GEOTECHNICAL INVESTIGATION

PROPOSED MIXED-USE MULTI-FAMILY RESIDENTIAL DEVELOPMENT 207 EAST SEASIDE WAY LONG BEACH, CALIFORNIA

GEOCON
WEST, INC.

GEOTECHNICAL
ENVIRONMENTAL
MATERIALS

PREPARED FOR

ENSEMBLE INVESTMENTS, LLC
LONG BEACH, CALIFORNIA

PROJECT NO. A9124-06-01

MAY 12, 2014

Project No. A9124-06-01
May 12, 2014

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

Attention: Mr. Kambiz Babaoff

Subject: GEOTECHNICAL INVESTIGATION
PROPOSED MULTI-FAMILY RESIDENTIAL DEVELOPMENT
207 EAST SEASIDE WAY, 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 mixed-use multi-family residential development located at 207 East Seaside Way 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.

Petrina Zen

PROFESSIONAL GEOLOGIST SUSAN FRANZEN KIRKGARD No. 1754 CERTIFIED ENGINEERING GEOLOGIST STATE OF CALIFORNIA

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GEOTECHNICAL INVESTIGATION

1. PURPOSE AND SCOPE

This report presents the results of our geotechnical investigation for a proposed mixed-use multi-family residential development located at 207 East Seaside Way 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 16, 2014 by excavating two 4⅞-inch diameter borings utilizing a mud rotary drilling machine. The borings were advanced to depths of 50½ 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 CONDITIONS & PROJECT DESCRIPTION

The subject site is located at 207 East Seaside Way in 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 an alley way and multi-story mixed-use residential structure to the north, by Collins Way to the east, by Seaside Way to the south, and by South Locust Avenue 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 on site consists of trees and bushes in isolated planter areas.

Based on the information provided to us by the Client, it is our understanding that the proposed development will consist of a five-story mixed-use multi-family residential structure to be constructed over two levels of podium parking to be constructed at or near present site grade. It is our further understanding that the proposed development will connect with a proposed pedestrian bridge between Collins Way and the future Convention Center Walkway on the podium level.

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 800 kips, and wall loads are estimated to be up to 8 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 2.5 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

A thin layer of artificial fill was encountered throughout the area of proposed development. Artificial fill was observed in both borings 1 and 2 to depths of 1½ feet and 4 feet, respectively. The artificial fill generally consists of dark brown sandy clay. The artificial fill is characterized as slightly moist and stiff, with varying amounts of brick fragments. 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 silty sand, and silt with varying amounts of shell fragments and minor clay layers. 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 5 feet below the existing ground surface. Based on the depth of groundwater observed in our borings, groundwater could be encountered during construction. 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 7.14).

6. GEOLOGIC HAZARDS

6.1 Surface Fault Rupture

The numerous faults in Southern California include active, potentially active, and inactive faults. The criteria for these major groups are based on criteria developed by the California Geological Survey (CGS) 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 2.5 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 4.4 miles southwest, 12 miles west-northwest, 18 miles northeast, 23½ miles northwest, and 24 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 6.2 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, 18½ miles north-northwest, and 19 miles north 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	68	E
Near Redlands	July 23, 1923	6.3	56	ENE
Long Beach	March 10, 1933	6.4	16	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	21	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 0.662g.

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	1.607g	Figure 1613.3.1(1)
MCE_R Ground Motion Spectral Response Acceleration – Class B (1 sec), S_1	0.604g	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}	1.607g	Section 1613.3.3 (Eqn 16-37)
Site Class Modified MCE_R Spectral Response Acceleration – (1 sec), S_{M1}	0.906g	Section 1613.3.3 (Eqn 16-38)
5% Damped Design Spectral Response Acceleration (short), S_{DS}	1.071g	Section 1613.3.4 (Eqn 16-39)
5% Damped Design Spectral Response Acceleration (1 sec), S_{D1}	0.604g	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	0.627g	Figure 22-7
Site Coefficient, F_{PGA}	1.0	Table 11.8-1
Site Class Modified MCE_G Peak Ground Acceleration, PGA_M	0.627g	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 5 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 analysis, 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 northeast 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 The depth of existing artificial fill encountered on the site during the field explorations was observed to be variable with a maximum depth of approximately four feet. The existing fill encountered is believed to be the result of past grading and demolition activities at the site. Deeper fill may exist in other areas of the site that were not directly explored. It is our opinion that the existing fill, in its present condition, is not suitable for direct support of proposed foundations or slabs. The existing fill and site soils are suitable for re-use as engineered fill provided the recommendations in the *Grading* section of this report are followed (see Section 7.4).

