UPDATED GEOTECHNICAL INVESTIGATION

PROPOSED CITY HALL PARKING STRUCTURE PROJECT
8300 SANTA MONICA BOULEVARD
WEST HOLLYWOOD, CALIFORNIA

PREPARED FOR
CITY OF WEST HOLLYWOOD
WEST HOLLYWOOD, CALIFORNIA

PROJECT NO. A8635-06-02A
NOVEMBER 27, 2012
Project No. A8635-06-02A
November 27, 2012

VIA OVERNIGHT COURIER

City of West Hollywood
City Hall - City Clerk
8300 Santa Monica Boulevard
West Hollywood, CA 90069

Subject: UPDATED GEOTECHNICAL INVESTIGATION
PROPOSED CITY HALL PARKING STRUCTURE PROJECT
8300 SANTA MONICA BOULEVARD
WEST HOLLYWOOD, CALIFORNIA

Ladies and Gentlemen:

In accordance with your authorization of our proposal dated October 11, 2012, we have performed an updated geotechnical investigation for the proposed City Hall Parking Structure located at 8300 Santa Monica Boulevard in the City of West Hollywood, California. The accompanying report presents the findings of our study, and our conclusions and recommendations pertaining to the geotechnical aspects of 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 in this report are followed and implemented during construction.

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

Very truly yours,

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1. PURPOSE

This report presents the results of an updated geotechnical investigation for the proposed City Hall Parking Structure located at 8300 Santa Monica Boulevard in the City of West Hollywood, California (see Figure 1, Vicinity Map). The purpose of the investigation was to evaluate subsurface soil and geologic conditions throughout the area of proposed construction and based on conditions encountered provide conclusions and recommendations pertaining to the geotechnical aspects of design and construction.

The scope of our investigation included a site reconnaissance, field exploration, laboratory testing, engineering analysis, and the preparation of this report. The site was initially explored on August 15, 2008 by drilling four 8-inch diameter borings utilizing a truck mounted hollow stem-auger drilling machine. The borings were conducted to depths between 25½ and 35½ feet below the ground surface. A supplemental site investigation was conducted on August 12, 2011 by excavating two 8-inch diameter borings utilizing a truck-mounted hollow stem-auger drilling machine. The borings were advanced to depths of 25½ feet below the existing ground surface. The approximate locations of the exploratory borings are depicted on the Site Plan, Figure 2. A detailed discussion of the field exploration, 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.

2. SITE DESCRIPTION

The area of proposed development is located at 8300 Santa Monica Boulevard in the City of West Hollywood, California. The area is bounded by the existing 3-story City Hall building to the north, by a two-story multi-family residential building to the south, by a two-story commercial building (Gelson’s Market) to the west, and by Sweetzer Avenue to the east. The site gently slopes to the south and southeast with approximately four feet of total relief across the site. Surface water drainage at the site appears to be by sheet flow along the existing ground contours to city streets and storm drains. Vegetation of the site consists of trees and bushes located in isolated planter areas.

It is our understanding that the proposed construction will consist of a new five-story Automated Vehicle Storage and Retrieval System (AVSRS) parking structure to be constructed at or near present grade. Based on information provided by the project structural engineer, column loads are anticipated to be up to 650 kips (dead + live load), and wall loads are anticipated to be up to 18.5 kips per linear foot (dead + live loads). It is our
further understanding that the proposed structure will be supported on a deepened foundation system consisting of drilled, cast-in-place piles connected to a pile cap and grade beam system.

In addition to the proposed parking structure, it is our further understanding that improvement to the intersection of Santa Monica Boulevard and Sweetzer Avenue will be performed concurrently with the parking structure construction. The improvements may include street widening, realignment of sidewalks and existing underground utilities, and the construction of underground electrical utilities.

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 in the northwestern edge of the Los Angeles Basin just south of the southern edge of the eastern 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 the Santa Ana Mountains and San Joaquin Hills on the south. The Los Angeles Basin is located in the northern portion of the Peninsular Ranges geomorphic province and is a northwest-trending alluviated lowland plain, sometimes called the Coastal Plain of Los Angeles. The basin is underlain by a deep structural depression which has been filled by both marine and continental sedimentary deposits, which rest on a basement complex of presumably igneous and metamorphic composition (Yerkes, et al., 1965). The basement surface within the central portion of the basin extends to a maximum depth of 32,000 feet below sea level. The prominent structural features within the Los Angeles Basin include the central lowland plain, the uplifted Palos Verdes Hills, and the northwest trending line of low hills and mesas (underlain by the Newport-Inglewood Fault Zone).

4. SOIL AND GEOLOGIC CONDITIONS

Based on our field investigation, the area of proposed construction is mantled by artificial fill soils underlain by Holocene age Alluvial Fan deposits derived from the southern flank of the nearby Santa Monica Mountains (Dibblee, 1991). The soil and geologic units encountered at the site are discussed below. Detailed stratigraphic profiles are provided in the Boring Logs in Figures A1 and A6 in Appendix A.

4.1 Artificial Fill

Various amounts of artificial fill were encountered throughout the area of proposed development. Artificial fill was observed in our field explorations to a maximum depth of 1½ feet below existing ground surface. The fill generally consists of soft to medium dense, dry to slightly moist, brown, clay, silty sand, and sandy silt. The fill
is believed to be the result of past grading and construction activities on the site. Deeper fill may exist between borings and on other parts of the site that were not directly explored.

4.2 Alluvial Fan Deposits

The artificial fill is underlain by Holocene Age alluvial fan deposits. The alluvial fan deposits consist primarily of light brown to dark brown, sandy clay and clayey sand. The Holocene Age alluvial fan soils are generally slightly moist to moist, soft to stiff and medium dense.

5. GROUNDWATER

Based on a review of the Seismic Hazard Zone Report for the Hollywood 7.5 Minute Quadrangle, Los Angeles County, California (California Geological Survey, formerly California Division of Mines and Geology, 1998), the historically highest groundwater in the area is greater than 70 feet beneath the ground surface. Groundwater information presented in this document is generated from data collected in the early 1900’s to 1998. Based on current groundwater basin management practices, it is unlikely that groundwater levels will ever reach the historic high levels.

Groundwater was not encountered in our field explorations, drilled to a maximum of 35½ feet below the ground surface. Based on these considerations, groundwater is neither expected to be encountered during construction, nor have a detrimental effect on the project. However, it is not uncommon for groundwater levels to vary seasonally or for groundwater 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 region. Proper surface drainage of irrigation and precipitation will be critical to future performance of the project. Recommendations for drainage are provided in the Surface Drainage section of this report (see Section 7.14).

6. GEOLOGIC HAZARDS

6.1 Surface Fault Rupture

The numerous faults in Southern California include active, potentially active, and inactive faults. The criteria for these major groups are based on criteria developed by the California Geological Survey (CGS, formerly known as the California Division of Mines and Geology) for the Alquist-Priolo Earthquake Fault Zone Program (Bryant and Hart, 2007). By 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 location of the site with respect to local active and potentially active faults is shown on Figure 3, Regional Fault Map.
The site is not within a currently established Alquist-Priolo Earthquake Fault Zone for surface fault rupture hazards. 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. The site, however, 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 closest surface trace of an active fault to the site is the Hollywood Fault located approximately 1,240 feet north-northwest of the site (KFM Geoscience, 2010; Bing Yen & Associates, 2001). Other nearby active faults are the Santa Monica Fault, the Newport-Inglewood Fault Zone, the Raymond Fault and the Verdugo Fault located 3.6 miles west-southwest, 4.8 miles south, 7.5 miles east-northeast and 8.3 miles northeast of the site, respectively (Ziony and Jones, 1989). The active San Andreas Fault zone is located approximately 34 miles northeast of the site.

The closest potentially active faults to the site are the MacArthur Park Fault, the Overland Fault, the Charnock Fault and the Coyote Pass Fault located approximately 3.4 miles east, 6.0 miles southwest, 7.3 miles southwest and 10 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 buried thrust faults. These thrust faults are not exposed at the surface and do not present a potential surface fault rupture hazard; however, these active features are capable of generating future earthquakes.

6.2 Seismicity

As with all of Southern California, the site has experienced historic earthquakes from various regional faults. The epicenters of recorded earthquakes with magnitudes equal to or greater than 4.0 within a radius of 60 miles of the site are depicted on Figure 4, Regional Seismicity Map.

