Appendix G

Geotechnical Investigations

Appendix G.1

Residential Option Geotechnical Report

GEOCON
WEST, INC.

GEOTECHNICAL
ENVIRONMENTAL
MATERIALS

GEOTECHNICAL INVESTIGATION

PROPOSED HIGH-RISE REDEVELOPMENT
6254-6274 W. DE LONGPRE AVENUE
1334 & 1348-1360 N. VINE STREET
6241-6265 W. AFTON PLACE
LOS ANGELES, CALIFORNIA
TRACT: 1210, BLOCK A, LOT: 11-23

PREPARED FOR

ONNI CAPITAL, LLC
VANCOUVER, BRITISH COLUMBIA

PROJECT NO. A9382-06-01

REVISED SEPTEMBER 2016

GEOCON
WEST, INC.
GEOTECHNICAL ■ ENVIRONMENTAL ■ MATERIALS

Project No. A9382-06-01
March 15, 2016
Revised September 21, 2016

Mr. Daniel Bell
Onni Group, LLC
300 - 550 Robson Street
Vancouver, British Columbia   V6B 2B7

Subject: GEOTECHNICAL INVESTIGATION
PROPOSED HIGH-RISE REDEVELOPMENT
6254-6274 W. DE LONGPRE AVENUE, 1334 & 1348-1360 N. VINE STREET
6241 -6265 W. AFTON PLACE, LOS ANGELES, CALIFORNIA
TRACT 1210, BLOCK A, LOTS 11-23

Dear Mr. Bell:

In accordance with your authorization of our proposal dated February 3, 2016, we have performed a geotechnical investigation for the proposed high-rise development located at the southeast corner of De Longpre Avenue and Vine Street in the Hollywood area of Los Angeles, California. The accompanying report presents the findings of our study, and our conclusions and recommendations pertaining to the geotechnical aspects of proposed design and construction. Based on the results of our investigation, it is our opinion that the site can be developed as proposed, provided the recommendations of this report are followed and implemented during design and construction.

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

Very truly yours,

GEOCON WEST, INC.

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GEOTECHNICAL INVESTIGATION

1. PURPOSE AND SCOPE

This report presents the results of a geotechnical investigation for the proposed multi-family residential development located at the corner of De Longpre Avenue and Vine Street in the Hollywood area of Los Angeles, California (see Vicinity Map, Figure 1). The purpose of the investigation was to evaluate subsurface soil and geologic conditions underlying the site and, based on conditions encountered, to provide conclusions and recommendations pertaining to the geotechnical aspects of design and construction.

The scope of this investigation included a site reconnaissance, a review of documents on file with LADBS, field exploration, laboratory testing, engineering analysis, and the preparation of this report. The site was explored on February 25, 2016 and February 26, 2016, by excavating two 8-inch diameter borings to depths of approximately 101½ feet below the existing ground surface utilizing a truck-mounted hollow-stem auger drilling machine. The approximate locations of the exploratory borings are depicted on the Site Plan (see Figure 2). A detailed discussion of the field investigation, including boring logs, is presented in Appendix A.

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

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

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

2. SITE AND PROJECT DESCRIPTION

The subject site is located at the southeast corner of De Longpre Avenue and Vine Street in the City of Los Angeles, California. The site includes the following addresses: 6254-6274 W. De Longpre Avenue, 1334 & 1348-1360 N. Vine Street, and 6241-6265 W. Afton Place, Los Angeles, California. The site is an approximately rectangular-shaped parcel and is currently occupied by several one-story single-family residential lots, a two-story multi-family residential structure, and one- to two story commercial structures. The site is bounded by Vine Street to the west, De Longpre Avenue to the north, Afton Place to the south, and by multi-family residential structures to the east. The site is relatively level with no pronounced highs or lows. Surface water drainage at the site appears to be by sheet flow along the existing ground contours to the city streets. Vegetation onsite consists of grass and trees, which are located in isolated planter areas.

Based on the information provided by the Client, it is our understanding that the proposed development will consist of a 20-story tower underlain by four levels of subterranean parking. The tower will occupy only the western portion of the site; the eastern portion of the site will have low-rise structures. The proposed construction is depicted on the Site Plan and Cross-Section (see Figures 2, 3A, and 3B).

Anticipated column loads were provided by the project structural engineer. It is anticipated that column loads will range from 1,350 kips for the low-rise portion of the structure to 3,700 kips for the proposed high-rise tower.

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

3. BACKGROUND REVIEW

As a part of this investigation, we performed research at the City of Los Angeles Records Department to review any prior geotechnical studies for the subject site and vicinity. Our search did not find any prior reports for the subject site or adjacent sites. However as a result of our research, we did review the following prior report on file for a site located approximately a quarter mile to the northeast:

> *Response to Soils Report Correction Letter, Proposed Mixed-Use Development, 6121-6125 Sunset Boulevard, 1500-1550 N. El Centro Avenue, and 1525-1575 N. Gower Street, Hollywood, California,* dated August 14, 2013.

The response letter references additional reports and addenda for the proposed project which is described as a mixed-use development comprised of a 20-story tower and multiple mid-rise office buildings, underlain by five levels of subterranean parking. Although all reports and addenda associated with this other project were not reviewed at this time, the referenced response letter contains information on a down-hole seismic survey. This information might be used in the future to supplemental a site-specific ground motion hazard analysis for the proposed project. If data from the referenced report is used, a copy of the report will be attached to a future report or addendum letter.

4. GEOLOGIC SETTING

The site is located in the northern portion of the Los Angeles Basin, a coastal plain bounded by the Santa Monica Mountains on the north, the Elysian Hills and Repetto Hills on the northeast, the Puente Hills and Whittier Fault on the east, the Palos Verdes Peninsula and Pacific Ocean on the west and south, and the Santa Ana Mountains and San Joaquin Hills on the southeast. The basin is underlain by a deep structural depression which has been filled by both marine and continental sedimentary deposits underlain by a basement complex of igneous and metamorphic composition (Yerkes, et al., 1965). The basement surface within the central portion of the basin extends to a maximum depth of approximately 32,000 feet below sea level. Regionally, the site is located within the northern portion of the Peninsular Ranges geomorphic province. This geomorphic province is characterized by northwest-trending physiographic and geologic features such as the Newport-Inglewood Fault Zone located approximately 6.0 miles to the southwest. The northern boundary of this province is the active Hollywood Fault, located approximately 0.5 mile to the north.

5. SOIL AND GEOLOGIC CONDITIONS

Based on our field investigation and published geologic maps of the area, the site is underlain by artificial fill and slightly to moderately consolidated Pleistocene age deposits consisting of silt, sand, clay and gravel (Dibblee, 1991; California Geological Survey, 2010). Detailed stratigraphic profiles are provided on the boring logs in Appendix A. The subsurface distribution of the geologic materials and groundwater conditions encountered at the site are shown in Figure 3A.

5.1 Artificial Fill

Artificial fill was encountered in our field explorations to a maximum depth of 13 feet below existing ground surface. The artificial fill varied in composition across the site. In boring B1 (located in the northwestern corner of the site), the fill consists of brown silty sand to sandy silt. In boring B2 (located in the southeastern portion of the site, the fill consists of dark brown clay with trace fine-grained sand. The artificial fill is characterized as slightly moist and loose or very soft to soft. The fill is likely the result of past grading or construction activities at the site. Deeper fill may exist between excavations and in other portions of the site that were not directly explored.

5.2 Older Alluvium

Pleistocene age alluvium was encountered beneath the artificial fill and consists primarily of reddish brown, yellowish brown, and brown interbedded silty sand, clayey sand, sand with various amounts of silt and gravel, silty clay and sandy clay. The older alluvial soils are primarily moist to wet and medium dense to very dense or firm to hard.

6. GROUNDWATER

Review of the Seismic Hazard Zone Report for the Hollywood Quadrangle (California Division of Mines and Geology [CDMG], 1998) indicate the historically highest groundwater level in the area is approximately 45 feet beneath the ground surface. Groundwater information presented in this document is generated from data collected in the early 1900’s to the late 1990s. Based on current groundwater basin management practices, it is unlikely that groundwater levels will ever exceed the historic high levels.

The Los Angeles County Department of Public Works (LACDPW) has maintained various wells in the vicinity of the subject site over the past 50 years. The closest groundwater monitoring well to the site is Well No. 2671A (State No. 1S14W14E01) located approximately 0.6 mile to the south (LACDPW, 2016a). Due to the distance of this well to the site and the known variation of the groundwater levels in the immediate area, the groundwater monitoring data for this well is not considered representative of historic groundwater levels at the site.

Groundwater was encountered in borings B1 and B2 at depths of 48 and 39 feet below the existing ground surface, respectively. These groundwater levels are not static groundwater levels but represent the first water encountered in the borings. The water levels encountered in the borings, particularly in boring B2, likely represent perched water since they are approximately the same elevation or at a higher elevation than the historic high groundwater levels reported by CDMG (1998) for this area. It should be noted that the water encountered in boring B2 was immediately above a less permeable clayey sand bed that strongly suggests this is a perched water condition. Considering the historic high groundwater levels (CDMG, 1998) and the depth to perched water encountered in our borings, groundwater may be encountered during construction. It is not uncommon for groundwater levels to vary seasonally or for groundwater seepage conditions to develop where none previously existed, especially in impermeable fine-grained soils which are heavily irrigated or after seasonal rainfall. In addition, recent requirements for stormwater infiltration could result in shallower seepage conditions in the immediate site vicinity. Proper surface drainage of irrigation and precipitation will be critical for future performance of the project. Recommendations for drainage are provided in the S*urface Drainage* section of this report (see Section 8.26).

7. GEOLOGIC HAZARDS

7.1 Surface Fault Rupture

The numerous faults in Southern California include active, potentially active, and inactive faults. The criteria for these major groups are based on criteria developed by the California Geological Survey (CGS, formerly known as CDMG) for the Alquist-Priolo Earthquake Fault Zone Program (CGS, 2016; Bryant and Hart, 2007). By definition, an active fault is one that has had surface displacement within Holocene time (about the last 11,000 years). A potentially active fault has demonstrated surface displacement during Quaternary time (approximately the last 1.6 million years), but has had no known Holocene movement. Faults that have not moved in the last 1.6 million years are considered inactive.

The site is not within a state-designated Alquist-Priolo Earthquake Fault Zone (Bryant and Hart, 2007) or a city-designated Preliminary Fault Rupture Study Area (City of Los Angeles, 2016) for surface fault rupture hazards. No active or potentially active faults with the potential for surface fault rupture are known to pass directly beneath the site. Therefore, the potential for surface rupture due to faulting occurring beneath the site during the design life of the proposed development is considered low. However, the site is located in the seismically active Southern California region, and could be subjected to moderate to strong ground shaking in the event of an earthquake on one of the many active Southern California faults. The faults in the vicinity of the site are shown in Figure 4, Regional Fault Map.

The closest surface trace of an active fault to the site is the Hollywood Fault located approximately 0.5 mile to the north (Ziony and Jones, 1989). Other nearby active faults include the Raymond Fault, the Newport-Inglewood Fault Zone, the Santa Monica Fault, and the Verdugo Fault located approximately 4.5 miles east, 5.4 miles west, 5.6 miles west, and 6.5 miles northeast of the site, respectively (Ziony and Jones, 1989). The active San Andreas Fault Zone is located approximately 33 miles north of the site.

The closest potentially active fault to the site is the MacArthur Park Fault located approximately 1.1 miles to the southeast (Ziony and Jones, 1989). Other nearby potentially active faults are the Overland Avenue Fault, the Charnock Fault, and the Coyote Pass Fault located approximately 6.9 miles southwest, 7.7 miles southwest, and 7.9 miles southeast of the site, respectively (Ziony and Jones, 1989).

Several buried thrust faults, commonly referred to as blind thrusts, underlie the Los Angeles Basin at depth. These faults are not exposed at the ground surface and are typically identified at depths greater than 3.0 kilometers. The October 1, 1987 M_w 5.9 Whittier Narrows earthquake and the January 17, 1994 M_w 6.7 Northridge earthquake were a result of movement on the Puente Hills Blind Thrust and the Northridge Thrust, respectively. These thrust faults and others in the Los Angeles area are not exposed at the surface and do not present a potential surface fault rupture hazard at the site; however, these deep thrust faults are considered active features capable of generating future earthquakes that could result in moderate to significant ground shaking at the site.

7.2 Seismicity

As with all of Southern California, the site has experienced historic earthquakes from various regional faults. The seismicity of the region surrounding the site was formulated based on research of an electronic database of earthquake data. The epicenters of recorded earthquakes with magnitudes equal to or greater than 5.0 in the site vicinity are depicted on Figure 5, Regional Seismicity Map. A partial list of moderate to major magnitude earthquakes that have occurred in the Southern California area within the last 100 years is included in the following table.

LIST OF HISTORIC EARTHQUAKES

Earthquake (Oldest to Youngest)	Date of Earthquake	Magnitude	Distance to Epicenter (Miles)	Direction to Epicenter
San Jacinto-Hemet area	April 21, 1918	6.8	80	ESE
Near Redlands	July 23, 1923	6.3	62	E
Long Beach	March 10, 1933	6.4	39	SE
Tehachapi	July 21, 1952	7.5	74	NW
San Fernando	February 9, 1971	6.6	22	NNW
Whittier Narrows	October 1, 1987	5.9	14	E
Sierra Madre	June 28, 1991	5.8	22	ENE
Landers	June 28, 1992	7.3	108	E
Big Bear	June 28, 1992	6.4	86	E
Northridge	January 17, 1994	6.7	15	WNW
Hector Mine	October 16, 1999	7.1	122	ENE

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

7.3 Seismic Design Criteria

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

2013 CBC SEISMIC DESIGN PARAMETERS

Parameter	Value	2013 CBC Reference
Site Class	D	Table 1613.3.2
MCE_R Ground Motion Spectral Response Acceleration – Class B (short), S_S	2.336g	Figure 1613.3.1(1)
MCE_R Ground Motion Spectral Response Acceleration – Class B (1 sec), S_1	0.863g	Figure 1613.3.1(2)
Site Coefficient, F_A	1.0	Table 1613.3.3(1)
Site Coefficient, F_V	1.5	Table 1613.3.3(2)
Site Class Modified MCE_R Spectral Response Acceleration (short), S_{MS}	2.336g	Section 1613.3.3 (Eqn 16-37)
Site Class Modified MCE_R Spectral Response Acceleration – (1 sec), S_{M1}	1.295g	Section 1613.3.3 (Eqn 16-38)
5% Damped Design Spectral Response Acceleration (short), S_{DS}	1.557g	Section 1613.3.4 (Eqn 16-39)
5% Damped Design Spectral Response Acceleration (1 sec), S_{D1}	0.863g	Section 1613.3.4 (Eqn 16-40)

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

ASCE 7-10 PEAK GROUND ACCELERATION

Parameter	Value	ASCE 7-10 Reference
Mapped MCE_G Peak Ground Acceleration, PGA	0.901g	Figure 22-7
Site Coefficient, F_{PGA}	1.0	Table 11.8-1
Site Class Modified MCE_G Peak Ground Acceleration, PGA_M	0.901g	Section 11.8.3 (Eqn 11.8-1)

The Maximum Considered Earthquake Ground Motion (MCE) is the level of ground motion that has a 2 percent chance of exceedance in 50 years, with a statistical return period of 2,475 years. According to the 2013 California Building Code and ASCE 7-10, the MCE is to be utilized for the evaluation of liquefaction, lateral spreading, seismic settlements, and it is our understanding that the intent of the Building code is to maintain "Life Safety" during a MCE event. The Design Earthquake Ground Motion (DE) is the level of ground motion that has a 10 percent chance of exceedance in 50 years, with a statistical return period of 475 years.

Deaggregation of the MCE peak ground acceleration was performed using the USGS 2008 Probabilistic Seismic Hazard Analysis (PSHA) Interactive Deaggregation online tool. The result of the deaggregation analysis indicates that the predominant earthquake contributing to the MCE peak ground acceleration is characterized as a 6.68 magnitude event occurring at a hypocentral distance of 5.2 kilometers from the site.

Deaggregation was also performed for the Design Earthquake (DE) peak ground acceleration, and the result of the analysis indicates that the predominant earthquake contributing to the DE peak ground acceleration is characterized as a 6.66 magnitude occurring at a hypocentral distance of 9.6 kilometers from the site.

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

7.4 Site-Specific Ground Motion Hazard Analysis

It is anticipated that a site-specific ground motion hazard analysis will be necessary in order to satisfy the requirements of the City of Los Angeles Building Code and the Los Angeles Tall Buildings Structural Design Council. The analysis will generate a site-specific target response spectrum which will be used to match earthquake time history records for the structural engineer's use in analyzing the seismic response of the structure. It is recommended that the site-specific ground motion hazard analysis be performed once the structural engineer is able to provide input relating to the ground motion study.

7.5 Liquefaction Potential

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

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

The State of California Seismic Hazard Zone Map for the Hollywood Quadrangle (1999) indicates that the site is not located in an area designated as "liquefiable." In addition, a review of the County of Los Angeles Seismic Safety Element (Leighton, 1990) indicates that the site is potentially located within an area identified as having a potential for liquefaction. Due to the relatively dense to stiff older alluvial deposits underlying the site and the depth of the historic high groundwater level in the site vicinity, it is our opinion that the potential for liquefaction and associated ground settlement and lateral spread to affect the site is very low.

7.6 Slope Stability

The topography at the site is relatively level and the topography in the immediate site vicinity slopes gently to the south-southwest. The site is not located within a City of Los Angeles Hillside Grading Area and is not within a Hillside Ordinance Area (City of Los Angeles, 2016). The County of Los Angeles Safety Element (Leighton, 1990), indicates the site is not within an area identified as having a potential for slope instability. Additionally, the site is not within an area identified as having a potential for seismic slope instability (CDMG, 1999). There are no known landslides near the site, nor is the site in the path of any known or potential landslides. Therefore, the potential for slope stability hazards to adversely affect the proposed development is considered low.

7.7 Earthquake-Induced Flooding

Earthquake-induced flooding is inundation caused by failure of dams or other water-retaining structures due to earthquakes. The Los Angeles County Safety Element (Leighton, 1990) indicates that the site is located within the Mulholland Dam inundation area. However, this reservoir, as well as others in California, are continually monitored by various governmental agencies (such as the State of California Division of Safety of Dams and the U.S. Army Corps of Engineers) to guard against the threat of dam failure. Current design, construction practices, and ongoing programs of review, modification, or total reconstruction of existing dams are intended to ensure that all dams are capable of withstanding the maximum considered earthquake (MCE) for the site. Therefore, the potential for inundation at the site as a result of an earthquake-induced dam failure is considered low.

7.8 Tsunamis, Seiches, and Flooding

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

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

The site is within an area of minimal flooding (Zone X) as defined by the Federal Emergency Management Agency (LACDPW, 2016b).

7.9 Oil Fields & Methane Potential

Based on a review of the California Division of Oil, Gas and Geothermal Resources (DOGGR) Oil and Gas Well Location Map W1-5, the site is not located within the limits of an oilfield and oil or gas wells are not located in the immediate site vicinity. However, due to the voluntary nature of record reporting by the oil well drilling companies, wells may be improperly located or not shown on the location map and undocumented wells could be encountered during construction. Any wells encountered during construction will need to be properly abandoned in accordance with the current requirements of the DOGGR.

The site is not located within the boundaries of a city-designated Methane Zone or Methane Buffer Zone (City of Los Angeles, 2016). Also, since the site is not located within the boundaries of a known oil field, the potential for the presence of methane or other volatile gases at the site is considered low. However, should it be determined that a methane study is required for the proposed development it is recommended that a qualified methane consultant be retained to perform the study and provide mitigation measures as necessary.

7.10 Subsidence

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

8. CONCLUSIONS AND RECOMMENDATIONS

8.1 General

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

8.1.2 Up to 13 feet of existing artificial fill was encountered during the site investigation. The existing fill encountered is believed to be the result of past grading and construction activities at the site. Deeper fill may exist in other areas of the site that were not directly explored. The existing fill and site soils are suitable for re-use as engineered fill provided the recommendations in the *Grading* section of this report are followed (see Section 8.4).

8.1.3 Excavation for the subterranean portion of the structure is anticipated to penetrate through the existing artificial fill and expose undisturbed alluvial soils throughout the excavation bottom.

8.1.4 It is anticipated that the proposed tower may be supported on reinforced concrete mat foundations, and the low-rise portion of the project supported on conventional spread foundations. Recommendations for mat foundations and conventional spread foundations are provided herein as Sections 8.8 through 8.10. All foundations should derive support in the competent undisturbed alluvial soils generally found at or below the anticipated foundation depth of 45 feet below the existing ground surface. Foundations should be deepened as necessary to extend into satisfactory soils and must be observed and approved in writing by the Geotechnical Engineer (a representative of Geocon West, Inc.).

8.1.5 Groundwater was encountered at depths of 39 and 48 feet below the ground surface, but are not considered static groundwater, and likely represent perched groundwater conditions. The historic high groundwater level is reported at a depth of 45 feet below the ground surface. Excavation for construction of the proposed subterranean levels is anticipated to extend to depths of approximately 45 feet below the ground surface, including foundation excavations. Based on these considerations, it is anticipated that groundwater may be encountered at or near the bottom of the proposed excavation during construction. Due to the depth of the proposed excavation and the potential for seasonal fluctuation in the groundwater level, temporary dewatering measures may be required to mitigate groundwater during excavation and construction. Recommendations for temporary dewatering are discussed in Section 8.2 of this report.

8.1.6 If the subterranean portion of the structure extends below a depth of 45 feet below the ground surface and is not designed for full hydrostatic pressure, a permanent dewatering system will be required to relieve and mitigate the water pressure. Based on correspondence with the project structural engineer, the proposed structure and foundations are not anticipated to extend below a depth of 45 feet. However, recommendations for permanent dewatering are provided in Section 8.3 of this report should they be necessary.

8.1.7 The alluvial soils anticipated to be exposed at the excavation bottom may be very moist and could be subject to excessive pumping. Operation of rubber tire equipment on the subgrade soils may cause excessive disturbance of the soils. Excavation activities to establish the finished subgrade elevation must be conducted carefully and methodically to avoid excessive disturbance to the subgrade. Stabilization of the excavation bottom may be required in order to provide a firm working surface upon which heavy equipment can operate. Recommendations for bottom stabilization and earthwork are provided in the *Grading* section of this report (see Section 8.6).

8.1.8 Due to the depth of the excavation and the proximity to the property lines, city streets and adjacent offsite structures, excavations will require sloping and/or shoring measures in order to provide a stable excavation. Where shoring is required it is recommended that a soldier pile shoring system be utilized. In addition, where the proposed excavation will be deeper than and adjacent to an offsite structure, the proposed shoring should be designed to resist the surcharge imposed by the adjacent offsite structure. Recommendations for *Temporary Excavations* are provided in Section 8.19 of this report.

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

8.1.10 Any changes in the design, location or elevation, as outlined in this report, should be reviewed by this office. Once the foundation loading configuration and design elevations for the existing and proposed structures proceeds to a more finalized plan, the recommendations within this report should be reviewed and revised, as necessary. Based on the final foundation loading configurations and building elevations, the potential for settlement should be reevaluated by this office.

8.2 Temporary Dewatering

8.2.1 Groundwater seepage was encountered at depths between 39 and 48 feet below the ground surface during site exploration. Based on the conditions encountered at the time of exploration, groundwater may be encountered during construction activities. The depth to groundwater at the time of construction can be further verified during initial dewatering well or shoring pile installation. If groundwater is present above the depth of the proposed foundation excavation bottom, temporary dewatering will be necessary to maintain a safe working environment during excavation and construction activities.

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

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

8.3 Permanent Dewatering

8.3.1 If the subterranean portion of the structure extends below the historic high groundwater depth (45 feet below the ground surface) and is not designed for full hydrostatic pressure and buoyancy, a permanent dewatering system will be required to relieve and mitigate the water pressure. Based on correspondence with the project structural engineer, the proposed structure and foundations are not anticipated to extend below a depth of 45 feet. However, recommendations for permanent dewatering are provided below should they be necessary.

8.3.2 A subdrainage system consisting of perforated pipe placed in gravel-filled trenches may be installed beneath the subterranean slab-on-grade to intercept and control groundwater. This system can be combined with the perimeter retaining wall drainage system provided backflow valves are installed at the base of the wall drainage system.

8.3.3 A typical permanent sub-slab drainage system would consist of a 12-inch-thick layer of ¾-inch gravel that is placed upon a layer of filter fabric (Miami 500X or equivalent), and vibrated to a dense state. Subdrain pipes leading to sump areas, provided with automatic pumping units, should drain the gravel layer. The drain lines should consist of perforated pipe, placed with perforations down, in trenches that are at least six inches below the gravel layer. The excavation bottom, as well as the trench bottoms should be lined with filter fabric prior to placing and compacting gravel. The trenches should be spaced approximately 40 feet apart at most, within the interior, and should extend along to the perimeter of the building. Subsequent to the installation of the drainage system, the waterproofing system and building slab may then be placed on the densified gravel. A mud- or rat-slab may be placed below and over the waterproofing system for protection during placement of rebar and mat slab construction.

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

8.4 Soil and Excavation Characteristics

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

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

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

8.4.4 Based on depth of the proposed subterranean levels, the proposed structure would not be prone to the effects of expansive soils.

8.5 Minimum Resistivity, pH, and Water-Soluble Sulfate

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

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

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

8.6 Grading

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

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

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

8.6.4 All foundations should derive support in the competent undisturbed alluvial soils generally found at or below the anticipated foundation depth of 45 feet below the existing ground surface. Foundations should be deepened as necessary to extend into satisfactory soils and must be observed and approved in writing by the Geotechnical Engineer (a representative of Geocon West, Inc.).

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

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

8.6.7 Where temporary or permanent dewatering is not required, an alternative method of subgrade stabilization would consist of introducing a thin lift of three to six-inch diameter crushed angular rock into the soft excavation bottom. The use of crushed concrete will also be acceptable. The crushed rock should be spread thinly across the excavation bottom and pressed into the soils by track rolling or wheel rolling with heavy equipment. It is very important that voids between the rock fragments are not created so the rock must be thoroughly pressed or blended into the soils. All subgrade soils must be properly compacted and proof-rolled in the presence of the Geotechnical Engineer (a representative of Geocon West, Inc.).

8.6.8 The City of Los Angeles Department of Building and Safety requires a minimum compactive effort of 95 percent of the laboratory maximum dry density in accordance with ASTM D 1557 (latest edition) where the soils to be utilized in the fill have less than 15 percent finer than 0.005 millimeters. Soils with more than 15 percent finer than 0.005 millimeters may be compacted to 90 percent of the laboratory maximum dry density in accordance with ASTM D 1557 (latest edition). It is anticipated that the soils encountered by this firm would require the minimum 95 percent compaction requirement; however additional laboratory testing can be performed during construction to verify the compaction requirement. All fill and backfill soils should be placed in horizontal loose layers approximately 6 to 8 inches thick, moisture conditioned to optimum moisture content, and properly compacted to the required degree of compaction in accordance with ASTM D 1557 (latest edition).

8.6.9 Prior to construction of exterior slabs, the upper 12 inches of the subgrade should be moisture conditioned to optimum moisture content and properly compacted to at least 95 percent relative compaction, as determined by ASTM Test Method D1557 (latest edition).

8.6.10 Although not anticipated for this project, all imported fill shall be observed, tested, and approved by Geocon West, Inc. prior to bringing soil to the site. Rocks larger than 6 inches in diameter shall not be used in the fill. If necessary, import soils used as structural fill should have an expansion index less than 20 and corrosivity properties that are equally or less detrimental to that of the existing onsite soils (see Figure B6).

8.6.11 Utility trenches should be properly backfilled in accordance with the requirements of the Green Book (latest edition). The pipe should be bedded with clean sands (Sand Equivalent greater than 30) to a depth of at least one foot over the pipe, and the bedding material must be inspected and approved in writing by the Geotechnical Engineer (a representative of Geocon). The use of gravel is not acceptable unless used in conjunction with filter fabric to prevent the gravel from having direct contact with soil. The remainder of the trench backfill may be derived from onsite soil or approved import soil, compacted as necessary, until the required compaction is obtained. The use of minimum 2-sack slurry is also acceptable as backfill (see Section 8.7). Prior to placing any bedding materials or pipes, the excavation bottom must be observed and approved in writing by the Geotechnical Engineer (a representative of Geocon).

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

8.7 Controlled Low Strength Material (CLSM)

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

Standard Requirements

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

5. The excavation bottom shall be accepted by the soil engineer and the City Inspector prior to placing CLSM.

Requirements for CLSM that will be used for support of footings

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

8.8 Foundation Design

8.8.1 It is anticipated that the tower structure will be supported on reinforced concrete mat foundations, and the low-rise portion of the structure will be supported on conventional spread foundations. All foundations should derive support in the competent undisturbed alluvial soils generally found at or below the anticipated foundation depth of 45 feet below the existing ground surface. Foundations should be deepened as necessary to extend into satisfactory soils and must be observed and approved in writing by the Geotechnical Engineer (a representative of Geocon West, Inc.).

8.8.2 No special subgrade presaturation is required prior to placement of concrete. However, the slab and foundation subgrade should be sprinkled as necessary; to maintain a moist condition as would be expected in any concrete placement.

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

A waterproofing consultant should be retained in order to recommend a product or method, which would provide protection to subterranean walls, floor slabs and foundations.

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

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

8.9 Conventional Foundation Design

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

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

8.9.3 The allowable soil bearing pressure above may be increased by 250 psf and 700 psf for each additional foot of foundation width and depth, respectively, up to a maximum allowable soil bearing pressure of 8,000 psf.

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

8.9.5 If depth increases are utilized for the perimeter foundations, this office should be provided a copy of the final construction plans so that the excavation recommendations presented herein could be properly reviewed and revised if necessary.

8.9.6 Continuous footings should be reinforced with four No. 4 steel reinforcing bars, two placed near the top of the footing and two near the bottom. Reinforcement for spread footings should be designed by the project structural engineer.

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

8.10 Mat Foundation Design

8.10.1 It is anticipated that the mat foundation constructed for support of the tower will impart an average pressure of approximately 5,000 psf to 8,000 psf. The recommended maximum allowable bearing value is 8,000 psf. The allowable bearing pressure may be increased by up to one-third for transient loads due to wind or seismic forces.

8.10.2 A vertical modulus of subgrade reaction of 20 pounds per cubic inch (pci) may be used in the design of mat foundations deriving support in competent alluvial soils generally found at or below the anticipated foundation depth of 45 feet below the existing ground surface. This value takes into consideration the estimated mat foundation size, but should be reevaluated once foundation loads and dimensions become available.

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

8.10.4 If a portion of the proposed structure will extend below the historic high groundwater table, that portion should be designed for full hydrostatic pressure. The recommended floor slab uplift pressure to be used in design would be 62.4(H) in units of pounds per square foot, where "H" is the height of the water above the bottom of the mat foundation in feet. If a permanent dewatering system is not implemented then the structure must be designed for hydrostatic pressure based on the historic high groundwater of 45 feet below ground surface.

8.10.5 For seismic design purposes, a coefficient of friction of 0.35 may be utilized between the concrete mat and alluvium without a moisture barrier, and 0.15 for slabs underlain by a moisture barrier.

8.11 Foundation Settlement

8.11.1 The maximum expected static settlement for conventional foundations deriving support in the recommended bearing materials and designed with a maximum bearing pressure of 8,000 psf is estimated to be approximately ¾ inch and occur below the heaviest loaded structural element. Differential settlement is not expected to exceed ½ inch over a distance of 20 feet.

8.11.2 The maximum expected static settlement for a mat foundation deriving support in competent alluvial soils and utilizing a maximum allowable bearing pressure of 8,000 psf is estimated to be approximately 3 inches and occur below the central portion of the mat. The differential settlement between the center and corner of the mat is estimated to be less than 2 inches.

8.11.3 Differential settlement between the mat foundations and conventional foundations is expected to be less than 1 inch.

8.11.4 A majority of the settlement of the foundation system is expected to occur on initial application of loading; however, minor additional settlements are expected within the first 12 months.

8.11.5 Once the design and foundation loading configuration proceeds to a more finalized plan, the recommendations within this report should be reviewed and revised, if necessary. Based on the final foundation loading configuration, the potential for settlement should be reevaluated by this office.

8.12 Lateral Design

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

8.12.2 Passive earth pressure for the sides of foundations and slabs poured against the alluvial soils may be computed as an equivalent fluid having a density of 250 pcf with a maximum earth pressure of 2,500 pcf. When combining passive and friction for lateral resistance, the passive component should be reduced by one-third.

8.13 Concrete Slabs-on-Grade

8.13.1 The project structural engineer may determine and design the necessary slab thickness and reinforcing for this structure. Unless specifically analyzed and designed by the project structural engineer, the slab-on-grade and ramp for the subterranean parking garage should be a minimum of 5 inches concrete reinforced with No. 3 steel reinforcing bars placed 18 inches on center in both horizontal directions and positioned vertically near the slab midpoint. The concrete slab-on-grade may bear directly on competent alluvial soils. Any disturbed soils should be properly compacted for slab support.

8.13.2 Slabs-on-grade at the ground surface that may receive moisture-sensitive floor coverings or may be used to store moisture-sensitive materials should be underlain by a vapor retarder placed directly beneath the slab. The vapor retarder and acceptable permeance should be specified by the project architect or developer based on the type of floor covering that will be installed. The vapor retarder design should be consistent with the guidelines presented in Section 9.3 of the American Concrete Institute's (ACI) Guide for Concrete Slabs that Receive Moisture-Sensitive Flooring Materials (ACI 302.2R-06) and should be installed in general conformance with ASTM E 1643 (latest edition) and the manufacturer's recommendations. A minimum thickness of 15 mils extruded polyolefin plastic is recommended; recycled content or woven materials are not recommended. The material should have a permeance of less than 0.01 perms demonstrated by testing before and after mandatory conditioning. The vapor retarder should be installed in direct contact with the concrete slab with proper perimeter seal. If the Los Angeles Green Building Code requirements apply to this project, the vapor retarder should be underlain by 4 inches of clean aggregate. It is important that the vapor retarder be puncture resistant since it will be in direct contact with angular gravel. As an alternative to the clean aggregate suggested in the Los Angeles Green Building Code, it is our opinion that the concrete slab-on-grade may be underlain by a vapor retarder over 4-inches of clean sand (sand equivalent greater than 30), since the sand will serve a capillary break and will minimize the potential for punctures and damage to the vapor barrier.

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

8.13.4 For seismic design purposes, a coefficient of friction of 0.35 may be utilized between concrete slabs and soil without a moisture barrier and 0.15 for slabs underlain by a vapor retarder or methane barrier.

8.13.5 Exterior slabs, not subject to traffic loads, should be at least 4 inches thick and reinforced with No. 3 steel reinforcing bars placed 18 inches on center in both horizontal directions, positioned near the slab midpoint. Prior to construction of slabs, the upper 12 inches of subgrade should be moisture conditioned to optimum moisture content and properly compacted to at least 95 percent relative compaction, as determined by ASTM Test Method D 1557 (latest edition). Crack control joints should be spaced at intervals not greater than 10 feet and should be constructed using saw-cuts or other methods as soon as practical following concrete placement. Crack control joints should extend a minimum depth of one-fourth the slab thickness. The project structural engineer should design construction joints as necessary.

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

8.14 Retaining Walls Design

8.14.1 The recommendations presented below are generally applicable to the design of rigid concrete or masonry retaining walls having a maximum height of 45 feet. In the event that walls significantly higher than 45 feet are planned, Geocon should be contacted for additional recommendations.

8.14.2 Retaining wall foundations may be designed in accordance with the recommendations provided in the *Foundation Design* sections of this report (see Section 8.8 through 8.10).

8.14.3 Assuming that proper drainage and permanent dewatering is maintained, retaining walls with a level backfill surface that are not restrained at the top should be designed utilizing a triangular distribution of pressure (active pressure) of 49 pcf.

8.14.4 Restrained walls are those that are not allowed to rotate more than 0.001H (where H equals the height of the retaining portion of the wall in feet) at the top of the wall. Where walls are restrained from movement at the top, walls may be designed utilizing a triangular distribution of pressure (at-rest pressure) of 70 pcf. Calculation of the recommended earth pressures is provided as Figure 6.

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

8.14.6 Additional active pressure should be added for a surcharge condition due to sloping ground, vehicular traffic, or adjacent structures. Recommendations for the incorporation of surcharges are provided in section 8.25 of this report. Once the design becomes more finalized, an addendum letter can be prepared revising recommendations and addressing specific surcharge conditions throughout the project, if necessary.

8.14.7 In addition to the recommended earth pressure, the upper ten feet of the subterranean wall adjacent to the street and parking lot should be designed to resist a uniform lateral pressure of 100 pounds per square foot, acting as a result of an assumed 300 psf surcharge behind the walls due to normal street traffic. If the traffic is kept back at least 10 feet from the subterranean walls, the traffic surcharge may be neglected.

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

8.15 Dynamic (Seismic) Lateral Forces

8.15.1 The structural engineer should determine the seismic design category for the project in accordance with Section 1613 of the CBC. If the project possesses a seismic design category of D, E, or F, proposed retaining walls in excess of 6 feet in height should be designed with seismic lateral pressure (Section 1803.5.12 of the 2013 CBC).

8.15.2 A seismic load of 15 pcf should be used for design of walls that support more than 6 feet of backfill in accordance with Section 1803.5.12 of the 2013 CBC. The seismic load is applied as an equivalent fluid pressure along the height of the wall and the calculated loads result in a maximum load exerted at the base of the wall and zero at the top of the wall. This seismic load should be applied in addition to the active earth pressure. The earth pressure is based on half of two thirds of PGA_M calculated from ASCE 7-10 Section 11.8.3.

8.16 Retaining Wall Drainage

8.16.1 Retaining walls should be provided with a drainage system. At the base of the drain system, a subdrain covered with a minimum of 12 inches of gravel should be installed, and a compacted fill blanket or other seal placed at the surface (see Figure 7). The clean bottom and subdrain pipe, behind a retaining wall, should be observed by the Geotechnical Engineer (a representative of Geocon), prior to placement of gravel or compacting backfill.

8.16.2 As an alternative, a plastic drainage composite such as Miradrain or equivalent may be installed in continuous, 4-foot wide columns along the entire back face of the wall, at 8 feet on center. The top of these drainage composite columns should terminate approximately 18 inches below the ground surface, where either hardscape or a minimum of 18 inches of relatively cohesive material should be placed as a cap (see Figure 8). These vertical columns of drainage material would then be connected at the bottom of the wall to a collection panel or a 1-cubic-foot rock pocket drained by a 4-inch subdrain pipe.

8.16.3 Subdrainage pipes at the base of the retaining wall drainage system should outlet to an acceptable location via controlled drainage structures. Drainage should not be allowed to flow uncontrolled over descending slopes.

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

8.17 Elevator Pit Design

8.17.1 The elevator pit slab and retaining wall should be designed by the project structural engineer. Elevator pit walls may be designed in accordance with the recommendations in the *Retaining Wall Design* section of this report (see Section 8.14).

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

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

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

8.18 Elevator Piston

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

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

8.18.3 Casing may be required in the drilled excavation. The contractor should be prepared to use casing and should have it readily available at the commencement of drilling activities. The contractor should be prepared to mitigate the buoyant forces on the casing due to groundwater seepage, if encountered. Continuous observation of the drilling and installation of the elevator piston by the Geotechnical Engineer (a representative of Geocon West, Inc.) is required.

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

8.19 Temporary Excavations

8.19.1 Excavations on the order of 45 feet in height are anticipated for excavation and construction of the proposed subterranean level, foundation system, and dewatering measures. The excavations are expected to expose alluvial soils, which are suitable for vertical excavations up to 5 feet where loose soils or caving sands are not present or where not surcharged by adjacent traffic or structures.

8.19.2 Vertical excavations greater than five feet will require sloping and/or shoring measures in order to provide a stable excavation. Where sufficient space is available, temporary unsurcharged embankments could be sloped back at a uniform 1:1 slope gradient or flatter, up to a maximum of 12 feet in height. A uniform slope does not have a vertical portion. Where space is limited, shoring measures will be required. *Shoring* data is provided in Section 8.20 of this report.

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

8.20 Shoring – Soldier Pile Design and Installation

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

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

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

8.20.4 Drilled cast-in-place soldier piles should be placed no closer than 2 diameters on center. The minimum diameter of the piles is 18 inches. Structural concrete should be used for the soldier piles below the excavation; lean-mix concrete may be employed above that level. As an alternative, lean-mix concrete may be used throughout the pile where the reinforcing consists of a wideflange section. The slurry must be of sufficient strength to impart the lateral bearing pressure developed by the wideflange section to the soil. For design purposes, an allowable passive value for the soils below the bottom plane of excavation may be assumed to be 160 pounds per square foot per (value have been reduced for buoyant forces). Where piles are installed by vibration techniques, the passive pressure may be assumed to mobilize across a width equal to the 2 times the dimension of the beam flange. The allowable passive value may be doubled for isolated piles spaced a minimum of three the pile diameter. To develop the full lateral value, provisions should be implemented to assure firm contact between the soldier piles and the undisturbed soils.

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

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

8.20.7 Casing may be required if caving may occur in the saturated soils. If casing is used, extreme care should be employed so that the pile is not pulled apart as the casing is withdrawn. At no time should the distance between the surface of the concrete and the bottom of the casing be less than five feet. As an alternative, piles may be vibrated into place; however, there is always a risk that excessive vibrations in sandy soils could induce settlements and distress to adjacent offsite improvements. Continuous observation of the drilling and pouring of the piles by the Geotechnical Engineer (a representative of Geocon West, Inc.), is required.

8.20.8 If a vibratory method of solider pile installation is utilized, predrilling may be performed prior to installation of the steel beams. If predrilling is performed, it is recommended that the bore diameter be at least 2 inches smaller than the largest dimension of the pile to prevent excessive loss in the frictional component of the pile capacity. Predrilling should not be conducted below the proposed excavation bottom.

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

8.20.10 The level of vibration that results from the installation of the piles should not exceed a threshold where occupants of nearby structures are disturbed, despite higher vibration tolerances that a building may endure without deformation or damage. The main parameter used for vibration assessment is peak particle velocity in units of inch per second (in/sec). The acceptable range of peak particle velocity should be evaluated based on the age and condition of adjacent structures, as well as the tolerance of human response to vibration.

8.20.11 Based on Table 19 of the *Transportation and Construction Induced Vibration Guidance Manual* (Caltrans 2013), a continuous source of vibrations (ex. vibratory pile driving) which generates a maximum peak particle velocity of 0.5 in/sec is considered tolerable for modern industrial/commercial buildings and new residential structures. The Client should be aware that a lower value may be necessary if older or fragile structures are in the immediate vicinity of the site.

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

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

8.20.14 The frictional resistance between the soldier piles and retained soil may be used to resist the vertical component of the anchor load. The coefficient of friction may be taken as 0.4 based on uniform contact between the steel beam and lean-mix concrete and retained earth. The portion of soldier piles below the plane of excavation may also be employed to resist the downward loads. The downward capacity may be determined using a frictional resistance of 600 psf per foot (value has been reduced for buoyant forces).

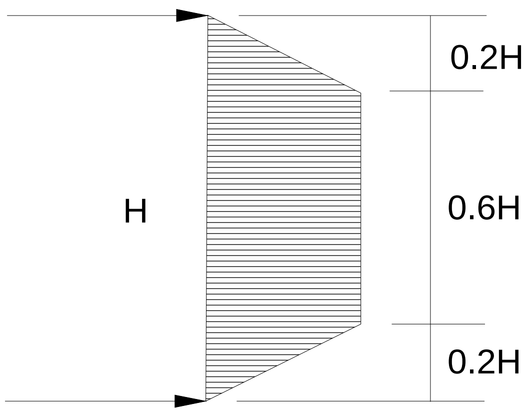
8.20.15 Due to the nature of the site soils, it is expected that continuous lagging between soldier piles will be required. However, it is recommended that the exposed soils be observed by the Geotechnical Engineer (a representative of Geocon West, Inc.), to verify the presence of any competent, cohesive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

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

8.20.17 It is recommended that an equivalent fluid pressure based on the following table, be utilized for design. A diagram depicting the trapezoidal pressure distribution of lateral earth pressure is provided below the table. Calculation of the recommended shoring pressures is provided as Figure 9.

HEIGHT OF SHORING (FEET)	EQUIVALENT FLUID PRESSURE (Pounds Per Cubic Foot) (ACTIVE PRESSURE)	EQUIVALENT FLUID PRESSURE (Pounds Per Square Foot per Foot) Trapezoidal (Where H is the height of the shoring in feet)
Up to 45	39	25H

Trapezoidal Distribution of Pressure



8.20.18 It is very important to note that active pressures can only be achieved when movement in the soil (earth wall) occurs. If movement in the soil is not acceptable, such as adjacent to an existing structure, or the pile is restrained from movement by bracing or a tie back anchor, an at-rest pressure of 61 pcf should be considered for design purposes.