7.1.3 The enclosed liquefaction analyses indicate that the site soils below the groundwater table could be prone to approximately 0.1 inch of total settlement as a result of the DBE ground motion. The differential settlement at the ground surface is anticipated to be negligible. The grading and foundation recommendations presented herein are intended to mitigate the effects of potential settlement on proposed improvements.

7.1.4 Based on the presence of shallow groundwater as well as the nature of the earth materials that will be exposed at the excavation bottom, it is recommended that the proposed structure be supported on a reinforced concrete mat foundation system. The mat foundation should derive support in newly placed engineered fill and or competent dense alluvial soils.

7.1.5 As a minimum, it is recommended that the upper four feet of existing site soils be excavated and properly compacted within the footprint area of the proposed structure. The excavation should extend laterally a minimum distance of three feet beyond the structure footprint area or for a distance equal to the depth of fill below the foundation, whichever is greater. Any encountered deeper fill or soft soils encountered above groundwater level should be completely over-excavated or stabilized as necessary at the direction of the Geocon representative. Recommendations for earthwork are provided in the *Grading* section of this report (see Section 7.4).

7.1.6 If the required lateral over-excavation cannot be performed, it may be acceptable to limit the engineered fill blanket to the footprint area of the mat foundation with a reduced lateral over-excavation; however, this is dependent upon the positioning of the structure and this must first be approved by the Geotechnical Engineer in writing. If the lateral over-excavation is not conducted, the lateral component of foundation capacity can rely solely on friction between the bottom of the mat and the underlying subgrade soils, and should not utilize passive pressure unless foundations are bounded by and in direct contact with newly placed engineered fill.

7.1.7 Prior to placing any fill, the excavation bottom must be proof-rolled in the presence of the Geotechnical Engineer (a representative of Geocon). If determined to be excessively soft, stabilization of the bottom of the excavation may be required in order to provide a firm working surface upon which engineered fill can be placed and heavy equipment can operate. All foundation excavations must be observed and approved by the Geotechnical Engineer (a representative of Geocon), prior to placing fill. Recommendations for earthwork and bottom stabilization are provided in the *Grading* section of this report (see Section 7.4).

7.1.8 Where new paving is to be placed, it is recommended that all existing fill be excavated and properly compacted for paving support. The client should be aware that excavation and compaction of all existing 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 soil should be scarified and properly compacted for paving support. Paving recommendations are provided in *Preliminary Pavement Recommendations* section of this report (see Section 7.10).

7.1.9 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.

7.1.10 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.

7.1.11 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 7.13).

7.2.4 The upper four feet of existing site soils encountered during the field investigation are considered to have a "moderate" expansive potential (EI = 65); and are classified as "expansive", based on the 2013 California Building Code (CBC) Section 1803.5.3. 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 Grading

7.4.1 The grading recommendations presented herein are intended to reduce the potential for settlement as well as prevent grading operations from extending into the shallow groundwater level. It is recommended that potholing be performed at each corner of the building footprint area prior to commencement of site grading activities to establish the groundwater level at the time of grading. The depth of the groundwater will influence the limits and depths of grading. Direction will be provided by the Geotechnical Engineer (a representative of Geocon West, Inc.) during grading activities.

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

7.4.3 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.

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

7.4.5 It is recommended that the proposed development be supported on a reinforced concrete mat foundation deriving support in newly placed engineered fill.

7.4.6 As a minimum, it is recommended that the upper four feet of existing site soils be excavated and properly compacted within the footprint area of the proposed structure. Any encountered deeper fill or soft soils above the groundwater level should be completely over-excavated as necessary at the direction of the Geotechnical Engineer. Where excavation and compaction is to be conducted, the excavation should extend laterally a minimum distance of three feet beyond the improvement footprint area or for a distance equal to the depth of fill below the foundation, whichever is greater. The limits of existing fill and/or soft soil removal will be verified by the Geocon representative during site grading activities.

