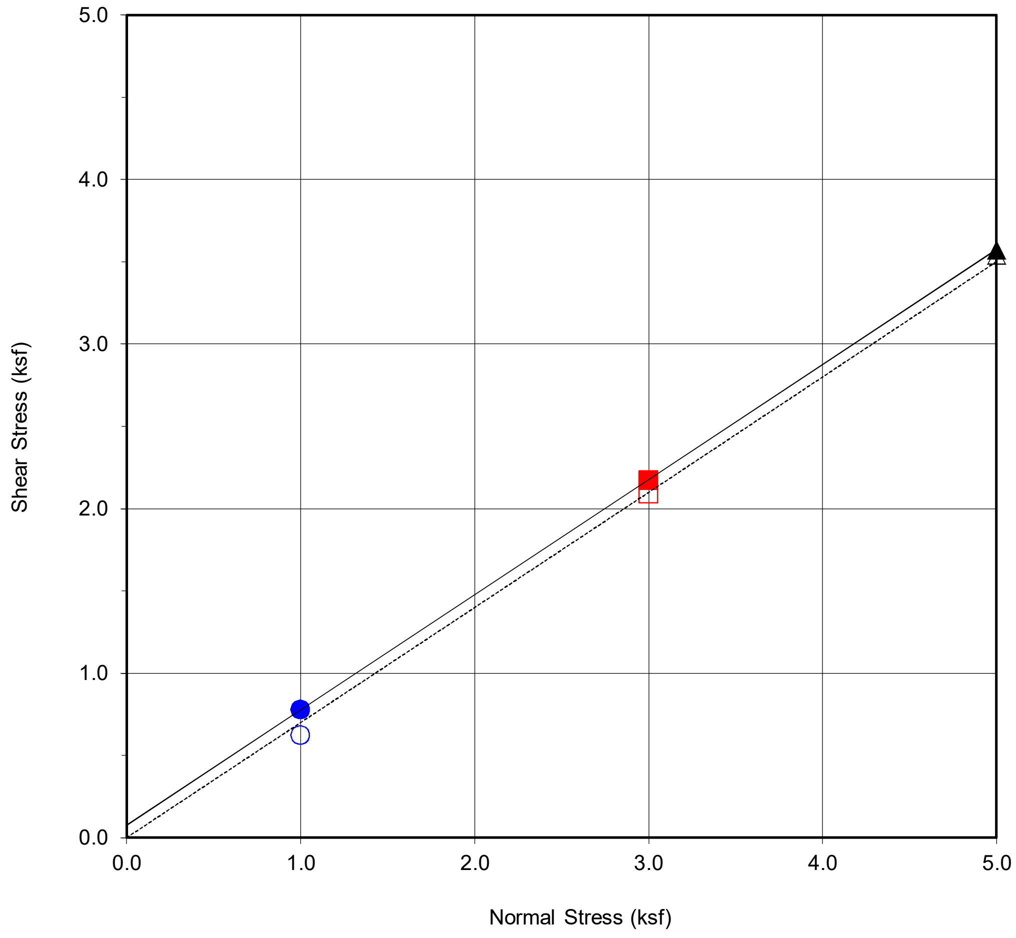
NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

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, corrosivity, and in-place dry density and moisture content. The results of the laboratory tests are summarized in Figures B1 through B15. The in-place dry density and moisture content of the samples tested are presented on the boring log, Appendix A.



Boring No.	**B1**
Sample No.	**B1@10**
Depth (ft)	**10**
Sample Type:	Ring

Normal Stress (kip/ft2)	1	3	5
Peak Shear Stress (kip/ft²)	● 0.78	■ 2.17	▲ 3.57
Shear Stress @ End of Test (ksf)	○ 0.62	□ 2.09	△ 3.54
Deformation Rate (in./min.)	0.05	0.05	0.05
Initial Sample Height (in.)	1.0	1.0	1.0
Ring Inside Diameter (in.)	2.375	2.375	2.375
Initial Moisture Content (%)	13.1	13.1	12.3
Initial Dry Density (pcf)	97.4	102.3	103.1
Initial Degree of Saturation (%)	48.5	54.4	52.5
Soil Height Before Shearing (in.)	1.2	1.2	1.2
Final Moisture Content (%)	18.1	16.4	14.5

Soil Identification:

Reddish Brown Silty Sand with Gravel

Strength Parameters

	C (psf)	ϕ (^{o})
Peak	75	35.0
Ultimate	0	35.0

GEOCON

DIRECT SHEAR TEST RESULTS

Consolidated Drained ASTM D-3080

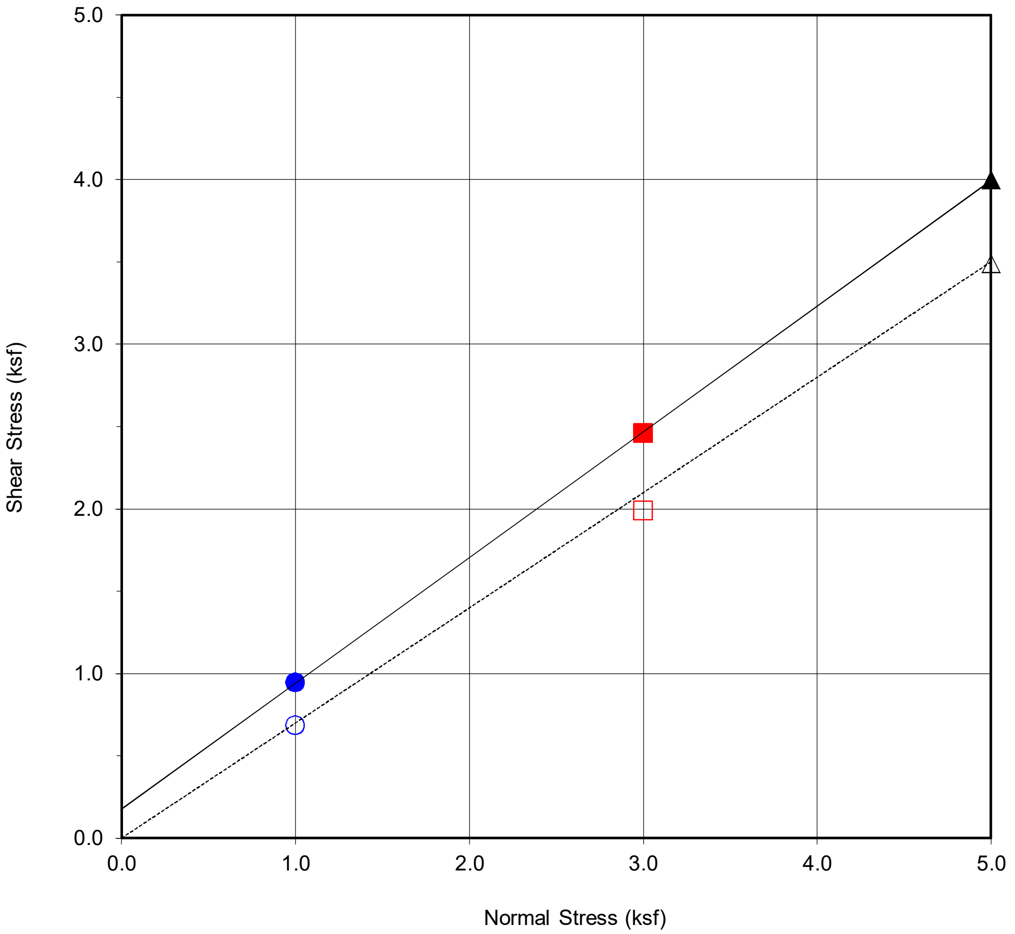
Checked by: JTA

Project No.: W1063-06-01

1708-1732 N. Cahuenga Blvd.
6381-6385 W. Hollywood Blvd.
Los Angeles, California

Oct 19

Figure B1



Boring No.	**B1**
Sample No.	**B1@20**
Depth (ft)	**20**
Sample Type:	Ring

Soil Identification:

Orangish Brown Sand

Strength Parameters

	C (psf)	ϕ (^{o})
Peak	177	37.4
Ultimate	0	35.0

Normal Stress (kip/ft2)	1	3	5
Peak Shear Stress (kip/ft²)	● 0.94	■ 2.46	▲ 4.00
Shear Stress @ End of Test (ksf)	○ 0.68	□ 1.99	△ 3.49
Deformation Rate (in./min.)	0.05	0.05	0.05
Initial Sample Height (in.)	1.0	1.0	1.0
Ring Inside Diameter (in.)	2.375	2.375	2.375
Initial Moisture Content (%)	4.4	5.7	6.9
Initial Dry Density (pcf)	95.2	95.1	96.3
Initial Degree of Saturation (%)	15.4	20.0	24.9
Soil Height Before Shearing (in.)	1.2	1.2	1.2
Final Moisture Content (%)	22.5	20.9	19.5

GEOCON

DIRECT SHEAR TEST RESULTS

Consolidated Drained ASTM D-3080

Checked by: JTA

Project No.: W1063-06-01

1708-1732 N. Cahuenga Blvd.
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Los Angeles, California

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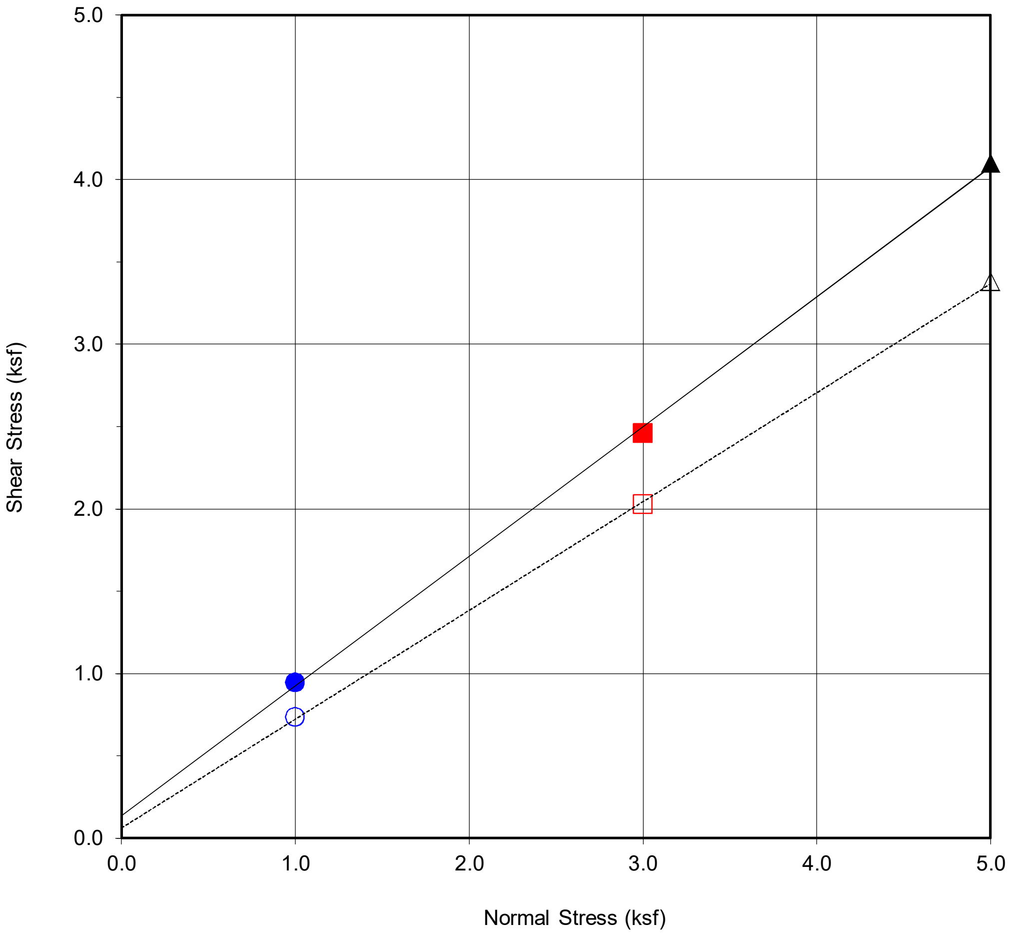
Figure B2



Boring No.	**B1**
Sample No.	**B1@35'**
Depth (ft)	**35**
Sample Type:	Ring

Normal Stress (kip/ft2)	1	3	5
Peak Shear Stress (kip/ft²)	● 0.94	■ 2.46	▲ 4.10
Shear Stress @ End of Test (ksf)	○ 0.73	□ 2.02	△ 3.38
Deformation Rate (in./min.)	0.05	0.05	0.05
Initial Sample Height (in.)	1.0	1.0	1.0
Ring Inside Diameter (in.)	2.375	2.375	2.375
Initial Moisture Content (%)	16.7	17.4	12.9
Initial Dry Density (pcf)	108.8	108.1	113.0
Initial Degree of Saturation (%)	82.3	84.1	70.8
Soil Height Before Shearing (in.)	1.2	1.2	1.2
Final Moisture Content (%)	18.7	17.5	15.3

Soil Identification:

Orangish Brown Silt

Strength Parameters

	C (psf)	ϕ (°)
Peak	135	38.3
Ultimate	62	33.5

GEOCON

DIRECT SHEAR TEST RESULTS

Consolidated Drained ASTM D-3080

Checked by: JTA

Project No.: W1063-06-01

1708-1732 N. Cahuenga Blvd.
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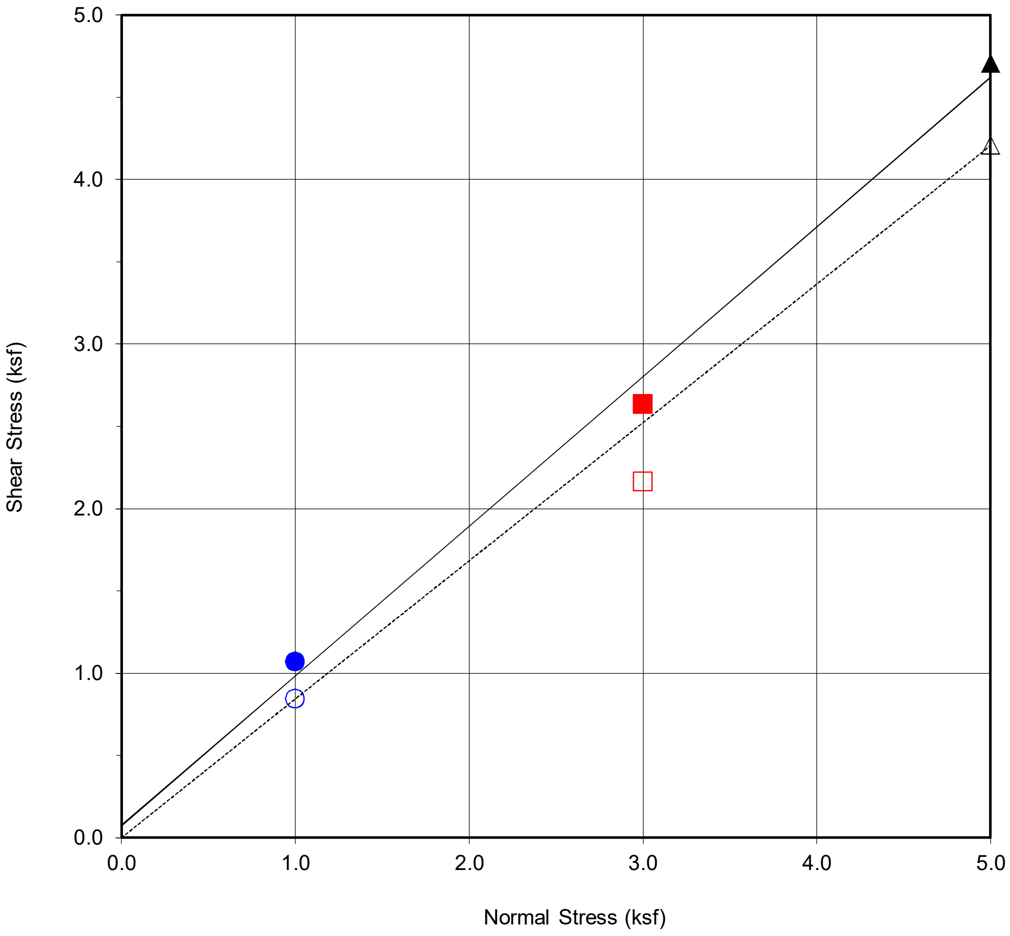
Figure B3



Boring No.	**B1**
Sample No.	**B1@50**
Depth (ft)	**50**
Sample Type:	Ring

Soil Identification:		
Orangish Brown Sand		
Strength Parameters		
	C (psf)	ϕ (°)
Peak	74	42.3
Ultimate	0	40.1

Normal Stress (kip/ft2)	1	3	5
Peak Shear Stress (kip/ft²)	● 1.07	■ 2.63	▲ 4.71
Shear Stress @ End of Test (ksf)	○ 0.84	□ 2.16	△ 4.21
Deformation Rate (in./min.)	0.05	0.05	0.05
Initial Sample Height (in.)	1.0	1.0	1.0
Ring Inside Diameter (in.)	2.375	2.375	2.375
Initial Moisture Content (%)	12.2	12.3	11.7
Initial Dry Density (pcf)	113.6	115.2	119.6
Initial Degree of Saturation (%)	68.2	71.9	77.2
Soil Height Before Shearing (in.)	1.2	1.2	1.2
Final Moisture Content (%)	16.0	14.4	13.3

GEOCON

DIRECT SHEAR TEST RESULTS

Consolidated Drained ASTM D-3080

Checked by: JTA

Project No.: W1063-06-01

1708-1732 N. Cahuenga Blvd.
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Los Angeles, California

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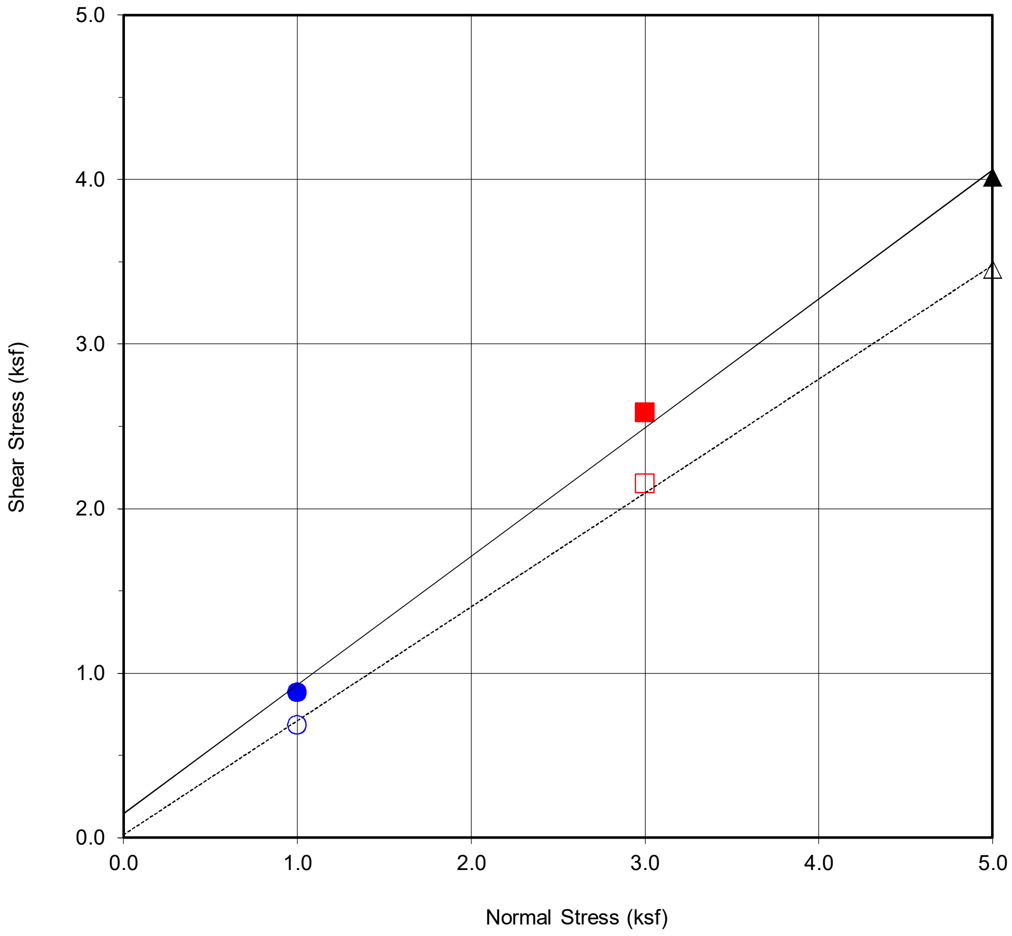
Figure B4



Boring No.	**B1**
Sample No.	**B1@70**
Depth (ft)	**70**
Sample Type:	Ring

Soil Identification:		
Orangish Brown Sand		
Strength Parameters		
	C (psf)	ϕ (°)
Peak	145	38.0
Ultimate	18	34.7

Normal Stress (kip/ft2)	1	3	5
Peak Shear Stress (kip/ft²)	● 0.88	■ 2.58	▲ 4.01
Shear Stress @ End of Test (ksf)	○ 0.68	□ 2.15	△ 3.45
Deformation Rate (in./min.)	0.05	0.05	0.05
Initial Sample Height (in.)	1.0	1.0	1.0
Ring Inside Diameter (in.)	2.375	2.375	2.375
Initial Moisture Content (%)	9.0	10.3	9.7
Initial Dry Density (pcf)	105.8	107.7	103.3
Initial Degree of Saturation (%)	41.1	49.4	41.4
Soil Height Before Shearing (in.)	1.2	1.2	1.2
Final Moisture Content (%)	17.7	16.4	16.8

GEOCON

DIRECT SHEAR TEST RESULTS

Consolidated Drained ASTM D-3080

Checked by: JTA

Project No.: W1063-06-01

1708-1732 N. Cahuenga Blvd.
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Los Angeles, California

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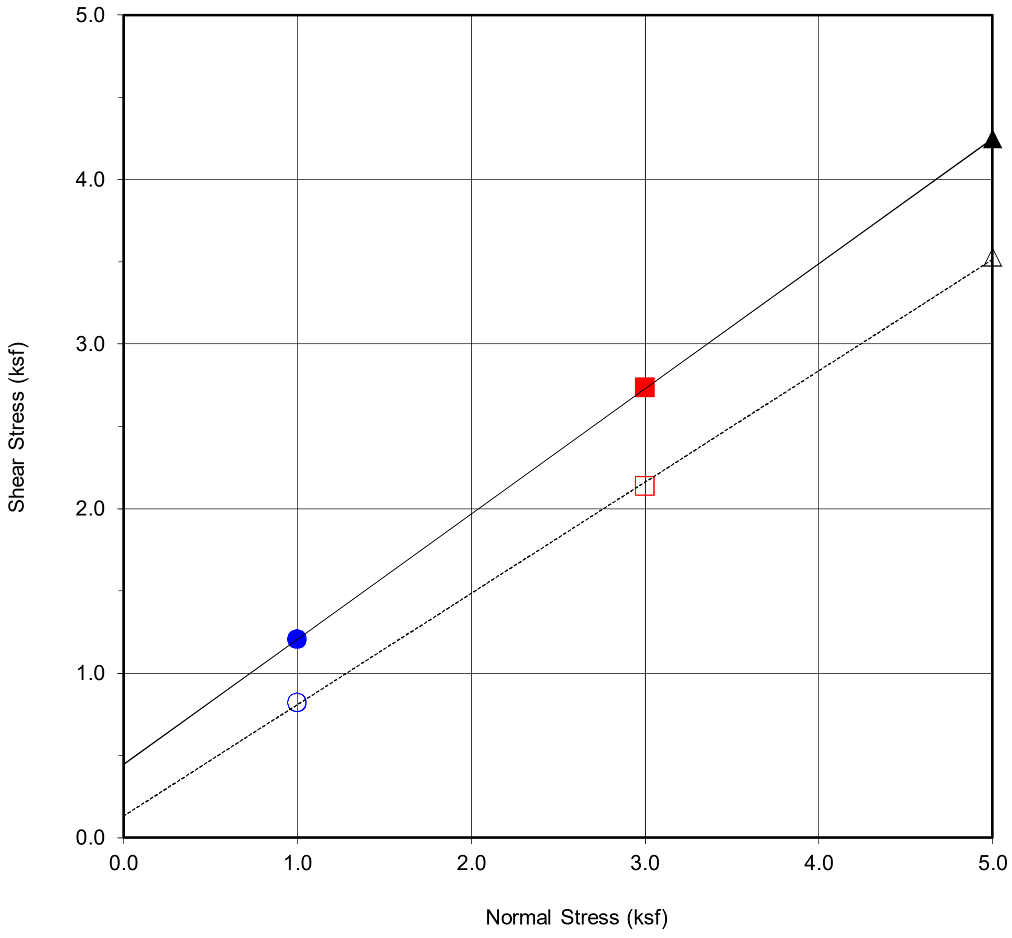
Figure B5



Boring No.	**B1**
Sample No.	**B1@95**
Depth (ft)	**95**
Sample Type:	Ring

Soil Identification:		
Orangish Brown Sand with Silt		
Strength Parameters		
	C (psf)	ϕ (°)
Peak	445	37.3
Ultimate	131	34.1

Normal Stress (kip/ft2)	1	3	5
Peak Shear Stress (kip/ft²)	● 1.20	■ 2.73	▲ 4.25
Shear Stress @ End of Test (ksf)	○ 0.82	□ 2.14	△ 3.53
Deformation Rate (in./min.)	0.05	0.05	0.05
Initial Sample Height (in.)	1.0	1.0	1.0
Ring Inside Diameter (in.)	2.375	2.375	2.375
Initial Moisture Content (%)	14.9	15.5	15.9
Initial Dry Density (pcf)	114.4	114.6	114.5
Initial Degree of Saturation (%)	85.0	88.7	91.0
Soil Height Before Shearing (in.)	1.2	1.2	1.2
Final Moisture Content (%)	15.8	14.8	14.8

GEOCON

DIRECT SHEAR TEST RESULTS

Consolidated Drained ASTM D-3080

Checked by: JTA

Project No.: W1063-06-01

1708-1732 N. Cahuenga Blvd.
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Los Angeles, California

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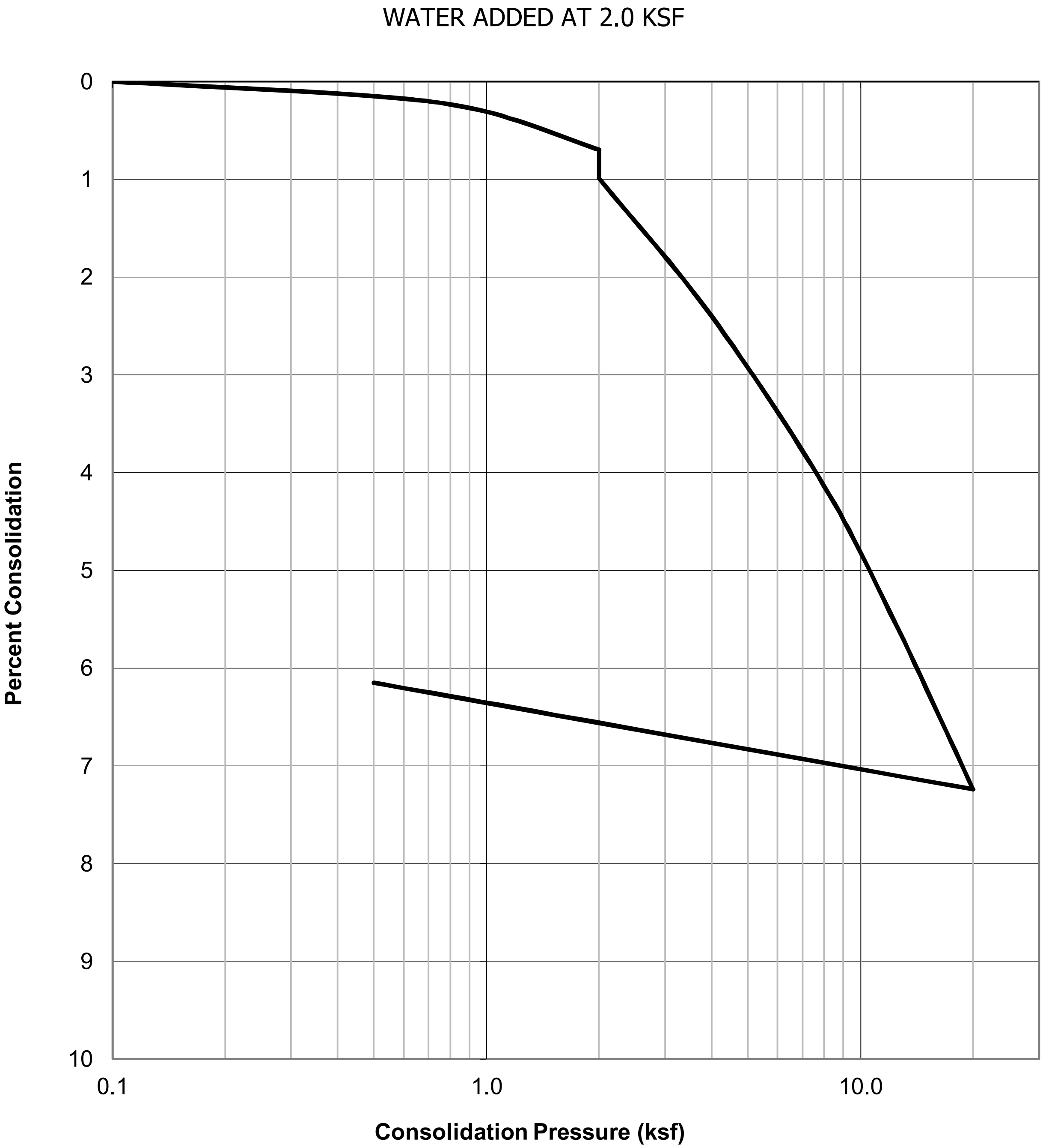
Figure B6



SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B1@15	Reddish Brown Silty Sand	107.9	11.3	14.2

GEOCON

CONSOLIDATION TEST RESULTS

ASTM D-2435

Checked by: JTA

Project No.: W1063-06-01

1708-1732 N. Cahuenga Blvd.
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Los Angeles, California

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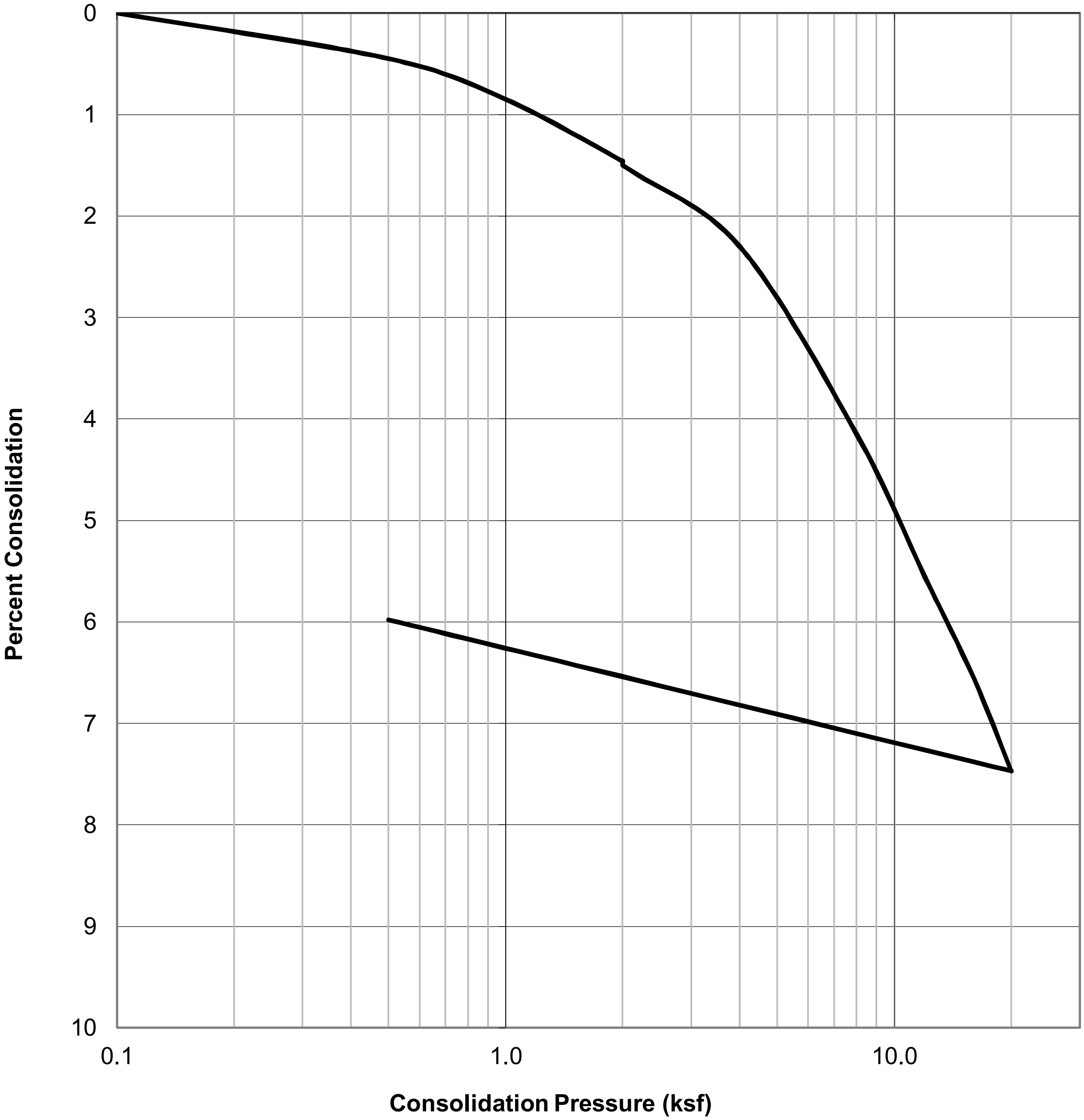
Figure B7

WATER ADDED AT 2.0 KSF



SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B1@25	Orangish Brown Silt (interbed)	111.3	14.4	13.9

GEOCON

CONSOLIDATION TEST RESULTS

ASTM D-2435

Checked by: JTA

Project No.: W1063-06-01

1708-1732 N. Cahuenga Blvd.
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Los Angeles, California

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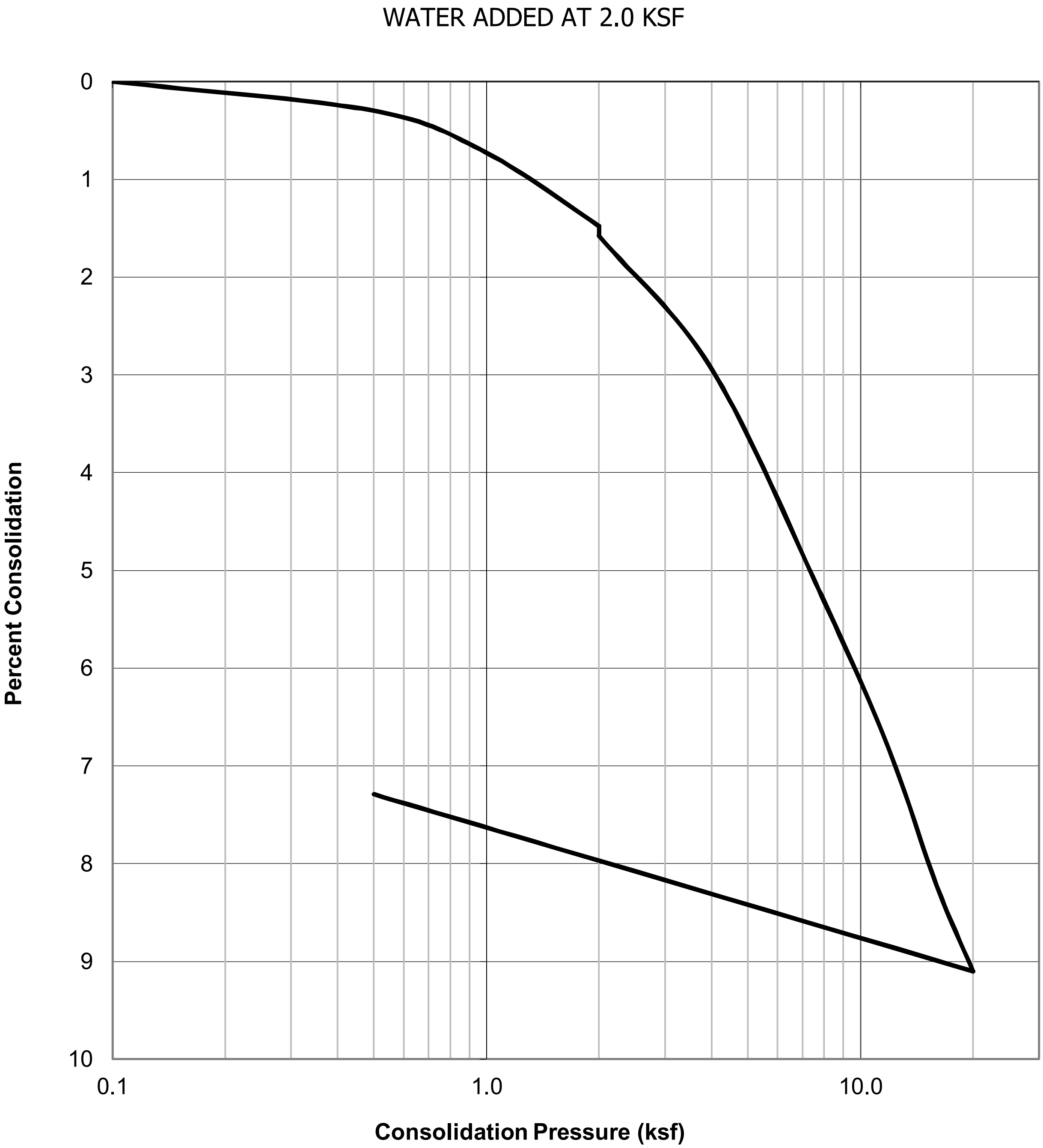
Figure B8



SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B1@35	Orangish Brown Silt	103.2	18.7	17.5

GEOCON

CONSOLIDATION TEST RESULTS

ASTM D-2435

Checked by: JTA

Project No.: W1063-06-01

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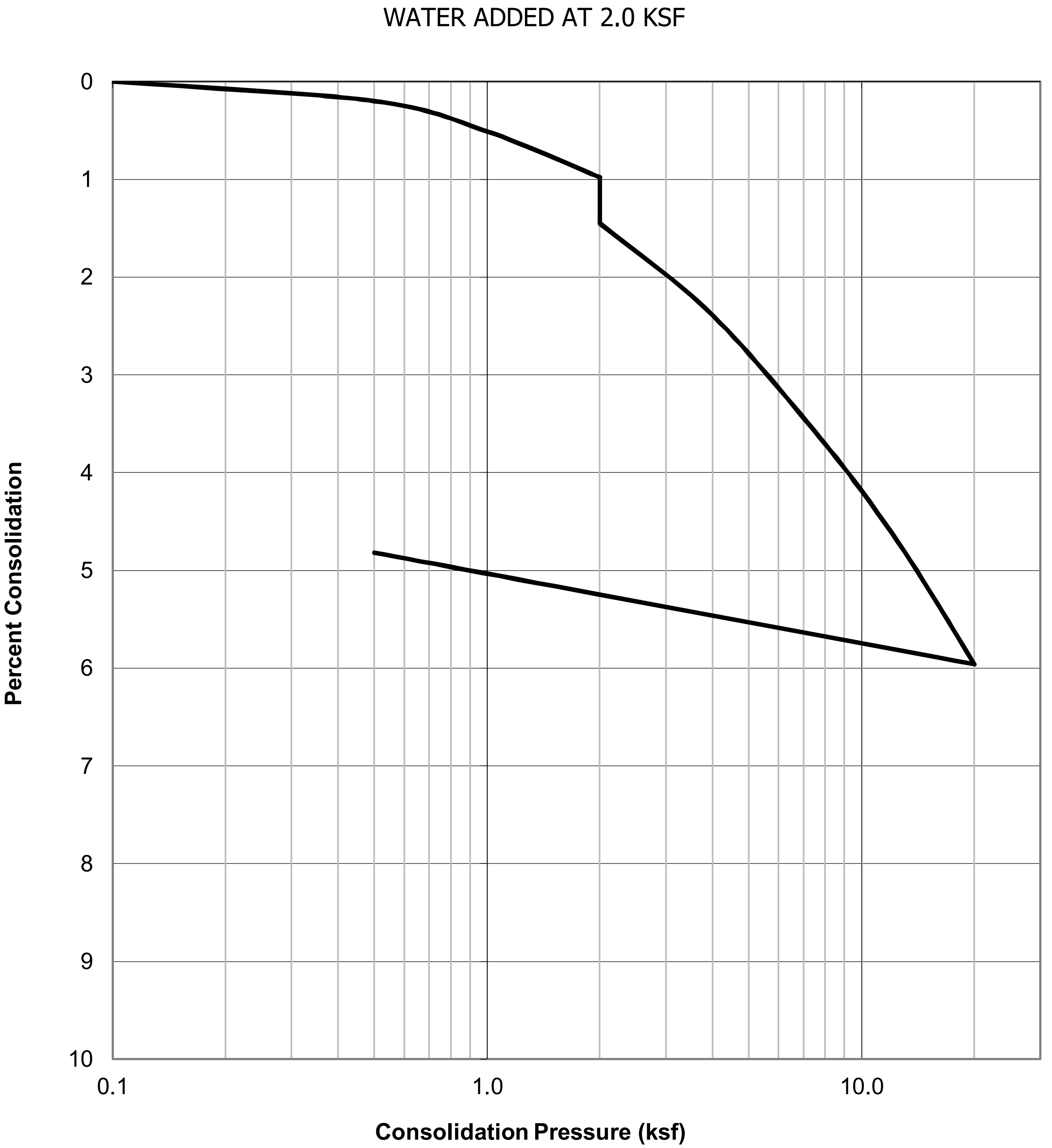
Figure B9



SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B1@45	Orangish Brown Silty Sand	104.9	11.7	17.2

GEOCON

CONSOLIDATION TEST RESULTS

ASTM D-2435

Checked by: JTA

Project No.: W1063-06-01

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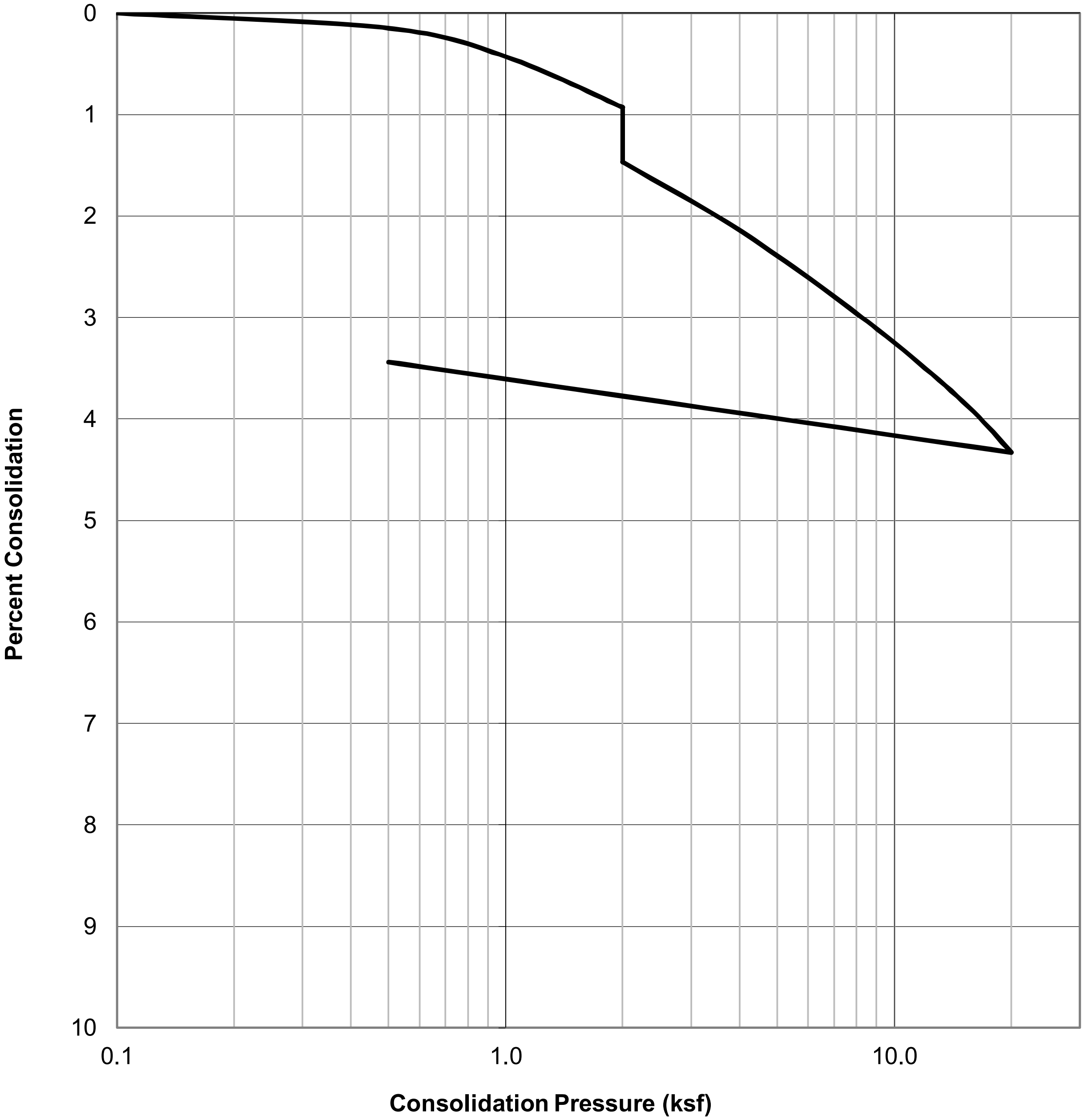
Los Angeles, California

Oct 19

WATER ADDED AT 2.0 KSF



SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B1@60	Orangish Brown Sand	112.9	9.7	13.7

GEOCON

CONSOLIDATION TEST RESULTS

ASTM D-2435

Checked by: JTA

Project No.: W1063-06-01

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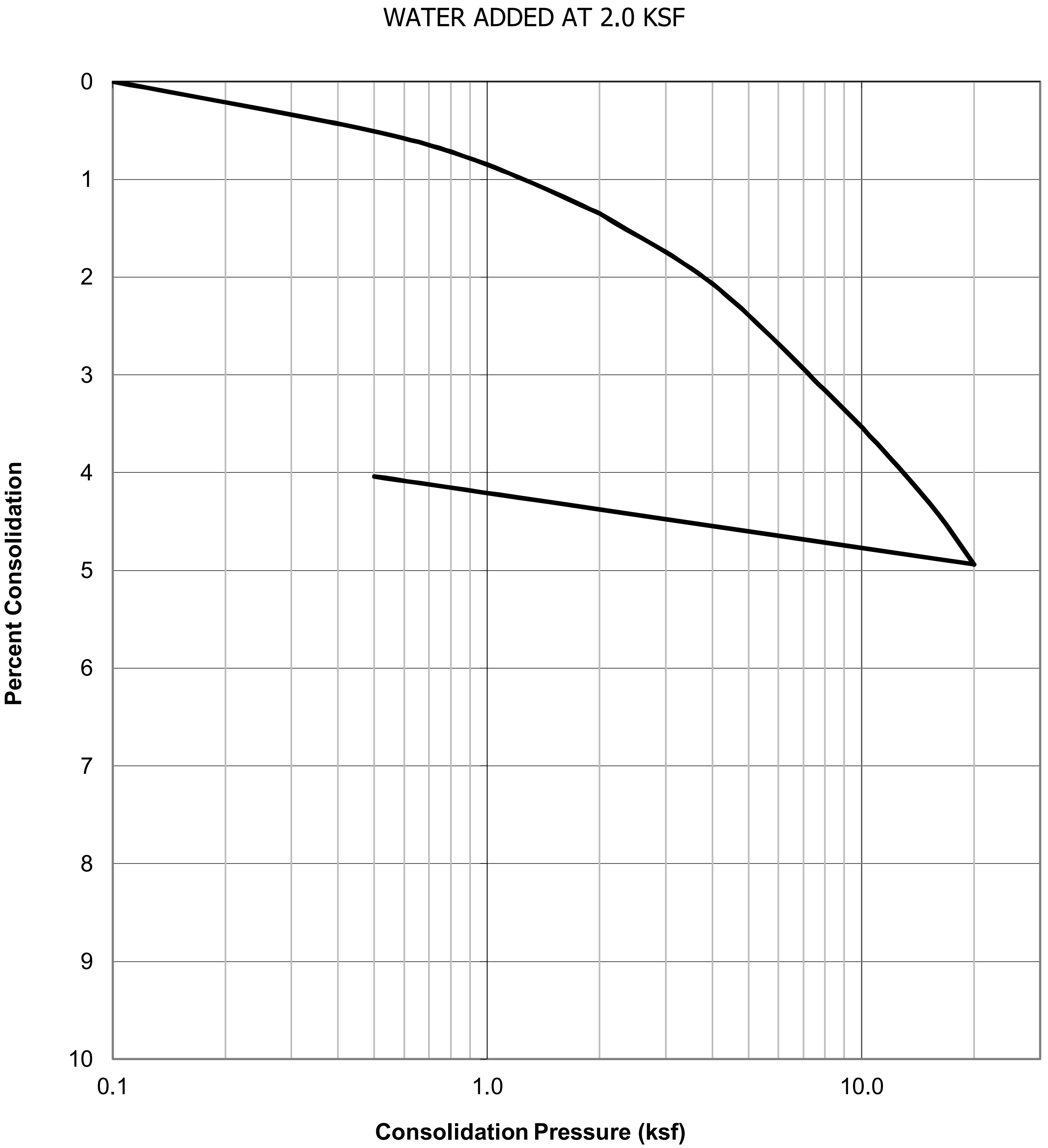

SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B1@89.5	Brown Silty Sand	115.0	13.8	13.0

GEOCON

CONSOLIDATION TEST RESULTS

ASTM D-2435

Checked by: JTA

Project No.: W1063-06-01

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Los Angeles, California

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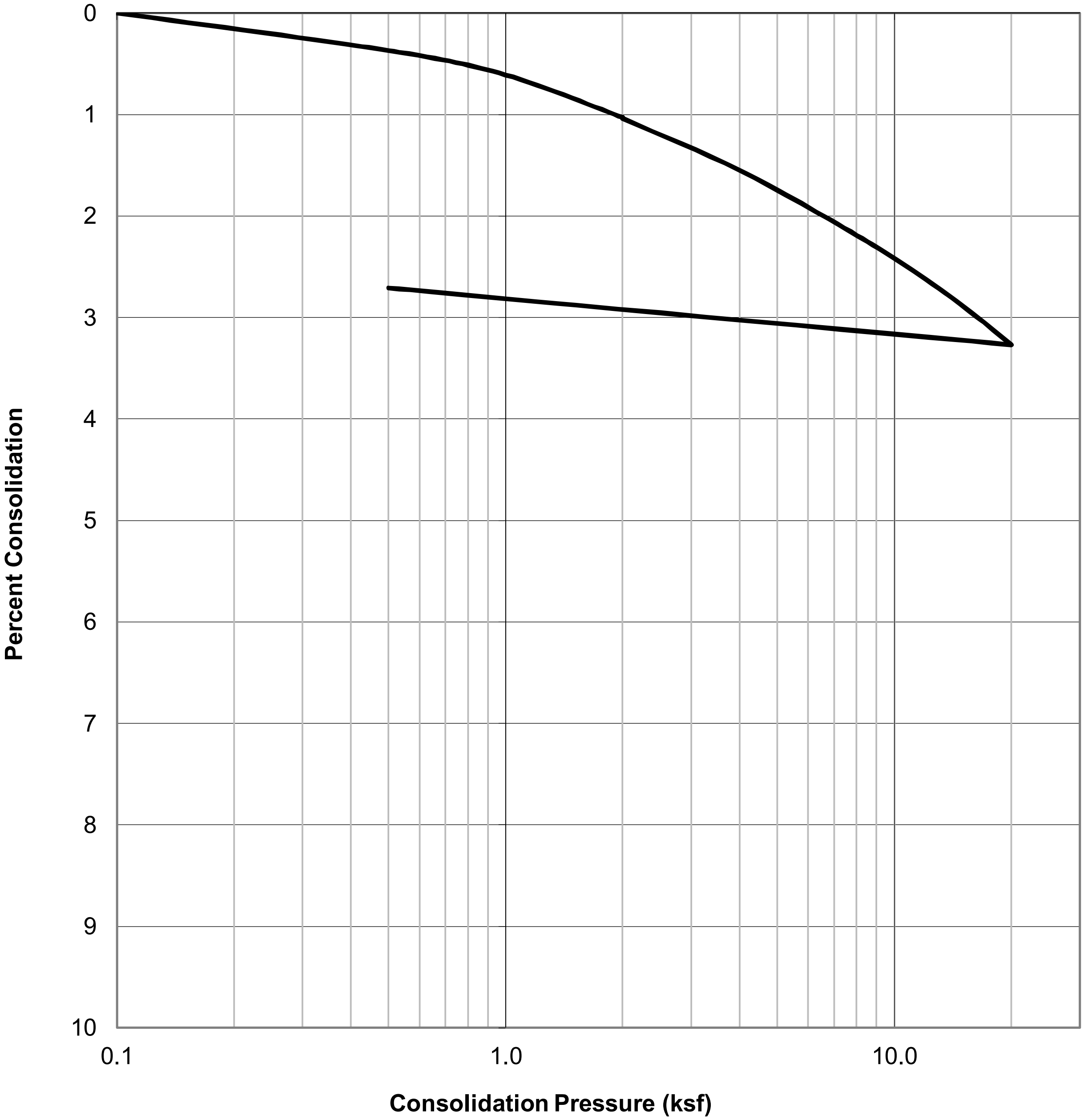
WATER ADDED AT 2.0 KSF



SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B1@110	Orangish Brown Sand	115.5	17.1	14.5

GEOCON

CONSOLIDATION TEST RESULTS

ASTM D-2435

Checked by: JTA

Project No.: W1063-06-01

1708-1732 N. Cahuenga Blvd.
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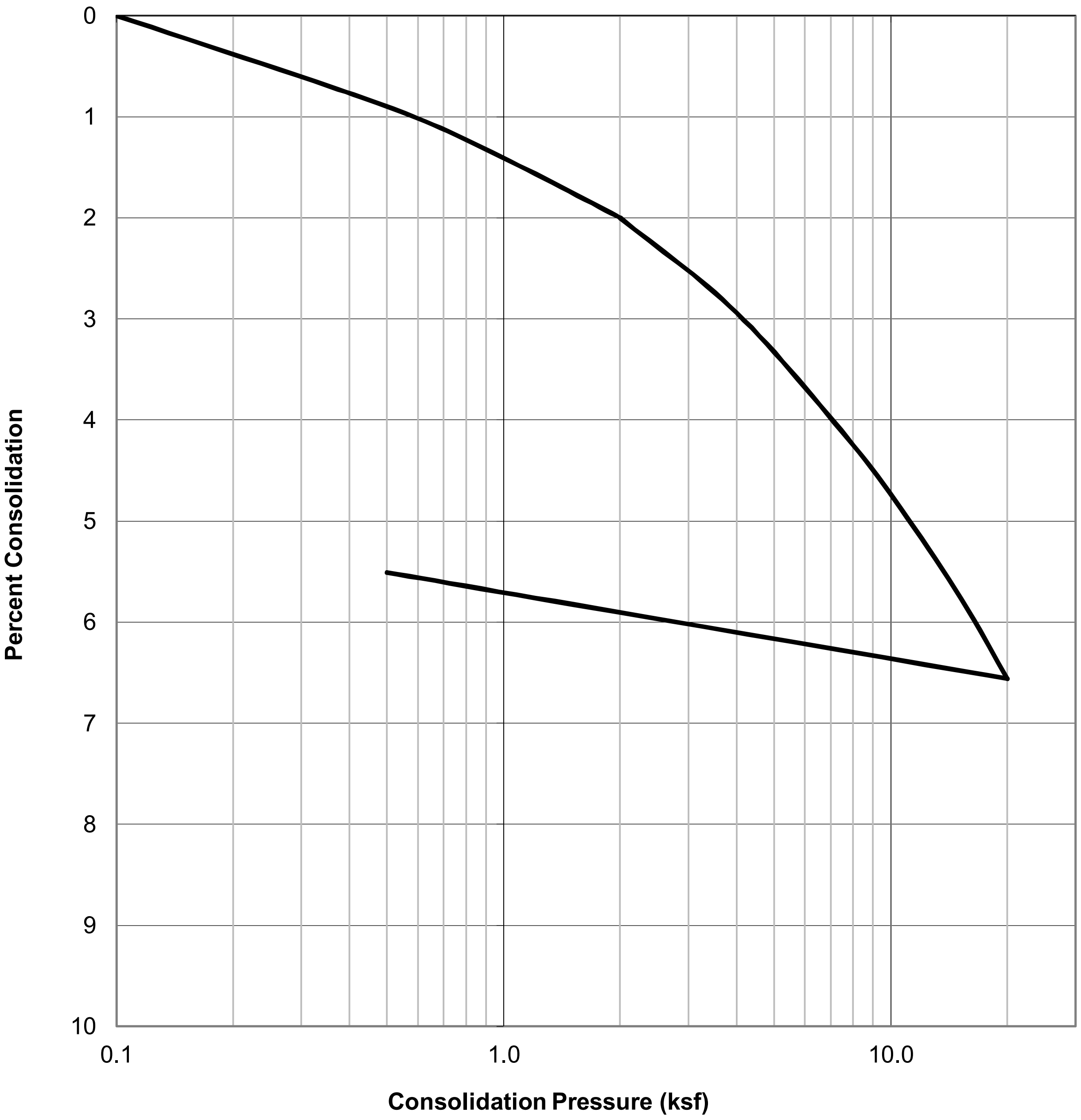
Figure B13

WATER ADDED AT 2.0 KSF



SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B1@150	Yellow Brown Silty Sand (SM)	115.2	17.0	14.7

GEOCON

CONSOLIDATION TEST RESULTS

ASTM D-2435

Checked by: JTA

Project No.: W1063-06-01

1708-1732 N. Cahuenga Blvd.
6381-6385 W. Hollywood Blvd.
Los Angeles, California

Oct 19

Figure B14

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

Sample No.	**pH**	**Resistivity (ohm centimeters)**
B1 @ 10-15'	8.5	2900 (Moderately Corrosive)
B1 @ 20-25'	8.2	5900 (Moderately Corrosive)

SUMMARY OF LABORATORY CHLORIDE CONTENT TEST RESULTS
EPA NO. 325.3

Sample No.	Chloride Ion Content (%)
B1@10-15	0.005
B1@20-25	0.001

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

Sample No.	**Water Soluble Sulfate (% SQ_4)**	**Sulfate Exposure***
B1@10-15'	0.000	S0
B1@20-25	0.000	S0

GEOCON

CORROSIVITY TEST RESULTS

Checked by: JTA

Project No.: W1063-06-01

1708-1732 N. Cahuenga Blvd.
6381-6385 W. Hollywood Blvd.
Los Angeles, California

Oct 19

Figure B15

APPENDIX C

APPENDIX C

PRIOR REPORTS

(CD ONLY)

REPORT OF GEOTECHNICAL INVESTIGATION PROPOSED BUILDING CONVERSION AND PARKING STRUCTURE

6381 HOLLYWOOD BOULEVARD
AND 1716-1720 NORTH CAHUENGA BOULEVARD
HOLLYWOOD DISTRICT OF LOS ANGELES, CALIFORNIA

Prepared for:

TEXAS ROCK, LLC

Oklahoma City, Oklahoma

June 21, 2007

MACTEC

MACTEC
engineering and constructing a better tomorrow

June 21, 2007

Mr. David Morgan, Esq.
Texas Rock, LLC
501 NW Grand Boulevard
Oklahoma City, Oklahoma 73118

Subject: **LETTER OF TRANSMITTAL**
Report of Geotechnical Investigation
Proposed Building Conversion and Parking Structure
6381 Hollywood Boulevard and 1716-1720 North Cahuenga Boulevard
Hollywood District of Los Angeles, California
MACTEC Project 4953-07-0921

Dear Mr. Morgan:

We are pleased to submit the results of our geotechnical investigation for the proposed building conversion and parking structure located at the northeast corner of Hollywood Boulevard and Cahuenga Boulevard in the Hollywood District of Los Angeles, California. This investigation was conducted in general accordance with our proposal dated May 31, 2007 which you authorized June 1, 2007.

