Appendix C

Geotechnical Report

GEOTECHNICAL INVESTIGATION

PROPOSED MIXED-USE DEVELOPMENT
8920 SUNSET BOULEVARD
WEST HOLLYWOOD, CALIFORNIA

GEOCON
WEST, INC.

GEOTECHNICAL
ENVIRONMENTAL
MATERIALS

PREPARED FOR

8920 SUNSET BOULEVARD LLC
NEW YORK, NEW YORK

PROJECT NO. A9286-06-01

JULY 2, 2015

GEOCON
WEST, INC.
GEOTECHNICAL ■ ENVIRONMENTAL ■ MATERIALS

Project No. A9286-06-01
July 2, 2015

8920 Sunset Boulevard LLC
c/o VE Equities
250 Bowery, 2nd Floor
New York, New York 1002

Attention: Mr. Zach Vella

Subject: GEOTECHNICAL INVESTIGATION
PROPOSED MIXED-USE DEVELOPMENT
8920 SUNSET BOULEVARD,
WEST HOLLYWOOD, CALIFORNIA

Dear Mr. Vella,

In accordance with your authorization of our proposal dated April 30, 2015, we have performed a geotechnical investigation for the proposed mixed-use development located at 8920 Sunset Boulevard in the West Hollywood District 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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TABLE OF CONTENTS

TABLE OF CONTENTS (Continued)

GEOTECHNICAL INVESTIGATION

1. PURPOSE AND SCOPE

This report presents the results of a geotechnical investigation for the proposed mixed-use development located at 8920 Sunset Boulevard in the West Hollywood District 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 scope of this investigation included a site reconnaissance, a review of prior reports in the project vicinity, field exploration, laboratory testing, engineering analysis, and the preparation of this report. The site was explored on May 30, 2015, by excavating two 8-inch diameter borings to depths of approximately 3 and 75½ feet below the existing ground surface using a truck-mounted hollow-stem auger drilling machine. The approximate locations of the exploratory borings are depicted on the Site Plan (see Figure 2). A detailed discussion of the field investigation, including boring logs, is presented in Appendix A.

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

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

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

2. SITE AND PROJECT DESCRIPTION

The subject site is located at 8920 Sunset Boulevard in the City of West Hollywood, California. The site is a rectangular shaped parcel and is currently occupied by a two-story commercial building with two levels of subterranean parking. Paved surface parking and a driveway are present at the rear of the existing building in the southern portion of the site. The site is bounded by Sunset Boulevard to the north, by a single-story commercial structure to the east, by Hilldale Avenue to the west, and by a three-story multi-family residential structure with a partial subterranean parking level to the south. Topography at the site slopes to the south-southeast and elevations at the site range from approximately 374 feet MSL (at the northwest corner of the site) to approximately 355 feet MSL (the southeast corner of the site). The natural topography in the immediate site vicinity slopes to the south at a gradient of approximately 6:1 (horizontal to vertical) or flatter. Surface water drainage at the site appears to be by sheet flow along the existing ground contours to the city streets. Vegetation onsite consists of shrubs and trees located in isolated planter areas.

Based on the information provided by the Client, it is our understanding that the proposed development will consist of demolishing the existing site structure and constructing an eight-story mixed-use structure constructed over four levels of subterranean parking. Due to the sloping nature of the site, it is anticipated that the subterranean parking will extend to depths of approximately 50 feet along Sunset Boulevard and to depths of approximately 30 feet along the south property line. The proposed development is depicted on the Site Plan and Cross-Section (see Figures 2 and 3).

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 residential structures will be up to 1,000 kips, and wall loads will be up to 10 kips per linear foot.

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

3. GEOLOGIC SETTING

The site is located along the northern margin of the Los Angeles Basin on a steep-sloping alluvial fan at the base of the southern flank of the Santa Monica Mountains. The Los Angeles Basin is a coastal plain between the Santa Monica Mountains to the north, the Puente Hills and Whittier Fault to the east, the Palos Verdes Peninsula and Pacific Ocean to the west and south, and the Santa Ana Mountains and San Joaquin Hills to the southeast. The basin is underlain by a deep structural depression which has been filled by both marine and continental sedimentary deposits that are underlain by a basement complex of igneous and metamorphic composition (Yerkes et al., 1965).

Regionally, the site is located in the southern portion of the Transverse Ranges geomorphic province, near the boundary of the Peninsular Ranges geomorphic province to the south. The Transverse Ranges is characterized by east-west trending geologic structures in contrast to the Peninsular Ranges that is characterized by northwest-trending geologic structures. The boundary between the two geomorphic provinces is a system of faults that include the active Malibu Coast, Santa Monica, Hollywood, Raymond, and Sierra Madre Fault zones. Based on published geologic maps, splays of the Hollywood fault zone are located approximately 150 feet south of the site (City of West Hollywood, 2011; City of West Hollywood, 2010).

4. SOIL AND GEOLOGIC CONDITIONS

Based on our field investigation and published geologic maps of the area, the site is underlain by artificial fill and Quaternary age alluvial fan deposits consisting of varying amounts of unconsolidated to moderately consolidated clayey sand and sandy clay originating from the Santa Monica Mountains to the north (Dibblee 1991, California Geological Survey, 2010). A previous geotechnical investigation was performed on the adjacent site to the west (8950 Sunset Boulevard) by GeoDesign, Inc. (GeoDesign, 2006). The prior report includes ten borings that were conducted by GeoDesign and three prior consultants. The borings indicate alluvial fan deposits consist of alternating layers of silty sand and clayey sand with lesser amounts of sandy clay that extend to depths of approximately 80 to 100 feet beneath the existing ground surface. Based on the logs from the GeoDesign (2006) report, the Holocene age alluvial deposits are estimated to be approximately 15 to 20 feet thick and are underlain by Pleistocene age alluvial fan deposits. The alluvial fan deposits are underlain by Cretaceous age quartz diorite bedrock.

Detailed stratigraphic profiles are provided on the boring logs in Appendix A. The logs from the GeoDesign (2006) report are included in Appendix C.

4.1 Artificial Fill

Artificial fill was encountered in our field explorations to a maximum depth of 9 feet below existing ground surface. The artificial fill generally consists of yellowish brown silty sand. The artificial fill is characterized as slightly moist and medium dense. The fill is likely the result of past grading or construction activities at the site. Deeper fill may exist between excavations and in other portions of the site that were not directly explored.

4.2 Alluvial Fan Deposits

Quaternary age alluvial fan deposits were encountered beneath the fill and consist primarily of yellowish brown to dark reddish brown poorly graded sand, sandy silt, clayey sand, and sand with clay. The alluvial soils are primarily slightly moist to wet and loose to very dense.

5. GROUNDWATER

Based on a review of the Seismic Hazard Zone Report for the Beverly Hills 7.5 Minute Quadrangle, Los Angeles County, California (California Division of Mines and Geology [CDMG], 1998), the historically highest groundwater level in the area is approximately 22 feet beneath the ground surface. The City of West Hollywood also reports that the historic high groundwater level in the vicinity of the site is at a depth of approximately 22 feet beneath the existing ground surface (City of West Hollywood, 2010).

Groundwater was encountered in boring B2 at a depth of 38 feet below the existing ground surface. In addition, groundwater levels encountered during previous investigations on the adjacent 8950 Sunset Boulevard site are summarized in the following table.

Summary of Groundwater Levels 8950 Sunset Boulevard

Boring Number	Drilled/Measured By	Date	Depth to Groundwater (feet)	Groundwater Elevation (above MSL)
B-1	Kovacs-Byer	August 1986	38.0	337.0
B-2	Kovacs-Byer	August 1986	29.0	335.0
B-3	Kovacs-Byer	August 1986	36.0	335.0
B-4	Kovacs-Byer	April 1991	46.0	335.0
B-5	Jerry Kovacs	November 1999	53.0	331.0
B-6	Jerry Kovacs	November 1999	34.0	331.0
B-7	Geotechnologies	June 2005	30.0	350.0
B-8	Geotechnologies	June 2005	22.5	343.5
B-9	GeoDesign	April 2006	37.5	339.5
B-10	GeoDesign	April 2006	27.0	338.0

Considering the historic high groundwater level, the depths groundwater encountered in previous borings on the adjacent site (8950 Sunset), and the depth to groundwater as encountered in our boring 2 drilled for this investigation, we anticipate that groundwater will be encountered during construction. Also, it is not uncommon for groundwater levels to vary seasonally or for groundwater seepage conditions to develop where none previously existed, especially in impermeable fine-grained soils which are heavily irrigated or after seasonal rainfall. In addition, recent requirements for stormwater infiltration could result in shallower seepage conditions in the immediate site vicinity. Proper surface drainage of irrigation and precipitation will be critical for future performance of the project. Recommendations for drainage are provided in the S*urface Drainage* section of this report (see Section 7.27).

6. GEOLOGIC HAZARDS

6.1 Surface Fault Rupture

6.1.1 General

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 California Division of Mines and Geology [CDMG]) for the Alquist-Priolo Earthquake Fault Zone Program (Bryant and Hart, 2007). By CGS definition, an active fault is one that has had surface displacement within Holocene time (about the last 11,000 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 site is not within a currently established Alquist-Priolo Earthquake Fault Zone for surface fault rupture hazards. Also, the site is not within a city-designated Fault Precaution Zone for the Hollywood Fault. No active or potentially active faults with the potential for surface fault rupture are known to pass directly beneath the site. Therefore, the potential for surface rupture due to faulting occurring beneath the site during the design life of the proposed development 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 4, Regional Fault Map.

6.1.2 Hollywood Fault

The closest active fault with the potential for surface fault rupture is the Hollywood Fault located approximately 150 feet south of the site (California Geological Survey, 2014b). The Hollywood Fault trends east-west along the base of the Santa Monica Mountains from the West Beverly Hills Lineament in the West Hollywood-Beverly Hills area to the Los Feliz area of Los Angeles. The fault is a ground-water barrier within Holocene age sediments. Scarps 1.8 to 2.7 meters high in Holocene age flood plain deposits have been suggested along the fault trace in the Atwater area (Weber et al. 1980). Studies by several investigators (Dolan et al., 2000; Dolan et al., 1997; Dolan and Sieh, 1992; and Crook and Proctor, 1992) have indicated that the fault is active, based on geomorphic evidence, stratigraphic correlation between exploratory borings, and fault trenching studies. Additionally, recent investigations performed in the Hollywood area have demonstrated that Holocene age alluvial sediments have been offset by several strands of the Hollywood Fault (California Geological Survey, 2014a). An Alquist-Priolo Earthquake Fault Zone has recently been established for the Hollywood Fault (California Geological Survey, 2014b). Also, the City of West Hollywood considers the Hollywood Fault active for planning purposes (City of West Hollywood. 2011; City of West Hollywood, 2010).

A previous fault rupture hazard investigation was performed for the adjacent properties to the west (8950 Sunset Boulevard) and south (1016, 1018 and 1020 Hilldale Avenue). The previous fault investigation at the 8950 Sunset site was performed by Applied Earth Science in 1998 and included drilling 9 hollow stem auger borings. The previous investigation at the 1016-1020 Hilldale Avenue site was performed by Advanced Geotechniques in 1998 and included drilling 5 hollow stem auger borings. Both of these investigations concluded that active faults did not traverse these sites (City of West Hollywood, 2010). Based on the absence of faulting observed on the adjacent sites and the location of known or suspected faults (City of West Hollywood General Plan, 2011; City of West Hollywood, 2010), the main trace of the Hollywood Fault is located approximately150 feet south of the site.

6.1.3 Other Nearby Active Faults

Other nearby active faults are the Newport-Inglewood Fault Zone, the Santa Monica Fault, and the Raymond Fault located approximately 2.0 miles west, 2.3 miles southwest, and 8.9 miles east of the site, respectively (Ziony and Jones, 1989). The active San Andreas Fault Zone is located approximately 35 miles northeast of the site.

The closest potentially active fault to the site is the Overland Avenue Fault located approximately 4.3 miles to the southwest (Ziony and Jones, 1989). Other nearby potentially active faults are the MacArthur Fault, the Charnock Fault, and the Coyote Pass Fault located approximately 5.6 miles southeast, 6.7 miles southwest, and 11.0 miles southeast of the site, respectively (Ziony and Jones, 1989).

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.

6.2 Seismicity

As with all of Southern California, the site has experienced historic earthquakes from various regional faults. The seismicity of the region surrounding the site was formulated based on research of an electronic database of earthquake data. The epicenters of recorded earthquakes with magnitudes equal to or greater than 5.0 in the site vicinity are depicted on Figure 5, Regional Seismicity Map. A partial list of moderate to major magnitude earthquakes that have occurred in the Southern California area within the last 100 years is included in the following table.

LIST OF HISTORIC EARTHQUAKES

Earthquake (Oldest to Youngest)	Date of Earthquake	Magnitude	Distance to Epicenter (Miles)	Direction to Epicenter
San Jacinto-Hemet area	April 21, 1918	6.8	83	ESE
Near Redlands	July 23, 1923	6.3	65	E
Long Beach	March 10, 1933	6.4	41	SE
Tehachapi	July 21, 1952	7.5	73	NW
San Fernando	February 9, 1971	6.6	22	N
Whittier Narrows	October 1, 1987	5.9	18	E
Sierra Madre	June 28, 1991	5.8	25	ENE
Landers	June 28, 1992	7.3	112	E
Big Bear	June 28, 1992	6.4	89	E
Northridge	January 17, 1994	6.7	12	NW
Hector Mine	October 16, 1999	7.1	126	ENE

The site could be subjected to strong ground shaking in the event of an earthquake. 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.

6.3 Seismic Design Criteria

The following table summarizes summarizes site-specific design criteria obtained from the 2013 California Building Code (CBC; Based on the 2012 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. The values presented below are for the risk-targeted maximum considered earthquake (MCE_R).

2013 CBC SEISMIC DESIGN PARAMETERS

Parameter	Value	2013 CBC Reference
Site Class	D	Table 1613.3.2
MCE_R Ground Motion Spectral Response Acceleration – Class B (short), S_S	2.446	Figure 1613.3.1(1)
MCE_R Ground Motion Spectral Response Acceleration – Class B (1 sec), S_1	0.889g	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}	2.446g	Section 1613.3.3 (Eqn 16-37)
Site Class Modified MCE_R Spectral Response Acceleration – (1 sec), S_{M1}	1.334g	Section 1613.3.3 (Eqn 16-38)
5% Damped Design Spectral Response Acceleration (short), S_{DS}	1.631g	Section 1613.3.4 (Eqn 16-39)
5% Damped Design Spectral Response Acceleration (1 sec), S_{D1}	0.889g	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	0.948g	Figure 22-7
Site Coefficient, F_{PGA}	1.0	Table 11.8-1
Site Class Modified MCE_G Peak Ground Acceleration, PGA_M	0.948g	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 2013 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 2008 Probabilistic Seismic Hazard Analysis (PSHA) Interactive Deaggregation online tool. The result of the deaggregation analysis indicates that the predominant earthquake contributing to the MCE peak ground acceleration is characterized as a 6.71 magnitude event occurring at a hypocentral distance of 5.0 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 6.68 magnitude occurring at a hypocentral distance of 10.7 kilometers from the site.

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

6.4 Liquefaction Potential

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

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

The State of California Seismic Hazard Zone Map for the Beverly Hills Quadrangle (1998) indicates that the site is not located in an area designated as "liquefiable." In addition, a review of the City of West Hollywood General Plan (2011) indicates that the site is not located within an area identified as having a potential for liquefaction. As previously discussed, the alluvial materials beneath the groundwater table are consolidated, older alluvial deposits that are not prone to liquefaction. Based on these considerations, it is our opinion that the potential for liquefaction and associated ground deformations beneath the site is very low.

6.5 Slope Stability

The topography of the site vicinity slopes to the south-southeast with approximately 18 feet of vertical relief. According to the City of West Hollywood (2011), the site is not within an area identified as having a potential for slope instability. Additionally, the site is not within an area identified as having a potential for seismic slope instability (CDMG, 1999). There are no known landslides near the site, nor is the site in the path of any known or potential landslides. Therefore, the potential for slope stability hazards to adversely affect the proposed development is considered low.

6.6 Earthquake-Induced Flooding

Earthquake-induced flooding is inundation caused by failure of dams or other water-retaining structures due to earthquakes. Based on a review of the Los Angeles County Safety Element (Leighton, 1990) and the City of West Hollywood General Plan (2011), the site is not located within a potential inundation area for an earthquake-induced dam failure. Therefore, the probability of earthquake-induced flooding is considered very low.

6.7 Tsunamis, Seiches, and Flooding

The site is not located within a coastal area. Therefore, tsunamis, seismic sea waves, 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. Flooding 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, 2015; LACDPW, 2015b).

6.8 Oil Fields & Methane Potential

Based on a review of the California Division of Oil, Gas and Geothermal Resources (DOGGR) Oil and Gas Well Location Map W1-5, the site is not located within the limits of an oilfield and oil wells are not located in the immediate site vicinity. 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 will need to be properly abandoned in accordance with the current requirements of the DOGGR.

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

6.9 Subsidence

Subsidence occurs when a large portion of land is displaced vertically, usually due to the withdrawal of groundwater, oil, or natural gas or as a result of decomposition of natural organic materials. Soils that are particularly subject to subsidence include those with high silt or clay content and/or high organic content. The site is located outside the boundary of an area identified in the City of West Hollywood Safety Element as a former marsh. In addition, significant organic materials were not encountered in our exploration at the site. Therefore, the potential for subsidence related to decomposition of organic materials at the site is considered low. Also, the potential for subsidence related to fluid or gas withdrawal is also considered low at the site. Only marginal activity currently exists in the Salt Lake Oilfield and water injection and flooding operations as part of secondary recovery is believed to have largely mitigated hazards related to fluid or gas withdrawal in the area (City of West Hollywood, 2010).

7. CONCLUSIONS AND RECOMMENDATIONS

7.1 General

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

7.1.2 Up to 9 feet of existing artificial fill was encountered during site exploration. The existing fill encountered is believed to be the result of past grading and construction activities at the site. Deeper fill may exist in other areas of the site that were not directly explored. Demolition of the existing structures which occupy the site are anticipated to disturb the upper few feet of soil below the existing subterranean levels. The existing fill and site soils are suitable for re-use as engineered fill provided the recommendations in the *Grading* section of this report are followed (see Section 7.6).

7.1.3 It is anticipated that the proposed subterranean parking levels will extend to depths of between 35 and 55 feet below the existing ground surface, including foundation depths. In addition, the extents of the existing subterranean levels are unknown; it is suggested that the extents of the existing subterranean walls be checked for conflicts with the proposed structure.

7.1.4 Excavation of the proposed subterranean levels is anticipated to penetrate through the existing artificial fill and alluvial soils throughout the excavation bottom.

7.1.5 Groundwater has been reported at depths ranging from 22½ to 53 feet below the ground surface, and the historic high groundwater level is at a depth of 22 feet below the ground surface. Excavation for construction of the proposed subterranean levels is anticipated to extend to depths of approximately 55 feet below the ground surface, including foundation excavations. Based on these considerations, it is anticipated that groundwater will be encountered during construction. Due to the depth of the proposed excavation and the potential for seasonal fluctuation in the groundwater level, temporary dewatering measures will likely be required to mitigate groundwater during excavation and construction. Recommendations for temporary dewatering are discussed in Section 7.2 of this report.

7.1.6 If the subterranean portion of the structure which extends below a depth of 22 feet below the ground surface is not designed for full hydrostatic pressure, a permanent dewatering system will be required to relieve and mitigate the water pressure. Recommendations for permanent dewatering are discussed in Section 7.3 of this report.

7.1.7 Based on the assumed depth of the proposed subterranean levels, it is recommended that the proposed structure be supported on a foundation system which derives support exclusively in the undisturbed, competent alluvium found at or below a depth of 35 feet.

7.1.8 If a permanent dewatering system is implemented, it is recommended that a conventional foundation system or reinforced concrete mat foundation system be utilized for support of the proposed structure provided foundations derive support in the competent alluvium. Foundations should be deepened as necessary to penetrate through any soft or unsuitable soils at the direction of the Geotechnical Engineer. All foundation excavations must be observed and approved by the Geotechnical Engineer (a representative of Geocon), prior to placing steel or concrete. Recommendations for foundation design are provided in Sections 7.8 through 7.10 of this report.

7.1.9 If a permanent dewatering system is not implemented, then the structure must be designed for hydrostatic pressure based on the historic high groundwater level of 22 feet below the ground surface. The hydrostatic design will result in uplift forces on the slab that that must be resisted by structural design. Based on these considerations, a reinforced concrete mat foundation system deriving support in the competent alluvium is recommended for support of the proposed structure.

7.1.10 The concrete slab-on-grade, if present, and ramp for the subterranean level may bear directly on the undisturbed alluvial soils at the excavation bottom. Any soils that are disturbed should be properly compacted for slab and ramp support. Where necessary, the existing artificial fill and alluvial soils are suitable for re-use as an engineered fill provided the procedures outlined in the *Grading* section of this report are followed (see Section 7.6).

7.1.11 Excavations up to 55 feet in vertical height are anticipated for construction of the subterranean levels, including excavations for foundations and/or dewatering systems. Due to the depth of the excavation and the proximity to the property lines, city streets and adjacent offsite structures, excavation of the proposed subterranean levels will likely require sloping and 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 shoring are provided in Section 7.21 of this report.

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

7.1.13 Foundations for small outlying structures, such as block walls up to 6 feet in height, planter walls or trash enclosures, which will not be tied-in to the existing structure, may be supported on conventional foundations bearing on a minimum of 12 inches of newly placed engineered fill. Where excavation and compaction cannot be performed or is undesirable, such as adjacent to property lines, foundations may derive support in the undisturbed alluvial soils found at or below a depth of 2 feet and should be deepened as necessary to maintain a minimum 12 inch embedment into the recommended bearing materials. Excavation on the order of 3 to 9 feet may be required in some areas. If the soils exposed in the excavation bottom are soft, compaction of the soft soils will be required prior to placing steel or concrete. Compaction of the foundation excavation bottom is typically accomplished with a compaction wheel or mechanical whacker and must be observed and approved by a Geocon representative.

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

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

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

7.2 Temporary Dewatering

7.2.1 Groundwater has been reported at depths ranging from 22½ to 53 feet below the ground surface. Based on the conditions encountered during site exploration, groundwater may be encountered during construction activities. The depth to groundwater at the time of construction can be further verified during initial dewatering well or shoring pile installation. If groundwater is present above the depth of the subterranean level, temporary dewatering will be necessary to maintain a safe working environment during excavation and construction activities.

7.2.2 It is recommended that a qualified dewatering consultant be retained to design the dewatering system. Recommendations for design flow rates for the temporary dewatering system should be determined by a qualified contractor or dewatering consultant. Temporary dewatering may consist of perimeter wells with interior well points as well as gravel filled trenches (French drains) placed adjacent to the shoring system and interior of the site. The number and locations of the wells or French drains can be adjusted during excavation activities as necessary to collect and control any encountered seepage. The French drains will then direct the collected seepage to a sump where it will be pumped out of the excavation.

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

7.3 Permanent Dewatering

7.3.1 If the subterranean portion of the structure which extends below a depth of 22 feet below the ground surface is not designed for full hydrostatic pressure and buoyancy, a permanent dewatering system will be required to relieve and mitigate the water pressure. A subdrainage system consisting of perforated pipe placed in gravel-filled trenches may be installed beneath the subterranean slab-on-grade to intercept and control groundwater. This system can be combined with the perimeter retaining wall drainage system provided backflow valves are installed at the base of the wall drainage system.

7.3.2 A typical permanent sub-slab drainage system would consist of a 3-inch thick layer of ¾-inch gravel that is placed upon a layer of filter fabric (Miami 500X or equivalent), and vibrated to a dense state. Subdrain pipes leading to sump areas, provided with automatic pumping units, should drain the gravel layer. The drain lines should consist of perforated pipe, placed with perforations down, in trenches that are at least six inches below the gravel layer. The excavation bottom, as well as the trench bottoms should be lined with filter fabric prior to placing and compacting gravel. The trenches should be spaced approximately 50 feet apart at most, within the interior, and should extend along to the perimeter of the building. Subsequent to the installation of the drainage system, the waterproofing system and building slab may then be placed on the densified gravel. A mud- or rat-slab may be placed over the waterproofing system for protection during placement of rebar and mat slab construction.

7.3.3 Recommendations for design flow rates for the permanent dewatering system should be determined by a qualified contractor or dewatering consultant.

7.4 Soil and Excavation Characteristics

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

7.4.2 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.

7.4.3 All onsite excavations must be conducted in such a manner that potential surcharges from existing structures, construction equipment, and vehicle loads are resisted. The surcharge area may be defined by a 1:1 projection down and away from the bottom of an existing foundation or vehicle load. Penetrations below this 1:1 projection will require special excavation measures such as sloping or shoring. Excavation recommendations are provided in the *Temporary Excavations* section of this report (see Section 7.20).

7.5 Minimum Resistivity, pH, and Water-Soluble Sulfate

7.5.1 Potential of Hydrogen (pH) and resistivity testing as well as chloride content testing were performed on representative samples of soil to generally evaluate the corrosion potential to surface utilities. The tests were performed in accordance with California Test Method Nos. 643 and 422 and indicate that the soils are considered “corrosive” with respect to corrosion of buried ferrous metals on site. The results are presented in Appendix B (Figure B7) and should be considered for design of underground structures.

7.5.2 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 B7) and indicate that the on-site materials possess “negligible” sulfate exposure to concrete structures as defined by 2013 CBC Section 1904 and ACI 318-08 Sections 4.2 and 4.3.

7.5.3 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 on buried metal pipes and concrete structures in direct contact with the soils.

7.6 Grading

7.6.1 Grading is anticipated to include excavation for the subterranean portion of the proposed structure, foundations, and utility trenches, as well as placement of backfill for walls, ramps, and trenches.

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

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

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

7.6.5 It is recommended that the proposed structure be supported on a foundation system which derives support exclusively in the undisturbed, competent alluvium found at or below a depth of 35 feet. Foundations should be deepened as necessary to penetrate through soft or unsuitable soils at the direction of the Geotechnical Engineer. All foundation excavations must be observed and approved in writing by the Geotechnical Engineer (a representative of Geocon), prior to placing steel or concrete.

7.6.6 The concrete slab-on-grade, if present, and ramp for the subterranean portion of the proposed structure may bear directly on the undisturbed alluvial soils found at the excavation bottom. Any disturbed soils should be properly compacted for slab support.

7.6.7 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 a minimum 95 percent of the maximum dry density in accordance with ASTM D 1557 (latest edition).

