GEOCON
WEST, INC.
GEOTECHNICAL
ENVIRONMENTAL
MATERIALS

GEOTECHNICAL INVESTIGATION

PROPOSED MULTI-FAMILY RESIDENTIAL DEVELOPMENT
444 WEST OCEAN BOULEVARD
LONG BEACH, CALIFORNIA

PREPARED FOR

444 W. OCEAN, LLC
LONG BEACH, CALIFORNIA

PROJECT NO. A9125-06-01

MAY 14, 2014

Project No. A9125-06-01
May 14, 2014

444 W. Ocean, LLC
444 W. Ocean Blvd., Suite 1108
Long Beach, CA 90802

Attention: Mr. Kambiz Babaoff

Subject: GEOTECHNICAL INVESTIGATION
PROPOSED MULTI-FAMILY RESIDENTIAL DEVELOPMENT
444 WEST OCEAN BOULEVARD, LONG BEACH, CALIFORNIA

Dear Mr. Babaoff:

In accordance with your authorization of our proposal dated March 18, 2014, we have prepared this geotechnical investigation report for the proposed multi-family residential development located at 444 West Ocean Boulevard in the City of Long Beach, California. The accompanying report presents the findings of our study and our conclusions and recommendations pertaining to the geotechnical aspects of proposed design and construction. Based on the results of our investigation, it is our opinion that the site can be developed as proposed, provided the recommendations of this report are followed and implemented during design and construction.

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

Very truly yours,

GEOCON WEST, INC.

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GEOTECHNICAL INVESTIGATION

1. PURPOSE AND SCOPE

This report presents the results of our geotechnical investigation for a proposed multi-family residential development located at 444 West Ocean Boulevard in the City of Long Beach, California (see Vicinity Map, Figure 1). The purpose of this investigation was to evaluate subsurface soil and geologic conditions underlying the property, and based on conditions encountered, to provide conclusions and recommendations pertaining to the geotechnical aspects of proposed design and construction.

The scope of this investigation included a site reconnaissance, field exploration, laboratory testing, engineering analysis, and the preparation of this report. The site was explored on April 15, 2014 by excavating two 4⅞-inch diameter borings utilizing a mud rotary drilling machine. The borings were advanced to depths of 60½ and 62½ feet below the existing ground surface. The approximate locations of the exploratory borings are depicted on the Site Plan (Figure 2). A detailed discussion of the field investigation, including boring logs, is presented in Appendix A.

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

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

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

2. SITE AND PROJECT DESCRIPTION

The subject site is located at 444 West Ocean Boulevard in the City of Long Beach, California. The site is a rectangular shaped parcel and is currently occupied by an asphalt paved parking lot. The site is bounded by a multi-story commercial structure over three levels of podium parking to the north, by a five level parking structure with one level subterranean parking to the east, by Seaside Way to the south, and by Queens Way overpass and a parking lot to the west.

The site is relatively flat with no significant highs or lows. Surface water drainage at the site appears to be by sheet flow along the existing ground contours toward the city streets. Vegetation is nonexistent due to the paved nature of the site.

Based on the information provided to us by the Client, it is our understanding that the proposed development consists of a six-story multi-family residential structure to be constructed over three levels of podium parking. The lowest level of parking will be subterranean parking which is anticipated to extend to depths up to 12 feet below the ground surface. It is our further understanding that the proposed parking levels will connect with the existing parking levels located on the north side of the proposed site.

Based on the preliminary nature of the design at this time, wall and column loads were not available. It is anticipated that column loads for the proposed residential structure will be up to 900 kips, and wall loads are estimated to be up to 9 kips per linear foot.

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

3. GEOLOGIC SETTING

The site is located in the southern edge of the Los Angeles Basin, coastal plain between the Santa Monica Mountains to the north, the Puente Hills and Whittier faults to the east, the Palos Verdes Peninsula and Pacific Ocean to the west and south, and the Santa Ana Mountains and San Joaquin Hills to the southeast. The Los Angeles Basin is a deep structural depression which has been filled by both marine and continental sedimentary deposits over a basement complex of presumably igneous and metamorphic composition (Yerkes, et al., 1965). Regionally, the site is in the Peninsular Ranges geomorphic province characterized by northwest-trending mountains, hills, alluviated valleys, and geologic structures such as the Newport-Inglewood Fault Zone located approximately 2.7 miles to the northeast (California Division of Mines and Geology [CDMG], 1986).

4. SOIL AND GEOLOGIC CONDITIONS

Based on our field investigation and published geologic maps of the area, the soils underlying the site consist of artificial fill over young alluvial and estuarine deposits (Poland and Piper, 1956; CDMG, 1998). The soil and geologic units encountered at the site are discussed below. Detailed stratigraphic profiles are provided on the Boring Logs in Appendix A.

4.1 Artificial Fill

Artificial fill was encountered in our borings to a depth of 1½ feet below the ground surface and generally consists of light brown silty sand. The artificial fill is characterized as slightly moist and medium dense. The fill is likely the result of past grading and/or construction activities at the site. Deeper fill may occur between borings and on other parts of the site that were not directly explored.

4.2 Alluvium

The artificial fill is underlain by Holocene Age alluvium and estuarine deposits. These deposits generally consist of yellowish brown to olive brown sand, silty sand, silt, sandy silt, and sandy clay with varying amounts of shell fragments. The alluvium is primarily moist, medium dense to very dense, or stiff to hard, and becomes denser with increased depth.

5. GROUNDWATER

The historically highest groundwater level in the area is less than 10 feet beneath the ground surface (CDMG, 1998). Groundwater level information in the CDMG publication is based on data collected from the early 1900's to the late 1990's. Based on current groundwater basin management practices, it is unlikely that the groundwater levels will ever exceed the historic high levels.

Groundwater was encountered in both borings 1 and 2 at a depth of 7 feet and 7½ feet below the existing ground surface, respectively. Based on the depth of groundwater observed in our borings, groundwater will be encountered during excavation of the subterranean level. It is common 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* section of this report (see Section 7.23).

6. GEOLOGIC HAZARDS

6.1 Surface Fault Rupture

The numerous faults in Southern California include active, potentially active, and inactive faults. The criteria for these major groups are based on criteria developed by the California Geological Survey for the Alquist-Priolo Earthquake Fault Zone Program (Bryant and Hart, 2007). By definition, an active fault is one that has had surface displacement within Holocene time (about the last 11,000 years). A potentially active fault has demonstrated surface displacement during Quaternary time (approximately the last 1.6 million years), but has had no known Holocene movement. Faults that have not moved in the last 1.6 million years are considered inactive.

The site is not within a currently established Alquist-Priolo Earthquake Fault Zone for surface fault rupture hazards (Bryant and Hart, 2007). 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 3, Regional Fault Map.

The closest surface trace of an active fault to the site is the Newport-Inglewood Fault Zone located approximately 2.7 miles to the northeast (Ziony and Jones, 1989). Other nearby active faults are the Palos Verdes Hills Fault Zone, the Redondo Canyon Fault, the Whittier Fault, the Santa Monica Fault, and the Hollywood Fault located approximately 4.2 miles southwest, 12 miles west-northwest, 18 miles northeast, 23 miles northwest, and 24½ miles north of the site, respectively (Ziony and Jones, 1989). The active San Andreas Fault Zone is located approximately 50 miles northeast of the site (Ziony and Jones, 1989).

The closest potentially active fault to the site is the Los Alamitos Fault located approximately 6.4 miles to the northeast (Ziony and Jones, 1989). Other nearby potentially active faults are the Norwalk Fault, the El Modeno Fault, the Charnock Fault, the Coyote Pass Fault, the Overland Fault, and the MacArthur Park Fault located approximately 12 miles northeast, 16½ miles northeast, 18 miles north-northwest, 18 miles north, 18 miles north-northwest, and 19 miles 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. This thrust fault and other in the Los Angeles Basin are not exposed at the surface and do not present a potential surface fault rupture hazard; however, these active features are capable of generating future earthquakes.

6.2 Seismicity

As with all of Southern California, the site has experienced historic earthquakes from various regional faults. The 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 4, Regional Seismicity Map. A partial list of moderate to major magnitude earthquakes that have occurred in the Southern California area within the last 100 years is included in the following table.

LIST OF HISTORIC EARTHQUAKES

Earthquake (Oldest to Youngest)	Date of Earthquake	Magnitude	Distance to Epicenter (Miles)	Direction to Epicenter
San Jacinto-Hemet area	April 21, 1918	6.8	69	E
Near Redlands	July 23, 1923	6.3	57	ENE
Long Beach	March 10, 1933	6.4	17	SE
Tehachapi	July 21, 1952	7.5	97	NNW
San Fernando	February 9, 1971	6.6	46	N
Whittier Narrows	October 1, 1987	5.9	22	N
Sierra Madre	June 28, 1991	5.8	36	NNE
Landers	June 28, 1992	7.3	105	ENE
Big Bear	June 28, 1992	6.4	84	ENE
Northridge	January 17, 1994	6.7	37	NNW

The site could be subjected to strong ground shaking in the event of an earthquake. This hazard is common in Southern California and the effects of ground shaking can be mitigated if the proposed structures are designed and constructed in conformance with current building codes and engineering practices.

6.3 Estimation of Peak Ground Accelerations

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

6.3.1 Deterministic Analysis

Table 1 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) modeling the soil underlying the site as Site Class “D”. The Site Class determination is based on the discussion in Section 1613.3.2 of the 2013 CBC and Table 20.3-1 of ASCE 7-10. 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 7.1 event on the Newport-Inglewood Fault Zone. Such an event would be expected to generate peak horizontal accelerations at the site of 0.647g.

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

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

6.3.2 Probabilistic Analysis

The computer program *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 the 2013 California Building Code and ASCE 7-10, the MCE is to be utilized for the design of critical structures such as schools and hospitals. The Design Earthquake Ground Motion (DE) is the level of ground motion that has a 10 percent chance of exceedance in 50 years, with a statistical return period of 475 years. The DE is typically used for the design of non-critical structures.

Based on the computer program *FRISKSP* (Blake, 2000), the MCE and DE is expected to generate ground motions at the site of approximately 0.68g and 0.40g, respectively. Graphical representation of the analysis is presented on Figure 5.

6.4 Seismic Design Criteria

The following table summarizes summarizes site-specific design criteria obtained from the 2013 California Building Code (CBC; Based on the 2012 International Building Code [IBC] and ASCE 7-10), Chapter 16 Structural Design, Section 1613 Earthquake Loads. The data was calculated using the computer program *U.S. Seismic Design Maps*, provided by the USGS. The short spectral response uses a period of 0.2 second. The values presented below are for the risk-targeted maximum considered earthquake (MCE_R).

2013 CBC SEISMIC DESIGN PARAMETERS

Parameter	Value	2013 CBC Reference
Site Class	D	Table 1613.3.2
MCE_R Ground Motion Spectral Response Acceleration – Class B (short), S_S	1.613g	Figure 1613.3.1(1)
MCE_R Ground Motion Spectral Response Acceleration – Class B (1 sec), S_1	0.607g	Figure 1613.3.1(2)
Site Coefficient, F_A	1.0	Table 1613.3.3(1)
Site Coefficient, F_V	1.5	Table 1613.3.3(2)
Site Class Modified MCE_R Spectral Response Acceleration (short), S_{MS}	1.613g	Section 1613.3.3 (Eqn 16-37)
Site Class Modified MCE_R Spectral Response Acceleration – (1 sec), S_{M1}	0.911g	Section 1613.3.3 (Eqn 16-38)
5% Damped Design Spectral Response Acceleration (short), S_{DS}	1.075g	Section 1613.3.4 (Eqn 16-39)
5% Damped Design Spectral Response Acceleration (1 sec), S_{D1}	0.607g	Section 1613.3.4 (Eqn 16-40)

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

ASCE 7-10 PEAK GROUND ACCELERATION

Parameter	Value	ASCE 7-10 Reference
Mapped MCE_G Peak Ground Acceleration, PGA	0.631g	Figure 22-7
Site Coefficient, F_{PGA}	1.0	Table 11.8-1
Site Class Modified MCE_G Peak Ground Acceleration, PGA_M	0.631g	Section 11.8.3 (Eqn 11.8-1)

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

6.5 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 117, Guidelines for Analyzing and Mitigating Liquefaction in California" and "Special Publication 117A, Guidelines for Evaluating and Mitigating Seismic Hazards in California" requires liquefaction analysis to a depth of 50 feet below the lowest portion of the proposed structure. Liquefaction typically occurs in areas where the soils below the water table are composed of poorly consolidated, fine to medium-grained, primarily sandy soil. In addition to the requisite soil conditions, the ground acceleration and duration of the earthquake must also be of a sufficient level to induce liquefaction.

The site is within an area with a potential for liquefaction (Leighton, 1990; CDMG, 1999; city of Long Beach, 2004).

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. The liquefaction potential evaluation was performed by utilizing the historic high groundwater table of 7 feet below the ground surface, a magnitude 7.1 earthquake, and a peak horizontal acceleration of 0.631g (PGA_M). This semi-empirical method is based on a correlation between values of Standard Penetration Test (SPT) resistance and field performance data.

The enclosed liquefaction analyses, included herein for borings B1 and B2, indicate that the alluvial soils below the historic high groundwater depth could be prone to approximately 0.1 inches of total settlement during PGA_M ground motion (see enclosed calculation sheets, Figures 6 through 9). Differential settlement at the ground surface is anticipated to be negligible.

6.6 Slope Stability

The topography at the site is relatively level and the site is not within an area identified as having a potential for seismic slope instability (CDMG, 1999). No landslides have been identified at the site or in close proximity to the site. Also, 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.

6.7 Earthquake-Induced Flooding

Earthquake-induced flooding is inundation caused by failure of dams or other water-retaining structures due to earthquakes. The Safety Element of the Los Angeles County General Plan (Leighton, 1990) and the Public Safety Element of the Long Beach General Plan (2004), indicate that the site is not located within the inundation boundaries of upgradient dams or reservoirs. The probability of earthquake-induced flooding is considered very low.

6.8 Tsunamis, Seiches and Flooding

According to the California Geological Survey (2009), the site is located within a tsunami inundation area. Due to the presence of the Palos Verdes Peninsula, Channel Islands, and the harbor breakwater, the Long Beach coastline and harbor are somewhat protected from tsunami inundation (Woodward-Clyde Consultants, 1988). However, the harbor and coastline are vulnerable to tsunamis generated in the South Seas and offshore Southern California (Woodward-Clyde Consultants, 1988). Published estimates of recurrence intervals indicate maximum wave heights of up to 7.0 feet and 9.7 feet for 100 and 500 year recurrence intervals, respectively (Houston and Garcia, 1974) and 3.0 feet for 50 year recurrence interval (City of Long Beach, 2004). Such events are not expected to cause major damage to on-shore features. However, there is considerable potential for damage to boats, harbor facilities, and light, seafront structures during such events. Warning times of approximately 6 to 12 hours would be expected for distant events. The potential for death or injury from this source is not considered great, although shoreline property damage could be substantial (City of Long Beach, 2004).

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

The site is in an area of a 0.2% chance annual flood hazard zone (Zone X) as defined by the Federal Emergency Management Agency (FEMA, 2008).

6.9 Oil Fields and Methane Potential

Based on a review of the California Division of Oil, Gas and Geothermal Resources (DOGGR) Oil and Gas Well Location Map 131, the site is located within the boundaries of the Wilmington Oil Field. The site is located approximately 1,200 feet northwest of an oil production facility containing 11 wells belonging to Tidelands Oil Production Company and 4 wells belonging to Chevron Texaco. Tidelands Oil Production Company's wells are slanted, plugged and abandoned oil wells and Chevron Texaco's wells are completed oil wells. Due to the voluntary nature of record reporting by the oil well drilling companies, wells may be improperly located or not shown on the location map. Undocumented wells could be encountered during construction. Any wells encountered will need to be properly abandoned in accordance with the current requirements of the DOGGR.

The site is located within the boundaries of the Wilmington Oil Field. Therefore, there could be a potential for methane and other volatile gases to occur at the site which could require a permanent methane gas control system beneath the proposed buildings. Should it be determined that a methane study is required for the proposed development it is recommended that a qualified methane consultant be retained to perform the study and provide mitigation measures as necessary.

6.10 Subsidence

Subsidence occurs when a large portion of land is displaced vertically, usually due to the withdrawal of groundwater, oil, or natural gas. Subsidence commonly occurs in such small magnitudes and over such large areas that is it generally imperceptible at an individual locality. Accordingly, it affects only regionally extensive structures sensitive to slight elevation changes, such as canals and pipelines. The rate of elevation change is usually uniform over a large enough area that it does not result in differential settlements that would cause damage to individual buildings. Soils that are particularly subject to subsidence include those with high silt or clay content.

Within the Long Beach area, a substantial level of subsidence has occurred between 1926 through 1967 due to petroleum production from the Wilmington Oil Field. As much as 30 feet of subsidence has been recorded near the Navy drydock on Terminal Island between 1926 through 1967 (City of Long Beach, 2004).

