Appendix F1

Geotechnical Investigation

GEOTECHNICAL INVESTIGATION

PROPOSED MIXED-USE HIGH-RISE DEVELOPMENT
1000, 1014, & 1020
NORTH LA BREA AVENUE
WEST HOLLYWOOD, CALIFORNIA

APN: 5531-014-015, 5531-014-016, & 5531-014-17

GEOCON
WEST, INC.

GEOTECHNICAL
ENVIRONMENTAL
MATERIALS

PREPARED FOR

1014 N LA BREA OWNER, LLC
C/O CIM GROUP, LP
LOS ANGELES, CALIFORNIA

PROJECT NO. W1718-06-01

MAY 10, 2023

GEOCON
W E S T, I N C.
G E O T E C H N I C A L ■ E N V I R O N M E N T A L ■ M A T E R I A L S

Project No. W1718-06-01
May 10, 2023

1014 N La Brea Owner, LLC
c/o CIM Group, LP
4700 Wilshire Boulevard
Los Angeles, California 90010

Attention: Mr. Paul Kim

Subject: GEOTECHNICAL INVESTIGATION
PROPOSED MIXED-USE HIGH-RISE DEVELOPMENT
1000, 1014, & 1020 NORTH LA BREA AVENUE
WEST HOLLYWOOD, CALIFORNIA
APN: 5531-014-015, 5531-014-016, & 5531-014-017

Dear Mr. Kim:

In accordance with your authorization of our proposal dated January 5, 2023, we have performed a geotechnical investigation for the proposed mixed-use high-rise development located at 1000, 1014, and 1020 North La Brea Avenue in the City of West Hollywood, 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 mixed-use high-rise development located at 1000, 1014, and 1020 North La Brea Avenue in the City of West Hollywood, 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 review of prior geotechnical reports prepared for the site, a site reconnaissance, field exploration, laboratory testing, engineering analysis, and the preparation of this report. The site was explored on January 30, 2023, by excavating one 7-inch diameter boring using a truck-mounted hollow-stem auger drilling machine. The boring was excavated to a depth of approximately 120½ feet below the existing ground surface. The approximate location of the exploratory boring is depicted on the Site Plan (see Figure 2A). A detailed discussion of the field investigation, including boring log, is presented in Appendix A.

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

The recommendations presented herein are based on analysis of the data obtained during our investigation, as well as data from prior investigation reports provided for our review, and our experience with similar soil and geologic conditions. The prior reports are summarized in Section 3, *Prior Investigations*. 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 1000, 1014, and 1020 North La Brea Avenue in the City of West Hollywood, California. The site is currently occupied by a concrete batch plant, concrete asphalt paved parking areas and a single-story commercial structure. The site is bounded by a single-story commercial structure with associated parking lot to the north, by North La Brea Avenue to the west, by Romaine Street to the south and by another portion of the concrete batch plant to the east. In the future, the parcel to the east is proposed to be developed as a 7-story mixed-use structure with 2 subterranean parking levels. The site is relatively level and surface water drainage at the site appears to flow to the city streets or area drains. Vegetation at the site is nonexistent.

It is our understanding that the proposed development will consist of a 34-story mixed-use high-rise structure with two subterranean parking levels that will extend to a depth up to approximately 30 feet below the ground surface, including foundation depths and dewatering elements. The ground floor of the structure will include retail space. Above the retail space, there will be 4 additional parking levels. The remaining 28 levels will be used for multi-family residential units. The proposed site conditions are depicted on the Site Plan and Cross Sections (see Figures 2A and 2B).

Preliminary bearing pressures of the foundation system that will support the proposed structure were provided by the project structural engineer, Magnusson Klemencic Associates. It is anticipated that the 34-story tower core foundation will impart pressures of up to 8,000 psf, the 17-story tower foundation will impart pressures of up to 4,500 psf, the 6-story parking garage foundation will impart pressures of up to 2,000 psf, and the plaza area will impart pressures of up to 700 psf on the underlying soil.

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. PRIOR INVESTIGATIONS

As a part of the preparation of this report, we reviewed two prior geotechnical report for the site (1010, 1014, & 1020 La Brea) and the adjacent property to the east (1011 Sycamore), provided by the Client:

> *Preliminary Geotechnical Engineering Investigation, Proposed Mixed-Use High-Rise Development, 1010, 1014 and 1020 North La Brea Avenue, West Hollywood, California,* prepared by Geotechnologies, Inc., File No. 21848, dated October 24, 2019.
>
> *Preliminary Geotechnical Engineering Investigation, Proposed Mixed-Use Development, 1011 North Sycamore Avenue, Los Angeles, California,* prepared by Geotechnologies, Inc., File No. 21849, dated October 25, 2019.

Preliminary geotechnical investigations of the subject site and the adjacent property were performed by Geotechnologies in 2019. The prior investigation report for the site was prepared for a proposed 48-story structure to be constructed over three levels of subterranean parking. The investigation included the excavation and logging of two borings to depths of 130 and 180 feet below the ground surface. The 180-foot boring was excavated on the property line that separates the site from the adjacent site to the east. The subsurface information from this boring was presented in both of Geotechnologies' reports and is referenced as boring B2. Artificial fill was encountered in both borings to a maximum depth of 8 feet below the existing ground surface. The existing fill was underlain by older alluvial soils consisting primarily of stiff to very stiff clays and silts, as well as medium dense to very dense silty sand and clayey sand. Siltstone was encountered at depths of 105 and 107½ feet below the ground surface (see additional discussion in Section 5.2 herein). Static groundwater was encountered in the borings at depths of 18½ and 19 feet below the ground surface.

Laboratory testing was performed on select soil samples. Downhole seismic testing to measure the shear wave velocity of the site soils was conducted by GeoPentech in the 180-foot boring (B2) and the results are presented in the appendices of both Geotechnologies reports. The shear wave velocities for the upper 0 to 100 feet, 10 to 110 feet, 20 to 120 feet, 30 to 130 feet, 40 to 140 and 49 to 149 feet are presented in the report. A *Soil Corrosivity Evaluation Report*, by Project X Corrosion Engineering is also presented in the appendices of both Geotechnologies reports. Seven samples taken between depths of 5 to 75 feet below ground surface from the 180-foot boring were analyzed by Project X Corrosion Engineering.

Geocon West, Inc. has reviewed the referenced reports prepared by Geotechnologies (2019), and we assume responsibility for the utilization of the exploration and laboratory data presented within those reports. The recommendations presented herein are based on analysis of the subsurface and laboratory data obtained from these prior reports, as well as our own subsurface and laboratory data. Geocon West, Inc. is the Geotechnical Consultant of Record and will be providing all necessary geotechnical consultation, plan review, design recommendations, inspection and testing services for this project. Where differing, the recommendations presented herein supersede all previous recommendations. Copies of the reports prepared by Geotechnologies are provided in Appendix C.

4. GEOLOGIC SETTING

The property is located within the northern portion of the Los Angeles Basin on a southerly sloping alluvial fan formed from sediments derived from the Santa Monica Mountains, located approximately 1.3 miles to the north. The Los Angeles Basin is a coastal plain that is bound by the Santa Monica Mountains on the north, the Elysian Park 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 east and southeast. The basin is underlain by a deep structural depression which has been infilled by both marine and continental sedimentary deposits that are underlain by a basement complex of igneous and metamorphic composition.

Regionally, the site is located in the northern portion of the Peninsular Ranges geomorphic province, near the southern boundary of the Transverse Ranges geomorphic province. The Peninsular Ranges geomorphic province is characterized by northwest-trending physiographic and geologic structures in contrast to the Transverse Ranges geomorphic province that is characterized by east-west trending geologic structures. The Hollywood Fault, located approximately 1.0 mile to the north (CGS, 2014; City of West Hollywood, 2011), forms the boundary between the Peninsular Ranges geomorphic province and the Transverse Ranges geomorphic province 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 that is in turn underlain by Pleistocene age alluvial sediments (California Geological Survey [CGS], 2012), A detailed stratigraphic profile of the geologic materials encountered at the site is provided on the boring log in Appendix A.

5.1 Artificial Fill

Artificial fill was encountered in our boring to a maximum depth of 3 feet below existing ground surface. The artificial fill generally consists of dark brown to black clay that can be characterized as moist and soft to firm. Geotechnologies encountered fill to a maximum depth of approximately 8 feet and consisted of clay silt and sand. The fill can be characterized as moist, stiff, or medium dense. 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

The artificial fill is underlain by Pleistocene age alluvium and consisted of yellowish brown to reddish brown, or grayish brown, interbedded clays, silts, and sand with varying amounts of fine gravel. The alluvium is characterized as slightly moist to wet and loose to very dense or firm to hard.

Geotechnologies encountered siltstone at depths of 105 and 107½ feet beneath the ground surface at the site. The siltstone was classified as Puente Formation bedrock. Based on published geologic information (CDWR, 1961), we consider the siltstone encountered in the prior Geotechnologies borings to be classified as Pleistocene age sediments and not Miocene age sedimentary bedrock of the Puente Formation.

6. GROUNDWATER

Based on a review of the Seismic Hazard Evaluation Report for the Hollywood 7.5 Minute Quadrangle (California Division of Mines and Geology [CDMG], 1998) and the City of West Hollywood General Plan (2011), the historic high groundwater level beneath the site is approximately 15 to 20 feet below the existing ground surface. Groundwater information presented in these publications are based on data collected from the early 1900’s to the late 1990’s.

Groundwater was encountered in our boring at a depth of approximately 17 feet below the existing ground surface. Also, groundwater was reported as encountered at depths of approximately 18½ and 19 feet beneath the ground surface in the prior borings at the site (Geotechnologies, 2019). The groundwater measurements were performed in a manner that is typical of a geotechnical exploration and should not be interpreted as representing a fully equalized water level. Based on the depth to groundwater encountered in the current and prior borings at the site and the depth of proposed construction, static groundwater is anticipated to be encountered during construction. Also, 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 8.27).

7. GEOLOGIC HAZARDS

7.1 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 within a state-designated Alquist-Priolo Earthquake Fault Zone (CGS, 2014; CGS, 2023b) or a city-designated Fault Precaution Zone (City of West Hollywood, 2011) 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 3, Regional Fault Map.

The closest surface trace of an active fault to the site is the Hollywood Fault located approximately 1.0 mile to the north (CGS, 2014). Other nearby active faults include the Newport-Inglewood Fault Zone, the Santa Monica Fault, and the Raymond Fault, located approximately 4.0 miles southwest, 4.4 miles northwest, and 7.3 miles east-northeast of the site, respectively (CGS, 2014; USGS, 2006; Ziony and Jones, 1989). The active San Andreas Fault Zone is located approximately 34 miles northeast of the site.

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 greater 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 4, Regional Seismicity Map. A partial list of moderate to major magnitude earthquakes that have occurred in the Southern California area within the last 100 years is included in the following table.

LIST OF HISTORIC EARTHQUAKES

Earthquake (Oldest to Youngest)	Date of Earthquake	Magnitude	Distance to Epicenter (Miles)	Direction to Epicenter
Near Redlands	July 23, 1923	6.3	63	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	15	E
Sierra Madre	June 28, 1991	5.8	23	ENE
Landers	June 28, 1992	7.3	109	E
Big Bear	June 28, 1992	6.4	87	E
Northridge	January 17, 1994	6.7	14	NW
Hector Mine	October 16, 1999	7.1	123	ENE
Ridgecrest	July 5, 2019	7.1	123	NNE

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

7.3 Site-Specific Ground Motion Hazard Analysis

A site-specific ground motion hazard analyses was performed in accordance with ASCE 7-16 Chapter 21 and Section 1613 of the 2022 CBC using the online applications developed by USGS.