The scope of our services was planned with Ms. Sheri Bonstelle of Jeffer, Mangels, Butler & Marmaro, LLP. (JMBM). Structural features and loadings of the proposed building conversion and parking structure are not available at this time. Supplemental geotechnical consultation may be needed when structural information becomes available.

The results of our investigation and design recommendations are presented in this report. Please note that you or your representative should submit copies of this report to the appropriate governmental agencies for their review and approval prior to obtaining a building permit.

It has been a pleasure to be of professional service to you. Please call if you have any questions or if we can be of further assistance.

Sincerely,

MACTEC Engineering and Consulting, Inc.

Marshall

REGISTERED PROFESSIONAL ENGINEER
MARSHALL LEW
No. 522
Exp. 3-31-09
GEOTECHNICAL
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(4 copies submitted)

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REPORT OF GEOTECHNICAL INVESTIGATION
PROPOSED BUILDING CONVERSION AND PARKING STRUCTURE

6381 HOLLYWOOD BOULEVARD
AND 1716-1720 NORTH CAHUENGA BOULEVARD
HOLLYWOOD DISTRICT OF LOS ANGELES, CALIFORNIA

Prepared for:

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EXECUTIVE SUMMARY

We have completed our geotechnical investigation of the site of the proposed Building Conversion and Parking Structure in the Hollywood District of Los Angeles for Texas Rock, LLC. Our subsurface explorations, engineering analyses, and foundation design recommendations are summarized below.

We explored the soil conditions by drilling two borings at the site. Fill soils, up to 6½ feet thick, were found in our borings. The fill soils consist of silty sand with some debris. Deeper soils may be encountered beneath the site at locations not explored. The natural soils underlain the fill consist mainly of silty sand with layers of silt and well graded sand. Corrosion test results indicate that the site soils are moderately corrosive to ferrous metals and aggressive to copper when saturated and have negligible potential for sulfate attack on concrete.

Ground water was not encountered in the borings to the maximum depth explored. The historical ground water for the site as reported by the California Geologic Survey is at a depth greater than 80 feet below the ground surface.

Based on the information available from the foundation plan, the existing building at 6381 Hollywood Boulevard is underlain by one subterranean level and the structure is supported on spread foundations established below Elevation 365.99. New or enlarged footings for the proposed conversion of the existing office building may be established in the natural soils.

The planned excavation for the subterranean parking levels will extend below the depth of the existing fill soils. The proposed structure can be supported on shallow spread footings established in the undisturbed natural soils. The on-site soils are suitable for use as compacted fill, and the building floor slab can be supported on grade. Shoring should be used where sloped excavations are not possible.

1.0 SCOPE

This report provides foundation design information for the proposed building conversion (located at 6381 Hollywood Boulevard) and parking structure (to be located at 1716-1720 North Cahuenga Boulevard) in Hollywood, California. The location of the site, the proposed building conversion and parking structure, existing buildings, and our exploration borings are shown on Figure 1, Plot Plan.

This investigation was authorized to determine the static physical characteristics of the soils at the site of the proposed building conversion and parking structure, and to provide recommendations for foundation design, floor slab support, and grading for the development. We were to evaluate the existing soil and ground-water conditions at the site, including the corrosion potential of the soils, and develop recommendations for the following:

- Provide recommendations for appropriate foundations together with the necessary design parameters.
- A feasible foundation system design along with the necessary design parameters, including the estimated settlement due to the expected loadings for the existing and new foundations of the proposed building conversion and parking structure.
- Provide recommendations for excavation and shoring.
- Provide recommendations for design of walls below grade.
- Provide recommendations for lateral surcharge pressures on the shoring and walls below grade.
- Provide recommendations for floor slab support.
- Grading, including site preparation, excavation and slopes, the placing of compacted fill, and quality control measures relating to earthwork.

The scope of this investigation did not include geologic or seismic studies for the site. Accordingly, our conclusions and recommendations are for static loading conditions only; however, this does not imply that there is a geologic or seismic hazard affecting the site. Also, the assessment of general site environmental conditions for the presence of contaminants in the soils and groundwater of the site was beyond the scope of this investigation.

Our recommendations are based on the results of our current field explorations, laboratory tests, and appropriate engineering analyses. The results of the field explorations and laboratory tests, which form the basis of our recommendations, are presented in the Appendix.

2.0 PROJECT DESCRIPTION

It is planned to convert an existing office building located at the northeast corner of Hollywood Boulevard and Cahuenga Boulevard to a luxury hotel. The existing structure is a historical structure and is seven stories in height with a full basement. The proposed conversion will consist mainly of interior renovations and retrofitting, including potential additions of a stairwell and shear walls for seismic strengthening of the existing structure.

The proposed parking structure is planned just north of the proposed luxury hotel. The proposed parking will have four levels; two levels will be above grade and two levels will be below grade. The site of the proposed parking structure is currently occupied by a single-story restaurant building and adjacent paving used for surface parking.

Structural information for the buildings is not available at this time. Based on our experience with similar projects, we have assumed a maximum dead-plus-live column loads will be about 800 kips for the proposed parking structure.

The finished floor elevation of the proposed new parking structure is anticipated to extend below a depth of about 20 feet below the existing grade for the two subterranean levels.

3.0 SITE CONDITIONS

The site is located at the northeast corner of Hollywood Boulevard and Cahuenga Boulevard in the Hollywood District of Los Angeles, California; see Figure 1, Plot Plan. The site is currently occupied by a seven story historic building with one basement level to be renovated. The location of the proposed parking structure is covered with an existing single story restaurant building and adjacent paving for parking to be demolished as part of the construction.

Various existing buildings located north and east of the site. To the north of the site of the proposed parking structure is a three-story masonry building; the building appears to be a seismically retrofitted unreinforced masonry building. There is also an existing building immediately east of the existing restaurant building. The ground surface of the site slopes gently in a northwest direction from Elevation 382.2 to 385.4. Various underground utilities cross the site.

4.0 EXPLORATIONS AND LABORATORY TESTS

The soil conditions beneath the site were explored by drilling two borings to depths of 50 and 50½ feet below the existing grade at the locations shown on Figure 1. Details of the explorations and the logs of the borings are presented in the Appendix.

Laboratory tests were performed on selected samples obtained from the borings to aid in the classification of the soils and to determine the pertinent engineering properties of the foundation soils. The following tests were performed:

- Moisture content and dry density determinations.
- Direct shear.
- Consolidation.
- Compaction.

In addition, we have retained Schiff Associates, Consulting Corrosion Engineers, to perform chemical testing to evaluate the corrosion and sulfate attack potential of the site soils.

All testing was done in general accordance with applicable ASTM specifications. Details of the laboratory testing program and test results are presented in the Appendix.

5.0 SOIL CONDITIONS

Fill soils, up to 6½ feet thick, were found in our borings and are not uniformly compacted. The fill soils consist of silty sand with some debris. Deeper soils may be encountered beneath the site at locations not explored.

The natural soils underlain by the fill consist mainly of silty sand with layers of silt and well graded sand.

Water was not encountered within the 50½-foot depth explored. The historical ground water for the site as reported by the California Geologic Survey is at a depth greater than 80 feet below the ground surface.

The corrosion studies indicate that the on-site soils are moderately corrosive to ferrous metals, negligible to portland cement concrete and aggressive to copper. The report of corrosion studies presented in the Appendix should be referred to for a discussion of the corrosion potential of the soils, and for potential mitigation measures.

6.0 LIQUEFACTION POTENTIAL

Liquefaction potential is greatest where the ground water level is shallow, and submerged loose, fine sands occur within a depth of about 50 feet or less. Liquefaction potential decreases as grain size and clay and gravel content increase. As ground acceleration and shaking duration increase during an earthquake, liquefaction potential increases.

The site is not within a State of California designated Liquefaction Hazard Zone. Based on the ground-water level and the general nature of the soils at the site below the building foundations, the liquefaction potential at the site is considered to be low.

7.0 RECOMMENDATIONS

New or enlarged footings for the proposed conversion of the existing office building may be established in the natural soils. Based on the information available from the foundation plan, the existing building at 6381 Hollywood Boulevard is underlain by one subterranean level and the structure is supported on spread foundations established below Elevation 365.99.

The planned excavation for the subterranean parking levels will extend below the depth of the existing fill soils. The proposed structure can be supported on shallow spread footings established in the undisturbed natural soils. If the recommendations on grading are followed, the floor slab can be supported on grade. Shoring should be used where sloped excavations are not possible.

New foundations for the proposed conversion and parking structure should be located below a 1:1 (horizontal to vertical) plane extending upward from the bottom of the adjacent building foundations so that new foundations do not surcharge the existing foundations.

Adjacent to any existing foundations, unshored excavations should not extend below a 1½:1 (horizontal to vertical) plane extending downward from the bottoms of the existing foundations and care should be exercised not to undermine the existing slabs on grade.

7.1 FOUNDATIONS

In this section, data are given for the following foundation design considerations:

- Bearing value for both major structures and structurally separate minor structures for the existing foundations and the proposed parking structure.
- Estimated settlement of the structures.
- Lateral resistance.

Bearing Value

Spread footings carried at least 1 foot into the undisturbed natural soils and at least 2 feet below the lowest adjacent grade or floor level can be designed to impose a net dead-plus-live load pressure of 5,000 pounds per square foot. The excavations should be deepened as necessary to extend into satisfactory soils.

For preliminary design, existing footings of the office building may be analyzed for assuming a dead-plus-live load pressure of 5,000 pounds per square foot if additional loads are imposed. Higher bearing pressures may be possible once the details of the structural strengthening are known and specific analyses can be performed.

A one-third increase can be used for wind or seismic loads. The recommended bearing value is a net value, and the weight of concrete in the footings can be taken as 50 pounds per cubic foot; the weight of soil backfill can be neglected when determining the downward loads.

Footings for minor structures (loading dock walls, minor retaining walls, and free-standing walls) that are structurally separate from the building conversion and parking structure can be designed to impose a net dead-plus-live load pressure of 1,000 pounds per square foot at a depth of 1½ feet below the lowest adjacent grade. Such footings can be established in either properly compacted fill soils or undisturbed natural soils.

Settlement

We estimate the settlement of the proposed parking structure with an assumed maximum dead-plus-live load of 800 kips, supported on spread footings in the manner recommended, will be about than 1 inch. At least half of the total settlement is expected to occur during construction, shortly after dead loads are imposed.

Lateral Resistance

Lateral loads can be resisted by soil friction and by the passive resistance of the soils. A coefficient of friction of 0.5 can be used between the existing and new structure footings and the floor slab and the supporting soils. The passive resistance of natural soils or properly compacted fill soils can be assumed to be equal to the pressure developed by a fluid with a density of 300 pounds per cubic foot. A one-third increase in the passive value can be used for wind or seismic loads. The frictional resistance and the passive resistance of the soils can be combined without reduction in determining the total lateral resistance.

7.2 SITE COEFFICIENT AND SEISMIC ZONATION

The site coefficient, S, can be determined as established in the Earthquake Regulations under Section 1628 of the Los Angeles Building Code (LABC), for seismic design of the proposed building conversion and parking structure. Based on a review of the local soil and geologic conditions, the site may be classified as Soil Profile Type S_D as specified in the LABC. The site is located within UBC Seismic Zone 4.

The site is near the Hollywood fault, which has been determined to be a Type B seismic source by the California Division of Mines and Geology. According to Map M-32 in the 1998 publication from the International Conference of Building Officials entitled "Maps of Known Active Fault Near-Source Zones in California and Adjacent Portions of Nevada," the proposed building conversion and parking structure is located at a distance of less than 2 kilometers from the Hollywood fault. At this distance for a seismic source type B, the near source factors, N_a and N_v, are to be taken as 1.3 and 1.6, respectively, based on Tables 16-S and 16-T of the LABC.

7.3 FLOOR SLAB SUPPORT

If the subgrade is prepared as recommended in the following section on grading, the building floor slab of the proposed parking structure can be supported on grade.

Construction activities and exposure to the environment can cause deterioration of the prepared subgrade. Therefore, we recommend our that our field representative observe the condition of the final subgrade soils immediately prior to slab-on-grade construction, and, if necessary, perform further density and moisture content tests to determine the suitability of the final prepared subgrade.

If vinyl or other moisture-sensitive floor covering is planned, we recommend that the floor slab in those areas be underlain by a capillary break consisting of a vapor-retarding membrane over a 4-inch-thick layer of gravel. A 2-inch-thick layer of sand should be placed between the gravel and the membrane to decrease the possibility of damage to the membrane. We suggest the following gradation for the gravel:

Sieve Size	Percent Passing
¾"	90 - 100
No. 4	0 - 10
No. 100	0 - 3

A low-slump concrete should be used to reduce possible curling of the slab. A 2-inch-thick layer of coarse sand can be placed over the vapor retarding membrane to reduce slab curling. If this sand bedding is used, care should be taken during the placement of the concrete to prevent displacement of the sand. The concrete slab should be allowed to cure properly before placing vinyl or other moisture-sensitive floor covering.

7.4 EXCAVATION AND SLOPES

Excavation of about 20 feet deep will be required for proposed parking structure. Ground water was not encountered in our borings.

Where the necessary space is available, temporary drained unsurcharged embankments deeper than 4 feet and less than 15 feet in height may be sloped back at 1:1 without shoring. Unsurcharged embankments 4 feet or less in height may be cut vertically. Adjacent to any existing structure supported on shallow foundations, the excavation should not extend below a plane drawn downward at 1½:1 (horizontal to vertical) from the bottoms of the existing foundations. Where space is not available for sloped excavations, shoring will be required. Data for design of shoring are presented in the following section.

Where sloped embankments are used, the tops of the slopes should be barricaded to prevent vehicles and storage loads within at least 7 feet of the tops of the slopes. A greater setback may be necessary when considering heavy vehicles, such as concrete trucks and cranes; we should be advised of such heavy vehicle loadings so that specific setback requirements can be established. If the temporary constructions embankments are to be maintained during the rainy season, berms are suggested along the tops of the slopes, where necessary, to prevent runoff water from entering the excavation and eroding the slope faces.

The excavation should be observed by personnel of our firm so that any necessary modifications based on variations in the soil conditions encountered can be made.

7.5 SHORING

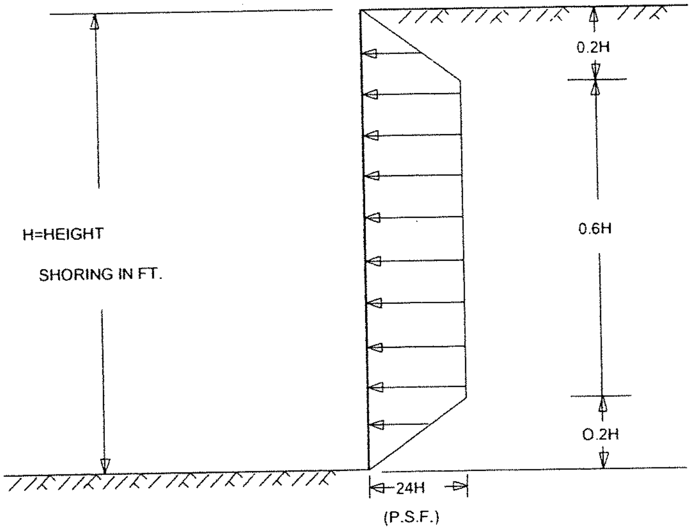
General

Where sufficient space for sloped embankments is not available, shoring will be required. One method of shoring would consist of steel soldier beams placed in drilled holes, and the holes backfilled with concrete and tied-back with earth anchors. Some caving and raveling may occur during the drilling of the soldier piles through the silty sand and sand deposits. Water was not encountered in our borings. If there is not sufficient space to install the tie-back anchors to the desired lengths on any side of the excavation, the soldier piles of the shoring system may be internally braced.

The following information on the design and installation of the shoring is as complete as possible at this time. We can furnish any additional required data as the design progresses. Also, we suggest that our firm review the final shoring plans and specifications prior to bidding or negotiating with a shoring contractor.

Lateral Pressures

For the design of tied-back or braced shoring, we recommend the use of a trapezoidal distribution of earth pressure. The recommended pressure distribution, for the case where the grade is level behind the shoring, is illustrated in the following diagram with the maximum pressure equal to 24H in pounds per square foot, where H is the height of the shoring in feet. (Where a combination of sloped embankment and shoring is used, the pressure would be greater and must be determined for each combination.)



In addition to the recommended earth pressure, the upper 10 feet of shoring adjacent to the streets and traffic areas should be designed to resist a uniform lateral pressure of 100 pounds per square foot, acting as a result of an assumed 300 pounds per square foot surcharge behind the shoring due to normal street traffic. If the traffic is kept back at least 10 feet from the shoring, the traffic surcharge may be neglected. Also, the shoring system adjacent to the existing structures should also be designed to support the lateral surcharge pressures imposed by the foundations of the adjacent structures unless the structures are underpinned. In addition, the shoring system should be designed to support the lateral surcharge pressures imposed by concrete trucks and other heavy construction equipment, including cranes, placed near the shoring system.

Design of Soldier Piles

For the design of soldier piles spaced at least two diameters on centers, the allowable lateral bearing value (passive value) of the soils below the level of excavation may be assumed to be 500 pounds per square foot per foot of depth at the excavated surface, up to a maximum of 5,000 pounds per square foot. To develop the full lateral value, provisions should be taken to assure firm contact between the soldier piles and the retained earth. The tremie method should be used in pouring the concrete in the soldier piles; however, if the tremie method is used, the compressive strength of the concrete should be increased by 1,000 pounds per square inch below the water. The concrete placed in the soldier pile excavations may be a lean-mix concrete. However, the concrete used in that portion of the soldier pile which is below the planned excavated level should be of sufficient strength to adequately transfer the imposed loads to the surrounding soils.

The frictional resistance between the soldier piles and the retained earth may be used in resisting the downward component of the anchor load. The coefficient of friction between the soldier piles and the retained earth may be taken as 0.3. (This value is based on the assumption that uniform full bearing will be developed between the steel soldier beam and the lean-mix concrete and between the lean-mix concrete and the retained earth.). In addition, provided that the portion of the soldier piles below the excavated level is backfilled with structural concrete, the soldier piles below the excavated level may be used to resist downward loads. For resisting the downward loads, the frictional resistance between the concrete soldier piles and the soils below the excavated level may be taken equal to 250 pounds per square foot.

Lagging

Continuous lagging will be required between the soldier piles. The soldier piles and anchors should be designed for the full anticipated lateral pressure. However, the pressure on the lagging will be less due to arching in the soils. For clear spans of up to 6 feet, we recommend that the lagging be designed for a triangular distribution of earth pressure where the maximum pressure of 400 pounds per square foot is at the mid-line between the soldier piles, and 0 pounds per square foot at the soldier piles.

Anchor Design

Tie-back friction anchors may be used to resist lateral loads. For design purposes, it may be assumed that the active wedge adjacent to the shoring is defined by a plane drawn at 35 degrees with the vertical through the bottom of the excavation. The anchors should extend at least 20 feet beyond the potential active wedge to develop the desired capacities.

The capacities of anchors should be determined by testing of the initial anchors as outlined in the anchor testing section. For design purposes, it may be estimated that drilled friction anchors will develop an average friction value of 500 pounds per square foot. For post-grouted anchors, it is estimated that the anchors could develop an average friction of up to 1,500 pounds per square foot. Only the frictional resistance developed beyond the active wedge would be effective in resisting lateral loads. If the anchors are spaced at least 6 feet on centers, no reduction in the capacity of the anchors need be considered due to group action.

Anchor Installation

The anchors may be installed at angles of 15 to 40 degrees below the horizontal. Caving of the anchor holes should be anticipated and provisions made to minimize such caving. The anchors should be filled with concrete placed by pumping from the tip out, and the concrete should extend from the tip of the anchor to the active wedge. To minimize chances of caving, we suggest 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 may contain a small amount of cement to allow the sand to be placed by pumping.

Anchor Testing

Our representative should select two of the initial anchors for 24-hour 200% tests, and five additional anchors for each main structure for quick 200% tests. The purpose of the 200% tests is to verify the friction value assumed in design. The anchors should be tested to develop twice the assumed friction value. 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.

For post-grouted anchors where concrete is used to backfill the anchor along its entire length, the test load should be computed as that required to develop the appropriate friction along the entire bonded length of the anchor. The test load should therefore be computed as:

$$P_{test} = P_{design} * \frac{L_b}{L_a} * M / 100$$

where L_a=Length of Anchor beyond the Active Wedge
L_b=Bonded Length of Anchor
M=150% or 200%, depending on the test performed

The total deflection during the 24-hour 200% test should not exceed 12 inches during loading; the anchor deflection should not exceed 0.75 inch during the 24-hour period, measured after the 200% test load is applied. If the anchor movement after the 200% load has been applied for 12 hours is less than 0.5 inch, and the movement over the previous 4 hours has been less than 0.1 inch, the test may be terminated.

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

All of the production anchors should be pretested to at least 150% of the design load; the total deflection during the tests should not exceed 12 inches. The rate of creep under the 150% test should not exceed 0.1 inch over a 15-minute period for the anchor to be approved for the design loading.

After a satisfactory test, each production anchor should be locked-off at the design load. The locked-off load should be verified by rechecking the load in the anchor. If the locked-off load varies by more than 10% from the design load, the load should be reset until the anchor is locked-off within 10% of the design load.

The installation of the anchors and the testing of the completed anchors should be observed by our firm.

Internal Bracing

As alternative to tie-back anchors, raker bracing may be used to internally brace the soldier piles. If used, raker bracing could be supported laterally by temporary concrete footings (deadmen). For design of such temporary footings, poured with the bearing surface normal to the rakers inclined at 45 to 60 degrees with the vertical, a bearing value of 2,500 pounds per square foot may be used, provided the shallowest point of the footing is at least 1 foot below the lowest adjacent grade. To reduce the movement of the shoring, the rakers should be tightly wedged against the footings and the shoring system.

Deflection

It is difficult to accurately predict the amount of deflection of a shored embankment. It should be realized, however, that some deflection will occur. We estimate that this deflection could be on the order of 1 inch at the top of the shored embankment. If greater deflection occurs during construction, additional bracing may be necessary to minimize settlement. If desired to reduce the deflection of the shoring, a greater active pressure could be used in the shoring design.

Monitoring

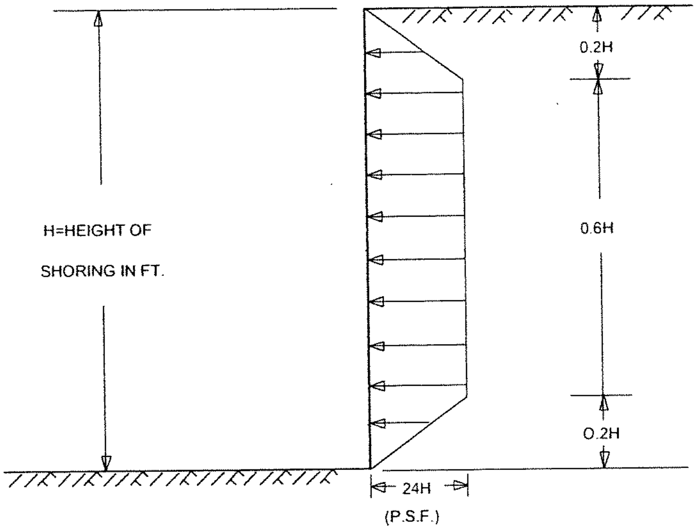
Some means of monitoring the performance of the shoring system is recommended. The monitoring should consist of periodic surveying of the lateral and vertical locations of the tops of all the soldier piles. Adjacent buildings should be surveyed prior to shoring installation and monitored during the shoring installation. We will be pleased to discuss this further with the design consultants and the contractor when the design of the shoring system has been finalized.

7.6 WALLS BELOW GRADE

Lateral Earth Pressure

For design of cantilevered walls below grade and retaining walls, where the surface of the backfill is level, it may be assumed that drained soils will exert a lateral pressure equal to that developed by a fluid with a density of 35 pounds per cubic foot.

Walls restrained at the top should be designed to resist a trapezoidal distribution of lateral earth pressure plus any surcharges from adjacent loads. The recommended pressure distribution, for the case where the grade is level behind the shoring, is shown below with the maximum pressure equal to 24H in pounds per square foot, where H is the height of the shoring in feet. Where a combination of sloped embankment and shoring is used, the pressure would be greater and must be determined for each combination.



The recommended earth pressure assumes that a drainage system will be installed behind the walls below grade so that external water pressure will not develop against the walls.

In addition to the recommended earth pressures, the upper 10 feet of walls adjacent to vehicular traffic areas should be designed to resist a uniform lateral pressure of 100 pounds per square foot, acting as a result of an assumed 300 pounds per square foot surcharge behind the walls due to normal traffic. If the traffic is kept back at least 10 feet from the walls, the traffic surcharge may be neglected.

Backfill

All required soil backfill should be mechanically compacted, in layers not more than 8 inches thick, to at least 90% of the maximum density obtainable by the ASTM Designation D1557-02 method of compaction. The backfill should be sufficiently impermeable when compacted to restrict the inflow of surface water. Some settlement of the deep backfill should be allowed for in planning sidewalks and utility connections.

Drainage

The upper basement walls should be waterproofed or at least damproofed, depending upon the degree of moisture protection desired.

If the backfill is placed and compacted as recommended and good surface drainage is provided, infiltration of water into the backfill should be minimal. Since the subsurface soils are generally granular in nature, water infiltrating through the upper backfill and natural soils should readily dissipate through the underlying granular soils. Nonetheless, to avoid accidental buildup of hydrostatic pressure against the basement walls above the water table, we recommend that vertical drains to be installed against the upper basement walls so as to provide drainage of incidental water from non ground-water sources to the soils below the water level.

Drainage behind the basement walls may be provided by vertical strips of geosynthetic drainage composite. In our opinion, Miradrain 6000 (or the equivalent), attached to the back of the wall before backfilling, would provide satisfactory drainage. The Miradrain strips may be placed at a depth starting at about 4 feet below the existing grade. The strips should be at least 4 feet wide and placed 8 feet on centers. The Miradrain should be continuous within the lower 4 feet of the wall.

The vertical drainage may be connecting to a collector pipe or a horizontal continuous strip of Miradrain or equivalent to be installed at the base of the walls to serve as a collector pipe to carry water from behind the building walls to a sump and pump system.

The pipe should be sloped at least 2 inches in 100 feet and surrounded by filter gravel. The filter gravel should meet the requirements of Class 2 Permeable Material as defined in the current State of California, Department of Transportation, Standard Specifications. If Class 2 Permeable Material is not available, ¾-inch crushed rock or gravel separated from the surrounding soils by an appropriate filter fabric can be used. The crushed rock or gravel should have less than 5% passing a No. 200 sieve.

As an alternative, the walls could be designed for hydrostatic pressure, in addition to the lateral earth pressures.

7.7 GRADING

The existing fill soils are not uniformly well compacted. The existing fill soils were not observed and tested during placement and are not considered suitable for support of or paving or floor slabs on grade. The existing fill soils should be excavated and replaced as properly compacted fill. All required fill should be uniformly well compacted and observed and tested during placement. The on-site soils can be used in any required fill.

- This section gives recommendations for the following grading considerations:
- Site preparation (includes specifications for compaction of natural soils).
- Excavations and temporary slopes.
- Compaction (specifications for fill compaction).
- Backfill (specifications for backfill compaction).
- Material for fill (specifications for on-site and import materials).

Site Preparation

After the site is cleared and any existing fill soils are excavated as recommended, the exposed natural soils should be carefully observed for the removal of all unsuitable deposits. Next, the exposed soils should be rolled with heavy compaction equipment. At least the upper 6 inches of the exposed soils should be compacted to at least 90% of the maximum dry density obtainable by the ASTM Designation D1557-02 method of compaction.

Excavations and Temporary Slopes

Where excavations are deeper than about 4 feet, the sides of the excavations should be sloped back at 1:1 (horizontal to vertical) or shored for safety. Unshored excavations should not extend below a plane drawn at 1½:1 (horizontal to vertical) extending downward from adjacent existing footings.

Excavations should be observed by personnel of our firm so that any necessary modifications based on variations in the soil conditions can be made. All applicable safety requirements and regulations, including OSHA regulations, should be met.

Compaction

Any required fill should be placed in loose lifts not more than 8-inches-thick and compacted. The fill should be compacted to at least 90% of the maximum density obtainable by the ASTM

Designation D1557-02 method of compaction. The moisture content of the on-site soils at the time of compaction should vary no more than 2% below or above optimum moisture content.

Backfill

All required backfill should be mechanically compacted in layers; flooding should not be permitted. Proper compaction of backfill will be necessary to reduce settlement of the backfill and to reduce settlement of overlying slabs and paving. Backfill should be compacted to at least 90% of the maximum dry density obtainable by the ASTM Designation D1557-02 method of compaction. The on-site soils can be used in the compacted backfill. The exterior grades should be sloped to drain away from the foundations to prevent ponding of water.

Some settlement of the backfill should be expected, and any utilities supported therein should be designed to accept differential settlement, particularly at the points of entry to the building. Also, provisions should be made for some settlement of concrete walks supported on backfill.

Material for Fill

The on-site soils, less any debris or organic matter, can be used in required fills. Cobbles larger than 4 inches in diameter should not be used in the fill. Any required import material should consist of relatively non-expansive soils with an expansion index of less than 35. The imported materials should contain sufficient fines (binder material) so as to be relatively impermeable and result in a stable subgrade when compacted. All proposed import materials should be approved by our personnel prior to being placed at the site.

7.8 GEOTECHNICAL OBSERVATION

The reworking of the upper soils and the compaction of all required fill should be observed and tested during placement by a representative of our firm. This representative should perform at least the following duties:

- Observe the clearing and grubbing operations for proper removal of all unsuitable materials.
- Observe the exposed subgrade in areas to receive fill and in areas where excavation has resulted in the desired finished subgrade. The

representative should also observe proofrolling and delineation of areas requiring overexcavation.

- Evaluate the suitability of on-site and import soils for fill placement; collect and submit soil samples for required or recommended laboratory testing where necessary.
- Observe the fill and backfill for uniformity during placement.
- Test backfill for field density and compaction to determine the percentage of compaction achieved during backfill placement.
- Observe and probe foundation materials to confirm that suitable bearing materials are present at the design foundation depths.

The governmental agencies having jurisdiction over the project should be notified prior to commencement of grading so that the necessary grading permits can be obtained and arrangements can be made for required inspection(s). The contractor should be familiar with the inspection requirements of the reviewing agencies.

8.0 GENERAL LIMITATIONS AND BASIS FOR RECOMMENDATIONS

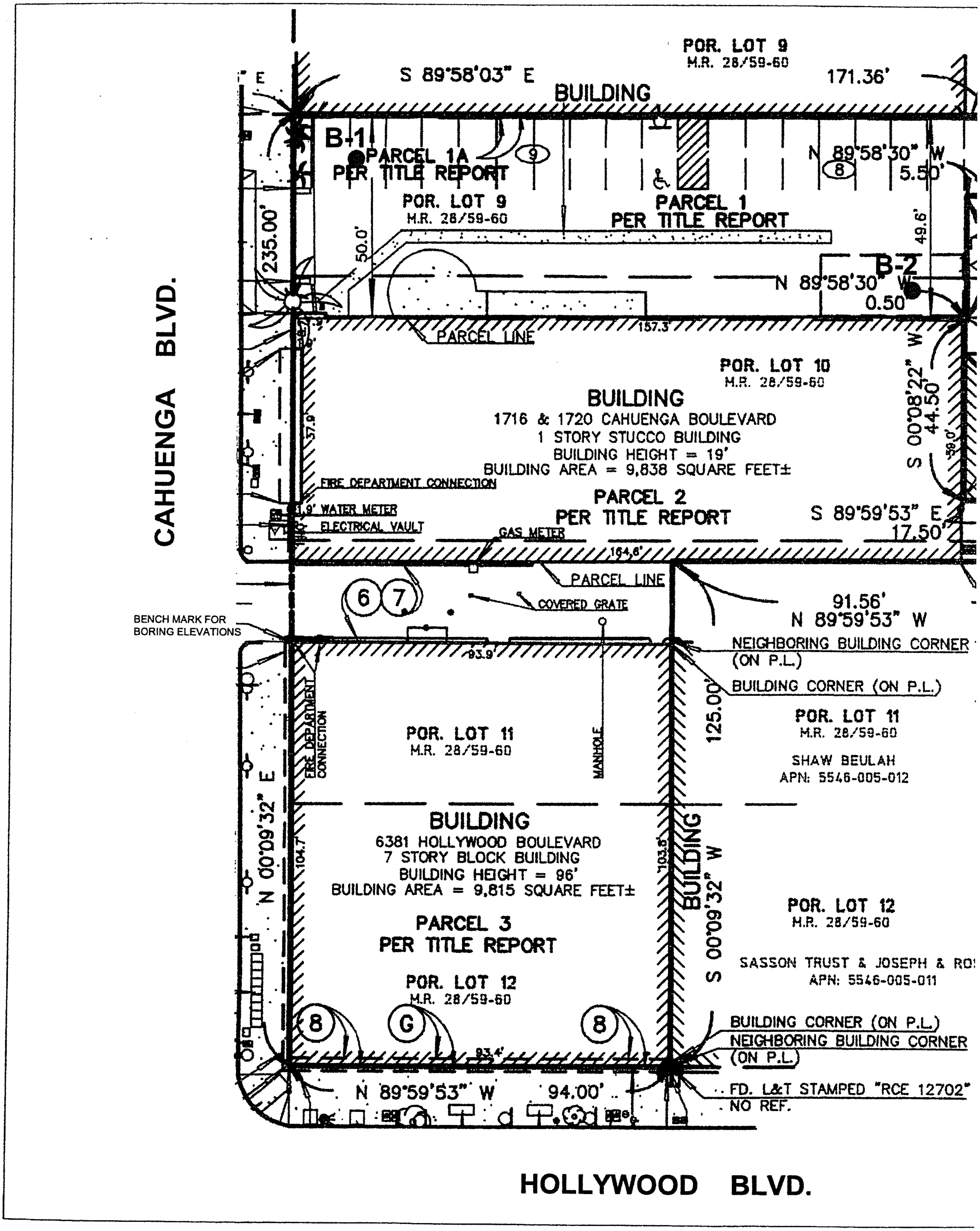
Our professional services have been performed using that degree of care and skill ordinarily exercised, under similar circumstances, by reputable geotechnical consultants practicing in this or similar localities. No other warranty, expressed or implied, is made as to the professional advice included in this report. This report has been prepared for Texas Rock, LLC and their design consultants to be used solely in the design of the proposed building conversion and parking structure. The report has not been prepared for use by other parties, and may not contain sufficient information for purpose of other parties or other uses.

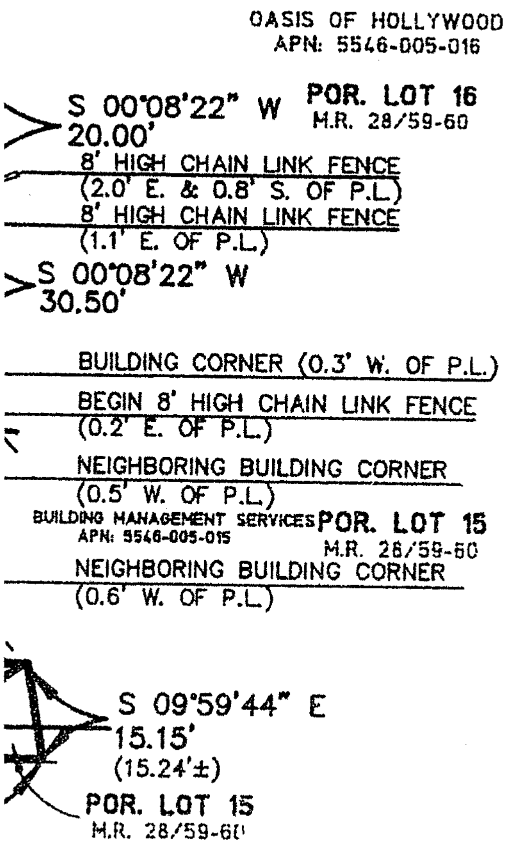
The recommendations provided in this report are based upon our understanding of the described project information and on our interpretation of the data collected during our subsurface explorations. We have made our recommendations based upon experience with similar subsurface conditions under similar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

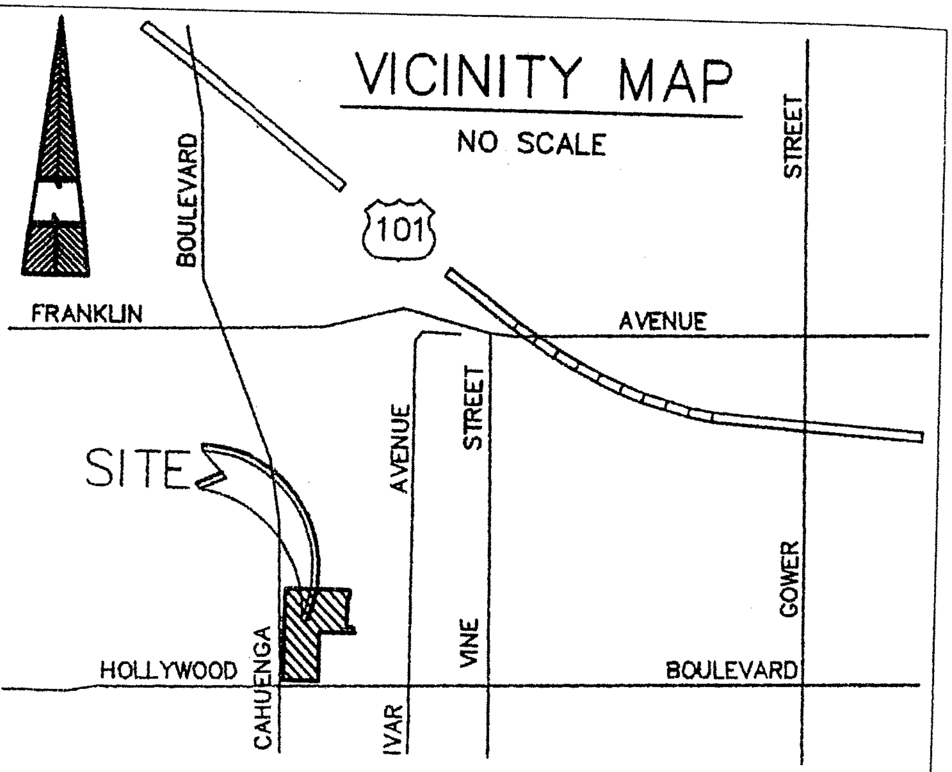
The recommendations provided in this report are also based upon the assumption that the necessary geotechnical observations and testing during construction will be performed by representatives of our firm. The field observation services are considered a continuation of the geotechnical investigation and essential to verify that the actual soil conditions are as expected. This also provides for the procedure whereby the client can be advised of unexpected or changed conditions that would require modifications of our original recommendations. If another firm is retained for the geotechnical observation services, our professional responsibility and liability would be limited to the extent that we would not be the geotechnical engineer of record.

FIGURES

FIGURE

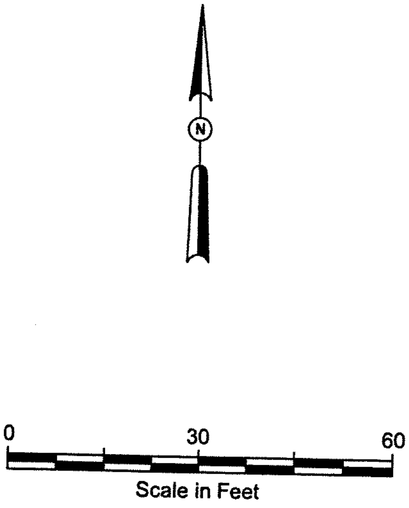

POR. LOT 9
M.R. 28/59-60
S 89°58'03" E
BUILDING
171.36'
B-1
PARCEL 1A
PER TITLE REPORT
9
8
N 89°58'30" W
5.50'
POR. LOT 9
M.R. 28/59-60
PARCEL 1
PER TITLE REPORT
235.00'
50.0'
49.6'
B-2
N 89°58'30" W
0.50'
157.3'
PARCEL LINE
POR. LOT 10
M.R. 28/59-60
BUILDING
1716 & 1720 CAHUENGA BOULEVARD
1 STORY STUCCO BUILDING
BUILDING HEIGHT = 19'
BUILDING AREA = 9,838 SQUARE FEET±
S 00°08'22" W
44.50'
58.0'
57.9'
FIRE DEPARTMENT CONNECTION
PARCEL 2
PER TITLE REPORT
1.9' WATER METER
ELECTRICAL VAULT
GAS METER
S 89°59'53" E
17.50'
164.6'
CAHUENGA BLVD.
PARCEL LINE
6
7
COVERED GRATE
91.56'
N 89°59'53" W
BENCH MARK FOR
BORING ELEVATIONS
93.9'
NEIGHBORING BUILDING CORNER
(ON P.L.)
BUILDING CORNER (ON P.L.)
125.00'
POR. LOT 11
M.R. 28/59-60
FIRE DEPARTMENT CONNECTION
MANHOLE
POR. LOT 11
M.R. 28/59-60
SHAW BEULAH
APN: 5546-005-012
N 00°09'32" E
104.7'
BUILDING
6381 HOLLYWOOD BOULEVARD
7 STORY BLOCK BUILDING
BUILDING HEIGHT = 96'
BUILDING AREA = 9,815 SQUARE FEET±
103.8'
BUILDING
S 00°09'32" W
PARCEL 3
PER TITLE REPORT
POR. LOT 12
M.R. 28/59-60
SASSON TRUST & JOSEPH & RO
APN: 5546-005-011
POR. LOT 12
M.R. 28/59-60
8
G
8
BUILDING CORNER (ON P.L.)
NEIGHBORING BUILDING CORNER
(ON P.L.)
93.4'
N 89°59'53" W
94.00'
FD. L&T STAMPED "RCE 12702"
NO REF.
HOLLYWOOD BLVD.





REFERENCE:
A.L.T.A. / A.C.S.M. LAND TITLE SURVEY
(DATED 5/3/07) BY O.K.O. ENGINEERING INC.

LEGEND:
B-2 ● BORING LOCATION
AND NUMBER



MACTEC
5628 E. SLAUSON AVENUE
LOS ANGELES, CALIFORNIA 90040
(323) 889-5300 FAX (323) 889-5398

FIGURE 1

PLOT PLAN

Proposed Building Conversion & Parking Structure
6381 Hollywood Blvd. & 1716 & 1720 N. Cauenga Blvd.
Hollywood District, Los Angeles County, California

PROJECT NO. 4953-07-0921	REVISION:
DATE: 6/11/07	
SCALE: 1" = 30'	
DWG BY: T.T.	CHECKED BY:

APPENDIX

APPENDIX

SUBSURFACE EXPLORATIONS AND LABORATORY TESTS

APPENDIX

SUBSURFACE EXPLORATIONS AND LABORATORY TESTS

EXPLORATIONS

The soil conditions beneath the site were explored by drilling two borings at the locations shown on Figure 1. The borings were drilled to depths of 50 and 50½ feet below the existing grade using 8-inch-diameter hollow stem auger-type drilling equipment.

The soils encountered were logged by our field technician, and undisturbed and bulk samples were obtained for laboratory inspection and testing. The logs of the borings are presented on Figures A-1.1 and A-1.2; the depths at which undisturbed samples were obtained are indicated to the left of the boring logs. The number of blows required to drive the Crandall sampler 12 inches using a 140 pound hammer falling 30 inches is indicated on the logs. In addition to obtaining undisturbed samples, standard penetration tests (SPT) were performed in the borings; the results of the tests are indicated on the logs. The soils are classified in the accordance with the Unified Soil Classification System described on Figure A-2.

LABORATORY TESTS

Laboratory tests were performed on selected samples obtained from the borings to aid in the classification of the soils and to determine their engineering properties.

The field moisture content and dry density of the soils encountered were determined by performing tests on the undisturbed samples. The results of the tests are shown to the left on the boring logs.

Direct shear tests were performed on selected undisturbed samples to determine the strength of the soils. The tests were performed at field moisture content and after soaking to near saturated moisture content and at various surcharge pressures. The yield-point values determined from the direct shear tests are presented on Figure A-3, Direct Shear Test Data.

Confined consolidation tests were performed on two undisturbed samples to determine the compressibility of the soils. The results of the tests are presented on Figures A-4, Consolidation Test Data.

The optimum moisture content and maximum dry density of the upper soils were determined by performing a compaction test on a sample obtained from Boring 1. The test was performed in accordance with the ASTM Designation D1557-02 method of compaction. The results of the tests are presented on Figure A-5, Compaction Test Data.

Soil corrosivity studies were performed on samples of the on-site soils. The results of the study and recommendations for mitigating procedures are presented at end of this Appendix.

BORING 1

DATE DRILLED: June 7, 2007
EQUIPMENT USED: Hollow Stem Auger
HOLE DIAMETER (in.) 8
ELEVATION: 385.4**

ELEVATION (ft)	DEPTH (ft)	"N" VALUE STD.PEN.TEST	MOISTURE (% of dry wt.)	DRY DENSITY (pcf)	BLOW COUNT* (blows/ft)	SAMPLE LOC.		
								3.5" thick Asphaltic Paving
385							SM	FILL - SILTY SAND - moist, light brown, fine to medium, occasional gravel
	5							Dark brown, trace gravel
380			6.4	105	10	▨		
							SM	SILTY SAND - moist, brown, fine to medium, some coarse
			8.0	101	11	▨		
375	10		12.8	105	15	▨		Alternating with layers of Sandy Silt
		8				☒		Sandier
370	15						SW	WELL GRADED SAND - moist, light brown, fine to coarse
			2.9	101	13	▨		
							SM	SILTY SAND - moist, light brown, fine to medium
365	20	18				☒	SW	WELL GRADED SAND with gravel - moist, light brown, fine to coarse
			12.6	102	10	▨	ML	SANDY SILT - moist, brown, fine to medium, some coarse
							SM	SILTY SAND - moist, light brown, fine to medium
360	25	16				☒		
			11.1	110	15	▨		Occasional gravel
355	30	14				☒		
			7.0	110	19	▨		
350	35	15				☒		Some gravel and cobbles
	40							

THIS RECORD IS A REASONABLE INTERPRETATION OF SUBSURFACE CONDITIONS AT THE EXPLORATION LOCATION. SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND AT OTHER TIMES MAY DIFFER. INTERFACES BETWEEN STRATA ARE APPROXIMATE. TRANSITIONS BETWEEN STRATA MAY BE GRADUAL.

(CONTINUED ON FOLLOWING FIGURE)

Field Tech: AR
Prepared By: HP
Checked By:

Building Conversion
& Parking Structure
Hollywood District, California

MACTEC

LOG OF BORING
Project: 4953-07-0921 Figure: A-1.1a

BORING 1 (Continued)

DATE DRILLED: June 7, 2007
EQUIPMENT USED: Hollow Stem Auger
HOLE DIAMETER (in.) 8
ELEVATION: 385.4**

THIS RECORD IS A REASONABLE INTERPRETATION OF SUBSURFACE CONDITIONS AT THE EXPLORATION LOCATION. SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND AT OTHER TIMES MAY DIFFER. INTERFACES BETWEEN STRATA ARE APPROXIMATE. TRANSITIONS BETWEEN STRATA MAY BE GRADUAL.

ELEVATION (ft)	DEPTH (ft)	"N" VALUE STD.PEN.TEST	MOISTURE (% of dry wt.)	DRY DENSITY (pcf)	BLOW COUNT* (blows/ft)	SAMPLE LOC.	
345			9.8	102	16	▩	
		11				☒	Thin Sandy Silt layer
340	45						
			9.2	113	28	▩	
335	50	15				☒	Thin Sandy Silt layer
							END OF BORING AT 50.5 FEET
330	55						
325	60						
320	65						
315	70						
310	75						
	80						

NOTES:

Ground water not encountered. Boring backfilled with soil cutting and tamped.

* Number of blows required to drive the Crandall sampler 12 inches using a 140 pounds hammer falling 30 inches.

** Elevation surveyed based on bench mak location shown on Figure 1.
(Elevation of bench mark = 382.2').

Field Tech: AR
Prepared By: HP
Checked By: LT

Building Conversion & Parking Structure Hollywood District, California

MACTEC

LOG OF BORING

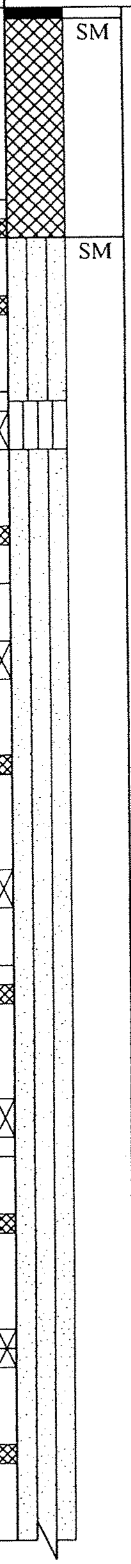
Project: 4953-07-0921 Figure: A-1.1b

B12SOIL CRANDALL(DECIMAL ELE) 70921.GPJ LAW CRAN.GDT 6/21/07

THIS RECORD IS A REASONABLE INTERPRETATION OF SUBSURFACE CONDITIONS AT THE EXPLORATION LOCATION. SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND AT OTHER TIMES MAY DIFFER. INTERFACES BETWEEN STRATA ARE APPROXIMATE. TRANSITIONS BETWEEN STRATA MAY BE GRADUAL.

BORING 2

DATE DRILLED: June 7, 2007
EQUIPMENT USED: Hollow Stem Auger
HOLE DIAMETER (in.) 8
ELEVATION: 384.7**



ELEVATION (ft)	DEPTH (ft)	"N" VALUE STD.PEN.TEST	MOISTURE (% of dry wt.)	DRY DENSITY (pcf)	BLOW COUNT* (blows/ft)	SAMPLE LOC.		
							SM	3" thick Asphaltic Paving FILL - SILTY SAND - moist, light brown, fine to medium, brick fragments (up to 3" in size)
380	5							
			14.9	90	5	▩	SM	SILTY SAND - moist, brown, fine to medium, some coarse
			9.4	106	8	▩		
375	10							
		7				☒		Layer of Sandy Silt - moist, brown, fine to medium
								Occasional small gravel
			11.2	102	10	▩		
370	15							
		4				☒		Thin layer of fine to coarse sand Light browish-yellow, fine to coarse
			7.0	106	10	▩		
365	20							
		9				☒		
360	25							Thin layer of Silt
			10.8	102	10	▩		
		11				☒		
355	30							
			11.7	106	14	▩		
350	35	10				☒		
			9.6	113	19	▩		
345	40							

Field Tech: AR
Prepared By: HP
Checked By: LT

(CONTINUED ON FOLLOWING FIGURE)

**Building Conversion
& Parking Structure
Hollywood District, California**

MACTEC

LOG OF BORING
Project: 4953-07-0921
Figure: A-1.2a

BORING 2 (Continued)

DATE DRILLED: June 7, 2007
EQUIPMENT USED: Hollow Stem Auger
HOLE DIAMETER (in.) 8
ELEVATION: 384.7**

ELEVATION (ft)	DEPTH (ft)	"N" VALUE STD.PEN.TEST	MOISTURE (% of dry wt.)	DRY DENSITY (pcf)	BLOW COUNT* (blows/ft)	SAMPLE LOC.	
		10				☒	Layer of Silt
			11.1	117	13	▨	
340	45						
		18				☒	
335	50		12.1	110	15	▨	END OF BORING AT 50 FEET
330	55						
325	60						
320	65						
315	70						
310	75						
305	80						

NOTES:

Ground water not encountered. Boring backfilled with soil cuttings and tamped.

THIS RECORD IS A REASONABLE INTERPRETATION OF SUBSURFACE CONDITIONS AT THE EXPLORATION LOCATION. SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND AT OTHER TIMES MAY DIFFER. INTERFACES BETWEEN STRATA ARE APPROXIMATE. TRANSITIONS BETWEEN STRATA MAY BE GRADUAL.

Field Tech: AR
Prepared By: HP
Checked By: LT

Building Conversion
& Parking Structure
Hollywood District, California

MACTEC

LOG OF BORING
Project: 4953-07-0921 Figure: A-1.2b

MAJOR DIVISIONS			GROUP SYMBOLS		TYPICAL NAMES
COARSE GRAINED SOILS (More than 50% of material is LARGER than No. 200 sieve size)	GRAVELS (More than 50% of coarse fraction is LARGER than the No. 4 sieve size)	CLEAN GRAVELS (Little or no fines)	●	GW	Well graded gravels, gravel - sand mixtures, little or no fines.
			○	GP	Poorly graded gravels or grave - sand mixtures, little or no fines.
		GRAVELS WITH FINES (Appreciable amount of fines)	◌	GM	Silty gravels, gravel - sand - silt mixtures.
			▨	GC	Clayey gravels, gravel - sand - clay mixtures.
	SANDS (More than 50% of coarse fraction is SMALLER than the No. 4 Sieve Size)	CLEAN SANDS (Little or no fines)	⁘	SW	Well graded sands, gravelly sands, little or no fines.
				SP	Poorly graded sands or gravelly sands, little or no fines.
		SANDS WITH FINES (Appreciable amount of fines)	▥	SM	Silty sands, sand - silt mixtures
			▨	SC	Clayey sands, sand - clay mixtures.
FINE GRAINED SOILS (More than 50% of material is SMALLER than No. 200 sieve size)	SILTS AND CLAYS (Liquid limit LESS than 50)		▥	ML	Inorganic silts and very fine sands, rock flour, silty of clayey fine sands or clayey silts and with slight plasticity.
			▨	CL	Inorganic lays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays.
			☰	OL	Organic silts and organic silty clays of low plasticity.
	SILTS AND CLAYS (Liquid limit GREATER than 50)		▥	MH	Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts.
			▨	CH	Inorganic clays of high plasticity, fat clays
			≈	OH	Organic clays of medium to high plasticity, organic silts.
HIGHLY ORGANIC SOILS			▩	PT	Peat and other highly organic soils.