6.3 Estimation of Peak Ground Accelerations

The seismic exposure of the site may be investigated in two ways. The deterministic approach recognizes the Maximum Earthquake, which is the theoretical maximum event that could occur along a fault. The deterministic method assigns a maximum earthquake to a fault derived from formulas that correlate the length and other characteristics of the fault trace to the theoretical maximum magnitude earthquake. The probabilistic method considers the probability of exceedance of various levels of ground motion and is calculated by consideration of risk contributions from regional faults.
6.3.1 Deterministic Analysis

Table 1 shows known faults within a 60 mile radius of the site. The maximum earthquake magnitude is indicated for each fault. In order to measure the distance of known faults to the site, the computer program \textit{EQFAULT}, (Blake, 2000), was utilized. Principal references used within \textit{EQFAULT} in selecting faults to be included are Jennings (1994), Anderson (1984) and Wesnousky (1986). For this investigation, the ground motion generated by maximum earthquakes on each of the faults is assumed to attenuate to the site per the attenuation relation by Sadigh et al. (1997), modeling the soil underlying the site as a Site Class “D”, as defined by CBC Table 1613.5.2. The resulting calculated peak horizontal accelerations at the site are shown on Table 1. These values are one standard deviation above the mean.

Using this methodology, the maximum earthquake resulting in the highest peak horizontal accelerations at the site would be a magnitude 6.4 event on the Hollywood Fault. Such an event would be expected to generate peak horizontal accelerations at the site of 1.08g.

While listing of peak accelerations is useful for comparison of potential effects of fault activity in a region, other considerations are important in seismic design, including the frequency and duration of motion and the soil conditions underlying the site.

The site could be subjected to moderate to severe ground shaking in the event of a major earthquake on any of the faults referenced above or other faults in Southern California. With respect to seismic shaking, the site is considered comparable to the surrounding developed area.

6.3.2 Probabilistic Analysis

The computer program \textit{FRISKSP} (Blake, 2000) was used to perform a site-specific probabilistic seismic hazard analysis. The program is a modified version of \textit{FRISK} (McGuire, 1978) that models faults as lines to evaluate site-specific probabilities of exceedance of given horizontal accelerations for each line source. Geologic parameters not included in the deterministic analysis are included in this analysis. The program operates under the assumption that the occurrence rate of earthquakes on each mapped Quaternary Fault is proportional to the fault's slip rate. The program accounts for fault rupture length as a function of earthquake magnitude, and site acceleration estimates are made using the earthquake magnitude and closest distance from the site to the rupture zone.

Uncertainty in each of following are accounted for: (1) earthquake magnitude, (2) rupture length for a given magnitude, (3) location of the rupture zone, (4) maximum magnitude of a given earthquake, and (5) acceleration at the site from a given earthquake along each fault. After calculating the expected accelerations from all earthquake sources, the program then calculates the total average annual expected number of occurrences of the site acceleration greater than a specified value. Attenuation relationships suggested by Sadigh et al. (1997) were utilized in the analysis.
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 2,500 years. According to 2010 California Building Code and ASCE 7-05, the MCE is to be utilized for the design of critical structures such as schools and hospitals. The Design-Basis Earthquake Ground Motion (DBE) 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. The DBE is typically used for the design of non-critical structures. Based on the computer program FRISKSP (Blake, 2000), the MCE and DBE is expected to generate ground motions at the site of approximately 1.10g and 0.64g, respectively. Graphical representation of the analysis is presented on Figure 5.

6.4 Seismic Design Criteria

The following table summarizes site-specific design criteria obtained from the 2010 California Building Code (CBC; Based on the 2009 International Building Code [IBC]), Chapter 16 Structural Design, Section 1613 Earthquake Loads. The values were derived using the computer program Seismic Hazard Curves and Uniform Hazard Response Spectra, provided by the USGS. The short spectral response uses a period of 0.2 second.

CBC SEISMIC DESIGN PARAMETERS

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
<th>2010 CBC Reference</th>
</tr>
</thead>
<tbody>
<tr>
<td>Site Class</td>
<td>D</td>
<td>Table 1613.5.2</td>
</tr>
<tr>
<td>Spectral Response – Class B (short), $S_S$</td>
<td>1.760g</td>
<td>Figure 1613.5(3)</td>
</tr>
<tr>
<td>Spectral Response – Class B (1 sec), $S_1$</td>
<td>0.600g</td>
<td>Figure 1613.5(4)</td>
</tr>
<tr>
<td>Site Coefficient, $F_s$</td>
<td>1.0</td>
<td>Table 1613.5.3(1)</td>
</tr>
<tr>
<td>Site Coefficient, $F_v$</td>
<td>1.5</td>
<td>Table 1613.5.3(2)</td>
</tr>
<tr>
<td>Maximum Considered Earthquake Spectral Response Acceleration (short), $S_{MS}$</td>
<td>1.760g</td>
<td>Section 1613.5.3 (Eqn 16-36)</td>
</tr>
<tr>
<td>Maximum Considered Earthquake Spectral Response Acceleration (1 sec), $S_{M1}$</td>
<td>0.900g</td>
<td>Section 1613.5.3 (Eqn 16-37)</td>
</tr>
<tr>
<td>5% Damped Design Spectral Response Acceleration (short), $S_{DS}$</td>
<td>1.173g</td>
<td>Section 1613.5.4 (Eqn 16-38)</td>
</tr>
<tr>
<td>5% Damped Design Spectral Response Acceleration (1 sec), $S_{D1}$</td>
<td>0.600g</td>
<td>Section 1613.5.4 (Eqn 16-39)</td>
</tr>
</tbody>
</table>

Conformance to the criteria in the above table 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 intent of the code is “Life Safety,” not to completely prevent damage to the structure, since such design may be economically prohibitive.
6.5 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 117A, Guidelines for Analyzing and Mitigating Liquefaction in California” requires liquefaction analysis to a depth of fifty 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.

According to the City of West Hollywood Seismic Safety Element (Bing Yen & Associates, 2001) and General Plan Update (KFM Geoscience, 2010), the County of Los Angeles Seismic Safety Element (Leighton, 1990) and the State of California Seismic Hazard Zone, Hollywood Quadrangle Map (CDMG, 1999), the site is not located in an area designated as “liquefiable.” As previously stated, groundwater was not encountered during our site explorations, drilled to a maximum depth of 35½ feet beneath the existing ground surface and the historically highest groundwater in the area is reported to be approximately 70 feet beneath the ground surface. Based on these considerations, it is our opinion that the potential for liquefaction of the site soils is very low. Further, no surface manifestations of liquefaction are expected at the subject site.

6.6 Seismically-Induced Settlement

Dynamic compaction of dry and loose sands may occur during a major earthquake. Typically, settlements occur in thick beds of such soils. Based on the primarily fine-grained and relatively dense and consolidated nature of the site soils, it is our opinion that the potential for appreciable seismically-induced settlements is very low.

6.7 Landslides

According to the City of West Hollywood Seismic Safety Element (Bing Yen & Associates, 2001) and General Plan Update (KFM Geoscience, 2101), and the Los Angeles County Seismic Safety Element (Leighton, 1990), the site is not within an area identified as having a potential for slope instability. Additionally, according to the California Geological Survey (1999), the site is not located within an area identified as having a potential for seismic slope instability. The site and surrounding vicinity is gently sloping to the south. There are no known landslides near the site, nor is the site in the path of any known or potential landslides. We do not consider the potential for a landslide to be a hazard to this project.
6.8 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 County of Los Angeles Flood and Inundation Hazards Map (Leighton, 1990), the site is located within a potential inundation area for an earthquake-induced dam failure from the Mulholland Dam.

However, this dam, as well as others in California, are continually monitored by various governmental agencies (such as the State of California Division of Safety of Dams and the U.S. Army Corps of Engineers) to guard against the threat of dam failure.

Current design and construction practices and ongoing programs of review, modification, or total reconstruction of existing dams are intended to ensure that all dams are capable of withstanding the maximum considered earthquake (MCE) for the site. Therefore, the potential for inundation at the site as a result of an earthquake-induced dam failure is considered low.

6.9 Tsunamis and Seiches

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 in an area of minimal flooding potential (Zone C) as defined by the Federal Insurance Administration.

6.10 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 boundaries of an oil field. No oil wells are located in the immediate vicinity of the site. 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. 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.
The site is not located within the boundaries of a known oil field; therefore, the potential for the presence of methane 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.

6.11 Subsidence

The site is not within an area of known subsidence associated with fluid withdrawal (groundwater or petroleum), peat oxidation, or hydrocompaction.