7.3.1 Site-Specific Shear Wave Velocity

During the site exploration program performed by Geotechnologies Inc. in August, 2019, GeoPentech collected geophysical measurements for the determination of shear wave velocities as a function of depth. Approximately 150 feet of 2-inch diameter PVC casing pipe was placed inside of the 8-inch diameter hollow-stem auger boring (referred to as boring B2 in both Geotechnologies reports). The annular space between the PVC casing and the boring was backfilled with bentonite-cement grout. The seismic source used to generate the seismic waves was offset 5 feet horizontally from the boring and struck with a sledgehammer to generate seismic waves. A downhole receiver was positioned at selected depths to record the arrival of the seismic waves from the source. In-situ horizontal shear and compression wave velocity measurements were collected at 5-foot intervals from 5 feet below ground surface to a depth of 149 feet below existing ground surface. The methodologies used by GeoPentech for the data acquisition and analysis are presented in appendices of the October 2019 reports by Geotechnologies, Inc. Copies of the reports are provided in Appendix C

Based on the results of the suspension P-S logging performed by GeoPentech, the site-specific soil shear wave velocity for the upper 30 meters of soil(V_{S30}) as measured from the midpoint of the proposed subterranean levels is calculated as 352 meters/second. In accordance with Section 1613.3.2 of the 2022 California Building Code and Table 20.3-1 of ASCE 7-16, the calculated soil shear wave velocity falls within the boundaries of a Site Class "D".

7.3.2 Probabilistic Seismic Hazard Analysis

The risk-targeted Maximum Considered Earthquake (MCE_R) probabilistic response spectrum consists of the spectral response accelerations which are expected to achieve a 1 percent probability of collapse within a 50-year period, evaluated at 5 percent damping.

The mean spectral response accelerations having a 2 percent chance of exceedance in 50 years were evaluated at 5 percent damping using the PSHA hazard platform used in the National Seismic Hazard Mapping Project, NSHMP-HAZ. The soil underlying the site was modeled as a Site Class "D" with a corresponding average shear wave velocity (V_{S30}) of 352meters per second.

The web application uses the ground motion prediction equations (GMPEs) from the NGA-West 2 project: Abrahamson-et al. (2014) NGA West 2, Boore et al. (2014) NGA West 2, Campbell-Bozorgnia (2014) NGA West 2, and Chiou-Youngs (2014) NGA West 2. Each GMPE was assigned an equal weight and the mean value of the four GMPEs was evaluated. The mean spectral accelerations were rotated to maximum direction using the period specific ratios from Shahi et al. (2013 & 2014).

The GMPE of Campbell and Borzorgnia requires that the depth to where the shear wave velocity reaches 2.5 kilometers per second ($Z_{2.5}$) be defined. Additionally, the GMPEs of Abrahamson-et al., Boore et al. and Chiou-Youngs require that the depth to where the shear wave velocity reaches 1 kilometer per second ($Z_{1.0}$) be defined. The values of $Z_{2.5}$ and $Z_{1.0}$ were estimated using data from the Community Velocity Model (CVM) Version 4 developed by Southern California Earthquake Data Center (SCEDC) accessed by the OpenSHA Site Data Application (v1.5.2).

The MCE uniform hazard response spectra were adjusted to risk-targeted spectral accelerations corresponding to a 1 percent chance of collapse in 50 years by using the USGS Risk-Targeted Ground Motion Calculator and following ASCE 7-16 Section 21.2.1.2 Method 2.

The risk-targeted Maximum Considered Earthquake (MCE_R) probabilistic response spectrum is provided on Figure 5.

7.3.3 Deterministic Seismic Hazard Analysis

In order to define the deterministic scenario events, deaggregation of the uniform hazard probabilistic response spectrum was performed using the USGS Uniform Hazard Tool. The inversion approach used by UCERF-3 allows for a large number of variations for each source scenario, including multi-fault ruptures. Therefore, deaggregation of UCERF-3 consists of the contributions from multi-fault ruptures rather than individual source contributions. To address this, the USGS Unified Hazard Tool aggregates the contributions on a per-fault-section basis, with rupture contributions only ever counted once. The Unified Hazard Tool deaggregation contributor list shows the fault sections which contribute most to hazard at a site and report a mean earthquake magnitude for each section identified by a 'parent' fault name and section index. Based on the deaggregation, we have considered scenario events with the greatest contribution to the deterministic ground motions.

The characteristics of the deterministic scenario events were defined using the closest distance (R_{rup}) from the Uniform Hazard Tool deaggregation results and supplemented by the fault source parameters specified in the BSSC2014 Scenario Catalog. The values of $Z_{2.5}$ and $Z_{1.0}$ were estimated using data from the Community Velocity Model (CVM) Version 4, Iteration 26, Basin Depth developed by Southern California Earthquake Data Center (SCEDC) accessed by the OpenSHA Site Data Application (v1.5.2).

The input values used to evaluate the deterministic scenarios are provided in Figure 8. The deterministic median and standard deviation (sigma) for the scenario events were evaluated using the USGS NSHMP-HAZ-WS Response Spectra application. The deterministic analysis used the same four GMPEs, equally weighted, to generate the median and standard deviation of ground motion which were then used to calculate the 84^{th} percentile at 5% damping. The geometric median spectral accelerations were rotated to maximum direction using the period specific ratios from Shahi et al. (2013 & 2014). The four deterministic scenarios were compared, and the event occurring on the Santa Monica fault is considered the controlling deterministic event.

The resulting 84th percentile maximum rotated component deterministic response spectrum for the controlling deterministic event was compared to the deterministic lower bound envelope as defined by ASCE 7-16, Section 21.2.2, and the maximum values taken as the deterministic MCE_R response spectrum (see Figure 6).

7.3.4 Site-Specific Response Spectrum

The lesser of the probabilistic and deterministic MCE_R response spectrums is the Site-Specific MCE_R. Two thirds of the Site-Specific MCE_R is the Design Earthquake (DE) Response Spectrum, provided the results are not less than 80 percent of the General Design Response Spectrum determined by ASCE 7-16 Section 11.4.6 with F_a and F_v determined as specified in Section 21.3.

Graphical representations of the analyses are presented on Figures 5 and 6. The Site-Specific Design Earthquake response spectrum at 5 percent damping is presented on Figure 6 and in tabular form on Figure 7.

7.3.5 Site-Specific Seismic Design Criteria

Based the site-specific ground motion hazard analysis performed, and in accordance with the ASCE 7-16 Section 21.4, site-specific design acceleration parameters shall be derived using the results of the site-specific ground motion hazard analysis.

The parameter S_{DS} shall be taken as equal to 90 percent of the maximum spectral acceleration obtained from the site-specific analysis at any period within the range from 0.2 to 5 seconds, inclusive. The parameter S_{D1} shall be taken as the maximum value of the product of the spectral acceleration and period for periods from 1 to 5 seconds, inclusive. The values of S_{MS} and S_{M1} shall be taken as 1.5 times the site-specific values of S_{DS} and S_{D1}. The site-specific design acceleration parameters shall not be less than 80 percent of the general seismic design values determined by ASCE 7-16 Section 11.4.

The following table presents the site-specific seismic design parameters based on the site-specific ground motion hazard analysis.

SITE-SPECIFIC DESIGN ACCELERATION PARAMETERS

Parameter	Value
Site Class Modified MCE_R Spectral Response Acceleration (short), S_{MS}	2.353g
Site Class Modified MCE_R Spectral Response Acceleration – (1 sec), S_{M1}	1.5g
5% Damped Design Spectral Response Acceleration (short), S_{DS}	1.568g
5% Damped Design Spectral Response Acceleration (1 sec), S_{D1}	1g

7.3.6 Site-Specific Peak Ground Acceleration

The site-specific Maximum Considered Earthquake (MCE_G) geometric mean peak ground acceleration was evaluated in accordance with ASCE 7-16 Section 21.5.

The probabilistic geometric mean peak ground acceleration and the deterministic 84th percentile geometric mean peak ground acceleration were analyzed using the same approaches as described above. The analysis used the same Site Class and scenario earthquake.

The deterministic MCE_G shall not be less than $0.5F_{PGA}$, where F_{PGA} is determined from ASCE 7-16 Table 11.8-1 with the value of PGA taken as 0.5g. The site-specific MCE_G peak ground acceleration is taken as the lesser of the probabilistic and deterministic MCE_G, provided the value is not less than 80 percent of the value of PGA_M as determined by ASCE 7-16 Equation 11.8.1.

ASCE 7-16 SITE-SPECIFIC PEAK GROUND ACCELERATION

Parameter	Value	ASCE 7-16 Reference
Site-Specific MCE_G Peak Ground Acceleration, PGA_M	0.977g	Section 21.5

7.4 Liquefaction Potential

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

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

A review of the Seismic Hazard Zone Map for the Hollywood Quadrangle (CGS, 2014; CDMG, 1999) indicates that the site is not located in an area designated as having a potential for liquefaction. In addition, the City of West Hollywood Seismic Hazard Zones Map (City of West Hollywood, 2010) indicates that the site is not located within an area identified as having a potential for liquefaction. The site is underlain by Pleistocene age older alluvial sediments that are typically not prone to liquefaction. Based on these considerations, it is our opinion that the potential for liquefaction and associated ground deformations beneath the site is very low.

7.5 Slope Stability

The topography in the general site vicinity slopes gently to the southwest and the site topography is relatively level. The City of West Hollywood (2011) and the County of Los Angeles (Leighton, 1990) indicate that the site is not located within a designated "hillside area" or an area identified as having a potential for slope instability. Also, according to the California Geological Survey (CDMG, 1999; CGS, 2014) and the City of West Hollywood (2010 and 2011), the site is not located within an area identified as having a potential for seismic slope instability. No landslides have been identified at the site or in close proximity to the site, and the site is not in the path of any known or potential landslides (USGS, 2023a). Therefore, the potential for slope stability hazards to adversely affect the proposed development is considered low.

7.6 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) and the City of West Hollywood (2010 and 2011) indicate 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.

7.7 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 from a seismically induced seiche is considered unlikely.

The site is located within an area of minimal flooding (Zone X) as defined by the Federal Emergency Management Agency (FEMA, 2023; LACDPW, 2023). Therefore, flooding is not anticipated to adversely impact the site.

7.8 Oil Fields & Methane Potential

Based on a review of the California Geologic Energy Management Division (CalGEM) Well Finder website, the site is not located within the limits of a known oil field and oil wells are not located in the immediate site vicinity (CalGEM, 2023). However, due to the voluntary nature of reporting by oil and gas 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 CalGEM.

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.

7.9 Subsidence

Subsidence occurs when a large portion of land is displaced vertically, usually due to the withdrawal of groundwater, oil, or natural gas or as a result of decomposition of natural organic materials. Soils that are particularly subject to subsidence include those with high silt or clay content and/or high organic content. The site is not located within the limits of a former marsh (Mendenhall, 1907). In addition, organic materials were not encountered in our borings at the site. Therefore, the potential for subsidence related to decomposition of organic materials at the site is considered low. Also, the potential for subsidence at the site related to fluid or gas withdrawal is considered low (USGS, 2023b). Oil or gas extraction within the nearby Salt Lake Oil Field (approximately 2,000 feet to the south) is considered to have marginal activity. Water injection and flooding operations as part of secondary recovery have largely mitigated hazards related to fluid or gas withdrawal in the area (City of West Hollywood, 2010).

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 3 feet of existing artificial fill was encountered during our site exploration. Up to 8 feet of existing artificial fill was encountered in the prior explorations at the site. 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. It is our opinion that the existing fill, in its present condition, is not suitable for direct support of proposed foundations or slabs. Excavation for the proposed subterranean levels is anticipated to penetrate through the existing fill and expose undisturbed alluvial soils throughout the excavation bottom. Where necessary, the existing artificial fill and alluvial soils are suitable for re-use as engineered fill provided the procedures outlined in the *Grading* section of this report are followed (see Section 8.6).