8.20.19 Where a combination of sloped embankment and shoring is utilized, the pressure will be greater and must be determined for each combination. Additional active pressure should be added for a surcharge condition due to slopes, vehicular traffic or adjacent structures and should be designed for each condition. The surcharge pressure should be evaluated in accordance with the recommendations in Section 8.25 of this report.

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

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

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

8.20.23 Due to the depth of the depth of the excavation and proximity to adjacent structures, it is suggested that prior to excavation the existing improvements be inspected to document the present condition. For documentation purposes, photographs should be taken of preconstruction distress conditions and level surveys of adjacent grade and pavement should be considered. During excavation activities, the adjacent structures and pavement should be periodically inspected for signs of distress. In the even that distress or settlement is noted, an investigation should be performed and corrective measures taken sot that continued or worsened distress or settlement is mitigated. Documentation and monitoring of the offsite structures and improvements is not the responsibility of the geotechnical engineer.

8.21 Tie-Back Anchors

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

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

- 10 feet below the top of the excavation – 650 pounds per square foot
- 25 feet below the top of the excavation – 1,000 pounds per square foot
- 40 feet below the top of the excavation – 1,500 pounds per square foot

8.21.3 Depending on the techniques utilized, and the experience of the contractor performing the installation, a maximum allowable friction capacity of 5.0 kips per linear foot for post-grouted anchors (for a minimum 20-foot length beyond the active wedge) may be assumed for design purposes. Only the frictional resistance developed beyond the active wedge should be utilized in resisting lateral loads. Higher capacity assumptions may be acceptable, but must be verified by testing.

8.22 Anchor Installation

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

8.23 Anchor Testing

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

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

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

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

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

8.24 Internal Bracing

8.24.1 Rakers may be utilized to brace the soldier piles in lieu of tieback anchors. The raker bracing could be supported laterally by temporary concrete footings (deadmen) or by the permanent, interior footings. For design of such temporary footings or deadmen, poured with the bearing surface normal to rakers inclined at 45 degrees, a bearing value of 3,500 psf may be used, provided the shallowest point of the footing is at least one foot below the lowest adjacent grade. The structural engineer should review the shoring plans to determine if raker footings conflict with the structural foundation system. The client should be aware that the utilization of rakers could significantly impact the construction schedule do to their intrusion into the construction site and potential interference with equipment.

8.25 Surcharge from Adjacent Structures and Improvements

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

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

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

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

and

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

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

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

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

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

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

and

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

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

then

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

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

8.26 Surface Drainage

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

8.26.2 All site drainage should be collected and controlled in non-erosive drainage devices. Drainage should not be allowed to pond anywhere on the site, and especially not against any foundation or retaining wall. The site should be graded and maintained such that surface drainage is directed away from structures in accordance with 2013 CBC 1804.3 or other applicable standards. In addition, drainage should not be allowed to flow uncontrolled over any descending slope. Discharge from downspouts, roof drains and scuppers are not recommended onto unprotected soils within five feet of the building perimeter. Planters which are located adjacent to foundations should be sealed to prevent moisture intrusion into the soils providing foundation support. Landscape irrigation is not recommended within 5 feet of the building perimeter footings except when enclosed in protected planters.

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

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

8.27 Plan Review

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

LIMITATIONS AND UNIFORMITY OF CONDITIONS

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

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

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

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

LIST OF REFERENCES

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

California Department of Water Resources, 2016, *Groundwater Level Data by Township, Range, and Section,* Web Site Address: http://www.water.ca.gov/waterdatalibrary/groundwater/hydrographs/index_trs.cfm.

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

California Division of Mines and Geology, 1998; *State of California Seismic Hazard Zones, Hollywood Quadrangle,* Official Map, Released: March 25, 1998.

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

California Division of Oil, Gas and Geothermal Resources, 2016. Division of Oil, Gas, and Geothermal Resources Well Finder, http://maps.conservation.ca.gov.doggr/index.html#close. Accessed February, 19, 2016.

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

California Geological Survey, 2016, www.quake.ca.gov/gmaps,WH/regulatory maps.htm.

California Geological Survey, 2010, *Geologic Compilation of Quaternary Surficial Deposits in Southern California, Los Angeles 30' X 60' Quadrangle*, A Project for the Department of Water Resources by the California Geological Survey, Compiled from existing sources by Trinda L. Bedrossian, CEG and Peter D. Roffers, CGS Special Report 217, Plate 9, Scale 1:100,000.

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

FEMA, 2016, Online Flood Hazard Maps, *Flood Insurance Rate Map, Los Angeles County, California and Unincorporated Areas, Map Number 06037C1595F, Date Accessed: August 26, 2016,* http://www.esri.com/hazards/index.html.

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

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

Los Angeles, City of, 2016, NavigateLA website, http://navigatela.lacity.org.

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

LIST OF REFERENCES (CONTD.)

Los Angeles, City of, 1996, *Safety Element of the Los Angeles City General Plan*.

Los Angeles County Department of Public Works, 2016a, Ground Water Wells Website, http://dpw2.co.la.ca.us/website/wells/viewer.asp.

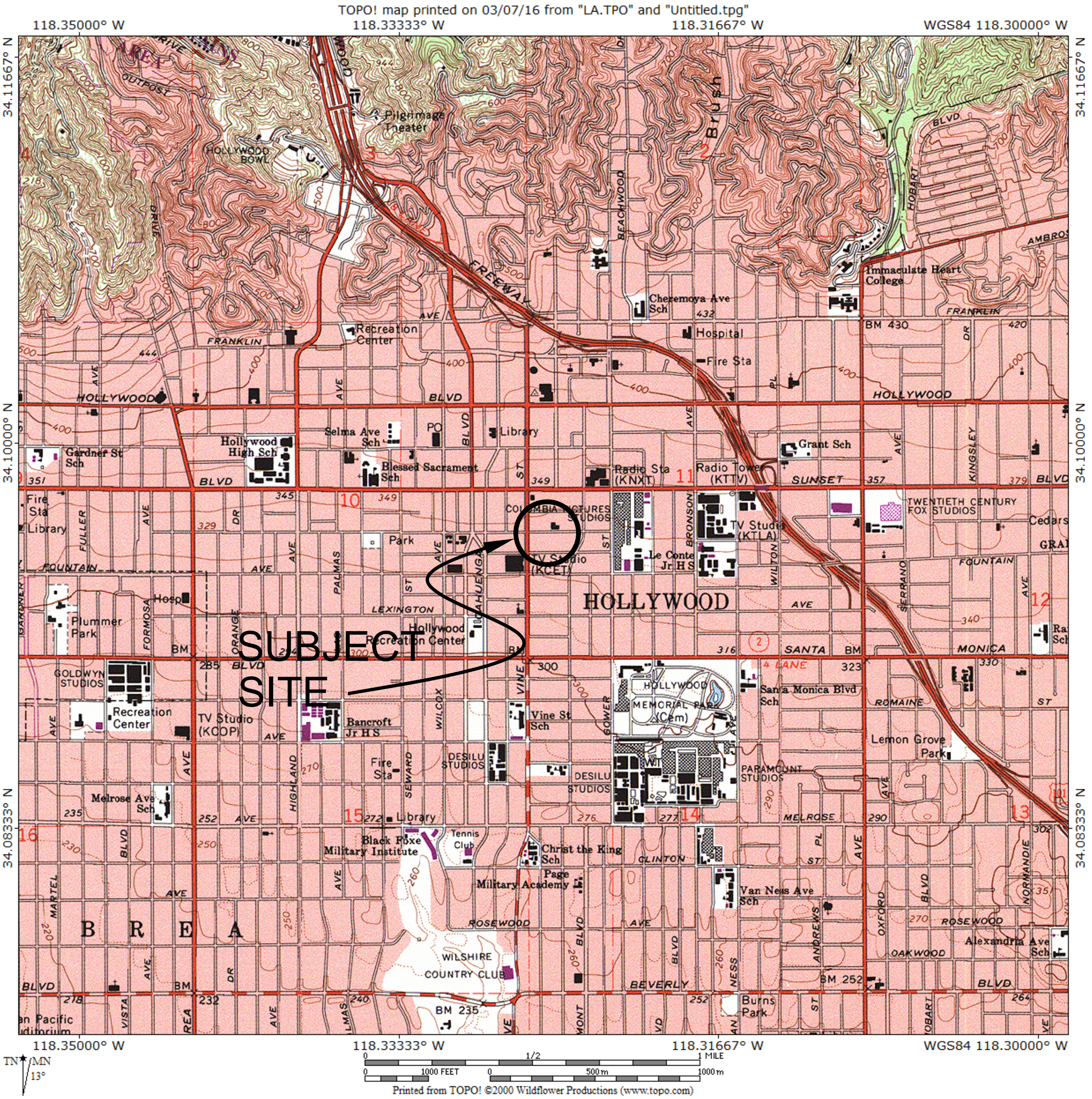
Los Angeles County Department of Public Works, 2016b, Flood Zone Determination Website, http://dpw.lacounty.gov/apps/wmd/floodzone/map.htm.

Toppozada, T., Branum, D., Petersen, M, Hallstrom, C., and Reichle, M., 2000, *Epicenters and Areas Damaged by M> 5 California Earthquakes, 1800 – 1999,* California Geological Survey, Map Sheet 49.

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

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

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



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

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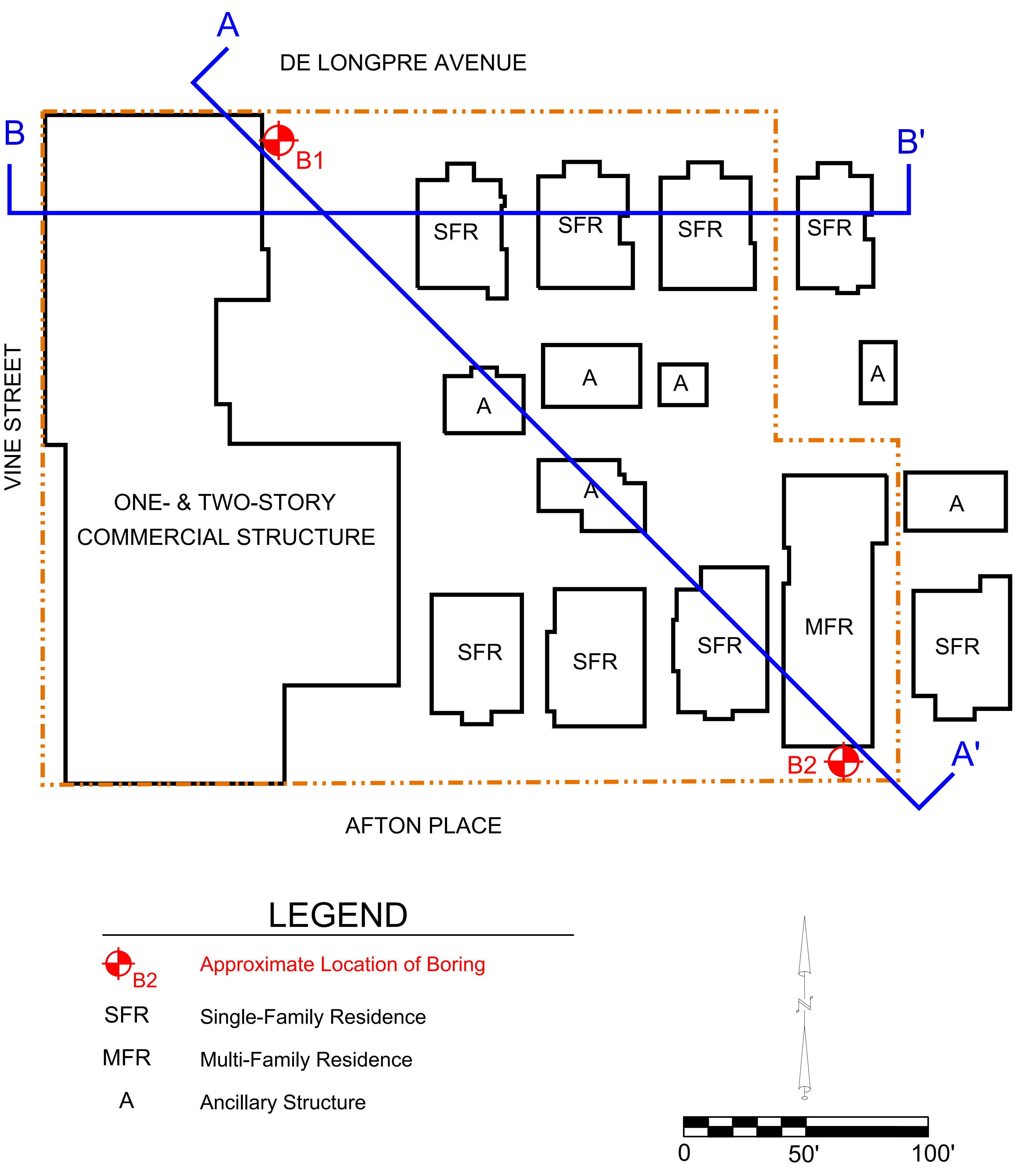
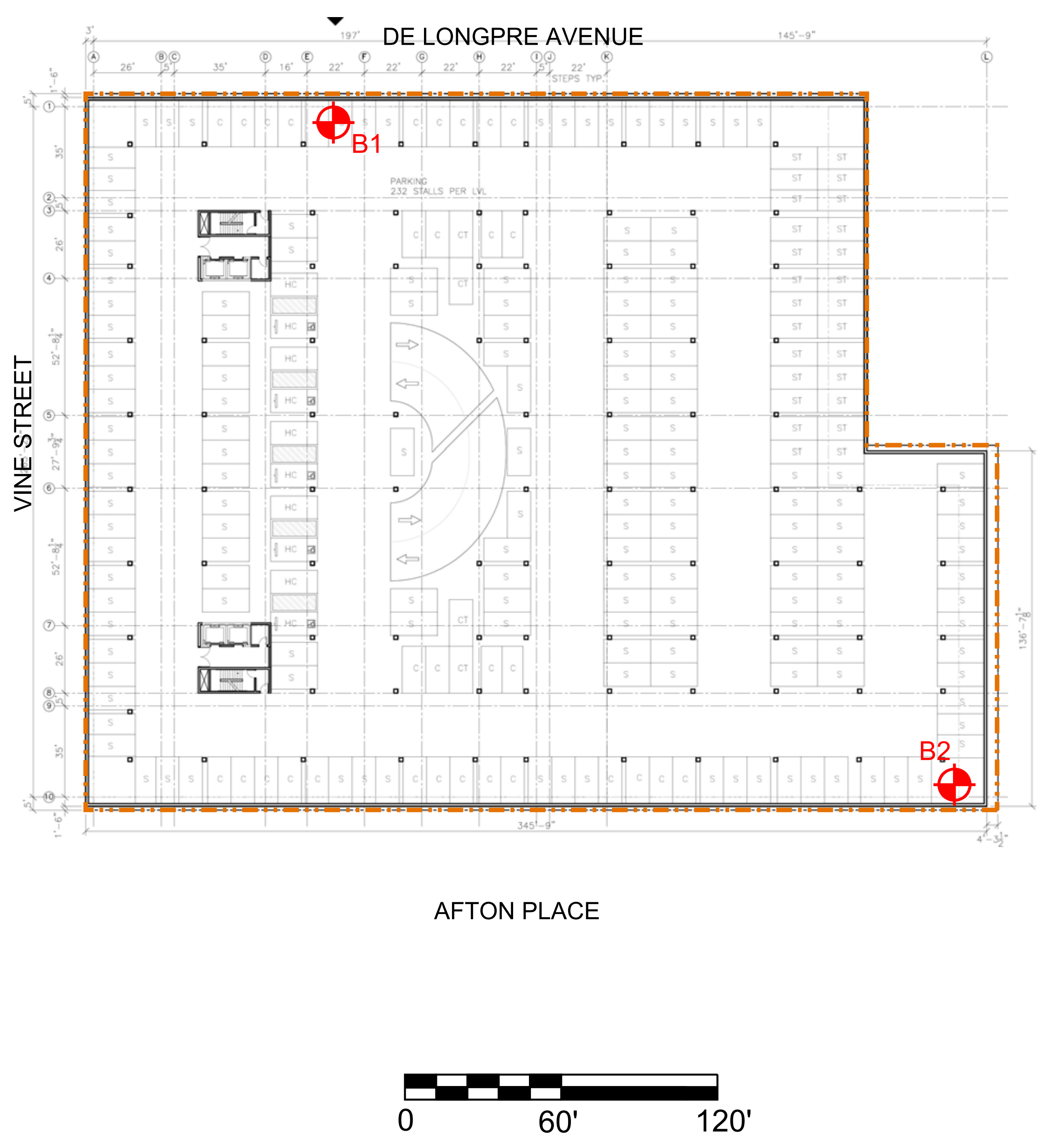
VICINITY MAP

ONNI CAPITAL, LLC
DE LONGPRE AVENUE & VINE STREET
LOS ANGELES, CALIFORNIA

SEPT 2016	PROJECT NO. A9382-06-01	FIG. 1





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SITE PLAN

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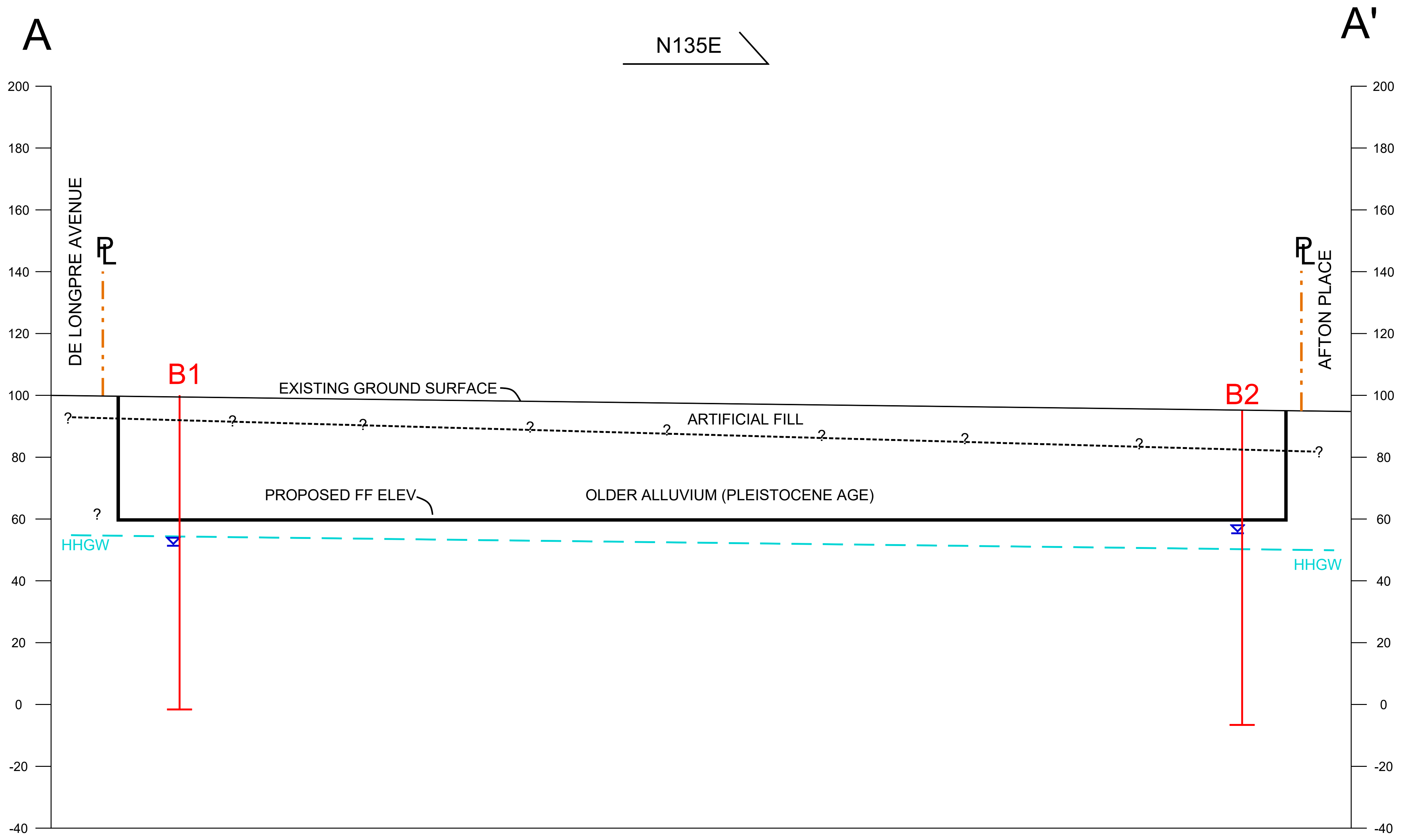
SEPT 2016 | PROJECT NO. A9382-06-01 | FIG. 2



SCALE: 1" = 40' (H&V)

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CROSS-SECTION

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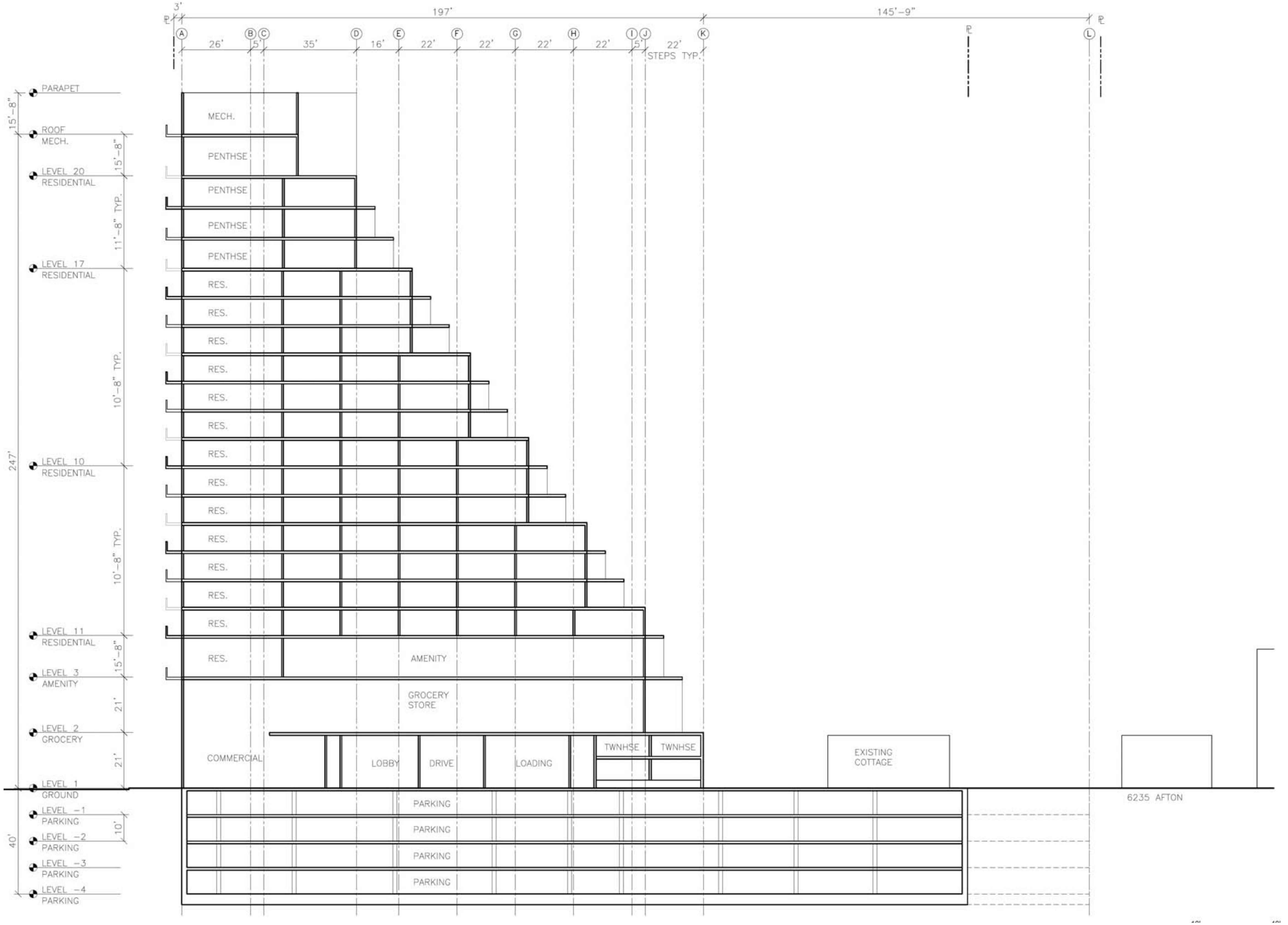
LOS ANGELES, CALIFORNIA

SEPT 2016	PROJECT NO. A9382-06-01	FIG. 3A



Architectual Section B-B'

Plan by: Natoma Architects, Inc.

SCALE: 1" = 40' (H&V)

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CROSS-SECTION

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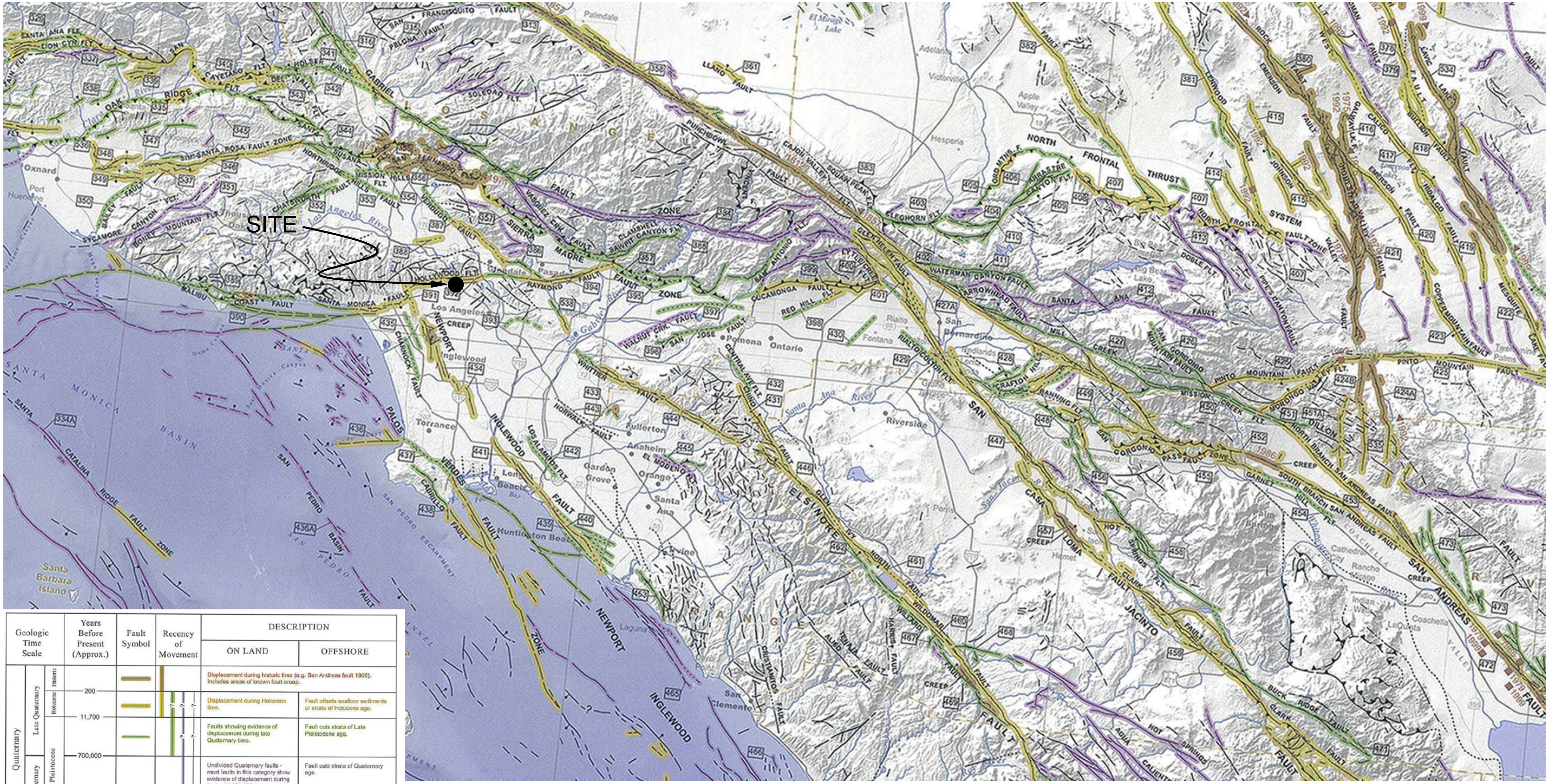
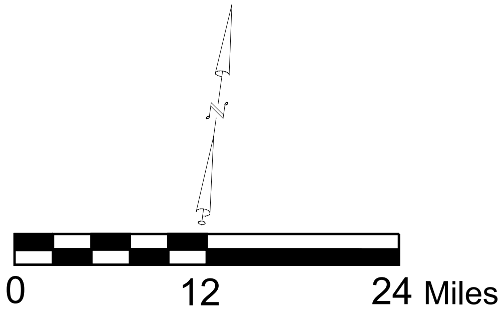
SEPT 2016 | PROJECT NO. A9382-06-01 | FIG. 3B

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



Geologic Time Scale			Years Before Present (Approx.)	Fault Symbol	Recency of Movement	DESCRIPTION: ON LAND	DESCRIPTION: OFFSHORE
Quaternary	Late Quaternary	Historic		—		Displacement during historic time (e.g. San Andreas fault 1906). Includes areas of known fault creep.	
		Holocene	200	—	? ? ?	Displacement during Holocene time.	Fault offsets seafloor sediments or strata of Holocene age.
		Pleistocene	11,700	—	? ?	Faults showing evidence of displacement during late Quaternary time.	Fault cuts strata of Late Pleistocene age.
	Early Quaternary		700,000	—	?	Undivided Quaternary faults - most faults in this category show evidence of displacement during the last 1,600,000 years; possible exceptions are faults which displace rocks of undifferentiated Plio-Pleistocene age.	Fault cuts strata of Quaternary age.
Pre-Quaternary			1,600,000* 4.5 billion (Age of Earth)	—		Faults without recognized Quaternary displacement or showing evidence of no displacement during Quaternary time. Not necessarily inactive.	Fault cuts strata of Pliocene or older age.

* Quaternary now recognized as extending to 2.6 Ma (Walker and Geissman, 2009). Quaternary faults in this map were established using the previous 1.6 Ma criterion.



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REGIONAL FAULT MAP

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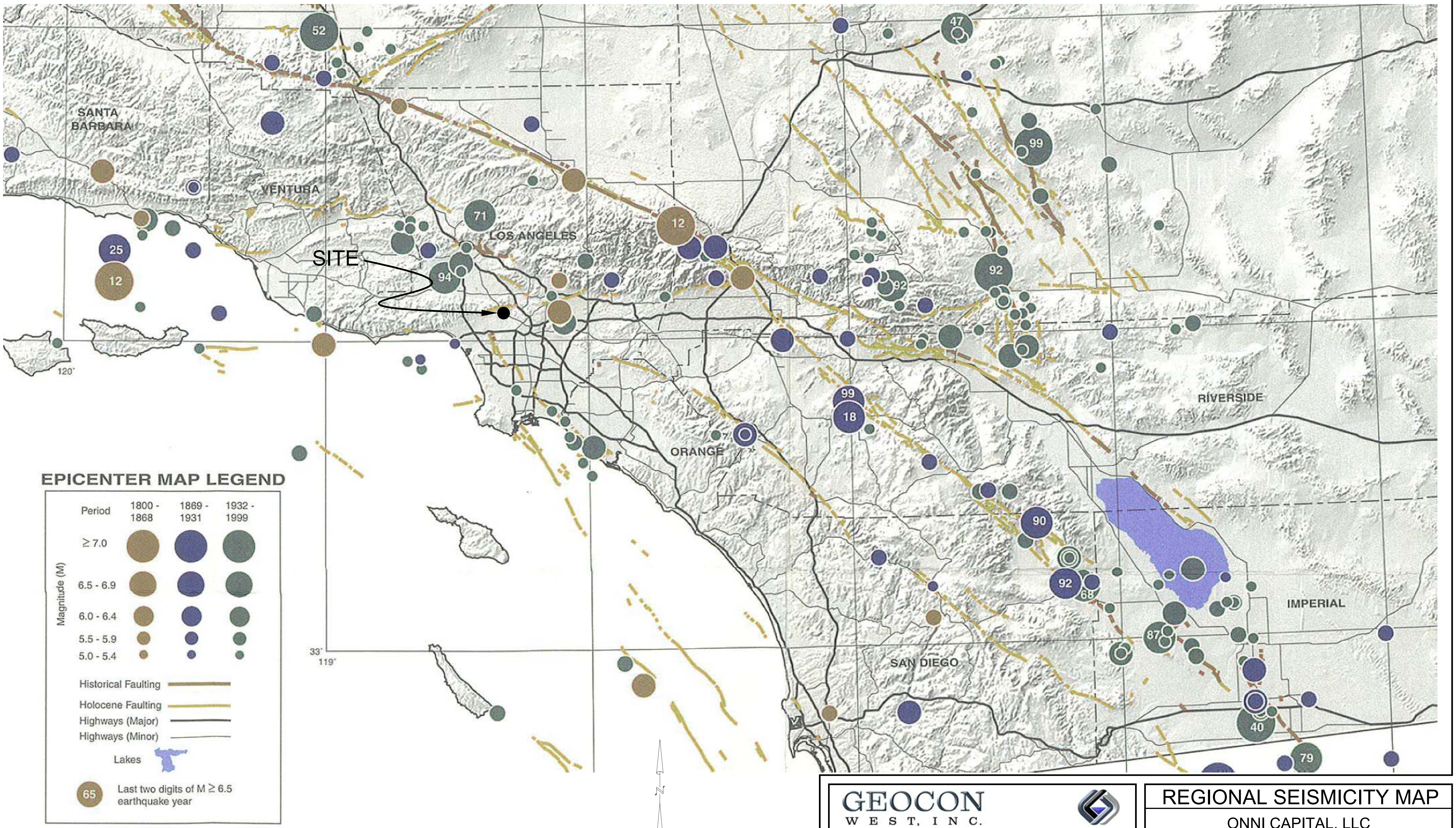
LOS ANGELES, CALIFORNIA

SEPT 2016 | PROJECT NO. A9382-06-01 | FIG. 4



Reference: Toppozada, T., Branum, D., Petersen, M., Hallstrom, C., Cramer, C., and Reichle, M., 2000, Epicenters and Areas Damaged by M≥5 California Earthquakes, 1800 - 1999, California Geological Survey, Map Sheet 49.

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DRAFTED BY: RA | CHECKED BY: SFK

REGIONAL SEISMICITY MAP

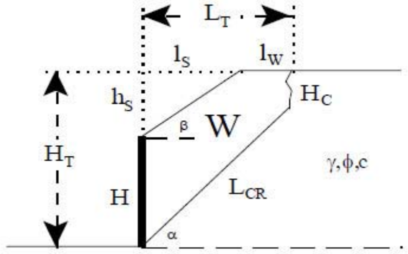
ONNI CAPITAL, LLC
DE LONGPRE AVENUE & VINE STREET
LOS ANGELES, CALIFORNIA

SEPT 2016	PROJECT NO. A9382-06-01	FIG.5

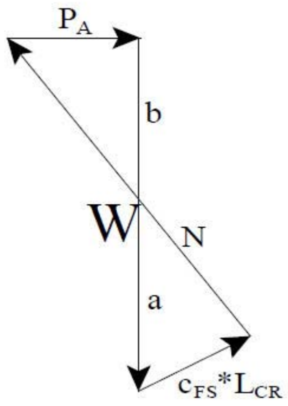
Retaining Wall Design with Transitioned Backfill
(Vector Analysis)

Input:

Retaining Wall Height	(H)	45.00	feet
Slope Angle of Backfill	(β)	0.0	degrees
Height of Slope above Wall	(h_s)	0.0	feet
Horizontal Length of Slope	(l_s)	0.0	feet
Total Height (Wall + Slope)	(H_T)	45.0	feet
Unit Weight of Retained Soils	(γ)	128.0	pcf
Friction Angle of Retained Soils	(φ)	29.0	degrees
Cohesion of Retained Soils	(c)	350.0	psf
Factor of Safety	(FS)	1.50	
Factored Parameters:	(ϕ_{FS})	20.3	degrees
	(c_{FS})	233.3	psf



Failure Angle (α) degrees	Height of Tension Crack (H_C) feet	Area of Wedge (A) feet^2	Weight of Wedge (W) lbs/lineal foot	Length of Failure Plane (L_{CR}) feet	a lbs/lineal foot	b lbs/lineal foot	Active Pressure (P_A) lbs/lineal foot
45	5.8	996	127459.8	55.5	29028.5	98431.2	45312.3
46	5.7	962	123164.7	54.7	27574.1	95590.6	46043.2
47	5.6	930	118998.1	53.9	26240.5	92757.6	46690.2
48	5.5	898	114953.0	53.2	25014.7	89938.3	47256.2
49	5.4	867	111022.8	52.4	23885.2	87137.6	47743.5
50	5.4	838	107201.0	51.7	22842.2	84358.8	48154.0
51	5.3	808	103481.8	51.1	21877.1	81604.7	48489.4
52	5.3	780	99859.2	50.4	20982.1	78877.1	48751.1
53	5.3	753	96328.0	49.8	20150.7	76177.3	48940.2
54	5.2	726	92883.0	49.1	19376.7	73506.2	49057.3
55	5.2	699	89519.2	48.5	18655.0	70864.2	49103.0
56	5.2	674	86232.0	48.0	17980.8	68251.2	49077.4
57	5.3	649	83017.2	47.4	17349.9	65667.4	48980.4
58	5.3	624	79870.6	46.8	16758.4	63112.2	48811.6
59	5.3	600	76788.2	46.3	16203.0	60585.2	48570.4
60	5.4	576	73766.3	45.8	15680.5	58085.8	48255.9
61	5.4	553	70801.5	45.3	15188.2	55613.3	47866.6
62	5.5	530	67890.2	44.8	14723.4	53166.8	47401.1
63	5.6	508	65029.4	44.3	14283.7	50745.7	46857.5
64	5.6	486	62215.9	43.8	13866.9	48348.9	46233.5
65	5.8	464	59446.7	43.3	13471.0	45975.7	45526.5
66	5.9	443	56719.1	42.8	13094.0	43625.1	44733.6
67	6.0	422	54030.3	42.4	12734.0	41296.3	43851.3
68	6.2	401	51377.6	41.9	12389.2	38988.4	42875.8
69	6.3	381	48758.4	41.4	12057.8	36700.6	41802.9
70	6.6	361	46170.1	40.9	11738.1	34432.1	40627.8



Design Equations (Vector Analysis):

$a = c_{FS}*L_{CR}*\sin(90+\phi_{FS})/\sin(\alpha-\phi_{FS})$

$b = W-a$

$P_A = b*\tan(\alpha-\phi_{FS})$

$EFP = 2*P_A/H^2$

Maximum Active Pressure Resultant

$P_{A,\,max}$ 49102.97 lbs/lineal foot

Equivalent Fluid Pressure (per lineal foot of wall)

$EFP = 2*P_A/H^2$

EFP 48.5 pcf 70.0 pcf

Design Wall for an Equivalent Fluid Pressure: 49 pcf 70 pcf

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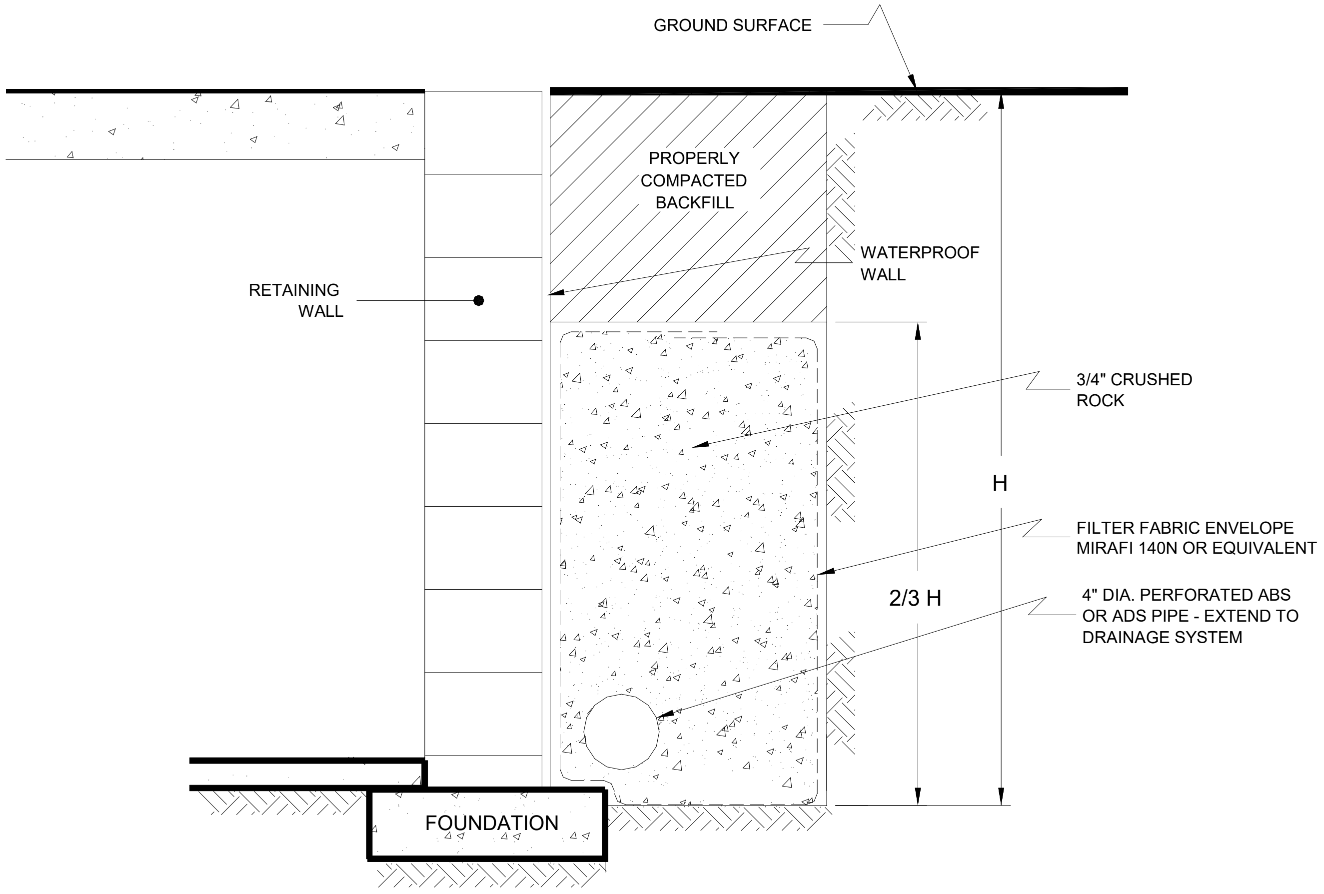
RETAINING WALL PRESSURE CALCULATION

