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August 18, 2017

Los Angeles City Council
c/o Office of the City Clerk
City Hall, Room 395
Los Angeles, California 90012

Attention: PLUM Committee

Dear Honorable Members:

CONSIDERATION OF ERRATA TO MND NO. ENV-2016-2229-MND FOR POTENTIAL AMENDMENTS TO PROPOSED PROJECT AT 12444 VENICE BOULEVARD; CF 17-0537

INTRODUCTION

On July 13, 2017, the City Planning Commission approved the construction of a 6-story with a mezzanine level, mixed-use development totaling 62,652 square feet. The project includes 77 dwelling units, 2,100 square feet of retail space, and 8,075 square feet of open space. The project will reserve 11 percent, or 7 dwelling units, of the 58 total base dwelling units permitted for Very Low Income household occupancy for a period of 55 years. The project will utilize Assembly Bill 744 (California Government Code Section 65915 (p)(2)) to allow for the provision of reduced parking at 46 residential parking spaces. The Project includes one at-grade level of parking and one subterranean parking level.

An Initial Study and Mitigated Negative Declaration (IS/MND) was prepared for the subject project by the Department of City Planning and circulated for public review on September 22, 2016 for a period of 20 days (identified hereafter as the circulated IS/MND). The review period for the circulated IS/MND ended on October 12, 2016.

The attached errata to the MND was prepared on August 18, 2017 to analyze a potential amended project description for consideration by City Council. The subject amendments involve the removal of at-grade parking and the addition of one new subterranean parking level.

ANALYSIS OF POTENTIAL CHANGES TO THE PROJECT

After review of the IS/MND and preparation of the Errata in light of the potential changes to the project, it is determined that the potential changes to the project are not expected to result in new or different significant impacts requiring any new or modified mitigation measures. The changes to the project are not expected to result in any changes to the environment that are different in kind or nature from those identified in the IS/MND. Further analysis and information is provided in the attached Errata.

The Errata does not identify any new, significant effect or mitigation measures or project revisions to reduce an effect to insignificance. No new measures or revisions are required. None of the changes described in this Errata constitute a substantial revision of the MND. Rather, the Errata clarifies, amplifies, and/or makes minor modifications to the circulated IS/MND. Therefore, based on the analysis including herein and pursuant to State CEQA Guidelines §15073.5, recirculation of the MND is not required.

Sincerely,

VINCENT P. BERTONI, AICP
Director of Planning

Faisal Roble
Principal City Planner

VPB: FR: DL: cc

Enclosures
Initial Study / Mitigated Negative Declaration Errata

City of Los Angeles
Department of City Planning

Initial Study/Mitigated Negative Declaration
Errata

Project Title: ENV-2016-2229-MND
Case No.: DIR-2016-304-DB-SPR
Project Location: 12444 W VENICE BLVD

Introduction

An Initial Study and Mitigated Negative Declaration (IS/MND) was prepared for the subject project by the Department of City Planning and circulated for public review on September 22, 2016 for a period of 20 days (identified hereafter as the circulated IS/MND). The review period for the circulated IS/MND ended on October 12, 2016. Through the project's design development process, the number of subterranean parking levels fluctuated between one and two levels. The Project Description in the circulated IS/MND identifies the project as including two levels of subterranean parking. However, the project plans only identify one level of subterranean parking. Thus, for clarity and for thorough analysis purposes, this Errata considers the potential environmental impacts of two levels of subterranean parking, if such potential impacts were not fully evaluated in the circulated IS/MND. This Errata has been prepared by the City to ensure that it has fulfilled its responsibility as the lead agency for the project pursuant to the California Environmental Quality Act (CEQA).

Project Considered in this Errata

The IS/MND prepared for the project and circulated on September 22, 2016 evaluated a proposed project that consisted of demolition of an existing two-story commercial building and the construction of a new mixed-use building with 77 residential units and 2,100 square feet of ground floor retail space. As described in the circulated IS/MND, the proposed building contains six above-ground stories and two levels of subterranean parking. However, the project plans identify only one level of subterranean parking. For clarity, the project considered in this Errata includes a second level of subterranean parking. By including the second subterranean level, the project would slightly deviate from the project plans. Such deviations include eliminating the at-grade/tuck-under parking that is shown in the rear (south) portion of the ground floor level, and modifications to the first subterranean level to accommodate vehicular access to the second subterranean level (i.e., a ramp). Including a second subterranean level would not change the proposed uses or the height of the proposed building, and no changes to the floorplans for stories two through six are anticipated.

Statutory Background

State CEQA Guidelines §15073.5(a) requires that a lead agency recirculate a negative declaration “when the document must be substantially revised.” As described in State CEQA Guidelines §15073.5(b), a “substantial revision” means: (1) a new, avoidable significant effect is identified and mitigation measures or project revisions must be added in order to reduce the effect to insignificance; or (2) the lead agency determines that the proposed mitigation measures or project revisions will not reduce potential effects to less than significance and new measures or revisions must be required. Pursuant to State CEQA Guidelines §15073.5(c) recirculation is not required under the following circumstances:

(1) Mitigation measures are replaced with equal or more effective measures pursuant to Section 15074.1.

(2) New project revisions are added in response to written or verbal comments on the project’s effects identified in the proposed negative declaration which are not new avoidable significant effects.

(3) Measures or conditions of project approval are added after circulation of the negative declaration which are not required by CEQA, which do not create new significant environmental effects and are not necessary to mitigate an avoidable significant effect.

(4) New information is added to the negative declaration which merely clarifies, amplifies, or makes insignificant modifications to the negative declaration.

Environmental Analysis

The environmental analysis in this Errata focuses on the environmental topics where impacts could be different that those described in the circulated IS/MND—namely Air Quality, Geology/Soils, Greenhouse Gas Emissions, and Noise. The paragraphs below consider the potential impacts respective to each of these topics. The number of subterranean parking levels would not affect the analysis of potential environmental impacts related to Aesthetics, Agriculture and Forestry Resources, Biological Resources, Cultural Resources, Hazards & Hazardous Materials, Hydrology/Water Quality, Land Use/Planning, Mineral Resources, Population/Housing, Public Services, Recreation, Transportation/Traffic, or Utilities/Service Systems. Therefore, such topics are not discussed further in this Errata.

Air Quality

Given that the number of subterranean parking levels would not change the type or intensity of the proposed uses, there would be no change in the operation phase air pollutant impacts described in the circulated IS/MND. Likewise, regardless of the number of subterranean parking levels the proposed project would not conflict with or obstruct implementation of the applicable air quality plan (see the circulated IS/MND). The number of subterranean parking levels would, however, affect the project’s construction activities, particularly related to grading and excavation. Thus, construction phase air pollutant impacts are considered herein.

The air quality analysis in the circulated IS/MND considered 20,000 cubic yards of soil export. Given the size of the site (20,896 square feet), 20,000 cubic yards of excavation equates to a grading depth of approximately 26 feet. Such a depth is anticipated to accommodate two levels of subterranean parking. Thus, no additional analysis is required. For clarity, the difference in potential construction phase air quality impacts between a project with one and two levels of subterranean parking, is largely related to the total volume of air pollutants that would be generated over the course of the grading phase. Daily grading activities would be largely the same regardless of whether there would be one or two levels of subterranean parking. However, in the two-level scenario, grading would occur over a longer period of time and, thus, would generate a greater total volume of air pollutants. When evaluating air quality impacts from construction pursuant to the guidance provided the South Coast Air Quality Management District (SCQAMD), the effective measurement is the maximum daily air pollutant emissions and not the total volume of air pollutants generated over time, as the concentration of air pollution is related to the duration over which pollutants are emitted. Since daily grading activities would be largely the same in both the one- and two-subterranean parking level scenarios, air quality impacts are substantially similar in both scenarios.

Geology/Soils

Prior to circulation of the IS/MND for public review, the applicant had submitted two geotechnical evaluations: a Geotechnical Engineering Exploration report prepared by Byer Geotechnical in October 2014 that evaluated a proposed building consisting of five above-ground stories over two levels of subterranean parking; and a Geotechnical Engineering Update prepared by Byer Geotechnical in March 2017 that considered a version of the project comprising six stories over one subterranean level. After circulation of the IS/MND, the City commissioned a geotechnical evaluation of a version of the project that comprises six stories over two subterranean levels—see the Geotechnical Update Report prepared by R.T. Frankian & Associates, Inc. (RTF&A) in August 2017 included as Appendix A to this Errata. As concluded in this Geotechnical Update Report, a project that comprises six stories over two subterranean levels is feasible from a geotechnical/geologic perspective. The information provided in the RTF&A Geotechnical Update Report does not change the analysis or conclusions in the circulated IS/MND related to geology and soils. The project would continue to have no impacts related to fault rupture, liquefaction, or landslides and would continue to have less than significant impacts related to seismic ground shaking, soil erosion and the loss of topsoil, geologic stability, and expansive soil.

Greenhouse Gas Emissions

The GHG analysis in the circulated IS/MND does not specify the amount of grading or the number of levels of subterranean parking that were evaluated. Thus, for conservative purposes, this Errata assumes the GHG analysis in the circulated IS/MND considered one level of subterranean parking.

Given that the number of subterranean parking levels would not change the type or intensity of the proposed uses, there would be no change in the operation phase greenhouse gas (GHG) emissions impacts described in the circulated IS/MND. Likewise, regardless of the number of subterranean parking levels, the proposed project would not conflict with an applicable plan, policy or regulation adopted for the purpose of reducing GHG emissions, as the project would be required to implement the Los Angeles Green Building Code (LAGBC), is an in-fill mixed-use development located in a Transit Priority Area, and would implement various green building practices (see Response VII.b in the circulated IS/MND). The number of subterranean parking levels would, however, affect the project's construction activities, particularly related to grading and excavation. Thus, construction phase GHG emissions are considered herein.

The difference in potential construction phase GHG emission between a project with one and two levels of subterranean parking, is largely related to the total volume of GHG emissions that would be generated over the course of the grading phase. Daily grading activities would be largely the same regardless of whether there would be one or two levels of subterranean parking. However, in the two-level scenario, grading would occur over a longer period of time and, thus, would generate a greater total volume of GHG pollutants. Pursuant to guidance provided by the SCAQMD, when evaluating a project's potential GHG impacts, the construction phase GHG emissions should be amortized over 30 years (i.e., divided by 30) and then added to the GHG emissions from one year of project operations (i.e., the project's total annual GHG emissions). Even if the project involved two stories of subterranean parking, GHG emissions generated during grading would be a fraction of the GHG emission generated by the project's construction, which in turn would be a small fraction of the annual GHG emissions generated by project operation. The difference in the project's total annual GHG emissions between the one- and two-subterranean parking level scenarios would be negligible. Therefore, the project's GHG impacts remain less than significant with the implementation of the mitigation measures identified in the circulated IS/MND.