8.1.3 Groundwater was encountered during site exploration at a depth of approximately 17 feet below existing ground surface. The groundwater measurements were performed in a manner that is typical of a geotechnical exploration and should not be interpreted as representing a fully equalized water level. In the prior site investigation conducted by Geotechnologies in 2019, groundwater water was encountered at depths of approximately 18½ and 19 feet below the ground surface. Excavation for construction of the proposed subterranean levels is anticipated to extend to a maximum depth of approximately 30 feet below the ground surface, including foundation excavations and dewatering elements. Due to the depth of the proposed excavation and the static groundwater level, the contractor should be prepared to implement temporary dewatering measures to mitigate groundwater during excavation and construction. The depth to groundwater at the time of construction can be confirmed during installation of dewatering wells or during initial drilling of soldier piles. Recommendations for temporary dewatering are discussed in Section 8.5 of this report.

8.1.4 Temporary dewatering measures will require additional analyses to evaluate the potential impacts the proposed dewatering at the subject site will have on the adjacent structures and public streets. The additional analyses are anticipated to include estimating a dewatering drawdown curve and the resulting settlements. Onsite flow testing as well as coordination with a dewatering contractor and a hydrogeologist will be required for this evaluation.

8.1.5 The historically high groundwater at the site is approximately 15 to 20 feet below the ground surface. The proposed structure must be designed for hydrostatic pressure for any portion that extends below a depth of 15 feet. The hydrostatic design will result in uplift forces on the structure that must be resisted by counterweight or structural design measures. If the proposed structure does not provide sufficient dead load to resist the buoyant forces then uplift mitigation will be required. Recommendations for uplift resistance are provided in Section 8.10 of this report.

8.1.6 Due to variation in the anticipated bearing pressures imparted by the proposed structure's components, which range from 700 to 8,000 psf, it is recommended that ground improvement consisting of stone columns be used to reduce the differential settlement amongst the various structural components. The ground improvement system is able to provide variable soil stiffness by changing the pier spacing. This allows the ground improvement system to be tailored to match the structure's bearing pressure variations and to limit settlement to tolerable levels.

8.1.7 Subsequent to the completion of the recommended ground improvement, the proposed structure may be supported on a reinforced concrete mat foundation system that derives support in the improved soils. In addition, a mat foundation is more accommodating to waterproofing and hydrostatic design. The foundation should be designed to derive vertical support from the improved soils and may develop lateral resistance at the foundation perimeter, as well as by friction beneath the foundations, if necessary. The ground improvement and foundation recommendations presented herein are intended to reduce the effects of differential settlement on the proposed structure. In order to minimize differential settlement between the ramp, ramp walls, and basement level, it is recommended that the ramp and ramp walls for the subterranean parking garage be structurally supported on the mat foundation. All foundation excavations must be observed and approved in writing by the Geotechnical Engineer prior to placement of steel or concrete. Recommendations for ground improvement and the design of a mat foundation system are provided in Section 8.7 and Section 8.8, respectively, of this report.

8.1.8 As an alternative, the proposed structure may be supported on a deepened foundation system consisting of auger-cast pressure grouted displacement (APGD) piles. The APGD piles have the benefit of not generating soil spoils; however, the use of APGD piles will require a comprehensive load testing program. The Client should be aware that APGD piles are designed and installed by a specialty geotechnical contractor. Recommendations for the design of APGD piles are provided in Section 8.9

8.1.9 Where new foundations are constructed immediately adjacent to existing foundations (such as adjacent to the future structure on the east), the new foundation should be deepened to match or exceed the depth of the existing foundation to prevent a surcharge on the existing foundation. Where a proposed foundation will be deeper than an existing adjacent foundation, the proposed foundation must be designed to resist the surcharge imposed by the existing foundation. The surcharge area may be defined by a 1:1 projection down and away from the bottom of an existing foundation.

8.1.10 All excavations must be observed and approved in writing by the Geotechnical Engineer (a representative of Geocon) prior to placing any fill or foundation construction. Due to the potential for high-moisture content soils at the excavation bottom or if the excavation bottom is saturated, stabilization of the bottom of the excavation may be required in order to provide a firm working surface upon which engineered fill can be placed and heavy equipment can operate. Recommendations for earthwork and bottom stabilization are provided in the *Grading* section of this report (see Section 8.6).

8.1.11 The grading contractor should be aware that the existing soils are currently at or above optimum moisture content. If the site soils are oversaturated at the time of grading, they will likely require some spreading and drying activities in order to achieve proper compaction; however, this could change seasonally.

8.1.12 Excavations up to 30 feet in vertical height are anticipated for construction of the proposed subterranean levels, including foundations and dewatering elements. Due to the depth of the excavation and the proximity to the property lines, city streets and adjacent offsite structures, excavation of the proposed subterranean level will likely require sloping and/or shoring measures in order to provide a stable excavation. Where shoring is required, a soldier pile shoring system is the most common type of shoring system. However, further study will be required to determine if a soldier pile and lagging system is compatible with the temporary dewatering system. 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 shoring are provided in Section 8.21 of this report.

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

8.1.14 Foundations for small outlying structures, such as block walls up to 6 feet high, planter walls or trash enclosures, which will not be tied to the proposed structure, may be supported on conventional foundations bearing on a minimum of 12 inches of newly placed engineered fill which extends laterally at least 12 inches beyond the foundation area. Where excavation and proper compaction cannot be performed, foundations may derive support directly in the undisturbed alluvial soils and should be deepened as necessary to maintain a minimum 12-inch embedment into the recommended bearing materials. If the soils exposed in the excavation bottom are soft or loose, compaction of the soils will be required prior to placing steel or concrete. Compaction of the foundation excavation bottom is typically accomplished with a compaction wheel or mechanical whacker and must be observed and approved by a Geocon representative.

8.1.15 Where new paving is to be placed, it is recommended that all existing fill soils be excavated and properly compacted for paving support. The client should be aware that excavation and compaction of all existing fill in the area of new paving is not required, however, paving constructed over existing uncertified fill or unsuitable soils may experience increased settlement and/or cracking and may therefore have a shorter design life and increased maintenance costs. As a minimum, the upper twelve inches of soil should be scarified and properly compacted. Paving recommendations are provided in the *Preliminary Pavement Recommendations* section of this report (see Section 8.14).

8.1.16 Based on the depth of static groundwater, historic high groundwater levels, and the depth of proposed subterranean parking levels, a stormwater infiltration system is not recommended for this project. It is suggested that stormwater be retained, filtered, and discharged in accordance with the requirements of the local governing agency.

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

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

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

8.1.20 The requirements of the 2022 CBC, 2023 LA County Building Code, and ASCE 7-16 shall apply to the project.

8.2 Mandatory County of Los Angeles Statement

8.2.1 This statement is made in accordance with the County of Los Angeles, Section 111. It is the opinion of this office that, provided our recommendations are followed and properly maintained, (1) the proposed grading and proposed structures will be safe for its intended use against hazard from landslide, settlement or slippage and (2) the proposed grading and proposed structures will have no adverse effect on the stability of the site or adjoining properties.

8.3 Soil and Excavation Characteristics

8.3.1 The in-situ soils can be excavated with moderate effort using conventional excavation equipment. Some caving should be anticipated in unshored excavations, especially where granular soils are encountered. The contractor should be aware that casing will be required during shoring pile installation.

8.3.2 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 existing adjacent improvements. The site soils may be classified as OSHA Type B in the upper 8 feet and as OSHA Type C below a depth of 8 feet.

8.3.3 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 or shoring. Excavation recommendations are provided in the *Temporary Excavations* section of this report (see Section 8.20).

8.3.4 The soils encountered during the prior investigation at a depth between 1 and 5 feet below the existing ground surface are considered to have a “medium” to “high” expansive potential (EI = 82 & 94) and are classified as “expansive” in accordance with the 2022 California Building Code (CBC) Section 1803.5.3. The recommendations of this report assume that near surface elements, such as paving or hardscape, may derive support in these soils. Furthermore, based on depth of the proposed subterranean levels, the proposed structure would not be prone to the effects of expansive soils.

8.4 Minimum Resistivity, pH and Water-Soluble Sulfate

8.4.1 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 indicate that the site soils are considered "corrosive" to "severely corrosive" with respect to corrosion of buried ferrous metals on site. The results are presented in the *Soil Corrosivity Evaluation Report* by Project X Corrosion Engineering (Appendix C) and should be considered for design of underground structures.

8.4.2 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 indicate that the site soils possess a sulfate exposure class of "S0" to concrete structures as defined by 2022 CBC Section 1904 and ACI 318-19 Chapter 19. The complete *Soil Corrosivity Evaluation Report* by Project X Corrosion Engineering is presented in Appendix C.

8.4.3 Geocon West, Inc. does not practice in the field of corrosion engineering and mitigation. If corrosion sensitive improvements are planned, the recommendations in the *Soil Corrosivity Evaluation Report* by Project X Corrosion Engineering should be implemented. A copy of the report is embedded within the reports presented in Appendix C of this report.

8.5 Temporary Dewatering

8.5.1 Groundwater was encountered during site exploration at a depth of approximately 17 feet below ground surface. The groundwater measurements were performed in a manner that is typical of a geotechnical exploration and should not be interpreted as representing a fully equalized water level. In a prior site investigation conducted by Geotechnologies in 2019, groundwater water encountered at depths of approximately 18½ and 19 feet below the ground surface. Based on the conditions encountered at the time of exploration, groundwater should be expected during construction activities. The depth to groundwater at the time of construction can be confirmed during installation of dewatering wells or during shoring pile installation. If groundwater is present above the depth of the subterranean level, temporary dewatering will be necessary to maintain a safe working environment during excavation and construction activities.

8.5.2 It is recommended that a qualified dewatering consultant be retained to design the dewatering system. Recommendations for design flow rates for the temporary dewatering system should be determined by a qualified contractor or dewatering consultant. The dewatering consultant should also provide the minimum depth that the temporary dewatering be effective to, and also the potential effects of temporary dewatering on adjacent structures and the public right of way. Additional analyses will be required in the future to evaluate the settlement based on the proposed dewatering system and associated drawdown curves.

8.5.3 Temporary dewatering typically consists 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 will be determined by qualified dewatering consultant.

8.5.4 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.

8.6 Grading

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

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

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

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

8.6.5 The grading contractor should be aware that the site soils are currently at or above optimum moisture content. Conditions could change seasonally. If the soils are in excess of 3 percent above optimum moisture content at the time of construction and grading, the soils will likely require some spreading and drying activities in order to achieve proper compaction.

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

8.6.7 Bottom stabilization, if necessary, may be achieved placing 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.).

8.6.8 An additional method of subgrade stabilization would be to place a minimum 12-inch thick layer of aggregate base over Tensar InterAx NX850 geogrid or equivalent extruded (nonwoven) geotextile. The Tensar geogrids should be installed taut and should overlap in accordance with the manufacturer's recommendations. Prior to placing the geogrid, excessively soft or wet materials should be removed and the resulting excavation bottom should be free of loose material. Non-vibratory compaction methods should be used for compaction of the base material. The aggregate base should be compacted to a dry density of at least 95 percent of the laboratory maximum density near the optimum moisture. If pumping of the subgrade continues, a thicker layer of aggregate base may be placed. It is very important that subgrade stabilization be performed uniformly across the entire excavation bottom.

8.6.9 The proposed structure may be supported on a reinforced concrete mat foundation system deriving support in improved soils at and below a depth of 30 feet below the 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.).

