GEOTECHNICAL INVESTIGATION

PROPOSED COMMERCIAL BUILDING
601 OCEAN FRONT WALK
VENICE DISTRICT
LOS ANGELES, CALIFORNIA
TRACT: GOLDEN BAY TRACT
BLOCK: BLK 6   LOTS: 287

PREPARED FOR
BOARDWALK SUNSET LLC
VENTURA, CALIFORNIA

PROJECT NO. A9297-06-01

JULY 15, 2015
Project No. A9297-06-01
July 15, 2015

Boardwalk Sunset LLC
c/o Pacific Coast Management
PO BOX 25070
Ventura, California 93002

Attention: Mr. Gordon Freitas

Subject: GEOTECHNICAL INVESTIGATION
PROPOSED COMMERCIAL BUILDING
601 OCEAN FRONT WALK
VENICE DISTRICT, LOS ANGELES, CALIFORNIA
TRACT: GOLDEN BAY TRACT
BLOCK: BLK 6    LOT: 287

Dear Mr. Freitas:

In accordance with your authorization of our proposal dated June 18, 2014, we have performed a geotechnical investigation for the proposed commercial building located at 601 Ocean Front Walk in the Venice District of the City of Los Angeles, California. The accompanying report presents the findings of our study, and our conclusions and recommendations pertaining to the geotechnical aspects of proposed design and construction. Based on the results of our investigation, it is our opinion that the site can be developed as proposed, provided the recommendations of this report are followed and implemented during design and construction.

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

Very truly yours,

GEOCON WEST, INC.

Thai La
Staff Engineer

Susan F. Kirkgard
CEG 1754

Harry Derkalousdian
PE 79694

Addressee
City of Los Angeles, Submitted by Geocon
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GEOTECHNICAL INVESTIGATION

1. PURPOSE AND SCOPE

This report presents the results of a geotechnical investigation for the proposed commercial building located at 601 Ocean Front Walk in the Venice District of the City of Los Angeles, California (see Vicinity Map, Figure 1). The purpose of the investigation was to evaluate subsurface soil and geologic conditions underlying the site and, based on conditions encountered, to provide conclusions and recommendations pertaining to the geotechnical aspects of 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 June 27, 2015, by excavating one 4\(\frac{7}{8}\) inch diameter boring to a depth of approximately 75½ feet below the existing ground surface utilizing a truck-mounted reverse circulation mud rotary drilling machine. The approximate location of the exploratory boring is depicted on the Site Plan (see Figure 2). A detailed discussion of the field investigation, including boring log, is presented in Appendix A.

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

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

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

2. SITE AND PROJECT DESCRIPTION

The subject site is located at 601 Ocean Front Walk in the Venice District of the City of Los Angeles, California. The site is a rectangular shaped parcel and is currently occupied by an on-grade paved parking lot. The site is bounded by Speedway (alley) to the northeast, by an on-grade three story commercial structure to the southeast, by Ocean Front Walk to the southwest, and by Sunset Avenue to the northwest. The site is gently sloping to the west-southwest. Surface water drainage at the site appears to be by sheet flow along the existing ground contours to the city streets.

Based on information provided by the client, it is our understanding that the proposed development will consist of a three-story commercial structure to be constructed over two subterranean parking levels (see Site Plan and Cross Section, Figures 2 and 2A).
Based on the preliminary nature of the design at this time, wall and column loads were not available. It is anticipated that column loads for the proposed residential structures will be up to 800 kips, and wall loads will be up to 8 kips per linear foot.

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

3. GEOLOGIC SETTING

The site is located in the northwestern portion of the Los Angeles Basin, a coastal plain between the Santa Monica Mountains to the north, the Puente Hills and Whittier Fault to the east, the Palos Verdes Peninsula and Pacific Ocean to the south and west, and the Santa Ana Mountains and San Joaquin Hills to the southeast. The prominent structural features within the Los Angeles Basin include the central lowland plain, the uplifted Palos Verdes Hills, and a northwest-trending line of low hills and mesas within the Newport-Inglewood Fault Zone. Regionally, the site is located in the northern portion of the Peninsular Ranges geomorphic province, characterized by northwest-trending physiographic and geologic features.

4. SOIL AND GEOLOGIC CONDITIONS

Based on our field exploration and published geologic maps of the area, the site is underlain by artificial fill and Holocene age alluvium consisting of fine- to coarse-grained sand, silt and gravel (Dibblee, 2007). Detailed stratigraphic profiles are provided on the boring log in Appendix A.

4.1 Artificial Fill

Artificial fill was encountered during site in our boring to a maximum depth of 3 feet below ground surface. The artificial fill generally consists of yellowish brown silty sand with some fine gravel. The artificial fill is characterized as slightly moist and medium dense. The fill is likely the result of past grading and construction activities at the site. Deeper fill may exist in other portions of the site that were not directly explored.

4.2 Alluvium

The artificial fill is underlain by Holocene age alluvium that is in turn underlain by Pleistocene age alluvium. Alluvial soils were encountered to a depth of approximately 75½ feet beneath the existing ground surface (maximum depth explored). The alluvium generally consists of pale brown to yellowish brown poorly graded sand, silty sand, and sand with gravel. The deposits range from fine- to coarse-grained and are characterized as primarily moist to wet and medium dense to very dense.
5. GROUNDWATER

Based on a review of the Seismic Hazard Evaluation of the Venice Quadrangle (California Division of Mines and Geology, 1998), the historic high groundwater level beneath the site is 5 feet below the existing ground surface. The historic high groundwater level is based on available groundwater records from the early 1900’s to late 1990’s. Based on current groundwater basin management practices, it is unlikely that the groundwater levels will ever exceed the historic high levels.

The Los Angeles County Department of Public Works (LACDPW) has maintained various groundwater monitoring wells in the vicinity of the subject site over the past 50 years. The closest LACDPW monitoring well to the site is Well ID No. 2539L (State No. 2S15W09N09) located approximately 1.3 miles to the northeast (LACDPW, 2015a). Due to the distance between the site and this monitoring well, groundwater level measurements from this well would not be representative of the groundwater conditions at the site (LACDPW, 2015a).

The proposed development is planned to be constructed over two levels of subterranean parking. Groundwater was encountered in our boring at a depth of 17½ feet below ground surface. Based on the depth to groundwater encountered in our boring and the historic high groundwater level, we anticipate groundwater will be encountered during construction.

It is not uncommon for groundwater levels to vary seasonally or for perched groundwater conditions to develop where none previously existed, especially in impermeable fine-grained soils which are subjected to irrigation or precipitation. In addition, recent requirements for stormwater infiltration could result in shallower seepage conditions in the region. Proper surface drainage of irrigation and precipitation will be critical to future performance of the project. Recommendations for drainage are provided in the Surface Drainage section of this report (see Section 7.24).

6. GEOLOGIC HAZARDS

6.1 Surface Fault Rupture

The numerous faults in Southern California include active, potentially active, and inactive faults. The criteria for these major groups are based on criteria developed by the California Geological Survey (CGS, formerly known as California Division of Mines and Geology [CDMG]) 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. No active or potentially active faults with the potential for surface fault rupture are known to pass directly beneath the site. Therefore, the potential for surface rupture due to faulting occurring beneath the site during the design life of the proposed development is considered low. However, the site is located in the seismically active Southern California region, and could be subjected to moderate to strong ground shaking in the event of an earthquake on one of the many active Southern California faults. The faults in the vicinity of the site are shown in Figure 3, Regional Fault Map.

The closest trace of an active fault to the site is the Santa Monica Fault located approximately 2.8 miles to the north (Ziony and Jones, 1989). Other nearby active faults are the Newport-Inglewood Fault Zone, the Hollywood Fault, the Malibu Coast Fault, and the Palos Verdes Hills Fault Zone located approximately 6.1 miles east-northeast, 7.5 miles northeast, 10.7 miles northwest and 12.8 miles south-southeast of the site, respectively (Ziony and Jones, 1989). The active San Andreas Fault Zone is located approximately 42 miles northeast of the site.