Noise

The Noise analysis in the circulated IS/MND does not specify the amount of grading or the number of levels of subterranean parking that were evaluated. Thus, for conservative purposes, this Errata assumes the Noise analysis in the circulated IS/MND considered one level of subterranean parking.

Given that the number of subterranean parking levels would not change the type or intensity of the proposed uses, there would be no change in the operation phase noise impacts described in the circulated IS/MND. Likewise, regardless of the number of subterranean parking levels, the proposed project would not result in any noise impacts related to proximity to an airport or private airstrip. The number of subterranean parking levels would, however, affect the project's construction activities, particularly related to grading and excavation. Thus, construction phase noise impacts are considered herein.

The difference in potential construction phase noise and vibration impacts between a project with one and two levels of subterranean parking, is largely the duration over which construction noise and vibration would be generated. Daily grading activities would be largely the same regardless of whether there would be one or two levels of subterranean parking. Thus, the noise and vibration levels generated during construction would be substantially similar in both the one- and two-subterranean parking level scenarios. In the two-level scenario, grading would occur over a longer period of time and, thus, would generate noise and vibration or a longer duration. When evaluating noise impacts from construction pursuant to the City of Los Angeles General Plan Noise Element and Ordinance No. 161,574, the effective measurement is the maximum noise level and the duration of noise and vibration generation is not a critical factor. Since daily grading activities would be largely the same in both the one- and two-subterranean parking level scenarios, noise and vibration impacts are substantially similar in both scenarios. Therefore, the project's noise and vibration impacts remain less than significant with the implementation of the mitigation measures identified in the circulated IS/MND.

Determination

This Errata does not identify any new, significant effect or mitigation measures or project revisions to reduce an effect to insignificance. No new measures or revisions are required. None of the changes described in this Errata constitute a substantial revision of the MND. Rather, this Errata clarifies, amplifies, and/or makes minor modifications to the circulated IS/MND. Therefore, based on the analysis including herein and pursuant to State CEQA Guidelines §15073.5, recirculation of the MND is not required.

End of Errata.

Appendix A

Geotechnical Update Report
R.T. Frankian & Associates, Inc., August 18, 2017

GEOTECHNICAL ENGINEERING & ENGINEERING GEOLOGY

August 18, 2017

Michael Baker International
3760 Kilroy Airport Way, Suite 270
Long Beach, California 90806

Job No. 2017-012-060

Attention: Mr. John Bellas

Subject: Geotechnical Update Report
Proposed Six-Story Building over
Two-Level Subterranean Parking Garage
12444 Venice Boulevard
Roseboro Villa Tract, Block A, Portions of Lots 8, 9 and 10
Los Angeles, California

References: See attached References

Ladies/Gentlemen:

R. T. Frankian & Associates, Inc., (RTF&A) is pleased to present this geotechnical update report for the proposed construction of a six-story residential building underlain by a two-level subterranean parking garage at 12444 Venice Boulevard in Los Angeles, California. This update report has been prepared for Michael Baker International, under the direction of the City of Los Angeles Department of City Planning (LADCP), and provides a geotechnical feasibility evaluation of the project, based upon our review of the referenced geotechnical reports prepared by Byer Geotechnical, Inc. (BGI). Recommendations presented herein are based on RTF&A's review of the BGI reports and our engineering and geologic analyses. No additional subsurface exploration or laboratory testing was performed as part of this Geotechnical Update Report.

The assessment of general site environmental conditions for the presence of contaminants in the soils and groundwater at the site was beyond the scope of this update report. Our professional services have been performed using that degree of care and skill ordinarily exercised, under similar circumstances, by reputable geotechnical engineers and geologists practicing in this or similar localities. No other warranty, expressed or implied, is made as to the professional advice included in this report. This report has been prepared for Michael Baker International and LADCP to be used solely for planning and design of the 12444 Venice Boulevard project.

BACKGROUND

The geotechnical conditions beneath the 12444 Venice Boulevard property (herein referred to as "the project") were initially investigated by BGI in 2014. At that time, the investigation consisted of the excavation of two borings to evaluate the nature, distribution, and engineering properties of the geologic materials beneath the site, with respect to the planned construction of a five-story residential building underlain by two levels of a subterranean parking garage. The borings were excavated to depths of 31½ and 61 feet below the existing grade. Representative samples of the subsurface materials were obtained during the drilling of the borings for laboratory analyses. BGI determined that the project was feasible from a geotechnical engineering standpoint as discussed in the referenced "Geotechnical Engineering Exploration" report (BGI, 2014) and provided geotechnical recommendations to be incorporated into the plans and implemented during project construction. The "Geotechnical Engineering Exploration" report (BGI, 2014) was approved by the City of Los Angeles Department of Building and Safety (LADBS) in their "Soils Report Approval Letter" dated November 25, 2014 (LADBS, 2014).

In 2017, BGI prepared an update letter to address a redesigned project, consisting of a six-story residential structure over a single-level subterranean parking garage. BGI concluded that the previous recommendations presented in their "Geotechnical Engineering Exploration" report (BGI, 2014) remained applicable to the redesigned project.

As part of processing the proposed entitlements for the project, LADCP is looking for options to make the project more palatable for the surrounding community. Consequently, LADCP requested a geotechnical update to evaluate the geotechnical feasibility of a version of the project comprising a six-story residential building underlain by a two-level subterranean parking garage. This submittal serves as that geotechnical update.

SITE DESCRIPTION

The project site is located on the southeast corner of Venice Boulevard and Wasatch Avenue, in the Mar Vista district of the City of Los Angeles. The property is level and essentially rectangular shaped, measuring approximately 160 feet along Venice Boulevard (on the north) and 180 feet along Wasatch Avenue (on the west). The rear of the property is bordered by an alley and the eastern portion bounded by a commercial development. Centinela Avenue and the San Diego Freeway (Interstate 405) are located approximately 400 feet and 1 mile, respectively, to the east.

Presently, the property is occupied by a two-story commercial building in the southern portion of the property and a single-story commercial building in the eastern portion of the site. A paved asphalt parking lot covers the remaining portion of the property.

PROPOSED DEVELOPMENT

It is our understanding that it is being proposed to construct a six-story residential building underlain by a two-level subterranean parking garage. We further understand that the proposed structure will occupy the majority of the site, essentially extending to the property lines on all sides of the project. For the purposes of preparing this update report, we will assume that the subterranean parking garage will consist of concrete construction and that the lowest portion of the garage will extend approximately 20 feet below the present grade. It will further be assumed that the at-grade portion of the structure will consist of wood-frame or lightweight steel construction and may incorporate block walls.

We have not been presented with load data for the proposed structure. For the purposes of presenting this submittal, we will assume that the structure will generate isolated or point loads of approximately 800 kips at column locations and loads of approximately 5 kips per lineal foot along continuous foundations.

The project described above was the basis for the preparation of the recommendations presented in this report. We should be notified if the description of the project is inaccurate or if significant changes to the development are proposed.

SOIL CONDITIONS

As previously discussed, two test borings (Borings B1 and B2) were previously drilled at the site and a geotechnical investigation was performed as summarized in the referenced "Geotechnical Engineering Exploration" (BGI, 2014) report. The locations of those borings are indicated on the attached Plot Plan; the Site Plan from the referenced "Geotechnical Engineering Exploration" (BGI, 2014) report was used as the base map for the Plot Plan. The following descriptions of the soil conditions at the site are based on our review of the boring logs presented as part of that report.

The upper portion of the site is mantled by alluvial soils primarily consisting of silty to sandy clays and sandy silts. Alluvial soils consisting of silty to gravelly sands were also present in Boring B1 at depths between 10 and 20 feet below the existing grade. In general, the alluvial soils were observed as being slightly moist to moist. The clays and silts were described as being medium stiff to very stiff and the sands as being medium dense. The alluvial soils extended to depths of approximately 20 feet below the existing grade.

The alluvial soils were underlain by marine deposits that extended to the depths of exploration. The marine deposits consisted of well-graded and poorly-graded gravelly sands. The marine deposits were observed to be slightly moist and becoming wet below a depth of 50 feet. The marine deposits were described as being medium dense to dense at depths ranging from 20 to 25 feet and becoming dense to very dense at depths greater than 25 feet below the existing grade.

Variations of the materials encountered are indicated on the BGI boring logs that have been included in Appendix A of this report. As noted on Log of Boring B1, groundwater was encountered at a depth of 51 feet; groundwater was not encountered during the drilling of Boring B2.

LABORATORY TESTS

Laboratory tests were performed on selected samples of soil obtained from the test borings as summarized in the referenced "Geotechnical Engineering Exploration" (BGI, 2014) report. The tests were performed to aid in the classification of the soils and to determine the pertinent engineering properties of the site soils. The following tests were performed:

- moisture content and dry density determinations;
- maximum dry density test;
- expansion index test;
- direct shear tests; and
- consolidation tests;

For a review of the laboratory test results, the reader is referred to the referenced "Geotechnical Engineering Exploration" (BGI, 2014) report.

GEOLOGIC-SEISMIC CONDITIONS

GENERAL

The following provides a summary of the project geologic, hydrogeologic, and seismic conditions. The summary is based on our review of the BGI reports (BGI 2014 and 2017) and a review of geologic-seismic data relevant to the project site. Included is a brief discussion of geologic seismic hazards and their potential impact to the project.

GEOLOGIC MATERIALS

The project site is mantled by a thin veneer of Holocene age alluvial deposits that are underlain at depth by upper Pleistocene age marine terrace deposits and alluvial plain sediments (Hoots, 1930; Poland et al, 1959; and Dibblee, 1991). As indicated by the BGI exploratory borings, the Holocene alluvial deposits extend from ground surface to a depth of 20 feet beneath the project site, where they directly overlie the Pleistocene units. The Holocene deposits consist of interlayered clay, silty clay, sandy silt, and sand/gravelly sand that range from medium stiff to stiff, and medium dense (BGI, 2014). The Pleistocene deposits are composed of medium dense to very dense sand and gravelly sand units.