8.6.10 All fill and backfill soils should be placed in horizontal loose layers approximately 6 to 8 inches thick, moisture conditioned to at least two percent above optimum moisture content (higher moisture contents may be acceptable, provided the minimum required compaction is achieved), and properly compacted to a minimum 90 percent of the maximum dry density in accordance with ASTM D 1557 (latest edition).

8.6.11 Prior to construction of exterior slabs or paving, the upper 12 inches of the subgrade should be moisture conditioned to at least two percent above optimum moisture content (higher moisture contents may be acceptable, provided the minimum required compaction is achieved), and properly compacted to at least 92 percent relative compaction, as determined by ASTM Test Method D1557 (latest edition).

8.6.12 Foundations for small outlying structures, such as block walls up to 6 feet high, planter walls or trash enclosures, which will not be tied to the proposed structure, may be supported on conventional foundations deriving support on a minimum of 12 inches of newly placed engineered fill which extends laterally at least 12 inches beyond the foundation area. Where excavation and proper compaction cannot be performed, foundations may derive support directly in the undisturbed alluvial soils, and should be deepened as necessary to maintain a minimum 12-inch embedment into the recommended bearing materials. If the soils exposed in the excavation bottom are soft or loose, compaction of the soils will be required prior to placing steel or concrete. Compaction of the foundation excavation bottom is typically accomplished with a compaction wheel or mechanical whacker and must be observed and approved by a Geocon representative.

8.6.13 Although not anticipated for this project, all imported fill shall be observed, tested, and approved by Geocon West, Inc. prior to bringing soil to the site. Rocks larger than six inches in diameter shall not be used in the fill. If necessary, import soils used as structural fill should have an expansion index less than 50 and soil corrosivity properties that are equally or less detrimental to that of the existing onsite soils (see *Soil Corrosivity Evaluation Report* by Project X Corrosion Engineering in Appendix C).

8.6.14 Utility trenches should be properly backfilled in accordance with the following requirements. 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 as backfill is also acceptable. Prior to placing any bedding materials or pipes, the excavation bottom must be observed and approved in writing by the Geotechnical Engineer (a representative of Geocon).

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

8.7 Ground Improvement– Stone Columns

8.7.1 It is recommended that soil improvement consisting of stone columns be performed below the proposed structure. This type of ground improvement system has multiple trade names and is designed and installed by a specialty contractor. Subsequent to construction of the stone column system, the proposed structure may be supported on a mat foundation system deriving support in the improved soils. The foundation system should be designed to derive vertical support from the improved soils and may develop lateral resistance at the foundation perimeter, as well as by friction beneath the foundations, if necessary.

8.7.2 The RAP system is based on soil improvement that consists of installing densified, aggregate columns to depths typically ranging up to about 25 feet below the proposed foundations. The system increases density and lateral stress in the surrounding soil and claims improvement in bearing capacity and potential settlement . Stone column elements are constructed by creating shafts (commonly 30 inches in diameter) by drilling or displacement methods, and backfilling the open shaft with specially rammed/compacted, open graded crushed rock and Class 2 AB in 10- to 12-inch lifts. It should be noted that creating the shaft using the displacement method, advancing the shaft with a displacement mandrel, reduces the soil cuttings generated during the creation of the shaft. It is anticipated that the displacement method will be suitable for penetrations in the alluvial soils underlying the site.

8.7.3 The pattern and depth of ground improvements may vary depending upon the purposes of mitigation and stratigraphic conditions. The specialty contractor should design the RAP to incorporate allowable static settlements in accordance with the recommendations of the project structural engineer. The contractor is also responsible for evaluating the post-installation static settlement within the ground improvement zone and shall provide this information to the project structural engineer to confirm if the planned structure can tolerate the planned settlements after the installation of the ground improvement.

8.7.4 Spacing and diameter should be selected by the specialty contractor to obtain level of improvement as outlined herein. The ground improvement should extend laterally outside the edge of planned building structures, where practical.

8.7.5 The ground improvement design should be based on settlement criteria of a maximum differential settlement of 1½ inches between adjacent columns, or as specified by the project structural engineer.

8.7.6 The ground improvement design package should be submitted to Geocon West, Inc. for review at least two weeks prior to mobilization for construction. Within the design package, the specialty contractor should outline a performance and load testing program to verify the effectiveness of the ground improvement and to confirm the bearing capacity of the improved soils with a full-scale load test. During the load testing, a representative of Geocon should be present to observe the ground improvement installation and testing. The information obtained from the load testing should be used to modify the depth necessary to achieve design capacities, as well as develop installation criteria that can be used during construction.

8.7.7 Geocon should be present continuously during installation of the ground improvements. Geocon's QA/QC observations and documentation will include pier ID, location, depth, diameter, number of lifts, type of aggregate placed, lift thickness, and any changed conditions.

8.8 Mat Foundation Design

8.8.1 Based on the depth of proposed construction and potential hydrostatic pressures, it is recommended that a reinforced concrete mat foundation be utilized for support of the proposed structure

8.8.2 The proposed structure must be designed for hydrostatic pressure for any portion of the structure below a depth of 15 feet. 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. If the proposed structure does not provide sufficient dead load to resist the buoyant forces, then uplift mitigation will be required (see Section 8.10).

8.8.3 It is anticipated that the mat foundation will impart pressures ranging from 700 to 8,000 psf. For preliminary design purposes, the aforementioned bearing pressures may be assumed; however, the design bearing pressures should be provided by the ground improvement contractor and verified with testing at the completion of the ground improvement. The allowable bearing pressure may be increased by up to one-third for transient loads due to wind or seismic forces.

8.8.4 For preliminary design purposes, a modulus of subgrade reaction of 150 pci may be utilized for design of the mat foundation where directly underlain by improved soil. However, the ground improvement contractor should provide the structural engineer with a revised modulus value incorporating the planned improvement techniques. This value is a unit value for use with a 1-foot square footing. The modulus should be reduced in accordance with the following equation when used with larger foundations:

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

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

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

8.8.6 For seismic design purposes, a coefficient of friction of 0.4 may be utilized between concrete slab and subgrade soils without a moisture barrier, and 0.15 for slabs underlain by a moisture barrier.

8.8.7 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.

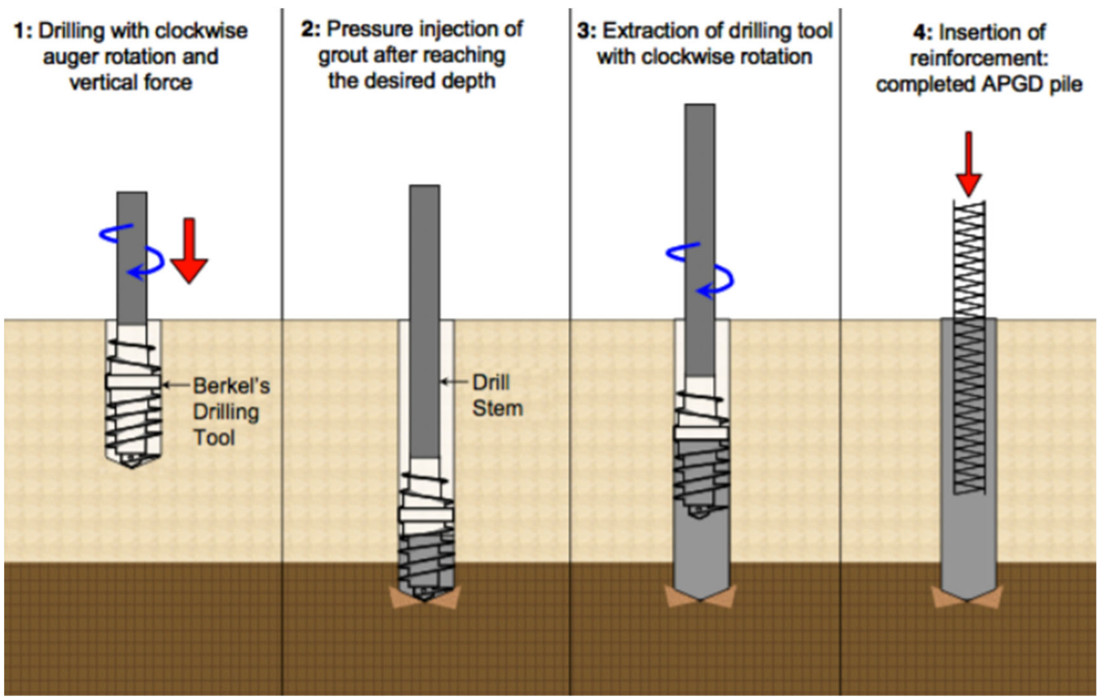
8.8.8 Waterproofing of subterranean walls and slabs is recommended for this project. Particular care should be taken in the design and installation of waterproofing to avoid moisture problems, or actual water seepage into the structure through any normal shrinkage cracks which may develop in the concrete 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.

8.8.9 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.

8.8.10 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.

8.9 Auger-Cast Displacement Piles

8.9.1 As an alternative, it is recommended that the proposed structure be supported on Auger-Cast Pressure Grouted Displacement (APGD) piles deriving support in competent alluvium generally found below the basement level. Auger-cast pressure grouted displacement (APGD) piles are installed by advancing a hollow-stem auger with a diameter equivalent to that of the pile to the desired pile tip elevation. The specialized hollow-stem auger bit displaces the penetrated soils laterally away from the auger as it is advanced, creating increased pile capacity and minimizing the amount of soil spoils. Once the desired pile tip elevation is achieved, grout is pumped under pressure from the tip of the auger as it is withdrawn and then the pile reinforcing steel is placed in the grout.



8.9.2 The proposed structure must be designed for hydrostatic pressure for any portion of the structure below a depth of 15 feet. 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. If the proposed structure does not provide sufficient dead load to resist the buoyant forces, then uplift mitigation will be required (see Section 8.10).

8.9.3 The Client should be aware that APGD piles are typically designed and installed by a specialty geotechnical contractor. The pile recommendations presented herein may be used for preliminary design purposes. Actual pile capacities are verified by load testing after installation.

8.9.4 For preliminary design purposes, 24-, 30-, and 36-inch diameter APGD piles have been assumed, and preliminary ultimate pile capacities are provided in the following table.

AUGER-CAST GROUTED DISPLACEMENT ULTIMATE PILE CAPACITIES

Embedment Below Bottom of Excavation (feet)	24-Inch Diameter Pile Capacity (kips)	30-Inch Diameter Pile Capacity (kips)	36-Inch Diameter Pile Capacity (kips)
50 feet	890	1390	1840
60 feet	1065	1660	2390
70 feet	1235	1925	2770
80 feet	1410	2190	3150

8.9.5 Single pile uplift capacity can be taken as 50 percent of the downward capacity.

8.9.6 The axial capacity of the APGD piles should be verified by the design-build contractor and confirmed based upon pile load testing. Geocon should review, and if necessary, can assist the design-build contractor in developing a suitable testing program. During pile load testing, a representative of Geocon should be present to observe pile installation and testing. The information obtained from the pile load testing should be used to evaluate the need to modify pile lengths to achieve design capacities, as well as develop installation criteria that can be used during construction of production piles.

8.9.7 It is recommended that at least two pre-production piles or one percent of the production pile quantity be constructed, and load tested to at least 200 percent of the design load. Additional information on the indicator pile test program is provided in Appendix D.

8.9.8 During pile load testing, a representative of Geocon must be present to observe pile installation and testing procedures. The information obtained from the pile load testing program should be used to verify the suitability of the preliminary design parameters, or to modify pile design and installation criteria prior to construction of production piles.

8.9.9 Proof testing of production piles should also be performed by the design-build contractor and verified by the Geotechnical Engineer. It is recommended that at least 5 percent of production piles be constructed, and load tested to at least 160 percent of the design load. In addition, Thermal Integrity Profiling will be required for 10 percent of the production piles. The testing program and acceptance criteria should be configured to satisfy the requirements of the building official.