<u>BOUNDARY CLASSIFICATIONS</u>: Soils possessing characteristics of two groups are designated by combinations of group symbols.

■	Undisturbed Sample	▮	Auger Cuttings
☒	Standard Penetration Test	≋	Bulk Sample
▥	Rock Core	▩	Crandall Sampler
▩	Dilatometer	⁘	Pressure Meter
□	Packer	○	No Recovery
▽	Water Table at time of drilling	▼	Water Table after drilling

Correlation of Penetration Resistance with Relative Density and Consistency

SAND & GRAVEL		SILT & CLAY	
No. of Blows	Relative Density	No. of Blows	Consistency
0 - 4	Very Loose	0 - 1	Very Soft
5 - 10	Loose	2 - 4	Soft
11 - 30	Medium Dense	5 - 8	Medium Stiff
31 - 50	Dense	9 - 15	Stiff
Over 50	Very Dense	16 - 30	Very Stiff
		Over 30	Hard

SILT OR CLAY	SAND			GRAVEL		Cobbles	Boulders
	Fine	Medium	Coarse	Fine	Coarse		

No.200 No.40 No.10 No.4 3/4" 3" 12"

U.S. STANDARD SIEVE SIZE

<u>Reference:</u> The Unified Soil Classification System, Corps of Engineers, U.S. Army Technical Memorandum No. 3-357, Vol. 1, March, 1953 (Revised April, 1960)

KEY TO SYMBOLS AND DESCRIPTIONS

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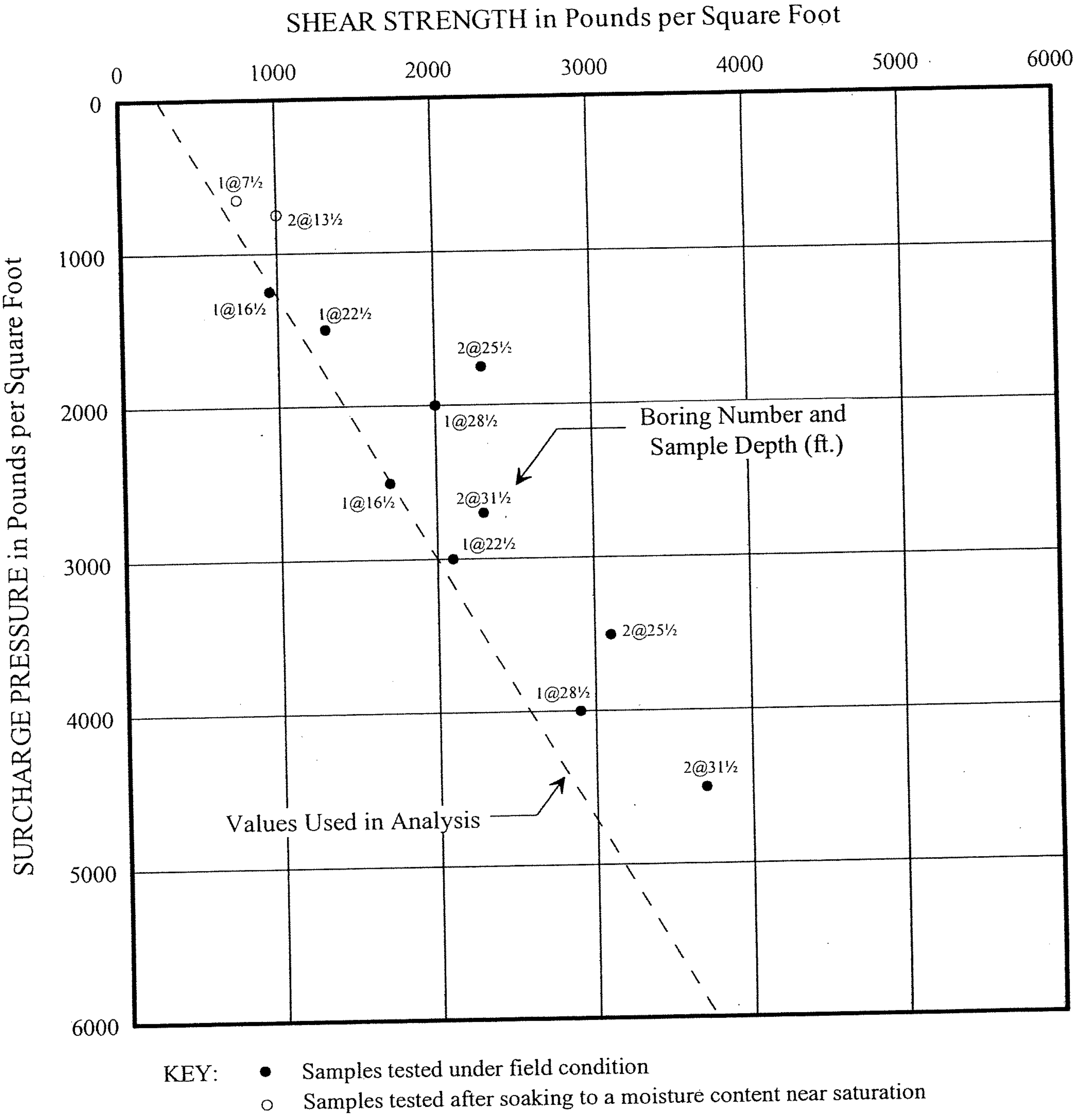
Figure A-2

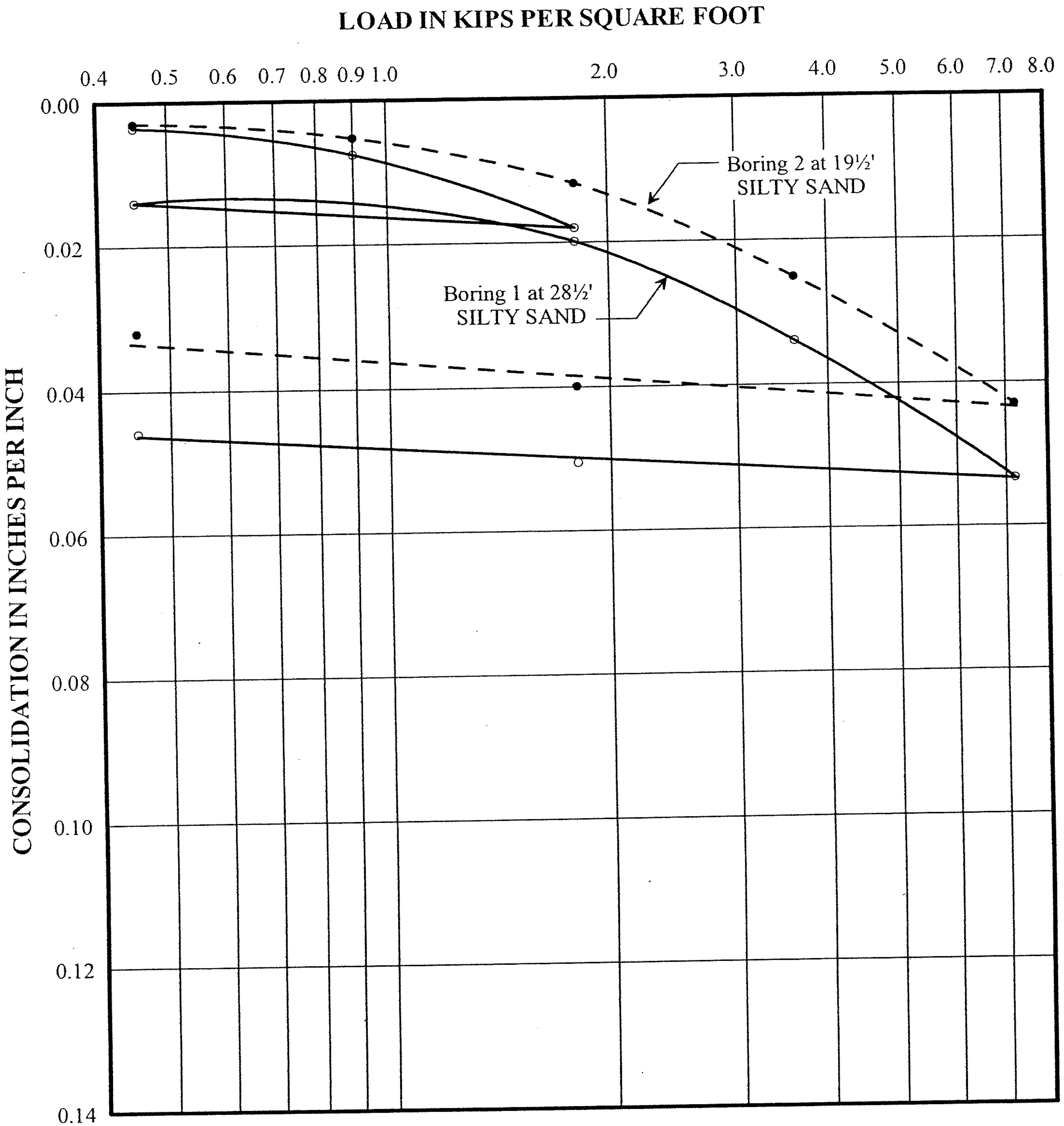


Prepared/Date: EI 06/19/07
Checked/Date: LT

Proposed Bldg. Conversion
& Parking Structure
Hollywood District, California

MACTEC

DIRECT SHEAR TEST DATA
Project No. 4953-07-0921
Figure A-3



NOTE: Samples tested at field moisture content.

Prepared/Date: LT 6/19/2007
Checked/Date: EI 6/19/2007

Proposed Bldg. Conversion
& Parking Structure
Hollywood District, California

MACTEC

CONSOLIDATION TEST DATA
Project No. 4953-07-0921
Figure A-4

TEST PIT NUMBER AND SAMPLE DEPTH:	1 at 0' to 5'
SOIL TYPE:	FILL - SILTY SAND
MAXIMUM DRY DENSITY: (lbs./cu.ft.)	129.6
OPTIMUM MOISTURE CONTENT: (%)	8.3

TEST METHOD: ASTM Designation D1557

Prepared/Date: LT 06/19/07
Checked/Date: EI 06/19/07

Proposed Bldg. Conversion
& Parking Structure
Hollywood District, California

MACTEC

COMPACTION TEST DATA
Project 4953-07-0921
Figure A-5

SCHIFF ASSOCIATES

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Consulting Corrosion Engineers – Since 1959

Jun 21, 2007 via email: LTran@mactec.com

MACTEC
5628 East Slauson Avenue
Los Angeles, CA 90040

Attention: Ms. Lan-Anh Tran

Re: Soil Corrosivity Study
Texas Rock, LLC
Hollywood, California
MAC #4953-07-0921, SA #07-0845SCS

INTRODUCTION

Laboratory tests have been completed on one soil sample provided for the referenced project. The purpose of these tests was to determine if the soil might have deleterious effects on underground utility piping and concrete structures. Schiff Associates assumes that the sample provided is representative of the most corrosive soils at the site.

The proposed construction consists of parking structure. The site is located at 6381 Hollywood Boulevard and 1716-1720 North Cahuenga Boulevard, Hollywood District, Los Angeles, California. The water table is reportedly greater than 50 feet deep.

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

LABORATORY SOIL CORROSIVITY TESTS

The electrical resistivity of the sample was measured in a soil box per ASTM G57 in its as-received condition and again after saturation with distilled water. Resistivities are at about their lowest value when the soil is saturated. The pH of the saturated sample was measured. A 5:1 water:soil extract from the sample was chemically analyzed for the major soluble salts commonly found in soils and for ammonium and nitrate. Test results are shown in Table 1.

SOIL CORROSIVITY

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

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

Soil Resistivity in ohm-centimeters			Corrosivity Category
over		10,000	mildly corrosive
2,000	to	10,000	moderately corrosive
1,000	to	2,000	corrosive
below		1,000	severely corrosive

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

The electrical resistivity was in the moderately corrosive category with as-received moisture. When saturated, the resistivity was in the moderately category. The resistivity dropped with added moisture because the samples were dry as-received.

The soil pH value was 7.8. This is mildly alkaline.

The soluble salt content of the sample was moderate.

Ammonium was detected in a low concentration. The nitrate concentration was high enough to be deleterious to copper.

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

This soil is classified as moderately corrosive to ferrous metals and aggressive to copper.

CORROSION CONTROL RECOMMENDATIONS

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

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

Steel Pipe

1. Bond underground steel pipe with rubber gasketed, mechanical, grooved end, or other nonconductive type joints for electrical continuity. Electrical continuity is necessary for corrosion monitoring and cathodic protection.
2. Install corrosion monitoring test stations to facilitate corrosion monitoring and the application of cathodic protection:
3. To prevent dissimilar metal corrosion cells and to facilitate the application of cathodic protection, electrically isolate each buried steel pipeline per NACE Standard RP0286 from:
    - a. Dissimilar metals.
    - b. Dissimilar coatings (cement-mortar vs. dielectric).
    - c. Above ground steel pipe.
4. Apply a suitable dielectric coating intended for underground use such as:
    - a. Polyurethane per AWWA C222 *or*
    - b. . Extruded polyethylene per AWWA C215 *or*
    - c. A tape coating system per AWWA C214 *or*
    - d. Hot applied coal tar enamel per AWWA C203 *or*
    - e. Fusion bonded epoxy per AWWA C213.
5. Apply cathodic protection to steel piping as per NACE Standard RP0169. The amount of cathodic protection current needed can be minimized by coating the pipe.
6. As an alternative to dielectric coating and cathodic protection, apply a ¾-inch cement mortar coating per AWWA C205 or encase in concrete 3 inches thick, using any type of cement. Joint bonds, test stations, and insulated joints are still required for these alternatives.
7. Some steel piping systems, such as for oil, gas, and high-pressure piping systems, have special corrosion and cathodic protection requirements that must be evaluated for each specific application.

Iron Pipe

For iron pipe, implement *all* the following measures:

1. Encase pressurized cast and ductile iron piping per AWWA Standard C105; *or* coat with epoxy for underground use; *or* polyurethane intended for underground use; *or* with wax tape per AWWA C217. The thin factory-applied asphaltic coating applied to ductile iron pipe for transportation and aesthetic purposes does not constitute a corrosion control coating.
2. Electrically insulate underground iron pipe from dissimilar metals and from above ground iron pipe with insulating joints per NACE International Standard RP0286-2002.

3. Bond all nonconductive type joints for electrical continuity.
4. Install corrosion monitoring test stations as necessary to facilitate corrosion monitoring and the application of cathodic protection.
5. Apply cathodic protection to cast and ductile iron piping as per NACE International Standard RP0169-2002.

Copper Tubing

Protect buried copper tubing by one of the following measures:

1. Prevention of soil contact. Soil contact may be prevented by routing the tubing above ground.
2. Installation of a factory-coated copper pipe with a minimum 25-mil thickness such as Kamco's Aqua Shield™, Mueller's Streamline Protec™, or similar products. Polyethylene coating protects against elements that corrode copper and prevents contamination between copper and sleeving. However, it must be continuous with no cuts or defects if installed underground.
3. Wrapping of copper with 12-mil polyethylene pipe wrapping tape with butyl rubber mastic over a suitable primer. Protect wrapped copper tubing by applying cathodic protection per NACE International Standard RP0169-2002. The amount of cathodic protection current needed can be minimized by coating the tubing.

Plastic and Vitrified Clay Pipe

No special precautions are required for plastic and vitrified clay piping placed underground from a corrosion viewpoint. Protect all metallic fittings and valves with wax tape per AWWA Standard C217-99 or epoxy.

All Pipe

On all pipes, appurtenances, and fittings not protected by cathodic protection, coat bare metal such as valves, bolts, flange joints, joint harnesses, and flexible couplings with wax tape per AWWA Standard C217-99 after assembly.

Where metallic pipelines penetrate concrete structures such as building floors, vault walls, and thrust blocks use plastic sleeves, rubber seals, or other dielectric material to prevent pipe contact with the concrete and reinforcing steel.

Concrete

Any type of cement may be used for concrete structures and pipe because the sulfate concentration is negligible, 0 to 0.1 percent, per 1997 Uniform Building Code (UBC) Table 19-A-4 and American Concrete Institute (ACI-318) Table 4.3.1.

Standard concrete cover over reinforcing steel may be used for concrete structures and pipe in contact with these soils.

CLOSURE

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

Please call if you have any questions.

Respectfully Submitted,
SCHIFF ASSOCIATES

Leobardo Solis

Leobardo Solis

Reviewed by,

Steve R Fox

Steven R. Fox, P. E.

STEVEN R. FOX

Steve R Fox

Enc: Table 1

SCHIFF ASSOCIATES
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Table 1 - Laboratory Tests on Soil Samples

MACTEC
Texas Rock, LLC, Hollywood District, Los Angeles, CA
Your #4953-07-0921, SA #07-0845SCS
11-Jun-07

Sample ID			B-1 @ 5.5' Fill
Resistivity		**Units**	
as-received		ohm-cm	7,340
saturated		ohm-cm	2,484
pH			7.8
Electrical Conductivity		mS/cm	0.25
Chemical Analyses			
Cations			
calcium	Ca^{2+}	mg/kg	130
magnesium	Mg^{2+}	mg/kg	19
sodium	Na^{1+}	mg/kg	74
potassium	K^{1+}	mg/kg	46
Anions			
carbonate	CO_3^{2-}	mg/kg	ND
bicarbonate	HCO_3^{1-}	mg/kg	400
flouride	F^{1-}	mg/kg	ND
chloride	Cl^{1-}	mg/kg	9.2
sulfate	SO_4^{2-}	mg/kg	83
phosphate	PO_4^{3-}	mg/kg	13
Other Tests			
ammonium	NH_4^{1+}	mg/kg	1.8
nitrate	NO_3^{1-}	mg/kg	75.1
sulfide	S^{2-}	qual	na
Redox		mV	na

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

FAULT RUPTURE HAZARD EXPLORATION

SECURITY PACIFIC BANK BUILDING
6381 HOLLYWOOD BLVD. AND
1716 – 1726 NORTH CAHUENGA BLVD.
LOS ANGELES, CALIFORNIA

Submitted to:
SPBB, LLC
% Mr. Billy Reed
501 N.W. Grand Blvd.
Oklahoma City, OK 73118

Prepared by:
ENGEO Incorporated

January 23, 2015

Project No:
11613.000.000

ENGEO
Expect Excellence

GEOTECHNICAL
ENVIRONMENTAL
WATER RESOURCES
CONSTRUCTION SERVICES

Project No.
11613.000.000

January 23, 2015

SPBB, LLC
℅ Mr. Billy Reed
501 N.W. Grand Blvd.
Oklahoma City, OK 73118

Subject: Security Pacific Bank Building
6381 Hollywood Blvd and
1716 – 1726 North Cahuenga Blvd
Los Angeles, California

FAULT RUPTURE HAZARD EXPLORATION

Dear Mr. Reed:

ENGEO prepared this report describing the results of our fault rupture hazard exploration as outlined in our revised September 29, 2014, proposal for the Security Pacific Bank Building and adjacent properties located at the intersection of Hollywood and Cahuenga Boulevards, Los Angeles, California. The conclusions of this report are based on the findings of the California Geological Survey's Fault Evaluation Report (FER) 253 Supplement No. 1 and on our recent exploration.

Based upon our findings, we conclude that the data collected demonstrates the absence of active (Holocene) faulting on the subject parcels and a minimum of 50 feet beyond the northernmost parcel.

If you have any questions or comments regarding this report, please call and we will be glad to discuss them with you.

Sincerely,

ENGEO Incorporated

PATRICK LAM
No. 2490
Exp. 3/31/2017

PHILIP STUECHELI
No. 1640
Exp. 3/31/2016

Patrick S. Lam, CEG

Philip J. Stuecheli, CEG

psl/pjs/jf

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www.engeo.com

TABLE OF CONTENTS

1.0 INTRODUCTION

1.1 PURPOSE AND SCOPE

The purpose of this exploration is to provide an evaluation of potential surface fault rupture hazards at the subject properties (Figure 1). The project that is currently entitled and permitted by the City of Los Angeles includes construction of a new parking garage, office and retail space fronting 1716 and 1720 Cahuenga Boulevard, and seismic upgrades to the existing Security Pacific Bank Building (SPBB). The seven-story SPBB was built during the early 1920s and has been on the National Register of Historic Places since 1983.

A comprehensive regional evaluation of the Hollywood fault zone was previously completed by others (e.g. Dolan et al., 2007; Hernandez and Treiman, 2014; Hernandez, 2014). Traces of the fault zone were found to be sufficiently active and well defined for zoning under the Alquist-Priolo Earthquake Fault Zone (AP Zone) Act. The study presented herein is based upon portions of the recently completed California Geological Survey (CGS) Fault Evaluation Reports and is intended to address the potential for site-specific fault rupture hazard.

The scope of services included review of published geologic maps, review of selected published geologic reports, and review of the 2014 CGS Fault Evaluation Reports. We also examined aerial photographs; however, due to urbanization, the photographs provided nominal value for geomorphic mapping at this site.

The field exploration consisted of advancing twelve Cone Penetrometer Tests (CPTs) and four continuous dry core borings. A summary table of the CPTs and Borings utilized in this study is presented in Table 2.1-1. Samples collected from the cores were tested for physical properties to facilitate correlation with CPT data. Charcoal and organic sediment samples were also obtained for commercial ^{14}C age dating. Selected samples were retained for post-infrared infrared stimulated luminescence (post-IR IRSL) measurements at the University of California at Los Angeles (UCLA). Our team appreciates the assistance of Galpin Motors representatives, who graciously provided access to the adjoining parcel for data collection. We also thank CGS representatives for providing portions of the referenced Metro Rail geotechnical report (Converse et al., 1984).

This report was prepared for the exclusive use of our client and their consultants for project design. In the event that any changes are made in the character, design or layout of the development, we must be contacted to review the conclusions and recommendations contained in this report to determine whether modifications are necessary. This document may not be reproduced in whole or in part by any means whatsoever, nor may it be quoted or excerpted without our express written consent.

1.2 PROJECT DESCRIPTION AND BACKGROUND

The properties and current uses of the properties included in this study (from south to north) are identified in Table 1.2-1 below. All parcels are within the recently established Hollywood AP

Zone. Additional investigation was performed offsite on the Galpin Truck Rentals property in order to clear a minimum of 50 feet north of the northern property.

TABLE 1.2-1

Address	Existing	Status
6381 Hollywood Blvd	Security Pacific Bank Building	Occupied
1716 N Cahuenga Blvd	Sharky's Woodfired Mexican Grill	Occupied
1720 N Cahuenga Blvd	Sharky's Parking Lot	Occupied
1724 N Cahuenga Blvd	Pink Building	Vacant
1726 N Cahuenga Blvd	Star Parking	Occupied
1750 N Cahuenga Blvd	Galpin Truck Rentals (Offsite)	Occupied

1.3 REGIONAL GEOLOGY, FAULTING, AND SEISMICITY

The Hollywood fault is part of a network of east-west trending reverse, oblique-slip, and left-lateral strike-slip faults that extends along the southern edge of the Transverse Ranges (Dolan et al., 2007). The Hollywood fault juxtaposes bedrock along the Santa Monica Mountains against younger alluvial sediments. The bedrock in the footwall north of the site, is predominantly sedimentary Upper and Middle Topanga Formation (Figure 2); to the northwest, a significant body of Cretaceous quartz diorite is mapped (Dibblee and Ehrenspeck, 1991). The following is a brief summary of bedrock geologic units mapped within the vicinity of the present-day Cahuenga Boulevard drainage:

TABLE 1.3-1

Unit	Age	Name	Description
Ttusi	Miocene	Upper Topanga	Gray micaceous clay shale or claystone, crumbly where weathered, and thin interbeds of gray to tan semi-friable sandstone
Tts	Miocene	Middle Topanga	Dark gray sandstone of basaltic grains
Tvb	Miocene	Middle Topanga	Basaltic volcanic rocks, dark gray to black
qd	Cretaceous	Quartz diorite	Medium to light gray, massive to vaguely gneissoid.

Dibblee and Ehrenspeck, 1991

The hanging wall and portions of the incised footwall consist of a series of alluvial fans that are the outlets from several north-south trending canyons within the Santa Monica Mountains. Dolan et al. (2007) initially identified the alluvial fans by evaluating historical topographic maps. Cahuenga Boulevard essentially runs north-south down the axis an alluvial fan. Alluvial fans are also present at the mouths of other canyons along the range front.

Regional geologic maps provide the following descriptions for the alluvial sediments mapped on the site and in the vicinity.

TABLE 1.3-2

Source	Unit	Age	Name	Description
Dibblee and Ehrenspeck, 1991	Qa	Holocene	Alluvium	Clay, sand and gravel; includes gravel and sand of minor stream channels (Surficial sediments, unconsolidated detrital sediments)
Dibblee and Ehrenspeck, 1991	Qae	Late Pleistocene	Alluvium, elevated	Similar to Qa, but slightly elevated and dissected; includes alluvuial fan sediments (Older surficial sediments unconsolidated to weakly consolidated
Bedrossian et al., 2012	Qof	Late to Middle Pleistocene	Old Alluvial Fan Deposits	Slightly to moderately consolidated, moderately dissected boulder, cobble, gravel, sand, and silt deposits issued from a confined valley or canyon

As discussed in the FERs, the fault location has been confirmed north of the site by several studies, including Dolan et al. (1997) and Converse (1984).

The segment of the Hollywood fault that is closest to the site is locally known as the Yucca Street strand (Hernandez, 2014). The Yucca Street strand strikes roughly west-northwest and dips steeply to the north. The fault is relatively discontinuous and obscured by the Cahuenga Boulevard fan in the vicinity of Cahuenga Boulevard. The presence of the Yucca Street strand has been confirmed near the intersection of Cahuenga Boulevard and Yucca Street in a deep boring (Boring 28B; Converse, 1984) and inferred by the change in slope east of Cahuenga Boulevard along Ivar and Vine Streets (see Locality A3 in Hernandez and Treiman, 2014).

The mapped (inferred) Yucca Street strand is located approximately 200 to 230 feet north of the northern most property line (1726 North Cahuenga) that is subject of this study (Figure 3).

The Uniform California Earthquake Rupture Forecast, Version 2 (UCERF 2) calculation of the probability of damaging earthquakes is the report issued by the 2007 Working Group on California Earthquake Probabilities (2008). Using the recent data and numerical models, the Working Group assigned a 97 percent probability of a M6.7 or greater earthquake in southern California during the next 30 years. Based on the historic seismicity, the proximity of known active faults and the estimated earthquake probabilities for the California as a whole, it should be expected that the site will experience strong seismic ground shaking during the lifetime of the proposed improvements. The ground shaking hazard levels at the site are similar to those for most of the Southern California.

2.0 FIELD EXPLORATION

Prior to the field exploration activities, Underground Service Alert and a private utility locator identified potential subsurface utilities and other potential obstructions at the study areas.

Due to urbanization, existing structures and tenants at the site, and anticipated thickness of Holocene material, ENGEO utilized a series of soil cores and CPTs to investigate the potential for fault activity. The exploration transect (Figure 4) was chosen based relative accessibility and proximity to previously published borings by Converse (1984) and Mactec (2007) and clearance from existing utilities.

Several utilities, including gas, electrical, water, communications, and sewer all run along the east side of North Cahuenga Boulevard. The utility congestion limited our ability to explore in front of the existing buildings; however, it is our opinion that the consistency amongst our findings is adequate to support our conclusion.

To clear a minimum 50-foot buffer north of 1726 North Cahuenga Boulevard, Galpin Motors representatives provided access to allow for continuation of the exploration transect. The transect extended approximately 80 feet north of the property line. Given the Yucca Street strand was mapped in an approximately east-southeast orientation, geometric constructions were utilized to confirm the transect length required to clear and shadow a 50-foot zone beyond the subject site (Figure 4).

CPTs were advanced by Middle Earth Geo Testing. The 25-ton CPT rig has a compression-type cone with a 10-square-centimeter (cm^2) tip, and a friction sleeve with a surface area of 150 cm^2. The cone, connected with a series of rods, is pushed into the ground at a constant rate. Cone readings are taken at approximately 5-cm intervals with a penetration rate of 2 cm per second in accordance with ASTM D-3441. Measurements include the tip resistance to penetration of the cone (Qc), the resistance of the surface sleeve (Fs), and pore pressure (U) (Robertson and Campanella, 1988). Logs depicting soil behavior type (SBT) and post-processed logs depicting normalized soil behavior type (SBTn) and normalized soil behavior index (I_c) were utilized to evaluate subsurface stratigraphy. The CPT field and post-processed logs are included in Appendix A.

Drilling and dry core sampling was completed by Martini Drilling Corporation under the direct observation of a California-licensed ENGEO Certified Engineering Geologist from October 7 to October 9, 2014, and on November 20, 2014. Borings were advanced using a CME-75 drill rig equipped with 8-inch-diameter hollow-stem slip auger (4.25 ID) with threaded AW sample rods. Samples were collected using 5-foot by 3.5-inch ID continuous split sample tubes. Sample runs were advanced with a sand catcher in the cutting shoe. Each sampler was advanced in either 2.5-foot or 5-foot runs as noted on the logs.

The cores were extracted from the tubes, place into wooden core boxes, photographed, and logged in the field. As-drilled boring locations and depths are shown in Table 2.1-1.

The logs and profile graphically depict the subsurface conditions encountered at the time of the exploration, in general accordance with the Unified Soil Classification System (USCS). Soil features described on the log include consistency, estimated grain size, the soil color based on the Munsell color chart, the relative development soil structure (if present), the relative accumulation of translocated clay as films on soils grains and fracture surfaces (if present), depositional layering, and contacts between differing soil layers. The geologic profile is presented in Figure 5. The core logs are attached in Appendix B. Photographs of the logs are included in Appendix C.

Where charcoal or other potentially dateable material was encountered, the location was marked for further laboratory evaluation. Samples from C-2 and C-3 were also collected and sealed in the field for post-IR IRSL measurements.

Subsurface conditions at other locations may differ from conditions occurring at these boring locations, and the passage of time may result in altered subsurface conditions. In addition, stratification lines represent the approximate boundaries between soil types, and the transitions may be gradual. Select samples recovered during exploration activities were tested to determine various soil characteristics as described in Appendix D.

Onsite auger borings (C-1 through C-3) were backfilled with cuttings and tamped upon completion of sampling; the offsite boring (A-C-4) was sealed with cement grout placed with a tremie pipe upon completion.

Upon completion of the field logging, the core boxes were transported to an offsite location. The core boxes were placed side by side, opened, and correlations between each core were noted.

2.1 EXPLORATION LOCATIONS

Personnel from Brandow and Johnston, Inc. (BJSCE) provided surveying services as part of the project development team. On October 29, 2014, BJSCE surveyed the as-built locations of the historical Mactec (2007) borings and ENGEO's then-completed CPTs and borings. The locations and elevations of the boring and CPTs completed on the Galpin property were approximated based upon BJSCE's surveyed topographic map and measurements from existing features. The locations of the Converse (1984) borings were plotted and calculated using the referenced report. The locations are reported in California State Plane Zone V NAD83 (feet) coordinates with elevations in NAVD 1988 datum.

The location and elevations of our explorations are approximate should be considered accurate only to the degree implied by the method used.

TABLE 2.1-1
Exploration Survey Locations

Name	By*	Date	Location Source	Elev.*** (Feet)	N**** (Feet)	E**** (Feet)	Depth (Feet)	Refusal?
28	C	1/5/1981	Report	388.8	1859630	6461930	202.0	--
28-5	C	11/19/1983	Report	387.5	1859660	6461880	100.0	--
28-4	C	11/20/1983	Report	392.0	1859830	6461880	85.0	--
B-1	M	6/7/2007	As-built**	393.3	1859790	6461960	50.5	--
B-2	M	6/7/2007	As-built**	392.6	1859770	6462090	50.0	--
CPT-01	E	10/6/2014	As-built**	396.2	1859880	6461950	59.7	Refusal
CPT-02	E	10/6/2014	As-built**	395.8	1859860	6461960	86.9	Refusal

Name	By*	Date	Location Source	Elev.*** (Feet)	N**** (Feet)	E**** (Feet)	Depth (Feet)	Refusal?
CPT-03	E	10/6/2014	As-built**	395.7	1859850	6461960	87.4	Refusal
CPT-04	E	10/6/2014	As-built**	393.2	1859790	6461960	87.1	Refusal
CPT-05	E	10/6/2014	As-built**	392.7	1859770	6461960	86.5	Refusal
C-1	E	10/7/2014	As-built**	396.4	1859900	6461950	90.0	--
C-2	E	10/8/2014	As-built**	393.4	1859800	6461960	90.0	--
C-3	E	10/9/2014	As-built**	390.6	1859690	6461960	90.0	--
CPT-06	E	11/19/2014	As-built**	390.6	1859690	6461950	86.0	Refusal
CPT-07	E	11/19/2014	As-built**	392.8	1859780	6462090	91.7	Refusal
A-C-4	E	11/20/2014	Approx.	397.5	1859950	6461960	90.0	--
A-CPT-08	E	12/15/2014	Approx.	398.0	1859990	6461960	83.7	Refusal
A-CPT-09	E	12/15/2014	Approx.	398.0	1859970	6461970	80.4	Refusal
A-CPT-10	E	12/15/2014	Approx.	397.5	1859960	6461970	81.0	Refusal
A-CPT-11	E	12/15/2014	Approx.	396.9	1859940	6461960	89.1	Refusal
A-CPT-12	E	12/15/2014	Approx.	396.8	1859920	6461960	60.4	Refusal

Notes:
* C-Converse (1984), M-Mactec (1997), E-ENGEO (this study)
** As-built field survey by BJSCE 10/29/2014
*** NAVD 1988 (feet)
**** State Plane Zone V (NAD83, feet)

2.2 SOIL UNITS

Soil behavior type (SBTn) processed from the CPT data was utilized during the preparation of the geologic profile and the results were compared to the core logs. In addition, the CPT data was post processed and presented on the exploration profile as a normalized soil behavior index (I_c). Identifying contacts using I_c can be particularly helpful, especially if the transition is subtle and occurs within a single SBT unit. In other words, a sandy silt lens (I_c=2.55) within a silt layer (I_c=2.8) would potentially be missed in a SBT plot because the units all behave like SBT Zone 4. The use of I_c also allows for determining differences between sharp and gradational soil contacts.

Furthermore, Robertson (1990) stressed that the CPT responds to the in-situ mechanical behavior of the soil and not directly to soil classification criteria based on grain-size distribution. Thus, our evaluation looked at relative I_c differences within a single CPT and between separate CPTs. The following is a summary of correlations for soil, SBT, and I_c (Jefferies and Davies, 1993).

TABLE 2.2-1

Soil Classification	SBT Zone (Robertson, 1990)	I_c Range
Organic Clay Soils	2	$I_c >3.22$
Clay	3	$2.82 < I_c < 3.22$
Silt Mixtures	4	$2.54 < I_c < 2.82$
Sand Mixtures	5	$1.90 < I_c < 2.54$
Sands	6	$1.25 < I_c < 1.90$
Gravelly Sands	7	$I_c < 1.25$

Our nomenclature for the soil units follows closely what was set forth by the USGS and CGS (Bedrossian et al., 2012), for classification of Quaternary deposits. Units are described below.

TABLE 2.2-2
Soil Units Utilized in Geologic Profile

Unit	Age	Name	Description
Qaf	Latest Holocene	Artificial Fill	Fill consists of aggregate base or silty sand, placed or disturbed during site development.
Qyf	Holocene	Young alluvial fan	From increasing to decreasing order of occurrence, deposits generally consist of interbedded sandy clay, clayey silt, silty sand, and very thinly bedded gravel. Generally exhibits 10YR to 7.5YR hues.
Qof	Late Pleistocene	Old alluvial fan	From increasing to decreasing order of occurrence, deposits generally consist of interbedded silty sand, silt, silty/sandy clay, and gravel. Some intervals are strongly cemented. Predominantly exhibits 5YR hues. Ages confirmed with ^{14}C.

2.2.1 Surficial fill (Qaf)

Surficial fill material (Qaf) is described as relatively loose silty sand. Qaf was encountered in each of the borings and ranges in thickness from 5 to 8 feet. The fill material appears to be locally derived.

Each core, CPT, and attempted CPT within 1726 N Cahuenga Blvd (Star Parking) encountered a layer of concrete beneath the existing pavement. Failed attempts at CPTs at the eastern end of the property encountered concrete between 4 and 5 feet bgs.

Core C-3 was advanced in the alley behind 6381 Hollywood Boulevard; because of the basement within the building, the greater thickness of fill was expected. In Figure 5, the profile of the basement is shown as Qaf, and the thickness was based upon BJSCE's basement elevation survey.

2.2.2 Young alluvial fan (Qyf)

Young alluvial fan deposits (Qyf) are described as of interbedded clayey sand/sandy clay, silty sand, silt, and gravel. The deposits generally vary with dark yellowish brown, brown, and yellowish brown.

Several distinct and laterally continuous fine-grained layers are interpreted on the geologic profile (Figure 5). One distinct and laterally continuous 2- to 3-foot-thick silty clay was encountered at approximately 10 feet below ground surface bgs. Within the central portion of the profile between 17 and 22 feet bgs, two continuous silty clay layers were observed. Just above 25 feet, an unbroken fine-grained layer mantling silty sand was observed in all the ENGEO CPTs and borings; two of the previous borings (B-1 and 28) also reported a fine-grained interval at the same depth.

A continuous and unbroken fine-grained layer between 35 and 38 feet up to 4 feet thick is located within the base of the Holocene. There is a slight increase in reddish hues within the transition between approximately 35 and 40 feet. Given the lack of dateable charcoal within this zone, we conservatively place the Holocene-Late Pleistocene boundary below 40 feet.

2.2.3 Old alluvial fan (Qof)

Old alluvial fan deposits (Qof) are described as interbedded silty sand, silt, silty/sandy clay, and gravel. The soil has predominantly (yet subtlety distinct from Qyf) strong brown, reddish brown, and yellowish red hues.

The top of Qof is delineated by a laterally continuous silty clay to clayey silt up to 4 feet thick, starting at approximately 41 to 42 feet bgs. In A-C-4, millimeter-scale laminations were observed at approximately 41 feet bgs, suggesting climactic quiescence.

Increased occurrences of several-feet-thick coarse-grained sediments and gravel are present within Qof. A continuous granular layer including silty sand and gravel was encountered at the north end of the profile between A-CPT-08 and CPT-03 at depths below 55 to 60 feet. Two CPTs (A-CPT-12 and CPT-01) hit refusal at 60 feet. South of this layer (between C-2 and C-3) is a distinct and continuous interbedded package of clay, silt, and sand.

A distinct and laterally continuous fine-grained layer was observed between approximately 70 and 75 feet bgs. The thickness of the layers ranged from 1 to 3 feet thick. The layers yielded dateable charcoal in three of the four cores (Table 2.4-1). The radiocarbon ages are late Pleistocene and are consistent with the soil hue observations. The age range between the youngest oldest samples within that horizon is approximately 2,400 years. It is interesting to note that strong pedogenic structures were generally absent; where observed, the structures tended to be subtle and platy.

A very hard gravel and cemented sand layer was also encountered towards the base of the drilled borings (below 80 to 85 feet); the same layer was also documented in nearby Converse (1984) borings. Refusal was encountered in this layer by all the CPTs that were advanced to at least

80 feet. Significant clay films were also observed on the gravels. Some of the cobbles encountered were dioritic.

2.3 LABORATORY TESTING

The summary of laboratory test results is below. The lab testing results were used to help refine the field logs. Laboratory testing was completed by AP Testing, a City of Los Angeles certified laboratory. Laboratory test reports are included in Appendix D.

TABLE 2.3-1
Summary of Laboratory Testing

Boring	Depth (feet)	Gravel (%)	Sand (%)	Fines (%)	Plasticity Index	USCS	Moisture (%)	pH
C-1	26	6	90	4	--	SP	1	7.5
C-1	41.5	3	46	51	21	CL	15.1	7.4
C-1	66	18	75	7	--	SW-SM	2.2	7.6
C-2	13	1	60	39	15	SC	--	7.7
C-2	29	1	47	52	20	CL	14.9	7.6
C-2	34	3	77	20	--	SM	7	7.6
C-2	39	2	72	26	--	SM	7	7.6
C-2	49	7	58	35	--	SM	10.3	7.4
C-2	52	1	55	44	--	SM	--	7.4
C-2	60	0	34	66	23	CL	19.9	7.2
C-2	61	1	54	45	15	SC	--	7.5
C-2	68	6	72	22	--	SM	7.7	7.6
C-2	81	2	63	35	7	SC	--	7.6
C-3	18	5	91	4	--	SP	--	7.9
C-3	29	2	58	40	--	SM	11.5	7.6
C-3	39	1	60	39	--	SM	12.6	7.5
C-3	42	3	73	24	--	SM	--	7.8
C-3	49	1	60	39	--	SM	11.8	7.5
C-3	59	0	50	50	18	CL	16.1	7.6
C-3	69	17	57	26	--	SM	9.3	7.3
C-3	78.5	1	67	32	10	SC	9.7	7.3

2.4 RADIOMETRIC AGE DATING

After the core boxes were delivered to the laboratory, we performed secondary examination of charcoal-bearing zones. Charcoal and/or organic sediment was obtained from the core interval and wrapped in aluminum foil after excess soil was removed. The material was wrapped bagged, labeled, and shipped to Beta Analytic for radiocarbon age dating. The samples collected and measured are presented in Table 2.4-1 below. Beta Analytic's laboratory reports are included in Appendix E.

Given the age concordance of the organic sediment sample (11613 C-3@67.5) and the deeper charcoal sample (11613 C-3@72), it is possible that the shallower sample is reworked sediment.

TABLE 2.4-1
Results of ^{14}C Age Dating

Sample ID	Type*	Unit	Core Elevation	Sample Elevation	Cal BC	Cal BP
11613 A-C-4@73.5	C	Qof	397.5	324.0	15,535 to 15,240	17,485 to 17,190
11613 C-1@72.75	C	Qof	396.4	323.7	15,970 to 15,730	17,920 to 17,680
11613 C-3@67.5	S	Qof	390.6	323.1	13,610 to 13,315	15,560 to 15,265
11613 C-3@72	C	Qof	390.6	318.6	13,825 to 13,610	15,775 to 15,560

Notes:
* C-Charcoal, S-Organic Sediment

3.0 GROUNDWATER

Groundwater was not encountered in the borings drilled onsite (C-1 through C-3). Perched groundwater was encountered to the north within A-C-4 at approximately 72½ feet bgs and extended to 76¼ feet bgs. A moist (neither wet nor saturated) silty clay aquitard was present below the saturated zone. Permeable soils below the aquitard were moist (neither wet nor saturated) down to the bottom of the core. At approximately 87½ feet bgs, visible water was present, but the soil was not saturated.

The Yucca Street strand and other Hollywood fault segments are known to act as a groundwater barrier (Hernandez and Treiman, 2014). Review of the Converse (1984) report also indicates a significant change in depth to groundwater across the Yucca Street strand.

We interpret the two water bearing zones as perched groundwater zones that are either slowly permeating through the fault zone or cascading down over the less permeable fault zone. It also appears that the groundwater elevations have decreased during the past 30-plus years given C-1 through C-3 is dry in an area that encountered groundwater during the early 1980s.

4.0 DISCUSSION AND CONCLUSIONS

Despite the challenge of identifying continuous sedimentary units within an alluvial environment given the cyclical nature of fine-grained and coarse-grained sediment deposition, the exploration data herein has yielded several distinct units across the site that indicates that the fault rupture hazard for the site is low. The additional clearance on the Galpin Truck Rental property (offsite) north of 1726 North Cahuenga Boulevard provides at least 50 feet of clearance beyond the subject properties.

The borings and CPTs show several vertically overlapping, distinct, continuous, and unbroken units. At least three distinct fine-grained units within Qyf demonstrate that no disruption has occurred at the study area during the Holocene. Within Qof, a single fine-grained charcoal-containing layer was encountered; this layer ties together continuity amongst coarse-grained layers at the north end of the transect, and a continuous fine-grained layer at the south end of the transect. Thus, Qof has not been disrupted during the late Pleistocene.

Not all potentially correlateable units within the CPT traces have been shown on the geologic profile (Figure 5). The correlated units shown on the geologic profile were noted as distinct in the field, and confirmed when the cores were examined side-by-side.

Considering the site is located at the fringe of the AP fault zone and several hundred feet away from the Yucca Street strand, by inspection, we considered the likelihood of the presence of Holocene fault rupture prior to our exploration to be low. Based on the results of our research and subsurface exploration, there is no evidence that Holocene surface fault rupture has occurred along the exploration transect.

Considering previous workers have noted that the Hollywood fault acts as a groundwater barrier (e.g. Hernandez, 2014 and the references therein), the perched groundwater encountered in only the northernmost boring (A-C-4) suggests that we are potentially encroaching closer to the actual fault.

Preliminary analysis of K-feldspar sediment grains collected for post-IR IRSL suggest an igneous provenance. The axis of the present-day alluvial fan runs north-south along North Cahuenga Boulevard. The bedrock mapped by others (e.g. Dibblee, 1991) show only older sedimentary formations directly up canyon. The dioritic cobbles and igneous-derived (K-feldspar) sediment found at depth; therefore, may have been deposited during a time when the present-day alluvial fan was aligned with canyons to the west, where bedrock exposures of granodiorite are mapped. Such a reconstruction is consistent with documented components of left-lateral movement along the Hollywood fault.

5.0 LIMITATIONS AND UNIFORMITY OF CONDITIONS

This geological study is issued with the understanding that it is the responsibility of the owner to transmit the information and recommendations of this report to others involved with the project including but not limited to contractors, buyers, architects, engineers, and designers for the project so that the necessary steps can be taken by the contractors and subcontractors to carry out such recommendations in the field. The conclusions and recommendations contained in this report are solely professional opinions.

The professional staff of ENGEO Incorporated strives to perform its services in a proper and professional manner with reasonable care and competence but is not infallible. There are risks of earth movement and property damages inherent in the design and construction of facilities and utilities. We are unable to eliminate all risks or provide insurance; therefore, we are unable to guarantee or warrant the results of our services.

This study is based upon field and other conditions discovered at the time of preparation of ENGEO's documents of service. This document must not be subject to unauthorized reuse; that is, reuse without written authorization of ENGEO. Such authorization is essential because it requires ENGEO to evaluate the document's applicability given new circumstances, not the least of which is passage of time. Actual field or other conditions will necessitate clarifications, adjustments, modifications or other changes to ENGEO's documents of service. Therefore, ENGEO must be engaged to prepare the necessary clarifications, adjustments, modifications or other changes before construction activities commence or further activity proceeds. If ENGEO's scope of services does not include a design-level geotechnical exploration, onsite construction observation, or if other persons or entities are retained to provide such services, ENGEO cannot be held responsible for any or all claims, including, but not limited to claims arising from or resulting from the performance of such services by other persons or entities, and any or all claims arising from or resulting from clarifications, adjustments, modifications, discrepancies or other changes necessary to reflect changed field or other conditions.

If ENGEO's scope of services does not include onsite construction observation, or if other persons or entities are retained to provide such services, ENGEO cannot be held responsible for any or all claims, including, but not limited to claims arising from or resulting from the performance of such services by other persons or entities, and any or all claims arising from or resulting from clarifications, adjustments, modifications, discrepancies or other changes necessary to reflect changed field or other conditions.

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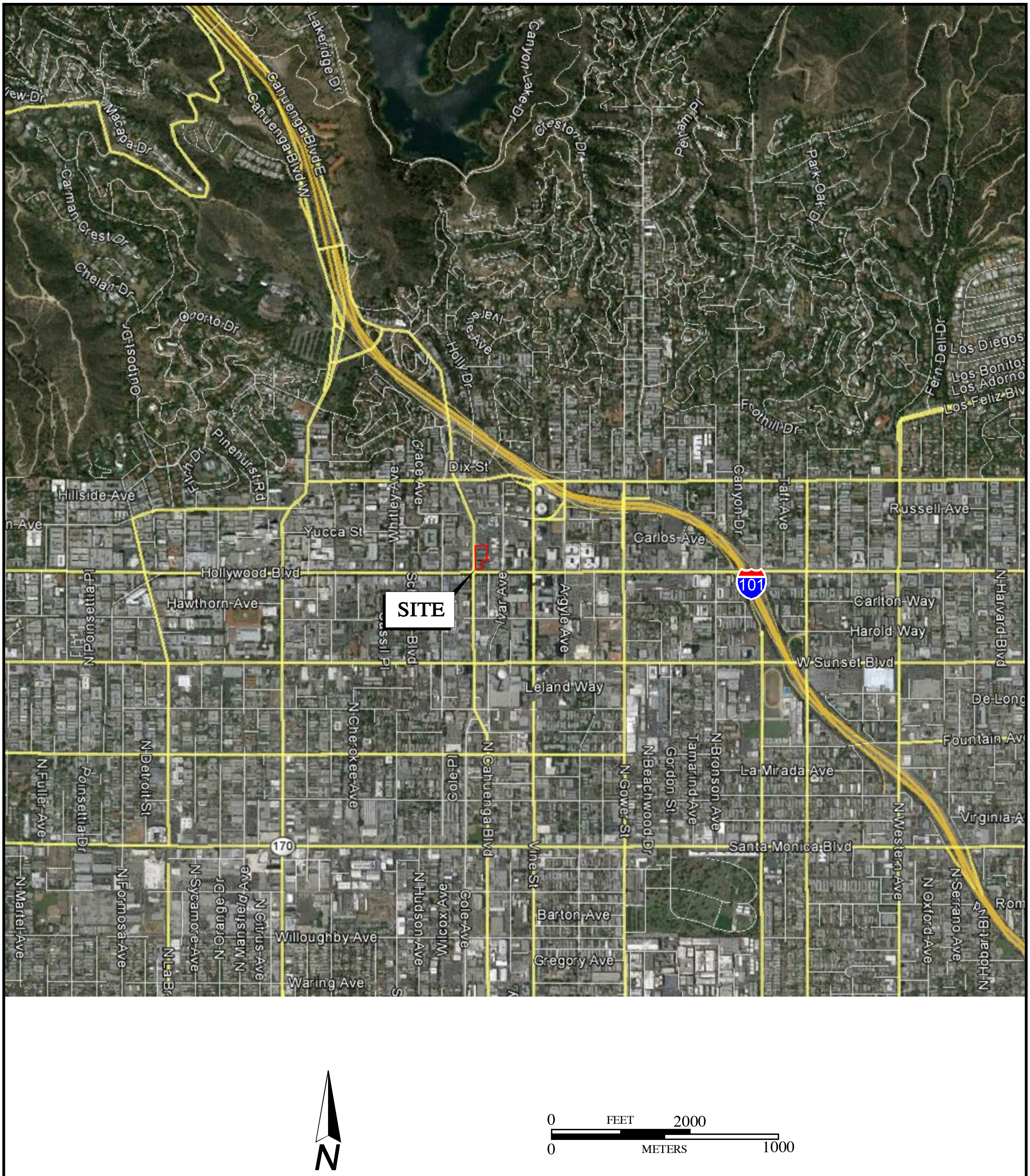
FIGURES

FIGURES



BASE MAP SOURCE: GOOGLE EARTH PRO

ENGEO
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VICINITY MAP

SECURITY PACIFIC BANK BUILDING
LOS ANGELES, CALIFORNIA

PROJECT NO.: 11613.000.000

SCALE: AS SHOWN

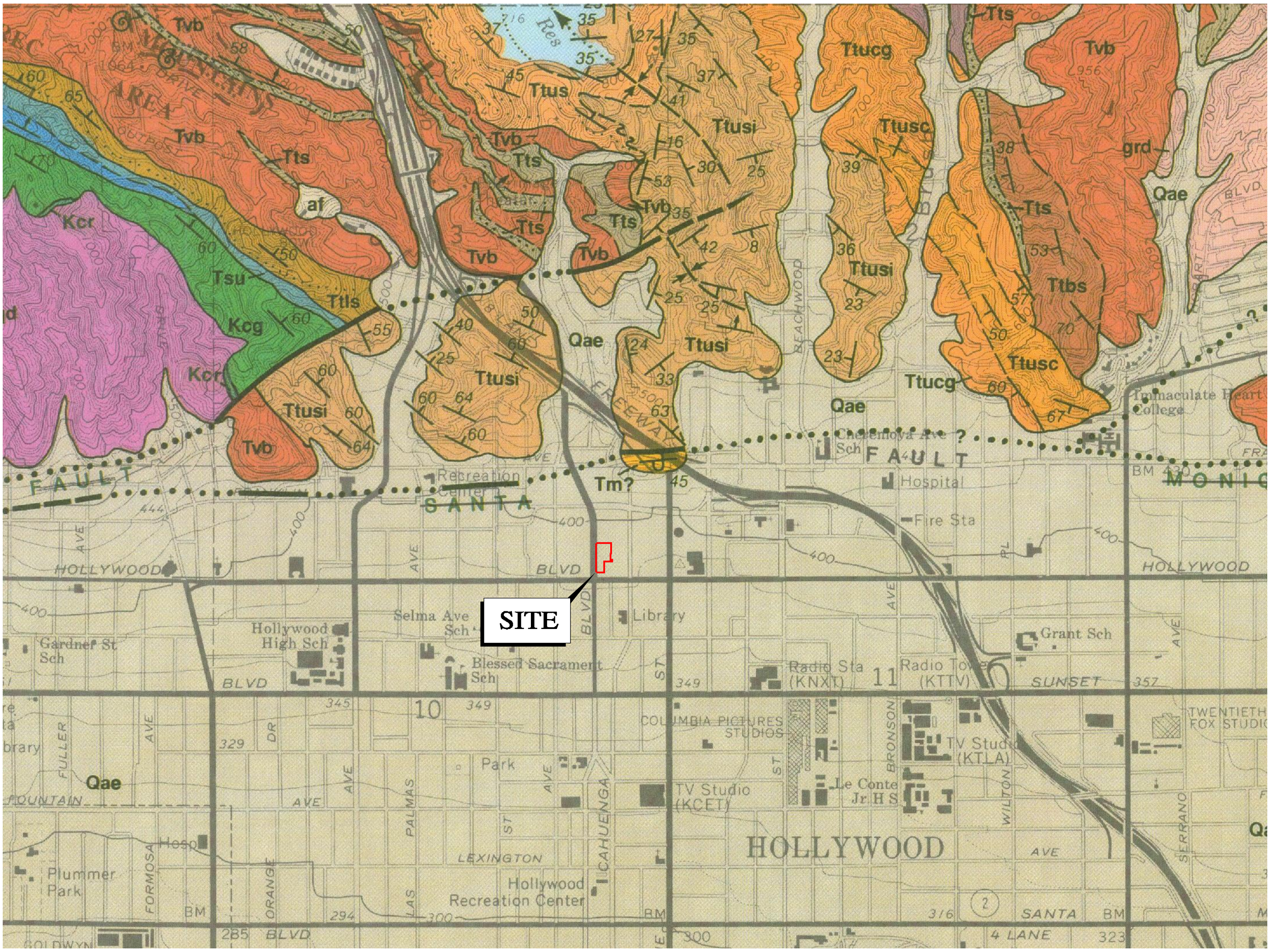
DRAWN BY: SRP

CHECKED BY: PS

FIGURE NO. 1

G:\Drafting\DRAFTING2_Dwg_10000 to 12999\11613\000\ESA\11613000000-ESA-1-VicinityMap-0115.dwg Plot Date: 2-27-15 spatters

ORIGINAL FIGURE PRINTED IN COLOR



EXPLANATION

GEOLOGIC CONTACT-DASHED WHERE GRADATIONAL OR APPROXIMATELY LOCATED

FAULT-DASHED WHERE INFERRED, DOTTED WHERE CONCEALED, QUERIED WHERE EXISTENCE IS DOUBTFUL. SAWTEETH ARE ON UPPER PLATE OF LOW ANGLE THRUST FAULT.