7.6.8 Foundations for small outlying structures, such as block walls up to 6 feet in height, planter walls or trash enclosures, which will not be tied-in to the existing structure, may be supported on conventional foundations bearing on a minimum of 12 inches of newly placed engineered fill. Where excavation and compaction cannot be performed or is undesirable, such as adjacent to property lines, foundations may derive support in the undisturbed alluvial soils found at or below a depth of 2 feet and should be deepened as necessary to maintain a minimum 12 inch embedment into the recommended bearing materials. Excavation on the order of 3 to 9 feet may be required in some areas. If the soils exposed in the excavation bottom are soft, compaction of the soft soils will be required prior to placing steel or concrete. Compaction of the foundation excavation bottom is typically accomplished with a compaction wheel or mechanical whacker and must be observed and approved by a Geocon representative.

7.6.9 Although not anticipated for this project, all imported fill shall be observed, tested, and approved by Geocon West, Inc. prior to bringing soil to the site. Rocks larger than six inches in diameter shall not be used in the fill. If necessary, import soils used as structural fill should have an expansion index less than 20 and soil corrosivity properties that are equally or less detrimental to that of the existing onsite soils (see Figure B7).

7.6.10 Utility trenches should be properly backfilled in accordance with the requirements of the Green Book (latest edition). The pipe should be bedded with clean sands (Sand Equivalent greater than 30) to a depth of at least one 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. Prior to placing any bedding materials or pipes, the excavation bottom must be observed and approved in writing by the Geotechnical Engineer (a representative of Geocon).

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

7.7 Foundation Design

7.7.1 If a permanent dewatering system is implemented, it is recommended that a conventional foundation system or reinforced concrete mat foundation system be utilized for support of the proposed structure provided foundations derive support in the competent alluvium found at or below a depth of 35 feet. Foundations should be deepened as necessary to penetrate through any soft or unsuitable alluvium at the direction of the Geotechnical Engineer.

7.7.2 If a permanent dewatering system is not implemented then the structure must be designed for hydrostatic pressure based on the historic high groundwater level of 22 feet below the ground surface. The hydrostatic design will result in uplift forces on the slab that that must be resisted by structural design. The recommended floor slab uplift pressure to be used in design would be 62.4(H) in units of pounds per square foot, where "H" is the height of the water above the bottom of the mat foundation in feet. Based on these considerations, a reinforced concrete mat foundation system deriving support in the competent alluvium is recommended for support of the proposed structure.

7.7.3 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 dewatering system (if necessary, reinforcing steel and concrete to verify that the excavations and exposed soil conditions are consistent with those anticipated. If unanticipated soil conditions are encountered, foundation modifications may be required.

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

7.8 Conventional Foundation Design

7.8.1 Continuous footings may be designed for an allowable bearing capacity of 4,000 pounds per square foot (psf), and should be a minimum of 12 inches in width, 18 inches in depth below the lowest adjacent grade, and 12 inches into the recommended bearing materials.

7.8.2 Isolated spread foundations may be designed for an allowable bearing capacity of 4,500 psf, and should be a minimum of 12 inches in width, 18 inches in depth below the lowest adjacent grade, and 12 inches into the recommended bearing materials.

7.8.3 The soil bearing pressure above may be increased by 500 psf and 800 psf for each additional foot of foundation width and depth, respectively, up to a maximum allowable soil bearing pressure of 8,000 psf.

7.8.4 The allowable bearing pressure may be increased by up to one-third for transient loads due to wind or seismic forces.

7.8.5 Continuous footings should be reinforced with a minimum of four No. 4 steel reinforcing bars, two placed near the top of the footing and two near the bottom. Reinforcement for spread footings should be designed by the project structural engineer.

7.8.6 If depth increases are utilized for the exterior wall footings, this office should be provided a copy of the final construction plans so that the excavation recommendations presented herein could be properly reviewed and revised if necessary.

7.8.7 The above foundation dimensions and minimum reinforcement recommendations are based on soil conditions and building code requirements only, and are not intended to be used in lieu of those required for structural purposes.

7.8.8 No special subgrade presaturation is required prior to placement of concrete. However, the moisture in the foundation subgrade should be sprinkled as necessary to maintain a moist condition at the time of concrete placement.

7.9 Mat Foundation Design

7.9.1 It is anticipated that the proposed mat foundation will impart an average pressure of less than 4,000 psf, with locally higher pressures up to 8,000 psf. The recommended maximum allowable bearing value is 8,000 pounds per square foot. The allowable bearing pressure may be increased by up to one-third for transient loads due to wind or seismic forces.

7.9.2 It is recommended that a modulus of subgrade reaction of 200 pounds per cubic inch (pci) be utilized for the design of the mat foundation on undisturbed alluvium, or where underlain by a permanent dewatering system. This value is a unit value for use with a one-foot square footing. The modulus should be adjusted in accordance with the following equation when used with larger foundations:

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

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

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

7.9.4 For seismic design purposes, a coefficient of friction of 0.40 may be utilized between the concrete mat and alluvium without a moisture barrier, and 0.15 for slabs underlain by a moisture barrier.

7.9.5 If the proposed structure is to be designed for full hydrostatic pressure, the recommended floor slab uplift pressure to be used in design would be 62.4(H) in units of pounds per square foot, where “H” is the height of the water above the bottom of the mat foundation in feet. For design purposes the water table may be assumed at 22 feet below the existing ground surface.

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

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

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

7.10 Foundation Settlement

7.10.1 If a permanent dewatering system is installed to relieve hydrostatic pressure and buoyancy, the maximum expected settlement for the structure supported on a foundation system deriving support in the competent alluvium found at and below a depth of 35 feet and with a maximum allowable bearing pressure of 8,000 psf is estimated to be less than ½ inch and occur below the heaviest loaded structural element. Differential settlement is not expected to exceed ¾ inch over a distance of twenty feet, or between the center and corner of the mat.

7.10.2 If a permanent dewatering system is not installed to relieve hydrostatic pressure and buoyancy, an uplift pressure of 62.4(H) in units of pounds per square foot (where "H" is the height of the water above the bottom of the mat foundation in feet) must be mitigated based on an assumed water depth of 22 feet below existing ground surface (the historic high groundwater depth). Recommendations for resistance to uplift are provided in Section 7.11.

7.10.3 Once the design and foundation loading configurations for the proposed structures proceeds to a more finalized plan, the estimated settlements presented in this report should be reviewed and revised, if necessary. If the final foundation loading configurations are greater than the assumed loading conditions, the potential for settlement should be reevaluated by this office.

7.11 Uplift Resistance

7.11.1 Foundation uplift may be resisted by the weight of structure as well as friction along the sides of foundations. If additional uplift resistance is required, the perimeter shoring piles may be utilized provided the toes of the piles are poured with structural concrete and are designed as permanent piles. Recommendations for the design of shoring are provided in Section 7.21.

7.11.2 Uplift resistance may also be generated by additional piles constructed within the interior of the structure. It is recommended that post-grouted friction piles be utilized. The uplift capacity may be determined using a frictional resistance of 500 psf (⅔ the downward capacity).

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

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

7.12 Lateral Design

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

7.12.2 Passive earth pressure for the sides of foundations and slabs poured against the alluvial soils or properly compacted engineered fill below the groundwater table may be computed as an equivalent fluid having a density of 140 pcf with a maximum earth pressure of 1,400 pcf (these values have been adjusted for buoyant forces). If a permanent dewatering system is installed to relieve hydrostatic pressure and buoyancy, or for soils above the groundwater table, passive pressure may be computed as 280 pcf with a maximum earth pressure of 2,800 pcf. When combining passive and friction for lateral resistance, the passive component should be reduced by one-third.

7.13 Miscellaneous Foundations

7.13.1 Foundations for small outlying structures, such as block walls less than 6 feet in height, planter walls or trash enclosures, which will not be structurally supported by the proposed building, may be supported on conventional foundations deriving support on a minimum of 12 inches of newly placed engineered fill which extends laterally at least 12 inches beyond the foundation area. Where excavation and compaction cannot be performed, such as adjacent to property lines, foundations may derive support in the undisturbed alluvial soils found at or below a depth of 2 feet, and should be deepened as necessary to maintain a minimum 12 inch embedment into the recommended bearing materials. Excavation on the order of 3 to 9 feet may be required in some areas.

7.13.2 If the soils exposed in the excavation bottom are loose, compaction of the soils will be required prior to placing steel or concrete. Compaction of the foundation excavation bottom is typically accomplished with a compaction wheel or mechanical whacker and must be observed and approved by a Geocon representative. Miscellaneous foundations may be designed for a bearing value of 1,500 psf, and should be a minimum of 12 inches in width, 18 inches in depth below the lowest adjacent grade and 12 inches into the recommended bearing material. The allowable bearing pressure may be increased by up to one-third for transient loads due to wind or seismic forces.

7.13.3 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 to verify that the excavations and exposed soil conditions are consistent with those anticipated.

7.14 Concrete Slabs-on-Grade

7.14.1 Unless specifically evaluated and designed by a qualified structural engineer, the slab-on-grade for the subterranean parking garage should be a minimum of 5 inches of concrete reinforced with No. 3 steel reinforcing bars placed 18 inches on center in both horizontal directions and positioned vertically near the slab midpoint. The concrete slab-on-grade may bear directly on the undisturbed alluvial soils found at the excavation bottom. The ramp may derive support in the undisturbed alluvial soils and/or engineered fill. It is recommended that where artificial is exposed along the ramp, the upper 12 inches of soil should be properly compacted to at least 95 percent relative compaction, as determined by ASTM Test Method D 1557 (latest edition) for ramp support. Any disturbed soils should be properly compacted for slab and ramp support.

7.14.2 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 is recommended. 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.

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

7.14.4 For seismic design purposes, a coefficient of friction of 0.40 may be utilized between concrete slabs and subgrade soils without a moisture barrier, and 0.15 for slabs underlain by a moisture barrier.

7.14.5 Exterior slabs, not subject to traffic loads, 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. Prior to construction of slabs, the upper 12 inches of subgrade should be moistened 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. Construction joints should be designed by the project structural engineer.

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

7.15 Retaining Walls Design

7.15.1 The recommendations presented below are generally applicable to the design of rigid concrete or masonry retaining walls having a maximum height of 50 feet. In the event that walls significantly higher than 50 feet are planned, Geocon should be contacted for additional recommendations.

7.15.2 Retaining wall foundations may be designed in accordance with the recommendations provided in the *Foundation Design* sections of this report (see Section 7.7 through 7.10).

7.15.3 Retaining walls with a level backfill surface that are not restrained at the top should be designed utilizing a triangular distribution of pressure (active pressure) of 40 pcf.

7.15.4 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) of 60 pcf.

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

7.15.6 The wall pressures provided above assume that the proposed retaining walls will support relatively undisturbed alluvial soils. If sloping techniques are to be utilized for construction of proposed walls, which would result in a wedge of engineered fill behind the retaining walls, revised earth pressures may be required. This should be evaluated once the use of sloping measures is established and once the geotechnical characteristics of the engineered backfill soils can be further evaluated.

7.15.7 Additional active pressure should be added for a surcharge condition due to sloping ground, vehicular traffic, or adjacent structures. Recommendations for the incorporation of surcharges are provided in section 7.26 of this report. Once the design becomes more finalized, an addendum letter can be prepared revising recommendations and addressing specific surcharge conditions throughout the project, if necessary.

7.15.8 In addition to the recommended earth pressure, the upper ten 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.

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

7.16 Dynamic (Seismic) Lateral Forces

7.16.1 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 2013 CBC).

7.16.2 A seismic load of 25 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 2013 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 one half of two thirds of PGA_M calculated from ASCE 7-10 Section 11.8.3.

7.17 Retaining Wall Drainage

7.17.1 Retaining walls should be provided with a drainage system extended at least two-thirds the height of the wall. 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 6). The clean bottom and subdrain pipe, behind a retaining wall, should be observed by the Geotechnical Engineer (a representative of Geocon), prior to placement of gravel or compacting backfill.

7.17.2 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 7). These vertical columns of drainage material would then be connected at the bottom of the wall to a collection panel or a 1-cubic-foot rock pocket drained by a 4-inch subdrain pipe.

7.17.3 Subdrainage pipes at the base of the retaining wall drainage system should outlet to an acceptable location via controlled drainage structures.

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

7.18 Elevator Pit Design

7.18.1 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. 3 steel reinforcing bars placed 18 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 7.7 to 7.10 and 7.15).

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

7.18.3 If retaining wall drainage is to be provided, the drainage system should be designed in accordance with the *Retaining Wall Drainage* section of this report (see Section 7.17).

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

7.19 Elevator Piston

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

7.19.2 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.

7.19.3 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. The contractor should be prepared to mitigate the buoyant forces on the casing due to groundwater seepage, if encountered. Continuous observation of the drilling and installation of the elevator piston by the Geotechnical Engineer (a representative of Geocon West, Inc.) is required.

7.19.4 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.

7.20 Temporary Excavations

7.20.1 Excavations on the order of 55 feet in vertical height may be required during excavation and construction of the proposed subterranean levels and foundations, including excavations that may be required for foundations and dewatering systems. The excavations are expected to expose artificial fill and alluvial soils, which may be suitable for vertical excavations up to 5 feet in height where loose soils or caving sands are not present and where excavations are not surcharged by adjacent traffic or structures.

7.20.2 Vertical excavations greater than 5 feet or where surcharged by existing structures or traffic loads will require sloping or shoring measures 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 height of 15 feet. A uniform slope does not have a vertical portion. Where space is limited, shoring measures will be required. Shoring recommendations are provided in Section 7.21 of this report.

7.20.3 Where sloped embankments 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 construction embankments are to be maintained during the rainy season, berms are suggested along the tops of the slopes where necessary to prevent runoff water from entering the excavation and eroding the slope faces. The soils exposed in the cut slopes should be inspected during excavation by our personnel so that modifications of the slopes can be made if variations in the soil conditions occur. All excavations should be stabilized within 30 days of initial excavation.

7.21 Shoring – Soldier Pile Design and Installation

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

7.21.2 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.

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

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

7.21.5 Due to the preliminary nature of the project, the extents of the existing subterranean levels are unknown. It is suggested that the extents of the existing subterranean walls be checked for conflicts with the proposed structure.

7.21.6 Drilled cast-in-place soldier piles should be placed no closer than 2 diameters on center. The minimum diameter of the piles is 18 inches. Structural concrete should be used for the soldier piles below the excavation; lean-mix concrete may be employed above that level. As an alternative, lean-mix concrete may be used throughout the pile where the reinforcing consists of a wideflange section. The slurry must be of sufficient strength to impart the lateral bearing pressure developed by the wideflange section to the soil. For design purposes, an allowable passive value for the soils below the bottom plane of excavation may be assumed to be 140 pounds per square foot per (value have been reduced for buoyant forces). Where piles are installed by vibration techniques, the passive pressure may be assumed to mobilize across a width equal to the 2 times the dimension of the beam flange. The allowable passive value may be doubled for isolated piles spaced a minimum of three the pile diameter. To develop the full lateral value, provisions should be implemented to assure firm contact between the soldier piles and the undisturbed soils.

7.21.7 Groundwater was encountered during exploration and the contractor should be prepared for groundwater during pile installation. Piles placed below the water level require the use of a tremie to place the concrete into the bottom of the hole. A tremie should consist of a rigid, water-tight tube having a diameter of not less than 6 inches with a hopper at the top. The tube should be equipped with a device that will close the discharge end and prevent water from entering the tube while it is being charged with concrete. The tremie should be supported so as to permit free movement of the discharge end over the entire top surface of the work and to permit rapid lowering when necessary to retard or stop the flow of concrete. The discharge end should be closed at the start of the work to prevent water entering the tube and should be entirely sealed at all times, except when the concrete is being placed. The tremie tube should be kept full of concrete. The flow should be continuous until the work is completed and the resulting concrete seal should be monolithic and homogeneous. The tip of the tremie tube should always be kept about 5 feet below the surface of the concrete and definite steps and safeguards should be taken to insure that the tip of the tremie tube is never raised above the surface of the concrete.

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

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

7.21.10 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, it is recommended that the bore diameter be at least 2 inches smaller than 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.

7.21.11 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.

7.21.12 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.

7.21.13 Based on Table 19 of the *Transportation and Construction Induced Vibration Guidance Manual* (Caltrans 2004), 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.

7.21.14 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.

7.21.15 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.

7.21.16 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 0.40 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 700 pounds per square foot.

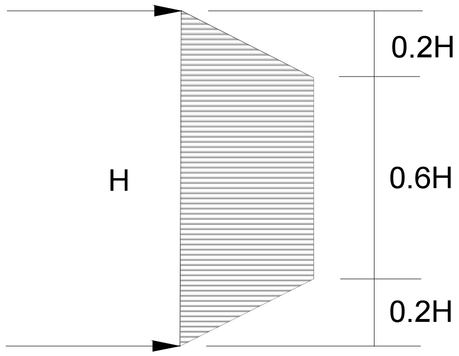
7.21.17 Due to the nature of the site soils, it is expected that continuous lagging between soldier piles will be required. However, it is recommended that the exposed soils be observed by the Geotechnical Engineer (a representative of Geocon West, Inc.), to verify the presence of any cohesive soils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

7.21.18 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.

7.21.19 Assuming that a temporary dewatering system is implemented just outside the shoring system, and that pumping is continuously maintained throughout the excavation and construction process, it is recommended that an equivalent fluid pressure based on the following table, be utilized for design. A diagram depicting the trapezoidal pressure distribution of lateral earth pressure is provided below the table.

HEIGHT OF SHORING (FEET)	EQUIVALENT FLUID PRESSURE (Pounds Per Cubic Foot) (ACTIVE PRESSURE)	EQUIVALENT FLUID PRESSURE (Pounds Per Cubic Foot) (AT-REST PRESSURE)
Up to 55	33	21H

Trapezoidal Distribution of Pressure



7.21.20 It is very important to note that active pressures can only be achieved when movement in the soil (earth wall) occurs. If movement in the soil is not acceptable, such as adjacent to an existing structure, or the pile is restrained from movement by bracing or a tie back anchor, an at-rest pressure of 53 pcf should be considered for design purposes.

7.21.21 Where a combination of sloped embankment and shoring is utilized, the pressure will be greater and must be determined for each combination. Additional active pressure should be added for a surcharge condition due to slopes, vehicular traffic or adjacent structures and should be designed for each condition. The surcharge pressure should be evaluated in accordance with the recommendations in Section 7.26 of this report.

7.21.22 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.

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

7.21.24 Because of the depth of the excavation, some means of monitoring the performance of the shoring system is suggested. The monitoring should consist of periodic surveying of the lateral and vertical locations of the tops of all soldier piles and the lateral movement along the entire lengths of selected soldier piles.

7.21.25 Due to the depth of 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 even that distress or settlement is noted, an investigation should be performed and corrective measures taken sot that continued or worsened distress or settlement is mitigated. Documentation and monitoring of the offsite structures and improvements is not the responsibility of the geotechnical engineer.

7.22 Tie-Back Anchors

7.22.1 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.

7.22.2 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 (reduced for buoyancy) as follows:

- 7 feet below the top of the excavation – 1,000 pounds per square foot
- 20 feet below the top of the excavation – 1,350 pounds per square foot
- 30 feet below the top of the excavation – 1,600 pounds per square foot

7.22.3 Depending on the techniques utilized, and the experience of the contractor performing the installation, a maximum allowable friction capacity of 3½ 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.

7.23 Anchor Installation

7.23.1 Tied-back anchors are typically installed between 20 and 40 degrees below the horizontal; however, occasionally alternative angles are necessary to avoid existing improvements and utilities. The locations and depths of all offsite utilities should be thoroughly checked prior to design and installation of the tie-back anchors. Caving of the anchor shafts, particularly within sand and gravel deposits or seepage zones, should be anticipated during installation and provisions should be implemented in order to minimize such caving. It is suggested that hollow-stem auger drilling equipment be used to install the anchors. The anchor shafts should be filled with concrete by pumping from the tip out, and the concrete should extend from the tip of the anchor to the active wedge. In order to minimize the chances of caving, it is recommended that the portion of the anchor shaft within the active wedge be backfilled with sand before testing the anchor. This portion of the shaft should be filled tightly and flush with the face of the excavation. The sand backfill should be placed by pumping; the sand may contain a small amount of cement to facilitate pumping.

7.24 Anchor Testing

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

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

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

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

7.24.5 After a satisfactory test, each anchor should be locked-off at the design load. This should be verified by rechecking the load in the anchor. The load should be within 10 percent of the design load. A representative of this firm should observe the installation and testing of the anchors.

7.25 Internal Bracing

7.25.1 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 4,000 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.

7.26 Surcharge from Adjacent Structures and Improvements

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

7.26.2 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:

$$For\ \ {}^{x}\!/\!{}_{H} \le 0.4$$

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

and

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

$$\sigma_H(x,z) = \frac{1.26\left(\frac{x}{H}\right)^2\left(\frac{z}{H}\right)}{\left[\left(\frac{x}{H}\right)^2+\left(\frac{z}{H}\right)^2\right]^2}\frac{Q_L}{H}$$

where x is the distance from the face of the excavation to the vertical line-load, H is the distance from the bottom of the footing to the bottom of excavation, z is the depth at which the horizontal pressure is desired, QL is the vertical line-load and σH is the horizontal pressure at depth z.

7.26.3 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:

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

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

and

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

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

then

$$\sigma_H^{'}(z) = \sigma_H(z)\cos^2(1.1\theta)$$

where x is the distance from the face of the excavation 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, σ is the vertical pressure at depth z, Θ is the angle between a line perpendicular to the bulkhead and a line from the point-load to half the pile spacing at the bulkhead, and σ_H is the horizontal pressure at depth z.

7.26.4 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.

7.27 Surface Drainage

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

7.27.2 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 2013 CBC 1804.3 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 five feet of the building perimeter footings except when enclosed in protected planters.

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

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

7.28 Plan Review

7.28.1 Grading and foundation 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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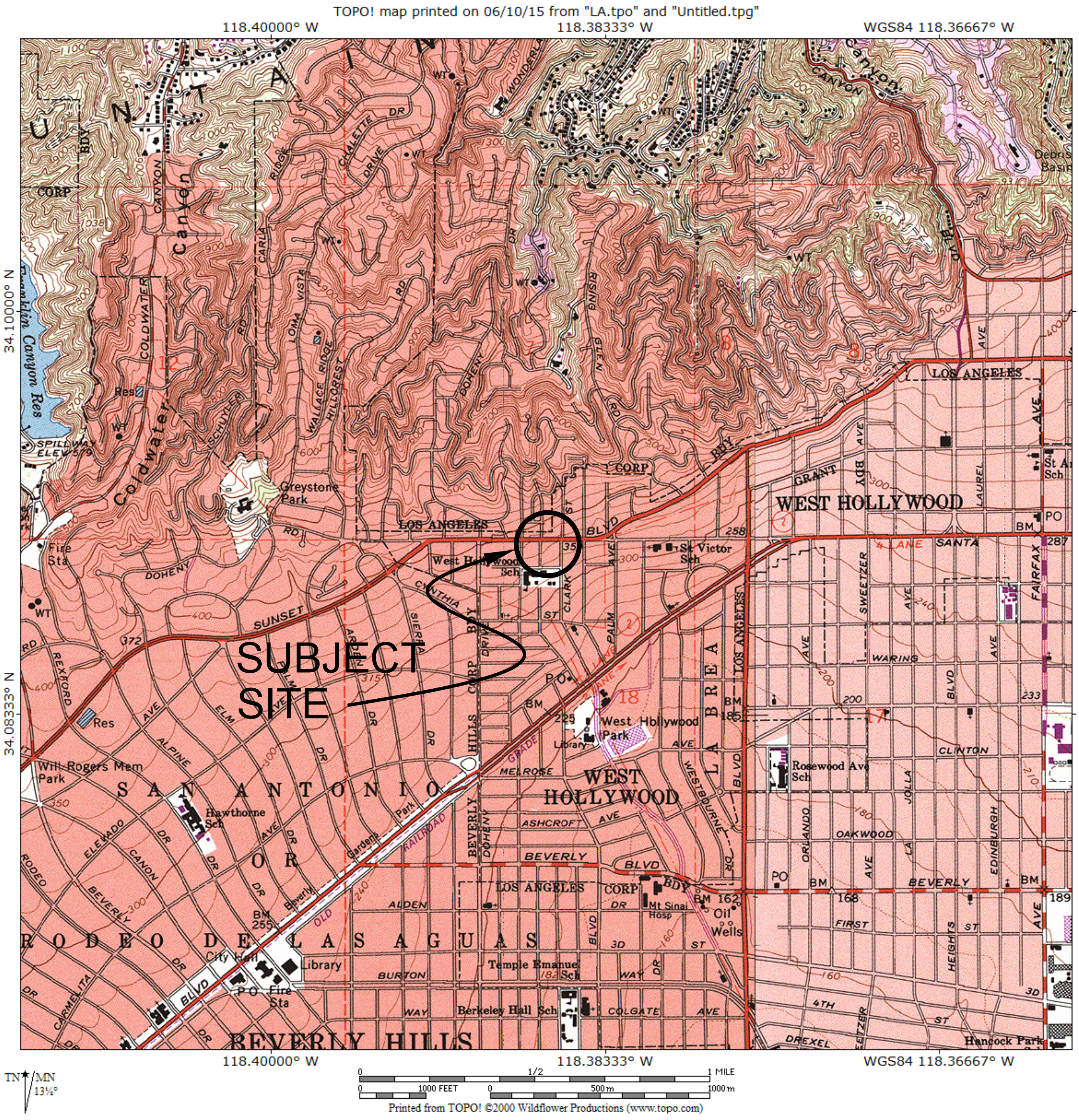

REFERENCE:U.S.G.S. TOPOGRAPHIC MAPS, 7.5 MINUTE SERIES, HOLLYWOOD, CA QUADRANGLE

GEOCON
WEST, INC.