8.9.10 APGD pile construction should be performed under continuous observation of the Geotechnical Engineer (a representative of Geocon) to observe that soil conditions do not differ from those anticipated and to observe that construction of the APGD piles is performed in accordance with the project plans and specifications. Additional specifications for APGD installation are provided in Appendix D.

8.9.11 If piles are spaced at least at least 3 diameters on center, no reduction in axial capacity is considered necessary for group effects. If pile spacing is closer than three pile diameters, an evaluation for group effects including appropriate reductions should be incorporated into the pile design based on pile dimension, spacing, and the direction of loading.

8.9.12 For increased resistance to differential foundation movement and lateral drift, the pile tops should be interconnected in two horizontal directions with grade beams or tied with a structural slab. The project structural engineer should provide slab and grade beam design, reinforcement and spacing dependent on anticipated loading. However, for grade beams we recommend a minimum embedment depth below lowest adjacent pad grade of 24 inches and a minimum width of 12 inches. In addition, minimum reinforcement should consist of four No. 5 steel reinforcing bars; two placed near the top of the grade beam and two near the bottom. The use of a structural slab may be more accommodating to waterproofing and hydrostatic design

8.9.13 APGD piles should be designed based on settlement criteria of a maximum static differential settlement of 1½ inches between adjacent columns.

8.10 Uplift Resistance

8.10.1 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 are provided in Section 7.21.

8.10.2 If the structural design will rely on uplift resistance along the sides of the foundations and waterproofing is present, the waterproofing manufacturer should specify the allowable coefficient of friction based on their product's material properties.

8.10.3 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 290 psf (⅔ the downward capacity).

8.10.4 Post-grouted friction piles should be a minimum of 12 inches in diameter and uniformly spaced at least 3 times the diameter on-center. If so spaced, no reduction of the axial capacity for group effects will be necessary. The allowable uplift capacity may be increased by one-third when considering transient wind or seismic loads.

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

8.11 Miscellaneous Foundations

8.11.1 Foundations for small outlying structures, such as block walls up to 6 feet in height, planter walls or trash enclosures, which will not be tied to the proposed structure, may be supported on conventional foundations bearing on a minimum of 12 inches of newly placed engineered fill which extends laterally at least 12 inches beyond the foundation area. Where excavation and compaction cannot be performed, such as adjacent to property lines, foundations may derive support in the undisturbed alluvium, and should be deepened as necessary to maintain a minimum 12-inch embedment into the recommended bearing materials.

8.11.2 If the soils exposed in the excavation bottom are soft, compaction of the soft soils will be required prior to placing steel or concrete. Compaction of the foundation excavation bottom is typically accomplished with a compaction wheel or mechanical whacker and must be observed and approved by a Geocon representative. Miscellaneous foundations may be designed for a bearing value of 1,500 psf, and should be a minimum of 12 inches in width, 30 inches in depth below the lowest adjacent grade, and 12 inches into the recommended bearing material. The allowable bearing pressure may be increased by up to one-third for transient loads due to wind or seismic forces.

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

8.12 Lateral Design

8.12.1 Resistance to lateral loading may be provided by friction acting at the base of foundations and by passive earth pressure. An allowable coefficient of friction of 0.4 may be used with the dead load force in the undisturbed alluvium or in properly compacted fill. The recommended coefficient of friction is an allowable value based on a factor of safety of 1.5.

8.12.2 Based on a factor of safety of 1.5, passive earth pressure for the sides of foundations poured against undisturbed alluvium or in properly compacted fill may be computed as an equivalent fluid having a density of 260 pounds per cubic foot with a maximum earth pressure of 2,600 psf. Passive earth pressure for the sides of foundations and slabs poured against the alluvial soils below the groundwater table may be computed as an equivalent fluid having a density of 130 pounds per cubic foot with a maximum earth pressure of 1,300 psf (these values have been adjusted for buoyant forces). When combining passive and friction for lateral resistance, the passive component should be reduced by one-third.

8.12.3 Ultimate lateral capacities for ¼ inch deflection of fixed and free-head drilled cast-in place piles are presented in the table below. No factors of safety have been applied to the lateral load values calculated to induce ¼-inch lateral deflection. Lateral capacities provided are for 24-, 30-, and 36-inch diameter APGD piles, penetrating the earth materials encountered during the course of this investigation. Assumed as part of these lateral capacity calculations are a concrete modulus of elasticity of at least 3,000,000 psi.

LATERAL LOAD CAPACITIES OF DRILLED CAST-IN-PLACE PILES

FIXED HEAD (NO HEAD ROTATION)

PILE NUMBER	PILE DIAMETER (INCHES)	Lateral Load Capacity "P" (KIPS)	Maximum Positive Moment "Mp" (LAT FORCE =P)	Maximum Negative Moment "Mp" (LAT FORCE =P)	Depth to Max Pos. Moment (Feet)	Depth to Zero Moment (Feet)	Depth to Inflection Point (Feet)	MINIMUM PILE LENGTH FOR APPLICABILITY OF LATERAL DESIGN DATA (FEET)
1	24	**43**	**1.4** P	**-5.1** P	**12**	**25**	**6.4**	**25**
2	30	**61**	**1.7** P	**-6.1** P	**15**	**28**	**7.6**	**28**
3	36	**79**	**1.9** P	**-7.1** P	**17**	**30**	**8.8**	**30**

FREE HEAD (HINGED)

PILE NUMBER	PILE DIAMETER (INCHES)	Lateral Load Capacity "P" (KIPS)	Maximum Moment "Mp" (LAT FORCE =P)	Depth to Zero Moment (Feet)	Depth to Maximum Moment (Feet)
1	24	**17**	**4.3** P	**23**	**7**
2	30	**25**	**5.2** P	**28**	**9**
3	36	**32**	**6.0** P	**31**	**10**

Lateral capacities are based on 1/4-inch deflection.
Moment magnitudes are presented as a function of the applied lateral load "P".
"P" is entered in units of kips and the moment magnitude will be in units of kip-feet.
The maximum negative moment is at the rigid, pile to pile cap or grade beam connection at the top of the pile.

8.13 Exterior Concrete Slabs-on-Grade

8.13.1 Exterior concrete slabs-on-grade, at the ground surface, subject to vehicle loading should be designed in accordance with the recommendations in the *Preliminary Pavement Recommendations* section of this report (Section 8.14).

8.13.2 Exterior slabs, not subject to traffic loads, should be at least 4 inches thick and reinforced with No. 3 steel reinforcing bars placed 18 inches on center in both horizontal directions, positioned near the slab midpoint. Prior to construction of slabs, the upper 12 inches of subgrade should be moistened to at least two percent over optimum moisture content (higher moisture contents may be acceptable, provided the minimum required compaction is achieved), and properly compacted to at least 92 percent relative compaction, as determined by ASTM Test Method D 1557 (latest edition). Crack control joints should be spaced at intervals not greater than 8 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.

8.13.3 The moisture content of the slab subgrade at or near the ground surface should be maintained at least two percent over optimum moisture content prior to and at the time of concrete placement.

8.13.4 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.

8.14 Preliminary Pavement Recommendations

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

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

8.14.3 The Traffic Indices listed below are estimates. Geocon does not practice in the field of traffic engineering. The actual Traffic Index for each area should be determined by the project civil engineer. If pavement sections for Traffic Indices other than those listed below are required, Geocon should be contacted to provide additional recommendations. Pavement thicknesses were determined following procedures outlined in the *California Highway Design Manual* (Caltrans). It is anticipated that the majority of traffic will consist of automobile and large truck traffic.

PRELIMINARY PAVEMENT DESIGN SECTIONS

Location	Estimated Traffic Index (TI)	Asphalt Concrete (inches)	Class 2 Aggregate Base (inches)
Automobile Parking And Driveways	4.0	3.0	4.0
Trash Truck & Fire Lanes	7.0	4.0	12.0

8.14.4 Asphalt concrete should conform to Section 203-6 of the "*Standard Specifications for Public Works Construction*" (Green Book). Class 2 aggregate base materials should conform to Section 26-1.02A of the "*Standard Specifications of the State of California, Department of Transportation*" (Caltrans). The use of Crushed Miscellaneous Base (CMB) in lieu of Class 2 aggregate base is acceptable. Crushed Miscellaneous Base should conform to Section 200-2.4 of the "*Standard Specifications for Public Works Construction*" (Green Book).

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

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

8.15 Retaining Wall Design

8.15.1 The recommendations presented below are generally applicable to the design of rigid concrete or masonry retaining walls having a maximum height of 26 feet. In the event that walls higher than 26 feet are planned, Geocon should be contacted for additional recommendations.

8.15.2 Retaining wall foundations may be designed in accordance with the recommendations provided in the *Mat Foundation Design* section of this report (see Section 8.8).

8.15.3 Retaining walls with a level backfill surface that are not restrained at the top should be designed utilizing a triangular distribution of pressure (active pressure). Restrained walls are those that are not allowed to rotate more than 0.001H (where H equals the height of the retaining portion of the wall in feet) at the top of the wall. Where walls are restrained from movement at the top, walls may be designed utilizing a triangular distribution of pressure (at-rest pressure). The table below presents recommended pressures to be used in retaining wall design, assuming that proper drainage will be maintained.

RETAINING WALL WITH LEVEL BACKFILL SURFACE

HEIGHT OF RETAINING WALL (Feet)	ACTIVE PRESSURE EQUIVALENT FLUID PRESSURE (Pounds Per Cubic Foot)	AT-REST PRESSURE EQUIVALENT FLUID PRESSURE (Pounds Per Cubic Foot)
Up to 26	32	61

8.15.4 The wall pressures provided above assume that the proposed retaining walls will support relatively undisturbed alluvial soils. If sloping techniques are to be utilized for construction of proposed walls, which would result in a wedge of engineered fill behind the retaining walls, revised earth pressures may be required to account for the expansive potential of the soil placed as engineered fill. This should be evaluated once the use of sloping measures is established and once the geotechnical characteristics of the engineered backfill soils can be further evaluated.

8.15.5 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, an at-rest equivalent fluid pressure of 93 should be used in design of undrained, restrained walls for the full height of the wall. The value includes hydrostatic pressures plus buoyant lateral earth pressures. If a partially drained wall is proposed, Geocon should be contacted to provide additional recommendations.

8.15.6 Additional 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. 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.

8.15.7 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 or a distance from the shoring equal to at least half the shoring height, whichever is greater, the traffic surcharge may be neglected.

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

8.16 Dynamic (Seismic) Lateral Forces

8.16.1 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 2022 CBC).

8.16.2 A seismic load of 20 and 45 pcf should be used for design of displacing and non-displacing walls, respectively, which support more than 6 feet of backfill in accordance with Section 1803.5.12 of the 2022 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 static earth pressure. The earth pressure is based on a free field PGA of $S_{DS}/2.5$ and with a mean dynamic earth pressure coefficient of 0.16 and 0.36 for displacing and non-displacing walls, respectively (Mikola, 2013).

8.17 Retaining Wall Drainage

8.17.1 Unless designed for hydrostatic pressures, 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 9). 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.

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

8.17.3 Retaining wall plans or pipe submittals should be provided to Geocon for review and approval.

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

8.17.5 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.

8.18 Elevator Pit Design

8.18.1 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 *Mat Foundation Design* and *Retaining Wall Design* sections of this report (see Sections 8.8 and 8.15).

8.18.2 Additional 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.

8.18.3 The proposed structure must be designed for hydrostatic pressure for any portion of the structure below a depth of 15 feet. The hydrostatic design will result in uplift forces on the slab that that must be resisted by structural design. 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.

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

8.19 Elevator Piston

8.19.1 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.