GROUNDWATER

The project site is located within the Santa Monica Subbasin of the Coastal Plain of Los Angeles Groundwater Basin. This subbasin is bounded by impermeable rocks of the Santa Monica Mountains on the north and the Ballona escarpment on the south. It extends from the Pacific Ocean on the west to the Inglewood fault on the east. Ballona Creek is the dominant hydrologic feature in the subbasin and drains surface waters to the Pacific Ocean. Holocene age alluvium forms much of the surficial deposits for the central part of the subbasin and fills the Ballona gap, an erosional channel cutting into and across the Inglewood fault. These deposits include the clay-rich Bellflower aquiclude and underlying gravels of the Ballona aquifer (California Department of Water Resources [DWR], 1961).

Groundwater levels within the vicinity of the project are monitored by both the Los Angeles County Department of Public Works (LACDPW) and DWR. Water well records from LACDPW and DWR indicate several inactive and active wells within a mile of the project. The closest well was located approximately 600 feet to the east, on Centinela Avenue, just south of Venice Boulevard. This inactive well, designated LACDPW Well No. 2569B, was monitored from August 1, 1934 through December 1, 1952. During that period the highest observed water level was measured on November 1, 1940, at a depth of 53 feet below ground surface. The ground surface elevation was estimated to be about 64 feet above mean sea level (msl) at the well site; therefore, the corresponding water surface elevation was approximately 11 feet msl. The last measurement from Well No. 2569B, recorded on December 1, 1952, indicated water at a depth of 59.4 feet. The closest active wells include DWR Well No. 340161N1184263W004 and 005, and

LACDPW Well Nos. 2578AA and 2578X, located about 1 mile to the northeast. The DWR wells have a limited measurement record, with water levels recorded between January 19, 2011 and March 7, 2017. The highest recorded levels in the DWR wells was approximately 80 feet below ground surface (water surface elevation of approximately 13.5 feet msl), measured on March 7, 2017. Measurements in the two LACDPW wells date back to 1954, with no measurements after October 29, 2010. The highest water levels recorded in LACDPW Well Nos. 2578AA and 2578X were 66 feet (water surface elevation of 27 feet msl) and 89.5 feet (water surface elevation of 3.5 feet msl), respectively. The levels were measured on October 29, 2010 in Well No. 2578AA, and November 29, 1999 in Well No. 2578X.

Groundwater was encountered at a depth of 51 feet in BGI Boring B1 in August 2014. The groundwater depth corresponds to a water surface elevation of 5 feet msl, based on a ground surface elevation of 56 feet msl. Data from the California Geological Survey (formerly known as the California Division of Mines and Geology [CDMG]) indicates the historic high groundwater level as being approximately 40 feet below ground surface (water surface elevation of approximately 16 feet msl) within the vicinity of the project site (CDMG, 1998).

GEOLOGIC HAZARDS

General: Potential geologic and geotechnical hazards include, but are not limited to, primary earthquake hazards (ground shaking and ground rupture), and secondary earthquake hazards from earthquake ground shaking (such as liquefaction, tsunamis, and seiches). Earthquakes have the potential to inflict the greatest loss of life and property damage. Consequently, the location of a site to active or potentially active faults is a key element in assessing the potential for earthquake damage.

Faults: The numerous faults in California include both active and potentially active faults. In accordance with criteria established by the CGS for the Alquist-Priolo Earthquake Fault Zoning program (Hart and Bryant, 1999), a fault can be considered active if it has demonstrated movement within the Holocene epoch, or approximately the last 11,000 years. Faults that have demonstrated

Quaternary movement (last 1.6 million years), but lack strong evidence of Holocene movement, are classified as potentially active. Faults that have not moved since the beginning of the Quaternary period are deemed inactive.

There are no active or potentially active faults beneath the site and the site is not within an Alquist-Priolo Earthquake Fault Zone. Therefore, the potential for fault surface rupture occurring onsite is judged to be low.

Local active faults that could cause significant ground shaking at the site in the event of an earthquake include the Newport-Inglewood, Santa-Monica-Hollywood, and Malibu Coast fault zones. Seismic considerations and design criteria are presented in the "Seismic Coefficients and Factors" section of this report. At their closest point to the site, the Newport-Inglewood and Santa Monica-Hollywood fault zones are situated approximately 2¾ miles northeast and 3½ miles northwest of the site, respectively.

Liquefaction: The Beverly Hills Quadrangle of the State of California Earthquake Zones of Required Investigation map (Seismic Hazard Zone Map), dated March 25, 1999, indicates that the subject site is not within an area classified as being potentially susceptible to liquefaction. Accordingly, the potential for liquefaction impacting the site is judged to be low.

Landslides and Slope Stability: The project site is on relatively level ground, not near ascending or descending slopes, nor in the path of any existing or potential landslides. Consequently, the impact from landslides or potential slope instability is negligible.

Tsunamis and Seiches: The project site is not in a Tsunamis Inundation Area as defined by CGS (2009), and therefore not susceptible to tsunamis (seismic sea waves). Additionally, the site is not located downslope of any large impounded bodies of water that would adversely impact the site as a result of seiches (oscillations in a body of water due to earthquake shaking). Therefore, the site is considered safe from hazards associated with tsunamis or seiches.

Methane Gas: Following the 1985 methane gas fire in the Fairfax area of Los Angeles, the City of Los Angeles adopted Ordinance No. 175790, allowing the City to withhold permits for projects located within either a Methane or Methane Buffer Zone, until such a time as detailed plans addressing methane gas mitigation are developed and approved. A review of the City *Methane and Methane Buffer Zones* map (2004) indicates that the project is not situated within either a methane of methane buffer zone. The closest such zone lies approximately 1½ miles south of the site in the Playa Vista area.

SEISMIC COEFFICIENTS AND FACTORS

Under Section 1613 Earthquake Loads of the CBC, the following coefficients and factors apply to seismic force design of structures at the subject site. The parameters were determined using the Ground Motion Parameter Calculator (Version 5.0.8) at the United States Geological Survey (USGS) Earthquake Hazards website.

Site Class	D
Ss	1.867
S1	0.688
SMs	1.867
SM1	1.032
SDs	1.245
SD1	0.688
PGA	0.684

DISCUSSSION

Based on our understanding of the proposed development and our review of the referenced reports prepared by BGI, the currently proposed subject development is feasible from a geotechnical/geologic perspective. As previously discussed in the "Proposed Development" section of this report, it is anticipated that the subterranean garage portion of the proposed building will essentially extend to the property lines on all sides of the project. Accordingly, the installation of a temporary shoring system is recommended to allow for vertical excavations in association with the construction of the subterranean garage. Once the excavation for the subterranean garage has been made, conventional spread foundations may be utilized for the support of the proposed residential building and parking garage. The bearing soils for the foundations should consist of marine deposits, which are expected to occur at the bottom of the subterranean garage level. Specific recommendations for the installation of temporary shoring and foundations are presented in the following sections of this report.

RECOMMENDATIONS

GENERAL

The recommendations presented in this update report are applicable to the loading conditions and planned construction as described in the previous "Proposed Development" section of this report. If our description of the proposed development is inaccurate because of revisions to the plans or other reasons, we should be informed so that we may review the geotechnical recommendations presented herein and determine if they will remain applicable to the planned construction.

INFILTRATION OF STORM-WATER

Comments regarding the collection of storm-water into Low Impact Development (LID) devices and the infiltration of water into the soil subgrade were discussed in the referenced "Geotechnical Engineering Exploration" (BGI, 2014) report. As discussed in that report, the installation of infiltration basins is not recommended since the entire site will be occupied by the development of the subject residential building.

As an alternative, a biofiltration system may be utilized, provided it is installed in accordance with the requirements of the current City of Los Angeles Best Management Practices. Planter boxes may be used to capture and treat storm-water runoff through different soil layers before being discharged into the off-site, city-maintained storm drain system. Planter boxes should consist of impermeable structures equipped with underdrains to prevent infiltration of water into the underlying subsurface earth materials. For further details regarding the utilization of the biofiltration system, the reader is referred to the referenced "Geotechnical Engineering Exploration" (BGI, 2014) report.

GRADING

No significant grading, such as removal and recompaction of unsuitable soils, is anticipated to be required for the project. It is likely that some grading will be required to compact the subgrade soils exposed at the bottom of the subterranean garage level if those soils become disturbed or loosened as a result of excavation operations or related construction procedures.

If existing fill or otherwise unsuitable naturally deposited soils are present at the current site grade, and those materials are to be used to provide support for concrete slabs, driveways, or foundations located outside the proposed subterranean structure, those materials should be removed and replaced as compacted fill. The expansion potential of the recompacted fill should be determined, and expansive soil recommendations should be provided as necessary, in areas where it is proposed to construct concrete slabs.

In areas where fill is to be placed, the exposed subgrade should be scarified, adjusted to approximately optimum moisture content, and compacted to at least 90 percent of the maximum dry density of the soil as determined by the current ASTM Soil Compaction Method D 1557. The bottoms of areas to be filled should be observed and approved by the representative of the Geotechnical Engineer of Record prior to placement of fill. It will likely be required to also have a representative of the City of Los Angeles, Grading Division, observe and approve bottom areas prior to placement of compacted fill.

Fill should be placed in layers not exceeding 8 inches in loose thickness, adjusted to approximately optimum moisture content, and compacted to at least 90 percent of the maximum dry density of the soil as determined by the current ASTM Soil Compaction Method D 1557.

Soils having low cohesion that are to be placed as compacted fill and are determined to have particle sizes less than 15 percent finer than 0.005 millimeters shall be compacted to at least 95 percent of the maximum dry density of the soil.

Organic and decomposable material should be excluded from the fill, as should solid material exceeding 8 inches in maximum dimension. Fill soils should be placed under the observation and testing services of a representative of the Geotechnical Engineer of Record.

GENERAL GRADING REQUIREMENTS

1. All fills, unless otherwise specifically designed, shall be compacted to at least 90 percent of the maximum dry unit weight as determined by the current ASTM D1557 Method of Soil Compaction.

2. Soils having low cohesion that are to be placed as compacted fill and are determined to have particle sizes less than 15 percent finer than 0.005 millimeters shall be compacted to at least 95 percent of the maximum dry density of the soil.

3. No fill shall be placed until the area to receive the fill has been adequately prepared and subsequently approved by the Geotechnical Engineer of Record or his representative.

4. Fill soils should be kept free of debris and organic material.

5. Rocks or hard fragments larger than 12 inches may not be placed in the fill without approval from the Geotechnical Engineer of Record or his representative, and in a manner specified for each occurrence.