7.4.7 If the required lateral over-excavation cannot be performed, it may be acceptable to limit the engineered fill blanket to the footprint area of the mat foundation with a reduced lateral over-excavation; however, this is dependent upon the positioning of the structure and this must first be approved by the Geotechnical Engineer in writing. If the lateral over-excavation is not conducted, the lateral foundation capacity can rely solely on friction between the bottom of the mat and the underlying subgrade soils, and should not utilize passive pressure unless foundations are bounded by and in direct contact with newly placed engineered fill or competent alluvial soils.

7.4.8 Prior to placing any fill, the excavation bottom must be proof-rolled in the presence of the Geotechnical Engineer (a representative of Geocon) and approved in writing. If determined to be excessively soft, stabilization of the bottom of the excavation may be required in order to provide a firm working surface upon which engineered fill can be placed and heavy equipment can operate.

7.4.9 Due to the presence of high-moisture content soils at the bottom of the excavation, pumping of the soils may occur during operation of heavy equipment. 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. It is suggested that excavation and grading be performed during the summer season to promote moisture control of the soils. In addition, the use of track equipment should be considered to minimize disturbance to the soils if they become wet at the excavation bottom. Bottom stabilization, if necessary, may be achieved by introducing a thin lift of three to six-inch diameter crushed angular rock into the soft excavation bottom. The use of crushed concrete will also be acceptable. The crushed rock should be spread thinly across the excavation bottom and pressed into the soils by track rolling or wheel rolling with heavy equipment. It is very important that voids between the rock fragments are not created so the rock must be thoroughly pressed or blended into the soils.

7.4.10 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 compacted to at least 90 percent relative compaction, as determined by ASTM Test Method D 1557 (latest edition). The upper twelve inches of subgrade soil below the slab-on-grade or paving section should be compacted to at least 92 percent relative compaction.

7.4.11 All imported fill shall be observed, tested, and approved by Geocon West, Inc. prior to bringing soil to the site. Rocks larger than six inches in diameter shall not be used in the fill. Imported soils should have an expansion index less than 65 and soil corrosivity properties that are equally or less detrimental than that of the existing onsite soils.

7.4.12 Where new paving is to be placed, it is recommended that all existing fill and soft alluvial soils be excavated and properly compacted for paving support. The client should be aware that excavation and compaction of all existing fill and soft soils in the area of new paving is not required; however, paving constructed over existing uncertified fill or unsuitable alluvial soil may experience increased settlement and/or cracking, and may therefore have a shorter design life and increased maintenance costs. As a minimum, the upper twelve inches of soil should be scarified, moisture conditioned to optimum moisture content, and compacted to at least 92 percent relative compaction for paving support. Paving recommendations are provided in *Preliminary Pavement Recommendations* section of this report (see Section 7.10).

7.4.13 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).

7.4.14 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.

7.5 Shrinkage

7.5.1 Shrinkage results when a volume of material removed at one density is compacted to a higher density. A shrinkage factor of between 2 and 7 percent should be anticipated when excavating and compacting the existing earth materials on the site to an average relative compaction of 92 percent. Based on the potential for shrinkage during compaction, import soils may be required to maintain site elevations.

7.6 Mat Foundation Design

7.6.1 Subsequent to the recommended grading, a reinforced concrete mat foundation may be utilized for support of the proposed structure. The mat foundations should derive support in the newly placed engineered fill and or competent dense alluvial soils.

7.6.2 The recommended maximum allowable bearing value is 3,500 pounds per square foot. The allowable bearing pressure may be increased by up to one-third for transient loads due to wind or seismic forces.

7.6.3 A vertical modulus of subgrade reaction of 150 pounds per cubic inch may be used in the design of mat foundations deriving support in newly placed engineered fill and or competent dense alluvial soils. 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: KR = reduced subgrade modulus
K = unit subgrade modulus
B = foundation width (in feet)

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

7.6.5 For seismic design purposes, a coefficient of friction of 0.30 may be utilized between concrete slab and engineered fill or stabilized subgrade, and 0.15 for slabs underlain by a vapor retarder.

7.6.6 If any portion of the proposed structure will penetrate below the water table (assumed 5 feet below ground surface) such as an elevator pit, it must be designed for 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.