STRIKE AND DIP OF STRATA

INCLINED

AXIS OF FOLD

ANTICLINE

SYNCLINE

Qa	ALLUVIUM
Qae	ALLUVIUM SLIGHTLY ELEVATED
Tush	UNNAMED SHALE
Tm	MONTEREY FORMATION
Ttusc	UPPER TOPANGA FORMATION
Ttbs	DARK GRAY SANDSTONE
Tvb	BASALTIC VOLCANIC ROCKS
Ttusi	GRAY MICACEOUS CLAY SHALE
qd	QUARTZ DIORITE
grd	GRANODIORITE

N

0 FEET 2000
0 METERS 1000

NOTE: MAP IS PRESENTED TO EMPHASIZE UPSLOPE BEDROCK FORMATIONS; REFER TO HERNANDEZ (2014) FOR DETAILED FAULT DELINEATION

BASE MAP SOURCE: DIBBLEE, 1991

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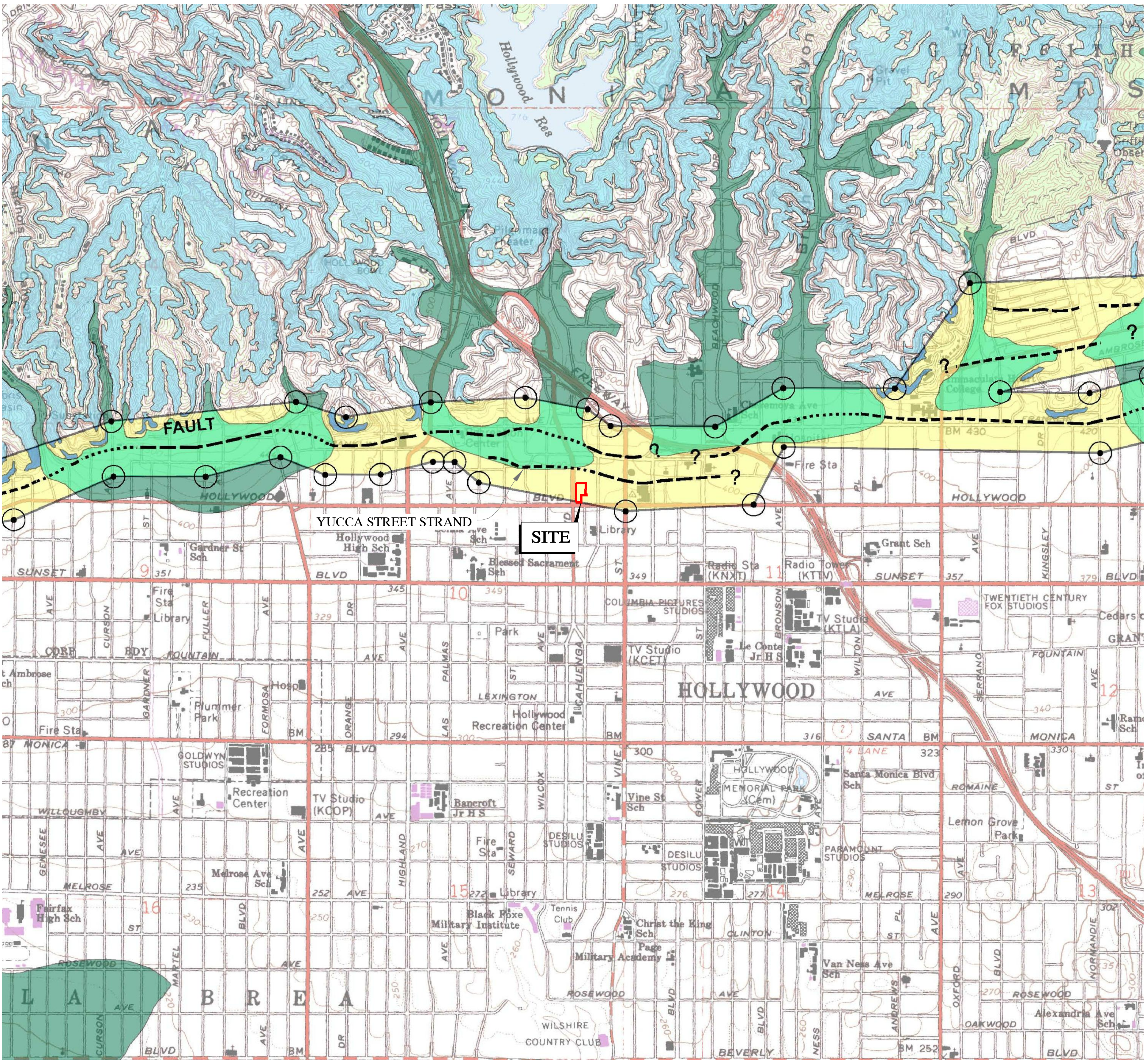
GEOLOGIC MAP
SECURITY PACIFIC BANK BUILDING
LOS ANGELES, CALIFORNIA

PROJECT NO.: 11613.000.000
SCALE: AS SHOWN
DRAWN BY: SRP
CHECKED BY: PS

FIGURE NO.
2

G:\Drafting\DRAFTING2_Dwg_10000 to 12999\11613\000\ESA\1161300000-ESA-3-ZonesOfInvestigationMap-0115.dwg Plot Date: 2-27-15 spatters



MAP EXPLANATION

ALQUIST-PRIOLO EARTHQUAKE FAULT ZONES

Earthquake Fault Zones
Zone boundaries are delineated by straight-line segments that connect encircled turning points; 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.

Active Fault Traces
Faults considered to have been active during Holocene time and to have potential for surface rupture; solid line where accurately located, long dash where approximately located, short dash where inferred, dotted 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.

1906 C

? ?

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.

OVERLAPPING ALQUIST-PRIOLO AND SEISMIC HAZARD ZONES

Overlap of Earthquake Fault Zone and Liquefaction Zone
Areas that are covered by both Earthquake Fault Zone and Liquefaction Zone.

Overlap of Earthquake Fault Zone and Earthquake-Induced Landslide Zone
Areas that are covered by both Earthquake Fault Zone and Earthquake-Induced Landslide Zone.

**Note: Mitigation methods differ for each zone –
AP Act only allows avoidance; Seismic Hazard Mapping Act allows mitigation by engineering/geotechnical design as well as avoidance.**

N

0 FEET 2000
0 METERS 1000

BASE MAP SOURCE: CALIFORNIA GEOLOGIC SURVEY

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ZONES OF REQUIRED INVESTIGATION
SECURITY PACIFIC BANK BUILDING
LOS ANGELES, CALIFORNIA

PROJECT NO.: 11613.000.000	
SCALE: AS SHOWN	
DRAWN BY: SRP	CHECKED BY: PS

FIGURE NO
3

ORIGINAL FIGURE PRINTED IN COLOR

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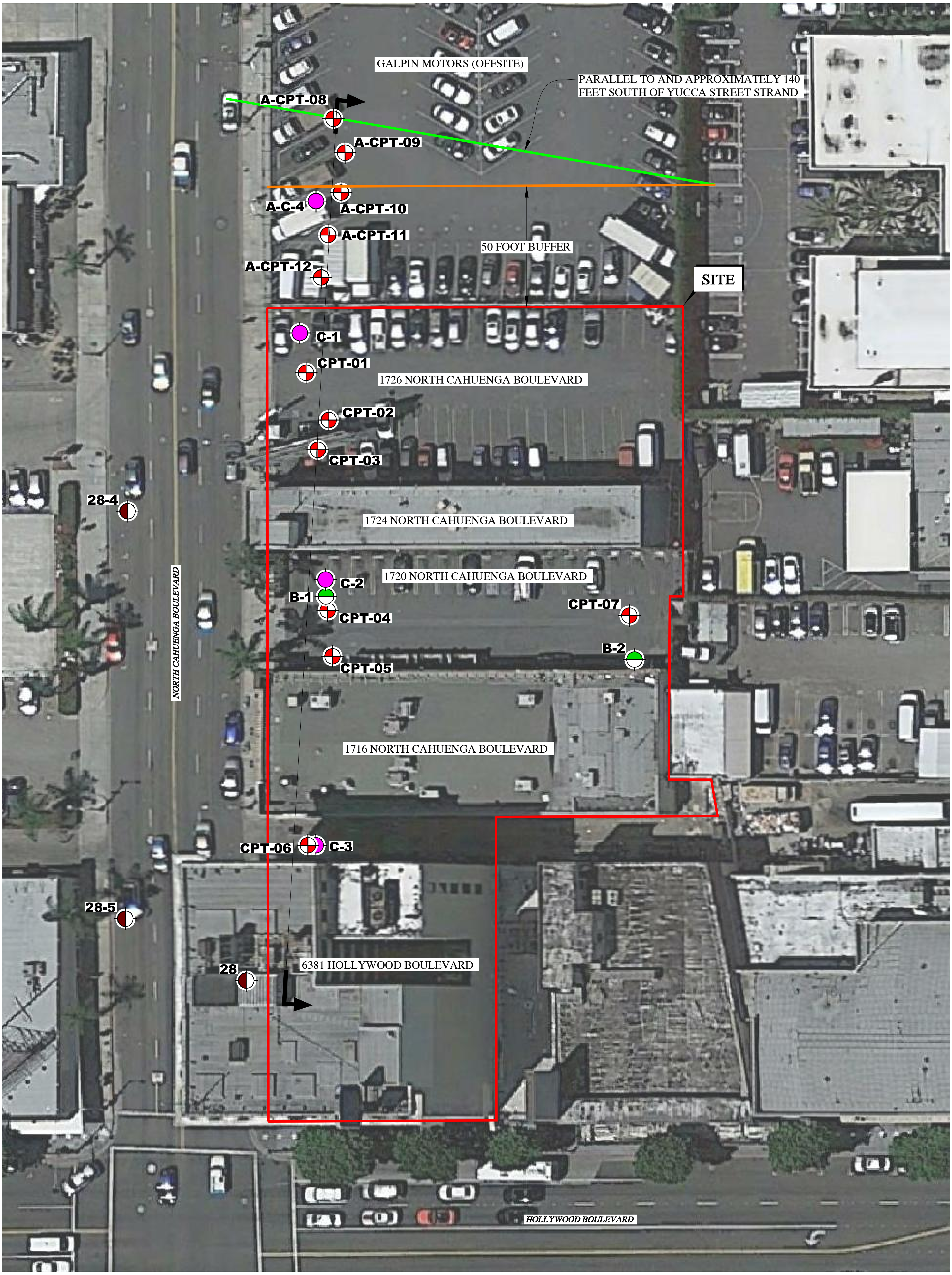

EXPLANATION

ALL LOCATIONS ARE APPROXIMATE

A-C-4 ● BORING (ENGEO, 2014)

B-2 ● BORING (MACTEC, 2007)

28-5 ● BORING (CONVERSE, 1984)

A-CPT-12 ● CONE PENETRATION TEST (ENGEO, 2014)

GEOLOGIC PROFILE LOCATION

N

0 FEET 40
0 METERS 20

BASE MAP SOURCE: GOOGLE EARTH PRO

ENGEO
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SITE PLAN
SECURITY PACIFIC BANK BUILDING
LOS ANGELES, CALIFORNIA

PROJECT NO.: 11613.000.000
SCALE: AS SHOWN
DRAWN BY: SRP
CHECKED BY: PS

FIGURE NO
4

ORIGINAL FIGURE PRINTED IN COLOR

PLAN VIEW

EXPLANATION

ALL LOCATIONS ARE APPROXIMATE, LOCATION SURVEYED BY BJCSE IF CIRCLED

🟢	CPT (ENGEO)	🔵	BORING (MACTEC, 2007)
🔵	CORE (ENGEO)	🟣	BORING (CONVERSE, 1984)

PROFILE

KEY TO BORING LOGS

MAJOR TYPES			DESCRIPTION
COARSE-GRAINED SOILS MORE THAN HALF OF MAT'L LARGER THAN #200 SIEVE	GRAVELS MORE THAN HALF COARSE FRACTION IS LARGER THAN NO. 4 SIEVE SIZE	CLEAN GRAVELS WITH LITTLE OR NO FINES	GW - Well graded gravels or gravel-sand mixtures GP - Poorly graded gravels or gravel-sand mixtures
		GRAVELS WITH OVER 12 % FINES	GM - Silty gravels, gravel-sand and silt mixtures GC - Clayey gravels, gravel-sand and clay mixtures
	SANDS MORE THAN HALF COARSE FRACTION IS SMALLER THAN NO. 4 SIEVE SIZE	CLEAN SANDS WITH LITTLE OR NO FINES	SW - Well graded sands, or gravelly sand mixtures SP - Poorly graded sands or gravelly sand mixtures
		SANDS WITH OVER 12 % FINES	SM - Silty sand, sand-silt mixtures SC - Clayey sand, sand-clay mixtures
FINE-GRAINED SOILS MORE THAN HALF OF MAT'L SMALLER THAN #200 SIEVE	SILTS AND CLAYS LIQUID LIMIT 50 % OR LESS		ML - Inorganic silt with low to medium plasticity CL - Inorganic clay with low to medium plasticity OL - Low plasticity organic silts and clays
	SILTS AND CLAYS LIQUID LIMIT GREATER THAN 50 %		MH - Inorganic silt with high plasticity CH - Inorganic clay with high plasticity OH - Highly plastic organic silts and clays
	HIGHLY ORGANIC SOILS		PT - Peat and other highly organic soils

SOIL CLASSIFICATION	GRAVELLY SANDS	SANDS	SAND MIXTURES	SILT MIXTURES	CLAY	ORGANIC CLAY SOILS
Ic RANGE	Ic < 1.25	1.25 < Ic < 1.90	1.90 < Ic < 2.54	2.54 < Ic < 2.82	2.82 < Ic < 3.22	Ic > 3.22

(JEFFERIES AND DAVIES, 1993)

EXPLANATION

🟫	Qaf	ARTIFICIAL FILL
🟨	Qyf	QUATERNARY YOUNG ALLUVIAL FAN (HOLOCENE)
🟨	Qof	QUATERNARY OLD ALLUVIAL FAN (LATE PLEISTOCENE)
▨	CL/ML	DISTINCT FINE-GRAINED HORIZON WITHIN MAPPED UNIT
▨	GM/SM	DISTINCT COARSE-GRAINED HORIZON WITHIN MAPPED UNIT

NOTE:
BLUE - LINED LOGS AND CPTS BY ENGEO (THIS STUDY)
BLACK - LINED LOGS BY OTHERS AS NOTED

BASE MAP SOURCE: BJCSC

ENGEO
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EXPLORATION PROFILE
SECURITY PACIFIC BANK BUILDING
LOS ANGELES, CALIFORNIA

PROJECT NO.: 11613.000.000
SCALE: AS SHOWN
DRAWN BY: SRP
CHECKED BY: PS
FIGURE NO. 5

ORIGINAL FIGURE PRINTED IN COLOR

APPENDIX A

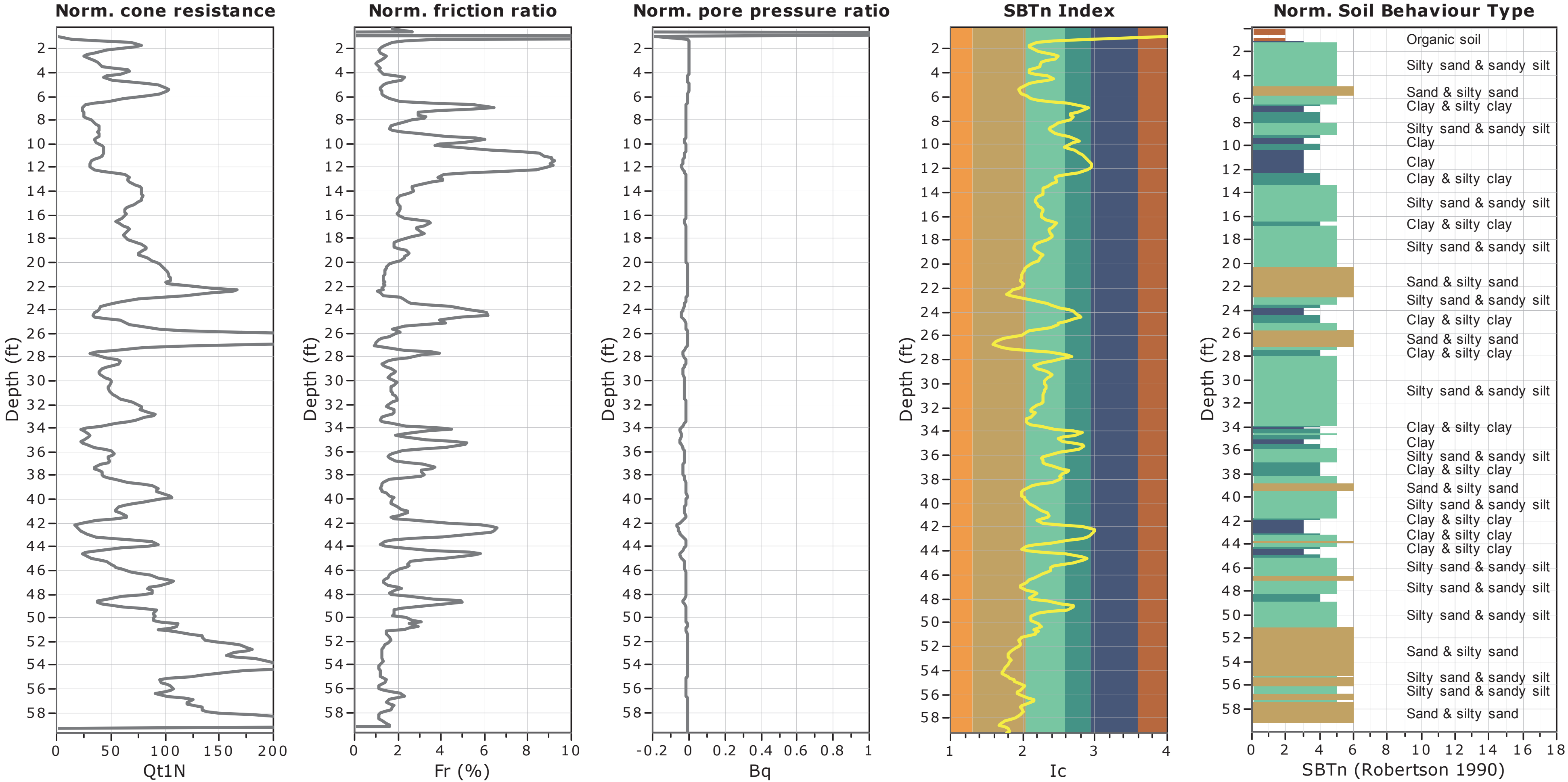
Cone Penetrometer Test Logs and Interpretations

Project:

Location:

CPT: CPT-01

Total depth: 59.71 ft, Date: 10/6/2014



SBTn 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

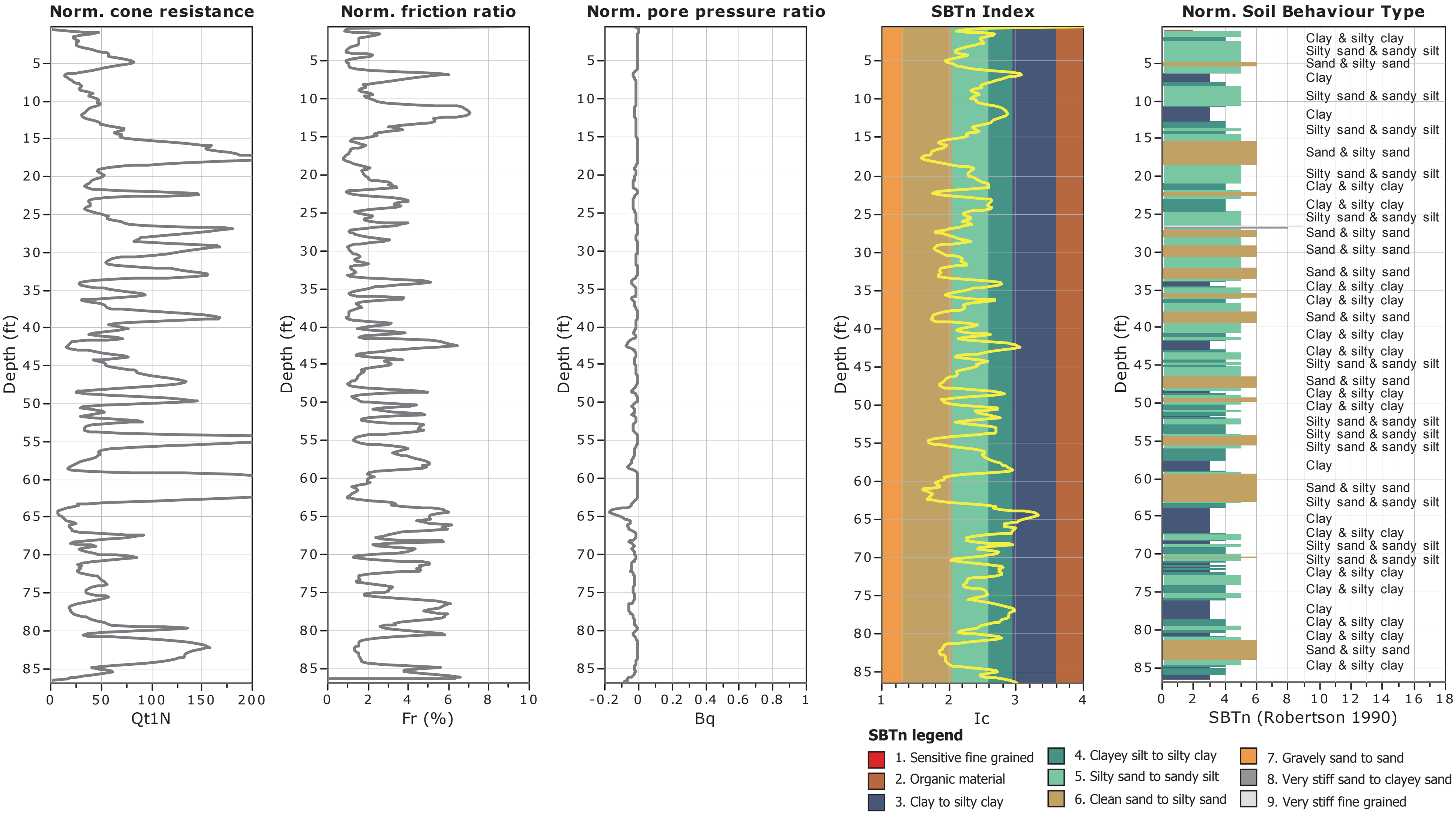
⬜ 9. Very stiff fine grained

Project:

Location:

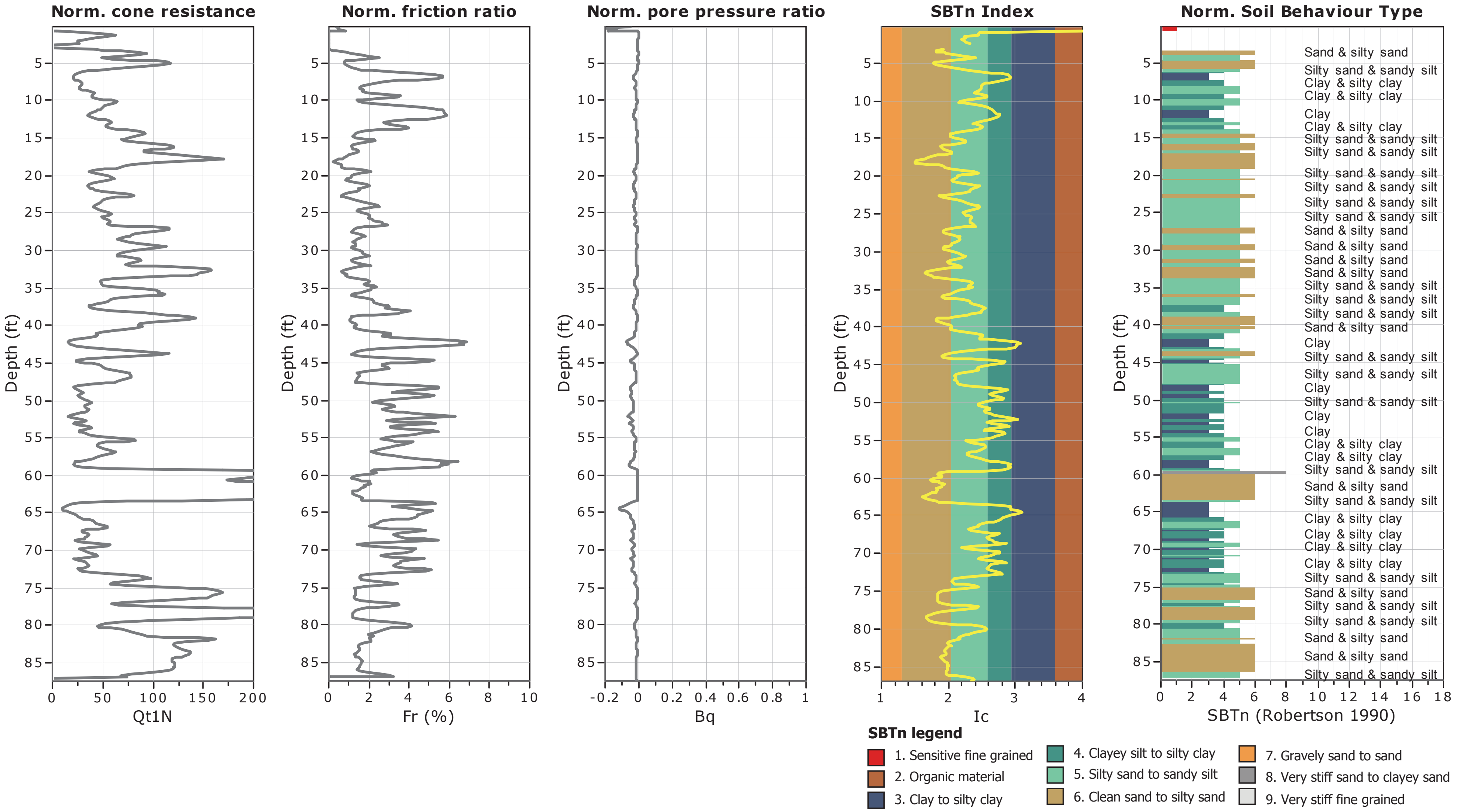
CPT: CPT-02

Total depth: 86.94 ft, Date: 10/6/2014



GEOLOGISMIKI
Geotechnical Software

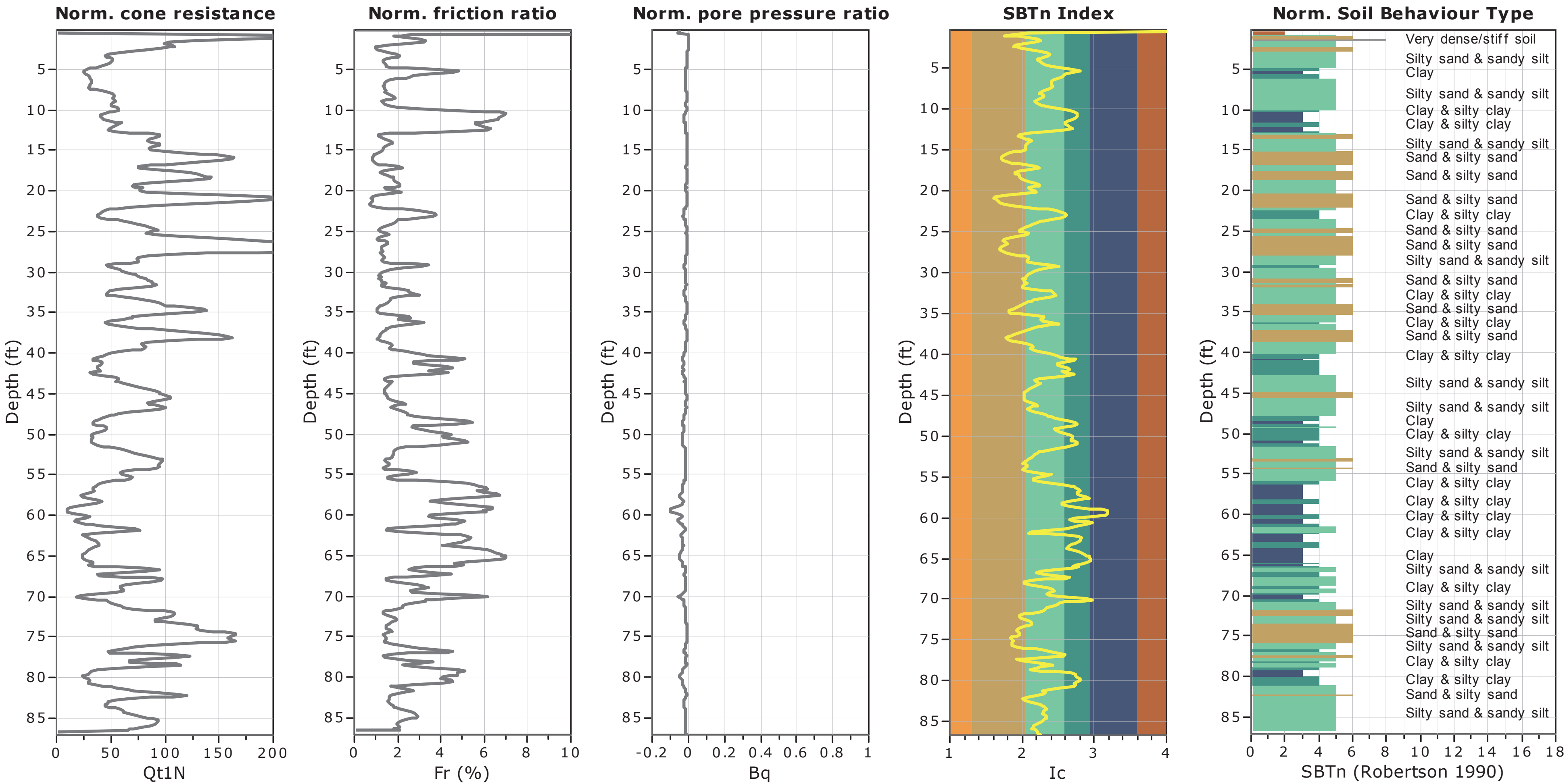
Project: **CPT: CPT-03**
Location: Total depth: 87.43 ft, Date: 10/6/2014



GEOLOGISMIKI
Geotechnical Software

Project: **CPT: CPT-04**

Location: Total depth: 87.11 ft, Date: 10/6/2014



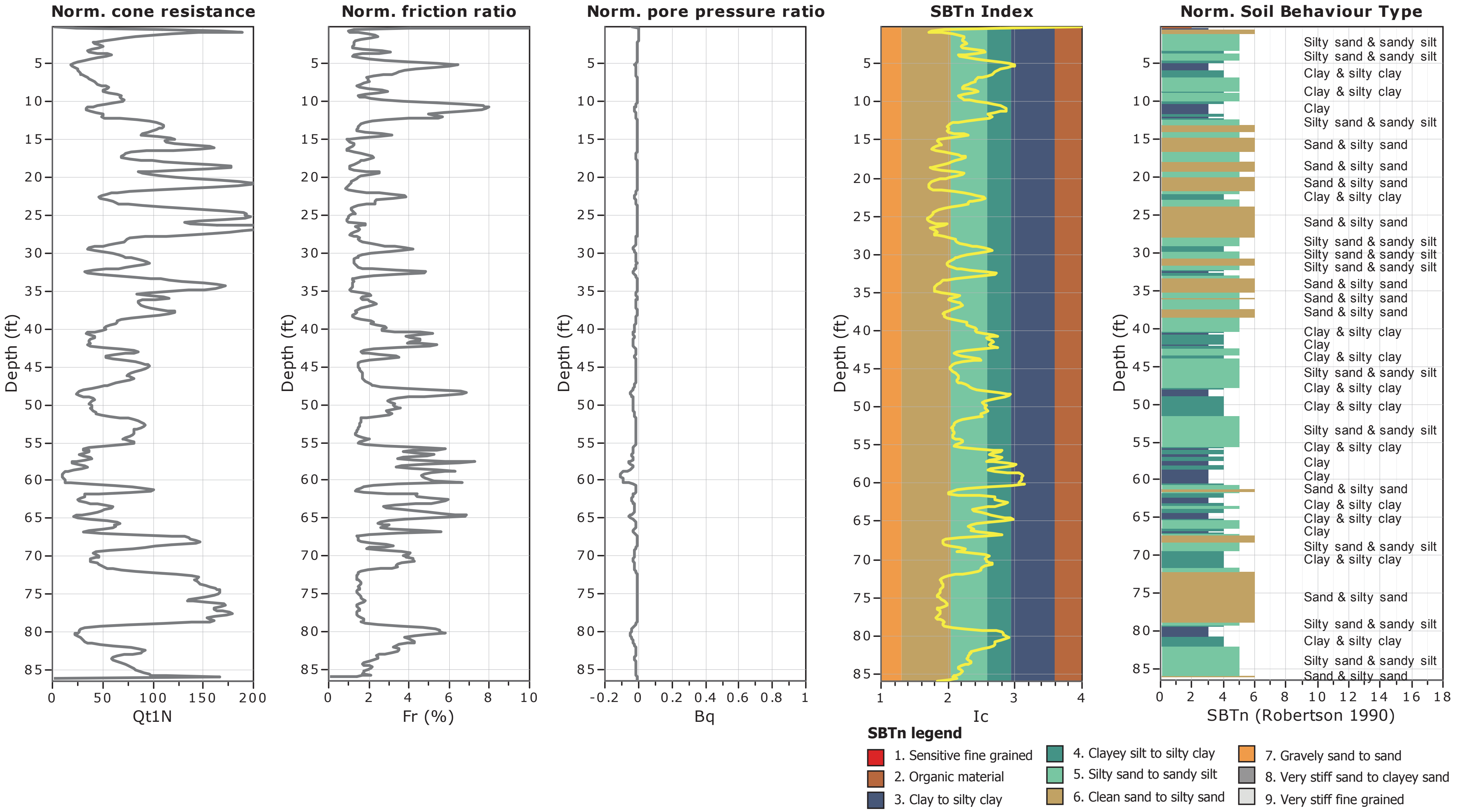
SBTn legend

- 🟥 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

GEOLOGISMIKI
Geotechnical Software

Project:
Location:

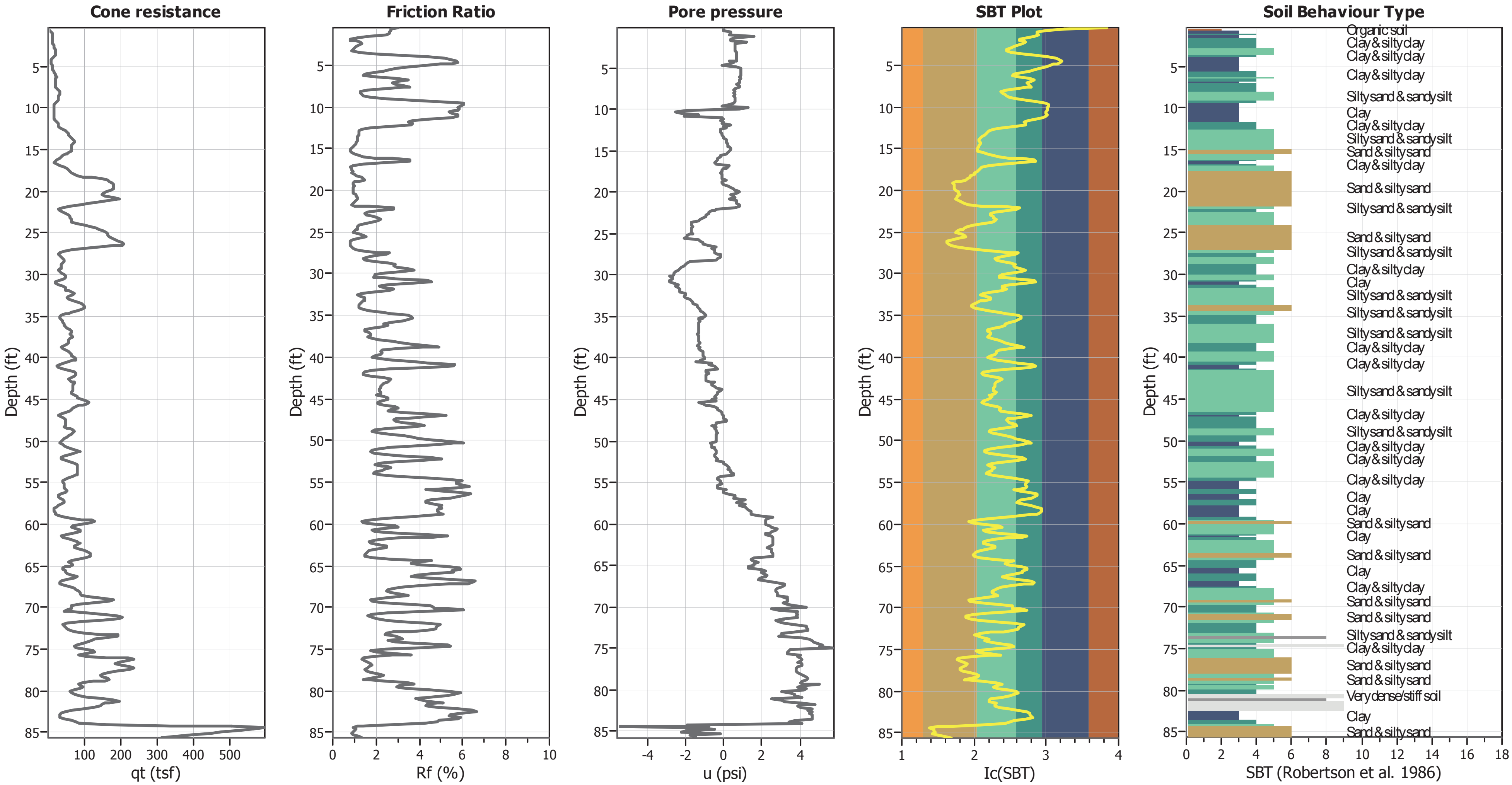
CPT: CPT-05
Total depth: 86.45 ft, Date: 10/6/2014


Norm. cone resistance
Norm. friction ratio
Norm. pore pressure ratio
SBTn Index
Norm. Soil Behaviour Type

SBTn 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 basic interpretation plots



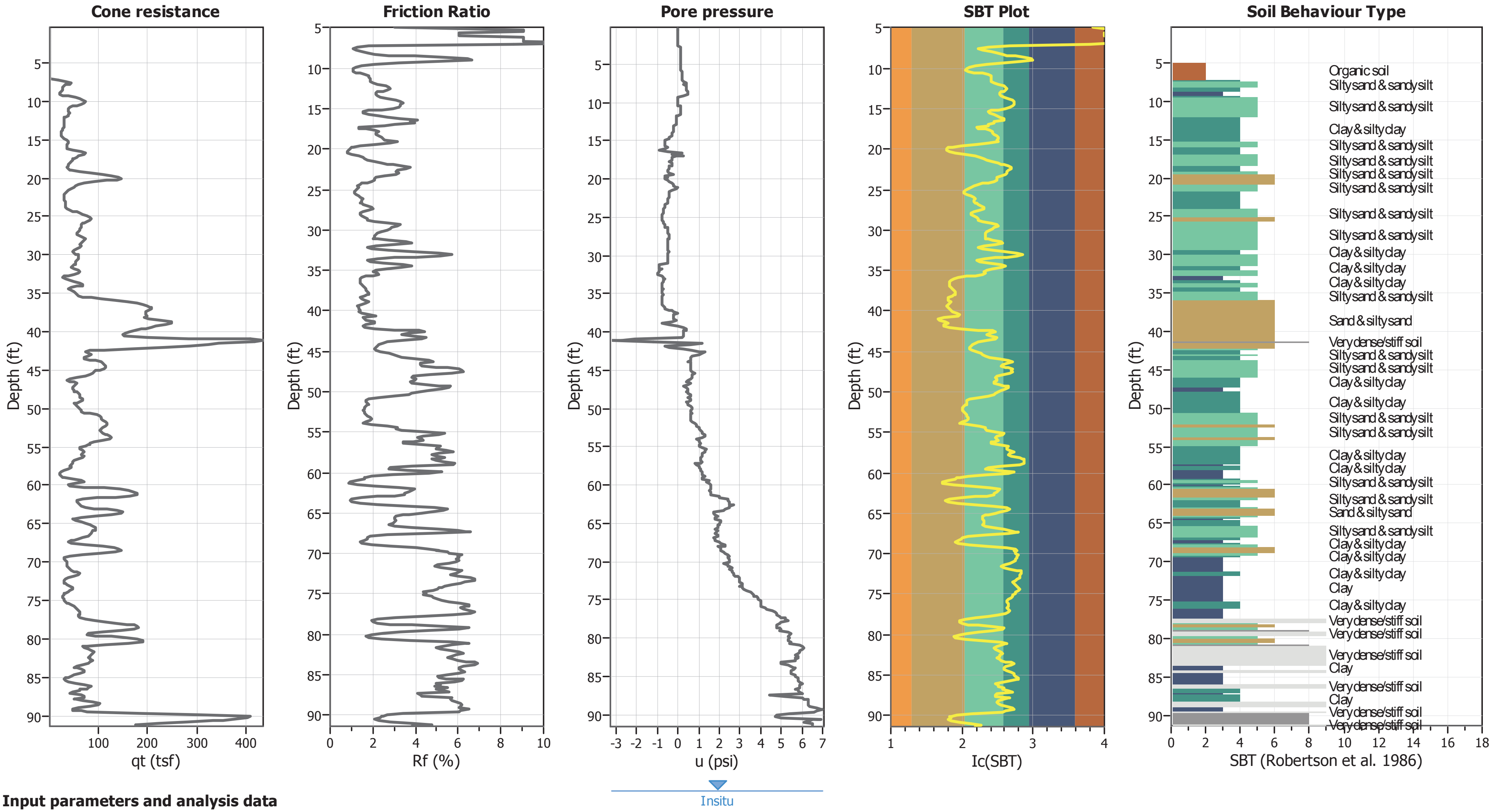
Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	100.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_o applied:	Yes
Earthquake magnitude M_w:	6.68	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.99	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	100.00 ft	Fill height:	N/A	Limit depth:	N/A

SBT legend

- 🟥 1. Sensitive fine grained
- 🟫 2. Organic material
- 🟦 3. Clay to silty clay
- 🟩 4. Clayey silt to silty
- 🟩 5. Silty sand to sandy silt
- 🟨 6. Clean sand to silty sand
- 🟧 7. Gravely sand to sand
- ⬛ 8. Very stiff sand to
- ⬜ 9. Very stiff fine grained

CPT basic interpretation plots



Input parameters and analysis data

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Fines correction method: NCEER (1998)
Points to test: Based on Ic value
Earthquake magnitude M_w: 6.68
Peak ground acceleration: 0.99
Depth to water table (insitu): 100.00 ft

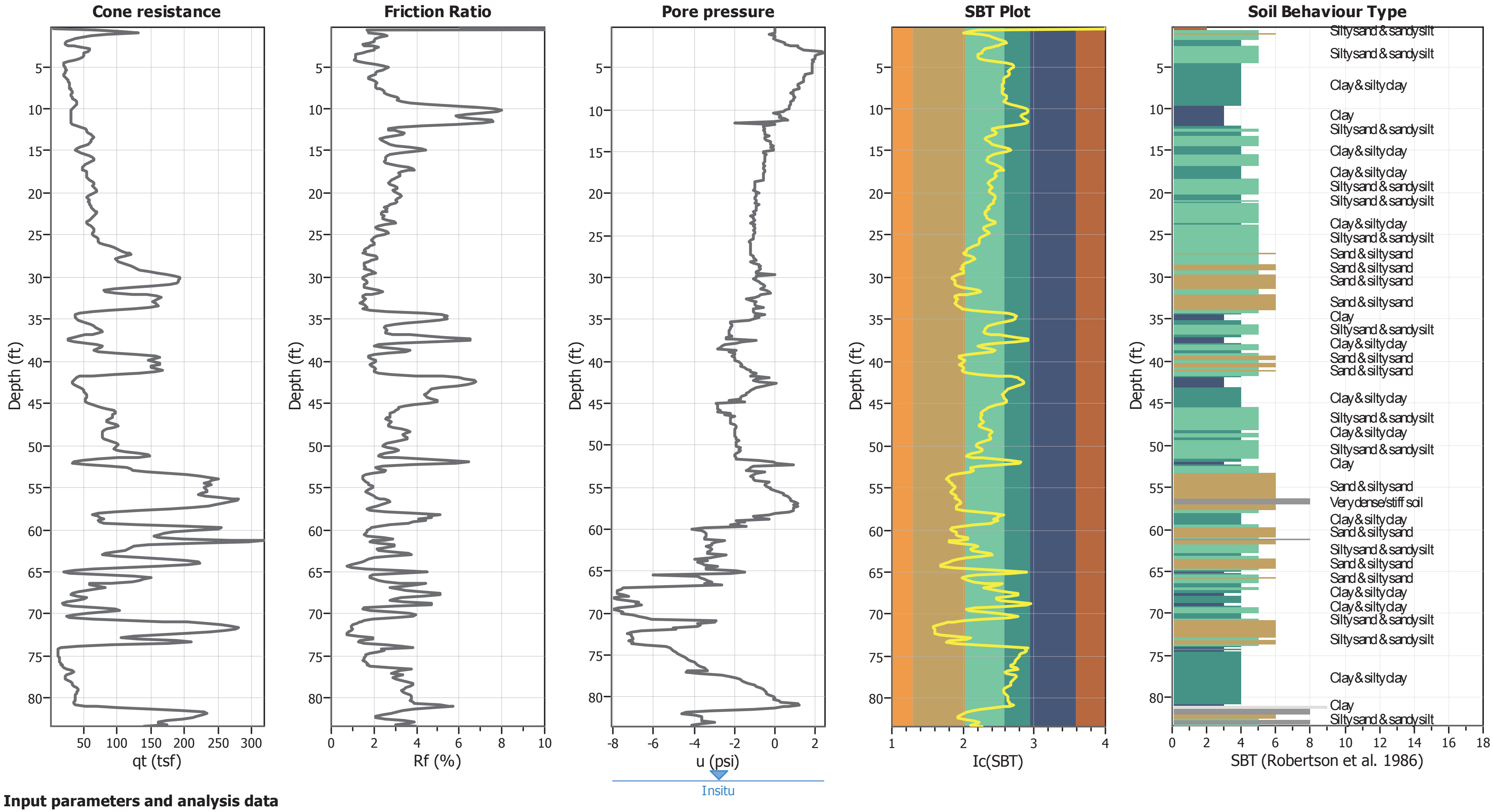
Depth to water table (erthq.): 100.00 ft
Average results interval: 3
Ic cut-off value: 2.60
Unit weight calculation: Based on SBT
Use fill: No
Fill height: N/A

Fill weight: N/A
Transition detect. applied: No
K_σ applied: Yes
Clay like behavior applied: Sands only
Limit depth applied: No
Limit depth: N/A

SBT legend

- 🟥 1. Sensitive fine grained
- 🟫 2. Organic material
- 🟦 3. Clay to silty clay
- 🟩 4. Clayey silt to silty
- 🟩 5. Silty sand to sandy silt
- 🟫 6. Clean sand to silty sand
- 🟧 7. Gravely sand to sand
- ⬛ 8. Very stiff sand to
- ⬜ 9. Very stiff fine grained

CPT basic interpretation plots



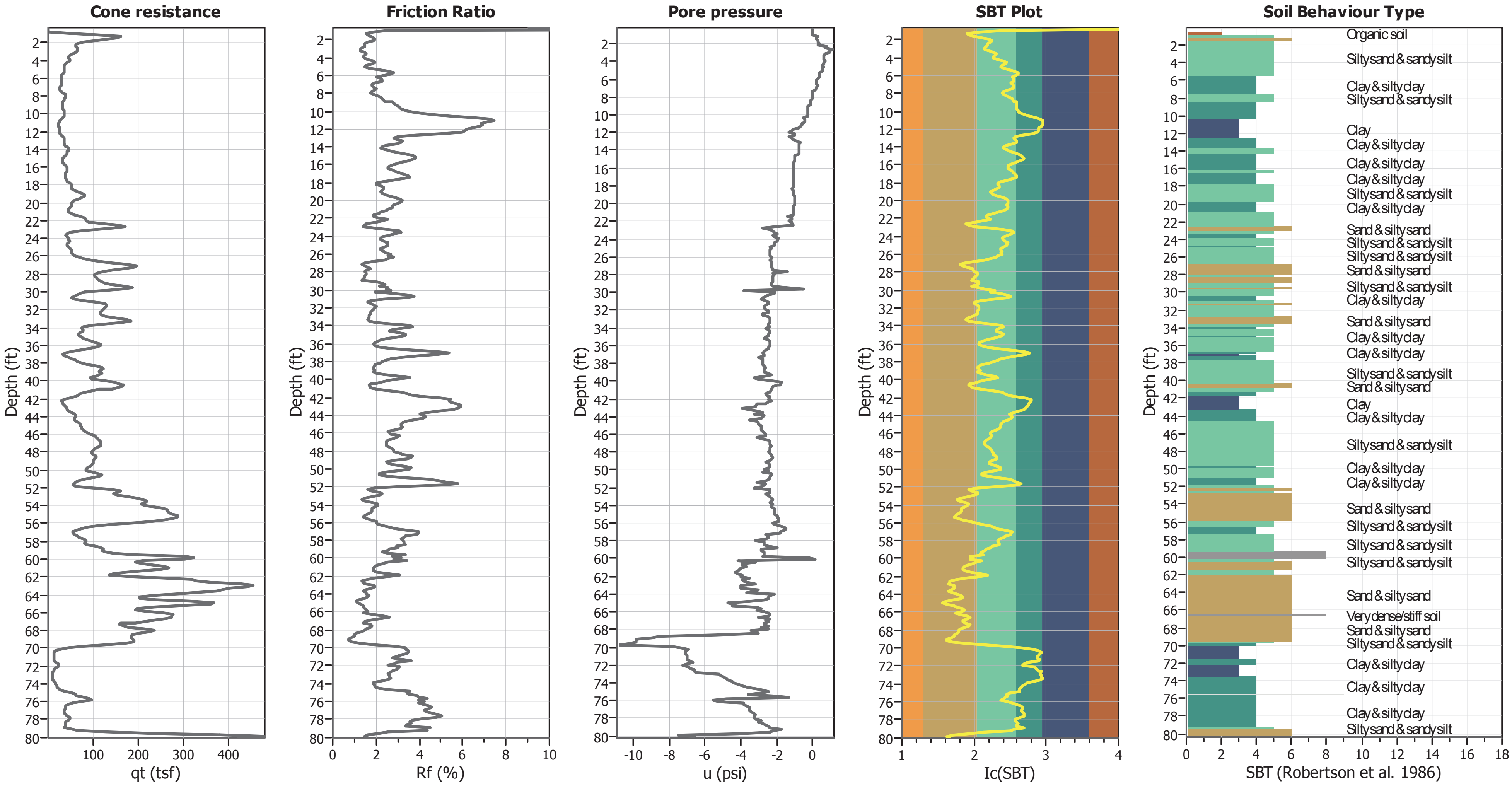
Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	90.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	6.68	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	1.00	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	90.00 ft	Fill height:	N/A	Limit depth:	N/A

SBT legend

🟥 1. Sensitive fine grained	🟩 4. Clayey silt to silty	🟧 7. Gravely sand to sand
🟫 2. Organic material	🟩 5. Silty sand to sandy silt	⬛ 8. Very stiff sand to
🟦 3. Clay to silty clay	🟨 6. Clean sand to silty sand	⬜ 9. Very stiff fine grained

CPT basic interpretation plots



Input parameters and analysis data

Insitu

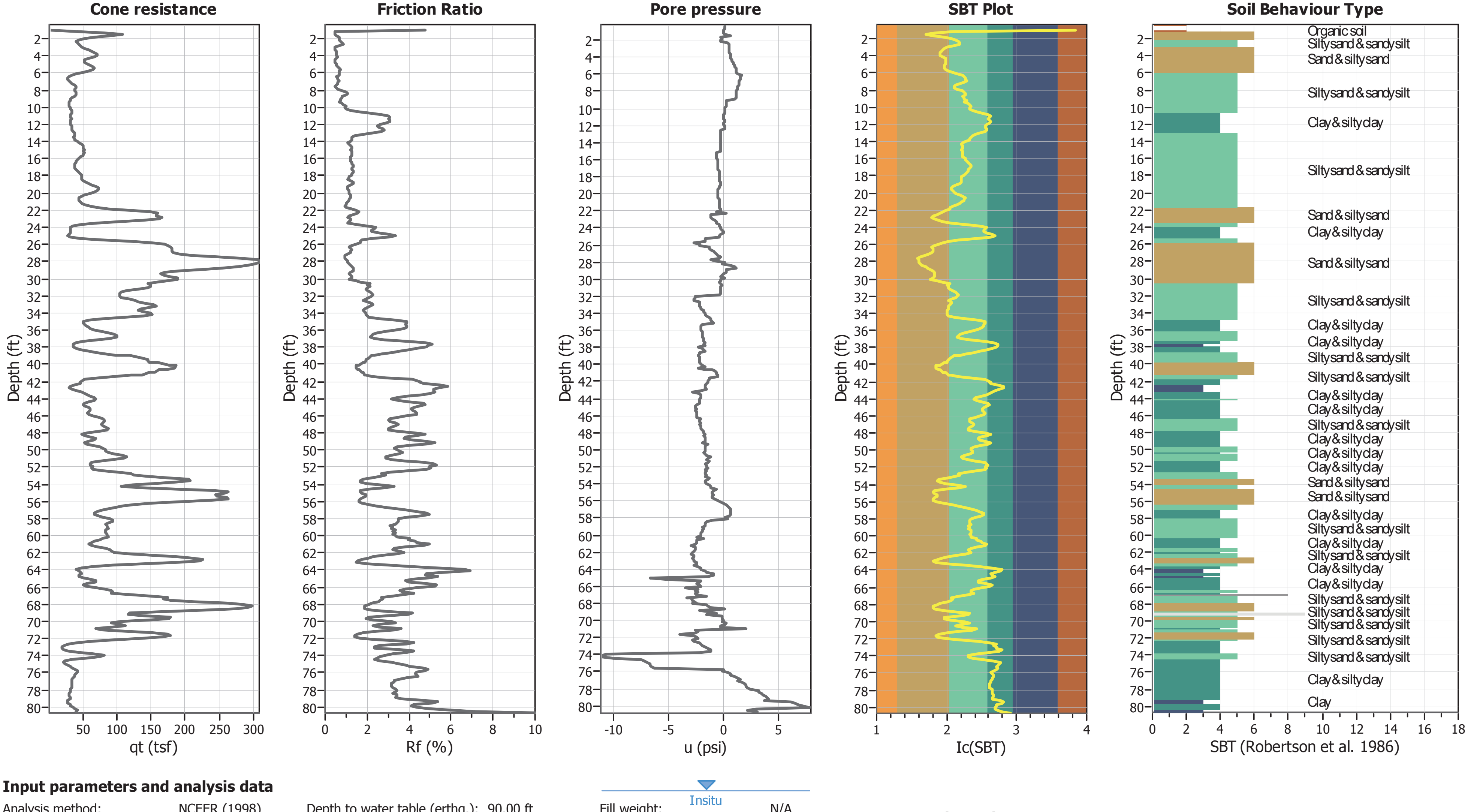
Analysis method:	NCEER (1998)	Depth to water table (erthq.):	90.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_o applied:	Yes
Earthquake magnitude M_w:	6.68	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	1.00	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	90.00 ft	Fill height:	N/A	Limit depth:	N/A

SBT legend

🟥 1. Sensitive fine grained	🟩 4. Clayey silt to silty	🟧 7. Gravely sand to sand
🟫 2. Organic material	🟩 5. Silty sand to sandy silt	⬜ 8. Very stiff sand to
🟦 3. Clay to silty clay	🟫 6. Clean sand to silty sand	⬜ 9. Very stiff fine grained

Project file: G:\Active Projects_10000 to 11999\11613\Exploration\CPT Data\2014351 DATA 11-19-14\Analysis\CPT8-12 CLiq.clq

