APPENDIX D

GEOTECHNICAL INVESTIGATION

GEOCON
WEST, INC.
GEOTECHNICAL
ENVIRONMENTAL
MATERIALS

UPDATED GEOTECHNICAL INVESTIGATION

PROPOSED MIXED-USE DEVELOPMENT
7141-7155 SANTA MONICA BOULEVARD
WEST HOLLYWOOD, CALIFORNIA

PREPARED FOR
GLJ PARTNERS
CARLSBAD, CALIFORNIA

PROJECT NO. A8936-06-01

JUNE 1, 2012

Project No. A8936-06-01
June 1, 2012

GLJ Partners
5780 Fleet Street, Suite 130
Carlsbad, CA 92008

Attention: Mr. Tony Ditteaux

Subject: UPDATED GEOTECHNICAL INVESTIGATION
PROPOSED MIXED-USE DEVELOPMENT
7141-7155 SANTA MONICA BOULEVARD
WEST HOLLYWOOD, CALIFORNIA

Reference: Geotechnical Engineering Investigation, Proposed Apartment Building, 7141 - 7155 Santa Monica Boulevard, West Hollywood, California, prepared by Geotechnologies, Inc., File No. 19079, dated April 11, 2008.

Dear Mr. Ditteaux:

In accordance with your authorization of our proposal dated May 11, 2012, we have prepared an updated geotechnical investigation for the proposed mixed-use development located at 7141 - 7155 Santa Monica Boulevard, West Hollywood, California. The accompanying report presents the findings of our study, and our conclusions and recommendations pertaining to the geotechnical aspects of proposed design and construction. Based on the results of our investigation, it is our opinion that the site can be developed as proposed, provided the recommendations in this report are followed and implemented during design and construction.

As a part of this investigation we have reviewed the referenced report by Geotechnologies, Inc., and we have incorporated pertinent information into this report, and accept responsibility for its use. Geocon West, Inc. is the Geotechnical Consultant of Record and will be providing all necessary geotechnical consultation, plan review, design recommendations, inspection and testing services for this project. Where differing, the recommendations presented herein supersede all previous recommendations.

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

Very truly yours,
GEOCON WEST, INC.

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APPENDIX A
PRIOR GEOTECHNICAL REPORT

UPDATED GEOTECHNICAL INVESTIGATION

1. PURPOSE

This report presents the results of an updated geotechnical investigation for the proposed mixed-use development located at 7141 – 7155 Santa Monica Boulevard, West Hollywood, California (see Figure 1, Vicinity Map). The purpose of the investigation was to evaluate subsurface soil and geologic conditions underlying the area of proposed development and based on conditions encountered, to provide conclusions and recommendations pertaining to the geotechnical aspects of proposed design and construction.

The scope of our investigation included review of a prior geotechnical investigation report pertaining to a prior development planned at the site, engineering analysis, and the preparation of this report.

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

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

2. SITE AND PROJECT DESCRIPTION

The subject property is located at 7141 – 7155 Santa Monica Boulevard, West Hollywood, California. The property consists of five parcels and is occupied by multiple one- and two-story commercial buildings and at-grade parking. The property is bounded by one- and two-story on-grade apartment buildings to the north, Detroit Street to the east, Formosa Avenue to the west, and Santa Monica Boulevard to the south (Figure 2).

The majority of the site slopes gently to the south-southwest, with up to 5 feet of vertical relief across the existing pad. Site elevations range from 289 MSL at the northeast corner of the site to 284 MSL at the southwest corner of the site. Surface water drainage at the site appears to be by sheet flow along the existing ground contours to the city street and site boundaries.

The proposed mixed-use development will consist of between 2 and 5 levels of multi-family residential construction over one level of retail and residential construction, with heights of up to 72 feet above the ground surface. One full subterranean parking level (P-1) and a partial second subterranean parking level (P-2) are planned as part of the proposed development. Excavations for subterranean parking levels are anticipated to be on the order of 11 to 23 feet. Based on progress Design Plans prepared by Studio One Eleven, finish floor elevations for the subterranean parking levels range from 278.3 feet Mean Sea Level (MSL) to 263.5 feet MSL. The planned finish floor elevations and limits of the P-1 and P-2 subterranean levels are indicated on the Site Plan and Cross-Sections (see Figures 2 and 3).

Due to the preliminary nature of the design at this time, wall and column loads were not made available. It is estimated that wall loads for the proposed structure could be up to 8 kips per linear foot, and column loads could up to 800 kips.

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

3. BACKGROUND REVIEW

A previous geotechnical investigation was performed at the site by Geotechnologies, Inc. The investigation included the drilling and logging of four hollow stem-auger borings at the locations shown on Figure 2. The borings were drilled on November 28 and 29, 2005 to depths between 40 and 70 feet below the existing ground surface. Groundwater was encountered in all borings at a depth of 21 feet below the existing ground surface. Laboratory tests were performed on selected soil samples obtained during the site exploration.

The recommendations presented herein are based on the data obtained from the investigation by Geotechnologies, Inc., as well as our own analysis of the data. We have reviewed the report by Geotechnologies, Inc., and we concur with and assume responsibility for the utilization of the exploration and laboratory data presented therein. Geocon West, Inc. is the Geotechnical Consultant of Record and will be providing all necessary geotechnical consultation, plan review, design recommendations, inspection and testing services for this project. Where differing, the recommendations presented herein supersede all previous recommendations. A copy of the report by Geotechnologies, Inc. is presented in Appendix A of this report.

4. GEOLOGIC SETTING

The site is located in the northern portion of the Los Angeles Basin on a gently south to southeast sloping surface approximately one mile south of the Santa Monica Mountains. This topographic feature is known as the La Brea Plain, an elevated and dissected older alluvial surface that has been folded into an east-west anticlinal structure (CDWR, 1961).

Regionally, the site is located within the Peninsular Ranges geomorphic province, near the boundary of the Transverse Ranges geomorphic province. The Peninsular Ranges is characterized by northwest-trending geologic structures and physiographic features such as the Newport-Inglewood fault zone located approximately 4 miles to the west. The Hollywood fault zone located approximately 0.6 mile to the north forms the boundary between the Peninsular Ranges and the Transverse Ranges geomorphic provinces.

5. GEOLOGIC MATERIALS

Based on our review of the available geologic maps of the area, as well as the referenced geotechnical report, the site is underlain by artificial fill and Quaternary alluvial soils (CDMG, 1998; Dibblee, 1991; CDWR, 1961). The alluvial soils are mapped as young sediments (Holocene age) by CDMG and older alluvial sediments (Pleistocene age) by Dibblee and CDWR. The young alluvial deposits and underlying older alluvial deposits are composed of sediments derived from the nearby Santa Monica Mountains. The alluvial soils are underlain by Tertiary age sedimentary rocks at depth. The geologic conditions at the site with respect to the proposed development are described below.

5.1 Artificial Fill

Up to 3 feet of artificial fill was encountered in the borings. The artificial fill generally consists of medium stiff to stiff clay and silty clay and medium dense silty sand. The fill is likely the result of past grading and construction activities at the site. Deeper fill may exist between excavations and in other portions of the site that were not explored.

5.2 Alluvium

The artificial fill materials are underlain by alluvial deposits. Based on published geologic information, younger alluvium may be present at the site. Based on blow counts recorded on the Geotechnologies boring logs, the younger alluvial soils, if present, are less than seven feet thick and consist of clayey sand and sand with minor gravel. The older alluvial deposits encountered in the borings are predominantly fine-grained soils consisting of clay, silt and fine grained clayey sand, silty sand and sand.

6. GROUNDWATER

Based on a review of the Seismic Hazard Evaluation of the Hollywood 7.5 Minute Quadrangle, Los Angeles County, California (CGS, formerly California Division of Mines & Geology, 1998), the historic high ground water in the vicinity of the site is at a depth of approximately 20 feet below the existing ground surface. Groundwater information presented in this document is generated from data collected in the early 1900's to the date of publication.

Groundwater was encountered at a depth of 21 feet below the existing ground surface in all prior borings drilled at the site by Geotechnologies in 2005. The depth to groundwater corresponds to elevations of 264½ feet MSL at the southwest corner of the site and 268 feet MSL at the northeast corner of the site. As reported by Geotechnologies, these groundwater level elevations are consistent with water levels summarized in a Groundwater Monitoring and Sampling report by Professional Services Industries, Inc. (PSI) dated March 26, 2008 where groundwater levels measured in on-site wells range from Elevation 262.5 to Elevation 267.5 at the northeast and southwest corners of the site, respectively. It should be noted that the PSI report was not available for our review and the monitoring period duration for these water level measurements is not reported.

Based on the data presented above, the project should be designed considering the historic high groundwater level of 20 feet. Due to the sloping nature of the site, this corresponds to an elevation of 269 feet MSL, at the northeast corner of the site and an elevation of 264 feet, MSL, at the southwest corner of the site.

It is not uncommon for groundwater levels to vary seasonally or for perched groundwater conditions to develop where none previously existed, especially in impermeable fine-grained soils which are subjected to irrigation or precipitation. 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 S*urface Drainage* of this report (see Section 8.25).

7. GEOLOGIC HAZARDS

7.1 Surface Fault Rupture

The numerous faults in Southern California include active, potentially active, and inactive faults. The criteria for these major groups are based on criteria developed by the California Geological Survey for the Alquist-Priolo Earthquake Fault Zone Program (Hart, 1999). 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 site is not located 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 faults in the vicinity of the site are shown in Figure 4, Regional Fault Map.

The closest surface trace of an active fault to the site is the Hollywood Fault located approximately 0.6 mile to the north (Ziony and Jones, 1989). Other nearby active faults are the Santa Monica Fault, the Newport-Inglewood Fault Zone, the Raymond Fault, and the Verdugo Fault located 3.4 miles south-southwest, 4.1 miles west, 6.2 miles east-northeast, and 7.5 miles northeast of the site, respectively (Ziony and Jones, 1989). The active San Andreas Fault Zone is located approximately 49 miles northeast of the site.

The closest potentially active fault to the site is the MacArthur Park Fault located approximately 2.1 miles east-southeast of the site. Other nearby potentially active fault are the Overland Fault, the Charnock Fault, and the Coyote Pass Fault located approximately 5.5 miles southwest, 6.5 miles southwest, and 9.0 miles southeast of the site, respectively (Ziony and Jones, 1989).

Several buried thrust faults, commonly referred to as blind thrusts, underlie the Los Angeles Basin at depth. These faults are not exposed at the ground surface and are typically identified at depths greater than 3.0 kilometers. The October 1, 1987 M_w 5.9 Whittier Narrows earthquake, and the January 17, 1994 M_w 6.7 Northridge earthquake were a result of movement on the buried thrust faults. The site is not located within the vertical surface projection of these mapped blind thrusts. However, even though these faults are not exposed at the surface and do not present a potential surface fault rupture hazard, these faults are considered active and capable of generating future earthquakes.

7.2 Seismicity

As with all of Southern California, the site has experienced historic earthquakes from various regional faults. The seismicity of the region surrounding the site was formulated based on research of an electronic database of earthquake data. The epicenters of recorded earthquakes with magnitudes equal to or greater than 4.0 within a radius of 60 miles of the site are depicted on Figure 5, Regional Seismicity Map. A number of earthquakes of moderate to major magnitude have occurred in the Southern California area within the last 100 years. A partial list of these earthquakes is included in the following table.

LIST OF HISTORIC EARTHQUAKES

Earthquake (Oldest to Youngest)	Date of Earthquake	Magnitude	Distance to Epicenter (Miles)	Direction to Epicenter
Long Beach	March 10, 1933	6.4	39	SSE
San Fernando	February 9, 1971	6.6	22	N
Whittier Narrows	October 1, 1987	5.9	15	E
Sierra Madre	June 28, 1991	5.8	23	NE
Landers	June 28, 1992	7.3	105	E
Big Bear	June 28, 1992	6.4	85	E
Northridge	January 17, 1994	6.7	14	NW
Hector Mine	October 16, 1999	7.1	120	NE

7.3 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.

7.3.1 Deterministic Analysis

Table 1 provides a list of 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 *EQFAULT*, (Blake, 2000), was utilized. Principal references used within *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). The resulting calculated peak horizontal accelerations at the site are indicated 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 1.021g.

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.

7.3.2 Probabilistic Analysis

The computer program *FRISKSP* (Blake, 2000) was used to perform a site-specific probabilistic seismic hazard analysis. The program is a modified version of FRISK (McGuire, 1978) that models faults as lines to evaluate site-specific probabilities of exceedance for 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 faults' 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 1.04g and 0.61g, respectively. Graphical representation of the analysis is presented on Figure 6.

7.4 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

Parameter	Value	2010 CBC Reference
Site Class	C	Table 1613.5.2
Spectral Response – Class B (short), S_S	1.702g	Figure 1613.5(3)
Spectral Response – Class B (1 sec), S_1	0.6g	Figure 1613.5(4)
Site Coefficient, F_a	1.0	Table 1613.5.3(1)
Site Coefficient, F_v	1.0	Table 1613.5.3(2)
Maximum Considered Earthquake Spectral Response Acceleration (short), S_{MS}	1.702g	Section 1613.5.3 (Eqn 16-36)
Maximum Considered Earthquake Spectral Response Acceleration – (1 sec), S_{M1}	0.60g	Section 1613.5.3 (Eqn 16-37)
5% Damped Design Spectral Response Acceleration (short), S_{DS}	1.135g	Section 1613.5.4 (Eqn 16-38)
5% Damped Design Spectral Response Acceleration (1 sec), S_{D1}	0.400g	Section 1613.5.4 (Eqn 16-39)

Conformance to the criteria in the previous 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.

7.5 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 State of California Seismic Hazard Zone Maps, (California Division of Mines and Geology, 1999) the site is not located within an area identified as having a potential for liquefaction. In addition, according to the City of West Hollywood Safety Element (2001), the site is not located within an area identified as having a potential for liquefaction.

Liquefaction analysis of the soil underlying the site is presented in the Geotechnologies, Inc. report and is based on SPT data obtained from boring B2 during the site investigation. The liquefaction potential evaluation was performed by utilizing the historic high groundwater table of 17 feet, a magnitude 7.1 earthquake, and a peak horizontal acceleration of 0.75g (DBE).

The results of the liquefaction analysis indicate that the alluvial soils underlying the site would not be prone to liquefaction during DBE ground motion.

7.6 Slope Stability

The topography at the site is gently sloping and the site is not within an area identified as having a potential for seismic slope instability (City of West Hollywood, 2001; CDMG, 1999). Additionally, according to the City of West Hollywood Safety Element (2001) the site is not located within a hillside area identified as having a potential for slope instability. No landslides have been identified at the site or in close proximity to the site. Additionally, the site is not in the path of any known or potential landslides. Therefore, the potential for slope stability hazards to adversely affect the proposed development is considered low.

7.7 Earthquake-induced Flooding

Earthquake-induced flooding is inundation caused by failure of dams or other water-retaining structures due to earthquakes. The site is located within an area identified as having a potential for inundation as a result of a failure or breech of Mulholland Dam (West Hollywood, 2001). 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. The possibility of dam failures during an earthquake has been addressed by the California Division of Mines and Geology in the earthquake planning scenarios for a magnitude 8.3 earthquake on the San Andreas fault zone (Davis et al., 1982) and a magnitude 7.0 earthquake on the Newport-Inglewood fault zone (Toppozada et al., 1988). As stated in both reports, catastrophic failure of a major dam as a result of a scenario earthquake is regarded as unlikely. 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.

7.8 Tsunamis, Inundation, and Flooding

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

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

According to the city of West Hollywood (2008), the site is in an area of minimal flooding potential (Zone X) as defined by the Federal Emergency Management Agency (FEMA).

7.9 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. Other 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 a methane zone is considered low. However, should it be determined that a methane study is required for the proposed development it is recommended that a qualified methane consultant be retained to perform the study and provide mitigation measures as necessary.

7.10 Subsidence

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

8. CONCLUSIONS AND RECOMMENDATIONS

8.1 General

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

8.1.2 Up to 3 feet of existing artificial fill was encountered during the site investigation. The existing fill encountered is believed to be the result of past grading and construction activities at the site. Deeper fill may exist in other areas of the site that were not directly explored. Future demolition of the existing structures and improvements which occupy the site will likely disturb the upper few feet of existing site soils. Excavation of the subterranean level is anticipated to penetrate through the existing fill and expose competent alluvial soils throughout the excavation bottom.

8.1.3 Groundwater was encountered during prior site exploration at a depth of 21 feet below the existing ground surface, corresponding to elevations of 264½ and 268 feet MSL. Excavation for the subterranean level is anticipated to extend to depths of up to 25 feet below the ground surface, including foundation excavations. The lowest elevation corresponding to excavation of the subterranean level is approximately 261½ feet MSL. Based on conditions encountered at the time of exploration, as well as consideration of the historic high depth to groundwater, groundwater is anticipated to be encountered during excavation. Due to the subterranean nature of the proposed structure and the potential for seasonal fluctuation in the groundwater level, temporary dewatering measures will be required to mitigate groundwater during excavation and construction.

8.1.4 If the subterranean level, which extends below the historic high groundwater level, is not designed for full hydrostatic pressure, a permanent dewatering system will be required to relieve and mitigate the water pressure. The historic high groundwater depth corresponds to an elevation of 269 feet MSL at the northeast corner of the site and 264 feet MSL at the southwest corner of the site. Recommendations for temporary and permanent dewatering are discussed in Sections 8.4 and 8.5 of this report.

8.1.5 Based on these considerations, a conventional foundation system may be utilized for support of the proposed structure provided foundations derive support in the competent alluvium found at or below a depth of 8 feet. Foundations should be deepened as necessary to penetrate through unsuitable soils and derive support in the competent alluvial soils. Any soils unintentionally disturbed should be properly compacted. The concrete slab-on-grade and ramp for the subterranean level may bear directly on the alluvial soils exposed at the excavation bottom as well as compacted soils if necessary. All foundation excavations must be observed and approved in writing by the Geotechnical Engineer prior to placement of steel or concrete.

8.1.6 As an alternative to spread foundations, a reinforced concrete mat foundation may also be utilized for support of the proposed structure. Recommendations for the design of a mat foundation system are provided in Section 8.9.

8.1.7 In order to minimize differential settlement across the stepped transition between the parking levels P-1 and P-2, the transition area will likely require a more heavily reinforced structural connection which should be designed by the project structural engineer.

8.1.8 Foundations for small outlying structures, such as block walls less than 6 feet in height, planter walls or trash enclosures, which will not be structurally tied-to the building foundations, 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 excavation and compaction cannot be performed, such as adjacent to property lines, foundations may bear in the undisturbed alluvial soils found at or below a depth of 18 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.

8.1.9 Due to the depth of the excavation and the proximity to the property lines, city streets and adjacent offsite structures, excavation of the proposed subterranean levels will require sloping and shoring measures in order to provide a stable excavation. Where shoring is required it is recommended that a soldier pile shoring system be utilized. In addition, where the proposed excavation will be deeper than and adjacent to an offsite structure, the proposed shoring should be designed to resist the surcharge imposed by the adjacent offsite structures. Recommendations for shoring are provided in Section 8.20 of this report.

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

8.1.11 Based on the high nature of groundwater at the subject site and depth of the subterranean level, a stormwater infiltration system is not recommended for this site. It is suggested that stormwater be retained, filtered and discharged in accordance with the requirements of the local governing agency.

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

8.1.13 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.

8.2 Soil and Excavation Characteristics

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

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

8.2.3 All onsite excavations must be conducted in such a manner that potential surcharges from existing structures, construction equipment, and vehicle loads are resisted. The surcharge area may be defined by a 1:1 projection down and away from the bottom of an existing foundation or vehicle load. Penetrations below this 1:1 projection will require special excavation measures such as sloping and shoring. Excavation recommendations are provided in the *Temporary Excavations* section of this report (see Section 8.19).

8.2.4 The soils encountered near the proposed subterranean level are considered to have a “low” expansive potential (EI = 45); and these soils are classified as “expansive” based on the 2010 California Building Code (CBC) Section 1803.5.3. The recommendations presented in this report assume that exterior slabs will derive support in these materials.

8.3 Minimum Resistivity, pH, Chloride and Water-Soluble Sulfate

8.3.1 Potential of Hydrogen (pH) and resistivity testing as well as chloride content testing were performed on representative samples of soil near the anticipated subterranean levels to generally evaluate the corrosion potential to surface utilities. The test results indicate that a potential for corrosion of buried ferrous metals exists on site and should be considered for design of underground structures.

8.3.2 Laboratory tests were performed on representative samples of soil near the anticipated subterranean levels to measure the percentage of water-soluble sulfate content. Results from the laboratory water-soluble sulfate tests 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.

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

8.4 Temporary Dewatering

8.4.1 Groundwater was encountered during prior site exploration at a depth of 21 feet below the ground surface, corresponding to elevations of 264½ and 268 feet MSL. The depth to groundwater at the time of construction can be further verified during initial dewatering well or shoring pile installation. If groundwater is present above the depth of the subterranean level, temporary dewatering will be necessary to maintain a safe working environment during excavation and construction activities.

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

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

8.4.4 Geocon can assist with water quality testing as well as obtaining discharge permits required for dewatering.

8.5 Permanent Dewatering

8.5.1 If the subterranean level which extends below the historic high groundwater level is not designed for full hydrostatic pressure, is not designed for hydrostatic pressure, a permanent dewatering system must be implemented to prevent the groundwater table from impacting the structure. The historic high groundwater depth corresponds to an elevation of 269 feet MSL at the northeast corner of the site and 264 at the southwest corner of the site. A subdrainage system consisting of perforated pipe placed in gravel-filled trenches may be installed beneath the subterranean slab-on-grade to intercept and control groundwater. This system can be combined with the perimeter retaining wall drainage system provided backflow valves are installed at the base of the wall drainage system

8.5.2 A typical permanent sub-slab drainage system would consist of a twelve-inch thick layer of ¾-inch gravel that is placed upon a layer of filter fabric (Miami 500X or equivalent), and vibrated to a dense state. Subdrain pipes leading to sump areas, provided with automatic pumping units, should drain the gravel layer. The drain lines should consist of perforated pipe, placed with perforations down, in trenches that are at least six inches below the gravel layer. The excavation bottom, as well as the trench bottoms should be lined with filter fabric prior to placing and compacting gravel. The trenches should be spaced approximately 40 feet apart at most, within the interior, and should extend along to the perimeter of the building. Subsequent to the installation of the drainage system, the waterproofing system and building slab may then be placed on the densified gravel. A mud- or rat-slab may be placed over the waterproofing system for protection during placement of rebar and mat slab construction.

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

8.6 Grading

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

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

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

8.6.4 Grading should commence with the removal of all existing vegetation and existing improvements from the area to be graded. Once a clean excavation bottom has been established it must be observed and approved in writing by the Geotechnical Engineer (a representative of Geocon West, Inc. Deleterious debris such as wood and root structures should be exported from the site and should not be mixed with the fill soils. Asphalt and concrete should not be mixed with the fill soils unless approved 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.

8.6.5 Due to the potential for high-moisture content soils at the excavation bottom, or if construction is performed during the rainy season and the excavation bottom becomes saturated, stabilization measures may have to be implemented to prevent excessive disturbance the excavation bottom.

Should this condition exist, rubber tire equipment should not be allowed in the excavation bottom until it is stabilized or extensive soil disturbance could result.

8.6.6 If a permanent dewatering system is to be installed, subgrade stabilization may be accomplished by placing a one-foot thick layer of washed, angular 3/4-inch gravel atop a stabilization fabric (Mirafi 500X or equivalent), subsequent to subgrade approval. This procedure should be conducted in sections until the entire excavation bottom has been blanketed by fabric and gravel. Heavy equipment may operate upon the gravel once it has been placed. The gravel should be compacted to a dense state utilizing a vibratory drum roller. The placement of gravel at the subgrade level should be coordinated with the temporary or permanent dewatering of the site. The gravel and fabric system will function as both a permeable material for any necessary dewatering procedures as well as a stable material upon which heavy equipment may operate. It is recommended that the contractor meet with the Geotechnical Engineer to discuss this procedure in more detail.

8.6.7 Where temporary or permanent dewatering is not required, an alternative method of subgrade stabilization would consist of 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. All subgrade soils must be properly compacted and proof-rolled in the presence of the Geotechnical Engineer (a representative of Geocon West, Inc.).

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

8.6.9 Foundations for small outlying structures, such as block walls less than 6 feet high, planter walls or trash enclosures, which will not be structurally tied-to the proposed building, 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 excavation and proper compaction cannot be performed or is undesirable, foundations may derive support directly in the undisturbed alluvial soils found at or below a depth of 18 inches below the ground surface, and should be deepened as necessary to maintain a minimum 12 inch embedment into the recommended bearing materials. If the 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.

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

8.6.11 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 40 and corrosivity properties that are equally or less detrimental to that of the existing onsite soils.

8.6.12 All 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.

8.7 Foundation Design - General

8.7.1 A conventional foundation system may be utilized for support of the proposed structure, provided foundations derive support in the competent alluvial soils found at or below a depth of 8 feet and/or the stabilized subgrade. Recommendations for a conventional foundation system are provided in Section 8.8 of this report.

8.7.2 As an alternative to spread foundations, a reinforced concrete mat foundation may also be utilized for support of the proposed structure. The mat foundation may derive support in the competent alluvial soils found at or below a depth of 8 feet below the existing ground surface and/or the stabilized subgrade. The use of a mat foundation system may improve construction efficiency and save time. Recommendations for a reinforced concrete mat foundation system are provided in Section 8.9 of this report.

8.7.3 If the proposed structure is to be designed for full hydrostatic pressure, the recommended floor slab uplift pressure to be used in design would be 62.4(H) in units of pounds per square foot, where "H" is the height of the water above the bottom of the mat foundation in feet. For design purposes the water table may be assumed at 20 feet below the existing ground surface. The historic high groundwater level corresponds to an elevation of 269 feet MSL at the northeast corner of the site and 264 feet MSL at the southwest corner of the site.

8.7.4 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. Footings should be deepened if necessary to extend into satisfactory bearing materials. Footing excavations should be cleaned of all loose soils prior to placing steel and concrete. All required footing backfill should be mechanically compacted; flooding is not permitted.

8.8 Conventional Foundation Design

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

8.8.2 Isolated spread foundations may be designed for an allowable bearing capacity of 4,500 pounds per square foot, and should be a minimum of 24 inches in width, 18 inches in depth below the lowest adjacent grade, and 12 inches into the recommended bearing material.

8.8.3 The soil bearing pressure above may be increased by 150 psf and 500 psf for each additional foot of foundation width and depth, respectively, up to a maximum allowable soil bearing pressure of 6,500 psf.

8.8.4 If depth increases are utilized for the exterior wall footings, this office should be provided a copy of the final construction plans so that the excavation recommendations presented herein could be properly reviewed and revised if necessary. Foundation depths should be established prior to finalization of the shoring design to ensure that the embedment of the shoring pile toes is maintained and accounted for in the shoring design.

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

8.8.6 Continuous footings should be reinforced with a minimum of four No. 4 steel reinforcing bars, two placed near the top of the footing and two near the bottom. The project structural engineer should design reinforcement for spread footings.

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

8.8.8 Due to the expansive potential of the anticipated subgrade soils at the subterranean level, the moisture content in the slab and foundation subgrade should be maintained at 2 percent above optimum moisture content prior to and at the time of concrete placement.

8.9 Mat Foundation Design

8.9.1 It is anticipated that the mat foundation will impart an average pressure of less than 2,500 psf, with locally higher pressures up to 4,000 psf. The recommended maximum allowable bearing value is 6,500 pounds per square foot. The allowable bearing pressure may be increased by up to one-third for transient loads due to wind or seismic forces.

8.9.2 It is recommended that a modulus of subgrade reaction of 200 pounds per cubic inch (pci) be utilized for the design of the mat foundation bearing in the competent alluvial soils. If the subgrade is stabilized in accordance with the recommendation of this report a modulus of subgrade reaction of 300 pounds per cubic inch (pci) may be utilized.

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

8.9.4 For seismic design purposes, a coefficient of friction of 0.33 may be utilized between the concrete mat and undisturbed alluvial soils, and 0.15 for slabs underlain by a moisture barrier.

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

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

8.10 Miscellaneous Foundations

8.10.1 Foundations for small outlying structures, such as block walls less than 6 feet in height, planter walls or trash enclosures, which will not be structurally supported by the proposed building, may be supported on conventional foundations bearing on a minimum of 12 inches of newly placed engineered fill which extends laterally at least 12 inches beyond the foundation area. Where excavation and compaction cannot be performed, such as adjacent to property lines, foundations may bear in the undisturbed alluvial soils found at or below a depth of 18 inches.

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

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

8.11 Foundation Settlement

8.11.1 The maximum expected static settlement for a structure supported on a conventional foundation system utilizing a maximum allowable soil bearing pressure of 6,500 psf and deriving support in the competent alluvial soils found at or below a depth of 8 feet is estimated to be less than 1 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 ½ inch over a distance of twenty feet.

8.11.2 The maximum anticipated static settlement for a reinforced concrete mat foundation with a maximum allowable bearing value of 6,500 psf deriving support in the older alluvial soils is estimated to be less than 1 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 ½ inch over a distance of twenty feet.

8.11.3 Where separated by a stepped transition, differential settlement between subterranean levels P-1 and P-2 could be on the order of ½ inch and will likely require a heavily reinforced structural connection, or a structural separation to account for the anticipated differential movements.

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

8.12 Lateral Design

8.12.1 Resistance to lateral loading may be provided by friction acting at the base of foundations, slabs and by passive earth pressure. An allowable coefficient of friction of 0.33 may be used with the dead load forces in the competent alluvium or in properly compacted engineered fill.

8.12.2 Passive earth pressure for the sides of foundations and slabs poured against the alluvial soils, stabilized subgrade, or properly compacted engineered fill below the groundwater table may be computed as an equivalent fluid having a density of 100 pcf with a maximum earth pressure of 1,500 pcf (these values have been adjusted for buoyant forces). Passive earth pressure for the sides of foundations and slabs poured against the alluvial soils, stabilized subgrade, or properly compacted engineered fill above the groundwater table may be computed as an equivalent fluid having a density of 220 pcf with a maximum earth pressure of 2,200 pcf. When combining passive and friction for lateral resistance, the passive component should be reduced by one-third.

8.13 Concrete Slabs-on-Grade

8.13.1 Unless specifically evaluated and designed by a qualified structural engineer, the slab-on-grade and ramp for the subterranean parking garage (properly drained to relieve hydrostatic pressure) 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 for the parking garage and ramp may bear directly on the competent alluvial soils found at the excavation bottom and/or engineered fill. Any disturbed soils should be properly compacted for slab support.

8.13.2 Slabs-on-grade at the ground surface that may receive moisture-sensitive floor coverings or may be used to store moisture-sensitive materials should be underlain by a vapor retarder placed directly beneath the slab. The vapor retarder and acceptable permeance should be specified by the project architect or developer based on the type of floor covering that will be installed. The vapor retarder design should be consistent with the guidelines presented in Section 9.3 of the American Concrete Institute's (ACI) *Guide for Concrete Slabs that Receive Moisture-Sensitive Flooring Materials* (ACI 302.2R-06) and should be installed in general conformance with ASTM E 1643-09 and the manufacturer's recommendations. If the California Green Code requirements apply to this project, the vapor retarder should be underlain by 4 inches of ½-inch clean 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.

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

8.13.4 For seismic design purposes, a coefficient of friction of 0.33 may be utilized between concrete slabs and subgrade soils without a moisture barrier, and 0.15 for slabs underlain by a moisture barrier.

8.13.5 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 12 feet and should be constructed using saw-cuts or other methods as soon as practical following concrete placement. Crack control joints should extend a minimum depth of one-fourth the slab thickness. The project structural engineer should design construction joints as necessary.

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

8.14 Retaining Walls

8.14.1 The recommendations presented below are generally applicable to the design of rigid concrete or masonry retaining walls having a maximum height of 25 feet. In the event that walls higher than 25 feet are planned, Geocon should be contacted for additional recommendations.