6. The fill material shall be placed in layers which, when compacted, shall not exceed 8 inches in thickness. Each layer shall be spread evenly and shall be thoroughly mixed during the spreading to ensure uniformity of material and moisture.

7. When the moisture content of the fill material is too low to obtain adequate compaction, water shall be added and thoroughly dispersed until the soil is approximately two percent over optimum moisture content.

8. When the moisture content of the fill material is too high to obtain adequate compaction, the fill material shall be adjusted until the soil is approximately two percent over optimum moisture content.

9. Fill and cut slopes should not be constructed at gradients steeper than 2:1 (horizontal:vertical).

DEWATERING

As discussed in the "Groundwater" section of this report, groundwater was encountered at a depth of 51 feet below the present grade within BGI Boring B1. The California Geological Survey indicates that the historic high groundwater level in the vicinity of the site is approximately 40 feet below the ground surface. Since the lowest level of construction is anticipated to extend approximately 26 feet below the present grade, dewatering of the site is not anticipated to be required.

TEMPORARY EXCAVATIONS

It is anticipated that the temporary excavation for the construction of the subterranean garage will extend to various depths. Assuming that spread foundations will extend at least 2 feet below grade (see following "Foundations" section of this report), it is anticipated that the depth of the excavation along the perimeter of the subterranean garage will extend approximately 22 feet below the present grade. It is further anticipated that isolated areas within the interior of the garage, for items such as elevator pits, may extend as much as 26 feet below the existing grade. Due to adjacent existing buildings and property line constraints, a shoring system will be required to support the sides of the temporary excavation for the construction of the proposed subterranean garage.

It is anticipated that temporary excavations of relatively minor heights will be required for the construction of items such as vehicle ramps, elevator pits, utility line trenches, and foundation excavations. Temporary excavations having relatively minor heights may be made without the use of shoring. Temporary excavations that are not subject to surcharge loads and are less than 5 feet in height may be cut vertically. Temporary excavations that are not subject to surcharge loads and are between 5 and 8 feet in height should be sloped at a gradient no steeper than ¾:1 (horizontal:vertical); temporary excavations greater than 8 feet in height should be sloped at a gradient no steeper than 1:1.

Personnel from our firm should observe the temporary excavations and shoring installation so that necessary modifications, based on variations in the soil conditions encountered, can be made. Since the project is located within the City of Los Angeles, it will be required to employ a Los Angeles City Registered Grading Deputy to observe/test all aspects of the shoring installation; we can provide a Registered Grading Deputy to perform the required observation/testing services. All applicable safety requirements and regulations, including OSHA regulations, should be met.

TEMPORARY SHORING

General: Shoring piles should be installed to provide support for the subterranean garage excavation. Cantilevered shoring piles or shoring piles supplemented by the use of conventional or pressure-grouted tie-back earth anchors, raker-braces, cross-bracing, or a combination of those methods may be utilized. The recommendations for temporary shoring presented in this report include a factor of safety of greater than 1.5. The Geotechnical Engineer of Record should review and approve the final shoring plans and specifications prior to negotiations with a shoring contractor. Further details of shoring construction are presented in Appendix B of this report.

Perimeter Structures: Prior to excavating near the perimeter of the planned excavation, all existing structures located on or near the sides of the excavation should be underpinned, braced, and/or fully supported and restrained by the installed shoring. Otherwise, existing structures around the perimeter of the excavation should be removed. It is recommended that a thorough survey of the condition of adjacent existing improvements, including photo documentation, be performed prior to the initiation of excavation or shoring installation.

Cantilevered Shoring: Cantilevered shoring piles may be used to provide temporary support for the subterranean garage excavation. Since the excavation around the perimeter of the subterranean garage will extend approximately 22 feet below the present grade, and portions of the excavation will be subject to surcharge loads, it is anticipated that the lengths of cantilevered shoring piles would be substantial. It is anticipated that groundwater will be encountered within pile excavations that approach 50 feet in depth. It should be noted that it will be required to increase the design strength of concrete by 1,000 pounds per square inch (psi) should it become necessary to utilize a rigid tremie pipe and cast concrete in standing water more than 12 inches in depth.

The referenced "Geotechnical Engineering Exploration" report (BGI, 2014) presented an Active Equivalent Fluid Pressure to be used in the design of temporary cantilevered shoring piles. The recommended design pressures included applicable surcharge loads. Those recommended design pressures remain applicable and are summarized below:

North Property Line	39 pcf
South Property Line	39 pcf
East Property Line	43 pcf
West Property Line	39 pcf

Tieback/Braced Shoring: For the design of tied-back or braced shoring, we recommend the use of a uniform distribution of earth pressure. The recommended pressure distribution, for the case where the grade is level behind the shoring, is a uniform lateral rectangular earth pressure equal to 26H in psf (exclusive of hydrostatic pressure), throughout the depth of excavation, where H is the height of the shoring in feet. This lateral pressure does not incorporate the adjacent surcharge loads from existing streets, traffic, and buildings. Additional surcharge loads from items such as cranes, adjacent buildings, and traffic should be added to the recommended lateral pressure, where applicable. Surcharge loads should be assumed to be distributed at 45 degree angles from the bottom of the existing structures.

The design of the upper 10 feet of walls below grade should include a uniform rectangular lateral pressure of 100 psf due to adjacent traffic unless the traffic is kept at least 10 feet away from the perimeter of the shoring. This additional lateral pressure would be the result of an assumed 300 psf surcharge behind the wall due to normal street loads.

Rakers: If raker braces are utilized for shoring, the following provides general data regarding their use. Raker braces for shoring would be installed at an angle. Footings for the rakers (deadmen) would be used to resist the axial load of the rakers. Such inclined loads would be less than what could be supported by a conventional footing with a vertical load. Design data for raker footings would depend on the type of supporting soil and raker angle, among other factors. As guidance for raker footing needs, a continuous footing founded in the anticipated dense sand, supporting a raker installed at 45 degrees from the vertical, with the raker footing extending at least two feet below lowest adjacent grade and being at least 5 feet in width, would have a design bearing value of 2,500 psf. We can provide bearing values, footing sizes, and embedment depths for specific cases upon request.

Design of Shoring Piles: For the design of shoring piles spaced at least 2½ diameters on centers, the allowable lateral bearing value (passive value) of the soils below the bottom of the excavation may be assumed to be zero at the excavated surface, increasing at the rate of 600 psf per foot of depth, to a maximum of 6,000 psf. To develop the full lateral value, provisions should be taken to assure firm contact between the piles and the undisturbed soils. The concrete placed in the pile excavations above the excavation bottom may be a lean-mix concrete. The concrete used in the portion of the shoring pile that is below the planned excavated level should be of sufficient strength to adequately transfer the imposed loads to the surrounding soils. It should be noted that it will be required to increase the design strength of concrete by 1,000 pounds per square inch (psi) should it become necessary to utilize a rigid tremie pipe and cast concrete in standing water more than 12 inches in depth.

The piles below the excavated level may be used to resist downward loads, provided that those portions of the piles below the excavated level consist of structural concrete. The frictional resistance between the concrete soldier piles and the soils below the excavated level may be taken as equal to 500 psf. (This value is based on the assumption that uniform full bearing will be developed between the steel shoring beam and the lean-mix concrete and between the lean-mix concrete and the retained earth.)

Shoring Pile Installation: Groundwater is not anticipated to be encountered during the drilling of shoring piles and/or tie-back earth anchors. Significant caving is not anticipated to occur during the drilling of shoring piles and/or tie-back earth anchors. However, if localized caving or sloughing of soil occurs, it may become necessary to utilize casing or other means of maintaining an "open" excavation during the installation of the shoring elements. The boring logs presented in this report (Appendix A) should be reviewed by the shoring contractor for a description of the anticipated soil conditions.

Lagging: Lagging most likely will be required between the shoring piles if sandy soils are exposed in the excavation; accordingly, it is recommended that it be planned that lagging should be installed in all areas. Any areas where it is desired to omit lagging should be observed and approved by a representative of the Geotechnical Engineer of Record. The governing agency (the City of Los Angeles) should be contacted regarding any restrictions related to the elimination of lagging. Lagging may not be eliminated in areas where the excavation will be subject to surcharge loads from existing structures or items such as cranes or other construction equipment.

Lagging must be installed in a manner that will allow the lagging boards to be in direct contact with the retained soils. It is recommended that any voids that are present behind lagging boards be backfilled with lean-mix concrete (sand-cement slurry). No more than 5 vertical feet of lagging should be installed without being properly backfilled with lean-mix concrete backfill. Installation of lagging should be performed under the observation of a Los Angeles City Registered Grading Deputy.

The shoring piles and anchors should be designed for the full anticipated lateral pressure. The pressure on the lagging, however, will be less due to arching in the soils. The lagging should be designed for the recommended earth pressure but limited to a maximum value of 400 psf.

Anchor Design: Conventional tie-back friction anchors or pressure-grouted tie-back earth anchors may be used to resist lateral loads in areas where they are permitted to be installed beyond the subject property. For design purposes, it may be assumed that the active wedge adjacent to the shoring is defined by an imaginary plane projected at 55 degrees from the horizontal to the bottom of the excavation. The anchors should extend at least 15 feet beyond the potential active wedge formed by this plane, and to a greater length as necessary, to develop the desired capacities.

The capacities of anchors should be determined by testing of the initial anchors as outlined in the following "Anchor Installation" subheading of this report. Only the frictional resistance developed beyond the active wedge should be considered as effective in resisting lateral loads. If the anchors are spaced at least 6 feet on centers, no reduction in the capacity of the anchors need be considered due to group action.

Anchor Capacities: The soil/concrete strength at the interface along the length of an anchor will determine the capacity of that anchor. The anchor strengths presented in the following paragraph are suggested as the maximum values likely to be obtained. It is expected that the Shoring Engineer would use these (or lesser) values to determine the anchor test loads as opposed to the anchor design loads.

It is anticipated that the majority of the tie-back anchors will be founded in medium stiff to very stiff silts and clays and/or medium dense to dense sands. It is suggested that an ultimate friction of 500 psf for initial anchors be assumed for the bond length when designing conventional anchors in open shafts with concrete placed through a standard tremie pipe. For pressure-grouted or post-grouted anchors, a value of 1,800 psf is suggested as a friction design value for the initial testing of anchors.