The closest potentially active fault to the site is the Charnock Fault located approximately 3.2 miles to the east-northeast (Ziony and Jones, 1989). Other nearby potentially active faults include the Overland Fault and the MacArthur Park Fault located approximately 4.2 miles east and 12.3 miles northeast 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 Mw 5.9 Whittier Narrows earthquake and the January 17, 1994 Mw 6.7 Northridge earthquake were a result of movement on the Puente Hills Blind Thrust and the Northridge Thrust, respectively. These thrust faults are not exposed at the surface and do not present a potential surface fault rupture hazard at the site; however, these are active features capable of generating future earthquakes that could generate strong ground shaking at the site.

6.2 Seismicity

As with all of Southern California, the site has experienced historic earthquakes from various regional faults. The seismicity of the region surrounding the site was formulated based on research of an electronic database of earthquake data. The epicenters of recorded earthquakes with magnitudes equal to or greater than 5.0 in the site vicinity are depicted on Figure 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

<table>
<thead>
<tr>
<th>Earthquake (Oldest to Youngest)</th>
<th>Date of Earthquake</th>
<th>Magnitude</th>
<th>Distance to Epicenter (Miles)</th>
<th>Direction to Epicenter</th>
</tr>
</thead>
<tbody>
<tr>
<td>San Jacinto-Hemet area</td>
<td>April 21, 1918</td>
<td>6.8</td>
<td>86</td>
<td>E</td>
</tr>
<tr>
<td>Near Redlands</td>
<td>July 23, 1923</td>
<td>6.3</td>
<td>70</td>
<td>E</td>
</tr>
<tr>
<td>Long Beach</td>
<td>March 10, 1933</td>
<td>6.4</td>
<td>39</td>
<td>SE</td>
</tr>
<tr>
<td>Tehachapi</td>
<td>July 21, 1952</td>
<td>7.5</td>
<td>76</td>
<td>NNW</td>
</tr>
<tr>
<td>San Fernando</td>
<td>February 9, 1971</td>
<td>6.6</td>
<td>29</td>
<td>NNE</td>
</tr>
<tr>
<td>Whittier Narrows</td>
<td>October 1, 1987</td>
<td>5.9</td>
<td>23</td>
<td>E</td>
</tr>
<tr>
<td>Sierra Madre</td>
<td>June 28, 1991</td>
<td>5.8</td>
<td>33</td>
<td>ENE</td>
</tr>
<tr>
<td>Landers</td>
<td>June 28, 1992</td>
<td>7.3</td>
<td>118</td>
<td>E</td>
</tr>
<tr>
<td>Big Bear</td>
<td>June 28, 1992</td>
<td>6.4</td>
<td>95</td>
<td>E</td>
</tr>
<tr>
<td>Northridge</td>
<td>January 17, 1994</td>
<td>6.7</td>
<td>16</td>
<td>NNW</td>
</tr>
<tr>
<td>Hector Mine</td>
<td>October 16, 1999</td>
<td>7.1</td>
<td>133</td>
<td>ENE</td>
</tr>
</tbody>
</table>

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

6.3 Seismic Design Criteria

The following table summarizes 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 (MCEₚ).
### 2013 CBC Seismic Design Parameters

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
<th>2013 CBC Reference</th>
</tr>
</thead>
<tbody>
<tr>
<td>Site Class</td>
<td>D</td>
<td>Table 1613.3.2</td>
</tr>
<tr>
<td>$\text{MCE}_R$ Ground Motion Spectral Response Acceleration – Class B (short), $S_s$</td>
<td>1.863g</td>
<td>Figure 1613.3.1(1)</td>
</tr>
<tr>
<td>$\text{MCE}_R$ Ground Motion Spectral Response Acceleration – Class B (1 sec), $S_t$</td>
<td>0.691g</td>
<td>Figure 1613.3.1(2)</td>
</tr>
<tr>
<td>Site Coefficient, $F_A$</td>
<td>1.0</td>
<td>Table 1613.3.3(1)</td>
</tr>
<tr>
<td>Site Coefficient, $F_V$</td>
<td>1.5</td>
<td>Table 1613.3.3(2)</td>
</tr>
<tr>
<td>Site Class Modified $\text{MCE}<em>R$ Spectral Response Acceleration (short), $S</em>{SM}$</td>
<td>1.863g</td>
<td>Section 1613.3.3(Eqn 16-37)</td>
</tr>
<tr>
<td>Site Class Modified $\text{MCE}<em>R$ Spectral Response Acceleration – (1 sec), $S</em>{SM1}$</td>
<td>1.037g</td>
<td>Section 1613.3.3(Eqn 16-38)</td>
</tr>
<tr>
<td>5% Damped Design Spectral Response Acceleration (short), $S_{DS}$</td>
<td>1.242g</td>
<td>Section 1613.3.4(Eqn 16-39)</td>
</tr>
<tr>
<td>5% Damped Design Spectral Response Acceleration (1 sec), $S_{D1}$</td>
<td>0.691g</td>
<td>Section 1613.3.4(Eqn 16-40)</td>
</tr>
</tbody>
</table>

The table below presents the mapped maximum considered geometric mean ($\text{MCE}_G$) 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

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
<th>ASCE 7-10 Reference</th>
</tr>
</thead>
<tbody>
<tr>
<td>Mapped $\text{MCE}_G$ Peak Ground Acceleration, PGA</td>
<td>0.693g</td>
<td>Figure 22-7</td>
</tr>
<tr>
<td>Site Coefficient, $F_{PGA}$</td>
<td>1.0</td>
<td>Table 11.8-1</td>
</tr>
<tr>
<td>Site Class Modified $\text{MCE}_G$ Peak Ground Acceleration, $PGA_M$</td>
<td>0.693g</td>
<td>Section 11.8.3(Eqn 11.8-1)</td>
</tr>
</tbody>
</table>

The Maximum Considered Earthquake Ground Motion (MCE) is the level of ground motion that has a 2 percent chance of exceedance in 50 years, with a statistical return period of 2,475 years. According to the 2013 California Building Code and ASCE 7-10, the MCE is to be utilized for the evaluation of liquefaction, lateral spreading, seismic settlements, and it is our understanding that the intent of the Building code is to maintain “Life Safety” during a MCE event. The Design Earthquake Ground Motion (DE) is the level of ground motion that has a 10 percent chance of exceedance in 50 years, with a statistical return period of 475 years.
Deaggregation of the MCE peak ground acceleration was performed using the USGS 2008 Probabilistic Seismic Hazard Analysis (PSHA) Interactive Deaggregation online tool. The result of the deaggregation analysis indicates that the predominant earthquake contributing to the MCE peak ground acceleration is characterized as a 6.81 magnitude event occurring at a hypocentral distance of 9.7 kilometers from the site.

Deaggregation was also performed for the Design Earthquake (DE) peak ground acceleration, and the result of the analysis indicates that the predominant earthquake contributing to the DE peak ground acceleration is characterized as a 6.73 magnitude occurring at a hypocentral distance of 16.3 kilometers from the site.

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

6.4 Liquefaction Potential

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

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

A review of the State of California Seismic Hazard Zone Map for the Venice Quadrangle (CDMG, 1999) and the County of Los Angeles General Plan (1990) indicate the site is located within an area identified as having a potential for liquefaction.

Liquefaction analysis of the soils underlying the site was performed using an updated version of the spreadsheet template LIQ2_30.WQ1 developed by Thomas F. Blake (1996). This program utilizes the 1996 NCEER method of analysis. This semi-empirical method is based on a correlation between values of Standard Penetration Test (SPT) resistance and field performance data.
The liquefaction analysis was performed for a Design Earthquake level by using a historic high groundwater table of less than 5 feet below the ground surface, a magnitude 6.73 earthquake, and a peak horizontal acceleration of 0.586g ($\frac{2}{3}$PGAM). The enclosed liquefaction analyses, included herein for boring B1, indicates that the alluvial soils below the bottom of the proposed subterranean level would not be susceptible to liquefaction settlement during Design Earthquake ground motion (see enclosed calculation sheets, Figures 5 and 6).

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

The liquefaction analysis was also performed for Maximum Considered Earthquake levels by using a historic high groundwater table of less than 5 feet below the ground surface, a magnitude 6.81 earthquake, and a peak horizontal acceleration of 0.693g (PGAM). The enclosed liquefaction analysis, included herein for boring B1, indicates that the alluvial soils below the bottom of the proposed subterranean level would not be susceptible to liquefaction settlement during Maximum Considered Earthquake ground motion (see enclosed calculation sheets, Figures 7 and 8).