7. CONCLUSIONS AND RECOMMENDATIONS

7.1 General

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

7.1.2 A maximum of 1½ feet of artificial fill was encountered during site exploration. The existing fill is believed to be the result of past grading and/or construction activities at the site. Deeper fill may exist in other areas of the site that were not directly explored. It is our opinion that the existing fill, in its present condition, is not suitable for direct support of proposed foundations or slabs. The existing fill and site soils are suitable for re-use as engineered fill provided the recommendations in the Grading section of this report are followed (see Section 7.4).

7.1.3 The results of laboratory testing indicate that the existing upper alluvial soils are moderately compressible in their current condition, and could yield excessive static and differential settlements when subject to the anticipated foundation loading.

7.1.4 In addition, based on the proximity of the proposed foundations to existing offsite foundations, as well as the anticipated wall and column loads, it is critical that the loads induced to the soil by new foundations do not cause excessive settlement and damage to existing, adjacent structures. The grading and foundation design recommendations are intended to minimize induced settlements and reduce the potential for distress to the existing offsite structures.

7.1.5 It is our understanding that the proposed structure will be supported on a deepened foundation system consisting of drilled, cast-in-place pile foundations connected with a system of pile caps and grade beams. The pile foundations may derive support in the undisturbed alluvial soils found at or below a depth of 2 feet. Recommendations for the design of pile foundations are provided in Section 7.6.
7.1.6 In order to generate resistance to lateral loads acting on the proposed caisson foundations, a pile cap or a continuous grade beam foundation may be placed across the top of the pile foundations. The appropriate span between piles should be determined by a qualified structural engineer. Based on the results of the laboratory testing discussed above, the pile cap or grade beam may generate lateral resistance in the undisturbed alluvial soils found at or below a depth of 2 feet.

7.1.7 The parking structure slab-on-grade may derive support on a blanket of newly placed engineered fill. As a minimum, all existing artificial fill should be excavated and properly recompacted for slab support. Based on observations during site exploration, the contractor should be prepared for excavations on the order of 18 inches in depth. Deeper excavation should be conducted as necessary at the direction of the Geotechnical Engineer (a representative of Geocon) to completely remove all encountered artificial fill or soft, unsuitable alluvium.

7.1.8 As an alternative to creating an engineered fill blanket for slab support, the concrete slab may be designed as a structural slab that derives all support from the deepened foundation system, eliminating permanent reliance on the soil immediately underlying the slab. However, if the subgrade soils are soft or disturbed, compaction of the soils will be required to create a stable subgrade prior to placing steel or concrete for structural slab construction. Compaction should be performed at the direction of the Geotechnical Engineer (a representative of Geocon).

7.1.9 Foundations for small outlying structures, such as block walls less than 6 feet in height, planter walls or trash enclosures, and which will not be tied-in to the proposed structures, 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, such as adjacent to property lines, foundations may bear in the competent alluvial soil found at or below a depth of 30 inches. 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.

7.1.10 Where new paving is to be placed, it is recommended that all existing fill and soft alluvial soil be excavated and properly compacted for paving support. The client should be aware that excavation and compaction of all existing fill and soft estuarine deposits in the area of new paving is not required; however, paving constructed over existing uncertified fill or unsuitable alluvial soil may experience increased settlement and/or cracking, and may therefore have a shorter design life and increased maintenance costs. As a minimum, the upper twelve inches of soil should be scarified and properly compacted for paving support. Paving recommendations are provided in Preliminary Pavement Recommendations section of this report (see Section 7.10).
7.1.11 It is anticipated that the majority of the recommended grading can be achieved with sloping measures. However, performing open excavations adjacent to or deeper than adjacent foundation systems could potentially remove lateral support and/or undermine the existing foundations. Due to the proximity of the proposed structure to the western property line and existing offsite structure, special excavation measures may be required in order to protect the offsite structure during grading operations. Excavation recommendations are provided in the Temporary Excavations section of this report (Section 7.13).

7.1.12 Based on the presence of expansive and impermeable clay at the subject site, a stormwater infiltration system is not recommended for this development. It is suggested that stormwater be retained, filtered and discharged in accordance with the requirements of the local governing agency.

7.1.13 Once the design and foundation loading configuration for the proposed addition proceeds to a more finalized plan, the recommendations within this report should be reviewed and revised, if necessary. Based on the final foundation loading configurations, the potential for settlement should be re-evaluated by this office.

7.1.14 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 Soil and Excavation Characteristics

7.2.1 The in-situ soils can be excavated with moderate effort using conventional excavation equipment. Minor caving should be anticipated in unshored vertical excavations.

7.2.2 It is the responsibility of the contractor to ensure that all excavations and trenches are properly shored and maintained in accordance with applicable OSHA rules and regulations to maintain safety and maintain the stability of adjacent existing improvements.

7.2.3 All onsite excavations must be conducted in such a manner that potential surcharges from existing structures, construction equipment, and vehicle loads are resisted. The surcharge area may be defined by a 1:1 projection down and away from the bottom of an existing foundation or vehicle load. Penetrations below this 1:1 projection will require special excavation measures such as sloping and shoring. Excavation recommendations are provided in the Temporary Excavations section of this report (see Section 7.13).

7.2.4 The upper five feet of existing site soils are considered to have a “medium” expansive potential (EI=59) and is classified as “expansive”, based on the 2010 California Building Code (CBC) Section 1803.5.3. The recommendations presented herein assume that shallow foundations and slabs will derive support in these materials.
7.3 Minimum Resistivity, pH, Chloride and Water-Soluble Sulfate

7.3.1 Potential of Hydrogen (pH) and resistivity testing as well as chloride content testing were previously 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 “extremely corrosive” with respect to corrosion of buried ferrous metals on site. The results are presented in Appendix B (Figure B12) and should be considered for design of underground structures.

7.3.2 Laboratory tests were performed on representative samples of the site materials to measure the percentage of water-soluble sulfate content. Results from the laboratory water-soluble sulfate tests are presented in Appendix B (Figure B12) and indicate that the on-site materials possess “negligible” sulfate exposure to concrete structures as defined by 2010 CBC Section 1904.3 and ACI 318-08 Sections 4.2 and 4.3.

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

7.4 Grading

7.4.1 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.

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

7.4.3 Grading should commence with the removal of all existing vegetation and existing improvements from the area to be graded. Deleterious debris such as wood and root structures should be exported from the site and should not be mixed with the fill soils. Asphalt and concrete should not be mixed with the fill soils unless approved by the Geotechnical Engineer. All existing underground improvements planned for removal should be completely excavated and the resulting depressions properly backfilled in accordance with the procedures described herein.
7.4.4 As a minimum, it is recommended that all existing artificial fill and soft, unsuitable alluvium be excavated and properly compacted for slab support. Based on observations during site exploration, the contractor should be prepared for excavations on the order of 18 inches in depth. Deeper excavation should be conducted as necessary at the direction of the Geotechnical Engineer (a representative of Geocon) to completely remove all encountered artificial fill or soft, unsuitable alluvium.

7.4.5 As an alternative to creating a fill blanket for slab support, the concrete building slab may be designed as a structural slab that derives all support from the deepened foundation system, eliminating permanent reliance on the soil immediately underlying the slab. However, if the subgrade soils are soft or disturbed, compaction of the soils will be required to create a stable subgrade prior to placing steel or concrete for structural slab construction.

7.4.6 The excavation bottom must be observed and approved in writing by the Geotechnical Engineer (a representative of Geocon) prior to placing and compacting fill. If soils exposed at the bottom of the excavation are determined to be soft or disturbed, additional removals may be required at the direction of the Geotechnical Engineer.

7.4.7 If subgrade stabilization is required at the excavation bottom, rubber tire equipment should not be allowed in the excavation bottom until it is stabilized or extensive soil disturbance could result. It is suggested that excavation and grading be performed during the summer season to promote moisture control of the soils. In addition, the use of track equipment should be considered to minimize disturbance to the soils if they become wet at the excavation bottom. Bottom stabilization, if necessary, may be achieved by introducing a thin lift of three to six-inch diameter crushed angular rock into the soft excavation bottom. The use of crushed concrete will also be acceptable. The crushed rock should be spread thinly across the excavation bottom and pressed into the soils by track rolling or wheel rolling with heavy equipment. It is very important that voids between the rock fragments are not created so the rock must be thoroughly pressed or blended into the soils.

7.4.8 All fill and backfill soils should be placed in horizontal loose layers approximately 6 to 8 inches thick, moisture conditioned to at least 2 percent above optimum moisture content, and properly compacted to a minimum of 90 percent of the laboratory maximum dry density in accordance with ASTM D 1557 (latest edition).