8.14.2 Retaining wall foundations may be designed in accordance with the recommendations provided in the *Foundation Design* section of this report (see Section 8.8).

8.14.3 Assuming that proper drainage and permanent dewatering is maintained, retaining walls with a level backfill surface that are not restrained at the top should be designed utilizing a triangular distribution of pressure (active pressure) of 30 pcf.

8.14.4 Restrained walls are those that are not allowed to rotate more than 0.001H (where H equals the height of the retaining portion of the wall in feet) at the top of the wall. Assuming that proper drainage and permanent dewatering is maintained, where walls are restrained from movement at the top, walls may be designed utilizing a triangular distribution of pressure (at-rest pressure) of 50 pcf.

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

8.14.6 Additional active pressure should be added for a surcharge condition due to sloping ground, vehicular traffic or adjacent structures and should be designed for each condition as the project progresses. The anticipated surcharge pressure from the adjacent one- and two-story offsite structures to the north are provided on the Cross-Section/Surcharge Calculation sheets (see Figures 7 and 8). Due to the preliminary nature of the project at this time, information regarding the depth of existing offsite foundations, the presence of subterranean levels, and actual offsite building loads were not available at the time this report was prepared; therefore, the surcharge calculations presented herein are preliminary, and likely conservative. Once the design becomes more finalized, an addendum letter can be prepared revising recommendations and addressing specific surcharge conditions throughout the project, if necessary.

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

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

8.15 Dynamic (Seismic) Lateral Earth Pressure

8.15.1 In accordance with the 2010 California Building Code, if the project possesses a seismic design category of D, E, or F, the proposed retaining walls should be designed with seismic lateral earth pressure. The structural engineer should determine the seismic design category for the project. The dynamic (seismic) lateral pressure is equal to the sum of the static active pressure and the dynamic (seismic) pressure increment.

8.15.2 Braced retaining walls should be designed for the greater of either the at-rest earth pressure or the dynamic (seismic) lateral earth pressure (sum of the static active pressure and the dynamic (seismic) pressure increment).

8.15.3 The application of seismic loading should be performed at the discretion of the project Structural Engineer and in accordance with the requirements of the Building Official. If seismic loading is to be applied, we recommend a dynamic (seismic) pressure increment of 13½H be used for design. The seismic pressure is dependent on the retained height where H is the height of the wall, in feet, and the calculated loads result in pounds per square foot (psf) applied uniformly along the wall height. This dynamic (seismic) pressure increment is for horizontal backfill behind the wall and does not account for an inclined backfill surface. The seismic pressure is based on a peak ground acceleration of 0.45g (S_{DS}/2.5) and by applying a pseudo-static coefficient of 0.5.

8.16 Retaining Wall Drainage

8.16.1 Retaining walls should be provided with a drainage system extended at least two-thirds the height of the wall. At the base of the drain system, a subdrain covered with a minimum of 12 inches of gravel should be installed, and a compacted fill blanket or other seal placed at the surface (see Figure 9). The clean bottom and subdrain pipe, behind a retaining wall, should be observed by the Geotechnical Engineer (a representative of Geocon), prior to placement of gravel or compacting backfill.

8.16.2 As an alternative, a plastic drainage composite such as Miradrain or equivalent may be installed in continuous, 4-foot wide columns along the entire back face of the wall, at 8 feet on center. The top of these drainage composite columns should terminate approximately 18 inches below the ground surface, where either hardscape or a minimum of 18 inches of relatively cohesive material should be placed as a cap (see Figure 10). These vertical columns of drainage material would then be connected at the bottom of the wall to a one-cubic-foot rock pocket drained by a 4-inch subdrain pipe.

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

8.17 Elevator Pit Design

8.17.1 The elevator pit slab and retaining wall should be designed by the project structural engineer. As a minimum the slab-on-grade for the elevator pit bottom should be at least 4 inches thick and reinforced with No. 3 steel reinforcing bars placed 18 inches on center in both horizontal directions, positioned near the slab midpoint. Elevator pit walls may be designed in accordance with the recommendations in the *Foundation Design and Retaining Wall Design* section of this report (see Sections 8.8 and 8.16).

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

8.17.3 If retaining wall drainage is to be provided, the drainage system should be designed in accordance with the *Retaining Wall Drainage* section of this report (see Section 8.16).

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

8.18 Elevator Piston

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

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

8.18.3 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. The contractor should also be prepared to mitigate buoyant forces during installation of the piston casing. Continuous observation of the drilling and installation of the elevator piston by the Geotechnical Engineer (a representative of Geocon West, Inc.) is required.

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

8.19 Temporary Excavations

8.19.1 Excavations on the order of 12 to 25 feet in height are anticipated for excavation and construction of the proposed subterranean levels and foundations. The excavations are expected to expose artificial 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.

8.19.2 Excavation for the subterranean level will require sloping or shoring measures in order to provide a stable excavation. *Shoring* data is provided in Section 8.20 of this report.

8.19.3 Where sufficient space is available, temporary unsurcharged embankments could be sloped back at a uniform 1:1 slope gradient or flatter up to a maximum height of 12 feet. A uniform slope does not have a vertical portion. Slopes in excess of 12 feet in height should be sloped back at a uniform 1½:1 gradient or flatter.

8.19.4 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. Geocon personnel should inspect the soils exposed in the cut slopes during excavation so that modifications of the slopes can be made if variations in the soil conditions occur. All excavations should be stabilized within 30 days of initial excavation.

8.20 Shoring – Soldier Pile Design and Installation

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

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

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

8.20.4 Drilled cast-in-place soldier piles should be placed no closer than 2 diameters on center. The minimum diameter of the piles is 18 inches. Structural concrete should be used for the soldier piles below the excavation; lean-mix concrete may be employed above that level. As an alternative, lean-mix concrete may be used throughout the pile where the reinforcing consists of a wideflange section. The slurry must be of sufficient strength to impart the lateral bearing pressure developed by the wideflange section to the soil. For design purposes, an allowable passive value for the soils below the bottom plane of excavation may be assumed to be 265 pounds per square foot per foot for the portion of the pile above the water table, and 120 pounds per square foot per foot for the portion of the pile below the water table (value has been reduced for buoyant forces). The allowable capacity may be doubled for isolated piles spaced more than twice the diameter. To develop the full lateral value, provisions should be implemented to assure firm contact between the soldier piles and the undisturbed soils.

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

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

8.20.7 Casing may be required since caving may occur in the saturated soils. If casing is used, extreme care should be employed so that the pile is not pulled apart as the casing is withdrawn. At no time should the distance between the surface of the concrete and the bottom of the casing be less than five feet. Continuous observation of the drilling and pouring of the piles by the Geotechnical Engineer (a representative of Geocon West, Inc.), is required.

8.20.8 The frictional resistance between the soldier piles and retained soil may be used to resist the vertical component of the anchor load. The coefficient of friction may be taken as 0.33 based on uniform contact between the steel beam and lean-mix concrete and retained earth. The portion of soldier piles below the plane of excavation may also be employed to resist the downward loads. The downward capacity may be determined using a frictional resistance of 650 pounds per square foot for the portion of the pile above the water table, and 400 pounds per square foot per foot for the portion of the pile below the water table (value has been reduced for buoyant forces).

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

8.20.10 The time between lagging excavation and lagging placement should be as short as possible Soldier piles should be designed for the full-anticipated pressures. Due to arching in the soils, the pressure on the lagging will be less. It is recommended that the lagging be designed for the full design pressure but be limited to a maximum of 400 pounds per square foot.

8.20.11 Assuming that a permanent dewatering system is implemented just outside the shoring system, and that pumping is continuously maintained throughout the excavation and construction process it is recommended that an equivalent fluid pressure based on the table below, be utilized for shoring design with a level backfill surface.

HEIGHT OF CANTILEVERED SHORING (FEET)	EQUIVALENT FLUID PRESSURE (Pounds Per Cubic Foot) (ACTIVE PRESSURE)	EQUIVALENT FLUID PRESSURE (Pounds Per Cubic Foot) (AT-REST PRESSURE)
Up to 25	25	45

8.20.12 It is very important to note that active pressures can only be achieved when movement in the soil (earth wall) occurs. If movement in the soil is not acceptable, such as adjacent to an existing structure, the at-rest pressure should be considered for design purposes.

8.20.13 Where a combination of sloped embankment and shoring is utilized, the pressure will be greater and must be determined for each combination. Additional active pressure should be added for a surcharge condition due to sloping ground, vehicular traffic, or adjacent structures and must be determined for each combination. The anticipated surcharge pressure from the adjacent one- and two-story offsite structures to the north are provided on Figures 7 and 8 and should be incorporated into the shoring design as necessary. Information regarding the depth of existing adjacent foundations, the presence of subterranean levels, actual offsite building loads, and location of the proposed excavation were not available at the time this report was prepared; therefore, the surcharge calculations presented herein are preliminary and should be reviewed as the design progresses. Once design becomes more finalized, an addendum letter can be prepared revising recommendations and addressing specific surcharge conditions throughout the project, if necessary.

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

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

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

8.21 Tie-Back Anchors

8.21.1 Tie-back anchors may be used to resist lateral loads. Friction anchors are recommended. For design purposes, it may be assumed that the active wedge adjacent to the shoring is defined by a plane drawn 35 degrees with the vertical through the bottom plane of the excavation. Friction anchors should extend a minimum of 20 feet beyond the potentially active wedge and to greater lengths if necessary to develop the desired capacities. The locations and depths of all offsite utilities should be thoroughly checked and incorporated into the drilling angle design for the tie-back anchors.

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

- Up to 5 feet below the top of the excavation – 750 pounds per square foot (dry condition).
- Up to 12 feet below the top of the excavation – 800 pounds per square foot (value has been reduced for buoyant forces).
- Up to 17 feet below the top of the excavation – 900 pounds per square foot (value has been reduced for buoyant forces).

8.21.3 Depending on the techniques utilized, and the experience of the contractor performing the installation, it is anticipated that a friction capacity in excess of 2.5 kip per linear foot could be utilized for post-grouted anchors. The maximum allowable friction capacity is 2.8 kips per linear foot (for a 20 foot length beyond active wedge). Only the frictional resistance developed beyond the active wedge should be utilized in resisting lateral loads.

8.22 Anchor Installation

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

8.23 Anchor Testing

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

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

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

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

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

8.24 Internal Bracing

8.24.1 Rakers may be utilized to brace the soldier piles in lieu of tieback anchors. The raker bracing could be supported laterally by temporary concrete footings (deadmen) or by the permanent, interior footings. For design of such temporary footings or deadmen, poured with the bearing surface normal to rakers inclined at 45 degrees, a bearing value of 1,500 pounds per square foot may be used, provided the shallowest point of the footing is at least one foot below the lowest adjacent grade. The client should be aware that the utilization of rakers could significantly impact the construction schedule do to their intrusion into the construction site and potential interference with equipment. In addition, it is extremely important the project structural engineer and project shoring engineer review each other’s plans for potential foundation conflicts.

8.25 Surface Drainage

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

8.25.2 All site drainage should be collected and controlled in non-erosive drainage devices. Drainage should not be allowed to pond anywhere on the site, and especially not against any foundation or retaining wall. The site should be graded and maintained such that surface drainage is directed away from structures in accordance with 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.

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

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

8.26 Plan Review

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

LIMITATIONS AND UNIFORMITY OF CONDITIONS

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

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

3. The findings of this report are valid as of the 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

Anderson, J. G., 1984, *Synthesis of Seismicity and Geologic Data in California*, U.S. Geological Survey Open File Report 84-424.

Blake, T.F., 2000, EQFAULT, *A Computer Program for the Deterministic Prediction of Peak Horizontal Acceleration from Digitized California Faults*, Version 2.20.

Blake, T.F., 2000, EQSEARCH, *A Computer Program for the Estimation of Peak Horizontal Acceleration from California Historical Earthquake Catalogs*, Version 2.20.

Blake, T.F., 2000, FRISKSP, *A Computer Program for the Probabilistic Estimation of Uniform-Hazard Spectra Using 3-D Faults as Earthquake Sources.*

Boore, D.M., Joyner, W.B., and Fumal, T.E., 1997, *Equations for Estimating Horizontal Response Spectra and Peak Acceleration from Western North American Earthquakes*, A *Summary of Recent Work*, Seismological Research Letters, Vol. 68, No. 1, pp. 128-153.

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

California Department of Conservation, Division of Mines and Geology: *Digital Images of Official Maps of Alquist-Priolo Earthquake Fault Zones of California, Central Coast Region*, DMG, CD 2000-004.

California Division of Mines and Geology, 1999, Seismic Hazard Zone Map, Hollywood Quadrangle, Los Angeles County, California.

California Division of Mines and Geology, 1997, “Guidelines for Evaluating and Mitigating Seismic Hazards in California,” Special Publication 117, revised 2008.

California Division of Mines and Geology, 1998, Seismic Hazard Evaluation of the Hollywood 7.5-*Minute Quadrangle, Los Angeles County, California*, Open-File Report 98-17.

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

Davis, J. F., Bennett, J. H., Borchardt, G. A., Kahle, J. E., Rice, S. J., Silva, M. A., 1982, *Earthquake Planning Scenario for a Magnitude 8.3 Earthquake on the San Andreas Fault in Southern California*, California Division of Mines and Geology Special Publication 60.

Dibblee, T. W. Jr., 1991, *Geologic Map of the Hollywood and Burbank (South ½) Quadrangles, Los Angeles County*, California, Dibblee Geological Foundation Map #DF-30.

FEMA and ESRI, 2010, Online Flood Hazard Maps, http://www.esri.com/hazards/index.html.

Geotechnologies, Inc., 2005, *Geotechnical Engineering Investigation, Proposed Apartment Building, 7141-7155 Santa Monica Boulevard, West Hollywood, California*, File No. 19709, report dated December 9, 2005, revised April 11, 2008.

Hart, E. W., 1973, revised 1999, *Fault-Rupture Hazard Zones in California, Alquist-Priolo Earthquake Fault Zoning Act with Index to Earthquake Fault Zone Maps*, California Division of Mines and Geology Special Publication 42.

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

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.*

Lamar, D.L., 1970, *Geology of the Elysian Park-Repetto Hills Area, Los Angeles County, California*, California Division of Mines and Geology Special Report 101.

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

Los Angeles Department of Public Works, 2004, *Methane and Methane Buffer Zones, Citywide Methane Ordinance Map A-20960,* City Ordinance No. 175,790

Los Angeles County Department of Public Works, 2012, Ground Water Wells Website, http://dpw2.co.la.ca.us/website/wells/viewer.asp

Sadigh, K., Chang, C.Y., Egan, J.A., Makdisi, F., and Youngs, R.R., 1997, *Attenuation Relationships for Shallow Crustal Earthquakes Based on California Strong Motion Data*, Seismological Research Letters, Vol. 68, No. 1.

Tinsley, J.C., Youd, T.L., Perkins, D.M., and Chen, A.T.F., 1985, *Evaluating Liquefaction Potential in Evaluating Earthquake Hazards in the Los Angeles Region-An Earth Science Perspective*, U.S. Geological Survey Professional Paper 1360, edited by J.I. Ziony, U.S. Government Printing Office, pp. 263-315.

Toppozada, T. R., Bennett, J. H., Borchardt, G. A., Saul, R., and Davis, J. F., "1988, "Planning Scenario for a Major Earthquake on the Newport–Inglewood Fault Zone," *California Division of Mines and Geology Special Publication 99.*

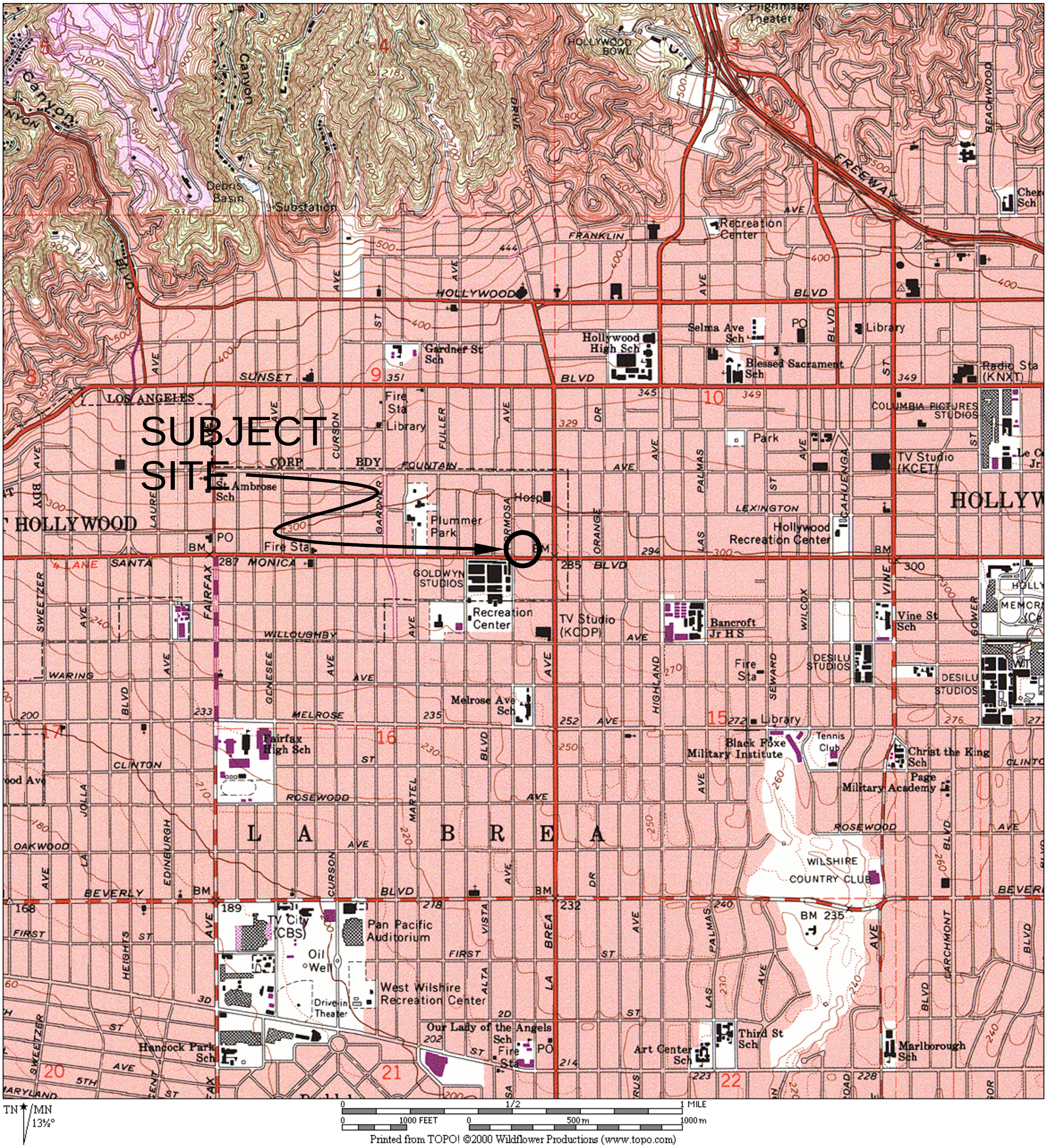
U.S. Geological Survey, 1972, *Hollywood 7.5-Minute Topographic Map*.

Wesnousky, S. G., 1986, "Earthquakes, Quaternary Faults and Seismic Hazard in California," *Journal of Geophysical Research*, Vol. 91, No. B12, pp. 12,587–12,631.

West Hollywood, City of, 2008, *Approval of Revised Flood Insurance Designation Letter of Map Revision (LOMR), Case No.:08-09-0191P, West Hollywood, CA*, Effective Date May 30, 2008.

West Hollywood, City of, 2001, *Safety Element of the General Plan*.

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



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

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VICINITY MAP

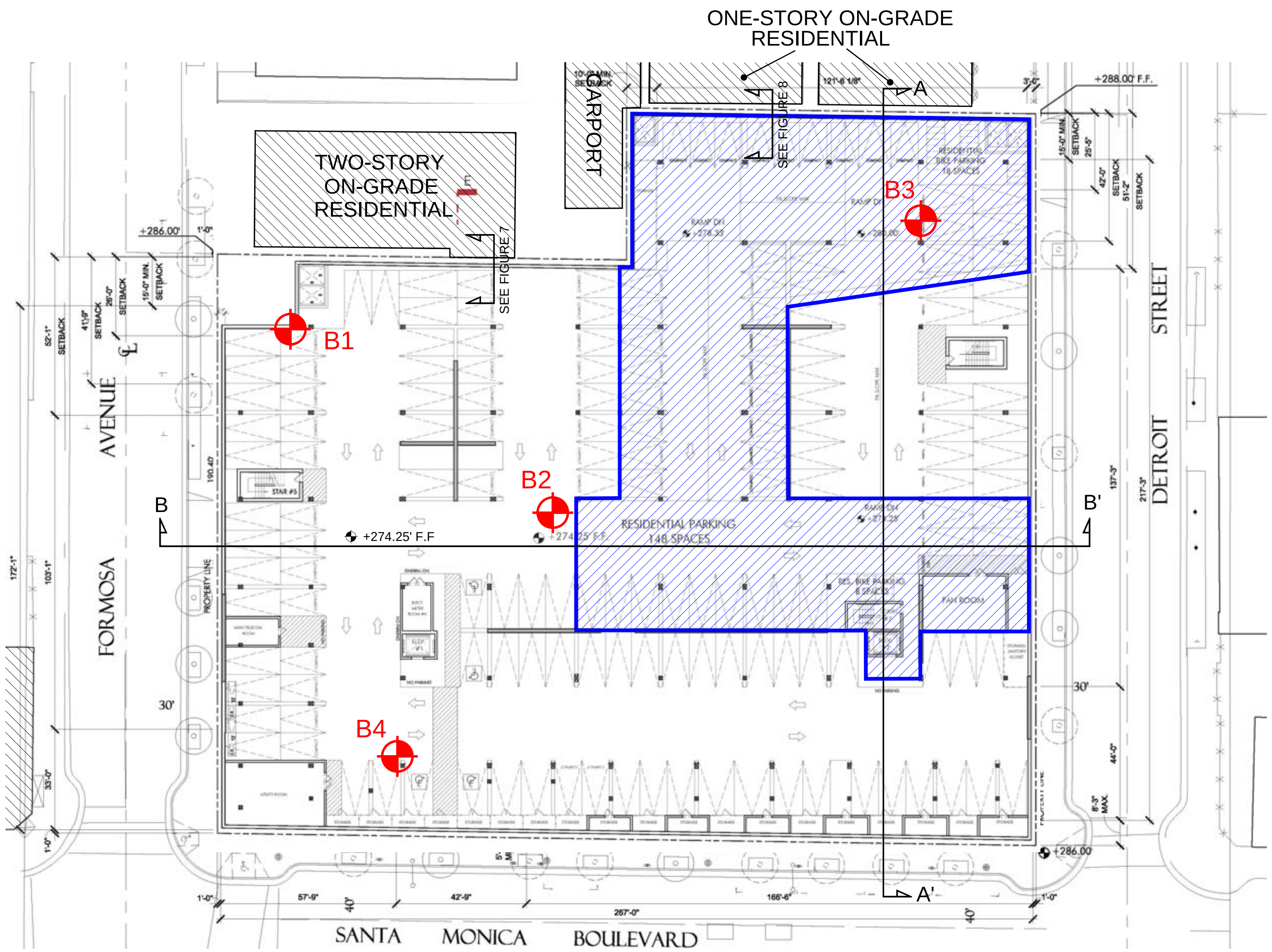
GLJ PARTNERS

7141-7155 SANTA MONICA BOULEVARD

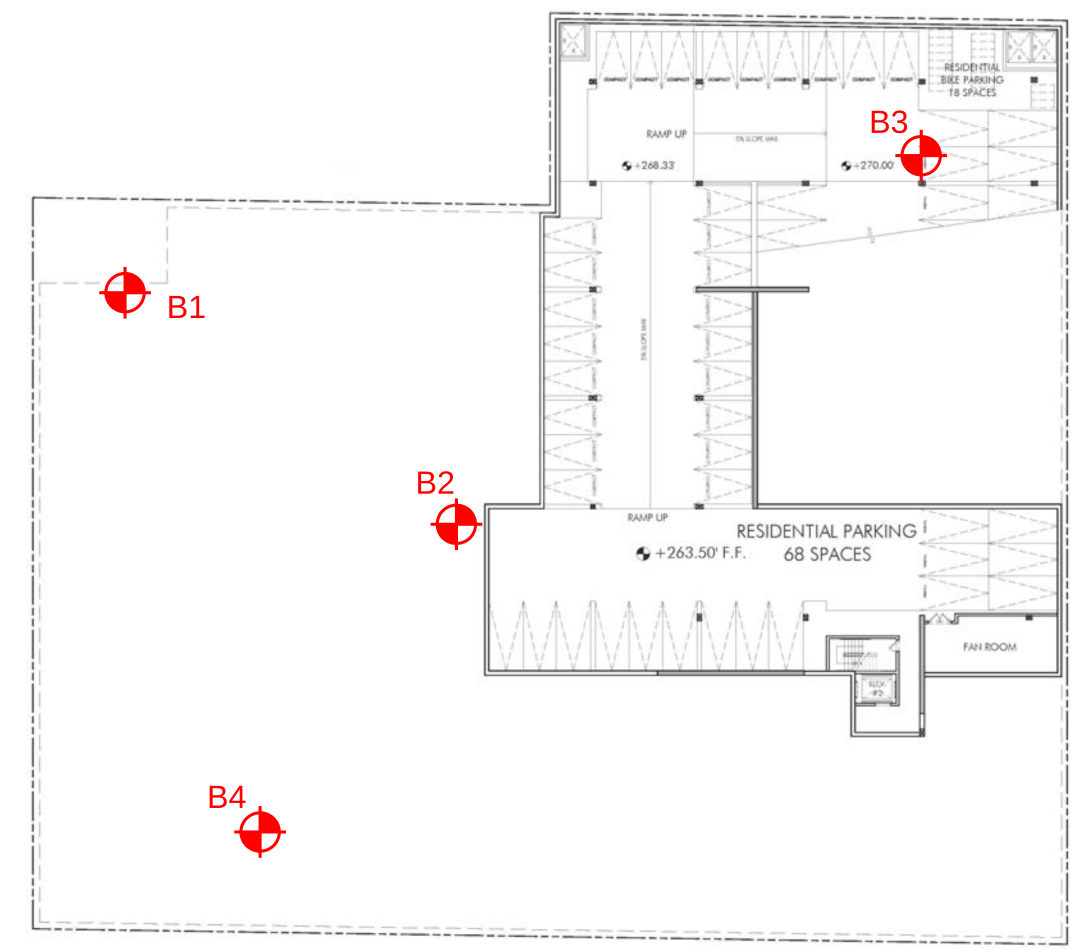
WEST HOLLYWOOD, CALIFORNIA

CHL		8000	June 1, 2012	PROJECT NO. A8936-06-01	FIG. 1

PLAN BY: Studio One Eleven



SUBTERRANEAN LEVEL P1



SUBTERRANEAN LEVEL P2

LEGEND

B4 Approximate Location of Boring (Geotechnologies,Inc. 2005)

Approximate Limits of Subterranean Level P2

Approximate Location of Offsite Structures

0 50' 100'

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SITE PLAN

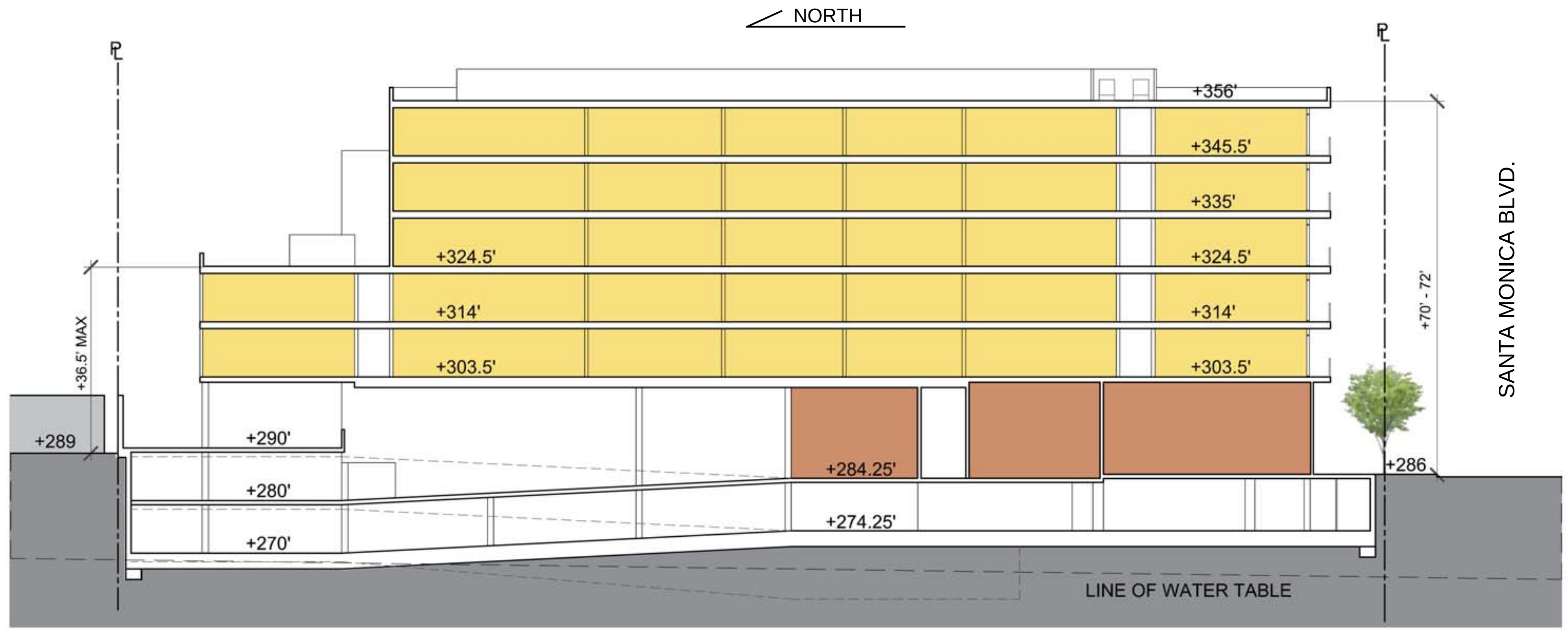
GLJ PARTNERS

7141-7155 SANTA MONICA BOULEVARD

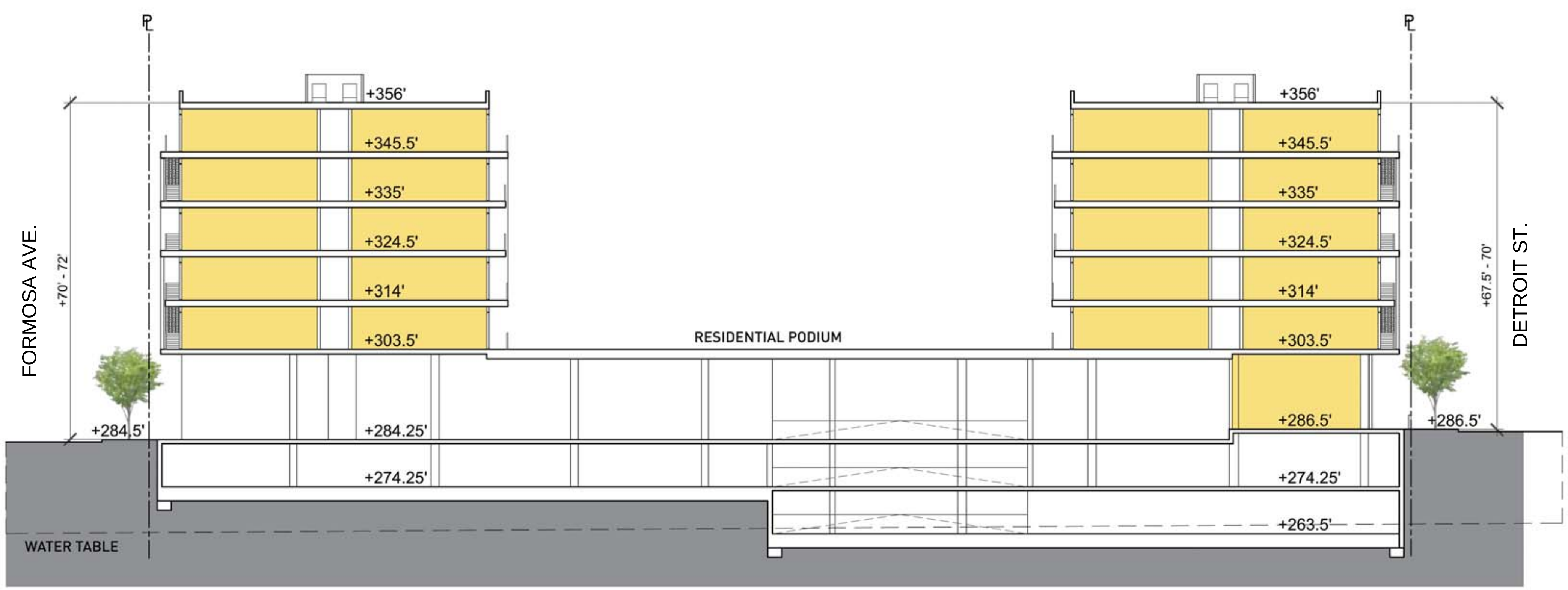
WEST HOLLYWOOD, CALIFORNIA

June 1, 2012	PROJECT NO. A8936-06-01	FIG. 2

PLAN BY: Studio One Eleven



SECTION A-A'