8.19.2 Casing will likely 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 also be prepared to mitigate buoyant forces during installation of the piston casing. Continuous observation of the drilling and installation of the elevator piston by the Geotechnical Engineer (a representative of Geocon West, Inc.) is required.

8.19.3 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.

8.20 Temporary Excavations

8.20.1 Excavations on the order of 30 feet in vertical height may be required during excavation and construction of the proposed subterranean levels, including foundations and dewatering elements. The excavations are expected to expose artificial fill and alluvial soils. Where excavations are limited to the upper 8 feet (OSHA Type B soils), vertical excavations up to 4 feet in height may be attempted and where not surcharged by adjacent traffic or structures.

8.20.2 Vertical excavations greater than four 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 1½:1 slope gradient or flatter up to a maximum height of 12 feet. A uniform slope does not have a vertical portion. Where space is limited, shoring measures will be required. *Shoring* recommendations are provided in Section 8.21 of this report.

8.20.3 Where temporary construction slopes 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 slopes 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. The soils exposed in the cut slopes should be inspected during excavation by our personnel and the contractor's competent person 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.

8.21 Shoring – Soldier Pile Design and Installation

8.21.1 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.

8.21.2 One method of shoring would consist of steel soldier piles, placed in drilled holes and backfilled with concrete. This is the most common type of shoring system. However, further study will be required to determine if a soldier pile and lagging system is compatible with the temporary dewatering system. The steel soldier piles may also be installed utilizing high frequency vibration; however, if vibration installation techniques are considered, the potential impacts on adjacent properties must be evaluated and found acceptable and additional recommendations will be required. 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.

8.21.3 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 foundations and/or adjacent drainage systems.

8.21.4 The proposed soldier piles may also be designed as permanent piles. The required pile depth, dimension, and 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 8.14).

8.21.5 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 that are above the static groundwater level may be assumed to be 260 psf per foot. An allowable passive value for the soils below the plane of excavation below groundwater may be assumed to be 130 psf per foot (value has been reduced for buoyant forces). The allowable passive value may be doubled for isolated piles, spaced a minimum of 3 times the pile diameter. To develop the full lateral value, provisions should be implemented to assure firm contact between the soldier piles and the undisturbed alluvium.

8.21.6 Groundwater was encountered during our exploration at a depth of approximately 17 feet below the ground surface. 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.

8.21.7 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 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.

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

8.21.9 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 an allowable frictional resistance of 440 psf (value has been reduced for buoyant forces).

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

8.21.11 The time between lagging excavation and lagging placement should be as short as possible. Soldier piles should be designed for the full-anticipated pressures. Due to arching in the soils, the pressure on the lagging will be less. It is recommended that the lagging be designed for the full design pressure but be limited to a maximum of 400 psf. The 400 psf surcharge pressure on the lagging is due to the earth pressure only. Any potential surcharges should be evaluated in addition to earth pressures provided herein. The proper strength lagging board should be selected and incorporated into the shoring design by a qualified shoring engineer.

8.21.12 It is recommended that shoring be designed in accordance the equations provided below, which have been derived from the *State of California Department of Transportation Trenching and Shoring Manual, dated August 2011* for cohesionless soils. We reviewed the equations for both cohesionless and cohesive soils with respect to the soil properties underlying the site, and it is our opinion that the equations for cohesionless soil may be used. Diagrams depicting the trapezoidal pressure distribution of lateral earth pressure are provided on the following page.

For shoring with a single level of anchors or braces:	*For shoring with multilevel anchors:*
$\sigma_a = \frac{1.3P}{\frac{2}{3}H}$	$\sigma_a = \frac{1.3P}{\left[H - {}^{1}/_{3}(H_1 + H_{n+1})\right]}$

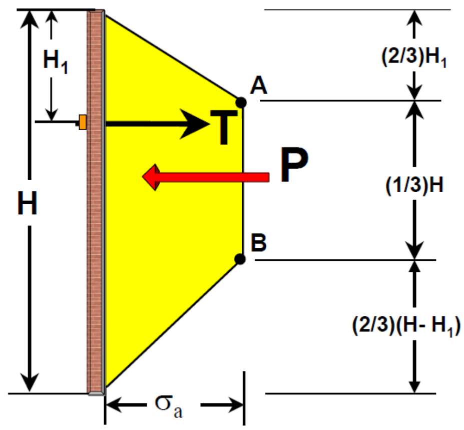


Figure 7-2. Pressure Diagram for Single Anchored/Braced Wall

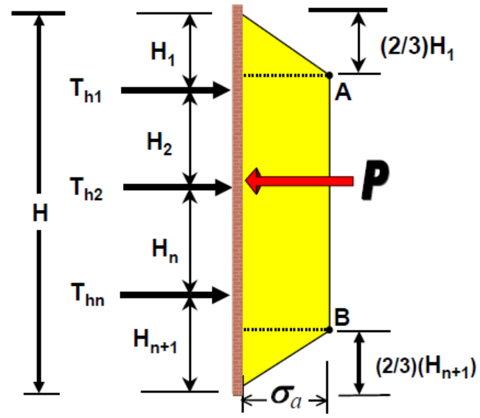


Figure 7-3. Pressure Diagram for Multi Anchored/Braced Wall

where: $P = 20H^2$
H = total height of shoring (in feet)
H_1 = distance from top of shoring to uppermost anchor (in feet)
H_{n+1} = distance from bottom of shoring to lowermost anchor (in feet)
n = number of anchors

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

8.21.14 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 or a distance from the shoring equal to at least half the shoring height, whichever is greater, the traffic surcharge may be neglected.

8.21.15 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.

8.21.16 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.

8.21.17 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.

8.22 Temporary Tie-Back Anchors

8.22.1 Temporary tie-back anchors may be used with the soldier 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.

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

- 7 feet below the top of the excavation – 740 psf
- 15 feet below the top of the excavation – 630 psf*

8.22.3 Depending on the techniques utilized, and the experience of the contractor performing the installation, a maximum allowable friction capacity of 2 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.

8.23 Anchor Installation

8.23.1 It is recommended that tie-back anchors be installed between 10 and 30 degrees below the horizontal in accordance with PTI DC35.1-14. Alternative angles are necessary to avoid existing improvements and utilities, and should be reviewed by the Geotechnical Engineer. 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.

8.24 Anchor Testing

8.24.1 It is recommended that anchor testing be performed in accordance with the procedures outlined in PTI DC35.1-14.

8.24.2 The deflection criteria should be based on the theoretical elongation of each anchor. The theoretical elongation of the anchors should be provided by the project shoring engineer.

8.24.3 At least 2 to 3 anchors should be selected for performance tests, and a minimum of 2 percent of the remaining anchors should be performance tested. 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.

8.24.4 All anchors not subjected to a performance test should be proof tested. 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.

8.25 Internal Bracing

8.25.1 Rakers may be utilized to brace the soldier piles in lieu of tieback anchors. The raker bracing could be supported laterally by temporary concrete footings (deadmen) or by the permanent, interior footings. For design of such temporary footings or deadmen, poured with the bearing surface normal to rakers inclined at 45 degrees, a bearing value of 2,500 psf in competent alluvial soil, provided the shallowest point of the footing is at least one foot below the lowest adjacent grade. The client should be aware that the utilization of rakers could significantly impact the construction schedule due to their intrusion into the construction site and potential interference with equipment. The structural engineer should review the shoring plan to determine if the raker footings conflict with the structural foundation system.

8.26 Surcharge from Adjacent Structures and Improvements

8.26.1 Additional 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.

8.26.2 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 or wall to the vertical line-load, H is the distance from the bottom of the footing to the bottom of excavation or wall, z is the depth at which the horizontal pressure is desired, Q_L is the vertical line-load and $\sigma_H(z)$ is the horizontal pressure at depth z.

8.26.3 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/wall to the vertical point-load, H is distance from the outrigger/bottom of column footing to the bottom of excavation, z is the depth at which the horizontal pressure is desired, Q_P is the vertical point-load, $\sigma_H(z)$ is the horizontal pressure at depth z, θ is the angle between a line perpendicular to the excavation/wall and a line from the point-load to location on the excavation/wall where the surcharge is being evaluated, and $\sigma_H(z)$ is the horizontal pressure at depth z.

8.27 Surface Drainage

8.27.1 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.

8.27.2 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 2022 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 five feet of the building perimeter footings except when enclosed in protected planters.

8.27.3 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.

8.27.4 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.

8.28 Plan Review

8.28.1 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.

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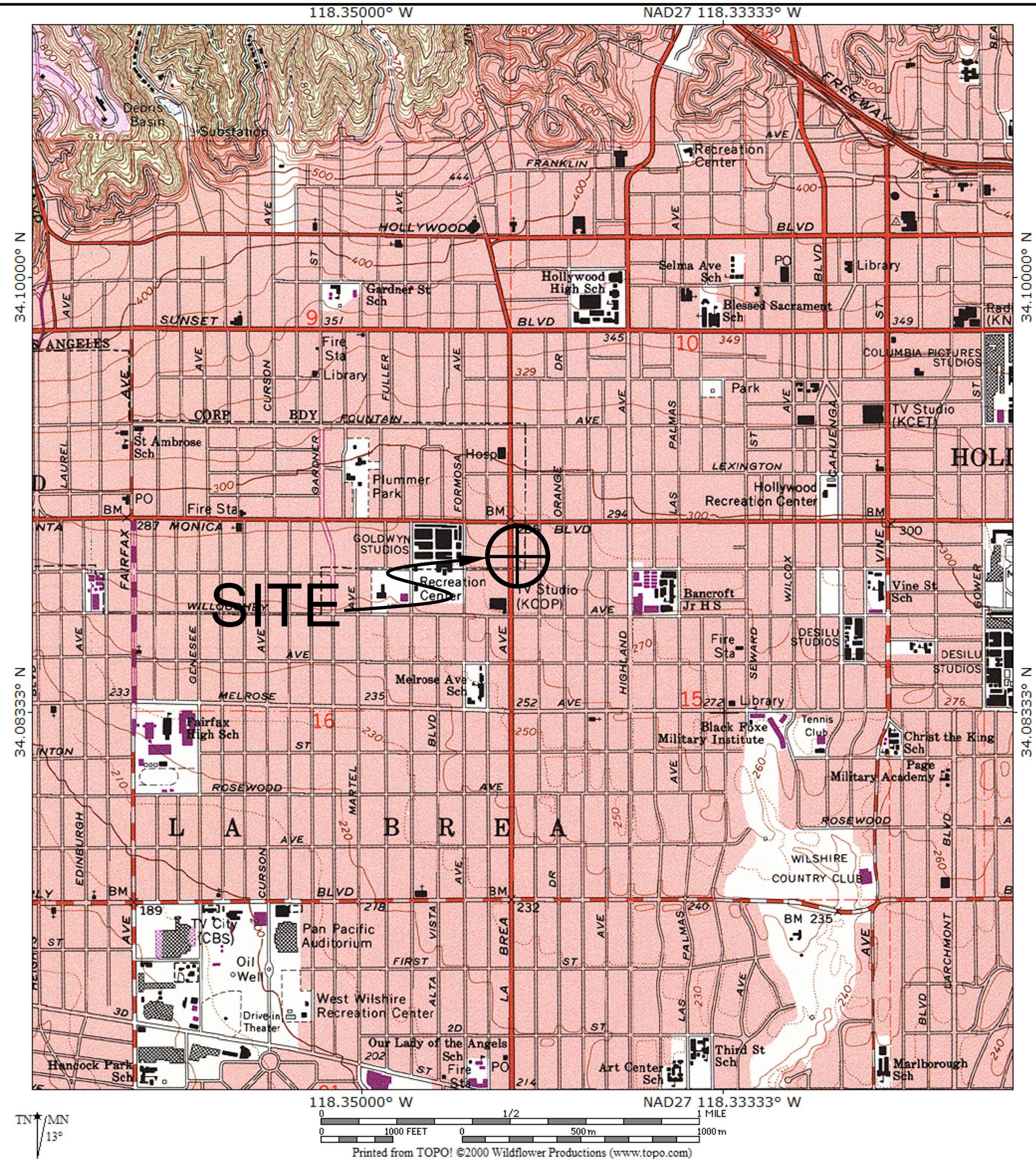