ENVIRONMENTAL GEOTECHNICAL MATERIALS
3303 N. SAN FERNANDO BLVD. - SUITE 100 - BURBANK, CA 91504
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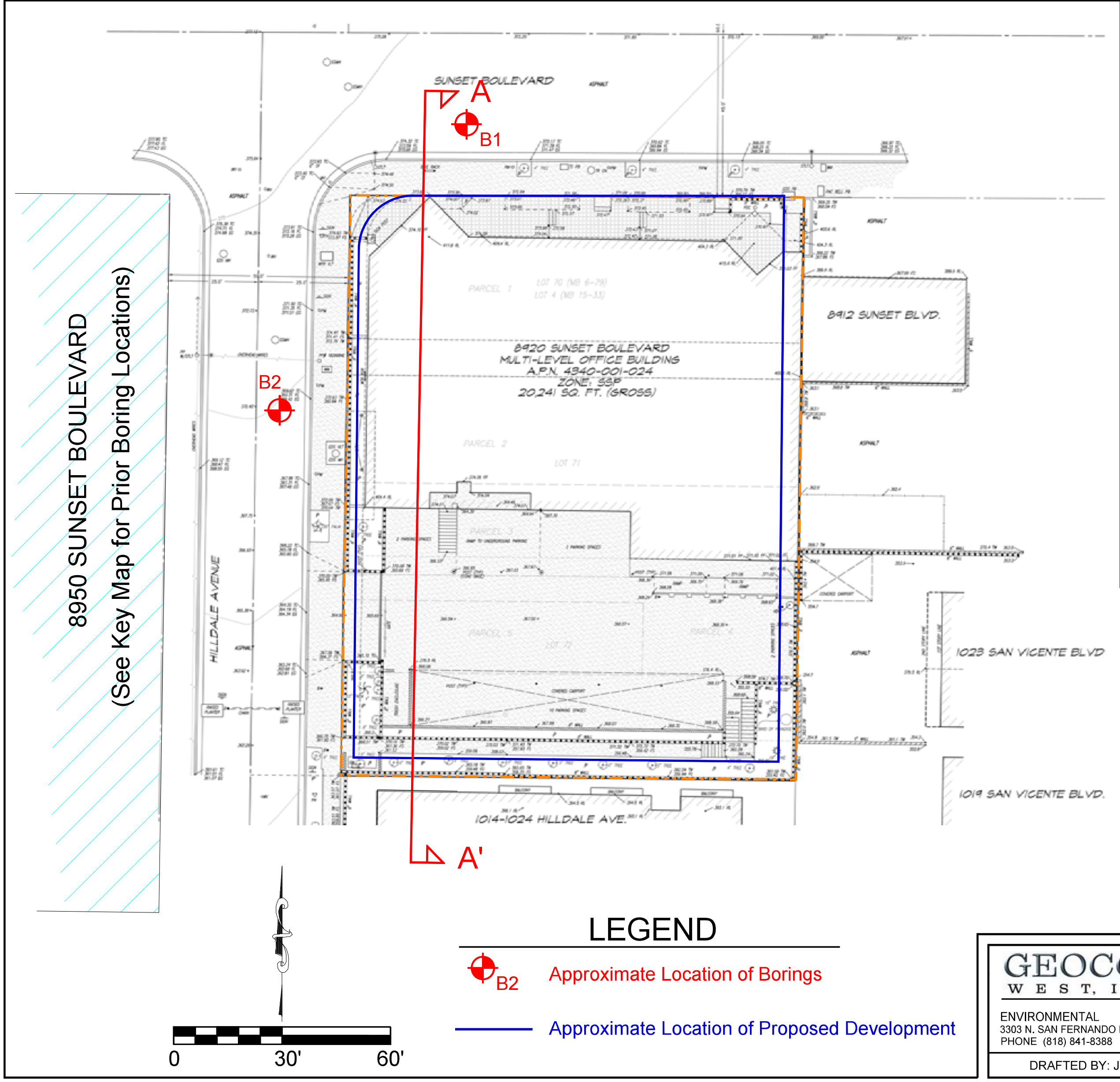
DRAFTED BY: RMA
CHECKED BY: SFK

VICINITY MAP

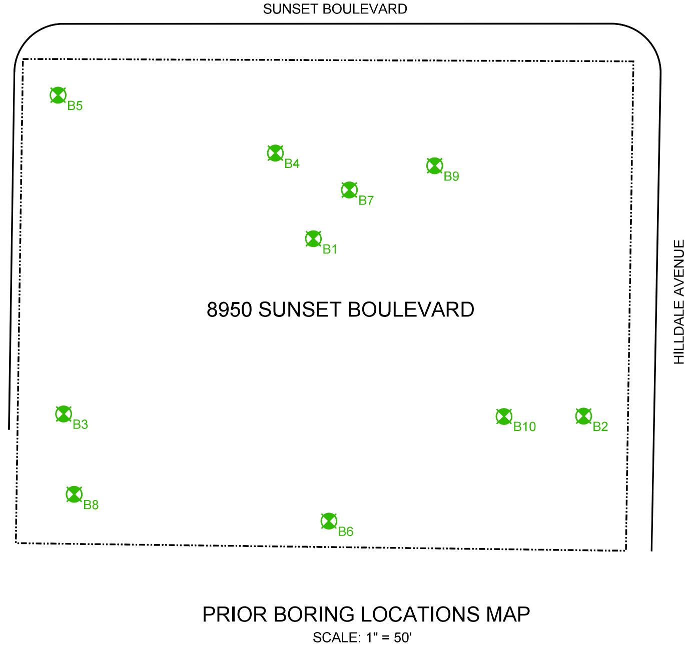
8920 SUNSET BOULEVARD LLC
8920 SUNSET BOULEVARD
WEST HOLLYWOOD, CALIFORNIA

JULY 2015
PROJECT NO. A9286-06-01
FIG. 1



LEGEND

- B2 Approximate Location of Borings
- Approximate Location of Proposed Development



PRIOR BORING LOCATIONS MAP

SCALE: 1" = 50'

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SITE PLAN

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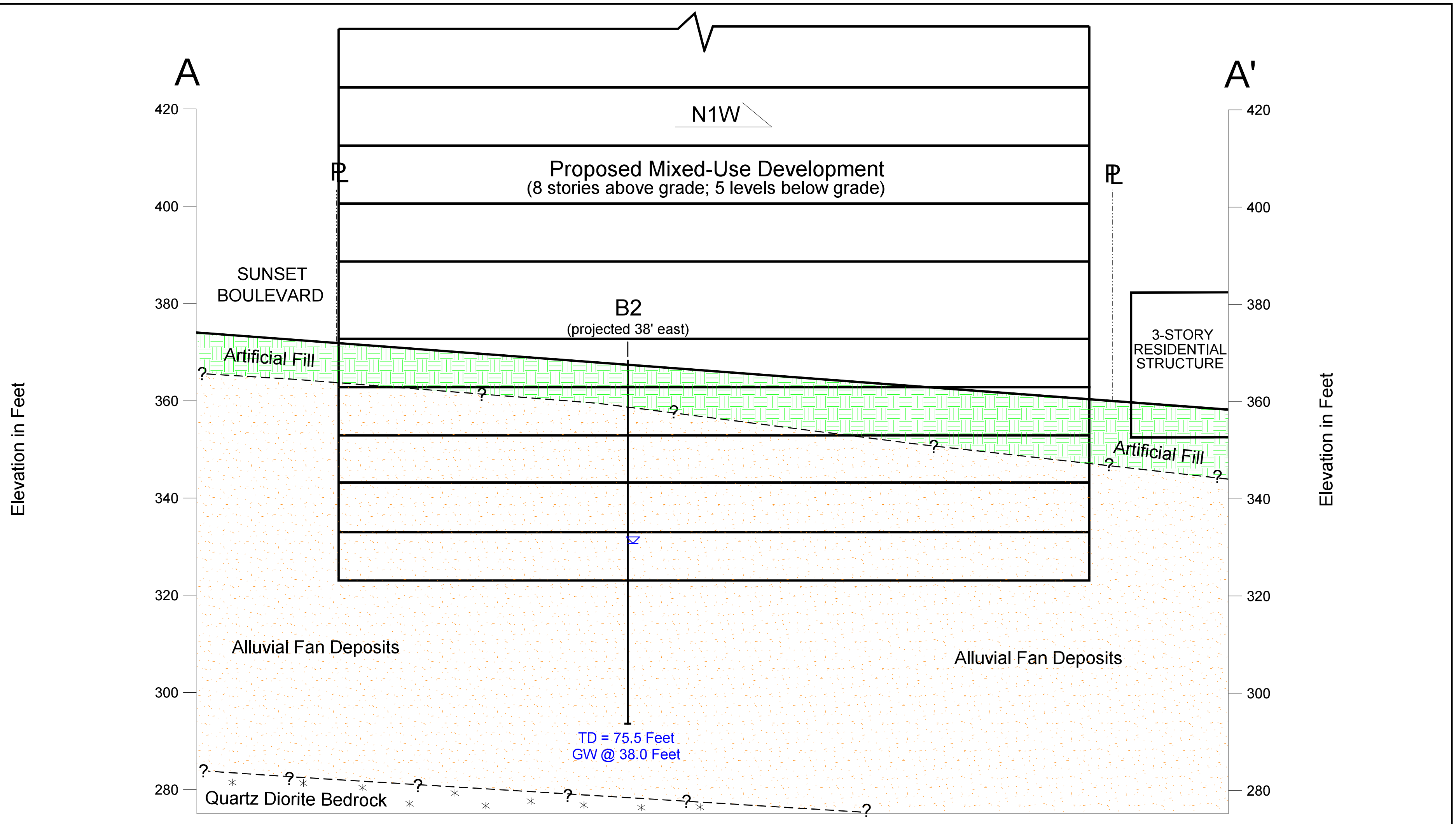
JULY 2015	PROJECT NO. A9286-06-01	FIG. 2



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GEOLOGIC SECTION A - A'

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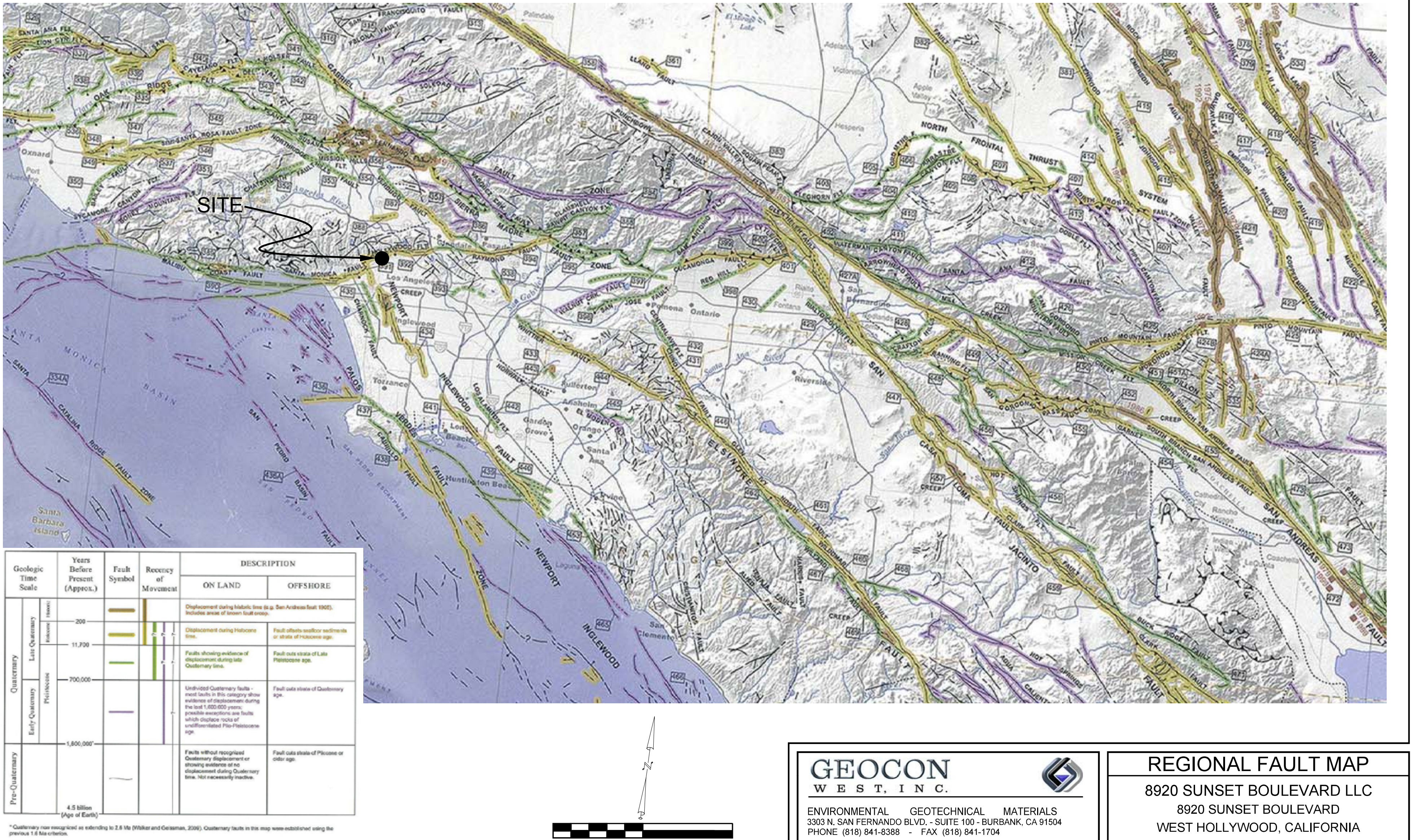
JULY 2015
PROJECT NO. A9286-06-01
FIG. 3

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



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REGIONAL FAULT MAP

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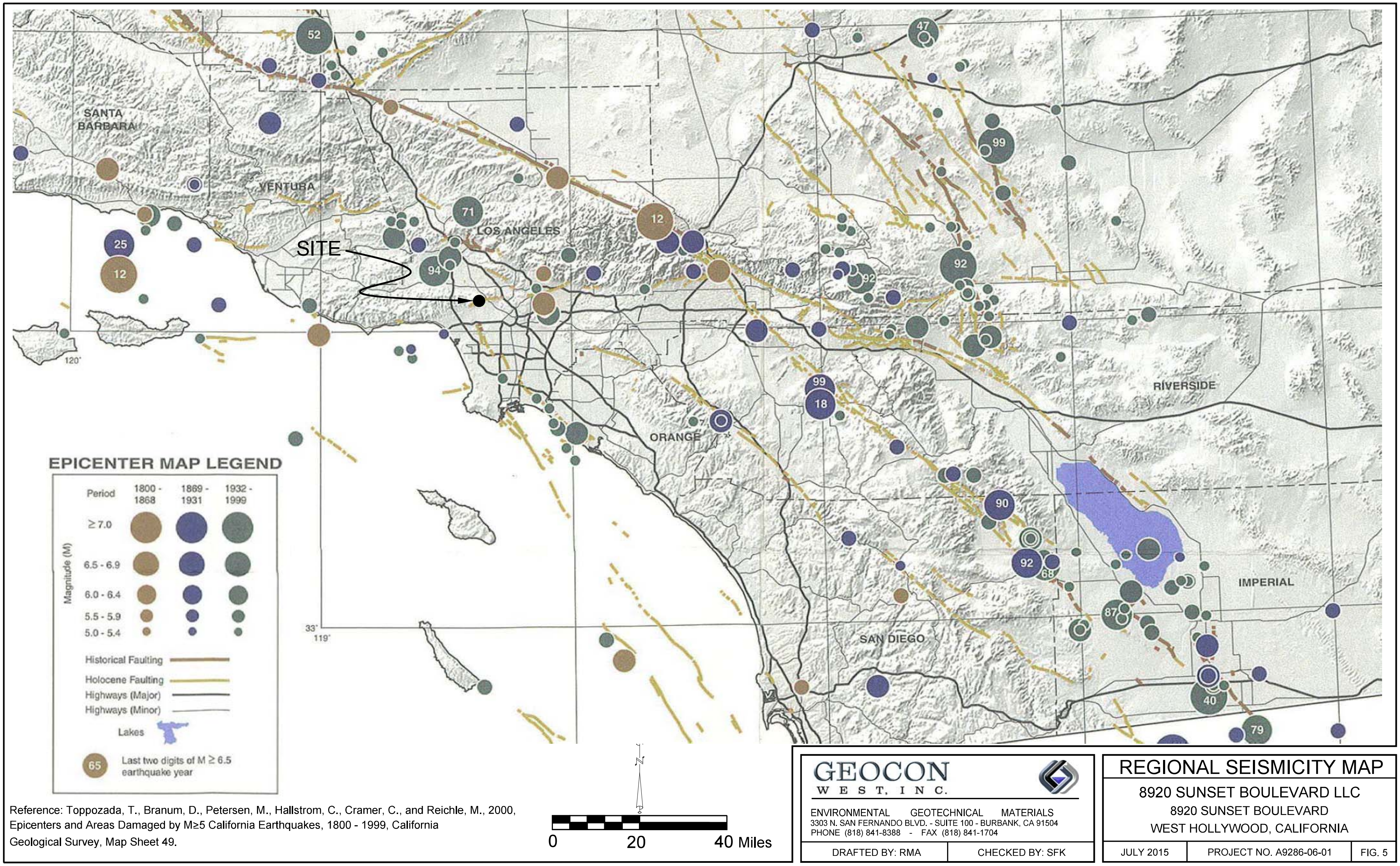
JULY 2015	PROJECT NO. A9286-06-01	FIG. 4



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

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REGIONAL SEISMICITY MAP

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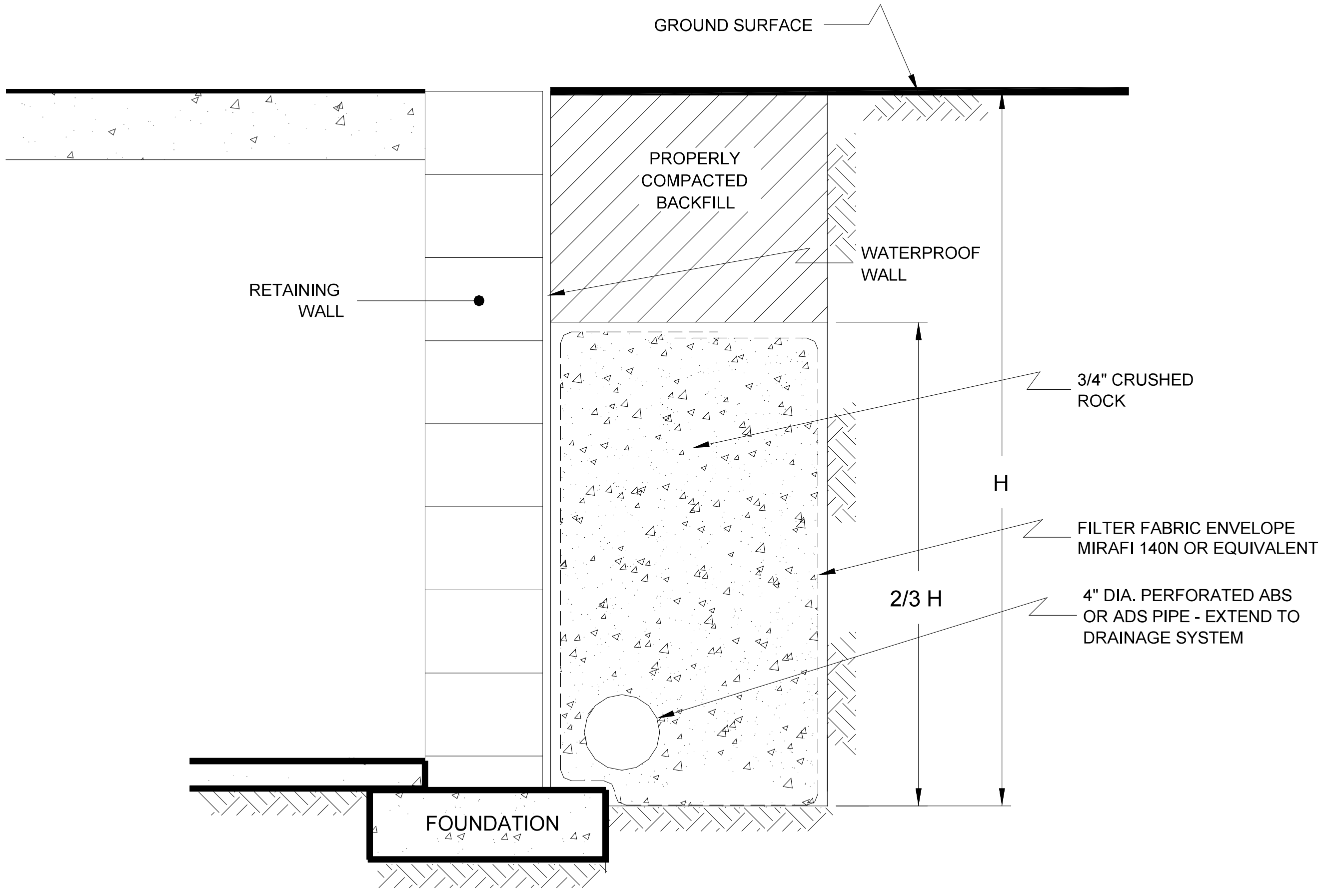
JULY 2015 PROJECT NO. A9286-06-01 FIG. 5



NO SCALE

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RETAINING WALL DRAIN DETAIL

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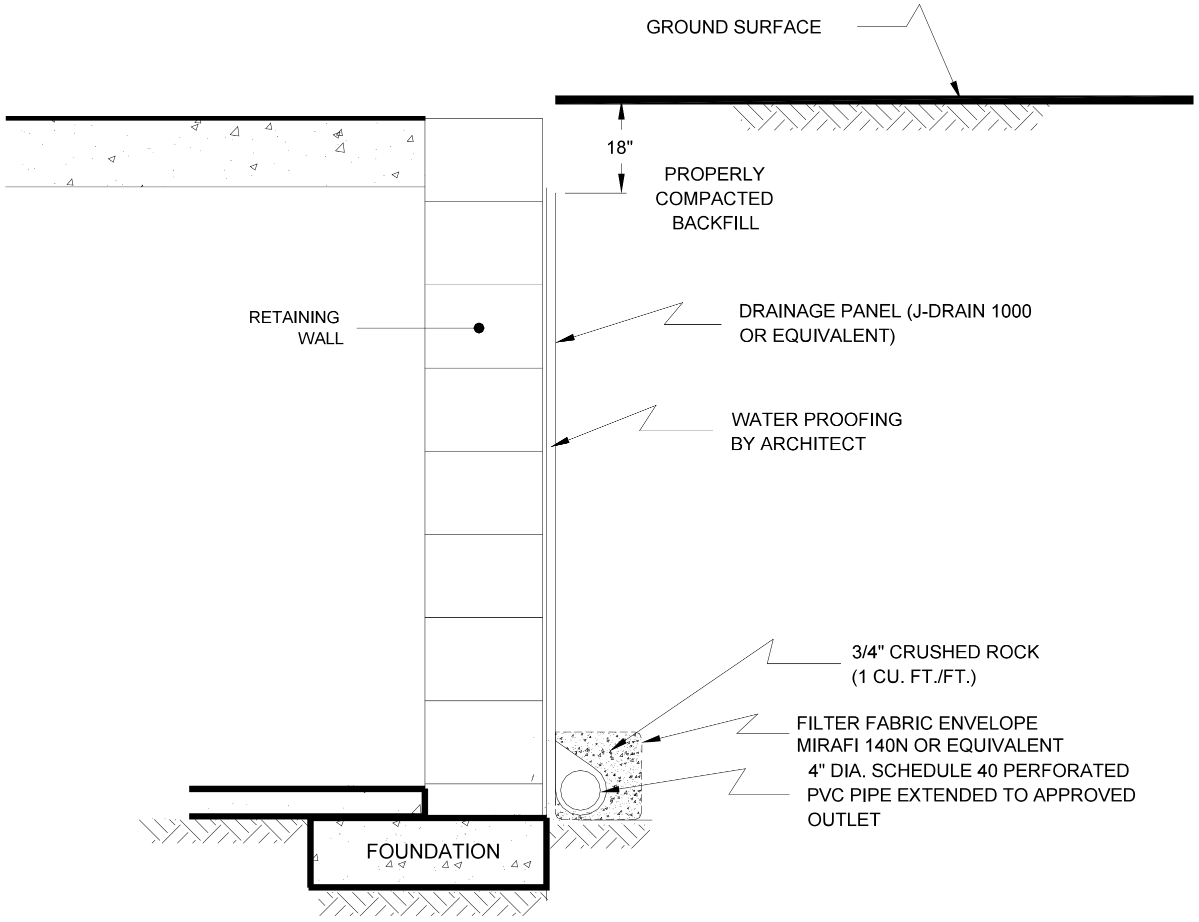
JULY 2015 | PROJECT NO. A9286-06-01 | FIG. 6



NO SCALE

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RETAINING WALL DRAIN DETAIL

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JULY 2015 | PROJECT NO. A9286-06-01 | FIG. 7

APPENDIX A

APPENDIX A

FIELD INVESTIGATION

The site was explored on May 30, 2015 by excavating two 8-inch-diameter borings, one in Sunset Boulevard and one in Hilldale Avenue, using a truck-mounted hollow-stem auger drilling machine. Borings could not be drilled within the site boundaries due to the existing basement that encompasses the entire property. The borings were excavated to a depth of approximately 3 to 75½ feet below the existing ground surface, respectively. Boring B1 was planned to be drilled to a depth of 75½ feet below the existing ground surface; however, the presence of underground utilities in Sunset Boulevard resulted in abandonment of boring B1 after three attempts to drill.

Representative and relatively undisturbed samples were obtained by driving a 3 inch, O. D., California Modified Sampler into the "undisturbed" soil mass with blows from a 140-pound auto-hammer falling 30 inches. The California Modified Sampler was equipped with 1-inch high by 2 ³/₈-inch diameter brass sampler rings to facilitate soil removal and testing. Bulk samples were also obtained.

The soil conditions encountered in the borings were visually examined, classified and logged in general accordance with the Unified Soil Classification System (USCS). Logs of the borings are presented on Figures A1 and A2. The logs depict the soil and geologic conditions encountered and the depth at which samples were obtained.