SECTION B-B'

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

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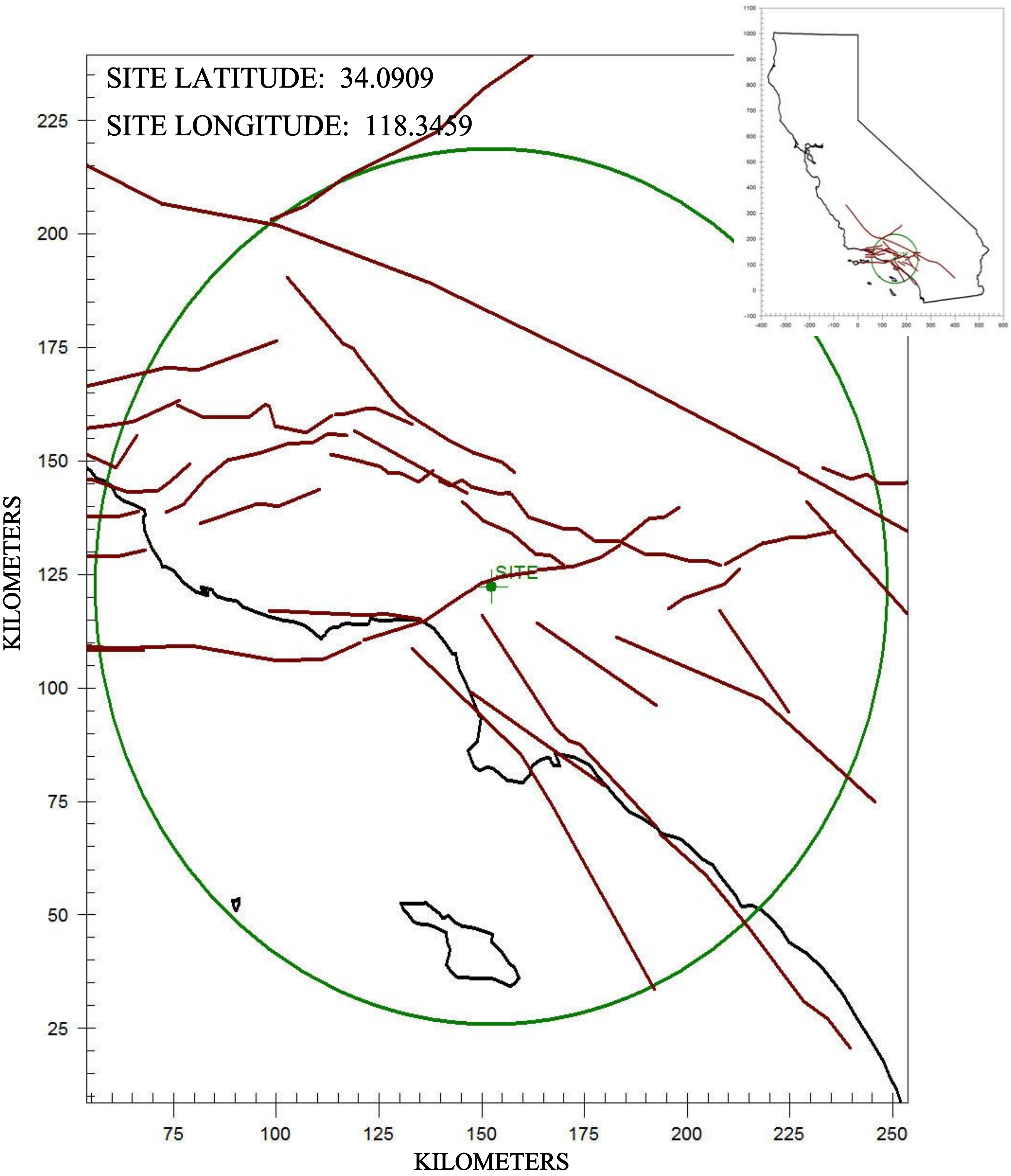
NDB		8000
CROSS-SECTIONS		
GLJ PARTNERS 7141-7155 SANTA MONICA BOULEVARD WEST HOLLYWOOD, CALIFORNIA		
June 1, 2012	PROJECT NO. A8936-06-01	FIG. 3



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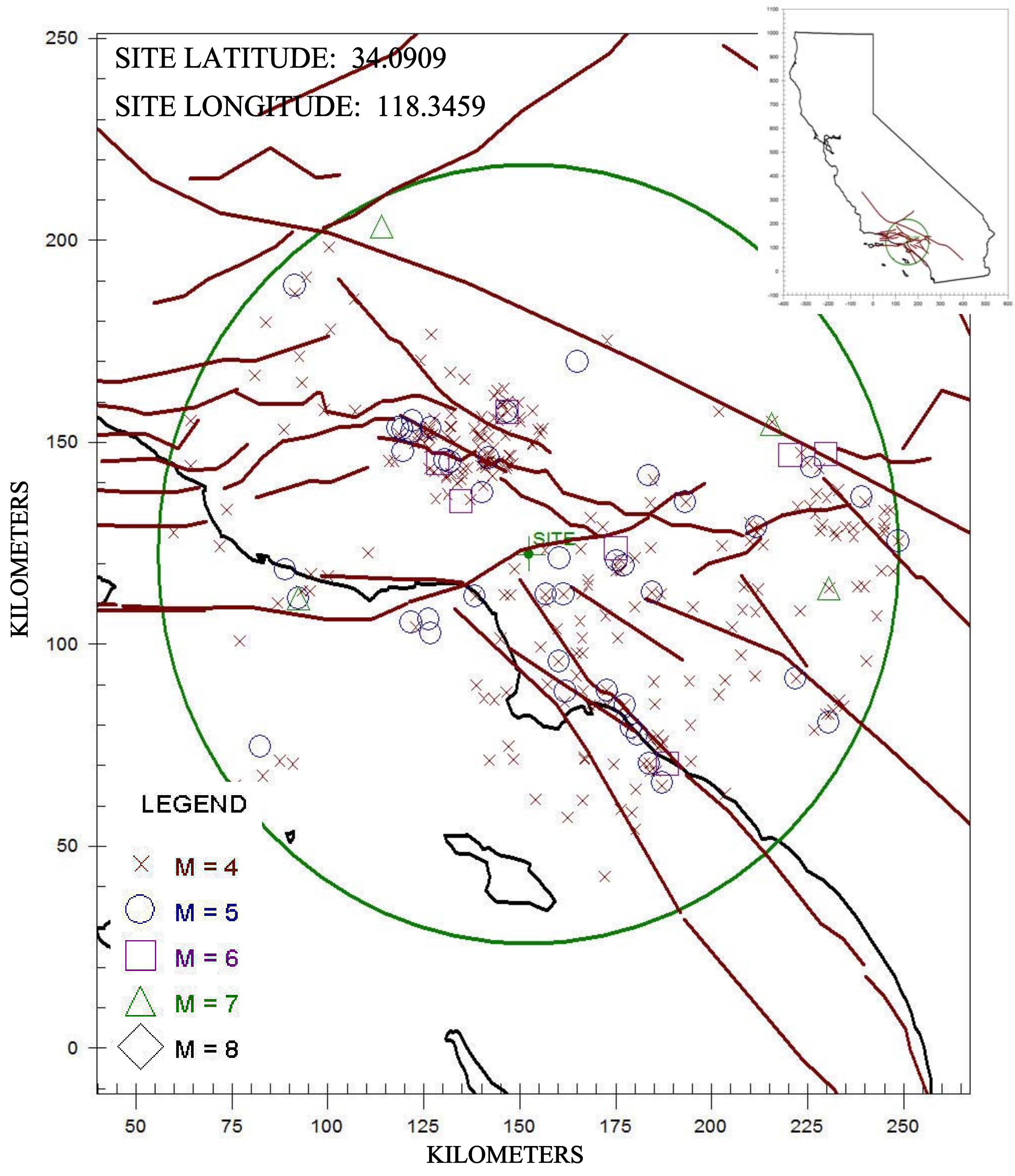
REGIONAL FAULT MAP

GLJ PARTNERS
7141-7155 SANTA MONICA BOULEVARD
WEST HOLLYWOOD, CALIFORNIA

June 1, 2012	PROJECT NO. A8936-06-01	FIG. 4



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REGIONAL SEISMICITY MAP

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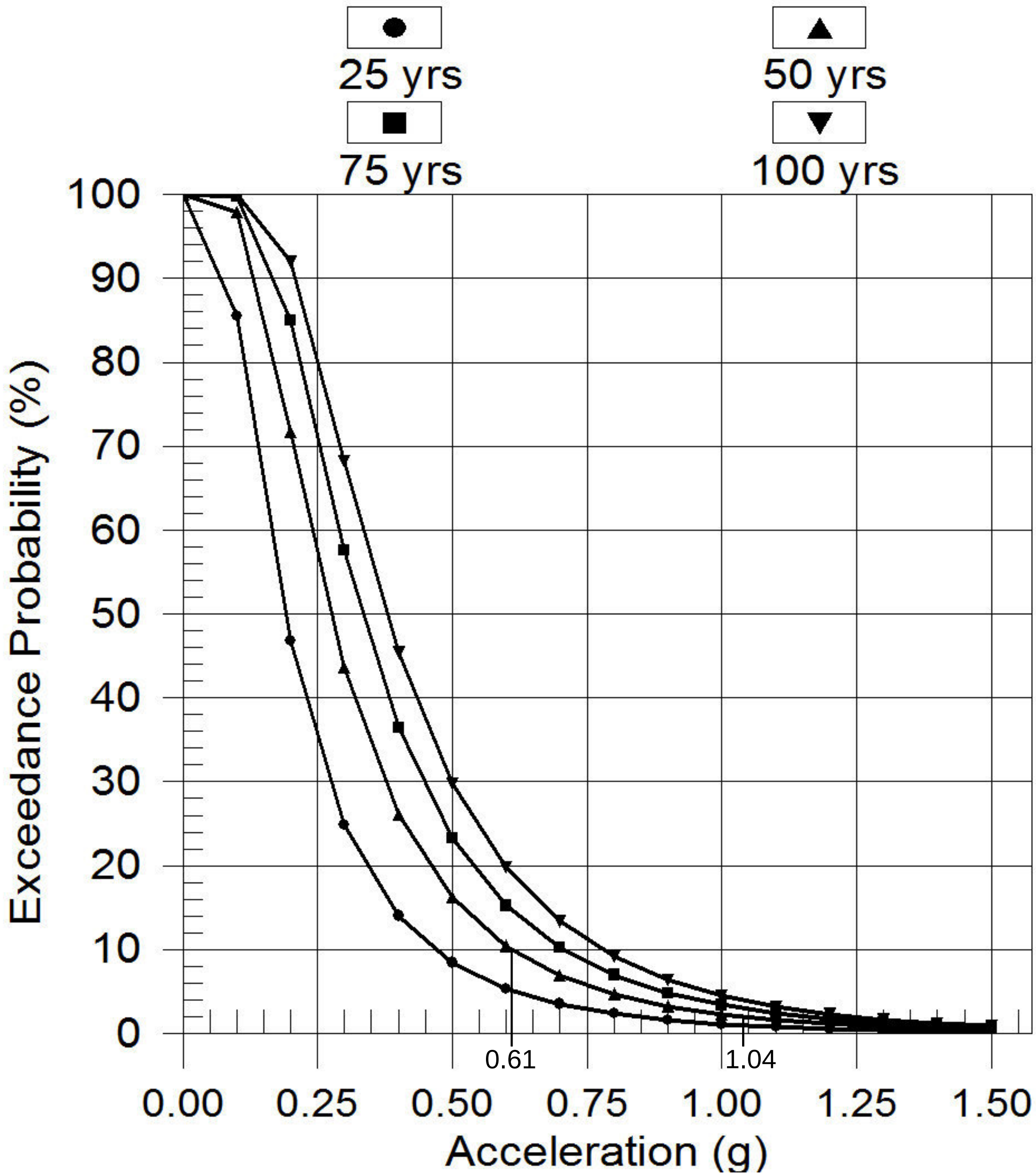
WEST HOLLYWOOD, CALIFORNIA

June 1, 2012	PROJECT NO. A8936-06-01	FIG. 5

PROBABILITY OF EXCEEDANCE

SADIGH ET AL. (1997) DEEP SOIL 1



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PROBABILITY OF EXCEEDANCE

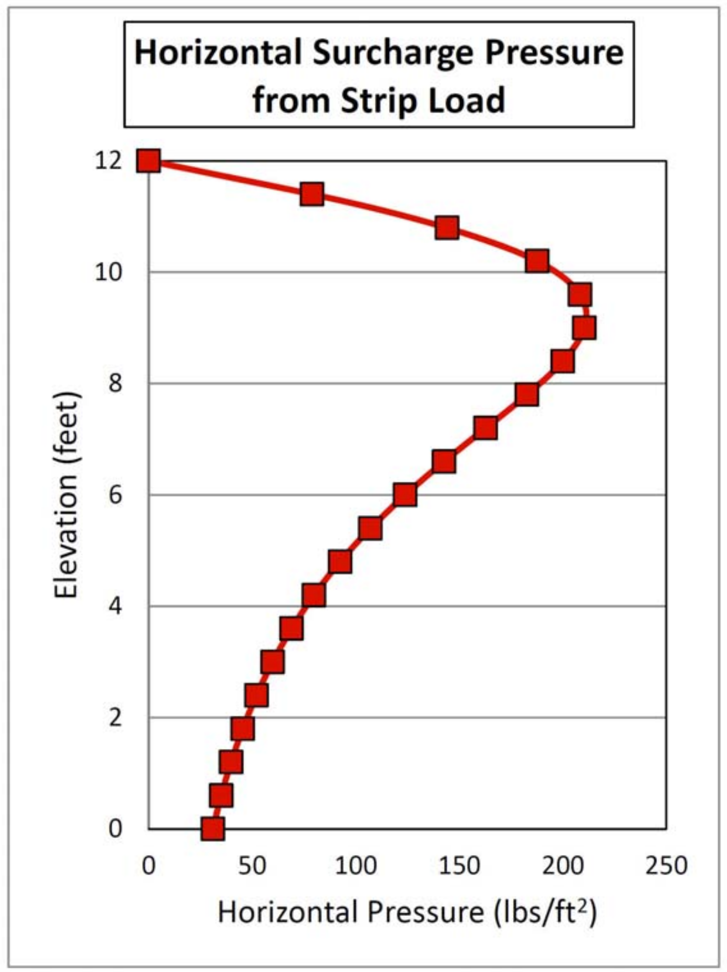
GLJ PARTNERS
7141-7155 SANTA MONICA BOULEVARD
WEST HOLLYWOOD, CALIFORNIA

CHL		8000	June 1, 2012	PROJECT NO. A8936-06-01	FIG. 6

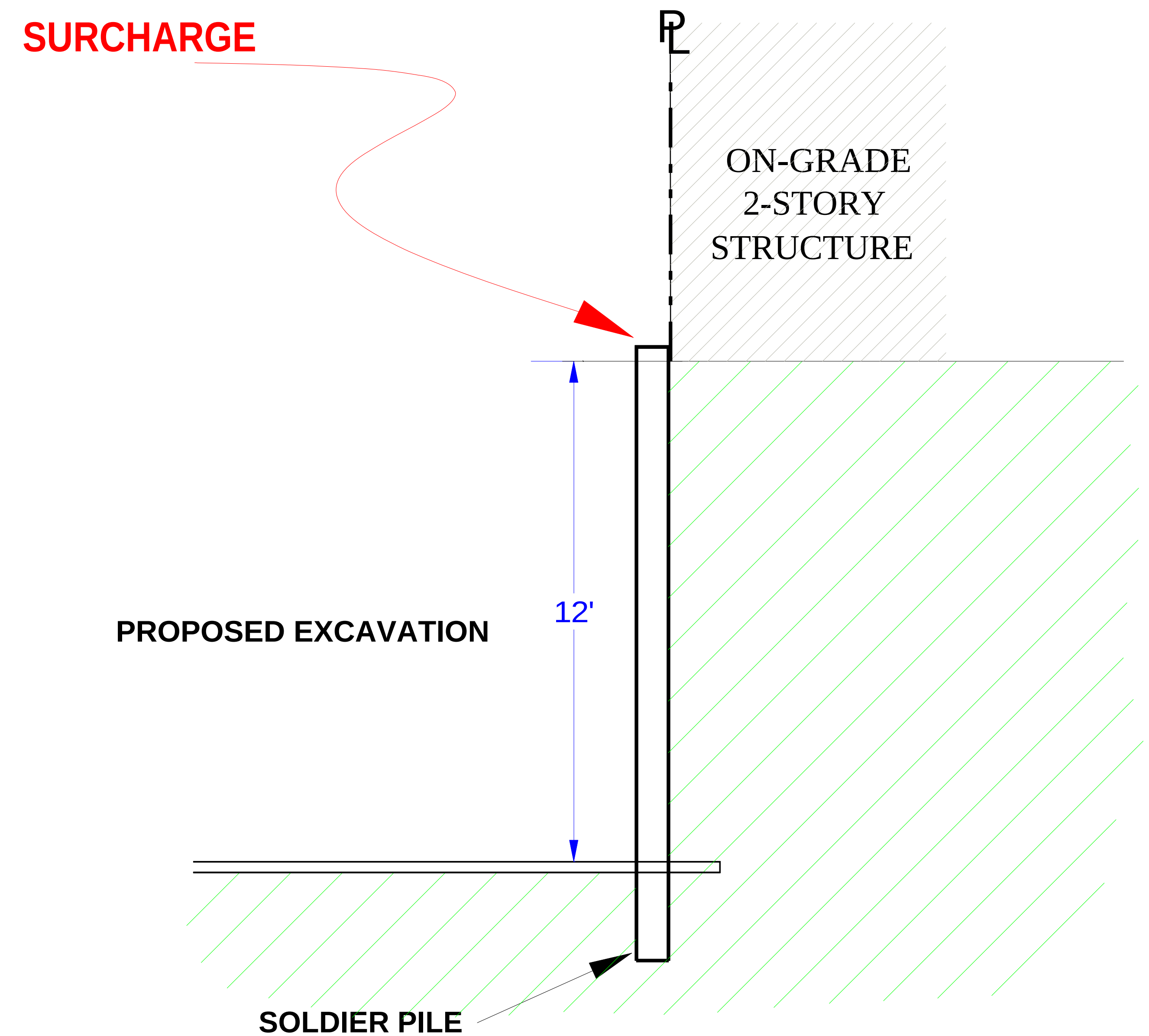
The load and property line setback of the adjacent structure was assumed.

Horizontal Surcharge Pressure from Strip Load
Stip Load Ql= 2500 lbs/lf
Height of Cut H= 12 ft
Distance Away X1= 0 ft
m = 0

Elevation (feet)	n-value	Horizontal Pressure (lbs/ft^2)
12	0	0.00
11.4	0.05	78.90
10.8	0.1	144.18
10.2	0.15	187.65
9.6	0.2	208.33
9	0.25	210.41
8.4	0.3	200.00
7.8	0.35	182.73
7.2	0.4	162.76
6.6	0.45	142.69
6	0.5	123.93
5.4	0.55	107.13
4.8	0.6	92.46
4.2	0.65	79.82
3.6	0.7	69.03
3	0.75	59.87
2.4	0.8	52.08
1.8	0.85	45.48
1.2	0.9	39.86
0.6	0.95	35.06
0	1	30.97



Maximum Pressure = 210.41 lbs/ft^2
Total Load per Lineal Foot of Wall = 1342.71 lbs/ft



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

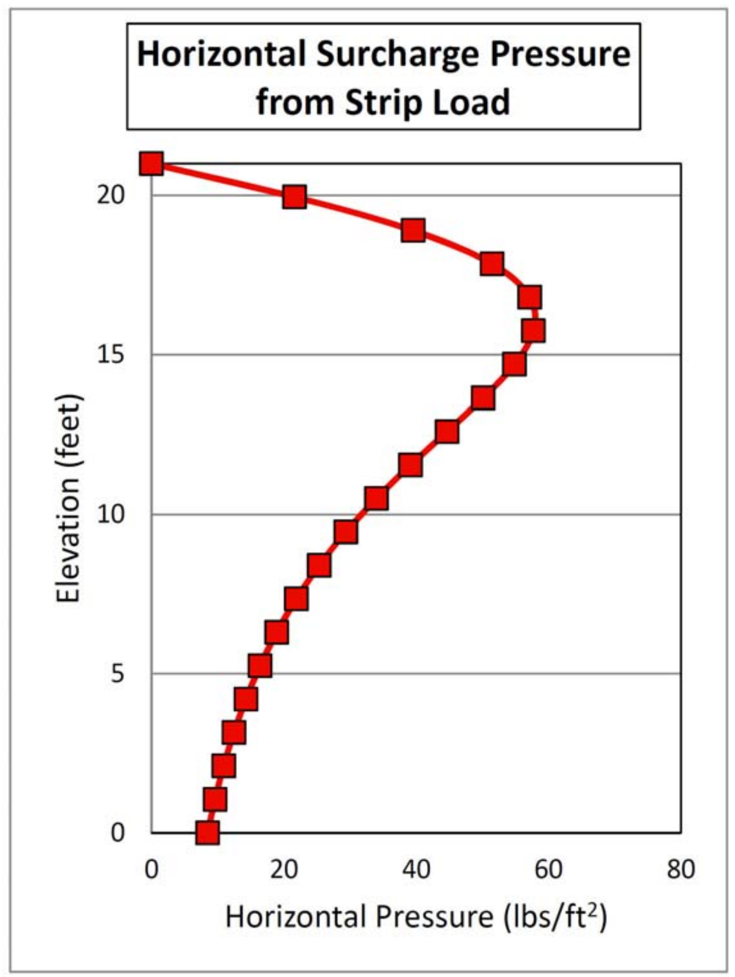
CROSS-SECTION/SURCHARGE CALCULATION		
GLJ PARTNERS 7141-7155 SANTA MONICA BOULEVARD WEST HOLLYWOOD, CALIFORNIA		
June 1, 2012	PROJECT NO. A8936-06-01	FIG. 7

The load and property line setback of the adjacent structure was assumed.

Horizontal Surcharge Pressure from Strip Load

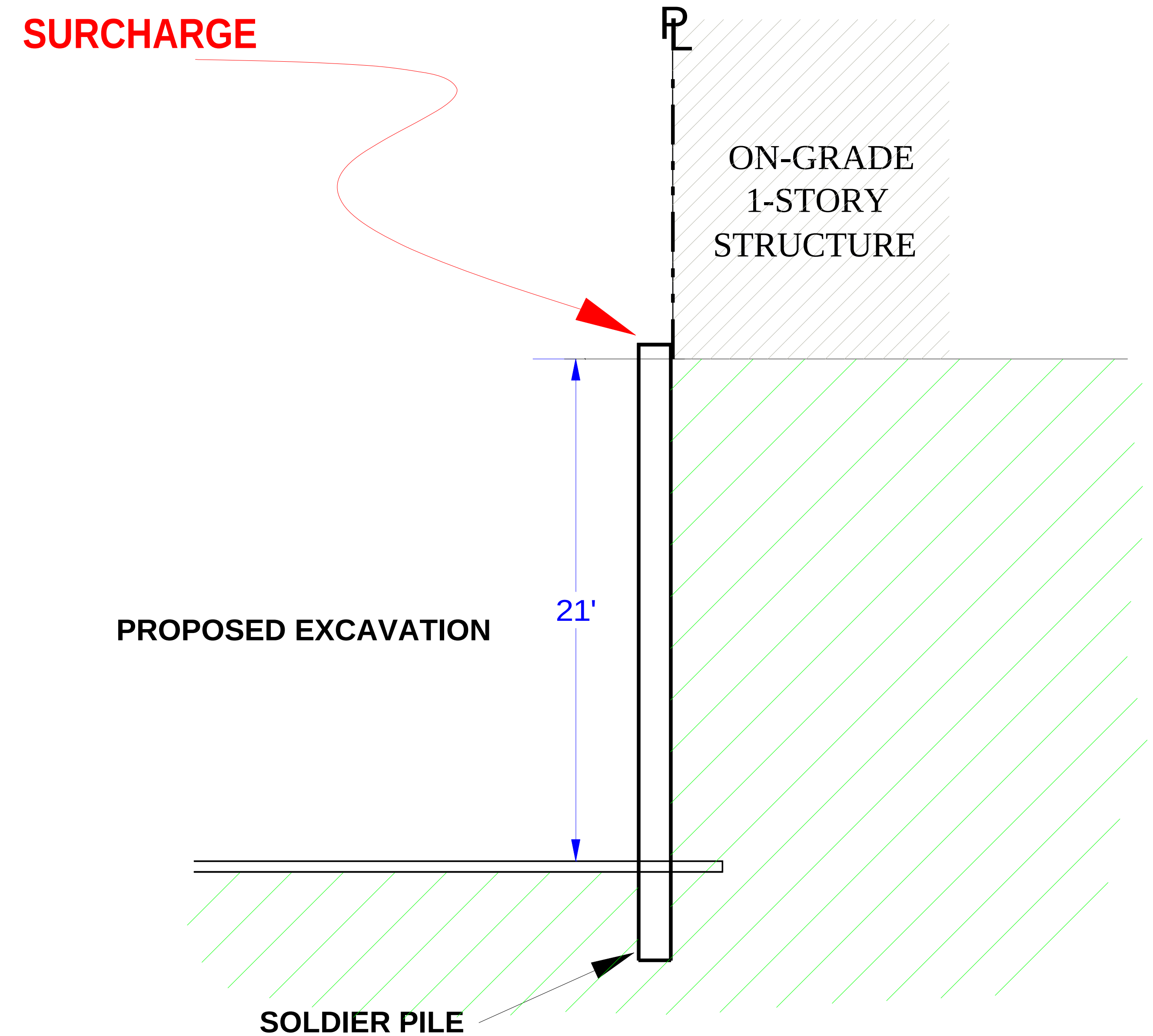
Stip Load Ql=	1200 lbs/lf
Height of Cut H=	21 ft
Distance Away X1=	0 ft
m =	0

Elevation (feet)	n-value	Horizontal Pressure (lbs/ft^2)
21	0	0.00
19.95	0.05	21.64
18.9	0.1	39.55
17.85	0.15	51.47
16.8	0.2	57.14
15.75	0.25	57.71
14.7	0.3	54.86
13.65	0.35	50.12
12.6	0.4	44.64
11.55	0.45	39.14
10.5	0.5	33.99
9.45	0.55	29.39
8.4	0.6	25.36
7.35	0.65	21.89
6.3	0.7	18.93
5.25	0.75	16.42
4.2	0.8	14.29
3.15	0.85	12.47
2.1	0.9	10.93
1.05	0.95	9.62
0	1	8.49



Maximum Pressure = 57.71 lbs/ft^2

Total Load per Lineal Foot of Wall = 644.50 lbs/ft



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CROSS-SECTION/SURCHARGE CALCULATION

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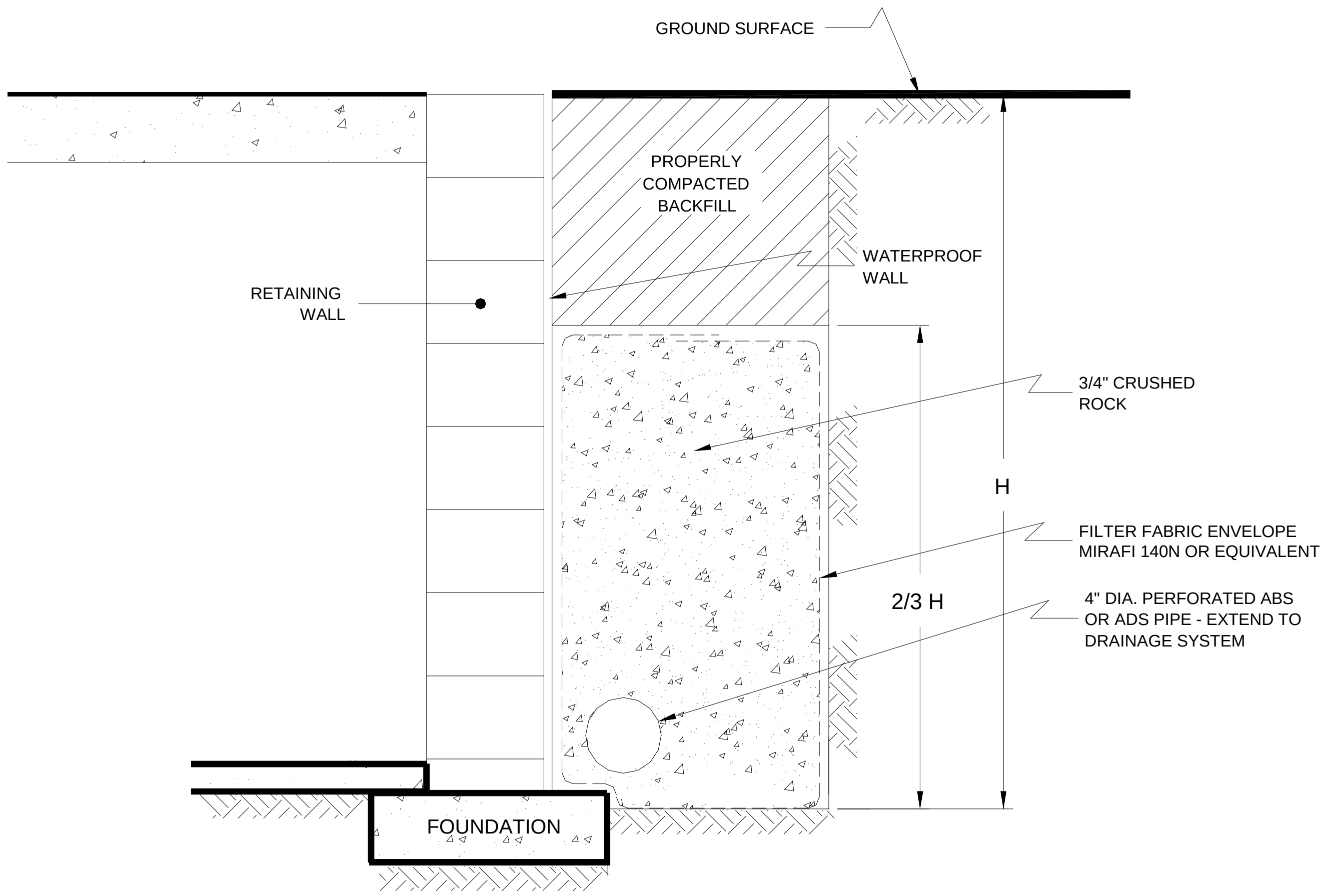
WEST HOLLYWOOD, CALIFORNIA