CPT basic interpretation plots



Input parameters and analysis data

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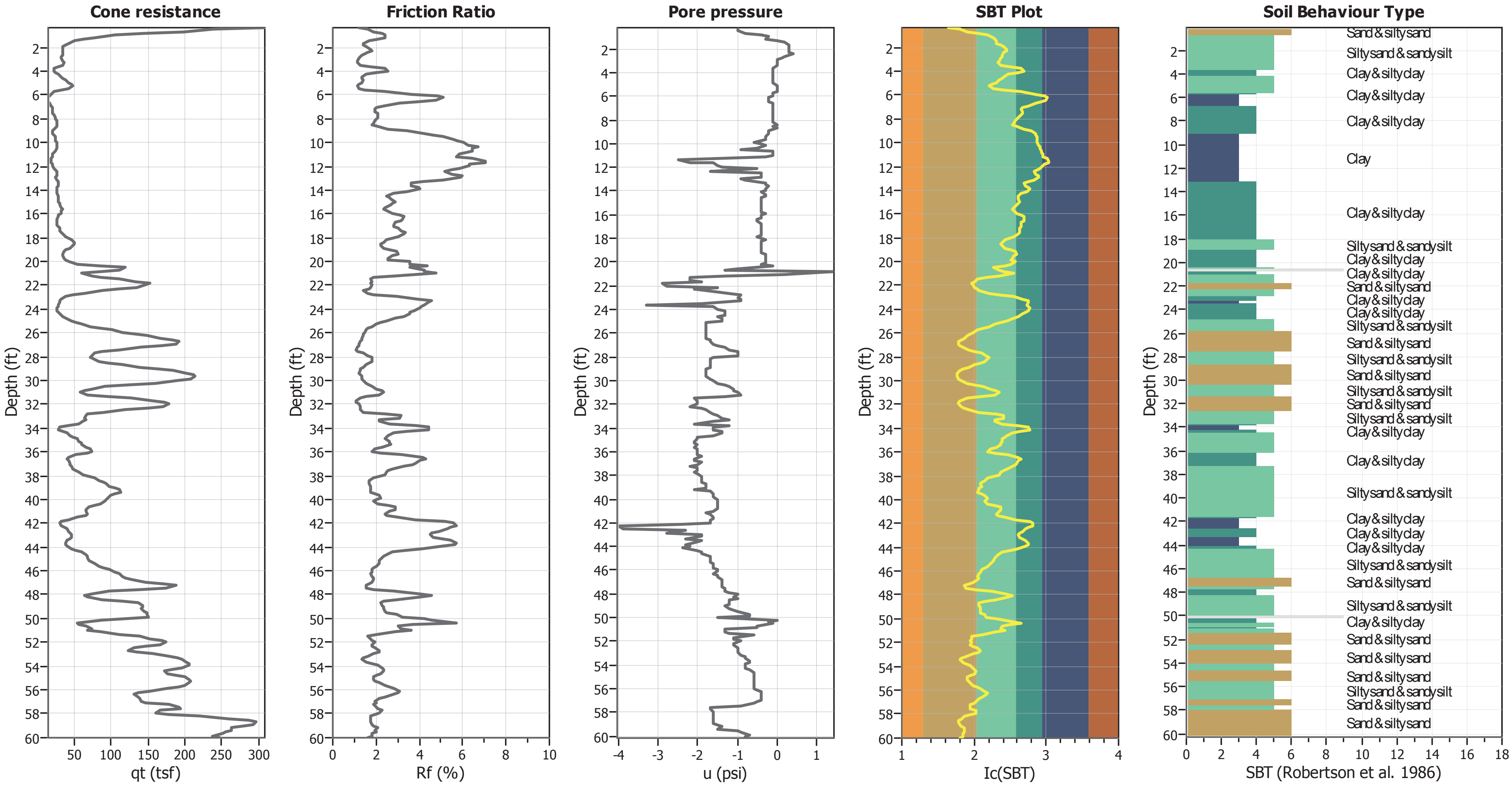
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_o applied:	Yes
Earthquake magnitude M_w:	6.68	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	1.00	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	90.00 ft	Fill height:	N/A	Limit depth:	N/A

SBT legend

🟥 1. Sensitive fine grained	🟩 4. Clayey silt to silty	🟧 7. Gravely sand to sand
🟫 2. Organic material	🟩 5. Silty sand to sandy silt	⬛ 8. Very stiff sand to
🟦 3. Clay to silty clay	🟨 6. Clean sand to silty sand	⬜ 9. Very stiff fine grained

CPT basic interpretation plots



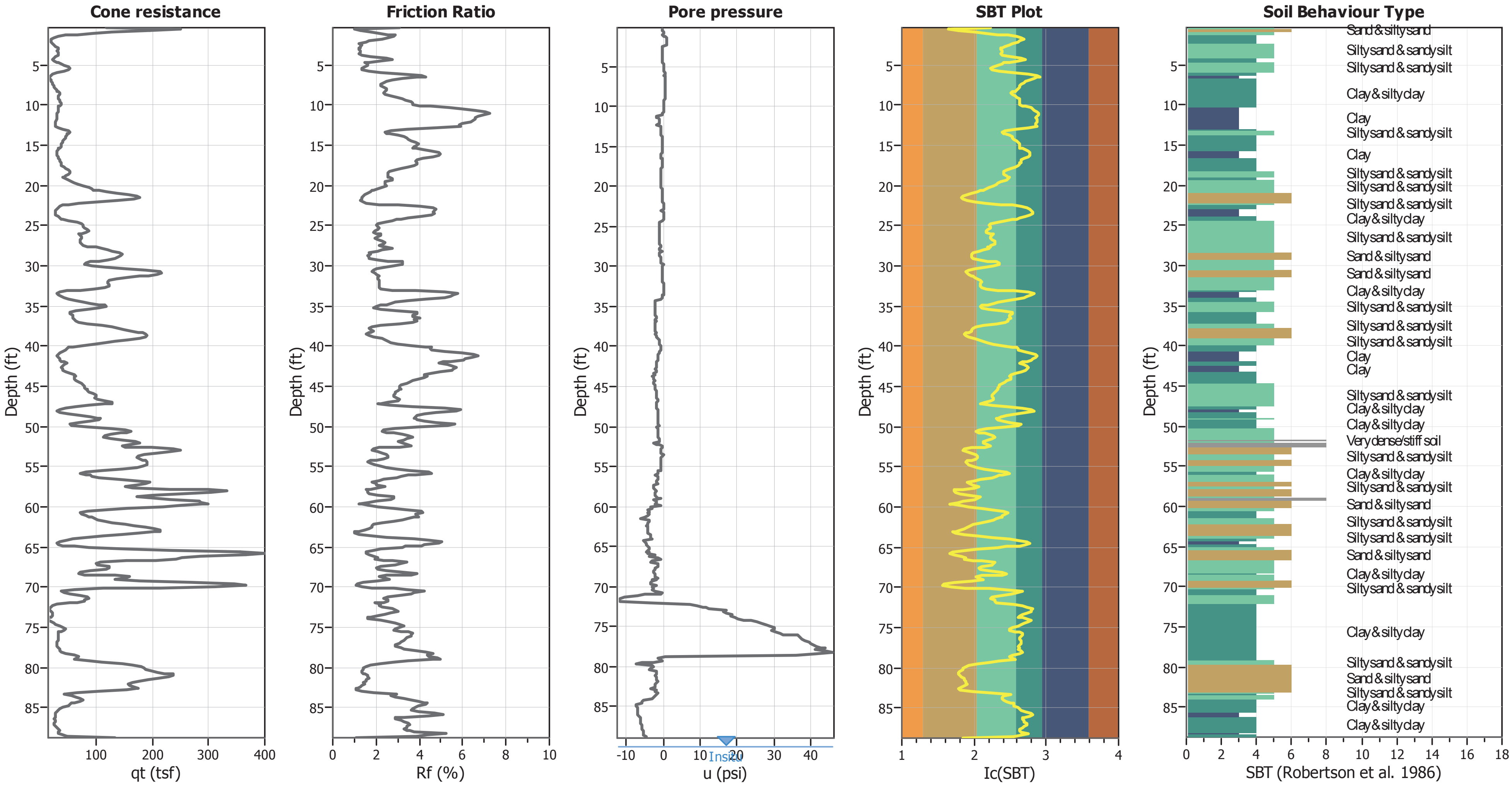
Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	90.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	6.68	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	1.00	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	90.00 ft	Fill height:	N/A	Limit depth:	N/A

SBT legend

- 🟥 1. Sensitive fine grained
- 🟫 2. Organic material
- 🟦 3. Clay to silty clay
- 🟩 4. Clayey silt to silty
- 🟩 5. Silty sand to sandy silt
- 🟫 6. Clean sand to silty sand
- 🟧 7. Gravely sand to sand
- ⬛ 8. Very stiff sand to
- ⬜ 9. Very stiff fine grained

CPT basic interpretation plots



Input parameters and analysis data

Analysis method: NCEER (1998)
Fines correction method: NCEER (1998)
Points to test: Based on Ic value
Earthquake magnitude M_w: 6.68
Peak ground acceleration: 1.00
Depth to water table (insitu): 90.00 ft

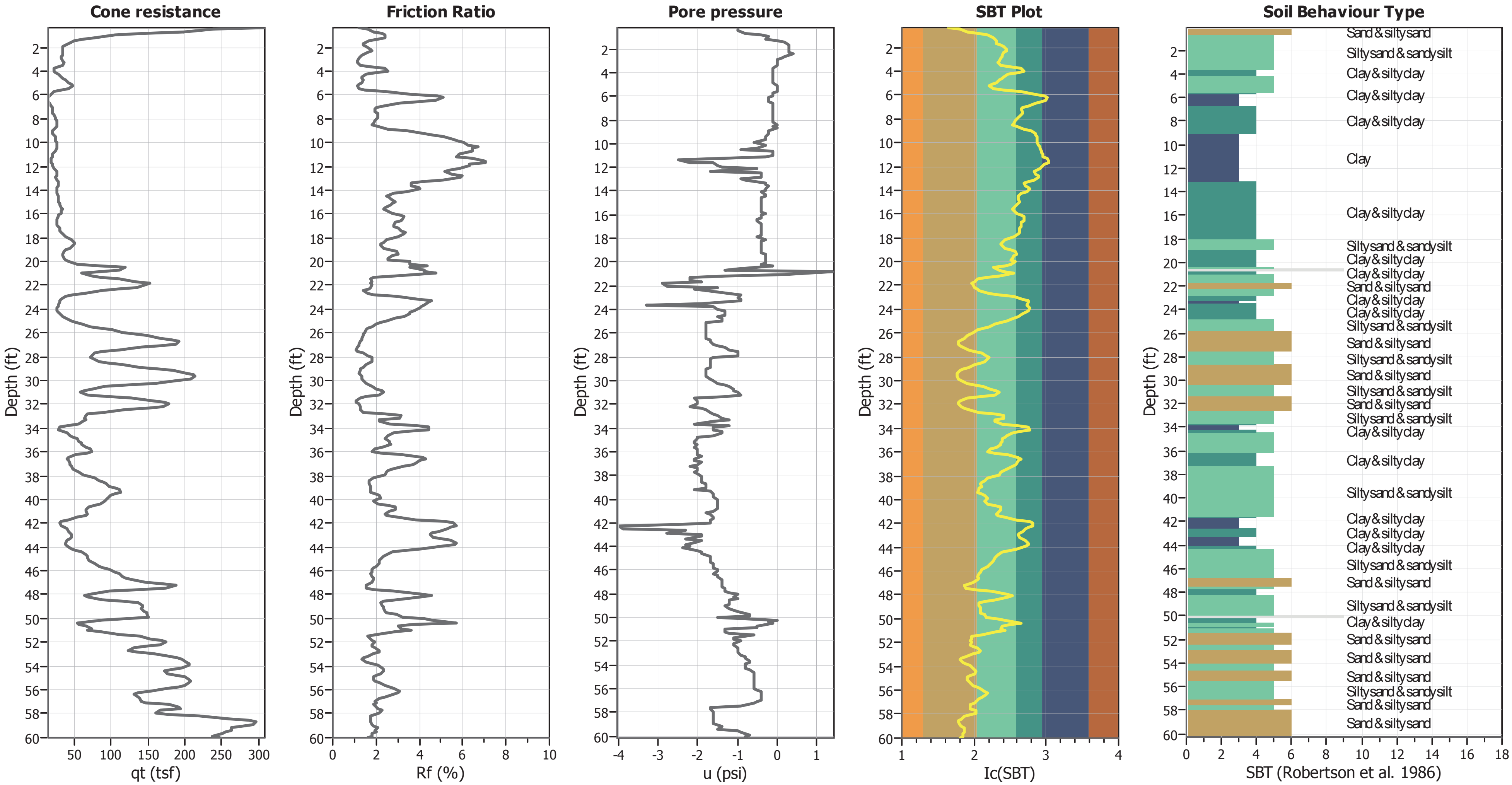
Depth to water table (erthq.): 90.00 ft
Average results interval: 3
Ic cut-off value: 2.60
Unit weight calculation: Based on SBT
Use fill: No
Fill height: N/A

Fill weight: N/A
Transition detect. applied: No
K_σ applied: Yes
Clay like behavior applied: Sands only
Limit depth applied: No
Limit depth: N/A

SBT legend

🟥 1. Sensitive fine grained	🟩 4. Clayey silt to silty	🟧 7. Gravely sand to sand
🟫 2. Organic material	🟩 5. Silty sand to sandy silt	⬛ 8. Very stiff sand to
🟦 3. Clay to silty clay	🟨 6. Clean sand to silty sand	⬜ 9. Very stiff fine grained

CPT basic interpretation plots



Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	90.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	6.68	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	1.00	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	90.00 ft	Fill height:	N/A	Limit depth:	N/A

SBT legend

🟥 1. Sensitive fine grained
🟫 2. Organic material
🟦 3. Clay to silty clay
🟩 4. Clayey silt to silty
🟩 5. Silty sand to sandy silt
🟫 6. Clean sand to silty sand
🟧 7. Gravely sand to sand
⬛ 8. Very stiff sand to
⬜ 9. Very stiff fine grained

APPENDIX B

Core Logs

APPENDIX B

ENGEO
Expect Excellence

LOG OF BORING C-1

Field Exploration
SPBB Hollywood
Los Angeles, California
11613.000.000

DATE DRILLED: 10/7/2014
HOLE DEPTH: 90 ft.
HOLE DIAMETER: 8.0 in.
SURF ELEV (FT-AMSL): 396.43 ft.

LOGGED / REVIEWED BY: P. Lam / PJS
DRILLING CONTRACTOR: Martini Drilling
DRILLING METHOD: Hollow Stem Auger
HAMMER TYPE: Dry Core

Depth in Feet	Depth in Meters	Formation	DESCRIPTION	Log Symbol	Water Level	Run Number	Recovery	Moisture Content (% dry weight)	Gravel Content (%)	Sand Content (%)	Fines Content (%)	Notes
		Qaf (Fill)	ASPHALT									
			CONCRETE									
	1		SILTY SAND FILL (SM), hand auger									
5	2	Qyf (Young fan)	SANDY SILT (ML), very dark brown / dusky yellowish brown (10YR 2/2), moist			1	3.5/5					
			SILTY SAND (SM), brown (10YR 4/3)									
			SANDY CLAY (CL), dark yellowish brown (10YR 3/6) Fine grained sand									
10	3		NO RECOVERY	NR								
			SANDY CLAY (CL), decreased sand									
			SILTY CLAY (CL-ML), dark yellowish brown (10YR 3/6), moist			2	2.5/2.5					
	4		SILTY SAND (SM), yellowish brown (10YR 5/8), moist, fine grained sand			3	2.5/2.5					
15	5		Increasing silt, trace carbonate			4	2/2.5					
			Brown (10YR 5/3) SILTY SAND brown (10YR 5/3), fine grained sand			5	2.5/2.5					
20	6					6	2/2.5					
	7		Increasing gravel up to 1/2"-diameter									
			SANDY CLAY (CL), dark yellowish brown (10YR 4/6)			7	2.5/2.5					
25			SANDY SILT (ML), strong brown (7.5YR 5/6), trace gravel at base									
	8		POORLY GRADED SAND (SP), trace gravel, fine to medium grained sand Trace cobbles Trace cobbles, fine grained sand, trace gravel			8	2/2.5	1	6	90	4	
	9		SANDY SILT (ML), strong brown (7.5YR 5/6), fine grained sand, trace cobbles at base			9	1.5/2.5					
30			SILTY SAND (SM), yellowish brown (10YR 5/8), trace cobbles, fine to coarse grained sand			10	2/2.5					
	10		Trace cobbles									
			Trace gravel			11	2.5/2.5					
35			SILTY CLAY (CL-ML), dark yellowish brown (10YR 4/6), fine grained sand									

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LOG OF BORING C-1

Field Exploration
SPBB Hollywood
Los Angeles, California
11613.000.000

DATE DRILLED: 10/7/2014
HOLE DEPTH: 90 ft.
HOLE DIAMETER: 8.0 in.
SURF ELEV (FT-AMSL): 396.43 ft.

LOGGED / REVIEWED BY: P. Lam / PJS
DRILLING CONTRACTOR: Martini Drilling
DRILLING METHOD: Hollow Stem Auger
HAMMER TYPE: Dry Core

Depth in Feet	Depth in Meters	Formation	DESCRIPTION	Log Symbol	Water Level	Run Number	Recovery	Moisture Content (% dry weight)	Gravel Content (%)	Sand Content (%)	Fines Content (%)	Notes
	11	Qyf (Young fan)	SILTY SAND (SM), reddish brown / moderate brown (5YR 4/4), trace coarse-grained sand, ~40-50% silt									
			SANDY CLAY (CL), yellowish red (5YR 4/6), fine grained sand, increasing silt, trace FeO staining			12	5/5					
40	12		SILTY SAND (SM), fine to medium grained sand, trace gravel 1-4 cm diameter at 39'									
			SANDY CLAY (CL), strong brown (7.5YR 4/6)									
			SILTY SAND (SM), fine grained sand					15.1	3	46	51	
	13	Qof (Old fan)	CLAYEY SILT (ML), strong brown (7.5YR 5/6), trace sand			13	5/5					
			SILTY SAND (SM)									
			SANDY SILT (ML), increasing sand with depth									
45	14		SILTY SAND (SM), strong brown (7.5YR 4/6), fine to coarse grained sand			14	5/5					
			SILT (ML)									
	15		SILTY SAND (SM), trace gravel									
50			SILT (ML)									
			SILTY SAND (SM), yellowish brown (10YR 5/8), trace cobble at 51', increasing medium to coarse sand with depth									
	16		SILTY CLAY (CL-ML), yellowish red / light brown (5YR 5/6)			15	5/5					
			SILTY SAND (SM), trace coarse-grained sand, trace gravel and cobbles									
			SILT (ML), reddish brown / moderate brown (5YR 4/4)									
55	17		SILTY SAND (SM), yellowish red / light brown (5YR 5/6), fine to medium grained sand									
			SILT (ML)									
			SILTY SAND (SM), yellowish red (5YR 4/6), fine grained sand, ~15-20% silt Trace cobble, increased silt with depth			16	4.5					
60	18		CEMENTED ROCK FRAGMENTS (GM), yellowish brown (10YR 5/8), sandstone/cemented sand									
	19		SILTY SAND (SM), yellowish brown (10YR 5/8), with gravel, fine to coarse grained sand			17	4/5					
			SANDY SILTY GRAVEL (GM), brownish yellow / dark yellowish orange (10YR 6/6), cemented sand, 3" cobbles at top, 3" cobble at base									
65			NO RECOVERY	NR								
	20		SAND WITH SILT (SW-SM), brownish yellow / dark yellowish orange (10YR 6/6), with gravel, fine to coarse grained sand					2.2	18	75	7	
			SILTY GRAVEL (GM), 3" cobble, 2" cobbles			18	4.5/5					
			SILTY SAND (SM), trace gravel, coarse grained sand									
	21		SILTY CLAY (CL-ML), yellowish red / light brown (5YR 5/6), trace fine-grained sand	NR								
70												

ENGEO
Expect Excellence

LOG OF BORING C-1

Field Exploration
SPBB Hollywood
Los Angeles, California
11613.000.000

DATE DRILLED: 10/7/2014
HOLE DEPTH: 90 ft.
HOLE DIAMETER: 8.0 in.
SURF ELEV (FT-AMSL): 396.43 ft.

LOGGED / REVIEWED BY: P. Lam / PJS
DRILLING CONTRACTOR: Martini Drilling
DRILLING METHOD: Hollow Stem Auger
HAMMER TYPE: Dry Core

Depth in Feet	Depth in Meters	Formation	DESCRIPTION	Log Symbol	Water Level	Run Number	Recovery	Moisture Content (% dry weight)	Gravel Content (%)	Sand Content (%)	Fines Content (%)	Notes
		Qof (Old fan)	SILTY SAND (SM), fine to medium grained sand, trace gravel up to 1"-diameter									
	22		NO RECOVERY									
			SILT (ML)			19	5/5					
			SILTY SAND (SM), yellowish red / light brown (5YR 5/6), fine to coarse grained sand									14C Age: 17,920-17,680 Cal BP (Charcoal)
			SILT (ML), yellowish brown (10YR 5/8)									
75	23		SILTY SAND (SM), fine grained sand									
			CLAYEY SILT (ML), reddish brown / moderate brown (5YR 4/4), fine grained sand, trace charcoal									
			SILTY SAND (SM), yellowish red / light brown (5YR 5/6), fine to medium grained sand			20	5/5					
	24		(SM), with gravel, fine sand and gradational at base									
			SANDY SILT (ML), fine grained sand									
80			SILTY CLAY (CL-ML), very dark brown mottled with reddish brown / moderate brown (5YR 4/4), fine to medium grained sand, fine platy ped structure									
	25		SILTY SAND (SM), yellowish red (5YR 4/6), trace coarse-grained sand, fine to medium grained sand Yellowish red / light brown (5YR 5/6), trace gravel up to 1"-diameter, slight color change			21	5/5					
85			SILT (ML), yellowish red (5YR 4/6)									
	26		SILTY SAND (SM), yellowish red (5YR 4/6), trace gravel, increasing silt with depth									
			SANDY SILT (ML), dark reddish brown / moderate brown (5YR 3/4), ~45% fine sand			22	5/5					
	27		CEMENTED SAND/SANDSTONE (GM), driller hit rocks at 88'									
90			No groundwater encountered.									

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LOG OF BORING C-2

Field Exploration
SPBB Hollywood
Los Angeles, California
11613.000.000

DATE DRILLED:	10/8/2014
HOLE DEPTH:	90 ft.
HOLE DIAMETER:	8.0 in.
SURF ELEV (FT-AMSL):	393.42 ft.
LOGGED / REVIEWED BY:	P. Lam / PJS
DRILLING CONTRACTOR:	Martini Drilling
DRILLING METHOD:	Hollow Stem Auger
HAMMER TYPE:	Dry Core

Depth in Feet	Depth in Meters	Formation	DESCRIPTION	Log Symbol	Water Level	Run Number	Recovery	Moisture Content (% dry weight)	Gravel Content (%)	Sand Content (%)	Fines Content (%)	Notes
		Qaf (Fill)	ASPHALT									
			SILTY SAND (SM), reddish brown / moderate brown (5YR 4/4)									
	1											
5												
		Qyf (Young fan)	Fine grained sand, trace rootlets			1	2.5/2.5					
	2		Brown (7.5YR 4/4), trace gravel at 9'									
						2	2.5/2.5					
10	3		SILT (ML), dark yellowish brown (10YR 3/6), with fine-grained sand, trace pores, increased pore size at 13'									
						3	2.5/2.5					
	4								1	60	39	
						4	2.5/2.5					
15			SILTY SAND (SM), yellowish brown (10YR 5/8), trace coarse-grained sand, fine grained sand, ~20-30% silt Trace gravel									
	5											
			SILT (ML), with fine-grained sand, ~30-40% sand			5	4.75/5					
			SILTY SAND (SM), ~20-30% silt, trace gravel up to 1/2"-diameter									
20	6		SILT (ML), dark yellowish brown (10YR 3/6), trace fine-grained sand									
			SILTY SAND (SM), yellowish brown (10YR 5/8), trace coarse-grained sand, fine grained sand, increased coarse sand with depth									
						6	4.5/5					
	7		SANDY SILT (ML), fine to medium grained sand, trace gravel at base									
			CLAYEY SILT (ML), rocky drilling at base									
25			NO RECOVERY									
	8		POORLY GRADED SAND (SP), strong brown (7.5YR 5/6), fine to coarse grained sand			7	2.5/2.5					
			SANDY CLAY (CL), strong brown (7.5YR 5/6), fine grained sand			8	2.25/2.5	14.9	1	47	52	(OSL)
	9											
30			SILTY SAND (SM), yellowish brown (10YR 5/8), fine to coarse grained sand, trace cobbles			9	2.5/2.5					
			Trace coarse sand									
	10											
			CLAYEY SILT (ML), dark yellowish brown (10YR 4/6), micaceous			10	2.25/2.5	7	3	77	20	(OSL)
35												

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LOG OF BORING C-2

Field Exploration
SPBB Hollywood
Los Angeles, California
11613.000.000

DATE DRILLED: 10/8/2014
HOLE DEPTH: 90 ft.
HOLE DIAMETER: 8.0 in.
SURF ELEV (FT-AMSL): 393.42 ft.

LOGGED / REVIEWED BY: P. Lam / PJS
DRILLING CONTRACTOR: Martini Drilling
DRILLING METHOD: Hollow Stem Auger
HAMMER TYPE: Dry Core

Depth in Feet	Depth in Meters	Formation	DESCRIPTION	Log Symbol	Water Level	Run Number	Recovery	Moisture Content (% dry weight)	Gravel Content (%)	Sand Content (%)	Fines Content (%)	Notes
	11	Qyf (Young fan)	SILTY SAND (SM), reddish yellow (5YR 6/6), trace coarse-grained sand, fine to medium grained sand, trace gravel up to 1.5"-diameter									
			SILTY CLAY (CL-ML), yellowish red (5YR 4/6), fine grained sand, micaceous			11	5/5					
40	12		SILTY SAND (SM), trace gravel up to 1"-diameter					7	2	72	26	(OSL)
	13	Qof (Old fan)	CLAYEY SILT (ML), yellowish red / light brown (5YR 5/6), trace coarse-grained sand			12	5/5					
45	14 15		SILTY SAND (SM), yellowish red / light brown (5YR 5/6), trace coarse-grained sand, fine grained sand, increasing fines with depth			13	5/5	10.3	7 1	58 55	35 44	(OSL)
50	16		CLAYEY SILT (ML), strong brown (7.5YR 5/8), trace carbonate Yellowish red (5YR 4/6), clay films on gravel			14	5/5					
55	17		SILT WITH SAND (ML)									
			CLAYEY SILT (ML), yellowish red (5YR 4/6), trace gravel up to 1/2"-1"-diameter at 56'									
			SILTY CLAY (CL-ML), trace fine gravel at 57'			15	5/5					
			SANDY SILT (ML)									
60	18		SILTY CLAY (CL-ML), brown (7.5YR 4/3), trace fine- to medium-grained sand					19.9	0	34	66	(OSL)
			SILTY SAND (SM)									
	19		CLAYEY SAND (SC), strong brown (7.5YR 4/6), trace coarse-grained sand, trace mottling yellowish red (5YR 5/6), increased silt with depth			16	5/5		1	54	45	
			SANDY CLAY (CL), reddish brown / moderate brown (5YR 4/4), fine to coarse grained sand, ~30-40% fines									
65	20		SILTY SAND (SM), yellowish red (5YR 4/6)									
			COBBLES									
			CLAYEY SILT (ML), strong brown (7.5YR 5/6), trace coarse-grained sand			17	4/5					
70	21		SILTY SAND (SM), reddish yellow (5YR 6/6), fine grained sand, ~20-30% fines, increasing coarse sand and gravel with depth					7.7	6	72	22	(OSL)

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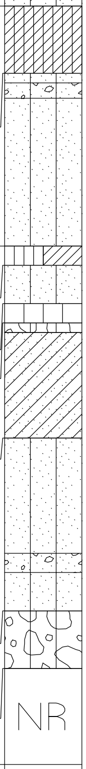
LOG OF BORING C-2

Field Exploration
SPBB Hollywood
Los Angeles, California
11613.000.000

DATE DRILLED: 10/8/2014
HOLE DEPTH: 90 ft.
HOLE DIAMETER: 8.0 in.
SURF ELEV (FT-AMSL): 393.42 ft.

LOGGED / REVIEWED BY: P. Lam / PJS
DRILLING CONTRACTOR: Martini Drilling
DRILLING METHOD: Hollow Stem Auger
HAMMER TYPE: Dry Core



Depth in Feet	Depth in Meters	Formation	DESCRIPTION	Log Symbol	Water Level	Run Number	Recovery	Moisture Content (% dry weight)	Gravel Content (%)	Sand Content (%)	Fines Content (%)	Notes
	22	Qof (Old fan)	SILTY CLAY (CL-ML), yellowish red / light brown (5YR 5/6), trace coarse-grained sand Reddish brown / moderate brown (5YR 4/4), trace coarse-grained sand, sharp color change, increasing sand with depth			18	4.5/5					
			SILTY SAND (SM), yellowish red (5YR 4/6), with gravel, cobble in sampler at 72 1/2', increasing coarse sand to 75'									
75	23		Fine to medium grained sand, ~20-30% fines									
			CLAYEY SILT (ML), yellowish red / light brown (5YR 5/6), with sand			29	4.5/5					
			SILTY SAND (SM), reddish yellow (5YR 6/8), fine to coarse grained sand									
	24		SANDY SILT (ML)									
80			GRAVELLY SILTY SAND gravel up to 1 1/2" diameter									
			CLAYEY SAND (SC), brown (7.5YR 4/4), fine to coarse grained sand						2	63	35	
	25		SILTY SAND (SM), reddish brown (5YR 4/3), trace gravel at 83', increasing silt with depth			20	5/5					
85	26		3" cobbles in sampler									
			SILTY SAND / SANDY SILT reddish brown / moderate brown (5YR 4/4), significant rig chatter									
	27		SILTY GRAVEL AND COBBLES (GM), strong brown (7.5YR 4/6), (strong brown matrix), coarse sand in matrix, very hard drilling and no recovery beyond 87.5'	NR		21	2.5/5					
90			No groundwater encountered.									

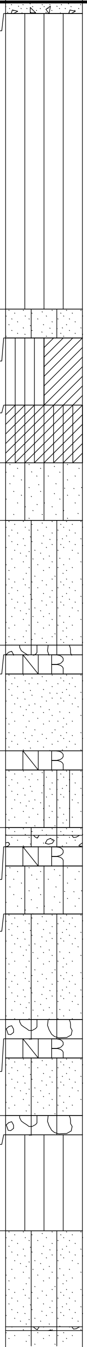
LOG OF BORING C-3

Field Exploration
SPBB Hollywood
Los Angeles, California
11613.000.000

DATE DRILLED: 10/9/2014
HOLE DEPTH: 90 ft.
HOLE DIAMETER: 8.0 in.
SURF ELEV (FT-AMSL): 390.64 ft.

LOGGED / REVIEWED BY: P. Lam / PJS
DRILLING CONTRACTOR: Martini Drilling
DRILLING METHOD: Hollow Stem Auger
HAMMER TYPE: Dry Core

Formation	DESCRIPTION
FILL	CONCRETE
	SANDY SILT (ML), dark reddish brown (5YR 3/3), trace coarse-grained sand, fine to medium grained sand, ~30-40% sand, hand auger to 5'
Qyf (Young fan)	SILTY SAND (SM), dark reddish brown / moderate brown (5YR 3/4)
	CLAYEY SILT (ML), dark yellowish brown (10YR 3/6), with sand, trace pores, trace charcoal
	SILTY CLAY (CL), trace pores at top
	SANDY SILT (ML), yellowish brown (10YR 5/8), fine to coarse grained sand
	SILTY SAND (SM), brown (7.5YR 4/4), fine grained sand, ~30-40% fines, decreasing silt with depth
	Strong brown (7.5YR 4/6), trace fine gravel, increasing sand
	GRAVEL (GM), 2" cobble
	POORLY GRADED SAND (SP), brown (10YR 5/3), trace silt, fine to medium grained sand
	SAND WITH SILT (SP-SM), yellowish brown (10YR 5/8)
	SILTY SAND (SM), with gravel, fine to coarse grained sand, 2" cobble
	SANDY SILT (ML), dark yellowish brown (10YR 4/6), increasing sand with depth
	SILTY SAND (SM), strong brown (7.5YR 5/6), fine to medium grained sand
	GRAVEL (GM), fine to coarse gravel, increasing coarse sand with depth
	SILTY SAND (SM), strong brown (7.5YR 5/6)
	GRAVEL (GM), with coarse-grained sand, 2" cobble
	SILT (ML), reddish brown / moderate brown (5YR 4/4), trace coarse-grained sand, fine grained sand
	SILTY SAND (SM), yellowish brown (10YR 5/8), fine grained sand, ~30-40% silt



Run Number	Recovery	Moisture Content (% dry weight)	Gravel Content (%)	Sand Content (%)	Fines Content (%)	Notes
1	2.5/2.5					
2	2.5/2.5					
3	2.5/2.5					
4	2.5/2.5					
5	2/2.5					
6	2/2.5		5	91	4	
7	2/2.5					
8	2.5/2.5					
9	2/2.5					
10	2/2.5	11.5	2	58	40	(OSL)
11	5/5					

Depth in Feet: 5, 10, 15, 20, 25, 30, 35

Depth in Meters: 1, 2, 3, 4, 5, 6, 7, 8, 9, 10

ENGEO
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LOG OF BORING C-3

Field Exploration
SPBB Hollywood
Los Angeles, California
11613.000.000

DATE DRILLED: 10/9/2014
HOLE DEPTH: 90 ft.
HOLE DIAMETER: 8.0 in.
SURF ELEV (FT-AMSL): 390.64 ft.

LOGGED / REVIEWED BY: P. Lam / PJS
DRILLING CONTRACTOR: Martini Drilling
DRILLING METHOD: Hollow Stem Auger
HAMMER TYPE: Dry Core

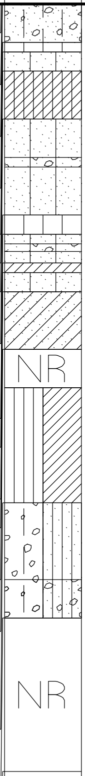
Depth in Feet	Depth in Meters	Formation	DESCRIPTION	Log Symbol	Water Level	Run Number	Recovery	Moisture Content (% dry weight)	Gravel Content (%)	Sand Content (%)	Fines Content (%)	Notes
		Qyf (Young fan)	Trace fine gravel, ~20-30% silt									
	11		SANDY SILT (ML), brown (7.5YR 5/4), fine to medium grained sand									
			SILTY SAND (SM), yellowish red (5YR 4/6), trace fine gravel			12	5/5					
			Fine to medium grained sand									
								12.6	1	60	39	(OSL)
	12		Trace gravel, gravel up to 1"-diameter, increasing with depth									
40												
		Qof (Old fan)	CLAYEY SAND (SC), trace fine gravel									
									3	73	24	
			SILTY SAND (SM), yellowish red (5YR 4/6)			13	5/5					
	13											
			Trace fine gravel									
45			Trace fine gravel									
	14											
			SILTY SAND (SM), reddish brown (5YR 4/3), trace coarse-grained sand, fine to medium grained sand, clay films on sand			14	5/5					
	15							11.8	1	60	39	(OSL)
50												
			Trace mottling with yellowish red (5YR 4/6)									
	16		Coarse sand lens			15	5/5					
			SANDY CLAY (CL), reddish brown (5YR 4/3)									
55			SILTY SAND (SM), yellowish red (5YR 4/6), with fine gravel									
	17		SANDY CLAY (CL), reddish brown / moderate brown (5YR 4/4), fine to medium grained sand									
			Trace carbonate, poorly developed platy ped structure			16	5/5					
			Fine to medium sand lens									
	18		Fine to medium sand lens, trace carbonate filaments					16.1	0	50	50	(OSL)
60												
			SILTY SAND (SM), yellowish red (5YR 4/6), fine to coarse grained sand									
	19		SANDY CLAY (CL)			17	5/5					
			SILTY SAND (SM), yellowish red (5YR 5/8), ~15-20% fines									
			SANDY SILT (ML), reddish brown / moderate brown (5YR 4/4), transitioning to dark reddish brown (5YR 3/4) then to yellowish red (5YR 4/6)									
65	20		SILTY SAND (SM), yellowish red (5YR 5/8), fine to medium grained sand									
			SILTY CLAY (CL-ML), yellowish red (5YR 4/6)			18	5/5					14C Age: 15,560-15,265 Cal BP (Organic sediment)
			SILT (ML)									
	21		SILTY CLAY (CL-ML), yellowish red (5YR 4/6), with mottling dark reddish brown (5YR 2.5/2)					9.3	17	57	26	
70												

LOG OF BORING C-3

Field Exploration
SPBB Hollywood
Los Angeles, California
11613.000.000

DATE DRILLED: 10/9/2014
HOLE DEPTH: 90 ft.
HOLE DIAMETER: 8.0 in.
SURF ELEV (FT-AMSL): 390.64 ft.

LOGGED / REVIEWED BY: P. Lam / PJS
DRILLING CONTRACTOR: Martini Drilling
DRILLING METHOD: Hollow Stem Auger
HAMMER TYPE: Dry Core



Depth in Feet	Depth in Meters	Formation	DESCRIPTION	Log Symbol	Water Level	Run Number	Recovery	Moisture Content (% dry weight)	Gravel Content (%)	Sand Content (%)	Fines Content (%)	Notes
		Qof (Old fan)	SILT (ML)									(OSL)
			CLAYEY SIILT (ML), dark reddish brown / moderate brown (5YR 3/4)									
	22		SILTY SAND (SM), yellowish red (5YR 4/6), fine to medium grained sand, ~30-40% fines			19	4.5/5					14C Age: 15,775-15,560 Cal BP (Charcoal)
			SANDY CLAY (CL), dark reddish brown / moderate brown (5YR 3/4)									
75	23		SILTY SAND WITH GRAVEL (SM), yellowish red (5YR 4/6), fine gravel, fine to coarse grained sand, ~20-30% fines									
			SILT (ML), yellowish red (5YR 4/6)									
			SILTY SAND (SM), fine to medium grained sand, 2" cobble									
	24		SILTY CLAY (CL-ML), dark reddish brown (5YR 2.5/2), increasingsand with depth			20	4/5	9.7	1	67	32	(OSL)
80			SILTY SAND (SM), yellowish red (5YR 4/6), with fine to coarse gravel, transitioning to strong brown (7.5YR 5/8), 3" cobble at 74'	NR								
			SILT (ML), trace gravel									
	25		SILTY SAND (SM), strong brown (7.5YR 5/6), fine grained sand, 2" cobble			21	4.5/5					
			SILTY CLAY (CL-ML), yellowish red (5YR 4/6)									
			SILTY SAND (SM), yellowish red / light brown (5YR 5/6), fine gravel, fine to medium grained sand, ~40-50% fines									
85	26		CLAYEY SAND (SC), dark brown (7.5YR 3/3), fine to medium grained sand									
	27		CLAYEY SILT (ML), dark reddish brown (5YR 3/3), trace fine gravel, fine to medium grained sand, decreasing sand with depth Reddish brown / moderate brown (5YR 4/4)	NR		22	1/5					
90			SANDY SILTY GRAVEL (GM), yellowish red (5YR 5/8), broken cobbles in core Cobble in sampler									
			No recovery from 86' to 90', hard drilling and rig chatter for entire run									

No groundwater encountered.

ENGEO
Expect Excellence

LOG OF BORING A-C-4

Field Exploration
SPBB Hollywood
Los Angeles, California
11613.000.000

DATE DRILLED: 11/20/2014
HOLE DEPTH: 90 ft.
HOLE DIAMETER: 8.0 in.
SURF ELEV (FT-AMSL): Approx. 397½ ft.

LOGGED / REVIEWED BY: P. Lam / PJS
DRILLING CONTRACTOR: Martini Drilling
DRILLING METHOD: Hollow Stem Auger
HAMMER TYPE: Dry Core

Depth in Feet	Depth in Meters	Formation	DESCRIPTION	Log Symbol	Water Level
		Qaf (Fill)	ASPHALT		
			AGGREGATE BASE		
	1		SILTY SAND (SM), hand auger		
5		Qyf (Young fan)	SILTY SAND (SM), strong brown (7.5YR 5/6), fine to medium grained sand		
	2		SILT WITH SAND (ML), dark brown (7.5YR 3/4)		
			GRAVEL (GM), fine to coarse gravel		
			SILTY SAND (SM), strong brown (7.5YR 4/6)		
10	3		SANDY SILT (ML), brown (7.5YR 4/4), increasing silt with depth		
			SILTY CLAY (CL-ML), dark yellowish brown (10YR 3/6), increasing silt with depth		
	4		SILT (ML)		
			SILTY SAND (SM), yellowish brown (10YR 5/8)		
15	5		SILT (ML), brown (10YR 5/3), with fine- to medium-grained sand, trace pores, trace gravel, increasing sand with depth		
			SANDY SILT (ML), brown (10YR 5/3)		
20	6		SILTY SAND (SM), brown (10YR 5/3), fine grained sand		
	7		SAND WITH SILT (SP-SM), yellowish brown (10YR 5/8), trace fine gravel, fine to medium grained sand, <10% fines, decreasing sand with depth		
25			SILTY CLAY (CL-ML), dark yellowish brown (10YR 4/6), trace gravel		
	8		SILTY SAND (SM), yellowish brown (10YR 5/8), fine to medium grained sand		
			SILT (ML)		
			POORLY GRADED GRAVEL WITH SAND (GM), fine gravel, coarse grained sand		
30	9		SILTY SAND (SM), yellowish brown (10YR 5/8), fine to medium grained sand		
	10		POORLY GRADED SAND (SP), brown (7.5YR 5/3), trace fine gravel, fine to medium grained sand Increasing coarse sand with depth		
			SILT (ML)		
35			POORLY GRADED SAND (SP-SM)	NR	

Run Number	Recovery	Moisture Content (% dry weight)	Gravel Content (%)	Sand Content (%)	Fines Content (%)	Notes
1	2.25/2.5					
2	2/2.5					
3	2.5/2.5					
4	2/2.5					
5	2.5/2.5					
6	2.25/2.5					
7	2.5/2.5					
8	2/2.5					
9	5/5					
10	4/5					

ENGEO
Expect Excellence

LOG OF BORING A-C-4

Field Exploration
SPBB Hollywood
Los Angeles, California
11613.000.000

DATE DRILLED: 11/20/2014
HOLE DEPTH: 90 ft.
HOLE DIAMETER: 8.0 in.
SURF ELEV (FT-AMSL): Approx. 397½ ft.

LOGGED / REVIEWED BY: P. Lam / PJS
DRILLING CONTRACTOR: Martini Drilling
DRILLING METHOD: Hollow Stem Auger
HAMMER TYPE: Dry Core

Depth in Feet	Depth in Meters	Formation	DESCRIPTION	Log Symbol	Water Level	Run Number	Recovery	Moisture Content (% dry weight)	Gravel Content (%)	Sand Content (%)	Fines Content (%)	Notes
	11	Qyf (Young fan)	SILTY SAND (SM), reddish yellow (5YR 6/8), trace fine gravel, fine to medium grained sand									
	12		SILTY CLAY (CL-ML), yellowish red / light brown (5YR 5/6), subtle platy ped structure, possible laminations, weak bleaching @ 37.5', increased silt and color change to strong brown (5YR 4/6)			11	5/5					
40			SILTY SAND (SM), yellowish red (5YR 4/6), trace gravel, medium grained sand, trace clay									
			SILTY CLAY (CL-ML), yellowish red (5YR 4/6), finely laminated at base									
45	13	Qof (Old fan)	SILTY CLAY (CL-ML), yellowish red (5YR 4/6)			12	4.5/5					
	14		SILT WITH SAND (ML), yellowish red / light brown (5YR 5/6), fine grained sand									
			SILTY SAND (SM), yellowish red (5YR 5/8), fine grained sand, ~40% fines			13	5/5					
	15		POORLY GRADED SAND WITH SILT (SP-SM), trace fine gravel, fine to medium grained sand									
50			SILTY CLAY (CL-ML), yellowish red / light brown (5YR 5/6), trace fine-grained sand									
			SILT (ML), strong brown (7.5YR 4/6)									
	16		SILTY CLAY (CL-ML)			14	5/5					
			SILT (ML)									
			SILTY CLAY (CL-ML), strong brown (7.5YR 5/8), trace fine-grained sand									
55			SILT WITH SAND (SP-SM), reddish brown (5YR 5/4)									
	17		SILTY CLAY (CL-ML), strong brown (7.5YR 4/6), some sand									
			SANDY CLAY (CL), brownish yellow (10YR 6/8), fine to coarse grained sand, increasing sand with depth			15	2/5					
60	18			NR								
			SAND WITH SILT (SP-SM), strong brown (7.5YR 4/6), fine to medium grained sand, increasing silt with depth Rig chatter									
	19		SILTY CLAY (CL-ML), brown (7.5YR 5/4), trace sand			16	4.75/5					
65			GRAVELLY SAND (GP), fine gravel									
	20		SILT (ML), strong brown (7.5YR 4/6)									
			SILTY CLAY (CL-ML), brown (7.5YR 4/4)									
			POORLY GRADED SAND (SP-SM), yellowish brown / moderate yellowish (10YR 5/4), trace gravel, ~10% fines			17	4.25/5					
70	21		Crushed weathered rock, partially cemented, cobbles up to 3", rig chatter									

ENGEO
Expect Excellence

LOG OF BORING A-C-4

Field Exploration
SPBB Hollywood
Los Angeles, California
11613.000.000

DATE DRILLED: 11/20/2014
HOLE DEPTH: 90 ft.
HOLE DIAMETER: 8.0 in.
SURF ELEV (FT-AMSL): Approx. 397½ ft.

LOGGED / REVIEWED BY: P. Lam / PJS
DRILLING CONTRACTOR: Martini Drilling
DRILLING METHOD: Hollow Stem Auger
HAMMER TYPE: Dry Core

Depth in Feet	Depth in Meters	Formation	DESCRIPTION	Log Symbol	Water Level	Run Number	Recovery	Moisture Content (% dry weight)	Gravel Content (%)	Sand Content (%)	Fines Content (%)	Notes
		Qof (Old fan)										
			SILT (ML), strong brown (7.5YR 4/6), trace fine gravel			18	1.5/2.5					
	22		POORLY GRADED SAND (SP-SM), yellowish brown / moderate yellowish (10YR 5/4), ~10% fines	NR	▽							
			NO RECOVERY									
			SANDY SILT (ML), brown (7.5YR 5/4), saturated, fine to medium grained sand, increasing fine sand with depth			19	2.5/2.5					14C Age: 17,485-17,190 Cal BP (Charcoal)
75	23		SILTY CLAY (CL-ML), dark reddish brown (5YR 2.5/2), wet, grades to dark reddish brown (5YR 3/4)									
			SANDY CLAY (CL), dark reddish brown / moderate brown (5YR 3/4), wet, fine grained sand			20	5/5					
80	24		SILTY CLAY (CL-ML), reddish brown (5YR 4/3), moist, trace fine gravel, decreasing moisture with depth Dark reddish brown (2.5YR 3/4), trace sand, dry to moist Grades to dark reddish brown (5YR 3/4)									
			GRAVEL AND COBBLES (GP), rig chatter									
	25		Rock in sample	NR		21	3.5/5					
			Decomposed granodiorite in sample	NR								
85	26		SILTY CLAY (CL), reddish brown (2.5YR 4/4), trace fine-grained sand, trace gravel, increasing sand with depth									
			Cobbles									
			SANDY CLAY (CL)									
	27		SILTY SAND (SM), trace fine gravel, medium to coarse grained sand, moist to wet, increasing sand with depth			22	5/5					
90			Bottom of boring at 90 feet. Groundwater encountered at 72.5 feet.									

APPENDIX C

Core Photographs

C-1 05-15

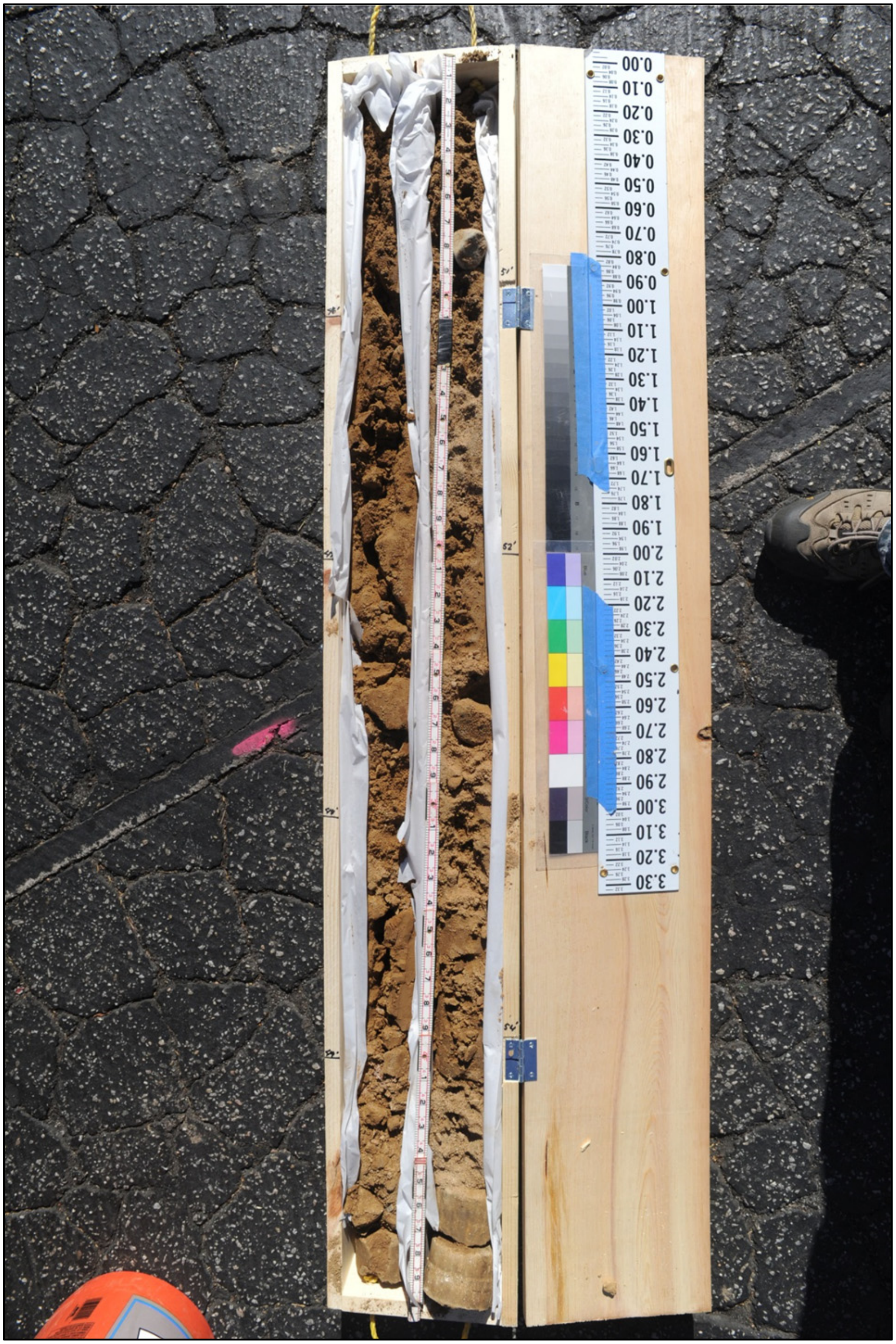
11613.000.000
January 23, 2015

C-1 15-25

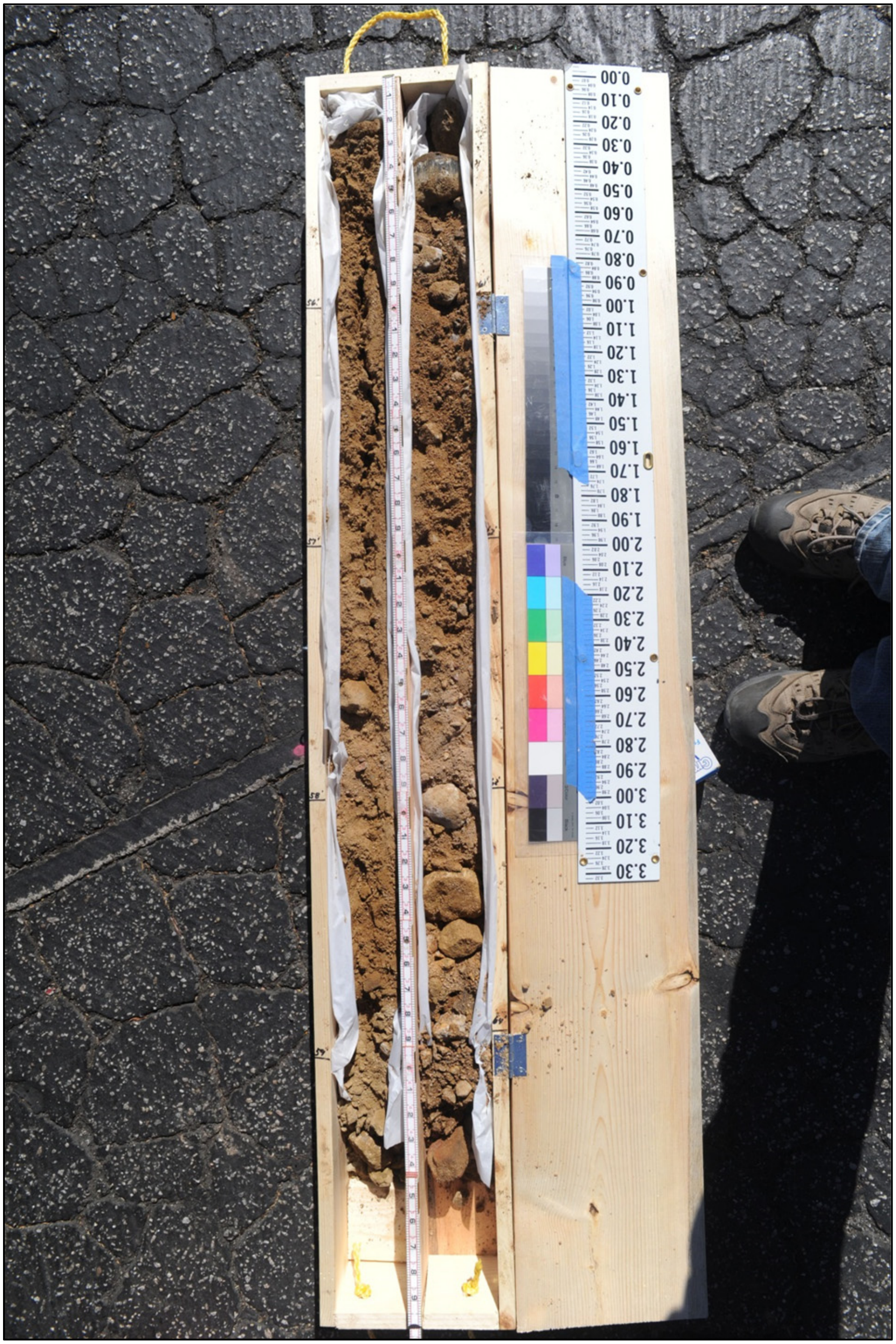
C-1 25-35

11613.000.000
January 23, 2015

C-1 35-45



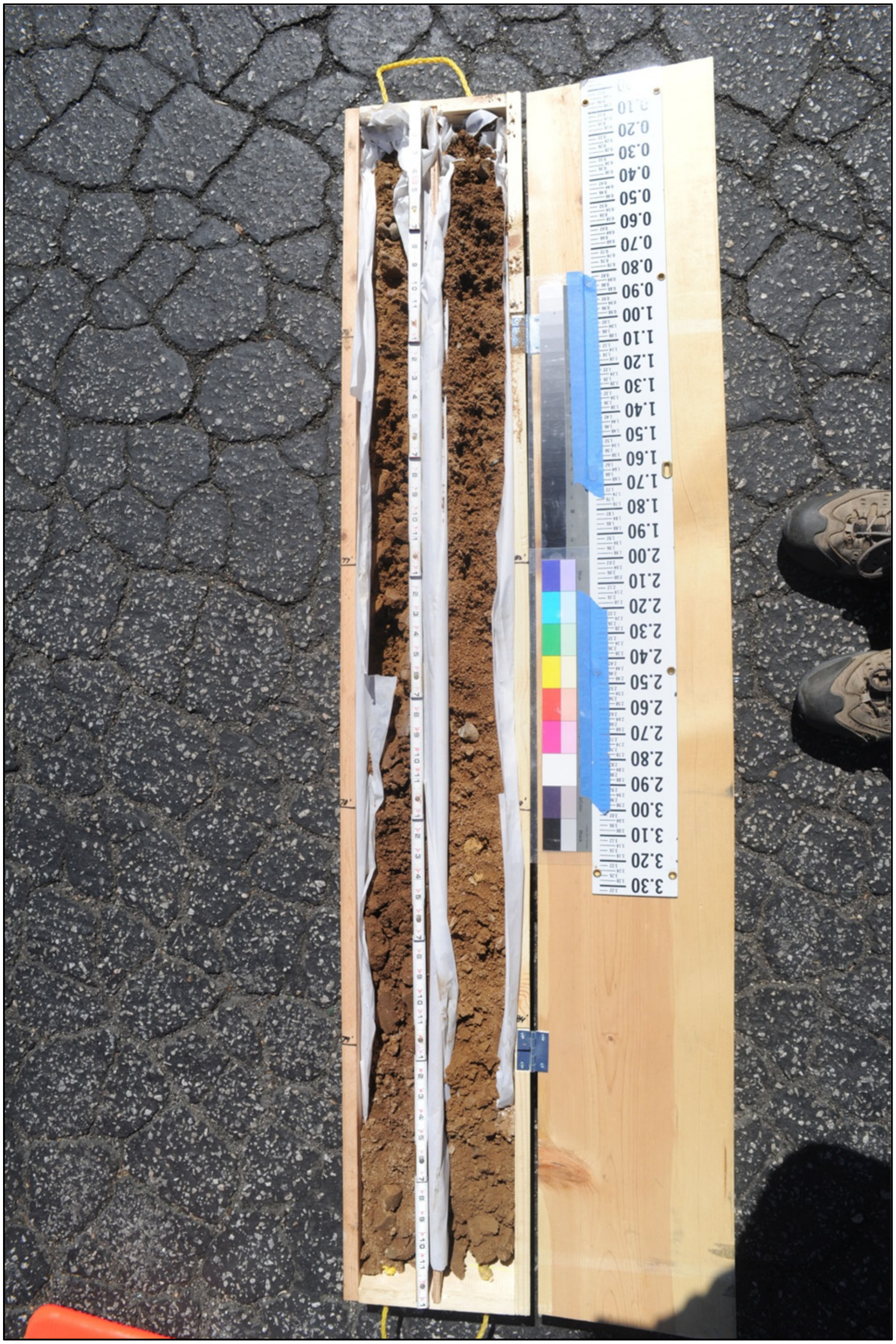
C-1 45-55



C-1 55-65

C-1 65-75

11613.000.000
January 23, 2015



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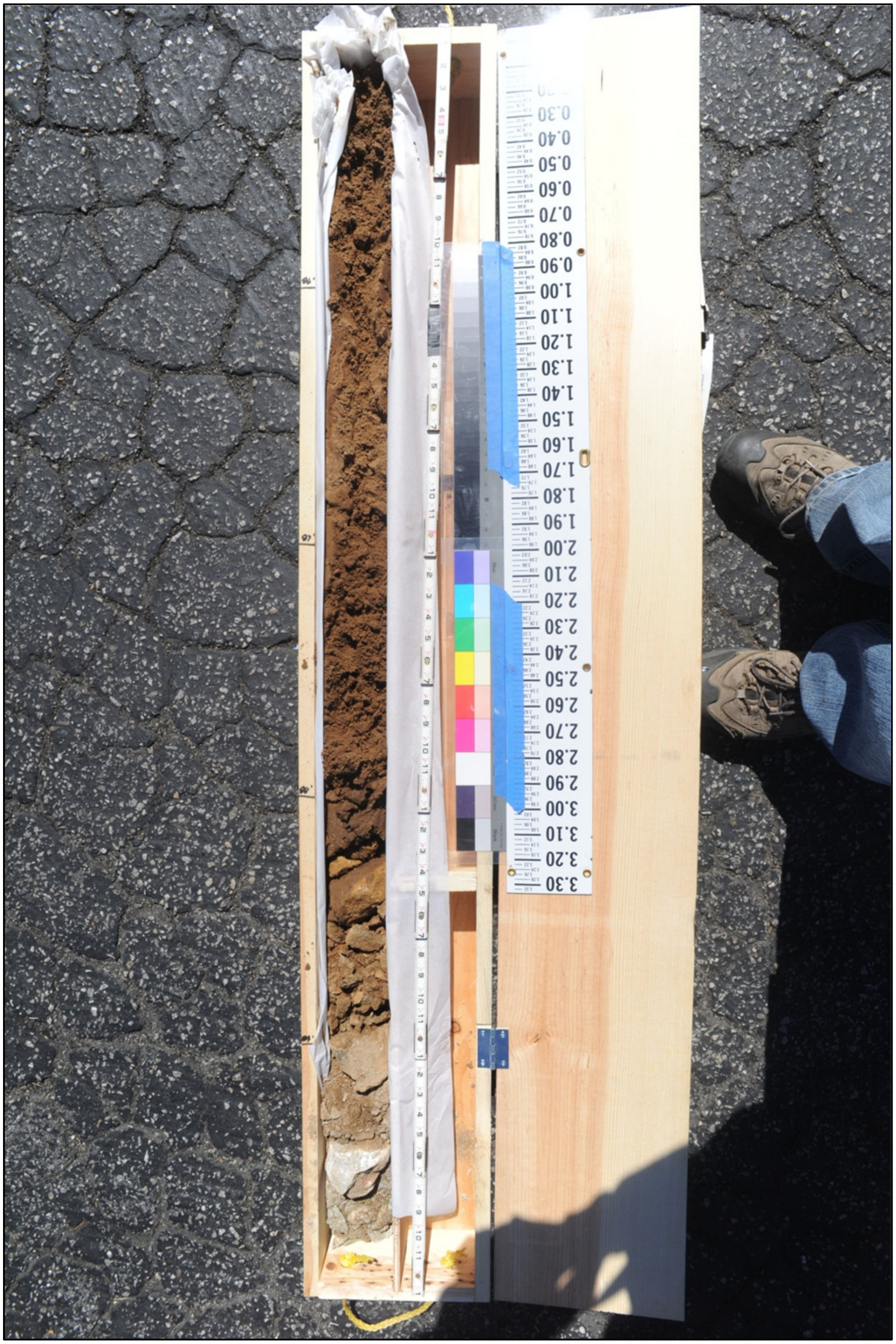
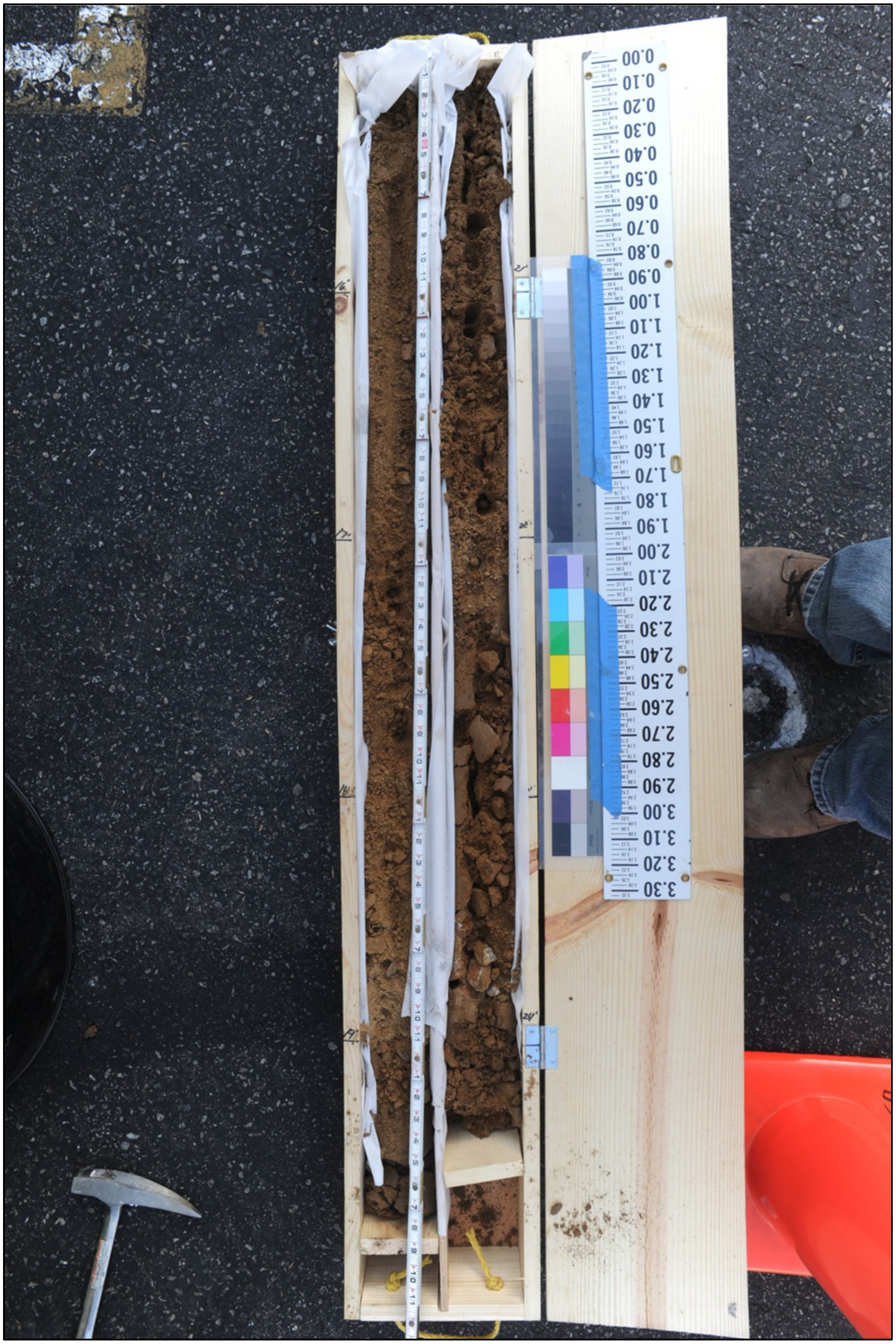