PROJECT NO. A9286-06-01

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

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

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

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

GEOCON

PROJECT NO. A9286-06-01

BORING 2

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

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

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

A9286-06-01 BORING LOGS.GPJ

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

SAMPLE SYMBOLS

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

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

GEOCON

PROJECT NO. A9286-06-01

BORING 2

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

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

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

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

A9286-06-01 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. A9286-06-01

BORING 2

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

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

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

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

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

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

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

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

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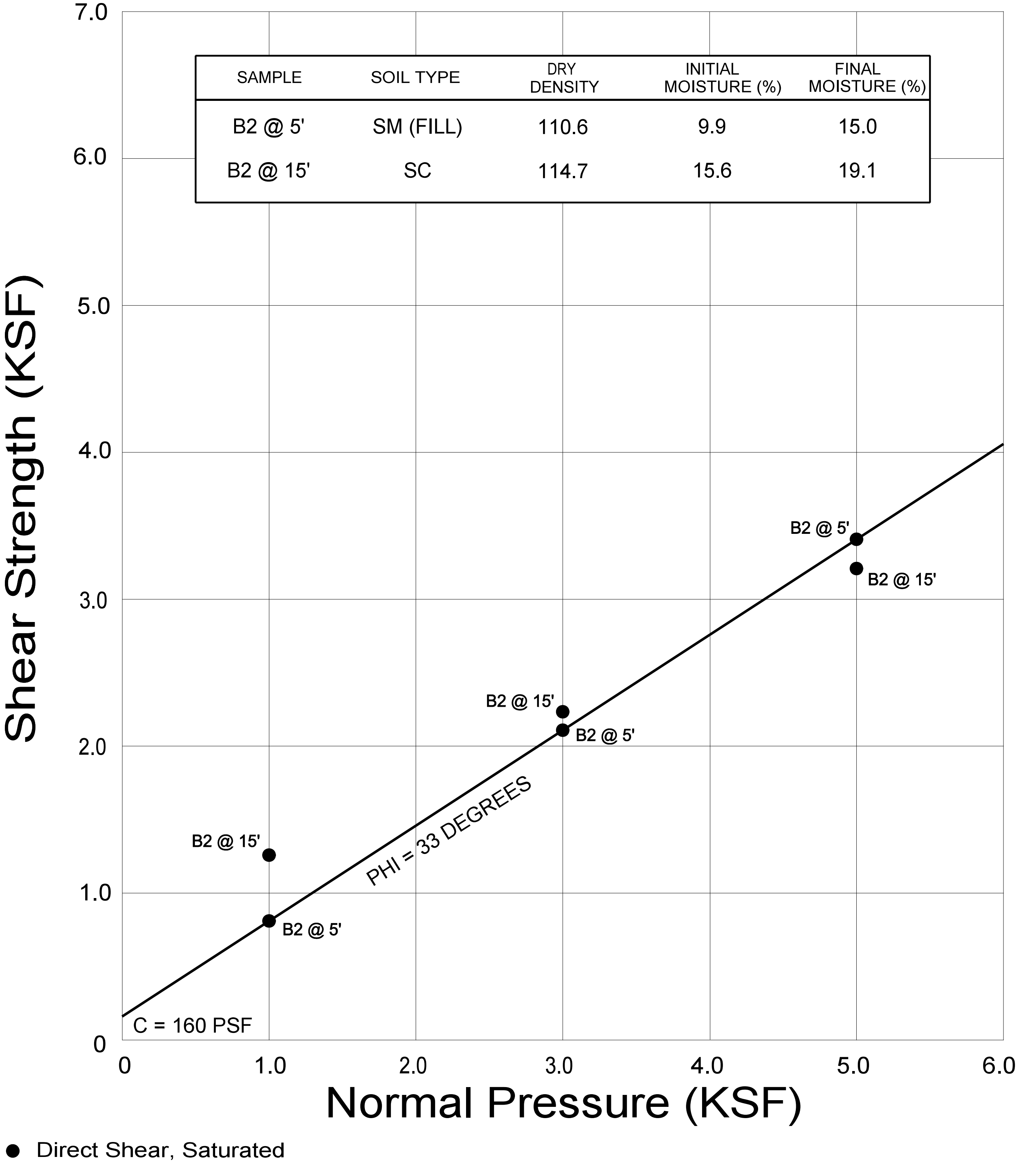
APPENDIX B

APPENDIX B

LABORATORY TESTING

Laboratory tests were performed in accordance with generally accepted test methods of the "American Society for Testing and Materials (ASTM)", or other suggested procedures. Selected samples were tested for direct shear strength, consolidation and expansion characteristics, corrosivity, in-place dry density and moisture content. The results of the laboratory tests are summarized in Figures B1 through B7. The in-place dry density and moisture content of the samples tested are presented on the boring logs, Appendix A.



SAMPLE	SOIL TYPE	DRY DENSITY	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B2 @ 5'	SM (FILL)	110.6	9.9	15.0
B2 @ 15'	SC	114.7	15.6	19.1

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DIRECT SHEAR TEST RESULTS

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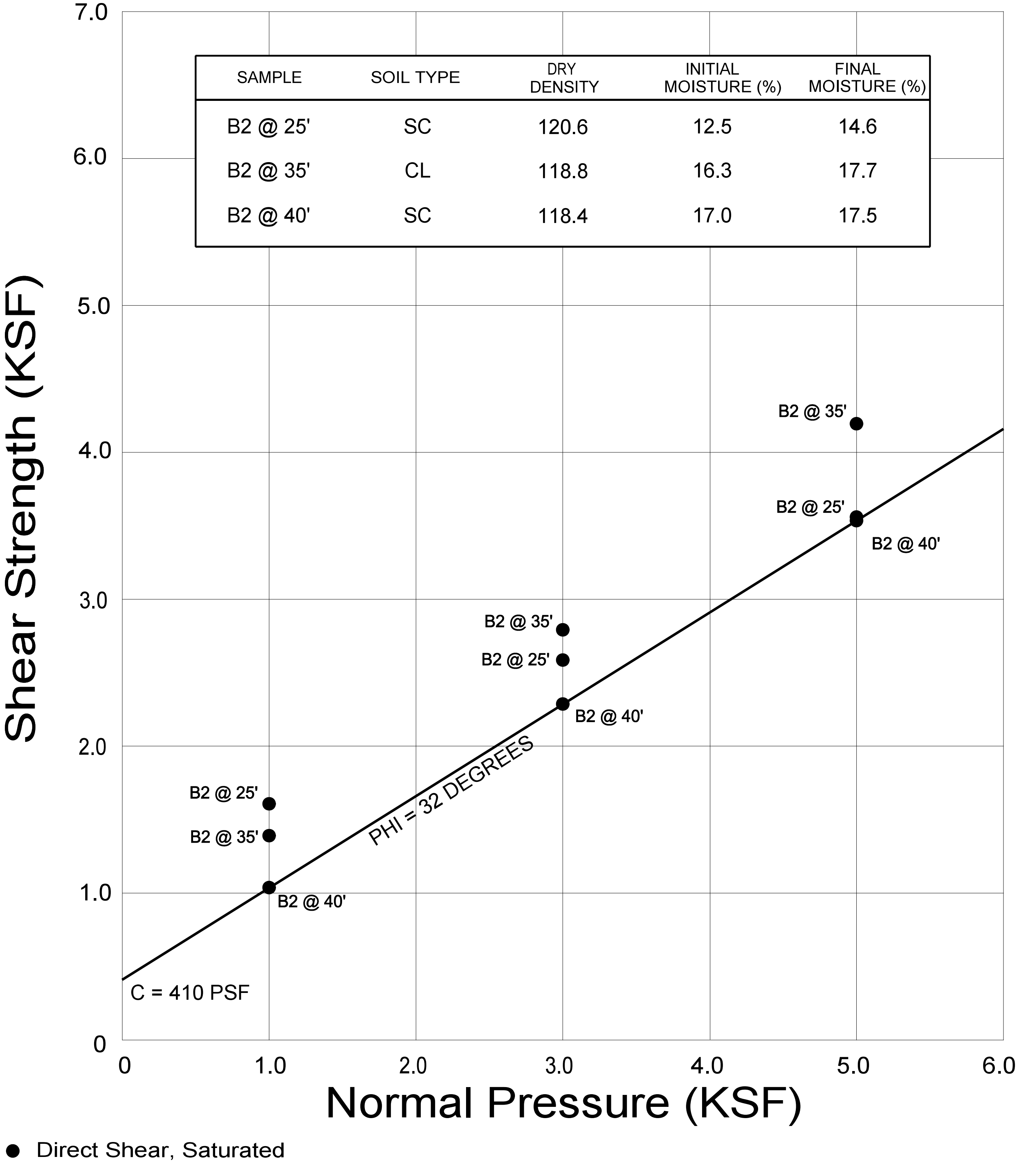
DRAFTED BY: PZ	CHECKED BY: JMT	JULY 2015	PROJECT NO. A9286-06-01	FIG. B1



SAMPLE	SOIL TYPE	DRY DENSITY	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B2 @ 25'	SC	120.6	12.5	14.6
B2 @ 35'	CL	118.8	16.3	17.7
B2 @ 40'	SC	118.4	17.0	17.5

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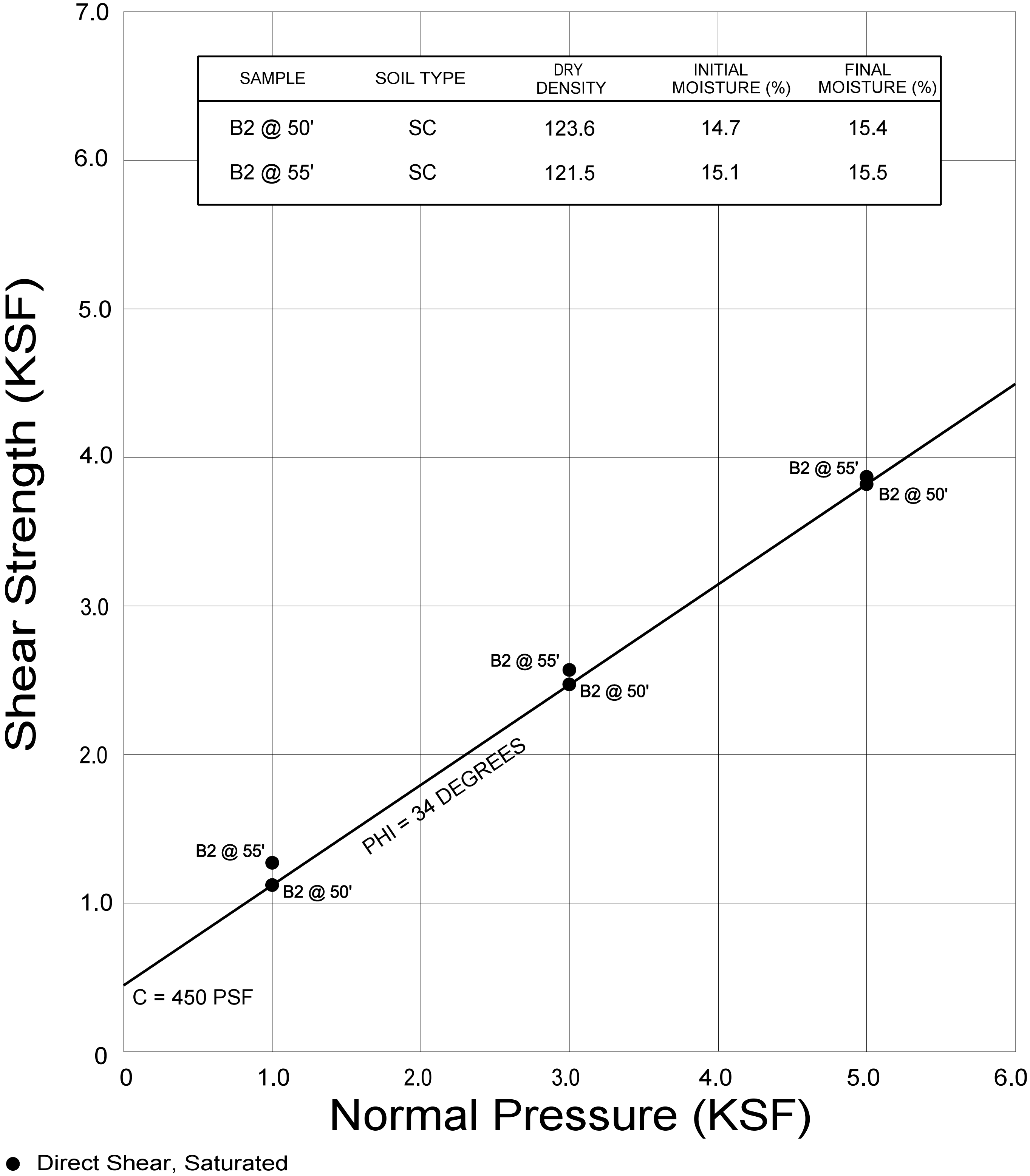
DRAFTED BY: PZ	CHECKED BY: JMT	JULY 2015	PROJECT NO. A9286-06-01	FIG. B2



SAMPLE	SOIL TYPE	DRY DENSITY	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B2 @ 50'	SC	123.6	14.7	15.4
B2 @ 55'	SC	121.5	15.1	15.5

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DIRECT SHEAR TEST RESULTS

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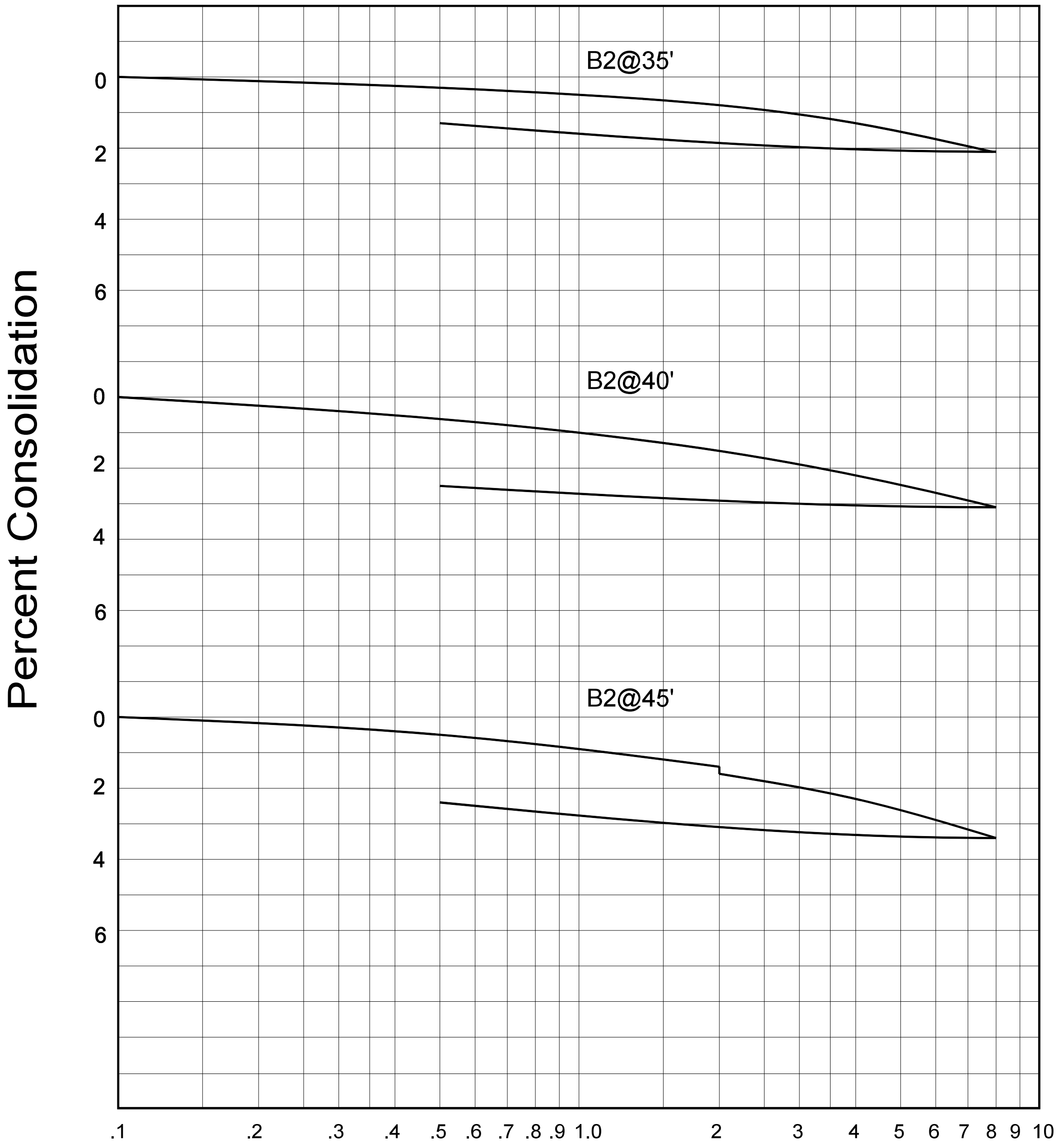
DRAFTED BY: PZ	CHECKED BY: JMT	JULY 2015	PROJECT NO. A9286-06-01	FIG. B3

WATER ADDED AT 2 KSF



Consolidation Pressure (KSF)

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CONSOLIDATION TEST RESULTS

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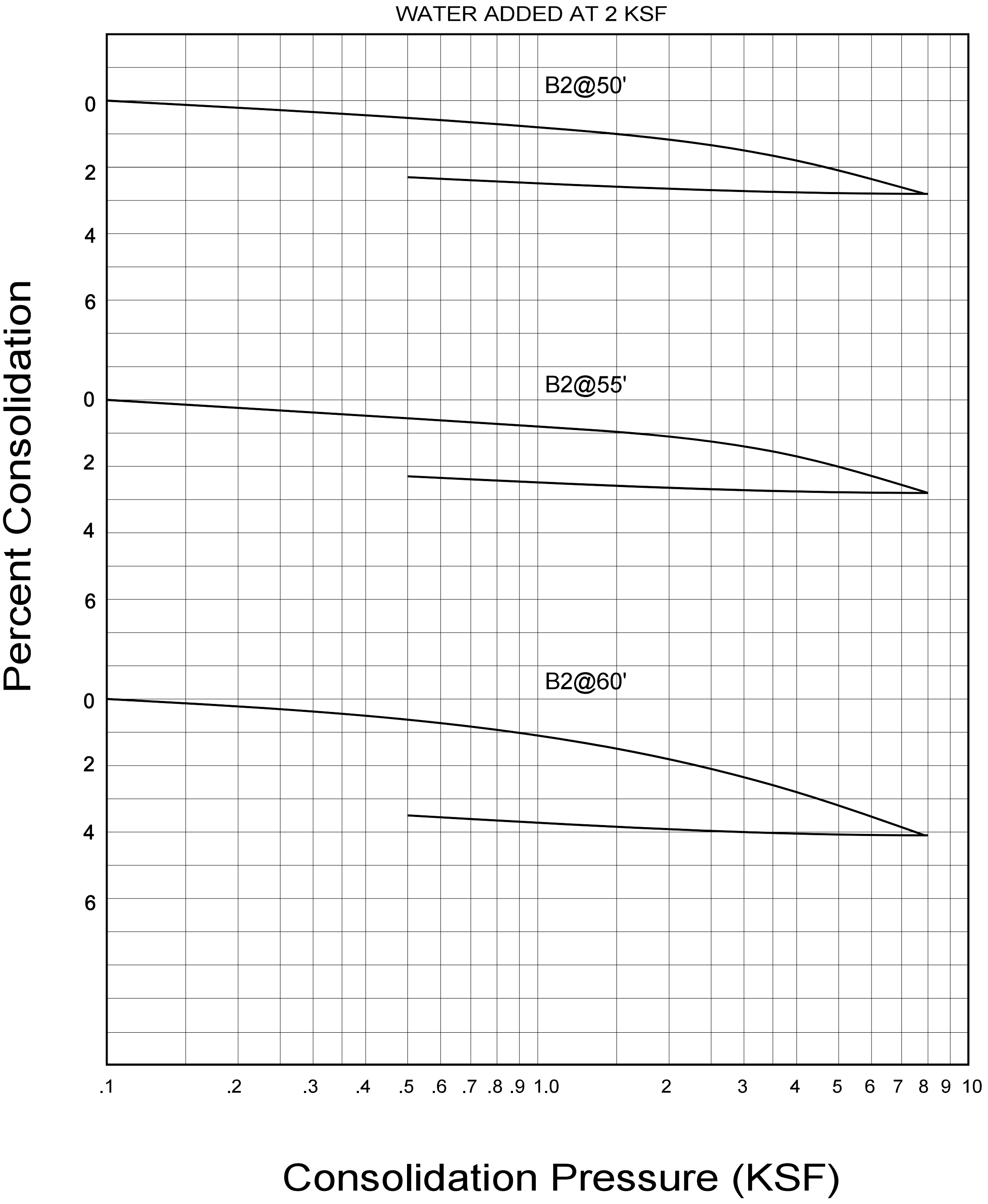
DRAFTED BY: PZ	CHECKED BY: JMT	JULY 2015	PROJECT NO. A9286-06-01	FIG. B4



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CONSOLIDATION TEST RESULTS

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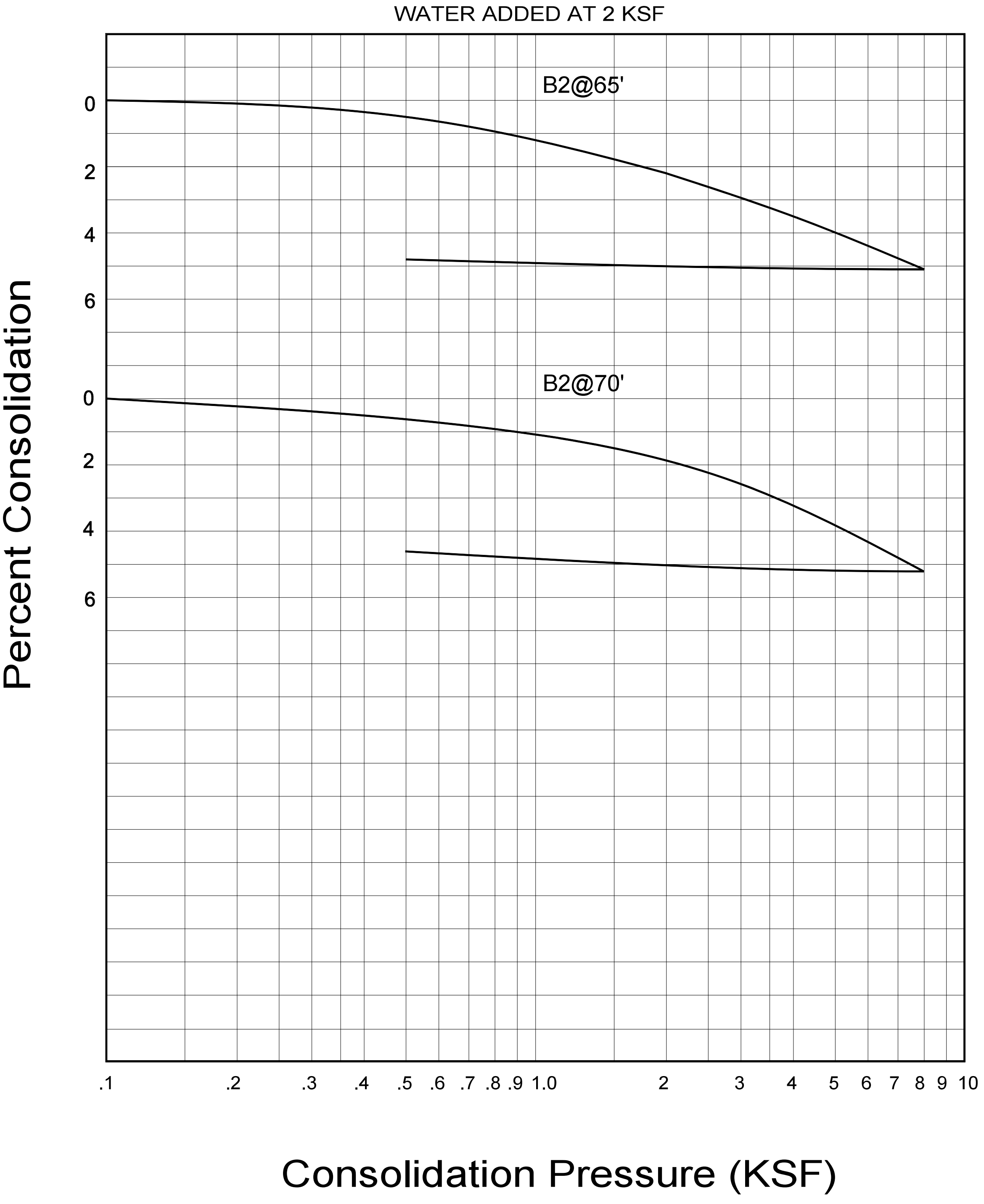
DRAFTED BY: PZ	CHECKED BY: JMT	JULY 2015	PROJECT NO. A9286-06-01	FIG. B5



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CONSOLIDATION TEST RESULTS

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

Sample No.	**pH**	**Resistivity (ohm centimeters)**
B2 @ 45-50'	8.22	1700 (Corrosive)

SUMMARY OF LABORATORY CHLORIDE CONTENT TEST RESULTS
EPA NO. 325.3

Sample No.	**Chloride Ion Content (%)**
B2 @ 45-50'	0.008

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

Sample No.	**Water Soluble Sulfate (% SO_4)**	**Sulfate Exposure***
B2 @ 45-50'	0.003	Negligible

* Reference: 2013 California Building Code, Section 1904.3 and ACI 318-11 Section 4.3.

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CORROSIVITY TEST RESULTS

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APPENDIX C

APPENDIX C

PREVIOUS BORINGS - 8950 SUNSET BOULEVARD

BORING LOG NUMBER 1

Drilling Date August 13, 1986 **Elevation** 375'

Project KB 14298-S Communication Art House

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

BORING LOG NUMBER 1 - Continued

Drilling Date August 13, 1986 **Elevation**

Project KB 14298-S Communication Art House

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

BORING LOG NUMBER 1 - Continued

Drilling Date August 13, 1986 **Elevation**

Project KB 14298-S Communication Art House

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

BORING LOG NUMBER 2

Drilling Date August 13, 1986 **Elevation** 364'

Project KB 14298-S Communication Art House

Sample Depth ft.	Blows per ft.	Moisture Content %	Dry Unit Weight p.c.f.	Depth in feet	Graphic Log	Description Surface Conditions 1½ inch A.C.
				0		
						FILL: Silty Sand brown, slightly moist, loose, medium to coarse grained, trace gravel
2	Push	7.0	106.1	2		roots to 1/8 inch, root hairs
				4		light brown
5	1/Tap	5.7	107.8			SAND TO SILTY SAND, light brown, slightly moist, medium dense, coarse grained
				6		
					SP/SM	SAND TO SILTY SAND, light reddish-brown, slightly moist, medium dense, some gravel
				8		
10	2	6.6	105.0	10		porous
				12		
				14		
15	5	14.7	113.8			
					SM/SC	SILTY SAND TO CLAYEY SAND, reddish-brown, slightly moist, medium dense, medium to coarse grained, some gravel
				16		
						light reddish-brown, sandier
				18		
20	9	13.1	122.8	20		dense
					SC/SM	CLAYEY SAND TO SILTY SAND, reddish-brown, moist, dense, medium to coarse grained, some gravel
				22		
				24		

BORING LOG NUMBER 2 - Continued

Drilling Date August 13, 1986 **Elevation**

Project KB 14298-S Communication Art House

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

BORING LOG NUMBER 3

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

Project KB 14298-S Communication Art House

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

BORING LOG NUMBER 3 - Continued

Drilling Date August 14, 1986 **Elevation** ____

Project KB 14298-S Communication Art House

Sample Depth ft.	Blows per ft.	Moisture Content %	Dry Unit Weight p.c.f.	Depth in feet	Graphic Log	Description Surface Conditions Weeds and Loose Soil
25	9	10.5	113.5	25		CLAYEY SAND TO SILTY SAND - Continued
				27		
					SM	SILTY SAND, light orange-brown, moist, dense, trace gravel, medium to coarse grained, clay binder
				29		
30	26	10.4	124.4			
				31		reddish-brown
				33		
35	7	15.6	116.6	35	SM/SC	SILTY SAND TO CLAYEY SAND, reddish-brown, very moist, dense, wet
						free water
				37		
				39		
40	7	20.2	112.4			
				41		END at 40 feet; No Caving; Fill to 4 feet; Free Water at 36 feet.