June 1, 2012 | PROJECT NO. A8936-06-01 | FIG. 8



NO SCALE

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RETAINING WALL DRAIN DETAIL

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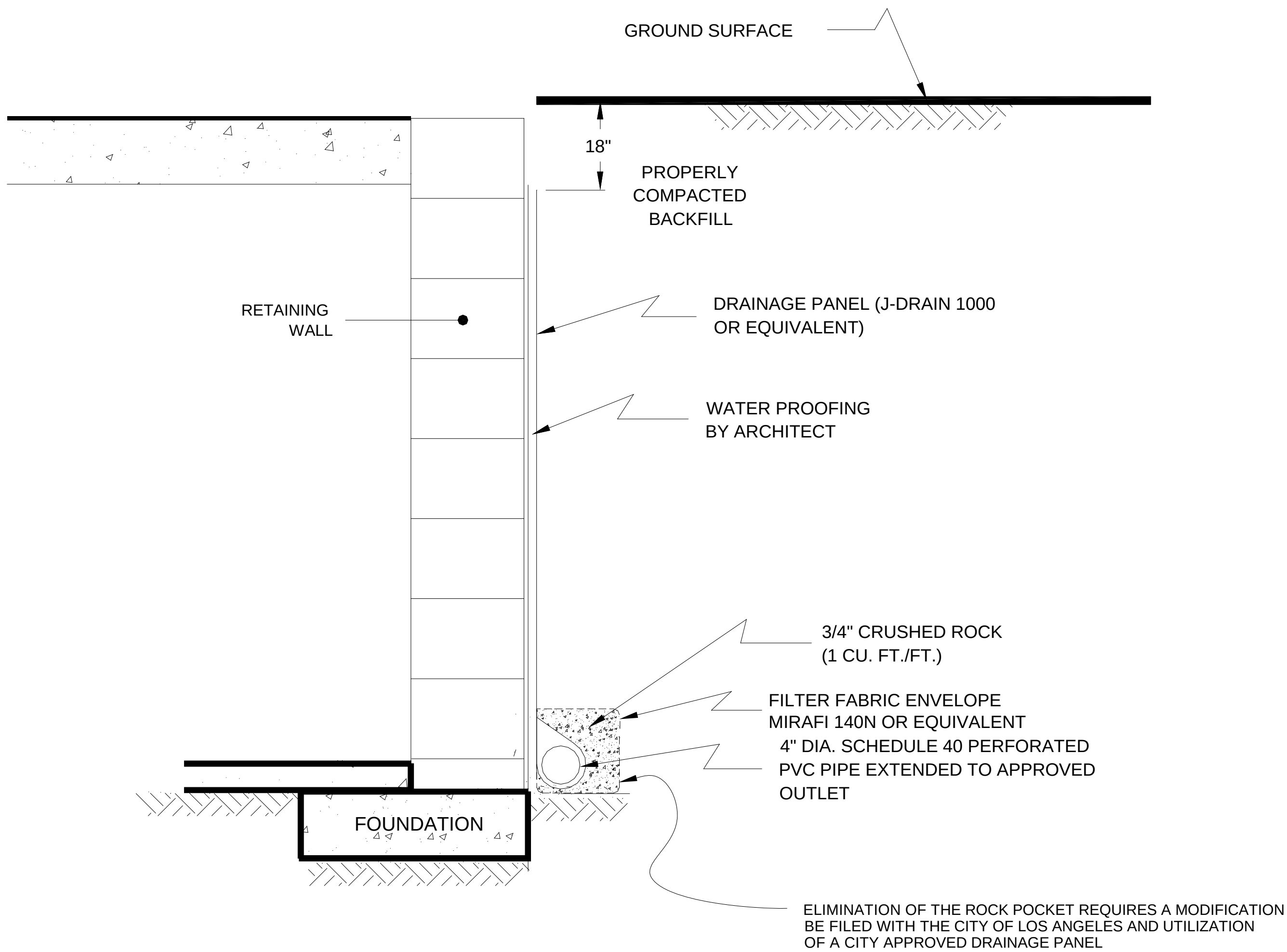
WEST HOLLYWOOD, CALIFORNIA

June 1, 2012	PROJECT NO. A8936-06-01	FIG. 9



NO SCALE

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RETAINING WALL DRAIN DETAIL

GLJ PARTNERS
7141-7155 SANTA MONICA BOULEVARD
WEST HOLLYWOOD, CALIFORNIA

NDB		8000	June 1, 2012	PROJECT NO. A8936-06-01	FIG. 10

GEOCON

TABLE 1
FAULTS WITHIN 60 MILES OF THE SITE
DETERMINISTIC SITE PARAMETERS

ABBREVIATED FAULT NAME	APPROXIMATE DISTANCE mi (km)	ESTIMATED MAX. EARTHQUAKE EVENT: MAXIMUM EARTHQUAKE MAG.(Mw)	PEAK SITE ACCEL. g	EST. SITE INTENSITY MOD.MERC.
HOLLYWOOD	0.9 (1.5)	6.4	1.021	XI
SANTA MONICA	3.7 (5.9)	6.6	0.747	XI
NEWPORT-INGLEWOOD (L.A.Basin)	4.2 (6.8)	6.9	0.563	X
RAYMOND	7.4 (11.9)	6.5	0.503	X
VERDUGO	7.7 (12.4)	6.7	0.513	X
COMPTON THRUST	9.6 (15.4)	6.8	0.452	X
ELYSIAN PARK THRUST	10.5 (16.9)	6.7	0.410	X
SIERRA MADRE	11.1 (17.8)	7.0	0.430	X
MALIBU COAST	11.4 (18.4)	6.7	0.383	X
NORTHRIDGE (E. Oak Ridge)	12.7 (20.5)	6.9	0.374	IX
SIERRA MADRE (San Fernando)	12.8 (20.6)	6.7	0.348	IX
PALOS VERDES	14.2 (22.9)	7.1	0.288	IX
SAN GABRIEL	16.2 (26.0)	7.0	0.246	IX
SANTA SUSANA	18.1 (29.2)	6.6	0.240	IX
WHITTIER	20.1 (32.4)	6.8	0.183	VIII
CLAMSHELL-SAWPIT	20.3 (32.6)	6.5	0.203	VIII
ANACAPA-DUME	21.3 (34.2)	7.3	0.289	IX
HOLSER	24.4 (39.2)	6.5	0.165	VIII
SAN JOSE	26.6 (42.8)	6.5	0.148	VIII
OAK RIDGE (Onshore)	29.3 (47.1)	6.9	0.164	VIII
SIMI-SANTA ROSA	29.3 (47.2)	6.7	0.147	VIII
CHINO-CENTRAL AVE. (Elsinore)	32.8 (52.8)	6.7	0.128	VIII
SAN CAYETANO	33.7 (54.2)	6.8	0.131	VIII
CUCAMONGA	34.2 (55.1)	7.0	0.144	VIII
SAN ANDREAS - 1857 Rupture	34.5 (55.5)	7.8	0.187	VIII
SAN ANDREAS - Mojave	34.5 (55.5)	7.1	0.119	VII
NEWPORT-INGLEWOOD (Offshore)	42.4 (68.3)	6.9	0.081	VII
SAN ANDREAS - Carrizo	43.1 (69.4)	7.2	0.098	VII
ELSINORE-GLEN IVY	43.7 (70.4)	6.8	0.073	VII
SANTA YNEZ (East)	46.3 (74.5)	7.0	0.077	VII
VENTURA - PITAS POINT	48.8 (78.5)	6.8	0.082	VII
SAN JACINTO-SAN BERNARDINO	49.1 (79.0)	6.7	0.059	VI
SAN ANDREAS - San Bernardino	49.2 (79.1)	7.3	0.089	VII
SAN ANDREAS - Southern	49.2 (79.1)	7.4	0.096	VII
OAK RIDGE(Blind Thrust Offshore)	51.3 (82.5)	6.9	0.081	VII
CLEGHORN	52.7 (84.8)	6.5	0.047	VI
CHANNEL IS. THRUST (Eastern)	52.9 (85.1)	7.4	0.113	VII
M.RIDGE-ARROYO PARIDA-SANTA ANA	53.7 (86.5)	6.7	0.067	VI
MONTALVO-OAK RIDGE TREND	54.4 (87.6)	6.6	0.062	VI
RED MOUNTAIN	57.6 (92.7)	6.8	0.065	VI
GARLOCK (West)	59.6 (95.9)	7.1	0.060	VI

41 FAULTS FOUND WITHIN THE SPECIFIED SEARCH RADIUS.
THE HOLLYWOOD FAULT IS CLOSEST TO THE SITE.
IT IS ABOUT 0.9 MILES (1.5 km) AWAY.
LARGEST MAXIMUM-EARTHQUAKE SITE ACCELERATION: 1.0207 g

APPENDIX A

PRIOR GEOTECHNICAL REPORT

Geotechnologies, Inc.
Consulting Geotechnical Engineers

439 Western Avenue
Glendale, California 91201-2837
818.240.9600 • Fax 818.240.9675

December 9, 2005
Revised April 11, 2008
File No. 19079

Hanover West, Inc.
333 North Glenoaks Boulevard, Suite 500
Burbank, California 91502

Attention: Daniel Hale

Subject: Geotechnical Engineering Investigation
Proposed Apartment Building
7141 - 7155 Santa Monica Boulevard, West Hollywood, California

Dear Mr. Hale:

This letter transmits the Geotechnical Engineering Investigation for the subject property prepared by Geotechnologies, Inc. This report provides geotechnical recommendations for the development of the site, including earthwork, seismic design, retaining walls, and foundations. Engineering for the proposed project should not begin until approval of the geotechnical investigation is granted by the local building official. Significant changes in the geotechnical recommendations may result due to the building department review process.

The validity of the recommendations presented herein is dependant upon review of the geotechnical aspects of the project during construction by this firm. The subsurface conditions described herein have been projected from limited subsurface exploration and laboratory testing. The exploration and testing presented in this report should in no way be construed to reflect any variations which may occur between the exploration locations or which may result from changes in subsurface conditions.

Should you have any questions, please feel free to contact the undersigned.

Respectfully submitted,
GEOTECHNOLOGIES, INC.

REGISTERED PROFESSIONAL ENGINEER
MICHAEL A. CAZENEUVE
No. 71490
Exp. 12/31/09
CIVIL
STATE OF CALIFORNIA

MICHAEL A. CAZENEUVE
R.C.E. 71490

MAC:km

Distribution: (7) Addressee

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GEOTECHNICAL ENGINEERING INVESTIGATION

PROPOSED APARTMENT BUILDING

7141 - 7155 SANTA MONICA BOULEVARD

WEST HOLLYWOOD, CALIFORNIA

INTRODUCTION

This report presents the results of the geotechnical engineering investigation performed on the subject property. The purpose of this investigation was to evaluate the nature of the soils underlying the site, to ascertain their engineering properties, and to provide recommendations for site preparation, grading, foundation design, retaining wall design, expansive soils, resistance to lateral loading, floor slabs, temporary excavations and shoring. In addition, a seismic hazard evaluation of the site was performed.

This investigation included excavating four exploratory borings, obtaining representative samples, laboratory testing, engineering analysis, review of available geotechnical engineering information, and the preparation of this report. The exploratory boring locations are shown on the enclosed Plot Plan. The results of the exploration and the laboratory tests are shown in the Appendix of this report.

PROPOSED DEVELOPMENT

Information concerning the proposed development was provided by the client, VCA Engineers, Studio One Eleven, and Earth Support Systems. The proposed project consists of the construction of a 3 to 6 story apartment building with a subterranean parking garage. The lowest finished floor elevation is proposed to vary between 271 and 273 feet. This corresponds to depths between 11 and 19 feet below the existing ground surface. Wall loads are expected to be between 8 and 10 kips per foot. Column loading is expected to be between 400 and 900 kips. Grading will consist of excavations up to approximately 23 feet for the proposed subterranean garage and perimeter footings.

Any changes in the design of the project or location of the proposed structure, as outlined in this report, should be reviewed by this office. The recommendations contained in this report should not be considered valid until reviewed and modified or reaffirmed, in writing, subsequent to such review.

SITE CONDITIONS

The subject site is located at 7141 - 7155 Santa Monica Boulevard, in the city of West Hollywood, California. The site of the proposed development is currently occupied by one to two story commercial buildings and at-grade parking. The subject site is bounded by Santa Monica Boulevard to the south, Formosa Avenue to the west, residential structures to the north, and Detroit Street to the east. The neighboring development consists of commercial and residential structures.

The site gently descends towards the southwest with a total topographic relief on the order of 5 feet. Site elevations vary from approximately 289 feet at the northeastern corner to approximately 284 feet at the southwestern corner. Vegetation on the site consists of some trees and shrubs located in isolated planters. Drainage on the site is by sheetflow along the existing contours to the city streets. The subject site and site elevations are shown relative to city streets and offsite improvements on the enclosed Plot Plan.

GEOTECHNICAL EXPLORATION

FIELD EXPLORATION

The subject site was explored on November 28 and 29, 2005, by drilling four exploratory borings to depths between 40 and 70 feet below the existing ground surface. The borings were drilled with the aid of a truck mounted, hollow-stem auger drilling machine, and were approximately 8 inches in diameter.

The boring locations are shown on the enclosed Plot Plan, and the soils encountered in the borings are logged on Plates A-1 through A-4. Undisturbed and bulk samples of the soils encountered in the borings were obtained and transported to the laboratory. The results of the laboratory tests are given in the Appendix.

EARTH MATERIALS

Fill materials were encountered during exploration to depths between 1 and 3 feet below the existing site grade. The fill materials consist of clay, silty clay, sandy clay and silty sand, which are brown to dark brown, moist, medium dense to stiff, fine to coarse grained, and contain some gravel.

The native soils underlying the site consist of various mixtures of silt, clay and sand, which range from medium brown to orange brown, and are slightly moist to wet, dense to very stiff and hard, fine to coarse grained, and contain some occasional gravel.

The native earth materials consist of older alluvial materials, typical to this area of Los Angeles County. More detailed soil profiles may be obtained from the individual boring logs.

GROUNDWATER

Groundwater was encountered during exploration by this firm at a depth of 21 feet below the existing site grade. This corresponds to groundwater surface elevations ranging from approximately 268 feet at the northeast corner of the site to 263 feet at the southwest corner of the site.

This office has reviewed the draft report by Professional Service Industries, Inc., titled, "Well Installation and Groundwater Sampling Report, 7141 and 7155 Santa Monica Boulevard and 1107-1117 Detroit Street," dated March 26, 2008. The report presents groundwater surface elevation contours across the subject site that are based on nine groundwater wells constructed on the site. Water surface elevations range from 267.5 feet at the northeast corner of the site to 262.5 feet at the southwest corner of the site. Groundwater surface elevations of 263 and 265 feet are reported at the northwest and southeast corners of the site, respectively.

Based on groundwater data provided in the Seismic Hazard Zone Report of the Hollywood 7½-Minute Quadrangle (CDMG Seismic Hazard Zone Report 026), the historic-high groundwater level for the site was 17 feet below the ground surface (elevation 268 feet) at the southeast corner of the site. The historic high water level increased to 20 feet below the ground surface at the southwest, northwest, and northeast corners of the site (historic water surface elevations of 264, 266, and 269 feet, respectively). A copy of the Historically Highest Groundwater Levels Map is provided in the Appendix.

Fluctuations in the level of groundwater may occur due to variations in rainfall, temperature, and other factors not evident at the time of the measurements reported herein. Fluctuations also may occur across the site. Higher groundwater levels can result in changed conditions.

Drilling Conditions

The borings were excavated with the aid of a hollow-stem auger drill rig, in which the boring is essentially cased by the augers, and caving is not possible. Unusually difficult drilling conditions were not encountered.

REGIONAL GEOLOGY AND SEISMICITY

REGIONAL SETTING

The subject property is located in the Los Angeles Basin and within the northern portion of the Peninsular Ranges Geomorphic Province. The Peninsular Ranges are characterized by northwest-trending blocks of mountain ridges and sediment-floored valleys. The dominant geologic structural features are northwest trending fault zones that either die out to the northwest or terminate at east-west trending reverse faults that form the southern margin of the Transverse Ranges.

The Los Angeles Basin is located at the northern end of the Peninsular Ranges Geomorphic Province. The basin is bounded by the east and southeast by the Santa Ana Mountains and San Joaquin Hills, to the northwest by the Santa Monica Mountains. Over 22 million years ago the Los Angeles basin was a deep marine basin formed by tectonic forces between the North American and Pacific plates. Since that time, over 5 miles of marine and non-marine sedimentary rock as well as intrusive and extrusive igneous rocks have filled the basin. During the last 2 million years, defined by the Pleistocene and Holocene epochs, the Los Angeles basin and surrounding mountain ranges have been uplifted to form the present day landscape. Erosion of the surrounding mountains, has resulted in deposition of unconsolidated sediments in low-lying areas by rivers such as the Los Angeles River. Areas that have experienced subtle uplift have been eroded with gullies.

The site is underlain by older alluvial sediments deposited by river and stream action, that are likely deeper than 150 feet.

REGIONAL FAULTING

Based on criteria established by the California Division of Mines and Geology (CDMG) now called California Geologic Survey (CGS), faults may be categorized as active, potentially active, or inactive. Active faults are those which show evidence of surface displacement within the last 11,000 years (Holocene-age). Potentially-active faults are those that show evidence of most recent surface displacement within the last 1.6 million years (Quaternary-age). Faults showing no evidence of surface displacement within the last 1.6 million years are considered inactive for most purposes, with the exception of design of some critical structures.

Buried thrust faults are faults without a surface expression but are a significant source of seismic activity. They are typically broadly defined based on the analysis of seismic wave recordings of hundreds of small and large earthquakes in the southern California area. Due to the buried nature of these thrust faults, their existence is usually not known until they produce an earthquake. The risk for surface rupture potential of these buried thrust faults is inferred to be low (Leighton, 1990). However, the seismic risk of these buried structures in terms of recurrence and maximum potential magnitude, is not well established. Therefore, the potential for surface rupture on these surface-verging splays at magnitudes higher than 6.0 cannot be precluded.

Using the computer program EQFAULT (2000) significant faults within a 60 mile radius of the site and their distance to the site is presented in Table I in the Appendix. The program EQFAULT, measures the shortest distance to faults in a three dimensional system. Some of the attenuation relationships utilized in the program returns a distance of 0.0 miles where the depth to a dipping fault plane is less than 10 km. For depths greater than 10 km, these attenuation relationships cause the program to return the inferred depth to the fault plane minus 10 km.

HISTORIC SEISMICITY

The epicenters of earthquakes with magnitudes of 5.0 or greater, and located within a radius of 60 miles of the site are listed on Table II, Historical Earthquake Epicenters, in the Appendix. The location of the earthquake epicenters is shown on Figure II, Earthquake Epicenters Map. Other pertinent information regarding these earthquakes is also provided on Table II.

SEISMIC HAZARDS

The primary geologic hazard at the site is moderate to strong ground motion (acceleration) caused by an earthquake on any of the local or regional faults. The potential for other earthquake-induced hazards was also evaluated including surface rupture, liquefaction, dynamic settlement, inundation and landsliding.

Ground Motion

The seismic exposure of the site may be investigated in two ways. The deterministic method calculates an estimated maximum earthquake magnitude for a fault based on formulas which correlate the fault trace to the theoretical maximum magnitude earthquake. The probabilistic method considers the probability of exceedance of various levels of ground motion (acceleration) and is calculated by consideration of risk contributions from all possible earthquake scenarios on all faults within a prescribed search radius. The CGS database of faults and historical earthquakes is used for both methods.

Deterministic Method

The deterministic method is used to predict a unique outcome for a given earthquake scenario. All known faults within the defined search radius are assigned an estimated maximum earthquake magnitude based on their length. Then, the resulting ground acceleration that the earthquake is capable of producing is calculated based on an appropriate attenuation relationship. The selected ground motion is simply the highest attenuated ground motion.

Table I in the Appendix shows known faults within a 60-mile radius of the site based on the current understanding of regional seismo-tectonics. For this investigation, the attenuation relationship of Bozorgnia Campbell Niazi (1999) Hor.-Pleist. Soil-Uncor. was selected. The resulting peak site accelerations at the site from the maximum-earthquake for each fault are shown on Table I.

Using this methodology, the maximum earthquake resulting in the largest estimated maximum earthquake acceleration at the site would be a magnitude 7.1 event on the Puente Hills Blind Thrust Fault. Such an event would be expected to generate peak horizontal accelerations at the site of 1.09g.

Probabilistic Method

The probabilistic method uses earthquake activity rates, maximum earthquake magnitude distributions, and other parameters for all faults that have an effect on the earthquake hazard for the site. For each identified fault, the range of potential earthquake magnitudes and activity rates are assigned probabilities of occurrence. Then, a distance to each potential earthquake hypocenter is measured and the ground motion is reduced by the appropriate attenuation relationship. This exercise is repeated for each fault in the selected radius from the site, typically 60 miles.

Based on a summation of all the possible earthquake probabilities and their corresponding ground accelerations, a series of curves are developed that provide the probability of exceedance of various levels of ground motion. This type of model is commonly used throughout the world for seismic hazard analysis of important facilities.

Figure III in the Appendix indicates the return periods of various levels of mean peak horizontal acceleration. Typical earthquakes used for design are often taken as those with 2 percent and 10 percent probability of exceedance in a 50 year structural life.

The 10 percent probability earthquake has a return period of 475 years. The 2001 California Building Code (2001 CBC) defines this ground motion as the Design Basis Earthquake (DBE). The DBE ground motion is expressed as Peak Ground Acceleration (PGA_{DBE}). It is used as a basis for structural design in the 2001 CBC and as a design basis ground motion for analysis of liquefaction hazard in California.

The 2 percent probability earthquake has a return period of 2,475 years. The 2003 National Earthquake Hazards Reduction Program Provisions (2003 NEHRP) defines this ground motion as the Maximum Considered Earthquake (MCE). The 2003 NEHRP provides the basis for the seismic design of structures for the 2007 California Building Code (2007 CBC). The MCE ground motion is expressed as Peak Ground Acceleration (PGA_{MCE}). It is used as a basis for structural design in the 2007 CBC.

The enclosed probabilistic seismic hazard analysis was performed utilizing the computer program, FRISKSP V. 4.00, by Thomas F. Blake (2000). The attenuation relation of Bozorgnia Campbell Niazi (1999) Hor.-Pleist. Soil-Uncor was utilized in the analysis The results of the probabilistic seismic hazard analysis is presented in Figure IV. The results indicated a PGA_{DBE} for the site of 0.75g, and a MCE_{DBE} for the site of 1.22g.

The data used for performing the probabilistic seismic hazard analysis includes recorded and measured quantities such as slip-rate and fault rupture length. The analysis does not take into account the potential hazards from unknown buried thrust faults, many of which may still be identified.

OTHER SEISMIC EFFECTS

Surface Rupture

In 1972, the Alquist-Priolo Special Studies Zones Act (now known as the Alquist-Priolo Earthquake Fault Zoning Act) was passed into law. The Act defines "active" and "potentially active" faults utilizing the same aging criteria as that used by California Geological Survey (CGS). However, established state policy has been to zone only those faults which have direct evidence of movement within the last 11,000 years. It is this recency of fault movement that the CGS considers as a characteristic for faults that have a relatively high potential for ground rupture in the future.

CGS policy is to delineate a boundary from 200 to 500 feet wide on each side of the known fault trace based on the location precision, the complexity, or the regional significance of the fault. If a site lies within an Earthquake Fault Zone, a geologic fault rupture investigation must be performed that demonstrates that the proposed building site is not threatened by surface displacement from the fault before development permits may be issued.

Ground rupture is defined as surface displacement which occurs along the surface trace of the causative fault during an earthquake. Based on research of available literature and results of site reconnaissance, no known active or potentially active faults underlie the subject site. The subject site is not located within an Alquist-Priolo Earthquake Fault Zone. In addition, the subject site is not located within the Fault Precaution Zones, FP-1 or FP-2, as delineated in the Seismic Safety Element of the City of West Hollywood. Based on these considerations, the potential for surface ground rupture at the subject site is considered low.

Liquefaction

Liquefaction is a phenomenon in which saturated silty to cohesionless soils below the groundwater table are subject to a temporary loss of strength due to the buildup of excess pore pressure during cyclic loading conditions such as those induced by an earthquake. Liquefaction-related effects include loss of bearing strength, amplified ground oscillations, lateral spreading, and flow failures.

Liquefaction typically occurs in areas where groundwater is less than 50 feet from the surface, and where the soils are composed of poorly consolidated, fine to medium-grained sand. In addition to the necessary soil conditions, the ground acceleration and duration of the earthquake must also be of a sufficient level to initiate liquefaction.

The Seismic Hazards Maps of the State of California (CDMG, 1999), do not classify the site as part of a "Liquefiable" area. This determination is based on groundwater depth records, soil type and distance to a fault capable of producing a substantial earthquake. A copy of this map is provided in the Appendix.

The Seismic Safety Element of the City of West Hollywood indicates the site is not located in an area that is susceptible to liquefaction. However, in accordance with the City of West Hollywood requirements, the enclosed liquefaction analysis has been performed.

Groundwater was encountered during exploration at a depth of 21 feet below the existing site grade. According to the Seismic Hazard Zone Report, the highest historic groundwater level for the site was 17 feet below the ground surface. The enclosed liquefaction analysis was completed utilizing the high groundwater level of 17 feet below the existing site grade. Based on the deterministic and probabilistic seismic analyses provided herein, a Magnitude Scaling Factor (M_w) of 7.1 and a peak ground acceleration (PGA_{DBE}) of 0.75g were used in the analysis.

The enclosed liquefaction analysis of the soils underlying the site was performed using the spreadsheet template LIQ2_30.WQ1 developed by Thomas F. Blake (1996). This program utilizes the 1996 NCEER method of analysis. This semi-empirical method is based on a correlation between measured values of Standard Penetration Test (SPT) resistance and field performance data.

The enclosed "Empirical Estimation of Liquefaction Potential" is based on Boring B2. Standard Penetration Test (SPT) data were collected at 5-foot intervals. Due to the high corrected SPT blow counts, the liquefaction analysis indicates the site soils would not be prone to liquefaction.

Landsliding

The probability of seismically-induced landslides affecting the subject development is considered to be remote, due to the lack of significant slopes on the site and in surrounding areas.

Earthquake-Induced Flooding

The subject site is high enough and far enough from the ocean and any lakes to preclude potential flooding from a tsunami or seiche. Review of the County of Los Angeles Flood and Inundation Hazards Map, (Leighton ,1990), indicates that the site lies within the potential inundation zone of the Mulholland Dam. A determination of whether a higher site elevation would remove the site from the potential inundation zone is beyond the scope of this investigation.

Seismically-Induced Settlement (Dynamic Dry 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 relatively dense nature of the soils underlying the site, as evidenced by the very high blow counts for the soils between the bottom of the proposed structure and the groundwater level, the potential for dynamic settlement would be considered negligible.

CONCLUSIONS AND RECOMMENDATIONS

Based upon the exploration, laboratory testing, and research, it is the finding of this firm that construction of the proposed development is considered feasible from a geotechnical engineering standpoint, provided the advice and recommendations presented herein are followed and implemented during construction.

Existing fill was encountered on the site to depths between 1 and 3 feet below the existing site grade. It is anticipated that all of the existing fill materials will be removed during excavation of the proposed subterranean parking garage. Conventional spread footings bearing in the competent native soils found at the proposed subterranean garage level may be utilized for foundation support.

Foundations for miscellaneous small outlying structures, such as property line walls and trash enclosures, which will not be rigidly connected to the proposed structure may be supported on conventional foundations bearing in native soils, and/or newly placed compacted fill.

Due to the depth of the proposed excavations and proximity to the property lines, shoring will be required in all areas in order to allow for safe excavation of the proposed subterranean garage level.

Based on the exploration and research, groundwater occurs beneath the site at elevations ranging between approximately 262.5 and 268 feet, which is below the proposed building pad elevations that range between approximately 270 and 272 feet. However, it is anticipated that some foundation excavations will extend below the groundwater level. Temporary dewatering will be required for construction of these foundations, as discussed in the "Dewatering" section of this report.

Although the proposed pad elevations are above the groundwater level, the soils at the proposed subgrade level should be expected to be well above their optimum moisture level. These wet soils may be susceptible to disturbance from construction activities. It will most likely be necessary to protect or stabilize the subgrade soils with a gravel mat, as discussed in the "Dewatering" section of this report.

It is the understanding of this firm that the proposed basement walls will be designed without backdrainage systems to eliminate maintenance of the drainage systems and potential handling of water. The walls will be constructed above the groundwater level. However, in order to prevent hydrostatic build up from other sources such as irrigation and damaged utilities, the walls shall be designed to resist hydrostatic forces based on a water level at the ground surface. Hydrostatic forces on the basement walls are addressed in the "Retaining Wall Design" section of this report.

The validity of the conclusions and design recommendations presented herein is dependant upon review of the geotechnical aspects of the proposed construction by this firm. The subsurface conditions described herein have been projected from borings on the site as indicated and should in no way be construed to reflect any variations which may occur between these borings or which may result from changes in subsurface conditions.

2007 CBC SEISMIC DESIGN CONSIDERATIONS

In accordance with Section 1613.5.2 and Table 1613.5.2 of the 2007 CBC, the subject site is classified as Site Class D, which corresponds to a "Stiff Soil" Profile. The following table outlines the Mapped Spectral Accelerations and Site Coefficients per the 2007 CBC which may be used by the structural engineer for the seismic design and analysis of structures.

2007 CBC SEISMIC PARAMETERS	
Site Class	D
Mapped Spectral Acceleration at Short Periods (S_S)	1.704g
Site Coefficient (F_a)	1.0
Maximum Considered Earthquake Spectral Response for Short Periods (S_{MS})	1.704g
Five-Percent Damped Design Spectral Response Acceleration at Short Periods (S_{DS})	1.136g
Mapped Spectral Acceleration at One-Second Period (S_1)	0.600g
Site Coefficient (F_v)	1.5
Maximum Considered Earthquake Spectral Response for One-Second Period (S_{M1})	0.900g
Five-Percent Damped Design Spectral Response Acceleration for One-Second Period (S_{D1})	0.600g

DEWATERING

Temporary Dewatering

Foundation excavations in the northeastern section of the building are expected to extend into the groundwater. Other foundation excavations may or may not encounter groundwater. Water should be removed from the footing excavations prior to placement of steel and concrete. It is anticipated the water could be mitigated on an "as-encountered" basis through the use of pumping equipment and/or small channels at the base of the excavations leading to temporary sump pits. However, it is recommended a qualified dewatering consultant be retained during the design stage of the project to provide recommendations regarding the handling of groundwater during construction.

Wet Subgrade Soils

Soils at the proposed subgrade level should be expected to be well above their optimum moisture level. At this time, pumping, rutting, and disturbance of the high-moisture content soils should be expected to occur during operation of heavy equipment. A representative of this office should observe the subgrade as it becomes exposed so that the recommendations provided herein may be revised or reaffirmed as necessary. In order to minimize disturbance of the subgrade soils, provide a firm working surface, and provide a subgrade suitable for support of the proposed floor slab , it is recommended the subgrade be protected and/or stabilized as it becomes exposed.

Protection or stabilization of the subgrade may be accomplished by placement of a an approximate 1 to 1½ foot thick layer of angular ¾-inch gravel. Depending on the condition of the exposed subgrade, a thicker mat of gravel may be required if pumping of the subgrade continues. The elevation at the bottom of excavation will require adjustment to provide space for the gravel mat. The gravel should be placed and vibrated to a dense state as the subgrade becomes exposed. It is not recommended that rubber tire construction equipment attempt to operate directly on the subgrade soils prior to placing the gravel. Direct operation of rubber tire equipment on soft subgrade soils will likely result in excessive disturbance to the soils, which in turn could result in a delay to the construction schedule. Extreme care should be utilized to place gravel as the subgrade becomes exposed.