7.4.9 Foundations for small outlying structures, such as block walls less than 6 feet in height, planter walls or trash enclosures, which will not be tied-in to the proposed structures, may be supported on conventional foundations bearing on a minimum of 12 inches of newly placed engineered fill which extends laterally at least 12 inches beyond the foundation area. Where removal and recompaction cannot be performed, foundations may bear in the undisturbed alluvial soils at or below a depth of 30
inches, and should be deepened as necessary to maintain a minimum 12 inch embedment into undisturbed alluvium. If the alluvial soils exposed in the excavation bottom are soft or 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.

7.4.10 Although not anticipated for this project, all imported fill shall be observed, tested, and approved by Geocon West, Inc. prior to bringing soil to the site. Rocks larger than 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 50 and corrosivity properties that are equally or less detrimental to that of the existing onsite soils (see Figure B12). Import soils placed in the building area must be placed uniformly across the building footprint or in a manner that is approved by the Geotechnical Engineer (a representative of Geocon).

7.4.11 Prior to construction of concrete slabs or paving, the upper 12 inches of the subgrade should be moisture conditioned to 2 percent above the optimum moisture content and properly compacted to at least 92 percent relative compaction, as determined by ASTM Test Method D 1557 (latest edition). Paving recommendations are provided in Preliminary Pavement Recommendations section of this report (see Section 7.10).

7.4.12 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).

7.4.13 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 materials, fill, steel, gravel or concrete.

7.5 Shrinkage

7.5.1 Shrinkage results when a volume of material removed at one density is compacted to a higher density. A shrinkage factor between 5 and 10 percent should be anticipated when excavating and compacting the upper existing earth materials on the site to an average relative compaction of 92 percent.
7.6 Deepened Foundation Design - Drilled Cast-in-Place Concrete Friction Piles

7.6.1 Drilled cast-in-place, concrete friction piles should be a minimum of 24 inches in diameter and should derive support in the competent alluvial soil found at or below a depth of 2 feet. Piles should be embedded a minimum of 20 feet below the ground surface. Piles may be designed based on the Pile Capacity chart presented below. The allowable axial capacities are based on skin friction.

![Friction Pile Capacity Chart](image)

7.6.2 Single pile uplift capacity may be taken as $\frac{2}{3}$ of the allowable downward capacity.

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

7.6.4 If pile spacing is at least three times the maximum dimension of the pile, no reduction in axial capacity or lateral load capacity is considered necessary for group effects. If pile spacing is closer than three pile diameters, an evaluation for group effects including appropriate reductions should be performed by Geocon based on pile dimension and spacing.
7.6.5 Total pile settlement is expected to be less than ½ inch. The majority of settlement is anticipated to occur on initial application of loading during construction.

7.6.6 The piles do not require the complete removal of all loose earth materials from the bottom of the excavation, since end-bearing capacity is not being considered; however, a cleanout of the excavation bottom will be required. All drilled pile excavations should be continuously observed by personnel of this firm to verify adequate penetration into the recommended bearing materials.

7.6.7 For increased resistance to differential foundation movement and lateral drift, the piles should be interconnected in two horizontal directions with a pile cap and grade beams, or tied with a structural slab. The appropriate span between friction piles should be determined by a qualified structural engineer.

7.6.8 Casing may be required to prevent caving during excavation of the pile foundations. The contractor should have casing available prior to the commencement of drilling activities.

7.6.9 Closely spaced friction piles should be drilled and filled alternately, with the concrete permitted to set at least eight hours before drilling an adjacent hole. Friction pile excavations should be filled with concrete as soon after drilling and inspection as possible; the holes should not be left open overnight unless approved by the Geotechnical Engineer.

7.7 Miscellaneous Foundations

7.7.1 Small outlying miscellaneous structures, such as property line walls less than 6 feet in height, planter walls and trash enclosures, which will not be rigidly connected to a proposed structure, may be supported on a minimum of 12 inches of newly placed engineered fill. Where excavation and proper compaction cannot be performed or is undesirable, foundations may bear directly in the undisturbed alluvial soils at or below a depth of 30 inches below the ground surface, and should be deepened as necessary to maintain a minimum 12 inch embedment into undisturbed alluvium.

7.7.2 If the alluvial 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. Miscellaneous foundations may be designed for a bearing value of 1,500 pounds per square foot, and should be a minimum of 12 inches in width, 24 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. The maximum expected static settlement for a miscellaneous foundation supported in the recommended bearing materials is estimated to be less than ¾ inch and occur below the heaviest loaded structural
element. Settlement of the foundation system is expected to occur on initial application of loading. Differential settlement is not expected to exceed \(\frac{1}{2}\) inch over a distance of twenty feet.

7.7.3 Foundation excavations should be observed 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.

7.8  **Lateral Design**

7.8.1 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 0.30 may be used with the dead load forces in newly placed engineered fill, 0.18 in the alluvial soils found above a depth of 10 feet below the existing ground surface, and 0.3 in the alluvial soils found below a depth of 10 feet.

7.8.2 Passive earth pressure for the sides of foundations and slabs poured against newly placed engineered fill may be computed as an equivalent fluid having a density of 200 pcf with a maximum earth pressure of 2,000 pcf.

7.8.3 Passive earth pressure for the sides of foundations and slabs poured against the alluvial soils found above a depth of 10 feet below the existing ground surface may be computed as an equivalent fluid having a density of 150 pcf with a maximum earth pressure of 1,500 pcf; and within the competent alluvial soils found below a depth of 10 feet as an equivalent fluid having a density of 300 pcf for the competent alluvial soils found below a depth of 10 feet, with a maximum earth pressure of 3,000 pcf. When combining passive and friction for lateral resistance, the passive component should be reduced by one-third. The allowable capacity may be doubled for isolated piles spaced more than twice the diameter.

7.8.4 Maximum recommended allowable lateral capacities for \(\frac{1}{4}\) inch deflection of fixed and free-head piles are presented in the table on the following table.
### Lateral Load Capacities of Drilled Cast-In-Place Piles

#### Fixed Head (No Head Rotation)

<table>
<thead>
<tr>
<th>Pile Number</th>
<th>Pile Diameter (Inches)</th>
<th>Lateral Load Capacity &quot;P&quot; (Kips)</th>
<th>Maximum Positive Moment &quot;Mp&quot; (LAT FORCE =P)</th>
<th>Maximum Negative Moment &quot;Mp&quot; (LAT FORCE =P)</th>
<th>Depth to Max Pos Moment (Feet)</th>
<th>Depth to Zero Moment (Feet)</th>
<th>Depth to Inflection Point (Feet)</th>
<th>Minimum Pile Length for Applicability of Lateral Design Data (Feet)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>24</td>
<td>35</td>
<td>1.5 P</td>
<td>-5.5 P</td>
<td>13</td>
<td>27</td>
<td>6.8</td>
<td>27</td>
</tr>
<tr>
<td>2</td>
<td>30</td>
<td>51</td>
<td>1.8 P</td>
<td>-6.5 P</td>
<td>16</td>
<td>32</td>
<td>8.1</td>
<td>32</td>
</tr>
<tr>
<td>3</td>
<td>36</td>
<td>68</td>
<td>2.1 P</td>
<td>-7.5 P</td>
<td>18</td>
<td>37</td>
<td>9.4</td>
<td>37</td>
</tr>
</tbody>
</table>

#### Free Head (Hinged)

<table>
<thead>
<tr>
<th>Pile Number</th>
<th>Pile Diameter (Inches)</th>
<th>Lateral Load Capacity &quot;P&quot; (Kips)</th>
<th>Maximum Moment &quot;Mp&quot; (LAT FORCE =P)</th>
<th>Depth to Zero Moment (Feet)</th>
<th>Depth to Maximum Moment (Feet)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>24</td>
<td>14</td>
<td>4.6 P</td>
<td>24</td>
<td>8</td>
</tr>
<tr>
<td>2</td>
<td>30</td>
<td>21</td>
<td>5.5 P</td>
<td>29</td>
<td>9</td>
</tr>
<tr>
<td>3</td>
<td>36</td>
<td>28</td>
<td>6.4 P</td>
<td>33</td>
<td>11</td>
</tr>
</tbody>
</table>

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

7.8.5 No factors of safety have been applied to the lateral load values calculated to induce 1/4-inch lateral deflection. Lateral capacities provided are for 24, 30 and 36-inch diameter drilled cast-in-place piles, penetrating the earth materials encountered during the course of this investigation. Assumed as part of these lateral capacity calculations are a concrete modulus of elasticity of at least 3,000,000 pounds per square inch (psi).