As of 1958 local agencies began full-scale-water injection operations to impede further subsidence within the within the Long Beach area. In addition, subsidence is continually monitored by a network of 5 microearthquake monitoring stations that have been in operation since 1971 (City of Long Beach, 2004). As a result no further manifestation of subsidence has occurred in the area since the implementation of this system. As long as the water injection operations are implemented and the ground surface is monitored to control elevation changes, the potential for subsidence to impact the proposed development is low.

7. CONCLUSIONS AND RECOMMENDATIONS

7.1 General

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

7.1.2 Up to 1½ feet of existing artificial fill was encountered during site exploration. The existing fill encountered is believed to be the result of past grading and construction activities at the site. Deeper fill may exist in other areas of the site that were not directly explored. Excavations for the subterranean level are anticipated to penetrate through the existing artificial fill and expose undisturbed granular alluvial soils throughout the excavation bottom.

7.1.3 The enclosed liquefaction analyses and seismically-induced settlement calculations indicate that the site soils below the lowest subterranean level could be prone to approximately 0.1 inch of total settlement as a result of the DBE ground motion. The differential settlement at the lowest subterranean level is anticipated to be negligible. The foundation recommendations presented herein are intended to minimize the potential for differential settlement.

7.1.4 Groundwater was encountered in both borings at depths between 7 and 7½ feet below existing ground surface. Excavation for the lowest subterranean level is anticipated to extend to depths of at least 12 feet below ground surface. Therefore, based on conditions encountered at the time of exploration, 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. If the subterranean portion of the structure is not designed for full hydrostatic pressure, a permanent dewatering system will be required to relieve and mitigate the water pressure. Recommendations for temporary and permanent dewatering are discussed in Section 7.4 and Section 7.5 of this report.

7.1.5 It is recommended that a reinforced concrete mat foundation system be utilized for support of the proposed structure provided foundations derive support in the competent alluvial soils found at or below the proposed subterranean level. The mat foundation system allows for more efficient construction when performed in conjunction with the waterproofing installation Recommendations for foundation design are provided in Section 7.7 of this report.

7.1.6 If a permanent dewatering system is not implemented then the structure should be designed for hydrostatic pressure based on the historic high groundwater level of 5 feet below the ground surface. The hydrostatic design will result in uplift forces on the structure that that must be resisted by the building weight and/or structural design.

7.1.7 The proposed structure may bear directly in the competent undisturbed alluvial soils. If any wet or disturbed soils are present in the excavation bottom, the operation of rubber tire equipment on these soils may cause excessive disturbance of the soils. Excavation activities to establish the finished subgrade elevation must be conducted carefully and methodically to avoid excessive disturbance to the subgrade. Track equipment should be considered for these construction activities. If wet or soft soils are encountered, stabilization of the bottom of the excavation may be required in order to provide a firm working surface upon which heavy equipment can operate. Recommendations for bottom stabilization and earthwork are provided in the *Grading* section of this report (see Section 7.6).

7.1.8 Excavations on the order of 14 feet in vertical height may be required for excavation of the subterranean level including the dewatering system. 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 level will require shoring measures in order to provide a stable excavation. Where shoring is required it is recommended that a soldier pile shoring system be utilized. Recommendations for shoring are provided in Section 7.18 of this report.

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

7.1.10 Foundations for small outlying structures, such as block walls less than 6 feet high, planter walls or trash enclosures, which will independent of the proposed structure, 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 1½ feet 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.

7.1.11 Based on the relatively high groundwater level at the subject site, a storm water infiltration system is not recommended for this development. It is suggested that storm water be retained, filtered and discharged in accordance with the requirements of the local governing agency.

7.1.12 Once the design and foundation loading configuration proceeds to a more finalized plan, the recommendations within this report should be reviewed and revised, if necessary. If the proposed building loads will exceed those presented herein, the potential for settlement should be reevaluated by this office.

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

7.2 Soil and Excavation Characteristics

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

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

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

7.2.4 The soils encountered at the lowest subterranean levels are primarily granular in nature and are considered to be "non-expansive". The recommendations presented in this report assume that foundations and slabs will derive support in these materials.

7.3 Minimum Resistivity, pH and Water-Soluble Sulfate

7.3.1 Potential of Hydrogen (pH) and resistivity testing as well as chloride content testing were performed on representative samples of soil to generally evaluate the corrosion potential to surface utilities. The tests were performed in accordance with California Test Method Nos. 643 and 422 and indicate that a potential for corrosion of buried ferrous metals exists on site. The results are presented in Appendix B (Figure B6) and should be considered for design of underground structures.

7.3.2 Laboratory tests were performed on representative samples of the site materials to measure the percentage of water-soluble sulfate content. Results from the laboratory water-soluble sulfate tests are presented in Appendix B (Figure B6) and indicate that the on-site materials possess "negligible" sulfate exposure to concrete structures as defined by 2013 CBC Section 1904 and ACI 318-11 Section 4.2 and 4.3.

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

7.4 Temporary Dewatering

7.4.1 Groundwater was observed during site exploration at depths of 7 and 7½ feet below ground surface. The depth to groundwater at the time of construction can be further verified during initial dewatering well or shoring pile installation. If groundwater is present above the depth of the subterranean level, temporary dewatering will be necessary to maintain a safe working environment during excavation and construction activities.

7.4.2 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.

7.4.3 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.

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

7.5 Permanent Dewatering

7.5.1 If a permanent dewatering system is not implemented then the structure must be designed for hydrostatic pressure based on the historic high groundwater level of 5 feet below the ground surface. If permanent dewatering is to be utilized, a sub-slab drainage system consisting of perforated pipes placed in gravel-filled trenches may be installed beneath the subterranean slab-on-grade to intercept and control groundwater. A separate retaining wall drainage system is also required around the perimeter of the structure. The sub-slab drainage system can be combined with the perimeter retaining wall drainage system provided backflow valves are installed at the base of the wall drainage system.

7.5.2 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.

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

7.6 Grading

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

7.6.2 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.

7.6.3 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.

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

7.6.5 The concrete foundation ramp for the subterranean portion of the proposed structure may bear directly on the competent undisturbed alluvial soils at the excavation bottom. Any disturbed soils should be either removed or properly compacted for slab support.

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

7.6.7 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.

7.6.8 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.).

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

7.6.10 Where new surface paving is to be placed, it is recommended that all existing fill and soft or unsuitable alluvial soils be excavated and properly compacted for paving support. The client should be aware that excavation and compaction of all soft or unsuitable alluvial soils in the area of new paving is not required, however, paving constructed over existing unsuitable soils may experience increased settlement and/or cracking, and may therefore have a shorter design life and increased maintenance costs. As a minimum, the upper 12 inches of the subgrade should be moisture conditioned to optimum moisture content and properly compacted to at least 95 percent relative compaction, as determined by ASTM Test Method D1557 (latest edition).

7.6.11 Foundations for small outlying structures, such as block walls up to 6 feet high, 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 proper compaction cannot be performed or is undesirable, foundations may derive support directly in the undisturbed alluvial soils generally found at a depth of 1½ feet below the existing ground surface, and should be deepened as necessary to maintain a minimum 12 inch embedment into undisturbed alluvium. If the alluvial soils exposed in the excavation bottom are loose or disturbed, compaction of the soils will be required prior to placing steel or concrete. Compaction of the foundation excavation bottom is typically accomplished with a compaction wheel or mechanical whacker and must be observed and approved by a Geocon representative.

7.6.12 Although not anticipated for this project, all imported fill shall be observed, tested, and approved by Geocon West, Inc. prior to bringing soil to the site. Rocks larger than six inches in diameter shall not be used in the fill. If necessary, import soils used as structural fill should have an expansion index less than 20 and soil corrosivity properties that are equally or less detrimental to that of the existing onsite soils (see Figure B5).

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

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

7.7 Foundation Design – Mat Foundation System

7.7.1 Subsequent to the recommended grading, a reinforced concrete mat foundation may be utilized for support of the proposed structure. The mat foundation should derive support in the competent dense alluvial soils or stabilized subgrade.

7.7.2 The recommended maximum allowable bearing value is 4,000 pounds per square foot. The allowable bearing pressure may be increased by up to one-third for transient loads due to wind or seismic forces.

7.7.3 It is recommended that a modulus of subgrade reaction of 220 pounds per cubic inch (pci) be utilized for the design of the mat foundation bearing on the competent alluvial soils or stabilized subgrade. This value is a unit value for use with a one-foot square footing. The modulus should be reduced in accordance with the following equation when used with larger foundations:

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

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

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

7.7.5 For seismic design purposes, a coefficient of friction of 0.35 may be utilized between concrete slab and undisturbed alluvial soils and/or new placed engineered fill without a vapor retarder, and 0.15 for slabs underlain by a vapor retarder.

7.7.6 If the proposed structure is to be designed for full hydrostatic pressure, the recommended floor slab uplift pressure to be used in design would be 62.4(H) in units of pounds per square foot, where "H" is the height of the water above the bottom of the mat foundation in feet. If a permanent dewatering system is not implemented then the structure must be designed for hydrostatic pressure based on the groundwater at 5 feet below ground surface.

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

7.7.8 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 exposed soil conditions are consistent with those anticipated. If unanticipated soil conditions are encountered, foundation modifications may be required.

7.7.9 The maximum expected static settlement for proposed improvements supported on a mat foundation system deriving support in competent alluvium or stabilized subgrade 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. Once the design and foundation loading configuration for the proposed structure proceeds to a more finalized plan, the estimated settlements presented in this report should be reviewed and revised, if necessary. If the final foundation loading configurations are greater than the assumed loading conditions, the potential for settlement should be reevaluated by this office.

7.8 Miscellaneous Foundations

7.8.1 Foundations for small outlying structures, such as block walls up to 6 feet high, planter walls or trash enclosures, which will not be tied-in to the proposed structure, may be supported on conventional foundations bearing on a minimum of 12 inches of newly placed engineered fill. Where excavation and compaction cannot be performed, foundations may bear in the undisturbed alluvial soils. If the soils exposed in the excavation bottom are loose, compaction of the soils will be required prior to placing steel or concrete. Compaction of the foundation excavation bottom is typically accomplished with a compaction wheel or mechanical whacker and must be observed and approved by a Geocon representative. Miscellaneous foundations may be designed for a bearing value of 1,500 pounds per square foot, and should be a minimum of 12 inches in width, 24 inches in depth below the lowest adjacent grade and 12 inches into the recommended bearing material. The allowable bearing pressure may be increased by up to one-third for transient loads due to wind or seismic forces.

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

7.9 Lateral Design

7.9.1 Resistance to lateral loading may be provided by friction acting at the base of foundations, slabs and by passive earth pressure. An allowable coefficient of friction of 0.35 may be used with the dead load forces in the undisturbed alluvial soils or stabilized subgrade.

7.9.2 Passive earth pressure for the sides of foundations and slabs poured against undisturbed alluvial soils or stabilized subgrade may be computed as an equivalent fluid having a density of 300 pcf with a maximum earth pressure of 3,000 pcf. When combining passive and friction for lateral resistance, the passive component should be reduced by one-third.

7.10 Concrete Slabs-on-Grade

7.10.1 Unless specifically evaluated and designed by a qualified structural engineer, the slab-on-grade and ramp subject to vehicle loading that is not part of the mat foundation system should be a minimum of 6 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 and ramp may derive support in the undisturbed alluvial soils and/or engineered fill and the upper 12 inches of subgrade soils directly beneath the ramp should be properly compacted to at least 95 percent relative compaction, as determined by ASTM Test Method D 1557 (latest edition) for ramp support.

7.10.2 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 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. As an alternative to clean aggregates suggested, it is our opinion that the concrete slab-on-grade may be underlain by a vapor retarded over 4-inches of clean sand (sand equivalent greater than 30), since the sand will serve a capillary break and will minimize the potential for punctures and damage to the vapor barrier.

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

7.10.4 For seismic design purposes, a coefficient of friction of 0.35 may be utilized between concrete slabs and subgrade soils without a vapor retarder, and 0.15 for slabs underlain by a vapor retarder.

7.10.5 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 moistened to near optimum moisture content and properly compacted to at least 95 percent relative compaction, as determined by ASTM Test Method D 1557 (latest edition). Crack control joints should be spaced at intervals not greater than 12 feet and should be constructed using saw-cuts or other methods as soon as practical following concrete placement. Crack control joints should extend a minimum depth of one-fourth the slab thickness. The project structural engineer should design construction joints as necessary.

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

7.11 Preliminary Pavement Recommendations

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

7.11.2 The following pavement sections are based on an assumed R-Value of 20. Once site grading activities are complete an R-Value should be obtained by laboratory testing to confirm the properties of the soils serving as paving subgrade, prior to placing pavement. This is especially important where import soils are utilized in proposed parking areas. Pavement thicknesses were determined following procedures outlined in the *California Highway Design Manual* (Caltrans). It is anticipated that the majority of traffic will consist of automobile traffic.

PRELIMINARY PAVEMENT DESIGN SECTIONS

Location	Estimated Traffic Index (TI)	Asphalt Concrete (inches)	Class 2 Aggregate Base (inches)
Automobile Parking and Driveways	5.0	3.0	7.0
Trash Truck & Fire Lanes	7.0	4.0	12.5

7.11.3 Asphalt concrete should conform to Section 203-6 of the "*Standard Specifications for Public Works Construction*" (Green Book). Class 2 aggregate base materials should conform to Section 26-1.02A of the "*Standard Specifications of the State of California, Department of Transportation*" (Caltrans).

7.11.4 Unless specifically designed and evaluated by the project structural engineer, where concrete paving will be utilized for support of vehicles, it is recommended that the concrete be a minimum of 6 inches of 4,000 psi Portland cement concrete reinforced with No. 3 steel reinforcing bars placed 18 inches on center in both horizontal directions. Concrete paving supporting vehicular traffic should be underlain by a minimum of 4 inches of aggregate base and a properly compacted subgrade. The subgrade and base material should be compacted to at least 92 and 95 percent relative compaction, respectively, as determined by ASTM Test Method D 1557 (latest edition).

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

7.12 Retaining Walls

7.12.1 The recommendations presented below are generally applicable to the design of rigid concrete or masonry retaining walls having a maximum height of 12 feet. In the event that walls higher than 12 feet are planned, Geocon should be contacted for additional recommendations.

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

7.12.3 Retaining walls with a level backfill surface that are not restrained at the top should be designed utilizing a triangular distribution of pressure (active pressure) of 30 pcf.

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

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

7.12.6 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.

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

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

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

and

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

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

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

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

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

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

and

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

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

then

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

where x is the distance from the face of the excavation to the vertical point-load, H is distance from the outrigger/bottom of column footing to the bottom of excavation, z is the depth at which the horizontal pressure is desired, Q_P is the vertical point-load, σ is the vertical pressure at depth z, Θ is the angle between a line perpendicular to the bulkhead and a line from the point-load to half the pile spacing at the bulkhead, and σ_H is the horizontal pressure at depth z.

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

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

7.13 Dynamic (Seismic) Lateral Forces

7.13.1 In accordance with the 2013 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 pressure. The structural engineer should determine the seismic design category for the project. The maximum dynamic active pressure is equal to the sum of the initial static pressure and the dynamic (seismic) pressure increment.

7.13.2 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).

7.13.3 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 seismic load of 26 pounds per cubic foot be used for design applied as a triangular distribution of pressure 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 ground acceleration of 0.43 (S_{DS}/2.5 ground motion) and by applying a pseudo-static coefficient of 0.5.

7.14 Retaining Wall Drainage

7.14.1 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 10). The clean bottom and subdrain pipe, behind a retaining wall, should be observed by the Geotechnical Engineer (a representative of Geocon), prior to placement of gravel or compacting backfill.

7.14.2 As an alternative, a plastic drainage composite such as Miradrain or equivalent may be installed in continuous, 4-foot wide columns along the entire back face of the wall, at 8 feet on center. The top of these drainage composite columns should terminate approximately 18 inches below the ground surface, where either hardscape or a minimum of 18 inches of relatively cohesive material should be placed as a cap (see Figure 11). These vertical columns of drainage material would then be connected at the bottom of the wall to a collection panel or a one-cubic-foot rock pocket drained by a 4-inch subdrain pipe.

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

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

7.15 Elevator Pit Design

7.15.1 The elevator pit slab and retaining wall should be designed by the project structural engineer. Elevator pit walls may be designed in accordance with the recommendations in the *Foundation Design and Retaining Wall Design* section of this report (see Sections 7.7 and 7.12).

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

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

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

7.16 Elevator Piston

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

7.16.2 Caving is expected and the contractor should be prepared to use casing and should have it readily available at the commencement of drilling activities. Continuous observation of the drilling and installation of the elevator piston by the Geotechnical Engineer (a representative of Geocon West, Inc.) is required.

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

7.17 Temporary Excavations

7.17.1 Excavations on the order of 14 feet in height may be required for excavation and construction of the proposed subterranean level and dewatering measures. The excavations are expected to expose artificial fill and alluvial soils, which may be subject to excessive caving. Vertical excavations up to five feet in height may be attempted where not surcharged by adjacent traffic or structures; however, the contractor should be prepared for caving sands in open excavations.