**6.6 Slope Stability**

The site slopes at a gradient flatter than 13:1 (H:V) toward the west-southwest and topographic relief across the site is on the order of 10 feet. The site is not located within a City of Los Angeles Hillside Grading Area or Hillside Ordinance Area (City of Los Angeles, 2015). The County of Los Angeles Safety Element (Leighton, 1990), indicates the site is not within an area identified as having a potential for slope instability. Additionally, the site is not within an area identified as having a potential for seismic slope instability (CDMG, 1999). There are no known landslides near the site, nor is the site in the path of any known or potential landslides. Therefore, the potential for slope stability hazards to adversely affect the proposed development is considered low.

**6.7 Earthquake-Induced Flooding**

Earthquake-induced flooding is inundation caused by failure of dams or other water-retaining structures due to earthquakes. A review of the Los Angeles County Safety Element (Leighton, 1990), indicates that the site is not located within the inundation boundaries of upgradient dams or reservoirs. Therefore, the potential for inundation at the site as a result of an earthquake-induced dam failure is considered low.
6.8 Tsunamis, Seiches, and Flooding

According to the California Geological Survey (2009) and the County of Los Angeles General Plan (Leighton, 1990), the site is located within the limits of a tsunami inundation area. There is a potential for tsunamis to impact the site.

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

The site is located within an area of minimal flooding potential (Zone X) as defined by the Federal Emergency Management Agency (LACDPW, 2015b).

6.9 Oil Fields & Methane Potential

Based on a review of the California Division of Oil, Gas and Geothermal Resources (DOGGR) Oil and Gas Well Location Map W1-5, the site is located approximately 200 feet north of the Venice Beach Oil Field and no oil wells are located in the immediate vicinity of the site. Due to the voluntary nature of record reporting by the oil well drilling companies, wells may be improperly located or not shown on the well 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 a City of Los Angeles Methane Zone (City of Los Angeles, 2015). A methane study is required for the proposed development and 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. Soils that are particularly subject to subsidence include those with high silt or clay content. The site is not located within an area of known ground subsidence. No large-scale extraction of groundwater, gas, oil, or geothermal energy is occurring or planned at the site or in the general site vicinity. There appears to be little or no potential for ground subsidence due to withdrawal of fluids or gases at the site.
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 commercial structure provided the recommendations presented herein are followed and implemented during design and construction.

7.1.2 Up to 3 feet of existing artificial fill was encountered during the site investigation. The existing fill encountered is believed to be the result of past grading and construction activities at the site. Deeper fill may exist in other areas of the site that were not directly explored. Excavation for the proposed subterranean parking level is anticipated to penetrate through the existing fill and expose competent alluvial soils throughout the excavation bottom).

7.1.3 Groundwater was encountered in boring B1 at a depth of 17.5 feet below the existing ground surface. Excavation for the proposed subterranean parking level is anticipated to extend to depths of approximately 30 feet below ground surface, including foundation construction. Based on conditions encountered at the time of exploration, groundwater should be anticipated during construction of the subterranean parking levels. Due to subterranean nature of the proposed structure and potential for seasonal fluctuation in the groundwater level, temporary dewatering measures will be required to mitigate groundwater seepage during excavation and construction. Recommendations for design flow rates for the dewatering system should be determined by a qualified contractor or dewatering consultant.

7.1.4 Based on a review of the Seismic Hazard Zone Report for the Venice Quadrangle, Los Angeles County, California, the historic high groundwater level beneath the site is less than 5 feet below the existing ground surface. If the subterranean portion of the structure, which extends below the historic high groundwater level, is not designed for full hydrostatic pressure, a permanent dewatering system will be required to relieve and mitigate the water pressure. Recommendations for Temporary and Permanent Dewatering are discussed in Section 7.4 and Section 7.5 of this report.

7.1.5 Based on these considerations, it is recommended a conventional reinforced concrete mat foundation system be utilized for support of the proposed structure provided foundations derive support in the competent alluvial soils found at the excavation bottom. Foundations should be deepened as necessary to penetrate through unsuitable soils and derive support in the competent alluvial soils. Any soils unintentionally disturbed should be properly compacted. All foundation excavations must be observed and approved in writing by the Geotechnical Engineer prior to placement of steel or concrete. Recommendations for the design of a foundation system are provided in Sections 7.8 through 7.10 of this report.
7.1.6 If a permanent dewatering system is not implemented, the structure must be designed for hydrostatic pressure based on the historic high groundwater. The hydrostatic design will result in uplift forces on the structure that must be resisted by counterweight or structural design measures.

7.1.7 The concrete ramp for the subterranean level should derive support from the mat foundation system.

7.1.8 It is recommended that the soils exposed at the excavation bottom be proof rolled prior to placing construction materials. Any disturbed soils should be properly compacted. The existing site soils are suitable for re-use as an engineered fill provided the procedures outlined in the Grading section of this report are followed (see Section 7.6).

7.1.9 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 in order to minimize soil disturbance. 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).

7.1.10 Excavations on the order of 30 feet in vertical height may be required for construction of the subterranean level, including the excavations for the dewatering system and foundation system, as indicated on Cross-Sections A-A’ (see Figure 2A). 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 sloping and/or shoring measures in order to provide a stable excavation. Where shoring is required it is recommended that a soldier pile shoring system be utilized. In addition, where the proposed excavation will be deeper than and adjacent to an offsite structure, the proposed retaining wall and shoring systems should be designed to resist the surcharge imposed by the adjacent offsite structure. Recommendations for Shoring are provided in Section 7.19.

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

7.1.12 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 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 36 inches below existing ground surface, and should be deepened as necessary to maintain a minimum of 12-inch embedment into recommended bearing materials. If the soils exposed in the excavation bottom are soft or loose, compaction of the soft soils will be required prior to placing steel or concrete. Compaction of the foundation excavation bottom is typically accomplished with a compaction wheel or mechanical whacker and must be observed and approved in writing by a Geocon representative.

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

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

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

7.1.16 The most recent ASTM standards apply to this project and must be utilized, even if older ASTM standards are indicated in 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 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 7.18).

7.2.4 The soils encountered near the proposed subterranean level are considered to have a “very low” expansive potential based on the granular nature of the soil near the proposed subterranean level, and are classified as “non-expansive” in accordance with the 2013 California Building Code (CBC) Section 1803.5.3. The recommendations presented herein assume that the building 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 the soils are considered “severely corrosive” with respect to corrosion of buried ferrous metals on site. The results are presented in Appendix B (Figure B3) and should be considered for design of underground structures. Due to the corrosive potential of the soils, it is recommended that ABS pipes are utilized in lieu of cast-iron for subdrains and retaining wall drains.

7.3.2 Laboratory tests were performed on representative samples of the on-site soil to measure the percentage of water-soluble sulfate content. Results from the laboratory water-soluble sulfate tests are presented in Appendix B (Figure B3) and indicate that the upper on-site soils possess a “negligible” sulfate exposure to concrete structures as defined by 2013 CBC Section 1904.3 and ACI 318-11 Section 4.2 and 4.3. However, the presence of seawater in the soil justifies the use of a higher cement type (possibly Type-V) to resist seawater attack on concrete.

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

7.4.1 Groundwater was encountered during site exploration at depths 17½ feet below the existing ground surface. The depth to groundwater at the time of construction can be further verified during initial dewatering well or shoring pile installation. If the proposed excavation depth will extend below the groundwater 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 12-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 6 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 
away 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.

7.5.4 Do to the requirement for a passive methane mitigation system, it is recommended that 
the methane consultant communicate with the dewatering consultant and/or project civil 
engineer to eliminate any redundancy with the methane and dewatering designs.

7.6 Grading

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

7.6.2 Earthwork should be observed, and compacted fill tested by representatives of Geocon West, 
Inc. The existing fill and alluvial soil 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.

7.6.3 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 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 in writing 
by the Geotechnical Engineer. All existing underground improvements planned for removal 
should be completely excavated and the resulting depressions properly backfilled in 
accordance with the procedures described herein. 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.) and the City of Los Angeles Inspector.