7.6.7 Slabs-on-grade at the ground surface that may receive moisture-sensitive floor coverings or may be used to store moisture-sensitive materials should be underlain by a vapor retarder placed directly beneath the slab. The vapor retarder and acceptable permeance should be specified by the project architect or developer based on the type of floor covering that will be installed. The vapor retarder design should be consistent with the guidelines presented in Section 9.3 of the American Concrete Institute's (ACI) *Guide for Concrete Slabs that Receive Moisture-Sensitive Flooring Materials* (ACI 302.2R-06) and should be installed in general conformance with ASTM E 1643-09 and the manufacturer's recommendations. If the Green Building 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 the clean aggregate suggested in the Green Building Code, it is our opinion that the concrete slab-on-grade may be underlain by a vapor retarder over 4-inches of clean sand (sand equivalent greater than 30), since the sand will serve as a capillary break and will minimize the potential for punctures and damage to the vapor barrier.

7.6.8 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.

7.6.9 The maximum expected static settlement for proposed improvements supported on a mat foundation system deriving support in newly placed engineered fill or competent alluvium 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.

7.6.10 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.

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

7.7 Miscellaneous Foundations

7.7.1 Foundations for small outlying structures, such as property line walls less than 6 feet in height, planter walls or trash enclosures, which will not be tied-in to the proposed structures, may be supported on conventional foundations bearing on a minimum of 12 inches of newly placed engineered fill. Where removal and compaction cannot be performed, such as adjacent to property lines, foundations may bear in the undisturbed alluvial soils. 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. Should the soils exposed in the excavation bottom be soft, compaction of the soft soils will be required prior to placing steel or concrete. Compaction of the foundation excavation bottom is typically accomplished with a compaction wheel or mechanical whacker. As an alternative, excavations should be deepened as necessary to extend into satisfactory soils.

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

7.8 Lateral Design

7.8.1 Resistance to lateral loading may be provided by friction acting at the base of foundations, slabs and by passive earth pressure. An allowable coefficient of friction of 0.30 may be used with the dead load forces in the properly compacted engineered fill, stabilized subgrade, and undisturbed alluvial soils.

7.8.2 Passive earth pressure for the sides of foundations and slabs poured against properly compacted engineered fill, stabilized subgrade, and undisturbed alluvial soils 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.

7.8.3 If the required `lateral over-excavation cannot be performed, it may be acceptable to limit the engineered fill blanket to the footprint area of the mat foundation with no lateral over-excavation; however, this is dependent upon the positioning and load distribution of the structure on the mat foundation and must first be approved by the Geotechnical Engineer. If the lateral over-excavation is not conducted, the lateral foundation capacity can rely solely on friction between the bottom of the mat and the underlying subgrade soils, and should not utilize passive pressure unless foundations are bounded by and in direct contact with newly placed engineered fill.

7.9 Exterior Concrete Slabs-on-Grade

7.9.1 Concrete slabs-on-grade subject to vehicle loading that are not a part of the mat foundation system should be designed in accordance with the recommendations in the *Preliminary Pavement Recommendations* section of this report (Section 7.10).

7.9.2 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 subgrade should be moisture conditioned to near optimum moisture content and properly compacted. Crack control joints should be spaced at intervals not greater than 10 feet and should be constructed using saw-cuts or other methods as soon as practical following concrete placement. Crack control joints should extend a minimum depth of one-fourth the slab thickness. Construction joints should be designed by the project structural engineer.

7.9.3 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.

7.10 Preliminary Pavement Recommendations

7.10.1 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).

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

7.10.3 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).

7.10.4 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).

7.10.5 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.

7.11 Elevator Pit Design

7.11.1 The elevator pit slab and retaining wall should be designed by the project structural engineer. Provided retaining walls are properly drained, walls not restrained at the top and having a level backfill surface should be designed utilizing an equivalent fluid pressure of 30 pounds per cubic foot.

The equivalent fluid pressure to be used in design of the non-drained elevator pit retaining walls would be 90 pounds per cubic foot. The value includes hydrostatic pressures plus buoyant lateral earth pressures.

7.11.2 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.

7.11.3 If retaining wall drainage is to be provided, the drainage system should extend at least two-thirds the height of the wall. At the base of the drain system, a subdrain covered with a minimum of 12 inches of gravel should be installed, and a compacted fill blanket or other seal placed at the surface (see Figure 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.