EXPANSIVE SOILS

Test performed on a representative sample of the onsite soils showed the soils to be in the low expansion range, with an Expansion Index of 45. Reinforcing recommendations are provided in the "Foundation Design" and "Slabs on Grade" sections of this report.

HYDROCONSOLIDATION POTENTIAL

Hydroconsolidation is a phenomena in which the underlying soils collapse when wetted. Hydroconsolidation could potentially result in significant foundation movements, over a long period of time of wetting.

The underlying native soils were observed to be dense to stiff. Samples of the native soils exhibited minor hydroconsolidation strains ranging between 0 and 0.2 percent during consolidation testing. Settlements resulting from the indicated hydroconsolidation strains are expected to be negligible.

It shall be the responsibility of the property owner to maintain proper drainage of the subject site throughout the life of the structure. All utility and irrigation lines, and drainage devices should be checked periodically and maintained. In addition, landscape irrigation should be properly controlled, in order to prevent saturation of the underlying soils, which provide support to the proposed structure. The "Site Drainage" section of this report should be followed and implemented into the final construction documents.

SOIL CORROSIVITY

Soil corrosivity testing was performed on three representative samples of the onsite soils by the laboratory of Schiff Associates. The reader is referred to the attached report by their office for complete results, discussion of results and recommended mitigating measures.

Briefly, the results of the corrosivity testing indicate that the electrical resistivities of the soils are in the mildly and moderately corrosive categories at field moisture conditions, and in the corrosive to severely corrosive categories when saturated. Soil pH values of the samples range between 7.2 and 7.4, indicating neutral to slightly alkaline conditions. The soluble salt content was low to moderate. Ammonium and nitrate were detected at levels high enough to be deleterious to copper. The soil is classified as severely corrosive to ferrous metals, aggressive to copper, and negligible for sulfate attack on concrete.

GRADING GUIDELINES

The following guidelines may be used in preparation of the grading plan and job specifications for any areas where fill or recompaction may be required.

Site Preparation

All vegetation, existing fill, and soft or disturbed earth materials should be removed from the areas to receive controlled fill. The excavated areas shall be observed by the geotechnical engineer prior to placing compacted fill.

Any vegetation or associated root system located within the area to be graded should be removed. Any existing or abandoned utilities located within the area to be graded should be removed or relocated as appropriate. All fill materials and disturbed earth materials resulting from grading operations should be removed and properly recompacted.

It is very important that the limits of the area to be graded are accurately located so that the grading operation proceeds efficiently.

Subsequent to the indicated removals, the exposed grade shall be scarified to a depth of six inches, moistened to optimum moisture content, and recompacted in excess of the minimum required comparative density.

Compaction

Fill, consisting of soil approved by a representative of this firm shall be placed in loose lifts not more than 8 inches in thickness. The loose materials shall be compacted with suitable compaction equipment. Once a layer has been adequately compacted, the next loose lift may be placed.

Fill materials shall be moisture conditioned to within 3 percent of optimum moisture content and sufficiently blended prior to placement as controlled fill. Materials larger than 6 inches in maximum dimension shall not be used in the fill.

All fill shall be compacted to at least 90 percent of the maximum laboratory density. The maximum density shall be determined by the laboratory operated by Geotechnologies, Inc., using test method ASTM D 1557-02 or equivalent.

Field observation and testing shall be performed by a representative of the geotechnical engineer during grading to assist the contractor in obtaining the required degree of compaction and the proper moisture content. Where compaction is less than required, additional compactive effort shall be made with adjustment of the moisture content, as necessary, until a minimum of 90 percent compaction is obtained.

Acceptable Materials

The excavated onsite materials are considered satisfactory for reuse in the controlled fills as long as any debris and/or organic matter is removed.

Any imported materials shall be observed and tested by the representative of the geotechnical engineer prior to use in fill areas. Imported materials should contain sufficient fines so as to be relatively impermeable and result in a stable subgrade when compacted. Any required import materials should consist of granular soils with an expansion index of less than 50. The water-soluble sulfate content of the import materials should be less than 0.10 percentage by weight.

Imported materials should be free from chemical or organic substances which could affect the proposed development. A competent professional should be retained in order to test imported materials and address environmental issues and organic substances which might affect the proposed development.

Utility Trench Backfill

Utility trenches should be backfilled with controlled fill. The utility should be bedded with clean sands at least one foot over the crown. The remainder of the backfill may be onsite soil compacted to 90 percent of the laboratory maximum density. Utility trench backfill should be tested by representatives of this firm in accordance with ASTM D-1556-00 or ASTM D-2922-96.

Geotechnical Observations and Testing During Grading

Geotechnical observations and testing during grading are considered to be a continuation of the geotechnical investigation. It is critical that the geotechnical aspects of the project be reviewed by this firm during the construction process. Compliance with the design concepts, specifications or recommendations during construction requires review by this firm during the course of construction. Any fill which is placed should be observed, tested, and verified if used for engineered purposes. Please advise this office at least twenty-four hours prior to any required site visit.

FOUNDATION DESIGN

It is recommended that the proposed structure be supported on a system of conventional spread foundations bearing exclusively in competent native soils present at the proposed subterranean garage level. Wall foundations may be designed for an allowable bearing value of 5,000 pounds per square foot, and should be a minimum of 24 inches in width, 18 inches in depth below the lowest adjacent grade and 18 inches into the recommended bearing material. Column foundations may be designed for an allowable bearing value of 6,000 pounds per square foot, and should be a minimum of 24 inches in width, 18 inches in depth below the lowest adjacent grade and 18 inches into the recommended bearing material. A factor of safety of 3 was utilized in the calculations.

A subgrade modulus of 175 pounds per cubic inch may be utilized for the design of larger combined and shear wall footings.

Conventional foundations for miscellaneous small structures not rigidly connected to the proposed structure may bear in native soils, and/or newly placed compacted fill. These footings may be designed for a bearing value of 1,000 pounds per square foot, and should be a minimum of 12 inches in width, 18 inches in depth below the lowest adjacent grade and 18 inches into the recommended bearing material. No bearing value increases are recommended. The applicability of these recommendations for miscellaneous structures should be confirmed as detailed plans become available.

The bearing values indicated above are for the total of dead and frequently applied live loads, and may be increased by one third for short duration loading, which includes the effects of wind or seismic forces.

Since the recommended bearing value is a net value, the weight of concrete in the foundations may be taken as 50 pounds per cubic foot and the weight of the soil backfill may be neglected when determining the downward load on the foundations.

All continuous foundations should be reinforced with a minimum of four #4 steel bars. Two should be placed near the top of the foundation, and two should be placed near the bottom.

Lateral Design

Resistance to lateral loading may be provided by friction acting at the base of foundations and by passive earth pressure. An allowable coefficient of friction of 0.33 may be used with the dead load forces. Passive earth pressure for the sides of foundations poured against undisturbed or recompacted soil may be computed as an equivalent fluid having a density of 200 pounds per cubic foot with a maximum earth pressure of 2,000 pounds per square foot. When combining passive and friction for lateral resistance, the passive component should be reduced by one third. The above values are allowable values, with a factor of safety of 1.5.

Foundation Settlement

Settlement of the foundation system is expected to occur on initial application of loading. The maximum settlement is expected to be 3/4 inch, and will occur below the heaviest loaded columns. Differential settlement is not expected to exceed 1/4 inch.

Foundation Observations

It is critical that all foundation excavations are observed by a representative of this firm to verify penetration into the recommended bearing materials. The observation should be performed prior to the placement of reinforcement. Foundations should be deepened to extend into satisfactory earth materials, if necessary.

Foundation excavations should be cleaned of all loose soils prior to placing steel and concrete. Any required foundation backfill should be mechanically compacted, flooding is not permitted.

RETAINING WALL DESIGN

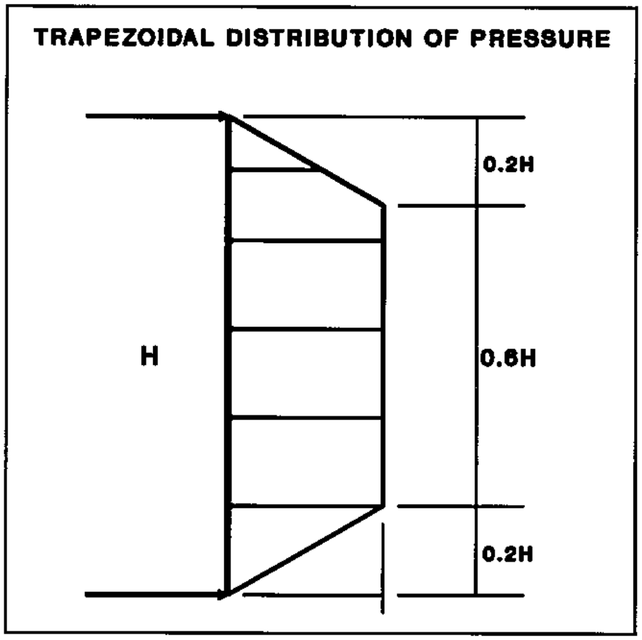
Retaining walls up to approximately 20 feet in height will be required for the proposed subterranean levels. It is anticipated these walls will be restrained. The basement retaining walls will be designed to resist hydrostatic forces in lieu of installation of back-drainage systems. The following restrained wall design criteria includes the full hydrostatic design.

In addition to the wall pressure recommended below, the upper ten feet of the retaining wall adjacent to streets, driveways or parking areas should be designed to resist a uniform lateral pressure of 100 pounds per square foot, acting as a result of an assumed 300 pounds per square foot surcharge behind the walls due to normal street traffic. If the traffic is kept back at least ten feet from the retaining walls, the traffic surcharge may be neglected.

Additional active pressure should be added for any additional surcharge conditions, such as sloping ground, or adjacent traffic and structures. Foundations may be designed in accordance with the "Foundation Design" section above.

Restrained Retaining Walls

Restrained retaining walls may be designed to resist a trapezoidal pressure distribution of at rest pressure as indicated in the diagram below.



Design restrained walls as follows:

HEIGHT OF WALL "H" (feet)	DESIGN WALL FOR (Where H is the height of the wall) Includes Hydrostatic Pressure
Up to 20 feet	62H psf

Dynamic (Seismic) Lateral Forces

Retaining walls exceeding 12 feet in height shall be designed to resist the additional earth pressure caused by seismic ground shaking. An inverse triangular pressure distribution should be utilized for seismic loads, with an equivalent fluid pressure of 32 pounds per cubic foot. Utilizing this inverse triangular pressure distribution, the earthquake load would be zero at the base of the wall, and would increase linearly to a maximum of 32(H) pounds per square foot at the top of the wall, where H is the height of the retaining wall.

Waterproofing

Moisture affecting retaining walls is one of the most common post-construction complaints. Poorly applied or omitted waterproofing can lead to efflorescence or standing water inside the building. Efflorescence is a process in which a powdery substance is produced on the surface of the concrete by the evaporation of water. The white powder usually consists of soluble salts such as gypsum, calcite, or common salt. Efflorescence is common to retaining walls and does not affect their strength or integrity.

It is recommended that retaining walls be waterproofed. Waterproofing design and inspection of its installation is not the responsibility of the geotechnical engineer. A waterproofing consultant should be retained in order to recommend a product or method which would provide protection to below grade walls.

Retaining Wall Drainage

Since the proposed basement retaining walls will be designed to resist hydrostatic forces, back drains may be omitted from the design.

Retaining Wall Backfill

Any required backfill should be mechanically compacted in layers not more than 8 inches thick, to at least 90 percent of the maximum density obtainable by the ASTM Designation D 1557-02 method of compaction. Flooding should not be permitted. Proper compaction of the backfill will be necessary to reduce settlement of overlying walks and paving. Some settlement of required backfill should be anticipated, and any utilities supported therein should be designed to accept differential settlement, particularly at the points of entry to a structure.

TEMPORARY EXCAVATIONS

Excavations up to a maximum of approximately 23 feet in height will be required for construction of the proposed basement and foundation elements. The excavations are expected to expose fill and native soils, which are suitable for vertical excavations up to 5 feet where not surcharged by adjacent traffic or structures. Where there is sufficient space, unsurcharged excavations over 5 feet in height may be made at a uniform 1:1 gradient up to a maximum height of 15 feet. Where there is insufficient space to excavate at a uniform 1:1 gradient, or where the excavations will be surcharged by adjacent traffic or structures, the excavations should be shored. Based on our current understanding of the proposed development, excavations around the proposed building perimeter will require shoring.

The tops of any temporary unshored excavations should be barricaded to prevent vehicles and storage loads within a 1:1 line projected upward from the bottom of the excavation, or a minimum of 5 feet, whichever is greater. If the temporary construction embankments, including shored excavations, are to be maintained during the rainy season, berms are suggested along the tops of the excavations where necessary to prevent runoff from entering the excavation and eroding the slope faces. The soils exposed in the excavations should be inspected during excavation by personnel from this office so that modifications can be made if variations in the soil conditions occur. All unshored excavations should be stabilized within 30 days of initial excavation.

Shoring

The following information on the design and installation of shoring is as complete as possible at this time. It is suggested that a review of the final shoring plans and specifications be made by this office prior to bidding or negotiating with a shoring contractor.

The recommended method of shoring consists of steel soldier piles, placed in drilled holes and backfilled with concrete. The soldier piles may be designed either as cantilevers, or may be laterally braced utilizing drilled tieback anchors or raker braces.

Drilled cast-in-place soldier piles should be placed no closer than 2½ diameters on center. The minimum diameter of the piles is 18 inches. Depending on the design, structural concrete may be used below the excavation, and lean-mix concrete may be employed above that level. As an alternative, lean-mix concrete may be used throughout the pile where the designer utilizes only the width of the steel beam flange in the determination of passive earth pressure resistance. The slurry must have sufficient strength to impart the lateral bearing pressure developed by the flange to the soil. For design purposes, an allowable passive value for the earth materials below the bottom plane of excavation may be assumed to be 400 pounds per square foot per foot of depth, up to a maximum of 4,000 pounds per square foot. To develop the full lateral value, provisions should be implemented to assure firm contact between the soldier piles and the undisturbed soils.

The frictional resistance between the soldier piles and retained soil may be used to resist the vertical component of the anchor load. The coefficient of friction may be taken as 0.33 based on uniform contact between the steel beam, lean-mix concrete and retained earth.

The portion of soldier piles below the plane of excavation may also be employed to resist the downward loads. The downward capacity may be determined using a frictional resistance of 500 pounds per square foot, for that portion of the pile embedded in the undisturbed native earth materials. The minimum depth of embedment for shoring piles is 5 feet below the bottom of the footing excavation or 7 feet below the bottom of excavated plane whichever is deeper.

Soldier Pile Installation

Casing or polymer drilling fluid may be required should caving be experienced in the saturated earth materials. If casing is used, extreme care should be employed so that the pile is not pulled apart as the casing is withdrawn. At no time should the distance between the surface of the concrete and the bottom of the casing be less than 5 feet.

Groundwater was encountered during exploration at a depth of 21 feet below the existing site grade. Depending on the length of the proposed piles, it is anticipated that the piles will likely encounter water. Piles placed below the water level will require the use of a tremie to place the concrete into the bottom of the hole. A tremie shall consist of a water-tight tube having a diameter of not less than 6 inches with a hopper at the top. The tube shall be equipped with a device that will close the discharge end and prevent water from entering the tube while it is being charged with concrete. The tremie shall be supported so as to permit free movement of the discharge end over the entire top surface of the work and to permit rapid lowering when necessary to retard or stop the flow of concrete. The discharge end shall be closed at the start of the work to prevent water entering the tube and shall be entirely sealed at all times, except when the concrete is being placed. The tremie tube shall be kept full of concrete. The flow shall be continuous until the work is completed and the resulting concrete seal shall be monolithic and homogeneous. The tip of the tremie tube shall always be kept about five feet below the surface of the concrete and definite steps and safeguards should be taken to insure that the tip of the tremie tube is never raised above the surface of the concrete.

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

Lagging

If the clear spacing between soldier piles does not exceed approximately 4 feet, lagging between soldier piles could possibly be omitted within the cohesive soils. In the less cohesive soils, lagging would be necessary. At this time, it is anticipated the entire excavation will require lagging. It is recommended that the exposed soils be observed by a representative of the soils engineer to verify the cohesive nature of the soils and the areas where lagging may be omitted.

Soldier piles and anchors should be designed for the full anticipated pressures. Due to arching in the soils, the pressure on the lagging will be less. It is recommended that the lagging be designed for the full design pressure but be limited to a maximum of 400 pounds per square foot.

Lateral Pressures

A triangular distribution of lateral earth pressure should be utilized for the design of cantilever shoring. A trapezoidal distribution of lateral earth pressure (as diagramed in the "Retaining Wall

Design" section of this report) would be appropriate where shoring is to be restrained at the top by tie backs. Pressures for the design of cantilevered and restrained shoring are presented in the following table.

Height of Shoring (feet)	Cantilever Shoring System Equivalent Fluid Pressure (pcf) Triangular Distribution of Pressure	Restrained Shoring System Lateral Earth Pressure (psf)* Trapezoidal Distribution of Pressure
Up to 23 feet	30 pcf	22H psf

Additional pressure should be added for a surcharge condition due to sloping ground, vehicular traffic, or adjacent structures.

Tieback Anchor Design and Installation

Tie-back anchors may be used to resist lateral loads. Friction anchors are recommended. For design purposes, it may be assumed that the active wedge adjacent to the shoring is defined by a plane drawn 35 degrees with the vertical through the bottom plane of the excavation. Friction anchors should extend a minimum of 20 feet beyond the potentially active wedge, and to greater lengths if necessary to develop the desired capacities.

Tieback anchors may be installed between 20 and 40 degrees below the horizontal. Caving may occur within granular materials. Where caving occurs the following provisions should be implemented in order to minimize such caving. The anchor shafts should be filled with concrete by pumping from the tip out, and the concrete should extend from the tip of the anchor to the active wedge. In order to minimize the chances of caving, it is recommended that the portion of the anchor shaft within the active wedge be backfilled with sand before testing the anchor. This portion of the shaft should be filled tightly and flush with the face of the excavation. The sand backfill should be placed by pumping. The sand may contain a small amount of cement to facilitate pumping.

The capacities of the anchors should be determined by testing of the initial anchors as outlined in a following section. Drilled friction anchors constructed without utilizing post-grouting techniques may be designed for a skin friction of 600 pounds per square foot. Depending on the techniques utilized, and the experience of the contractor performing the installation, it is anticipated that a skin friction of 2,500 pounds per square foot could be utilized for post-grouted anchors, provided the system does not rely on end-bearing plates to develop the necessary resistance. Only the frictional resistance developed beyond the active wedge should be utilized in resisting lateral loads. Anchors should be placed at least 6 feet on center to be considered isolated.

Tieback Anchor Testing

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

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

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

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

After a satisfactory test, each anchor should be locked-off at the design load. This should be verified by rechecking the load in the anchor. The load should be within 10 percent of the design load. Where satisfactory tests are not attained, the anchor diameter and/or length should be increased or additional anchors installed until satisfactory test results are obtained. Where post-grouted anchors are utilized, additional post-grouting may be required. The installation and testing of the anchors should be observed by a representative of the soils engineer.

Deflection

It is difficult to accurately predict the amount of deflection of a shored embankment. It should be realized that some deflection will occur. It is recommended that the shoring be designed for maximum deflection of ½ inch at the top of the shored embankment. If greater deflection occurs during construction, additional bracing may be necessary to minimize earth movement in adjacent areas.

Monitoring

Some means of monitoring the performance of the shoring system is suggested. The monitoring should consist of periodic surveying of the lateral and vertical locations of the tops of all soldier piles, and the lateral movement along the entire lengths of selected soldier piles. Also, some means of periodically checking the load on selected anchors will be necessary, where applicable.

Some movement of the shored embankments should be anticipated. It is recommended that photographs of the existing buildings and other improvements on the adjacent properties be made before and during construction to record any movements for use in the event of a dispute.

SLABS ON GRADE

Concrete Slabs-on Grade

Interior building floor slabs-on-grade should be a minimum of 5 inches in thickness. Outdoor concrete flatwork should be a minimum of 4 inches in thickness. Concrete slabs should be cast over undisturbed natural soils and/or properly controlled fill materials. Any earth materials loosened or over-excavated should be wasted from the site or properly compacted to at least 90 percent of the maximum dry density.

Exterior concrete paving subject to vehicular traffic shall be a minimum of 6 inches in thickness and underlain by 4 inches of aggregate base. A subgrade modulus of 150 pounds per cubic inch may be assumed for design of concrete paving.

Aggregate base should be compacted to a minimum of 95 percent of the ASTM D 1557-02 laboratory maximum dry density. Base materials should conform with Sections 200-2.2 or 200-2.4 of the "Standard Specifications for Public Works Construction", (Green Book), 1991 Edition.

Slab Reinforcing

All concrete floor slabs should be reinforced with a minimum of #3 steel bars on 18-inch centers each way.

Design Of Slabs That Receive Moisture-Sensitive Floor Coverings

In any areas where dampness would be objectionable, it is recommended that the floor slab should be supported on a vapor retarder. The design of the slab and the installation of the vapor retarder should comply with ASTM E 1643-98. Where a vapor retarder is used, a low-slump concrete should be used to minimize possible curling of the slabs. The barrier should be covered with a thin layer of sand, to prevent punctures and aid in the concrete cure.

Concrete Crack Control

The recommendations presented in this report are intended to reduce the potential for cracking of concrete slabs-on-grade due to settlement. However even where these recommendations have been implemented, foundations, stucco walls and concrete slabs-on-grade may display some cracking due to minor soil movement and/or concrete shrinkage. The occurrence of concrete cracking may be reduced and/or controlled by limiting the slump of the concrete used, proper concrete placement and curing, and by placement of crack control joints at reasonable intervals, in particular, where re-entrant slab corners occur.

For standard crack control maximum expansion joint spacing of 12 feet should not be exceeded. Lesser spacings would provide greater crack control. Joints at curves and angle points are recommended. The crack control joints should be installed as soon as practical following concrete placement. Crack control joints should extend a minimum depth of one-fourth the slab thickness. Construction joints should be designed by a structural engineer.

Complete removal of the existing fill soils beneath outdoor flatwork and exterior concrete pavement is not required. However, due to the rigid nature of concrete, some cracking, a shorter design life and increased maintenance costs should be anticipated. In order to provide uniform support beneath the flatwork it is recommended that a minimum of 12 inches of the exposed subgrade beneath the flatwork be scarified and recompacted to 90 or 95 percent of the maximum density.

SITE DRAINAGE

Proper surface drainage is critical to the future performance of the project. Saturation of a soil can cause it to lose internal shear strength and increase its compressibility, resulting in a change in the designed engineering properties. Proper site drainage should be maintained at all times.

All site drainage should be collected and transferred to the street in non-erosive drainage devices. The proposed structure should be provided with roof drainage. Discharge from downspouts, roof drains and scuppers should not be permitted on unprotected soils within five feet of the building perimeter. Drainage should not be allowed to pond anywhere on the site, and especially not against any foundation or retaining wall. Drainage should not be allowed to flow uncontrolled over any descending slope. Planters which are located within retaining wall backfill should be sealed to prevent moisture intrusion into the backfill.

DESIGN REVIEW

Engineering of the proposed project should not begin until approval of the geotechnical report by the Building Official is obtained in writing. Significant changes in the geotechnical recommendations may result during the building department review process.

It is recommended that the geotechnical aspects of the project be reviewed by this firm during the design process. This review provides assistance to the design team by providing specific recommendations for particular cases, as well as review of the proposed construction to evaluate whether the intent of the recommendations presented herein are satisfied.

CONSTRUCTION MONITORING

Geotechncial observations and testing during construction is considered to be a continuation of the geotechnical investigation. Therefore, it is critical that the geotechnical aspects of the project be reviewed by this firm during the construction process. Compliance with the design concepts, specifications or recommendations during construction requires review by this firm during the course of construction. All foundations should be observed by a representative of this firm prior to placing concrete or steel. Any fill which is placed should be observed, tested, and verified if used for engineered purposes. Please advise this office at least twenty-four hours prior to any required site visit.

If conditions encountered during construction appear to differ from those disclosed herein, notify this office immediately so the need for modifications may be considered in a timely manner.

It is the responsibility of the contractor to ensure that all excavations and trenches are properly sloped or shored. All temporary excavations should be cut and maintained in accordance with applicable OSHA rules and regulations.

CLOSURE AND LIMITATIONS

The purpose of this report is to aid in the design and completion of the described project. Implementation of the advice presented in this report is intended to reduce certain risks associated with construction projects. The professional opinions and geotechnical advice contained in this report are sought because of special skill in engineering and geology and were prepared in accordance with generally accepted geotechnical engineering practice. Geotechnologies, Inc. has a duty to exercise the ordinary skill and competence of members of the engineering profession. Those who hire Geotechnologies, Inc. are not justified in expecting infallibility, but can expect reasonable professional care and competence.

The scope of the geotechnical services provided did not include any environmental site assessment for the presence or absence of organic substances, hazardous/toxic materials in the soil, surface water, groundwater, or atmosphere, or the presence of wetlands.

Proper compaction is necessary to reduce settlement of overlying improvements. Some settlement of compacted fill should be anticipated. Any utilities supported therein should be designed to accept differential settlement. Differential settlement should also be considered at the points of entry to the structure.

GEOTECHNICAL TESTING

Classification and Sampling

The soil is continuously logged by a representative of this firm and classified by visual examination in accordance with the Unified Soil Classification system. The field classification is verified in the laboratory, also in accordance with the Unified Soil Classification System. Laboratory classification may include visual examination, Atterberg Limit Tests and grain size distribution. The final classification is shown on the boring logs.

Samples of the earth materials encountered during exploration were collected and transported to the laboratory. Undisturbed samples of soil are obtained at frequent intervals. Unless noted on the boring logs as an SPT sample, samples acquired while utilizing a hollow-stem auger drill rig are obtained by driving a thin-walled, Modified California Sampler with successive 30-inch drops of a 140-pound hammer. The soil is retained in brass rings of 2.50 inches inside diameter and 1.00 inches in height. The central portion of the samples are stored in close fitting, waterproof containers for transportation to the laboratory. Samples noted on the boring logs as SPT samples are obtained in accordance with ASTM (D-1586-99). Samples are retained for 30 days after the date of the geotechnical report.

Moisture and Density Relationships

The field moisture content and dry unit weight are determined for each of the undisturbed soil samples, and the moisture content is determined for SPT samples by ASTM (D-4959-03) or ASTM (D-4643-00). This information is useful in providing a gross picture of the soil consistency between borings and any local variations. The dry unit weight is determined in pounds per cubic foot and shown on the "Boring Logs," A-Plates. The field moisture content is determined as a percentage of the dry unit weight.

Direct Shear Testing

Shear tests are performed by ASTM (D-3080-04) with a strain controlled, direct shear machine manufactured by GeoMatic. The rate of deformation is approximately 0.005 inches per minute. Each sample is sheared under varying confining pressures in order to determine the Mohr-Coulomb shear strength parameters of the cohesion intercept and the angle of internal friction. Samples are generally tested in an artificially saturated condition. Depending upon the sample location and future site conditions, samples may be tested at field moisture content. The results are plotted on the "Shear Test Diagram," B-Plates.

Consolidation Testing

Settlement predictions of the soil's behavior under load are made on the basis of the consolidation tests ASTM (D-2435-04). The consolidation apparatus is designed to receive a single one-inch high ring. Loads are applied in several increments in a geometric progression, and the resulting deformations are recorded at selected time intervals. Porous stones are placed in contact with the top and bottom of each specimen to permit addition and release of pore fluid. Samples are generally tested at increased moisture content to determine the effects of water on the bearing soil. The normal pressure at which the water is added is noted on the drawing. Results are plotted on the "Consolidation Test," C-Plates.

Expansion Index Testing

The expansion tests performed on soil samples are in accordance with the Expansion Index testing procedures, as described in the ASTM D4829-03. The soil sample is compacted into a metal ring at a saturation degree of 50 percent. The ring sample is then placed in a consolidometer, under a vertical confining pressure of 1 lbf/square inch and inundated with distilled water. The deformation of the specimen is recorded for a period of 24 hours or until the rate of deformation becomes less than 0.0002 inches/hour, whichever occurs first. The expansion index, EI, is determined by dividing the difference between final and initial height of the ring sample by the initial height, and multiplied by 1,000.

Laboratory Compaction Characteristics

The maximum dry unit weight and optimum moisture content of a soil are determined by use of ASTM D 1557-02. A soil at a selected moisture content is placed in five layers into a mold of given dimensions, with each layer compacted by 25 blows of a 10 pound hammer dropped from a distance of 18 inches subjecting the soil to a total compactive effort of about 56,000 pounds per cubic foot. The resulting dry unit weight is determined. The procedure is repeated for a sufficient number of moisture contents to establish a relationship between the dry unit weight and the water content of the soil. The data when plotted, represent a curvilinear relationship know as the compaction curve. The values of optimum moisture content and modified maximum dry unit weight are determined from the compaction curve.

REFERENCES

Blake, T.F., 2000, EQFAULT - A Computer Program for the Deterministic Prediction of Peak Horizontal Acceleration from Digitized California Faults, Version 2.20.

Blake, T.F., 2000, EQSEARCH - A Computer Program for the Estimation of Peak Horizontal Acceleration from California Historical Earthquake Catalogs, Version 2.20.

Blake, T.F., 2000, FRISKSP - A Computer Program for the Probabilistic Estimation of Uniform-Hazard Spectra Using 3-D Faults as Earthquake Sources.

Blake, T.F., 1994-1996, LIQ2_30.WQ1 - A QUATTRO-PRO Spreadsheet Computer Program for the determination of liquefaction potential using the NCEER (1996) semi-empirical method.

California Department of Conservation, Division of Mines and Geology, 1999, Seismic Hazard Zones Map, Hollywood 7½-minute Quadrangle, CDMG Seismic Hazard Zone Mapping Act of 1990.

California Department of Conservation, Division of Mines and Geology, 1998 (Revised 2006), Seismic Hazard Zone Report of the Hollywood 7½-Minute Quadrangle, Los Angeles County, California., C.D.M.G. Seismic Hazard Zone Report 026, map scale 1:24,000.

Fain, Hsai-Yang, ed., 1991, Foundation Engineering Handbook, Second Edition: New York, Van Nostrand Reinhold, pp. 630-631.

Ishihara, K. 1985, Stability of Natural Deposits During Earthquakes, Proceedings of the Eleventh International Conference on Soil Mechanics and Foundation Engineering, San Francisco, CA, Volume 1, P. 321-376, August.

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

Martin, G.R., and Lew, M., 1999, Co-chairs and Editors of the Implementation Committee, "Recommended Procedures for Implementation of DMG Special Publication 117, Guidelines for Analyzing and Mitigating Liquefaction Hazards in California," Organized through the Southern California Earthquake Center, University of Southern California.

REFERENCES (continued)

O'Rourke, T.D., Pease, J.W. (1997), Mapping Liquefiable Layer Thickness for Seismic Hazard Assessment, Journal of the Geotechnical Engineering Division, American Society of Civil Engineers, Vol. 123, no. 1, pp. 46-56.