C-1 85-90

11613.000.000
January 23, 2015

C-2 05-15



C-2 15-25

11613.000.000
January 23, 2015

C-2 25-35

C-2 35-45

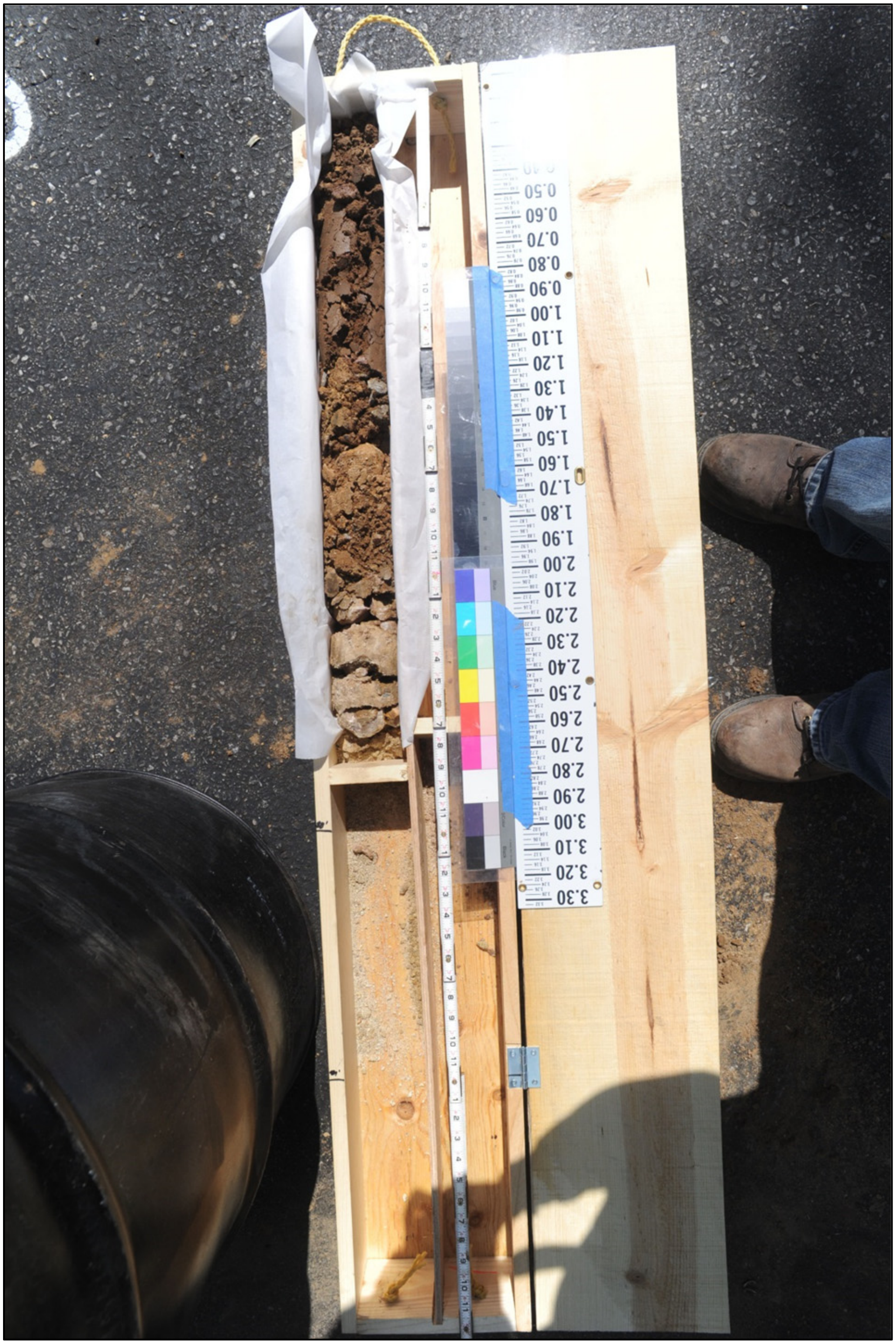
C-2 45-55

C-2 55-65

C-2 65-75

11613.000.000
January 23, 2015

C-2 75-85



C-2 85-90

C-3 05-15

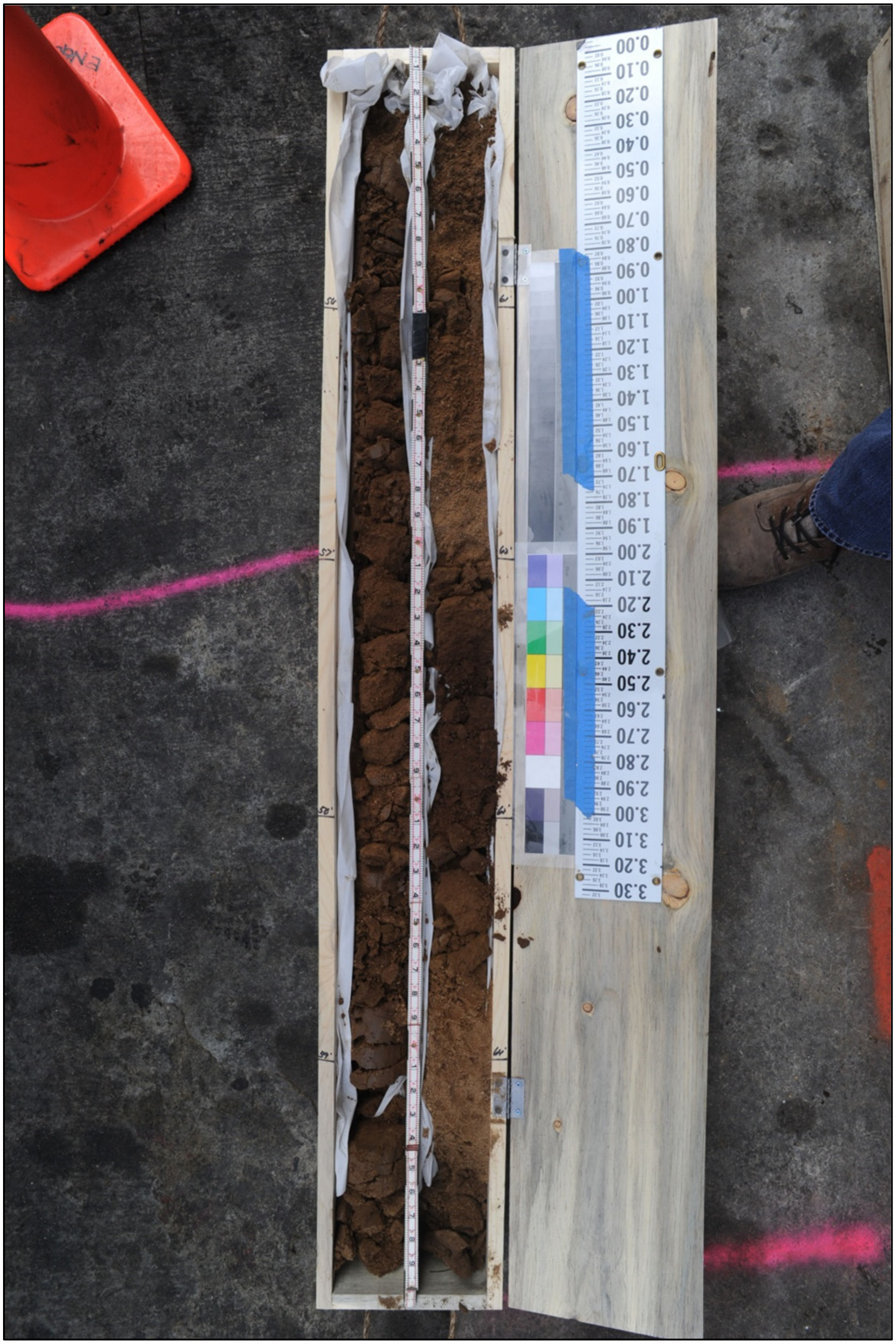
C-3 15-25

C-3 25-35

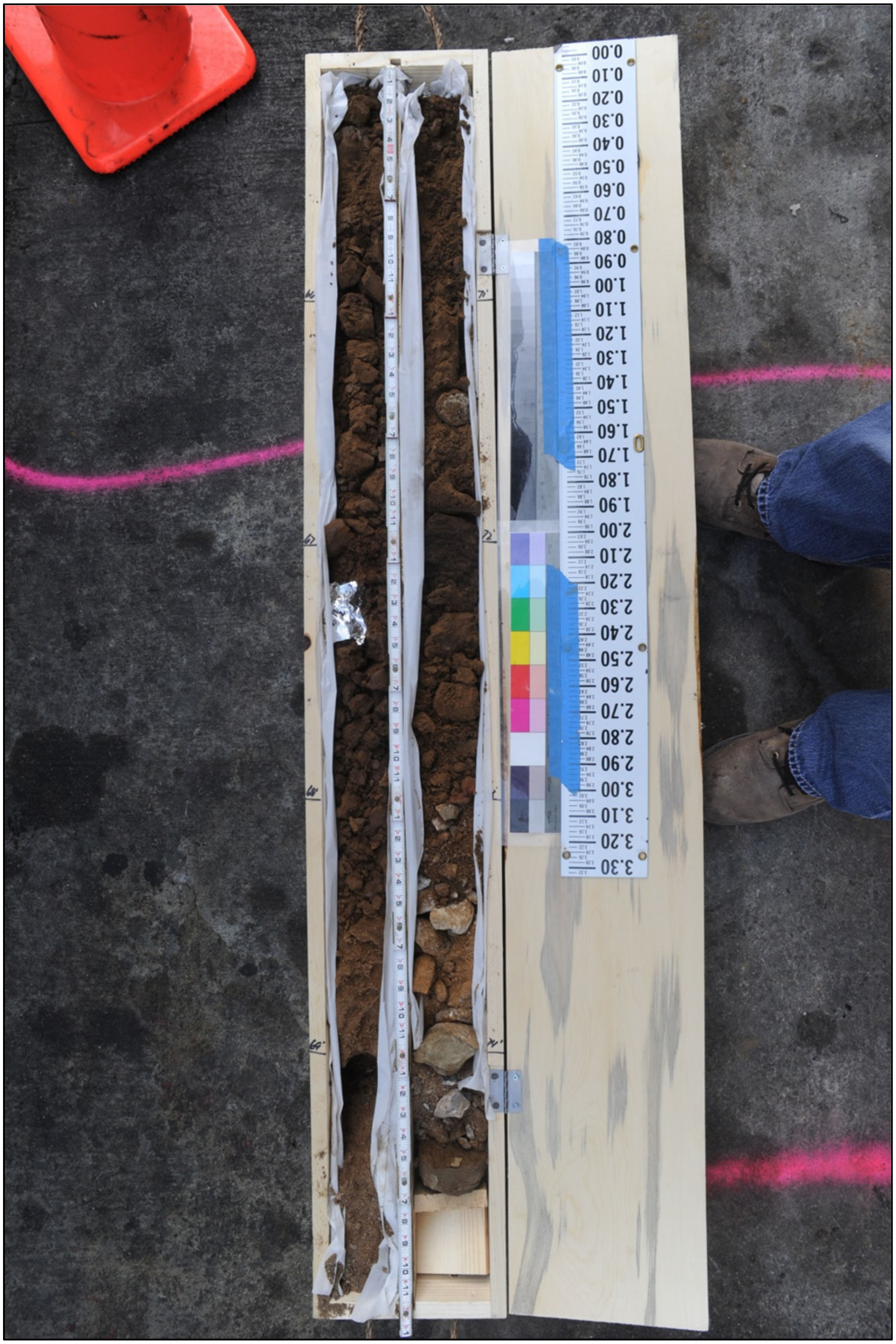
11613.000.000
January 23, 2015

C-3 35-45

C-3 45-55



C-3 55-65



C-3 65-75

C-3 75-85

11613.000.000
January 23, 2015

C-3 85-90

A-C-4 05-15

A-C-4 15-25

11613.000.000
January 23, 2015

A-C-4 25-35

A-C-4 35-45

A-C-4 45-55

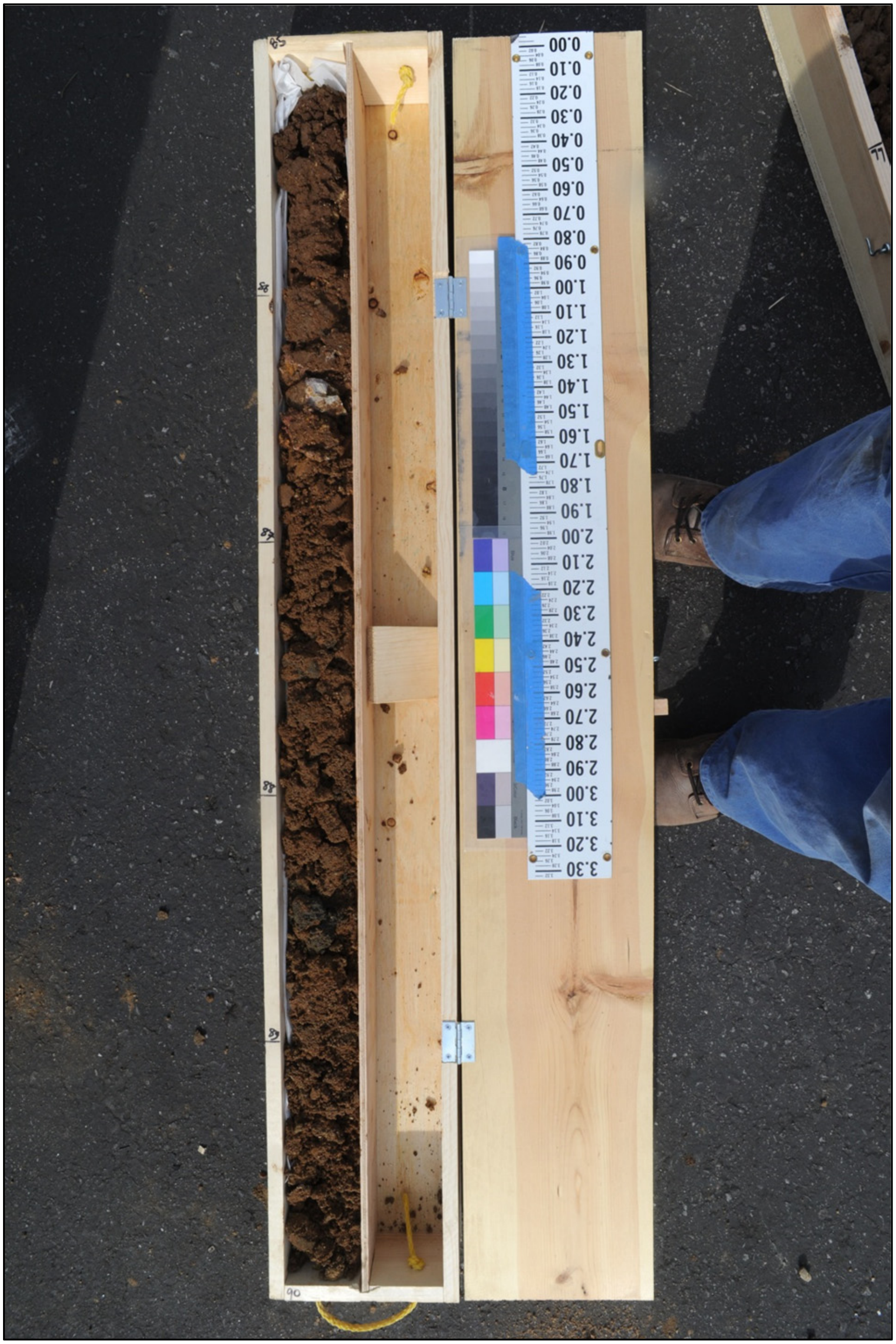
A-C-4 55-65

11613.000.000
January 23, 2015

A-C-4 65-75

11613.000.000
January 23, 2015

A-C-4 75-85



A-C-4 85-90

APPENDIX D

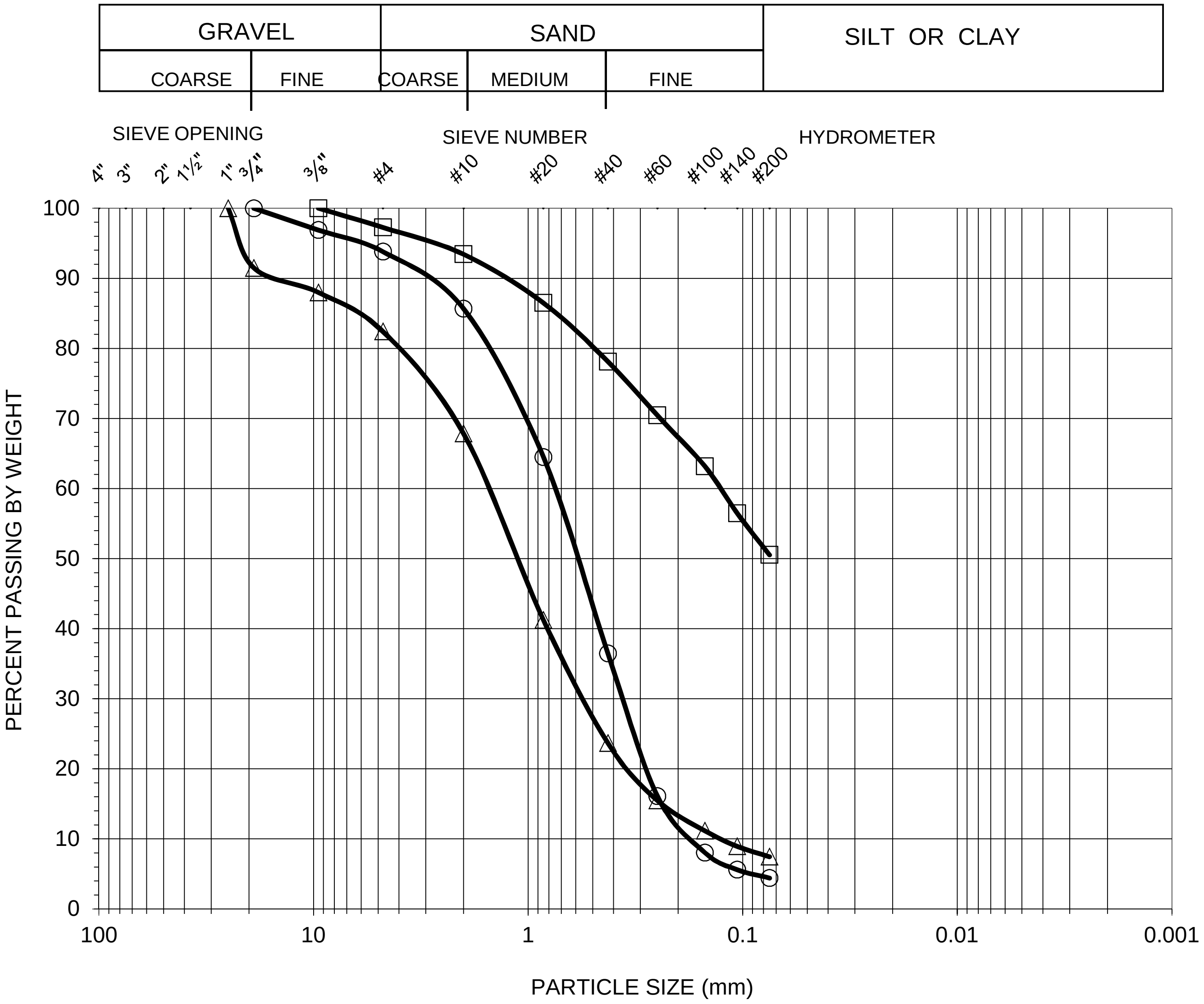
APPENDIX D

Laboratory Test Data
(AP Testing)

AP Engineering & Testing, Inc.

GRAIN SIZE DISTRIBUTION CURVE
ASTM D 6913

Client Name: EnGeo Tested by: KM Date: 10/28/14
Project Name: Hollywood Properties (City of Los Angeles) Computed by: KM Date: 11/11/14
Project No.: 11613.000.000 Checked by: AP Date: 11/11/14



Symbol	Boring No.	Sample No.	Sample Depth (feet)	Percent			Atterberg Limits LL:PL:PI	Soil Symbol ASTM D 2487
				Gravel	Sand	Fines		
○	C-1	-	26	6	90	4	N/A	SP
□	C-1	-	41.5	3	46	51	37:16:21	CL
△	C-1	-	66	18	75	7	N/A	SW-SM

AP Engineering & Testing, Inc.

GRAIN SIZE DISTRIBUTION CURVE
ASTM D 6913

Client Name: EnGeo | Tested by: KM | Date: 10/28/14
Project Name: Hollywood Properties (City of Los Angeles) | Computed by: KM | Date: 11/11/14
Project No.: 11613.000.000 | Checked by: AP | Date: 11/11/14

Symbol	Boring No.	Sample No.	Sample Depth (feet)	Percent			Atterberg Limits LL:PL:PI	Soil Symbol ASTM D 2487
				Gravel	Sand	Fines		
○	C-2	-	13	1	60	39	29:14:15	SC
□	C-2	-	29	1	47	52	33:13:20	CL
△	C-2	-	34	3	77	20	N/A	SM

AP Engineering & Testing, Inc.

GRAIN SIZE DISTRIBUTION CURVE
ASTM D 6913

Client Name: EnGeo
Tested by: KM Date: 10/28/14
Project Name: Hollywood Properties (City of Los Angeles)
Computed by: KM Date: 11/11/14
Project No.: 11613.000.000
Checked by: AP Date: 11/11/14

Symbol	Boring No.	Sample No.	Sample Depth (feet)	Percent			Atterberg Limits LL:PL:PI	Soil Symbol ASTM D 2487
				Gravel	Sand	Fines		
○	C-2	-	39	2	72	26	N/A	SM
□	C-2	-	49	7	58	35	N/A	SM
△	C-2	-	52	1	55	44	N/A	SM

GRAIN SIZE DISTRIBUTION CURVE
ASTM D 6913

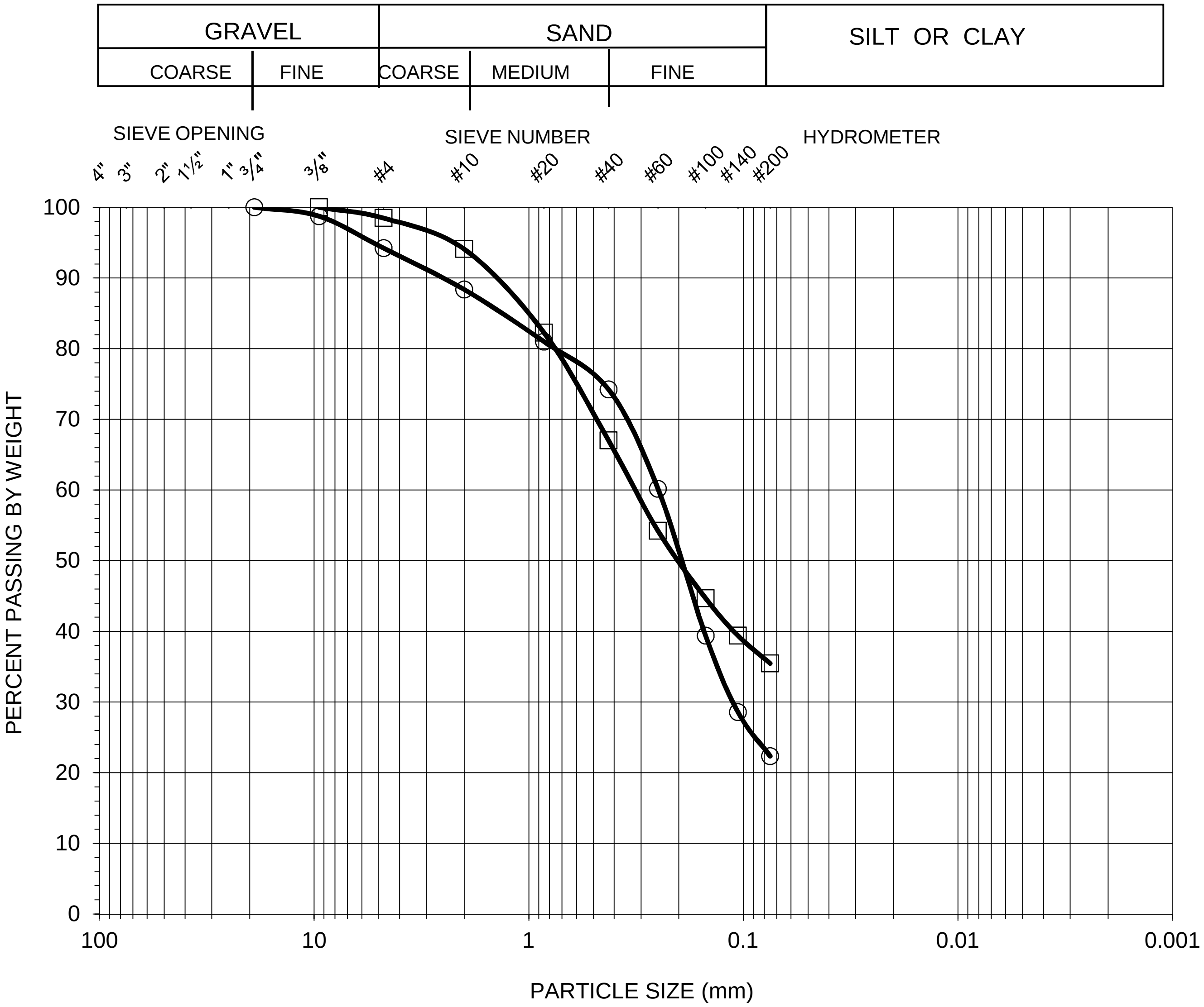
Client Name:	EnGeo	Tested by:	KM	Date:	10/28/14
Project Name:	Hollywood Properties (City of Los Angeles)	Computed by:	KM	Date:	11/11/14
Project No.:	11613.000.000	Checked by:	AP	Date:	11/11/14

Symbol	Boring No.	Sample No.	Sample Depth (feet)	Percent			Atterberg Limits LL:PL:PI	Soil Symbol ASTM D 2487
				Gravel	Sand	Fines		
○	C-2	-	60	0	34	66	40:17:23	CL
☐	C-2	-	61	1	54	45	29:14:15	SC

AP Engineering & Testing, Inc.

GRAIN SIZE DISTRIBUTION CURVE
ASTM D 6913

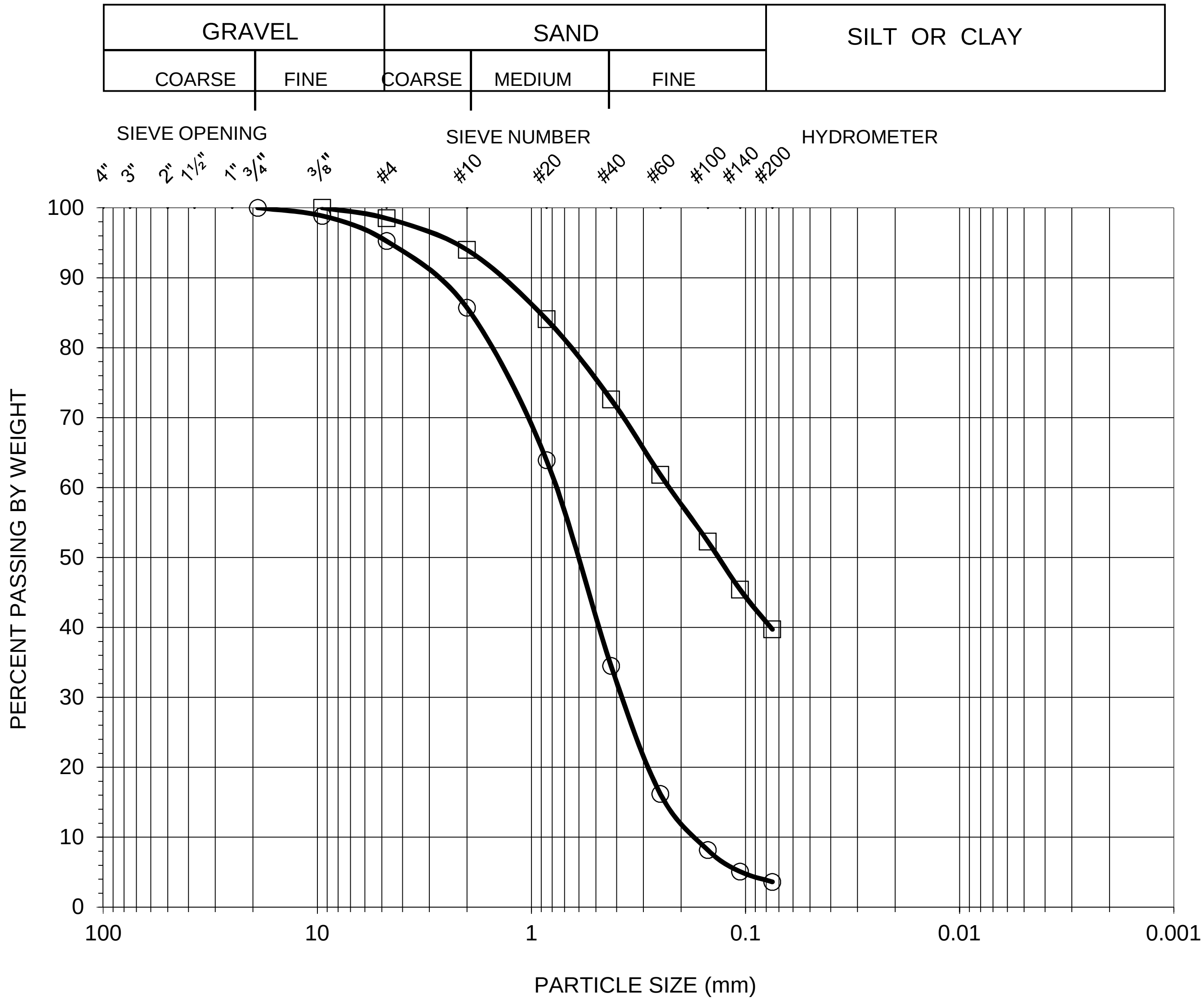
Client Name: EnGeo Tested by: KM Date: 10/28/14
Project Name: Hollywood Properties (City of Los Angeles) Computed by: KM Date: 11/11/14
Project No.: 11613.000.000 Checked by: AP Date: 11/11/14



Symbol	Boring No.	Sample No.	Sample Depth (feet)	Percent			Atterberg Limits LL:PL:PI	Soil Symbol ASTM D 2487
				Gravel	Sand	Fines		
○	C-2	-	68	6	72	22	N/A	SM
□	C-2	-	81	2	63	35	20:13:7	SC

GRAIN SIZE DISTRIBUTION CURVE
ASTM D 6913

Client Name:	EnGeo	Tested by:	KM	Date:	10/28/14
Project Name:	Hollywood Properties (City of Los Angeles)	Computed by:	KM	Date:	11/11/14
Project No.:	11613.000.000	Checked by:	AP	Date:	11/11/14

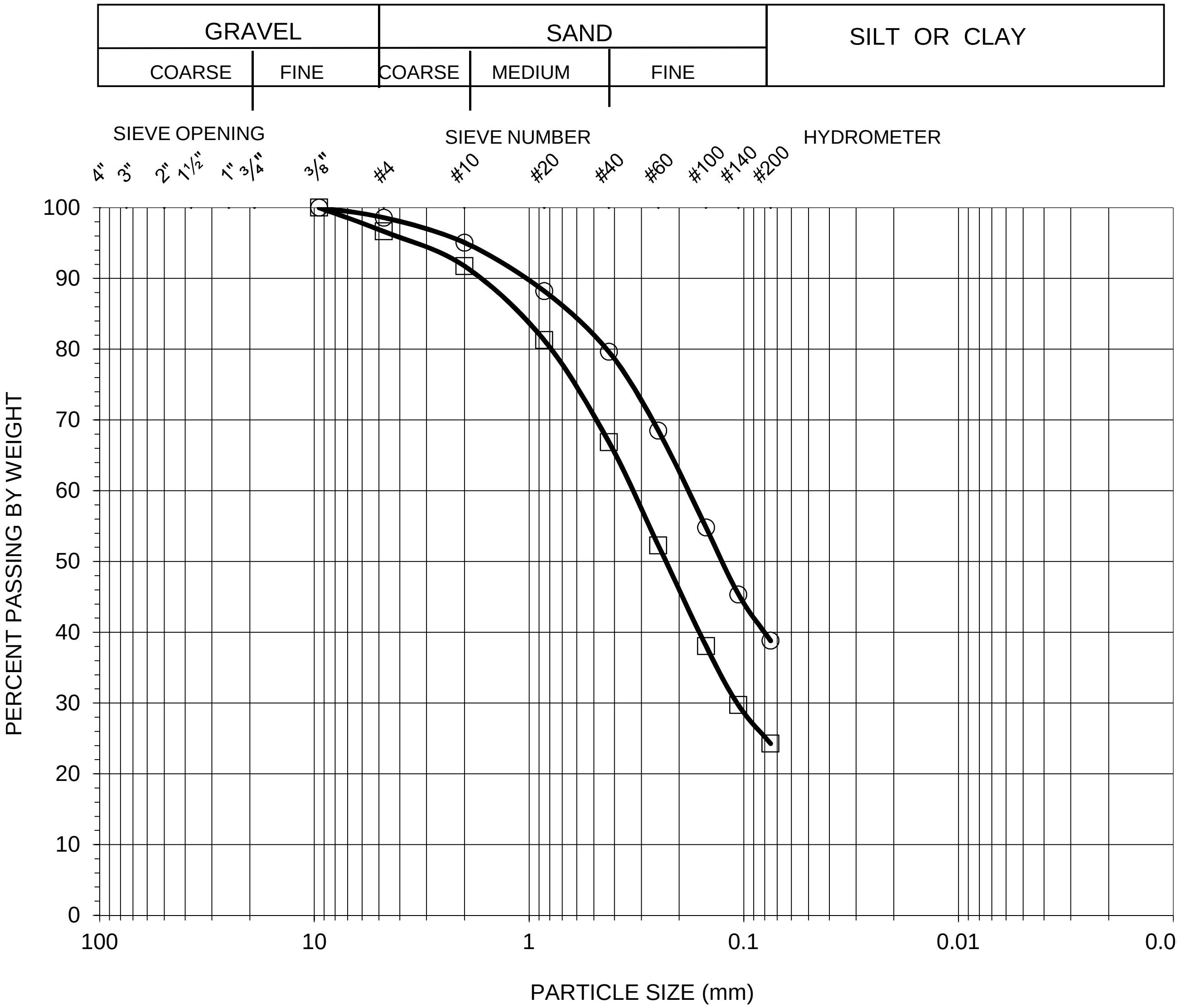


Symbol	Boring No.	Sample No.	Sample Depth (feet)	Percent			Atterberg Limits LL:PL:PI	Soil Symbol ASTM D 2487
				Gravel	Sand	Fines		
○	C-3	-	18	5	91	4	N/A	SP
□	C-3	-	29	2	58	40	N/A	SM

AP Engineering & Testing, Inc.

GRAIN SIZE DISTRIBUTION CURVE
ASTM D 6913

Client Name:	EnGeo	Tested by:	KM	Date:	10/28/14
Project Name:	Hollywood Properties (City of Los Angeles)	Computed by:	KM	Date:	11/11/14
Project No.:	11613.000.000	Checked by:	AP	Date:	11/11/14



Symbol	Boring No.	Sample No.	Sample Depth (feet)	Percent			Atterberg Limits LL:PL:PI	Soil Symbol ASTM D 2487
				Gravel	Sand	Fines		
○	C-3	-	39	1	60	39	N/A	SM
□	C-3	-	42	3	73	24	N/A	SM

AP Engineering & Testing, Inc.

GRAIN SIZE DISTRIBUTION CURVE
ASTM D 6913

Client Name: EnGeo Tested by: KM Date: 10/28/14
Project Name: Hollywood Properties (City of Los Angeles) Computed by: KM Date: 11/11/14
Project No.: 11613.000.000 Checked by: AP Date: 11/11/14

Symbol	Boring No.	Sample No.	Sample Depth (feet)	Percent			Atterberg Limits LL:PL:PI	Soil Symbol ASTM D 2487
				Gravel	Sand	Fines		
○	C-3	-	49	1	60	39	N/A	SM
□	C-3	-	59	0	50	50	33:15:18	CL

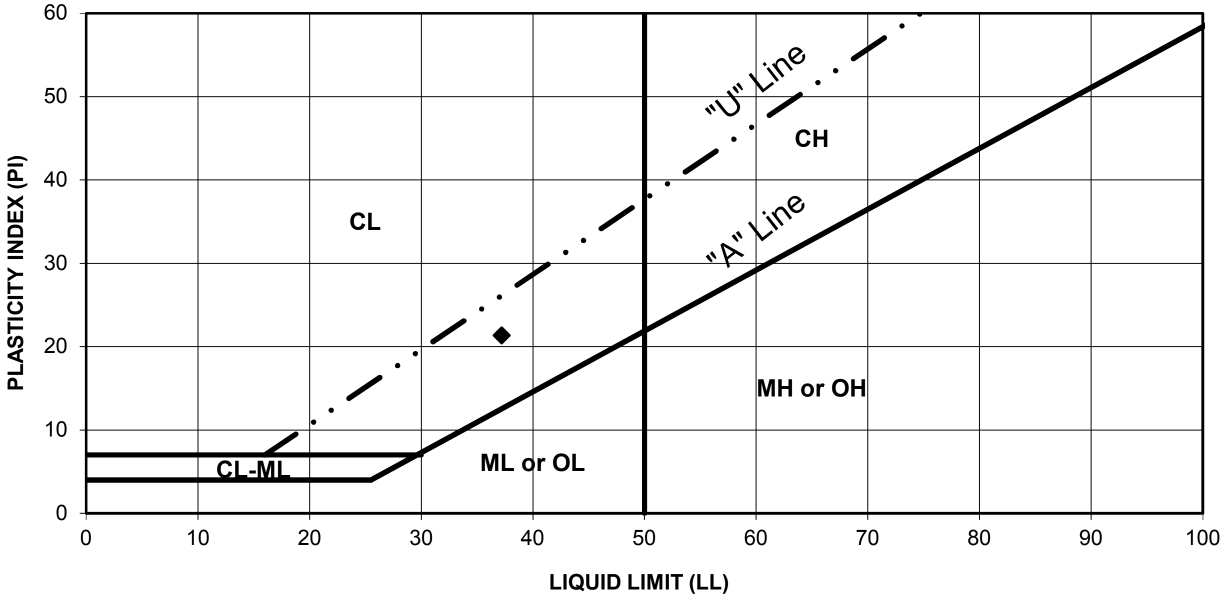
GRAIN SIZE DISTRIBUTION CURVE
ASTM D 6913

Client Name:	EnGeo	Tested by:	KM	Date:	10/28/14
Project Name:	Hollywood Properties (City of Los Angeles)	Computed by:	KM	Date:	11/11/14
Project No.:	11613.000.000	Checked by:	AP	Date:	11/11/14

Symbol	Boring No.	Sample No.	Sample Depth (feet)	Percent			Atterberg Limits LL:PL:PI	Soil Symbol ASTM D 2487
				Gravel	Sand	Fines		
○	C-3	-	69	17	57	26	N/A	SM
□	C-3	-	78.5	1	67	32	22:12:10	SC

ATTERBERG LIMITS
ASTM D 4318

Project Name: Hollywood Properties (City of Los Angeles) **Tested By:** DK Date: 11/01/14
Project No.: 11613.000.000 **Checked By:** AP Date: 11/11/14



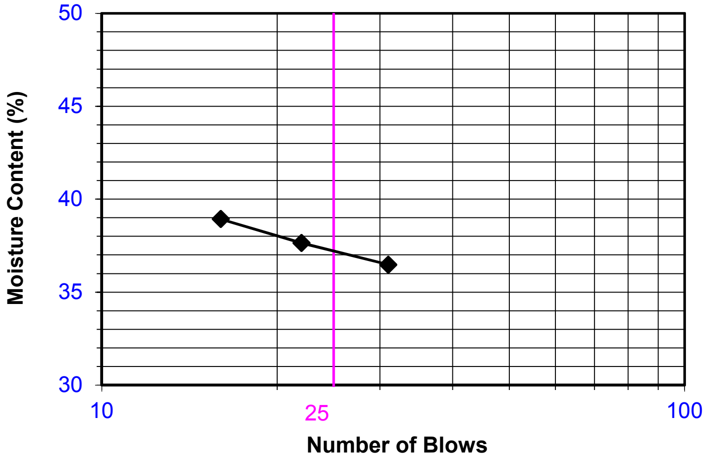
PROCEDURE USED

☐ Wet Preparation

☒ Dry Preparation

☒ Procedure A
Multipoint Test

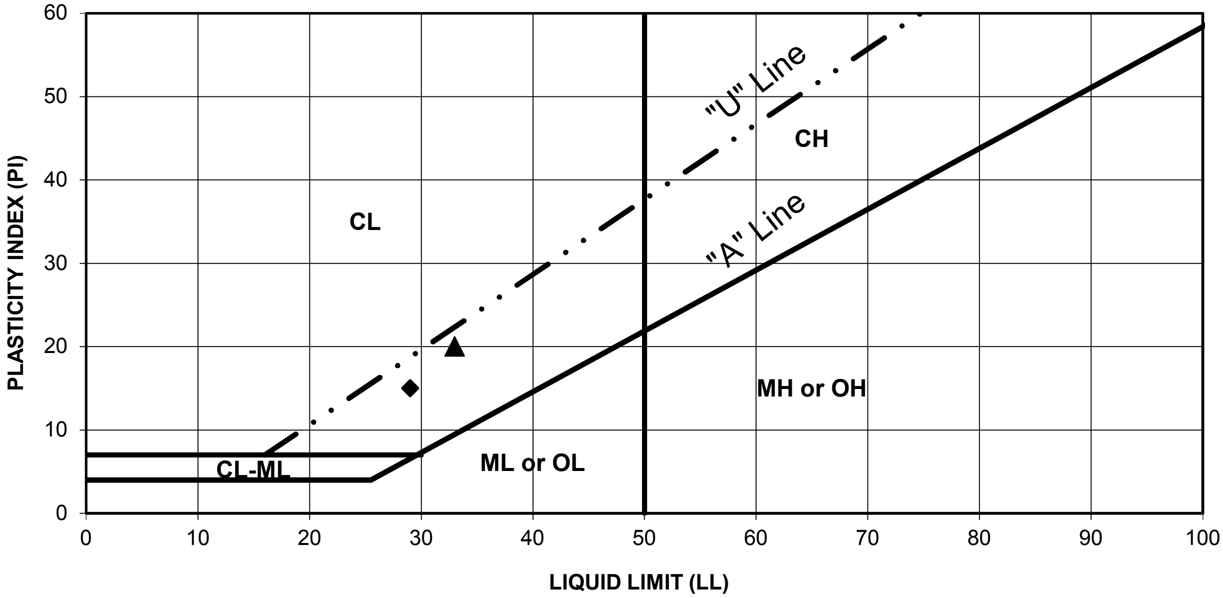
☐ Procedure B
One-point Test



Symbol	Boring Number	Sample Number	Depth (feet)	LL	PL	PI	Plasticity Chart Symbol
♦	C-1	-	41.5	37	16	21	CL

ATTERBERG LIMITS
ASTM D 4318

Project Name: Hollywood Properties (City of Los Angeles) **Tested By:** DK Date: 11/01/14
Project No.: 11613.000.000 **Checked By:** AP Date: 11/11/14



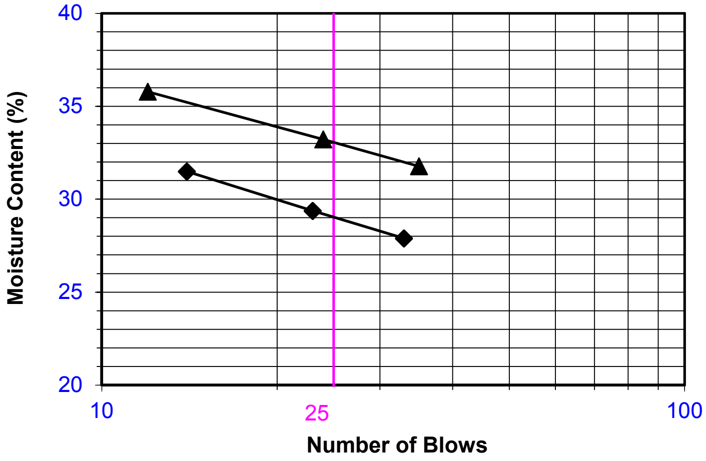
PROCEDURE USED

☐ Wet Preparation

☒ Dry Preparation

☒ Procedure A
Multipoint Test

☐ Procedure B
One-point Test



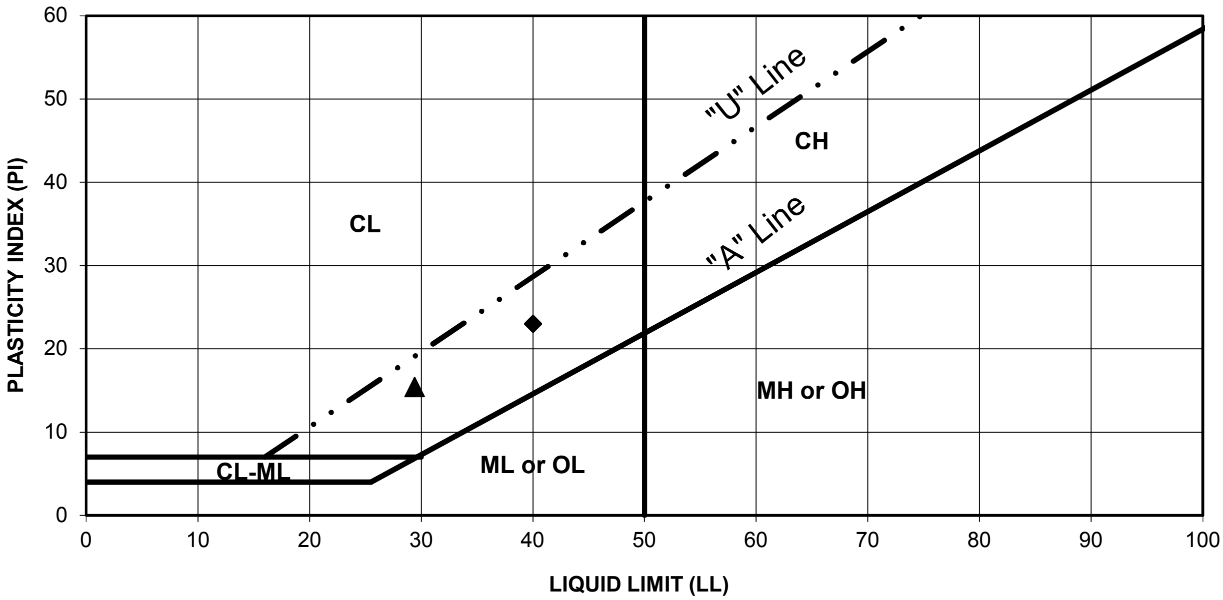
Symbol	Boring Number	Sample Number	Depth (feet)	LL	PL	PI	Plasticity Chart Symbol
◆	C-2	-	13	29	14	15	CL
▲	C-2	-	29	33	13	20	CL

AP Engineering & Testing, Inc.