7.17.2 Vertical excavations greater than five feet will require sloping and/or shoring measures in order to provide a stable excavation. Where sufficient space is available, temporary unsurcharged embankments up to 8 feet high could be sloped back at a uniform 1:1 slope gradient or flatter. A uniform slope does not have a vertical portion. Where space is limited, shoring measures will be required. *Shoring* data is provided in Section 7.18 of this report.

7.17.3 Where 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.

7.18 Shoring – Soldier Pile Design and Installation

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

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

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

7.18.4 The proposed soldier piles may also be designed as permanent piles. The required pile depth, dimension, spacing and underpinning connection to existing offsite foundation should be determined and designed by the project structural and shoring engineers. All piles utilized for shoring can also be incorporated into a permanent retaining wall system (shotcrete wall) and should be designed in accordance with the earth pressure provided in the *Retaining Walls* section of this report (see Section 7.12).

7.18.5 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 135 pounds per square foot per foot, for the portion of the pile below the water table (these values have been adjusted for buoyant forces). The allowable passive value may be doubled for isolated piles, spaced a minimum of three times the pile diameter. To develop the full lateral value, provisions should be implemented to assure firm contact between the soldier piles and the undisturbed alluvium.

7.18.6 Casing will likely be required since caving is expected in the granular soils, and the contractor should have casing available prior to commencement of drilling activities. When casing is used, extreme care should be employed so that the pile is not pulled apart as the casing is withdrawn. At no time should the distance between the surface of the concrete and the bottom of the casing be less than five feet.

Continuous observation of the drilling and pouring of the piles by the Geotechnical Engineer (a representative of Geocon West, Inc.), is required.

7.18.7 The frictional resistance between the soldier piles and retained soil may be used to resist the vertical component of the load. The coefficient of friction may be taken as 0.35 based on uniform contact between the steel beam and lean-mix concrete and alluvium. The portion of soldier piles below the plane of excavation may also be employed to resist the downward loads. The downward capacity may be determined using a frictional resistance of 750 pounds per square foot for the portion of the pile above the water table, and as 550 pounds per square foot per foot for the portion of the pile below the water table (value has been reduced for buoyant forces).

7.18.8 Groundwater was encountered at depths of 7 and 7½ feet during exploration. The contractor should be prepared for groundwater during pile installation. Piles placed below the water level require the use of a tremie to place the concrete into the bottom of the hole. A tremie should consist of a rigid, water-tight tube having a diameter of not less than 6 inches with a hopper at the top. The tube should be equipped with a device that will close the discharge end and prevent water from entering the tube while it is being charged with concrete. The tremie should be supported so as to permit free movement of the discharge end over the entire top surface of the work and to permit rapid lowering when necessary to retard or stop the flow of concrete. The discharge end should be closed at the start of the work to prevent water entering the tube and should be entirely sealed at all times, except when the concrete is being placed. The tremie tube should be kept full of concrete. The flow should be continuous until the work is completed and the resulting concrete seal should be monolithic and homogeneous. The tip of the tremie tube should always be kept about 5 feet below the surface of the concrete and definite steps and safeguards should be taken to insure that the tip of the tremie tube is never raised above the surface of the concrete.

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

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

7.18.11 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.

7.18.12 For design of shoring, it is recommended that an equivalent fluid pressure based on the following table, be utilized for design.

HEIGHT OF SHORING (FEET)	EQUIVALENT FLUID PRESSURE (Pounds Per Cubic Foot) (ACTIVE PRESSURE)	EQUIVALENT FLUID PRESSURE (Pounds Per Cubic Foot) (AT-REST PRESSURE)
Up to 14	25	45

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

7.18.14 Where a combination of sloped embankment and shoring is utilized, the pressure will be greater and must be determined for each combination. Additional active pressure should be added for a surcharge condition due to sloping ground, vehicular traffic, or adjacent structures and must be determined for each combination.

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

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

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

and

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

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

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

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

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

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

and

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

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

then

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

where x is the distance from the face of the excavation to the vertical point-load, H is distance from the outrigger/bottom of column footing to the bottom of excavation, z is the depth at which the horizontal pressure is desired, Q_p is the vertical point-load, σ is the vertical pressure at depth z, Θ is the angle between a line perpendicular to the bulkhead and a line from the point-load to half the pile spacing at the bulkhead, and σ_H is the horizontal pressure at depth z.

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

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

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

7.19 Tie-Back Anchors

7.19.1 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.

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

- Up to 5 feet below the top of the excavation – 500* pounds per square foot
  (*reduced for buoyant forces)

7.19.3 An allowable friction capacity of 2 kips per linear foot may be utilized for a 20 foot length beyond the active wedge. Additional tieback length will yield higher capacity. The maximum allowable friction capacity is 3 kips per linear foot. Only the frictional resistance developed beyond the active wedge should be utilized in resisting lateral loads.

7.20 Anchor Installation

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

7.21 Anchor Testing

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

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

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

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

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

7.22 Internal Bracing

7.22.1 Rakers may be utilized to brace the soldier piles in lieu of tieback anchors. The raker bracing could be supported laterally by temporary concrete footings (deadmen) or by the permanent, interior footings. For design of such temporary footings or deadmen, poured with the bearing surface normal to rakers inclined at 45 degrees, a bearing value of 2,000 pounds per square foot in competent alluvial soil, 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.

7.23 Surface Drainage

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

7.23.2 All site drainage should be collected and controlled in non-erosive drainage devices. Drainage should not be allowed to pond anywhere on the site, and especially not against any foundation or retaining wall. The site should be graded and maintained such that surface drainage is directed away from structures in accordance with 2013 CBC 1804.3 or other applicable standards. In addition, drainage should not be allowed to flow uncontrolled over any descending slope. Discharge from downspouts, roof drains and scuppers are not recommended onto unprotected soils within five feet of the building perimeter. Planters which are located adjacent to foundations should be sealed to prevent moisture intrusion into the soils providing foundation support. Landscape irrigation is not recommended within five feet of the building perimeter footings except when enclosed in protected planters.

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

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

7.23.5 Based on the relatively high groundwater level at the subject site, a storm water infiltration system is not recommended for this development. It is suggested that storm water be retained, filtered and discharged in accordance with the requirements of the local governing agency.

7.24 Plan Review

7.24.1 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.

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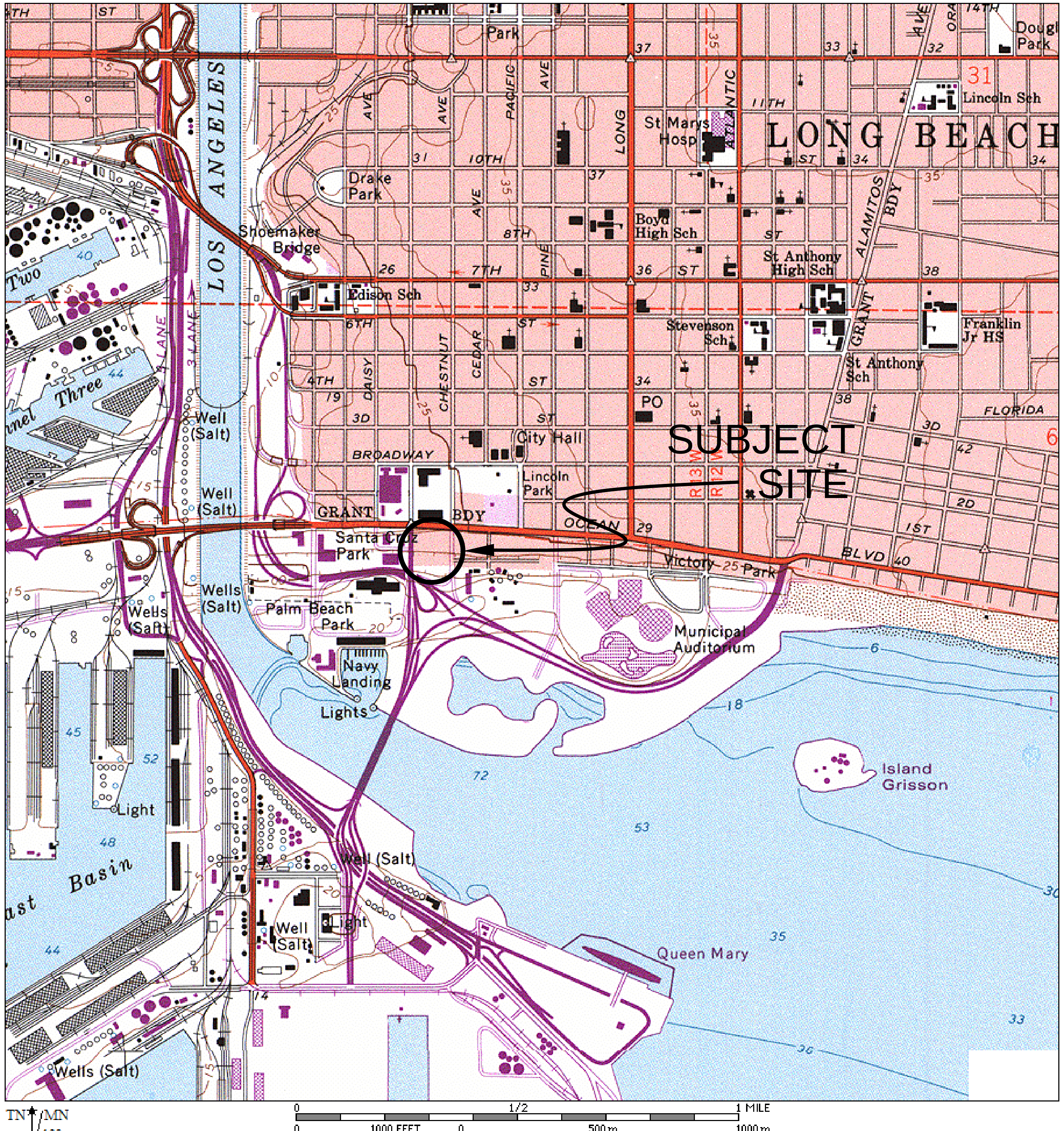

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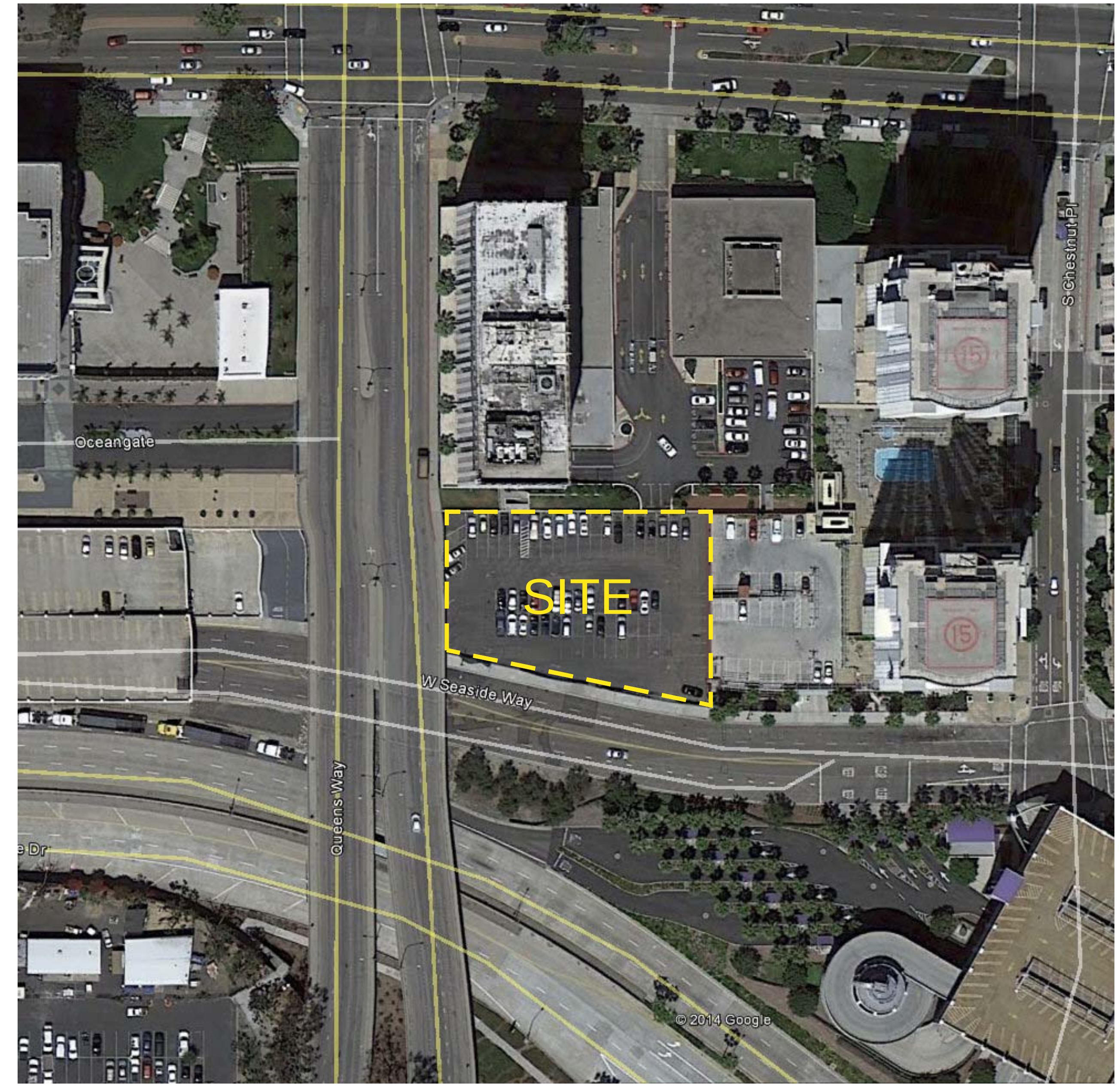
REFERENCE:U.S.G.S. TOPOGRAPHIC MAPS, 7.5 MINUTE SERIES, LONG BEACH, CA QUADRANGLE