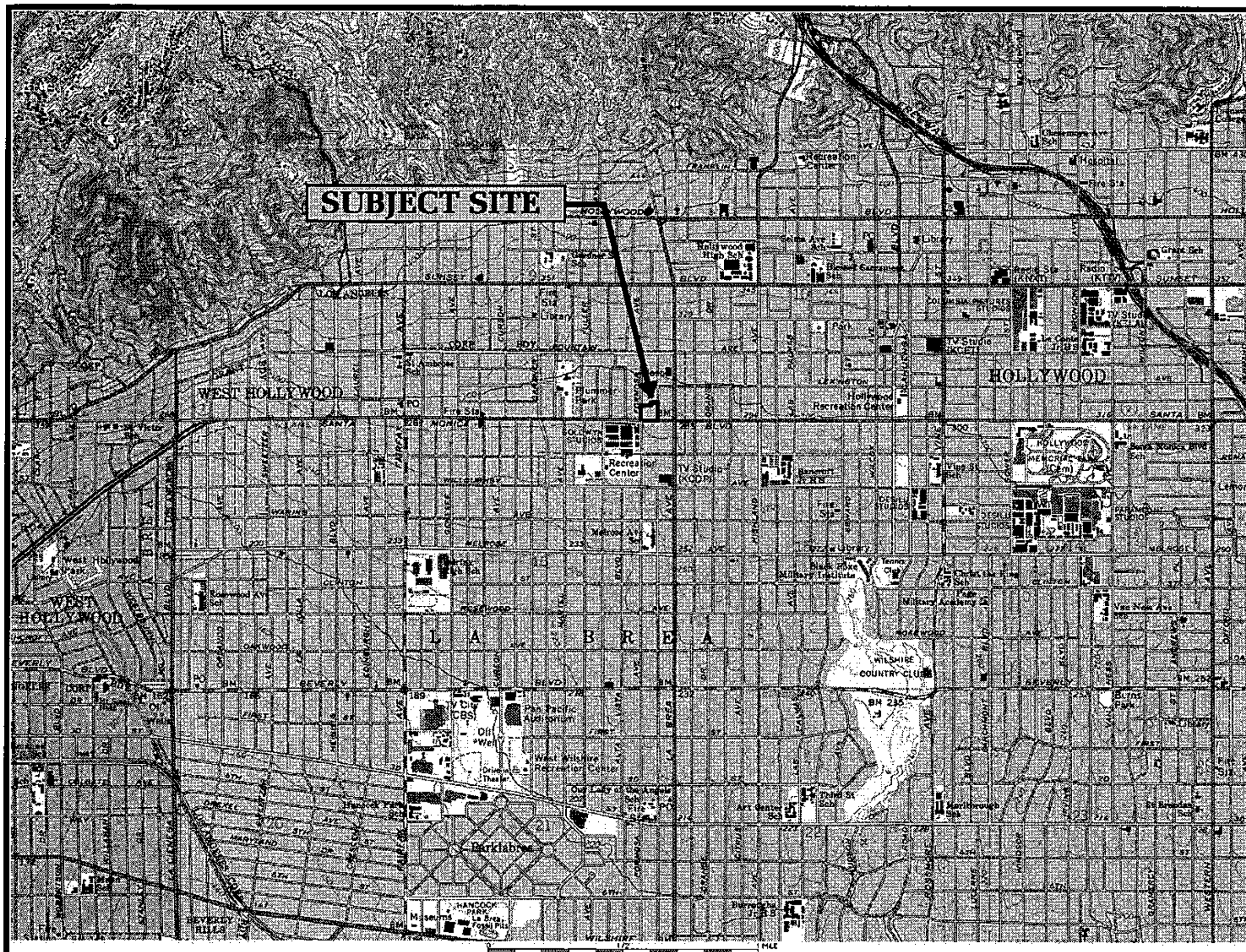
Seed, H.B. , Idriss, I.M., and Arango, I., 1983, Evaluation of Liquefaction Potential Using Field Performance Data, Journal of the Geotechnical Engineering Division, American Society of Civil Engineers, vol. 109, no. 3, pp. 458-482.

Tokimatsu, K., and Yoshimi, Y., 1983, Empirical Correlation of Soil Liquefaction Based on SPT N-Value and Fines Content, Soils and Foundations, Japanese Society of Soil Mechanics and Foundation Engineering, vol. 23, no. 4, pp. 56-74.

Tokimatsu, K. and Seed, H. B., 1987, Evaluation of Settlements in Sands Due to Earthquake Shaking, Journal of Geotechnical Engineering, ASCE, Vol. 113, No. 8.

Yerkes, R.F., McCulloh, T.H., Schoellhamer, J.E., Vedder, J.G., Geology of the Los Angeles Basin, Southern California- An Introduction, U.S. Geological Professional Paper 420-A.

Youd, T. L. and Garris, C. T., 1995, Liquefaction-Induced Ground Surface Disruption, Journal of Geotechnical Engineering, ASCE, Vol. 121, No. 11, P. 805-809.



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N

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

VICINITY MAP

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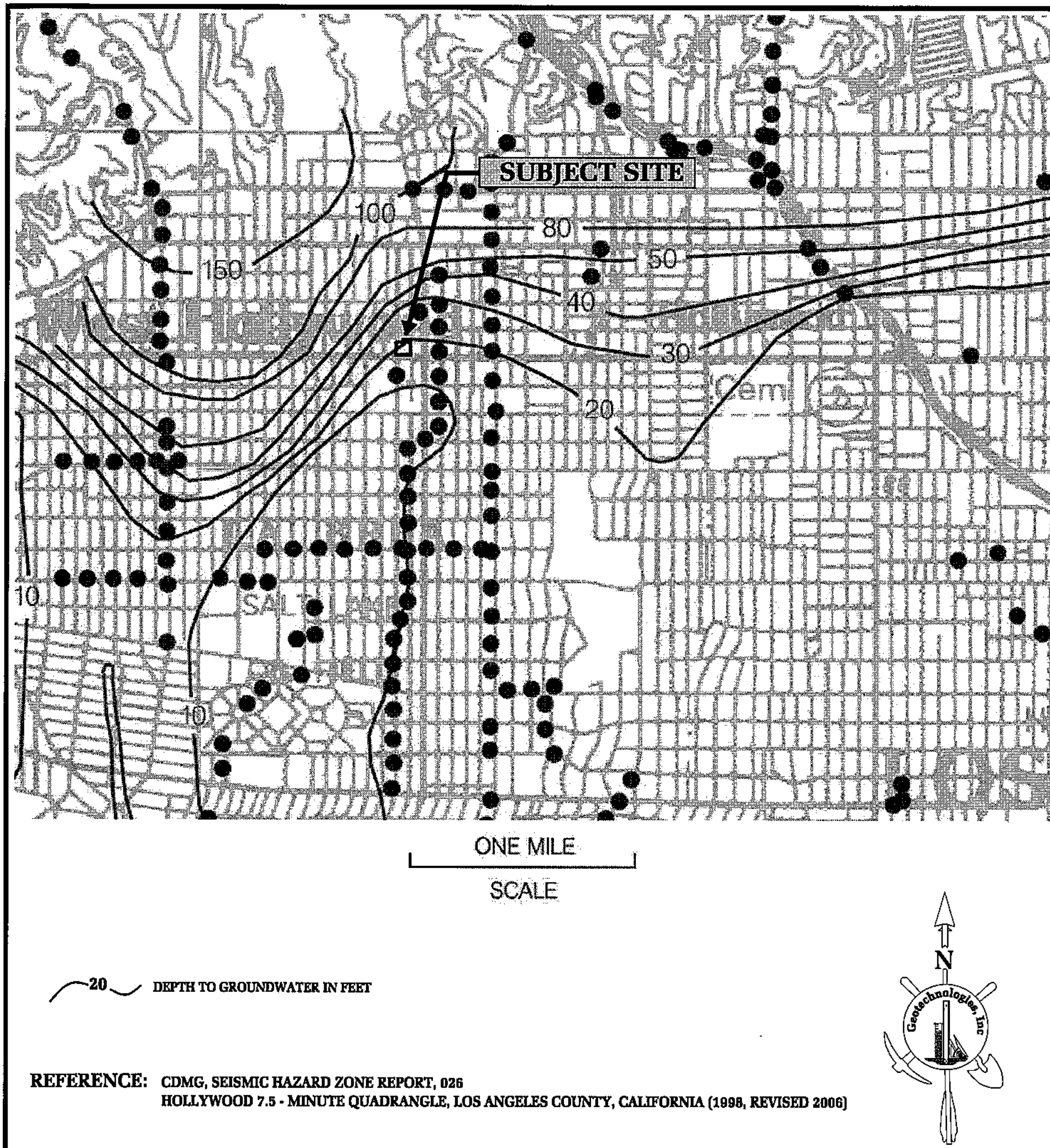
Consulting Geotechnical Engineers

HANOVER WEST, INC.

FILE NO. 19079



20 DEPTH TO GROUNDWATER IN FEET

REFERENCE: CDMG, SEISMIC HAZARD ZONE REPORT, 026
HOLLYWOOD 7.5 - MINUTE QUADRANGLE, LOS ANGELES COUNTY, CALIFORNIA (1998, REVISED 2006)

HISTORICALLY HIGHEST GROUNDWATER LEVELS

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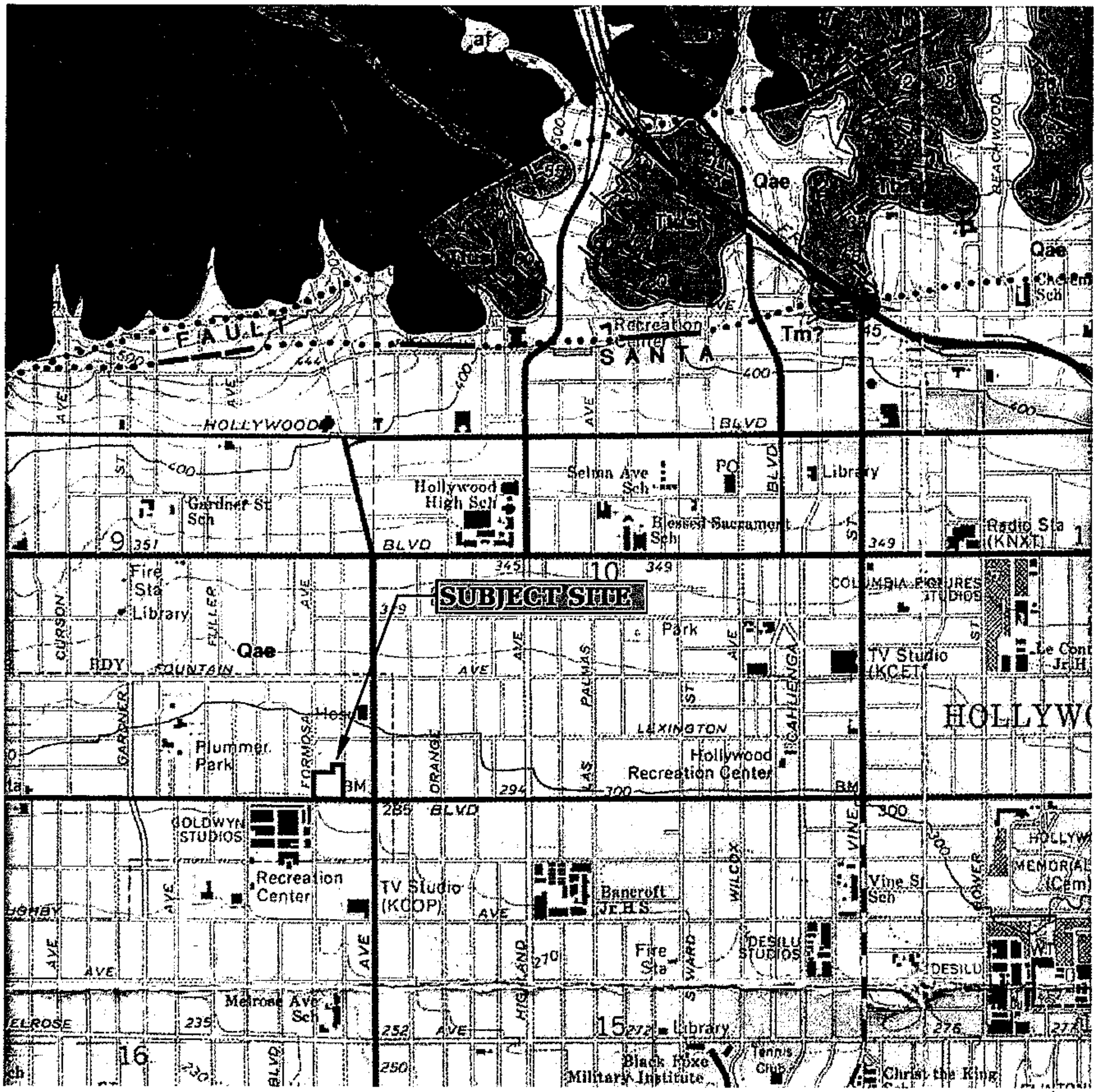

SCALE IN FEET

0 1000 2000 4000

N

LEGEND

af: Artifcial cut and fill
Qae: Older Surficial Sediments - similar to Qa but slightly elevated and dissected; includes alluvilal fan sediments
Qd: Granitic Rocks - quartz diorite, medium to light gray, massive to vaguely gneissoid
Kcr: Unnamed Strata - "Trabuco Formation: rusty-brown conglomerate, reddish sandstone and claystone
Kcg: Unnamed Strata - gray to brown, crudely bedded conglomerate of cobbles and pebbles of metavolcanic and granitic rocks and qurtzite in bronw sandy matrix
Tm: Monterey Formation - white weathering, thin bedded, platy semi-siliceous shale
Tsu: Santa Susana Formation - light gray to tan fine grained sandstone and gray, vaguely bedded micaceous silty claystone
Ttls: Lower Topanga Formation - tan, moderately hard, thick-bedded arkosic sandstone
Tts: Middle Topanga Formation and Volcanic Rocks - light gray, to tan moderately hard sandstone; locally bedded
Ttusl: Upper Topanga Formation - mostly gay micaceaous clay sharl or claystone, crumbly where weathered, and thin interbeds of gray to tan semi-friable sandstone
Tvb: Middle Topanga Formation and Volcanic Rocks - basaltic volcanic rocks: dask gray to black, fine grained, massive to locally vesicular and/or pillowed; composed of mafic minerals

REFERENCE: DIBBLEE, T.W., (1991) GEOLOGIC MAP OF THE HOLLYWOOD AND BURBANK (SOUTH 1/2) QUADRANGLES

LOCAL GEOLOGIC MAP

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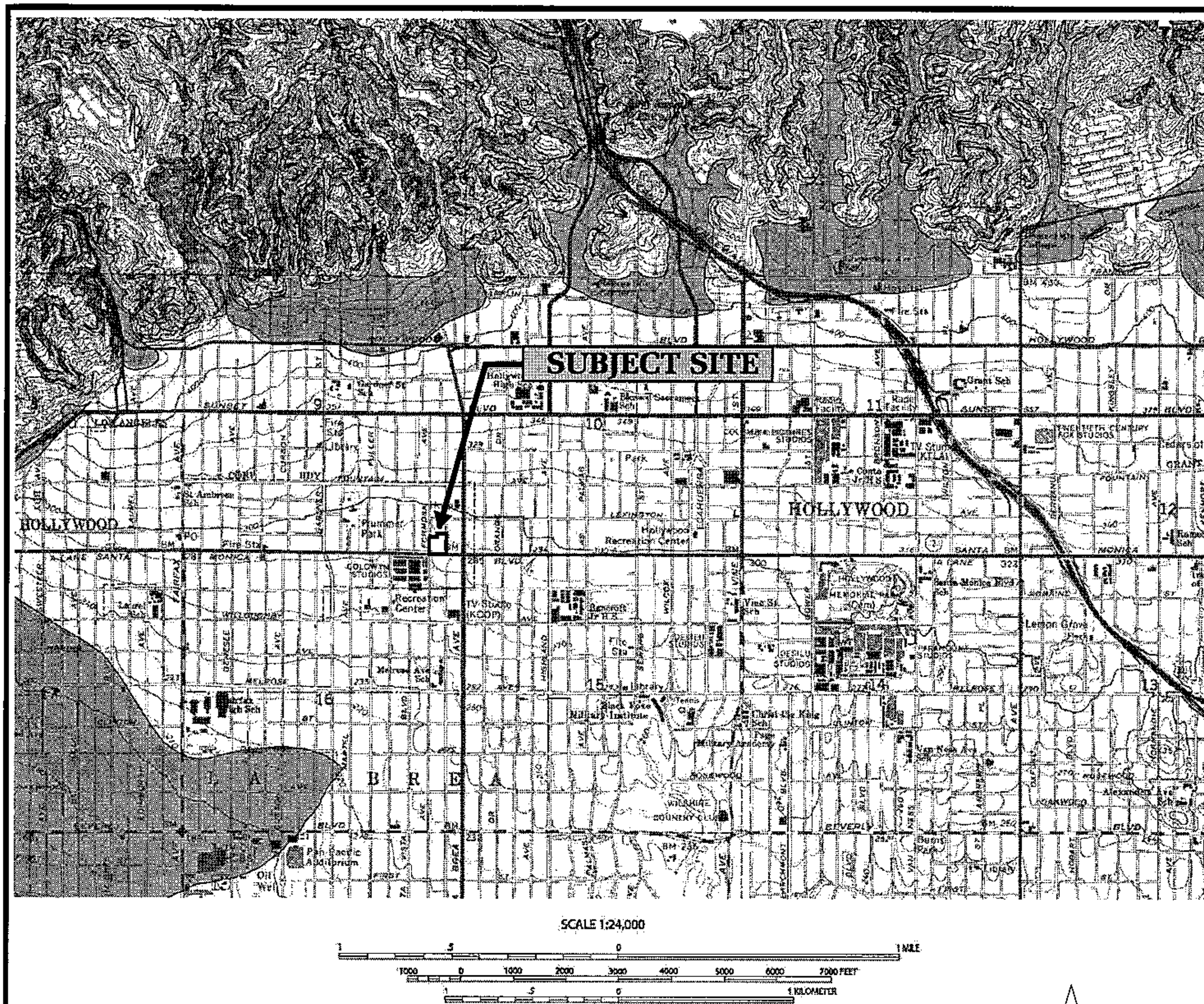
FILE NO. 19079



LIQUEFACTION AREA

REFERENCE: SEISMIC HAZARD ZONES, HOLLYWOOD QUADRANGLE OFFICIAL MAP (CDMG, 1999)

SEISMIC HAZARD ZONE MAP

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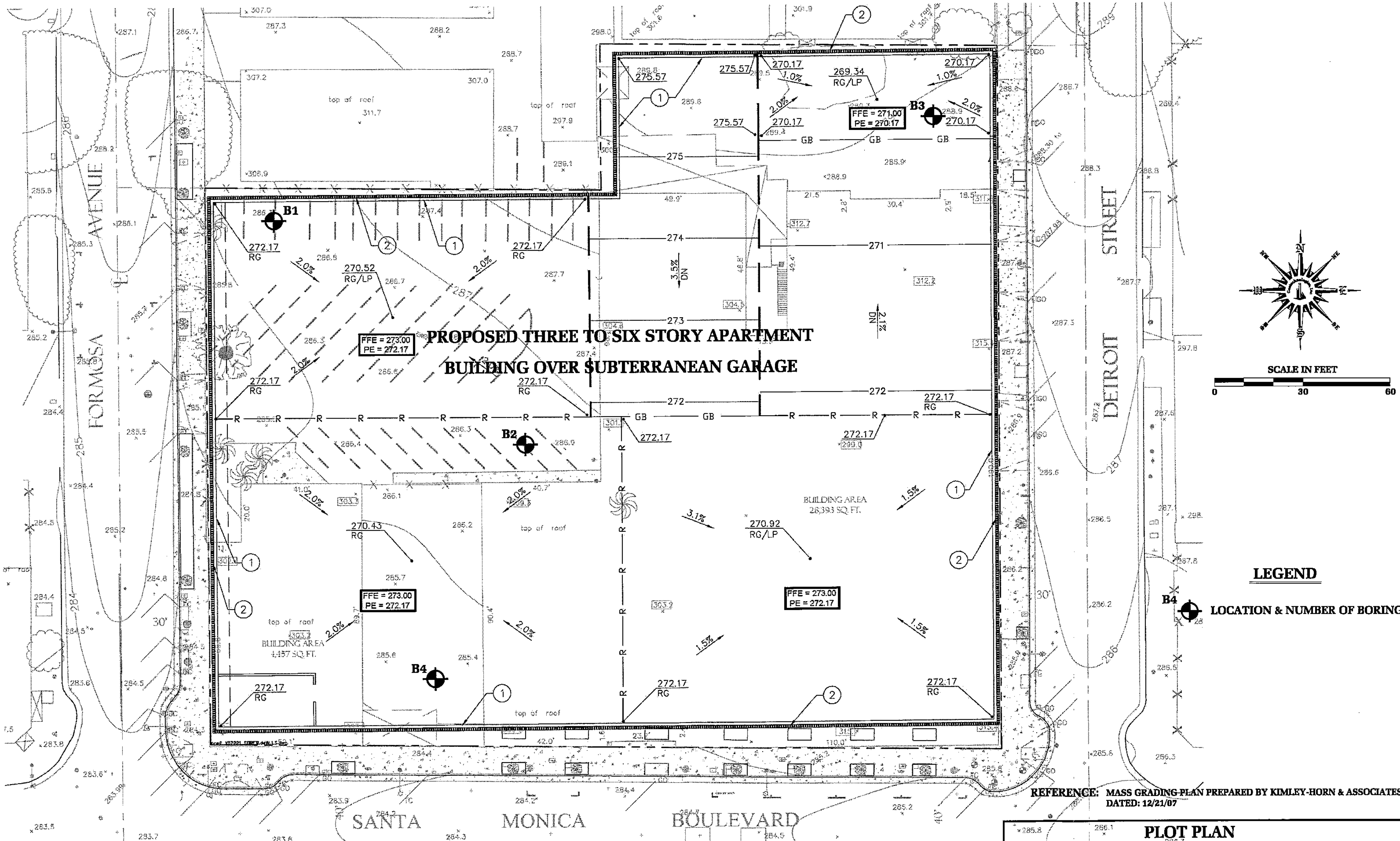
HANOVER WEST, INC.

FILE NO. 19079



REFERENCE: MASS GRADING PLAN PREPARED BY KIMLEY-HORN & ASSOCIATES DATED: 12/21/07

PLOT PLAN

HANOVER WEST, INC,

Geotechnologies, Inc.
Consulting Geotechnical Engineers

FILE No. 19079	DRAWN BY: BA
DATE: April '08	SHEET: 1 of 1

BORING LOG NUMBER 1

Drilling Date: 11/28/05 **Elevation: 286'**

Project: File No. 19079 **Hanover West, Inc.**

km

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description Surface Conditions: 4-inch Asphalt - Poor Condition, 4-inch Base
				0 --		FILL: Clay, dark brown, moist, medium stiff
				-		
				1 --		
				-		
2	70	19.7	107.1	2 --		
				-	CL	Silty to Sandy Clay, dark brown, moist, very stiff
				3 --		
				-		
				4 --		
				-		
5	74	17.5	112.7	5 --		
				-		
				6 --		
				-		
7	30 50/5"	19.5	110.3	7 --		
				-		medium brown, moist, very stiff to hard
				8 --		
				-		
				9 --		
				-		
10	30 50/4"	23.0	107.7	10 --		
				-	SC	Clayey Sand, orange-brown, slightly moist, very dense, fine grained
				11 --		
				-		
				12 --		
				-		
				13 --		
				-		
				14 --		
				-		
15	28 50/4"	23.8	102.5	15 --		
				-		
				16 --		
				-		
				17 --		
				-		
				18 --		
				-		
				19 --		
				-		
20	37 50/5"	26.4	98.9	20 --		
				-		occasional gravel
				21 --		
				-		
				22 --		
				-		
				23 --		
				-		
				24 --		
				-		
25	74	26.4	101.9	25 --		
				-	ML	Clayey Silt, orange-brown, moist, very stiff
				26 --		
				-		
				27 --		
				-		
				28 --		
				-		
				29 --		
				-		
30	87	31.1	95.0	30 --		
				-	SC	Clayey Sand, orange-brown, very moist, very dense, fine grained

BORING LOG NUMBER 1

Project: File No. 19079 **Hanover West, Inc.**

km

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description
				31		
				32		
				33		
				34		
35	76	30.7	95.8	35		
				36		
				37		
				38		
				39		
40	40 50/5"	21.4	108.8	40		
				41		Total depth: 40 feet Water at 21 feet Fill to 2 feet
				42		
				43		NOTE: The stratification lines represent the approximate boundary between earth types; the transition may be gradual
				44		
				45		Used 8-inch diameter Hollow-Stem Auger 140-lb. Slide Hammer, 30-inch drop Modified California Sampler used unless otherwise noted
				46		SPT=Standard Penetration Test
				47		
				48		
				49		
				50		
				51		
				52		
				53		
				54		
				55		
				56		
				57		
				58		
				59		
				60		

BORING LOG NUMBER 2

Drilling Date: 11/28/05 **Elevation: 286.5'**

Project: File No. 19079 **Hanover West, Inc.**

km

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description Surface Conditions: 5-inch Asphalt - Poor Condition, No Base
				0 --		FILL: Silty Clay, dark brown, moist, medium stiff to stiff
				1 --		
2	60	24.8	102.2	2 --		
				-	CL	Silty Clay, dark brown, moist, stiff
				3 --		
				4 --		
5	50	No Recovery	SPT	5 --		- - - - - - - - - - - - - -
				-		medium brown, moist, stiff, minor caliche
				6 --		
7	83	22.1	108.8	7 --		
				8 --		
				9 --		
10	68	12.1	SPT	10 --		
				-	SC	Clayey Sand, orange-brown, moist, very dense, fine grained
				11 --		
				12 --		
12½	100/7"	17.1	109.5	-		
				13 --		
				14 --		
15	58	24.7	SPT	15 --		
				16 --		
				17 --		
17½	81	25.8	100.9	-		
				18 --		
				19 --		
20	60	23.8	SPT	20 --		
				-	ML	Clayey Silt, orange-brown, very moist, very stiff
				21 --		
				22 --		
22½	90	25.9	101.5	-		
				23 --		
				24 --		
25	66	15.9	SPT	25 --		
				26 --		
				27 --		
27½	75/7"	31.7	100.9	-		
				28 --	SM	Silty Sand, orange-brown, wet, very dense, fine grained
				29 --		
30	46	33.7	SPT	30 --		
				-	CL	Sandy Clay, orange-brown, moist, stiff

BORING LOG NUMBER 2

Project: File No. 19079 **Hanover West, Inc.**

km

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description
				31 --		
				32 --		
32½	72	No Recovery		33 --		
				34 --		
35	42	33.4	SPT	35 --		
				36 --	SC	Clayey Sand, orange-brown, moist, dense, fine grained
				37 --		
37½	75/7"	30.6	98.7	38 --		
				39 --		
40	52	19.9	SPT	40 --		
				41 --	SP	Sand, medium brown, wet, dense, fine grained
				42 --		
42½	90	18.5	111.2	43 --		Clayey lense
				44 --		
45	55	17.6	SPT	45 --		
				46 --		
				47 --		
47½	40 50/3"	23.5	106.9	48 --	SC	Clayey Sand, orange-brown, moist, very dense, fine grained
				49 --		
50	56	23.5	SPT	50 --		
				51 --		
				52 --		
52½	77	30.0	96.3	53 --	CL	Silty Clay, gray and orange-brown, moist, very stiff
				54 --		
55	44	27.0	SPT	55 --		
				56 --		
				57 --		
57½	70	23.3	106.9	58 --		
				59 --		
60	50/6"	22.7	SPT	60 --		
				-	SC	Clayey Sand, orange and grayish-brown, moist, very dense, fine grained

BORING LOG NUMBER 2

Project: File No. 19079 **Hanover West, Inc.**

km

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description
				61 --		
				62 --		
62½	75/7"	19.4	113.4	-		
				63 --		
				64 --		
65	60	28.7	SPT	65 --		
				-	CL	Sandy Clay, orange-brown, moist, stiff
				66 --		
				67 --		
67½	30 50/5"	28.9	98.3	-		
				68 --		
				69 --		
70	72	22.1	SPT	70 --		
				-		Total depth: 70 feet Water at 21 feet Fill to 2 feet
				71 --		
				72 --		
				73 --		
				74 --		
				75 --		
				76 --		
				77 --		
				78 --		
				79 --		
				80 --		
				81 --		
				82 --		
				83 --		
				84 --		
				85 --		
				86 --		
				87 --		
				88 --		
				89 --		
				90 --		

BORING LOG NUMBER 3

Drilling Date: 11/29/05 | **Elevation: 289'**

Project: File No. 19079 | **Hanover West, Inc.**

km

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description Surface Conditions: 3-inch Asphalt - Fair to Good Condition, No Base
				0 --		FILL: Silty Clay, gray, brown, moist, stiff, some gravel
1	58	12.0	114.5	1 --		Sandy Clay to Sand with Gravel, grayish-brown, moist, dense, fine to coarse grained
				2 --		
3	26	10.3	120.1	3 --	SC	Clayey Sand with Gravel, brown, moist, dense, fine to coarse grained
				4 --		
5	32	18.6	108.7	5 --		interbedded Sand lenses
				6 --		
7	50 50/5"	9.1	116.2	7 --		
				8 --	ML	Sandy Silt, orange-brown, slightly moist, hard
				9 --		
10	75/7"	19.4	110.4	10 --		slightly porous
				11 --		
				12 --		
				13 --		
				14 --		
15	75/8"	18.3	107.5	15 --	SC	Clayey Sand, orange-brown, moist, very dense, fine grained, some gravel
				16 --		
				17 --		
				18 --		
				19 --		
20	32 50/5"	21.3	106.2	20 --		
				21 --		
				22 --		
				23 --		
				24 --		
25	62	22.8	105.3	25 --		
				26 --		
				27 --		
				28 --		
				29 --		
30	75	23.7	106.0	30 --		

BORING LOG NUMBER 3

Project: File No. 19079 **Hanover West, Inc.**

km

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description
				31		
				32		
				33		
				34		
35	50	32.5	93.8	35	SM	Silty Sand, orange-brown, very moist, dense, fine grained, some gravel
				36		
				37		
				38		
				39		
40	84	25.8	98.7	40	CL	Silty Clay, orange-brown, moist, hard
				41		
				42		
				43		
				44		
45	52	30.0	95.5	45		
				46		
				47		
				48		
				49		
50	77	26.0	102.2	50		Total depth: 50 feet Water at 21 feet Fill to 3 feet
				51		
				52		
				53		
				54		
				55		
				56		
				57		
				58		
				59		
				60		