ATTERBERG LIMITS
ASTM D 4318

Project Name: Hollywood Properties (City of Los Angeles) **Tested By:** DK Date: 11/01/14

Project No.: 11613.000.000 **Checked By:** AP Date: 11/11/14



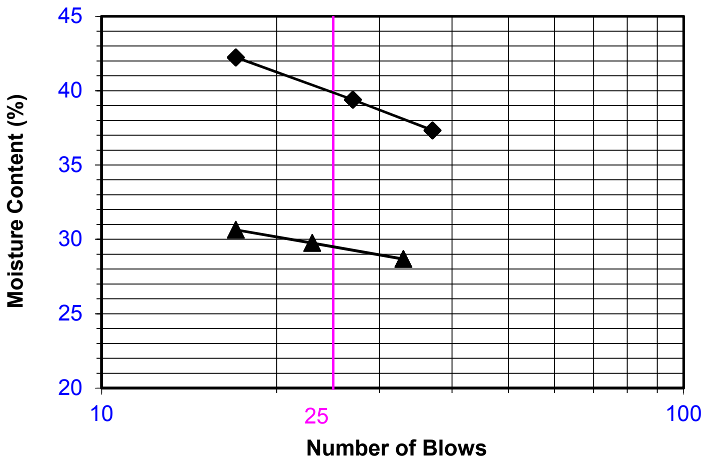
PROCEDURE USED

☐ Wet Preparation

☒ Dry Preparation

☒ Procedure A
Multipoint Test

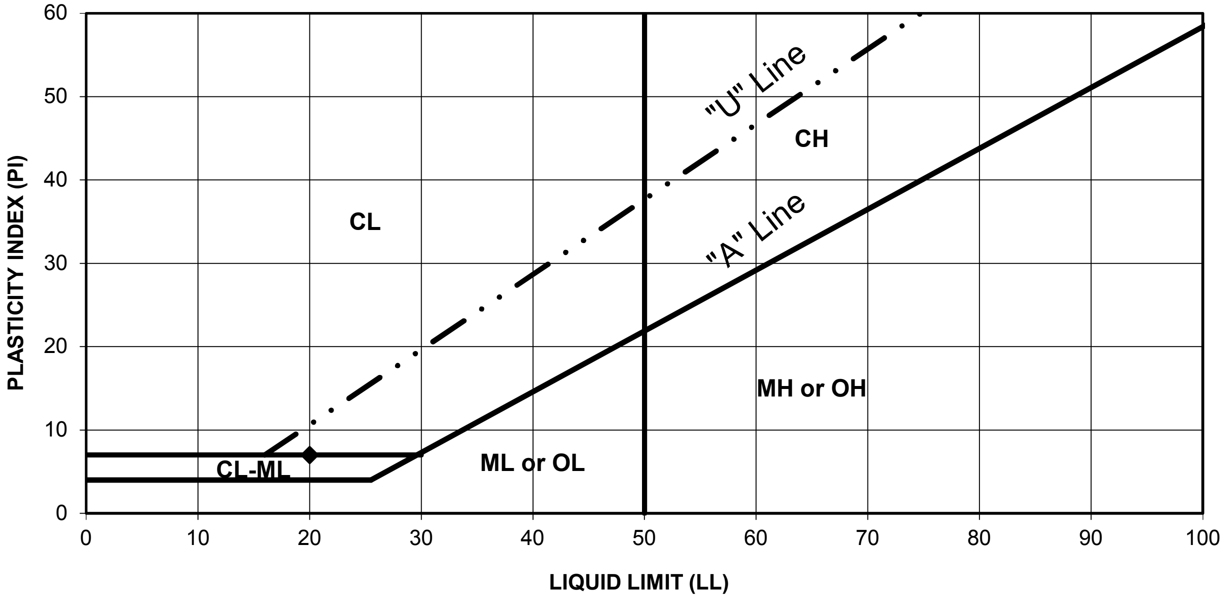
☐ Procedure B
One-point Test



Symbol	Boring Number	Sample Number	Depth (feet)	LL	PL	PI	Plasticity Chart Symbol
◆	C-2	-	60	40	17	23	CL
▲	C-2	-	61	29	14	15	CL

ATTERBERG LIMITS
ASTM D 4318

Project Name: Hollywood Properties (City of Los Angeles) **Tested By:** DK Date: 11/01/14
Project No.: 11613.000.000 **Checked By:** AP Date: 11/11/14



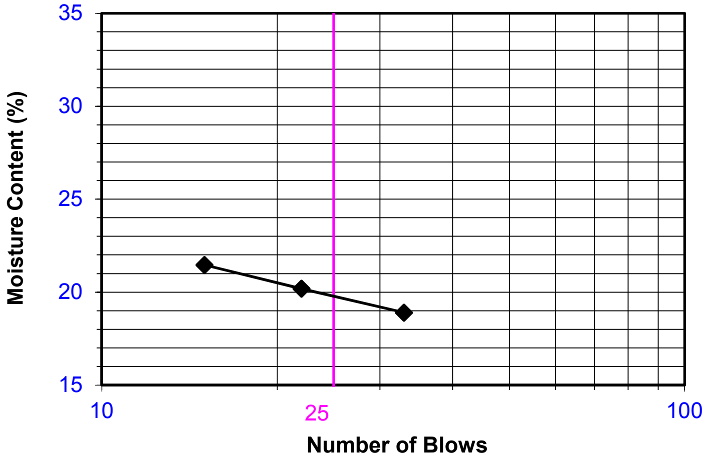
PROCEDURE USED

☐ Wet Preparation

☒ Dry Preparation

☒ Procedure A
Multipoint Test

☐ Procedure B
One-point Test

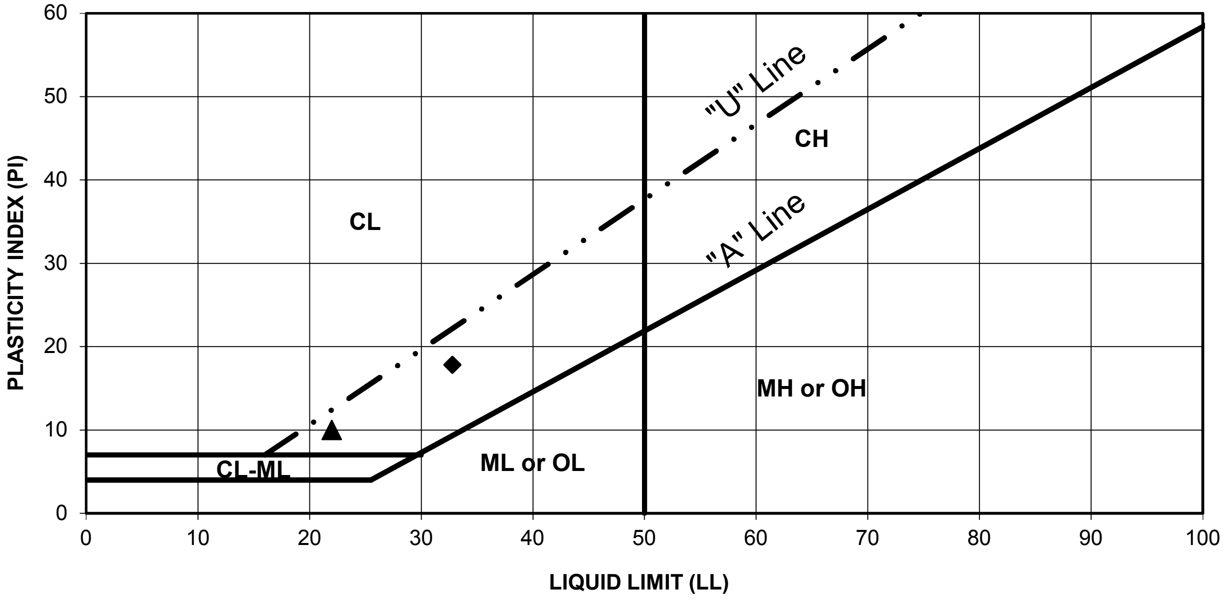


Symbol	Boring Number	Sample Number	Depth (feet)	LL	PL	PI	Plasticity Chart Symbol
♦	C-2	-	81	20	13	7	CL

AP Engineering & Testing, Inc.

ATTERBERG LIMITS
ASTM D 4318

Project Name: Hollywood Properties (City of Los Angeles) **Tested By:** DK Date: 11/01/14
Project No.: 11613.000.000 **Checked By:** AP Date: 11/11/14



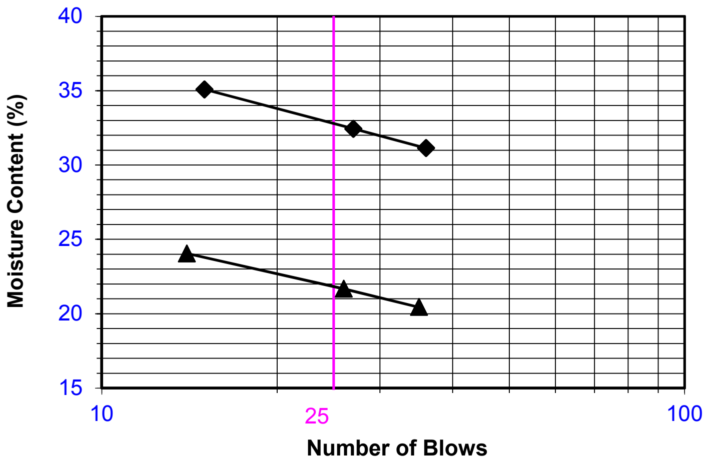
PROCEDURE USED

☐ Wet Preparation

☒ Dry Preparation

☒ Procedure A
Multipoint Test

☐ Procedure B
One-point Test



Symbol	Boring Number	Sample Number	Depth (feet)	LL	PL	PI	Plasticity Chart Symbol
♦	C-3	-	59	33	15	18	CL
▲	C-3	-	78.5	22	12	10	CL

AP Engineering & Testing, Inc.

MOISTURE AND DENSITY TEST RESULTS

Client: EnGeo

Project Name: Hollywood Properties (City of Los Angeles)

Project No.: 11613.000.000

Laboratory No.: 14-1067

Date: 10/28/14

Boring No.	Sample No.	Sample Depth (ft.)	Moisture Content (%)	Dry Density (pcf)
C-1	-	26	1.0	NA
C-1	-	41.5	15.1	NA
C-1	-	66	2.2	NA
C-2	-	29	14.9	NA
C-2	-	34	7.0	NA
C-2	-	39	7.0	NA
C-2	-	49	10.3	NA
C-2	-	60	19.9	NA
C-2	-	68	7.7	NA
C-3	-	29	11.5	NA
C-3	-	39	12.6	NA
C-3	-	49	11.8	NA
C-3	-	59	16.1	NA
C-3	-	69	9.3	NA
C-3	-	78.5	9.7	NA

CORROSION TEST RESULTS

Client Name: EnGeo
Project Name: Hollywood Properties (City of Los Angeles)
Project No.: 11613.000.000

AP Job No.: 14-1067
Date 11/10/14

Boring No.	Sample No.	Depth (feet)	Soil Type	Minimum Resistivity (ohm-cm)	pH	Sulfate Content (ppm)	Chloride Content (ppm)
C-1	-	26	SP	NR	7.5	NR	NR
C-1	-	41.5	CL	NR	7.4	NR	NR
C-1	-	66	SW-SM	NR	7.6	NR	NR
C-2	-	13	SC	NR	7.7	NR	NR
C-2	-	29	CL	NR	7.6	NR	NR
C-2	-	34	SM	NR	7.6	NR	NR
C-2	-	39	SM	NR	7.6	NR	NR
C-2	-	49	SM	NR	7.4	NR	NR
C-2	-	52	SM	NR	7.4	NR	NR
C-2	-	60	CL	NR	7.2	NR	NR

NOTES:

Resistivity Test and pH: California Test Method 643
Sulfate Content : California Test Method 417
Chloride Content : California Test Method 422
ND = Not Detectable
NA = Not Sufficient Sample
NR = Not Requested

CORROSION TEST RESULTS

Client Name: EnGeo

Project Name: Hollywood Properties (City of Los Angeles)

Project No.: 11613.000.000

AP Job No.: 14-1067

Date 11/10/14

Boring No.	Sample No.	Depth (feet)	Soil Type	Minimum Resistivity (ohm-cm)	pH	Sulfate Content (ppm)	Chloride Content (ppm)
C-2	-	61	SC	NR	7.5	NR	NR
C-2	-	68	SM	NR	7.6	NR	NR
C-2	-	81	SC	NR	7.6	NR	NR
C-3	-	18	SP	NR	7.9	NR	NR
C-3	-	29	SM	NR	7.6	NR	NR
C-3	-	39	SM	NR	7.5	NR	NR
C-3	-	42	SM	NR	7.8	NR	NR
C-3	-	49	SM	NR	7.5	NR	NR
C-3	-	59	CL	NR	7.6	NR	NR

NOTES:

Resistivity Test and pH: California Test Method 643
Sulfate Content : California Test Method 417
Chloride Content : California Test Method 422
ND = Not Detectable
NA = Not Sufficient Sample
NR = Not Requested

AP Engineering & Testing, Inc.

CORROSION TEST RESULTS

Client Name: EnGeo
Project Name: Hollywood Properties (City of Los Angeles)
Project No.: 11613.000.000

AP Job No.: 14-1067
Date 11/10/14

Boring No.	Sample No.	Depth (feet)	Soil Type	Minimum Resistivity (ohm-cm)	pH	Sulfate Content (ppm)	Chloride Content (ppm)
C-3	-	69	SM	NR	7.3	NR	NR
C-3	-	78.5	SC	NR	7.3	NR	NR

NOTES:
Resistivity Test and pH: California Test Method 643
Sulfate Content : California Test Method 417
Chloride Content : California Test Method 422
ND = Not Detectable
NA = Not Sufficient Sample
NR = Not Requested

APPENDIX E

^{14}C Measurements
(Beta Analytic)

APPENDIX E

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4985 SW 74 Court
Miami, Florida 33155 USA
Tel: 305 667 5167
Fax: 305 663 0964
Beta@radiocarbon.com
www.radiocarbon.com

Darden Hood
President

Ronald Hatfield
Christopher Patrick
Deputy Directors

October 27, 2014

Mr. Patrick Lam
ENGEO Incorporated
2010 Crow Canyon Place
Suite 250
San Ramon, CA 94583-4634
USA

RE: Radiocarbon Dating Results For Samples 11613 C-1@72.75, 11613 C-3@67.5, 11613 C-3@72

Dear Mr. Lam:

Enclosed are the radiocarbon dating results for three samples recently sent to us. As usual, the method of analysis is listed on the report with the results and calibration data is provided where applicable. The Conventional Radiocarbon Ages have all been corrected for total fractionation effects and where applicable, calibration was performed using 2013 calibration databases (cited on the graph pages).

The web directory containing the table of results and PDF download also contains pictures, a cvs spreadsheet download option and a quality assurance report containing expected vs. measured values for 3-5 working standards analyzed simultaneously with your samples.

Reported results are accredited to ISO/IEC 17025:2005 Testing Accreditation PJLA #59423 standards and all chemistry was performed here in our laboratories and counted in our own accelerators here in Miami. Since Beta is not a teaching laboratory, only graduates trained to strict protocols of the ISO/IEC 17025:2005 Testing Accreditation PJLA #59423 program participated in the analyses.

As always Conventional Radiocarbon Ages and sigmas are rounded to the nearest 10 years per the conventions of the 1977 International Radiocarbon Conference. When counting statistics produce sigmas lower than +/- 30 years, a conservative +/- 30 BP is cited for the result.

When interpreting the results, please consider any communications you may have had with us regarding the samples. As always, your inquiries are most welcome. If you have any questions or would like further details of the analyses, please do not hesitate to contact us.

The cost of the analysis was charged to the VISA card provided. Thank you. As always, if you have any questions or would like to discuss the results, don't hesitate to contact me.

Sincerely,

Darden Hood
Digital signature on file

REPORT OF RADIOCARBON DATING ANALYSES

Mr. Patrick Lam — Report Date: 10/27/2014

ENGEO Incorporated — Material Received: 10/17/2014

Sample Data	Measured Radiocarbon Age	13C/12C Ratio	Conventional Radiocarbon Age(*)
Beta - 393295 SAMPLE : 11613 C-1@72.75 ANALYSIS : AMS-PRIORITY delivery MATERIAL/PRETREATMENT : (charred material): acid/alkali/acid 2 SIGMA CALIBRATION : Cal BC 15970 to 15730 (Cal BP 17920 to 17680)	14560 +/- 50 BP	-21.3 o/oo	14620 +/- 50 BP
Beta - 393296 SAMPLE : 11613 C-3@67.5 ANALYSIS : AMS-PRIORITY delivery MATERIAL/PRETREATMENT : (organic sediment): acid washes 2 SIGMA CALIBRATION : Cal BC 13610 to 13315 (Cal BP 15560 to 15265)	12880 +/- 40 BP	-23.5 o/oo	12900 +/- 40 BP
Beta - 393297 SAMPLE : 11613 C-3@72 ANALYSIS : AMS-PRIORITY delivery MATERIAL/PRETREATMENT : (charred material): acid/alkali/acid 2 SIGMA CALIBRATION : Cal BC 13825 to 13610 (Cal BP 15775 to 15560)	13050 +/- 40 BP	-24.1 o/oo	13060 +/- 40 BP

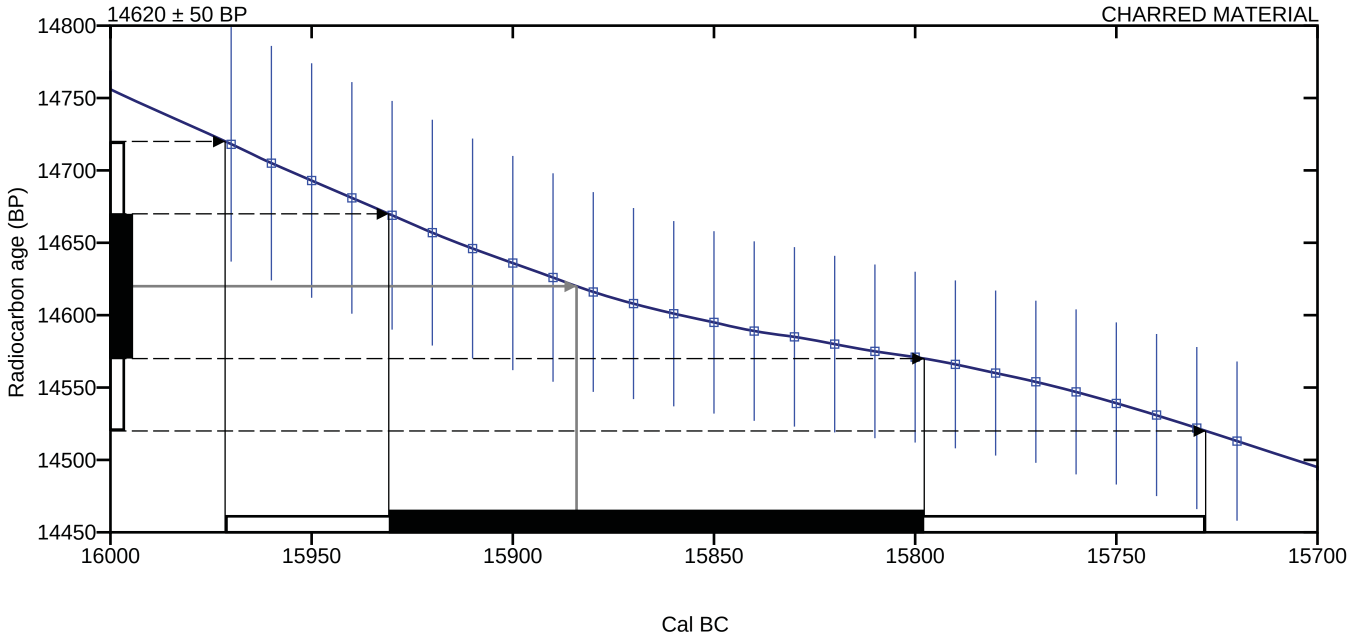
Dates are reported as RCYBP (radiocarbon years before present, "present" = AD 1950). By international convention, the modern reference standard was 95% the 14C activity of the National Institute of Standards and Technology (NIST) Oxalic Acid (SRM 4990C) and calculated using the Libby 14C half-life (5568 years). Quoted errors represent 1 relative standard deviation statistics (68% probability) counting errors based on the combined measurements of the sample, background, and modern reference standards. Measured 13C/12C ratios (delta 13C) were calculated relative to the PDB-1 standard.

The Conventional Radiocarbon Age represents the Measured Radiocarbon Age corrected for isotopic fractionation, calculated using the delta 13C. On rare occasion where the Conventional Radiocarbon Age was calculated using an assumed delta 13C, the ratio and the Conventional Radiocarbon Age will be followed by "*". The Conventional Radiocarbon Age is not calendar calibrated. When available, the Calendar Calibrated result is calculated from the Conventional Radiocarbon Age and is listed as the "Two Sigma Calibrated Result" for each sample.

CALIBRATION OF RADIOCARBON AGE TO CALENDAR YEARS

(Variables: C13/C12 = -21.3 o/oo : lab. mult = 1)

Laboratory number	**Beta-393295**
Conventional radiocarbon age	**14620 ± 50 BP**
2 Sigma calibrated result 95% probability	**Cal BC 15970 to 15730 (Cal BP 17920 to 17680)**
Intercept of radiocarbon age with calibration curve	Cal BC 15885 (Cal BP 17835)
1 Sigma calibrated results 68% probability	Cal BC 15930 to 15800 (Cal BP 17880 to 17750)



Database used

INTCAL13

References

Mathematics used for calibration scenario

A Simplified Approach to Calibrating C14 Dates, Talma, A. S., Vogel, J. C., 1993, Radiocarbon 35(2):317-322

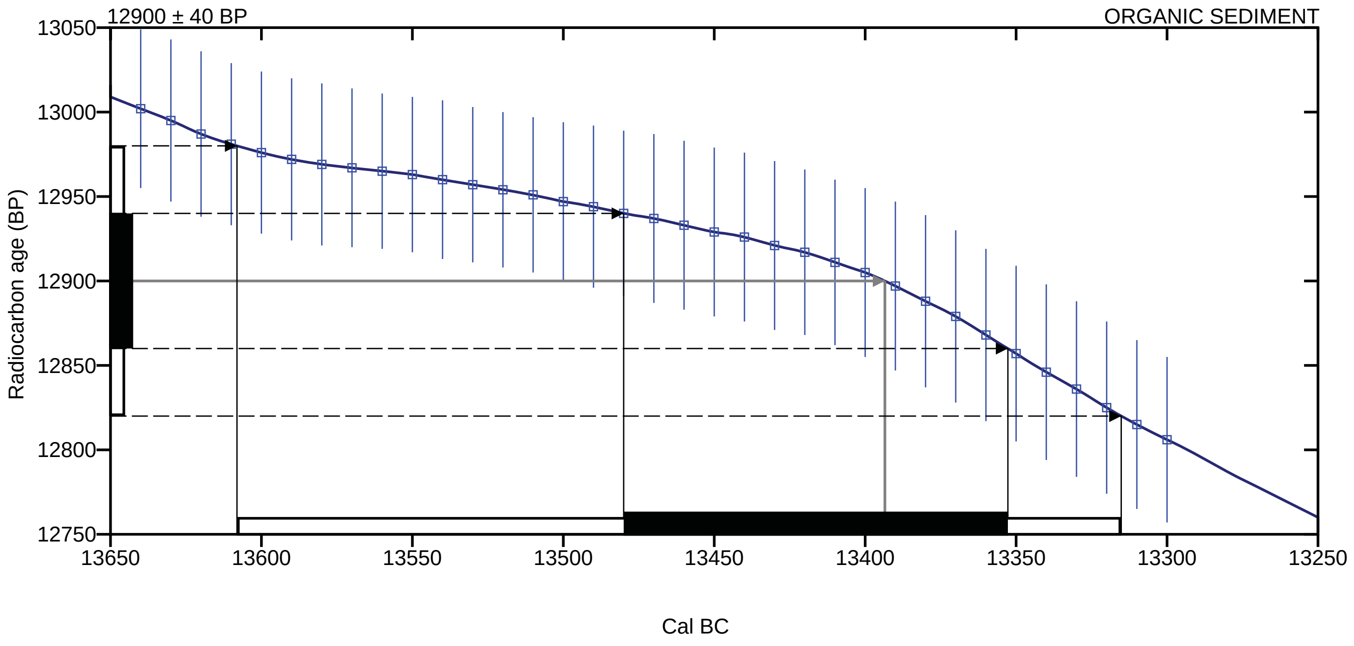
References to INTCAL13 database

Reimer PJ et al. IntCal13 and Marine13 radiocarbon age calibration curves 0– 50,000 years cal BP. Radiocarbon 55(4):1869– 1887.

CALIBRATION OF RADIOCARBON AGE TO CALENDAR YEARS

(Variables: C13/C12 = -23.5 o/oo : lab. mult = 1)

Laboratory number	**Beta-393296**
Conventional radiocarbon age	**12900 ± 40 BP**
2 Sigma calibrated result **95% probability**	**Cal BC 13610 to 13315 (Cal BP 15560 to 15265)**
Intercept of radiocarbon age with calibration curve	Cal BC 13395 (Cal BP 15345)
1 Sigma calibrated results 68% probability	Cal BC 13480 to 13355 (Cal BP 15430 to 15305)



Database used

INTCAL13

References

Mathematics used for calibration scenario

A Simplified Approach to Calibrating C14 Dates, Talma, A. S., Vogel, J. C., 1993, Radiocarbon 35(2):317-322

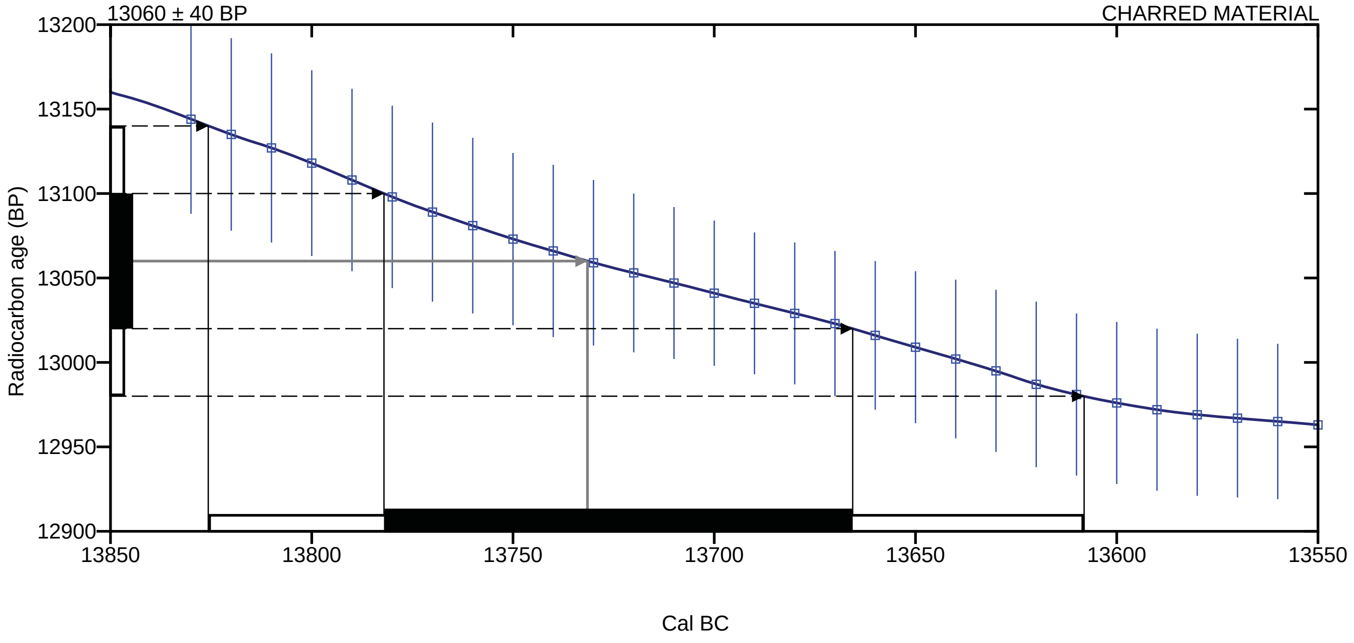
References to INTCAL13 database

Reimer PJ et al. IntCal13 and Marine13 radiocarbon age calibration curves 0– 50,000 years cal BP. Radiocarbon 55(4):1869– 1887.

CALIBRATION OF RADIOCARBON AGE TO CALENDAR YEARS

(Variables: C13/C12 = -24.1 o/oo : lab. mult = 1)

Laboratory number	**Beta-393297**
Conventional radiocarbon age	**13060 ± 40 BP**
2 Sigma calibrated result **95% probability**	**Cal BC 13825 to 13610 (Cal BP 15775 to 15560)**
Intercept of radiocarbon age with calibration curve	Cal BC 13730 (Cal BP 15680)
1 Sigma calibrated results 68% probability	Cal BC 13780 to 13665 (Cal BP 15730 to 15615)



Database used

INTCAL13

References

Mathematics used for calibration scenario

A Simplified Approach to Calibrating C14 Dates, Talma, A. S., Vogel, J. C., 1993, Radiocarbon 35(2):317-322

References to INTCAL13 database

Reimer PJ et al. IntCal13 and Marine13 radiocarbon age calibration curves 0– 50,000 years cal BP. Radiocarbon 55(4):1869– 1887.

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beta@radiocarbon.com
www.radiocarbon.com

Mr. Darden Hood
President

Mr. Ronald Hatfield
Mr. Christopher Patrick
Deputy Directors

The Radiocarbon Laboratory Accredited to ISO-17025 Testing Standards (PJLA Accreditation #59423)

Quality Assurance Report

This report provides the results of reference materials used to validate radiocarbon analyses prior to reporting. Known value reference materials were analyzed quasi-simultaneously with the unknowns. Results are reported as expected values vs measured values. Reported values are calculated relative to NIST SRM-4990B and corrected for isotopic fractionation. Results are reported using the direct analytical measure percent modern carbon (pMC) with one relative standard deviation.

Report Date: October 27, 2014
Submitter : Mr. Patrick Lam

QA MEASUREMENTS

Reference 1
Expected Value: 1.5 +/- 0.1 pMC
Measured Value: 1.6 +/- 0.1 pMC
Agreement: Accepted

Reference 2
Expected Value: 96.8 +/- 0.5 pMC
Measured Value: 96.9 +/- 0.4 pMC
Agreement: Accepted

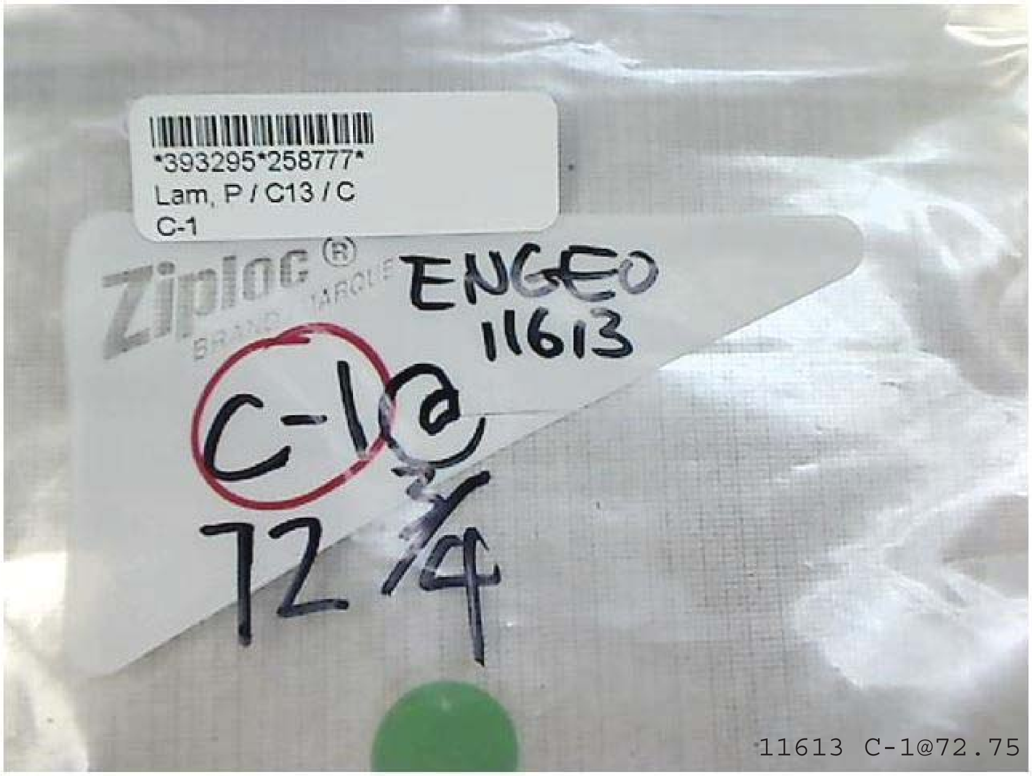
Reference 3
Expected Value: 57.2 +/- 0.3 pMC
Measured Value: 57.5 +/- 0.2 pMC
Agreement: Accepted

Reference 4
Expected Value: 27.4 +/- 0.2
Measured Value: 27.3 +/- 0.1 pMC
Agreement: Accepted

COMMENT: All measurements passed acceptance tests.

Validation: Darden Hood

Date: October 27, 2014

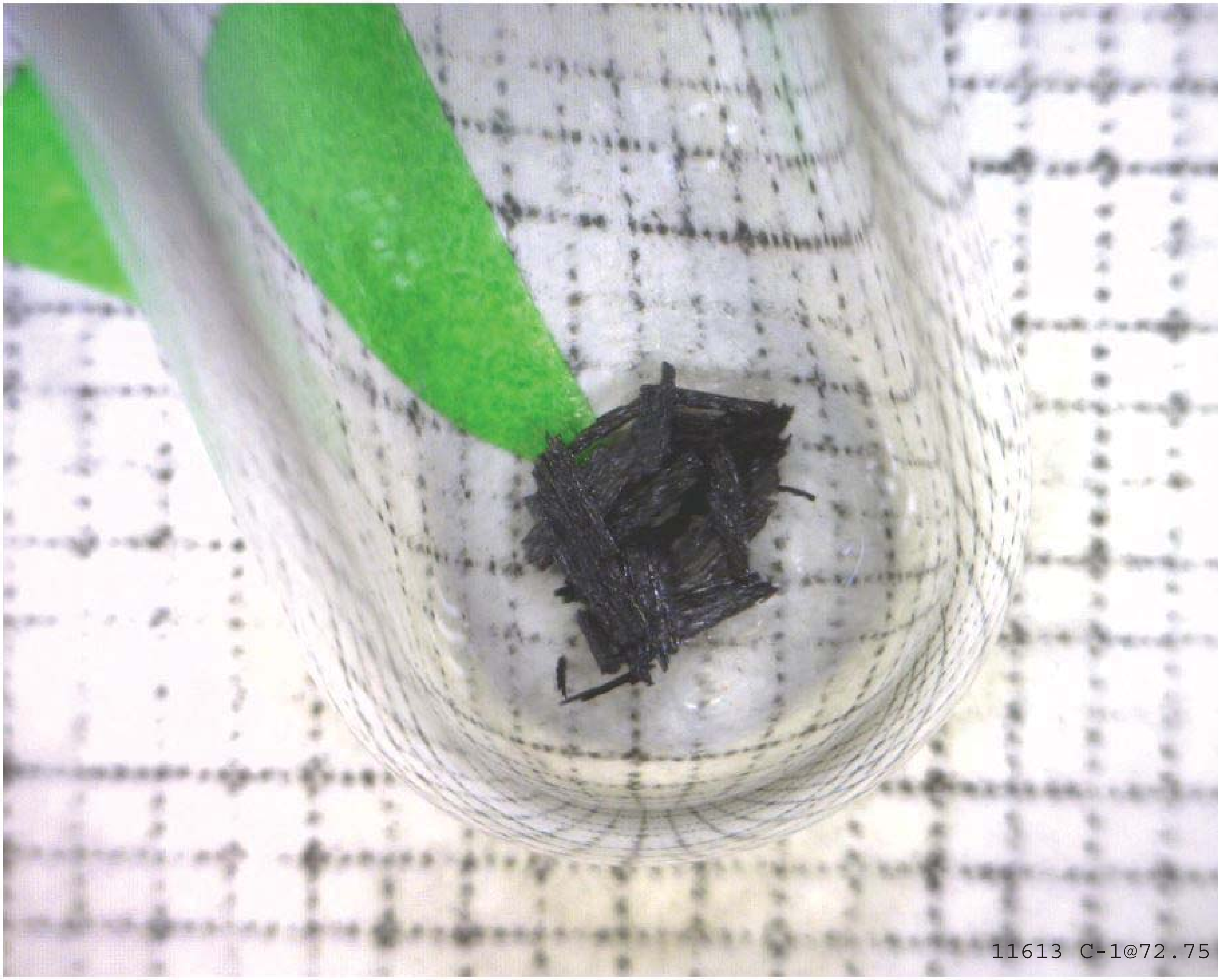


SAMPLE BAG

CLOSE ⊗

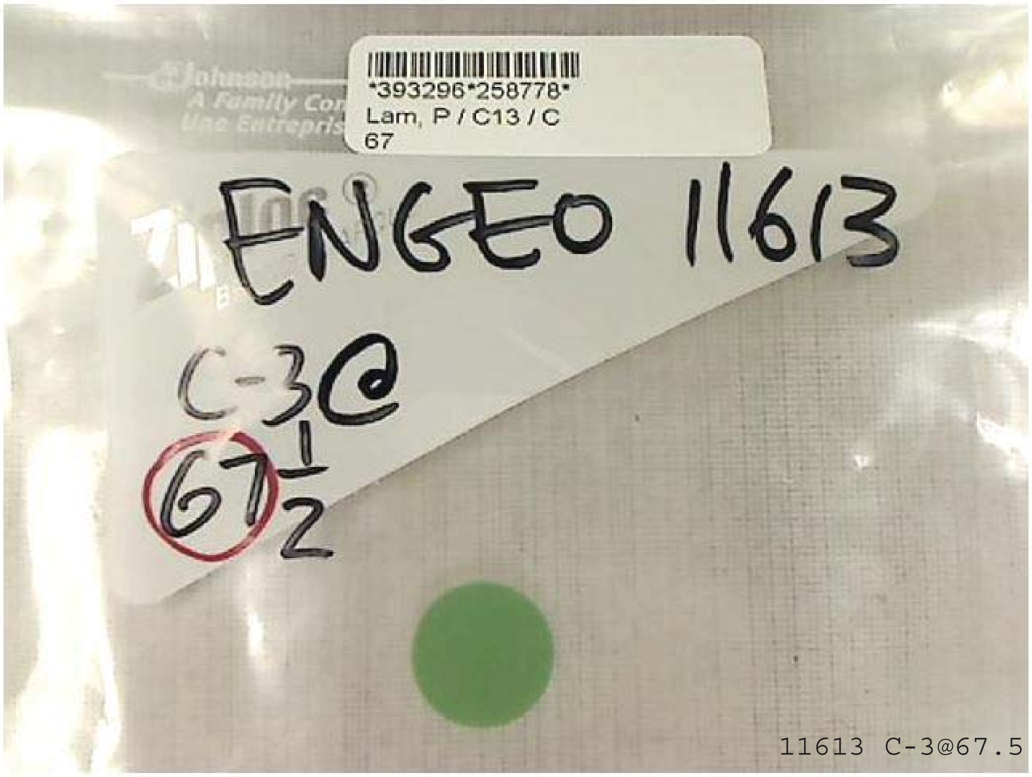
WHOLE SAMPLE

CLOSE ⊗



2.1MG ANALYZED (1MM X 1MM SCALE)

CLOSE ⓧ



11613 C-3@67.5

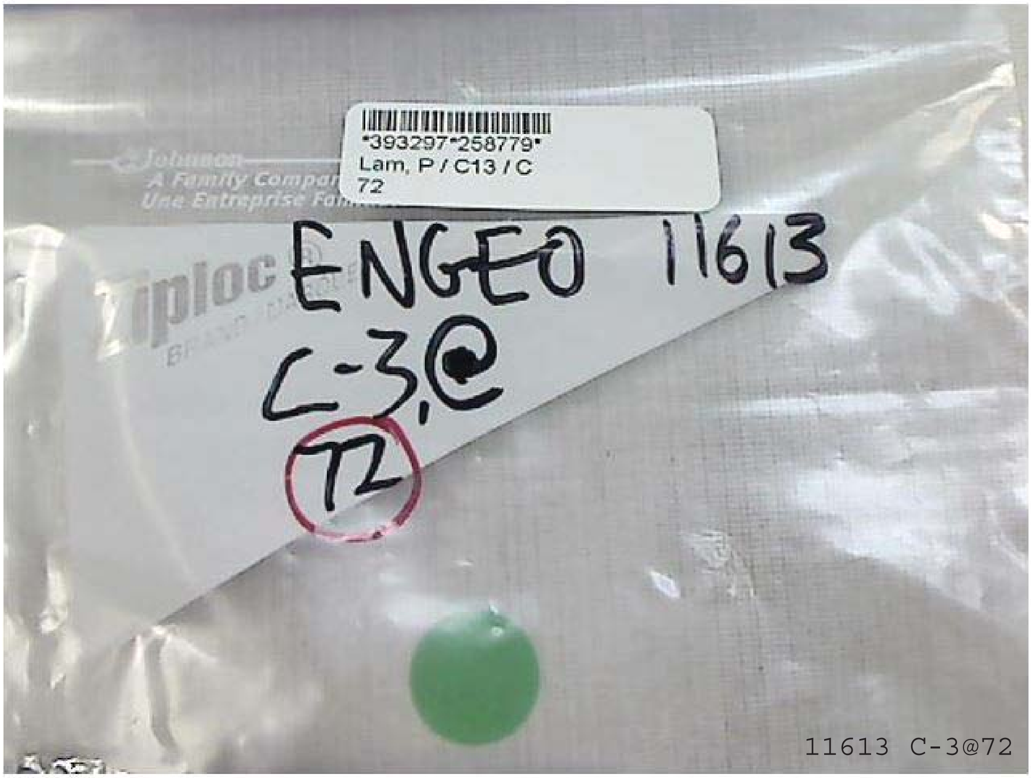
SAMPLE BAG

CLOSE ⊗


11613 C-3@67.5
WHOLE SAMPLE
CLOSE ⓧ

2.0 GRAMS ANALYZED (1MM X 1MM SCALE)

CLOSE ⊗

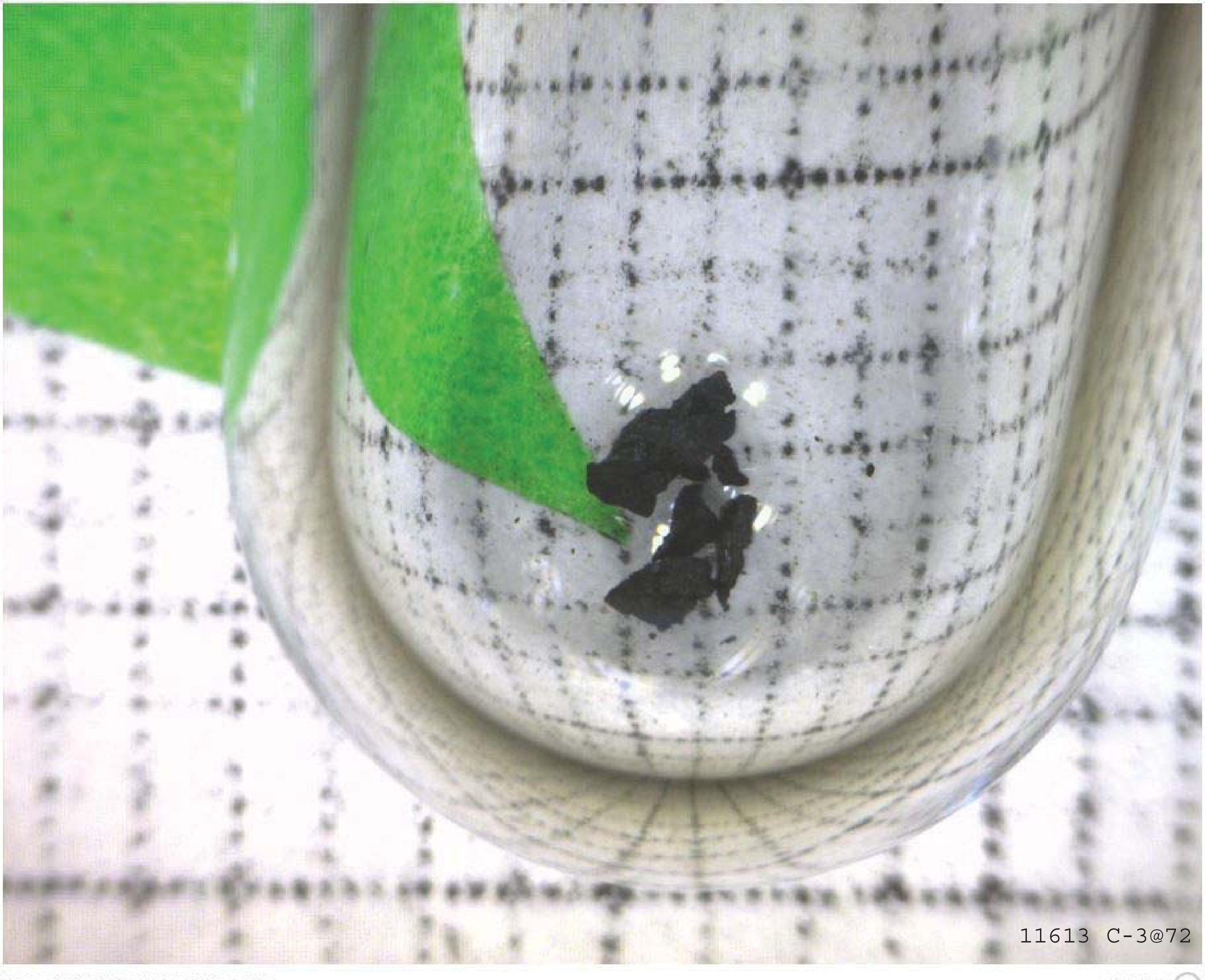


SAMPLE BAG

CLOSE ⊗

WHOLE SAMPLE

CLOSE ⊗



1.0MG ANALYZED (1MM X 1MM SCALE)

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www.radiocarbon.com

Darden Hood
President

Ronald Hatfield
Christopher Patrick
Deputy Directors

December 15, 2014

Mr. Patrick Lam
ENGEO Incorporated
2010 Crow Canyon Place, Suite 250
San Ramon, CA 94583-4634
USA

RE: Radiocarbon Dating Result For Sample 11613 A-C-4@73.5

Dear Mr. Lam:

Enclosed is the radiocarbon dating result for one sample recently sent to us. As usual, specifics of the analysis are listed on the report with the result and calibration data is provided where applicable. The Conventional Radiocarbon Age has been corrected for total fractionation effects and where applicable, calibration was performed using 2013 calibration databases (cited on the graph pages).

The web directory containing the table of results and PDF download also contains pictures, a cvs spreadsheet download option and a quality assurance report containing expected vs. measured values for 3-5 working standards analyzed simultaneously with your samples.

The reported result is accredited to ISO/IEC 17025:2005 Testing Accreditation PJLA #59423 standards and all pretreatments and chemistry were performed here in our laboratories and counted in our own accelerators here in Miami. Since Beta is not a teaching laboratory, only graduates trained to strict protocols of the ISO/IEC 17025:2005 Testing Accreditation PJLA #59423 program participated in the analysis.

As always Conventional Radiocarbon Ages and sigmas are rounded to the nearest 10 years per the conventions of the 1977 International Radiocarbon Conference. When counting statistics produce sigmas lower than +/- 30 years, a conservative +/- 30 BP is cited for the result.

When interpreting the result, please consider any communications you may have had with us regarding the sample. As always, your inquiries are most welcome. If you have any questions or would like further details of the analysis, please do not hesitate to contact us.

Thank you for prepaying the analyses. As always, if you have any questions or would like to discuss the results, don't hesitate to contact me.

Sincerely,

Darden Hood
Digital signature on file

REPORT OF RADIOCARBON DATING ANALYSES

Mr. Patrick Lam | Report Date: 12/15/2014

ENGEO Incorporated | Material Received: 11/25/2014

Sample Data	Measured Radiocarbon Age	13C/12C Ratio	Conventional Radiocarbon Age(*)
Beta - 397358	NA	NA	14240 +/- 50 BP

SAMPLE : 11613 A-C-4@73.5
ANALYSIS : AMS-Standard delivery
MATERIAL/PRETREATMENT : (charred material): acid/alkali/acid
2 SIGMA CALIBRATION : Cal BC 15535 to 15240 (Cal BP 17485 to 17190)

COMMENT: The original sample was too small to provide a 13C/12C ratio on the original material. However, a ratio including both natural and laboratory effects was measured during the 14C detection to calculate the true Conventional Radiocarbon Age.

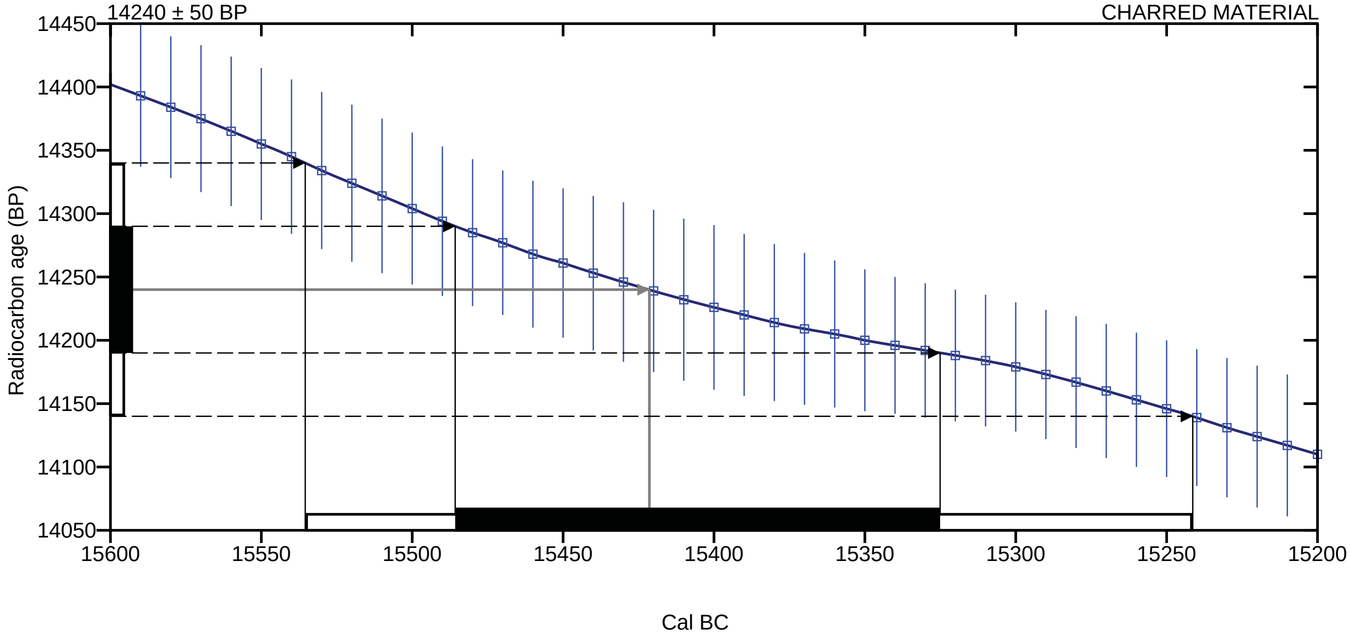
Dates are reported as RCYBP (radiocarbon years before present, "present" = AD 1950). By international convention, the modern reference standard was 95% the 14C activity of the National Institute of Standards and Technology (NIST) Oxalic Acid (SRM 4990C) and calculated using the Libby 14C half-life (5568 years). Quoted errors represent 1 relative standard deviation statistics (68% probability) counting errors based on the combined measurements of the sample, background, and modern reference standards. Measured 13C/12C ratios (delta 13C) were calculated relative to the PDB-1 standard.

The Conventional Radiocarbon Age represents the Measured Radiocarbon Age corrected for isotopic fractionation, calculated using the delta 13C. On rare occasion where the Conventional Radiocarbon Age was calculated using an assumed delta 13C, the ratio and the Conventional Radiocarbon Age will be followed by "*". The Conventional Radiocarbon Age is not calendar calibrated. When available, the Calendar Calibrated result is calculated from the Conventional Radiocarbon Age and is listed as the "Two Sigma Calibrated Result" for each sample.

CALIBRATION OF RADIOCARBON AGE TO CALENDAR YEARS

(Variables: C13/C12 = N/A : lab. mult = 1)

Laboratory number	**Beta-397358**
Conventional radiocarbon age	**14240 ± 50 BP**
2 Sigma calibrated result **95% probability**	**Cal BC 15535 to 15240 (Cal BP 17485 to 17190)**
Intercept of radiocarbon age with calibration curve	Cal BC 15420 (Cal BP 17370)
1 Sigma calibrated results 68% probability	Cal BC 15485 to 15325 (Cal BP 17435 to 17275)



Database used

INTCAL13

References

Mathematics used for calibration scenario

A Simplified Approach to Calibrating C14 Dates, Talma, A. S., Vogel, J. C., 1993, Radiocarbon 35(2):317-322

References to INTCAL13 database

Reimer PJ et al. IntCal13 and Marine13 radiocarbon age calibration curves 0– 50,000 years cal BP. Radiocarbon 55(4):1869– 1887., 2013.

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Mr. Darden Hood
President

Mr. Ronald Hatfield
Mr. Christopher Patrick
Deputy Directors

The Radiocarbon Laboratory Accredited to ISO/IEC 17025:2005 Testing Accreditation PJLA #59423

Quality Assurance Report

This report provides the results of reference materials used to validate radiocarbon analyses prior to reporting. Known value reference materials were analyzed quasi-simultaneously with the unknowns. Results are reported as expected values vs measured values. Reported values are calculated relative to NIST SRM-4990B and corrected for isotopic fractionation. Results are reported using the direct analytical measure percent modern carbon (pMC) with one relative standard deviation.

Report Date: December 16, 2014
Submitter : Mr. Patrick Lam

QA MEASUREMENTS

Reference 1
Expected Value: 27.4 +/- 0.2
Measured Value: 27.1 +/- 0.1 pMC
Agreement: Accepted

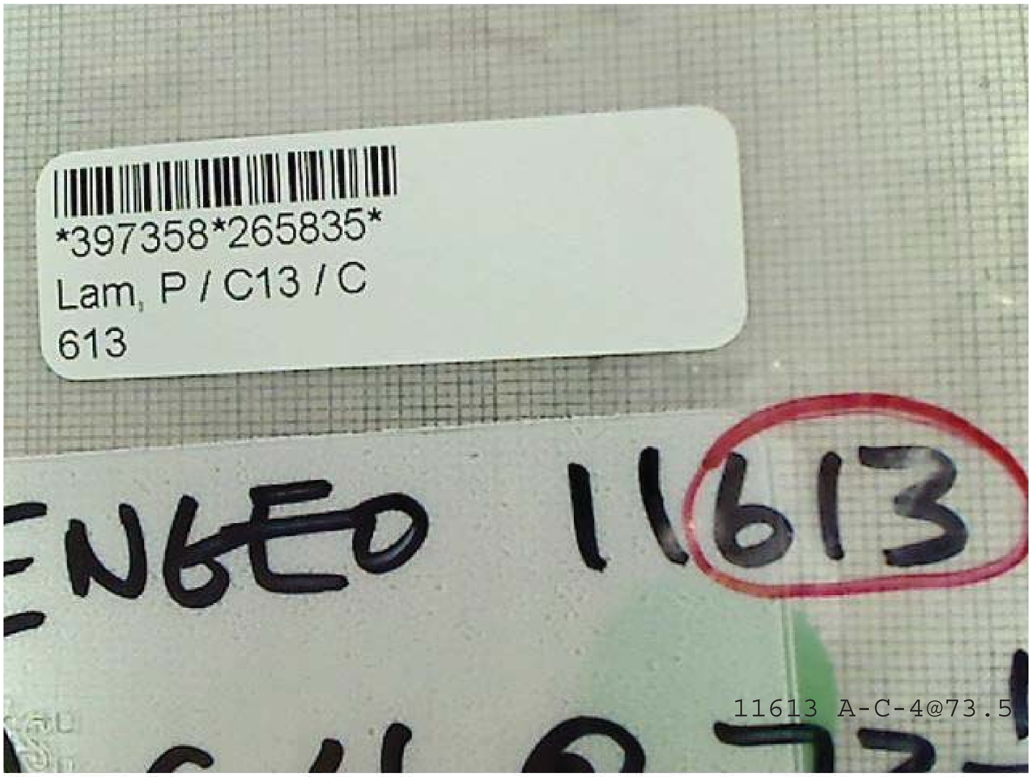
Reference 2
Expected Value: 4.4 +/- 0.2 pMC
Measured Value: 4.4 +/- 0.1 pMC
Agreement: Accepted

Reference 3
Expected Value: 96.8 +/- 0.5 pMC
Measured Value: 96.7 +/- 0.4 pMC
Agreement: Accepted

COMMENT: All measurements passed acceptance tests.