7.9 Concrete Slabs-on-Grade

7.9.1 Unless specifically evaluated and designed by a qualified structural engineer, the slab-on-grade for the parking garage, subject to vehicle loading, 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 derive support on at least 12 inches of newly placed engineered fill. Any disturbed soils should be properly compacted to at least 92 percent relative compaction, as determined by ASTM Test Method D 1557 (latest edition) for slab support.
7.9.2 If the concrete building slab is designed as a structural slab that derives all support from the deepened foundation system, the thickness and reinforcing of the slab should be designed by the project structural engineer.

7.9.3 Exterior concrete slabs-on-grade subject to vehicle loading should be designed in accordance with the recommendations in the Preliminary Pavement Recommendations section of this report (Section 7.10).

7.9.4 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 1643-09 and the manufacturer’s recommendations. If California Green Code requirements apply to this project, the vapor retarder should be underlain by 4 inches of ½-inch clean angular aggregate and the vapor retarder should be in direct contact with the concrete slab. It is important that the vapor retarder be puncture resistant since it will be in direct contact with angular gravel.

7.9.5 For seismic design purposes, a coefficient of friction of 0.3 may be utilized between conventional concrete slabs and engineered subgrade soils without a moisture barrier, and 0.15 for slabs underlain by a moisture barrier.

7.9.6 Exterior slabs, not subject to traffic loads, should be at least 4 inches thick and reinforced with No. 3 steel reinforcing bars placed 18 inches on center in both horizontal directions, positioned near the slab midpoint. Prior to construction of slabs, the upper 12 inches of subgrade should be moisture conditioned to 2 percent above optimum moisture content and properly compacted to at least 92 percent relative compaction, as determined by ASTM Test Method D 1557 (latest edition). Crack control joints should be spaced at intervals not greater than 8 feet and should be constructed using saw-cuts or other methods as soon as practical following concrete placement. Crack control joints should extend a minimum depth of one-fourth the slab thickness. The project structural engineer should design construction joints as necessary.

7.9.7 The recommendations of this report are intended to reduce the potential for cracking of slabs due to settlement. However, even with the incorporation of the recommendations presented herein, foundations, stucco walls, and slabs-on-grade may exhibit some cracking due to minor soil movement 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.

7.9.8 Slabs 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 1643-98 and the manufacturer’s recommendations.

7.10 Preliminary Pavement Recommendations

7.10.1 Where new paving is to be placed, it is recommended that all existing fill and soft alluvial soils be excavated and properly recompacted for paving support. The client should be aware that excavation and compaction of all existing artificial fill and soft alluvial soils in the area of new paving is not required; however, paving constructed over existing uncertified fill or unsuitable alluvial soils may experience increased settlement and/or cracking, and may therefore have a shorter design life and increased maintenance costs. As a minimum, the upper twelve inches of paving subgrade should be scarified, moisture conditioned to 2 percent above optimum moisture content, and properly compacted to at least 92 percent relative compaction, as determined by ASTM Test Method D 1557 (latest edition).

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

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

<table>
<thead>
<tr>
<th>Location</th>
<th>Estimated Traffic Index (TI)</th>
<th>Asphalt Concrete (inches)</th>
<th>Class 2 Aggregate Base (inches)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Automobile Parking &amp; Driveways</td>
<td>5</td>
<td>3</td>
<td>8</td>
</tr>
<tr>
<td>Trash Truck &amp; Fire Lanes</td>
<td>7</td>
<td>4</td>
<td>12</td>
</tr>
</tbody>
</table>
7.10.4 Asphalt concrete should conform to Section 203-6 of the “Standard Specifications for Public Works Construction” (Green Book). Class 2 aggregate base materials should conform to Section 26-1.02A of the “Standard Specifications of the State of California, Department of Transportation” (Caltrans). Crushed Miscellaneous Base should conform to Section 200-2.4 of the “Standard Specifications for Public Works Construction” (Green Book).

7.10.5 Unless specifically designed and evaluated by the project structural engineer, where exterior concrete paving will be utilized for support of vehicles, it is recommended that the concrete be a minimum of 6 inches of concrete reinforced with No. 3 steel reinforcing bars placed 18 inches on center in both horizontal directions. Concrete paving supporting vehicular traffic should be underlain by a minimum of 4 inches of aggregate base and a properly compacted subgrade. The subgrade and base material should be compacted to 92 and 95 percent relative compaction, respectively, as determined by ASTM Test Method D 1557 (latest edition).

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

7.11 Elevator Pit Design

7.11.1 The elevator pit slab and retaining wall should be designed by the project structural engineer. As a minimum the slab-on-grade 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. The elevator slab and retaining wall footings may derive support in the undisturbed alluvial soils anticipated to be exposed at the excavation bottom, or may be structurally supported either indirectly or directly by the mat and pile foundation system.

7.11.2 Provided the elevator pit retaining walls are properly drained, walls not restrained at the top and having a level backfill surface should be designed utilizing an equivalent fluid pressure of 30 pounds per cubic foot. The equivalent fluid pressure to be used in design of the non-drained elevator pit retaining walls would be 90 pounds per cubic foot. The value includes hydrostatic pressures plus buoyant lateral earth pressures. Additional active pressure should be added for a surcharge condition due to vehicular traffic or adjacent foundations and should be designed for each condition as the project progresses. Once the design becomes more finalized, an addendum letter can be prepared addressing specific surcharge conditions throughout the project, if necessary.
7.11.3 If retaining wall drainage is to be provided, the drainage system should extend 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.

7.11.4 Subdrainage pipes at the base of the retaining wall drainage system should outlet to a location acceptable to the building official.

7.11.5 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.

7.12 **Elevator Piston**

7.12.1 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 the drilled excavation could compromise the existing foundation support.

7.12.2 Casing may be required if caving is experienced in the drilled excavation. The contractor should be prepared to use casing and should have it readily available at the commencement of drilling activities. Continuous observation of the drilling and installation of the elevator piston by the Geotechnical Engineer (a representative of Geocon West, Inc.) is required.

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

7.13 **Temporary Excavations**

7.13.1 Excavations on the order of 5 feet in vertical height may be required for the recommended grading and foundation construction. The excavations are expected to expose fill and alluvial soils which are suitable for vertical excavations up to five feet where loose soils or caving sands are not present or where not surcharged by adjacent traffic or structures.

7.13.2 Vertical excavations greater than 5 feet will require sloping measures in order to provide a stable excavation. Where sufficient space is available, temporary unsurcharged embankments may be sloped back at a uniform 1:1 slope gradient or flatter. A uniform slope does not have a vertical portion. It is anticipated that sufficient space is available to complete the required earthwork and foundation construction for this project using sloping measures. If necessary, shoring and/or alternative temporary excavation recommendations will be provided under separate cover in an addendum.
7.13.3 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. Our personnel should inspect the soils exposed in the cut slopes during excavation so that modifications of the slopes can be made if variations in the soil conditions occur. All excavations should be stabilized within 30 days of initial excavation.

7.14 Surface Drainage

7.14.1 Proper surface drainage is critical to the future performance of the project. Uncontrolled infiltration of irrigation excess and storm runoff into the supporting soils can adversely affect the performance of the planned improvements. Saturation of supporting soils 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.

7.14.2 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 2010 CBC 1804.3 or other applicable standards. In addition, drainage should not be allowed to flow uncontrolled over any descending slope. The proposed structure should be provided with roof gutters. Discharge from downspouts, roof drains and scuppers 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 engineered fill providing foundation support. Landscape irrigation is not recommended within five feet of the building perimeter footings except when enclosed in protected planters.

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

7.14.4 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 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.
7.15 Plan Review

7.15.1 Grading, foundation and, if applicable, shoring plans should be reviewed by the Geotechnical Engineer 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, if necessary.
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 present date. 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.
LIST OF REFERENCES


California Division of Oil, Gas and Geothermal Resources, 2001; *Oil and Gas Well Location Map, Map Number W1-5*.


LIST OF REFERENCES (Continued)


Jennings, C. W., 1994, Fault Activity Map of California and Adjacent Areas with Locations and Ages of Recent Volcanic Eruptions, California Division of Mines and Geology Map No. 6.


PROBABILITY OF EXCEEDANCE
SADIGH ET AL. (1997) DEEP SOIL 1

![Graph showing probability of exceedance against acceleration (g) for different time periods: 25 yrs, 50 yrs, 75 yrs, 100 yrs. The graph plots acceleration (g) on the x-axis and exceedance probability (%) on the y-axis.](image_url)
RETAINING WALL DRAIN DETAIL

18” PROPERLY COMPACTED BACKFILL

GROUND SURFACE

RETAINING WALL

DRAINAGE PANEL (J-DRAIN 1000 OR EQUIVALENT)

WATER PROOFING BY ARCHITECT

3/4” CRUSHED ROCK (1 CU. FT./FT.)