NOTE: On Plates A-1 and A-7
The stratification lines represent the approximate boundary between earth types; the transition may be gradual.

NOTE on Plates A-1 and A-7:
Borings 1 through 3 excavated with 18" bucket-auger

BORING LOG NUMBER 4

Drilling Date April 19, 1991

Elevation 381'

Project KB 14298-S Communication Art House

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

BORING LOG NUMBER 4 (continued)

Drilling Date ______ **Elevation** ______

Project KB 14298-S Communication Art House

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

BORING LOG NUMBER 4 (continued)

Drilling Date

Elevation

Project KB 14298-S Communication Art House

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

BORING LOG NUMBER 4 (continued)

Drilling Date

Elevation

Project KB 14298-S Communication Art House

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

BORING LOG NUMBER 5

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

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

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

BORING LOG NUMBER 5 (continued)

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

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

Jerry Kovacs and Associates

BORING LOG NUMBER 5 (continued)

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

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

BORING LOG NUMBER 5 (continued)

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

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

BORING LOG NUMBER 6

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

Elevation: 365.0'
Sunset Heights, LLC

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

BORING LOG NUMBER 6 (continued)

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

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

BORING LOG NUMBER 6 (continued)

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

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

BORING LOG NUMBER 6 (continued)

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

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

BORING LOG NUMBER 7

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

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

km

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

BORING LOG NUMBER 7

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

km

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

BORING LOG NUMBER 7

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

km

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

BORING LOG NUMBER 7

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

km

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

BORING LOG NUMBER 8

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

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

km

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

BORING LOG NUMBER 8

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

km

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

BORING LOG NUMBER 8

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

km

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

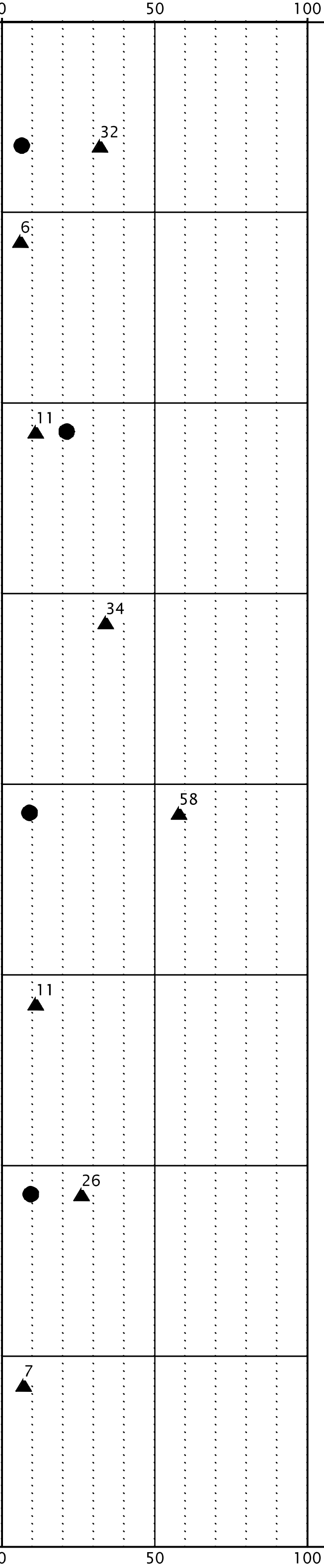
BORING LOG NUMBER 8

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

km

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

DEPTH FEET	GRAPHIC LOG	MATERIAL DESCRIPTION	ELEVATION DEPTH	TESTING	SAMPLE	▲ BLOW COUNT ● MOISTURE CONTENT % ▥ RQD% ▨ CORE REC%	INSTALLATION AND COMMENTS
0		Loose, brown, fine to medium, silty SAND; dry.	375.0				
				DD		32	DD = 117 pcf
5						6	
10				DD DS		11	DD = 97 pcf
		Hard, red-brown, fine to medium, sandy CLAY; dry.	361.0 14.0				
15						34	
20		becomes very stiff at 20.0 feet				58	
		Medium dense, red-brown, fine to medium, silty SAND with some clay; dry.	351.5 23.5				
25						11	
30		some alternating silt layers at 30.0 feet				26	
35		becomes very loose and moist at 35.0 feet				7	⩤ 04/06/06 Water measured at 37.5 feet at 8:40 AM.
40		becomes loose and wet with increasing clay at 39.5 feet					



DRILLED BY: 2-R Drilling, Inc.

LOGGED BY: LAS

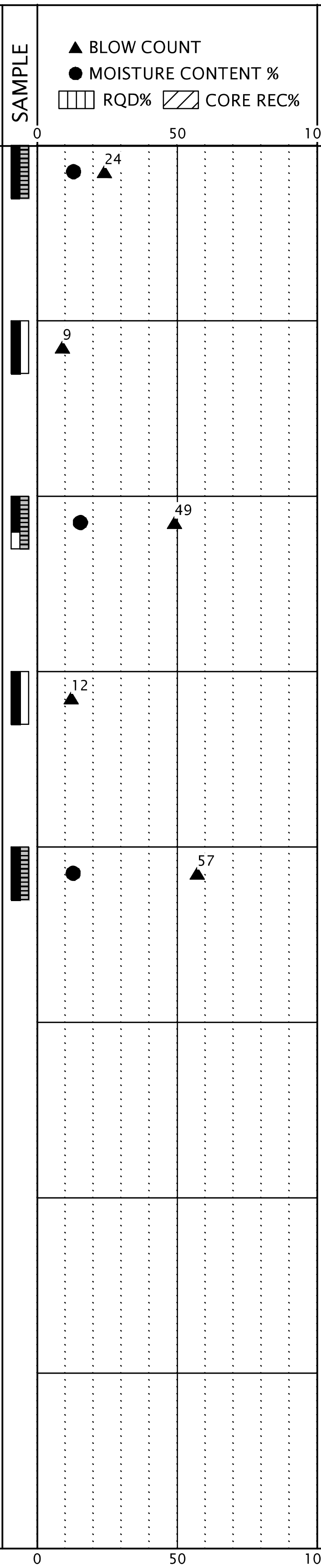
COMPLETED: 04/06/06

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

BORING BIT DIAMETER: 8.0 in

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

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



DRILLED BY: 2-R Drilling, Inc.

LOGGED BY: LAS

COMPLETED: 04/06/06

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

BORING BIT DIAMETER: 8.0 in

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

JAMESHOTEL-1-01

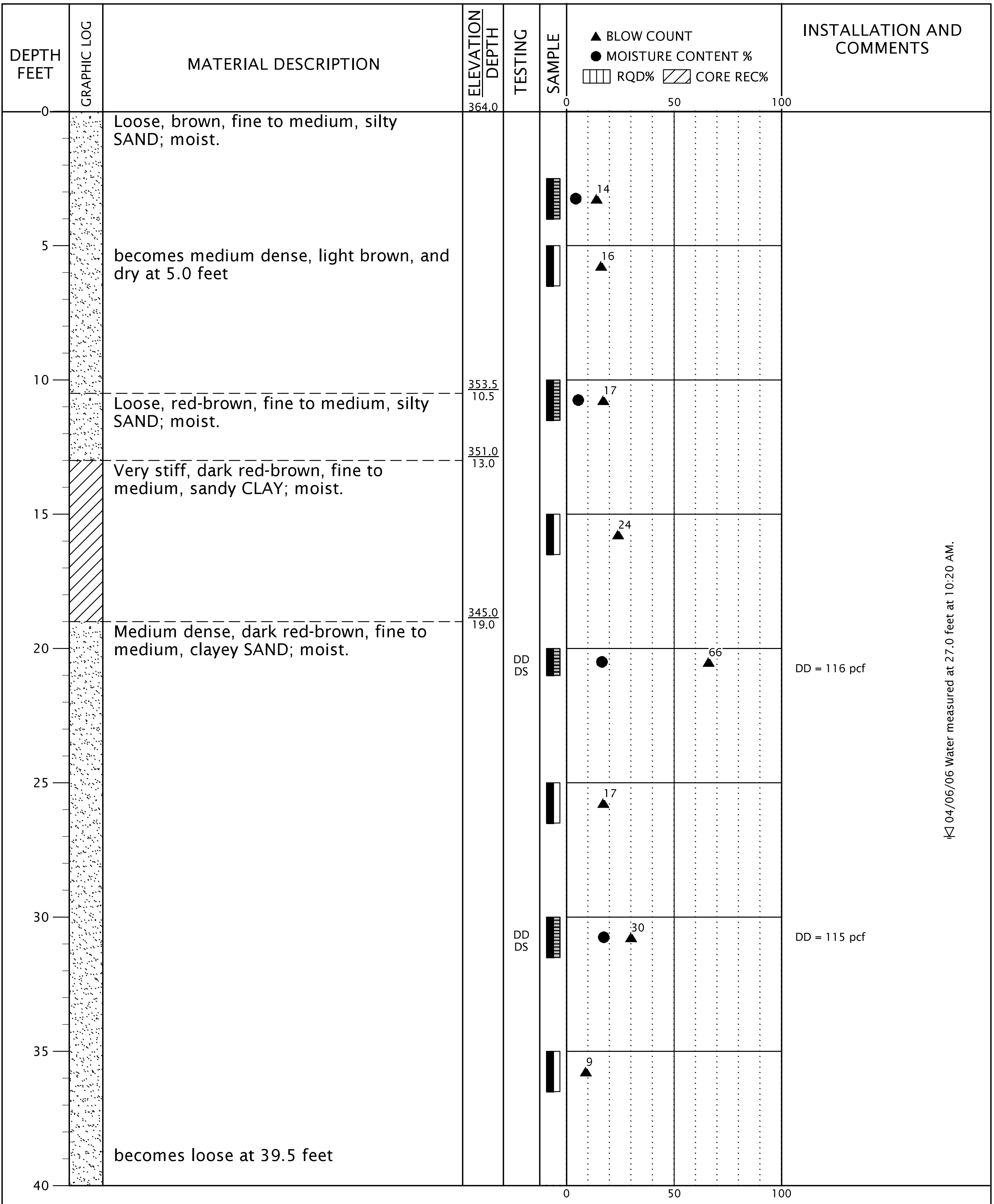
JUNE 2006

BORING B-9
(continued)

JAMES HOTEL
WEST HOLLYWOOD, CA

FIGURE A-1

BORING LOG JAMESHOTEL-1-01-B9&10.GPJ GEODESIGN.GDT PRINT DATE: 6/15/06:KYK



DEPTH FEET	MATERIAL DESCRIPTION	ELEVATION DEPTH
0	Loose, brown, fine to medium, silty SAND; moist.	364.0
5	becomes medium dense, light brown, and dry at 5.0 feet	
10.5	Loose, red-brown, fine to medium, silty SAND; moist.	353.5 10.5
13.0	Very stiff, dark red-brown, fine to medium, sandy CLAY; moist.	351.0 13.0
19.0	Medium dense, dark red-brown, fine to medium, clayey SAND; moist.	345.0 19.0
39.5	becomes loose at 39.5 feet	

DEPTH FEET	TESTING	BLOW COUNT	INSTALLATION AND COMMENTS
2.5		14	
5		16	
10		17	
15		24	
20	DD DS	66	DD = 116 pcf
25		17	
30	DD DS	30	DD = 115 pcf
35		9	

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

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

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

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

JAMESHOTEL-1-01

BORING B-10

JUNE 2006

JAMES HOTEL
WEST HOLLYWOOD, CA

FIGURE A-2

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

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

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

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

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

JAMESHOTEL-1-01

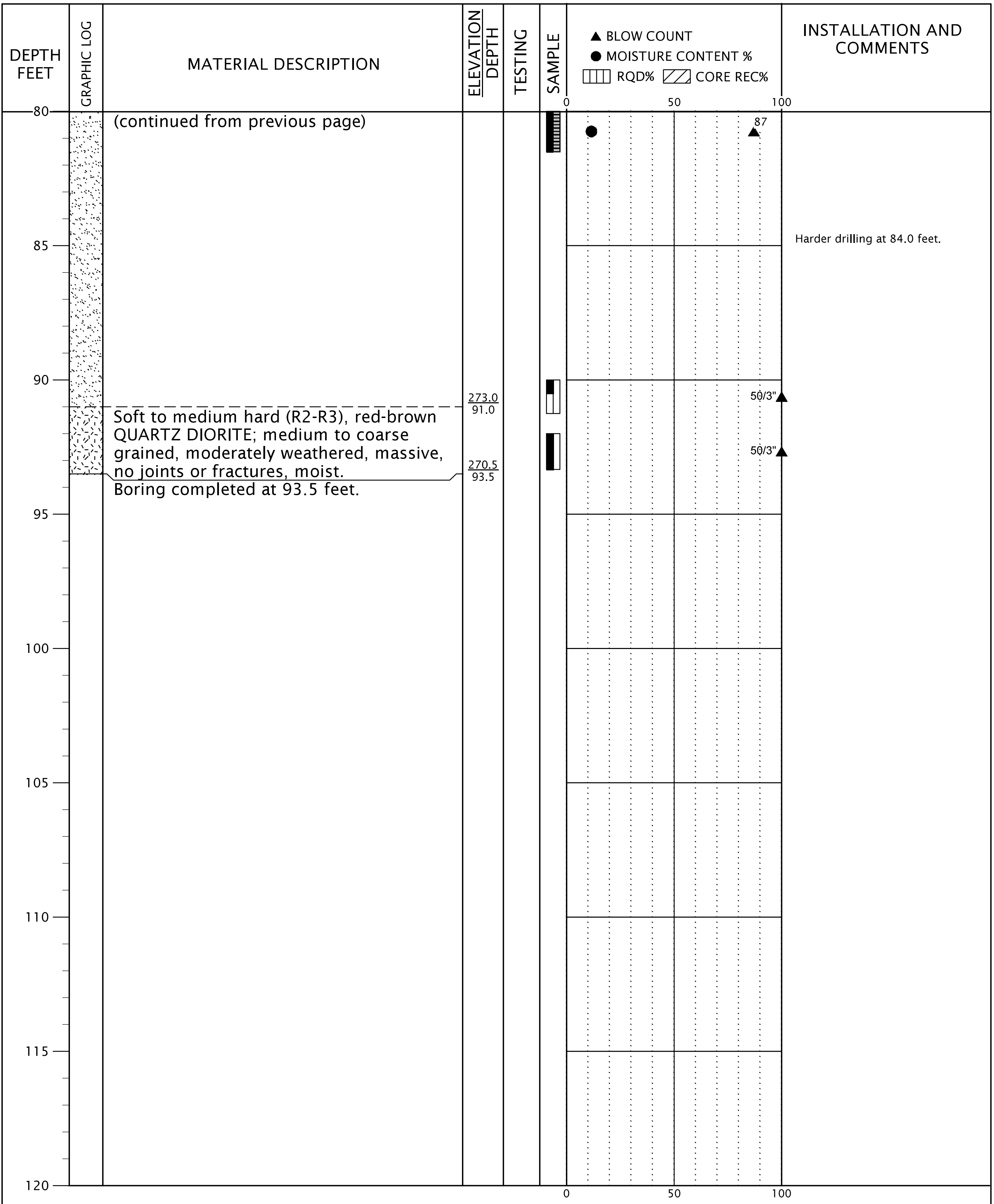
JUNE 2006

BORING B-10
(continued)

JAMES HOTEL
WEST HOLLYWOOD, CA

FIGURE A-2

BORING LOG JAMESHOTEL-1-01-B9&10.GPJ GEODESIGN.GDT PRINT DATE: 6/15/06:KYK



(continued from previous page)

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

Boring completed at 93.5 feet.

Harder drilling at 84.0 feet.

DRILLED BY: 2-R Drilling, Inc.

LOGGED BY: LAS

COMPLETED: 04/06/06

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

BORING BIT DIAMETER: 8.0 in

GEODESIGN INC

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

JAMESHOTEL-1-01

JUNE 2006

BORING B-10
(continued)

JAMES HOTEL
WEST HOLLYWOOD, CA

FIGURE A-2

GEOCON
WEST, INC.
GEOTECHNICAL ■ ENVIRONMENTAL ■ MATERIALS

Project No. A9286-06-01
May 24, 2016

VIA OVERNIGHT & E-MAIL

8920 Sunset Boulevard LLC
c/o VE Equities
250 Bowery, 2nd Floor
New York, New York 1002

Attention: Mr. Zach Vella

Subject: RESPONSE TO CITY OF WEST HOLLYWOOD REVIEW COMMENTS
PROPOSED MIXED-USE DEVELOPMENT
8920 SUNSET BOULEVARD,
WEST HOLLYWOOD, CALIFORNIA

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

City of West Hollywood Geotechnical, Geology, and Seismic Review Sheet, KFMg Project No. cWH 16-43E, dated January 28, 2016.

Dear Mr. Vella:

This letter has been prepared in response to the referenced January 28, 2016 City of West Hollywood Geotechnical, Geology, and Seismic Review Sheet. A copy of all City correspondence is appended herein. The Review Letter indicates that 24 comments be addressed. Where differing the recommendations presented herein supersede those in the above referenced reports and addenda.

Comment 1: Section 1, page 1: The Consultants present a single deep boring to characterize the subsurface conditions at the site, while utilizing additional data from a site to the west which was reviewed and approved 9 years ago. Given the size of the site and in consideration of the proposed deep excavation, this level of exploration is not the standard of practice in southern California. A more thorough subsurface investigation consisting of additional borings and/or CPT's is required to characterize the site.

Response 1: Due to existing site conditions, subsurface exploration throughout the site is not feasible. The existing site is occupied by a two-story commercial building underlain by two levels of subterranean parking. Due to the existing subterranean levels, borings and CPTs within the footprint of the existing structure are not practical. During the May 30, 2015 exploration program, an attempt was made to drill a boring along Sunset Avenue. However, there are two electrical conduits running parallel to Sunset Boulevard that preclude the ability to drill exploratory borings or advance CPTs within Sunset Boulevard.

An additional boring was conducted as a part of the preparation of this response letter. The drilling was placed as far south as possible, while leaving the driveway entry for the existing commercial building operational.

It is our opinion that the two borings conducted along Hilldale Avenue, along with supplemented with information from the adjacent sites to the west and south, provide sufficient information to characterize the anticipated subsurface conditions.

Comment 2: Section 6.4 Liquefaction Potential, page 9: Evaluation of liquefaction potential is an issue for this site. Although the site is not directly in the liquefaction hazard zone delineated by the California Geologic Survey (Beverly Hills Quad, OFR 98-14), it is immediately adjacent to the zone and the groundwater and blow count data from the single deep boring indicate a potential for liquefaction may exist. Low blow counts were also presented in Borings 9 and 10 (GeoDesign 2006). Please note that the previous investigation for the 1014 to 1020 Hilldale Avenue project, located just south of the subject project, encountered groundwater at depths of 15 to 18 feet. Please provide calculations for evaluation of the liquefaction potential of the on-site soils.

Response 2: In order to perform an evaluation of the liquefaction potential of the soils underlying the site, supplemental site exploration was performed on May 2, 2016, by excavating an 8-inch diameter boring utilizing a truck-mounted hollow-stem auger drilling machine to a depth of 83 feet below the existing ground surface. Representative and relatively undisturbed samples were obtained by driving a 3-inch, O. D., California Modified Sampler into the "undisturbed" soil mass with blows from a 140-pound auto-hammer falling 30 inches. The California Modified Sampler was equipped with 1-inch by 2³/₈-inch brass sampler rings to facilitate removal and testing. Standard Penetration Tests were also performed. The approximate location of the additional boring is depicted on the Site Plan (Figure 1).

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 supplemental boring is presented as Figure 2, appended herein. The log depicts the soil and geologic conditions encountered and the depth at which samples were obtained.

Liquefaction analysis of the soils underlying the site was performed using an updated version of the spreadsheet template LIQ2_30.WQ1 developed by Thomas F. Blake (1996). This program utilizes the 1996 NCEER method of analysis. This semi-empirical method is based on a correlation between values of Standard Penetration Test (SPT) resistance and field performance data. Laboratory test results used in the liquefaction analysis are presented as Figure 3.

The liquefaction analysis was performed for a Design Earthquake level by using a historic high groundwater table of 22 feet below the ground surface, a magnitude 6.68 earthquake, and a peak horizontal acceleration of 0.63g (⅔PGA$_M$). The enclosed liquefaction analyses, included herein for boring B3, indicates that the alluvial soils below the proposed subterranean structure could be susceptible to approximately 0.6 inches of settlement during Design Earthquake ground motion (see enclosed calculation sheets, Figures 4 and 5).

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

The liquefaction analysis was also performed for the Maximum Considered Earthquake level by using a historic high groundwater table of 22 feet below the ground surface, a magnitude 6.71 earthquake, and a peak horizontal acceleration of 0.948g (PGA_M). The enclosed liquefaction analyses, included herein for boring B3, indicates that the alluvial soils below the proposed subterranean structure could be susceptible to approximately 0.9 inches of settlement during Maximum Considered Earthquake ground motion (see enclosed calculation sheets, 6 and 7).

These settlements are in addition to static settlements and must be considered in the structural design. Updated Foundation Settlement recommendations are provided in the response to Comment 11.

Comment 3: Section 7.1.9, page 13: The topography at the project site slopes to the south-southeast and the existing grades at the site range from about 374 feet to 355 feet MSL. Please clarify the datum for the historic high groundwater level of 22 feet below the ground surface, or provide the groundwater table elevations in lieu of depth below the ground surface. Please note that the previous investigation for the 1014 to 1020 Hilldale Avenue project, located just south of the subject project, encountered groundwater at depths of 15 to 18 feet.

Response 3: As a part of the preparation of this response letter, the City of West Hollywood provided a copy of the following report for our review of the groundwater data contained therein:

> *Geological Fault Study, Proposed Residential Buildings, 1016, 1018, 1020 Hilldale Avenue, West Hollywood, California*, prepared by Advanced Geotechniques, dated July 8, 1998.

Based on our review of the prior report for 1016-1020 Hilldale Avenue, groundwater was encountered at depths ranging from 26 to 30 feet below the ground surface.

Groundwater levels encountered during exploration of 8920 Sunset Boulevard, 8950 Sunset Boulevard, and 1016-1020 Hilldale Avenue are summarized in the following tables.

Summary of Groundwater Levels 8920 Sunset Boulevard

Boring Number	Drilled/Measured By	Date	Depth to Groundwater (feet)	Groundwater Elevation (above MSL)
B2	Geocon	May 2015	38.0	332.0
B3	Geocon	May 2016	36.0	332.0

Summary of Groundwater Levels 8950 Sunset Boulevard

Boring Number	Drilled/Measured By	Date	Depth to Groundwater (feet)	Groundwater Elevation (above MSL)
B-1	Kovacs-Byer	August 1986	38.0	337.0
B-2	Kovacs-Byer	August 1986	29.0	335.0
B-3	Kovacs-Byer	August 1986	36.0	335.0
B-4	Kovacs-Byer	April 1991	46.0	335.0
B-5	Jerry Kovacs	November 1999	53.0	331.0
B-6	Jerry Kovacs	November 1999	34.0	331.0
B-7	Geotechnologies	June 2005	30.0	350.0
B-8	Geotechnologies	June 2005	22.5	343.5
B-9	GeoDesign	April 2006	37.5	339.5
B-10	GeoDesign	April 2006	27.0	338.0

Summary of Groundwater Levels 1016, 1018, 1020 Hilldale Avenue

Boring Number	Drilled/Measured By	Date	Depth to Groundwater (feet)	Groundwater Elevation (above MSL)
B-1	Advanced Geotechniques	April 1998	25.7	328.3
B-2	Advanced Geotechniques	April 1998	30	322.0
B-3	Advanced Geotechniques	April 1998	30	320.0
B-4	Advanced Geotechniques	April 1998	29	317.5
B-5	Advanced Geotechniques	April 1998	26	317.0

Considering the sloped nature of the site, the depth to groundwater encountered during exploration of the site and adjacent properties, and the historic high groundwater depth, it is recommended that the proposed structure be designed for a groundwater table corresponding to elevation 352.5 feet MSL at the northern extent of the property and 339 feet MSL at the southern extent of the property. These elevations are based on a historic high groundwater depth of 22 feet as measured from the highest ground surface elevation at the northern and southern extents of the site. The recommended design groundwater elevation is located above the previously encountered groundwater depths.

Comment 4: Section 7.1.13, page 14: The recommendations are somewhat ambiguous. The report states that footings may be placed on 12 inches of newly placed fill or on undisturbed alluvium. What about areas of existing undocumented fill? Is removal to alluvium required? Provisions for dealing with existing fill should be specifically included. This recommendation is repeated several times in the text (7.1.14, 7.6.8, etc.).

Response 4: Complete removal of all existing artificial fill is not required for the construction of miscellaneous footings, provided the project owner understands and accepts that miscellaneous footings constructed over existing uncertified fill or unsuitable alluvium material may experience increased settlement and/or cracking, and may therefore have a shorter design life and increased maintenance costs.

Comment 5: Section 7.2 Temporary Dewatering, page 14: (a) Provide minimum ground water level below the excavated subgrade for design of the temporary dewatering system; (b) Comment on potential effects of temporary dewatering on the adjacent buildings, structures, and public right of way.

Response 5: It is our opinion that the design of the temporary dewatering system, including the determination of the minimum depth that the temporary dewatering shall be effective to, should be performed by a qualified dewatering consultant. It is also our opinion that the qualified dewatering consultant should provide commentary on the potential effects of the temporary dewatering based on the design of the temporary dewatering system that will be implemented.

Comment 6: Section 7.3.1 Permanent Dewatering, page 15: Update the section with regard to the groundwater depth (i.e., 22 feet below the ground surface) based on Comment 3. The potential effects of permanent dewatering (potential ground settlement, on-going maintenance and replacement requirements, clogging issues, etc.) should be discussed.

Response 6: Based on correspondence with the project team, the proposed structure will be designed to withstand hydrostatic pressures; permeant dewatering will not be used.

Comment 7: Section 7.4.1 Soil and Excavation Characteristics, page 16: Please comment on soil type (category) per CalOSHA.

Response 7: The soils underlying the site may be classified as a CalOsha Type C soil.

Comment 8: Section 7.6.7 Grading, page 17: It is submitted for your consideration that all fill and backfill soils be moisture conditioned to at least optimum moisture content.

Response 8: The intention of our recommendations is for all fill and backfill soils to be moisture conditioned to at least optimum moisture content.

Comment 9: Section 7.7.2 Foundation Design, page 19: Update hydrostatic pressure based on Comment 3.

Response 9: Based on our response to Comment 3, paragraph 7.7.2 from the July 2, 2015 geotechnical report is revised below:

7.7.2 If a permanent dewatering system is not implemented then the structure must be designed for hydrostatic pressure based on a groundwater elevation of 352.5 feet MSL at the northern extent of the property and 339 feet MSL at the southern extent of the property. The hydrostatic design will result in uplift forces on the slab that that must be resisted by structural design. The recommended floor slab uplift pressure to be used in design would be 62.4(H) in units of pounds per square foot, where "H" is the height of the water above the bottom of the mat foundation in feet. Based on these considerations, a reinforced concrete mat foundation system deriving support in the competent alluvium is recommended for support of the proposed structure.