Validation: Darden Hood

Date: December 16, 2014



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613

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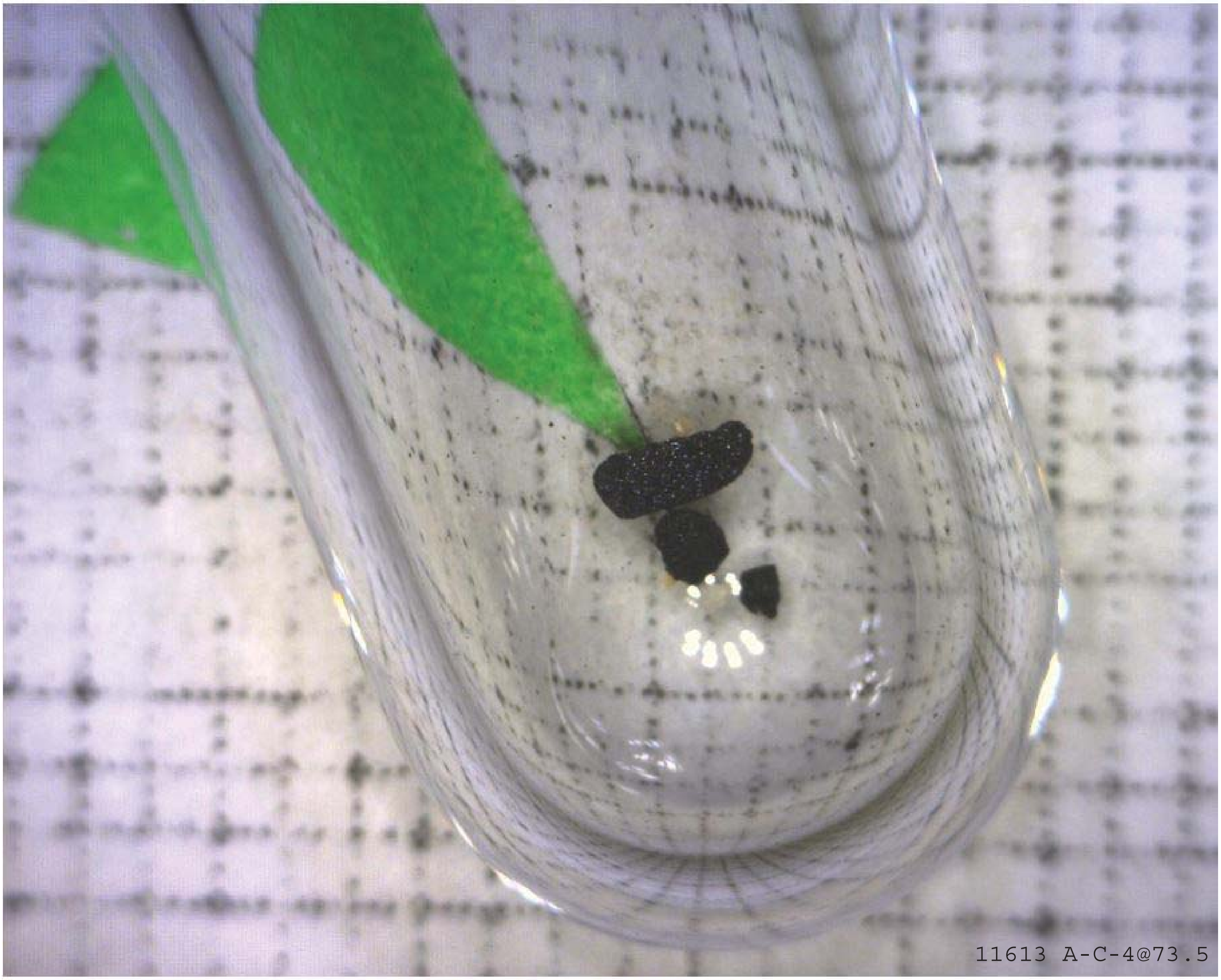
SAMPLE BAG

CLOSE

WHOLE SAMPLE

CLOSE ⊗



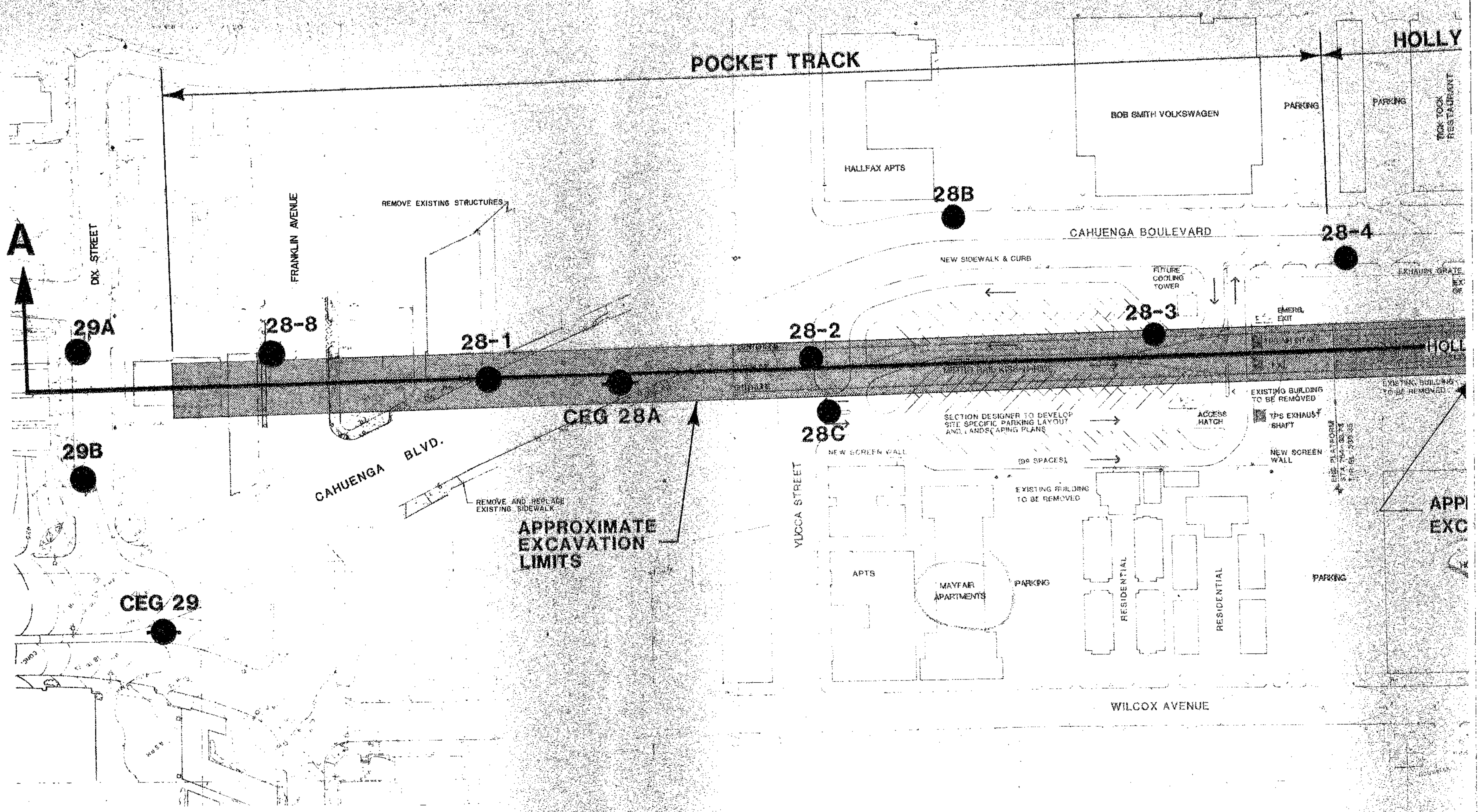
0.9MG ANALYZED (1MM X 1MM SCALE)

CLOSE ⊗

APPENDIX F

Historical Boring Logs and Cross Section

APPENDIX F



REFERENCES:

"PRELIMINARY HOLLYWOOD/CAHUENGA STATION SITE", DRAWING #C-17, SHEET 12 OF 24, PREPARED BY HARRY WEESE & ASSOCIATES, ORIGINAL SCALE 1"= 40' REDUCED TO 1"= 100', DATED 3-11-83.

"PRELIMINARY HOLLYWOOD/CAHUENGA STATION SITE", DRAWING #A-56, SHEET 10 OF 24, PREPARED BY HARRY WEESE & ASSOCIATES, ORIGINAL SCALE 1"= 40' REDUCED TO 1"= 100', DATED 5-11-83.

NOTES: 1.) FOR SUBSURFACE SECTION A-A' SEE DRAWING NO. 4
2.) FOR EXPLANATION OF SYMBOLS SEE DRAWING NO. 5

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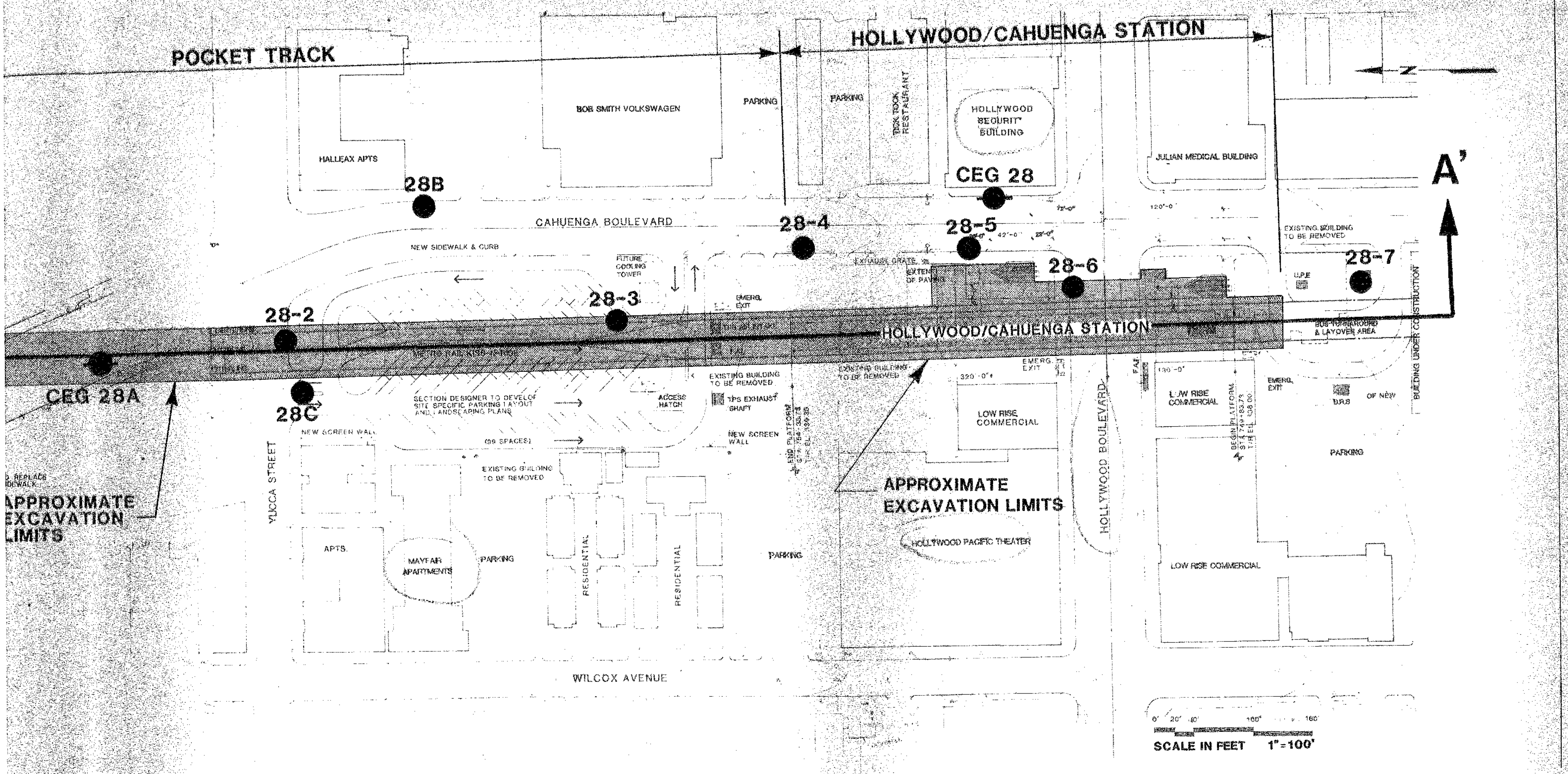


' 24, PREPARED BY
-11-83.

F 24, PREPARED BY
-11-83.

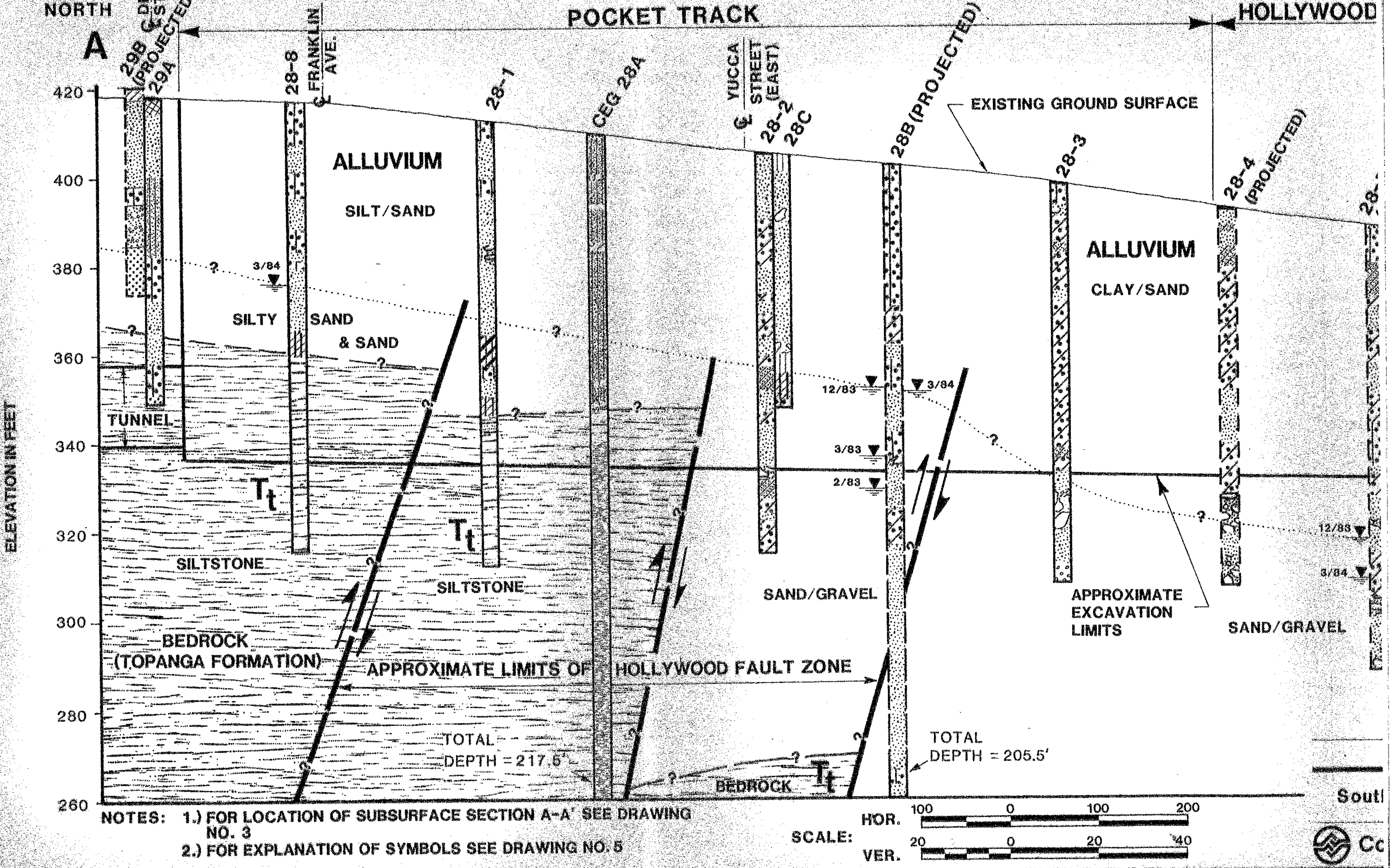
NO. 4
NO. 5

LOCATION OF BORINGS

DESIGN UNIT A350
Southern California Rapid Transit District
METRO RAIL PROJECT

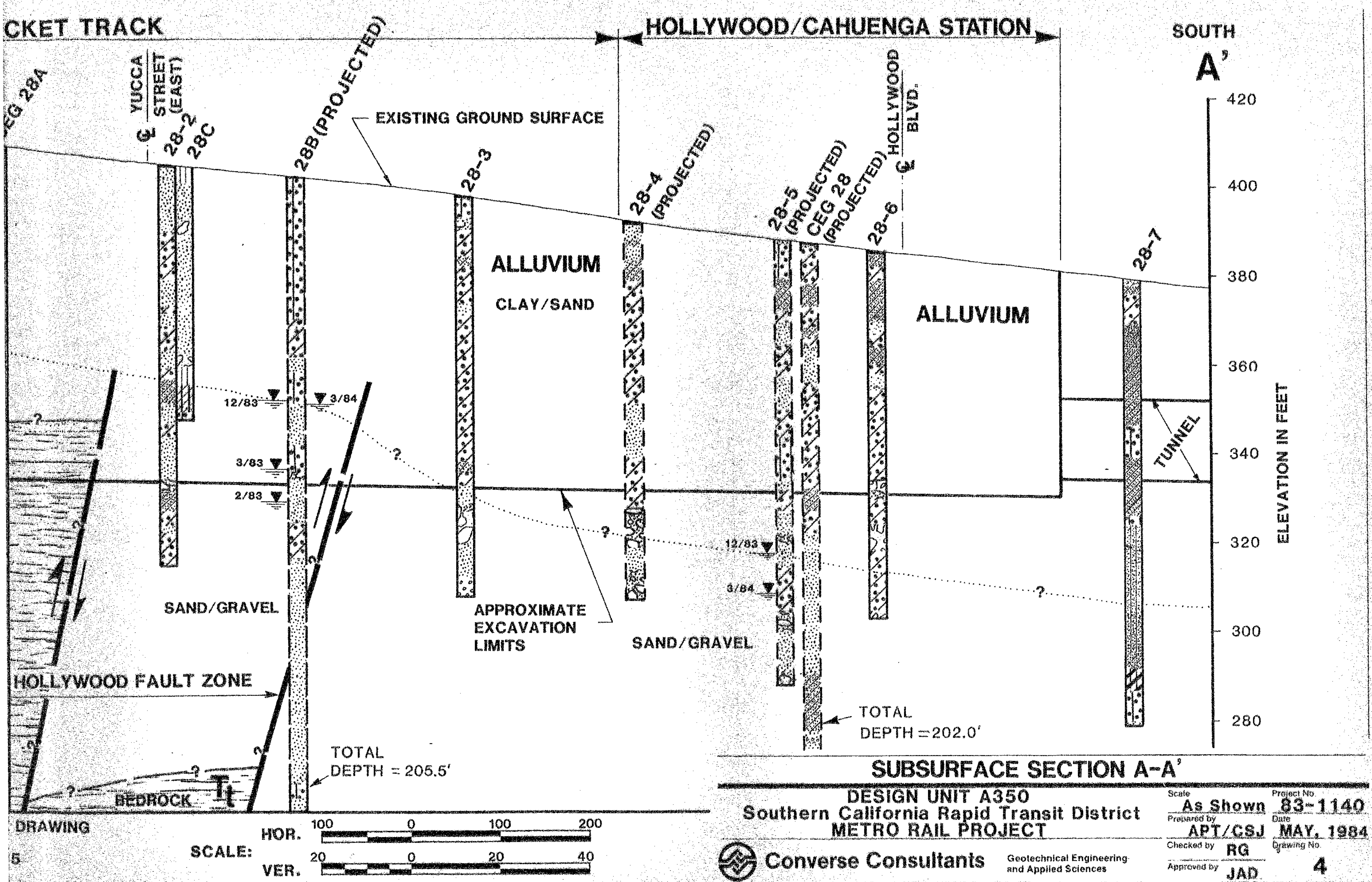
Converse Consultants
Geotechnical Engineering
and Applied Sciences

Scale As Shown	Project No
Date MAY, 1984	83-1140
Prepared by CSJ/RG	Drawing No
Checked by JAD	3
Approved By HAS	



NOTES: 1.) FOR LOCATION OF SUBSURFACE SECTION A-A' SEE DRAWING NO. 3

2.) FOR EXPLANATION OF SYMBOLS SEE DRAWING NO. 5



SUBSURFACE SECTION A-A'

DESIGN UNIT A350
Southern California Rapid Transit District
METRO RAIL PROJECT

Converse Consultants
Geotechnical Engineering and Applied Sciences

Scale As Shown	Project No. 83-1140
Prepared by APT/CSJ	Date MAY, 1984
Checked by RG	Drawing No. 4
Approved by JAD	

Converse Consultants, Inc.
Earth Sciences Associates
Geo/Resource Consultants

BORING LOG 28

Proj: DESIGN UNIT A-350
Date Drilled 1/5-7/81
Ground Elev. 385'
Drill Rig Failing 1500
Logged By L. Schoeberlein
Total Depth 202'
Hole Diameter 4 7/8"
Hammer Weight & Fall SS 140 lb @ 30" DR: 320 lbs @ 18"

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
0		0.0-1.2 ASPHALT			AD	Auger to 10'
2	SC	ALLUVIUM 1.2-9.0 CLAYEY SAND: dark yellowish brown; dry to moist; very loose; occasional fine gravel				
4						
6			J-1	2 1 2	SS	1.5/1.5 recovery
					AD	
8						
	CL	9.0-14.0 SANDY CLAY: dark yellowish brown; moist; stiff				
10			J-2	5 5 5	SS	1.3/1.5 recovery
12					RD	Rotary wash, 4 7/8" drag bit
		becoming more sandy				
14	SC	14.0-19.0 CLAYEY SAND: moderate yellowish brown; moist; loose				
16			J-3	3 3 3	SS	1.2/1.5 recovery
					RD	
18						
	CL	19.0-21.0 SANDY CLAY: moderate yellowish brown;				
20						

Sheet 1 of 9

Project DESIGN UNIT A-350 **Date Drilled** 1/5-7/81 **Hole No.** 28

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
20	CL	19.0-21.0 SANDY CLAY: (continued) wet; soft	C-1	2 2	DR	1.0/1.0 recovery
22	SC	21.0-23.0 CLAYEY SAND: moderate yellowish brown; wet; loose	J-4	5 3 6	SS	1.3/1.5 recovery
					RD	rig chatter
24	GP	23.0-24.0 GRAVEL: subangular to subrounded; fine to coarse				
	SP	24.0-31.0 SAND: moderate yellowish brown; dense; occasional gravel; wet	J-5	10 15 16	SS	0.7/1.5 recovery
26					RD	
28						
30			J	6 20 24	SS	0.0/1.5 recovery rock stuck in bit
32	SC	31.0-54.8 CLAYEY SAND: moderate yellowish brown; medium dense to dense; wet; occasional fine to coarse gravel			RD	
34						
36			J-6	9 11 12	SS	0.7/1.5 recovery 1/5/81 1/6/81 water at 15'
					RD	
38						
		becoming silty and dense	C-2	17 13	DR	0.7/1.0 recovery
40			J	17 19 20	SS	0.0/1.5 recovery
42					RD	
44						

Sheet 2 **of** 9

Project DESIGN UNIT A-350 Date Drilled 1/5-7/81 Hole No. 28

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
44	SC	31.0-54.8 CLAYEY SAND: (continued)			RD	
			J-7	8 11 11	SS	1.1/1.5 recovery
46						
					RD	
48						
50	(SP)	interbedded sand	J-8	9 13 13	SS	1.0/1.5 recovery
					RD	
52						
54	(GP)	- 54.5 thin gravel lens				
	CL	54.8-59.8 SANDY CLAY: moderate brown; moist; very stiff	J-9	5 8 8	SS	1.1/1.5 recovery
56						
					RD	
58						
			C-3	6 12	DR	0.7/1.0 recovery
60	SC	59.8-64.7 CLAYEY SAND: moderate yellowish brown; occasional gravel; moist; dense; interbeds of sandy clay and sand	J-10	11 17 17	SS	1.1/1,5 recovery
					RD	
62						
64						
	SP	64.7-96.5 SAND: moderate yellowish brown; moist; dense; occasional gravel	J-11	20 18 26	SS	1.1/1.5 recovery
66					RD	
68						

Sheet 3 of 9

Project DESIGN UNIT A-350 **Date Drilled** 1/5-7/81 **Hole No.** 28

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
68	SP	64.7-96.5 SAND: (continued)			RD	
70			J-12	29	SS	1.1/1.5 recovery
		becoming very dense		36		
				34		
72	(GW)	71.5-73.5' gravel lens			RD	chatter
74						
			J-13	30	SS	0.5/1.5 recovery
76				44		
				42		
					RD	
78						
				15		
	(SC)	moderate brown; clay increase	C-4	23	DR	0.7/1.0 recovery
80			J-14	37	SS	1.0/1.5 recovery
				35		
				40		
82		cobbles			RD	rig chatter @ 81.5', cemented sandstone in shoe of SPT
84						rig chatter
		weakly cemented; very dense	J-15	50	SS	0.25/0.25 recovery
					RD	
86						
		increased cementation				
88						
		moderate yellowish brown				
90			J-16	50	SS	0.2/0.2 recovery
92						

Sheet 4 **of** 9

Project DESIGN UNIT A-350 **Date Drilled** 1/5-7/81 **Hole No.** 28

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
92	SP	64.7-96.5 <u>SAND</u>: (continued)			RD	
	(CL)	93.0-94.0' sandy clay				
94						
	(GW)	gravel layer				
			J-17	50	SS	0.3/0.3 recovery
						intense chatter
96						
	CL	96.5-109.0 <u>SANDY CLAY</u>: moderate brown; moist; very stiff				
98						
			C-5	11 14	DR	1.0/1.0 recovery
100				9 11 16	SS	0.0/1.5 recovery
					RD	intermittent chatter
102						
104						
			J-18	18 18 23	SS	0.6/1.5 recovery
106					RD	
108						
						rig chatter
110	SP GW	109.0-113.5 <u>SAND/GRAVEL</u>: moderate brown; interbedded, sand with occasional well graded gravel	J-19	50 56	SS	0.7/1.0 recovery
					RD	
112						
114	CL	113.5-118.0 <u>SANDY CLAY</u>: moderate brown; interbedded with clayey sand; moist to wet; dense				
			J-20	18 28	SS	
116						

Sheet 5 **of** 9

Project DESIGN UNIT A-350 **Date Drilled** 1/5-7/81 **Hole No.** 28

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
116	CL	113.5-118.0 SANDY CLAY: (continued)	J-20	28	SS	1.1/1.5 recovery
					RD	
118	SC	118.0-125.0 CLAYEY SAND: moderate yellowish brown; moist; very dense; interbedded with sandy gravel and clayey gravel				
120			C-6	42 51	DR	1.0/1.0 recovery
			J-21	22 34 50	SS	1.2/1.3 recovery
122					RD	
124	(GW)	gravel lens				chatter
	SP	125.0-134.0 GRAVELLY SAND: moderate yellowish brown; moist to wet; very dense	J-22	62 51	SS	1.0/1.0 recovery
126					RD	
128						
						chatter
130			J-23	56	SS	0.2/0.5 recovery
					RD	
132						slight chatter
134	SC	134.0-156.0 CLAYEY SAND: moderate brown; occasional gravel; moist to wet; very dense				
			J-24	24 45 59	SS	1.3/1.5 recovery
136						
					RD	
		increasing clay with depth				
138						
			C-7	25 20	SS	1.0/1.0 recovery
140						

Sheet 6 of 9

Project DESIGN UNIT A-350 Date Drilled 1/5-7/81 Hole No. 28

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
140	SC	134.0-156.0 CLAYEY SAND: (continued)	J-25	41 50	SS	0.8/0.8 recovery
		becoming gravelly			RD	
142						
144						
			J-26	28 41 60	SS	1.3/1.5 recovery 1/6/81
146					RD	1/7/81
148						
150			J-27	43 50	SS	0.75/0.75 recovery
					RD	
152		becoming less clayey				
154						
			J-28	50	SS	
					RD	0.3/0.3 recovery
156	SP	156.0-178.6 SAND: moderate yellowish brown; moist; very dense; fine to coarse gravel				
158						
			C-8	65 68	DR	1.0/1.0 recovery
160		becoming silty	J	50	SS	0.0/0.25 recovery
					RD	intense chatter
162						
164						

Sheet 7 of 9

Project ____________________ Date Drilled ____________________ Hole No. __________

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
164	SP	156.0-178.6 SAND: (continued)			RD	
			J	50	SS	0.0/0.3 recovery
166						
	(SM)	167.0-168.5' silty sand				
168						
170		interbedded with fine sand	J-29	39 46 46	SS	1.4/1.5 recovery
					RD	
172						
174						
		clayey sand with occasional sand lenses	J	28 34 42	SS	0.0/1.5 recovery
176					RD	
178						
	SC	178.6-188.4 CLAYEY SAND: moderate yellowish brown; moist; very dense	C-9	42 50	DR	1.0/1.0 recovery
180			J-30	71 41 50	SS	1.0/1.3 recovery
		occasional gravel			RD	
182						chatter
184						
		thin gravel lenses	J	61	SS	0.0/0.5 recovery
186					RD	
188						

Sheet 8 of 9

Project DESIGN UNIT A-350 Date Drilled 1/5-7/81 Hole No. 28

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
188	SC	178.6-188.4 CLAYEY SAND: (continued)			RD	intense chatter
	SC GC	188.4-196.0 CLAYEY SAND/GRAVEL: moderate yellowish brown; moist; dense				
190			J	50	SS	0.0/0.25 recovery
						intense chatter
192						
194						
			J-31	50	SS	0.1/0.1 recovery
					RD	
196	CL	196.0-202.0 SANDY CLAY: moderate yellowish brown; moist; very stiff				
198						
200			C	100	DR	0.0/0.5 recovery
			J-32	58	SS	1.5/1.5 recovery
				51		
				36		
202		B.H. 202.0' Terminated hole; 1/7/81 downhole geophysical survey (GRC) 1/7/81 E-logs (ESA) 1/7/81 water at 75' 1/12/81 cased (4" PVC) and grouted to 100'				
204						
206						
208						
210						
212						

Sheet 9 of 9

THIS BORING LOG IS BASED ON FIELD CLASSIFICATION AND VISUAL SOIL DESCRIPTION, BUT IS MODIFIED TO INCLUDE RESULTS OF LABORATORY CLASSIFICATION TESTS WHERE AVAILABLE. THIS LOG IS APPLICABLE ONLY AT THIS LOCATION AND TIME. CONDITIONS MAY DIFFER AT OTHER LOCATIONS OR TIME.

Converse Consultants, Inc.
Earth Sciences Associates
Geo/Resource Consultants

BORING LOG 28-4

Proj: DESIGN UNIT A-350
Date Drilled 11/20/83
Ground Elev. 392'
Drill Rig Failing 1500
Logged By P. Moon
Total Depth 85.0'
Hole Diameter 4 7/8"
Hammer Weight & Fall SS140 lb @ 30", DR: 320 lbs @ 18"

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
0		0.0-0.4 ASPHALT CONCRETE			GB	
		0.4-1.0 GRAVEL BASE				
		ALLUVIUM				
	SP	1.0-3.5 SAND: moderate brown; wet; loose		4	DR	1.0/1.0 recovery
2			C-1	4		
					AD	
4	CH	3.5-6.0 SANDY CLAY: moderate greenish brown; wet; very soft	J-1	P P 2	SS	
6					RD	
	SW	6.0-9.0 SAND: moderate brown; wet; loose; trace of gravel		5	DR	
			C-2	13		1.0/1.0 recovery
8					RD	
	CL	9.0-13.5 SANDY CLAY: moderate yellowish brown; wet; stiff	J-2	1 3 6	SS	
10					RD	
12				6	DR	0.8/1.0 recovery
			C-3	12		
					RD	
14	SC	13.5-41.0 CLAYEY SAND: moderate yellowish brown; wet; medium dense	J-3	1 4 7	SS	
	SC & SM	with silty sand lenses				
16					RD	
				7	DR	1.0/1.0 recovery
			C-4	13		
18					RD	
		occasional fine gravel	J-4	9 11	SS	
20						

Sheet 1 of 4

Project DESIGN UNIT A-350 **Date Drilled** 11/20/83 **Hole No.** 28-4

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
20	SC	13.5-41.0 CLAYEY SAND: (continued) moderate yellowish brown; trace of gravel; wet; medium dense to dense	J-4	12	SS	
					RD	
22	SC & SM	with silty sand lenses		13	DR	0.7/1.0 recovery
			C-5	21		
					RD	
24			J-5	5 6 10	SS	
26					RD	
				20	DR	caliche nodule lodgec in drive shoe
28			C-6	46	RD	
			J-6	6 12 13	SS	
30					RD	
32				39	DR	1.0/1.0 recovery
			C-7	51		
					RD	
34			J-7	12 15 19	SS	
36					RD	
				33	DR	0.8/1.0 recovery
38			C-8	46		
					RD	
			J-8	14 8 10	SS	
40					RD	
	SW	41.0-53.0 SAND: moderate yellowish brown; wet; dense to very dense				
42				33	DR	1.0/1.0 recovery
			C-9	47	RD	
44						

Sheet 2 of 4

Project DESIGN UNIT A-350 Date Drilled 11/20/83 Hole No. 28-4

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
44	SW	41.0-53.0 SAND: (continued) moderate yellowish brown; wet; very dense	J-9	21 25 29	SS	
46					RD	
			C-10	23 29	DR	1.0/1.0 recovery
48		becoming silty			RD	
		thin interbeds of clayey fine sand	J-10	29 23 23	SS	
50					RD	
52			C-11	36 44	DR	1.0/1.0 recovery
					RD	
	SC	53.0-64.0 CLAYEY SAND: moderate yellowish brown; wet; medium dense				
54	SC & SM	with silty sand lenses	J-11	7 13 14	SS	
56					RD	
			C-12	18 37	DR	1.0/1.0 recovery
58					RD	
				10 11 19	SS	no recovery
60					RD	
62		scattered gravel	C-13	24 29	DR	1.0/1.0 recovery
					RD	
64	SW	64.0-73.5 GRAVELLY SAND: mottled yellowish brown and pinkish brown; wet; very dense; with silty sand	J-12	69	SS	
66					RD	
			C-14	72 105	DR	
68						

Sheet 3 of 4

Project DESIGN UNIT A-350 **Date Drilled** 11/20/83 **Hole No.** 28-4

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
68	SW	64.0-73.5 GRAVELLY SAND: (continued) moderate yellowish brown; wet; very dense			RD	
				72	SS	
70					RD	
72				76	DR	
			C-15	120		
					RD	
74	SW SM	73.5-78.0 SAND/SILTY SAND: moderate brown; wet; very dense; trace of gravel	J-13	33 40 51	SS	
76					RD	
			~~C-16~~	98 ~~98~~	DR	
78	GW	78.0-85.0 SANDY GRAVEL: moderate brown; wet; very dense			RD	
				15 9 17	SS	no recovery
80					RD	
82				200	DR	no recovery, cobble lodged in dirve shoe refusal at 5"
					RD	
84						
		B.H. 85.0' Terminated hole				
86						
88						
90						
92						

Sheet 4 of 4

THIS BORING LOG IS BASED ON FIELD CLASSIFICATION AND VISUAL SOIL DESCRIPTION, BUT IS MODIFIED TO INCLUDE RESULTS OF LABORATORY CLASSIFICATION TESTS WHERE AVAILABLE. THIS LOG IS APPLICABLE ONLY AT THIS LOCATION AND TIME. CONDITIONS MAY DIFFER AT OTHER LOCATIONS OR TIME.

Converse Consultants, Inc.
Earth Sciences Associates
Geo/Resource Consultants

BORING LOG 28-5

Proj: DESIGN UNIT A350
Date Drilled 11-19-83
Ground Elev. 387.5'
Drill Rig Failing 1500
Logged By P. Moon
Total Depth 100.0'
Hole Diameter 4 7/8"
Hammer Weight & Fall SS:140 lb, 30", DR 320 lbs. @ 18"

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
0		0.0-0.8 ASPHALT CONCRETE			GB	
		0.8-1.0 BASE ROCK				
		ALLUVIUM				
2	SM	1.0-8.5 SILTY SAND: moderate brown; moist; loose		9	DR	1.0/1.0 recovery
			C-1	9		
					AD	
4			J-1	2	SS	
				3		
				4		
6					RD	
				6	DR	1.0/1.0 recovery
			C-2	7		
8					RD	
	CL	8.5-16.0 SANDY CLAY: moderate yellowish brown; moist; stiff	J-2	4	SS	
10				4		
				5		
					RD	
12				4	DR	1.0/1.0 recovery
			C-3	7		
					RD	
14			J-3	4	SS	
				4		
				6		
					RD	
16	SC	16.0-18.5 CLAYEY SAND: moderate yellowish brown; moist; medium dense				
				9	DR	1.0/1.0 recovery
18			C-4	13		
					RD	
	SW	18.5-23.5 SAND: moderate yellowish brown;	J-4	10	SS	
20				12		

Sheet 1 of 5

Project DESIGN UNIT A350 Date Drilled 11-19-83 Hole No. 28-5

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
20	SW	18.5-23.5 SAND: continued medium dense to dense; wet	J-4	19	SS	
					RD	
22				113 30	DR	no recovery, refusal at 6-1/2"
					RD	
24	SC	23.5-31.0 CLAYEY SAND: moderate yellowish brown; wet; medium dense to dense		29 31 24	SS	no recovery
26					RD	
				31	DR	1.0/1.0 recovery
28			C-5	30		
					RD	
				10 13 16	SS	no recovery
30					RD	
	SW	31.0-38.0 SAND: moderate yellowish brown; wet; dense to very dense				
32				41	DR	1.0/1.0 recovery
			C-6	62		
					RD	
34			J-5	17 21 29	SS	
36					RD	
				41		0.8/1.0 recovery
			C-7	47		
38	SC	38.0-44.5 CLAYEY SAND: moderate yellowish brown; wet; dense			RD	
			J-6	12 13 17	SS	
40					RD	
42				21	DR	1.0/1.0 recovery
			C-8	24		6 rings
					RD	
44						

Sheet 2 of 5

Project DESIGN UNIT A350 **Date Drilled** 11-19-83 **Hole No.** 28-5

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
44	SC	38.0 -44.5 CLAYEY SAND: continued	J-7	21	SS	
	SM	44.5-53.5 SILTY SAND : moderate brown; wet; very dense		28		
				37		
					RD	
46						
				32	DR	1.0/1.0 recovery
			C-9	47		
48					RD	
			J-8	17	SS	
				18		
50				18		
					RD	
52				28	DR	1.0/1.0 recovery
			C-10	32		
					RD	
54	SW	53.5-56.0 SAND: moderate yellowish brown; wet; very dense; trace of gravel	J-9	22	SS	
				28		
				46		
					RD	
56	SC	56.0-61.0 CLAYEY SAND: moderate brown; wet; very dense				
				53	DR	1.0/1.0 recovery
			C-11	92		
58					RD	
			J-10	13	SS	
				30		
60				41		
					RD	
	SW	61.0-66.0 SAND: moderate brown; moist; very dense				
62				46	DR	1.0/1.0 recovery
			C-12	46		
					RD	
64		grading to GRAVELLY SAND	J-11	73	SS	refusal at 6"
66	GP	66.0-68.5 SANDY GRAVEL: wet; very dense				
				75	DR	0.6/1.0 recovery
68			C-13	75		

Project DESIGN UNIT A350 **Date Drilled** 11-19-83 **Hole No.** 28-5

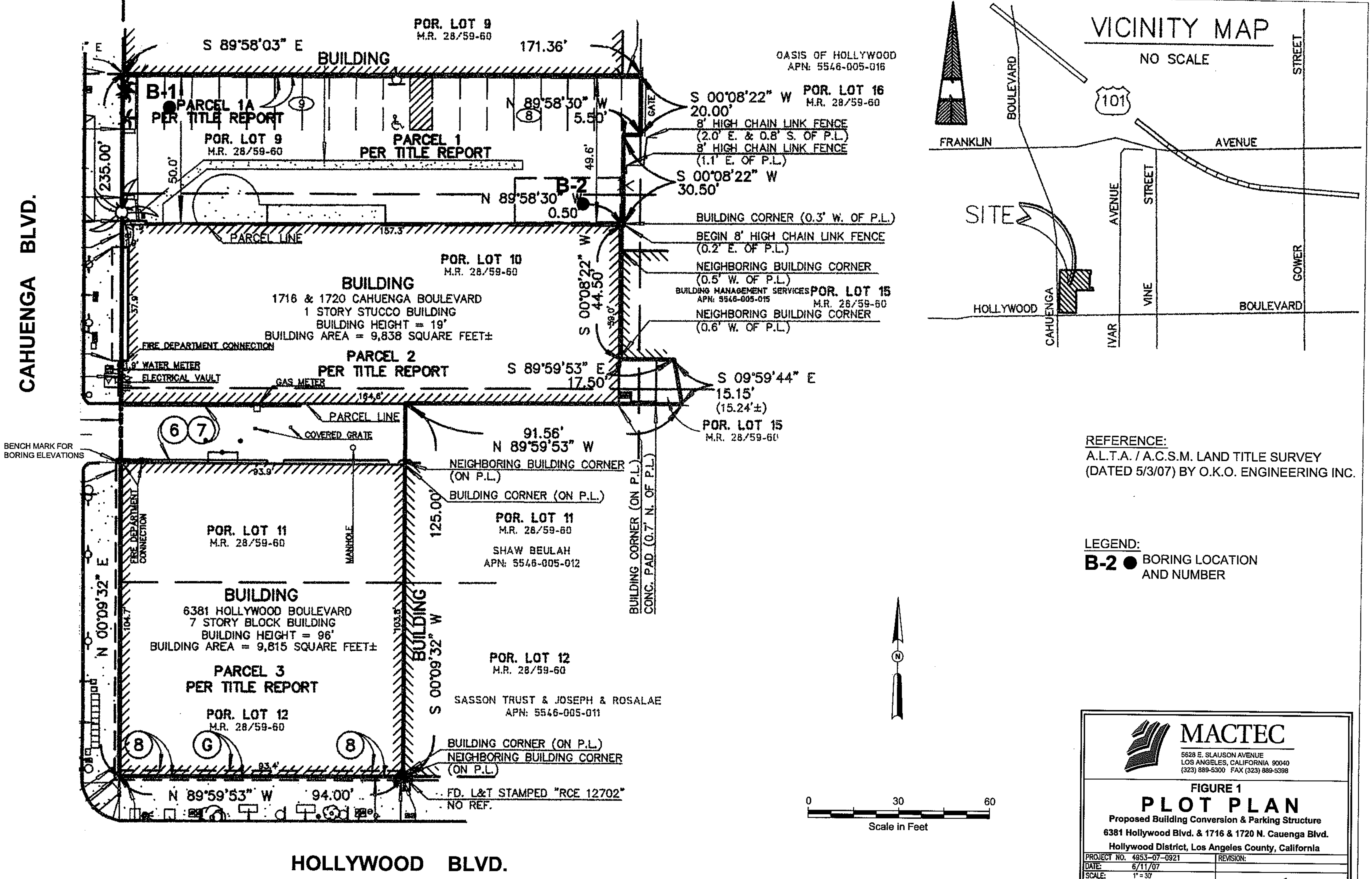
DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
68	GP	66.0-68.5 SANDY GRAVEL: continued			RD	
	SW	68.5-73.5 SAND: moderate yellowish brown; wet; very dense	J-12	62	SS	
					RD	
70						
72			C-14	130 60	DR	0.6/0.8 recovery refusal at 10"
					RD	
74	SM	73.5-83.5 SILTY SAND : moderate brown; wet; very dense	J-13	18 27 48	SS	
					RD	
76						
			C-15	29 35	DR	1.0/1.0 recovery
78					RD	
			J-11	7 11 18	SS	
80					RD	
82		scattered fine gravel	C-16	29 45	DR	1.0/1.0 recovery
					RD	
84	GW	83.5-88.0 SANDY GRAVEL: wet; very dense	J-15	76	SS	refusal at 6"
					RD	
86						
88	SW	88.0-97.0 SAND: wet; very dense				piezometer set at 100'
90						
92						

Sheet 4 of 5

Project DESIGN UNIT A350 Date Drilled 11-19-83 Hole No. 28-5

DEPTH	USCS	MATERIAL CLASSIFICATION	SAMPLE	BLOWS (6")	DRILL MODE	REMARKS
92	SW	88.0- 99.5 SAND: continued			RD	
94						
96						
	GW	grading gravelly				
98						
				32	DR	
	CL	99.5-100.0 SANDY CLAY: moderate yellowish brown; wet; very stiff	C-17	68		
100		B.O.H. 100'				following completion of drilling, prior to installation of piezometer, fluid level dropped to near T.D. = 100'
102						
104						
106						
108						
110						
112						
114						
116						

Sheet 5 of 5



REFERENCE:
A.L.T.A. / A.C.S.M. LAND TITLE SURVEY
(DATED 5/3/07) BY O.K.O. ENGINEERING INC.

LEGEND:
B-2 ● BORING LOCATION AND NUMBER

MACTEC
5628 E. SLAUSON AVENUE
LOS ANGELES, CALIFORNIA 90040
(323) 889-5300 FAX (323) 889-5398

FIGURE 1

PLOT PLAN

Proposed Building Conversion & Parking Structure
6381 Hollywood Blvd. & 1716 & 1720 N. Cauenga Blvd.
Hollywood District, Los Angeles County, California

PROJECT NO.	4953-07-0921	REVISION:
DATE:	6/11/07	
SCALE:	1" = 30'	
DWG BY:	T.T.	CHECKED BY:

B12SOIL CRANDALL(DECIMAL ELE) 70921.GPJ LAW CRAN.GDT 6/21/07

THIS RECORD IS A REASONABLE INTERPRETATION OF SUBSURFACE CONDITIONS AT THE EXPLORATION LOCATION. SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND AT OTHER TIMES MAY DIFFER. INTERFACES BETWEEN STRATA ARE APPROXIMATE. TRANSITIONS BETWEEN STRATA MAY BE GRADUAL.

BORING 1

DATE DRILLED: June 7, 2007
EQUIPMENT USED: Hollow Stem Auger
HOLE DIAMETER (in.) 8
ELEVATION: 385.4**

ELEVATION (ft)	DEPTH (ft)	"N" VALUE STD.PEN.TEST	MOISTURE (% of dry wt.)	DRY DENSITY (pcf)	BLOW COUNT* (blows/ft)	SAMPLE LOC.		
385							SM	3.5" thick Asphaltic Paving FILL - SILTY SAND - moist, light brown, fine to medium, occasional gravel
380	5		6.4	105	10	▩		Dark brown, trace gravel
							SM	SILTY SAND - moist, brown, fine to medium, some coarse
			8.0	101	11	▩		
375	10		12.8	105	15	▩		Alternating with layers of Sandy Silt
		8				☒		Sandier
370	15						SW	WELL GRADED SAND - moist, light brown, fine to coarse
			2.9	101	13	▩	SM	SILTY SAND - moist, light brown, fine to medium
365	20	18				☒	SW	WELL GRADED SAND with gravel - moist, light brown, fine to coarse
			12.6	102	10	▩	ML	SANDY SILT - moist, brown, fine to medium, some coarse
360	25						SM	SILTY SAND - moist, light brown, fine to medium
		16				☒		
			11.1	110	15	▩		Occasional gravel
355	30							
		14				☒		
350	35		7.0	110	19	▩		
		15				☒		Some gravel and cobbles
	40							

Field Tech: AR
Prepared By: HP
Checked By: LI

(CONTINUED ON FOLLOWING FIGURE)

B12SOIL_CRANDALL(DECIMAL_ELE) 70921.GPJ LAW_CRAN.GDT 6/21/07

THIS RECORD IS A REASONABLE INTERPRETATION OF SUBSURFACE CONDITIONS AT THE EXPLORATION LOCATION. SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND AT OTHER TIMES MAY DIFFER. INTERFACES BETWEEN STRATA ARE APPROXIMATE. TRANSITIONS BETWEEN STRATA MAY BE GRADUAL.

BORING 1 (Continued)

DATE DRILLED: June 7, 2007
EQUIPMENT USED: Hollow Stem Auger
HOLE DIAMETER (in.) 8
ELEVATION: 385.4**

ELEVATION (ft)	DEPTH (ft)	"N" VALUE STD.PEN.TEST	MOISTURE (% of dry wt.)	DRY DENSITY (pcf)	BLOW COUNT* (blows/ft)	SAMPLE LOC.	
345			9.8	102	16	▩	
		11				☒	Thin Sandy Silt layer
340	45						
			9.2	113	28	▩	
335	50	15				☒	Thin Sandy Silt layer
330	55						
325	60						
320	65						
315	70						
310	75						
	80						

END OF BORING AT 50.5 FEET

NOTES:

Ground water not encountered. Boring backfilled with soil cutting and tamped.

* Number of blows required to drive the Crandall sampler 12 inches using a 140 pounds hammer falling 30 inches.

** Elevation surveyed based on bench mak location shown on Figure 1.
(Elevation of bench mark = 382.2').

Field Tech: AR
Prepared By: HP
Checked By: LT

Building Conversion & Parking Structure Hollywood District, California

MACTEC

LOG OF BORING

Project: 4953-07-0921

Figure: A-1.1b

BORING 2

DATE DRILLED: June 7, 2007
EQUIPMENT USED: Hollow Stem Auger
HOLE DIAMETER (in.) 8
ELEVATION: 384.7**

THIS RECORD IS A REASONABLE INTERPRETATION OF SUBSURFACE CONDITIONS AT THE EXPLORATION LOCATION. SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND AT OTHER TIMES MAY DIFFER. INTERFACES BETWEEN STRATA ARE APPROXIMATE. TRANSITIONS BETWEEN STRATA MAY BE GRADUAL.

ELEVATION (ft)	DEPTH (ft)	"N" VALUE STD.PEN.TEST	MOISTURE (% of dry wt.)	DRY DENSITY (pcf)	BLOW COUNT* (blows/ft)	SAMPLE LOC.		
							SM	3" thick Asphaltic Paving FILL - SILTY SAND - moist, light brown, fine to medium, brick fragments (up to 3" in size)
380	5							
			14.9	90	5	▨		
							SM	SILTY SAND - moist, brown, fine to medium, some coarse
			9.4	106	8	▨		
375	10							
		7				☒		Layer of Sandy Silt - moist, brown, fine to medium
								Occasional small gravel
			11.2	102	10	▨		
370	15							
		4				☒		Thin layer of fine to coarse sand Light browish-yellow, fine to coarse
365	20		7.0	106	10	▨		
		9				☒		
360	25							
			10.8	102	10	▨		Thin layer of Silt
		11				☒		
355	30							
			11.7	106	14	▨		
350	35	10				☒		
			9.6	113	19	▨		
345	40							

(CONTINUED ON FOLLOWING FIGURE)

Field Tech: AR
Prepared By: HP
Checked By: LT

Building Conversion
& Parking Structure
Hollywood District, California

MACTEC

LOG OF BORING
Project: 4953-07-0921 Figure: A-1.2a

B12SOIL CRANDALL(DECIMAL ELE) 70921.GPJ LAW CRAN.GDT 6/21/07

BORING 2 (Continued)

DATE DRILLED: June 7, 2007
EQUIPMENT USED: Hollow Stem Auger
HOLE DIAMETER (in.) 8
ELEVATION: 384.7**

THIS RECORD IS A REASONABLE INTERPRETATION OF SUBSURFACE CONDITIONS AT THE EXPLORATION LOCATION. SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND AT OTHER TIMES MAY DIFFER. INTERFACES BETWEEN STRATA ARE APPROXIMATE. TRANSITIONS BETWEEN STRATA MAY BE GRADUAL.

ELEVATION (ft)	DEPTH (ft)	"N" VALUE STD.PEN.TEST	MOISTURE (% of dry wt.)	DRY DENSITY (pcf)	BLOW COUNT* (blows/ft)	SAMPLE LOC.	
		10				☒	Layer of Silt
			11.1	117	13	▩	
340	45						
		18				☒	
335	50		12.1	110	15	▩	END OF BORING AT 50 FEET
330	55						
325	60						
320	65						
315	70						
310	75						
305	80						

NOTES:

Ground water not encountered. Boring backfilled with soil cuttings and tamped.

Field Tech: AR
Prepared By: HP
Checked By: LT

B12SOIL CRANDALL(DECIMAL ELE) 70921.GPJ LAW CRAN.GDT 6/21/07

Building Conversion
& Parking Structure
Hollywood District, California

MACTEC

LOG OF BORING
Project: 4953-07-0921 Figure: A-1.2b

BOARD OF
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LOS ANGELES, CA 90012

RAYMOND S. CHAN, C.E., S.E.
GENERAL MANAGER

FRANK BUSH
EXECUTIVE OFFICER

GEOLOGY REPORT APPROVAL LETTER

March 26, 2015

LOG # 87442
SOILS/GEOLOGY FILE - 2
AP

SPBB LLC
501 N.W. Grand Boulevard
Oklahoma City, OK 73118

TRACT:	Hollywood (MR 148-5A187)
BLOCK:	20
LOT(S):	12 (Arb 3), 11 (Arb 3), 10 (Arbs 1 & 2) & 9 (Arb 1)
LOCATION:	6831 Hollywood Blvd., 1716-1726 N. Cahuenga Blvd.

CURRENT REFERENCE REPORT/LETTER(S)	REPORT No.	DATE(S) OF DOCUMENT	PREPARED BY
Geology Report	11613.000.000	01/23/2015	ENGEO Incorporated
Oversized Doc(s).	``	``	``

The Grading Division of the Department of Building and Safety has reviewed the referenced report that presents a fault rupture hazard investigation at 6831 Hollywood and 1716-1726 N. Cahuenga Boulevards for the future devolvement of the property. The site consists of several contiguous lots currently occupied by the Security Pacific Bank Building and two other commercial buildings and parking lots.

The property is located within an Alquist-Priolo (AP) Official Earthquake Fault Zone that was established (November 6, 2014) by the California Geological Survey for the Hollywood fault (on the USGS 7.5 minute Hollywood Quadrangle). The exploration consisted of a transect of CPT soundings and continuous core borings that extended about 80 feet north of the property. The southern edge of the property roughly coincides with the southern edge of the AP zone so there was no need to extend the transect to the south. Radio carbon dating was used to age-date select samples from the continous core borings. Data from previous investigations by Converse and Mactec were also used for the geologic analysis of the site.

The investigation documents overlapping continuous stratigraphy of Holocene "younger alluvial fan" (designated Qyf in the report) and Pleistocene "older" alluvial fan (designated as Qof). The age of the radio carbon dates obtained from the Qof ranged from about 13,000 to 17,000 years. No faults were observed within the Qof strata. Therefore, no building restrictions are recommended by ENGEO.

The referenced report is acceptable, provided the following conditions are complied with during site development:

(Note: Numbers in parenthesis ( ) refer to applicable sections of the 2014 City of LA Building Code. P/BC numbers refer the applicable Information Bulletin. Information Bulletins can be accessed on the internet at LADBS.ORG.)

1. Prior to issuance of any permit, a soil engineering report shall be submitted to the Grading Division to provide design recommendations for the proposed grading/construction.

2. During construction, the project engineering geologist shall observe all excavations that expose the natural alluvial soils. The project engineering geologist shall post a notice on the job site for the City Grading Inspector and the Contractor stating that the excavation (or portion thereof) has been observed and documented and meets the conditions of the report. No fill or lagging shall be placed until the LADBS Grading Inspector has verified the documentation.

3. A supplemental report that summarizes the geologist's observations (including photographs and simple logs of excavations) shall be submitted to the Grading Division of the Department upon completion of the excavations. If evidence of active faulting is observed, the Grading Division shall be notified immediately. (Code Section 91.7009)

DANIEL C. SCHNEIDEREIT
Engineering Geologist I

DCS/dcs
Log No. 87442
213-482-0480

cc: ENGEO Incorporated, Project Consultant
LA District Office

Appendix IS-2.2

Update of Geotechnical Investigation

GEOCON
WEST, INC.
GEOTECHNICAL ■ ENVIRONMENTAL ■ MATERIALS

Project No. W1063-06-01
May 18, 2021

Mr. Ben Spector
Onni Group
315 West 9th Street, Suite 801
Los Angeles, California 90015

Subject: UPDATE OF GEOTECHNICAL INVESTIGATION
PROPOSED HIGH-RISE DEVELOPMENT
1708-1732 NORTH CAHUENGA BOULEVARD
6381-6385 WEST HOLLYWOOD BOULEVARD
LOS ANGELES, CALIFORNIA
TRACT: HOLLYWOOD, BLOCK: 20, LOTS 8-12

Reference: *Geotechnical Investigation,* prepared by Geocon West, Inc., Project No. W1063-06-01, October 17, 2019.

Dear Mr. Spector:

At your request, this letter has been prepared to in support of the project Initial Study document. Based on the updated project description provided to us, it is our understanding that the development will consist of a 14-story tower with a maximum height of 213 feet underlain by 8 subterranean levels, one ground floor parking level, and one above grade paring level. Based on our site exploration and analyses presented in the referenced Geotechnical Report dated October 17, 2019, it is the opinion of the undersigned engineer that the proposed project is feasible from a geotechnical perspective. However, the recommendations provided in the referenced Geotechnical Report will need to be reviewed and updated based on design-level plans when available.

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.

REGISTERED PROFESSIONAL ENGINEER
JELISA THOMAS ADAMS
GE3092
GEOTECHNICAL
STATE OF CALIFORNIA

Jelisa Thomas Adams
GE 3092

(EMAIL) Addressee