BORING LOG NUMBER 4

Drilling Date: 11/29/05 **Elevation: 285.5'**

Project: File No. 19079 **Hanover West, Inc.**

km

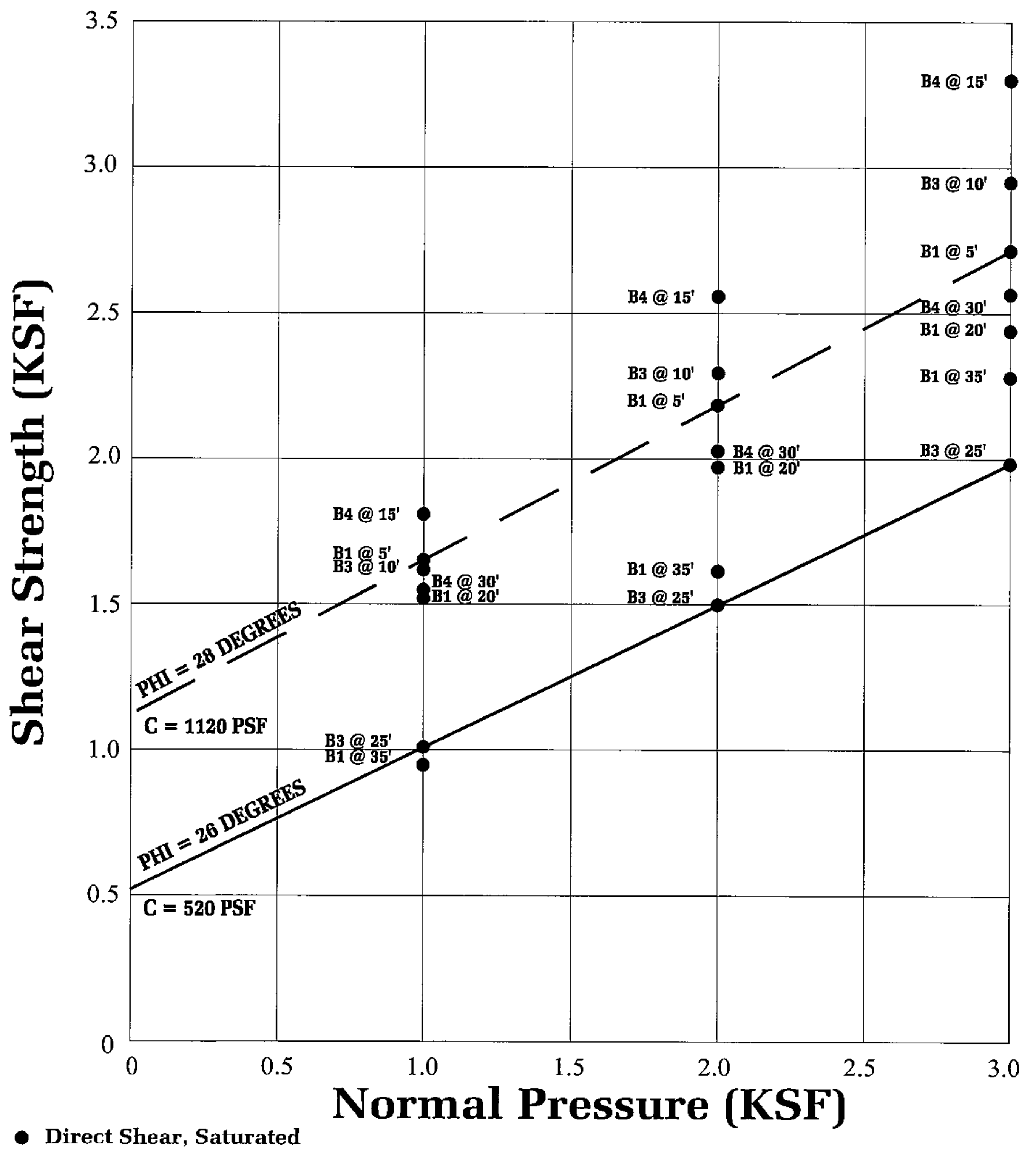
Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description Surface Conditions: 6-inch Asphalt, No Base
				0 --		FILL: Silty Sand, brown, moist, medium dense, fine grained, minor gravel
				-		
1	35	21.0	105.1	1 --		
				-	SC	Clayey Sand, dark brown, moist, dense, fine grained, some gravel
				2 --		
				-		
3	55	23.8	101.9	3 --		
				-	CL	Silty Clay, dark brown, moist, stiff, some gravel, hard
				4 --		
				-		
5	57	16.9	113.3	5 --		
				-	SC	Clayey Sand, orange-brown, moist, dense, fine grained, some gravel
				6 --		
				-		
7	73	20.1	109.4	7 --		
				-		
				8 --		
				-		
				9 --		
				-		
10	75/7"	19.9	103.3	10 --		
				-	ML	Sandy Silt, orange-brown, moist, hard, some gravel, some caliche
				11 --		
				-		
				12 --		
				-		
				13 --		
				-		
				14 --		
				-		
15	75/8"	14.8	111.7	15 --		
				-		
				16 --		
				-		
				17 --		
				-		
				18 --		
				-		
				19 --		
				-		
20	62	23.0	100.4	20 --		
				-	SC	Clayey Sand, orange-brown, moist, very dense, fine grained
				21 --		
				-		
				22 --		
				-		
				23 --		
				-		
				24 --		
				-		
25	70	25.4	100.5	25 --		
				-		
				26 --		
				-		
				27 --		
				-		
				28 --		
				-		
				29 --		
				-		
30	72	28.6	96.3	30 --		
				-	CL	Silty Clay, orange-brown, moist, hard

BORING LOG NUMBER 4

Project: File No. 19079 **Hanover West, Inc.**

km

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description
				31		
				32		
				33		
				34		
35	72	20.9	105.3	35		
				36	ML	Clayey Silt, orange-brown, moist, hard
				37		
				38		
				39		
40	76	20.3	112.6	40		
				41	SC	Clayey Sand, orange-brown, moist, very dense, fine grained, some gravel
				42		
				43		
				44		
45	72	19.7	110.4	45		
				46		
				47		
				48		
				49		
50	75	27.3	97.5	50	CL	Sandy Clay, orange-brown, gray mottling, moist, hard, some gravel
				51		Total depth: 50 feet Water at 21 feet Fill to 1 feet
				52		
				53		
				54		
				55		
				56		
				57		
				58		
				59		
				60		



SHEAR TEST DIAGRAM

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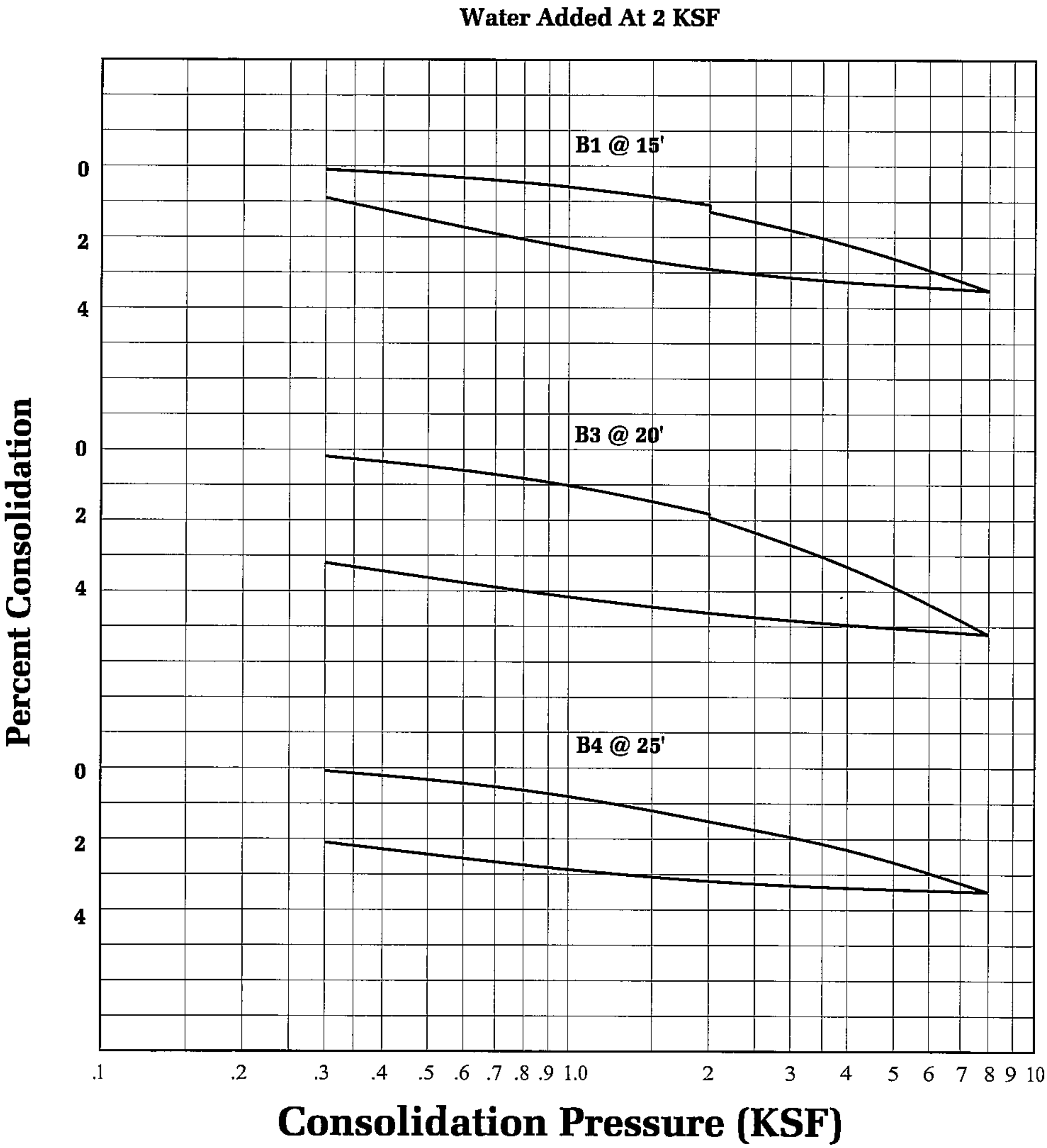
Consulting Geotechnical Engineers

HANOVER WEST, INC.	
FILE NO. 19079	PLATE: B

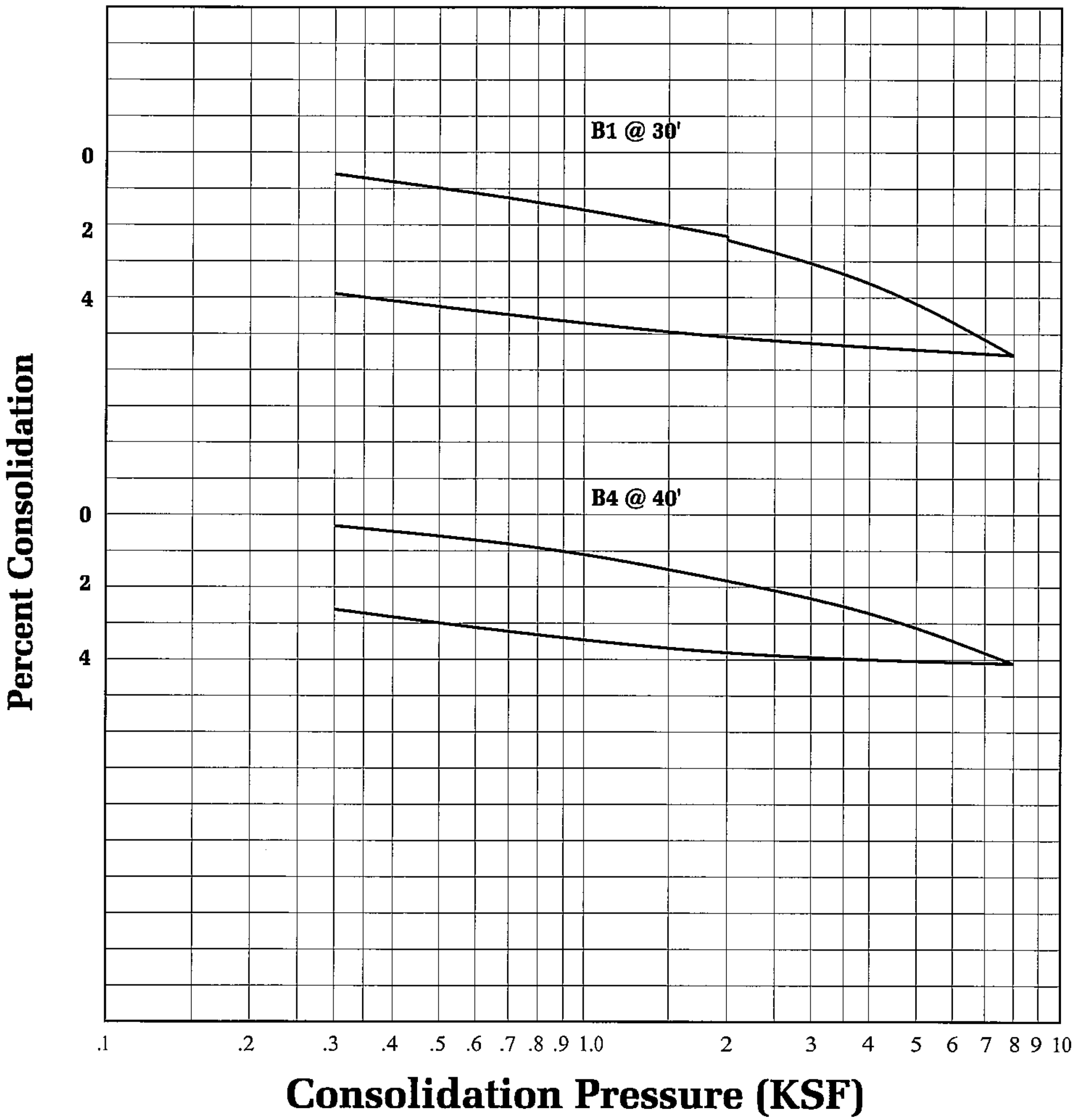


CONSOLIDATION TEST

Geotechnologies, Inc.
Consulting Geotechnical Engineers

HANOVER WEST, INC.	
FILE NO. 19079	PLATE: C-1

Water Added At 2 KSF



CONSOLIDATION TEST

Geotechnologies, Inc.
Consulting Geotechnical Engineers

HANOVER WEST, INC.	
FILE NO. 19079	PLATE: C-2

ASTM D 4829-03

SAMPLE	B3 @ 15'
SOIL TYPE:	SC
EXPANSION INDEX UBC STANDARD 18-2	45
EXPANSION CHARACTER	LOW

Geotechnologies, Inc.

Project: Hanover West, Inc.
File No.: 19079
Description: Liquefaction Analysis
Boring Number: 2

EMPIRICAL ESTIMATION OF LIQUEFACTION POTENTIAL

NCEER (1996) METHOD | By Thomas F. Blake (1994-1996) | LIQ2_30.WQ1

EARTHQUAKE INFORMATION:

Earthquake Magnitude:	7.1
Peak Horiz. Acceleration (g):	0.75
Calculated Mag.Wtg.Factor:	0.873

GROUNDWATER INFORMATION:

Current Groundwater Level (ft):	21.0
Historic Highest Groundwater Level* (ft):	17.0
Unit Wt. Water (pcf):	62.4

* Based on California Geological Survey Seismic Hazard Evaluation Report

ENERGY & ROD CORRECTIONS:

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

LIQUEFACTION CALCULATIONS:

Depth to Base (ft)	Total Unit Wt. (pcf)	Current Water Level (0 or 1)	FIELD SPT (N)	Depth of SPT (ft)	Liq.Sus. (0 or 1)	-200 (%)	Est. Dr (%)	CN Factor	Corrected $(N_1)_{60}$	Resist. CRR	rd Factor	Induced CSR	Liquefac. Safe.Fact.
1.0	127.6	0	NA	1.0	0	0.0		2.000	0.0	~	0.998	0.425	~
2.0	127.6	0	NA	1.0	0	0.0		#VALUE!	#VALUE!	~	0.993	0.423	~
3.0	127.6	0	NA	1.0	0	0.0		#VALUE!	#VALUE!	~	0.989	0.421	~
4.0	127.6	0	NA	1.0	0	0.0		#VALUE!	#VALUE!	~	0.984	0.419	~
5.0	127.6	0	NA	1.0	0	0.0		#VALUE!	#VALUE!	~	0.979	0.417	~
6.0	127.6	0	NA	1.0	0	0.0		#VALUE!	#VALUE!	~	0.975	0.415	~
7.0	127.6	0	NA	1.0	0	0.0		#VALUE!	#VALUE!	~	0.970	0.413	~
8.0	131.8	0	NA	1.0	0	0.0		#VALUE!	#VALUE!	~	0.966	0.411	~
9.0	131.8	0	NA	1.0	0	0.0		#VALUE!	#VALUE!	~	0.961	0.409	~
10.0	131.8	0	NA	1.0	0	0.0		#VALUE!	#VALUE!	~	0.957	0.407	~
11.0	128.2	0	NA	1.0	0	0.0		#VALUE!	#VALUE!	~	0.952	0.405	~
12.0	128.2	0	NA	1.0	0	0.0		#VALUE!	#VALUE!	~	0.947	0.403	~
13.0	128.2	0	NA	1.0	0	0.0		#VALUE!	#VALUE!	~	0.943	0.401	~
14.0	128.2	0	NA	1.0	0	0.0		#VALUE!	#VALUE!	~	0.938	0.399	~
15.0	128.2	0	NA	1.0	0	0.0		#VALUE!	#VALUE!	~	0.934	0.398	~
16.0	128.2	0	NA	1.0	0	0.0		#VALUE!	#VALUE!	~	0.929	0.396	~
17.0	128.2	0	NA	1.0	0	0.0		#VALUE!	#VALUE!	~	0.925	0.394	~
18.0	126.9	0	58.0	15.0	1	0.0	120	1.065	59.8	Infin.	0.920	0.392	Non-Liq.
19.0	126.9	0	58.0	15.0	1	0.0	120	1.065	59.8	Infin.	0.915	0.390	Non-Liq.
20.0	126.9	0	58.0	15.0	1	0.0	120	1.065	59.8	Infin.	0.911	0.388	Non-Liq.
21.0	127.8	0	60.0	20.0	1	0.0	113	0.919	59.2	Infin.	0.906	0.386	Non-Liq.
22.0	127.8	1	60.0	20.0	1	0.0	113	0.919	59.2	Infin.	0.902	0.388	Non-Liq.
23.0	127.8	1	60.0	20.0	1	0.0	113	0.919	59.2	Infin.	0.897	0.395	Non-Liq.
24.0	127.8	1	60.0	20.0	1	0.0	113	0.919	59.2	Infin.	0.893	0.401	Non-Liq.
25.0	127.8	1	60.0	20.0	1	0.0	113	0.919	59.2	Infin.	0.888	0.406	Non-Liq.
26.0	127.8	1	66.0	25.0	1	0.0	114	0.851	64.4	Infin.	0.883	0.411	Non-Liq.
27.0	127.8	1	66.0	25.0	1	0.0	114	0.851	64.4	Infin.	0.879	0.416	Non-Liq.
28.0	132.9	1	66.0	25.0	1	0.0	114	0.851	64.4	Infin.	0.874	0.421	Non-Liq.
29.0	132.9	1	66.0	25.0	1	0.0	114	0.851	64.4	Infin.	0.870	0.425	Non-Liq.
30.0	132.9	1	66.0	25.0	1	0.0	114	0.851	64.4	Infin.	0.865	0.428	Non-Liq.
31.0	132.9	1	46.0	30.0	1	0.0	92	0.805	44.5	Infin.	0.861	0.432	Non-Liq.
32.0	132.9	1	46.0	30.0	1	0.0	92	0.805	44.5	Infin.	0.856	0.435	Non-Liq.
33.0	132.9	1	46.0	30.0	1	0.0	92	0.805	44.5	Infin.	0.851	0.437	Non-Liq.
34.0	132.9	1	46.0	30.0	1	0.0	92	0.805	44.5	Infin.	0.847	0.440	Non-Liq.
35.0	132.9	1	46.0	30.0	1	0.0	92	0.805	44.5	Infin.	0.842	0.442	Non-Liq.
36.0	128.8	1	42.0	35.0	1	0.0	85	0.765	38.6	Infin.	0.838	0.444	Non-Liq.
37.0	128.8	1	42.0	35.0	1	0.0	85	0.765	38.6	Infin.	0.833	0.446	Non-Liq.
38.0	128.8	1	42.0	35.0	1	0.0	85	0.765	38.6	Infin.	0.829	0.448	Non-Liq.
39.0	128.8	1	42.0	35.0	1	0.0	85	0.765	38.6	Infin.	0.824	0.449	Non-Liq.
40.0	128.8	1	42.0	35.0	1	0.0	85	0.765	38.6	Infin.	0.819	0.451	Non-Liq.
41.0	131.8	1	52.0	40.0	1	0.0	92	0.732	45.7	Infin.	0.815	0.452	Non-Liq.
42.0	131.8	1	52.0	40.0	1	0.0	92	0.732	45.7	Infin.	0.810	0.453	Non-Liq.
43.0	131.8	1	52.0	40.0	1	0.0	92	0.732	45.7	Infin.	0.806	0.454	Non-Liq.
44.0	131.8	1	52.0	40.0	1	0.0	92	0.732	45.7	Infin.	0.801	0.454	Non-Liq.
45.0	131.8	1	52.0	40.0	1	0.0	92	0.732	45.7	Infin.	0.797	0.455	Non-Liq.
46.0	131.8	1	55.0	45.0	1	0.0	92	0.702	46.3	Infin.	0.792	0.455	Non-Liq.
47.0	131.8	1	55.0	45.0	1	0.0	92	0.702	46.3	Infin.	0.787	0.456	Non-Liq.
48.0	132.1	1	56.0	50.0	1	0.0	90	0.675	45.4	Infin.	0.783	0.456	Non-Liq.
49.0	132.1	1	56.0	50.0	1	0.0	90	0.675	45.4	Infin.	0.778	0.456	Non-Liq.
50.0	132.1	1	56.0	50.0	1	0.0	90	0.675	45.4	Infin.	0.774	0.456	Non-Liq.
51.0	132.1	1	56.0	50.0	1	0.0	90	0.675	45.4	Infin.	0.769	0.455	Non-Liq.
52.0	132.1	1	56.0	50.0	1	0.0	90	0.675	45.4	Infin.	0.765	0.455	Non-Liq.
53.0	125.3	1	44.0	55.0	1	0.0	78	0.652	34.4	Infin.	0.760	0.455	Non-Liq.
54.0	125.3	1	44.0	55.0	1	0.0	78	0.652	34.4	Infin.	0.755	0.454	Non-Liq.
55.0	125.3	1	44.0	55.0	1	0.0	78	0.652	34.4	Infin.	0.751	0.454	Non-Liq.
56.0	125.3	1	44.0	55.0	1	0.0	78	0.652	34.4	Infin.	0.746	0.454	Non-Liq.
57.0	125.3	1	44.0	55.0	1	0.0	78	0.652	34.4	Infin.	0.742	0.453	Non-Liq.
58.0	131.8	1	44.0	55.0	1	0.0	78	0.652	34.4	Infin.	0.737	0.452	Non-Liq.
59.0	131.8	1	44.0	55.0	1	0.0	78	0.652	34.4	Infin.	0.733	0.451	Non-Liq.
60.0	131.8	1	44.0	55.0	1	0.0	78	0.652	34.4	Infin.	0.728	0.450	Non-Liq.
61.0	135.4	1	100.0	60.0	1	0.0	115	0.632	75.8	Infin.	0.723	0.449	Non-Liq.
62.0	135.4	1	100.0	60.0	1	0.0	115	0.632	75.8	Infin.	0.719	0.448	Non-Liq.
63.0	135.4	1	100.0	60.0	1	0.0	115	0.632	75.8	Infin.	0.714	0.447	Non-Liq.
64.0	135.4	1	100.0	60.0	1	0.0	115	0.632	75.8	Infin.	0.710	0.445	Non-Liq.
65.0	135.4	1	100.0	60.0	1	0.0	115	0.632	75.8	Infin.	0.705	0.444	Non-Liq.
66.0	126.7	1	60.0	65.0	1	0.0	87	0.611	44.0	Infin.	0.701	0.443	Non-Liq.
67.0	126.7	1	60.0	65.0	1	0.0	87	0.611	44.0	Infin.	0.696	0.441	Non-Liq.
68.0	126.7	1	60.0	65.0	1	0.0	87	0.611	44.0	Infin.	0.691	0.440	Non-Liq.
69.0	126.7	1	60.0	65.0	1	0.0	87	0.611	44.0	Infin.	0.687	0.439	Non-Liq.
70.0	126.7	1	72.0	70.0	1	0.0	93	0.600	51.8	Infin.	0.682	0.437	Non-Liq.

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*                       *
*     E Q F A U L T     *
*                       *
*     Version 3.00      *
*                       *
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DETERMINISTIC ESTIMATION OF
PEAK ACCELERATION FROM DIGITIZED FAULTS

JOB NUMBER: 19079

DATE: 04-08-2008

JOB NAME: HANOVER WEST, INC.

CALCULATION NAME: HANOVER WEST, INC.

FAULT-DATA-FILE NAME: CGSFLTE.DAT

SITE COORDINATES:
SITE LATITUDE: 34.0911
SITE LONGITUDE: 118.3459

SEARCH RADIUS: 60 mi

ATTENUATION RELATION: 7) Bozorgnia Campbell Niazi (1999) Hor.-Pleist. Soil-Uncor.
UNCERTAINTY (M=Median, S=Sigma): S Number of Sigmas: 1.0
DISTANCE MEASURE: cdist
SCOND: 0
Basement Depth: 5.00 km Campbell SSR: 0 Campbell SHR: 0
COMPUTE PEAK HORIZONTAL ACCELERATION

FAULT-DATA FILE USED: CGSFLTE.DAT

MINIMUM DEPTH VALUE (km): 3.0

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Consulting Geotechnical Engineers

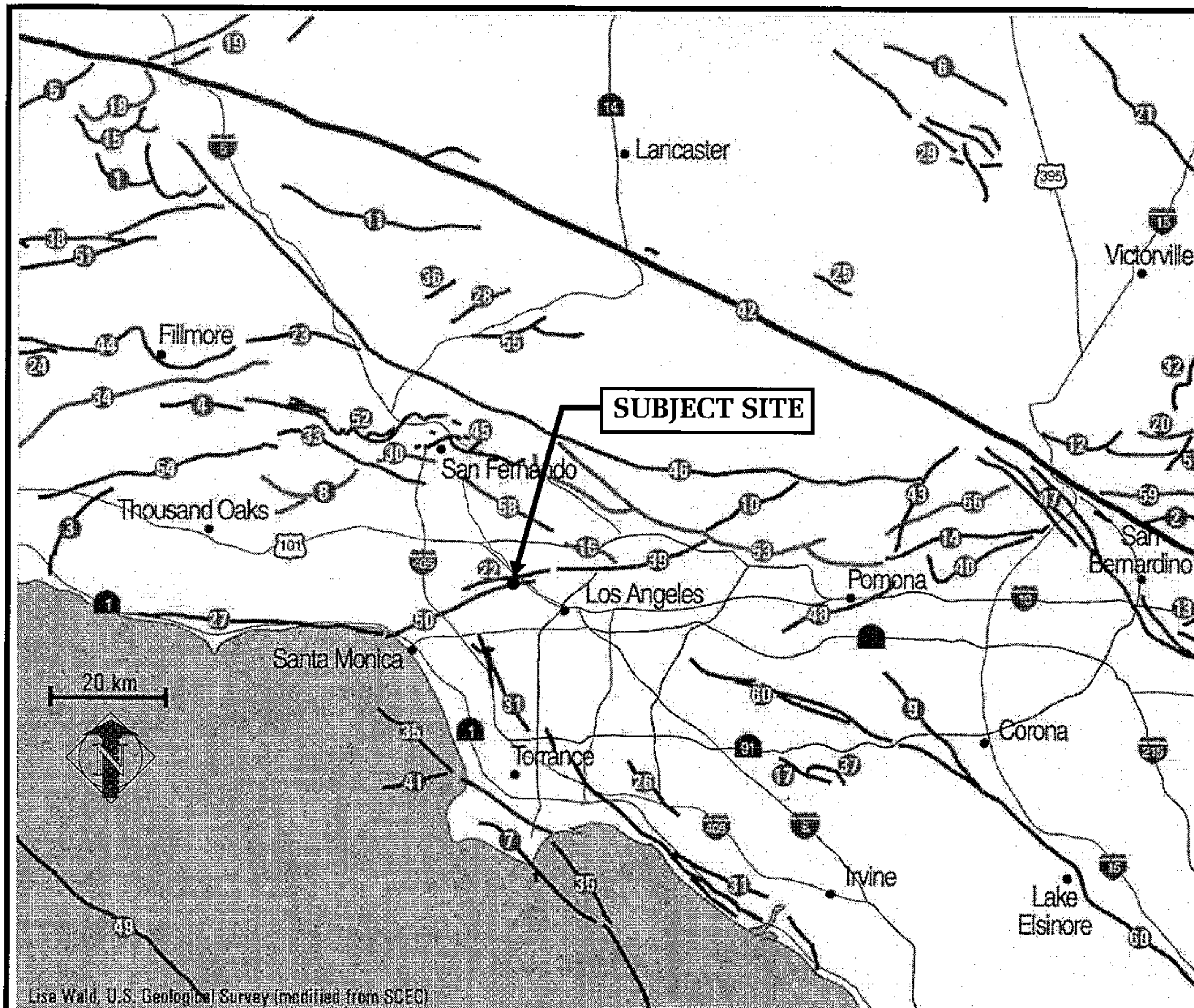
TABLE I - FAULTS IN THE VICINITY OF THE SITE

ABBREVIATED FAULT NAME	APPROXIMATE DISTANCE mi (km)	ESTIMATED MAX. EARTHQUAKE EVENT: MAXIMUM EARTHQUAKE MAG. (Mw)	PEAK SITE ACCEL. g	EST. SITE INTENSITY MOD.MERC.
HOLLYWOOD	2.4(3.9)	6.4	1.027	XI
UPPER ELYSIAN PARK BLIND THRUST	3.7(5.9)	6.4	1.012	XI
NEWPORT-INGLEWOOD (L.A.Basin)	4.5(7.3)	7.1	0.811	XI
PUENTE HILLS BLIND THRUST	4.7(7.6)	7.1	1.088	XI
SANTA MONICA	5.0(8.1)	6.6	0.807	XI
RAYMOND	7.8(12.5)	6.5	0.560	X
VERDUGO	9.7(15.6)	6.9	0.573	X
MALIBU COAST	11.4(18.4)	6.7	0.435	X
NORTHRIDGE (E. Oak Ridge)	12.7(20.5)	7.0	0.534	X
SIERRA MADRE	14.0(22.6)	7.2	0.482	X
SIERRA MADRE (San Fernando)	14.8(23.8)	6.7	0.329	IX
PALOS VERDES	16.0(25.8)	7.3	0.376	IX
SAN GABRIEL	16.2(26.1)	7.2	0.348	IX
SANTA SUSANA	19.4(31.2)	6.7	0.240	IX
WHITTIER	20.2(32.5)	6.8	0.206	VIII
CLAMSHELL-SAWPIT	20.3(32.6)	6.5	0.196	VIII
ANACAPA-DUME	21.4(34.5)	7.5	0.377	IX
HOLSER	24.4(39.2)	6.5	0.156	VIII
SIMI-SANTA ROSA	25.2(40.5)	7.0	0.219	IX
SAN JOSE	26.6(42.8)	6.4	0.129	VIII
OAK RIDGE (Onshore)	29.3(47.1)	7.0	0.182	VIII
CHINO-CENTRAL AVE. (Elsinore)	32.9(52.9)	6.7	0.125	VII
CUCAMONGA	34.4(55.4)	6.9	0.137	VIII
SAN ANDREAS - Whole M-1a	34.5(55.5)	8.0	0.262	IX
SAN ANDREAS - Mojave M-1c-3	34.5(55.5)	7.4	0.169	VIII
SAN ANDREAS - 1857 Rupture M-2a	34.5(55.5)	7.8	0.227	IX
SAN ANDREAS - Cho-Moj M-1b-1	34.5(55.5)	7.8	0.227	IX
SAN CAYETANO	34.8(56.0)	7.0	0.147	VIII
SAN JOAQUIN HILLS	37.2(59.9)	6.6	0.112	VII
NEWPORT-INGLEWOOD (Offshore)	42.5(68.4)	7.1	0.103	VII
SAN ANDREAS - Carrizo M-1c-2	43.1(69.3)	7.4	0.128	VIII
ELSINORE (GLEN IVY)	43.8(70.5)	6.8	0.078	VII
SANTA YNEZ (East)	46.3(74.5)	7.1	0.092	VII
VENTURA - PITAS POINT	49.1(79.0)	6.9	0.087	VII
SAN JACINTO-SAN BERNARDINO	49.2(79.1)	6.7	0.062	VI
SAN ANDREAS - San Bernardino M-1	50.5(81.2)	7.5	0.113	VII
SAN ANDREAS - SB-Coach. M-1b-2	50.5(81.2)	7.7	0.132	VIII
SAN ANDREAS - SB-Coach. M-2b	50.5(81.2)	7.7	0.132	VIII
OAK RIDGE(Blind Thrust Offshore)	51.3(82.5)	7.1	0.110	VII
CLEGHORN	52.7(84.8)	6.5	0.048	VI
CHANNEL IS. THRUST (Eastern)	52.9(85.1)	7.5	0.145	VIII
OAK RIDGE MID-CHANNEL STRUCTURE	54.4(87.6)	6.6	0.068	VI
M.RIDGE-ARROYO PARIDA-SANTA ANA	54.7(88.1)	7.2	0.096	VII
RED MOUNTAIN	58.7(94.4)	7.0	0.075	VII
GARLOCK (West)	59.1(95.1)	7.3	0.079	VII

-END OF SEARCH- 45 FAULTS FOUND WITHIN THE SPECIFIED SEARCH RADIUS.