Comment 10: Section 7.9.5 Mat Foundation Design, page 20: Update design groundwater table per Comment 3.

Response 10: Based on our response to Comment 3, paragraph 7.9.5 from the July 2, 2015 geotechnical report is revised below:

7.9.5 If the proposed structure is to be designed for full hydrostatic pressure, the recommended floor slab uplift pressure to be used in design would be 62.4(H) in units of pounds per square foot, where "H" is the height of the water above the bottom of the mat foundation in feet. For design purposes the water table may be assumed at an elevation of 352.5 feet MSL at the northern extent of the property and at an elevation of 339 feet MSL at the southern extent of the property.

Comment 11: Section 7.10.1 Foundation Settlement, page 21: Maximum differential settlement is presented as being larger than maximum total settlement. Please review and modify as needed.

Response 11: Paragraph 7.10.1 should be revised to read:

7.10.1 The maximum expected settlement for the structure supported on a foundation system deriving support in the competent alluvium found at and below a depth of 35 feet and with a maximum allowable bearing pressure of 8,000 psf is estimated to be less than 1½ inches and occur below the heaviest loaded structural element. Differential settlement is not expected to exceed ¾ inch over a distance of twenty feet, or between the center and corner of the mat.

Additional foundation settlement recommendations are needed based on the results of the liquefaction analysis. These additional recommendations are provided below:

> The enclosed liquefaction settlement analysis indicates that the site soils could be prone to approximately 0.6 inches of total settlement as a result of the Design Earthquake peak ground acceleration (⅔PGA_M). Based on seismic considerations, proposed structure should be designed for a combined static and seismically-induced differential settlement of 1 inch over a distance of twenty feet, or between the center and corner of the mat.

Comment 12: Section 7.10.2 Foundation Settlement, page 21: Update the uplift pressure based on groundwater level per Comment 3.

Response 12: Based on our response to Comment 3, paragraph 7.10.2 from the July 2, 2015 geotechnical report is revised below:

7.10.2 If a permanent dewatering system is not installed to relieve hydrostatic pressure and buoyancy, an uplift pressure of 62.4(H) in units of pounds per square foot (where "H" is the height of the water above the bottom of the mat foundation in feet) must be mitigated based on a groundwater elevation of 352.5 feet MSL at the northern extent of the property and at an elevation of 339 feet MSL at the southern extent of the property. Recommendations for resistance to uplift are provided in Section 7.11.

Comment 13: Section 7.11.12, Uplift Resistance, page 22: State whether the friction resistance is an allowable or ultimate value.

Response 13: The uplift capacity provided in paragraph 7.11.2 is an allowable capacity.

Comment 14: Section 7.16.2 Dynamic (Seismic) Lateral Forces, page 26: Provide calculations for the recommended seismic loads of 25 pcf. For evaluating seismic earth pressures, the Consultant must consider the procedures described in the publication "Seismic Earth Pressures on Retaining Structures in Cohesionless Soils" by Mikola and Sitar (March 2013, Report No.2 UCB GT 13-01).

It should be noted that in order to maintain consistency between projects and to provide a clear guidance to the geotechnical engineers practicing in the City of West Hollywood the procedures outlined by Sitar et al. will be adopted for review of projects in the City. Consultant's recommendations will be compared against the results yielded by the Sitar et al. procedures. It should be noted that the Sitar et al. procedures provide means to reflect the measure of risk by utilizing the mean and upper-bound correlations which should be considered by the Consultant depending on the structure. For analysis purposes, the free field PGA may be taken as SDS/2.5.

Response 14: Acknowledged. Calculation of the recommended seismic earth pressure has been revised based on the recommendations of Mikola and Sitar.

A seismic load of 44 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 2013 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 a free field PGA of $S_{DS}/2.5$ (0.65g) and a mean dynamic earth pressure coefficient of 0.57 (Mikola, 2013).

Comment 15: Section 7.17.1 Retaining Wall Drainage, page 26: Why limit the drainage system to the bottom 2/3 of wall? What about potential surface infiltration.

Response 15: Acknowledged. It is recommended that retaining wall drainage be provided for the full height of the wall.

Comment 16: Section 7.20.2 Temporary Excavation, page 28: Reference is made to sloped embankments, is this supposed to be excavation slopes or are fill embankments actually proposed?

Response 16: Based on the current set of project plans, no embankments or excavation slopes are proposed. The project will use a shoring system to provide a stable excavation.

Comment 17: Section 7.21.16 Shoring, page 31: Please provide calculations or references for the recommended values of frictional resistance between the soldier pile and the soils below the excavated level.

Response 17: Calculation of the recommended skin friction resistance for temporary shoring piles is provided below:

PROJECT NAME:	8920 Sunset	
PROJECT NUMBER:	A9286-06-01	
SHEAR SAMPLE:	**B2@40**	
COHESION =	410	psf
PHI =	32	degrees
FTG WIDTH =	1	feet
FTG DEPTH =	1.5	feet
DENSITY =	138	pcf
Soil or Bedrock	Soil	
Groundwater (Y/N)	Yes	
Effective Unit Weight =	75.6	pcf
Ny =	23.5	
Nq =	24.0	
Nc =	37.0	

SKIN FRICTION		
DRILLED, CONCRETE PILES		
Qu = Ca(πDL) + Khc(½yD)(TAN(¾Φ)(πDL)		
FILL DEPTH =	35	FEET
PILE DEPTH=	55	FEET
LENGTH =	20	FEET
PILE DIAM =	1	FEET
SAFETY FACTOR=		1.5
SKIN FRICTION(PSF) =		711.3
FRICT. CAP (KIPS) =		44.7

Comment 18: Section 7.21.19, page 32: (a) Provide calculations for active earth pressure of 33 pcf and at-rest pressure of 21H (this at-rest pressure value appears to be in error); (b) The equivalent fluid pressures presented in the table imply a triangular pressure distribution, yet a trapezoidal distribution is shown graphically. Update trapezoidal distribution graphic showing the maximum pressure ordinate; (c) explain the difference between active earth pressure of 33 pcf and the recommended active earth pressure of 40 pcf for retaining wall in Section 7.15.3; (d) explain the differences between at-rest pressure of 21H and the recommended at-rest earth pressure of 60 pcf in Section 7.15.4 and at-rest earth pressure of 53 pcf in Section 7.21.10.

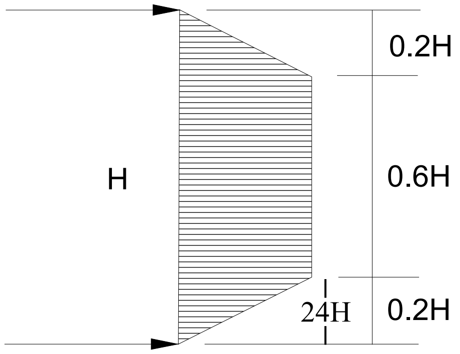
Response 18:

a) Calculation of the recommended shoring pressures are provided below. The calculation has been updated to take into account the in-situ moisture content of the soils, based on the consideration that temporary dewatering will be used. The table under paragraph 7.21.19 incorrectly referred to the trapezoidal distribution of the active pressure as an at-rest pressure. The recommended at-rest pressure is provided in paragraph 7.21.20 of the referenced July 2, 2015 report. Paragraphs 7.21.19 and 7.21.20 are restated below, with corrections, for clarity:

7.21.19 Assuming that a temporary dewatering system is implemented just outside the shoring system, and that pumping is continuously maintained throughout the excavation and construction process, it is recommended that an equivalent fluid pressure based on the following table, be utilized for design. A diagram depicting the trapezoidal pressure distribution of lateral earth pressure is provided below the table.

HEIGHT OF SHORING (FEET)	EQUIVALENT FLUID PRESSURE (Pounds Per Cubic Foot) (ACTIVE PRESSURE)	EQUIVALENT FLUID PRESSURE (Pounds Per Square Foot per Foot) Active Trapezoidal (Where H is the height of the shoring in feet)
Up to 55	38	24H

Trapezoidal Distribution of Pressure



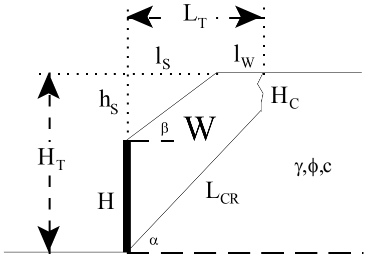
7.21.20 It is very important to note that active pressures can only be achieved when movement in the soil (earth wall) occurs. If movement in the soil is not acceptable, such as adjacent to an existing structure, or the pile is restrained from movement by bracing or a tie back anchor, an at-rest pressure of 61 pcf should be considered for design purposes.

Project Name: 8920 Sunset
Project No.: A9286-06-01
Description: B2@40'

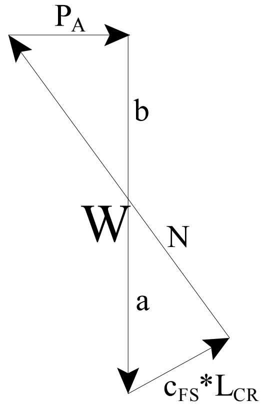
Shoring Design with Transitioned Backfill
(Vector Analysis)

Input:

Shoring Height	(H)	55.00	feet
Slope Angle of Backfill	(β)	0.0	degrees
Height of Slope above Shoring	(h_s)	0.0	feet
Horizontal Length of Slope	(l_s)	0.0	feet
Total Height (Shoring + Slope)	(H_T)	55.0	feet
Unit Weight of Retained Soils	(γ)	135.7	pcf
Friction Angle of Retained Soils	(ϕ)	32.0	degrees
Cohesion of Retained Soils	(c)	410.0	psf
Factor of Safety	(FS)	1.25	
Factored Parameters:	(ϕ_{FS})	26.6	degrees
	(c_{FS})	328.0	psf



Failure Angle (α) 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	9.7	1466	198867.5	64.1	59463.0	139404.5	46481.1
46	9.4	1418	192436.5	63.5	55938.3	136498.1	48175.0
47	9.1	1372	186145.3	62.8	52749.3	133396.0	49714.9
48	8.8	1327	179996.0	62.1	49854.4	130141.5	51106.1
49	8.6	1282	173988.0	61.4	47218.6	126769.5	52353.5
50	8.5	1239	168119.1	60.8	44811.4	123307.7	53461.6
51	8.3	1197	162385.5	60.1	42607.0	119778.5	54434.1
52	8.2	1156	156783.0	59.4	40582.9	116200.1	55274.7
53	8.1	1115	151306.7	58.8	38719.6	112587.1	55986.1
54	8.0	1076	145951.3	58.1	37000.1	108951.2	56570.9
55	7.9	1037	140711.7	57.5	35409.7	105302.0	57031.0
56	7.9	999	135582.5	56.9	33935.3	101647.2	57368.2
57	7.8	962	130558.3	56.2	32565.4	97992.9	57583.5
58	7.8	926	125634.1	55.6	31290.0	94344.1	57677.7
59	7.8	890	120804.5	55.0	30099.9	90704.6	57651.2
60	7.8	855	116064.8	54.4	28987.2	87077.6	57503.8
61	7.9	821	111409.9	53.9	27944.6	83465.3	57235.0
62	7.9	787	106835.3	53.3	26965.5	79869.8	56843.9
63	8.0	754	102336.2	52.7	26044.1	76292.2	56329.2
64	8.1	722	97908.4	52.2	25174.9	72733.6	55689.0
65	8.2	689	93547.5	51.6	24352.9	69194.6	54921.1
66	8.4	658	89249.3	51.0	23573.6	65675.7	54022.9
67	8.5	627	85009.7	50.5	22832.6	62177.0	52991.2
68	8.7	596	80824.6	49.9	22125.8	58698.8	51822.4
69	8.9	565	76690.1	49.3	21449.1	55241.0	50512.3
70	9.2	535	72602.2	48.7	20798.5	51803.6	49056.4



Design Equations (Vector Analysis):

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

$b = W-a$

$P_A = b*\tan(\alpha-\phi_{FS})$

$EFP = 2*P_A/H^2$

Maximum Active Pressure Resultant

$P_{A,\,max}$ 57677.72 lbs/lineal foot

Equivalent Fluid Pressure (per lineal foot of shoring)

$EFP = 2*P_A/H^2$

EFP 38.1 pcf 61.3 pcf

Design Shoring for an Equivalent Fluid Pressure: **38 pcf** Active **61 pcf** At-Rest

(b) As described in the response to part (a), the table below paragraph 7.21.19 had incorrectly labeled the trapezoidal pressure as at-rest. This has been corrected in the response to part (a).

(c) & (d) Different active and at-rest earth pressures are provided for the permanent retaining wall design and the temporary shoring design. For permanent retaining wall design a factor of safety of 1.5 was used and for temporary shoring a factor of safety of 1.25 was used.

Comment 19: Section 7.21.22 Surcharge Pressure, page 33: The report recommends that if surcharge is farther than 10 feet, it can be neglected from lateral pressure calculations. Please corroborate if this recommendation is applicable without limitation on the depth of the excavation for this project. It is submitted for your consideration that the traffic surcharge may be neglected if the traffic is kept back at least ten feet from the shoring wall and a distance from the shoring wall equal to at least half the wall height, whichever is greater.

Response 19: Acknowledged. Paragraphs 7.15.8, 7.21.22, and 7.26.4 are revised below:

7.15.8 In addition to the recommended earth pressure, the upper ten 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 ten feet from the subterranean walls or a distance from the subterranean walls equal to at least half the wall height, whichever is greater, the traffic surcharge may be neglected.

7.21.22 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 or a distance from the shoring equal to at least half the shoring height, whichever is greater, the traffic surcharge may be neglected.

7.26.4 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 or a distance from the shoring equal to at least half the shoring height, whichever is greater, the traffic surcharge may be neglected.

Comment 20: Section 7.22 Tie-Back Anchors, page 34: Please comment / verify: (a) will tie-back anchors be installed as permanent anchors? (b) Are there any property line encroachment or right-of-way issues for the anticipated anchor lengths?

Response 20: Based on conversations with the project team, tiebacks will be used for temporary support of the shoring; permanent anchors will not be used. The owner is responsible for obtaining agreements for installation of temporary tie-backs which extend beyond the property lines.

Comment 21: Section 7.24 Anchor Testing, page 35: It is submitted for your consideration that a sanctioned formal tieback testing procedure (e.g., FHWA, PTI, Caltrans, City of Los Angeles) be referenced in lieu of the testing descriptions in the report.

Response 21: Acknowledged.

Comment 22: Section 7.25 Internal Bracing, page 35: (a) Please provide calculations for the recommended shoring allowable bearing capacity of 4,000 psf for the rakers inclined at 45 degrees (the value seems very high for a footing base at that inclination). Raker footing design is typically governed by passive pressure limits); (b) provide any groundwater depth requirements to achieve the recommended allowable bearing capacity.

Response 22: Calculation of the raker bearing pressure is provided below. Groundwater should be maintained at a depth of at least 3 feet below the lowest portion of the raker footing.

COHESION =	410	psf
PHI =	32	degrees
FTG WIDTH =	1	feet
FTG DEPTH =	1.5	feet
DENSITY =	138	pcf
Soil or Bedrock	Soil	
Groundwater (Y/N)	No	
Effective Unit Weight =	138	pcf
Ny =	23.5	
Nq =	24.0	
Nc =	37.0	

RAKER FOOTING		
Width (B)=	2	FT
DEPTH (D) =	1	FT
Raker Inclination	45	Degrees
bc =	0.69	
bq =	0.26	
by =	0.26	
Qu = (CNcbc)+(yDNqbq)+(½yBNyby)		
Qu=	12209	psf
SAFETY FACTOR=	3	
Qall =	**4070**	psf

Comment 23: Section 7.26.4 Surcharge Pressure, page 37: Revise per Comment 19.

Response 23: Paragraph 7.26.4 has been revised as a part of the response to comment 19.

Comment 24: Paving Recommendations: Please provide paving recommendations for anticipated paving and/or hardscape at the site.

Response 24: Asphalt paving is not currently proposed as a part of the project. Recommendations for exterior concrete slab-on-grade (hardscape) are provided in paragraph 7.14.15 on page 24 of the referenced July 2, 2015 soils report.

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

Sincerely,

GEOCON WEST, INC.

JELISA M. THOMAS
74946

SUSAN FRANZEN KIRKGARD
No. 1754

Jelisa M. Thomas
PE 74946

Susan F. Kirkgard
CEG 1754

(3 + CD) Addressee (Please submit to City of West Hollywood)

Enclosures: City of West Hollywood Geotechnical, Geology, and Seismic Review Sheet
Figure 1, Site Plan
Figure 2, Boring Log
Figure 3, Grain Size Analysis
Figures 4 and 5, DE Empirical Estimation of Liquefaction Potential
Figures 6 and 7, MCE Empirical Estimation of Liquefaction Potential

City of West Hollywood
California 1984

CITY OF
WEST HOLLYWOOD

COMMUNITY DEVELOPMENT DEPARTMENT
PLANNING DIVISION

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

GEOTECHNICAL, GEOLOGY, AND SEISMIC REVIEW SHEET

Site Address: 8920 Sunset Blvd **KFMg Project No.:** cWH 16-43E
Lot/Block/Tract: NA
Owner: 8920 Sunset Boulevard, LLLC
Project Type: Geotechnical engineering evaluation for a new 8-story mixed-use structure with 4-stories of subterranean parking extending up to 55 feet to bottom of foundations.

Geotechnical Engineer: Geocon West, Inc. (Berliner, GE 2576)
Engineering Geologist: Geocon West, Inc. (Kirkgard, CEG 1754)
Report Dated: July 2, 2015 Project No. A9286-06-01)

ACTION:

___ Recommend **APPROVAL**

X Request **ADDITIONAL DATA** for review to address the comments listed below prior to approval

COMMENTS:

1. Section 1, page 1: The Consultants present a single deep boring to characterize the subsurface conditions at the site, while utilizing additional data from a site to the west which was reviewed and approved 9 years ago. Given the size of the site and in consideration of the proposed deep excavation, this level of exploration is not the standard of practice in southern California. A more thorough subsurface investigation consisting of additional borings and/or CPT’s is required to characterize the site.

2. Section 6.4 Liquefaction Potential, page 9: Evaluation of liquefaction potential is an issue for this site. Although the site is not directly in the liquefaction hazard zone delineated by the California Geologic Survey (Beverly Hills Quad, OFR 98-14), it is immediately adjacent to the zone and the groundwater and blow count data from the single deep boring indicate a potential for liquefaction may exist. Low blow counts were also presented in Borings 9 and 10 (GeoDesign 2006). Please

note that the previous investigation for the 1014 to 1020 Hilldale Avenue project, located just south of the subject project, encountered groundwater at depths of 15 to 18 feet. Please provide calculations for evaluation of the liquefaction potential of the on-site soils.

3. Section 7.1.9, page 13: The topography at the project site slopes to the south-southeast and the existing grades at the site range from about 374 feet to 355 feet MSL. Please clarify the datum for the historic high groundwater level of 22 feet below the ground surface, or provide the groundwater table elevations in lieu of depth below the ground surface. Please note that the previous investigation for the 1014 to 1020 Hilldale Avenue project, located just south of the subject project, encountered groundwater at depths of 15 to 18 feet.

4. Section 7.1.13, page 14: The recommendations are somewhat ambiguous. The report states that footings may be placed on 12 inches of newly placed fill or on undisturbed alluvium. What about areas of existing undocumented fill? Is removal to alluvium required? Provisions for dealing with existing fill should be specifically included. This recommendation is repeated several times in the text (7.1.14, 7.6.8, etc.).

5. Section 7.2 Temporary Dewatering, page 14: (a) Provide minimum ground water level below the excavated subgrade for design of the temporary dewatering system; (b) Comment on potential effects of temporary dewatering on the adjacent buildings, structures, and public right of way.

6. Section 7.3.1 Permanent Dewatering, page 15: Update the section with regard to the groundwater depth (i.e., 22 feet below the ground surface) based on Comment 3. The potential effects of permanent dewatering (potential ground settlement, on-going maintenance and replacement requirements, clogging issues, etc.) should be discussed.

7. Section 7.4.1 Soil and Excavation Characteristics, page 16: Please comment on soil type (category) per CalOSHA.

8. Section 7.6.7 Grading, page 17: It is submitted for your consideration that all fill and backfill soils be moisture conditioned to at least optimum moisture content.

9. Section 7.7.2 Foundation Design, page 19: Update hydrostatic pressure based on Comment 3.

10. Section 7.9.5 Mat Foundation Design, page 20: Update design groundwater table per Comment 3.

11. Section 7.10.1 Foundation Settlement, page 21: Maximum differential settlement is presented as being larger than maximum total settlement. Please review and modify as needed.

12. Section 7.10.2 Foundation Settlement, page 21: Update the uplift pressure based on groundwater level per Comment 3.

13. Section 7.11.12, Uplift Resistance, page 22: State whether the friction resistance is an allowable or ultimate value.

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23. Section 7.26.4 Surcharge Pressure, page 37: Revise per Comment 19.

24. Paving Recommendations: Please provide paving recommendations for anticipated paving and/or hardscape at the site.

CLOSURE:

This document and all associated written comments and responses are a part of the geotechnical design documentation for this project. It is the responsibility of the Geotechnical Consultant to distribute these and associated written comments and responses to the appropriate parties.

Edward H Sabins
Reviewed by: ______________________ Date: January 28, 2016
Edward H. Sabins, CEG 1571
Engineering Geology Reviewer

Reviewed by: ______________________ Date: January 28, 2016
Douglas R. Bell, GE 2140
Geotechnical Engineering Reviewer

CITY OF
WEST HOLLYWOOD

COMMUNITY DEVELOPMENT DEPARTMENT
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GEOTECHNICAL, GEOLOGY, AND SEISMIC REVIEW SHEET

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Engineering Geologist: Geocon West, Inc. (Kirkgard, CEG 1754)
Report Dated: July 2, 2015 Project No. A9286-06-01)

ACTION:

___ Recommend **APPROVAL**

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COMMENTS:

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6. Section 7.3.1 Permanent Dewatering, page 15: Update the section with regard to the groundwater depth (i.e., 22 feet below the ground surface) based on Comment 3. The potential effects of permanent dewatering (potential ground settlement, on-going maintenance and replacement requirements, clogging issues, etc.) should be discussed.

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8. Section 7.6.7 Grading, page 17: It is submitted for your consideration that all fill and backfill soils be moisture conditioned to at least optimum moisture content.

9. Section 7.7.2 Foundation Design, page 19: Update hydrostatic pressure based on Comment 3.

10. Section 7.9.5 Mat Foundation Design, page 20: Update design groundwater table per Comment 3.

11. Section 7.10.1 Foundation Settlement, page 21: Maximum differential settlement is presented as being larger than maximum total settlement. Please review and modify as needed.

12. Section 7.10.2 Foundation Settlement, page 21: Update the uplift pressure based on groundwater level per Comment 3.

13. Section 7.11.12, Uplift Resistance, page 22: State whether the friction resistance is an allowable or ultimate value.

14. Section 7.16.2 Dynamic (Seismic) Lateral Forces, page 26: Provide calculations for the recommended seismic loads of 25 pcf. For evaluating seismic earth pressures, the Consultant must consider the procedures described in the publication "Seismic Earth Pressures on Retaining Structures in Cohesionless Soils" by Mikola and Sitar (March 2013, Report No.2 UCB GT 13-01).

    It should be noted that in order to maintain consistency between projects and to provide a clear guidance to the geotechnical engineers practicing in the City of West Hollywood the procedures outlined by Sitar et al. will be adopted for review of projects in the City. Consultant's recommendations will be compared against the results yielded by the Sitar et al. procedures. It should be noted that the Sitar et al. procedures provide means to reflect the measure of risk by utilizing the mean and upper-bound correlations which should be considered by the Consultant depending on the structure. For analysis purposes, the free field PGA may be taken as SDS/2.5.

15. Section 7.17.1 Retaining Wall Drainage, page 26: Why limit the drainage system to the bottom 2/3 of wall? What about potential surface infiltration.

16. Section 7.20.2 Temporary Excavation, page 28: Reference is made to sloped embankments, is this supposed to be excavation slopes or are fill embankments actually proposed?

17. Section 7.21.16 Shoring, page 31: Please provide calculations or references for the recommended values of frictional resistance between the soldier pile and the soils below the excavated level.

18. Section 7.21.19, page 32: (a) Provide calculations for active earth pressure of 33 pcf and at-rest pressure of 21H (this at-rest pressure value appears to be in error); (b) The equivalent fluid pressures presented in the table imply a triangular pressure distribution, yet a trapezoidal distribution is shown graphically. Update trapezoidal distribution graphic showing the maximum pressure ordinate; (c) explain the difference between active earth pressure of 33 pcf and the recommended active earth pressure of 40 pcf for retaining wall in Section 7.15.3; (d) explain the differences between at-rest pressure of 21H and the recommended at-rest earth pressure of 60 pcf in Section 7.15.4 and at-rest earth pressure of 53 pcf in Section 7.21.10.

19. Section 7.21.22 Surcharge Pressure, page 33: The report recommends that if surcharge is farther than 10 feet, it can be neglected from lateral pressure calculations. Please corroborate if this recommendation is applicable without limitation on the depth of the excavation for this project. It is submitted for your consideration that the traffic surcharge may be neglected if the traffic is kept back at least ten feet from the shoring wall <u>and a distance from the shoring wall equal to at least half the wall height, whichever is greater.</u>

20. Section 7.22 Tie-Back Anchors, page 34: Please comment / verify: (a) will tie-back anchors be installed as permanent anchors? (b) Are there any property line encroachment or right-of-way issues for the anticipated anchor lengths?

21. Section 7.24 Anchor Testing, page 35: It is submitted for your consideration that a sanctioned formal tieback testing procedure (e.g., FHWA, PTI, Caltrans, City of Los Angeles) be referenced in lieu of the testing descriptions in the report.

22. Section 7.25 Internal Bracing, page 35: (a) Please provide calculations for the recommended shoring allowable bearing capacity of 4,000 psf for the rakers inclined at 45 degrees (the value seems very high for a footing base at that inclination). Raker footing design is typically governed by passive pressure limits); (b) provide any groundwater depth requirements to achieve the recommended allowable bearing capacity.