ONNI CAPITAL, LLC
DE LONGPRE AVENUE & VINE STREET
LOS ANGELES, CALIFORNIA

SEPT 2016 PROJECT NO. A9382-06-01 FIG. 6



NO SCALE

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RETAINING WALL DRAIN DETAIL

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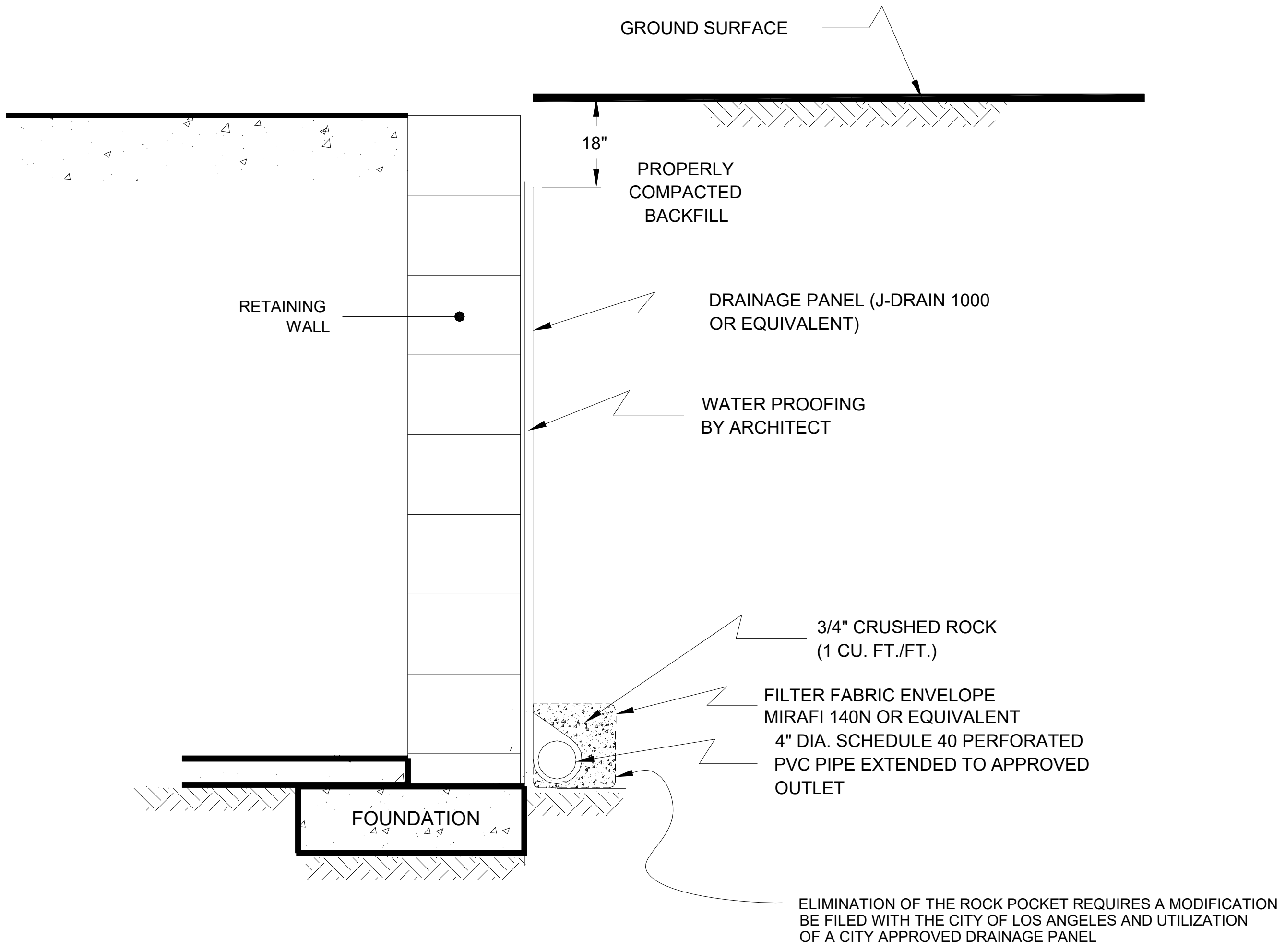
SEPT 2016 | PROJECT NO. A9382-06-01 | FIG. 5



NO SCALE

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RETAINING WALL DRAIN DETAIL

ONNI CAPITAL, LLC

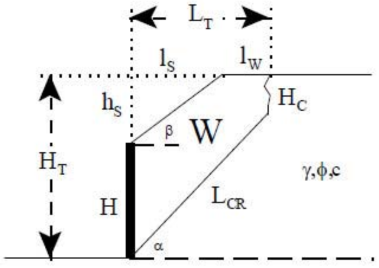
DE LONGPRE AVENUE & VINE STREET
LOS ANGELES, CALIFORNIA

SEPT 2016 | PROJECT NO. A9382-06-01 | FIG. 6

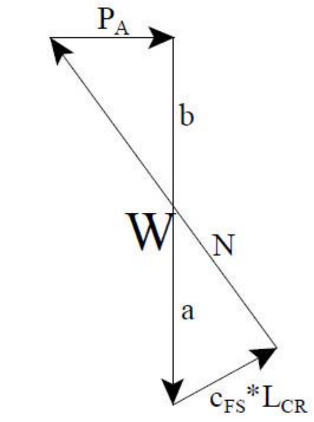
Shoring Design with Transitioned Backfill
(Vector Analysis)

Input:

Shoring Height	(H)	45.00	feet
Slope Angle of Backfill	(β)	0.0	degrees
Height of Slope above Shoring	(h_s)	0.0	feet
Horizontal Length of Slope	(l_s)	0.0	feet
Total Height (Shoring + Slope)	(H_T)	45.0	feet
Unit Weight of Retained Soils	(γ)	128.0	pcf
Friction Angle of Retained Soils	(ϕ)	29.0	degrees
Cohesion of Retained Soils	(c)	350.0	psf
Factor of Safety	(FS)	1.25	
Factored Parameters:	(ϕ_{FS})	23.9	degrees
	(c_{FS})	280.0	psf



Failure Angle (α) degrees	Height of Tension Crack (H_C) feet	Area of Wedge (A) feet^2	Weight of Wedge (W) lbs/lineal foot	Length of Failure Plane (L_{CR}) feet	a lbs/lineal foot	b lbs/lineal foot	Active Pressure (P_A) lbs/lineal foot
45	7.9	982	125645.2	52.5	37369.1	88276.1	34036.8
46	7.7	949	121530.3	51.9	35341.6	86188.7	34971.7
47	7.5	918	117516.6	51.3	33491.7	84024.9	35814.1
48	7.3	888	113602.0	50.7	31799.2	81802.9	36566.9
49	7.2	858	109784.0	50.1	30246.7	79537.3	37233.0
50	7.1	829	106059.1	49.5	28819.2	77240.0	37814.8
51	7.0	800	102423.9	48.9	27503.4	74920.5	38314.4
52	6.9	772	98874.5	48.4	26287.8	72586.7	38733.7
53	6.8	745	95407.3	47.8	25162.4	70244.9	39074.1
54	6.8	719	92018.2	47.2	24118.2	67900.0	39336.9
55	6.8	693	88703.6	46.7	23147.4	65556.2	39523.0
56	6.7	668	85459.8	46.2	22242.9	63216.8	39633.2
57	6.7	643	82283.0	45.6	21398.6	60884.4	39667.7
58	6.7	619	79169.9	45.1	20609.0	58560.9	39626.7
59	6.8	595	76116.9	44.6	19868.9	56248.0	39510.1
60	6.8	571	73120.9	44.1	19174.0	53946.9	39317.5
61	6.8	548	70178.6	43.6	18520.0	51658.5	39048.1
62	6.9	526	67286.9	43.1	17903.4	49383.4	38701.0
63	7.0	503	64442.9	42.7	17320.7	47122.1	38275.0
64	7.1	482	61643.6	42.2	16768.7	44874.9	37768.4
65	7.2	460	58886.4	41.7	16244.5	42641.9	37179.5
66	7.3	439	56168.4	41.2	15745.3	40423.1	36506.2
67	7.5	418	53487.0	40.7	15268.5	38218.5	35745.8
68	7.7	397	50839.5	40.3	14811.4	36028.1	34895.7
69	7.9	377	48223.4	39.8	14371.6	33851.8	33952.7
70	8.1	357	45636.1	39.3	13946.6	31689.5	32913.3



Design Equations (Vector Analysis):

$a = c_{FS}*L_{CR}*\sin(90+\phi_{FS})/\sin(\alpha-\phi_{FS})$

$b = W-a$

$P_A = b*\tan(\alpha-\phi_{FS})$

$EFP = 2*P_A/H^2$

Maximum Active Pressure Resultant

$P_{A,\ max}$ 39667.68 lbs/lineal foot

Equivalent Fluid Pressure (per lineal foot of shoring)

$EFP = 2*P_A/H^2$

EFP 39.2 pcf 61.1 pcf

Design Shoring for an Equivalent Fluid Pressure: 39 pcf 61 pcf

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SHORING PRESSURE CALCULATION

ONNI CAPITAL, LLC

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SEPT 2016 PROJECT NO. A9382-06-01 FIG. 9

APPENDIX A

APPENDIX A

FIELD INVESTIGATION

The site was explored on February 25, 2016 and February 26, 2016, by excavating two 8-inch diameter borings to depths of approximately 101½ feet below the existing ground surface utilizing a truck-mounted hollow-stem auger drilling machine. Representative and relatively undisturbed samples were obtained by driving a 3 inch, O. D., California Modified Sampler into the "undisturbed" soil mass with blows from a 140-pound auto-hammer falling 30 inches (auto-hammer). The California Modified Sampler was equipped with 1-inch high by 2 ⅜-inch diameter brass sampler rings to facilitate soil removal and testing. Bulk samples were also obtained.

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

PROJECT NO. A9382-06-01

BORING 1

ELEV. (MSL.) _______ DATE COMPLETED **2/25/16**

EQUIPMENT **HOLLOW STEM AUGER** BY: **MDS**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0		▒			**AC: 3"** **ARTIFICIAL FILL** Silty Sand to Sandy Silt, loose to very soft, slightly moist, brown, fine-grained.			
2		▒						
4	B1@5' ■	▒				15	101.8	9.5
6		▒						
8		⁞			**OLDER ALLUVIUM** Silty Sand, loose, slightly moist, reddish brown, fine-grained, trace medium-grained.			
10	B1@10' ■	⁞				15	103.5	9.8
12		⁞						
14	B1@15' ■	⁞			- medium dense	19	112.9	12.5
16		⁞						
18		⁞		SM				
20	B1@20' ■	⁞			- decrease in silt content, fine- to coarse-grained sand, trace fine gravel	24	113.6	14.8
22		⁞						
24	B1@25' ■	⁞				21	103.5	6.6
26		⁞						
28		⁞						

A9382-06-01 BORING LOGS.GPJ

Figure A1,
Log of Boring 1, Page 1 of 4

SAMPLE SYMBOLS

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

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

GEOCON

PROJECT NO. A9382-06-01

BORING 1

ELEV. (MSL.) ________ DATE COMPLETED **2/25/16**

EQUIPMENT **HOLLOW STEM AUGER** BY: **MDS**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
30	B1@30'			SM	- some oxidation staining	25	111.5	8.9
32								
34								
	B1@35'				- increase in silt content, no oxidation staining	34	129.5	9.4
36								
38								
				SP-SM	Sand with Silt, medium dense, slightly moist, reddish brown, fine- to coarse-grained, some gravel (to 1"), some oxidation staining, trace calcium carbonate, thin clay films.			
40	B1@40'					38	118.0	8.9
42								
44				SP-SC	Clay with Sand, stiff, slightly moist, brown, fine-grained, low plasticity.			
	B1@45'					39	117.5	16.1
46								
48			▼		- groundwater			
50	B1@50'					41	116.9	15.3
				SM	Silty Sand, dense, moist to wet, brown to yellowish brown, fine- to medium-grained.			
52								
	B1@53'			SP-SM	Sand with Silt, dense, wet, yellowish brown, fine- to medium-grained.	69	125.3	12.0
54								
56	B1@56'				- very dense	50 (5")	--	--
58								
				CL	Sandy Clay, stiff, moist, brown, fine-grained, low plasticity.			
	B1@59'					38	121.6	15.7

Figure A1,
Log of Boring 1, Page 2 of 4

A9382-06-01 BORING LOGS.GPJ

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

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

PROJECT NO. A9382-06-01

BORING 1

ELEV. (MSL.) _______ DATE COMPLETED **2/25/16**

EQUIPMENT **HOLLOW STEM AUGER** BY: **MDS**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
60				CL				
				CL	Silty Clay, stiff, moist, brown, low plasticity, trace fine-grained sand.			
62	B1@62'					40	112.3	13.5
64								
	B1@65'				Clayey/Silty Sand, medium dense, wet, yellowish brown, fine- to coarse-grained.	39	90.6	15.9
66								
68				SM-SC				
70	B1@70'				- very dense	50 (6")	139.2	18.0
72					Sand, poorly graded, medium dense to very dense, wet, yellowish brown, medium-grained.			
74				SP				
	B1@75'					44	114.0	17.8
76					Silty Sand, medium dense, wet, yellowish brown, fine- to medium-grained.			
78								
80	B1@80'				- saturated	43	116.4	14.6
82				SM				
84								
86					- dense, orangish brown with light gray mottles, some oxidation staining			
	B1@87'					54	123.3	15.6
88								

A9382-06-01 BORING LOGS.GPJ

Figure A1,
Log of Boring 1, Page 3 of 4

SAMPLE SYMBOLS

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

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

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PROJECT NO. A9382-06-01

BORING 1

ELEV. (MSL.) _______ DATE COMPLETED **2/25/16**

EQUIPMENT **HOLLOW STEM AUGER** BY: **MDS**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
90								
92								
94	B1@94' ■				- increase in silt content	67	116.0	17.4
96				SM				
98								
100	B1@100' ■				- medium dense, saturated	42	102.0	21.4

Total depth of boring: 101.5 feet
Fill to 8.5 feet.
Groundwater encountered at 48 feet.
Backfilled with soil cuttings and tamped.
Patched with concrete.

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

Figure A1,
Log of Boring 1, Page 4 of 4

A9382-06-01 BORING LOGS.GPJ

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

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

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PROJECT NO. A9382-06-01

BORING 2

ELEV. (MSL.) _______ DATE COMPLETED **2/26/16**

EQUIPMENT **HOLLOW STEM AUGER** BY: **MDS**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0					**ARTIFICIAL FILL** Clay, soft, slightly moist, dark brown, trace fine-grained sand.			
2								
4								
	B2@5'					7	94.6	20.4
6								
				CL	- brown, medium plasticity			
8								
10	B2@10'				- firm	15	101.0	20.5
12								
					OLDER ALLUVIUM Sandy Silt, firm, slightly moist, brown, fine-grained.			
14								
	B2@15'			SP		13	102.3	17.2
16								
18					Sand with Silt, loose, slightly moist, yellowish brown, fine- to medium-grained.			
20	B2@20'			SP		11	99.6	10.3
22								
					Silty Sand, medium dense, moist, brown, fine- to medium-grained, trace coarse-grained sand.			
24								
	B2@25'					22	120.6	12.1
26								
28				SM				

Figure A2,
Log of Boring 2, Page 1 of 4

A9382-06-01 BORING LOGS.GPJ

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

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

PROJECT NO. A9382-06-01

BORING 2

ELEV. (MSL.) _______ DATE COMPLETED **2/26/16**

EQUIPMENT **HOLLOW STEM AUGER** BY: **MDS**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
30	B2@30'	■				26	125.3	12.5
32								
34					Silty Sand with Gravel, medium dense, moist, orangish brown, fine- to medium-grained, fine gravel, some oxidation staining, thin clay films.			
	B2@35'	■				36	125.5	10.9
36				SC				
38			▼					
					- groundwater			
40	B2@40'	■			Clayey Sand, medium dense, wet, brown, fine- to medium-grained.	21	164.7	15.4
42				SM				
44					Silty Sand, medium dense, wet, yellowish brown, fine- to coarse-grained, trace clay.			
	B2@45'	■				40	171.6	13.8
46								
48								
50	B2@50'	■			- dense, some gravel	79	173.8	13.8
52				SM				
54								
	B2@55'	■			- clay, hard, moist, brown, some silt, some fine-grained sand	62	171.4	11.5
56								
58								

Figure A2,
Log of Boring 2, Page 2 of 4

A9382-06-01 BORING LOGS.GPJ

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

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

GEOCON

PROJECT NO. A9382-06-01

BORING 2

ELEV. (MSL.) _______ DATE COMPLETED **2/26/16**

EQUIPMENT **HOLLOW STEM AUGER** BY: **MDS**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
60	B2@60'					38	117.9	14.5
	B2@65'			ML	Silty Sand with Clay and Gravel, medium dense, wet, brown, fine- to coarse-grained.	42	168.8	17.4
70	B2@70'				- decrease in silt and clay content, dense to very dense	50 (6")	171.7	14.0
	B2@75'				- medium dense	41	124.8	13.1
80	B2@80'			CL	Sandy Silt, stiff, moist to wet, orangish brown with light gray mottles, some oxidation staining, fine-grained.	39	118.6	15.4
	B2@85'			ML	Silty Clay, hard, wet, orangish brown, medium plasticity.	51	105.7	26.7

Figure A2,
Log of Boring 2, Page 3 of 4

A9382-06-01 BORING LOGS.GPJ

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

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GEOCON

PROJECT NO. A9382-06-01

BORING 2

ELEV. (MSL.) _______ DATE COMPLETED **2/26/16**

EQUIPMENT **HOLLOW STEM AUGER** BY: **MDS**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
90	B2@90'					53	108.2	22.4
92								
94				ML	Silt with Sand, stiff, orangish brown, moist, fine-grained, oxidation staining.			
96	B2@95'					25	114.8	20.9
98								
100	B2@100'			SP	Sand, poorly graded, dense, wet, yellowish brown, fine- to medium-grained.	71	127.6	8.0

Total depth of boring: 101.5 feet
Fill to 13 feet.
Groundwater encountered at 39 feet.
Backfilled with soil cuttings and tamped.
Grass divot replaced.

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

A9382-06-01 BORING LOGS.GPJ

**Figure A2,
Log of Boring 2, Page 4 of 4**

SAMPLE SYMBOLS

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

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

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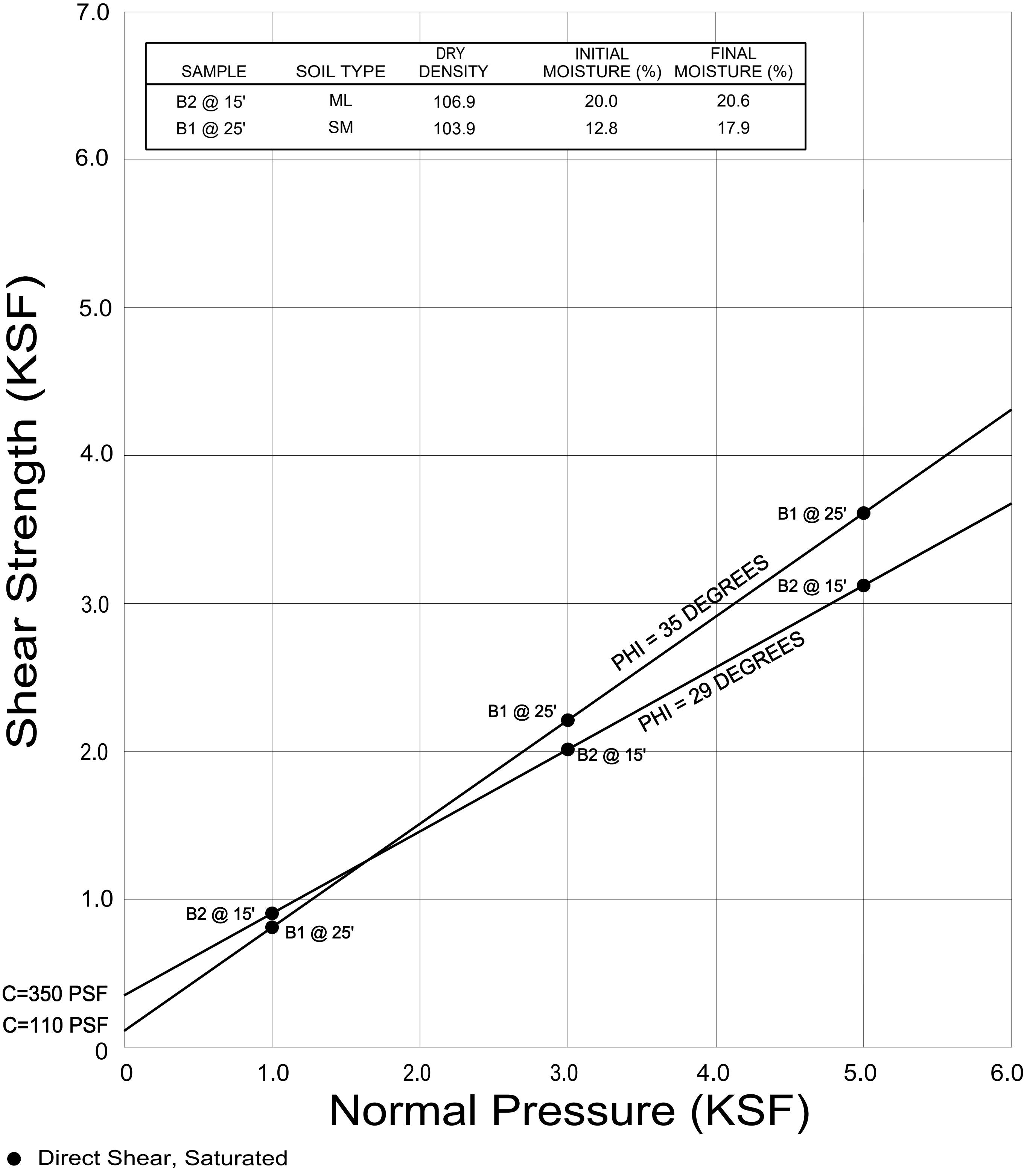
APPENDIX B

APPENDIX B

LABORATORY TESTING

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



SAMPLE	SOIL TYPE	DRY DENSITY	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B2 @ 15'	ML	106.9	20.0	20.6
B1 @ 25'	SM	103.9	12.8	17.9

GEOCON
W E S T, I N C.

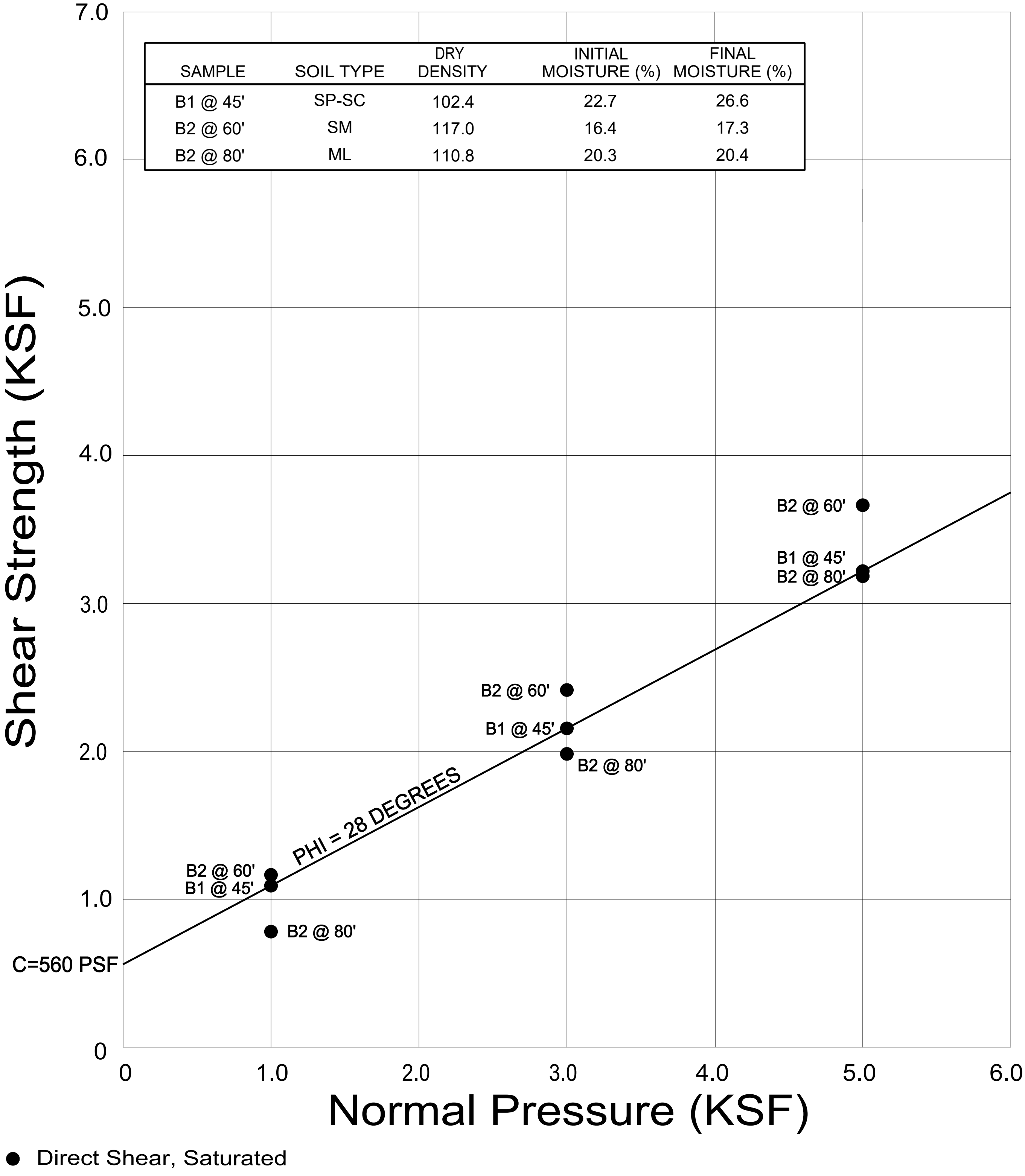
ENVIRONMENTAL GEOTECHNICAL MATERIALS
3303 N. SAN FERNANDO BLVD. - SUITE 100 - BURBANK, CA 91504
PHONE (818) 841-8388 - FAX (818) 841-1704

Drafted by: JMT
Checked by: NDB

DIRECT SHEAR TEST RESULTS

ONNI CAPITAL, LLC
DE LONGPRE AVENUE & VINE STREET
LOS ANGELES, CALIFORNIA

SEPT 2016
PROJECT NO. A9382-06-01
FIG. B1



SAMPLE	SOIL TYPE	DRY DENSITY	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B1 @ 45'	SP-SC	102.4	22.7	26.6
B2 @ 60'	SM	117.0	16.4	17.3
B2 @ 80'	ML	110.8	20.3	20.4

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

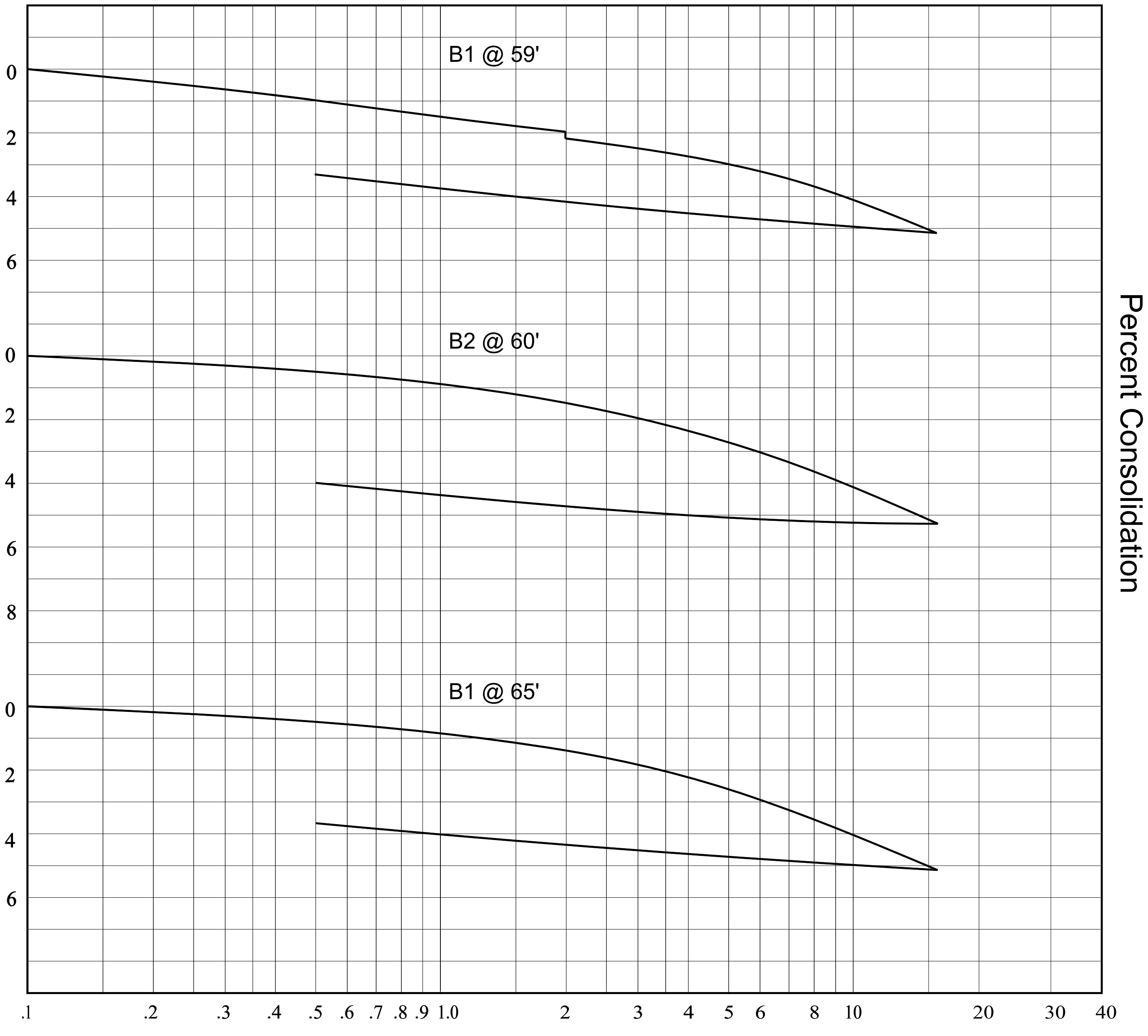
Drafted by: JMT | Checked by: NDB

DIRECT SHEAR TEST RESULTS

ONNI CAPITAL, LLC
DE LONGPRE AVENUE & VINE STREET
LOS ANGELES, CALIFORNIA

SEPT 2016 | PROJECT NO. A9382-06-01 | FIG. B2

WATER ADDED AT 2 KSF



GEOCON
W E S T, I N C.

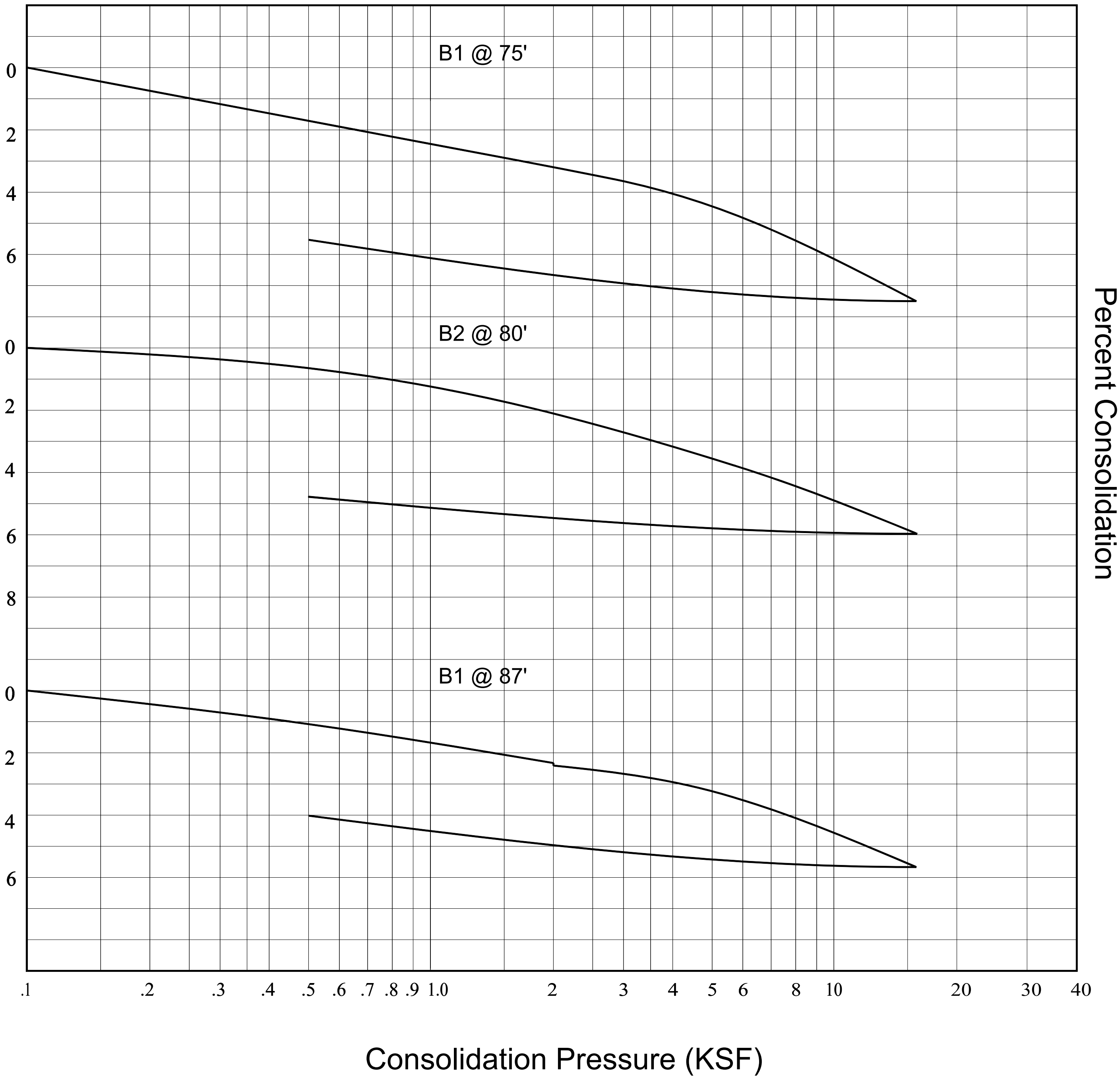
ENVIRONMENTAL GEOTECHNICAL MATERIALS
3303 N. SAN FERNANDO BLVD. - SUITE 100 - BURBANK, CA 91504
PHONE (818) 841-8388 - FAX (818) 841-1704

CONSOLIDATION TEST RESULTS

ONNI CAPITAL, LLC
DE LONGPRE AVENUE & VINE STREET
LOS ANGELES, CALIFORNIA

Drafted By: JMT	Checked By: NDB	SEPT 2016	PROJECT NO. A9382-06-01	FIG. B3

WATER ADDED AT 2 KSF



GEOCON
W E S T, I N C.

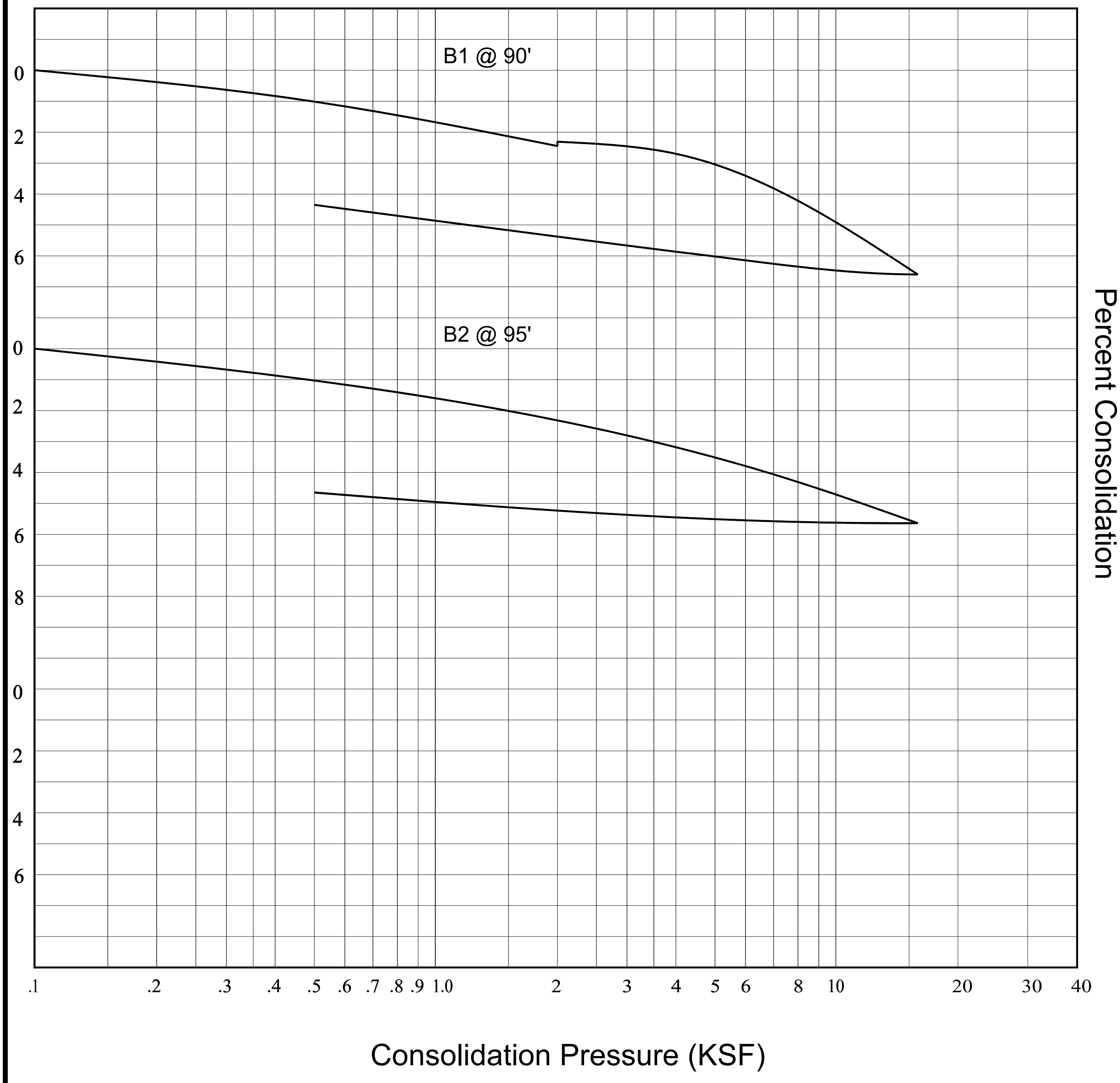
ENVIRONMENTAL GEOTECHNICAL MATERIALS
3303 N. SAN FERNANDO BLVD. - SUITE 100 - BURBANK, CA 91504
PHONE (818) 841-8388 - FAX (818) 841-1704

CONSOLIDATION TEST RESULTS

ONNI CAPITAL, LLC
DE LONGPRE AVENUE & VINE STREET
LOS ANGELES, CALIFORNIA

Drafted By: JMT	Checked By: NDB	SEPT 2016	PROJECT NO. A9382-06-01	FIG. B4

WATER ADDED AT 2 KSF



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

Drafted By: JMT	Checked By: NDB

CONSOLIDATION TEST RESULTS

ONNI CAPITAL, LLC
DE LONGPRE AVENUE & VINE STREET
LOS ANGELES, CALIFORNIA

SEPT 2016	PROJECT NO. A9382-06-01	FIG. B5

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

Sample No.	pH	Resistivity (ohm centimeters)
B1 @ 59'	7.52	1400 (Corrosive)

SUMMARY OF LABORATORY CHLORIDE CONTENT TEST RESULTS
EPA NO. 325.3

Sample No.	Chloride Ion Content (%)
B1 @ 59'	0.012

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

Sample No.	Water Soluble Sulfate (% SO_4)	Sulfate Exposure*
B1 @ 59'	0.009	Negligible

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

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

Drafted by: JMT | Checked by: NDB

CORROSIVITY TEST RESULTS

ONNI CAPITAL, LLC
DE LONGPRE AVENUE & VINE STREET
LOS ANGELES, CALIFORNIA

SEPT 2016 | PROJECT NO. A9382-06-01 | FIG. B6

Appendix G.2

Residential Option Geotechnical Report Approval Letter

BOARD OF
BUILDING AND SAFETY COMMISSIONERS

VAN AMBATIELOS
PRESIDENT

E. FELICIA BRANNON
VICE PRESIDENT

JOSELYN GEAGA-ROSENTHAL
GEORGE HOVAGUIMIAN
JAVIER NUNEZ

CITY OF LOS ANGELES
CALIFORNIA

ERIC GARCETTI
MAYOR

DEPARTMENT OF
BUILDING AND SAFETY
201 NORTH FIGUEROA STREET
LOS ANGELES, CA 90012

FRANK BUSH
GENERAL MANAGER

OSAMA YOUNAN, P.E.
EXECUTIVE OFFICER

SOILS REPORT APPROVAL LETTER

October 18, 2016

LOG # 95056
SOILS/GEOLOGY FILE - 2

Onni Capital
300-550 Robson Street
Vancouver, Canada

TRACT: 1210
BLOCK: A
LOT(S): 11 / 12 /13 /14 / 15 / 16 / 17 / 18 / 19 / 20 / 21 / 22 /23
LOCATION: 6254, 6254 1/2 / 6256-6258 / 6262, 6264 / 6268 / 6272, 6274 W De Longpre Ave / 1348-1360 / 1330, 1334 N Vine St / 6265 / 6261 / 6255 / 6249-6253 1/2 / 6245 / 6241 W Afton Pl

CURRENT REFERENCE REPORT/LETTER(S)	REPORT No.	DATE(S) OF DOCUMENT	PREPARED BY
Soils Report	A9382-06-01	09/21/2016	Geocon West, Inc.

The Grading Division of the Department of Building and Safety has reviewed the referenced report that provides recommendations for the proposed construction of a 20-story multi-family residential development underlain by a 4-level subterranean parking.

The earth materials at the subsurface exploration locations consist of up to 13 feet of uncertified fill underlain by alluvial deposits. The consultants recommend to support the proposed structure on conventional and mat-type foundations bearing on native undisturbed soils.

The referenced report is acceptable, provided the following conditions are complied with during site development:

Note: Numbers in parenthesis ( ) refer to applicable sections of the 2014 City of LA Building Code. P/BC numbers refer the applicable Information Bulletin. Information Bulletins can be accessed on the internet at LADBS.ORG.

1. Provide a notarized letter from all adjoining property owners allowing tie-back anchors on their property. (7006.6)

2. The geologist and soils engineer shall review and approve the detailed plans prior to issuance of any permits. This approval shall be by signature on the plans that clearly indicates the geologist and soils engineer have reviewed the plans prepared by the design

engineer and that the plans include the recommendations contained in their reports. (7006.1)

3. All recommendations of the report that are in addition to or more restrictive than the conditions contained herein shall be incorporated into the plans.

4. A copy of the subject and appropriate referenced reports and this approval letter shall be attached to the District Office and field set of plans. Submit one copy of the above reports to the Building Department Plan Checker prior to issuance of the permit. (7006.1)

5. A grading permit shall be obtained for all structural fill and retaining wall backfill. (106.1.2)

6. All man-made fill shall be compacted to a minimum 90 percent of the maximum dry density of the fill material per the latest version of ASTM D 1557. Where cohesionless soil having less than 15 percent finer than 0.005 millimeters is used for fill, it shall be compacted to a minimum of 95 percent relative compaction based on maximum dry density (D1556). Placement of gravel in lieu of compacted fill is allowed only if complying with Section 91.7011.3 of the Code. (7011.3)

7. Existing uncertified fill shall not be used for support of footings, concrete slabs or new fill. (1809.2, 7011.3)

8. Drainage in conformance with the provisions of the Code shall be maintained during and subsequent to construction. (7013.12)

9. Controlled Low Strength Material, CLSM (slurry) proposed to be used for backfill shall satisfy the requirements specified in P/BC 2014-121.

10. The applicant is advised that the approval of this report does not waive the requirements for excavations contained in the State Construction Safety Orders enforced by the State Division of Industrial Safety. (3301.1)

11. Temporary excavations that remove lateral support to the public way, adjacent property, or adjacent structures shall be supported by shoring or constructed using ABC slot cuts. Note: Lateral support shall be considered to be removed when the excavation extends below a plane projected downward at an angle of 45 degrees from the bottom of a footing of an existing structure, from the edge of the public way or an adjacent property. (3307.3.1)

12. Prior to the issuance of any permit which authorizes an excavation where the excavation is to be of a greater depth than are the walls or foundation of any adjoining building or structure and located closer to the property line than the depth of the excavation, the owner of the subject site shall provide the Department with evidence that the adjacent property owner has been given a 30-day written notice of such intent to make an excavation. (3307.1)

13. The soils engineer shall review and approve the shoring and/or underpinning plans prior to issuance of the permit. (3307.3.2)

14. Prior to the issuance of the permits, the soils engineer and/or the structural designer shall evaluate the surcharge loads used in the report calculations for the design of the retaining walls and shoring. If the surcharge loads used in the calculations do not conform to the

actual surcharge loads, the soil engineer shall submit a supplementary report with revised recommendations to the Department for approval.

15. Unsurcharged temporary excavation may be cut vertical up to 5 feet. Excavations over 5 feet up to a maximum height of 12 feet shall be trimmed back at a uniform gradient not exceeding 1:1 (horizontal to vertical), from top to bottom of excavation, as recommended.

16. Cantilever shoring shall be designed for a minimum EFP of 39 PCF; restrained shoring shall be designed for a trapezoidal distributed lateral earth pressure of 25H PSF; all surcharge loads shall be included into the design, as recommended. Total lateral load on shoring piles shall be determined by multiplying the recommended EFP by the pile spacing.

17. Shoring shall be designed for a maximum lateral deflection of ½ inch where a structure is within a 1:1 plane projected up from the base of the excavation, and for a maximum lateral deflection of 1 inch provided there are no structures within a 1:1 plane projected up from the base of the excavation, as recommended.