7.6.5 It is recommended that the exposed soils be proof rolled prior to placing construction materials. Any disturbed soils should be properly compacted for foundation support, as necessary.

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 3- to 6-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 The City of Los Angeles Department of Building and Safety requires a minimum compactive effort of 95 percent of the laboratory maximum dry density in accordance with ASTM D1557 (latest edition) where the soils to be utilized in the fill have less than 15 percent finer than 0.005 millimeter. Soils with more than 15 percent finer than 0.005 millimeter may be compacted to 90 percent of the laboratory maximum dry density in accordance with ASTM D1557 (latest edition). 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 95 percent of the maximum dry density in accordance with ASTM D1557 (latest edition).

7.6.10 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 compaction cannot be performed, such as adjacent to property lines, foundations may be deepened to maintain a minimum 12 inch embedment into undisturbed competent alluvial soils and must be observed and approved by a Geocon representative. 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.

7.6.11 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 6 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 30 and soil corrosivity properties that are equally or less detrimental to that of the existing onsite soils (see Figure B3).

7.6.12 Utility trenches should be properly backfilled in accordance with the requirements of the Green Book (latest edition). The pipe should be bedded with clean sands (Sand Equivalent greater than 30) to a depth of at least 1 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 (see Section 7.7).
7.6.13 All trench and foundation excavation bottoms must be observed and approved in writing by the Geotechnical Engineer (a representative of Geocon), prior to placing bedding sands, fill, steel, gravel, or concrete.

7.7 Controlled Low Strength Material (CLSM)

7.7.1 Controlled Low Strength Material (CLSM) may be utilized in lieu of compacted soil as engineered fill where approved in writing by the Geotechnical Engineer. Where utilized within the City of Los Angeles use of CLSM is subject to the following requirements:

**Standard Requirements**

1. CLSM shall be ready-mixed by a City of Los Angeles approved batch plant;
2. CLSM shall not be placed on uncertified fill, on incompetent natural soil, nor below water;
3. CLSM shall not be placed on a sloping surface with a gradient steeper than 5:1 (horizontal to vertical);
4. Placement of the CLSM shall be under the continuous inspection of a concrete deputy inspector;
5. The excavation bottom shall be accepted by the soil engineer and the City Inspector prior to placing CLSM.

**Requirements for CLSM that will be used for support of footings**

1. The cement content of the CLSM shall not be less than 188 pounds per cubic yard (min. 2 sacks);
2. The excavation bottom must be level, cleaned of loose soils and approved in writing by Geocon prior to placement of the CLSM;
3. The ultimate compressive strength of the CLSM shall be no less than 100 pounds per square inch when tested on the 28th-day per ASTM D4832 (latest edition), Standard Test Method for Preparation and Testing of Controlled Low Strength Material Test Cylinders. Compression testing will be performed in accordance with ASTM C39 and City of Los Angeles requirements;
4. Samples of the CLSM will be collected during placement, a minimum of one test (two cylinders) for each 50 cubic yards or fraction thereof;
5. Overexcavation for CLSM placement shall extend laterally beyond the footprint of any proposed footings as required for placement of compacted fill, unless justified otherwise by the soil engineer that footings will have adequate vertical and horizontal bearing capacity.
7.8 Mat Foundation Design

7.8.1 A reinforced concrete mat foundation system may be utilized for support of the proposed structure provided foundations derive support in the competent undisturbed alluvial soils found at the excavation bottom. The concrete ramp for the subterranean level should derive support from the mat foundation system. Foundations should be deepened as necessary to penetrate through unsuitable soils and derive support in the competent undisturbed alluvial soils.

7.8.2 It is anticipated that the mat foundation will impart an average pressure of less than 3,000 pounds per square foot (psf), with locally higher pressures up to 5,000 psf. The recommended maximum allowable bearing value is 6,000 psf. The allowable bearing pressure may be increased by up to one-third for transient loads due to wind or seismic forces.

7.8.3 A vertical modulus of subgrade reaction of 250 pounds per cubic inch (pci) may be used in the design of mat foundations deriving support in competent alluvial soils and/or a bed of gravel associated with the dewatering system. This value is a unit value for use with a 1-foot square footing. The modulus should be reduced in accordance with the following equation when used with larger foundations:

\[
K_r = 2 \left[ \frac{B+1}{2B} \right]^2
\]

where 
- \( K \) = unit subgrade modulus
- \( K_r \) = reduced subgrade modulus
- \( B \) = foundation width (in feet).

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

7.8.5 If a permanent dewatering system is not implemented the structure must be designed for hydrostatic pressure based on the historic high groundwater level, approximately 5 feet below the existing ground surface. The hydrostatic design will result in uplift forces on the mat foundation that that must be resisted by counterweight or structural design measures. The recommended floor slab uplift pressure to be used in design would be 62.4(H) in units of psf, where “H” is the height of the water above the bottom of the foundation in feet. If the proposed structure does not provide sufficient dead load to resist the buoyant forces then recommendations for uplift mitigation will be required. Such recommendations, if deemed necessary, will be provided under separate cover as the design becomes more finalized.
7.8.6 For seismic design purposes, a coefficient of friction of 0.40 may be utilized between the concrete mat and undisturbed alluvial soils without a moisture barrier, and 0.15 for slabs underlain by a moisture barrier.

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

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

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

7.9 Miscellaneous Foundations

7.9.1 Foundations for small outlying structures, such as block walls less than 6 feet in height, planter walls or trash enclosures, which will not be structurally supported by the proposed building, may be supported on conventional foundations deriving support on a minimum of 12 inches of newly placed engineered fill which extends laterally at least 12 inches beyond the foundation area. Where excavation and compaction cannot be performed, such as adjacent to property lines, foundations may derive support in the undisturbed alluvial soils found at or below a depth of 36 inches, and should be deepened as necessary to maintain a minimum 12-inch embedment into undisturbed competent alluvial soils and must be observed and approved by a Geocon representative.

7.9.2 If the soils exposed in the excavation bottom are soft, compaction of the soft soils will be required prior to placing steel or concrete. Compaction of the foundation excavation bottom is typically accomplished with a compaction wheel or mechanical whacker and must be observed and approved by a Geocon representative. Miscellaneous foundations may be designed for a bearing value of 1,500 psf, and should be a minimum of 12 inches in width, 18 inches in depth below the lowest adjacent grade and 12 inches into the recommended bearing material. The allowable bearing pressure may be increased by up to one-third for transient loads due to wind or seismic forces.
7.9.3 Foundation excavations should be observed and approved in writing by the Geotechnical Engineer (a representative of Geocon West, Inc.), prior to the placement of reinforcing steel and concrete to verify that the excavations and exposed soil conditions are consistent with those anticipated.

7.10 **Foundation Settlement**

7.10.1 The maximum expected total settlement for a structure supported on a mat foundation system deriving support in the recommended bearing material is estimated to be less than ½ 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 expected to be less than ¼ inch over a distance of 20 feet.

7.10.2 Once the design and foundation loading configurations 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.

7.11 **Lateral Design**

7.11.1 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 0.40 may be used with the dead load forces in the undisturbed alluvial soils and newly compacted engineered fill.

7.11.2 Passive earth pressure for the sides of foundations poured against undisturbed alluvial soils may be computed as an equivalent fluid having a density of 160 pounds per cubic foot (pcf) with a maximum earth pressure of 1,600 psf (these values have been adjusted for buoyant forces). When combining passive and friction for lateral resistance, the passive component should be reduced by one-third.

7.12 **Concrete Slabs-on-Grade**

7.12.1 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 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. The project structural engineer should design construction joints as necessary.
7.12.2 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 1643-11 and the manufacturer’s recommendations. A minimum thickness of 15 mils extruded polyolefin plastic is recommended; vapor retarders which contain recycled content or woven materials are not recommended. The vapor retarder should have a permeance of less than 0.01 perms demonstrated by testing before and after mandatory conditioning is recommended. The vapor retarder should be installed in direct contact with the concrete slab with proper perimeter seal. If the Los Angeles Green Building Code requirements apply to this project, the vapor retarder should be underlain by 4 inches of clean aggregate. It is important that the vapor retarder be puncture resistant since it will be in direct contact with angular gravel. As an alternative to the clean aggregate suggested in the Los Angeles Green Building Code, it is our opinion that the concrete slab-on-grade may be underlain by a vapor retarder over 4 inches of clean sand (sand equivalent greater than 30), since the sand will serve a capillary break and will minimize the potential for punctures and damage to the vapor barrier.