FILTER FABRIC ENVELOPE OR BURLAP ROCK-POCKET

APPROVED PIPE EXTENDED TO SUBDRAIN

ELIMINATION OF THE ROCK POCKET REQUIRES A MODIFICATION BE FILED WITH THE CITY OF LOS ANGELES AND UTILIZATION OF A CITY APPROVED DRAINAGE PANEL

NO SCALE
## TABLE 1
### FAULTS WITHIN 60 MILES OF THE SITE
#### DETERMINISTIC SITE PARAMETERS

<table>
<thead>
<tr>
<th>ABBREVIATED FAULT NAME</th>
<th>APPROXIMATE DISTANCE</th>
<th>ESTIMATED MAX. EARTHQUAKE EVENT</th>
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</thead>
<tbody>
<tr>
<td></td>
<td>mi (km)</td>
<td>MAG. (Mw)</td>
</tr>
<tr>
<td>------------------------</td>
<td>----------------------</td>
<td>-----------</td>
</tr>
<tr>
<td>HOLLYWOOD</td>
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<tr>
<td>SANTA MONICA</td>
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<tr>
<td>NEWPORT-INGLEWOOD (L.A. Basin)</td>
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<tr>
<td>UPPER ELYSIAN PARK BLIND THRUST</td>
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<td>VERDUGO</td>
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<td>RAYMOND</td>
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<td>MALIBU COAST</td>
<td>10.1 (16.2)</td>
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<td>NORTHRIIDGE (E. Oak Ridge)</td>
<td>12.2 (19.7)</td>
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<tr>
<td>SIERRA MADRE</td>
<td>13.0 (20.9)</td>
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<tr>
<td>SIERRA MADRE (San Fernando)</td>
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<td>PALOS VERDES</td>
<td>15.0 (24.2)</td>
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<td>SAN GABRIEL</td>
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<td>SANTA SUSANA</td>
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<tr>
<td>SAN JOSE</td>
<td>28.0 (45.1)</td>
<td>6.4</td>
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<tr>
<td>OAK RIDGE (Onshore)</td>
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<tr>
<td>SAN CAYETANO</td>
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<td>CHINO-CENTRAL AVE. (Elsinore)</td>
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<tr>
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<td>SAN ANDREAS - 1857 Rupture M-2a</td>
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<tr>
<td>SAN ANDREAS - Cho-Moj M-1b-1</td>
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<td>SAN JOAQUIN HILLS</td>
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<td>CHANNEL IS. THRUST (Eastern)</td>
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<tr>
<td>SAN ANDREAS - San Bernardino M-1</td>
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<td>SAN ANDREAS - SB-Coach. M-2b</td>
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<td>M.RIDGE-ARROYO PARIDA-SANTA ANA</td>
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<td>CLEGHORN</td>
<td>54.1 (87.0)</td>
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<tr>
<td>RED MOUNTAIN</td>
<td>56.3 (90.6)</td>
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<td>GARLOCK (West)</td>
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<tr>
<td>FLEITIO THRUST</td>
<td>59.3 (95.5)</td>
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</table>

46 FAULTS FOUND WITHIN THE SPECIFIED SEARCH RADIUS.
The HOLLYWOOD FAULT IS CLOSEST TO THE SITE.
IT IS ABOUT 0.6 MILES (0.9 km) AWAY.
LARGEST MAXIMUM-EARTHQUAKE SITE ACCELERATION: 1.0797 g
APPENDIX A

FIELD INVESTIGATION

The site was initially explored on August 15, 2008 by drilling four 8-inch diameter borings utilizing a truck mounted hollow stem-auger drilling machine. The borings were conducted to depths between 25½ and 35½ feet below the ground surface. A supplemental site investigation was conducted on August 12, 2011 by excavating two 8-inch diameter borings utilizing a truck-mounted hollow stem-auger drilling machine. The borings were advanced to depths of 25½ 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 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. 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 through A6. The logs depict the soil and geologic conditions encountered and the depth at which samples were obtained. The approximate locations of the borings are shown on the Site Plan, Figure 2.
### BORING 1

**ELEV. (MSL.) 260.5**  || **DATE COMPLETED 8/15/008**

**EQUIPMENT** HOLLOW STEM AUGER  || **BY: JMT**

<table>
<thead>
<tr>
<th>DEPTH IN FEET</th>
<th>SAMPLE NO.</th>
<th>LITHOLOGY</th>
<th>SOIL CLASS (USCS)</th>
<th>GROUNDWATER</th>
<th>PENETRATION RESISTANCE (BLOWS/FT)*</th>
<th>DRY DENSITY (P.C.F.)</th>
<th>MOISTURE CONTENT (%)</th>
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<tbody>
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<td>0</td>
<td></td>
<td>ASPHALT: 7&quot;</td>
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<tr>
<td>2</td>
<td></td>
<td>FILL</td>
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<tr>
<td>4</td>
<td></td>
<td>Clay, soft, dry, dark brown</td>
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<tr>
<td>6</td>
<td></td>
<td>ALLUVIAL FAN DEPOSITS</td>
<td>Sandy Clay, soft, moist, brown, fine- to coarse-grained</td>
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<td>8</td>
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</tr>
<tr>
<td>10</td>
<td>B1@10'</td>
<td>CL</td>
<td></td>
<td></td>
<td>Firm, decreased moisture content, increased medium- to coarse-grained sand</td>
<td>14</td>
<td>112.1</td>
</tr>
<tr>
<td>12</td>
<td>B1@12.5'</td>
<td>CL</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>14</td>
<td>B1@15'</td>
<td>CL</td>
<td></td>
<td></td>
<td>- Decreased moisture content, increased sand content</td>
<td>19</td>
<td>120.9</td>
</tr>
<tr>
<td>16</td>
<td></td>
<td>SC</td>
<td></td>
<td></td>
<td>Clayey Sand, medium dense, slightly moist, light brown, fine- to coarse-grained</td>
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</tr>
</tbody>
</table>

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

**SAMPLE SYMBOLS**
- .. sampling unsuccessful
- .. standard penetration test
- .. drive sample (undisturbed)
- .. disturbed or bag sample
- .. chunk sample
- .. water table or seepage

**NOTE:** The log of subsurface conditions shown hereon applies only at the specific boring or trench location and at the date indicated. It is not warranted to be representative of subsurface conditions at other locations and times.

GEOCON
**BORING 1**

<table>
<thead>
<tr>
<th>DEPTH IN FEET</th>
<th>SAMPLE NO.</th>
<th>LITHOLOGY</th>
<th>SOIL CLASS (USCS)</th>
<th>GROUNDWATER</th>
<th>ELEV. (MSL.)</th>
<th>DATE COMPLETED</th>
<th>EQUIPMENT</th>
<th>PENETRATION RESISTANCE (BLOWS/FT)</th>
<th>DRY DENSITY (P.C.F.)</th>
<th>MOISTURE CONTENT (%)</th>
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<td>30</td>
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<td>Cl</td>
<td></td>
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<td>260.5</td>
<td>8/15/008</td>
<td>HOLLOW STEM AUGER</td>
<td></td>
<td>33</td>
<td>112.7</td>
</tr>
</tbody>
</table>

**MATERIAL DESCRIPTION**

Clay with Sand, firm, moist, brown, fine- to coarse-grained, trace coarse-gravel

End at 35.5 feet
Fill to 1.5 feet
No groundwater encountered
Backfilled and tamped with soil cuttings

*Penetration resistance for 140 pound hammer falling 30 inches

---

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

**SAMPLE SYMBOLS**

- box: SAMPLING UNSUCCESSFUL
- line: STANDARD PENETRATION TEST
- square: DRIVE SAMPLE (UNDISTURBED)
- star: DISTURBED OR BAG SAMPLE
- triangle: CHUNK SAMPLE
- inverted triangle: 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.
<table>
<thead>
<tr>
<th>DEPTH IN FEET</th>
<th>SAMPLE NO.</th>
<th>LITHOLOGY</th>
<th>SOIL CLASS (USCS)</th>
<th>GROUNDWATER</th>
<th>WATER TABLE OR SEEPAGE</th>
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</tr>
<tr>
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<td>B2@5'</td>
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</tr>
<tr>
<td>8</td>
<td>B2@10'</td>
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<tr>
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<td>B2@12.5'</td>
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<tr>
<td>16</td>
<td>B2@25'</td>
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</tbody>
</table>