23. Section 7.26.4 Surcharge Pressure, page 37: Revise per Comment 19.

24. Paving Recommendations: Please provide paving recommendations for anticipated paving and/or hardscape at the site.

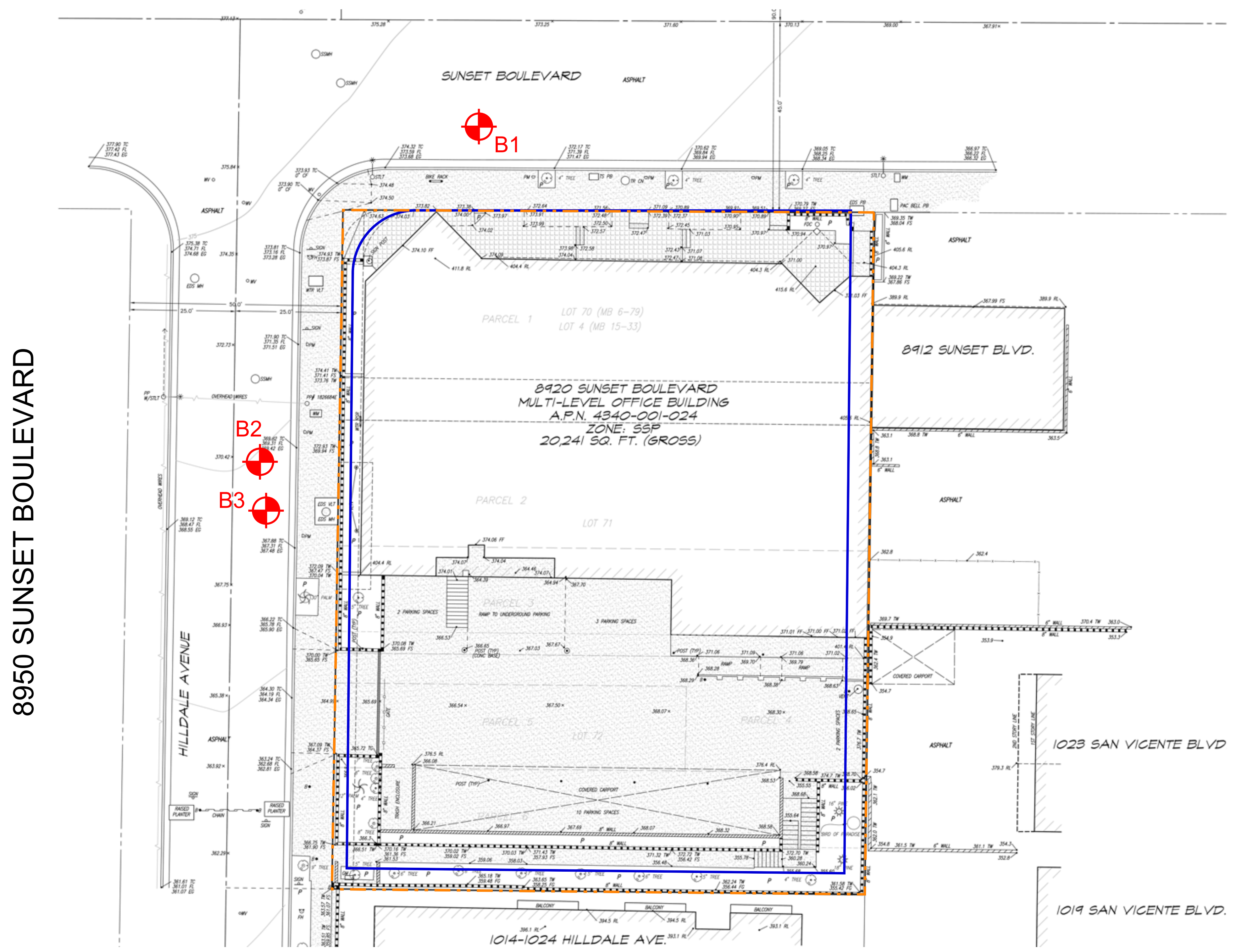
CLOSURE:

This document and all associated written comments and responses are a part of the geotechnical design documentation for this project. It is the responsibility of the Geotechnical Consultant to distribute these and associated written comments and responses to the appropriate parties.

Edward H Sabins

Reviewed by: ______________________________ Date: January 28, 2016
Edward H. Sabins, CEG 1571
Engineering Geology Reviewer

Reviewed by: ______________________________ Date: January 28, 2016
Douglas R. Bell, GE 2140
Geotechnical Engineering Reviewer



LEGEND

- B3 Approximate Location of Borings
- Approximate Location of Proposed Development

0 30' 60'

GEOCON
W E S T, I N C.

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

DRAFTED BY: JMT	CHECKED BY: SKF

SITE PLAN

8920 SUNSET BOULEVARD LLC
8920 SUNSET BOULEVARD
WEST HOLLYWOOD, CALIFORNIA

MAY 2016	PROJECT NO. A9286-06-01	FIG. 1

PROJECT NO. A9286-06-01

BORING 3

ELEV. (MSL.) **368** DATE COMPLETED **5/2/16**

EQUIPMENT **HOLLOW STEM AUGER** BY: **RMA**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0					**ARTIFICIAL FILL** Sand, poorly graded, medium dense, slightly moist, yellowish brown, fine- to medium-grained, trace rootlets, trace silt.			
2								
4								
6								
8								
10	B3@10'				**ALLUVIUM** Sand with Silt, poorly graded, medium dense, slightly moist, dark yellowish brown, fine- to medium-grained.	20	101.1	4.9
12								
14				SP-SM				
16								
18								
20	B3@20'				Sand with Silt, poorly graded, dense, slightly moist, brown, fine- to medium-grained, some coarse-grained, trace clay.	79	116.1	11.6
22								
24				SP-SM				
26								
28								

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

SAMPLE SYMBOLS

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

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

GEOCON

PROJECT NO. A9286-06-01

BORING 3

ELEV. (MSL.) **368** DATE COMPLETED **5/2/16**

EQUIPMENT **HOLLOW STEM AUGER** BY: **RMA**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
30	B3@30'			SM	Silty Sand, poorly graded, very dense, slightly moist, dark reddish brown, fine- to medium-grained, some coarse-grained, trace to some clay.	50 (6")	116.9	13.6
32	B3@32.5'					32	--	--
				SP-SC	Sand with Clay, dense, slightly moist, dark reddish brown, fine- to medium-grained, some coarse-grained, trace silt.			
34								
	B3@35'		▼		- very dense	50 (3")	122.1	15.3
36								
	B3@37.5'				- decrease in clay content, medium dense	24	--	--
38								
40	B3@40'			SM	Silty Sand, poorly graded, dense, wet, dark reddish brown, fine- to medium-grained, some coarse-grained, trace to some clay.	36	115.0	16.0
42	B3@42.5'					10	--	--
				SP-SC	Sand with Clay, poorly graded, medium dense, wet, dark reddish brown, fine- to medium-grained, some coarse-grained, moderate plasticity, some silt.			
44								
	B3@45'			SC	Clayey Sand, poorly graded, medium dense, wet, dark reddish brown, fine- to medium-grained, some coarse-grained, some silt.	35	117.6	15.3
46								
	B3@47.5'					22	--	--
48				SP-SC	Sand with Clay, poorly graded, medium dense, slightly moist to wet, dark reddish brown, fine- to medium-grained, some coarse-grained, some silt.			
50	B3@50'			SC	Clayey Sand, poorly graded, dense, wet, fine- to medium-grained, some coarse-grained, moderate plasticity.	34	123.2	14.2
52	B3@52.5'				- medium dense	23	--	--
54								
	B3@55'			CL	Clay with Sand, hard, dark reddish brown, fine- to medium-grained, trace coarse-grained, some silt.	62	108.4	13.4
56								
	B3@57.5'				- dense	31	--	--
58				SP	Sand, poorly graded, wet, medium dense, dark reddish brown, medium- to coarse-grained, trace fine-grained, friable.			

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

A9286-06-01 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. A9286-06-01

BORING 3

ELEV. (MSL.) **368** DATE COMPLETED **5/2/16**

EQUIPMENT **HOLLOW STEM AUGER** BY: **RMA**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
60	B3@60' ■			SP	Sand with Clay, poorly graded, dense, dark reddish brown, fine- to medium-grained, trace coarse-grained, some silt, moderate plasticity.	47	120.5	14.8
62	B3@62.5' ◧				- decrease in clay content, increase in silt content	45	--	--
64								
	B3@65' ■			SP-SM	Sand with Silt, poorly graded, very dense, wet, reddish brown to yellowish brown, fine- to medium-grained, trace to some clay, friable.	79	111.0	17.6
66								
	B3@67.5' ◧				- medium dense, trace coarse-grained sand	28	--	--
68								
70	B3@70' ■					50 (5")	122.5	15.5
72	B3@72.5' ◧					79	--	--
74								
	B3@75' ■			SP	Sand, poorly graded, very dense, wet, light brown, fine- to medium-grained, trace to some silt, friable.	50 (3")	108.4	17.7
76								
	B3@77.5' ◧				- increase in medium-grained sand content, very dense	50 (6")	--	--
78								
80	B3@80' ■				- medium- to coarse-grained sand	50 (5")	108.7	17.9
82	B3@82.5' ◧				- very dense	50 (4")	--	--

Total depth of boring: 83 feet
Fill to 9.5 feet.
Groundwater encountered at 36 feet.
Backfilled with Portland cement/bentonite mix.
Patched with black dye concrete.

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

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

A9286-06-01 BORING LOGS.GPJ

SAMPLE SYMBOLS			
	☐ ... SAMPLING UNSUCCESSFUL	◧ ... STANDARD PENETRATION TEST	■ ... DRIVE SAMPLE (UNDISTURBED)
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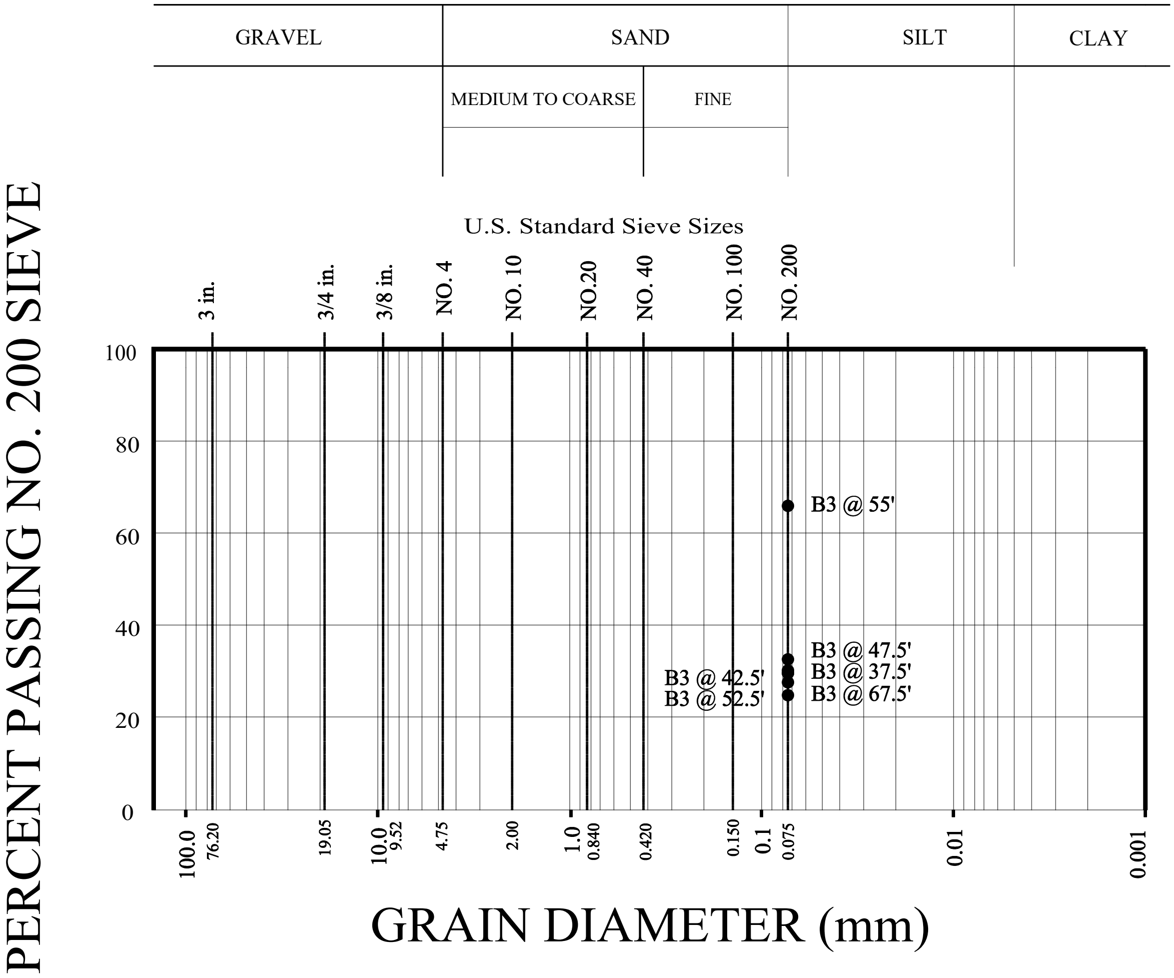

SAMPLE	PERCENT PASSING NO. 200 SIEVE
B3 @ 37.5'	30.1
B3 @ 42.5'	29.6
B3 @ 47.5'	32.4
B3 @ 52.5'	27.5
B3 @ 55'	65.9
B3 @ 67.5'	24.7

GEOCON
WEST, INC.

ENVIRONMENTAL GEOTECHNICAL MATERIALS
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PHONE (818) 841-8388 - FAX (818) 841-1704

Drafted by: JMT | Checked by: NDB

GRAIN SIZE ANALYSIS

8920 SUNSET BOULEVARD LLC
8920 SUNSET BOULEVARD
WEST HOLLYWOOD, CALIFORNIA

MAY 2016 | PROJECT NO. A9286-06-01 | FIG. 3

Client : 8920 Sunset, LLC
File No. : A9286-06-01
Boring : B3

EMPIRICAL ESTIMATION OF LIQUEFACTION POTENTIAL
DESIGN EARTHQUAKE

NCEER (1996) METHOD

By Thomas F. Blake (1994-1996)

EARTHQUAKE INFORMATION:

Earthquake Magnitude:	6.68
Peak Horiz. Acceleration PGA_M (g):	0.948
2/3 PGA_M (g):	0.632
Calculated Mag.Wtg.Factor:	0.747
Historic High Groundwater:	22.0
Groundwater Depth During Exploration:	36.0

ENERGY & ROD CORRECTIONS:

Energy Correction (CE) for N60:	1.25
Rod Len.Corr.(CR)(0-no or 1-yes):	1.0
Bore Dia. Corr. (CB):	1.15
Sampler Corr. (CS):	1.20
Use Ksigma (0 or 1):	1.0

LIQUEFACTION CALCULATIONS:

Unit Wt. Water (pcf): 62.4

Depth to Base (ft)	Total Unit Wt. (pcf)	Water (0 or 1)	FIELD SPT (N)	Depth of SPT (ft)	Liq.Sus. (0 or 1)	-200 (%)	Est. Dr (%)	CN Factor	Corrected (N1)60	Eff. Unit Wt. (psf)	Resist. CRR	rd Factor	Induced CSR	Liquefac. Safe.Fact.
1.0	107.0	0	11.0	1.0	0			2.000	28.5	107.0	~	0.998	0.306	~
2.0	107.0	0	11.0	2.0	0			2.000	28.5	107.0	~	0.993	0.305	~
3.0	107.0	0	11.0	3.0	0			2.000	28.5	107.0	~	0.989	0.304	~
4.0	107.0	0	11.0	4.0	0			2.000	28.5	107.0	~	0.984	0.302	~
5.0	107.0	0	11.0	5.0	0			2.000	28.5	107.0	~	0.979	0.301	~
6.0	107.0	0	11.0	6.0	0			1.884	26.8	107.0	~	0.975	0.299	~
7.0	107.0	0	11.0	7.0	0			1.733	24.7	107.0	~	0.970	0.298	~
8.0	107.0	0	11.0	8.0	0			1.613	23.0	107.0	~	0.966	0.297	~
9.0	107.0	0	11.0	9.0	0			1.515	21.6	107.0	~	0.961	0.295	~
10.0	107.0	0	11.0	10.0	0			1.433	20.4	107.0	~	0.957	0.294	~
11.0	107.0	0	11.0	10.0	0			1.364	19.4	107.0	~	0.952	0.292	~
12.0	107.0	0	11.0	10.0	0			1.303	18.5	107.0	~	0.947	0.291	~
13.0	107.0	0	11.0	10.0	0			1.250	17.8	107.0	~	0.943	0.290	~
14.0	107.0	0	11.0	10.0	0			1.203	17.1	107.0	~	0.938	0.288	~
15.0	107.0	0	11.0	10.0	0			1.160	16.5	107.0	~	0.934	0.287	~
16.0	107.0	0	11.0	10.0	0			1.122	16.0	107.0	~	0.929	0.285	~
17.0	107.0	0	11.0	10.0	0			1.088	15.5	107.0	~	0.925	0.284	~
18.0	107.0	0	11.0	10.0	0			1.056	15.0	107.0	~	0.920	0.282	~
19.0	107.0	0	11.0	10.0	0			1.027	14.6	107.0	~	0.915	0.281	~
20.0	129.6	0	40.0	20.0	0			0.998	61.6	129.6	~	0.911	0.280	~
21.0	129.6	0	40.0	20.0	0			0.968	59.8	129.6	~	0.906	0.278	~
22.0	129.6	1	40.0	20.0	0			0.941	58.1	67.2	~	0.902	0.281	~
23.0	129.6	1	40.0	20.0	0			0.917	56.6	67.2	~	0.897	0.286	~
24.0	129.6	1	40.0	20.0	0			0.894	55.2	67.2	~	0.893	0.291	~
25.0	129.6	1	40.0	20.0	0			0.872	53.9	67.2	~	0.888	0.296	~
26.0	129.6	1	40.0	20.0	0			0.852	52.6	67.2	~	0.883	0.301	~
27.0	129.6	1	40.0	20.0	0			0.834	51.5	67.2	~	0.879	0.305	~
28.0	129.6	1	40.0	20.0	0			0.816	50.4	67.2	~	0.874	0.308	~
29.0	129.6	1	40.0	20.0	0			0.800	49.4	67.2	~	0.870	0.312	~
30.0	129.6	1	40.0	20.0	0			0.785	48.4	67.2	~	0.865	0.315	~
31.0	132.8	1	32.0	32.5	1	30	87	0.770	48.3	70.4	Infin.	0.861	0.318	Non-Liq.
32.0	132.8	1	32.0	32.5	1	30	87	0.756	47.6	70.4	Infin.	0.856	0.320	Non-Liq.
33.0	140.8	1	32.0	32.5	1	30	87	0.742	46.8	78.4	Infin.	0.851	0.322	Non-Liq.
34.0	140.8	1	32.0	32.5	1	30	87	0.729	46.1	78.4	Infin.	0.847	0.324	Non-Liq.
35.0	140.8	1	32.0	32.5	1	30	87	0.716	45.4	78.4	Infin.	0.842	0.326	Non-Liq.
36.0	140.8	1	32.0	32.5	1	30	87	0.706	44.8	78.4	Infin.	0.838	0.328	Non-Liq.
37.0	140.8	1	24.0	37.5	1	30	73	0.700	34.8	78.4	Infin.	0.833	0.329	Non-Liq.
38.0	140.8	1	24.0	37.5	1	30	73	0.694	34.6	78.4	Infin.	0.829	0.330	Non-Liq.
39.0	140.8	1	24.0	37.5	1	30	73	0.687	34.3	78.4	Infin.	0.824	0.331	Non-Liq.
40.0	133.4	1	24.0	37.5	1	30	73	0.682	34.1	71.0	Infin.	0.819	0.332	Non-Liq.
41.0	133.4	1	24.0	37.5	1	30	73	0.676	33.9	71.0	Infin.	0.815	0.333	Non-Liq.
42.5	133.4	1	24.0	37.5	1	30	73	0.670	33.6	71.0	Infin.	0.809	0.334	Non-Liq.
43.0	133.4	1	10.0	42.5	1	30	46	0.667	17.3	71.0	0.172	0.805	0.333	0.52
44.0	133.4	1	10.0	42.5	1	30	46	0.661	17.2	71.0	0.171	0.801	0.334	0.51
45.0	133.4	1	10.0	42.5	1	30	46	0.656	17.2	71.0	0.170	0.797	0.335	0.51
46.0	135.6	1	22.0	47.5	1	32	66	0.651	31.1	73.2	Infin.	0.792	0.335	Non-Liq.
47.0	135.6	1	22.0	47.5	1	32	66	0.647	30.9	73.2	Infin.	0.787	0.335	Non-Liq.
48.0	135.6	1	22.0	47.5	1	32	66	0.642	30.8	73.2	Infin.	0.783	0.335	Non-Liq.
49.0	135.6	1	22.0	47.5	1	32	66	0.637	30.6	73.2	Infin.	0.778	0.335	Non-Liq.
50.0	135.6	1	22.0	47.5	1	32	66	0.633	30.4	73.2	Infin.	0.774	0.335	Non-Liq.
51.0	140.7	1	23.0	52.5	1	28	65	0.628	30.3	78.3	Infin.	0.769	0.335	Non-Liq.
52.0	140.7	1	23.0	52.5	1	28	65	0.624	30.1	78.3	Infin.	0.765	0.334	Non-Liq.
53.0	140.7	1	23.0	52.5	1	28	65	0.619	29.9	78.3	0.419	0.760	0.334	1.26
54.0	140.7	1	23.0	52.5	1	28	65	0.615	29.8	78.3	0.390	0.755	0.333	1.17
55.0	140.7	1	23.0	52.5	1	28	65	0.610	29.6	78.3	0.372	0.751	0.332	1.12
56.0	122.9	1	31.0	55.0	1	66	75	0.607	39.4	60.5	Infin.	0.746	0.332	Non-Liq.
57.0	122.9	1	31.0	55.0	1	66	75	0.603	39.3	60.5	Infin.	0.742	0.332	Non-Liq.
58.0	138.3	1	31.0	55.0	1	66	75	0.600	39.1	75.9	Infin.	0.737	0.331	Non-Liq.
59.0	138.3	1	31.0	55.0	1	66	75	0.596	38.9	75.9	Infin.	0.733	0.330	Non-Liq.
60.0	138.3	1	31.0	55.0	1	66	75	0.592	38.7	75.9	Infin.	0.728	0.329	Non-Liq.
61.0	138.3	1	45.0	62.5	1		86	0.588	45.7	75.9	Infin.	0.723	0.328	Non-Liq.
62.0	138.3	1	45.0	62.5	1		86	0.585	45.4	75.9	Infin.	0.719	0.327	Non-Liq.
63.0	138.3	1	45.0	62.5	1		86	0.581	45.1	75.9	Infin.	0.714	0.326	Non-Liq.
64.0	138.3	1	45.0	62.5	1		86	0.578	44.8	75.9	Infin.	0.710	0.325	Non-Liq.
65.0	129.4	1	28.0	67.5	1	25	66	0.574	32.4	67.0	Infin.	0.705	0.324	Non-Liq.
66.0	129.4	1	28.0	67.5	1	25	66	0.571	32.3	67.0	Infin.	0.701	0.323	Non-Liq.
67.0	129.4	1	28.0	67.5	1	25	66	0.568	32.1	67.0	Infin.	0.696	0.322	Non-Liq.
68.0	129.4	1	28.0	67.5	1	25	66	0.565	32.0	67.0	Infin.	0.691	0.321	Non-Liq.
69.0	129.4	1	28.0	67.5	1	25	66	0.563	31.8	67.0	Infin.	0.687	0.320	Non-Liq.
70.0	129.4	1	28.0	67.5	1	25	66	0.560	31.7	67.0	Infin.	0.682	0.319	Non-Liq.
71.0	141.5	1	79.0	72.5	1	25	109	0.557	80.5	79.1	Infin.	0.678	0.318	Non-Liq.
72.0	141.5	1	79.0	72.5	1	25	109	0.554	80.1	79.1	Infin.	0.673	0.316	Non-Liq.
73.0	141.5	1	79.0	72.5	1	25	109	0.550	79.7	79.1	Infin.	0.669	0.315	Non-Liq.
74.0	141.5	1	79.0	72.5	1	25	109	0.547	79.2	79.1	Infin.	0.664	0.313	Non-Liq.
75.0	141.5	1	79.0	72.5	1		109	0.544	74.1	79.1	Infin.	0.659	0.312	Non-Liq.