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

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

Retaining Walls Design

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

Retaining wall foundations may be designed in accordance with the recommendations provided in the Foundation Design sections of this report (see Section 7.8).

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). 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). The table below presents recommended pressures to be used in retaining wall design, assuming that proper drainage will be maintained. Calculations of the recommended retaining wall pressures are provided as Figures 9 and 10.

<table>
<thead>
<tr>
<th>HEIGHT OF RETAINING WALL (Feet)</th>
<th>ACTIVE PRESSURE EQUIVALENT FLUID PRESSURE (Pounds Per Cubic Foot)</th>
<th>AT-REST PRESSURE EQUIVALENT FLUID PRESSURE (Pounds Per Cubic Foot)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Up to 15</td>
<td>40</td>
<td>60</td>
</tr>
<tr>
<td>16 - 25</td>
<td>42</td>
<td>62</td>
</tr>
</tbody>
</table>

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.
7.13.5 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.

7.13.6 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 $\frac{x}{H} \leq 0.4$

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

and

For $\frac{x}{H} > 0.4$

$$\sigma_h(x, z) = \frac{1.26 \left( \frac{x}{H} \right)^2 \left( \frac{z}{H} \right) Q_L}{\left[ \left( \frac{x}{H} \right)^2 + \left( \frac{z}{H} \right)^2 \right]^{1.5}}$$

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, $Q_L$ is the vertical line-load and $\sigma H$ is the horizontal pressure at depth $z$. 
7.13.7 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:

\[
\text{For } \frac{x}{H} \leq 0.4 \\
\sigma(z) = \frac{0.28 \times \left( \frac{z}{H} \right)^2 \times \frac{Q_p}{H^2}}{0.16 + \left( \frac{z}{H} \right)^2} \\
\text{and} \\
\text{For } \frac{x}{H} > 0.4 \\
\sigma(z) = \frac{1.77 \times \left( \frac{x}{H} \right)^2 \times \left( \frac{z}{H} \right)^2 \times \frac{Q_p}{H^2}}{\left( \frac{x}{H} \right)^2 + \left( \frac{z}{H} \right)^2}
\]

then

\[
\sigma_{h}(z) = \sigma_{v}(z) \cos^{2}(1.1\theta)
\]

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, \(\sigma\) is the vertical pressure at depth \(z\), \(\Theta\) 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 \(\sigma_{h}\) is the horizontal pressure at depth \(z\).

7.13.8 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.

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

7.14 **Dynamic (Seismic) Lateral Forces**

7.14.1 The structural engineer should determine the seismic design category for the project in accordance with Section 1613 of the CBC. If the project possesses a seismic design category of D, E, or F, proposed retaining walls in excess of 6 feet in height should be designed with seismic lateral pressure (Section 1803.5.12 of the 2013 CBC).
7.14.2 A seismic load of 29 pcf should be used for design of walls that support more than 6 feet of backfill in accordance with Section 1803.5.12 of the 2013 CBC. The seismic load is applied as an equivalent fluid pressure along the height of the wall and the calculated loads result in a maximum load exerted at the base of the wall and zero at the top of the wall. This seismic load should be applied in addition to the active earth pressure. The earth pressure is based on half of two-thirds of \( \text{PGA}_M \) calculated from ASCE 7-10 Section 11.8.3.

7.15 Retaining Wall Drainage

7.15.1 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 13). 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.

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

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

7.15.4 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.
7.16 **Elevator Pit Design**

7.16.1 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 7.8 and 7.13).

7.16.2 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.

7.16.3 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 7.15).

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

7.17 **Elevator Piston**

7.17.1 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.

7.17.2 Casing will be required since groundwater and some caving is expected in the drilled excavation 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.

7.17.3 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.

7.18 **Temporary Excavations**

7.18.1 Excavations on the order of 30 feet in height may be required for excavation and construction of the proposed subterranean level, including a permanent dewatering system. The excavations are expected to expose artificial fill and alluvial soils, which may be subject to caving where granular soils are exposed. Vertical excavations up to 5 feet in height may be attempted where not surcharged by adjacent traffic or structures.
7.18.2 Vertical excavations greater than 5 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 12 feet high could be sloped back at a uniform 1½:1 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 7.19 of this report.

7.18.3 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.

### 7.19 Shoring – Soldier Pile Design and Installation

7.19.1 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.

7.19.2 A shoring system consisting of steel soldier beams is recommended for this project. The steel soldier beams may be placed in drilled holes and backfilled with concrete or may be installed utilizing high frequency vibration.

7.19.3 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.

7.19.4 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.
7.19.5 Drilled cast-in-place soldier piles should be placed no closer than three 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 200 psf per foot, (value has 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 alluvial soils.

7.19.6 Casing will be required since groundwater and caving are 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 5 feet. As an alternative, piles may be vibrated into place; however, there is always a risk that excessive vibrations in sandy soils could induce settlements and distress to adjacent offsite improvements. Continuous observation of the drilling and pouring of the piles by the Geotechnical Engineer (a representative of Geocon West, Inc.), is required.

7.19.7 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 0.40 based on uniform contact between the steel beam and lean-mix concrete and alluvial soils. 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 400 psf (value has been reduced for buoyant forces).

7.19.8 If a vibratory method is utilized, the owner should be aware of the potential risks associated with vibratory efforts, which typically involve inducing settlement within the vicinity of the pile which could result in a potential for damage to existing improvements in the area.

7.19.9 The level of vibration that results from the installation of the piles should not exceed a threshold where occupants of nearby structures are disturbed, despite higher vibration tolerances that a building may endure without deformation or damage. The main parameter used for vibration assessment is peak particle velocity in units of inch per second (in/sec). The acceptable range of peak particle velocity should be evaluated based on the age and condition of adjacent structures, as well as the tolerance of human response to vibration.
7.19.10 Based on Table 19 of the *Transportation and Construction Induced Vibration Guidance Manual* (Caltrans 2004), a continuous source of vibrations (ex. vibratory pile driving) which generates a maximum peak particle velocity of 0.5 in/sec is considered tolerable for modern industrial/commercial buildings and new residential structures. The Client should be aware that a lower value may be necessary if older or fragile structures are in the immediate vicinity of the site.

7.19.11 Vibrations should be monitored and recorded with seismographs during pile installation to detect the magnitude of vibration and oscillation experienced by adjacent structures. If the vibrations exceed the acceptable range during installation, the shoring contractor should modify the installation procedure to reduce the values to within the acceptable range. Vibration monitoring is not the responsibility of the Geotechnical Engineer.

7.19.12 Geocon does not practice in the field of vibration monitoring. If construction techniques will be implemented, it is recommended that qualified consultant be retained to provide site specific recommendations for vibration thresholds and monitoring.

7.19.13 Groundwater was encountered during exploration at a depth of 17½ feet below the existing ground surface and the contractor should be prepared for groundwater during pile installation. Piles placed below the water level require the use of a tremie to place the concrete into the bottom of the hole. A tremie should consist of a rigid, water-tight tube having a diameter of not less than 6 inches with a hopper at the top. The tube should be equipped with a device that will close the discharge end and prevent water from entering the tube while it is being charged with concrete. The tremie should be supported so as to permit free movement of the discharge end over the entire top surface of the work and to permit rapid lowering when necessary to retard or stop the flow of concrete. The discharge end should be closed at the start of the work to prevent water entering the tube and should be entirely sealed at all times, except when the concrete is being placed. The tremie tube should be kept full of concrete. The flow should be continuous until the work is completed and the resulting concrete seal should be monolithic and homogeneous. The tip of the tremie tube should always be kept about 5 feet below the surface of the concrete and definite steps and safeguards should be taken to insure that the tip of the tremie tube is never raised above the surface of the concrete.

7.19.14 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 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.
7.19.15 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.