18. A shoring monitoring program shall be implemented to the satisfaction of the soils engineer.

19. In the event shoring soldier beams/piles are installed using vibrating/driving equipment in the vicinity of existing structures, the following conditions shall be complied with:

    a. Ground vibrations shall be monitored during shoring installation adjacent to the pile driving operation.

    b. Peak particle velocities (PPV) for any single axis shall be limited to ½ inch/second.

    c. Settlement monitoring monuments shall be surveyed: prior to pile driving, daily during the first week of pile driving operations, and weekly thereafter, until completion of pile installation, as recommended.

    d. In the event any PPV is measured above the specified threshold (½ inch/second) or any settlement is measured/detected, pile driving shall be stopped and corrective actions shall be submitted to the Department for review before resuming pile driving.

20. All foundations shall derive entire support from native undisturbed soils, as recommended and approved by the geologist and soils engineer by inspection.

21. Footings supported on approved compacted fill or expansive soil shall be reinforced with a minimum of four (4) ½-inch diameter (#4) deformed reinforcing bars. Two (2) bars shall be placed near the bottom and two (2) bars placed near the top.

22. The building design shall incorporate provisions for anticipated total and differential settlements of 3 inches and 2 inches, respectively. (1808.2)

23. Special provisions such as flexible or swing joints shall be made for buried utilities and drain lines to allow for differential vertical displacement.

24. Slab on uncertified fill shall be designed as a structural slab. (7011.3)

25. Slabs placed on approved compacted fill shall be at least 5 inches thick and shall be reinforced with ½-inch diameter (#4) reinforcing bars spaced maximum of 16 inches on center each way.

26. The seismic design shall be based on a Site Class D as recommended. All other seismic design parameters shall be reviewed by LADBS building plan check.

27. Seismic design of the proposed building shall be peer-reviewed as required by Section 16.2.5 of the ASCE/SEI 7-10, and the publication "An Alternative Procedure for Seismic Analysis and Design of Tall Buildings Located in the Los Angeles Region", 2014 Edition. Notes: The peer review panel shall be approved by the LADBS Structural Plan Check Division prior to commencement of the review of time history data. This peer review is conducted in conjunction with the structural peer review of the structural framing system. The review and approval of the time histories is performed by the structural review panel approved by LADBS, and not during soils/geology report review process. For more information regarding the structural peer review and the time histories peer review, please contact Colin Kumabe, Assistant Deputy Superintendent of Building, Bureau of Engineering, (213)-482-0447.

28. This letter approves exclusively the option in which the structure is designed to withstand hydrostatic pressures, as a measure to control groundwater under permanent conditions. In the event a permanent dewatering system is planned to be implemented, a supplemental report prepared by a professional licensed by the State of California to perform groundwater studies, shall be submitted for review and approval containing, but not be limited to, justification that the proposed system is feasible and practical, specifics on the proposed dewatering system, and anticipated flow rates to lower groundwater levels to a depth no less than 6 inches below the lowest floor slab. (1805.1.3)

29. Traffic surcharge loads on the retaining walls and shoring shall be provided in accordance with Information Bulletin P/BC 2014-141.

30. Cantilever retaining walls with a level backfill shall be designed for a minimum EFP of 49 PCF, as specified on page 23 of the report. All other surcharge loads shall be incorporated into the design (P/BC 2014-083, P/BC 2014-141).

31. Retaining walls higher than 6 feet shall be designed for lateral earth pressure due to earthquake motions. A triangular pressure distribution with an equivalent fluid pressure of 24 PCF shall be utilized, as specified on page 21 of the report (1803.5.12).

32. Basement walls and other walls in which horizontal movement is restricted at the top shall be designed for a triangular pressure distribution with an equivalent fluid pressure of 70 PCF. When the restrained wall is designed for hydrostatic pressure, the EFP of 90 PCF shall be used, as specified on page 23 of the report (1610.1). All other surcharge loads shall be incorporated into the design (P/BC 2014-083, P/BC 2014-141).

33. All retaining walls shall be provided with a standard surface backdrain system and all drainage shall be conducted to the street in an acceptable manner and in a non-erosive device. (7013.11)

34. With the exception of retaining walls designed for hydrostatic pressure, all retaining walls shall be provided with a subdrain system to prevent possible hydrostatic pressure behind

the wall. Prior to issuance of any permit, the retaining wall subdrain system recommended in the soil report shall be incorporated into the foundation plan which shall be reviewed and approved by the soils engineer of record. (1805.4)

35. Installation of the subdrain system shall be inspected and approved by the soils engineer of record and the City grading/building inspector. (108.9)

36. Basement walls and floors shall be waterproofed/damp-proofed with an L.A. City approved "Below-grade" waterproofing/damp-proofing material with a research report number. (104.2.6)

37. Prefabricated drainage composites (Miradrain) (Geotextiles) may be only used in addition to traditionally accepted methods of draining retained earth.

38. Where the ground water table is lowered and maintained at an elevation not less than 6 inches below the bottom of the lowest floor, or where hydrostatic pressures will not occur, the floor and basement walls shall be damp-proofed. Where a hydrostatic pressure condition exists, and the design does not include a ground-water control system, basement walls and floors shall be waterproofed. (1803.5.4, 1805.1.3, 1805.2, 1805.3)

39. All roof or pad drainage shall be conducted to the street in an acceptable manner (7013.10)

40. All concentrated drainage shall be conducted in an approved device and disposed of in a manner approved by the LADBS. (7013.10)

41. Prior to issuance of a permit involving de-watering, clearance shall be obtained from the Department of Public Works and from the California Regional Water Quality Control Board.

201 N. Figueroa Street 3rd Floor, LA	(213) 482-7045
320 W. 4th Street, Suite 200	(213) 576-6600 (LARWQB)

42. The area shall be de-watered under the direction of the consultants prior to beginning the excavation. Note, that a permit from the State of California Regional Water Quality Control Board and Department of Public Works shall be obtained to discharge the water into a storm drain.

201 N. Figueroa Street 3rd Floor, LA	(213) 482-7045
320 W. 4th Street, Suite 200	(213) 576-6600 (LARWQB)

43. Any recommendations prepared by the geologist and/or the soils engineer for correction of geological hazards found during grading shall be submitted to the Grading Division of the Department for approval prior to utilization in the field. (7008.2, 7008.3)

44. The geologist and soils engineer shall inspect all excavations to determine that conditions anticipated in the report have been encountered and to provide recommendations for the correction of hazards found during grading. (7008 & 1705.6)

45. Prior to the pouring of concrete, a representative of the consulting soils engineer shall inspect and approve the footing excavations. He/She shall post a notice on the job site for the LADBS Building Inspector and the Contractor stating that the work so inspected meets the conditions of the report, but that no concrete shall be poured until the City Building

Inspector has also inspected and approved the footing excavations. A written certification to this effect shall be filed with the Grading Division of the Department upon completion of the work. (108.9 & 7008.2)

46. Prior to excavation, an initial inspection shall be called with LADBS Inspector at which time sequence of construction, protection fences and dust and traffic control will be scheduled. (108.9.1)

47. Installation of shoring shall be performed under the inspection and approval of the soils engineer and deputy grading inspector. (1705.6)

48. The installation and testing of tie-back anchors shall comply with the recommendations included in the report or the standard sheets titled "Requirement for Tie-back Earth Anchors", whatever is more restrictive. (Research Report #23835)

49. Prior to the placing of compacted fill, a representative of the soils engineer shall inspect and approve the bottom excavations. He/She shall post a notice on the job site for the City Grading Inspector and the Contractor stating that the soil inspected meets the conditions of the report, but that no fill shall be placed until the LADBS Grading Inspector has also inspected and approved the bottom excavations. A written certification to this effect shall be included in the final compaction report filed with the Grading Division of the Department. All fill shall be placed under the inspection and approval of the soils engineer. A compaction report together with the approved soil report and Department approval letter shall be submitted to the Grading Division of the Department upon completion of the compaction. In addition, an Engineer's Certificate of Compliance with the legal description as indicated in the grading permit and the permit number shall be included. (7011.3)

50. No footing/slab shall be poured until the compaction report is submitted and approved by the Grading Division of the Department.

DAN L. STOICA
Geotechnical Engineer I

DLS/dls
Log No. 95056
213-482-0480

cc: Geocon West, Inc., Project Consultant
LA District Office

Appendix G.3

Office Option Geotechnical Report

GEOCON
WEST, INC.

GEOTECHNICAL
ENVIRONMENTAL
MATERIALS

UPDATED GEOTECHNICAL INVESTIGATION

PROPOSED HIGH-RISE REDEVELOPMENT
6254-6274 W. DE LONGPRE AVENUE
1334 & 1348-1360 N. VINE STREET
6241-6265 W. AFTON PLACE
LOS ANGELES, CALIFORNIA
TRACT: 1210, BLOCK A, LOT: 11-23

PREPARED FOR

**ONNI CONTRACTING (CALIFORNIA), INC.
LOS ANGELES, CALIFORNIA**

PROJECT NO. A9382-06-02

REVISED AUGUST 17, 2020

GEOCON
WEST, INC.
GEOTECHNICAL ■ ENVIRONMENTAL ■ MATERIALS

Project No. A9382-06-02
Revised August 17, 2020

Mr. Mark Spector
Onni Contracting (California), Inc.
315 West 9th Street, Suite 801
Los Angeles, California 90015

Subject: UPDATED GEOTECHNICAL INVESTIGATION
PROPOSED HIGH-RISE REDEVELOPMENT – "1360 VINE"
6254-6274 W. DE LONGPRE AVENUE, 1334 & 1348-1360 N. VINE STREET
6241 -6265 W. AFTON PLACE, LOS ANGELES, CALIFORNIA
TRACT 1210, BLOCK A, LOTS 11-23

References: *Geotechnical Investigation*, prepared by Geocon West, Inc., dated Sept. 21, 2016;

City of Los Angeles Soils Report Approval Letter, Log No. 95056, dated Oct. 18, 2016.

Dear Mr. Spector:

In accordance with your authorization of our proposal dated April 1, 2020, we have performed an updated geotechnical investigation for the proposed high-rise development located at the southeast corner of De Longpre Avenue and Vine Street in the Hollywood area of Los Angeles, California. The accompanying report presents the findings of our study, and our conclusions and recommendations pertaining to the geotechnical aspects of proposed design and construction. Based on the results of our investigation, it is our opinion that the site can be developed as proposed, provided the recommendations of this report are followed and implemented during design and construction.

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

Very truly yours,

GEOCON WEST, INC.

PETRINA ZEN C87489 CIVIL

SUSAN FRANZEN KIRKGARD No. 1754 CERTIFIED ENGINEERING GEOLOGIST

JELISA THOMAS ADAMS GE3092 GEOTECHNICAL

Petrina Zen

Petrina Zen
PE 87489

Susan F. Kirkgard
CEG 1754

Jelisa Thomas Adams
GE 3092

(EMAIL) Addressee

TABLE OF CONTENTS

TABLE OF CONTENTS (Continued)

UPDATED GEOTECHNICAL INVESTIGATION

1. PURPOSE AND SCOPE

This report presents the results of an updated geotechnical investigation for the proposed high-rise development located at the corner of De Longpre Avenue and Vine Street in the Hollywood area of Los Angeles, California (see Vicinity Map, Figure 1). The purpose of the investigation was to evaluate subsurface soil and geologic conditions underlying the site and, based on conditions encountered, to provide conclusions and recommendations pertaining to the geotechnical aspects of design and construction.

The scope of this investigation included a site reconnaissance, a review of documents on file with LADBS, field exploration, laboratory testing, engineering analysis, and the preparation of this report. The site was explored on February 25, 2016 and February 26, 2016 by excavating two 8-inch diameter borings to depths of approximately 101½ feet below the existing ground surface utilizing a truck-mounted hollow-stem auger drilling machine. Additional site exploration was performed on June 22 and 23, 2020 by excavating one 4⅞-inch diameter boring to a depth of approximately 199½ feet below existing ground surface using a truck-mounted mud-rotary drilling machine. A geophysical survey consisting of down-hole suspension PS logging was performed in the boring as a part of the site exploration on June 24, 2020. The approximate locations of the exploratory borings are depicted on the Site Plan (see Figure 2). A detailed discussion of the field investigation, including boring logs, is presented in Appendix A.

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

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

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

2. SITE AND PROJECT DESCRIPTION

The subject site is located at the southeast corner of De Longpre Avenue and Vine Street in the City of Los Angeles, California. The site includes the following addresses: 6254-6274 W. De Longpre Avenue, 1334 & 1348-1360 N. Vine Street, and 6241-6265 W. Afton Place, Los Angeles, California. The site is an approximately rectangular-shaped parcel and is currently occupied by several one-story single-family residential lots, a two-story multi-family residential structure, and one- to two story commercial structures. The site is bounded by Vine Street to the west, De Longpre Avenue to the north, Afton Place to the south, and by multi-family residential structures to the east. The site is relatively level with no pronounced highs or lows. Surface water drainage at the site appears to be by sheet flow along the existing ground contours to the city streets. Vegetation onsite consists of grass and trees, which are located in isolated planter areas.

Based on the information provided by the Client, it is our understanding that the proposed development will consist of a 17-story tower underlain by 8 levels of subterranean parking. It is anticipated that the lowest subterranean level will extend to a depth of approximately 83 feet below the ground surface. The tower will occupy only the western portion of the site; the eastern portion of the site will consist of open space at the existing ground surface. The proposed construction is depicted on the Site Plan and Cross-Sections (see Figures 2, 3A, and 3B).

Based on the preliminary nature of the design at this time, wall and column loads were not available. It is estimated that wall loads may be up to 40 kips per linear foot and column loads may be up to 3,800 kips for the proposed tower.

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

3. GEOLOGIC SETTING

The site is located in the northern portion of the Los Angeles Basin, a coastal plain bounded by the Santa Monica Mountains on the north, the Elysian Hills and Repetto Hills on the northeast, the Puente Hills and Whittier Fault on the east, the Palos Verdes Peninsula and Pacific Ocean on the west and south, and the Santa Ana Mountains and San Joaquin Hills on the southeast. The basin is underlain by a deep structural depression which has been filled by both marine and continental sedimentary deposits underlain by a basement complex of igneous and metamorphic composition. The basement surface within the central portion of the basin extends to a maximum depth of approximately 32,000 feet below sea level (Yerkes et al., 1965). Regionally, the site is located within the northern portion of the Peninsular Ranges geomorphic province. This geomorphic province is characterized by northwest-trending physiographic and geologic features such as the Newport-Inglewood Fault Zone located approximately 5.4 miles to the west. The northern boundary of this province is the active Hollywood Fault, located approximately 0.5 mile to the north.

4. SOIL AND GEOLOGIC CONDITIONS

Based on our field investigation and published geologic maps of the area, the site is underlain by artificial fill and slightly to moderately consolidated Pleistocene age deposits consisting of silt, sand, clay and gravel (Dibblee, 1991; California Geological Survey, 2012). Detailed stratigraphic profiles are provided on the boring logs in Appendix A. The subsurface distribution of the geologic materials and groundwater conditions encountered at the site are shown in Figure 3A.

4.1 Artificial Fill

Artificial fill was encountered in our field explorations to a maximum depth of 13 feet below existing ground surface. The artificial fill varied in composition across the site. In borings B1 and B3 (located in the northwestern corner of the site), the fill consists of brown silty sand to sandy silt. In boring B2 (located in the southeastern portion of the site, the fill consists of dark brown clay with trace fine-grained sand. The artificial fill is characterized as dry to slightly moist and loose to medium dense or very soft to soft. The fill is likely the result of past grading or construction activities at the site. Deeper fill may exist between excavations and in other portions of the site that were not directly explored.

4.2 Older Alluvium

Pleistocene age alluvium was encountered beneath the artificial fill and consists primarily of reddish brown, yellowish brown, and brown interbedded silty sand, clayey sand, sand with various amounts of silt and gravel, silty clay and sandy clay. The older alluvial soils are primarily slightly moist to wet and medium dense to very dense or firm to hard.

5. GROUNDWATER

Review of the Seismic Hazard Zone Report for the Hollywood Quadrangle (California Division of Mines and Geology [CDMG], 1998) indicate the historically highest groundwater level in the area is approximately 45 feet beneath the ground surface. Groundwater information presented in this document is generated from data collected in the early 1900's to the late 1990s. Based on current groundwater basin management practices, it is unlikely that groundwater levels will ever exceed the historic high levels.

The Los Angeles County Department of Public Works (LACDPW) has maintained various wells in the vicinity of the subject site over the past 50 years. The closest groundwater monitoring well to the site is Well No. 2671A (State No. 1S14W14E01) located approximately 0.6 mile to the south (LACDPW, 2020a). Due to the distance of this well to the site and the known variation of the groundwater levels in the immediate area, the groundwater monitoring data for this well is not considered representative of historic groundwater levels at the site.

Groundwater was encountered in borings B1 and B2 at depths of 48 and 39 feet below the existing ground surface, respectively. The groundwater level in boring B3, drilled with a truck-mounted mud-rotary drill rig, was not established. These groundwater levels are not static groundwater levels but represent the first water encountered in the borings. The water levels encountered in the borings, particularly in boring B2, likely represent perched water since they are approximately the same elevation or at a higher elevation than the historic high groundwater levels reported by CDMG (1998) for this area. It should be noted that the water encountered in boring B2 was immediately above a less permeable clayey sand bed that strongly suggests this is a perched water condition. Considering the historic high groundwater levels (CDMG, 1998) and the depth to perched water encountered in our borings, groundwater may be encountered during construction. It is not uncommon for groundwater levels to vary seasonally or for groundwater seepage conditions to develop where none previously existed, especially in impermeable fine-grained soils which are heavily irrigated or after seasonal rainfall. In addition, recent requirements for stormwater infiltration could result in shallower seepage conditions in the immediate site vicinity. Proper surface drainage of irrigation and precipitation will be critical for future performance of the project. Recommendations for drainage are provided in the S*urface Drainage* section of this report (see Section 7.24).

6. GEOLOGIC HAZARDS

6.1 Surface Fault Rupture

The numerous faults in Southern California include Holocene-active, pre-Holocene, and inactive faults. The criteria for these major groups are based on criteria developed by the California Geological Survey (CGS, formerly known as CDMG) for the Alquist-Priolo Earthquake Fault Zone Program (CGS, 2018). By definition, a Holocene-active fault is one that has had surface displacement within Holocene time (about the last 11,700 years). A pre-Holocene fault has demonstrated surface displacement during Quaternary time (approximately the last 1.6 million years) but has had no known Holocene movement. Faults that have not moved in the last 1.6 million years are considered inactive.

The site is not located within a state-designated Alquist-Priolo Earthquake Fault Zone (CGS, 2020a; 2020b; CGS, 2014) nor a city-designated Preliminary Fault Rupture Study Area (City of Los Angeles, 2020) for surface fault rupture hazards. No Holocene-active or pre-Holocene faults with the potential for surface fault rupture are known to pass directly beneath the site. Therefore, the potential for surface rupture due to faulting occurring beneath the site during the design life of the proposed development is considered low. However, the site is located in the seismically active Southern California region, and could be subjected to moderate to strong ground shaking in the event of an earthquake on one of the many active Southern California faults. The faults in the vicinity of the site are shown in Figure 4, Regional Fault Map.

The closest surface trace of a Holocene-active fault to the site is the Hollywood Fault located approximately 0.5 mile to the north (CGS, 2014). Other nearby active faults include the Raymond Fault, the Newport-Inglewood Fault Zone, the Santa Monica Fault, and the Verdugo Fault located approximately 4.5 miles east, 5.4 miles west, 5.6 miles west, and 6.5 miles northeast of the site, respectively (Ziony and Jones, 1989; USGS, 2006). The active San Andreas Fault Zone is located approximately 33 miles north of the site (USGS, 2006).

Several buried thrust faults, commonly referred to as blind thrusts, underlie the Los Angeles Basin at depth. These faults are not exposed at the ground surface and are typically identified at depths greater than 3.0 kilometers. The October 1, 1987 M_w 5.9 Whittier Narrows earthquake and the January 17, 1994 M_w 6.7 Northridge earthquake were a result of movement on the Puente Hills Blind Thrust and the Northridge Thrust, respectively. These thrust faults and others in the Los Angeles area are not exposed at the surface and do not present a potential surface fault rupture hazard at the site; however, these deep thrust faults are considered active features capable of generating future earthquakes that could result in moderate to significant ground shaking at the site.

6.2 Seismicity

As with all of Southern California, the site has experienced historic earthquakes from various regional faults. The seismicity of the region surrounding the site was formulated based on research of an electronic database of earthquake data. The epicenters of recorded earthquakes with magnitudes equal to or greater than 5.0 in the site vicinity are depicted on Figure 5, Regional Seismicity Map. A partial list of moderate to major magnitude earthquakes that have occurred in the Southern California area within the last 100 years is included in the table on the following page.

LIST OF HISTORIC EARTHQUAKES

Earthquake (Oldest to Youngest)	Date of Earthquake	Magnitude	Distance to Epicenter (Miles)	Direction to Epicenter
Near Redlands	July 23, 1923	6.3	62	E
Long Beach	March 10, 1933	6.4	39	SE
Tehachapi	July 21, 1952	7.5	74	NW
San Fernando	February 9, 1971	6.6	22	NNW
Whittier Narrows	October 1, 1987	5.9	14	E
Sierra Madre	June 28, 1991	5.8	22	ENE
Landers	June 28, 1992	7.3	108	E
Big Bear	June 28, 1992	6.4	86	E
Northridge	January 17, 1994	6.7	15	WNW
Hector Mine	October 16, 1999	7.1	122	ENE
Ridgecrest	July 5, 2019	7.1	122	NNE

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

6.3 Site-Specific Shear Wave Velocity

During the site exploration program, GeoVision collected geophysical measurements for the determination of shear wave velocities as a function of depth. Suspension velocity measurements were taken in the uncased boring using an OYO PS Suspension Logging System. In-situ horizontal shear and compression wave velocity measurements were collected at 1.6-foot intervals to a depth of 187.01 feet below existing ground surface. The methodologies used by GeoVision for the data acquisition and analysis are presented in the July 17, 2020 report by GeoVision. A copy of the report is provided in Appendix C.

Based on the results of the suspension P-S logging performed by GeoVision Geophysical Services, the site-specific soil shear wave velocity for the soil to a depth of 30 meters below the ground surface (Vs30) is approximately 340 meters per second. According to the discussion in Section 1613.3.2 of the 2019 CBC and Table 20.3-1 of ASCE 7-16 and the shear wave velocity, the site falls within the boundaries of a Site Class “D”.

6.4 Seismic Design Criteria

The following table summarizes site-specific design criteria obtained from the 2019 California Building Code (CBC; Based on the 2018 International Building Code [IBC] and ASCE 7-16), Chapter 16 Structural Design, Section 1613 Earthquake Loads. The data was calculated using the online application *Seismic Design Maps*, provided by OSHPD. The short spectral response uses a period of 0.2 second. We evaluated the Site Class based on the discussion in Section 1613.2.2 of the 2019 CBC and Table 20.3-1 of ASCE 7-16. The values presented below are for the risk-targeted maximum considered earthquake (MCE_R).

2019 CBC SEISMIC DESIGN PARAMETERS

Parameter	Value	2019 CBC Reference
Site Class	D	Section 1613.2.2
MCE_R Ground Motion Spectral Response Acceleration – Class B (short), S_S	2.105g	Figure 1613.2.1(1)
MCE_R Ground Motion Spectral Response Acceleration – Class B (1 sec), S_1	0.748g	Figure 1613.2.1(2)
Site Coefficient, F_A	1	Table 1613.2.3(1)
Site Coefficient, F_V	1.7*	Table 1613.2.3(2)
Site Class Modified MCE_R Spectral Response Acceleration (short), S_{MS}	2.105g	Section 1613.2.3 (Eqn 16-36)
Site Class Modified MCE_R Spectral Response Acceleration – (1 sec), S_{M1}	1.272g*	Section 1613.2.3 (Eqn 16-37)
5% Damped Design Spectral Response Acceleration (short), S_{DS}	1.403g	Section 1613.2.4 (Eqn 16-38)
5% Damped Design Spectral Response Acceleration (1 sec), S_{D1}	0.848g*	Section 1613.2.4 (Eqn 16-39)

Note:

*Per Section 11.4.8 of ASCE/SEI 7-16, a ground motion hazard analysis shall be performed for projects for Site Class “E” sites with Ss greater than or equal to 1.0g and for Site Class “D” and “E” sites with S1 greater than 0.2g. Section 11.4.8 also provides exceptions which indicates that the ground motion hazard analysis may be waived provided the exceptions are followed. Using the code based values presented in the table above, in lieu of a performing a ground motion hazard analysis, requires the exceptions outlined in ASCE 7-16 Section 11.4.8 be followed.

The table below presents the mapped maximum considered geometric mean (MCE_G) seismic design parameters for projects located in Seismic Design Categories of D through F in accordance with ASCE 7-16.

ASCE 7-16 PEAK GROUND ACCELERATION

Parameter	Value	ASCE 7-16 Reference
Mapped MCE_G Peak Ground Acceleration, PGA	0.902g	Figure 22-7
Site Coefficient, F_{PGA}	1.1	Table 11.8-1
Site Class Modified MCE_G Peak Ground Acceleration, PGA_M	0.993	Section 11.8.3 (Eqn 11.8-1)

The Maximum Considered Earthquake Ground Motion (MCE) is the level of ground motion that has a 2 percent chance of exceedance in 50 years, with a statistical return period of 2,475 years. According to the 2019 California Building Code and ASCE 7-16, the MCE is to be utilized for the evaluation of liquefaction, lateral spreading, seismic settlements, and it is our understanding that the intent of the Building code is to maintain “Life Safety” during a MCE event. The Design Earthquake Ground Motion (DE) is the level of ground motion that has a 10 percent chance of exceedance in 50 years, with a statistical return period of 475 years.

Deaggregation of the MCE peak ground acceleration was performed using the USGS online Unified Hazard Tool, 2014 Conterminous U.S. Dynamic edition (v4.2.0). The result of the deaggregation analysis indicates that the predominant earthquake contributing to the MCE peak ground acceleration is characterized as a 6.83 magnitude event occurring at a hypocentral distance of 6.95 kilometers from the site.

Deaggregation was also performed for the Design Earthquake (DE) peak ground acceleration, and the result of the analysis indicates that the predominant earthquake contributing to the DE peak ground acceleration is characterized as a 6.71 magnitude occurring at a hypocentral distance of 10.55 kilometers from the site.

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

6.5 Site-Specific Ground Motion Hazard Analysis

It is anticipated that a site-specific ground motion hazard analysis will be necessary in order to satisfy the requirements of the City of Los Angeles Building Code and the Los Angeles Tall Buildings Structural Design Council. The analysis will generate a site-specific target response spectrum which will be used to match earthquake time history records for the structural engineer's use in analyzing the seismic response of the structure. It is recommended that the site-specific ground motion hazard analysis be performed once the structural engineer is able to provide input relating to the ground motion study.

6.6 Liquefaction Potential

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

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

The State of California Seismic Hazard Zone Map for the Hollywood Quadrangle (CDMG, 1999; CGS, 2014) indicates that the site is not located in an area designated as having a potential for liquefaction. Due to the relatively dense to stiff older alluvial deposits underlying the site and the depth of the historic high groundwater level in the site vicinity, it is our opinion that the potential for liquefaction and associated ground settlement and lateral spread to affect the site is very low.

6.7 Slope Stability

The topography at the site is relatively level and the topography in the immediate site vicinity slopes gently to the south-southwest. The site is not located within a City of Los Angeles Hillside Grading Area or a Hillside Ordinance Area (City of Los Angeles, 2020). Also, the County of Los Angeles Safety Element (Leighton, 1990) indicates the site is not located within an area identified as a "hillside area" or an area identified as having a potential for slope instability. The site is not located within an area identified as having a potential for seismic slope instability (CDMG, 1999; CGS, 2014). There are no known landslides near the site, nor is the site in the path of any known or potential landslides. Therefore, the potential for slope stability hazards to adversely affect the proposed development is considered low.

6.8 Earthquake-Induced Flooding

Earthquake-induced flooding is inundation caused by failure of dams or other water-retaining structures due to earthquakes. The Los Angeles County Safety Element (Leighton, 1990) indicates that the site is located within the Mulholland Dam inundation area. However, this reservoir, as well as others in California, are continually monitored by various governmental agencies (such as the State of California Division of Safety of Dams and the U.S. Army Corps of Engineers) to guard against the threat of dam failure. Current design, construction practices, and ongoing programs of review, modification, or total reconstruction of existing dams are intended to ensure that all dams are capable of withstanding the maximum considered earthquake (MCE) for the site. Therefore, the potential for inundation at the site as a result of an earthquake-induced dam failure is considered low.

6.9 Tsunamis, Seiches, and Flooding

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

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

The site is within an area of minimal flooding (Zone X) as defined by the Federal Emergency Management Agency (LACDPW, 2020b).

6.10 Oil Fields & Methane Potential

Review of the California Geologic Energy Management Division (CalGEM) Well Finder Website indicates that the site is not located within an oil field and oil or gas wells are not documented in the immediate site vicinity (CalGEM, 2020). However, due to the voluntary nature of record reporting by the oil well drilling companies, wells may be improperly located or not shown on the location map and undocumented wells could be encountered during construction. Any wells encountered during construction will need to be properly abandoned in accordance with the current requirements of the DOGGR.

The site is not located within the boundaries of a city-designated Methane Zone or Methane Buffer Zone (City of Los Angeles, 2020). Also, since the site is not located within the boundaries of a known oil field, the potential for the presence of methane or other volatile gases at the site is considered low. However, should it be determined that a methane study is required for the proposed development it is recommended that a qualified methane consultant be retained to perform the study and provide mitigation measures as necessary.

6.11 Subsidence

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

7. CONCLUSIONS AND RECOMMENDATIONS

7.1 General

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

7.1.2 Up to 13 feet of existing artificial fill was encountered during the site investigation. The existing fill encountered is believed to be the result of past grading and construction activities at the site. Deeper fill may exist in other areas of the site that were not directly explored. The existing fill and site soils are suitable for re-use as engineered fill provided the recommendations in the *Grading* section of this report are followed (see Section 7.5).

7.1.3 Excavation for the subterranean portion of the structure is anticipated to penetrate through the existing artificial fill and expose undisturbed alluvial soils throughout the excavation bottom.

7.1.4 Groundwater was encountered at depths of 39 and 48 feet below the ground surface, but are not considered static groundwater, and likely represent perched groundwater conditions. The historic high groundwater level is reported at a depth of 45 feet below the ground surface. Excavation for construction of the proposed subterranean levels is anticipated to extend to depths of approximately 83 feet below the ground surface, including foundation excavations. Based on these considerations, it is anticipated that groundwater may be encountered during construction. Due to the depth of the proposed excavation and the potential for seasonal fluctuation in the groundwater level, temporary dewatering measures may be required to mitigate groundwater during excavation and construction. Recommendations for temporary dewatering are discussed in Section 7.2 of this report.

7.1.5 The historically high groundwater level beneath the site is approximately 45 feet below the existing ground surface, and the proposed structure must be designed for hydrostatic pressure based on this groundwater level. The hydrostatic design will result in uplift forces on the structure that must be resisted by counterweight or structural design measures. The recommended floor slab uplift pressure to be used in design would be 62.4(H) in units of pounds per square foot (psf), where “H” is the height of the water above the bottom of the foundation in feet, assuming the water depth is 45 feet below the ground surface.

7.1.6 Based on these considerations, it is recommended that the proposed structure be supported on a reinforced concrete mat foundation system deriving support in the undisturbed alluvial soils at the excavation bottom. It is recommended that the ramp and ramp walls for the subterranean parking garage be structurally supported on the mat foundation. A mat foundation is more accommodating to subgrade stabilization, waterproofing, and hydrostatic design. All foundation excavations must be observed and approved in writing by the Geotechnical Engineer prior to placement of steel or concrete. Recommendations for the design of a mat foundation system are provided in Section 7.7 of this report.

7.1.7 Foundation uplift may be resisted by the weight of structure, as well as friction along the sides of foundations. If additional uplift resistance is required, the perimeter shoring piles may be utilized, and additional piles may be constructed within the interior of the structure. Recommendations for uplift resistance are provided in Section 7.9 of this report.

7.1.8 The alluvial soils anticipated to be exposed at the excavation bottom may be very moist and could be subject to excessive pumping. Operation of rubber tire equipment on the subgrade soils may cause excessive disturbance of the soils. Excavation activities to establish the finished subgrade elevation must be conducted carefully and methodically to avoid excessive disturbance to the subgrade. Stabilization of the excavation bottom may be required in order to provide a firm working surface upon which heavy equipment can operate. Recommendations for bottom stabilization and earthwork are provided in the *Grading* section of this report (see Section 7.5).

7.1.9 Due to the depth of the excavation and the proximity to the property lines, city streets and adjacent offsite structures, excavations will require sloping and/or shoring measures in order to provide a stable excavation. Where shoring is required it is recommended that a soldier pile shoring system be utilized. In addition, where the proposed excavation will be deeper than and adjacent to an offsite structure, the proposed shoring should be designed to resist the surcharge imposed by the adjacent offsite structure. Recommendations for *Temporary Excavations* are provided in Section 7.17 of this report.

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

7.1.11 Any changes in the design, location or elevation, as outlined in this report, should be reviewed by this office. Once the foundation loading configuration and design elevations for the existing and proposed structures proceeds to a more finalized plan, the recommendations within this report should be reviewed and revised, as necessary. Based on the final foundation loading configurations and building elevations, the potential for settlement should be reevaluated by this office.

7.2 Temporary Dewatering

7.2.1 Groundwater seepage was encountered at depths between 39 and 48 feet below the ground surface during site exploration. Based on the conditions encountered at the time of exploration, groundwater may be encountered during construction activities. The depth to groundwater at the time of construction can be further verified during initial dewatering well or shoring pile installation. If groundwater is present above the depth of the proposed foundation excavation bottom, temporary dewatering will be necessary to maintain a safe working environment during excavation and construction activities.

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

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

7.3 Soil and Excavation Characteristics

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

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

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

7.3.4 Based on depth of the proposed subterranean levels, the proposed structure would not be prone to the effects of expansive soils.

7.4 Minimum Resistivity, pH, and Water-Soluble Sulfate

7.4.1 Potential of Hydrogen (pH) and resistivity testing as well as chloride content testing were performed on representative samples of soil to generally evaluate the corrosion potential to surface utilities. The tests were performed in accordance with California Test Method Nos. 643 and 422 and indicate that the soils are considered “moderately corrosive” to “corrosive” with respect to corrosion of buried ferrous metals on site. The results are presented in Appendix B (Figure B8) and should be considered for design of underground structures.

7.4.2 Laboratory tests were performed on representative samples of the site materials to measure the percentage of water-soluble sulfate content. Results from the laboratory water-soluble sulfate tests are presented in Appendix B (Figure B8) and indicate that the on-site materials possess “negligible” sulfate exposure to concrete structures as defined by 2019 CBC Section 1904 and ACI 318-14 Table 19.3.1.1.

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

7.5 Grading

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

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

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

7.5.4 All foundations should derive support in the competent undisturbed alluvial soils generally found at or below the anticipated foundation depth of 83 feet below the existing ground surface. Foundations should be deepened as necessary to extend into satisfactory soils and must be observed and approved in writing by the Geotechnical Engineer (a representative of Geocon West, Inc.).

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

7.5.6 One method of subgrade stabilization would consist of introducing a thin lift of 3 to 6-inch diameter crushed angular rock into the soft excavation bottom. The use of crushed concrete will also be acceptable. The crushed rock should be spread thinly across the excavation bottom and pressed into the soils by track rolling or wheel rolling with heavy equipment. It is very important that voids between the rock fragments are not created so the rock must be thoroughly pressed or blended into the soils. All subgrade soils must be properly compacted and proof-rolled in the presence of the Geotechnical Engineer (a representative of Geocon West, Inc.).

7.5.7 Subgrade stabilization may also be accomplished by placing a 1-foot-thick layer of washed, angular 3/4-inch gravel atop a stabilization fabric (Mirafi 500X or equivalent), subsequent to subgrade approval. This gravel placement procedure should be conducted in sections until the entire excavation bottom has been blanketed by fabric and gravel. Heavy equipment may operate upon the gravel once it has been placed. The gravel should be compacted to a dense state utilizing a vibratory drum roller. The placement of gravel at the subgrade level should may be coordinated with the temporary or permanent dewatering of the site. The gravel and fabric system will function as both a permeable material for any necessary dewatering procedures as well as a stable material upon which heavy equipment may operate. It is recommended that the contractor meet with the Geotechnical Engineer to discuss this procedure in more detail.

7.5.8 The City of Los Angeles Department of Building and Safety requires a minimum compactive effort of 95 percent of the laboratory maximum dry density in accordance with ASTM D 1557 (latest edition) where the soils to be utilized in the fill have less than 15 percent finer than 0.005 millimeters. Soils with more than 15 percent finer than 0.005 millimeters may be compacted to 90 percent of the laboratory maximum dry density in accordance with ASTM D 1557 (latest edition). It is anticipated that the soils encountered by this firm would require the minimum 95 percent compaction requirement; however additional laboratory testing can be performed during construction to verify the compaction requirement. All fill and backfill soils should be placed in horizontal loose layers approximately 6 to 8 inches thick, moisture conditioned to optimum moisture content, and properly compacted to the required degree of compaction in accordance with ASTM D 1557 (latest edition).

7.5.9 Prior to construction of exterior slabs, the upper 12 inches of the subgrade should be moisture conditioned to optimum moisture content and properly compacted to at least 95 percent relative compaction, as determined by ASTM Test Method D1557 (latest edition).

7.5.10 Although not anticipated for this project, all imported fill shall be observed, tested, and approved by Geocon West, Inc. prior to bringing soil to the site. Rocks larger than 6 inches in diameter shall not be used in the fill. If necessary, import soils used as structural fill should have an expansion index less than 20 and corrosivity properties that are equally or less detrimental to that of the existing onsite soils (see Figure B8).

7.5.11 Utility trenches should be properly backfilled in accordance with the requirements of the Green Book (latest edition). The pipe should be bedded with clean sands (Sand Equivalent greater than 30) to a depth of at least one foot over the pipe, and the bedding material must be inspected and approved in writing by the Geotechnical Engineer (a representative of Geocon). The use of gravel is not acceptable unless used in conjunction with filter fabric to prevent the gravel from having direct contact with soil. The remainder of the trench backfill may be derived from onsite soil or approved import soil, compacted as necessary, until the required compaction is obtained. The use of minimum 2-sack slurry is also acceptable as backfill (see Section 7.6). Prior to placing any bedding materials or pipes, the excavation bottom must be observed and approved in writing by the Geotechnical Engineer (a representative of Geocon).

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

7.6 Controlled Low Strength Material (CLSM)

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

Standard Requirements

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

Requirements for CLSM that will be used for support of footings

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

7.7 Mat Foundation Design

7.7.1 It is recommended that a reinforced concrete mat foundation be utilized for support of the proposed structure. The reinforced concrete mat foundation may derive support in the undisturbed alluvial soils found at and below a depth of 83 feet below the existing ground surface. Foundations should be deepened as necessary to extend into satisfactory soils and must be observed and approved in writing by the Geotechnical Engineer (a representative of Geocon West, Inc.).

7.7.2 The recommended maximum allowable bearing value is 12,000 psf. The allowable bearing pressure may be increased by up to one-third for transient loads due to wind or seismic forces.

7.7.3 A vertical modulus of subgrade reaction of 20 pounds per cubic inch (pci) may be used in the design of mat foundations deriving support in competent alluvial soils generally found at or below the anticipated foundation depth of 83 feet below the existing ground surface. This value takes into consideration the estimated mat foundation size, but should be reevaluated once foundation loads and dimensions become available.

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

7.7.5 The proposed structure must be designed for hydrostatic pressure based on the groundwater level. The hydrostatic design will result in uplift forces on the structure that must be resisted by counterweight or structural design measures. The recommended floor slab uplift pressure to be used in design would be 62.4(H) in units of pounds per square foot (psf), where “H” is the height of the water above the bottom of the foundation in feet. For design purposes the groundwater table should be assumed to be at a depth of 45 feet below the ground surface. Considerations for uplift resistance are provided in Section 7.9 of this report.

7.7.6 For seismic design purposes, a coefficient of friction of 0.35 may be utilized between the concrete mat and alluvium without a moisture barrier, and 0.15 for slabs underlain by a moisture barrier.

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

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

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

7.8 Foundation Settlement

7.8.1 The maximum expected static settlement for a mat foundation deriving support in competent alluvial soils and utilizing a maximum allowable bearing pressure of 12,000 psf is estimated to be approximately 4 inches and occur below the central portion of the mat. The differential settlement between the center and corner of the mat is estimated to be less than 2 inches. The anticipated settlements are preliminary and should be verified once the project structural engineer can provide a final diagram of the anticipated mat foundation bearing pressures.

7.8.2 A majority of the settlement of the foundation system is expected to occur on initial application of loading; however, minor additional settlements are expected within the first 12 months.

7.8.3 Once the design and foundation loading configuration proceeds to a more finalized plan, the recommendations within this report should be reviewed and revised, if necessary. Based on the final foundation loading configuration, the potential for settlement should be reevaluated by this office.

7.9 Uplift Resistance

7.9.1 Foundation uplift may be resisted by the weight of structure, as well as friction along the sides of foundations. If additional uplift resistance is required, the perimeter shoring piles may be utilized provided the toes of the piles are poured with structural concrete and are designed as permanent piles. Recommendations for the design of shoring piles are provided in Section 7.18.

7.9.2 Uplift resistance may also be generated by additional piles constructed within the interior of the structure. It is recommended that post-grouted friction piles be utilized. The uplift capacity may be determined using a frictional resistance of 580 psf (⅔ the downward capacity, adjusted for buoyancy).

7.9.3 Post-grouted friction piles should be a minimum of 12 inches in diameter and should be uniformly spaced at least three times the diameter on-center. If so spaced, no reduction for group effects will be necessary. The allowable uplift capacity may be increased by one-third when considering transient wind or seismic loads.

7.9.4 Pile testing should be considered and performed as required by the building official to verify the uplift resistance prior to finalizing pile lengths or commencement of permanent pile installation.

7.10 Lateral Design

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

7.10.2 Passive earth pressure for the sides of foundations and slabs poured against the alluvial soils may be computed as an equivalent fluid having a density of 115 pcf with a maximum earth pressure of 1,150 pcf (values have been reduced for buoyancy). When combining passive and friction for lateral resistance, the passive component should be reduced by one-third.

7.11 Exterior Concrete Slabs-on-Grade

7.11.1 Exterior slabs, not subject to traffic loads, should be at least 4 inches thick and reinforced with No. 3 steel reinforcing bars placed 18 inches on center in both horizontal directions, positioned near the slab midpoint. Prior to construction of slabs, the upper 12 inches of subgrade should be moistened to near optimum moisture content and properly compacted to at least 95 percent relative compaction, as determined by ASTM Test Method D 1557 (latest edition). Crack control joints should be spaced at intervals not greater than 10 feet and should be constructed using saw-cuts or other methods as soon as practical following concrete placement. Crack control joints should extend a minimum depth of ¼ the slab thickness. The project structural engineer should design construction joints as necessary.

7.11.2 Slabs-on-grade at the ground surface that may receive moisture-sensitive floor coverings or may be used to store moisture-sensitive materials should be underlain by a vapor retarder placed directly beneath the slab. The vapor retarder and acceptable permeance should be specified by the project architect or developer based on the type of floor covering that will be installed. The vapor retarder design should be consistent with the guidelines presented in Section 9.3 of the American Concrete Institute's (ACI) Guide for Concrete Slabs that Receive Moisture-Sensitive Flooring Materials (ACI 302.2R-06) and should be installed in general conformance with ASTM E 1643 (latest edition) and the manufacturer's recommendations. A minimum thickness of 15 mils extruded polyolefin plastic is recommended; vapor retarders which contain recycled content or woven materials are not recommended. The vapor retarder should have a permeance of less than 0.01 perms demonstrated by testing before and after mandatory conditioning. The vapor retarder should be installed in direct contact with the concrete slab with proper perimeter seal. If the California Green Building Code requirements apply to this project, the vapor retarder should be underlain by 4 inches of clean aggregate. It is important that the vapor retarder be puncture resistant since it will be in direct contact with angular gravel. As an alternative to the clean aggregate suggested in the Green Building Code, it is our opinion that the concrete slab-on-grade may be underlain by a vapor retarder over 4 inches of clean sand (sand equivalent greater than 30), since the sand will serve a capillary break and will minimize the potential for punctures and damage to the vapor barrier.

7.11.3 For seismic design purposes, a coefficient of friction of 0.35 may be utilized between concrete slabs and subgrade soils without a moisture barrier.