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

7.11.5 It is suggested that the exterior walls and slab 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.

7.12 Elevator Piston

7.12.1 If a plunger-type elevator piston is installed for this project, a deep drilled excavation will be required. It is important to verify that the drilled excavation is not situated immediately adjacent to a foundation, or the drilled excavation could compromise the existing foundation support.

7.12.2 Casing will likely be required since caving is expected in the drilled excavation, especially below the groundwater level. The contractor should be prepared to use casing and should have it readily available at the commencement of drilling activities. The contractor should also be prepared to mitigate buoyant forces during installation of the piston casing. Continuous observation of the drilling and installation of the elevator piston by the Geotechnical Engineer (a representative of Geocon West, Inc.) is required.

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

7.13 Temporary Excavations

7.13.1 Excavations up to five feet in vertical height may be required for the grading activities. The excavations are expected to expose artificial fill and alluvial soils, which are suitable for vertical excavations up to five feet where loose soils or caving sands are not present. 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.

7.13.2 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 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. If necessary, shoring and/or alternative temporary excavation recommendations will be provided in an addendum.

7.13.3 Where sloped embankments are utilized, the top of the slope should be barricaded to prevent vehicles and storage loads at the top of the slope within a horizontal distance equal to the height of the slope. If the temporary construction embankments are to be maintained during the rainy season, berms are suggested along the tops of the slopes where necessary to prevent runoff water from entering the excavation and eroding the slope faces. Our personnel should inspect the soils exposed in the cut slopes during excavation so that modifications of the slopes can be made if variations in the soil conditions occur. All excavations should be stabilized within 30 days of initial excavation.

7.14 Surface Drainage

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

7.14.2 All site drainage should be collected and transferred to the street in non-erosive drainage devices. Drainage should not be allowed to pond anywhere on the site, and especially not against any foundation. Drainage should not be allowed to flow uncontrolled over any descending slope. The proposed structure should be provided with roof gutters. Discharge from downspouts, roof drains and scuppers not recommended onto unprotected soils within five feet of the building perimeter. Planters which are located adjacent to foundations should be sealed to prevent moisture intrusion into the engineered fill providing foundation support. Landscape irrigation is not recommended within five feet of the building perimeter footings except when enclosed in protected planters.

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

7.14.4 Landscaping planters immediately adjacent to paved areas are not recommended due to the potential for surface or irrigation water to infiltrate the pavement's subgrade and base course. Either a subdrain, which collects excess irrigation water and transmits it to drainage structures, or 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.

7.14.5 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.

7.15 Plan Review

7.15.1 Grading, foundation, and shoring plans should be reviewed by the Geotechnical Engineer prior to finalization to verify that the plans have been prepared in substantial conformance with the recommendations of this report and to provide additional analyses or recommendations, if necessary.

LIMITATIONS AND UNIFORMITY OF CONDITIONS

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

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

3. The findings of this report are valid as of the present date. However, changes in the conditions of a property can occur with the passage of time, whether they are due to natural processes or the works of man on this or adjacent properties. In addition, changes in applicable or appropriate standards may occur, whether they result from legislation or the broadening of knowledge. Accordingly, the findings of this report may be invalidated wholly or partially by changes outside our control. Therefore, this report is subject to review and should not be relied upon after a period of three years.

LIST OF REFERENCES

Abrahamson, N., and Silva, J. W., 2008, *Summary of the Abrahamson and Silva NGA Ground-Motion Relations,* Earthquake Spectra, Volume 24, pp 67-97.

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

Applied Technology Council, 1978, *Tentative Provisions for Development of Seismic Regulations for Buildings*, ATC Publication ATC 3-06, NBS Special Publication 510, NSF Publication 78-8.

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

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

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

Bryant, W.A. and Hart, E.W., 2007, *Fault-Rupture Hazard Zones in California, Alquist-Priolo Earthquake Fault Zoning Act with Index to Earthquake Fault Zone Maps*, California Division of Mines and Geology Special Publication 42, interim revision.

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

California Division of Mines and Geology, 1986; *Alquist Priolo Special Studies Zones, Long Beach Quadrangle, Los Angeles County, California*.

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

California Division of Mines and Geology, 1999; *Seismic Hazard Zones, Long Beach Quadrangle, Los Angeles County, California*, Official Map, Released: March 25, 1999.

California Division of Mines and Geology, 1998, *Seismic Hazard Evaluation of the Long Beach 7.5-Minute Quadrangles, Los Angeles County, California,* Open File Report 98-19.