The shear strength, which may be mobilized between the soil and the bond length of the earth anchor, will vary as a function of construction technique; the shear strength suggested above is considered to represent the upper limit. It is possible that some reduction of the available friction strength may result in areas where difficulties occur during the drilling and/or casting procedures. It is for this reason that we strongly recommend that initial anchor tests be conducted as early as possible to determine if the stated values are, in fact, available for use in production anchors.

Anchor Installation: Selected tie-back anchors should initially be installed and tested prior to installing production anchors. Based on the results of the initial anchor tests, the lengths, design friction values, or diameters of production anchors may be adjusted. Tieback anchors may be installed at angles of 15 to 40 degrees below the horizontal. For open shafts, the anchors should be filled with concrete placed by utilizing a tremie pipe and pumping concrete to the tip of the anchor. The anchors may also be of the pressure-grouted or post-grouted type. Large diameter, open shaft anchors could experience significant caving if relatively clean sands are encountered. Testing of anchors should be performed prior to backfilling the active wedge.

If caving soils are encountered in the active wedge during the drilling of anchors, we suggest that the portion of the anchor shaft within the active wedge be backfilled with sand before testing the anchor. The sand backfill may contain a small amount of cement to allow the sand to be placed by pumping.

Anchor Testing: In general, anchors are considered adequate if 150 percent of the design load is applied for a period of at least 15 minutes and the anchor deflection is less than 0.1 inch during that time period. The Geotechnical Engineer of Record should select at least one initial anchor on each tie-back level for a 24-hour 200 percent performance test. A quick 30-minute 200 percent test should be performed on 10 percent of the anchors. It is the responsibility of the Shoring Contractor to install an anchor rod or strand of sufficient strength to withstand the 200 percent test load. The purpose of the longer 200 percent tests is to verify the friction value assumed in design. The anchors should be tested to develop 2.0 times the assumed friction value. 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.

The total deflection during the 24-hour 200 percent test should not exceed 12 inches during loading; the anchor deflection should not exceed 0.75 inch during the 24-hour period, measured after the 200 percent test load is applied. If the anchor movement is less than 0.5 inch after the 200 percent load has been applied for 12 hours, and the movement over the previous 4 hours has been less than 0.1 inch, the test may be terminated.

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

It should be noted that deformation and/or twisting of shoring beams may occur during the application of the 200 percent test loads on the tie-back earth anchors. This often results in difficulties associated with maintaining a constant load application on the tie-back earth anchor. "Strapping" of shoring beams, or similar methods of decreasing the deformation and/or twisting of shoring beams, may be required and should be anticipated by the shoring contractor.

A test load of at least 150 percent of the design load should be applied to all of the production anchors. The total deflection during testing should not exceed 12 inches for each anchor. The deflection 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 load.

After a satisfactory test, each production anchor should be "locked off" at the design load. The actual "lock off" load should be verified by checking the "lift-off" load of the anchor. The "lock off" load must be between 95 percent and 110 percent of the design load of the anchor.

The installation of the soldier piles, tie-back anchors, and load testing of the anchors should be observed by a representative of the Geotechnical Engineer of Record. Since the project is located within the City of Los Angeles, it will be required to utilize a Los Angeles City Registered Grading Deputy to observe/test all aspects of the shoring installation. Observation of the removal of upper portions of tie-back rods and shoring beams located within the public right-of-way may be required after the building is under construction and the shoring no longer becomes necessary.

Deflection: It is difficult to accurately predict the amount of deflection of a shored embankment. It should be realized, however, that some deflection will occur. We estimate that this deflection could be on the order of one inch at the top of the shored embankment. If greater deflection occurs during construction, additional bracing may be necessary to reduce further deflection and minimize the settlement of adjacent structures and/or nearby utility lines. If it is desired to reduce the deflection of the shoring, a greater active pressure could be used in the shoring design.

Monitoring: It will be important to monitor the performance of the shoring system. The monitoring should consist of periodic surveying of the lateral and vertical locations of the tops of all the shoring piles. Further details of the monitoring procedures are provided in Appendix B.

FOUNDATIONS

The recommendations presented in this section are based on the assumption that the proposed building will have foundation loads similar to those described in the "Proposed Development" section of this report. Conventional spread foundations, founded in the Marine Deposit materials that are expected to occur at depths of approximately 20 feet below the existing grade, may be used to provide support for the proposed residential building and parking garage.

As previously discussed in this report, it is anticipated that the subterranean garage will be supported by the Marine Deposit materials that are expected to be present at the bottom of the subterranean garage. Foundations constructed at or near the present grade will likely be founded in naturally deposited clays or silts.

Foundations should be founded at depths of at least 24 inches below the lowest adjacent final grade. The foundations should be a minimum of 24 inches in width and be designed in accordance with the City of Los Angeles Building Code (CBC).

Foundations should be cast integrally and continuously under all exterior and interior bearing walls and should be reinforced continuously at the top and bottom of the foundations. Foundations for the proposed structure should have a minimum of four No. 4 steel reinforcement bars, two at the top of the footing and two at the bottom of the footing. Areas where access openings are proposed should include a concrete foundation and reinforcing steel below the proposed access openings. The design of foundation reinforcement is deferred to the project structural engineer.

Provided that the bearing material consists of the recommended marine deposits, foundations constructed in accordance with the recommendations presented above may be designed using a bearing value of 3,000 pounds per square foot (psf) for combined dead and frequently applied live loads. This bearing value applies to both continuous and isolated footings and may be increased by one-third for the total of all loads, including those attributed to seismic and wind forces. The recommended bearing value is a net value; i.e., the mass of concrete in footing pads may be neglected when computing the footing dimensions.

It is possible that foundations for incidental structures may be constructed at or near the present site grade. Although foundations for the subject building should be founded entirely in naturally deposited marine deposits that are expected to be present at depths of approximately 20 feet below the present grade, foundations for incidental structures and related construction can be founded in native soils and/or compacted fill. It may become necessary to deepen and/or compact the bottoms of foundations that are founded in naturally deposited soils if those soils are determined to be unsatisfactory. A bearing value of 1,500 psf may be used for the design of foundations for incidental structures, provided the recommendations of this report have been implemented.

Foundations should be deepened, where necessary, to prevent surcharge loads from being imposed upon adjacent foundations or utilities. Surcharge loads should be assumed to be distributed out from the bottom edges of foundations at 45-degree angles. Foundation excavations should be cleaned of all loose material and be observed and approved by a representative of the Geotechnical Engineer of Record prior to casting concrete.

The Foundation Plans for the proposed development should be reviewed by our firm prior to the initiation of construction. The Geotechnical Engineer of Record should sign and stamp the plans, provided the plans have been found to conform to the geotechnical recommendations presented in this report.

LATERAL DESIGN

Lateral resistance at the bases of footings or slabs may be assumed to be the product of the dead load and a coefficient of friction of 0.35. Passive pressure on the faces of footings and grade beams may also be used to resist lateral forces. A passive pressure of zero at the surface of finished grade, increasing at the rate of 230 psf per foot of depth, to a maximum of 6,000 psf, may be used for this project. When the passive pressure and friction are combined for lateral resistance, the passive component should be reduced by one-third.

SETTLEMENT

Provided that the proposed structure does not exceed the assumed structural loads and is founded in the recommended naturally deposited marine deposits, we estimate that the total static settlement will be up to about 1.0 inch. Static differential settlement is expected to be less than 0.75 inches within a horizontal distance of 30 feet.

FLOOR SLABS

General: The floor slab recommendations presented in this section are based on the assumption that the soil subgrade will consist of the Marine Deposit materials that are expected to be present at the bottom of the subterranean garage level and that concrete slabs will be subjected to normal loads with no special requirements. Any near-surface soils that become dried or disturbed during construction should be moisture-conditioned and compacted prior to casting slabs.

Concrete floor slabs should be reinforced with No. 4 bars spaced at 16 inches on center, in each direction. It is recommended that the soil subgrade be thoroughly moistened prior to casting the concrete slabs. Additional reinforcement may be required, depending on the floor loads and the structural requirements. The slab thicknesses and reinforcing should be determined by the Project Structural Engineer and designed in accordance with the requirements of the City of Los Angeles.

Expansive Soil Conditions: An expansion index test was performed by BGI on a sample of soil obtained from within the upper 5 feet of the subject site. Based on the results of the test, the soil was classified as having a “low” potential for expansion.

The soils that are expected to be present at the bottom of the subterranean garage level are expected to consist of marine deposits. The marine deposits primarily consist of well-graded and poorly-graded gravelly sands. Accordingly, those materials are not anticipated to be expansive and no special preparation of the soil subgrade, relative to expansive soil conditions, is anticipated to be required.

Moisture-Sensitive Flooring: Water vapor transmitted through floor slabs is a common cause of floor covering problems. An impermeable membrane "vapor barrier" should be installed to reduce excess vapor drive through floor slabs. The function of the impermeable membrane is to reduce the amount of water vapor transmitted through the floor slab. Vapor-related impacts should be expected in areas where a vapor barrier is not installed.

Floor slabs should be underlain by a vapor barrier surrounded by 2 inches of sand above and below the barrier. The vapor barrier should be at least 10 millimeters thick; care should be taken to preserve the continuity and integrity of the barrier beneath the floor slab. The sand should be sufficiently moist to remain in place and be stable during construction; however, if the sand above the barrier becomes saturated before placing concrete, the moisture in the sand can become a source of water vapor.

Another factor affecting vapor transmission through floor slabs is a high water-to-cement ratio in the concrete used for the floor slab. A high water-to-cement ratio increases the porosity of the concrete, thereby facilitating the transmission of water and water vapor through the slab. The Project Structural Engineer or a concrete mix specialist should provide recommendations for the design of concrete for footings and floor slabs in accordance with the CBC, with consideration of the above comments.

WALLS BELOW GRADE

Subterranean and braced walls should be designed to resist a uniform distribution of lateral earth pressure. The at-rest lateral earth pressure on the permanent subterranean walls for drained soils will be a rectangular earth pressure equal to 41H in psf, (where H equals the wall height) extending from the proposed ground surface to the bottom of the subterranean garage level. This lateral pressure does not incorporate the adjacent surcharge loads from existing streets, buildings, and vehicle traffic. Additional surcharge loads from those items should be added to the recommended lateral pressure, where applicable.

The upper 10 feet of walls below grade should be designed for an additional uniform rectangular lateral pressure of 100 psf due to adjacent traffic unless the traffic is kept at least 10 feet away from the perimeter of the shoring. This additional lateral pressure would be the result of an assumed 300 psf surcharge behind the wall due to normal street loads.