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

7.12 Retaining Wall Design

7.12.1 The recommendations presented below are generally applicable to the design of rigid concrete or masonry retaining walls having a maximum height of 83 feet. In the event that walls significantly higher than 83 feet are planned, Geocon should be contacted for additional recommendations.

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

7.12.3 Assuming that proper drainage is maintained, retaining walls with a level backfill surface that are not restrained at the top should be designed utilizing a triangular distribution of pressure (active pressure) of 53 pcf.

7.12.4 Restrained walls are those that are not allowed to rotate more than 0.001H (where H equals the height of the retaining portion of the wall in feet) at the top of the wall. Where walls are restrained from movement at the top, walls may be designed utilizing a triangular distribution of pressure (at-rest pressure) of 64 pcf. Calculation of the recommended earth pressures is provided as Figure 6.

7.12.5 The wall pressures provided above assume that the retaining wall will be properly drained preventing the buildup of hydrostatic pressure. If retaining wall drainage is not implemented, the equivalent fluid pressure to be used in design of undrained walls is 96 pcf. The value includes hydrostatic pressures plus buoyant lateral earth pressures.

7.12.6 Additional active pressure should be added for a surcharge condition due to sloping ground, vehicular traffic, or adjacent structures. Recommendations for the incorporation of surcharges are provided in Section 7.23 of this report. Once the design becomes more finalized, an addendum letter can be prepared revising recommendations and addressing specific surcharge conditions throughout the project, if necessary.

7.12.7 In addition to the recommended earth pressure, the upper ten feet of the subterranean wall adjacent to the street and parking lot should be designed to resist a uniform lateral pressure of 100 pounds per square foot, acting as a result of an assumed 300 psf surcharge behind the walls due to normal street traffic. If the traffic is kept back at least 10 feet from the subterranean walls, the traffic surcharge may be neglected.

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

7.13 Dynamic (Seismic) Lateral Forces

7.13.1 The structural engineer should determine the seismic design category for the project in accordance with Section 1613 of the CBC. If the project possesses a seismic design category of D, E, or F, proposed retaining walls in excess of 6 feet in height should be designed with seismic lateral pressure (Section 1803.5.12 of the 2019 CBC).

7.13.2 A seismic load of 15 pcf should be used for design of walls that support more than 6 feet of backfill in accordance with Section 1803.5.12 of the 2019 CBC. The seismic load is applied as an equivalent fluid pressure along the height of the wall and the calculated loads result in a maximum load exerted at the base of the wall and zero at the top of the wall. This seismic load should be applied in addition to the active earth pressure. The earth pressure is based on half of two thirds of PGA_M calculated from ASCE 7-16 Section 11.8.3.

7.14 Retaining Wall Drainage

7.14.1 Retaining walls should be provided with a drainage system. At the base of the drain system, a subdrain covered with a minimum of 12 inches of gravel should be installed, and a compacted fill blanket or other seal placed at the surface (see Figure 7). The clean bottom and subdrain pipe, behind a retaining wall, should be observed by the Geotechnical Engineer (a representative of Geocon), prior to placement of gravel or compacting backfill.

7.14.2 As an alternative, a plastic drainage composite such as Miradrain or equivalent may be installed in continuous, 4-foot wide columns along the entire back face of the wall, at 8 feet on center. The top of these drainage composite columns should terminate approximately 18 inches below the ground surface, where either hardscape or a minimum of 18 inches of relatively cohesive material should be placed as a cap (see Figure 8). These vertical columns of drainage material would then be connected at the bottom of the wall to a collection panel or a 1-cubic-foot rock pocket drained by a 4-inch subdrain pipe.

7.14.3 Subdrainage pipes at the base of the retaining wall drainage system should outlet to an acceptable location via controlled drainage structures. Drainage should not be allowed to flow uncontrolled over descending slopes.

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

7.15 Elevator Pit Design

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

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

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

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

7.16 Elevator Piston

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

7.16.2 Casing may be required in the drilled excavation. The contractor should be prepared to use casing and should have it readily available at the commencement of drilling activities. The contractor should be prepared to mitigate the buoyant forces on the casing due to groundwater seepage, if encountered. Continuous observation of the drilling and installation of the elevator piston by the Geotechnical Engineer (a representative of Geocon West, Inc.) is required.

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

7.17 Temporary Excavations

7.17.1 Excavations on the order of 83 feet in height are anticipated for excavation and construction of the proposed subterranean level, foundation system, and dewatering measures. The excavations are expected to expose alluvial soils, which are suitable for vertical excavations up to 5 feet where loose soils or caving sands are not present or where not surcharged by adjacent traffic or structures.

7.17.2 Vertical excavations greater than five feet will require sloping and/or shoring measures in order to provide a stable excavation. Where sufficient space is available, temporary unsurcharged embankments could be sloped back at a uniform 1:1 slope gradient or flatter, up to a maximum of 12 feet in height. A uniform slope does not have a vertical portion. Where space is limited, shoring measures will be required. *Shoring* data is provided in Section 7.18 of this report.

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

7.18 Shoring – Soldier Pile Design and Installation

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

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

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

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

7.18.5 Drilled cast-in-place soldier piles should be placed no closer than 3 diameters on center. The minimum diameter of the piles is 18 inches. Structural concrete should be used for the soldier piles below the excavation; lean-mix concrete may be employed above that level. As an alternative, lean-mix concrete may be used throughout the pile where the reinforcing consists of a wideflange section. The slurry must be of sufficient strength to impart the lateral bearing pressure developed by the wideflange section to the soil. For design purposes, an allowable passive value for the soils below the bottom plane of excavation may be assumed to be 130 pounds per square foot per (value have been reduced for buoyant forces). Where piles are installed by vibration techniques, the passive pressure may be assumed to mobilize across a width equal to the 2 times the dimension of the beam flange. The allowable passive value may be doubled for isolated piles spaced a minimum of three the pile diameter. To develop the full lateral value, provisions should be implemented to assure firm contact between the soldier piles and the undisturbed soils.

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

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

7.18.8 Casing may be required if caving may occur in the saturated soils. If casing is used, extreme care should be employed so that the pile is not pulled apart as the casing is withdrawn. At no time should the distance between the surface of the concrete and the bottom of the casing be less than five feet. As an alternative, piles may be vibrated into place; however, there is always a risk that excessive vibrations in sandy soils could induce settlements and distress to adjacent offsite improvements. Continuous observation of the drilling and pouring of the piles by the Geotechnical Engineer (a representative of Geocon West, Inc.), is required.

7.18.9 If a vibratory method of solider pile installation is utilized, predrilling may be performed prior to installation of the steel beams. If predrilling is performed, it is recommended that the bore diameter be at least 2 inches smaller than the largest dimension of the pile to prevent excessive loss in the frictional component of the pile capacity. Predrilling should not be conducted below the proposed excavation bottom.

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

7.18.11 The level of vibration that results from the installation of the piles should not exceed a threshold where occupants of nearby structures are disturbed, despite higher vibration tolerances that a building may endure without deformation or damage. The main parameter used for vibration assessment is peak particle velocity in units of inch per second (in/sec). The acceptable range of peak particle velocity should be evaluated based on the age and condition of adjacent structures, as well as the tolerance of human response to vibration.

7.18.12 Based on Table 19 of the *Transportation and Construction Induced Vibration Guidance Manual* (Caltrans 2013), a continuous source of vibrations (ex. vibratory pile driving) which generates a maximum peak particle velocity of 0.5 in/sec is considered tolerable for modern industrial/commercial buildings and new residential structures. The Client should be aware that a lower value may be necessary if older or fragile structures are in the immediate vicinity of the site.

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

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

7.18.15 The frictional resistance between the soldier piles and retained soil may be used to resist the vertical component of the anchor load. The coefficient of friction may be taken as 0.4 based on uniform contact between the steel beam and lean-mix concrete and retained earth. The portion of soldier piles below the plane of excavation may also be employed to resist the downward loads. The downward capacity may be determined using a frictional resistance of 880 psf per foot (value has been reduced for buoyant forces).

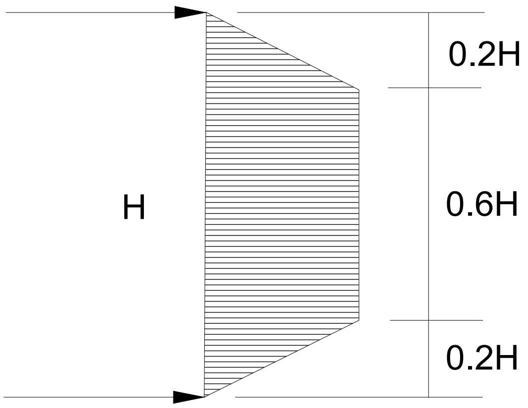
7.18.16 Due to the nature of the site soils, it is expected that continuous lagging between soldier piles will be required. However, it is recommended that the exposed soils be observed by the Geotechnical Engineer (a representative of Geocon West, Inc.), to verify the presence of any competent, cohesive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

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

7.18.18 It is recommended that an equivalent fluid pressure based on the following table, be utilized for design. A trapezoidal distribution of lateral earth pressure may be used where shoring will be restrained by bracing or tie backs. The recommended active and trapezoidal pressure are provided in the following table. A diagram depicting the trapezoidal pressure distribution of lateral earth pressure is provided below the table. Calculation of the recommended shoring pressures is provided as Figure 9.

HEIGHT OF SHORING (FEET)	**EQUIVALENT FLUID PRESSURE (Pounds Per Cubic Foot) (ACTIVE PRESSURE)**	**EQUIVALENT FLUID PRESSURE (Pounds Per Square Foot per Foot) Trapezoidal (Where H is the height of the shoring in feet)**
Up to 83	44	28H

Trapezoidal Distribution of Pressure



7.18.19 Where a combination of sloped embankment and shoring is utilized, the pressure will be greater and must be determined for each combination. Additional active pressure should be added for a surcharge condition due to slopes, vehicular traffic or adjacent structures and should be designed for each condition. The surcharge pressure should be evaluated in accordance with the recommendations in Section 7.23 of this report.

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

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

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

7.18.23 Due to the depth of the excavation and proximity to adjacent structures, it is suggested that prior to excavation the existing improvements be inspected to document the present condition. For documentation purposes, photographs should be taken of preconstruction distress conditions and level surveys of adjacent grade and pavement should be considered. During excavation activities, the adjacent structures and pavement should be periodically inspected for signs of distress. In the event that distress or settlement is noted, an investigation should be performed and corrective measures taken so that continued or worsened distress or settlement is mitigated. Documentation and monitoring of the offsite structures and improvements is not the responsibility of the geotechnical engineer.

7.19 Temporary Tie-Back Anchors

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

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

- 10 feet below the top of the excavation – 1,000 pounds per square foot
- 25 feet below the top of the excavation – 1,400 pounds per square foot
- 40 feet below the top of the excavation – 1,800 pounds per square foot
- 55 feet below the top of the excavation – 2,100 pounds per square foot
- 70 feet below the top of the excavation – 2,400 pounds per square foot

7.19.3 Depending on the techniques utilized, and the experience of the contractor performing the installation, a maximum allowable friction capacity of 5.0 kips per linear foot for post-grouted anchors (for a minimum 20-foot length beyond the active wedge) may be assumed for design purposes. Only the frictional resistance developed beyond the active wedge should be utilized in resisting lateral loads. Higher capacity assumptions may be acceptable, but must be verified by testing.

7.20 Anchor Installation

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

7.21 Anchor Testing

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

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

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

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

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

7.22 Internal Bracing

7.22.1 Rakers may be utilized to brace the soldier piles in lieu of tieback anchors. The raker bracing could be supported laterally by temporary concrete footings (deadmen) or by the permanent, interior footings. For design of such temporary footings or deadmen, poured with the bearing surface normal to rakers inclined at 45 degrees, a bearing value of 4,000 psf may be used, provided the shallowest point of the footing is at least one foot below the lowest adjacent grade. The structural engineer should review the shoring plans to determine if raker footings conflict with the structural foundation system. The client should be aware that the utilization of rakers could significantly impact the construction schedule due to their intrusion into the construction site and potential interference with equipment.

7.23 Surcharge from Adjacent Structures and Improvements

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

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

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

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

and

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

$$\sigma_H(z) = \frac{1.28 \times \left(\frac{x}{H}\right)^2 \times \left(\frac{z}{H}\right)}{\left[\left(\frac{x}{H}\right)^2 + \left(\frac{z}{H}\right)^2\right]^2} \times \frac{Q_L}{H}$$

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

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

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

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

and

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

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

then

$$\sigma'_H\ (z) = \sigma_H(z)cos^2(1.1\theta)$$

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

7.24 Surface Drainage

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

7.24.2 All site drainage should be collected and controlled in non-erosive drainage devices. Drainage should not be allowed to pond anywhere on the site, and especially not against any foundation or retaining wall. The site should be graded and maintained such that surface drainage is directed away from structures in accordance with 2019 CBC 1804.4 or other applicable standards. In addition, drainage should not be allowed to flow uncontrolled over any descending slope. Discharge from downspouts, roof drains and scuppers are not recommended onto unprotected soils within five feet of the building perimeter. Planters which are located adjacent to foundations should be sealed to prevent moisture intrusion into the soils providing foundation support. Landscape irrigation is not recommended within 5 feet of the building perimeter footings except when enclosed in protected planters.

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

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

7.25 Plan Review

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

LIMITATIONS AND UNIFORMITY OF CONDITIONS

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

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

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

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

LIST OF REFERENCES

California Department of Water Resources, 2020, *Groundwater Level Data by Township, Range, and Section,* http://www.water.ca.gov/waterdatalibrary/groundwater/ hydrographs/index_trs.cfm.

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

California Division of Mines and Geology, 1999; *State of California Seismic Hazard Zones, Hollywood Quadrangle,* Official Map, Released: March 25, 1999.

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

California Geologic Energy Management Division, 2020, CalGEM Resources Well Finder, http://maps.conservation.ca.gov.doggr/index.html#close. Accessed March 24, 2020.

California Geological Survey, 2020a, CGS Information Warehouse, Regulatory Map Portal, http://maps.conservation.ca.gov/cgs/informationwarehouse/index.html?map=regulatorymaps. Accessed March 24, 2020.

California Geological Survey, 2018, *Earthquake Fault Zones, A Guide for Government Agencies, Property Owners/Developers, and Geoscience Practitioners for Assessing Fault Rupture Hazards in California,* Special Publication 42, Revised 2018.

California Geological Survey, 2014, *Earthquake Zones of Required Investigation, Hollywood Quadrangle, dated November 6, 2014.*

California Geological Survey, 2012, *Geologic Compilation of Quaternary Surficial Deposits in Southern California, Los Angeles 30' X 60' Quadrangle*, A Project for the Department of Water Resources by the California Geological Survey, Compiled from existing sources by Trinda L. Bedrossian, CEG and Peter D. Roffers, CGS Special Report 217, Plate 9, Scale 1:100,000.

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

FEMA, 2020, Online Flood Hazard Maps, http://www.esri.com/hazards/index.html.

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

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

Los Angeles, City of, 2020, NavigateLA website, http://navigatela.lacity.org. Accessed June 23, 2020.

Los Angeles County Department of Public Works, 2020, Flood Zone Determination Website, http://dpw.lacounty.gov/apps/wmd/floodzone/map.htm. Accessed June 23, 2020.

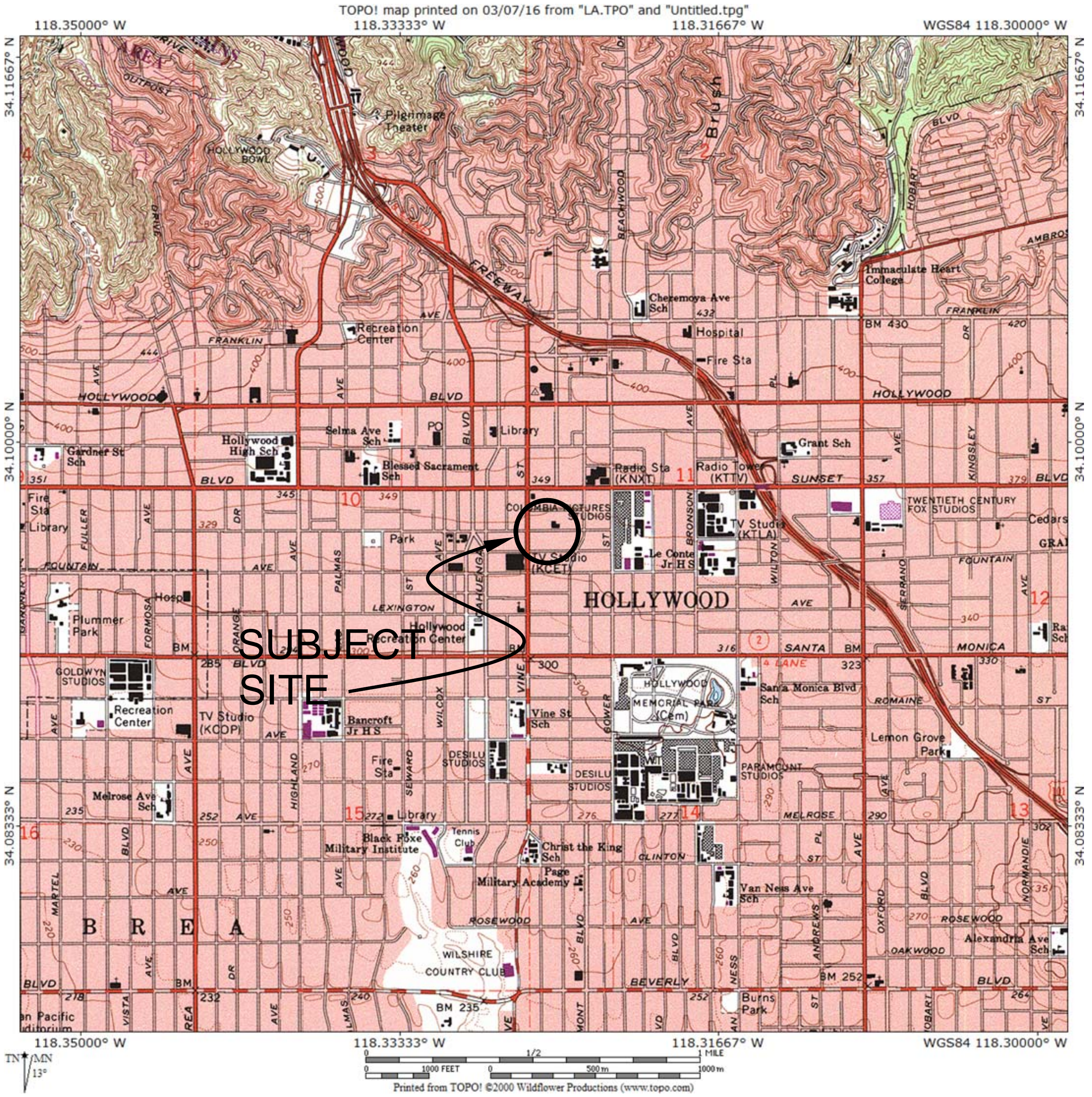
LIST OF REFERENCES (Continued)

Los Angeles, County of, 1990, *Safety Element of the General Plan.*

U.S. Geological Survey and California Geological Survey, 2006, *Quaternary Fault and Fold Database for the United States,* accessed June 23, 2020 from USGS web site: http//earthquake.usgs.gov/hazards/qfaults/.

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

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



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

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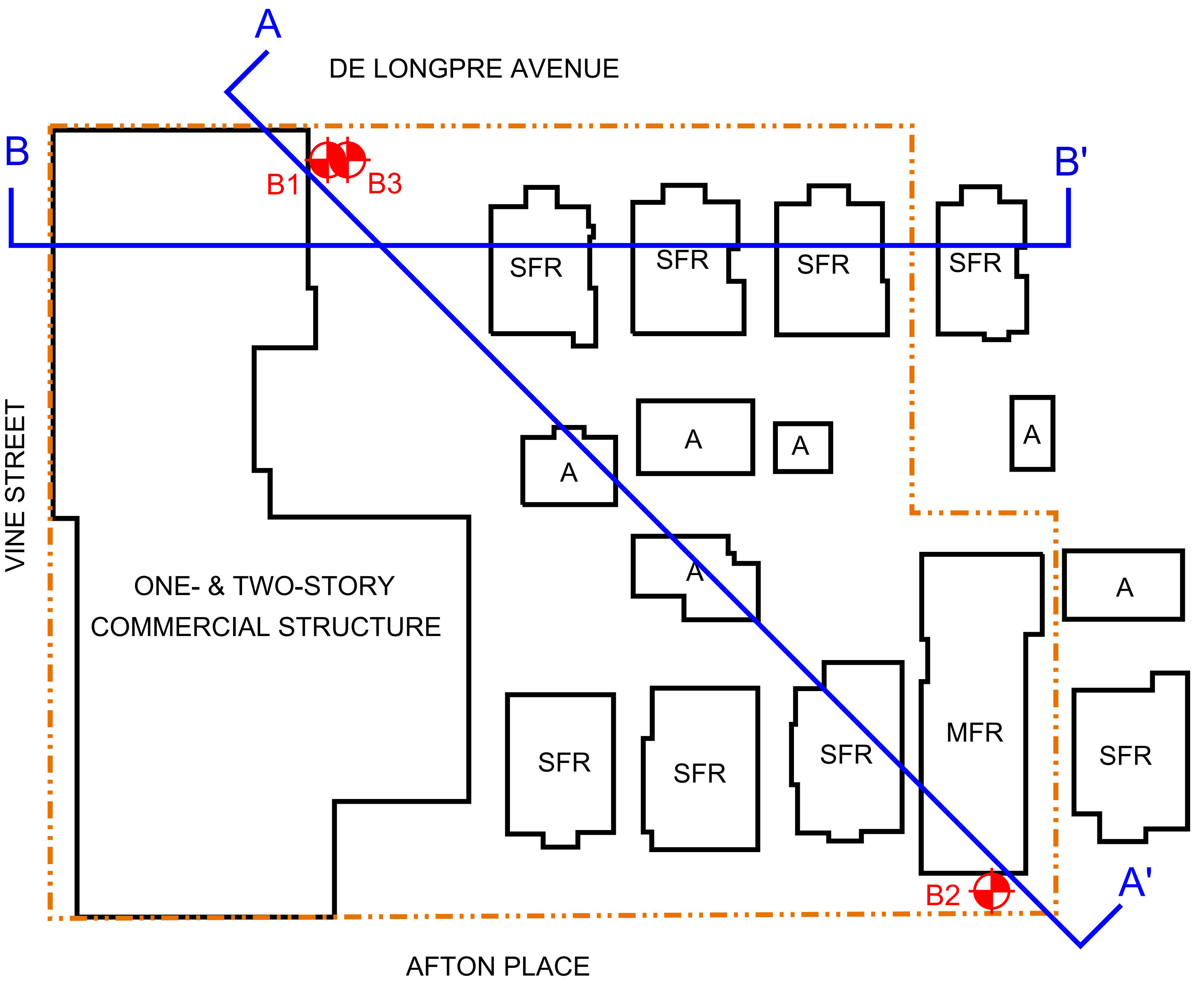
DRAFTED BY: RA CHECKED BY: SFK

VICINITY MAP

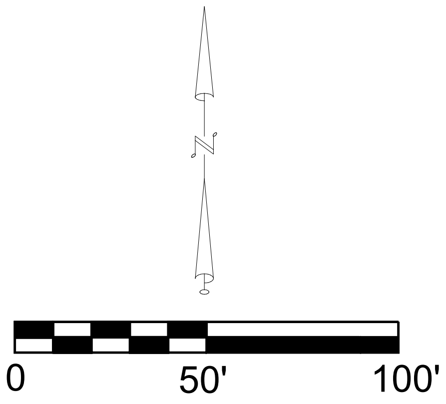
PROPOSED HIGH RISE
DE LONGPRE AVENUE & VINE STREET
LOS ANGELES, CALIFORNIA

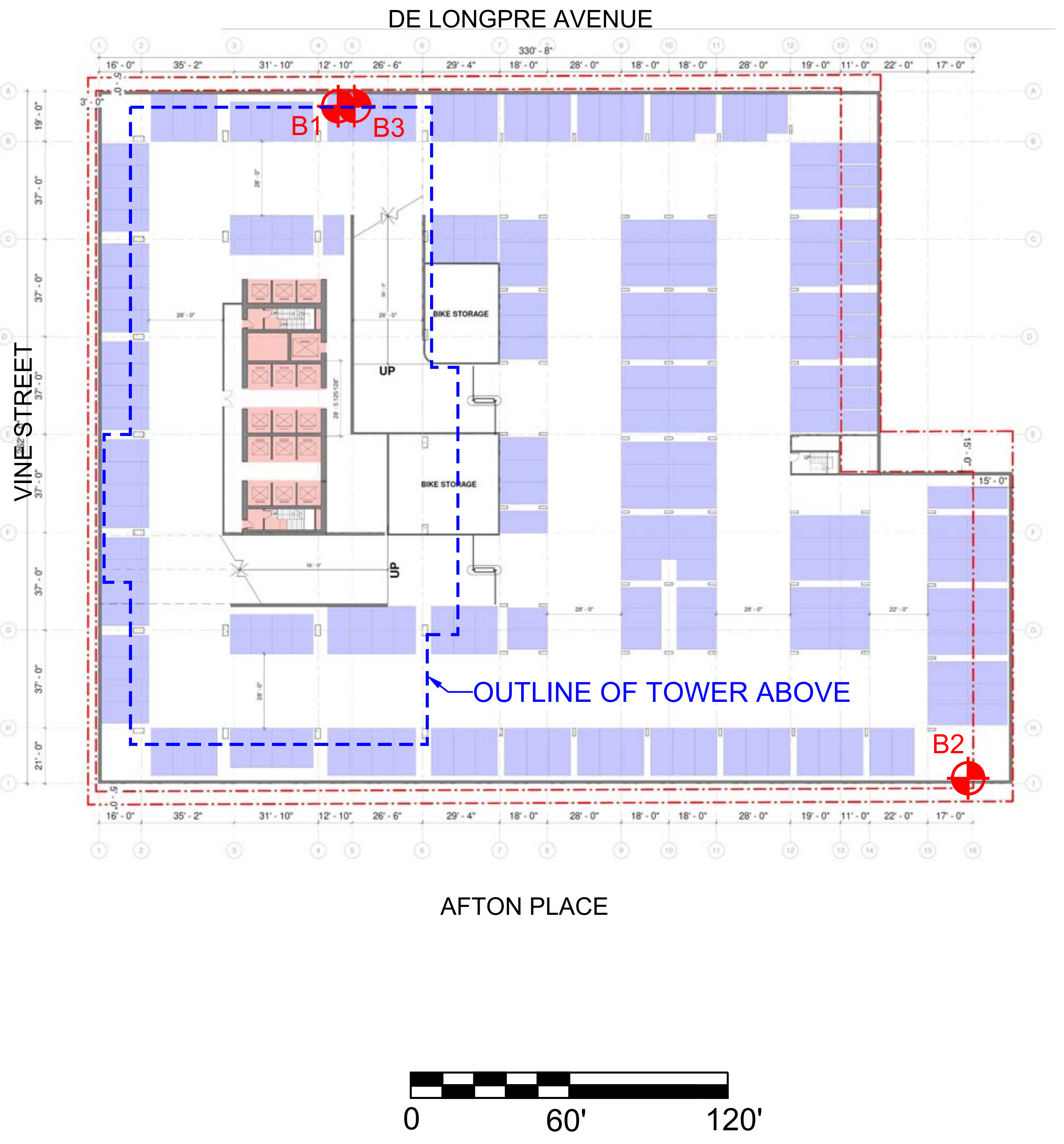
AUGUST 2020 PROJECT NO. A9382-06-02 FIG. 1



LEGEND

⊕ B3	Approximate Location of Boring
SFR	Single-Family Residence
MFR	Multi-Family Residence
A	Ancillary Structure





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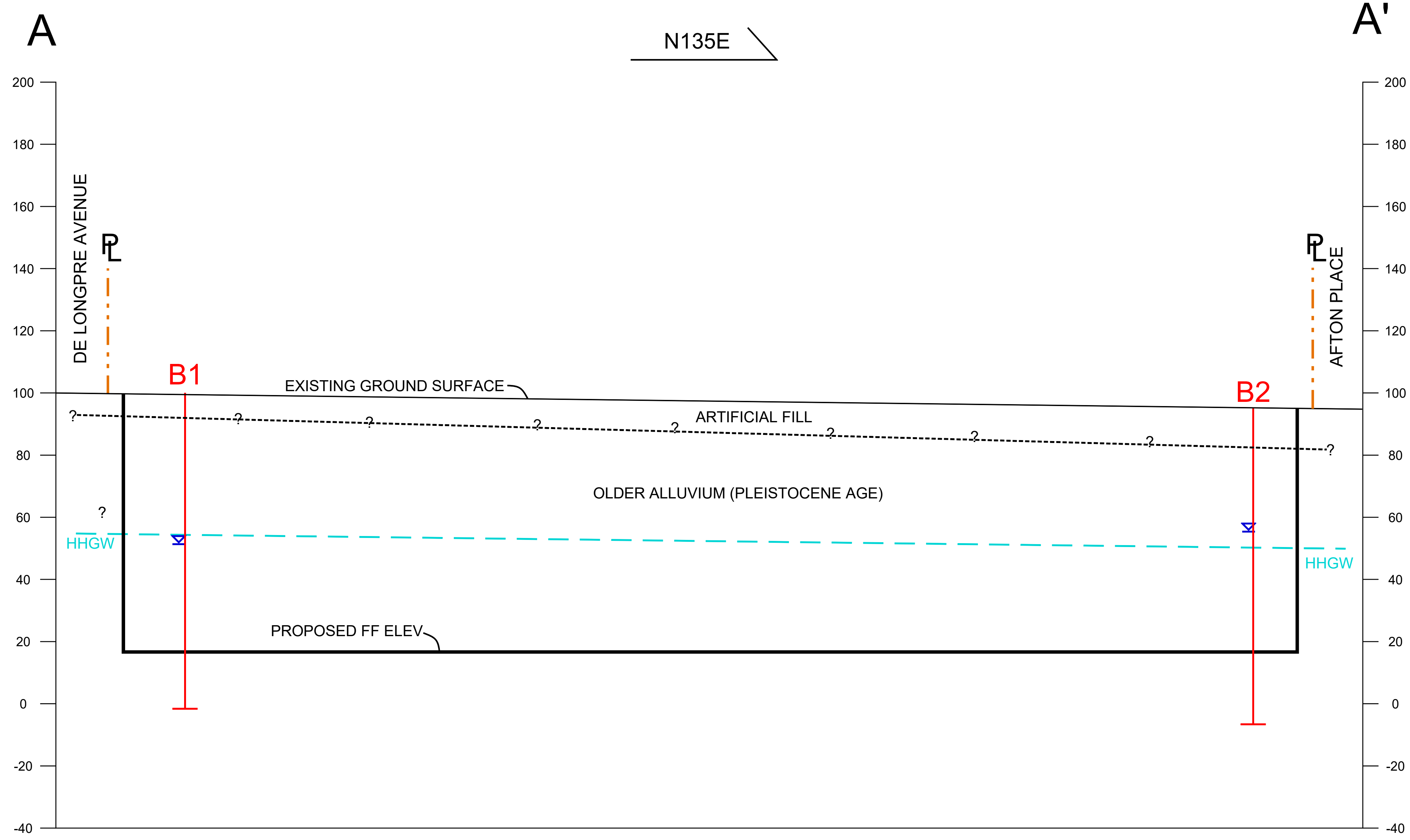
SITE PLAN

PROPOSED HIGH RISE
DE LONGPRE AVENUE & VINE STREET
LOS ANGELES, CALIFORNIA

AUGUST 2020 | PROJECT NO. A9382-06-02 | FIG. 2



SCALE: 1" = 40' (H&V)

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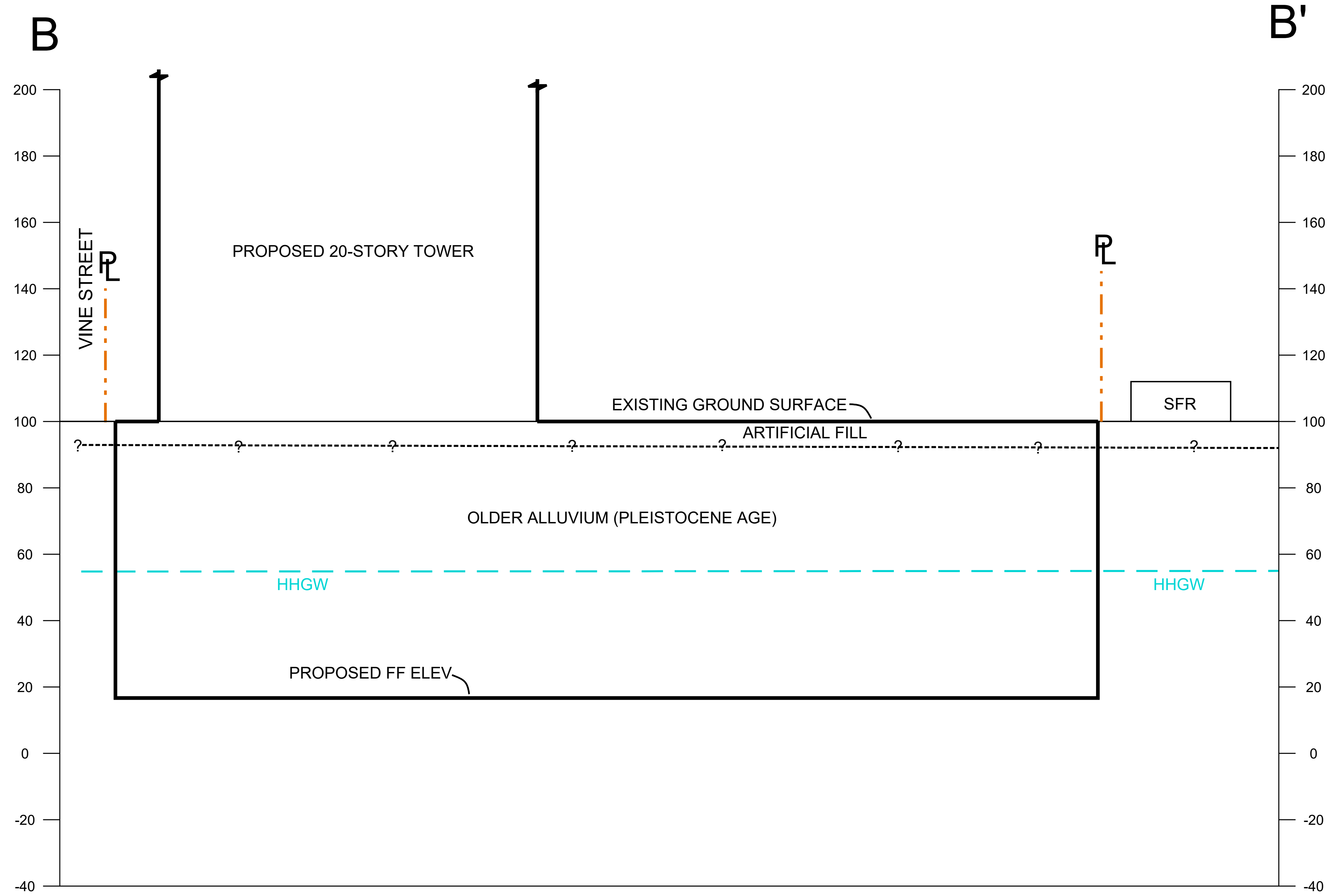
CROSS-SECTION
PROPOSED HIGH RISE
DE LONGPRE AVENUE & VINE STREET
LOS ANGELES, CALIFORNIA

AUGUST 2020	PROJECT NO. A9382-06-02	FIG. 3A



SCALE: 1" = 40' (H&V)

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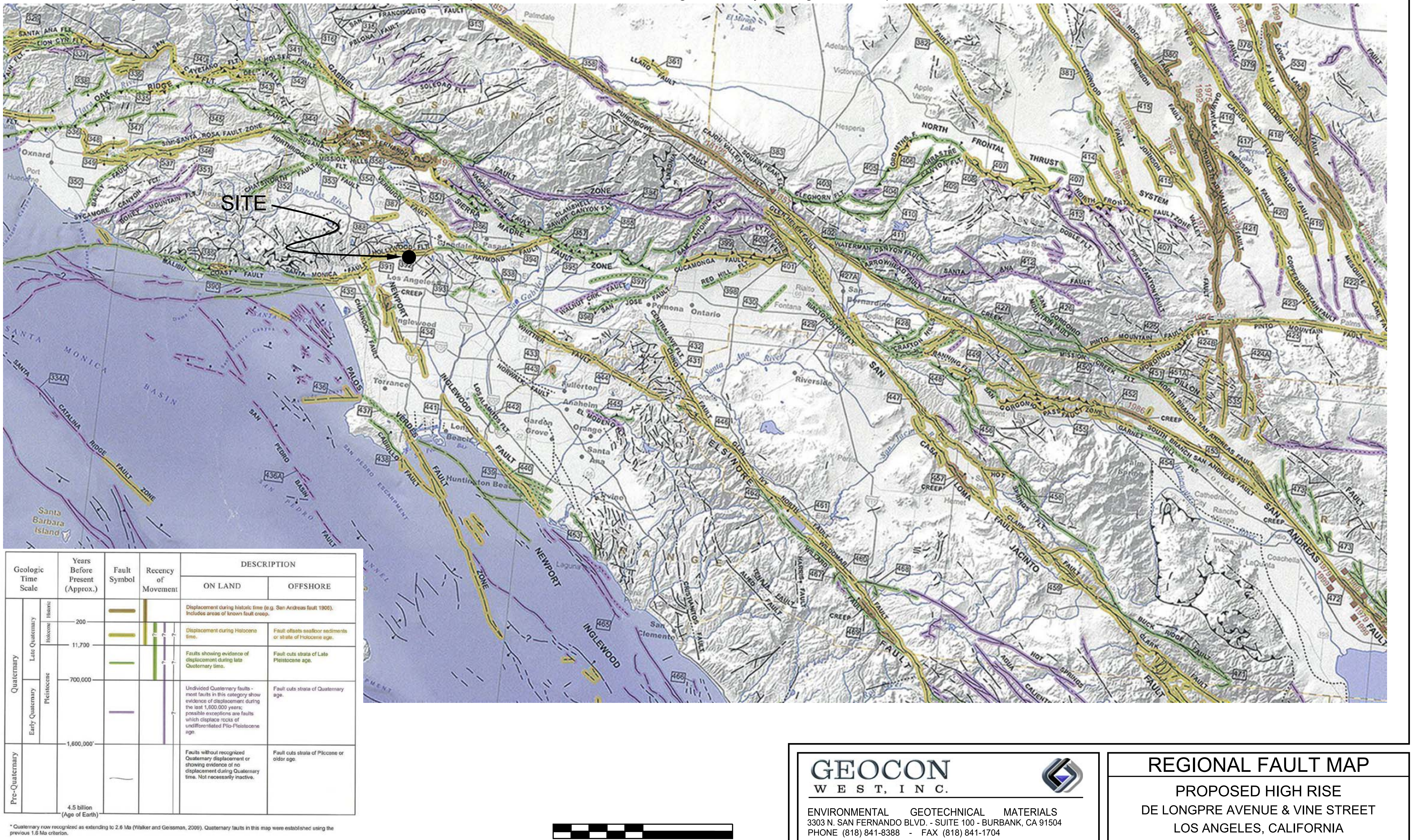
CROSS-SECTION

PROPOSED HIGH RISE
DE LONGPRE AVENUE & VINE STREET
LOS ANGELES, CALIFORNIA

AUGUST 2020	PROJECT NO. A9382-06-02	FIG. 3B

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



Geologic Time Scale			Years Before Present (Approx.)	Fault Symbol	Recency of Movement	DESCRIPTION ON LAND	DESCRIPTION OFFSHORE
Quaternary	Late Quaternary	Historic		━		Displacement during historic time (e.g. San Andreas fault 1906). Includes areas of known fault creep.	
		Holocene	200	━	? ? ?	Displacement during Holocene time.	Fault offsets seafloor sediments or strata of Holocene age.
		Pleistocene	11,700	━	? ?	Faults showing evidence of displacement during late Quaternary time.	Fault cuts strata of Late Pleistocene age.
	Early Quaternary		700,000	━	?	Undivided Quaternary faults - most faults in this category show evidence of displacement during the last 1,600,000 years; possible exceptions are faults which displace rocks of undifferentiated Plio-Pleistocene age.	Fault cuts strata of Quaternary age.
Pre-Quaternary			1,600,000* 4.5 billion (Age of Earth)	∼		Faults without recognized Quaternary displacement or showing evidence of no displacement during Quaternary time. Not necessarily inactive.	Fault cuts strata of Pliocene or older age.

* Quaternary now recognized as extending to 2.6 Ma (Walker and Geissman, 2009). Quaternary faults in this map were established using the previous 1.6 Ma criterion.

0 12 24 Miles

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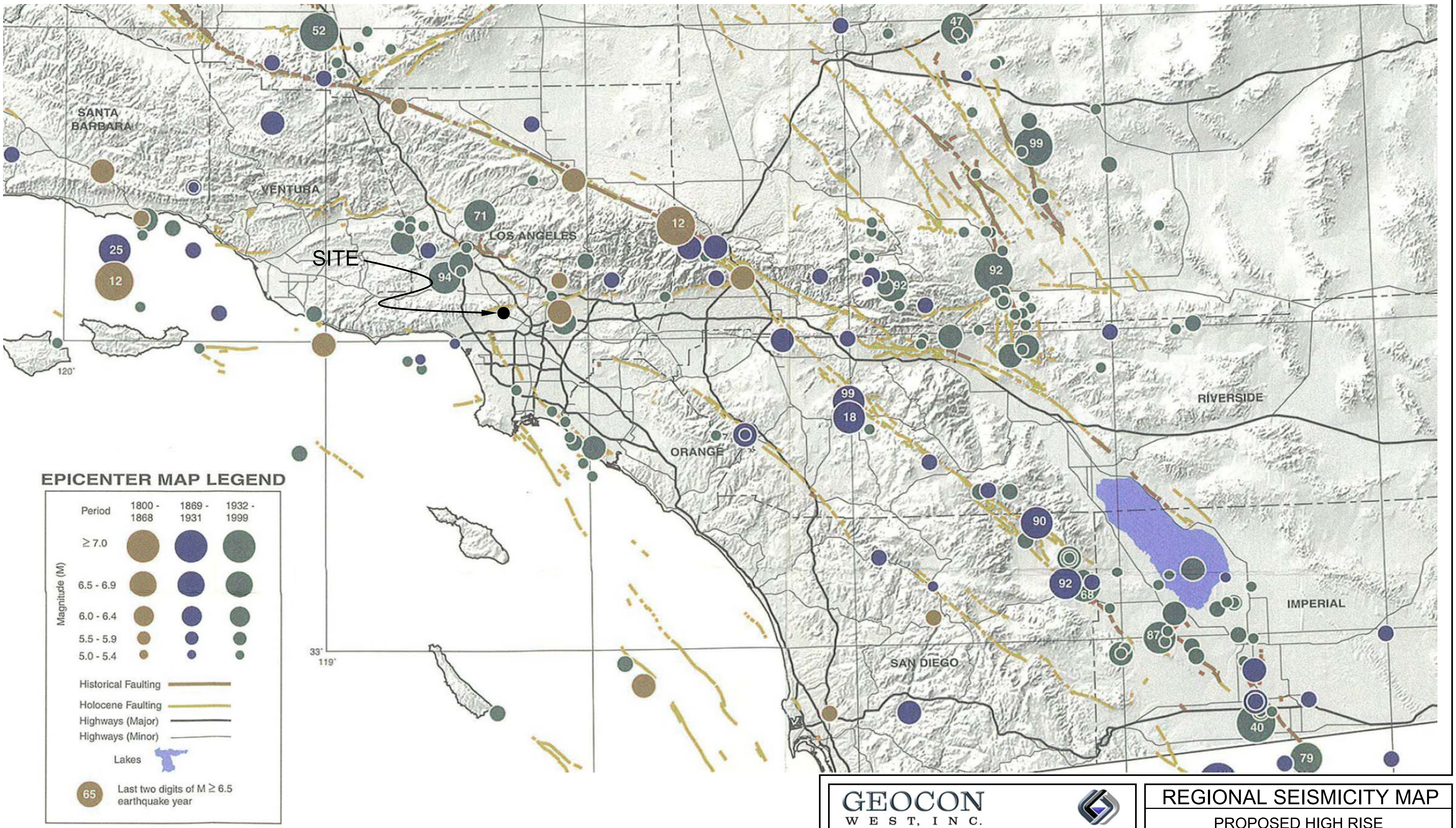
REGIONAL FAULT MAP

PROPOSED HIGH RISE
DE LONGPRE AVENUE & VINE STREET
LOS ANGELES, CALIFORNIA

AUGUST 2020 | PROJECT NO. A9382-06-02 | FIG. 4



Reference: Toppozada, T., Branum, D., Petersen, M., Hallstrom, C., Cramer, C., and Reichle, M., 2000, Epicenters and Areas Damaged by M≥5 California Earthquakes, 1800 - 1999, California Geological Survey, Map Sheet 49.