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

7.19.17 For the design of shoring, it is recommended that an equivalent fluid pressure based on the following table, be utilized for design. A diagram depicting the trapezoidal pressure distribution of lateral earth pressure is provided below the table.

<table>
<thead>
<tr>
<th>HEIGHT OF SHORING (FEET)</th>
<th>EQUIVALENT FLUID PRESSURE (Pounds Per Cubic Foot) (ACTIVE PRESSURE)</th>
<th>EQUIVALENT FLUID PRESSURE (Pounds Per Square Foot per Foot) Trapezoidal (Where H is the height of the shoring in feet)</th>
</tr>
</thead>
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<tr>
<td>Up to 15</td>
<td>30</td>
<td>19H</td>
</tr>
<tr>
<td>16 – 30</td>
<td>34</td>
<td>21H</td>
</tr>
</tbody>
</table>

Trapezoidal Distribution of Pressure

7.19.18 It is very important to note that active pressures can only be achieved when movement in the soil (earth wall) occurs. If movement in the soil is not acceptable, such as adjacent to an existing structure, or the pile is restrained from movement by bracing or a tie back anchor, an at-rest pressure of 55 pcf should be considered for design purposes.
7.19.19 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.

7.19.20 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:

\[
\text{For } \frac{x}{H} \leq 0.4 \quad \sigma_h(z) = \frac{0.20 \left( \frac{z}{H} \right)^2 Q_L}{0.16 + \left( \frac{z}{H} \right)^2 H},
\]

and

\[
\text{For } \frac{x}{H} > 0.4 \quad \sigma_h(x, z) = \frac{1.26 \left( \frac{x}{H} \right)^2 \left( \frac{z}{H} \right) Q_L}{\left[ \left( \frac{x}{H} \right)^2 + \left( \frac{z}{H} \right)^2 \right] H},
\]

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, \(Q_L\) is the vertical line-load and \(\sigma_h\) is the horizontal pressure at depth \(z\).
7.19.21 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 \( \frac{x}{H} \leq 0.4 \)

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

and

For \( \frac{x}{H} > 0.4 \)

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

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, \( \sigma \) is the vertical pressure at depth \( z \), \( \Theta \) 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 \( \sigma_{H} \) is the horizontal pressure at depth \( z \).

7.19.22 In addition to the recommended earth pressure, the upper 10 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.

7.19.23 It is difficult to accurately predict the amount of deflection of a shored embankment. It should be realized that some deflection will occur. It is recommended that the deflection be minimized to prevent damage to existing structures and adjacent improvements. Where public right-of-ways are present or adjacent offsite structures do not surcharge the shoring excavation, the shoring deflection should be limited to less than 1 inch at the top of the shored embankment. Where offsite structures are within the shoring surcharge area it is recommended that the beam deflection be limited to less than \( \frac{1}{2} \) 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.

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

7.20 Tie-Back Anchors

7.20.1 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.

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

- Up to 5 feet below the top of the excavation – 1050 pounds per square foot.
- Up to 15 feet below the top of the excavation – 950* pounds per square foot (*reduced for buoyancy).

7.20.3 Depending on the techniques utilized, and the experience of the contractor performing the installation, a maximum allowable friction capacity of 3.0 kips per linear foot for post-grouted anchors (for a 20-foot length beyond the active wedge) may be assumed for design purposes. Only the frictional resistance developed beyond the active wedge should be utilized in resisting lateral loads.
7.21 Anchor Installation

7.21.1 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.

7.22 Anchor Testing

7.22.1 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.

7.22.2 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.

7.22.3 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.

7.22.4 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.
7.22.5 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.

7.23 Internal Bracing

7.23.1 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 2,500 psf 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 due to their intrusion into the construction site and potential interference with equipment. The structural engineer should review the shoring plan to determine if the raker footings conflict with the structural foundation system.

7.24 Surface Drainage

7.24.1 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.

7.24.2 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 5 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 5 feet of the building perimeter footings except when enclosed in protected planters.

7.24.3 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.
7.24.4 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.

7.25 Plan Review

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

4. The firm that performed the geotechnical investigation for the project should be retained to provide testing and observation services during construction to provide continuity of geotechnical interpretation and to check that the recommendations presented for geotechnical aspects of site development are incorporated during site grading, construction of improvements, and excavation of foundations. If another geotechnical firm is selected to perform the testing and observation services during construction operations, that firm should prepare a letter indicating their intent to assume the responsibilities of project geotechnical engineer of record. A copy of the letter should be provided to the regulatory agency for their records. In addition, that firm should provide revised recommendations concerning the geotechnical aspects of the proposed development, or a written acknowledgement of their concurrence with the recommendations presented in our report. They should also perform additional analyses deemed necessary to assume the role of Geotechnical Engineer of Record.
LIST OF REFERENCES


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 Oil, Gas and Geothermal Resources (DOGGR), 2006, Regional Wildcat Map, Los Angeles and Orange Counties, Map W1-5.


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Los Angeles, City of, Department of Public Works, 2006, Methane and Methane Buffer Zones, Citywide Methane Ordinance Map A-20960, City Ordinance No. 175,790.

Los Angeles, City of, 1996, Safety Element of the Los Angeles City General Plan.
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Los Angeles County Department of Public Works, 2015a, Ground Water Wells Website, http://dpw2.co.la.ca.us/website/wells/viewer.asp.


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

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

ENVIRONMENTAL GEOTECHNICAL MATERIALS
BOARDWALK SUNSET LLC
601 OCEAN FRONT WALK
VENICE, CALIFORNIA

DRAFTED BY: SB CHECKED BY: SFK JULY 2015 PROJECT NO. A9297-06-01 FIG. 1
LEGEND

Approximate Location of Boring

Approximate Location of Proposed Development

SPEEDWAY (ALLEY)

PROPOSED THREE STORY COMMERCIAL OVER TWO SUBTERANEAN LEVEL

EXISTING THREE STORY COMMERCIAL

SUNSET AVENUE

OCEAN FRONT WALK

FIG. 2

PROPOSED THREE STORY COMMERCIAL OVER TWO SUBTERANEAN LEVEL
NOTE: CROSS-SECTIONS ARE FOR ILLUSTRATION PURPOSES ONLY, NOT FOR CONSTRUCTION

GROUNDWATER ELEVATION ENCOUNTERED DURING SITE INVESTIGATION

HISTORIC HIGH GROUNDWATER ELEVATION
SITE

### DESIGN EARTHQUAKE - EMPIRICAL ESTIMATION OF LIQUEFACTION POTENTIAL

**NCEER (1996) METHOD**

By Thomas F. Blake (1994-1996)

**EARTHQUAKE INFORMATION**

<table>
<thead>
<tr>
<th>Project Name</th>
<th>601 Ocean Front Walk</th>
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<td>A9297-06-01</td>
</tr>
<tr>
<td>Boring No.</td>
<td>1</td>
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</tbody>
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**LIQUEFACTION CALCULATIONS:**

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<th>Depth (ft)</th>
<th>Total SPT (N)</th>
<th>SPT (ft)</th>
<th>Field</th>
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<th>Ratio</th>
<th>Corrected</th>
<th>Safe/Fact.</th>
<th>Induced</th>
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**Figure 5**
### LIQUEFACTION SETTLEMENT ANALYSIS

**DESIGN EARTHQUAKE**

(SATURATED SAND AT INITIAL LIQUEFACTION CONDITION)

**NCEE (1996) METHOD**

**EARTHQUAKE INFORMATION**

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<th>Earthquake Magnitude</th>
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<td>0.933</td>
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**TERMINOLOGY**

- **SPEE (g)**: 1.2X SAT
- **SPE (g)**: 1.0X SAT

**DESIGN EARTHQUAKE**

- **Project Name**: 601 Ocean Front Walk
- **Project No.**: A9297-06-01
- **Boring No.**: 1

#### Historic High Groundwater:

- Depth to Base: [2/3 PGAM (g)]
- Depth to Base: [SATURATED SAND AT INITIAL LIQUEFACTION CONDITION]