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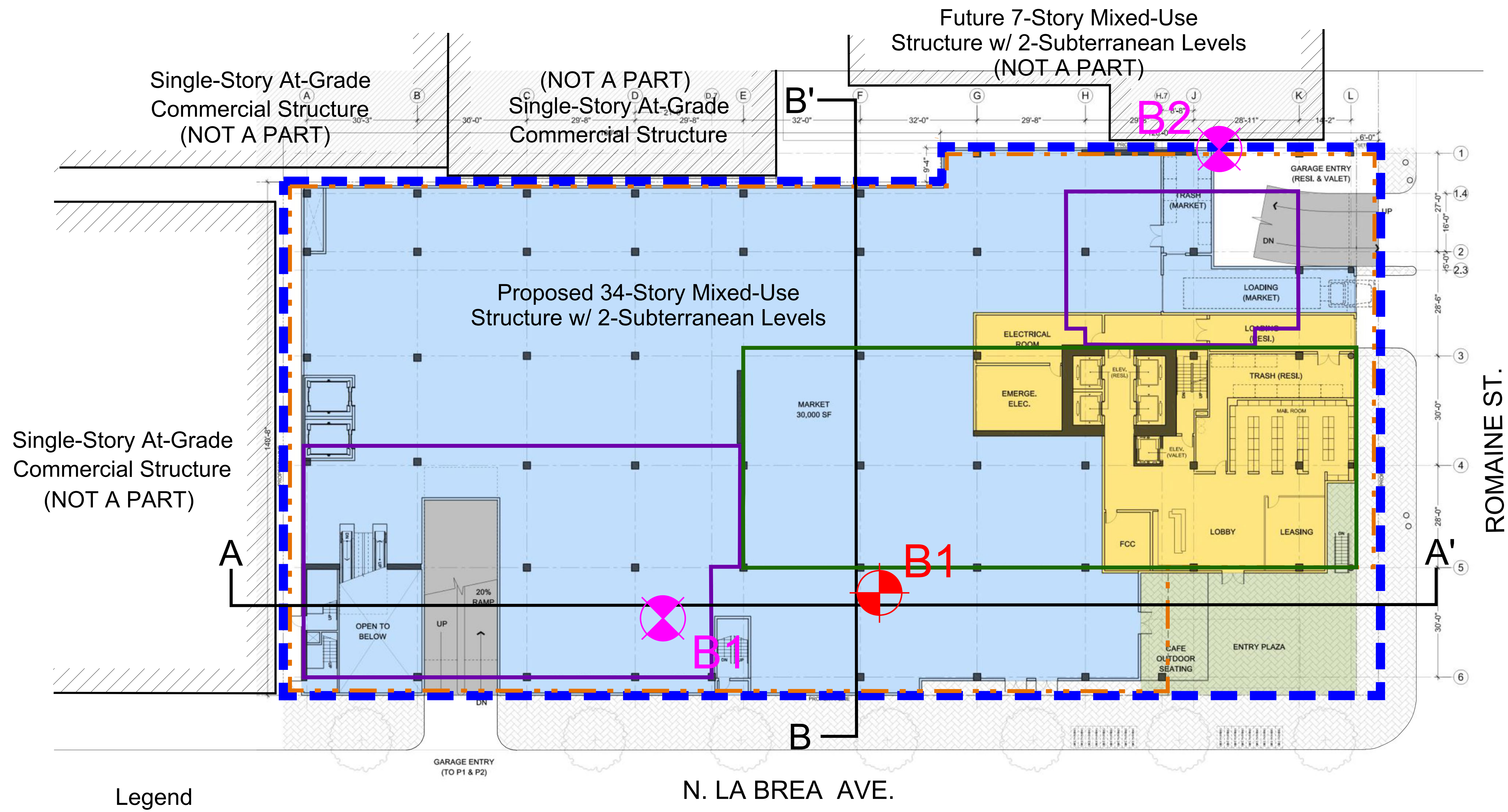
VICINITY MAP

1000, 1014 & 1020 NORTH LA BREA AVENUE
WEST HOLLYWOOD, CALIFORNIA

MAY 2023	PROJECT NO. W1718-06-01	FIG. 1



Legend

- B1 Boring Location (Geocon 2023)
- B2 Prior Boring Locations (Geotechnologies 2019)
- Limit of Subterranean Parking Levels
- Limits of 17-Story Portions of Tower
- Limit of 34-Story Portion of Tower
- Property Limits

0 30' 60'

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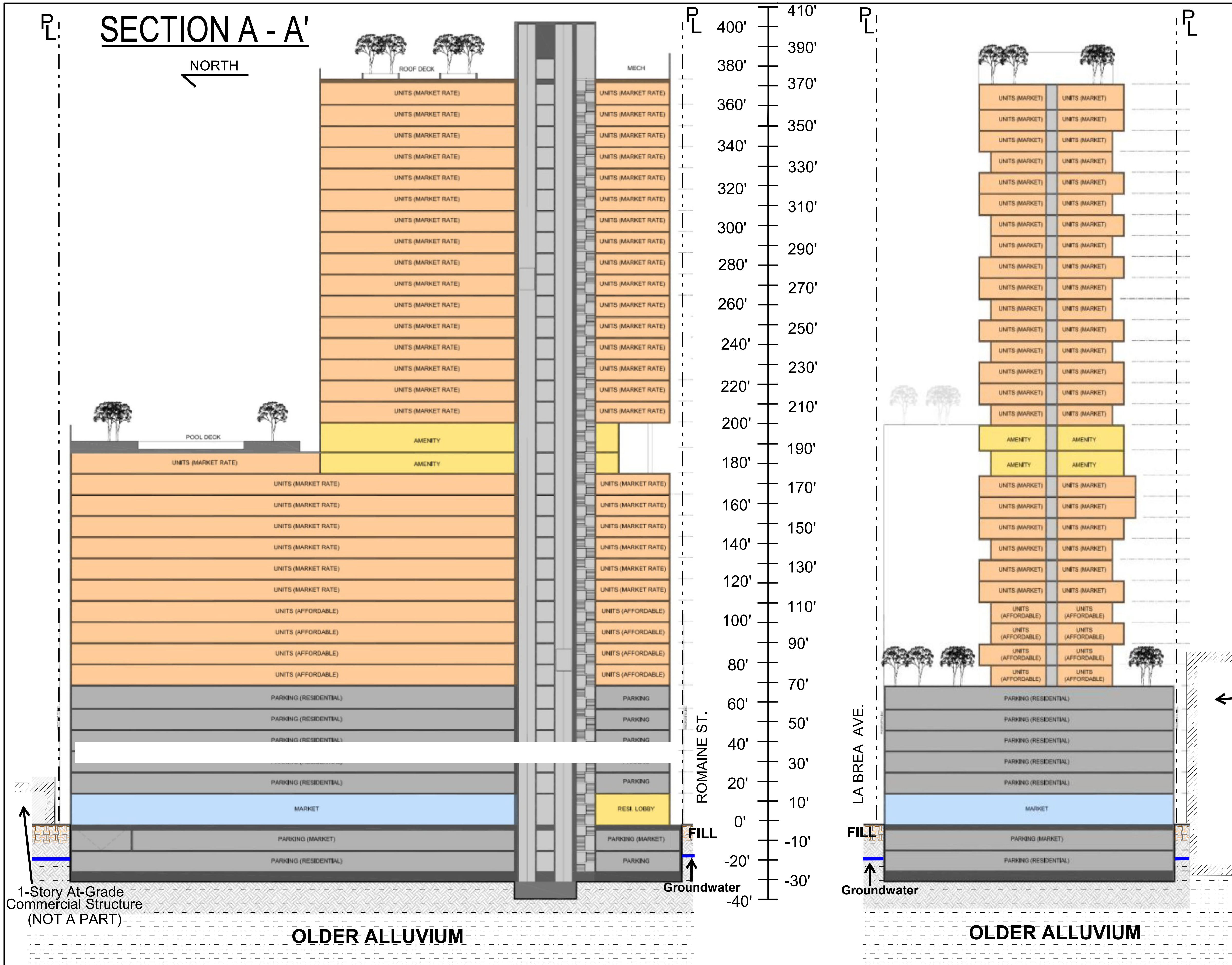
SITE PLAN

1000, 1014 & 1020 NORTH LA BREA AVENUE
WEST HOLLYWOOD, CALIFORNIA

MAY 2023 | PROJECT NO: W1718-06-01 | FIG. 2A



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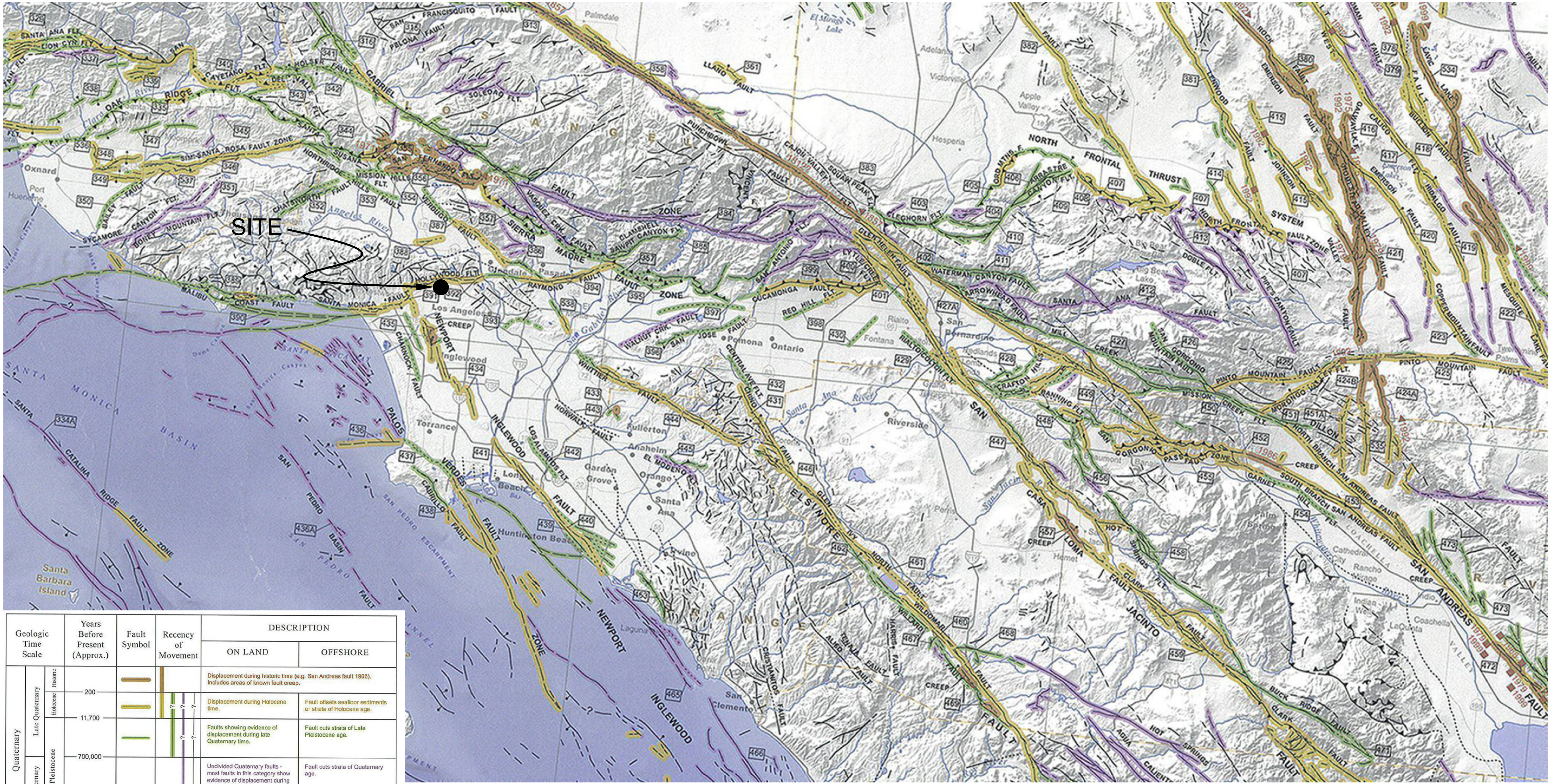
CROSS SECTIONS

1000, 1014, & 1020 NORTH LA BREA AVENUE

WEST HOLLYWOOD, CALIFORNIA

MAY 2023	PROJECT NO: W1718-06-01	FIG. 2B

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	200			Displacement during historic time (e.g. San Andreas fault 1906). Includes areas of known fault creep.	
Quaternary	Late Quaternary	Holocene	11,700		? ? ?	Displacement during Holocene time.	Fault offsets seafloor sediments or strata of Holocene age.
Quaternary	Late Quaternary	Pleistocene	700,000		? ?	Faults showing evidence of displacement during late Quaternary time.	Fault cuts strata of Late Pleistocene age.
Quaternary	Early Quaternary	Pleistocene	1,600,000*		?	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			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.