0 20 40 Miles

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REGIONAL SEISMICITY MAP

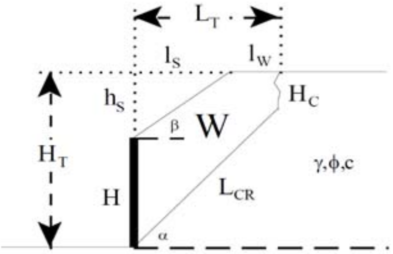
PROPOSED HIGH RISE
DE LONGPRE AVENUE & VINE STREET
LOS ANGELES, CALIFORNIA

AUGUST 2020 | PROJECT NO. A9382-06-02 | FIG. 5

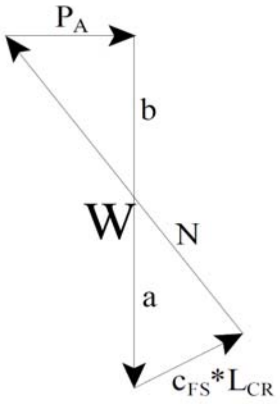
Retaining Wall Design with Transitioned Backfill
(Vector Analysis)

Input:

Retaining Wall Height	(H)	83.00	feet
Slope Angle of Backfill	(b)	0.0	degrees
Height of Slope above Wall	(h_s)	0.0	feet
Horizontal Length of Slope	(l_s)	0.0	feet
Total Height (Wall + Slope)	(H_T)	83.0	feet
Unit Weight of Retained Soils	(g)	125.0	pcf
Friction Angle of Retained Soils	(f)	29.0	degrees
Cohesion of Retained Soils	(c)	350.0	psf
Factor of Safety	(FS)	1.50	
Factored Parameters:	(f_{FS})	20.3	degrees
	(c_{FS})	233.3	psf



Failure Angle (a) degrees	Height of Tension Crack (H_C) feet	Area of Wedge (A) $feet^2$	Weight of Wedge (W) lbs/lineal foot	Length of Failure Plane (L_{CR}) feet	a lbs/lineal foot	b lbs/lineal foot	Active Pressure (P_A) lbs/lineal foot
45	5.9	3427	428370.9	109.0	57053.4	371317.5	170934.2
46	5.8	3310	413753.1	107.3	54121.9	359631.2	173223.7
47	5.7	3197	399605.7	105.7	51444.4	348161.2	175249.6
48	5.6	3087	385899.1	104.1	48992.3	336906.9	177020.7
49	5.6	2981	372606.2	102.6	46740.7	325865.4	178544.7
50	5.5	2878	359701.4	101.2	44668.5	315032.9	179828.2
51	5.4	2777	347161.2	99.8	42757.0	304404.2	180876.7
52	5.4	2680	334963.4	98.5	40990.0	293973.4	181694.5
53	5.4	2585	323087.6	97.2	39353.4	283734.1	182285.2
54	5.4	2492	311514.4	96.0	37834.6	273679.7	182651.1
55	5.4	2402	300225.9	94.8	36422.7	263803.2	182793.7
56	5.4	2314	289205.4	93.6	35107.8	254097.6	182713.7
57	5.4	2227	278437.1	92.6	33881.3	244555.8	182410.7
58	5.4	2143	267906.1	91.5	32735.4	235170.7	181883.5
59	5.4	2061	257598.7	90.5	31663.2	225935.5	181129.9
60	5.5	1980	247501.7	89.5	30658.4	216843.3	180146.7
61	5.5	1901	237602.8	88.6	29715.4	207887.4	178929.8
62	5.6	1823	227890.3	87.7	28829.1	199061.3	177474.0
63	5.7	1747	218353.3	86.8	27994.8	190358.5	175773.1
64	5.8	1672	208981.2	85.9	27208.3	181772.9	173819.8
65	5.9	1598	199764.1	85.1	26465.7	173298.4	171605.4
66	6.0	1526	190692.7	84.3	25763.4	164929.3	169120.0
67	6.2	1454	181757.7	83.5	25097.9	156659.8	166352.3
68	6.3	1384	172950.7	82.7	24466.2	148484.5	163289.6
69	6.5	1314	164263.3	81.9	23865.1	140398.2	159917.4
70	6.7	1246	155687.5	81.2	23291.8	132395.7	156219.3



Design Equations (Vector Analysis):

$a = c_{FS}*L_{CR}*\sin(90+f_{FS})/\sin(a-f_{FS})$

$b = W-a$

$P_A = b*\tan(a-f_{FS})$

$EFP = 2*P_A/H^2$

Maximum Active Pressure Resultant

$P_{A,\ max}$ 182793.7 lbs/lineal foot

Equivalent Fluid Pressure (per lineal foot of wall) At-Rest= γ*(1-sin(ϕ))

$EFP = 2*P_A/H^2$

EFP 53.1 pcf 64.4 pcf

Design Wall for an Equivalent Fluid Pressure: 53 pcf 64 pcf

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WEST, INC.

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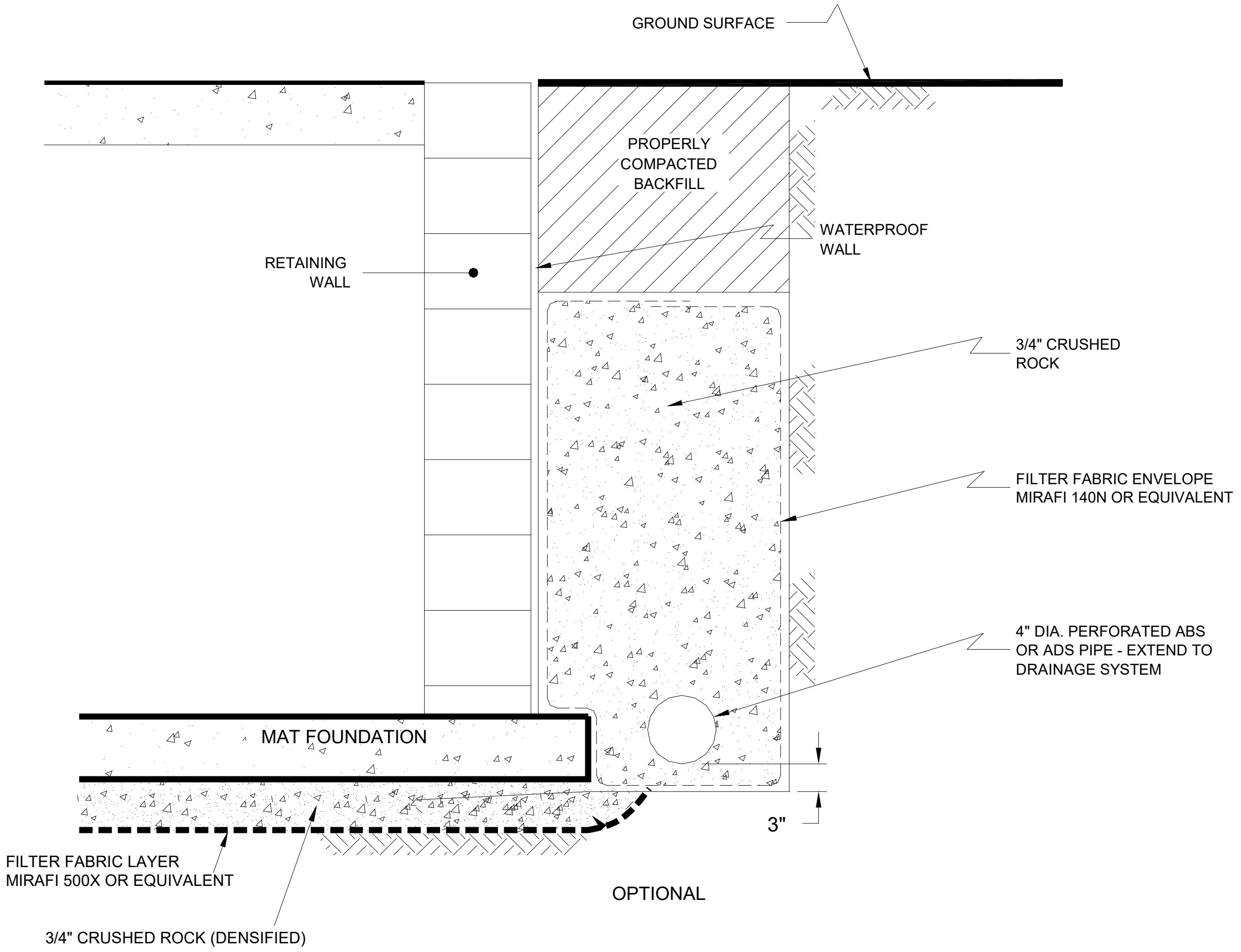
PHONE (818) 841-8388 - FAX (818) 841-1704

DRAFTED BY: PZ CHECKED BY: JTA

RETAINING WALL PRESSURE CALCULATION

PROPOSED HIGH RISE
DE LONGPRE AVENUE & VINE STREET
LOS ANGELES, CALIFORNIA

AUGUST 2020 PROJECT NO. A9382-06-02 FIG. 6



NO SCALE

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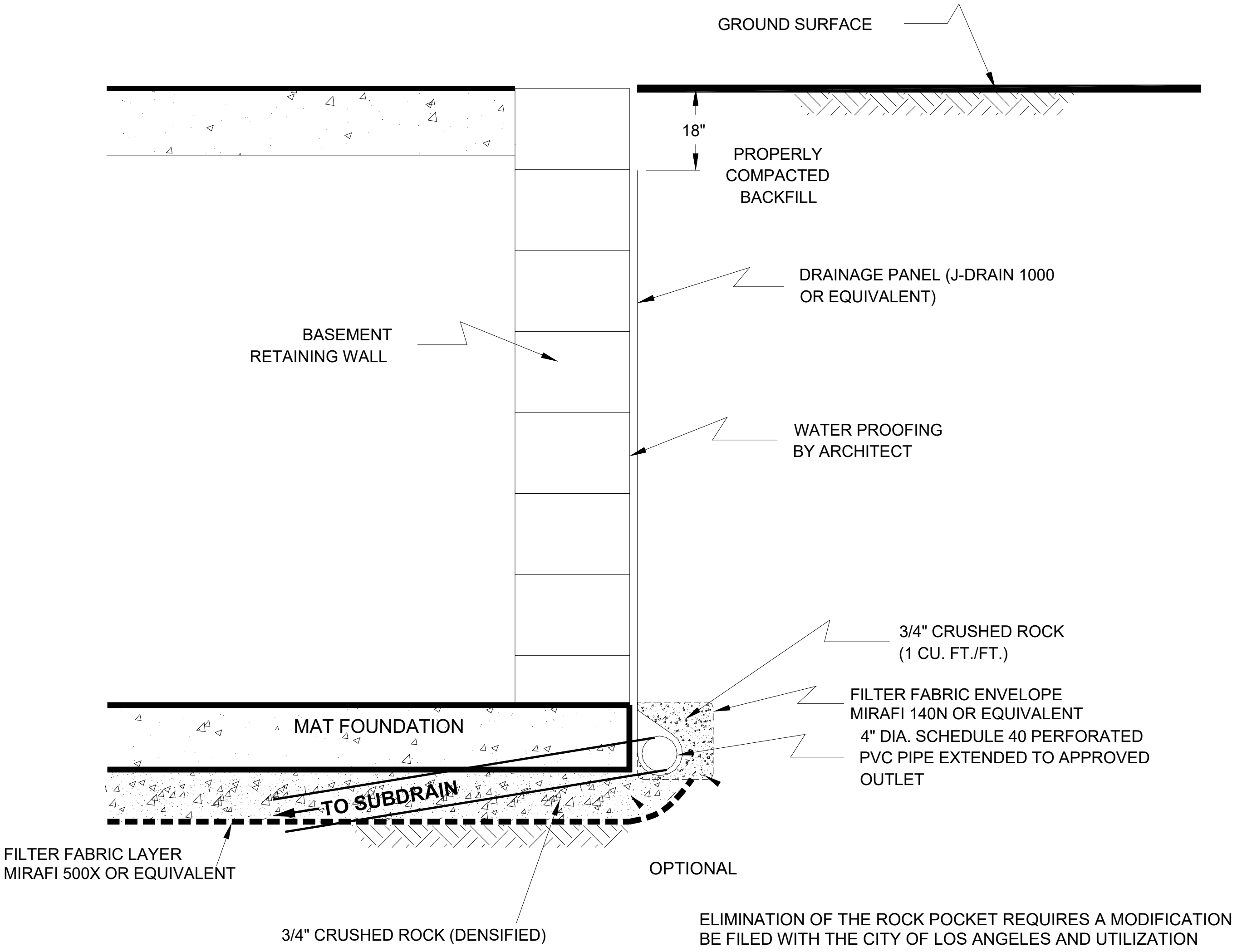
PHONE (818) 841-8388 - FAX (818) 841-1704

DRAFTED BY: PZ CHECKED BY: JTA

RETAINING WALL DRAIN DETAIL

PROPOSED HIGH RISE
DE LONGPRE AVENUE & VINE STREET
LOS ANGELES, CALIFORNIA

AUGUST 2020 PROJECT NO. A9382-06-02 FIG. 7



ELIMINATION OF THE ROCK POCKET REQUIRES A MODIFICATION
BE FILED WITH THE CITY OF LOS ANGELES AND UTILIZATION
OF A CITY APPROVED DRAINAGE PANEL

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

NO SCALE

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W E S T, I N C.

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RETAINING WALL DRAIN DETAIL

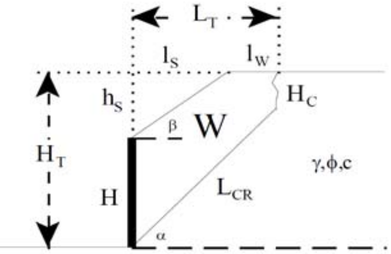
PROPOSED HIGH RISE
DE LONGPRE AVENUE & VINE STREET
LOS ANGELES, CALIFORNIA

DRAFTED BY: PZ	CHECKED BY: JTA	AUGUST 2020	PROJECT NO. A9382-06-02	FIG. 8

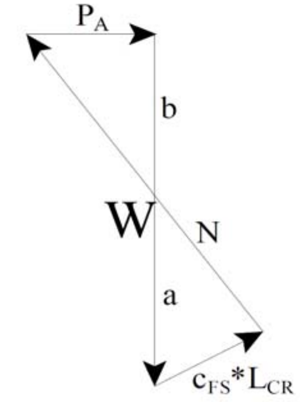
Shoring Design with Transitioned Backfill
(Vector Analysis)

Input:

Shoring Height	(H)	83.00	feet
Slope Angle of Backfill	(b)	0.0	degrees
Height of Slope above Shoring	(h_s)	0.0	feet
Horizontal Length of Slope	(l_s)	0.0	feet
Total Height (Shoring + Slope)	(H_T)	83.0	feet
Unit Weight of Retained Soils	(g)	125.0	pcf
Friction Angle of Retained Soils	(f)	29.0	degrees
Cohesion of Retained Soils	(c)	350.0	psf
Factor of Safety	(FS)	1.25	
Factored Parameters:	(f_{FS})	23.9	degrees
	(c_{FS})	280.0	psf



Failure Angle (a) degrees	Height of Tension Crack (H_C) feet	Area of Wedge (A) feet^2	Weight of Wedge (W) lbs/lineal foot	Length of Failure Plane (L_{CR}) feet	a lbs/lineal foot	b lbs/lineal foot	Active Pressure (P_A) lbs/lineal foot
45	8.0	3412	426512.8	106.0	75414.7	351098.1	135373.6
46	7.8	3297	412079.5	104.5	71130.5	340948.9	138342.7
47	7.7	3185	398088.5	103.0	67249.8	330838.7	141014.0
48	7.5	3076	384515.7	101.6	63722.7	320793.1	143398.5
49	7.4	2971	371337.7	100.2	60506.9	310830.8	145506.2
50	7.2	2868	358532.1	98.9	57566.4	300965.7	147345.5
51	7.1	2769	346077.9	97.6	54870.3	291207.7	148923.8
52	7.1	2672	333955.1	96.4	52391.9	281563.2	150247.6
53	7.0	2577	322144.7	95.2	50108.1	272036.5	151321.8
54	6.9	2485	310628.9	94.0	47999.0	262629.9	152150.8
55	6.9	2395	299390.8	92.9	46046.8	253343.9	152737.8
56	6.9	2307	288414.6	91.8	44236.3	244178.3	153085.1
57	6.9	2221	277685.2	90.8	42553.8	235131.4	153193.9
58	6.9	2138	267188.6	89.7	40987.3	226201.3	153064.8
59	6.9	2055	256911.3	88.8	39526.2	217385.1	152697.2
60	7.0	1975	246840.8	87.8	38160.9	208679.9	152089.6
61	7.0	1896	236965.0	86.9	36882.9	200082.1	151239.9
62	7.1	1818	227272.5	86.0	35684.5	191588.0	150144.5
63	7.2	1742	217752.7	85.1	34558.8	183193.9	148799.3
64	7.3	1667	208395.2	84.3	33499.4	174895.8	147199.0
65	7.4	1594	199190.4	83.4	32500.7	166689.7	145337.1
66	7.5	1521	190128.7	82.6	31557.2	158571.5	143206.2
67	7.7	1450	181201.4	81.8	30664.1	150537.3	140797.6
68	7.9	1379	172399.7	81.0	29816.7	142583.1	138101.4
69	8.1	1310	163715.5	80.3	29010.6	134704.9	135106.3
70	8.3	1241	155140.6	79.5	28241.6	126899.1	131799.7



Design Equations (Vector Analysis):

$a = c_{FS}*L_{CR}*\sin(90+f_{FS})/\sin(a-f_{FS})$

$b = W-a$

$P_A = b*\tan(a-f_{FS})$

$EFP = 2*P_A/H^2$

Maximum Active Pressure Resultant

$P_{A, max}$ 153193.9 lbs/lineal foot

Equivalent Fluid Pressure (per lineal foot of shoring)

$EFP = 2*P_A/H^2$

EFP 44.5 pcf

Design Shoring for an Equivalent Fluid Pressure: 44 pcf

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DRAFTED BY: PZ | CHECKED BY: JTA

SHORING PRESSURE CALCULATION

PROPOSED HIGH RISE

DE LONGPRE AVENUE & VINE STREET
LOS ANGELES, CALIFORNIA

AUGUST 2020 | PROJECT NO. A9382-06-02 | FIG. 9

APPENDIX A

APPENDIX A

FIELD INVESTIGATION

The site was explored on February 25, 2016 and February 26, 2016, by excavating two 8-inch diameter borings to depths of approximately 101½ feet below the existing ground surface utilizing a truck-mounted hollow-stem auger drilling machine. Additional site exploration was performed on June 22 and 23, 2020 by excavating one 4⅞-inch diameter boring to a depth of approximately 199½ feet below the existing ground surface using a truck-mounted mud-rotary drilling machine. Representative and relatively undisturbed samples were obtained by driving a 3 inch, O. D., California Modified Sampler into the “undisturbed” soil mass with blows from a 140-pound auto-hammer falling 30 inches (auto-hammer). The California Modified Sampler was equipped with 1-inch high by 2 ³/₈-inch diameter brass sampler rings to facilitate soil removal and testing. Bulk samples were also obtained. A geophysical survey consisting of down-hole suspension PS logging was performed in boring B3 as a part of the site exploration.

The soil conditions encountered in the borings were visually examined, classified and logged in general accordance with the Unified Soil Classification System (USCS). Logs of the borings are presented on Figures A1 through A3. The logs depict the soil and geologic conditions encountered and the depth at which samples were obtained. The logs also include our interpretation of the conditions between sampling intervals. Therefore, the logs contain both observed and interpreted data. We determined the lines designating the interface between soil materials on the logs using visual observations, penetration rates, excavation characteristics and other factors. The transition between materials may be abrupt or gradual. Where applicable, the logs were revised based on subsequent laboratory testing. The locations of the borings are shown on Figure 2.

PROJECT NO. A9382-06-01

BORING 1

ELEV. (MSL.) ________ DATE COMPLETED **2/25/16**

EQUIPMENT **HOLLOW STEM AUGER** BY: **MDS**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0					**AC: 3"** **ARTIFICIAL FILL** Silty Sand to Sandy Silt, loose to very soft, slightly moist, brown, fine-grained.			
2								
4								
	B1@5' ■					15	101.8	9.5
6								
8								
					OLDER ALLUVIUM Silty Sand, loose, slightly moist, reddish brown, fine-grained, trace medium-grained.			
10	B1@10' ■					15	103.5	9.8
12								
14								
	B1@15' ■				- medium dense	19	112.9	12.5
16								
18								
				SM				
20	B1@20' ■				- decrease in silt content, fine- to coarse-grained sand, trace fine gravel	24	113.6	14.8
22								
24								
	B1@25' ■					21	103.5	6.6
26								
28								

Figure A1,
Log of Boring 1, Page 1 of 4

SAMPLE SYMBOLS

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

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

PROJECT NO. A9382-06-01

BORING 1

ELEV. (MSL.) ______ DATE COMPLETED **2/25/16**

EQUIPMENT **HOLLOW STEM AUGER** BY: **MDS**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
30	B1@30' ■				- some oxidation staining	25	111.5	8.9
32								
34				SM				
	B1@35' ■				- increase in silt content, no oxidation staining	34	129.5	9.4
36								
38								
					Sand with Silt, medium dense, slightly moist, reddish brown, fine- to coarse-grained, some gravel (to 1"), some oxidation staining, trace calcium carbonate, thin clay films.			
40	B1@40' ■					38	118.0	8.9
				SP-SM				
42								
44								
					Clay with Sand, stiff, slightly moist, brown, fine-grained, low plasticity.			
	B1@45' ■					39	117.5	16.1
46								
				SP-SC				
48			▼		- groundwater			
50	B1@50' ■					41	116.9	15.3
					Silty Sand, dense, moist to wet, brown to yellowish brown, fine- to medium-grained.			
52				SM				
	B1@53' ■				Sand with Silt, dense, wet, yellowish brown, fine- to medium-grained.	69	125.3	12.0
54								
56	B1@56' ■			SP-SM	- very dense	50 (5")	--	--
58								
					Sandy Clay, stiff, moist, brown, fine-grained, low plasticity.			
	B1@59' ■			CL		38	121.6	15.7

Figure A1,
Log of Boring 1, Page 2 of 4

A9382-06-02 BORING LOGS.GPJ

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

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

BORING 1

ELEV. (MSL.) ______ DATE COMPLETED 2/25/16

EQUIPMENT HOLLOW STEM AUGER BY: MDS

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
60				CL				
61				CL	Silty Clay, stiff, moist, brown, low plasticity, trace fine-grained sand.			
62	■ B1@62'					40	112.3	13.5
64.5				SM-SC	Clayey/Silty Sand, medium dense, wet, yellowish brown, fine- to coarse-grained.			
65	■ B1@65'					39	90.6	15.9
70	■ B1@70'				- very dense	50 (6")	139.2	18.0
71				SP	Sand, poorly graded, medium dense to very dense, wet, yellowish brown, medium-grained.			
75	■ B1@75'					44	114.0	17.8
76				SM	Silty Sand, medium dense, wet, yellowish brown, fine- to medium-grained.			
80	■ B1@80'				- saturated	43	116.4	14.6
86					- dense, orangish brown with light gray mottles, some oxidation staining			
87	■ B1@87'					54	123.3	15.6

Figure A1,
Log of Boring 1, Page 3 of 4

SAMPLE SYMBOLS

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

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

PROJECT NO. A9382-06-01

BORING 1

ELEV. (MSL.) ________ DATE COMPLETED **2/25/16**

EQUIPMENT **HOLLOW STEM AUGER** BY: **MDS**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
90								
92								
94	B1@94' ■				- increase in silt content	67	116.0	17.4
96				SM				
98								
100	B1@100' ■				- medium dense, saturated	42	102.0	21.4

Total depth of boring: 101.5 feet
Fill to 8.5 feet.
Groundwater encountered at 48 feet.
Backfilled with soil cuttings and tamped.
Patched with concrete.

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

Figure A1,
Log of Boring 1, Page 4 of 4

A9382-06-02 BORING LOGS.GPJ

SAMPLE SYMBOLS

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

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

PROJECT NO. A9382-06-01

BORING 2

ELEV. (MSL.) _______ DATE COMPLETED **2/26/16**

EQUIPMENT **HOLLOW STEM AUGER** BY: **MDS**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0					**ARTIFICIAL FILL** Clay, soft, slightly moist, dark brown, trace fine-grained sand.			
2								
4								
	B2@5'					7	94.6	20.4
6								
				CL	- brown, medium plasticity			
8								
10	B2@10'				- firm	15	101.0	20.5
12								
					OLDER ALLUVIUM Sandy Silt, firm, slightly moist, brown, fine-grained.			
14								
	B2@15'					13	102.3	17.2
16				SP				
18					Sand with Silt, loose, slightly moist, yellowish brown, fine- to medium-grained.			
20	B2@20'			SP		11	99.6	10.3
22								
					Silty Sand, medium dense, moist, brown, fine- to medium-grained, trace coarse-grained sand.			
24								
	B2@25'					22	120.6	12.1
26								
28				SM				

Figure A2,
Log of Boring 2, Page 1 of 4

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

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

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PROJECT NO. A9382-06-01

BORING 2

ELEV. (MSL.) ______ DATE COMPLETED **2/26/16**

EQUIPMENT **HOLLOW STEM AUGER** BY: **MDS**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
30	B2@30' ■					26	125.3	12.5
32								
34					Silty Sand with Gravel, medium dense, moist, orangish brown, fine- to medium-grained, fine gravel, some oxidation staining, thin clay films.			
	B2@35' ■					36	125.5	10.9
36				SC				
38			▼					
					- groundwater			
40	B2@40' ■				Clayey Sand, medium dense, wet, brown, fine- to medium-grained.	21	164.7	15.4
42				SM				
44					Silty Sand, medium dense, wet, yellowish brown, fine- to coarse-grained, trace clay.			
	B2@45' ■					40	171.6	13.8
46								
48								
50	B2@50' ■				- dense, some gravel	79	173.8	13.8
				SM				
52								
54								
	B2@55' ■				- clay, hard, moist, brown, some silt, some fine-grained sand	62	171.4	11.5
56								
58								

Figure A2,
Log of Boring 2, Page 2 of 4

A9382-06-02 BORING LOGS.GPJ

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

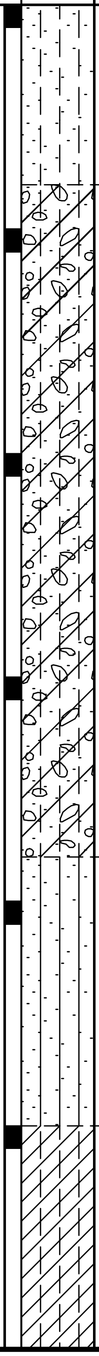
NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

PROJECT NO. A9382-06-01

BORING 2

ELEV. (MSL.) ________ DATE COMPLETED **2/26/16**

EQUIPMENT **HOLLOW STEM AUGER** BY: **MDS**



DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
60	B2@60'					38	117.9	14.5
62								
64					Silty Sand with Clay and Gravel, medium dense, wet, brown, fine- to coarse-grained.			
	B2@65'					42	168.8	17.4
66								
68								
70	B2@70'				- decrease in silt and clay content, dense to very dense	50 (6")	171.7	14.0
72				ML				
74								
	B2@75'				- medium dense	41	124.8	13.1
76								
78					Sandy Silt, stiff, moist to wet, orangish brown with light gray mottles, some oxidation staining, fine-grained.			
80	B2@80'					39	118.6	15.4
82				CL				
84								
	B2@85'				Silty Clay, hard, wet, orangish brown, medium plasticity.	51	105.7	26.7
86								
				ML				
88								

A9382-06-02 BORING LOGS.GPJ

Figure A2,
Log of Boring 2, Page 3 of 4

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

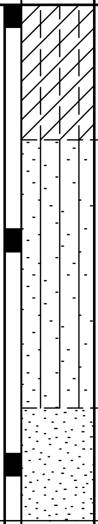
NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

GEOCON

BORING 2

ELEV. (MSL.) _______ DATE COMPLETED **2/26/16**

EQUIPMENT **HOLLOW STEM AUGER** BY: **MDS**



DEPTH IN FEET	SAMPLE NO.	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
90	B2@90'				53	108.2	22.4
92							
94			ML	Silt with Sand, stiff, orangish brown, moist, fine-grained, oxidation staining.			
96	B2@95'				25	114.8	20.9
98							
100	B2@100'		SP	Sand, poorly graded, dense, wet, yellowish brown, fine- to medium-grained.	71	127.6	8.0

Total depth of boring: 101.5 feet
Fill to 13 feet.
Groundwater encountered at 39 feet.
Backfilled with soil cuttings and tamped.
Grass divot replaced.

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

Figure A2,
Log of Boring 2, Page 4 of 4

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

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

PROJECT NO. A9382-06-01

BORING 3

ELEV. (MSL.) ________ DATE COMPLETED **6/22/20**

EQUIPMENT **MUD-ROTARY** BY: **JMH**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0					**ARTIFICIAL FILL** Silty Sand to Sandy Silt, loose to very soft, slightly moist, brown, fine-grained.			
2								
4					Silty Sand, poorly graded, medium dense, dry, brown, fine-grained, trace medium-grained.			
	B3@5'					32	96.6	8.9
6								
8								
10	B3@10'				- slightly moist	43	116.2	12.1
					OLDER ALLUVIUM Silty Sand, medium dense, slightly moist, brown, fine- to medium-grained.			
12								
14								
	B3@15'					53	116.9	14.7
16								
18								
				SM				
20	B3@20'				- reddish brown, trace coarse-grained	35	125.3	12.9
22								
24								
	B3@25'				- brown, fine-grained, trace medium- to coarse-grained	33	122.5	13.5
26								
28								

A9382-06-02 BORING LOGS.GPJ

Figure A3,
Log of Boring 3, Page 1 of 7

SAMPLE SYMBOLS

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

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GEOCON

PROJECT NO. A9382-06-01

BORING 3

ELEV. (MSL.) ________ DATE COMPLETED **6/22/20**

EQUIPMENT **MUD-ROTARY** BY: **JMH**

DEPTH IN FEET	SAMPLE NO.	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
30	B3@30' ■			- moist	48	126.9	8.6
32							
34							
	B3@35' ■		SM		68	129.2	10.8
36							
38							
40	B3@40' ■				78	128.0	12.1
42			SP-SM	Sand with Silt, poorly graded, dense, reddish brown, moist, fine-grained, trace medium-grained and fine gravel.			
44			CL	Clay with Sand, stiff, slightly moist, reddish brown, fine-grained.			
	B3@45' ■				45	115.8	19.4
46							
48							
50	B3@50' ■		SM	Silty Sand, poorly graded, dense, wet, reddish brown with yellowish brown mottles, fine- to medium-grained, trace fine gravel.	65	116.5	10.8
52							
54			SP-SM	Sand with Silt, wet, brown, fine- to medium-grained, trace fine gravel.			
	B3@55' ■				100	129.0	13.6
56							
58							
			CL	Sandy Clay, hard, moist, reddish brown.			

Figure A3,
Log of Boring 3, Page 2 of 7

A9382-06-02 BORING LOGS.GPJ

SAMPLE SYMBOLS

☐ ... SAMPLING UNSUCCESSFUL
◧ ... STANDARD PENETRATION TEST
■ ... DRIVE SAMPLE (UNDISTURBED)
☒ ... DISTURBED OR BAG SAMPLE
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PROJECT NO. A9382-06-01

BORING 3

ELEV. (MSL.) ______ DATE COMPLETED **6/22/20**

EQUIPMENT **MUD-ROTARY** BY: **JMH**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
60	B3@60'					77	131.7	12.1
62				CL				
64					Clayey Sand, poorly graded, very dense, moist, reddish brown, fine-grained, some medium-grained.			
	B3@65'					50 (4")	123.0	15.4
66								
				SC				
68								
70	B3@70'				- wet	50 (3")	125.4	12.5
					Sand, poorly graded, very dense, saturated, brown, fine- to medium-grained.			
72								
				SP				
74								
	B3@74.5'					50 (5")	119.1	13.9
					Silty Sand, very dense, saturated, brown, fine- to medium-grained.			
76								
78								
80	B3@80'					50 (3")	111.4	18.1
				SM				
82								
84								
	B3@85'				- dense	80	118.0	15.3
86								
88								

A9382-06-02 BORING LOGS.GPJ

Figure A3,
Log of Boring 3, Page 3 of 7

SAMPLE SYMBOLS

☐ ... SAMPLING UNSUCCESSFUL
◧ ... STANDARD PENETRATION TEST
■ ... DRIVE SAMPLE (UNDISTURBED)
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GEOCON

PROJECT NO. A9382-06-01

BORING 3

ELEV. (MSL.) ________ DATE COMPLETED **6/22/20**

EQUIPMENT **MUD-ROTARY** BY: **JMH**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
90	B3@90'				- very dense, reddish brown	50 (6")	115.2	16.6
92								
94								
	B3@94.5'					50 (5")	120.1	17.2
96				SM				
98								
100	B3@99.5'					50 (6")	121.9	14.9
102								
104								
	B3@105'				- wet, brown	93	115.0	19.7
106					Sand, poorly graded, very dense, saturated, brown, fine- to- medium-grained.			
				SP				
108								
110	B3@109.5				Sandy Silt, hard, moist, reddish brown.	50 (5")	127.9	13.7
				ML				
112								
114					Sandy Clay, hard, moist, reddish brown.			
	B3@115'					60	112.8	18.3
116								
				CL				
118								

A9382-06-02 BORING LOGS.GPJ

Figure A3,
Log of Boring 3, Page 4 of 7

SAMPLE SYMBOLS

☐ ... SAMPLING UNSUCCESSFUL
◧ ... STANDARD PENETRATION TEST
■ ... DRIVE SAMPLE (UNDISTURBED)
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PROJECT NO. A9382-06-01

BORING 3

ELEV. (MSL.) ________ DATE COMPLETED 6/22/20

EQUIPMENT MUD-ROTARY BY: JMH

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
120	B3@120'					50 (5")		
122								
124				CL				
	B3@125'				- brown	50 (6")	101.9	28.0
126								
128								
					Clayey Sand, poorly graded, very dense, slightly moist, brown, fine-grained, trace medium-grained.			
130	B3@130'					50 (5")	125.0	14.2
132								
134				SC				
	B3@135'				- dark brown	50 (6")	120.3	16.7
136								
138								
140	B3@140'					50 (6")	115.6	18.2
					Sand, well-graded, very dense, brown, saturated, fine- to coarse-grained.			
142				SW				
144								
	B3@144.5				Silty Sand, poorly graded, very dense, brown, wet, fine-grained, trace medium-grained.	50 (3")	121.9	16.0
146				SM				
148								

Figure A3,
Log of Boring 3, Page 5 of 7

A9382-06-02 BORING LOGS.GPJ

SAMPLE SYMBOLS

☐ ... SAMPLING UNSUCCESSFUL
◧ ... STANDARD PENETRATION TEST
■ ... DRIVE SAMPLE (UNDISTURBED)
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GEOCON

BORING 3

ELEV. (MSL.) ________ DATE COMPLETED 6/22/20

EQUIPMENT **MUD-ROTARY** BY: **JMH**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
150	B3@150' ■			SM	- some medium-grained	50 (5")	121.7	15.7
152								
154								
156								
158								
160	B3@160' ■			CL	Sandy Clay, hard, moist, brown.	50 (5")	113.2	23.6
162								
164								
166								
168								
170	B3@170' ■			SC	Clayey Sand, poorly graded, very dense, reddish brown, moist, fine-grained, trace medium-grained.	50 (5")	120.2	17.8
172								
174								
176								
178					- trace coarse-grained			

Figure A3,
Log of Boring 3, Page 6 of 7

A9382-06-02 BORING LOGS.GPJ

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

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PROJECT NO. A9382-06-01

BORING 3

ELEV. (MSL.) ______ DATE COMPLETED **6/22/20**

EQUIPMENT **MUD-ROTARY** BY: **JMH**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
180	B3@180' ■					50 (6")	102.7	16.6
182								
184				SC				
186								
188								
					Sandy Clay, hard, wet, reddish brown.			
190	B3@189.5 ■					50 (2")		
192								
194				CL				
196								
198								
	B3@199' ☐				- no recovery	50 (5")	110.7	19.6

Total depth of boring: 199.5 feet
Fill to 10.5 feet.
Groundwater level not established.
Backfilled with grout.
AC patched.

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

Figure A3,
Log of Boring 3, Page 7 of 7

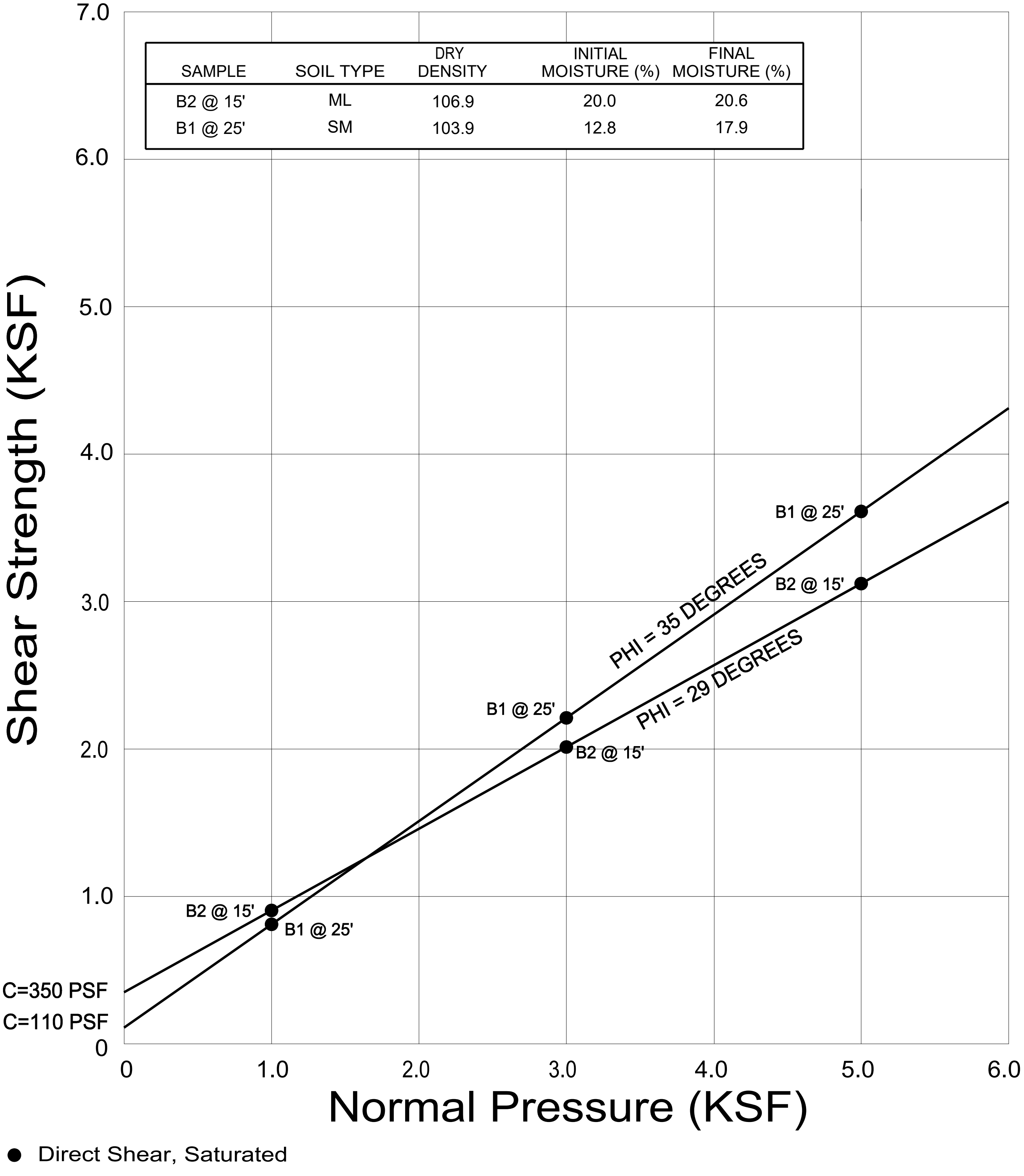
SAMPLE SYMBOLS			
	☐ ... SAMPLING UNSUCCESSFUL	◧ ... STANDARD PENETRATION TEST	■ ... DRIVE SAMPLE (UNDISTURBED)
	☒ ... DISTURBED OR BAG SAMPLE	◪ ... CHUNK SAMPLE	▼ ... WATER TABLE OR SEEPAGE

APPENDIX B

APPENDIX B

LABORATORY TESTING

Laboratory tests were performed in accordance with generally accepted test methods of the "American Society for Testing and Materials (ASTM)", or other suggested procedures. Selected samples were tested for direct shear strength, consolidation characteristics, corrosivity, in-place dry density and moisture content. The results of the laboratory tests are summarized in Figures B1 through B8. The in-place dry density and moisture content of the samples tested are presented on the boring logs, Appendix A.