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

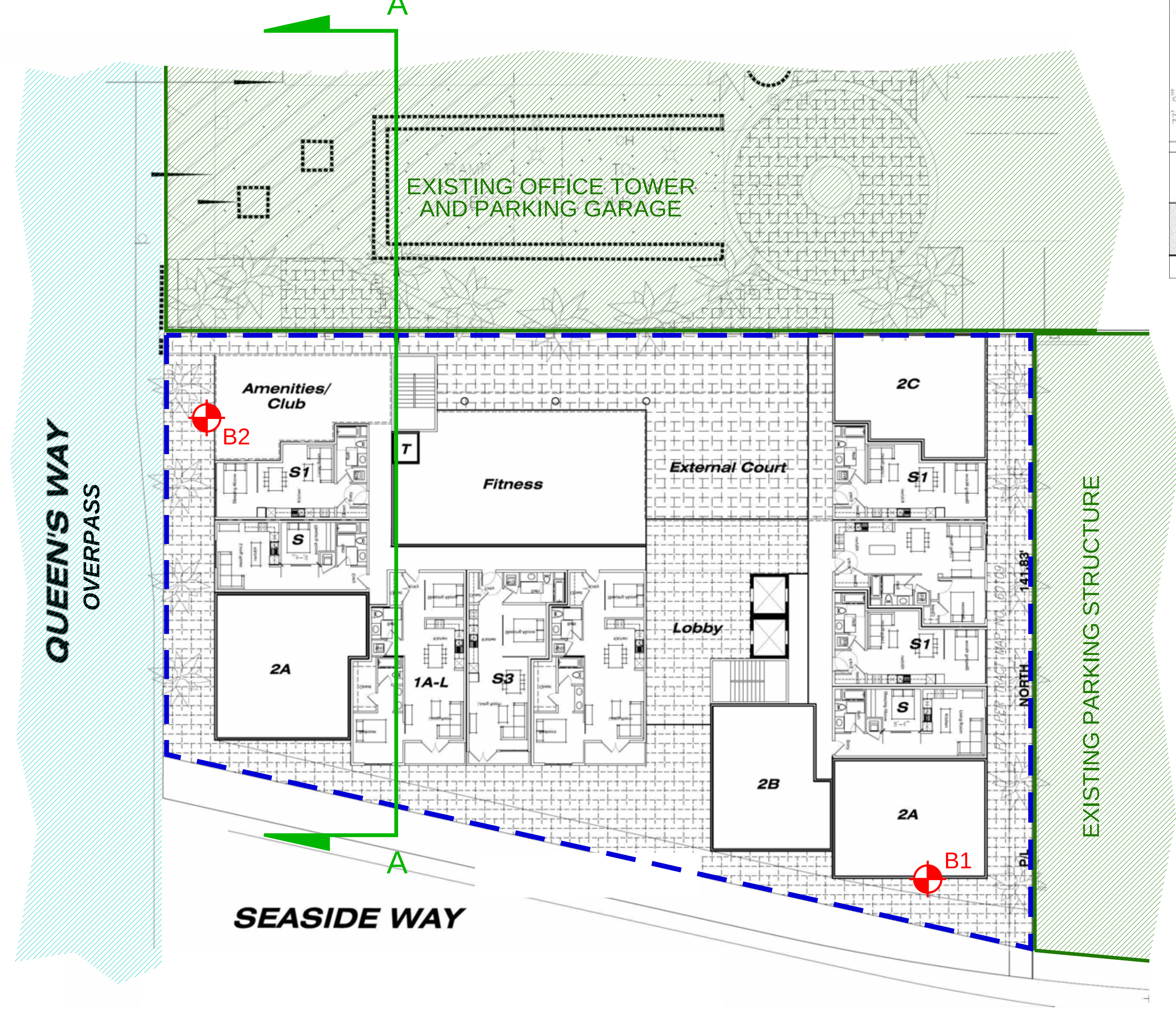
PZ		9000

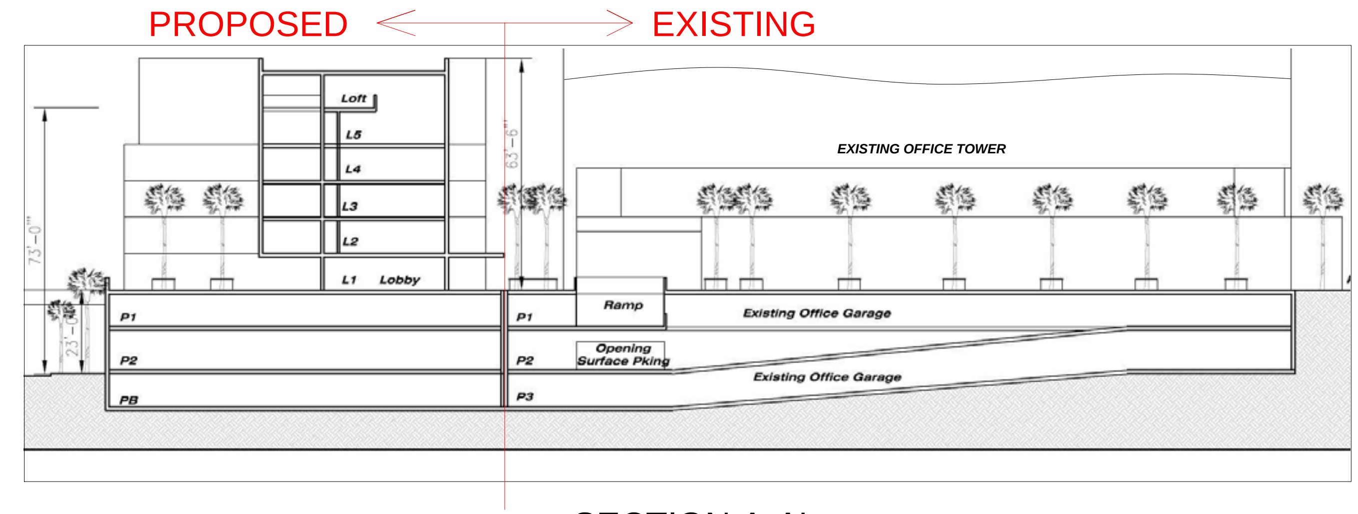
VICINITY MAP

444 W OCEAN, LLC
444 WEST OCEAN BOULEVARD
LONG BEACH, CALIFORNIA

MAY 14, 2014	PROJECT NO. A9125-06-01	FIG. 1

PLANS BY: STUDIO T-SQ





SECTION A-A'
NO SCALE

LEGEND

- B2 Approximate Location of Boring
- Approximate Location of Proposed Structure
- Approximate Location of Off Site Structures
- Approximate Location of Street Overpass

0 30' 60'

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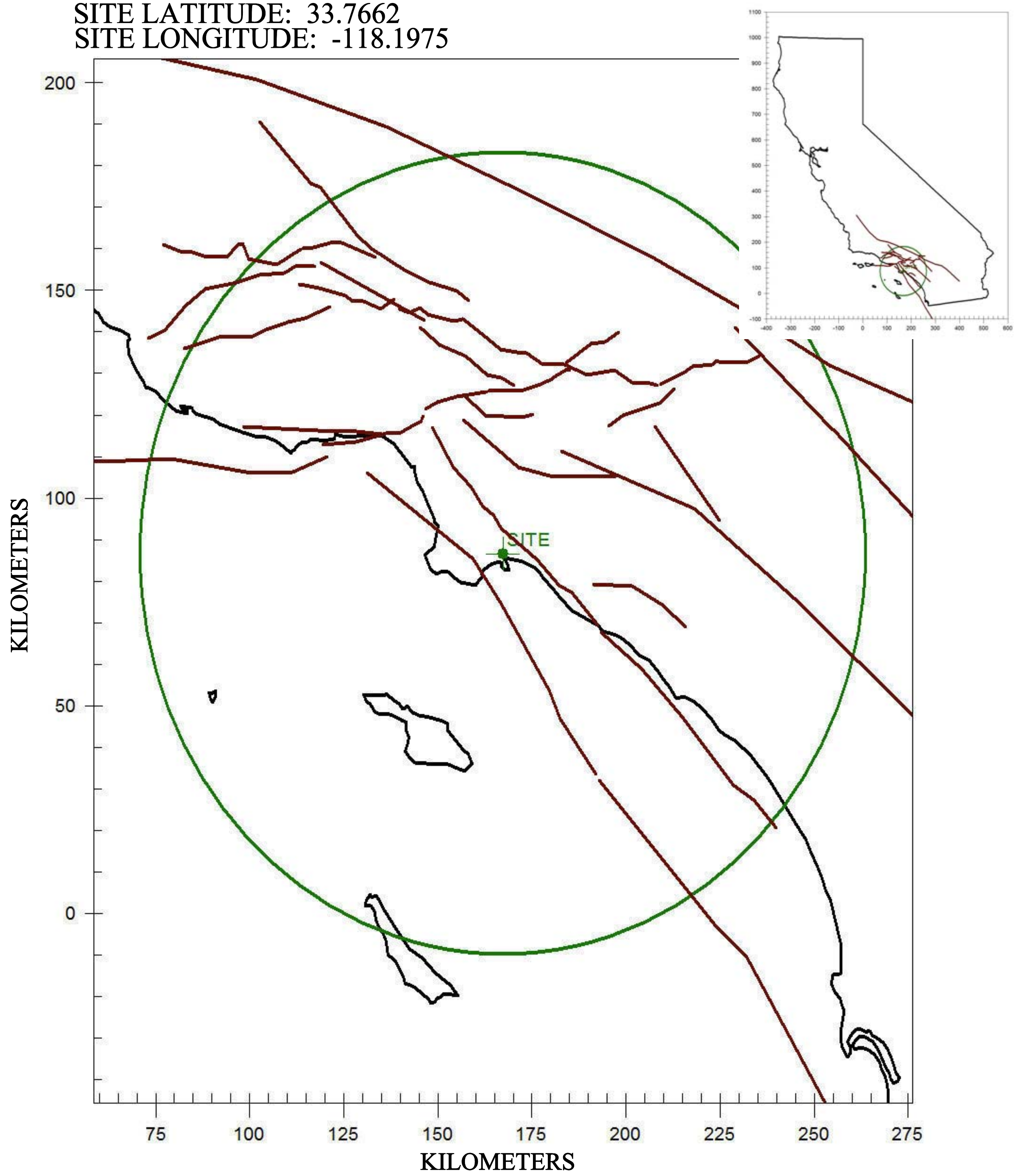
SITE PLAN

444 W OCEAN, LLC
444 WEST OCEAN BOULEVARD
LONG BEACH, CALIFORNIA

MAY 14, 2014	PROJECT NO. A9125-06-01	FIG. 2

SITE LATITUDE: 33.7662
SITE LONGITUDE: -118.1975



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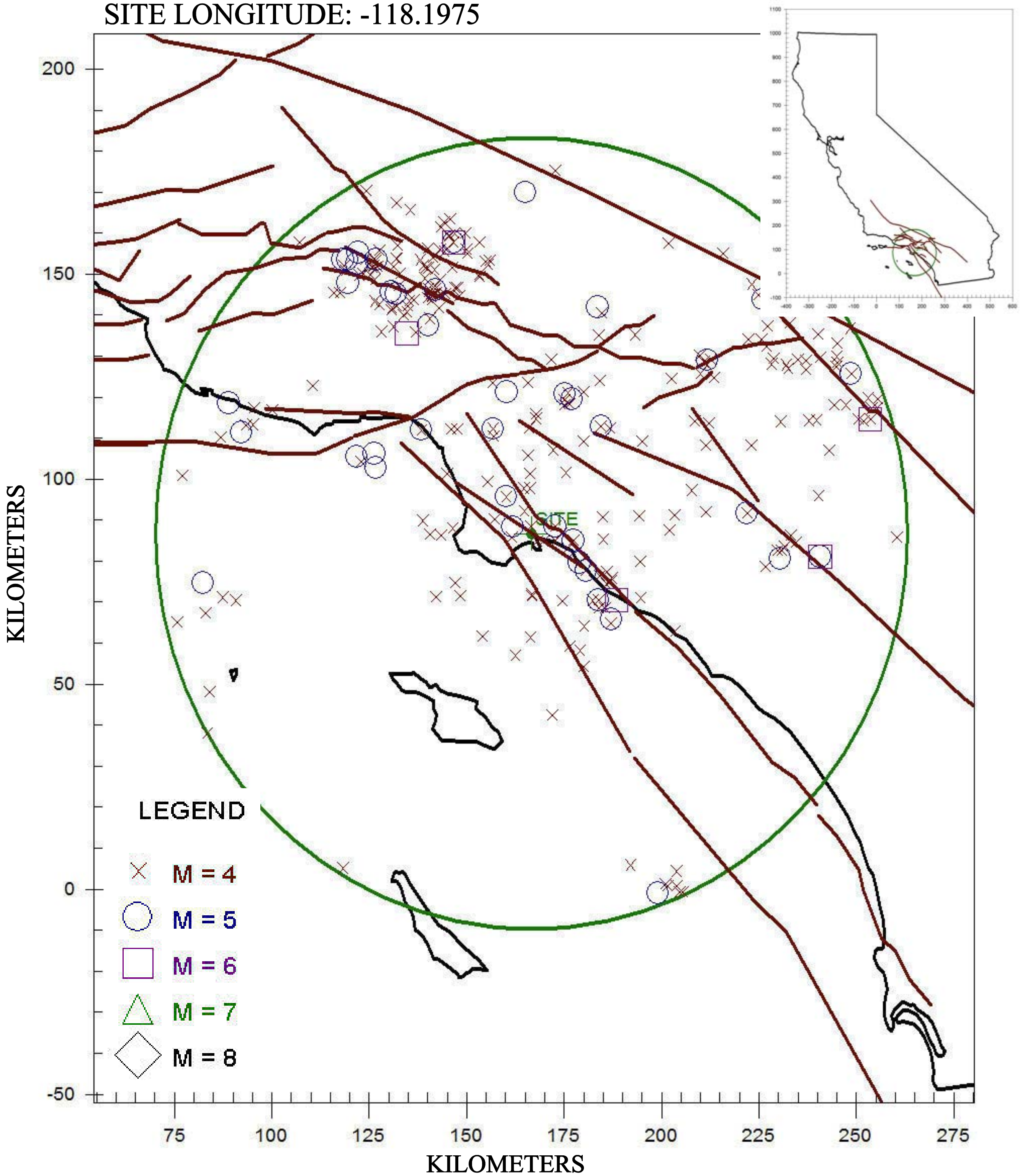
AL		9000

REGIONAL FAULT MAP

444 W OCEAN, LLC
444 WEST OCEAN BOULEVARD
LONG BEACH, CALIFORNIA

MAY 14, 2014	PROJECT NO. A9125-06-01	FIG. 3

SITE LATITUDE: 33.7662
SITE LONGITUDE: -118.1975



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REGIONAL SEISMICITY MAP

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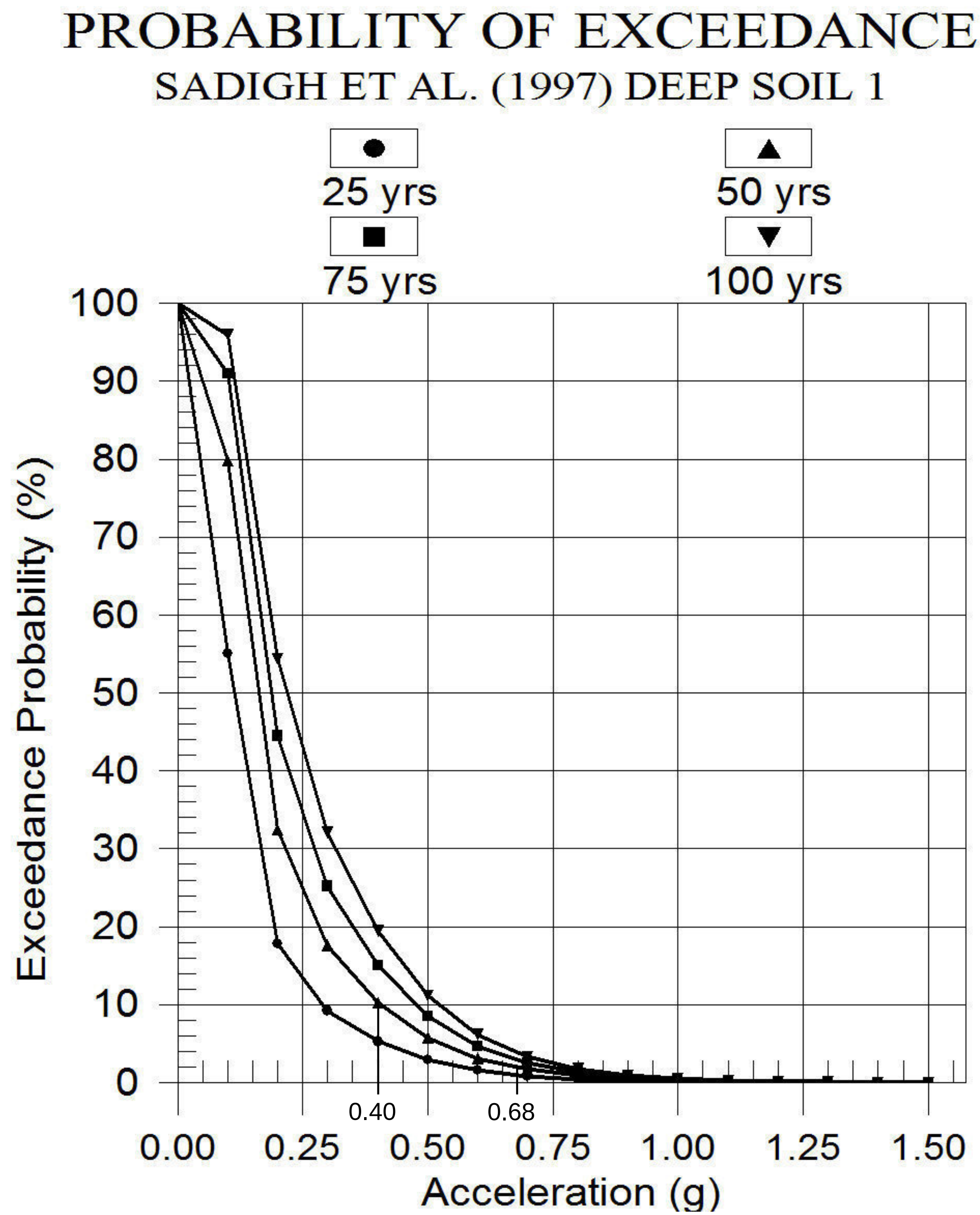
LONG BEACH, CALIFORNIA

AL		9000	MAY 14, 2014	PROJECT NO. A9125-06-01	FIG. 4



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PROBABILITY OF EXCEEDANCE

444 W OCEAN, LLC
444 WEST OCEAN BOULEVARD
LONG BEACH, CALIFORNIA

MAY 14, 2014	PROJECT NO. A9125-06-01	FIG. 5

Client:	Ocean Blvd
File No.	A9125-06-01
Boring	1

GEOCON

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):	0.63
Calculated Mag.Wtg.Factor:	0.873
Historic High Groundwater:	7.0
Groundwater Depth During Exploration	7.0

ENERGY & ROD CORRECTIONS:

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

LIQUEFACTION CALCULATIONS:

Unit Wt. Water (pcf):	62.4

Depth to Base (ft)	Total Unit Wt. (pcf)	Water (0 or 1)	FIELD SPT (N)	Depth of SPT (ft)	Liq.Sus. (0 or 1)	-200 (%)	Est. Dr (%)	CN Factor	Corrected (N1)60	Eff. Unit Wt. (psf)	Resist. CRR	rd Factor	Induced CSR	Liquefac. Safe.Fact.
1.0	125.0	0	12.0	2.5	0			2.000	27.0	125.0	~	0.998	0.357	~
2.0	125.0	0	12.0	2.5	0			2.000	27.0	125.0	~	0.993	0.356	~
3.0	125.0	0	12.0	2.5	0			2.000	27.0	125.0	~	0.989	0.354	~
4.0	125.0	0	12.0	2.5	0			2.000	27.0	125.0	~	0.984	0.352	~
5.0	125.0	0	12.0	2.5	0			1.927	26.0	125.0	~	0.979	0.351	~
6.0	125.0	0	12.0	2.5	0			1.743	23.5	125.0	~	0.975	0.349	~
7.0	125.0	1	23.0	7.5	0			1.635	42.3	62.6	~	0.970	0.361	~
8.0	125.0	1	23.0	7.5	0			1.573	40.7	62.6	~	0.966	0.384	~
9.0	125.0	1	23.0	7.5	0			1.518	39.3	62.6	~	0.961	0.404	~
10.0	125.0	1	23.0	7.5	0			1.468	38.0	62.6	~	0.957	0.420	~
11.0	125.0	1	23.0	7.5	1		99	1.423	36.8	62.6	Infin.	0.952	0.434	Non-Liq.
12.0	125.0	1	20.0	12.5	1		87	1.381	31.1	62.6	Infin.	0.947	0.446	Non-Liq.
13.0	125.0	1	20.0	12.5	1		87	1.343	30.2	62.6	Infin.	0.943	0.456	Non-Liq.
14.0	125.0	1	20.0	12.5	1	12	87	1.309	31.1	62.6	Infin.	0.938	0.465	Non-Liq.
15.0	125.0	1	20.0	12.5	1	12	87	1.276	30.3	62.6	Infin.	0.934	0.473	Non-Liq.
16.0	125.0	1	20.0	12.5	1	12	87	1.246	29.7	62.6	0.435	0.929	0.480	0.91
17.0	125.0	1	33.0	17.5	1		107	1.218	51.6	62.6	Infin.	0.925	0.485	Non-Liq.
18.0	125.0	1	33.0	17.5	1		107	1.192	50.5	62.6	Infin.	0.920	0.490	Non-Liq.
19.0	125.0	1	33.0	17.5	1		107	1.167	49.4	62.6	Infin.	0.915	0.495	Non-Liq.
20.0	125.0	1	33.0	17.5	1		107	1.144	48.5	62.6	Infin.	0.911	0.499	Non-Liq.
21.0	125.0	1	33.0	17.5	1		107	1.122	47.5	62.6	Infin.	0.906	0.502	Non-Liq.
22.0	125.0	1	34.0	22.5	1		104	1.102	52.1	62.6	Infin.	0.902	0.505	Non-Liq.
23.0	125.0	1	34.0	22.5	1		104	1.082	51.2	62.6	Infin.	0.897	0.507	Non-Liq.
24.0	125.0	1	34.0	22.5	1		104	1.064	50.3	62.6	Infin.	0.893	0.509	Non-Liq.
25.0	125.0	1	34.0	22.5	1		104	1.046	49.5	62.6	Infin.	0.888	0.511	Non-Liq.
26.0	125.0	1	34.0	22.5	1		104	1.029	48.7	62.6	Infin.	0.883	0.512	Non-Liq.
27.0	125.0	1	33.0	27.5	1		99	1.013	49.1	62.6	Infin.	0.879	0.513	Non-Liq.
28.0	125.0	1	33.0	27.5	1		99	0.998	48.4	62.6	Infin.	0.874	0.514	Non-Liq.
29.0	125.0	1	33.0	27.5	1		99	0.984	47.7	62.6	Infin.	0.870	0.514	Non-Liq.
30.0	125.0	1	33.0	27.5	1		99	0.970	47.0	62.6	Infin.	0.865	0.514	Non-Liq.
31.0	125.0	1	33.0	27.5	1		99	0.956	46.3	62.6	Infin.	0.861	0.515	Non-Liq.
32.0	125.0	1	33.0	27.5	1		99	0.943	45.7	62.6	Infin.	0.856	0.515	Non-Liq.
33.0	125.0	1	33.0	27.5	1		99	0.931	45.1	62.6	Infin.	0.851	0.514	Non-Liq.
34.0	125.0	1	33.0	27.5	1		99	0.919	44.5	62.6	Infin.	0.847	0.514	Non-Liq.
35.0	125.0	1	33.0	27.5	1		99	0.908	44.0	62.6	Infin.	0.842	0.513	Non-Liq.
36.0	125.0	1	33.0	27.5	1		99	0.897	43.5	62.6	Infin.	0.838	0.513	Non-Liq.
37.0	125.0	1	40.0	37.5	1		101	0.886	53.2	62.6	Infin.	0.833	0.512	Non-Liq.
38.0	125.0	1	40.0	37.5	1		101	0.876	52.6	62.6	Infin.	0.829	0.511	Non-Liq.
39.0	125.0	1	40.0	37.5	1		101	0.866	52.0	62.6	Infin.	0.824	0.510	Non-Liq.
40.0	125.0	1	40.0	37.5	1		101	0.856	51.4	62.6	Infin.	0.819	0.509	Non-Liq.
41.0	125.0	1	40.0	37.5	1		101	0.847	50.8	62.6	Infin.	0.815	0.508	Non-Liq.
42.0	125.0	1	49.0	47.5	1		105	0.838	61.6	62.6	Infin.	0.810	0.507	Non-Liq.
43.0	125.0	1	49.0	47.5	1		105	0.829	61.0	62.6	Infin.	0.806	0.505	Non-Liq.
44.0	125.0	1	49.0	47.5	1		105	0.821	60.3	62.6	Infin.	0.801	0.504	Non-Liq.
45.0	125.0	1	49.0	47.5	1		105	0.813	59.7	62.6	Infin.	0.797	0.502	Non-Liq.
46.0	125.0	1	49.0	47.5	1		105	0.805	59.2	62.6	Infin.	0.792	0.501	Non-Liq.
47.0	125.0	1	49.0	47.5	1		105	0.797	58.6	62.6	Infin.	0.787	0.499	Non-Liq.
48.0	125.0	1	49.0	47.5	1		105	0.790	58.0	62.6	Infin.	0.783	0.497	Non-Liq.
49.0	125.0	1	49.0	47.5	1		105	0.782	57.5	62.6	Infin.	0.778	0.496	Non-Liq.
50.0	125.0	1	49.0	47.5	1		105	0.775	57.0	62.6	Infin.	0.774	0.494	Non-Liq.
51.0	125.0	1	44.0	57.5	1		94	0.768	50.7	62.6	Infin.	0.769	0.492	Non-Liq.
52.0	125.0	1	44.0	57.5	1		94	0.762	50.3	62.6	Infin.	0.765	0.490	Non-Liq.
53.0	125.0	1	44.0	57.5	1		94	0.755	49.8	62.6	Infin.	0.760	0.488	Non-Liq.
54.0	125.0	1	44.0	57.5	1		94	0.749	49.4	62.6	Infin.	0.755	0.486	Non-Liq.
55.0	125.0	1	44.0	57.5	1		94	0.743	49.0	62.6	Infin.	0.751	0.484	Non-Liq.
56.0	125.0	1	44.0	57.5	1		94	0.737	48.6	62.6	Infin.	0.746	0.482	Non-Liq.
57.0	125.0	1	42.0	57.5	1		92	0.731	46.0	62.6	Infin.	0.742	0.480	Non-Liq.
58.0	125.0	1	42.0	57.5	1		92	0.725	45.7	62.6	Infin.	0.737	0.478	Non-Liq.
59.0	125.0	1	42.0	57.5	1		92	0.719	45.3	62.6	Infin.	0.733	0.475	Non-Liq.
60.0	125.0	1	42.0	57.5	1		92	0.714	45.0	62.6	Infin.	0.728	0.473	Non-Liq.

GEOCON

Client: Ocean Blvd
File No. A9125-06-01
Boring 1

LIQUEFACTION SETTLEMENT ANALYSIS
AMERICAN SOCIETY OF CIVIL ENGINEERS

(SATURATED SAND AT INITIAL LIQUEFACTION CONDITION)

NCEER (1996) METHOD
EARTHQUAKE INFORMATION:

Earthquake Magnitude:	7.1
Peak Horiz. Acceleration (g)	0.63
Calculated Mag.Wtg.Factor:	0.873
Historic High Groundwater:	7.0
Groundwater @ Exploration:	7.0

DEPTH TO BASE	BLOW COUNT N	WET DENSITY (PCF)	TOTAL STRESS O (TSF)	EFFECT STRESS O' (TSF)	REL. DEN. Dr (%)	ADJUST BLOWS (N1)60	Tav/O'o	LIQUEFACTION SAFETY FACTOR	Volumetric Strain [e_{15}} (%)	EQ. SETTLE. Pe (in.)
1	12	125	0.031	0.031		27	0.410	~	0.00	0.00
2	12	125	0.094	0.094		27	0.410	~	0.00	0.00
3	12	125	0.156	0.156		27	0.410	~	0.00	0.00
4	12	125	0.219	0.219		27	0.410	~	0.00	0.00
5	12	125	0.281	0.281		26	0.410	~	0.00	0.00
6	12	125	0.344	0.344		24	0.410	~	0.00	0.00
7	23	125	0.406	0.391		42	0.427	~	0.00	0.00
8	23	125	0.469	0.422		41	0.456	~	0.00	0.00
9	23	125	0.531	0.453		39	0.481	~	0.00	0.00
10	23	125	0.594	0.485		38	0.503	~	0.00	0.00
11	23	125	0.656	0.516	99	37	0.522	Non-Liq.	0.00	0.00
12	20	125	0.719	0.547	87	31	0.539	Non-Liq.	0.00	0.00
13	20	125	0.781	0.578	87	30	0.554	Non-Liq.	0.00	0.00
14	20	125	0.844	0.610	87	31	0.568	Non-Liq.	0.00	0.00
15	20	125	0.906	0.641	87	30	0.580	Non-Liq.	0.00	0.00
16	20	125	0.969	0.672	87	30	0.591	0.91	0.75	0.09
17	33	125	1.031	0.704	107	52	0.601	Non-Liq.	0.00	0.00
18	33	125	1.094	0.735	107	50	0.610	Non-Liq.	0.00	0.00
19	33	125	1.156	0.766	107	49	0.619	Non-Liq.	0.00	0.00
20	33	125	1.219	0.798	107	48	0.627	Non-Liq.	0.00	0.00
21	33	125	1.281	0.829	107	48	0.634	Non-Liq.	0.00	0.00
22	34	125	1.344	0.860	104	52	0.641	Non-Liq.	0.00	0.00
23	34	125	1.406	0.891	104	51	0.647	Non-Liq.	0.00	0.00
24	34	125	1.469	0.923	104	50	0.653	Non-Liq.	0.00	0.00
25	34	125	1.531	0.954	104	49	0.658	Non-Liq.	0.00	0.00
26	34	125	1.594	0.985	104	49	0.663	Non-Liq.	0.00	0.00
27	33	125	1.656	1.017	99	49	0.668	Non-Liq.	0.00	0.00
28	33	125	1.719	1.048	99	48	0.673	Non-Liq.	0.00	0.00
29	33	125	1.781	1.079	99	48	0.677	Non-Liq.	0.00	0.00
30	33	125	1.844	1.111	99	47	0.681	Non-Liq.	0.00	0.00
31	33	125	1.906	1.142	99	46	0.685	Non-Liq.	0.00	0.00
32	33	125	1.969	1.173	99	46	0.688	Non-Liq.	0.00	0.00
33	33	125	2.031	1.204	99	45	0.692	Non-Liq.	0.00	0.00
34	33	125	2.094	1.236	99	45	0.695	Non-Liq.	0.00	0.00
35	33	125	2.156	1.267	99	44	0.698	Non-Liq.	0.00	0.00
36	33	125	2.219	1.298	99	43	0.701	Non-Liq.	0.00	0.00
37	40	125	2.281	1.330	101	53	0.704	Non-Liq.	0.00	0.00
38	40	125	2.344	1.361	101	53	0.706	Non-Liq.	0.00	0.00
39	40	125	2.406	1.392	101	52	0.709	Non-Liq.	0.00	0.00
40	40	125	2.469	1.424	101	51	0.711	Non-Liq.	0.00	0.00
41	40	125	2.531	1.455	101	51	0.714	Non-Liq.	0.00	0.00
42	49	125	2.594	1.486	105	62	0.716	Non-Liq.	0.00	0.00
43	49	125	2.656	1.517	105	61	0.718	Non-Liq.	0.00	0.00
44	49	125	2.719	1.549	105	60	0.720	Non-Liq.	0.00	0.00
45	49	125	2.781	1.580	105	60	0.722	Non-Liq.	0.00	0.00
46	49	125	2.844	1.611	105	59	0.724	Non-Liq.	0.00	0.00
47	49	125	2.906	1.643	105	59	0.726	Non-Liq.	0.00	0.00
48	49	125	2.969	1.674	105	58	0.727	Non-Liq.	0.00	0.00
49	49	125	3.031	1.705	105	58	0.729	Non-Liq.	0.00	0.00
50	49	125	3.094	1.737	105	57	0.731	Non-Liq.	0.00	0.00

TOTAL SETTLEMENT = 0.1 INCHES

Figure 7

Client:	Ocean Blvd
File No.	A9125-06-01
Boring	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):	0.63
Calculated Mag.Wtg.Factor:	0.873
Historic High Groundwater:	7.0
Groundwater Depth During Exploration	7.5

ENERGY & ROD CORRECTIONS:

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

LIQUEFACTION CALCULATIONS:

Unit Wt. Water (pcf):	62.4

Depth to Base (ft)	Total Unit Wt. (pcf)	Water (0 or 1)	FIELD SPT (N)	Depth of SPT (ft)	Liq.Sus. (0 or 1)	-200 (%)	Est. Dr (%)	CN Factor	Corrected (N1)60	Eff. Unit Wt. (psf)	Resist. CRR	rd Factor	Induced CSR	Liquefac. Safe.Fact.
1.0	125.0	0	16.0	5.0	0			2.000	36.0	125.0	~	0.998	0.357	~
2.0	125.0	0	16.0	5.0	0			2.000	36.0	125.0	~	0.993	0.356	~
3.0	125.0	0	16.0	5.0	0			2.000	36.0	125.0	~	0.989	0.354	~
4.0	125.0	0	16.0	5.0	0			2.000	36.0	125.0	~	0.984	0.352	~
5.0	125.0	0	16.0	5.0	0			1.927	34.7	125.0	~	0.979	0.351	~
6.0	125.0	0	16.0	5.0	0			1.743	31.4	125.0	~	0.975	0.349	~
7.0	125.0	1	16.0	5.0	0			1.603	28.9	62.6	~	0.970	0.361	~
8.0	125.0	1	16.0	5.0	0			1.518	27.3	62.6	~	0.966	0.384	~
9.0	125.0	1	16.0	5.0	0			1.468	26.4	62.6	~	0.961	0.404	~
10.0	125.0	1	23.0	10.0	0			1.423	36.8	62.6	~	0.957	0.420	~
11.0	125.0	1	23.0	10.0	1		97	1.381	35.7	62.6	Infin.	0.952	0.434	Non-Liq.
12.0	125.0	1	23.0	10.0	1		97	1.344	34.8	62.6	Infin.	0.947	0.446	Non-Liq.
13.0	125.0	1	23.0	10.0	1		97	1.309	33.9	62.6	Infin.	0.943	0.456	Non-Liq.
14.0	125.0	1	23.0	10.0	1		97	1.276	33.0	62.6	Infin.	0.938	0.465	Non-Liq.
15.0	125.0	1	25.0	15.0	1		96	1.246	37.7	62.6	Infin.	0.934	0.473	Non-Liq.
16.0	125.0	1	25.0	15.0	1		96	1.218	36.8	62.6	Infin.	0.929	0.480	Non-Liq.
17.0	125.0	1	25.0	15.0	1		96	1.192	36.1	62.6	Infin.	0.925	0.485	Non-Liq.
18.0	125.0	1	25.0	15.0	1		96	1.167	35.3	62.6	Infin.	0.920	0.490	Non-Liq.
19.0	125.0	1	30.0	20.0	1		100	1.144	46.1	62.6	Infin.	0.915	0.495	Non-Liq.
20.0	125.0	1	30.0	20.0	1		100	1.122	45.2	62.6	Infin.	0.911	0.499	Non-Liq.
21.0	125.0	1	30.0	20.0	1		100	1.102	44.4	62.6	Infin.	0.906	0.502	Non-Liq.
22.0	125.0	1	30.0	20.0	1		100	1.082	43.6	62.6	Infin.	0.902	0.505	Non-Liq.
23.0	125.0	1	30.0	20.0	1		100	1.064	42.8	62.6	Infin.	0.897	0.507	Non-Liq.
24.0	125.0	1	30.0	20.0	1		100	1.046	42.1	62.6	Infin.	0.893	0.509	Non-Liq.
25.0	125.0	1	38.0	25.0	1		108	1.029	56.1	62.6	Infin.	0.888	0.511	Non-Liq.
26.0	125.0	1	38.0	25.0	1		108	1.013	55.2	62.6	Infin.	0.883	0.512	Non-Liq.
27.0	125.0	1	38.0	25.0	1		108	0.998	54.4	62.6	Infin.	0.879	0.513	Non-Liq.
28.0	125.0	1	38.0	25.0	1		108	0.984	53.6	62.6	Infin.	0.874	0.514	Non-Liq.
29.0	125.0	1	59.0	30.0	1		130	0.970	85.8	62.6	Infin.	0.870	0.514	Non-Liq.
30.0	125.0	1	59.0	30.0	1		130	0.956	84.6	62.6	Infin.	0.865	0.514	Non-Liq.
31.0	125.0	1	59.0	30.0	1		130	0.943	83.5	62.6	Infin.	0.861	0.515	Non-Liq.
32.0	125.0	1	59.0	30.0	1		130	0.931	82.4	62.6	Infin.	0.856	0.515	Non-Liq.
33.0	125.0	1	59.0	30.0	1		130	0.919	81.3	62.6	Infin.	0.851	0.514	Non-Liq.
34.0	125.0	1	59.0	30.0	1		130	0.908	80.3	62.6	Infin.	0.847	0.514	Non-Liq.
35.0	125.0	1	84.0	35.0	1		150	0.897	113.0	62.6	Infin.	0.842	0.513	Non-Liq.
36.0	125.0	1	84.0	35.0	1		150	0.886	111.7	62.6	Infin.	0.838	0.513	Non-Liq.
37.0	125.0	1	84.0	35.0	1		150	0.876	110.4	62.6	Infin.	0.833	0.512	Non-Liq.
38.0	125.0	1	84.0	35.0	1		150	0.866	109.1	62.6	Infin.	0.829	0.511	Non-Liq.
39.0	125.0	1	84.0	35.0	1		150	0.856	107.9	62.6	Infin.	0.824	0.510	Non-Liq.
40.0	125.0	1	84.0	35.0	1		150	0.847	106.7	62.6	Infin.	0.819	0.509	Non-Liq.
41.0	125.0	1	84.0	35.0	1		150	0.838	105.6	62.6	Infin.	0.815	0.508	Non-Liq.
42.0	125.0	1	84.0	35.0	1		150	0.829	104.5	62.6	Infin.	0.810	0.507	Non-Liq.
43.0	125.0	1	84.0	35.0	1		150	0.821	103.5	62.6	Infin.	0.806	0.505	Non-Liq.
44.0	125.0	1	69.0	45.0	1		127	0.813	84.1	62.6	Infin.	0.801	0.504	Non-Liq.
45.0	125.0	1	69.0	45.0	1		127	0.805	83.3	62.6	Infin.	0.797	0.502	Non-Liq.
46.0	125.0	1	69.0	45.0	1		127	0.797	82.5	62.6	Infin.	0.792	0.501	Non-Liq.
47.0	125.0	1	69.0	45.0	1		127	0.790	81.7	62.6	Infin.	0.787	0.499	Non-Liq.
48.0	125.0	1	69.0	45.0	1		127	0.782	81.0	62.6	Infin.	0.783	0.497	Non-Liq.
49.0	125.0	1	69.0	45.0	1		127	0.775	80.3	62.6	Infin.	0.778	0.496	Non-Liq.
50.0	125.0	1	69	45	1		127	0.768	79.5	62.6	Infin.	0.774	0.494	Non-Liq.
51.0	125.0	1	57.0	55.0	1		109	0.762	65.1	62.6	Infin.	0.769	0.492	Non-Liq.
52.0	125.0	1	57.0	55.0	1		109	0.755	64.6	62.6	Infin.	0.765	0.490	Non-Liq.
53.0	125.0	1	57.0	55.0	1		109	0.749	64.0	62.6	Infin.	0.760	0.488	Non-Liq.
54.0	125.0	1	57.0	55.0	1		109	0.743	63.5	62.6	Infin.	0.755	0.486	Non-Liq.
55.0	125.0	1	57.0	55.0	1		109	0.737	63.0	62.6	Infin.	0.751	0.484	Non-Liq.
56.0	125.0	1	57.0	55.0	1		109	0.731	62.5	62.6	Infin.	0.746	0.482	Non-Liq.
57.0	125.0	1	57.0	55.0	1		109	0.725	62.0	62.6	Infin.	0.742	0.480	Non-Liq.
58.0	125.0	1	57.0	55.0	1		109	0.719	61.5	62.6	Infin.	0.737	0.478	Non-Liq.
59.0	125.0	1	57.0	55.0	1		109	0.714	61.0	62.6	Infin.	0.733	0.475	Non-Liq.
60.0	125.0	1	57	55	1		109	0.708	60.6	62.6	Infin.	0.728	0.473	Non-Liq.

Client: Ocean Blvd
File No. A9125-06-01
Boring 2

LIQUEFACTION SETTLEMENT ANALYSIS
AMERICAN SOCIETY OF CIVIL ENGINEERS

(SATURATED SAND AT INITIAL LIQUEFACTION CONDITION)

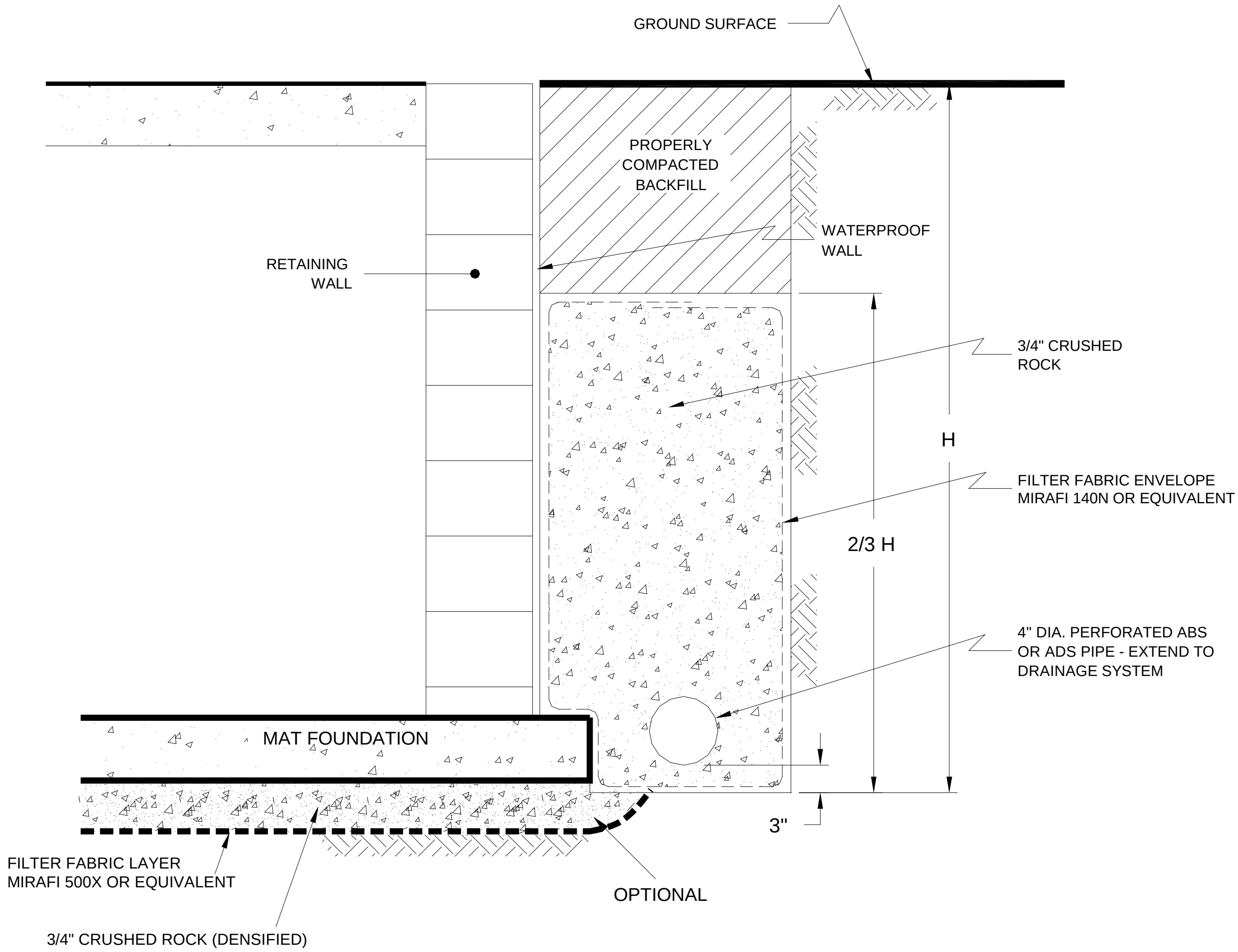
NCEER (1996) METHOD
EARTHQUAKE INFORMATION:

Earthquake Magnitude:	7.1
Peak Horiz. Acceleration (g)	0.63
Calculated Mag.Wtg.Factor:	0.873
Historic High Groundwater:	7.0
Groundwater @ Exploration:	7.5

DEPTH TO BASE	BLOW COUNT N	WET DENSITY (PCF)	TOTAL STRESS O (TSF)	EFFECT STRESS O' (TSF)	REL. DEN. Dr (%)	ADJUST BLOWS (N1)60	Tav/O'o	LIQUEFACTION SAFETY FACTOR	Volumetric Strain [e15} (%)	EQ. SETTLE. Pe (in.)
1	16	125	0.031	0.031		36	0.410	~	0.00	0.00
2	16	125	0.094	0.094		36	0.410	~	0.00	0.00
3	16	125	0.156	0.156		36	0.410	~	0.00	0.00
4	16	125	0.219	0.219		36	0.410	~	0.00	0.00
5	16	125	0.281	0.281		35	0.410	~	0.00	0.00
6	16	125	0.344	0.344		31	0.410	~	0.00	0.00
7	16	125	0.406	0.391		29	0.427	~	0.00	0.00
8	16	125	0.469	0.422		27	0.456	~	0.00	0.00
9	16	125	0.531	0.453		26	0.481	~	0.00	0.00
10	23	125	0.594	0.485		37	0.503	~	0.00	0.00
11	23	125	0.656	0.516	97	36	0.522	Non-Liq.	0.00	0.00
12	23	125	0.719	0.547	97	35	0.539	Non-Liq.	0.00	0.00
13	23	125	0.781	0.578	97	34	0.554	Non-Liq.	0.00	0.00
14	23	125	0.844	0.610	97	33	0.568	Non-Liq.	0.00	0.00
15	25	125	0.906	0.641	96	38	0.580	Non-Liq.	0.00	0.00
16	25	125	0.969	0.672	96	37	0.591	Non-Liq.	0.00	0.00
17	25	125	1.031	0.704	96	36	0.601	Non-Liq.	0.00	0.00
18	25	125	1.094	0.735	96	35	0.610	Non-Liq.	0.00	0.00
19	30	125	1.156	0.766	100	46	0.619	Non-Liq.	0.00	0.00
20	30	125	1.219	0.798	100	45	0.627	Non-Liq.	0.00	0.00
21	30	125	1.281	0.829	100	44	0.634	Non-Liq.	0.00	0.00
22	30	125	1.344	0.860	100	44	0.641	Non-Liq.	0.00	0.00
23	30	125	1.406	0.891	100	43	0.647	Non-Liq.	0.00	0.00
24	30	125	1.469	0.923	100	42	0.653	Non-Liq.	0.00	0.00
25	38	125	1.531	0.954	108	56	0.658	Non-Liq.	0.00	0.00
26	38	125	1.594	0.985	108	55	0.663	Non-Liq.	0.00	0.00
27	38	125	1.656	1.017	108	54	0.668	Non-Liq.	0.00	0.00
28	38	125	1.719	1.048	108	54	0.673	Non-Liq.	0.00	0.00
29	59	125	1.781	1.079	130	86	0.677	Non-Liq.	0.00	0.00
30	59	125	1.844	1.111	130	85	0.681	Non-Liq.	0.00	0.00
31	59	125	1.906	1.142	130	83	0.685	Non-Liq.	0.00	0.00
32	59	125	1.969	1.173	130	82	0.688	Non-Liq.	0.00	0.00
33	59	125	2.031	1.204	130	81	0.692	Non-Liq.	0.00	0.00
34	59	125	2.094	1.236	130	80	0.695	Non-Liq.	0.00	0.00
35	84	125	2.156	1.267	150	113	0.698	Non-Liq.	0.00	0.00
36	84	125	2.219	1.298	150	112	0.701	Non-Liq.	0.00	0.00
37	84	125	2.281	1.330	150	110	0.704	Non-Liq.	0.00	0.00
38	84	125	2.344	1.361	150	109	0.706	Non-Liq.	0.00	0.00
39	84	125	2.406	1.392	150	108	0.709	Non-Liq.	0.00	0.00
40	84	125	2.469	1.424	150	107	0.711	Non-Liq.	0.00	0.00
41	84	125	2.531	1.455	150	106	0.714	Non-Liq.	0.00	0.00
42	84	125	2.594	1.486	150	105	0.716	Non-Liq.	0.00	0.00
43	84	125	2.656	1.517	150	103	0.718	Non-Liq.	0.00	0.00
44	69	125	2.719	1.549	127	84	0.720	Non-Liq.	0.00	0.00
45	69	125	2.781	1.580	127	83	0.722	Non-Liq.	0.00	0.00
46	69	125	2.844	1.611	127	83	0.724	Non-Liq.	0.00	0.00
47	69	125	2.906	1.643	127	82	0.726	Non-Liq.	0.00	0.00
48	69	125	2.969	1.674	127	81	0.727	Non-Liq.	0.00	0.00
49	69	125	3.031	1.705	127	80	0.729	Non-Liq.	0.00	0.00
50	69	125	3.094	1.737	127	80	0.731	Non-Liq.	0.00	0.00

TOTAL SETTLEMENT = 0.0 INCHES

Figure 9



NO SCALE

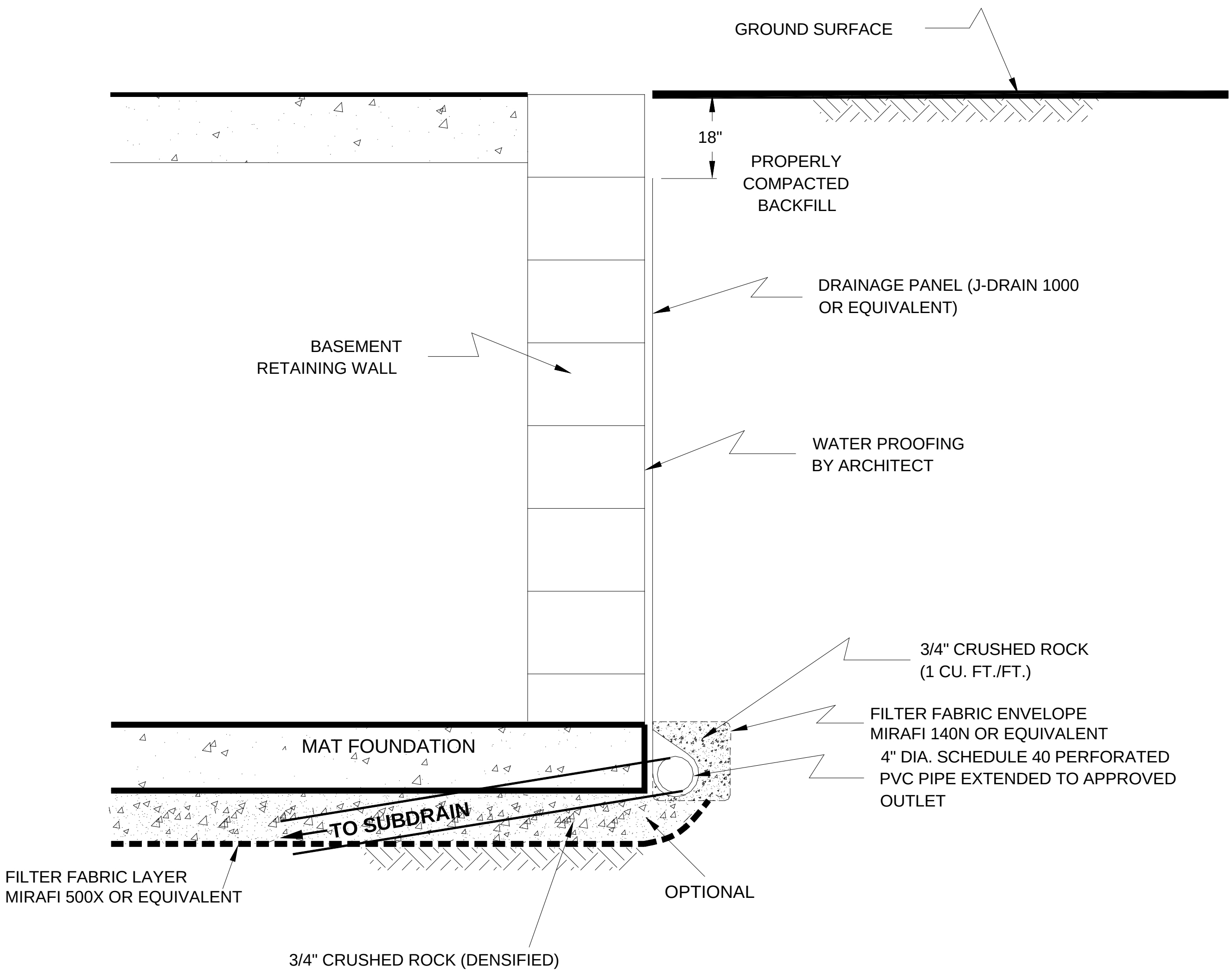
GEOCON WEST, INC.
ENVIRONMENTAL GEOTECHNICAL MATERIALS
3303 N. SAN FERNANDO BLVD. - SUITE 100 - BURBANK, CA 91504
PHONE (818) 841-8388 - FAX (818) 841-1704