California Division of Oil, Gas and Geothermal Resources, 2003, *Wilmington Oil Field (Eastern Portion), Los Angeles and Orange Counties, Map Number W131*.

California Division of Oil, Gas and Geothermal Resources, 2004, *Regional Wildcat Map, Counties: Los Angeles and Orange and Southern Los Angeles Basin, Map Number W1-6*.

California Geological Survey, 2013, www.quake.ca.gov/gmaps,WH/regulatory maps.htm.

LIST OF REFERENCES (Continued)

California Geological Survey, 2009, Tsunami Inundation Map for Emergency Planning, State of California, County of Los Angeles, Long Beach Quadrangle, March 1, 2009.

Chang, S.W., et al., 1994, *Ground Motions and Local Site Effects*, University of California at Berkeley Earthquake Engineering Research Center, Report No. UCB/EERC-94/08, p.28.

FEMA, 2008, *Flood Insurance Rate Map, Los Angeles, California and Incorporated Areas, Panel 1964 of 2350, Map Number 06037C1964F;* Online Flood Hazard Maps, http://www.esri.com/hazards/index.html.

Hauksson, E., 1992, *Seismicity, Faults, and Earthquake Potential in Los Angeles, Southern California,* Engineering Geology Practice in Southern California, Special Publication No. 4, Association of Engineering Geologists.

Houston, J. R., and Garcia, A. W., 1974, *Type 16 Flood Insurance Study: Tsunami Predictions for Pacific Coastal Communities*, U.S. Army Engineer Waterways Experiment Station, Hydraulic Laboratory.

Ishihara, K., 1985, Stability of Natural Deposits During Earthquakes, Proceedings of the Eleventh International Conference on Soil Mechanics and Foundation Engineering, A. A. Balkema Publishers, Rotterdam, Netherlands, vol. 1, pp. 321-376.

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

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

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

Los Angeles County Department of Public Works, 2014, Ground Water Wells Website, http://dpw2.co.la.ca.us/website/wells/viewer.asp.

Los Angeles County Department of Public Works, 2014, Flood Zone Determination Website, http://dpw.lacounty.gov/apps/wmd/floodzone/map.htm.

Long Beach, City of, 2004, *Public Safety Element of the Long Beach General Plan Program.*

McGuire, R. K., 1978, *FRISK-A computer based program for seismic risk analysis.* U. S. Geological Survey Open-File Report 78-1007.

Poland, J.F. and Piper, A.M., 1956, *Ground-water Geology of the Coastal Zone, Long Beach – Santa Ana Area, California*, U.S. Geological Survey, Water Supply Paper – 1109.

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

LIST OF REFERENCES (Continued)

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

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

Woodward-Clyde Consultants, 1988, *Revisions to City of Long Beach Seismic Safety , Prepared for the City of Long Beach, California, Prepared by Woodward-Clyde Consultants (Project No. 8743099A), dated August 9, 1988.*

Yerkes, R. F., McCulloch, T.H., Schoellhamer, J.E., and Vedder, J.G., 1965, "Geology of the Los Angeles Basin–An Introduction," *U.S. Geological Survey Professional Paper 420-A.*

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

APPENDIX A

FIELD INVESTIGATION

The site was explored on April 16, 2014 by excavating two 4⅞-inch diameter borings utilizing a mud rotary drilling machine. The borings were advanced to depths of 50½ 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 and bulk samples were also obtained.

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

PROJECT NO. A9124-06-01

BORING 1

ELEV. (MSL.) - -   DATE COMPLETED 4/16/14

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			
					Sandy Clay, stiff, slightly moist, dark brown, fine-grained			
2	B1@2.5'				**ALLUVIUM** Silty sand, medium dense, slightly moist, olive brown, fine-grained	26		22.7
4			▼					
	B1@5'				-Moist	62	94.1	23.9
6	B1@7.5'					19		31.3
8					-Some shells			
10	B1@10'				-Fine- to medium-grained, increase in silt content	52	98.1	20.7
12	B1@12.5'					20		27.7
14				SM	-Fine-grained			
	B1@15'					77	93.3	27.1
16	B1@17.5'					26		25.3
18					-Fine- to medium-grained			
					-Medium- to coarse-grained, some shells			
20	B1@20'				-Dense, fine-grained	73	94.9	24.8
22	B1@22.5'				-Trace medium-grained	31		24.8
24					-Very dense			
	B1@25'					54 (6")	101.9	21.5
26					-Reddish brown, medium- to coarse-grained			
	B1@27.5'				Clay, stiff, moist, olive brown with oxidation mottles, trace clay, trace fine-grained sand	23		33.4
28				CL				