SAMPLE	SOIL TYPE	DRY DENSITY	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B2 @ 15'	ML	106.9	20.0	20.6
B1 @ 25'	SM	103.9	12.8	17.9

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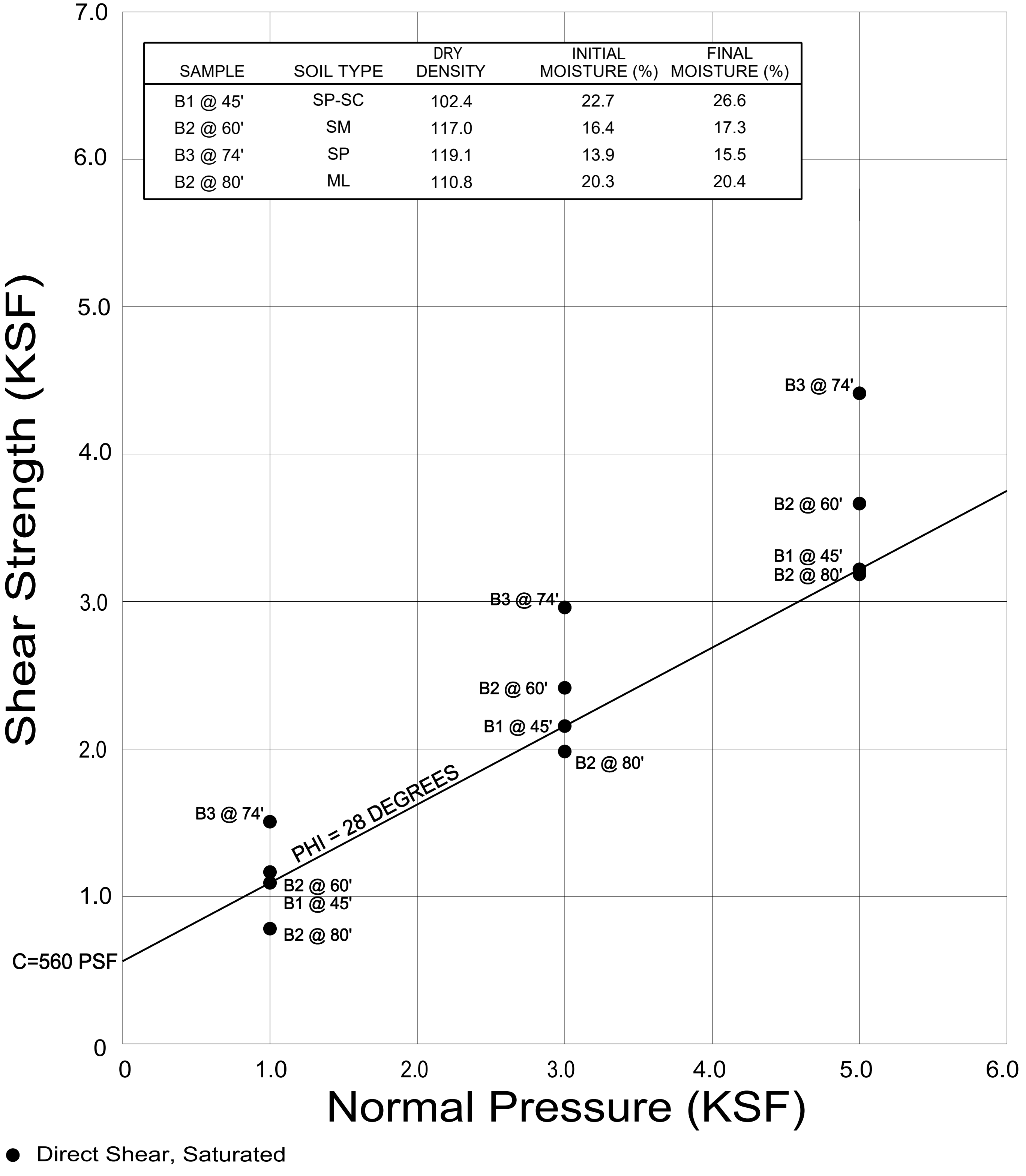
DIRECT SHEAR TEST RESULTS

PROPOSED HIGH RISE

DE LONGPRE AVENUE & VINE STREET
LOS ANGELES, CALIFORNIA

DRAFTED BY: JMT	CHECKED BY: NDB	AUGUST 2020	PROJECT NO. A9382-06-02	FIG. B1



SAMPLE	SOIL TYPE	DRY DENSITY	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B1 @ 45'	SP-SC	102.4	22.7	26.6
B2 @ 60'	SM	117.0	16.4	17.3
B3 @ 74'	SP	119.1	13.9	15.5
B2 @ 80'	ML	110.8	20.3	20.4

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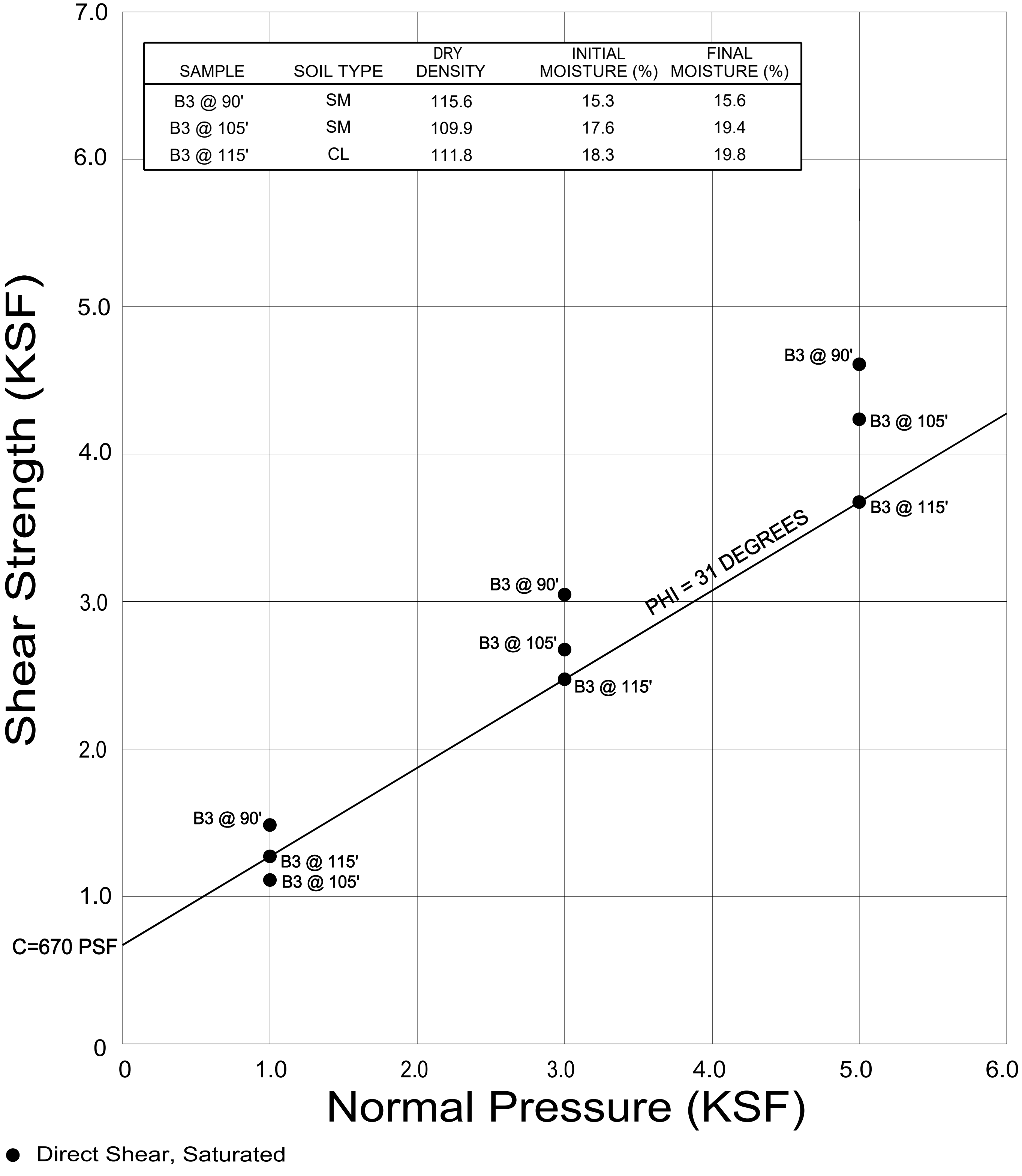
DRAFTED BY: JMT | CHECKED BY: NDB

DIRECT SHEAR TEST RESULTS

PROPOSED HIGH RISE
DE LONGPRE AVENUE & VINE STREET
LOS ANGELES, CALIFORNIA

AUGUST 2020 | PROJECT NO. A9382-06-02 | FIG. B2



SAMPLE	SOIL TYPE	DRY DENSITY	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B3 @ 90'	SM	115.6	15.3	15.6
B3 @ 105'	SM	109.9	17.6	19.4
B3 @ 115'	CL	111.8	18.3	19.8

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DRAFTED BY: PZ | CHECKED BY: NDB

DIRECT SHEAR TEST RESULTS

PROPOSED HIGH RISE
DE LONGPRE AVENUE & VINE STREET
LOS ANGELES, CALIFORNIA

AUGUST 2020 | PROJECT NO. A9382-06-02 | FIG. B3

WATER ADDED AT 2 KSF

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DRAFTED BY: JMT | CHECKED BY: NDB

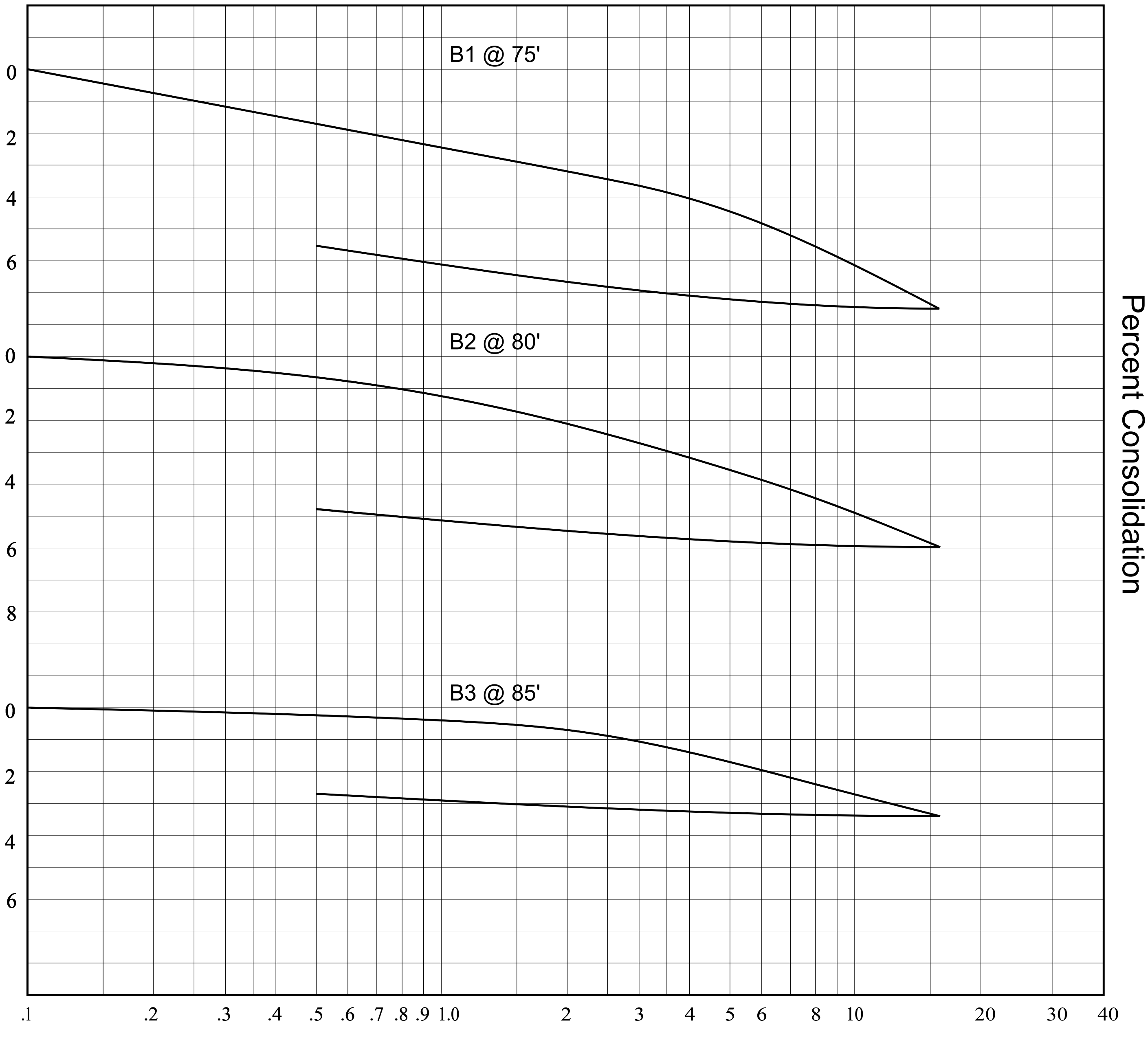
CONSOLIDATION TEST RESULTS

PROPOSED HIGH RISE

DE LONGPRE AVENUE & VINE STREET
LOS ANGELES, CALIFORNIA

AUGUST 2020 | PROJECT NO. A9382-06-02 | FIG. B4

WATER ADDED AT 2 KSF



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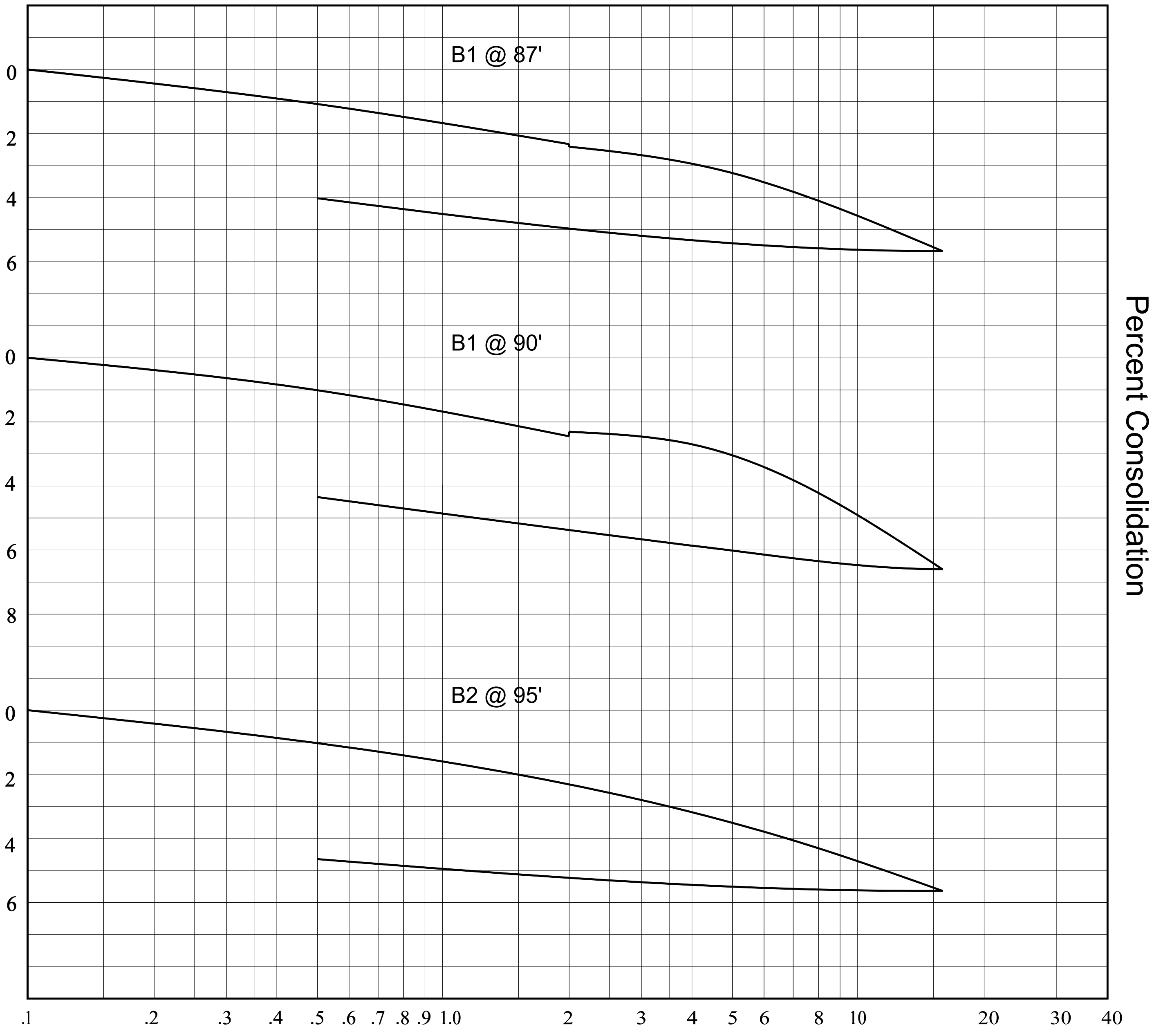
DRAFTED BY: JMT | CHECKED BY: NDB

CONSOLIDATION TEST RESULTS

PROPOSED HIGH RISE
DE LONGPRE AVENUE & VINE STREET
LOS ANGELES, CALIFORNIA

AUGUST 2020 | PROJECT NO. A9382-06-02 | FIG. B5

WATER ADDED AT 2 KSF



Consolidation Pressure (KSF)

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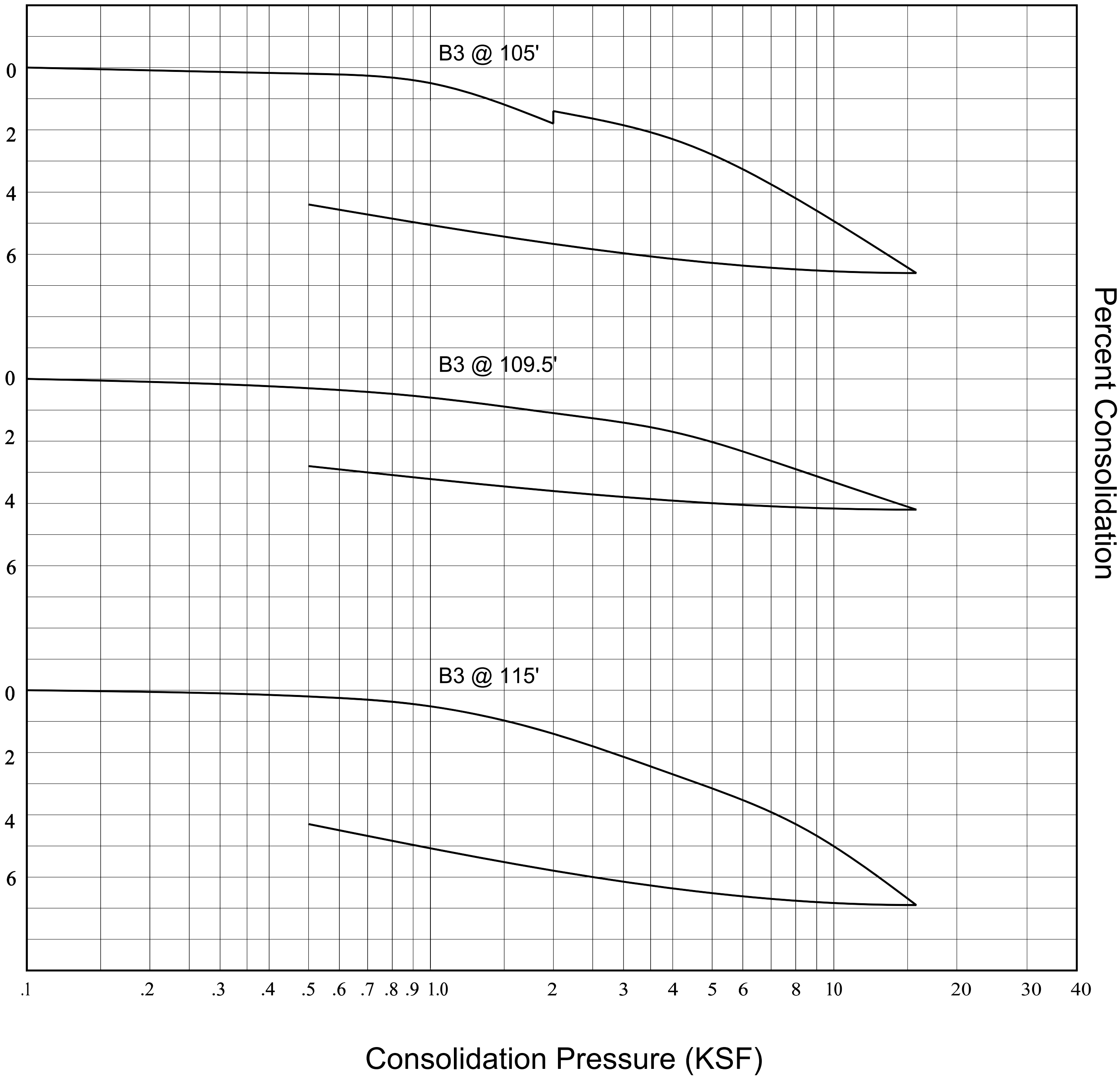
CONSOLIDATION TEST RESULTS

PROPOSED HIGH RISE

DE LONGPRE AVENUE & VINE STREET
LOS ANGELES, CALIFORNIA

DRAFTED BY: JMT	CHECKED BY: NDB	AUGUST 2020	PROJECT NO. A9382-06-02	FIG. B6

WATER ADDED AT 2 KSF



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CONSOLIDATION TEST RESULTS

PROPOSED HIGH RISE

DE LONGPRE AVENUE & VINE STREET
LOS ANGELES, CALIFORNIA

DRAFTED BY: JMT	CHECKED BY: NDB	AUGUST 2020	PROJECT NO. A9382-06-02	FIG. B7

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

Sample No.	**pH**	**Resistivity (ohm centimeters)**
B1 @ 59'	7.52	1400 (Corrosive)
B3 @ 80'	8.05	4600 (Moderately Corrosive)

SUMMARY OF LABORATORY CHLORIDE CONTENT TEST RESULTS
EPA NO. 325.3

Sample No.	Chloride Ion Content (%)
B1 @ 59'	0.012
B3 @ 80'	0.004

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

Sample No.	Water Soluble Sulfate (% SO_4)	Sulfate Exposure*
B1 @ 59'	0.009	S0
B3 @ 80'	0.004	S0

* Reference: 2019 California Building Code, Section 1904.3 and ACI 318 Table 19.3.1.1

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CORROSIVITY TEST RESULTS

PROPOSED HIGH RISE
DE LONGPRE AVENUE & VINE STREET
LOS ANGELES, CALIFORNIA

DRAFTED BY: JMT	CHECKED BY: NDB	AUGUST 2020	PROJECT NO. A9382-06-02	FIG. B8

APPENDIX C

BOREHOLE GEOPHYSICS

1360 VINE STREET

LOS ANGELES, CALIFORNIA

Prepared for

Geocon West. Inc.
3303 N. San Fernando Blvd, Suite 100
Burbank, CA 91504

Prepared by

GEOVision Geophysical Services
1124 Olympic Drive
Corona, California 92881
(951) 549-1234

July 17, 2020

Report 20202-01 rev 0

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APPENDICES

APPENDIX A **SUSPENSION VELOCITY MEASUREMENT QUALITY ASSURANCE SUSPENSION SOURCE TO RECEIVER ANALYSIS RESULTS**

APPENDIX B **GEOPHYSICAL LOGGING SYSTEMS - NIST TRACEABLE CALIBRATION RECORDS**

INTRODUCTION

GEO*Vision* acquired geophysical data in one borehole for the 1360 Vine Street Project in Los Angeles, California on June 24, 2020. The work was performed for Geocon West. A **GEO***Vision* professional Geophysicist or Engineer reviewed fieldwork, data analysis, and report.

SCOPE OF WORK

This report presents the results of geophysical data acquired in one borehole on June 24, 2020, as detailed in Table 1. The purpose of these measurements was to supplement data obtained during the drilling investigation by acquiring shear wave and compressional wave velocities as a function of depth.

An OYO PS Suspension Logging System was used to obtain in-situ horizontal shear (S_H), and compressional (P) wave velocity measurements in one borehole at 1.6-foot intervals. Measurements followed **GEO***Vision* Procedure for PS Suspension Seismic Velocity Logging, revision 1.5. Acquired data were analyzed and a profile of velocity versus depth was produced for both S_H and P waves.

A detailed reference for the PS Suspension velocity measurement techniques used in this study is:

> Guidelines for Determining Design Basis Ground Motions, Report TR-102293, Electric Power Research Institute, Palo Alto, California, November 1993, Sections 7 and 8.

INSTRUMENTATION

Suspension Velocity Instrumentation

Suspension velocity measurements were performed using the PS Suspension logging system, manufactured by OYO Corporation, and their subsidiary, Robertson Geo (RG). This system directly determines the average velocity of a 3.3-foot high segment of the soil column surrounding the borehole of interest by measuring the elapsed time between arrivals of a wave propagating upward through the soil column. The receivers that detect the wave, and the source that generates the wave, are moved as a unit in the borehole, producing relatively constant amplitude signals at all depths.

The suspension system probe consists of a combined reversible polarity solenoid horizontal shear-wave source and compressional-wave source, joined to two biaxial receivers by a flexible isolation cylinder, as shown in Figure 1. The separation of the two receivers is 3.3 feet, allowing average wave velocity in the region between the receivers to be determined by inversion of the wave travel time between the two receivers. The total length of the probe in these surveys is approximately 25 feet, with the center point of the receiver pair 12.5 feet above the bottom end of the probe.

The probe receives control signals from, and sends the digitized receiver signals to, the instrumentation on the surface via an armored multi-conductor cable. The cable is wound onto the drum of a winch and is used to support the probe. Cable travel is measured to provide probe depth data using a sheave of known circumference fitted with a digital rotary encoder.

The entire probe is suspended in the borehole by the cable; therefore, source motion is not coupled directly to the borehole walls; rather, the source motion creates a horizontally propagating impulsive pressure wave in the fluid filling the borehole and surrounding the source. This pressure wave is converted to P and S_H-waves in the surrounding soil and rock as it impinges upon the wall of the borehole. These waves propagate through the soil and rock surrounding the borehole, in turn causing a pressure wave to be generated in the fluid surrounding the receivers as the soil waves pass their location. Separation of the P and S_H-waves at the receivers is performed using the following steps:

1. The orientation of the horizontal receivers is maintained parallel to the axis of the source, maximizing the amplitude of the recorded S_H -wave signals.
2. At each depth, S_H-wave signals are recorded with the source actuated in opposite directions, producing S_H-wave signals of opposite polarity, providing a characteristic S_H-wave signature distinct from the P-wave signal.
3. The 6.3-foot separation of source and receiver 1 permits the P-wave signal to pass and damp significantly before the slower S_H-wave signal arrives at the receiver. In faster soils or rock, the isolation cylinder is extended to allow greater separation of the P- and S_H-wave signals.
4. In saturated soils, the received P-wave signal is typical of much higher frequency than the received S_H-wave signal, permitting additional separation of the two signals by low pass filtering.
5. Direct arrival of the original pressure pulse in the fluid is not detected at the receivers because the wavelength of the pressure pulse in the fluid is significantly greater than the dimension of the fluid annulus surrounding the probe (feet versus inches scale), preventing significant energy transmission through the fluid medium.

In operation, a distinct, repeatable pattern of impulses is generated at each depth as follows:

1. The source is fired in one direction producing dominantly horizontal shear with some vertical compression, and the signals from the horizontal receivers situated parallel to the axis of motion of the source are recorded.
2. The source is fired again in the opposite direction and the horizontal receiver signals are recorded.
3. The source is fired again and the vertical receiver signals are recorded. The repeated source pattern facilitates the picking of the P and S_H-wave arrivals; reversal of the source changes the polarity of the S_H-wave pattern but not the P-wave pattern.

The data from each receiver during each source activation is recorded as a different channel on the recording system. The PS Suspension system has six channels (two simultaneous recording channels), each with a 1024 sample record. The recorded data are displayed as six channels with a common time scale.

A review of the displayed data on the recorder or computer screen allows the operator to set the gains, filters, delay time, pulse length (energy), and sample rate to optimize the quality of the data before recording. Verification of the calibration of the PS Suspension digital recorder is performed at least every twelve months using a NIST traceable frequency source and counter, as presented in Appendix B.

MEASUREMENT PROCEDURES

Suspension Velocity Measurement Procedures

The boreholes were logged uncased and filled with fluid. Measurements followed the **GEO***Vision* Procedure for PS Suspension Seismic Velocity Logging, revision 1.5. Prior to the logging run, the probe was positioned with the top of the probe even with a stationary reference point. The electronic depth counter was set to the distance between the mid-point of the receiver and the top of the probe, minus the height of the stationary reference point, if any. Measurements were verified with a tape measure, and calculations recorded on a field log.

The probe was lowered to the bottom of the borehole, stopping at 1.6-foot intervals to collect data, as summarized in Table 2. At each measurement depth, the measurement sequence of two opposite horizontal records and one vertical record was performed. Gains were adjusted as required. The data from each depth were viewed on the computer display, checked, and saved to disk before moving to the next depth.

Upon completion of the measurements, the probe was returned to the surface, and the zero-depth indication at the depth reference point was verified prior to removal from the borehole.

DATA ANALYSIS

Suspension Velocity Analysis

Recorded digital waveforms were analyzed to locate the most prominent first minima, first maxima, or the first break on the vertical axis records, indicating the arrival of P-wave energy. The difference in travel time between receiver 1 and receiver 2 (R1-R2) arrivals was used to calculate the P-wave velocity for that 1.0-meter segment of the soil column. When observable, P-wave arrivals on the horizontal axis records were used to verify the velocities determined from the vertical axis data. The time picks were then transferred into a template to complete the velocity calculations based on the arrival time picks made in PSLOG. The Microsoft Excel® analysis file accompanies this report.

P-wave velocity over the 6.3-foot interval from source to receiver 1 (S-R1) was also picked, calculated, and plotted for quality assurance of the velocity derived from the travel time between receivers. In this analysis, the depth values as recorded were increased by 4.8 feet to correspond to the mid-point of the 6.33-foot S-R1 interval. Travel times were obtained by picking the first break of the P-wave signal at receiver 1 and subtracting the calculated and experimentally verified delay, in milliseconds, from source trigger pulse (beginning of record) to source impact. This delay corresponds to the duration of the acceleration of the solenoid before the impact.

As with the P-wave records, the recorded digital waveforms were analyzed to locate clear S_H-wave pulses, as indicated by the presence of opposite polarity pulses on each pair of horizontal records. Ideally, the S_H-wave signals from the 'normal' and 'reverse' source pulses are very nearly inverted images of each other. Digital Fast Fourier Transform – Inverse Fast Fourier Transform (FFT – IFFT) lowpass filtering was used to remove the higher frequency P-wave signal from the S_H-wave signal. Different filter cutoffs were used to separate P- and S_H-waves at different depths, ranging from 600 Hz in the slowest zones to 4000 Hz in the regions of highest velocity. At each depth, the filter frequency was selected to be at least twice the fundamental frequency of the S_H-wave signal being filtered.

Generally, the first maxima were picked for the 'normal' signals and the first minima for the 'reverse' signals, although other points on the waveform were used if the first pulse was distorted. The absolute arrival time of the 'normal' and 'reverse' signals may vary by +/- 0.2 milliseconds, due to differences in the actuation time of the solenoid source caused by constant mechanical bias in the source, or by borehole inclination. This variation does not affect the R1-R2 velocity determinations, as the differential time is measured between arrivals of waves created by the same source actuation. The final velocity value is the average of the values obtained from the 'normal' and 'reverse' source actuation.

As with the P-wave data, S_H-wave velocity calculated from the travel time over the 6.33-foot interval from source to receiver 1 was calculated and plotted for verification of the velocity derived from the travel time between receivers. In this analysis, the depth values were increased by 4.8 feet to correspond to the mid-point of the 6.33-foot S-R1 interval. Travel times were obtained by picking the first break of the S_H-wave signal at the near receiver and subtracting the calculated and experimentally verified delay, in milliseconds, from the beginning of the record at the source trigger pulse to source impact.

Poisson’s Ratio, **ν**, was calculated using the following formula:

$$\boldsymbol{\nu} = \frac{\left(\frac{V_s}{V_p}\right)^2 - 0.5}{\left(\frac{V_s}{V_p}\right)^2 - 1.0}$$

Figure 2 shows an example of R1 - R2 measurements on a sample filtered suspension record. In Figure 2, the time difference over the 3.3-foot interval of 1.88 milliseconds for the horizontal signals is equivalent to an S_H-wave velocity of 1745 feet/second. Whenever possible, time differences were determined from several phase points on the S_H-waveform records to verify the data obtained from the first arrival of the S_H-wave pulse. Figure 3 displays the same record before filtering the S_H-waveform record with a 1400 Hz FFT - IFFT digital lowpass filter, illustrating the presence of higher frequency P-wave energy at the beginning of the record, and distortion of the lower frequency S_H-wave by the residual P-wave signal.

Data and analyses were reviewed by a **GEO***Vision* Professional Geophysicist or Engineer as a component of the in-house data validation program.

Vs30 Analysis

The average shear wave velocity in the upper 30 meters (Vs30) was calculated using the NEHRP method. The PS Suspension logger measures directly the travel time over a 1 meter interval. However, data are logged at ½ meter intervals. The overlapped measurements (at nominal 0.5m intervals for your data) are overlapping travel times. These are then used to calculate the interval times, which are then accumulated to obtain the total travel time over 30 meters. Vs30 is 30 meters divided by this total travel time.

RESULTS

Suspension Velocity Results

Suspension R1-R2 P- and S_H-wave velocities for borehole B3 are plotted in Figure 4 and data are compiled in Table 3. The associated Microsoft Excel® analysis files accompany this report. Included in the Microsoft Excel® analysis files are Poisson's Ratio calculations, tabulated data, and plots.

P- and S_H-wave velocity data from R1-R2 analysis and quality assurance analysis of S-R1 data are plotted together in Figure A-1 in Appendix A to aid in visual comparison. Note that R1-R2 data are an average velocity over a 3.3-foot segment of the soil column; S-R1 data are an average over 6.3 feet, creating a significant smoothing relative to the R1-R2 plots. The S-R1 velocity data displayed in this figure are also compiled in Table A-1.

Vs30 Results

The Vs30 estimate for borehole B3 is 340 meters/second, or NEHRP site class D; stiff soil.

* Site Classifications taken from Table 1615 1.1 Site Class Definitions published in 2000 International Building code, International Code Council, Inc. on page 350

SUMMARY

Discussion of Suspension Velocity Results

PS Suspension velocity data for this project were collected in uncased, fluid-filled boreholes.

	Criteria	B3
1	Consistent data between receiver to receiver (R1 – R2) and source to receiver (S – R1) data.	Yes
2	Consistency between data from adjacent depth intervals.	Yes
3	Consistent relationship between P-wave and S_H -wave (excluding transition to saturated soils)	Yes Saturation occurs at about 45ft BGS
4	Clarity of P-wave and S_H-wave onset, as well as damping of later oscillations.	This is good data
5	Consistency of profile between adjacent borings, if available.	N/A

Quality Assurance

These borehole geophysical measurements were performed using industry-standard or better methods for measurements and analysis. All work was performed under **GEO***Vision* quality assurance procedures, which include:

- Use of NIST-traceable calibrations, where applicable, for field and laboratory instrumentation
- Use of standard field data logs
- Use of independent verification of velocity data by comparison of receiver-to-receiver and source-to-receiver velocities
- Independent review of calculations and results by a registered professional engineer, geologist, or geophysicist.

Suspension Velocity Data Reliability

P- and S_H-wave velocity measurement using the PS suspension method gives average velocities over a 3.3-foot interval of depth. This high resolution results in the scatter of values shown in the graphs. Individual measurements are very reliable, with an estimated precision of +/- 5%. Depth indications are very reliable with an estimated precision of +/- 0.2 feet. Standardized field procedures and quality assurance checks contribute to the reliability of these data.

CERTIFICATION

All geophysical data, analysis, interpretations, conclusions, and recommendations in this document have been prepared under the supervision of and reviewed by a **GEO***Vision* California Professional Geophysicist or Engineer.

Prepared by:

7/17/2020

Emily Feldman
Project Geophysicist
GEO*Vision* Geophysical Services

Date

Reviewed and approved by

VICTOR GONZALEZ
PGp 1074

7/17/2020

Victor M Gonzalez
California Professional Geophysicist, PGp 1074
GEO*Vision* Geophysical Services

Date

* This geophysical investigation was conducted under the supervision of a California Professional Geophysicist using industry-standard methods and equipment. A high degree of professionalism was maintained during all aspects of the project from the field investigation and data acquisition through data processing, interpretation and reporting. All original field data files, field notes and observations, and other pertinent information are maintained in the project files and are available for the client to review for a period of at least one year.

A professional geophysicist's certification of interpreted geophysical conditions comprises a declaration of his/her professional judgment. It does not constitute a warranty or guarantee, expressed or implied, nor does it relieve any other party of its responsibility to abide by contract documents, applicable codes, standards, regulations, or ordinances.

Table 1. Borehole locations and logging dates

BOREHOLE DESIGNATION	DATES LOGGED	COORDINATES [(1)] (US Survey Feet)		
		Northing	Easting	Elevation (ft)
B3	6/24/2020	-	-	-

(1) Coordinates not available

Table 2. Logging dates and depth ranges

BOREHOLE NUMBER	TOOL AND RUN NUMBER	DEPTH RANGE (FEET)	OPEN HOLE (FEET)	SAMPLE INTERVAL (FEET)	DATE LOGGED
B3	SUSPENSION DOWN01	11.48 – 187.01	200	1.6	6/24/2020

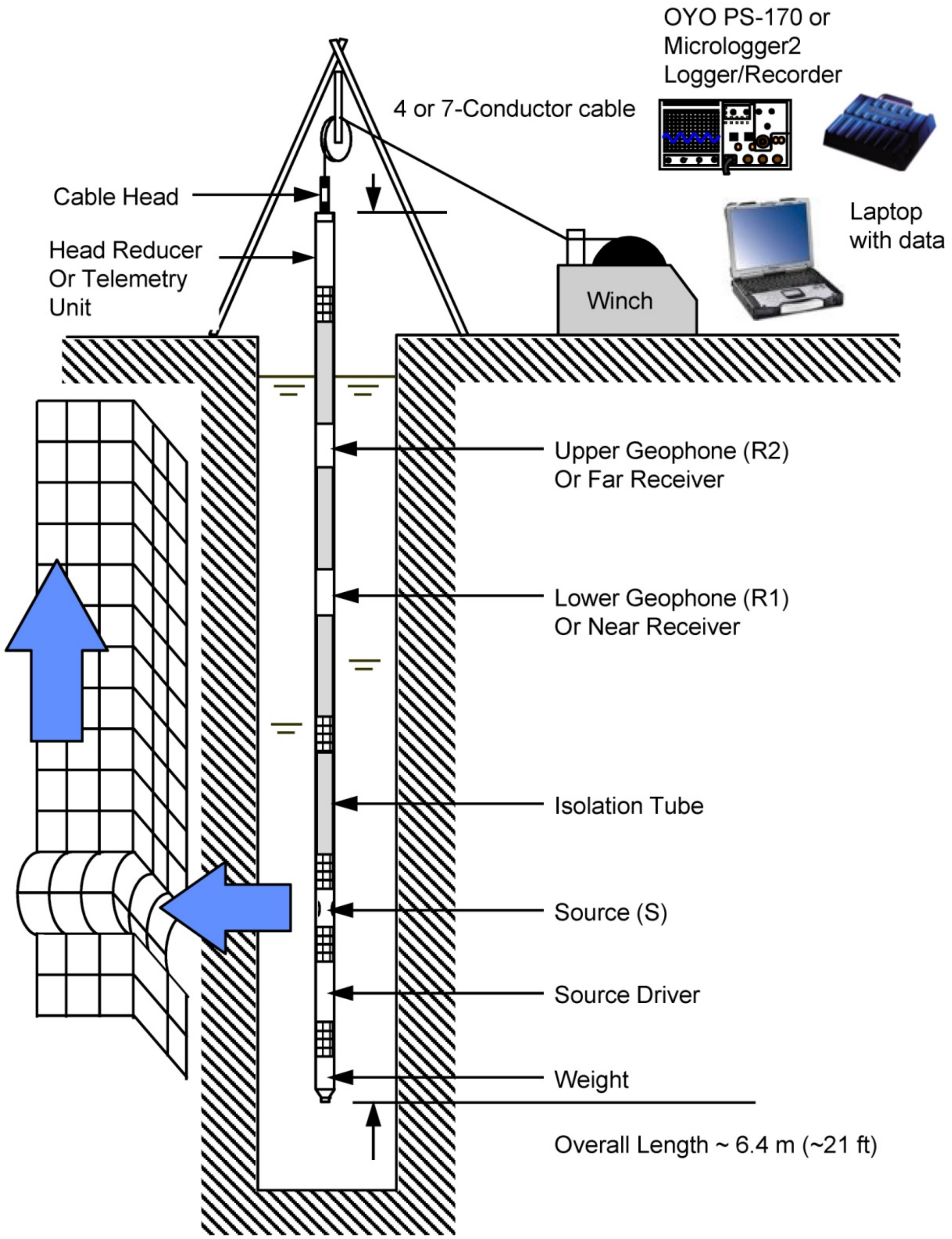

Figure 1: Concept illustration of PS logging system

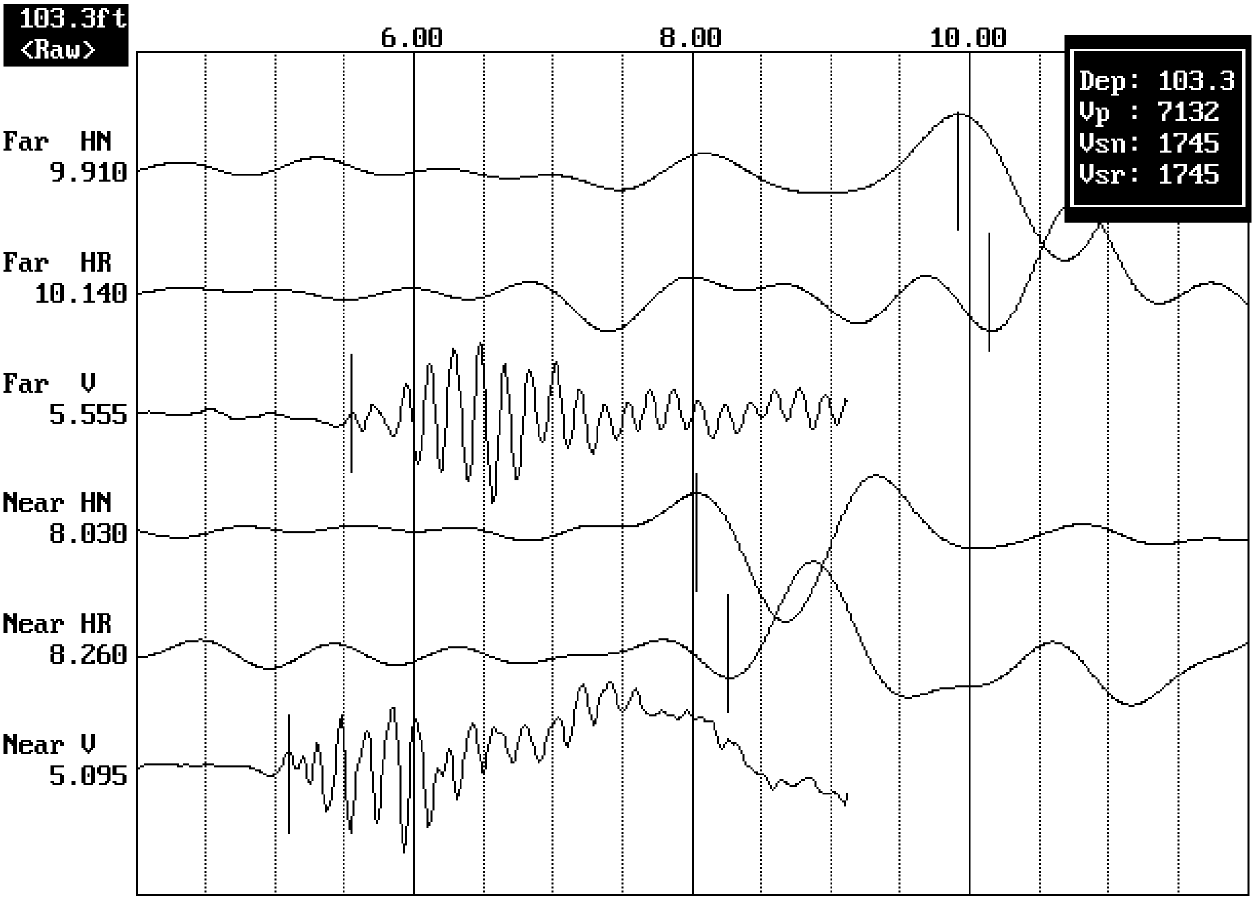

Figure 2: Example of filtered (1400 Hz lowpass) suspension record

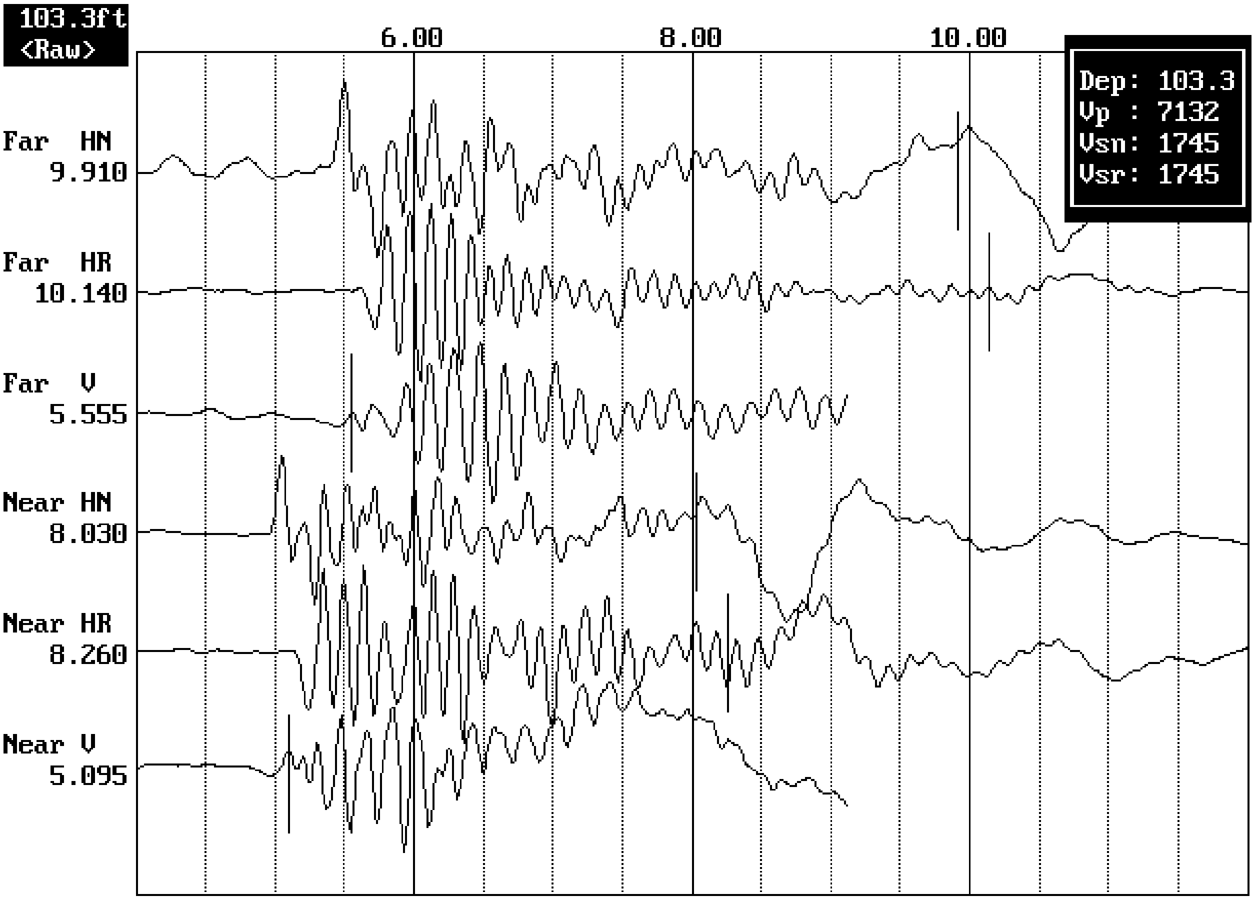

Figure 3. Example of unfiltered suspension record

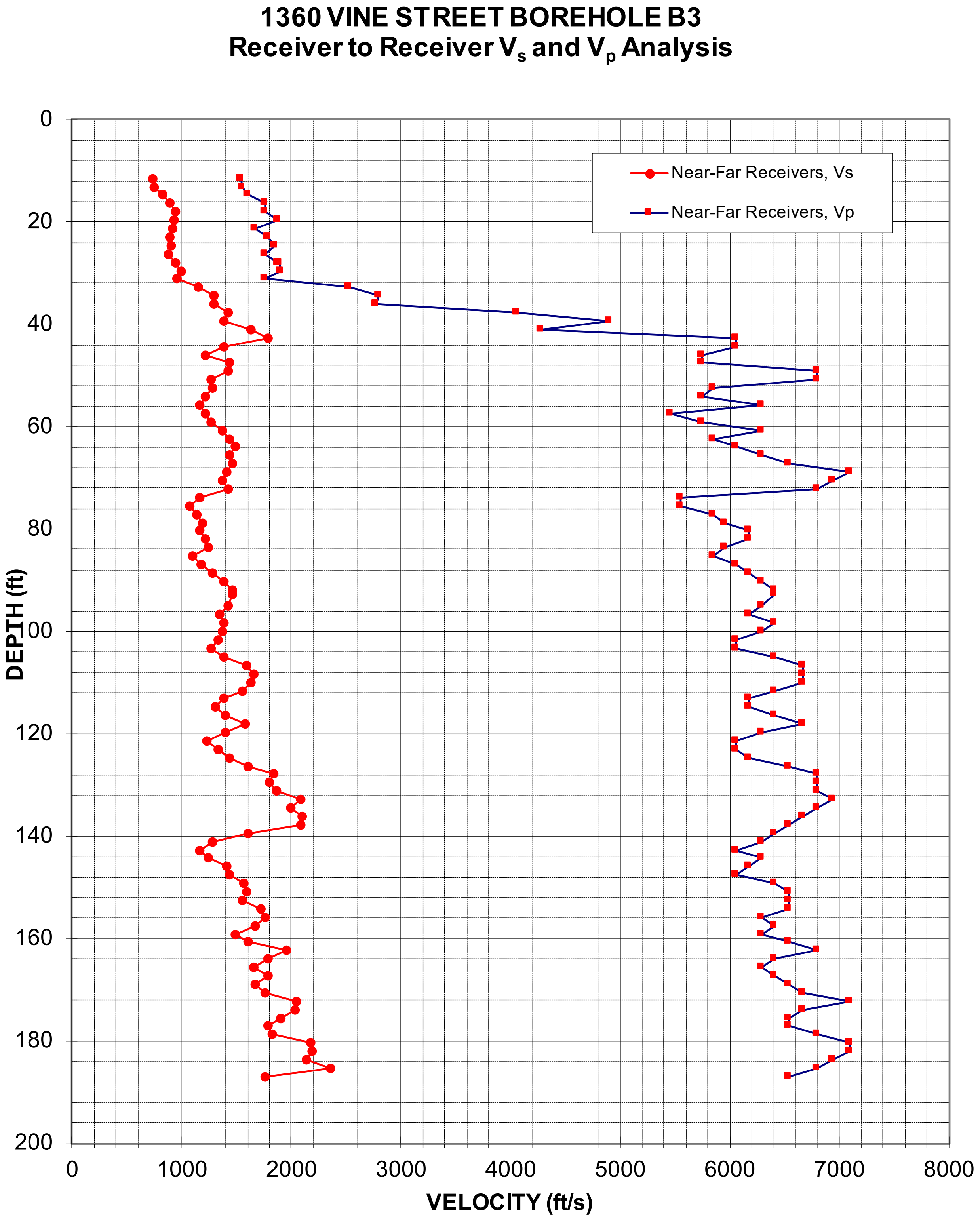

Figure 4: Borehole B3, Suspension R1-R2 P- and S_H-wave velocities

Table 3. Borehole B3, Suspension R1-R2 depths and P- and S_H-wave velocities

Summary of Compressional Wave Velocity, Shear Wave Velocity, and Poisson's Ratio
Based on Receiver-to-Receiver Travel Time Data - Borehole B3