PZ		9000

RETAINING WALL DRAIN DETAIL

444 W OCEAN, LLC
444 WEST OCEAN BOULEVARD
LONG BEACH, CALIFORNIA

MAY 14, 2014	PROJECT NO. A9125-06-01	FIG. 10



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

NO SCALE

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

PZ		9000

RETAINING WALL DRAIN DETAIL

444 W OCEAN, LLC
444 WEST OCEAN BOULEVARD
LONG BEACH, CALIFORNIA

MAY 14, 2014	PROJECT NO. A9125-06-01	FIG. 11

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.
NEWPORT-INGLEWOOD (L.A.Basin)	3.0(4.8)	7.1	0.647	X
PALOS VERDES	4.2(6.8)	7.3	0.603	X
PUENTE HILLS BLIND THRUST	13.7(22.0)	7.1	0.382	X
SAN JOAQUIN HILLS	16.0(25.8)	6.6	0.272	IX
WHITTIER	18.3(29.5)	6.8	0.202	VIII
NEWPORT-INGLEWOOD (Offshore)	20.3(32.6)	7.1	0.210	VIII
UPPER ELYSIAN PARK BLIND THRUST	20.7(33.3)	6.4	0.186	VIII
SANTA MONICA	24.0(38.6)	6.6	0.177	VIII
HOLLYWOOD	24.4(39.3)	6.4	0.153	VIII
RAYMOND	24.5(39.4)	6.5	0.164	VIII
VERDUGO	25.4(40.8)	6.9	0.193	VIII
SAN JOSE	26.3(42.3)	6.4	0.140	VIII
MALIBU COAST	26.5(42.7)	6.7	0.166	VIII
CHINO-CENTRAL AVE. (Elsinore)	29.0(46.7)	6.7	0.149	VIII
SIERRA MADRE	29.3(47.1)	7.2	0.197	VIII
CLAMSHELL-SAWPIT	30.6(49.3)	6.5	0.125	VII
ANACAPA-DUME	32.3(52.0)	7.5	0.214	VIII
ELSINORE (GLEN IVY)	32.7(52.7)	6.8	0.106	VII
NORTHRIDGE (E. Oak Ridge)	33.6(54.0)	7.0	0.147	VIII
SIERRA MADRE (San Fernando)	35.8(57.6)	6.7	0.115	VII
CUCAMONGA	36.6(58.9)	6.9	0.125	VIII
CORONADO BANK	37.7(60.7)	7.6	0.150	VIII
SAN GABRIEL	38.4(61.8)	7.2	0.112	VII
SANTA SUSANA	41.8(67.3)	6.7	0.094	VII
SIMI-SANTA ROSA	46.8(75.3)	7.0	0.097	VII
HOLSER	47.5(76.4)	6.5	0.070	VI
ELSINORE (TEMECULA)	49.5(79.7)	6.8	0.062	VI
SAN ANDREAS - Mojave M-1c-3	50.9(81.9)	7.4	0.092	VII
SAN ANDREAS - Whole M-1a	50.9(81.9)	8.0	0.140	VIII
SAN ANDREAS - 1857 Rupture M-2a	50.9(81.9)	7.8	0.122	VII
SAN ANDREAS - Cho-Moj M-1b-1	50.9(81.9)	7.8	0.122	VII
OAK RIDGE (Onshore)	51.0(82.0)	7.0	0.087	VII
SAN JACINTO-SAN BERNARDINO	51.6(83.0)	6.7	0.055	VI
SAN ANDREAS - SB-Coach. M-2b	54.1(87.0)	7.7	0.106	VII
SAN ANDREAS - SB-Coach. M-1b-2	54.1(87.0)	7.7	0.106	VII
SAN ANDREAS - San Bernardino M-1	54.1(87.0)	7.5	0.092	VII
CLEGHORN	56.5(90.9)	6.5	0.043	VI
SAN CAYETANO	56.5(90.9)	7.0	0.076	VII
SAN JACINTO-SAN JACINTO VALLEY	58.0(93.4)	6.9	0.053	VI

39 FAULTS FOUND WITHIN THE SPECIFIED SEARCH RADIUS.
THE NEWPORT-INGLEWOOD (L.A. Basin) FAULT IS CLOSEST TO THE SITE.
IT IS ABOUT 3.0 MILES (4.8 km) AWAY.
LARGEST MAXIMUM-EARTHQUAKE SITE ACCELERATION: 0.6470 g

APPENDIX A

FIELD INVESTIGATION

The site was explored on April 15, 2014 by excavating two 4⅞-inch diameter borings utilizing a mud rotary drilling machine. The borings were advanced to depths between 60½ and 62½ feet below the existing ground surface. Representative and relatively undisturbed samples were obtained by driving a 3 inch O. D., California Modified Sampler into the "undisturbed" soil mass with blows from a slide hammer. The California Modified Sampler was equipped with 1-inch high by 2⅜-inch diameter brass sampler rings to facilitate soil removal and testing. Standard Penetration Tests (SPTs) were performed in both borings.

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

BORING 1

ELEV. (MSL.) - - DATE COMPLETED 4/15/14

EQUIPMENT MUD ROTARY BY: PZ

DEPTH IN FEET	SAMPLE NO.	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT)*	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0				**ASPHALT: 1.5" CONCRETE: 5" BASE: 5"**			
				ARTIFICIAL FILL Silty Sand, medium dense, slightly moist, light brown, fine-grained			
2	B1@2.5'			**ALLUVIUM** Sand with silt, medium dense, slightly moist, light brown, fine-grained	12		24.0
4			SP	-Moist			
	B1@5'				36	89.8	16.8
6							
		▼		-Silty Sand, medium dense, moist, olive brown, fine-grained, some shells			
	B1@7.5'				23		30.7
8							
10	B1@10'				48	98.3	21.7
12	B1@12.5'				20		30.1
14							
				-Dense			
	B1@15'				61	101.8	21.1
16							
				-Increase in silt content, yellowish brown			
	B1@17.5'		SM		33		25.7
18							
20	B1@20'			-Medium dense	53	96.8	24.5
				-Dense			
22	B1@22.5'				34		24.8
24							
				-Very dense			
	B1@25'				50 (5")	92.2	22.3
26							
	B1@27.5'			-Dense	33		24.6
28							

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

A9125-06-01 BORING LOGS.GPJ

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

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

GEOCON

PROJECT NO. A9125-06-01

BORING 1

ELEV. (MSL.) - - DATE COMPLETED 4/15/14

EQUIPMENT MUD ROTARY BY: PZ

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT)*	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
32					-Trace fine gravel			
32.5	B1@32.5'			SM	-Some thin silt layers	80	99.9	22.5
37.5	B1@37.5'			ML	Silt with sand, hard, moist, olive brown with oxidation mottles, fine-grained	40		16.5
42.5	B1@42.5'			SM	Silty Sand, very dense, moist, olive brown, fine- to medium-grained, trace fine gravel	50 (5")	115.9	15.1
47.5	B1@47.5'			SC	Sandy Clay, hard, moist, yellowish brown, medium- to coarse-grained	49		15.1
52.5	B1@52.5'			ML	Sandy Silt, hard, moist, gray with yellowish brown mottles, fine-grained	50 (5.5")	100.9	20.4
57.5	B1@57.5'			ML	Silt, hard, moist, olive brown	44		36.3

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

A9125-06-01 BORING LOGS.GPJ

SAMPLE SYMBOLS
- ... SAMPLING UNSUCCESSFUL
- ... STANDARD PENETRATION TEST
- ... DRIVE SAMPLE (UNDISTURBED)
- ... DISTURBED OR BAG SAMPLE
- ... CHUNK SAMPLE
- ... WATER TABLE OR SEEPAGE

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

GEOCON

PROJECT NO. A9125-06-01

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	BORING 1 ELEV. (MSL.) - - DATE COMPLETED 4/15/14 EQUIPMENT MUD ROTARY BY: PZ	PENETRATION RESISTANCE (BLOWS/FT)*	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
					MATERIAL DESCRIPTION			
60								
62	B1@62'			ML	-Some fine-grained sand	53 (6")	102.7	21.3

Total depth of boring: 62.5 feet.
Fill to 1.5 feet.
Groundwater encountered at 7 feet.
Backfilled with bentonite chips and cement.
Asphalt patched.

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

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

A9125-06-01 BORING LOGS.GPJ

SAMPLE SYMBOLS			
	... SAMPLING UNSUCCESSFUL	... STANDARD PENETRATION TEST	... DRIVE SAMPLE (UNDISTURBED)
	... DISTURBED OR BAG SAMPLE	... CHUNK SAMPLE	... WATER TABLE OR SEEPAGE

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

GEOCON

BORING 2

ELEV. (MSL.) - - DATE COMPLETED 4/15/14

EQUIPMENT MUD ROTARY BY: PZ

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT)*	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0					**ASPHALT: 6"**			
					ARTIFICIAL FILL Silty Sand, medium dense, slightly moist, light brown, fine-grained			
2					**ALLUVIUM**			
	B2@2.5'			SP	Sand with Silt, medium dense, slightly moist, olive brown, fine-grained	32	88.9	16.2
4					Silty Sand, medium dense, moist, olive brown, fine-grained			
	B2@5'					16		16.2
6								
			▼		-Dense, fine- to medium-grained			
8	B2@7.5'					65	97.2	22.2
					-Grayish brown, trace shells			
10	B2@10'					23		26.6
12								
	B2@12.5'					53	95.3	23.1
14								
	B2@15'					25		22.4
16				SM	-Medium- to coarse-grained			
					-Increase in silt content, some coarse-grained, trace fine gravel, some shells			
18	B2@17.5'					60	113.7	12.1
					-Fine-grained			
					-Dense			
20	B2@20'					30		24.7
22								
	B2@22.5'					57	103.7	20.9
24					-Grayish brown			
	B2@25'					38		28.6
26								
					-Decrease in silt content			
28	B2@27.5'					78	93.5	27.5
					-Some medium-grained sand			

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

A9125-06-01 BORING LOGS.GPJ

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

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

PROJECT NO. A9125-06-01

BORING 2

ELEV. (MSL.) - - DATE COMPLETED 4/15/14

EQUIPMENT **MUD ROTARY** BY: **PZ**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT)*	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
30	B2@30'				-Some shells	59		22.2
32	B2@32.5'			SM	-Fine-grained	50 (4.5")	99.2	22.5
34	B2@35'				Sandy Silt, hard, moist, olive brown, fine-grained	84		15.6
36								
38				ML	-Decrease in sand content			
40	B2@40'				-Oxidation staining	55 (6")	110.0	19.0
42								
44	B2@45'				Silty Sand, dense, moist, olive brown, fine- to medium-grained	69		18.7
46								
48					-Very dense, medium- to coarse-grained, trace fine gravel			
50	B2@50'			SM	-Increase in silt content	52 (6")	111.4	17.5
52								
54	B2@55'					57		28.8
56					Silt, hard, moist, olive brown			
58				ML				

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

A9125-06-01 BORING LOGS.GPJ

SAMPLE SYMBOLS

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

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

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DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	BORING 2 ELEV. (MSL.) - - DATE COMPLETED 4/15/14 EQUIPMENT MUD ROTARY BY: PZ	PENETRATION RESISTANCE (BLOWS/FT)*	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
					MATERIAL DESCRIPTION			
60	B2@60'					86	87.8	34.6
					Total depth of boring: 60.5 feet. Fill to 1.5 feet. Groundwater encountered at 7.5 feet. Backfilled with bentonite chips and cement. Asphalt patched. *Penetration resistance for 140 pound hammer falling 30 inches by auto-hammer.			

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

A9125-06-01 BORING LOGS.GPJ

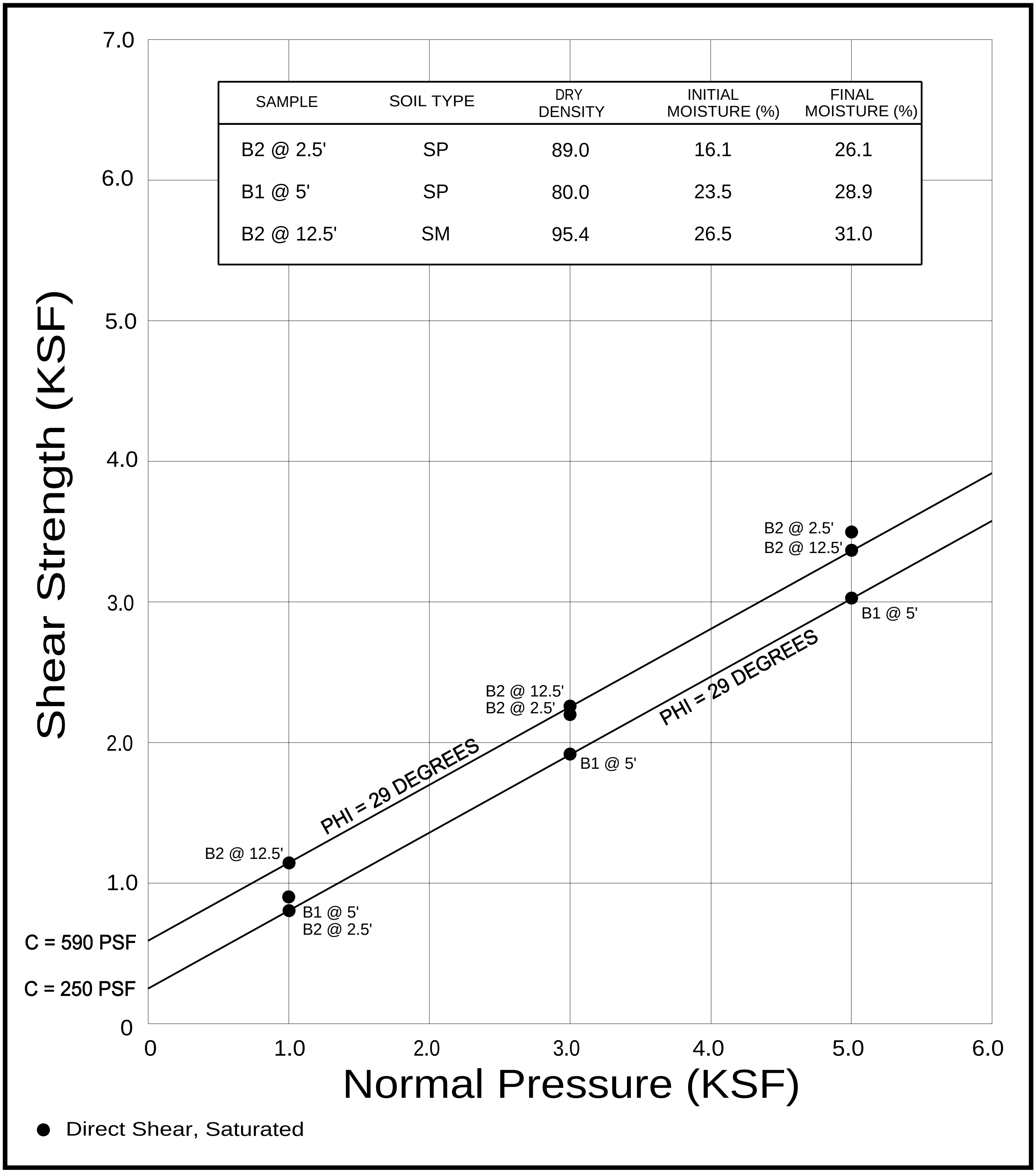
SAMPLE SYMBOLS			
	... SAMPLING UNSUCCESSFUL	... STANDARD PENETRATION TEST	... DRIVE SAMPLE (UNDISTURBED)
	... DISTURBED OR BAG SAMPLE	... CHUNK SAMPLE	... WATER TABLE OR SEEPAGE

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

APPENDIX B

LABORATORY TESTING

Laboratory tests were performed in accordance with generally accepted test methods of the American Society for Testing and Materials (ASTM), or other suggested procedures. Selected samples were tested for direct shear strength, consolidation and expansion characteristics, moisture density relationships, corrosivity, in-place dry density and moisture content. The results of the laboratory tests are summarized in Figures B1 through B6. The in-place dry density and moisture content of the samples tested are presented on the boring logs, Appendix A.