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

A9124-06-01 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 1

ELEV. (MSL.) **- -** DATE COMPLETED **4/16/14**

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	B1@30'			CL	-Dark yellowish brown	49	109.7	17.7
32	B1@32.5'					39		13.9
					-Olive brown, decrease in silt content			
34				SM	Silty sand, very dense, moist, olive brown, fine- to medium-grained			
	B1@35'					60 (6")	103.8	21.7
36					-Fine-grained with some medium-grained			
	B1@37.5'					47		27.0
38								
40	B1@40'				-Yellowish brown, fine-grained	53 (6")	94.2	26.7
42	B1@42.5'				-Fine- to medium-grained	60		22.8
44					-Olive brown, fine-grained			
	B1@45'					50 (5")	93.8	29.4
46								
	B1@47.5'					55		26.9
48								
					-Fine-grained with some medium-grained			
50	B1@50'					50 (4.5")	95.6	24.8

Total depth of boring: 50.5 feet.
Fill to 1.5 feet.
Groundwater encountered at 5 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 2 of 2

A9124-06-01 BORING LOGS.GPJ

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

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GEOCON

PROJECT NO. A9124-06-01

BORING 2

ELEV. (MSL.) **- -** DATE COMPLETED **4/16/14**

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	B2@0-4'				**ASPHALT: 3" BASE: 3"**			
					ARTIFICIAL FILL			
2					Sandy Clay, stiff, slightly moist, dark brown, fine-grained with some medium- to coarse-grained, some brick fragments, trace roots			
	B2@2.5'					34	112.3	12.5
					-Decrease in clay content			
4					**ALLUVIUM**			
	B2@5'		▼		Silty sand, dense, slightly moist, olive brown, fine-grained	31		22.2
					-Moist			
6								
8	B2@7.5'			SM		67	96.6	23.3
10	B2@10'					23		28.0
					-Fine- to medium-grained, increase in silt content, trace shells			
12					-Wet, oxidation staining, medium-grained with some coarse-grained, trace fine gravel, some shells			
	B2@12.5'					60	82.7	42.9
				CL	Clay, hard, moist, olive brown			
14					Silty sand, medium dense, wet, olive brown, fine-grained			
	B2@15'					29		27.6
16					-Oxidation staining			
18	B2@17.5'					63	95.7	24.7
				SM				
20	B2@20'					26		28.9
					-Fine-grained with trace medium-grained			
22	B2@22.5'					78	96.5	24.0
24	B2@25'				-Fine- to medium-grained, reddish brown, trace clay, trace fine gravel	35		35.3
26					-Fine-grained			
28	B2@27.5'					68	95.5	24.6
					Clay, hard, moist, olive brown			
				CL	-Gray			

A9124-06-01 BORING LOGS.GPJ

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

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

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

GEOCON

PROJECT NO. A9124-06-01

BORING 2

ELEV. (MSL.) - - DATE COMPLETED 4/16/14

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'			CL		18		34.5
					-Yellowish brown, some fine- to medium-grained sand			
32					Silty sand, very dense, moist, olive brown, fine- to medium-grained			
	B2@32.5'					50 (5")	125.8	11.3
34					-Yellowish brown, medium-grained with some coarse-grained, trace fine gravel			
	B2@35'					57		16.1
36					-Fine-grained			
38	B2@37.5'			SM		81	108.9	18.1
40	B2@40'					40		28.4
					-Increase in sand content			
42					-Very dense			
	B2@42.5'					50 (5")	102.0	23.7
					-Fine- to medium-grained			
44	B2@45'					51		27.6
46					-Fine-grained			
48	B2@47.5'					50 (5.5")	96.6	23.6
50	B2@50'					61		26.8

Total depth of boring: 50.5 feet.
Fill to 4 feet.
Groundwater encountered at 5 feet.
Backfilled with bentonite chips and cement.
Asphalt patched.

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

A9124-06-01 BORING LOGS.GPJ

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

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

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

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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.