THE HOLLYWOOD FAULT IS CLOSEST TO THE SITE.
IT IS ABOUT 2.4 MILES (3.9 km) AWAY.

LARGEST MAXIMUM-EARTHQUAKE SITE ACCELERATION: 1.0883 g



1 Alamo thrust
2 Arrowhead fault
3 Bailey fault
4 Big Mountain fault
5 Big Pine fault
6 Blake Ranch fault
7 Cabrillo fault
8 Chatsworth fault
9 Chino fault
10 Clamshell-Sawpit fault
11 Clearwater fault
12 Cleghorn fault
13 Crafton Hills fault zone
14 Cucamonga fault zone
15 Dry Creek fault
16 Eagle Rock fault
17 El Modeno fault
18 Frazier Mountain thrust
19 Garlock fault zone
20 Grass Valley fault
21 Helendale fault
22 Hollywood fault
23 Holser fault
24 Lion Canyon fault
25 Llano fault
26 Los Alamitos fault
27 Malibu Coast fault
28 Mint Canyon fault
29 Mirage Valley fault zone
30 Mission Hills fault
31 Newport Inglewood fault zone
32 North Frontal fault zone
33 Northridge Hills fault
34 Oak Ridge fault
35 Palos Verdes fault zone
36 Pelona fault
37 Peralta Hills fault
38 Pine Mountain fault
39 Raymond fault
40 Red Hill (Etiwanda Ave) fault
41 Redondo Canyon fault
42 San Andreas Fault
43 San Antonio fault
44 San Cayetano fault
45 San Fernando fault zone
46 San Gabriel fault zone
47 San Jacinto fault
48 San Jose fault
49 Santa Cruz-Santa Catalina Ridge f.z.
50 Santa Monica fault
51 Santa Ynez fault
52 Santa Susana fault zone
53 Sierra Madre fault zone
54 Simi fault
55 Soledad Canyon fault
56 Stoddard Canyon fault
57 Tunnel Ridge fault
58 Verdugo fault
59 Waterman Canyon fault
60 Whittier fault

REFERENCE: http://pasadena.wr.usgs.gov/info/images/LA%20Faults.pdf

SOUTHERN CALIFORNIA FAULT MAP

Geotechnologies, Inc.
Consulting Geotechnical Engineers

HANOVER WEST, INC.

FILE No. 19079

FIGURE I

Geotechnologies, Inc.
Consulting Geotechnical Engineers

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*   E Q S E A R C H    *
*                      *
*    Version 3.00      *
*                      *
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ESTIMATION OF
PEAK ACCELERATION FROM
CALIFORNIA EARTHQUAKE CATALOGS

JOB NUMBER: 19079

DATE: 04-08-2008

JOB NAME: HANOVER WEST, INC.

EARTHQUAKE-CATALOG-FILE NAME: F:\Seismic New\EQSEARCH\ALLQUAKE.DAT

SITE COORDINATES:
SITE LATITUDE: 34.0911
SITE LONGITUDE: 118.3459

SEARCH DATES:
START DATE: 1800
END DATE: 2008

SEARCH RADIUS:
60.0 mi
96.6 km

ATTENUATION RELATION: 7) Bozorgnia Campbell Niazi (1999) Hor.-Pleist. Soil-Uncor.
UNCERTAINTY (M=Median, S=Sigma): S Number of Sigmas: 1.0
ASSUMED SOURCE TYPE: DS [SS=Strike-slip, DS=Reverse-slip, BT=Blind-thrust]
SCOND: 0 Depth Source: A
Basement Depth: 5.00 km Campbell SSR: 0 Campbell SHR: 0
COMPUTE PEAK HORIZONTAL ACCELERATION

MINIMUM DEPTH VALUE (km): 3.0

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TABLE II - HISTORICAL EARTHQUAKE EPICENTERS

FILE CODE	LAT. NORTH	LONG. WEST	DATE	TIME (UTC) H M Sec	DEPTH (km)	QUAKE MAG.	SITE ACC. g	SITE MM INT.	APPROX. DISTANCE mi [km]
MGI	34.0800	118.2600	07/16/1920	18 8 0.0	0.0	5.00	0.244	IX	5.0(8.0)
MGI	34.0000	118.3000	09/03/1905	540 0.0	0.0	5.30	0.250	IX	6.8(11.0)
T-A	34.0000	118.2500	09/23/1827	0 0 0.0	0.0	5.00	0.159	VIII	8.3(13.4)
T-A	34.0000	118.2500	01/10/1856	0 0 0.0	0.0	5.00	0.159	VIII	8.3(13.4)
T-A	34.0000	118.2500	03/26/1860	0 0 0.0	0.0	5.00	0.159	VIII	8.3(13.4)
MGI	34.0000	118.5000	11/19/1918	2018 0.0	0.0	5.00	0.122	VII	10.8(17.4)
DMG	34.0000	118.5000	08/04/1927	1224 0.0	0.0	5.00	0.122	VII	10.8(17.4)
GSP	34.2310	118.4750	03/20/1994	212012.3	13.0	5.30	0.138	VIII	12.1(19.6)
GSP	34.2130	118.5370	01/17/1994	123055.4	18.0	6.70	0.353	IX	13.8(22.2)
MGI	34.1000	118.1000	07/11/1855	415 0.0	0.0	6.30	0.260	IX	14.1(22.6)
PAS	34.0730	118.0980	10/04/1987	105938.2	8.2	5.30	0.115	VII	14.2(22.9)
PAS	34.0610	118.0790	10/01/1987	144220.0	9.5	5.90	0.171	VIII	15.4(24.8)
DMG	34.3080	118.4540	02/09/1971	144346.7	6.2	5.20	0.091	VII	16.2(26.1)
DMG	33.8500	118.2670	03/11/1933	1425 0.0	0.0	5.00	0.071	VI	17.2(27.8)
DMG	33.9500	118.6320	08/31/1930	04036.0	0.0	5.20	0.074	VII	19.0(30.7)
GSB	34.3010	118.5650	01/17/1994	204602.4	9.0	5.20	0.074	VII	19.1(30.8)
GSP	34.3050	118.5790	01/29/1994	112036.0	1.0	5.10	0.065	VI	19.9(32.0)
PAS	33.9190	118.6270	01/19/1989	65328.8	11.9	5.00	0.059	VI	20.0(32.2)
DMG	34.3000	118.6000	04/04/1893	1940 0.0	0.0	6.00	0.131	VIII	20.5(32.9)
MGI	34.0000	118.0000	12/25/1903	1745 0.0	0.0	5.00	0.057	VI	20.8(33.4)
PAS	33.9440	118.6810	01/01/1979	231438.9	11.3	5.00	0.054	VI	21.7(34.9)
DMG	33.7830	118.2500	11/14/1941	84136.3	0.0	5.40	0.073	VII	22.0(35.3)
DMG	34.4110	118.4010	02/09/1971	14 041.8	8.4	6.40	0.160	VIII	22.3(35.9)
DMG	34.4110	118.4010	02/09/1971	14 244.0	8.0	5.80	0.100	VII	22.3(35.9)
DMG	34.4110	118.4010	02/09/1971	141028.0	8.0	5.30	0.066	VI	22.3(35.9)
DMG	34.4110	118.4010	02/09/1971	14 1 8.0	8.0	5.80	0.100	VII	22.3(35.9)
GSP	34.2620	118.0020	06/28/1991	144354.5	11.0	5.40	0.069	VI	22.9(36.9)
DMG	33.7830	118.1330	10/02/1933	91017.6	0.0	5.40	0.064	VI	24.5(39.5)
GSP	34.3780	118.6180	01/19/1994	211144.9	11.0	5.10	0.048	VI	25.2(40.5)
GSP	34.3260	118.6980	01/17/1994	233330.7	9.0	5.60	0.070	VI	25.8(41.6)
DMG	34.2000	117.9000	08/28/1889	215 0.0	0.0	5.50	0.062	VI	26.6(42.7)
GSP	34.3690	118.6720	04/26/1997	103730.7	16.0	5.10	0.045	VI	26.7(43.0)
GSP	34.3940	118.6690	06/26/1995	084028.9	13.0	5.00	0.039	V	27.9(44.9)
DMG	33.7500	118.0830	03/11/1933	323 0.0	0.0	5.00	0.039	V	28.0(45.0)
DMG	33.7500	118.0830	03/13/1933	131828.0	0.0	5.30	0.050	VI	28.0(45.0)
DMG	33.7500	118.0830	03/11/1933	910 0.0	0.0	5.10	0.042	VI	28.0(45.0)
DMG	33.7500	118.0830	03/11/1933	230 0.0	0.0	5.10	0.042	VI	28.0(45.0)
DMG	33.7500	118.0830	03/11/1933	2 9 0.0	0.0	5.00	0.039	V	28.0(45.0)
GSP	34.3770	118.6980	01/18/1994	004308.9	11.0	5.20	0.045	VI	28.2(45.3)
GSB	34.3790	118.7110	01/19/1994	210928.6	14.0	5.50	0.056	VI	28.8(46.3)
DMG	34.5190	118.1980	08/23/1952	10 9 7.1	13.1	5.00	0.034	V	30.7(49.4)
DMG	33.7000	118.0670	03/11/1933	51022.0	0.0	5.10	0.036	V	31.4(50.5)
DMG	33.7000	118.0670	03/11/1933	85457.0	0.0	5.10	0.036	V	31.4(50.5)
DMG	33.6830	118.0500	03/11/1933	658 3.0	0.0	5.50	0.047	VI	32.9(52.9)
GSP	34.1400	117.7000	02/28/1990	234336.6	5.0	5.20	0.032	V	37.1(59.7)
DMG	33.6170	118.0170	03/14/1933	19 150.0	0.0	5.10	0.029	V	37.8(60.8)
DMG	34.0000	119.0000	09/24/1827	4 0 0.0	0.0	7.00	0.131	VIII	37.9(61.1)
MGI	34.0000	119.0000	12/14/1912	0 0 0.0	0.0	5.70	0.046	VI	37.9(61.1)
DMG	33.6170	117.9670	03/11/1933	154 7.8	0.0	6.30	0.072	VI	39.3(63.2)
DMG	34.0650	119.0350	02/21/1973	144557.3	8.0	5.90	0.052	VI	39.4(63.5)
DMG	33.5750	117.9830	03/11/1933	518 4.0	0.0	5.20	0.028	V	41.3(66.4)
DMG	34.3700	117.6500	12/08/1812	15 0 0.0	0.0	7.00	0.108	VII	44.1(71.0)
DMG	34.3000	117.6000	07/30/1894	512 0.0	0.0	6.00	0.047	VI	45.0(72.4)

EARTHQUAKE SEARCH RESULTS

Page 2

FILE CODE	LAT. NORTH	LONG. WEST	DATE	TIME (UTC) H M Sec	DEPTH (km)	QUAKE MAG.	SITE ACC. g	SITE MM INT.	APPROX. DISTANCE mi [km]
MGI	33.8000	117.6000	04/22/1918	2115 0.0	0.0	5.00	0.020	IV	47.2(76.0)
DMG	34.2700	117.5400	09/12/1970	143053.0	8.0	5.40	0.027	V	47.7(76.7)
MGI	34.0000	117.5000	12/16/1858	10 0 0.0	0.0	7.00	0.095	VII	48.8(78.5)
DMG	34.3000	117.5000	07/22/1899	2032 0.0	0.0	6.50	0.061	VI	50.4(81.1)
PAS	33.6710	119.1110	09/04/1981	155050.3	5.0	5.30	0.022	IV	52.6(84.6)
DMG	34.2000	117.4000	07/22/1899	046 0.0	0.0	5.50	0.025	V	54.6(87.8)
DMG	33.6990	117.5110	05/31/1938	83455.4	10.0	5.50	0.024	V	55.0(88.5)
T-A	34.8300	118.7500	11/27/1852	0 0 0.0	0.0	7.00	0.080	VII	56.0(90.1)
DMG	34.7000	119.0000	10/23/1916	254 0.0	0.0	5.50	0.024	IV	56.2(90.4)
MGI	34.1000	117.3000	07/15/1905	2041 0.0	0.0	5.30	0.019	IV	59.8(96.2)

****

-END OF SEARCH- 63 EARTHQUAKES FOUND WITHIN THE SPECIFIED SEARCH AREA.

TIME PERIOD OF SEARCH: 1800 TO 2008

LENGTH OF SEARCH TIME: 209 years

THE EARTHQUAKE CLOSEST TO THE SITE IS ABOUT 5.0 MILES (8.0 km) AWAY.

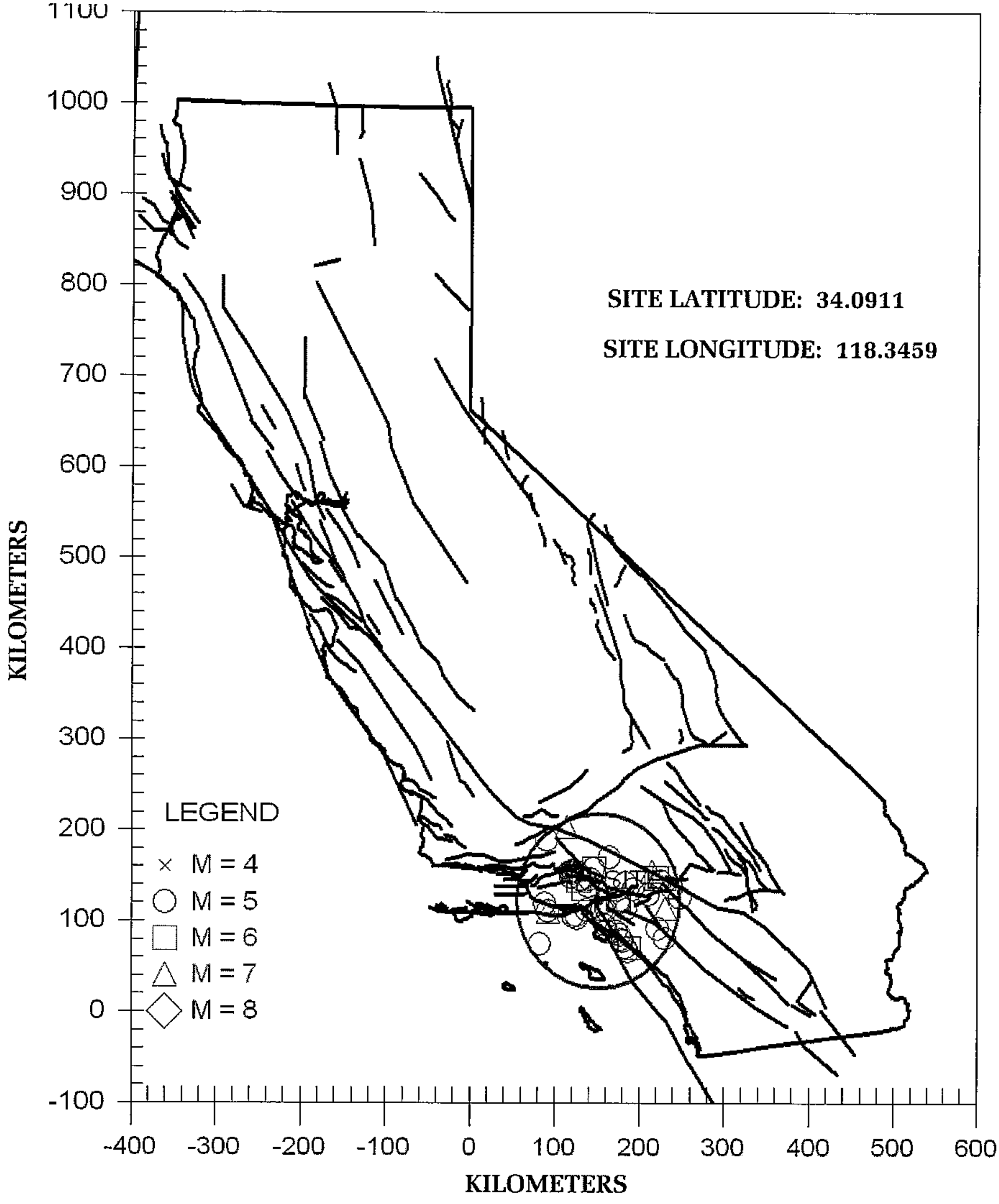
LARGEST EARTHQUAKE MAGNITUDE FOUND IN THE SEARCH RADIUS: 7.0

LARGEST EARTHQUAKE SITE ACCELERATION FROM THIS SEARCH: 0.353 g

COEFFICIENTS FOR GUTENBERG & RICHTER RECURRENCE RELATION:
a-value= 1.194
b-value= 0.391
beta-value= 0.900

TABLE OF MAGNITUDES AND EXCEEDANCES:

Earthquake Magnitude	Number of Times Exceeded	Cumulative No. / Year
4.0	63	0.30288
4.5	63	0.30288
5.0	63	0.30288
5.5	23	0.11058
6.0	11	0.05288
6.5	6	0.02885
7.0	4	0.01923



EARTHQUAKE EPICENTERS MAP

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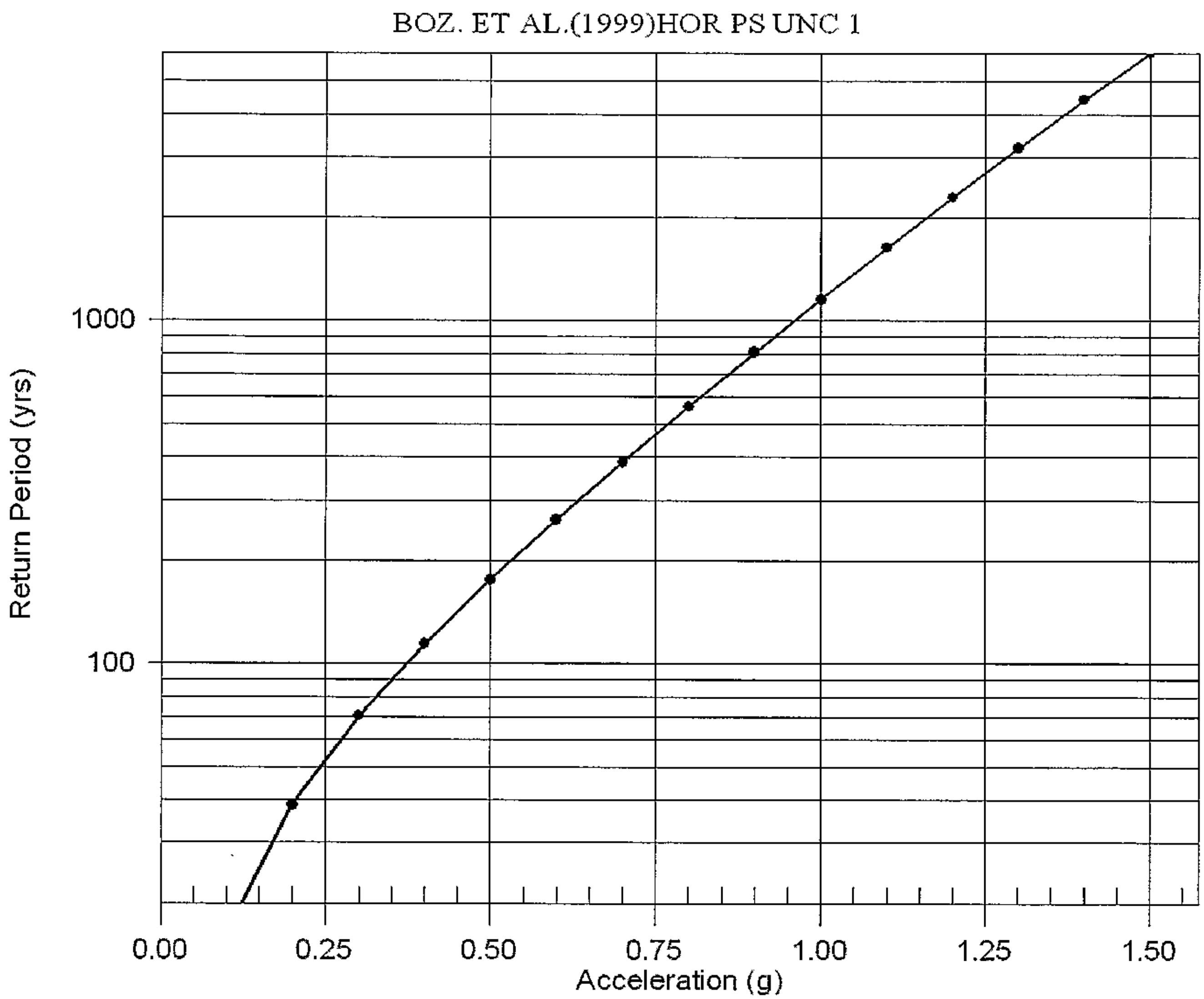
Consulting Geotechnical Engineers

HANOVER WEST, INC.

FILE No. 19079

FIGURE II



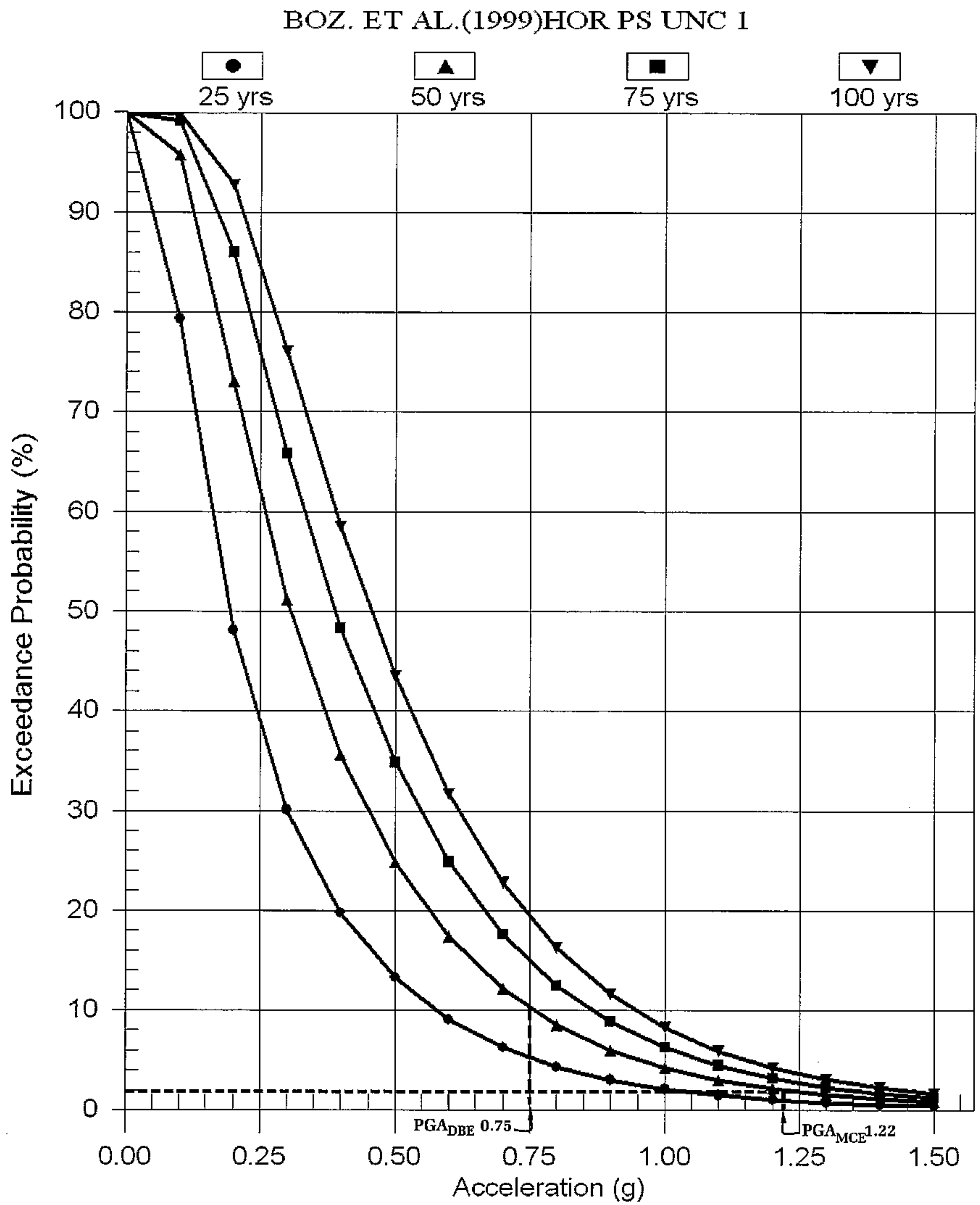
RETURN PERIOD vs ACCELERATION

Geotechnologies, Inc.
Consulting Geotechnical Engineers

HANOVER WEST, INC.

FILE No. 19079

FIGURE III



PROBABILITY OF EXCEEDANCE

Geotechnologies, Inc.
Consulting Geotechnical Engineers

HANOVER WEST, INC.

FILE No. 19079 | FIGURE IV

SCHIFF ASSOCIATES

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Consulting Corrosion Engineers – Since 1959

December 13, 2005 Fax: 818-240-9600

GEOTECHNOLOGIES, INC.
439 Western Avenue
Glendale, CA 91201.2837

Attention: Mr. Scott W. Moore
Manager of Engineering

Re: Soil Corrosivity Study
Hanover
West Hollywood, California
Geotechnologies #19079, SA #05-1740SCS

INTRODUCTION

Laboratory tests have been completed on three soil samples provided for the referenced project. The purpose of these tests was to determine if the soils might have deleterious effects on underground utility piping and concrete structures. We assume that the samples provided are representative of the most corrosive soils at the site.

The proposed project will consist of commercial buildings. Two basement levels are planned. Information transmitted with the soils states the water table is 21 feet deep.

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

LABORATORY SOIL CORROSIVITY TESTS

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

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SOIL CORROSIVITY

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

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

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

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

Electrical resistivities were in mildly and moderately corrosive categories with as received moisture. When saturated, the resistivities were corrosive and severely corrosive categories.

Soil pH values varied from 7.2 to 7.4. This range is neutral to slightly alkaline.

The soluble salt content of the samples was low and moderate.

Ammonium and nitrate were detected at levels high enough to be deleterious to copper.

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

This soil is classified as severely corrosive to ferrous metals, aggressive to copper, and negligible for sulfate attack on concrete.

CORROSION CONTROL RECOMMENDATIONS

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

Steel Pipe

Abrasive blast underground steel piping and apply a dielectric coating such as polyurethane, extruded polyethylene, a tape coating system, hot applied coal tar enamel, or fusion bonded epoxy intended for underground use.

Bond underground steel pipe with rubber gasketed, mechanical, grooved end, or other nonconductive type joints for electrical continuity. Electrical continuity is necessary for corrosion monitoring and cathodic protection.

Electrically insulate each buried steel pipeline from dissimilar metals and metals with dissimilar coatings (cement-mortar vs. dielectric), and above ground steel pipe to prevent dissimilar metal corrosion cells and to facilitate the application of cathodic protection.

Apply cathodic protection to steel piping as per NACE International Standard RP0169-2002. The amount of cathodic protection current needed can be minimized by coating the pipe.

As an alternative to dielectric coating and cathodic protection, apply a ¾-inch cement mortar coating or encase in concrete 3 inches thick, using any type of cement.

Some steel piping systems, such as for gas and oil, have special corrosion and cathodic protection requirements that must be evaluated for each specific application.

Hydraulic Elevator

Coat hydraulic elevator cylinders as described above for steel pipe. Electrically insulate each cylinder from building metals by installing dielectric material between the piston platen and car, insulating the bolts, and installing an insulated joint in the oil line. Apply cathodic protection to hydraulic cylinders as per NACE International Standard RP0169-2002. As an alternative to electrical insulation and cathodic protection, place each cylinder in a plastic casing with a plastic watertight seal at the bottom.

The elevator oil line should be placed above ground if possible but, if underground, should be protected by providing a bonded dielectric coating, electrically isolating the pipeline, and applying of cathodic protection to steel piping as per NACE International Standard RP0169-2002; or should be placed in a PVC casing pipe to prevent contact with soil and soil moisture.

Iron Pipe-Pressurized

Encase pressurized cast and ductile iron piping per AWWA Standard C105, coat with epoxy or polyurethane intended for underground use, or with wax tape per AWWA C217. The thin factory-applied asphaltic coating applied to ductile iron pipe for transportation and aesthetic purposes does not constitute a corrosion control coating. Electrically insulate underground iron pipe from dissimilar metals and from above ground iron pipe with insulating joints per NACE International Standard RP0286-2002. Bond all nonconductive type joints for electrical continuity. Apply cathodic protection to cast and ductile iron piping as per NACE International Standard RP0169-2002.

Iron Pipe-Non-Pressurized (Select one of the following alternatives for protection)

1. Polyethylene encase cast- and ductile-iron piping per AWWA Standard C105. Electrically insulate underground pipe from dissimilar metals and from above ground iron pipe with

insulating joints per NACE International Standard RP0286-2002. Protect all non-cast iron and non-ductile iron fittings and valves with wax tape per AWWA Standard C217-99 after assembly.

2. Concrete encase all buried portions of metallic piping so that there is a minimum of 3-inches of concrete cover provided over and around surfaces of pipe, fittings, and valves.
3. Apply cathodic protection to cast and ductile iron piping as per NACE International Standard RP0169-2002. The amount of cathodic protection current needed can be minimized by coating the piping.

Copper Tubing

Protect buried copper tubing by one of the following measures:

1. Prevention of soil contact. Soil contact may be prevented by routing the tubing above ground.
2. Installation of a factory coated copper pipe with a minimum of 100-mil thickness such as "Aqua Shield" or similar products. Polyethylene coating protects against elements that corrode copper and prevents contamination between copper and sleeving. However, it must be continuous with no cuts or defects if installed underground.
3. Wrapping of copper with 12-mil polyethylene pipe wrapping tape with butyl rubber mastic over a suitable primer. Protect wrapped copper tubing by applying cathodic protection per NACE International Standard RP0169-2002. The amount of cathodic protection current needed can be minimized by coating the tubing.

Plastic and Vitrified Clay Pipe

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

All Pipe

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

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

Concrete

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

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

CLOSURE

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

Please call if you have any questions.

Respectfully Submitted,
SCHIFF ASSOCIATES

Robert A. Pannell

Enc: Table 1

Reviewed by,

John W. French, P. E.

REGISTERED PROFESSIONAL ENGINEER
JOHN W. FRENCH
No. CR001023
12/31/05
CORROSION
STATE OF CALIFORNIA

Table 1 - Laboratory Tests on Soil Samples

Geotechnologies, Inc.
Hanover
Your #19079, MJS&A #05-1740SCS
5-Dec-05

Sample ID			B1 @ 2' Clay	B3 @ 7' Silt	B4 @ 20' Clay
Resistivity		**Units**			
as-received		ohm-cm	3,500	21,000	4,500
saturated		ohm-cm	1,000	1,300	1,300
pH			7.2	7.4	7.3
Electrical Conductivity		mS/cm	0.38	0.23	0.09
Chemical Analyses					
Cations					
calcium	Ca^{2+}	mg/kg	68	40	12
magnesium	Mg^{2+}	mg/kg	88	29	17
sodium	Na^{1+}	mg/kg	ND	34	17
Anions					
carbonate	CO_3^{2-}	mg/kg	ND	ND	ND
bicarbonate	HCO_3^{1-}	mg/kg	262	201	95
chloride	Cl^{1-}	mg/kg	ND	ND	ND
sulfate	SO_4^{2-}	mg/kg	128	125	57
Other Tests					
ammonium	NH_4^{1+}	mg/kg	11.6	6.0	1.3
nitrate	NO_3^{1-}	mg/kg	8.2	127.5	19.3
sulfide	S^{2-}	qual	na	na	na
Redox		mV	na	na	na

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