WATERPROOFING

Waterproofing needs are deferred to a waterproofing consultant for this project. Waterproofing should be installed for the full depth of the subterranean garage walls and floor slabs.

BACKFILL

Temporary sloped excavations may be utilized during the construction of the development as discussed in the previous “Temporary Excavations” section of this report. It will be necessary to place compacted backfill behind walls constructed for vehicle ramps, and similar items, in areas where temporary sloped excavations have been made. Efforts should be made to utilize soil backfill that is granular in nature. Backfill should not be placed until the bottom of the area to be filled has been observed and approved by a representative of the Geotechnical Engineer of Record. Local governing agencies may also require that they observe bottom areas prior to fill placement. Form lumber and other debris should be removed from the bottom of the area to be filled prior to placement of backfill.

It may prove economical to place clean, uniform-size gravel, rather than soil, for backfill in selected areas. Each 3-foot vertical thickness of gravel backfill should be well vibrated prior to placing additional gravel. Gravel backfill should be surrounded with a filter fabric (Mirafi 140N, or equivalent) to reduce soil infiltration into the gravel. A “soil cap” of at least 18 inches in thickness and consisting of relatively impermeable (cohesive) soil should be placed above all gravel that has been utilized as backfill, exclusive of areas where the backfill will be covered by asphalt or concrete pavement.

Soil backfill should be placed in layers not exceeding 8 inches in loose thickness, adjusted to approximately optimum moisture content, and compacted to at least 90 percent of the maximum dry density as determined by the current ASTM Soil Compaction Method D1557.

Organic and decomposable material should be excluded from the backfill, as should solid material exceeding 8 inches in maximum dimension. Backfill should be placed under the observation and/or testing of a representative of the Geotechnical Engineer of Record.

Some settlement of the compacted backfill should be expected, especially in areas where the backfill is relatively deep. Utility lines within the backfill should be designed to accommodate differential settlement, particularly in areas where the lines enter the proposed building.

WALL DRAINAGE

A drainage system should be provided behind retaining walls, or the walls should be designed to resist hydrostatic pressure equivalent to fluid pressure of 62 pcf. In addition to the subterranean garage walls, it will be necessary to provide a drainage system for backfill placed behind vehicle ramps.

Retaining wall backfill may be drained by utilizing a perforated pipe. The perforated pipe should be at least 4 inches in diameter and be placed at the base of the wall with the perforations pointed down. The pipe should be sloped to provide positive drainage, but in no instance shall the pipe be elevated more than 2 feet above the bottom of the wall. The pipe should be surrounded by at least 6 inches of uniform-sized gravel and be permitted to outlet onto a surface that would not be subject to erosion, or the drain should be connected to a suitable outlet device. The gravel should be separated from the surrounding soils by a filter fabric, such as Mirafi 140N or equivalent, wrapped around the gravel ("burrito-wrapped"). Alternatively, the filter fabric and gravel may be omitted when using a continuous slotted pipe and graded sand that conforms to the LACDPW "Graybook" F-1 Designated Filter Material.

Drainage panels (such as Miradrain, or equivalent) or a 6- to 12-inch-wide gravel chimney drain should be installed behind retaining walls that are greater than 3 feet in height. The top of the drainage panels or chimney drain should be capped with 18 to 24 inches of compacted on-site soil. The intent of installing the drainage panels, or chimney drain, would be to reduce the potential for build-up of water directly behind the walls. Excessive build-up of water could result in wall failure.

The installed drainage system should be observed by the Geotechnical Engineer of Record prior to backfilling the system. Observation of the drainage system may also be required by the reviewing governmental agencies prior to backfilling.

LIMITATIONS

Our professional services have been performed using that degree of care and skill ordinarily exercised, under similar circumstances, by reputable geotechnical engineers and engineering geologists practicing in this or similar localities. No other warranty, expressed or implied, is made as to the professional advice included in this report. This report has been prepared for Michael Baker International and LADCP, to be used solely for planning and design of 12444 Venice Boulevard. The report has not been prepared for use by other parties and may not contain sufficient information for purposes of other parties or other uses.

OBSERVATION/TESTING SERVICES

This report has been prepared assuming that R. T. Frankian & Associates will perform all geotechnical field observations and testing. If the recommendations presented in this report are utilized, and inspection of the geotechnical work is performed by others, the party performing the inspections must review this report and either assume responsibility for the recommendations presented herein, or provide its own report. That party would then assume the title "Geotechnical Engineer of Record" for the project and respond to any design and construction related issues that may arise.

A representative of the Geotechnical Engineer of Record should be present to observe excavation, shoring, grading, and backfilling operations, as well as all foundation excavations. A report presenting the results of these observations and related testing should be issued upon completion of these operations.

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The following are attached and complete this report:

- References
- Plot Plan
- Appendix A – Explorations
  – Unified Soil Classification System, Figure A-1
  – Log of Borings by BGI (Borings B1 and B2)
- Appendix B – Guide to Shoring Design and Construction

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PDF Distribution via Email:
– Michael Baker International, Attn: Mr. John Bellas

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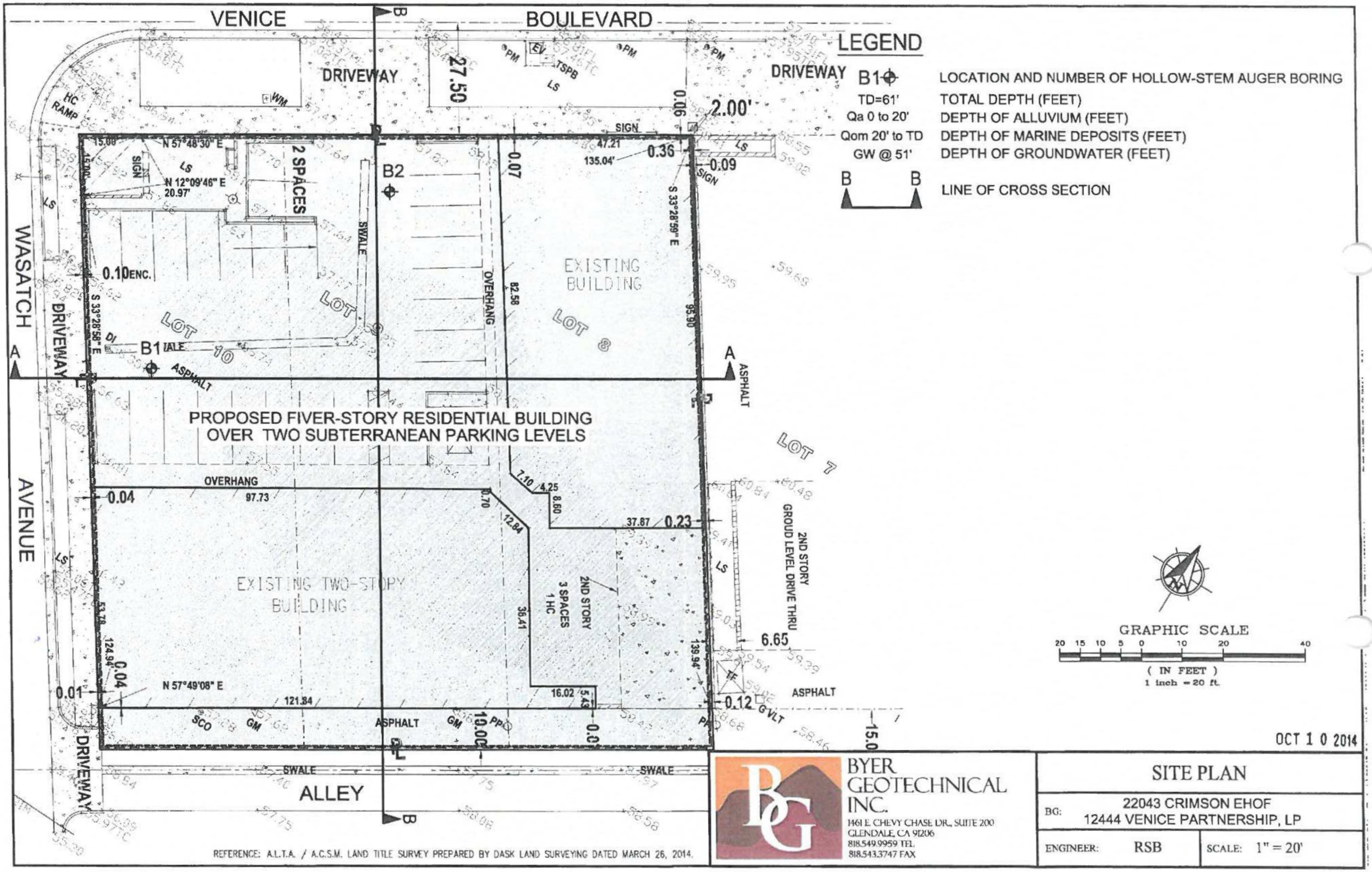

PLOT PLAN
Michael Baker International
August 18, 2017
2017-012-060

APPENDIX A

EXPLORATIONS

GEOTECHNICAL ENGINEERING & ENGINEERING GEOLOGY

APPENDIX A

EXPLORATIONS

The soil conditions beneath the site were explored by BGI as presented in the referenced "Geotechnical Engineering Exploration" report (BGI, 2014). Two separate test borings, Borings B1 and B2, were drilled on August 28, 2014, using a hollow-stem-auger drill rig. Samples of the soils encountered during the drilling of the test borings were obtained for laboratory testing as presented in following Appendix B. The approximate locations of the test borings are indicated on the attached Plot Plan. The logs of the test borings drilled under the observation of BGI are presented in this Appendix. The soils encountered were classified in accordance with the United Soil Classification System.