### MATERIAL DESCRIPTION

- **ASPHALT:** 7"
- **FILL**
  - Clay, soft, dry, dark brown

**ALLUVIUM FAN DEPOSITS**

- Sandy Clay, soft, brown, slightly moist, fine- to coarse-grained
  - Increased sand content
  - Firm, increased medium- to coarse-grained sand
  - Stiff
  - Firm, increased sand content
  - Clayey Sand, medium dense, slightly moist, brown, fine- to coarse-grained, trace fine-gravel

End at 25.5 feet
Fill to 1.5 feet
No groundwater encountered
Backfilled and tamped with soil cuttings

*Penetration resistance for 140 pound hammer falling 30 inches*

**Figure A2, Log of Boring 2, 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.
# BORING 3

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<th>DEPTH IN FEET</th>
<th>SAMPLE NO.</th>
<th>LITHOLOGY</th>
<th>SOIL CLASS (USCS)</th>
<th>GROUNDWATER</th>
<th>PENETRATION RESISTANCE (BLOWS/FT)*</th>
<th>DRY DENSITY (P.C.F.)</th>
<th>MOISTURE CONTENT (%)</th>
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<td>4</td>
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<td>-Increased sand content, firm</td>
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<td>Sandy Clay, stiff, slightly moist, brown, fine- to coarse-grained</td>
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<tr>
<td>10</td>
<td>B3@25'</td>
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<td>Clayey Sand, medium dense, slightly moist, light brown, fine- to coarse-grained</td>
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<td></td>
</tr>
<tr>
<td>22</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>-</td>
<td></td>
</tr>
<tr>
<td>24</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>-</td>
<td></td>
</tr>
<tr>
<td>26</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>-</td>
<td></td>
</tr>
<tr>
<td>28</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>-</td>
<td></td>
</tr>
</tbody>
</table>

---

**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.

---

**Figure A3, Log of Boring 3, Page 1 of 2**

**SAMPLE SYMBOLS**

- □ ... SAMPLING UNSUCCESSFUL
- □ ... STANDARD PENETRATION TEST
- □ ... DRIVE SAMPLE (UNDISTURBED)
- □ ... DISTURBED OR BAG SAMPLE
- □ ... CHUNK SAMPLE
- □ ... WATER TABLE OR SEEPAGE
**BORING 3**

<table>
<thead>
<tr>
<th>DEPTH IN FEET</th>
<th>SAMPLE NO.</th>
<th>LITHOLOGY</th>
<th>SOIL CLASS (USCS)</th>
<th>GROUNDWATER</th>
<th>ELEV. (MSL.)</th>
<th>DATE COMPLETED</th>
<th>EQUIPMENT</th>
<th>PENETRATION RESISTANCE (BLOWS/FT)</th>
<th>DRY DENSITY (P.C.F.)</th>
<th>MOISTURE CONTENT (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>30</td>
<td>B3@30’</td>
<td></td>
<td></td>
<td></td>
<td>258.5</td>
<td>8/15/08</td>
<td>HOLLOW STEM AUGER</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>32</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>34</td>
<td>B3@35’</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

- Increased clay content

- End at 35.5 feet
- No fill
- No groundwater encountered
- Backfilled and tamped with soil cuttings

*Penetration resistance for 140 pound hammer falling 30 inches

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

**NOTE:** THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.
### BORING 4

<table>
<thead>
<tr>
<th>DEPTH IN FEET</th>
<th>SAMPLE NO.</th>
<th>LITHOLOGY</th>
<th>SOIL CLASS (USCS)</th>
<th>GROUNDWATER</th>
<th>EQUIPMENT</th>
<th>PENETRATION RESISTANCE (BLOWS/FT)</th>
<th>DRY DENSITY (P.C.F.)</th>
<th>MOISTURE CONTENT (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>2</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>4</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td><strong>B4@5'</strong></td>
<td></td>
<td>ASPHALT: 7&quot;</td>
<td>FILL</td>
<td>CL</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>6</td>
<td></td>
<td>Clay, soft, dry, dark brown</td>
<td></td>
<td></td>
<td>12</td>
<td>112.1</td>
<td>15.0</td>
<td></td>
</tr>
<tr>
<td>8</td>
<td></td>
<td>ALLUVIAL FAN DEPOSITS</td>
<td>Clay with Sand, firm, slightly moist, dark brown, fine- to medium-grained</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td><strong>B4@10'</strong></td>
<td></td>
<td>Sandy Clay, firm, slightly moist, brown, fine- to coarse-grained</td>
<td>CL</td>
<td></td>
<td>18</td>
<td>112.4</td>
<td>14.8</td>
<td></td>
</tr>
<tr>
<td>10</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>12</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td><strong>B4@12.5'</strong></td>
<td></td>
<td>-Moist, increased medium- to coarse-grained sand</td>
<td>CL</td>
<td></td>
<td>12</td>
<td>121.9</td>
<td>13.2</td>
<td></td>
</tr>
<tr>
<td>14</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td><strong>B4@15'</strong></td>
<td></td>
<td>-Stiff, decreased moisture content</td>
<td>CL</td>
<td></td>
<td>14</td>
<td>112.4</td>
<td>10.7</td>
<td></td>
</tr>
<tr>
<td>16</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>18</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td><strong>B4@20'</strong></td>
<td></td>
<td>End at 25.5 feet</td>
<td>Fill to 1.5 feet</td>
<td>CL</td>
<td></td>
<td>13</td>
<td>116.0</td>
<td>8.1</td>
</tr>
<tr>
<td>22</td>
<td></td>
<td>No groundwater encountered</td>
<td>Backfilled and tamped with soil cuttings</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>24</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td><strong>B4@25'</strong></td>
<td></td>
<td>*Penetration resistance for 140 pound hammer falling 30 inches</td>
<td></td>
<td></td>
<td>28</td>
<td>121.9</td>
<td>8.7</td>
<td></td>
</tr>
</tbody>
</table>

**Figure A4, Log of Boring 4, Page 1 of 1**

**NOTE:** THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.
## BORING 5

**Artificial Fill**
Sandy Silt, soft, slightly moist, brown, fine- to medium-grained, moderate plasticity

**Alluvium**
Sandy Silt, soft, slightly moist, brown, fine- to medium-grained, moderate plasticity

- Firm

- Increase in sand content, fine- to coarse-grained

**Silty Sand**
Medium dense, slightly moist, brown, fine- to medium-grained

End at 25.5 feet.
Artificial fill to 1.5 feet.
No groundwater encountered.
Backfilled and tamped with soil cuttings.
Capped with asphalt patch.

---

**Figure A5, Log of Boring 5, 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.
<table>
<thead>
<tr>
<th>DEPTH IN FEET</th>
<th>SAMPLE NO</th>
<th>LITHOLOGY</th>
<th>SOIL CLASS (USCS)</th>
<th>GROUNDWATER</th>
<th>PENETRATION RESISTANCE (BLOWS/FT)*</th>
<th>DRY DENSITY (P.C.F.)</th>
<th>MOISTURE CONTENT (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>ELEV. (MSL.) 259</td>
<td>DATE COMPLETED 8/12/11</td>
<td></td>
</tr>
<tr>
<td>2</td>
<td>B2@2'</td>
<td>ARTIFICIAL FILL</td>
<td>Silty Sand, medium dense, slightly moist, brown, fine- to medium-grained</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>4</td>
<td>B2@4'</td>
<td>ALLUVIUM</td>
<td>Silty Sand to Sandy Silt, loose to soft, slightly moist, brown, fine- to medium-grained</td>
<td></td>
<td>18</td>
<td>124.4</td>
<td>9.6</td>
</tr>
<tr>
<td>6</td>
<td>B2@6'</td>
<td></td>
<td>Sandy Silt, soft, moist, brown, fine- to medium-grained, trace clay</td>
<td></td>
<td>6</td>
<td>111.4</td>
<td>13.7</td>
</tr>
<tr>
<td>8</td>
<td>B2@9'</td>
<td>Clay, firm, slightly moist, brown</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>10</td>
<td>B2@12'</td>
<td>-Stiff, trace fine- to medium-grained sand</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>12</td>
<td>B2@15'</td>
<td>CL</td>
<td>Sandy Clay, firm, slightly moist, brown, fine- to medium-grained</td>
<td></td>
<td>15</td>
<td>112.5</td>
<td>10.9</td>
</tr>
<tr>
<td>14</td>
<td>B2@20'</td>
<td></td>
<td>Silty Sand, loose, slightly moist, brown, fine- to medium-grained</td>
<td></td>
<td>15</td>
<td>108.5</td>
<td>6.7</td>
</tr>
<tr>
<td>16</td>
<td>B2@22'</td>
<td>SM</td>
<td>-Medium dense, fine- to coarse-grained sand</td>
<td></td>
<td>22</td>
<td>120.7</td>
<td>8.8</td>
</tr>
<tr>
<td>18</td>
<td>B2@25'</td>
<td></td>
<td>End at 25.5 feet. Artificial fill to 1.5 feet. No groundwater encountered. Backfilled and tamped with soil cuttings. Capped with asphalt patch.</td>
<td></td>
<td>27</td>
<td>121.5</td>
<td>11.0</td>
</tr>
</tbody>
</table>