SAMPLE	SOIL TYPE	DRY DENSITY	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B2 @ 2.5'	SP	89.0	16.1	26.1
B1 @ 5'	SP	80.0	23.5	28.9
B2 @ 12.5'	SM	95.4	26.5	31.0

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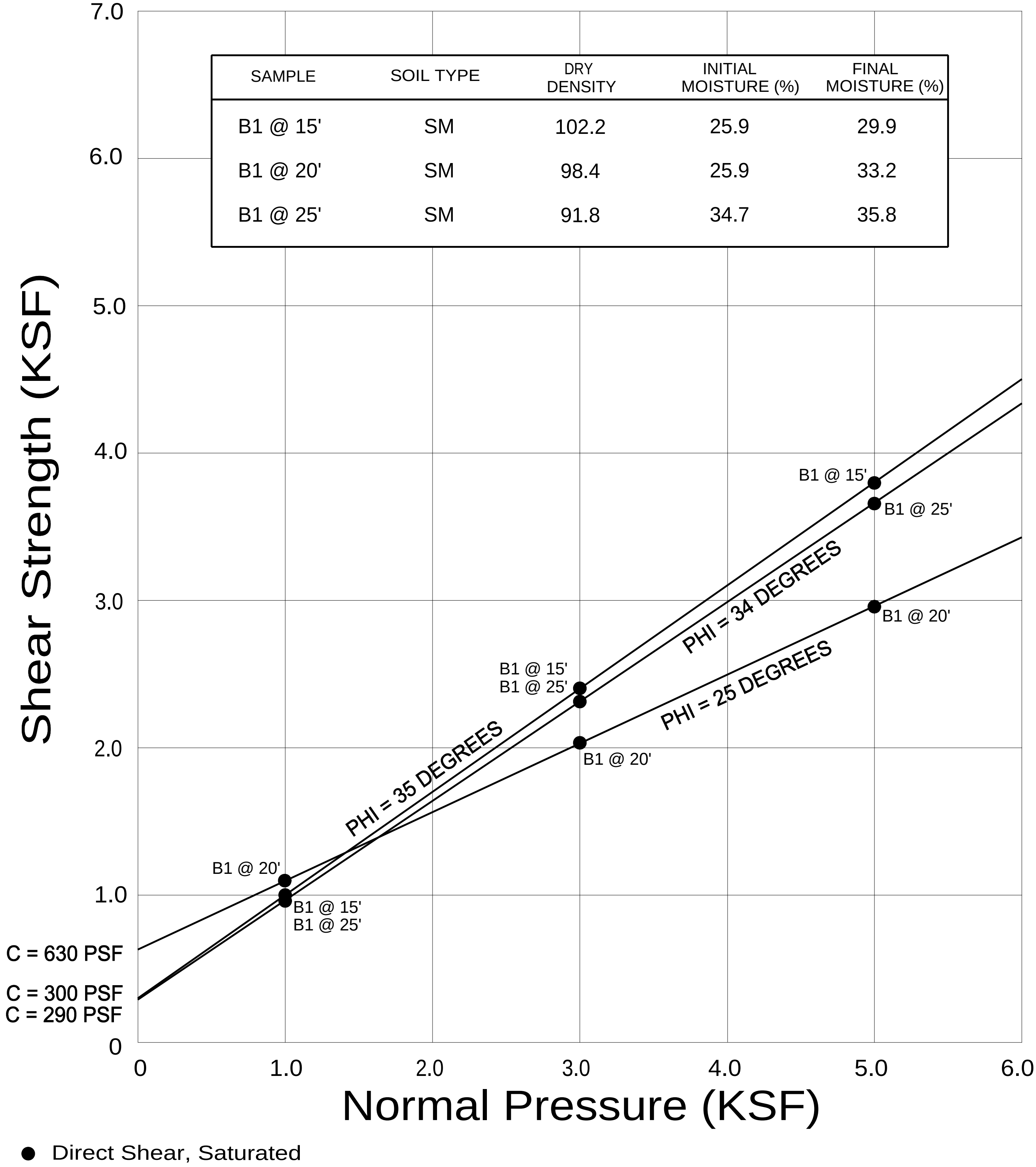
DIRECT SHEAR TEST RESULTS

444 W. OCEAN, LLC
444 WEST OCEAN BOULEVARD
LONG BEACH, CALIFORNIA

PZ		9000	MAY 14, 2014	PROJECT NO. A9125-06-01	FIG. B1



SAMPLE	SOIL TYPE	DRY DENSITY	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B1 @ 15'	SM	102.2	25.9	29.9
B1 @ 20'	SM	98.4	25.9	33.2
B1 @ 25'	SM	91.8	34.7	35.8

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PZ		9000

DIRECT SHEAR TEST RESULTS

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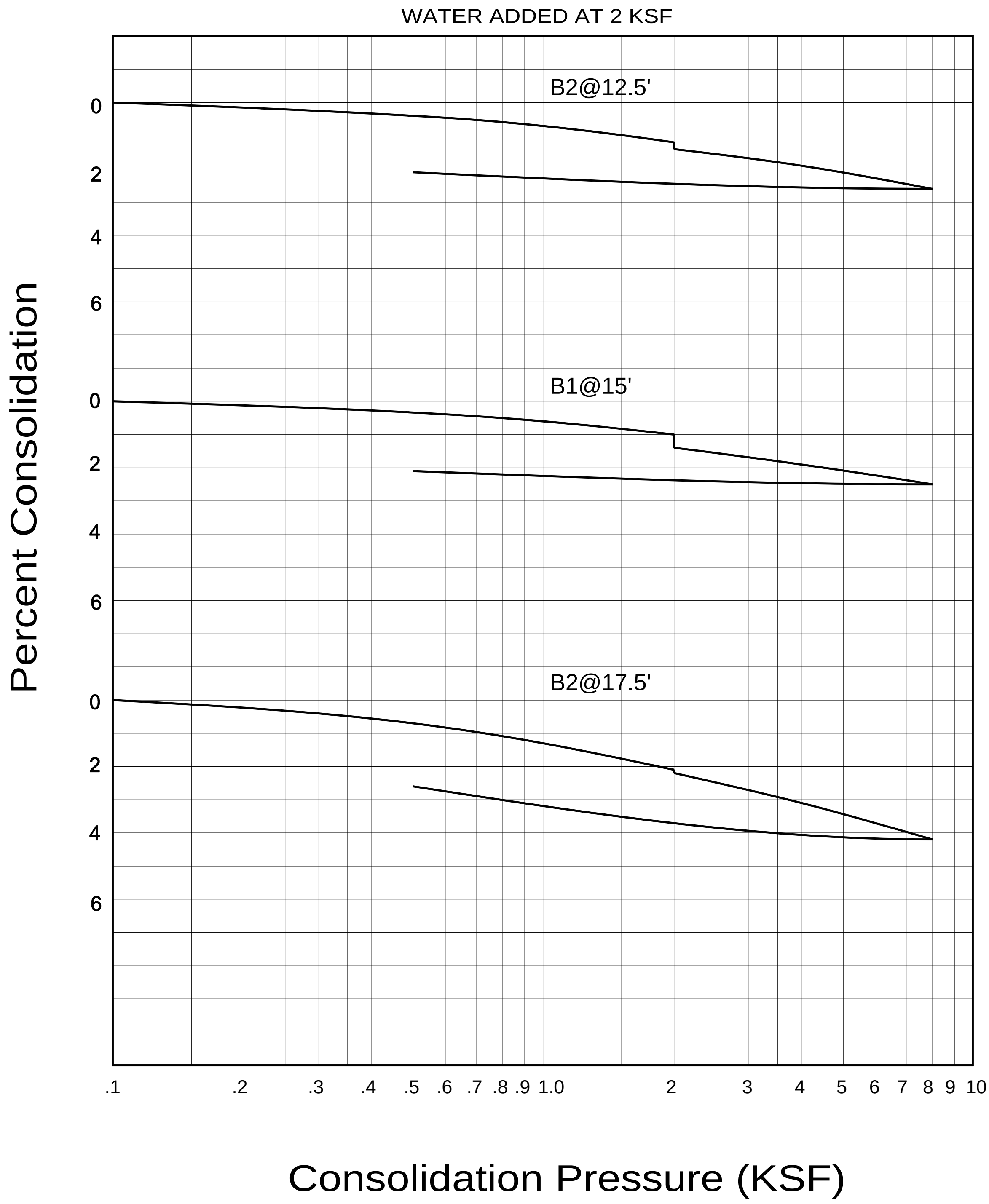
444 WEST OCEAN BOULEVARD
LONG BEACH, CALIFORNIA

MAY 14, 2014	PROJECT NO. A9125-06-01	FIG. B2



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PZ		9000

CONSOLIDATION TEST RESULTS

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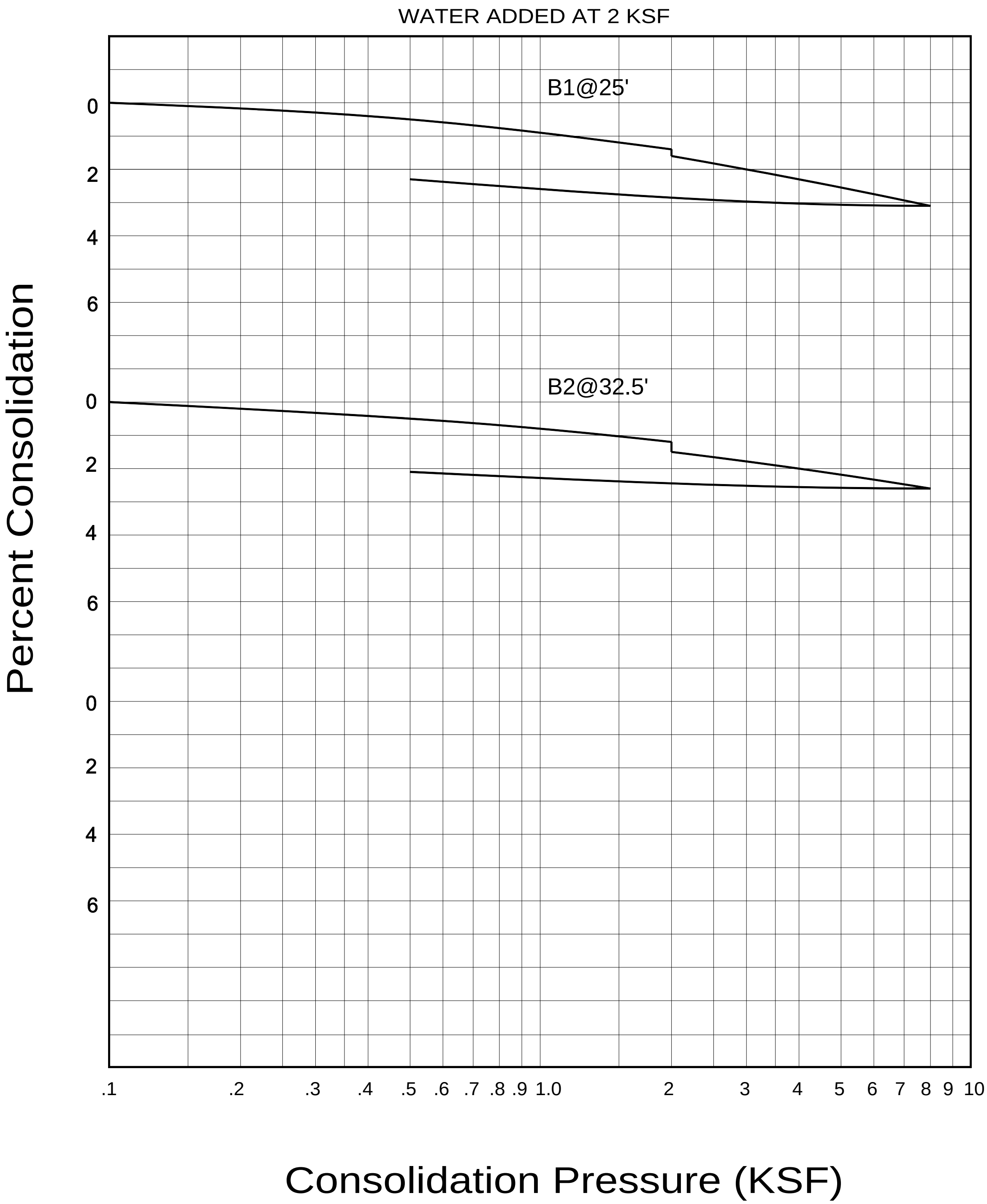
LONG BEACH, CALIFORNIA

MAY 14, 2014	PROJECT NO. A9125-06-01	FIG. B3



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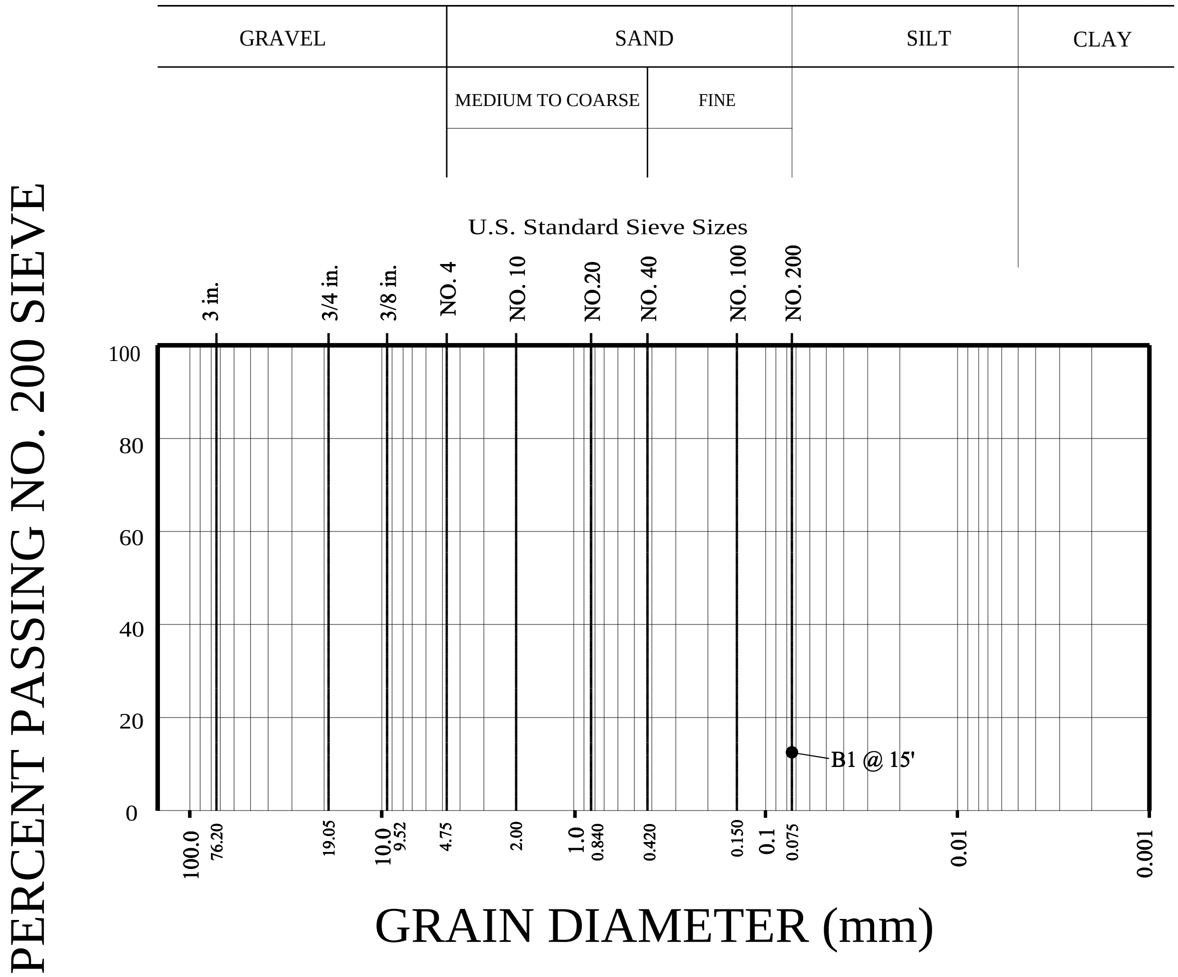
PZ		9000

CONSOLIDATION TEST RESULTS

444 W. OCEAN, LLC
444 WEST OCEAN BOULEVARD
LONG BEACH, CALIFORNIA

MAY 14, 2014	PROJECT NO. A9125-06-01	FIG. B4



SAMPLE	PERCENT PASSING NO. 200 SIEVE
B1 @ 15'	12.5

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PZ		9000

GRAIN SIZE ANALYSIS

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MAY 14, 2014	PROJECT NO. A9125-06-01	FIG. B5

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

Sample No.	pH	Resistivity (ohm centimeters)
B1 @ 12.5'	7.80	1600 (Corrosive)

SUMMARY OF LABORATORY CHLORIDE CONTENT TEST RESULTS EPA NO. 325.3

Sample No.	Chloride Ion Content (%)
B1 @ 12.5'	0.008

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

Sample No.	Water Soluble Sulfate (% SO_4)	Sulfate Exposure*
B1 @ 12.5'	0.013	Negligible

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