MAJOR DIVISION			GROUP SYMBOLS	TYPICAL NAMES
COARSE GRAINED SOILS More than 50% retained on No. 200 (75 µm) sieve*	GRAVELS 50% or more of coarse fraction retained on No. 4 (4.75mm) sieve	CLEAN GRAVELS (Little or no fines)	GW	Well graded gravels, gravel-sand mixtures, little or no fines
			GP	Poorly graded gravels, gravel-sand mixtures, little or no fines
		GRAVELS WITH FINES (Appreciable amount of fines)	GM	Silty gravel, gravel-sand-silt mixture
			GC	Clayey gravels, gravel-sand-clay mixture
	SANDS More than 50% of coarse fraction passes No. 4 (4.75 mm) sieve	CLEAN SANDS (Little or no fines)	SW	Well graded sands, gravelly-sands, little or no fines
			SP	Poorly graded sands, gravelly-sands, little or no fines
		SANDS WITH FINES (Appreciable amount of fines)	SM	Silty sands, sand-silt mixtures
			SC	Clayey sands, sand-clay mixtures
FINE-GRAINED SOILS 50% or more passes No. 200 (75 µm) sieve	SILTS AND CLAYS (Liquid limit LESS than 50)		ML	Inorganic silts and very fine sands, rock flour, silty or clayey fine sands or clayey silts with slight plasticity
			CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays
			OL	Organic silts and organic silty clays of low plasticity
	SILTS AND CLAYS (Liquid limit GREATER than 50)		MH	Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts
			CH	Inorganic clays of high plasticity, fat clays
			OH	Organic clays of medium to high plasticity, organic silts
HIGHLY ORGANIC SOILS			PT	Peat and other highly organic soils

*Based on the material passing the 3-inch (76 mm) sieve.

BOUNDARY CLASSIFICATIONS: Soils possessing characteristics of two groups are designated by a combination of group symbols.

PARTICLE SIZE LIMITS

SILT OR CLAY	SAND			GRAVEL		COBBLES	BOULDERS
	Fine	Medium	Coarse	Fine	Coarse		

No. 200 No. 40 No. 10 No. 4 3/4 in. 3 in. 12 in.

REFERENCE: ASTM D-2487

UNIFIED SOIL CLASSIFICATION SYSTEM

SAMPLE KEY:

FRANKIAN LINED-BARREL SAMPLER (3.50" O.D., 2.625" I.D., 8.0" LONG SAMPLE TUBE)

STANDARD PENETRATION TEST (ASTM D-1586)

CALIFORNIA SAMPLER

NO RECOVERY / DISTURBED SAMPLE

BULK SAMPLE

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LOG OF BORING B1

BG No. 22043

PAGE 1 OF 3

CLIENT Crimson EHOFF 12444 Venice Partnership, LP REPORT DATE 10/10/14 DRILL DATE 8/28/14

PROJECT LOCATION 12440 - 12492 West Venice Blvd., Los Angeles, CA LOGGED BY JHP

CONTRACTOR 2R Drilling DRILLING METHOD Hollow-Stem Auger HOLE SIZE 8-inch diameter

DRIVE WEIGHT 140-Pound Automatic Hammer HAMMER DROP 30 Inches ELEV. TOP OF HOLE 56 ft

ELEVATION (ft)	DEPTH (ft)	EARTH MATERIAL DESCRIPTION	GRAPHIC SYMBOL	USCS UNIT	SAMPLE TYPE & NUMBER	BLOW COUNT (Per 6 Inches)	MOISTURE CONTENT (%)	DRY UNIT WT. (pcf)	SATURATION (%)	TYPE OF TEST
	0	Surface: 3 inches asphalt over 4 inches base (parking lot).								
55		(CL) **ALLUVIUM (Qa):** 0.5'-2.5': CLAY, dark brown, slightly moist, medium plasticity.		CL						
		(CL-ML) 2.5': Silty CLAY, brown to dark brown, slightly moist, medium stiff, fine sand.		CL-ML	BAG1 S1	2 3 4	16			Max, EI
50	5	(ML) 5': Sandy SILT, brown, slightly moist, stiff, fine sand, some fine gravel up to 1/2 inches subangular.		ML	R1	5 10 11	14.9	114.7	89	Direct Shear
		(ML) 7.5': Sandy SILT, brown, moist, medium stiff to stiff, fine sand, some fine gravel up to 3/8 inches subangular.		ML	S2	2 4 4	14.7			
45	10	(SM) 10': Top: Silty SAND, brown, slightly moist, medium dense, fine sand, some fine gravel up to 3/8 inches subangular. (SW) Bottom: Gravelly SAND, greenish-brown, slightly moist, medium dense, fine to coarse sand, fine gravel up to 1/2 inches subangular to subrounded.		SM SW	R2	9 16 17	3.6	123	28	Direct Shear
40	15	(SW) 15': Gravelly SAND, dark grayish-brown, slightly moist, medium dense, fine to coarse sand, fine to coarse gravel up to 3 inches subangular to subrounded.		SW	S3	9 11 11	5.1			
35	20	(SW) **MARINE DEPOSITS (Qom):** 20': Top: Gravelly SAND, light brown, slightly moist, medium dense, fine to coarse sand, fine to coarse gravel up to 2.5 inches subangular to subrounded. (SP) Bottom: SAND, light brown, slightly moist, medium dense, fine sand, some medium sand, trace coarse sand.		SW SP	R3	17 20 21	3.1	112.5	17	Direct Shear, Consolidation
	25									

Bulk Sample Standard Penetration Test Ring Sample

BORING LOG BYER BY RSB - GINT STD US BYER.GDT - 10/10/14 12:51 - P:\22000 - 22999\22043 CRIMSON HOLDINGS, 12444 VENICE BLVD LA\22043 BORING LOGS.GPJ

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LOG OF BORING B1

BG No. 22043

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ELEVATION (ft)	DEPTH (ft)	EARTH MATERIAL DESCRIPTION	GRAPHIC SYMBOL	USCS UNIT	SAMPLE TYPE & NUMBER	BLOW COUNT (Per 6 Inches)	MOISTURE CONTENT (%)	DRY UNIT WT. (pcf)	SATURATION (%)	TYPE OF TEST
30	25	(SP) 25': SAND with Gravel, light brown, slightly moist, very dense, fine to medium sand, fine to coarse gravel up to 2 inches subrounded.		SP	S4	14 44 45	2.4			
25	30	(SW) 30': Gravelly SAND, light brown, slightly moist, dense, fine to coarse sand, fine to coarse gravel up to 1.5 inches subrounded.		SW	R4	46 50/4"	2.5	117.9	17	Consolidation
20	35	(SW) 35': Gravelly SAND, grayish-brown to brown, slightly moist, very dense, fine to coarse sand, fine to coarse gravel up to 3.5 inches subangular to subrounded, some silt pockets.		SW	S5	6 20 30	13.5			
15	40	(SW) 40': Gravelly SAND, brown to grayish-brown, slightly moist, dense, fine to coarse sand, fine to coarse gravel up to 3 inches subangular to subrounded.		SW	R5	50/5"	3.4	118.1	23	
10	45	(SW) 45': SAND with gravel, light brown to tan, slightly moist, very dense, fine to coarse sand, fine to coarse gravel up to 3.5 inches subangular to subrounded.		SW	S6	26 46 46	2.6			
	50									

Bulk Sample | Standard Penetration Test | Ring Sample

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LOG OF BORING B1

BG No. 22043

CLIENT Crimson EHOFF 12444 Venice Partnership, LP REPORT DATE 10/10/14 DRILL DATE 8/28/14

PROJECT LOCATION 12440 - 12492 West Venice Blvd., Los Angeles, CA LOGGED BY JHP

CONTRACTOR 2R Drilling DRILLING METHOD Hollow-Stem Auger HOLE SIZE 8-inch diameter

DRIVE WEIGHT 140-Pound Automatic Hammer HAMMER DROP 30 Inches ELEV. TOP OF HOLE 56 ft

ELEVATION (ft)	DEPTH (ft)	EARTH MATERIAL DESCRIPTION	GRAPHIC SYMBOL	USCS UNIT	SAMPLE TYPE & NUMBER	BLOW COUNT (Per 6 Inches)	MOISTURE CONTENT (%)	DRY UNIT WT. (pcf)	SATURATION (%)	TYPE OF TEST
5	50	(SW) 50': Gravelly SAND, light brown to brown, wet, dense, fine to coarse sand, fine to coarse gravel up to 3.5 inches subangular to subrounded. 51': Groundwater.		SW	R6	50/5"	9.2	123.5	72	Consolidation
0	55	(SW) 55': Gravelly SAND, yellowish-brown, wet, dense, fine to coarse sand, fine to coarse gravel up to 3.5 inches subangular to subrounded, fine sand at sampler tip.		SW	S7	21 23 20	19.8			
-5	60	(SP) 60': SAND, light brown, wet, dense, fine sand, some coarse sand.		SP	R7	47 50/3"	15.1	111.1	82	Consolidation

End at 61 Feet; Groundwater at 51 Feet.

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Bulk Sample   Standard Penetration Test   Ring Sample

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LOG OF BORING B2

BG No. 22043

CLIENT Crimson EHOFF 12444 Venice Partnership, LP REPORT DATE 10/10/14 DRILL DATE 8/28/14

PROJECT LOCATION 12440 - 12492 West Venice Blvd., Los Angeles, CA LOGGED BY JHP

CONTRACTOR 2R Drilling DRILLING METHOD Hollow-Stem Auger HOLE SIZE 8-inch diameter

DRIVE WEIGHT 140-Pound Automatic Hammer HAMMER DROP 30 Inches ELEV. TOP OF HOLE 57 ft

ELEVATION (ft)	DEPTH (ft)	EARTH MATERIAL DESCRIPTION	GRAPHIC SYMBOL	USCS UNIT	SAMPLE TYPE & NUMBER	BLOW COUNT (Per 6 Inches)	MOISTURE CONTENT (%)	DRY UNIT WT. (pcf)	SATURATION (%)	TYPE OF TEST
	0	Surface: 3.5 inches asphalt, 6 inches base (parking lot).								
55		(CL) **ALLUVIUM (Qa):** 0.75'-5': CLAY, dark brown, slightly moist, medium plasticity.		CL						
	5	(ML) 5': Sandy SILT, dark brown, slightly moist, medium stiff, fine sand.		ML	S1	1 2 3	16.9			
50										
	10	(CL) 10': CLAY, brown, slightly moist, very stiff, some fine sand.		CL	R1	8 18 26	15.8	115.6	97	
45										
	15	(CL) 15': Top: CLAY, brown, slightly moist, stiff, medium plasticity.		CL	S2	4 6 8	26.8			
40		(CL) Bottom: Sandy CLAY, brown, slightly moist, stiff, fine to medium sand, some fine gravel up to 1/2 inch subrounded.		CL						
	20	(SP) **MARINE DEPOSITS (Qom):** 20': SAND, yellowish-brown, slightly moist, dense, fine sand, some medium sand, some fine gravel up to 1/2 inches subrounded.		SP	R2	12 28 42	3.3	108.7	17	Direct Shear
35										
	25									