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<th>DEPTH TO BASE</th>
<th>BLOW COUNT N</th>
<th>BLOW</th>
<th>WET DENSITY (pcf)</th>
<th>TOTAL STRESS O (TSF)</th>
<th>EFFECT STRESS O (TSF)</th>
<th>REL. DEN. (Dr (%))</th>
<th>ADJUST BLOWS (N/100)</th>
<th>Tw/O2</th>
<th>LIQUEFACTION SAFETY FACTOR</th>
<th>Volumetric Strain (GPa) (%)</th>
<th>EQ SETTLE. (in.)</th>
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**TOTAL SETTLEMENT = 0.0 INCHES**

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Figure 6
### MAXIMUM CONSIDERED EARTHQUAKE - EMPIRICAL ESTIMATION OF LIQUEFACTION POTENTIAL

#### EERCE (1996) METHOD

**EARTHQUAKE INFORMATION**

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**PROJECT INFORMATION**

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**LIQUEFACTION CALCULATIONS:**

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<th>Eff. Unit (pcf)</th>
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### Figure 7
LIQUEFACTION SETTLEMENT ANALYSIS
MAXIMUM CONSIDERED EARTHQUAKE
(SATURATED SAND AT INITIAL LIQUEFACTION CONDITION)

NCREE (1996) METHOD

| DEPTH TO BASE | BLOW COUNT | WET DENSITY (PCF) | TOTAL STRESS O (TSP) | EFFECT STRESS O (TSP) | HEL. DEN. (br.%) | ADJUST BLOWS (IN. K) | TavO/ | LIQUEFACTION SAFETY FACTOR | VOLUMETRIC Strain (E%) | EL SETTLE. Pe (in.) |
|---------------|------------|-------------------|----------------------|----------------------|----------------|-----------------------|------|-----------------------------|-------------------|----------------|}
|               |            |                   |                      |                      |                |                       |      |                             |                   |                |}

CALCULATED MAG. WGT. FACTOR:

BASE TO 75

PROJECT NAME: 601 Ocean Front Walk

PROJECT NO. A9297-06-01

BORING NO.

Figure 8
Retaining Wall Design with Transitioned Backfill
(Vector Analysis)

Input:
- Retaining Wall Height (H): 15.00 feet
- Slope Angle of Backfill (θ): 0.00 degrees
- Height of Slope above Wall (h): 0.00 feet
- Horizontal Length of Slope (I): 0.00 feet
- Total Height (Wall + Slope): 15.00 feet
- Unit Weight of Retained Soils (γ): 125.0 pcf
- Friction Angle of Retained Soils (ϕ): 32.0 degrees
- Cohesion of Retained Soils (c): 150.0 pcf
- Factor of Safety (FS): 1.50

Factored Parameters:
- γₚ : 22.6 degrees
- γₛ : 100.0 pcf

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Maximum Active Pressure Resultant
$$P_{A,\text{max}} = 441.746 \text{ lbs/linear foot}$$

Equivalent Fluid Pressure (per linear foot of wall)
$$\text{EFP} = 2 \times P_{A} \times H^2$$
$$\text{EFP} = 39.3 \text{ pcf}$$

Design Wall for an Equivalent Fluid Pressure:
$$40 \text{ pcf}$$
Active
# Retaining Wall Design with Transitioned Backfill

## (Vector Analysis)

### Input:
- Retaining Wall Height (H) = 25.00 feet
- Slope Angle of Backfill (β) = 0.0 degrees
- Height of Slope above Wall (h) = 0.0 feet
- Horizontal Length of Slope (L) = 0.0 feet
- Total Height (Wall + Slope) (H_2) = 25.00 feet
- Unit Weight of Retained Soils (γ) = 125.0 pcf
- Friction Angle of Retained Soils (θ) = 37.0 degrees
- Cohesion of Retained Soils (c) = 100.0 psf
- Factor of Safety (FS) = 1.50

### Factored Parameters:
- \( \gamma' \) = 26.7 degrees
- \( \varphi' \) = 66.7 degrees

## Table: Retaining Wall Calculation (Fig. 10)

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<th>Failure Angle (α)</th>
<th>Height of Tension Crack (Hc)</th>
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### Maximum Active Pressure Resistant
- \( P_{A, max} = 12882.65 \text{ lbs/lineal foot} \)

### Equivalent Fluid Pressure (per lineal foot of wall)
- \( EFP = 2 \times P_A / H^2 \)
- \( EFP = 41.2 \text{ pcf} \)

### Design Wall for an Equivalent Fluid Pressure:
- \( P_{A, max} = 42 \text{ pcf} \)
- Active

---

**GEOCON WEST, Inc.**

ENVIRONMENTAL, GEOTECHNICAL, MATERIALS

3303 N. SAN FERNANDO BLVD. - SUITE 100 - BURBANK, CA 91504

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

DRAFTED BY: TL CHECKED BY: HHD

**RETTAINING WALL CALCULATION**

BOARDWALK SUNSET LLC

601 OCEAN FRONT WALK

VENICE, CALIFORNIA

JULY 2015 PROJECT NO. A9297-06-01 FIG. 10
Shoring Design with Transitioned Backfill
(Vector Analysis)

Input:
- Shoring Height \( H \): 15.00 feet
- Slope Angle of Backfill \( \beta \): 0.0 degrees
- Height of Slope above Shoring \( h_a \): 0.0 feet
- Horizontal Length of Slope \( L_a \): 0.0 feet
- Total Height (Shoring + Slope) \( H_T \): 15.0 feet

Unit Weight of Retained Soil: \( \gamma_b \) 125.0pcf
Friction Angle of Retained Soil: \( \phi_b \) 32.0 degrees
Cohesion of Retained Soil: \( c_b \) 150.0 pcf
Factor of Safety: \( F_S \) 1.25

Factored Parameters:
- \( \gamma_k \): 26.6 degrees
- \( \phi_k \): 120.0 degrees

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Maximum Active Pressure Resultant
\[ P_{a,\text{max}} \]
3383.02 lbs/lineal foot

Equivalent Fluid Pressure (per lineal foot of shoring)
\[ \text{EFP} = 2 \cdot P_a / H^2 \]
\[ \text{EFP} = 30.1 \text{pcf} \]

Design Shoring for an Equivalent Fluid Pressure:
\[ 30 \text{ pcf} \]
Active

---

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DRAFTED BY: TL CHECKED BY: HHD

---

**SHORING WALL CALCULATION**

**BOARDWALK SUNSET LLC**

601 OCEAN FRONT WALK

VENICE, CALIFORNIA

JULY 2015 PROJECT NO. A9297-06-01 FIG. 11
Shoring Design with Transient Backfill
(Vector Analysis)

Input:
Shoring Height \( (H) \) 30.00 feet
Slope Angle of Backfill \( (\beta) \) 0.0 degrees
Height of Slope above Shoring \( (h) \) 0.0 feet
Horizontal Length of Slope \( (L) \) 0.0 feet
Total Height (Shoring + Slope) \( (H_t) \) 30.00 feet
Unit Weight of Retained Soil \( (\gamma) \) 125.0 pcf
Friction Angle of Retained Soil \( (\phi) \) 37.0 degrees
Cohesion of Retained Soil \( (c) \) 100.0 pcf
Factor of Safety \( (FS) \) 1.25

Factored Parameters:
\( (\gamma_{fd}) \) 31.1 degrees
\( (\phi_{fd}) \) 80.0 pcf

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</tr>
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</table>

Maximum Active Pressure Resultant
\( P_{A,max} \) 15343.42 lbs/lineal foot

Equivalent Fluid Pressure (per lineal foot of shoring)
\( EFP = 2P_{A}/H^2 \)
EFP 34.1 pcf

Design Shoring for an Equivalent Fluid Pressure:
Active 34 pcf
3/4" CRUSHED ROCK
MIRAFI 500X OR EQUIVALENT
FILTER FABRIC LAYER
MIRAFI 140N OR EQUIVALENT
OPTIONAL
3/4" CRUSHED ROCK (DENSIFIED)
GROUND SURFACE
WATERPROOF WALL
PROPERLY COMPACTED BACKFILL
RETAINING WALL
2/3 H
3"
GROUND SURFACE

PROPERLY COMPACTED BACKFILL

DRAINAGE PANEL (J-DRAIN 1000 OR EQUIVALENT)

WATER PROOFING BY ARCHITECT

BASEMENT RETAINING WALL

FOUNDATION

TO SUBDRAIN

3/4" CRUSHED ROCK (DENSIFFIED)

FILTER FABRIC LAYER
MIRAFI 500X OR EQUIVALENT

3/4" CRUSHED ROCK (1 CU. FT./FT.)