American Units			
Depth at Midpoint Between Receivers	Velocity		Poisson's Ratio
	V_s	V_p	
(ft)	(ft/s)	(ft/s)	
11.5	740	1540	0.35
13.1	750	1560	0.35
14.8	820	1600	0.32
16.4	890	1750	0.33
18.0	940	1750	0.30
19.7	930	1870	0.34
21.3	920	1670	0.28
23.0	890	1790	0.33
24.6	900	1850	0.35
26.3	870	1760	0.34
27.9	940	1870	0.33
27.9	940	1890	0.34
29.5	990	1900	0.31
31.2	960	1750	0.29
32.8	1150	2530	0.37
34.5	1300	2800	0.36
36.1	1290	2780	0.36
37.7	1420	4070	0.43
39.4	1380	4900	0.46
41.0	1630	4270	0.41
42.7	1790	6060	0.45
44.3	1380	6060	0.47
45.9	1210	5750	0.48
47.6	1440	5750	0.47
49.2	1420	6800	0.48
50.9	1270	6800	0.48
52.5	1280	5850	0.47
54.1	1220	5750	0.48
55.8	1170	6290	0.48
57.4	1210	5460	0.47
59.1	1270	5750	0.47
60.7	1370	6290	0.48
62.3	1430	5850	0.47
64.0	1490	6060	0.47
65.6	1440	6290	0.47
67.3	1460	6540	0.47
68.9	1410	7090	0.48
70.5	1370	6940	0.48

Metric Units			
Depth at Midpoint Between Receivers	Velocity		Poisson's Ratio
	V_s	V_p	
(m)	(m/s)	(m/s)	
3.5	230	470	0.35
4.0	230	470	0.35
4.5	250	490	0.32
5.0	270	530	0.33
5.5	290	530	0.30
6.0	280	570	0.34
6.5	280	510	0.28
7.0	270	550	0.33
7.5	270	560	0.35
8.0	270	540	0.34
8.5	290	570	0.33
8.5	290	580	0.34
9.0	300	580	0.31
9.5	290	530	0.29
10.0	350	770	0.37
10.5	400	850	0.36
11.0	390	850	0.36
11.5	430	1240	0.43
12.0	420	1490	0.46
12.5	500	1300	0.41
13.0	550	1850	0.45
13.5	420	1850	0.47
14.0	370	1750	0.48
14.5	440	1750	0.47
15.0	430	2070	0.48
15.5	390	2070	0.48
16.0	390	1780	0.47
16.5	370	1750	0.48
17.0	360	1920	0.48
17.5	370	1670	0.47
18.0	390	1750	0.47
18.5	420	1920	0.48
19.0	440	1780	0.47
19.5	450	1850	0.47
20.0	440	1920	0.47
20.5	450	1990	0.47
21.0	430	2160	0.48
21.5	420	2120	0.48

Summary of Compressional Wave Velocity, Shear Wave Velocity, and Poisson's Ratio Based on Receiver-to-Receiver Travel Time Data - Borehole B3

American Units			
Depth at Midpoint Between Receivers	Velocity		Poisson's Ratio
	V_s	V_p	
(ft)	(ft/s)	(ft/s)	
72.2	1420	6800	0.48
73.8	1160	5560	0.48
75.5	1080	5560	0.48
77.1	1140	5850	0.48
78.7	1190	5950	0.48
80.4	1170	6170	0.48
82.0	1210	6170	0.48
83.7	1240	5950	0.48
85.3	1100	5850	0.48
86.9	1180	6060	0.48
88.6	1280	6170	0.48
90.2	1380	6290	0.47
91.9	1460	6410	0.47
92.9	1460	6410	0.47
95.1	1420	6290	0.47
96.8	1350	6170	0.47
98.4	1390	6410	0.48
100.1	1370	6290	0.48
101.7	1330	6060	0.47
103.4	1270	6060	0.48
105.0	1380	6410	0.48
106.6	1590	6670	0.47
108.3	1650	6670	0.47
109.9	1630	6670	0.47
111.6	1550	6410	0.47
113.2	1390	6170	0.47
114.8	1310	6170	0.48
116.5	1390	6410	0.48
118.1	1570	6670	0.47
119.8	1390	6290	0.47
121.4	1230	6060	0.48
123.0	1330	6060	0.47
124.7	1440	6170	0.47
126.3	1600	6540	0.47
128.0	1840	6800	0.46
129.6	1800	6800	0.46
131.2	1860	6800	0.46
132.9	2080	6940	0.45
134.5	2000	6800	0.45
136.2	2100	6670	0.45

Metric Units			
Depth at Midpoint Between Receivers	Velocity		Poisson's Ratio
	V_s	V_p	
(m)	(m/s)	(m/s)	
22.0	430	2070	0.48
22.5	350	1690	0.48
23.0	330	1690	0.48
23.5	350	1780	0.48
24.0	360	1810	0.48
24.5	360	1880	0.48
25.0	370	1880	0.48
25.5	380	1810	0.48
26.0	330	1780	0.48
26.5	360	1850	0.48
27.0	390	1880	0.48
27.5	420	1920	0.47
28.0	440	1950	0.47
28.3	450	1950	0.47
29.0	430	1920	0.47
29.5	410	1880	0.47
30.0	420	1950	0.48
30.5	420	1920	0.48
31.0	400	1850	0.47
31.5	390	1850	0.48
32.0	420	1950	0.48
32.5	490	2030	0.47
33.0	500	2030	0.47
33.5	500	2030	0.47
34.0	470	1950	0.47
34.5	420	1880	0.47
35.0	400	1880	0.48
35.5	430	1950	0.48
36.0	480	2030	0.47
36.5	430	1920	0.47
37.0	370	1850	0.48
37.5	410	1850	0.47
38.0	440	1880	0.47
38.5	490	1990	0.47
39.0	560	2070	0.46
39.5	550	2070	0.46
40.0	570	2070	0.46
40.5	640	2120	0.45
41.0	610	2070	0.45
41.5	640	2030	0.45

Summary of Compressional Wave Velocity, Shear Wave Velocity, and Poisson's Ratio Based on Receiver-to-Receiver Travel Time Data - Borehole B3

American Units			
Depth at Midpoint Between Receivers	**Velocity**		**Poisson's Ratio**
	V_s	**V_p**	
(ft)	(ft/s)	(ft/s)	
137.8	2080	6540	0.44
139.4	1600	6410	0.47
141.1	1280	6290	0.48
142.7	1170	6060	0.48
144.4	1240	6290	0.48
146.0	1410	6170	0.47
147.6	1430	6060	0.47
149.3	1560	6410	0.47
150.9	1590	6540	0.47
152.6	1550	6540	0.47
154.2	1720	6540	0.46
155.8	1750	6290	0.46
157.5	1680	6410	0.46
159.1	1480	6290	0.47
160.8	1600	6540	0.47
162.4	1960	6800	0.45
164.0	1780	6410	0.46
165.7	1660	6290	0.46
167.3	1780	6410	0.46
169.0	1680	6540	0.46
170.6	1760	6670	0.46
172.2	2040	7090	0.45
173.9	2030	6670	0.45
175.5	1900	6540	0.45
177.2	1790	6540	0.46
178.8	1830	6800	0.46
180.5	2180	7090	0.45
182.1	2190	7090	0.45
183.7	2140	6940	0.45
185.4	2360	6800	0.43
187.0	1750	6540	0.46

Metric Units			
Depth at Midpoint Between Receivers	**Velocity**		**Poisson's Ratio**
	V_s	**V_p**	
(m)	(m/s)	(m/s)	
42.0	640	1990	0.44
42.5	490	1950	0.47
43.0	390	1920	0.48
43.5	360	1850	0.48
44.0	380	1920	0.48
44.5	430	1880	0.47
45.0	440	1850	0.47
45.5	480	1950	0.47
46.0	480	1990	0.47
46.5	470	1990	0.47
47.0	520	1990	0.46
47.5	530	1920	0.46
48.0	510	1950	0.46
48.5	450	1920	0.47
49.0	490	1990	0.47
49.5	600	2070	0.45
50.0	540	1950	0.46
50.5	510	1920	0.46
51.0	540	1950	0.46
51.5	510	1990	0.46
52.0	540	2030	0.46
52.5	620	2160	0.45
53.0	620	2030	0.45
53.5	580	1990	0.45
54.0	550	1990	0.46
54.5	560	2070	0.46
55.0	660	2160	0.45
55.5	670	2160	0.45
56.0	650	2120	0.45
56.5	720	2070	0.43
57.0	530	1990	0.46

APPENDIX A

SUSPENSION VELOCITY MEASUREMENT QUALITY ASSURANCE SUSPENSION SOURCE TO RECEIVER ANALYSIS RESULTS

1360 VINE STREET BOREHOLE B3
Source to Receiver and Receiver to Receiver Analysis

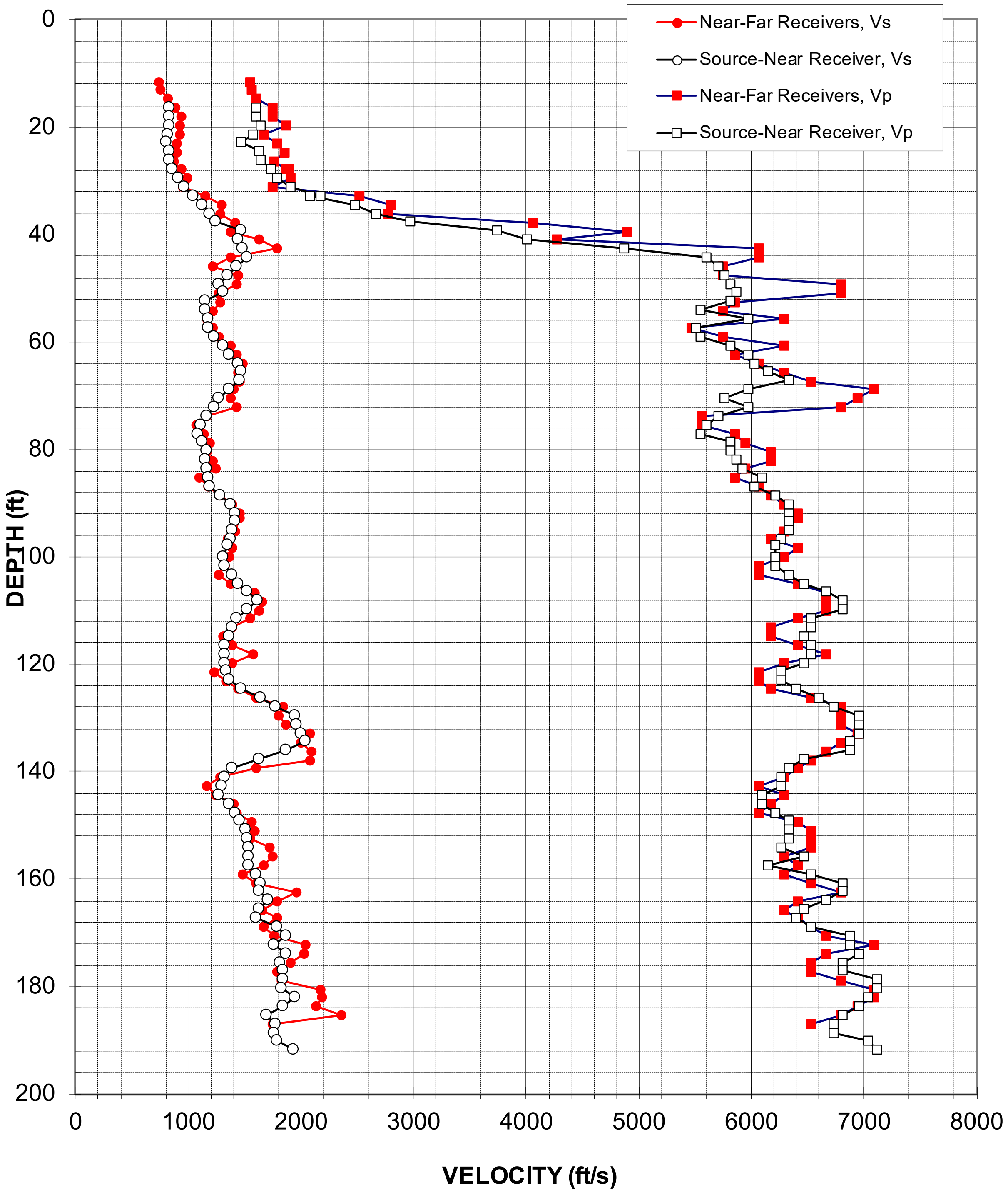

Figure A-1: Borehole B3, Suspension S-R1 P- and S_H-wave velocities

Table A-1. Borehole B3, S - R1 quality assurance analysis P- and S_H-wave data

Summary of Compressional Wave Velocity, Shear Wave Velocity, and Poisson's Ratio
Based on Source-to-Receiver Travel Time Data - Borehole B3

American Units				Metric Units			
Depth at Midpoint Between Source and Near Receiver	Velocity		Poisson's Ratio	Depth at Midpoint Between Source and Near Receiver	Velocity		Poisson's Ratio
	V_s	V_p			V_s	V_p	
(ft)	(ft/s)	(ft/s)		(m)	(m/s)	(m/s)	
16.3	840	1600	0.31	5.0	250	490	0.31
18.0	840	1600	0.31	5.5	250	490	0.31
19.6	840	1640	0.32	6.0	250	500	0.32
21.2	820	1570	0.31	6.5	250	480	0.31
22.9	810	1470	0.28	7.0	250	450	0.28
24.5	830	1620	0.32	7.5	250	490	0.32
26.2	840	1640	0.32	8.0	250	500	0.32
27.8	860	1730	0.33	8.5	260	530	0.33
29.4	910	1790	0.33	9.0	280	550	0.33
31.1	970	1910	0.33	9.5	290	580	0.33
32.7	1040	2180	0.35	10.0	320	660	0.35
32.7	1040	2080	0.33	10.0	320	630	0.33
34.4	1130	2480	0.37	10.5	340	760	0.37
36.0	1200	2670	0.37	11.0	360	810	0.37
37.6	1240	2970	0.39	11.5	380	910	0.39
39.3	1470	3750	0.41	12.0	450	1140	0.41
40.9	1440	4010	0.43	12.5	440	1220	0.43
42.6	1480	4870	0.45	13.0	450	1480	0.45
44.2	1520	5600	0.46	13.5	460	1710	0.46
45.8	1440	5700	0.47	14.0	440	1740	0.47
47.5	1340	5750	0.47	14.5	410	1750	0.47
49.1	1270	5810	0.47	15.0	390	1770	0.47
50.8	1310	5860	0.47	15.5	400	1790	0.47
52.4	1160	5810	0.48	16.0	350	1770	0.48
54.0	1150	5550	0.48	16.5	350	1690	0.48
55.7	1180	5970	0.48	17.0	360	1820	0.48
57.3	1170	5500	0.48	17.5	360	1680	0.48
59.0	1240	5550	0.47	18.0	380	1690	0.47
60.6	1310	5810	0.47	18.5	400	1770	0.47
62.2	1360	5970	0.47	19.0	410	1820	0.47
63.9	1440	6030	0.47	19.5	440	1840	0.47
65.5	1470	6150	0.47	20.0	450	1870	0.47
67.2	1450	6330	0.47	20.5	440	1930	0.47
68.8	1360	5970	0.47	21.0	420	1820	0.47
70.5	1270	5750	0.47	21.5	390	1750	0.47
72.1	1230	5970	0.48	22.0	370	1820	0.48

Summary of Compressional Wave Velocity, Shear Wave Velocity, and Poisson's Ratio Based on Source-to-Receiver Travel Time Data - Borehole B3

American Units			
Depth at Midpoint Between Source and Near Receiver	Velocity		Poisson's Ratio
	V_s	V_p	
(ft)	(ft/s)	(ft/s)	
73.7	1160	5700	0.48
75.4	1110	5600	0.48
77.0	1090	5550	0.48
78.7	1130	5810	0.48
80.3	1160	5810	0.48
81.9	1150	5860	0.48
83.6	1160	5920	0.48
85.2	1180	6090	0.48
86.9	1190	6030	0.48
88.5	1290	6210	0.48
90.1	1380	6330	0.47
91.8	1410	6330	0.47
93.4	1420	6330	0.47
95.1	1390	6330	0.47
96.7	1380	6270	0.47
97.7	1350	6210	0.48
100.0	1310	6210	0.48
101.6	1320	6210	0.48
103.3	1390	6330	0.47
104.9	1450	6460	0.47
106.5	1520	6660	0.47
108.2	1610	6810	0.47
109.8	1530	6810	0.47
111.5	1430	6530	0.47
113.1	1390	6530	0.48
114.7	1360	6460	0.48
116.4	1320	6530	0.48
118.0	1320	6530	0.48
119.7	1320	6460	0.48
121.3	1340	6270	0.48
122.9	1360	6270	0.48
124.6	1470	6390	0.47
126.2	1640	6590	0.47
127.9	1780	6730	0.46
129.5	1940	6960	0.46
131.1	1970	6960	0.46
132.8	2000	6960	0.45
134.4	2040	6880	0.45
136.1	1860	6880	0.46
137.7	1630	6460	0.47

Metric Units			
Depth at Midpoint Between Source and Near Receiver	Velocity		Poisson's Ratio
	V_s	V_p	
(m)	(m/s)	(m/s)	
22.5	350	1740	0.48
23.0	340	1710	0.48
23.5	330	1690	0.48
24.0	340	1770	0.48
24.5	350	1770	0.48
25.0	350	1790	0.48
25.5	350	1800	0.48
26.0	360	1860	0.48
26.5	360	1840	0.48
27.0	390	1890	0.48
27.5	420	1930	0.47
28.0	430	1930	0.47
28.5	430	1930	0.47
29.0	420	1930	0.47
29.5	420	1910	0.47
29.8	410	1890	0.48
30.5	400	1890	0.48
31.0	400	1890	0.48
31.5	420	1930	0.47
32.0	440	1970	0.47
32.5	460	2030	0.47
33.0	490	2070	0.47
33.5	470	2070	0.47
34.0	440	1990	0.47
34.5	420	1990	0.48
35.0	420	1970	0.48
35.5	400	1990	0.48
36.0	400	1990	0.48
36.5	400	1970	0.48
37.0	410	1910	0.48
37.5	420	1910	0.48
38.0	450	1950	0.47
38.5	500	2010	0.47
39.0	540	2050	0.46
39.5	590	2120	0.46
40.0	600	2120	0.46
40.5	610	2120	0.45
41.0	620	2100	0.45
41.5	570	2100	0.46
42.0	500	1970	0.47

Summary of Compressional Wave Velocity, Shear Wave Velocity, and Poisson's Ratio Based on Source-to-Receiver Travel Time Data - Borehole B3

American Units			
Depth at Midpoint Between Source and Near Receiver	Velocity		Poisson's Ratio
	V_s	V_p	
(ft)	(ft/s)	(ft/s)	
139.3	1390	6330	0.47
141.0	1330	6270	0.48
142.6	1300	6270	0.48
144.3	1280	6090	0.48
145.9	1360	6090	0.47
147.6	1420	6210	0.47
149.2	1450	6330	0.47
150.8	1510	6330	0.47
152.5	1530	6330	0.47
154.1	1540	6270	0.47
155.8	1540	6460	0.47
157.4	1540	6150	0.47
159.0	1610	6530	0.47
160.7	1640	6810	0.47
162.3	1620	6810	0.47
164.0	1710	6660	0.46
165.6	1620	6460	0.47
167.2	1600	6390	0.47
168.9	1790	6530	0.46
170.5	1860	6880	0.46
172.2	1760	6880	0.47
173.8	1870	6960	0.46
175.4	1820	6810	0.46
177.1	1840	6810	0.46
178.7	1840	7110	0.46
180.4	1830	7110	0.46
182.0	1950	7030	0.46
183.6	1840	6960	0.46
185.3	1690	6810	0.47
186.9	1780	6730	0.46
188.6	1770	6730	0.46
190.2	1790	7030	0.47
191.8	1930	7110	0.46

Metric Units			
Depth at Midpoint Between Source and Near Receiver	Velocity		Poisson's Ratio
	V_s	V_p	
(m)	(m/s)	(m/s)	
42.5	420	1930	0.47
43.0	410	1910	0.48
43.5	400	1910	0.48
44.0	390	1860	0.48
44.5	420	1860	0.47
45.0	430	1890	0.47
45.5	440	1930	0.47
46.0	460	1930	0.47
46.5	470	1930	0.47
47.0	470	1910	0.47
47.5	470	1970	0.47
48.0	470	1870	0.47
48.5	490	1990	0.47
49.0	500	2070	0.47
49.5	490	2070	0.47
50.0	520	2030	0.46
50.5	490	1970	0.47
51.0	490	1950	0.47
51.5	550	1990	0.46
52.0	570	2100	0.46
52.5	540	2100	0.47
53.0	570	2120	0.46
53.5	550	2070	0.46
54.0	560	2070	0.46
54.5	560	2170	0.46
55.0	560	2170	0.46
55.5	600	2140	0.46
56.0	560	2120	0.46
56.5	520	2070	0.47
57.0	540	2050	0.46
57.5	540	2050	0.46
58.0	550	2140	0.47
58.5	590	2170	0.46

APPENDIX B

BOREHOLE GEOPHYSICAL LOGGING SYSTEMS - NIST TRACEABLE CALIBRATION RECORDS

MICRO PRECISION

MICRO PRECISION CALIBRATION, INC
2165 N. Glassell St.,
Orange, CA 92865
714-901-5659

Certificate of Calibration

Date: May 4, 2020 **Cert No. 551220083593660**

Customer:
GEOVISION
1124 OLYMPIC DRIVE
CORONA CA 92881

Work Order #: LA-90046721
Purchase Order #: 19160-200422-01

MPC Control #: AM6768
Asset ID: 160024
Gage Type: LOGGER
Manufacturer: OYO
Model Number: 3403
Size: N/A
Temp/RH: 22.5°C / 42.9%
Location: Calibration performed at MPC facility

Serial Number: 160024
Department: N/A
Performed By: KYLE ANDERSON
Received Condition: IN TOLERANCE
Returned Condition: IN TOLERANCE
Cal. Date: April 30, 2020
Cal. Interval: 12 MONTHS
Cal. Due Date: April 30, 2021

Calibration Notes:

See attached data sheet for calculations. (1 Page)
Calibrated IAW customer supplied data form Rev 2.1
Frequency measurement uncertainty = 0.0005 Hz
Unit calibrated with Laptop Panasonic Model CF-29,s/n: 6AKSB01291
Calibrated To 4:1 Accuracy Ratio

Calibration performed in accordance with approved GEOVision calibration procedures included in work Instruction No. 06
Software: Geometrics seismodule controller ver 11.0.57, pickwin95.exe ver 3.2.0.1

Standards Used to Calibrate Equipment

I.D.	Description.	Model	Serial	Manufacturer	Cal. Due Date	Traceability #
DB8748	GPS TIME AND FREQUENCY RECEIVER	58503A	3625A01225	HEWLETT PACKARD	Apr 30, 2021	551220083021224
BD7715	UNIVERSAL COUNTER	53131A	3416A05377	HEWLETT PACKARD	Apr 30, 2021	551220082934517
LAS0052	ARB / FUNC GENERATOR	33250A	MY40029031	AGILENT	Oct 31, 2020	551220083302616

Calibrating Technician: KYLE ANDERSON

QC Approval: TYLER MCKEEN

STATEMENTS OF PASS OR FAIL CONFORMANCE: The uncertainty of measurement has been taken into account when determining compliance with specification, as per ILAC-G8:03/2009. All measurements and test results guard banded to ensure the probability of false-accept does not exceed 2% in compliance with ANSI/NCSL Z540.3-2006 and in case without guard banded the probability of false-accept depending on test uncertainty ratio.

THE CALIBRATION REPORT STATUS:
PASS- Term used when compliance statement is given, and the measurement result is PASS.
PASS^{z}- Term used when compliance statement is given, and the measurement result is conditional passed or PASS^{z}.
FAIL- Term used when compliance statement is given, and the measurement result is FAIL.
FAIL^{z}- Term used when compliance statement is given, and the measurement result is conditional failed or FAIL^{z}.
REPORT OF VALUE - Term used when reported measurement is not requiring compliance statement in report.
ADJUSTED- When adjustments are made to an instrument which changes the value of measurement from what was measured as found to new value as left.
LIMITED - When an instrument fails calibration but is still functional in a limited manner.

The expanded uncertainty of measurement is stated as the standard uncertainty of measurement multiplied by the coverage factor k=2, which for a normal distribution corresponds to a coverage probability of approximately 95%, unless otherwise stated. This calibration report complies with ISO/IEC 17025:2017 and ANSI/NCSL Z540.3. Calibration cycles and resulting due dates were submitted/approved by the customer. Any number of factors may cause an instrument to drift out of tolerance before the next scheduled calibration. Recalibration cycles should be based on frequency of use, environmental conditions and customer's established systematic accuracy. All standards are traceable to SI through the National Institute of Standards and Technology (NIST) and/or recognized national or international standards laboratories. Services rendered include proper manufacturer's service instruction and are warranted for no less than thirty (30) days. The information on this report pertains only to the instrument identified, this may not be reproduced in part or in a whole without the prior written approval of the issuing MP Calibration Laboratory.

Certificate of Calibration

Date: May 4, 2020 **Cert No. 551220083593660**

Procedures Used in this Event

Procedure Name	Description
GEOVISION SEISMIC Rev. 2.1	Seismic Logger/Recorder Calibration Procedure, Rev. 2.1

Calibrating Technician:

KYLE ANDERSON

QC Approval:

TYLER MCKEEN

STATEMENTS OF PASS OR FAIL CONFORMANCE: The uncertainty of measurement has been taken into account when determining compliance with specification, as per ILAC-G8:03/2009. All measurements and test results guard banded to ensure the probability of false-accept does not exceed 2% in compliance with ANSI/NCSL Z540.3-2006 and in case without guard banded the probability of false-accept depending on test uncertainty ratio.

THE CALIBRATION REPORT STATUS:
PASS- Term used when compliance statement is given, and the measurement result is PASS.
PASS^{Z}- Term used when compliance statement is given, and the measurement result is conditional passed or PASS^{Z}.
FAIL- Term used when compliance statement is given, and the measurement result is FAIL.
FAIL^{Z}- Term used when compliance statement is given, and the measurement result is conditional failed or FAIL^{Z}.
REPORT OF VALUE - Term used when reported measurement is not requiring compliance statement in report.
ADJUSTED- When adjustments are made to an instrument which changes the value of measurement from what was measured as found to new value as left.
LIMITED - When an instrument fails calibration but is still functional in a limited manner.

The expanded uncertainty of measurement is stated as the standard uncertainty of measurement multiplied by the coverage factor k=2, which for a normal distribution corresponds to a coverage probability of approximately 95%, unless otherwise stated. This calibration report complies with ISO/IEC 17025:2017 and ANSI/NCSL Z540.3. Calibration cycles and resulting due dates were submitted/approved by the customer. Any number of factors may cause an instrument to drift out of tolerance before the next scheduled calibration. Recalibration cycles should be based on frequency of use, environmental conditions and customer's established systematic accuracy. All standards are traceable to SI through the National Institute of Standards and Technology (NIST) and/or recognized national or international standards laboratories. Services rendered include proper manufacturer's service instruction and are warranted for no less than thirty (30) days. The information on this report pertains only to the instrument identified, this may not be reproduced in part or in a whole without the prior written approval of the issuing MP Calibration Laboratory.

GEOVision
geophysical services

SUSPENSION PS SEISMIC LOGGER/RECORDER CALIBRATION DATA FORM

INSTRUMENT DATA

System mfg.:	Oyo	Model no.:	3403
Serial no.:	160024	Calibration date:	4/30/2020
By:	Micro Precision	Due date:	4/30/2021
Counter mfg.:	Hewlett-Packard	Model no.:	53131A
Serial no.:	3416A05377	Calibration date:	4/23/2020
By:	Micro precision	Due date:	4/30/2021
Signal generator mfg.:	Agilent	Model no.:	33250A
Serial no.:	MY40029031	Calibration date:	10/31/2019
By:	Micro precision	Due date:	10/31/2020
Laptop controller mfg.:	Panasonic	Model no.:	Toughbook CF-29
Serial no.:	6AKSB01291	Calibration date:	N/A

SYSTEM SETTINGS:

Gain:	lowest setting - 2
Filter	10 KHz
Range:	200 to 5 microseconds
Delay:	0 msec
Stack (1 std)	1
System date = correct date and time	4/30/2020 15:40

PROCEDURE:

Set sine wave frequency to target frequency with amplitude of approximately 0.25 volt peak
Note actual frequency on data form.
Set sample period and record data file to disk. Note file name on data form.
Pick duration of 9 cycles using PSLOG.EXE program, note duration on data form, and save as .sps file. Calculate average frequency for each channel pair and note on data form.

Average frequency must be within +/- 1% of actual frequency at all data points.

Maximum error ((AVG-ACT)/ACT*100)% As found 0.11% As left 0.11%

Target Frequency (Hz)	Actual Frequency (Hz)	Sample Period (microS)	File Name	Time for 9 cycles Hn (msec)	Average Frequency Hn (Hz)	Time for 9 cycles Hr (msec)	Average Frequency Hr (Hz)	Time for 9 cycles V (msec)	Average Frequency V (Hz)
50.00	50.00	200	301	179.8	50.06	180.2	49.94	180	50.00
100.0	100.0	100	302	90	100.0	90	100.0	90.1	99.9
200.0	200.0	50	303	44.95	200.2	45.05	199.8	45.05	199.8
500.0	500.0	20	304	17.98	500.6	18	500.0	17.98	500.6
1000	1000	10	305	9	1000	9	1000	9	1000
2000	2000	5	306	4.5	2000	4.505	1998	4.5	2000

Calibrated by: Kyle Anderson 4/30/2020
Name Date Signature

Witnessed by: Emily Feldman 4/30/2020
Name Date Signature

Appendix G.4

Office Option Geotechnical Report
Approval Letter

CITY OF LOS ANGELES
CALIFORNIA

ERIC GARCETTI
MAYOR

BOARD OF
BUILDING AND SAFETY
COMMISSIONERS

VAN AMBATIELOS
PRESIDENT

JAVIER NUNEZ
VICE PRESIDENT

JOSELYN GEAGA-ROSENTHAL
GEORGE HOVAGUIMIAN
ELVIN W. MOON

DEPARTMENT OF
BUILDING AND SAFETY
201 NORTH FIGUEROA STREET
LOS ANGELES, CA 90012

OSAMA YOUNAN, P.E.
GENERAL MANAGER
SUPERINTENDENT OF BUILDING

JOHN WEIGHT
EXECUTIVE OFFICER

SOILS REPORT APPROVAL LETTER

December 1, 2021

LOG # 114518-02
SOILS/GEOLOGY FILE - 2

Onni Capital, LLC
315 W 9th Street, Suite 801
Los Angeles, CA 90015

TRACT: 1210
BLOCK: A
LOT(S): 11-23
LOCATION: 6254-6274 W De Longpre Ave, 1334-1360 N Vine St., 6241-6265 W Afton Pl.

CURRENT REFERENCE REPORT/LETTER(S)	REPORT No.	DATE OF DOCUMENT	PREPARED BY
Soils Report	A9382-06-02	10/28/2021	Geocon West, Inc.

PREVIOUS REFERENCE REPORT/LETTER(S)	REPORT No.	DATE OF DOCUMENT	PREPARED BY
Dept. Review Letter	114518-01	04/20/2021	LADBS
Soils Report	A9382-06-02	11/12/2020	Geocon West, Inc.
Dept. Review Letter	114518	09/30/2020	LADBS
Update Report	A9382-06-02	08/17/2020	Geocon West, Inc.
Dept. Approval Letter	95056	10/18/2016	LADBS
Soils Report	A9382-06-01	09/21/2016	Geocon West, Inc

The Grading Division of the Department of Building and Safety has reviewed the referenced report that provides recommendations for the proposed 17-story tower over 8 levels of subterranean parking. Retaining walls ranging up to 83 feet in height are proposed for the subterranean parking levels. The subterranean levels will extend over the entire site, while the tower will occupy only the western portion of the site.

The subject property was previously investigated by the consultant in 2016 to address a new 20-story multi-family residential development over 4 levels of subterranean parking. Retaining walls ranging up to 45 feet in height were proposed for the subterranean parking levels. Subsurface exploration performed by the consultant, at that time, consisted of two hollow-stem auger borings to a maximum depth of 101½ feet. The consultants recommend to support the proposed structure on mat-type foundations bearing on native undisturbed soils.

The earth materials at the subsurface exploration locations consist of up to 13 feet of uncertified fill underlain by alluvial deposits. Groundwater was encountered at depths of 48 and 39 feet below ground surface (bgs).

Additional subsurface exploration performed by the consultant (August 17-25, 2021) consisted of advancing two Cone Penetrometer Tests (CPTs) and 3 hollow-stem auger drilled borings. Temporary well casings were placed in all borings to monitor the groundwater level. According to the consultants, a static groundwater table is present at the site approximately 40 feet bgs. The historic high groundwater level in this area is approximately 45 feet bgs.

An analytical model was prepared to simulate the groundwater drawdown that would occur due to dewatering. The consultants proposed an impermeable shoring system (e.g. secant piles or sheet piles) installed around the perimeter to reduce the volume of the water to be removed.

The referenced reports are acceptable, provided the following conditions are complied with during site development:

1. Prior to issuance of grading/building permits, a design-level geotechnical report shall be submitted to the Grading Division to provide recommendations specific to the proposed development.

2. The design-level geotechnical report shall further evaluate the impact of the proposed improvements on the adjacent properties considering the seepage below the shoring system.

3. Prior to the issuance of any permit, secure approval from the Division of Land Unit of the Department of City Planning for the project.

DAN L. STOICA
Geotechnical Engineer I

DLS/dls
Log No. 114518-02
213-482-0480

cc: Geocon West, Inc., Project Consultant
VN District Office

CITY OF LOS ANGELES
DEPARTMENT OF BUILDING AND SAFETY
Grading Division

District	Log No. 114518-02

APPLICATION FOR REVIEW OF TECHNICAL REPORTS

INSTRUCTIONS

A. Address all communications to the Grading Division, LADBS, 201 N. Figueroa St., 3rd Fl., Los Angeles, CA 90012 Telephone No. (213)482-0480.

B. Submit two copies (three for subdivisions) of reports, one "pdf" copy of the report on a CD-Rom or flash drive, and one copy of application with items "1" through "10" completed.

C. Check should be made to the City of Los Angeles.

1. LEGAL DESCRIPTION
Tract: 1210
Block: A Lots: 11-23

2. PROJECT ADDRESS: 6254-6274 W De Longpre Ave, 1334-1360 N Vine St., 6241-6265 W Afton Pl.

3. OWNER: Onni Capital, LLC
Address: 315 W 9th Street, Suite 801
City: Los Angeles Zip: 90015
Phone (Daytime):

4. APPLICANT Geocon West Inc.
Address: 3303 N. San Fernando Blvd.
City: Burbank Zip: 91504
Phone (Daytime): 818-841-8388
E-mail address: berliner@geoconinc.com

5. Report(s) Prepared by: Geocon West, Inc. No. A9382-06-02A

6. Report Date(s): October 28, 2021

7. Status of project: ☐ Proposed ☑ Under Construction ☐ Storm Damage

8. Previous site reports? ☑ YES if yes, give date(s) of report(s) and name of company who prepared report(s)
Geocon West Inc. Project Numbers: A9382-06-01 (09/21/2016), A9382-06-02 (08/17/20)

9. Previous Department actions? ☑ YES if yes, provide dates and attach a copy to expedite processing.
Dates: Log No. 95056 (10/18/16), No. 114518 (09/30/20), No. 114518-01 (04/20/21)

10. Applicant Signature: Kelsey Filban (Kelsey Filban) Position: Admin

(DEPARTMENT USE ONLY)

REVIEW REQUESTED	FEES	REVIEW REQUESTED		FEES
☐ Soils Engineering	181^{50}	No. of Lots		
☐ Geology		No. of Acres		
☐ Combined Soils Engr. & Geol.		☐ Division of Land		
☐ Supplemental		Other		
☐ Combined Supplemental		☐ Expedite		90^{75}
☐ Import-Export Route		☐ Response to Correction		
Cubic Yards:		☐ Expedite ONLY		
		Sub-total		272^{25}
		One-Stop Surcharge		69^{91}
		TOTAL FEE		342^{16}

Fee Due: 342^{16}
Fee Verified By: Date: 11·2·21
(Cashier Use Only)

```
Los Angeles Department of Building
and Safety
Metro 4th Floor 11/03/2021 3:26:41
PM
User ID: htonsson
Receipt Ref Nbr: 2021307002-48
Transaction ID: 2021307002-48-1
GRADING REPORT $181.50
SYSTEMS DEV SURCH $16.34
GEN PLAN MAINT SURCH $19.06
DEV SERV CENTER SURCH $8.17
CITY PLAN SURCH $16.34
PLAN APPROVAL FEE $90.75
MISC OTHER $10.00
Amount Paid:        $342.16
PCIS Number: NA
Job Address: 1334-1360 N VINE ST, 6
254-6274 W DE LONGPRE, 6241-6265 W
AFTON PL
Owners Name: ONNI CAPITAL
```

ACTION BY:

THE REPORT IS: ☐ NOT APPROVED

☐ APPROVED WITH CONDITIONS ☐ BELOW ☐ ATTACHED

For Geology Date

For Soils Date

Appendix G.5

Supplemental Geotechnical Letter

GEOCON
WEST, INC.
GEOTECHNICAL ■ ENVIRONMENTAL ■ MATERIALS

Project No. A9382-06-02
March 31, 2022

Mr. Mark Spector
Onni Contracting (California), Inc.
315 West 9th Street, Suite 801
Los Angeles, California 90015

Subject: UPDATE OF GEOTECHNICAL INVESTIGATION
PROPOSED HIGH-RISE REDEVELOPMENT – "1360 VINE"
6254-6274 W. DE LONGPRE AVENUE, 1334 & 1348-1360 N. VINE STREET
6241 -6265 W. AFTON PLACE, LOS ANGELES, CALIFORNIA
TRACT 1210, BLOCK A, LOTS 11-23

References: *Geotechnical Investigation,* prepared by Geocon West, Inc., dated Sept. 21, 2016;

City of Los Angeles Approval Review Letter, Log No. 95056, dated Oct. 18, 2016;

Geotechnical Investigation prepared by Geocon West, Inc., dated Aug. 17, 2020;

City of Los Angeles Geology and Soils Report Review Letter, Log No. 114518, dated September 30, 2020;

Geotechnical Investigation prepared by Geocon West, Inc., dated Nov. 12, 2020;

City of Los Angeles Geology and Soils Report Review Letter, Log No. 114518-01, dated April 20, 2021;

Response to Soils Report Review Letter, prepared by Geocon West, Inc., dated October 28, 2021;

City of Los Angeles Soils Report Approval Letter, Log No. 114518-02, dated December 1, 2021.

Dear Mr. Spector:

At your request, this letter has been prepared to in support of the project EIR document. Based on the updated project description provided to us, it is our understanding that the development will consist of either a Residential Option costing of a 32-story tower with a maximum height of 360 feet 4 inches (including rooftop mechanical equipment) underlain by 4 levels of subterranean parking, or an Office Option consist of a 17-story tower with a maximum height of up to 303 feet (including rooftop mechanical equipment) underlain by 8 subterranean levels.

Geocon has evaluated development scopes for this property ranging from a 17-story tower to a 30-story tower underlain by 4- to 8-stories of subterranean parking. It is our opinion that there has been sufficient boring and laboratory testing as well as engineering analyses performed to confirm that the project is feasible from a geotechnical perspective and the intent of the geotechnical recommendations are applicable to either of the two proposed development schemes.

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

Very truly yours,

GEOCON WEST, INC.

REGISTERED PROFESSIONAL ENGINEER
JELISA THOMAS ADAMS
GE3092
GEOTECHNICAL
STATE OF CALIFORNIA

Jelisa Thomas Adams
GE 3092

(EMAIL) Addressee