Standard Penetration Test   Ring Sample

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LOG OF BORING B2

BG No. 22043

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PROJECT LOCATION 12440 - 12492 West Venice Blvd., Los Angeles, CA LOGGED BY JHP
CONTRACTOR 2R Drilling DRILLING METHOD Hollow-Stem Auger HOLE SIZE 8-inch diameter
DRIVE WEIGHT 140-Pound Automatic Hammer HAMMER DROP 30 Inches ELEV. TOP OF HOLE 57 ft

ELEVATION (ft)	DEPTH (ft)	EARTH MATERIAL DESCRIPTION	GRAPHIC SYMBOL	USCS UNIT	SAMPLE TYPE & NUMBER	BLOW COUNT (Per 6 Inches)	MOISTURE CONTENT (%)	DRY UNIT WT. (pcf)	SATURATION (%)	TYPE OF TEST
	25	(SP) 25': SAND, light brown, slightly moist, dense, fine sand, some medium to coarse sand, trace fine gravel up to 1/2 inches subrounded.		SP	S3	12 15 18	3			
30										
	30	(SW) 30': Gravelly SAND, grayish-brown, slightly moist, dense, fine to coarse sand, fine to coarse gravel up to 2 inches subangular to subrounded.		SW	R3	13 14 50/2"	2.6	123.8		

End at 31.5 Feet; No Groundwater

Standard Penetration Test   Ring Sample

Michael Baker International
August 18, 2017
2017-012-060

APPENDIX B

GUIDE TO SHORING DESIGN AND CONSTRUCTION

RTF&A
GEOTECHNICAL ENGINEERING & ENGINEERING GEOLOGY

APPENDIX B

GUIDE TO SHORING DESIGN AND CONSTRUCTION

INTRODUCTION

This Appendix further explains details for the construction and testing of shoring to be used to provide temporary support of the proposed excavation for the construction of the subject subterranean garage. Values for the design of the shoring system are presented within the body of this report.

INITIAL EXCAVATION

As previously mentioned in the "Temporary Shoring" section of this report, it is anticipated that either cantilevered shoring piles, or shoring piles supplemented by the use of conventional or pressure-grouted tie-back earth anchors, will be used to provide support for the temporary excavations. If cantilevered shoring piles are used, the excavation should proceed in 5-foot increments, allowing for the installation of timber lagging as recommended in this report. If shoring piles supplemented by the use of conventional or pressure-grouted tie-back earth anchors are used, the contractor shall excavate to install the upper row of tie-back earth anchors, following the installation of the soldier piles. The drill bench elevation is normally located approximately 3 feet below the elevation of the row of tie-backs. If the soldier piles consist of structural steel beams set into 24-inch-diameter borings and the borings are spaced no more than 8 feet, center to center, the initial excavation (to the drill bench elevation) may be extended as much as 8 feet below the ground surface.

TIE-BACK CONSTRUCTION

It is anticipated that tie-back earth anchors will be designed as friction anchors. The anchor length is that portion of the tie-back that extends beyond the active wedge. The active wedge is defined as a plane initiating at the bottom of the excavation/shoring beam interface, and extending upwards at 55 degrees from horizontal. It is anticipated that the anchors utilized on this project will be pressure grouted or post-grouted.

Should significant caving of the tieback anchor excavations occur, it would be necessary to modify the construction technique to produce a satisfactory condition. It is anticipated that the soils beneath the site will consist of materials with varying cohesive strengths.

Observation of the drilling of the tie-back anchor shafts should be provided by the representative of the Geotechnical Engineer of Record. Should unusually soft soils be encountered during the drilling to the design anchor length, consideration should be given to increasing the length, diameter, or other means of increasing the load capacity of the anchor.

INITIAL ANCHOR LOAD TESTING

It is recommended that initial anchors at each tie-back level be constructed and tested as early as is reasonably possible, and prior to installing other anchors. The intent would be to determine the actual friction capacity that can be developed at representative anchor locations.

CONCRETE PLACEMENT

Each anchor will be installed and the bond length of the anchor will be cast with concrete. The free anchor length (between the shoring piles and the active wedge) is not to be filled with concrete until the anchor has been tested, locked off, and accepted. Sand or fly-ash may be cast within the free anchor length to help maintain the excavation.

LAGGING

It is anticipated that the overall depth of the excavation will extend approximately 22 feet below the present grade and it is expected that construction would extend over a significant period of time. It is concluded that lagging will be required for most of the excavation. Where center to center spacing is 8 feet or less (i.e., where a free span of 6 feet or less applies) it is expected that lagging could be designed on the basis of a uniform 400 pounds per square foot between shoring piles.

Lagging may consist of treated timber. Timber lagging may swell when wetted and create a seal, resulting in a possible build-up of hydrostatic pressure outside the lagging. To alleviate such a condition, we suggest that a half-inch gap be left between lagging planks for each vertical two feet of lagging.

SURFACE LOADS

Conditions of known or anticipated surcharge loads are addressed in the shoring analysis; the shoring pressure recommended in the text includes the surcharge effects of streets. Should other surcharge loading be considered, such as crane loads or areas designated for significant construction equipment, the Geotechnical and Shoring Engineers should be notified in order to provide modifications to the shoring design.

ANCHOR TESTING

The design values for shoring are based on a factor of safety of at least 1.50. In order to obtain a factor of safety of 2 or greater for each of the anchors, it will be required to load-test each anchor to at least one and one-half times the design anchor load.

It will also be required to perform long term anchor load tests at each excavation face. If only one row of anchors is required, only one long-term (24-hour) test would be required at each face (north, south, east and west).

Each production anchor should be monitored by the representative of the Geotechnical Engineer of Record during the application of the test load. Loads are usually applied in increments of 50 percent, 100 percent, 125 percent, and 150 percent of design load. Once the full 150 percent load is applied, the test load would be maintained and the deflection of the anchor would be monitored. After the 150 percent load is applied, it will be required to obtain no more than 1/10 of an inch of deflection for a 15-minute period. The total deflection of the anchor should be less than 12 inches, although larger deflections may be accepted provided both the Shoring Engineer and the Geotechnical Engineer of Record approve each such anchor.

It will also be required to perform long-term anchor load tests. One long-term (24-hour) 200 percent test would be required for each level of tie-back anchors. It is the responsibility of the shoring contractor to install an anchor of sufficient size or capacity to withstand the 200 percent test load. 200 percent load tests often necessitate the installation of "straps" to reduce deformation or twisting of the shoring beam.

The long-term anchor load test is to be applied for 24 hours. The total anchor deflection should not exceed 12 inches during the test or more than 0.75 inch after application of the 200 percent test load. Should the deflection under 200 percent of the design load be less than 0.5 inch for a period of 12 hours, and less than 1/10 of an inch over the preceding 4-hour period, the test may be terminated.

A "quick," 30-minute, 200 percent test should be performed on 10 percent of the anchors. The test load should be maintained for 30 minutes, and the anchor deflection during that period should not exceed 0.25 inch in order for the anchor to be considered acceptable.

FAILED ANCHORS

Those anchors that do not meet the test requirements indicated in the text of this report are considered to be failed anchors. A stable load for each anchor, equal to two-thirds of the successful test load, may be assumed. It will be required to provide additional resistance equal to the difference between the design anchor load and the stable load.

It is generally expected that failed anchors would be locked off at two-thirds of that load which produced no more than 1/10 of an inch deflection during the 15-minute test period; i.e., normal procedures are to lock off at the stable load. Since the Shoring Contractor may be required to extend the excavation below the drill bench elevation to construct an additional replacement anchor, it may be advisable to lock off the failed anchor at some value between the stable load and the maximum test load attained by the anchor. The Geotechnical Engineer of Record and the Shoring Engineer should provide specific recommendations for the lock-off loads for each failed anchor.

LOCK-OFF LOADS

After each anchor has been tested and approved by the representative of the Geotechnical Engineer of Record, it is to be locked off at the design load. Verification of the lock-off load is to be obtained by "lift-off" or other acceptable means; the lock-off load should be no more than 10 percent above, or 5 percent below, the design load.

CONTINUED EXCAVATIONS

In no case are excavations (at any face) to be extended below the drill bench elevation until such time as the Geotechnical Engineer of Record has accepted each of the anchors at that elevation and has so notified the contractor that excavation may proceed. The Geotechnical Engineer of Record may permit localized excavations to be extended below the drill bench elevation where it would be required for construction of replacement anchors.

RAKER SUPPORT SYSTEM

Rakers may be used where permission is not obtained to extend earth anchors below adjacent property. The rakers will transfer shoring loads to the excavated subgrade by use of footings (deadmen); the footings may consist of isolated pads or continuous footings. At some locations, constructed portions of the spread foundation system for the proposed development may also be used to support rakers.

Footings for the rakers should be seated in bearing soils. Some deflection of the footings will, of course, occur as a function of consolidation of the supporting soil. It is suggested that the Shoring Contactor consider the possibility that shimming, or other means of extending the rakers, would be required where settlement of the raker footings results in unacceptable deflections of the support system.

MONITORING

It will be the responsibility of others to maintain an accurate monitoring record of the performance of the excavation. The intent of this program will be to produce an ongoing log of the horizontal and vertical deflections of the individual soldier piles.

It is anticipated that a licensed surveyor would be retained to establish and maintain the monitoring system. Both vertical and horizontal movements should be maintained on a weekly basis and the record of performance should be promptly submitted to both the Geotechnical Engineer of Record and the Shoring Engineer. Accuracy should be maintained within one one-hundredth of a foot and the record should be produced in a readily understood form. An acceptable form would be that which would list, in tabular form, the net and total deflections (both horizontal and vertical) of each soldier pile. The surveyor should submit to the Geotechnical Engineer of Record, prior to the start of excavation, a plan that would indicate the format used for presentation of deflection data.

It is suggested that an attempt be made to secure monuments, or survey points, for horizontal measurements of the subgrade located approximately 3 or 4 feet back of the shoring elements. The intent would be to determine whether the test load is creating bending and deflection of the soldier pile, or if the pressure of the soil behind the shoring is creating the pile deflection. It is suggested that at least four locations be selected on each side of the excavation, and the performance of such monuments be included within the monitoring data submitted each week.

Monitoring of the excavation performance (points established on each pile) should be started prior to the beginning of the initial excavation. The weekly schedule of performance monitoring may be modified as the job progresses. Once the subterranean garage has been constructed, monitoring of the performance will no longer be required.