FILTER FABRIC ENVELOPE
MIRAFI 140N OR EQUIVALENT

4" DIA. SCHEDULE 40 PERFORATED PVC PIPE EXTENDED TO APPROVED OUTLET

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

NO SCALE
APPENDIX A

FIELD INVESTIGATION

The site was explored on June 27, 2015, by excavating one 4⅞ inch diameter boring to a depth of approximately 75½ feet below the existing ground surface utilizing a truck-mounted reverse circulation mud rotary drilling machine. Representative and relatively undisturbed samples were obtained by driving a 3 inch, O. D., California Modified Sampler into the “undisturbed” soil mass with blows from a 140-pound auto-hammer falling 30 inches. The California Modified Sampler was equipped with 1-inch high by 2¼-inch diameter brass sampler rings to facilitate removal and testing. Standard Penetration Tests (SPTs) were performed in the borings and bulk samples were also obtained.

The soil conditions encountered in the boring were visually examined, classified and logged in general accordance with the Unified Soil Classification System (USCS). The boring log is presented as Figure A1. The log depicts the soil and geologic conditions encountered and the depth at which samples were obtained. The approximate location of the boring is indicated on the Site Plan (see Figure 2).
<table>
<thead>
<tr>
<th>DEPTH IN FEET</th>
<th>SAMPLE NO.</th>
<th>LITHOLOGY</th>
<th>SOIL CLASS (USCS)</th>
<th>GROUNDWATER</th>
<th>PENETRATION RESISTANCE (BLOWS/FT)</th>
<th>DRY DENSITY (P.C.F.)</th>
<th>MOISTURE CONTENT (%)</th>
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<td>B1@5'</td>
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<td></td>
<td></td>
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<td></td>
<td></td>
</tr>
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<td></td>
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<td>B1@27.5'</td>
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</table>

**MATERIAL DESCRIPTION**

**ARTIFICIAL FILL**
Silty Sand, medium dense, slightly moist, yellowish brown, fine- to medium-grained.
- some fine gravel

**ALLUVIUM**
Sand with Silt, poorly graded, medium dense, wet, pale brown, fine- to medium-grained, trace coarse-grained.
- loose, moist

Silty Sand, medium dense, moist, brown, fine- to medium-grained.
- some fine gravel

Sand, poorly graded, medium dense, moist to wet, pale brown, fine- to medium-grained.
- 19.5' - 20.0' gravel with sand layer, fine gravel
- very dense
- dense, fine- to coarse-grained, some fine gravel
- very dense

Sand with Gravel, dense, moist to wet, yellowish brown, fine- to medium-grained.

---

**ELEV. (MSL.)** DATE COMPLETED 6/27/15

**EQUIPMENT** MUD ROTARY BY: PZ

---

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.
## BORING 1

**ELEV. (MSL.)** | DATE COMPLETED | **6/27/15**
--- | --- | ---
**EQUIPMENT** | MUD ROTARY | **BY: PZ**

<table>
<thead>
<tr>
<th>DEPTH IN FEET</th>
<th>SAMPLE NO.</th>
<th>LITHOLOGY</th>
<th>SOIL CLASS (USCS)</th>
<th>GROUNDWATER</th>
<th>MATERIAL DESCRIPTION</th>
<th>PENETRATION RESISTANCE (BLOWS/FT)</th>
<th>DRY DENSITY (P.C.F.)</th>
<th>MOISTURE CONTENT (%)</th>
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<td>B1@30'</td>
<td>coarse-grained, fine gravel, some coarse gravel.</td>
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<td>75</td>
<td>132.5</td>
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<td>- very dense, decrease in gravel content</td>
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<td>12.1</td>
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<td>- no gravel content, pale brown, fine- to medium-grained</td>
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<td>- dense, trace coarse-grained</td>
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<td>14.7</td>
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<td>- yellowish brown, fine- to coarse-grained, fine gravel</td>
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<td></td>
<td>50 (5&quot;)</td>
<td>--</td>
<td>11.7</td>
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<tr>
<td>42</td>
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<td>Sand, poorly graded, very dense, moist, brown, very fine- to fine-grained.</td>
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<td></td>
</tr>
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<td>50 (5&quot;)</td>
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<td>76</td>
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<td>21.2</td>
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**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.

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

A9297-06-01 BORING LOGS.GPJ

GEOCON
<table>
<thead>
<tr>
<th>DEPTH IN FEET</th>
<th>SAMPLE NO.</th>
<th>LITHOLOGY</th>
<th>SOIL CLASS (USCS)</th>
<th>GROUNDWATER</th>
<th>PENETRATION RESISTANCE (BLOWS/FT)</th>
<th>DRY DENSITY (P.C.F.)</th>
<th>MOISTURE CONTENT (%)</th>
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</table>

- **Total depth of boring:** 75.5 feet
- **Fill to 3 feet.**
- **Groundwater encountered at 17.5 feet.**
- **Backfilled with cement/bentonite grout.**
- **Asphalt patched.**

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

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

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

**NOTE:** The log of subsurface conditions shown hereon applies only at the specific boring or trench location and at the date indicated. It is not warranted to be representative of subsurface conditions at other locations and times.
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, corrosivity, and in-place dry density and moisture content. The results of the laboratory tests are summarized in Figure B1. The in-place dry density and moisture content of the samples tested are presented in the boring logs, Appendix A.
### Direct Shear Test Results

**Boardwalk Sunset LLC**

601 Ocean Front Walk

Venice, California

<table>
<thead>
<tr>
<th>SAMPLE</th>
<th>SOIL TYPE</th>
<th>DRY DENSITY</th>
<th>INITIAL MOISTURE (%)</th>
<th>FINAL MOISTURE (%)</th>
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<tbody>
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<td>B1 @ 10'</td>
<td>SP-SM</td>
<td>95.1</td>
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<td>B1 @ 15'</td>
<td>SM</td>
<td>111.7</td>
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<td>16.7</td>
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<td>B1 @ 25'</td>
<td>SP</td>
<td>105.1</td>
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<td>15.3</td>
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<td>B1 @ 35'</td>
<td>SP</td>
<td>107.1</td>
<td>22.3</td>
<td>20.1</td>
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</table>

**Normal Pressure (KSF)**

**Shear Strength (KSF)**

- **C = 150 PSF**
- **C = 100 PSF**

**Direct Shear, Saturated**

**FIG. B1**

**Theta = 37 DEGREES**

**Theta = 34 DEGREES**
CONSOLIDATION TEST RESULTS

VENICE, CALIFORNIA

BOARDWALK SUNSET LLC
601 OCEAN FRONT WALK
VENICE, CALIFORNIA

JULY 2015 | PROJECT NO. A9297-06-01 | FIG. B2

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

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

DRAFTED BY: TL | CHECKED BY: HHD
### SUMMARY OF LABORATORY POTENTIAL OF HYDROGEN (pH) AND RESISTIVITY TEST RESULTS

**CALIFORNIA TEST NO. 643**

<table>
<thead>
<tr>
<th>Sample No.</th>
<th>pH</th>
<th>Resistivity (Ohm Centimeters)</th>
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<td>B1 @ 22.5-27.5'</td>
<td>8.7</td>
<td>420 (Severely Corrosive)</td>
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### SUMMARY OF LABORATORY CHLORIDE CONTENT TEST RESULTS

**EPA NO. 325.3**

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<th>Chloride Ion Content (%)</th>
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<td>B1 @ 22.5-27.5'</td>
<td>0.130</td>
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### SUMMARY OF LABORATORY WATER SOLUBLE SULFATE TEST RESULTS

**CALIFORNIA TEST NO. 417**

<table>
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<th>Water Soluble Sulfate (% SO₄)</th>
<th>Sulfate Exposure*</th>
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<tr>
<td>B1 @ 22.5-27.5'</td>
<td>0.017</td>
<td>Negligible</td>
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*Reference: 2013 California Building Code, Section 1904.3 and ACI 318-11 Section 4.3.*