Figure 4

GEOCON

Client : 8920 Sunset, LLC
File No. : A9286-06-01
Boring : B3

LIQUEFACTION SETTLEMENT ANALYSIS
DESIGN EARTHQUAKE

(SATURATED SAND AT INITIAL LIQUEFACTION CONDITION)

NCEER (1996) METHOD
EARTHQUAKE INFORMATION:

Earthquake Magnitude:	6.68
PGAM (g):	0.948
2/3 PGAM (g):	0.63
Calculated Mag.Wtg.Factor:	0.747
Historic High Groundwater:	22.0
Groundwater @ Exploration:	36.0

DEPTH TO BASE	BLOW COUNT N	WET DENSITY (PCF)	TOTAL STRESS O (TSF)	EFFECT STRESS O' (TSF)	REL. DEN. Dr (%)	ADJUST BLOWS (N1)60	Tav/σ'_o	LIQUEFACTION SAFETY FACTOR	Volumetric Strain [e_{15}} (%)	EQ. SETTLE. Pe (in.)
1	11	106.959	0.027	0.027		28	0.411	~	0.00	0.00
2	11	106.959	0.080	0.080		28	0.411	~	0.00	0.00
3	11	106.959	0.134	0.134		28	0.411	~	0.00	0.00
4	11	106.959	0.187	0.187		28	0.411	~	0.00	0.00
5	11	106.959	0.241	0.241		28	0.411	~	0.00	0.00
6	11	106.959	0.294	0.294		27	0.411	~	0.00	0.00
7	11	106.959	0.348	0.348		25	0.411	~	0.00	0.00
8	11	106.959	0.401	0.401		23	0.411	~	0.00	0.00
9	11	106.959	0.455	0.455		22	0.411	~	0.00	0.00
10	11	106.959	0.508	0.508		20	0.411	~	0.00	0.00
11	11	106.959	0.562	0.562		19	0.411	~	0.00	0.00
12	11	106.959	0.615	0.615		19	0.411	~	0.00	0.00
13	11	106.959	0.668	0.668		18	0.411	~	0.00	0.00
14	11	106.959	0.722	0.722		17	0.411	~	0.00	0.00
15	11	106.959	0.775	0.775		17	0.411	~	0.00	0.00
16	11	106.959	0.829	0.829		16	0.411	~	0.00	0.00
17	11	106.959	0.882	0.882		15	0.411	~	0.00	0.00
18	11	106.959	0.936	0.936		15	0.411	~	0.00	0.00
19	11	106.959	0.989	0.989		15	0.411	~	0.00	0.00
20	40	129.5676	1.049	1.049		62	0.411	~	0.00	0.00
21	40	129.5676	1.113	1.113		60	0.411	~	0.00	0.00
22	40	129.5676	1.178	1.162		58	0.417	~	0.00	0.00
23	40	129.5676	1.243	1.196		57	0.427	~	0.00	0.00
24	40	129.5676	1.308	1.230		55	0.437	~	0.00	0.00
25	40	129.5676	1.372	1.263		54	0.447	~	0.00	0.00
26	40	129.5676	1.437	1.297		53	0.456	~	0.00	0.00
27	40	129.5676	1.502	1.330		51	0.464	~	0.00	0.00
28	40	129.5676	1.567	1.364		50	0.472	~	0.00	0.00
29	40	129.5676	1.632	1.398		49	0.480	~	0.00	0.00
30	40	129.5676	1.696	1.431		48	0.487	~	0.00	0.00
31	32	132.7984	1.762	1.466	87	48	0.494	Non-Liq.	0.00	0.00
32	32	132.7984	1.828	1.501	87	48	0.501	Non-Liq.	0.00	0.00
33	32	140.7813	1.897	1.538	87	47	0.507	Non-Liq.	0.00	0.00
34	32	140.7813	1.967	1.577	87	46	0.513	Non-Liq.	0.00	0.00
35	32	140.7813	2.038	1.616	87	45	0.518	Non-Liq.	0.00	0.00
36	32	140.7813	2.108	1.655	87	45	0.523	Non-Liq.	0.00	0.00
37	24	140.7813	2.178	1.695	73	35	0.528	Non-Liq.	0.00	0.00
38	24	140.7813	2.249	1.734	73	35	0.533	Non-Liq.	0.00	0.00
39	24	140.7813	2.319	1.773	73	34	0.538	Non-Liq.	0.00	0.00
40	24	133.4	2.388	1.810	73	34	0.542	Non-Liq.	0.00	0.00
41	24	133.4	2.454	1.846	73	34	0.546	Non-Liq.	0.00	0.00
42.5	24	133.4	2.538	1.890	73	34	0.552	Non-Liq.	0.00	0.00
43	10	133.4	2.571	1.908	46	17	0.554	0.52	1.70	0.20
44	10	133.4	2.654	1.952	46	17	0.559	0.51	1.70	0.20
45	10	133.4	2.721	1.988	46	17	0.563	0.51	1.70	0.20
46	22	135.5928	2.788	2.024	66	31	0.566	Non-Liq.	0.00	0.00
47	22	135.5928	2.856	2.061	66	31	0.570	Non-Liq.	0.00	0.00
48	22	135.5928	2.924	2.097	66	31	0.573	Non-Liq.	0.00	0.00
49	22	135.5928	2.992	2.134	66	31	0.576	Non-Liq.	0.00	0.00
50	22	135.5928	3.060	2.170	66	30	0.579	Non-Liq.	0.00	0.00
51	23	140.6944	3.129	2.208	65	30	0.582	Non-Liq.	0.00	0.00
52	23	140.6944	3.199	2.247	65	30	0.585	Non-Liq.	0.00	0.00
53	23	140.6944	3.269	2.287	65	30	0.588	1.26	0.00	0.00
54	23	140.6944	3.340	2.326	65	30	0.590	1.17	0.00	0.00
55	23	140.6944	3.410	2.365	65	30	0.593	1.12	0.00	0.00
56	31	122.9256	3.476	2.400	75	39	0.595	Non-Liq.	0.00	0.00
57	31	122.9256	3.537	2.430	75	39	0.598	Non-Liq.	0.00	0.00
58	31	138.334	3.603	2.464	75	39	0.601	Non-Liq.	0.00	0.00
59	31	138.334	3.672	2.502	75	39	0.603	Non-Liq.	0.00	0.00
60	31	138.334	3.741	2.540	75	39	0.605	Non-Liq.	0.00	0.00
61	45	138.334	3.810	2.578	86	46	0.608	Non-Liq.	0.00	0.00
62	45	138.334	3.879	2.616	86	45	0.610	Non-Liq.	0.00	0.00
63	45	138.334	3.949	2.654	86	45	0.612	Non-Liq.	0.00	0.00
64	45	138.334	4.018	2.692	86	45	0.613	Non-Liq.	0.00	0.00
65	28	129.36	4.085	2.727	66	32	0.616	Non-Liq.	0.00	0.00
66	28	129.36	4.149	2.761	66	32	0.618	Non-Liq.	0.00	0.00
67	28	129.36	4.214	2.794	66	32	0.620	Non-Liq.	0.00	0.00
68	28	129.36	4.279	2.828	66	32	0.622	Non-Liq.	0.00	0.00
69	28	129.36	4.343	2.861	66	32	0.624	Non-Liq.	0.00	0.00
70	28	129.36	4.408	2.895	66	32	0.626	Non-Liq.	0.00	0.00
71	79	141.4875	4.476	2.931	109	81	0.628	Non-Liq.	0.00	0.00
72	79	141.4875	4.546	2.971	109	80	0.629	Non-Liq.	0.00	0.00
73	79	141.4875	4.617	3.010	109	80	0.630	Non-Liq.	0.00	0.00
74	79	141.4875	4.688	3.050	109	79	0.632	Non-Liq.	0.00	0.00
75	79	141.4875	4.759	3.090	109	74	0.633	Non-Liq.	0.00	0.00

TOTAL SETTLEMENT = 0.6 INCHES

Figure 5

GEOCON

Client : 8920 Sunset, LLC
File No. : A9286-06-01
Boring : B3

EMPIRICAL ESTIMATION OF LIQUEFACTION POTENTIAL
MAXIMUM CONSIDERED EARTHQUAKE

NCEER (1996) METHOD

By Thomas F. Blake (1994-1996)

EARTHQUAKE INFORMATION:

Earthquake Magnitude:	6.71
Peak Horiz. Acceleration PGA_M (g):	0.948
Calculated Mag.Wtg.Factor:	0.756
Historic High Groundwater:	22.0
Groundwater Depth During Exploration:	36.0

ENERGY & ROD CORRECTIONS:

Energy Correction (CE) for N60:	1.25
Rod Len.Corr.(CR)(0-no or 1-yes):	1.0
Bore Dia. Corr. (CB):	1.15
Sampler Corr. (CS):	1.20
Use Ksigma (0 or 1):	1.0

LIQUEFACTION CALCULATIONS:

Unit Wt. Water (pcf):	62.4

Depth to Base (ft)	Total Unit Wt. (pcf)	Water (0 or 1)	FIELD SPT (N)	Depth of SPT (ft)	Liq.Sus. (0 or 1)	-200 (%)	Est. Dr (%)	CN Factor	Corrected (N1)60	Eff. Unit Wt. (psf)	Resist. CRR	rd Factor	Induced CSR	Liquefac. Safe.Fact.
1.0	107.0	0	11.0	1.0	0			2.000	28.5	107.0	~	0.998	0.465	~
2.0	107.0	0	11.0	2.0	0			2.000	28.5	107.0	~	0.993	0.462	~
3.0	107.0	0	11.0	3.0	0			2.000	28.5	107.0	~	0.989	0.460	~
4.0	107.0	0	11.0	4.0	0			2.000	28.5	107.0	~	0.984	0.458	~
5.0	107.0	0	11.0	5.0	0			2.000	28.5	107.0	~	0.979	0.456	~
6.0	107.0	0	11.0	6.0	0			1.884	26.8	107.0	~	0.975	0.454	~
7.0	107.0	0	11.0	7.0	0			1.733	24.7	107.0	~	0.970	0.452	~
8.0	107.0	0	11.0	8.0	0			1.613	23.0	107.0	~	0.966	0.450	~
9.0	107.0	0	11.0	9.0	0			1.515	21.6	107.0	~	0.961	0.448	~
10.0	107.0	0	11.0	10.0	0			1.433	20.4	107.0	~	0.957	0.445	~
11.0	107.0	0	11.0	10.0	0			1.364	19.4	107.0	~	0.952	0.443	~
12.0	107.0	0	11.0	10.0	0			1.303	18.5	107.0	~	0.947	0.441	~
13.0	107.0	0	11.0	10.0	0			1.250	17.8	107.0	~	0.943	0.439	~
14.0	107.0	0	11.0	10.0	0			1.203	17.1	107.0	~	0.938	0.437	~
15.0	107.0	0	11.0	10.0	0			1.160	16.5	107.0	~	0.934	0.435	~
16.0	107.0	0	11.0	10.0	0			1.122	16.0	107.0	~	0.929	0.433	~
17.0	107.0	0	11.0	10.0	0			1.088	15.5	107.0	~	0.925	0.431	~
18.0	107.0	0	11.0	10.0	0			1.056	15.0	107.0	~	0.920	0.428	~
19.0	107.0	0	11.0	10.0	0			1.027	14.6	107.0	~	0.915	0.426	~
20.0	129.6	0	40.0	20.0	0			0.998	61.6	129.6	~	0.911	0.424	~
21.0	129.6	0	40.0	20.0	0			0.968	59.8	129.6	~	0.906	0.422	~
22.0	129.6	1	40.0	20.0	0			0.941	58.1	67.2	~	0.902	0.426	~
23.0	129.6	1	40.0	20.0	0			0.917	56.6	67.2	~	0.897	0.434	~
24.0	129.6	1	40.0	20.0	0			0.894	55.2	67.2	~	0.893	0.442	~
25.0	129.6	1	40.0	20.0	0			0.872	53.9	67.2	~	0.888	0.449	~
26.0	129.6	1	40.0	20.0	0			0.852	52.6	67.2	~	0.883	0.456	~
27.0	129.6	1	40.0	20.0	0			0.834	51.5	67.2	~	0.879	0.462	~
28.0	129.6	1	40.0	20.0	0			0.816	50.4	67.2	~	0.874	0.468	~
29.0	129.6	1	40.0	20.0	0			0.800	49.4	67.2	~	0.870	0.473	~
30.0	129.6	1	40.0	20.0	0			0.785	48.4	67.2	~	0.865	0.478	~
31.0	132.8	1	32.0	32.5	1	30	87	0.770	48.3	70.4	Infin.	0.861	0.482	Non-Liq.
32.0	132.8	1	32.0	32.5	1	30	87	0.756	47.6	70.4	Infin.	0.856	0.486	Non-Liq.
33.0	140.8	1	32.0	32.5	1	30	87	0.742	46.8	78.4	Infin.	0.851	0.489	Non-Liq.
34.0	140.8	1	32.0	32.5	1	30	87	0.729	46.1	78.4	Infin.	0.847	0.492	Non-Liq.
35.0	140.8	1	32.0	32.5	1	30	87	0.716	45.4	78.4	Infin.	0.842	0.494	Non-Liq.
36.0	140.8	1	32.0	32.5	1	30	87	0.706	44.8	78.4	Infin.	0.838	0.497	Non-Liq.
37.0	140.8	1	24.0	37.5	1	30	73	0.700	34.8	78.4	Infin.	0.833	0.499	Non-Liq.
38.0	140.8	1	24.0	37.5	1	30	73	0.694	34.6	78.4	Infin.	0.829	0.500	Non-Liq.
39.0	140.8	1	24.0	37.5	1	30	73	0.687	34.3	78.4	Infin.	0.824	0.502	Non-Liq.
40.0	133.4	1	24.0	37.5	1	30	73	0.682	34.1	71.0	Infin.	0.819	0.503	Non-Liq.
41.0	133.4	1	24.0	37.5	1	30	73	0.676	33.9	71.0	Infin.	0.815	0.505	Non-Liq.
42.5	133.4	1	24.0	37.5	1	30	73	0.670	33.6	71.0	Infin.	0.809	0.506	Non-Liq.
43.0	133.4	1	10.0	42.5	1	30	46	0.667	17.3	71.0	0.172	0.805	0.505	0.34
44.0	133.4	1	10.0	42.5	1	30	46	0.661	17.2	71.0	0.171	0.801	0.507	0.34
45.0	133.4	1	10.0	42.5	1	30	46	0.656	17.2	71.0	0.170	0.797	0.508	0.33
46.0	135.6	1	22.0	47.5	1	32	66	0.651	31.1	73.2	Infin.	0.792	0.508	Non-Liq.
47.0	135.6	1	22.0	47.5	1	32	66	0.647	30.9	73.2	Infin.	0.787	0.508	Non-Liq.
48.0	135.6	1	22.0	47.5	1	32	66	0.642	30.8	73.2	Infin.	0.783	0.508	Non-Liq.
49.0	135.6	1	22.0	47.5	1	32	66	0.637	30.6	73.2	Infin.	0.778	0.508	Non-Liq.
50.0	135.6	1	22.0	47.5	1	32	66	0.633	30.4	73.2	Infin.	0.774	0.508	Non-Liq.
51.0	140.7	1	23.0	52.5	1	28	65	0.628	30.3	78.3	Infin.	0.769	0.507	Non-Liq.
52.0	140.7	1	23.0	52.5	1	28	65	0.624	30.1	78.3	Infin.	0.765	0.507	Non-Liq.
53.0	140.7	1	23.0	52.5	1	28	65	0.619	29.9	78.3	0.419	0.760	0.506	0.83
54.0	140.7	1	23.0	52.5	1	28	65	0.615	29.8	78.3	0.390	0.755	0.505	0.77
55.0	140.7	1	23.0	52.5	1	28	65	0.610	29.6	78.3	0.372	0.751	0.504	0.74
56.0	122.9	1	31.0	55.0	1	66	75	0.607	39.4	60.5	Infin.	0.746	0.503	Non-Liq.
57.0	122.9	1	31.0	55.0	1	66	75	0.603	39.3	60.5	Infin.	0.742	0.503	Non-Liq.
58.0	138.3	1	31.0	55.0	1	66	75	0.600	39.1	75.9	Infin.	0.737	0.502	Non-Liq.
59.0	138.3	1	31.0	55.0	1	66	75	0.596	38.9	75.9	Infin.	0.733	0.501	Non-Liq.
60.0	138.3	1	31.0	55.0	1	66	75	0.592	38.7	75.9	Infin.	0.728	0.499	Non-Liq.
61.0	138.3	1	45.0	62.5	1		86	0.588	45.7	75.9	Infin.	0.723	0.498	Non-Liq.
62.0	138.3	1	45.0	62.5	1		86	0.585	45.4	75.9	Infin.	0.719	0.496	Non-Liq.
63.0	138.3	1	45.0	62.5	1		86	0.581	45.1	75.9	Infin.	0.714	0.495	Non-Liq.
64.0	138.3	1	45.0	62.5	1		86	0.578	44.8	75.9	Infin.	0.710	0.493	Non-Liq.
65.0	129.4	1	28.0	67.5	1	25	66	0.574	32.4	67.0	Infin.	0.705	0.492	Non-Liq.
66.0	129.4	1	28.0	67.5	1	25	66	0.571	32.3	67.0	Infin.	0.701	0.490	Non-Liq.
67.0	129.4	1	28.0	67.5	1	25	66	0.568	32.1	67.0	Infin.	0.696	0.489	Non-Liq.
68.0	129.4	1	28.0	67.5	1	25	66	0.565	32.0	67.0	Infin.	0.691	0.487	Non-Liq.
69.0	129.4	1	28.0	67.5	1	25	66	0.563	31.8	67.0	Infin.	0.687	0.486	Non-Liq.
70.0	129.4	1	28.0	67.5	1	25	66	0.560	31.7	67.0	Infin.	0.682	0.484	Non-Liq.
71.0	141.5	1	79.0	72.5	1	25	109	0.557	80.5	79.1	Infin.	0.678	0.482	Non-Liq.
72.0	141.5	1	79.0	72.5	1	25	109	0.554	80.1	79.1	Infin.	0.673	0.480	Non-Liq.
73.0	141.5	1	79.0	72.5	1	25	109	0.550	79.7	79.1	Infin.	0.669	0.477	Non-Liq.
74.0	141.5	1	79.0	72.5	1	25	109	0.547	79.2	79.1	Infin.	0.664	0.475	Non-Liq.
75.0	141.5	1	79.0	72.5	1		109	0.544	74.1	79.1	Infin.	0.659	0.473	Non-Liq.

Figure 6

GEOCON

Client : 8920 Sunset, LLC
File No. : A9286-06-01
Boring : B3

LIQUEFACTION SETTLEMENT ANALYSIS
MAXIMUM CONSIDERED EARTHQUAKE

(SATURATED SAND AT INITIAL LIQUEFACTION CONDITION)

NCEER (1996) METHOD
EARTHQUAKE INFORMATION:

Earthquake Magnitude:	6.71
PGA$_M$ (g):	0.948
Calculated Mag.Wtg.Factor:	0.756
Historic High Groundwater:	22.0
Groundwater @ Exploration:	36.0

DEPTH TO BASE	BLOW COUNT N	WET DENSITY (PCF)	TOTAL STRESS O (TSF)	EFFECT STRESS O' (TSF)	REL. DEN. Dr (%)	ADJUST BLOWS (N1)60	Tav/σ'_o	LIQUEFACTION SAFETY FACTOR	Volumetric Strain [e$_{15}$} (%)	EQ. SETTLE. Pe (in.)
1	11	106.959	0.027	0.027		28	0.616	~	0.00	0.00
2	11	106.959	0.080	0.080		28	0.616	~	0.00	0.00
3	11	106.959	0.134	0.134		28	0.616	~	0.00	0.00
4	11	106.959	0.187	0.187		28	0.616	~	0.00	0.00
5	11	106.959	0.241	0.241		28	0.616	~	0.00	0.00
6	11	106.959	0.294	0.294		27	0.616	~	0.00	0.00
7	11	106.959	0.348	0.348		25	0.616	~	0.00	0.00
8	11	106.959	0.401	0.401		23	0.616	~	0.00	0.00
9	11	106.959	0.455	0.455		22	0.616	~	0.00	0.00
10	11	106.959	0.508	0.508		20	0.616	~	0.00	0.00
11	11	106.959	0.562	0.562		19	0.616	~	0.00	0.00
12	11	106.959	0.615	0.615		19	0.616	~	0.00	0.00
13	11	106.959	0.668	0.668		18	0.616	~	0.00	0.00
14	11	106.959	0.722	0.722		17	0.616	~	0.00	0.00
15	11	106.959	0.775	0.775		17	0.616	~	0.00	0.00
16	11	106.959	0.829	0.829		16	0.616	~	0.00	0.00
17	11	106.959	0.882	0.882		15	0.616	~	0.00	0.00
18	11	106.959	0.936	0.936		15	0.616	~	0.00	0.00
19	11	106.959	0.989	0.989		15	0.616	~	0.00	0.00
20	40	129.5676	1.049	1.049		62	0.616	~	0.00	0.00
21	40	129.5676	1.113	1.113		60	0.616	~	0.00	0.00
22	40	129.5676	1.178	1.162		58	0.624	~	0.00	0.00
23	40	129.5676	1.243	1.196		57	0.640	~	0.00	0.00
24	40	129.5676	1.308	1.230		55	0.655	~	0.00	0.00
25	40	129.5676	1.372	1.263		54	0.669	~	0.00	0.00
26	40	129.5676	1.437	1.297		53	0.683	~	0.00	0.00
27	40	129.5676	1.502	1.330		51	0.696	~	0.00	0.00
28	40	129.5676	1.567	1.364		50	0.708	~	0.00	0.00
29	40	129.5676	1.632	1.398		49	0.719	~	0.00	0.00
30	40	129.5676	1.696	1.431		48	0.730	~	0.00	0.00
31	32	132.7984	1.762	1.466	87	48	0.741	Non-Liq.	0.00	0.00
32	32	132.7984	1.828	1.501	87	48	0.751	Non-Liq.	0.00	0.00
33	32	140.7813	1.897	1.538	87	47	0.760	Non-Liq.	0.00	0.00
34	32	140.7813	1.967	1.577	87	46	0.769	Non-Liq.	0.00	0.00
35	32	140.7813	2.038	1.616	87	45	0.777	Non-Liq.	0.00	0.00
36	32	140.7813	2.108	1.655	87	45	0.785	Non-Liq.	0.00	0.00
37	24	140.7813	2.178	1.695	73	35	0.792	Non-Liq.	0.00	0.00
38	24	140.7813	2.249	1.734	73	35	0.799	Non-Liq.	0.00	0.00
39	24	140.7813	2.319	1.773	73	34	0.806	Non-Liq.	0.00	0.00
40	24	133.4	2.388	1.810	73	34	0.813	Non-Liq.	0.00	0.00
41	24	133.4	2.454	1.846	73	34	0.819	Non-Liq.	0.00	0.00
42.5	24	133.4	2.538	1.890	73	34	0.827	Non-Liq.	0.00	0.00
43	10	133.4	2.571	1.908	46	17	0.830	0.34	1.70	0.20
44	10	133.4	2.654	1.952	46	17	0.838	0.34	1.70	0.20
45	10	133.4	2.721	1.988	46	17	0.843	0.33	1.70	0.20
46	22	135.5928	2.788	2.024	66	31	0.849	Non-Liq.	0.00	0.00
47	22	135.5928	2.856	2.061	66	31	0.854	Non-Liq.	0.00	0.00
48	22	135.5928	2.924	2.097	66	31	0.859	Non-Liq.	0.00	0.00
49	22	135.5928	2.992	2.134	66	31	0.864	Non-Liq.	0.00	0.00
50	22	135.5928	3.060	2.170	66	30	0.869	Non-Liq.	0.00	0.00
51	23	140.6944	3.129	2.208	65	30	0.873	Non-Liq.	0.00	0.00
52	23	140.6944	3.199	2.247	65	30	0.877	Non-Liq.	0.00	0.00
53	23	140.6944	3.269	2.287	65	30	0.881	0.83	0.75	0.09
54	23	140.6944	3.340	2.326	65	30	0.885	0.77	0.75	0.09
55	23	140.6944	3.410	2.365	65	30	0.889	0.74	0.75	0.09
56	31	122.9256	3.476	2.400	75	39	0.893	Non-Liq.	0.00	0.00
57	31	122.9256	3.537	2.430	75	39	0.897	Non-Liq.	0.00	0.00
58	31	138.334	3.603	2.464	75	39	0.901	Non-Liq.	0.00	0.00
59	31	138.334	3.672	2.502	75	39	0.904	Non-Liq.	0.00	0.00
60	31	138.334	3.741	2.540	75	39	0.908	Non-Liq.	0.00	0.00
61	45	138.334	3.810	2.578	86	46	0.911	Non-Liq.	0.00	0.00
62	45	138.334	3.879	2.616	86	45	0.914	Non-Liq.	0.00	0.00
63	45	138.334	3.949	2.654	86	45	0.917	Non-Liq.	0.00	0.00
64	45	138.334	4.018	2.692	86	45	0.920	Non-Liq.	0.00	0.00
65	28	129.36	4.085	2.727	66	32	0.923	Non-Liq.	0.00	0.00
66	28	129.36	4.149	2.761	66	32	0.926	Non-Liq.	0.00	0.00
67	28	129.36	4.214	2.794	66	32	0.929	Non-Liq.	0.00	0.00
68	28	129.36	4.279	2.828	66	32	0.932	Non-Liq.	0.00	0.00
69	28	129.36	4.343	2.861	66	32	0.935	Non-Liq.	0.00	0.00
70	28	129.36	4.408	2.895	66	32	0.938	Non-Liq.	0.00	0.00
71	79	141.4875	4.476	2.931	109	81	0.941	Non-Liq.	0.00	0.00
72	79	141.4875	4.546	2.971	109	80	0.943	Non-Liq.	0.00	0.00
73	79	141.4875	4.617	3.010	109	80	0.945	Non-Liq.	0.00	0.00
74	79	141.4875	4.688	3.050	109	79	0.947	Non-Liq.	0.00	0.00
75	79	141.4875	4.759	3.090	109	74	0.949	Non-Liq.	0.00	0.00

TOTAL SETTLEMENT = 0.9 INCHES

GEOCON
WEST, INC.
GEOTECHNICAL ■ ENVIRONMENTAL ■ MATERIALS

Project No. A9286-06-01
July 11, 2016

VIA OVERNIGHT & E-MAIL

8920 Sunset Boulevard LLC
c/o VE Equities
250 Bowery, 2nd Floor
New York, New York 1002

Attention: Mr. Zach Vella

Subject: CLARIFICATION LETTER – EXPANSIVE SOILS
PROPOSED MIXED-USE DEVELOPMENT
8920 SUNSET BOULEVARD,
WEST HOLLYWOOD, CALIFORNIA

References: *Geotechnical Investigation,* prepared by Geocon West, Inc., Project No. A9286-06-01, dated July 2, 2015;

City of West Hollywood Geotechnical, Geology, and Seismic Review Sheet, KFMg Project No. cWH 16-43E, dated January 28, 2016;

Response to City of West Hollywood Review Comments, prepared by Geocon West, Inc., Project No. A9286-06-01, dated May 24, 2016.

Dear Mr. Vella:

At your request, this letter has been prepared to clarify the effect, if any, the presence of expansive soils would have on the proposed structure. Based on depth of the proposed subterranean levels, the proposed structure would not be prone to the effects of expansive soils.

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

Sincerely,

GEOCON WEST, INC.

JELISA M. THOMAS
74946
CIVIL

NEAL D. BERLINER
2576
GEOTECHNICAL

Jelisa M. Thomas
PE 74946

Neal D. Berliner
GE 2576

(email) Addressee

City of West Hollywood
California 1984

CITY OF
WEST HOLLYWOOD

COMMUNITY DEVELOPMENT DEPARTMENT
PLANNING DIVISION

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

GEOTECHNICAL, GEOLOGY, AND SEISMIC REVIEW SHEET

Site Address: 8920 Sunset Blvd **KFMg Project No.:** cWH 16-43E
Lot/Block/Tract: NA
Owner: 8920 Sunset Boulevard, LLLC

Project Type: Geotechnical engineering evaluation for a new 8-story mixed-use structure with 4-stories of subterranean parking extending up to 55 feet to bottom of foundations.

Geotechnical Engineer: Geocon West, Inc. (Berliner, GE 2576)
Engineering Geologist: Geocon West, Inc. (Kirkgard, CEG 1754)
Report Dated: July 2, 2015 (Project No. A9286-06-01)
Response Letter Dated: May 24, 2016 (Project No. A9286-06-01)

ACTION:

__X__ Recommend **APPROVAL**

____ Request **ADDITIONAL DATA** for review to address the comments listed below prior to approval

CLOSURE:

This document and all associated written comments and responses are a part of the geotechnical design documentation for this project. It is the responsibility of the Geotechnical Consultant to distribute these and associated written comments and responses to the appropriate parties.

Edward H Sabins

Reviewed by: ______________________ Date: June 13, 2016
Edward H. Sabins, CEG 1571
Engineering Geology Reviewer

Reviewed by: ______________________ Date: June 13, 2016
Douglas R. Bell, GE 2140
Geotechnical Engineering Reviewer

8920 Sunset Blvd (Geocon) APPROVAL 2016-06-13 ES - DRB