**NOTE:** The log of subsurface conditions shown hereon applies only at the specific boring or trench location and at the date indicated. It is not warranted to be representative of subsurface conditions at other locations and times.
APPENDIX B
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 B12. The in-place dry density and moisture content of the samples tested are presented on the boring logs, Appendix A.
**DIRECT SHEAR TEST RESULTS**

**SAMPLE** | **SOIL TYPE** | **DENSITY** | **INITIAL MOISTURE (%)** | **FINAL MOISTURE (%)**
--- | --- | --- | --- | ---
B4 @ 5' | CL | 105.3 | 18.9 | 22.4
B6 @ 4' | ML | 116.1 | 13.1 | 14.8

**FIG. B1**

**CITY HALL PARKING STRUCTURE**
8300 SANTA MONICA BOULEVARD
WEST HOLLYWOOD, CALIFORNIA

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

HHD 8000

**NOV 27, 2012**  **PROJECT NO. A8635-06-02A**  **FIG. B1**
DIRECT SHEAR TEST RESULTS

CITY HALL PARKING STRUCTURE
CITY OF WEST HOLLYWOOD
8300 SANTA MONICA BOULEVARD
WEST HOLLYWOOD, CALIFORNIA

HHD 8000

NOV 27, 2012
PROJECT NO. A8635-06-02A
FIG. B2
WATER ADDED AT 2 KSF

B5 @ 5'
B5 @ 7'
B5 @ 10'

Percent Consolidation

Consolidation Pressure (KSF)

NOV 27, 2012 PROJECT NO. A8635-06-02A
CITY HALL PARKING STRUCTURE
8300 SANTA MONICA BOULEVARD
WEST HOLLYWOOD, CALIFORNIA

JMT 8000  PHONE (818) 841-8388  -  FAX (818) 841-1704

ENVIRONMENTAL GEOTECHNICAL MATERIALS
3303 N. SAN FERNANDO BLVD. - SUITE 100 - BURBANK, CA 91504
WATER ADDED AT 2 KSF

Percent Consolidation vs. Consolidation Pressure (KSF)

B5 @ 13'
B5 @ 17'
B5 @ 20'

NOV 27, 2012 PROJECT NO. A8635-06-02A
CITY HALL PARKING STRUCTURE
8300 SANTA MONICA BOULEVARD
WEST HOLLYWOOD, CALIFORNIA

CONSOLIDATION TEST RESULTS
CITY OF WEST HOLLYWOOD
8300 SANTA MONICA BOULEVARD
WEST HOLLYWOOD, CALIFORNIA

NOV 27, 2012 PROJECT NO. A8635-06-02A FIG. B5
Consolidation Test Results

Consolidation Pressure (KSF)

WATER ADDED AT 2 KSF

B6 @ 12'

B6 @ 15'

B6 @ 20'

Percent Consolidation

Consolidation Pressure (KSF)
CONSOLIDATION TEST RESULTS

WATER ADDED AT 2 KSF

Percent Consolidation

Consolidation Pressure (KSF)

B3 @ 30'

B1 @ 35'

FIG. B9

NOV 27, 2012 PROJECT NO. A8635-06-02A
CITY HALL PARKING STRUCTURE
8300 SANTA MONICA BOULEVARD
WEST HOLLYWOOD, CALIFORNIA

JMT 8000
PHONE (818) 841-8388 - FAX (818) 841-1704
ENVIRONMENTAL GEOTECHNICAL MATERIALS
3303 N. SAN FERNANDO BLVD. - SUITE 100 - BURBANK, CA 91504

CONSOLIDATION TEST RESULTS
CITY HALL PARKING STRUCTURE
CITY OF WEST HOLLYWOOD
8300 SANTA MONICA BOULEVARD
WEST HOLLYWOOD, CALIFORNIA

NOV 27, 2012 PROJECT NO. A8635-06-02A FIG. B9
CONSOLIDATION TEST RESULTS

WATER ADDED AT 2 KSF

Consolidation Pressure (KSF)

Percent Consolidation

B1 @ 5'

B1 @ 12.5'

CITY HALL PARKING STRUCTURE
CITY OF WEST HOLLYWOOD
8300 SANTA MONICA BOULEVARD
WEST HOLLYWOOD, CALIFORNIA

NOV 27, 2012 PROJECT NO. A8635-06-02A FIG. B10
### SUMMARY OF LABORATORY MAXIMUM DENSITY AND OPTIMUM MOISTURE CONTENT TEST RESULTS

**ASTM D 1557-12**

<table>
<thead>
<tr>
<th>Sample No.</th>
<th>Soil Description</th>
<th>Maximum Dry Density (pcf)</th>
<th>Optimum Moisture (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>B5 &amp; B6 @ 0-5'</td>
<td>Brown Sandy Silt</td>
<td>125.5</td>
<td>13.0</td>
</tr>
</tbody>
</table>

### SUMMARY OF LABORATORY EXPANSION INDEX TEST RESULTS

**ASTM D 4829-08A**

<table>
<thead>
<tr>
<th>Sample No.</th>
<th>Moisture Content (%) Before</th>
<th>Moisture Content (%) After</th>
<th>Dry Density (pcf)</th>
<th>Expansion Index</th>
<th>*UBC Classification</th>
<th>**CBC Classification</th>
</tr>
</thead>
<tbody>
<tr>
<td>B2 @ 10-13' &amp; B4 @ 10-14'</td>
<td>11.7</td>
<td>25.9</td>
<td>109.8</td>
<td>108</td>
<td>High</td>
<td>Expansive</td>
</tr>
<tr>
<td>B5 @ 5'</td>
<td>6.3</td>
<td>31.4</td>
<td>89.0</td>
<td>3</td>
<td>Very Low</td>
<td>Non Expansive</td>
</tr>
<tr>
<td>B5 &amp; B6 @ 0-5'</td>
<td>11.8</td>
<td>21.8</td>
<td>110.3</td>
<td>59</td>
<td>Medium</td>
<td>Expansive</td>
</tr>
</tbody>
</table>

* Reference: 1997 Uniform Building Code, Table 18-I-B.
** Reference: 2010 California Building Code, Section 1803.5.3
## SUMMARY OF LABORATORY POTENTIAL OF HYDROGEN (pH) AND RESISTIVITY TEST RESULTS
### CALIFORNIA TEST NO. 643

<table>
<thead>
<tr>
<th>Sample No.</th>
<th>pH</th>
<th>Resistivity (ohm centimeters)</th>
</tr>
</thead>
<tbody>
<tr>
<td>B2 @ 10-13' &amp; B4 @ 10-14'</td>
<td>7.8</td>
<td>1000 (Extremely Corrosive)</td>
</tr>
</tbody>
</table>

## SUMMARY OF LABORATORY CHLORIDE CONTENT TEST RESULTS
### CALIFORNIA TEST NO. 422

<table>
<thead>
<tr>
<th>Sample No.</th>
<th>Chloride Ion Content (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>B2 @ 10-13' &amp; B4 @ 10-14'</td>
<td>0.006</td>
</tr>
</tbody>
</table>

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

<table>
<thead>
<tr>
<th>Sample No.</th>
<th>Water Soluble Sulfate (% SO₄)</th>
<th>Sulfate Exposure*</th>
</tr>
</thead>
<tbody>
<tr>
<td>B2 @ 10-13' &amp; B4 @ 10-14'</td>
<td>0.013</td>
<td>Negligible</td>
</tr>
<tr>
<td>B5 &amp; B6 @ 0-5'</td>
<td>0.009</td>
<td>Negligible</td>
</tr>
</tbody>
</table>

*Reference: 2010 California Building Code, Section 1904.3 and ACI 381 Section 4.3.*