N

0 12 24 Miles

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REGIONAL FAULT MAP

1000 1014 & 1020 NORTH LA BREA AVENUE
WEST HOLLYWOOD, CALIFORNIA

MAY 2023 | PROJECT NO. W1718-06-01 | FIG. 3

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.

N

0 20 40 Miles

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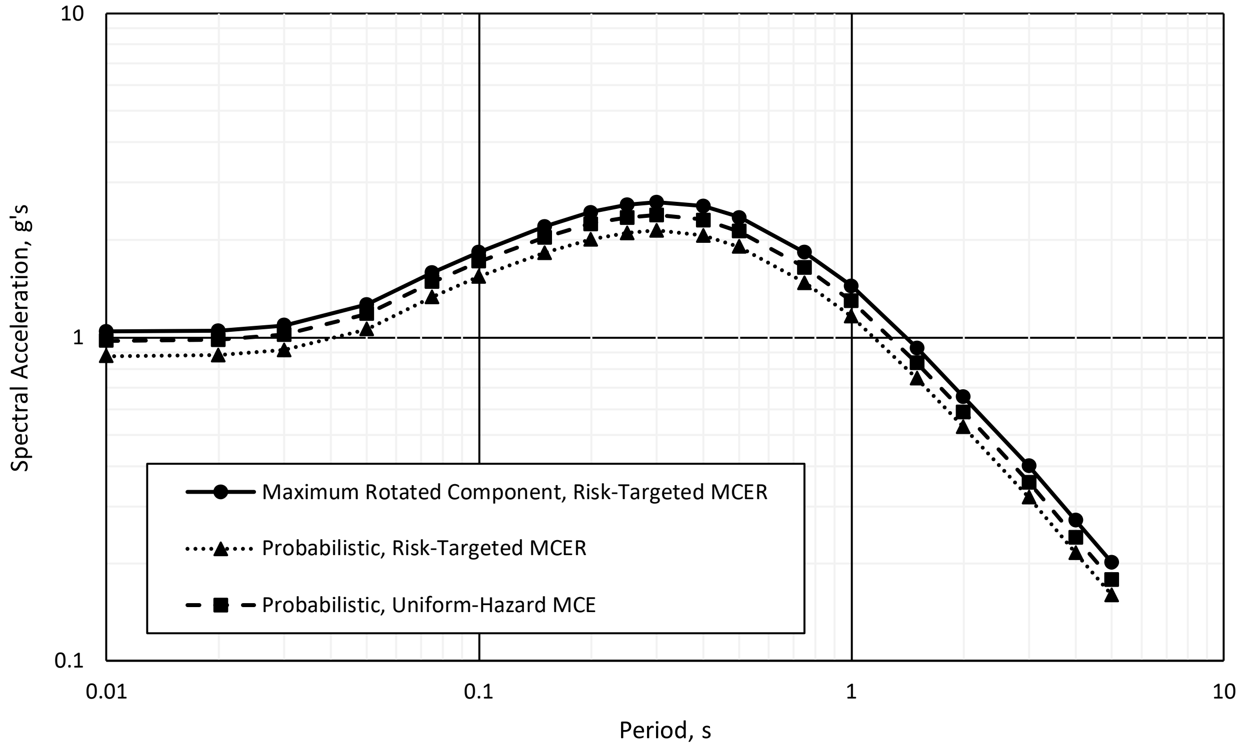
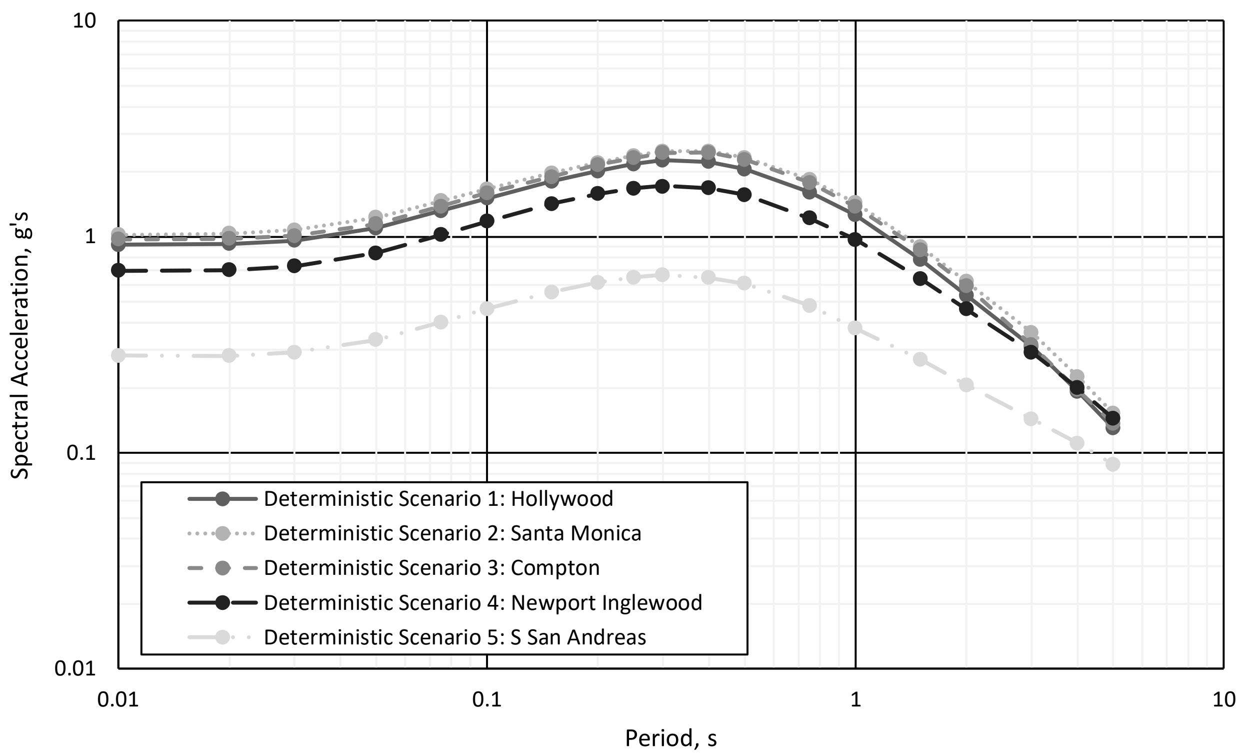
REGIONAL SEISMICITY MAP

1000 1014 & 1020 NORTH LA BREA AVENUE
WEST HOLLYWOOD, CALIFORNIA

DRAFTED BY: RA	CHECKED BY: SFK	MAY 2023	PROJECT NO. W1718-06-01	FIG.4





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DESIGN RESPONSE SPECTRUM

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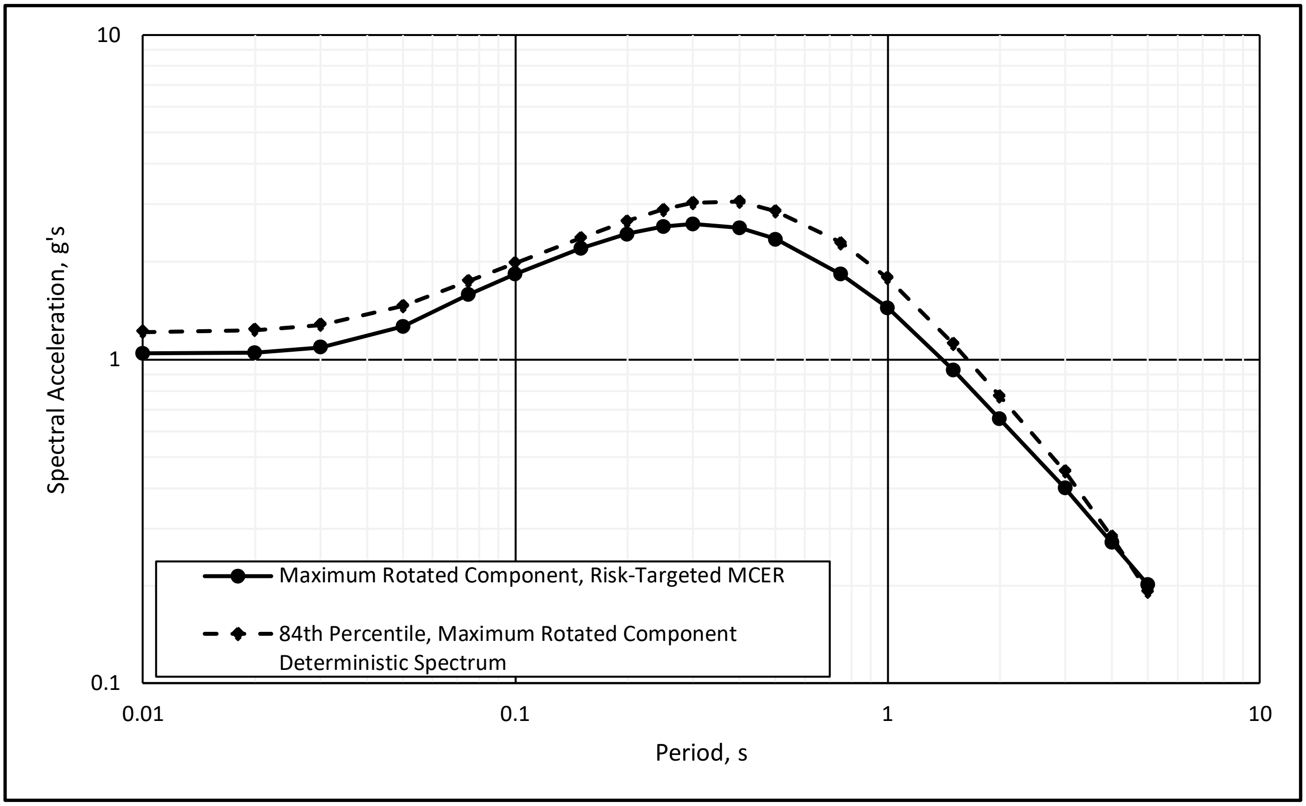
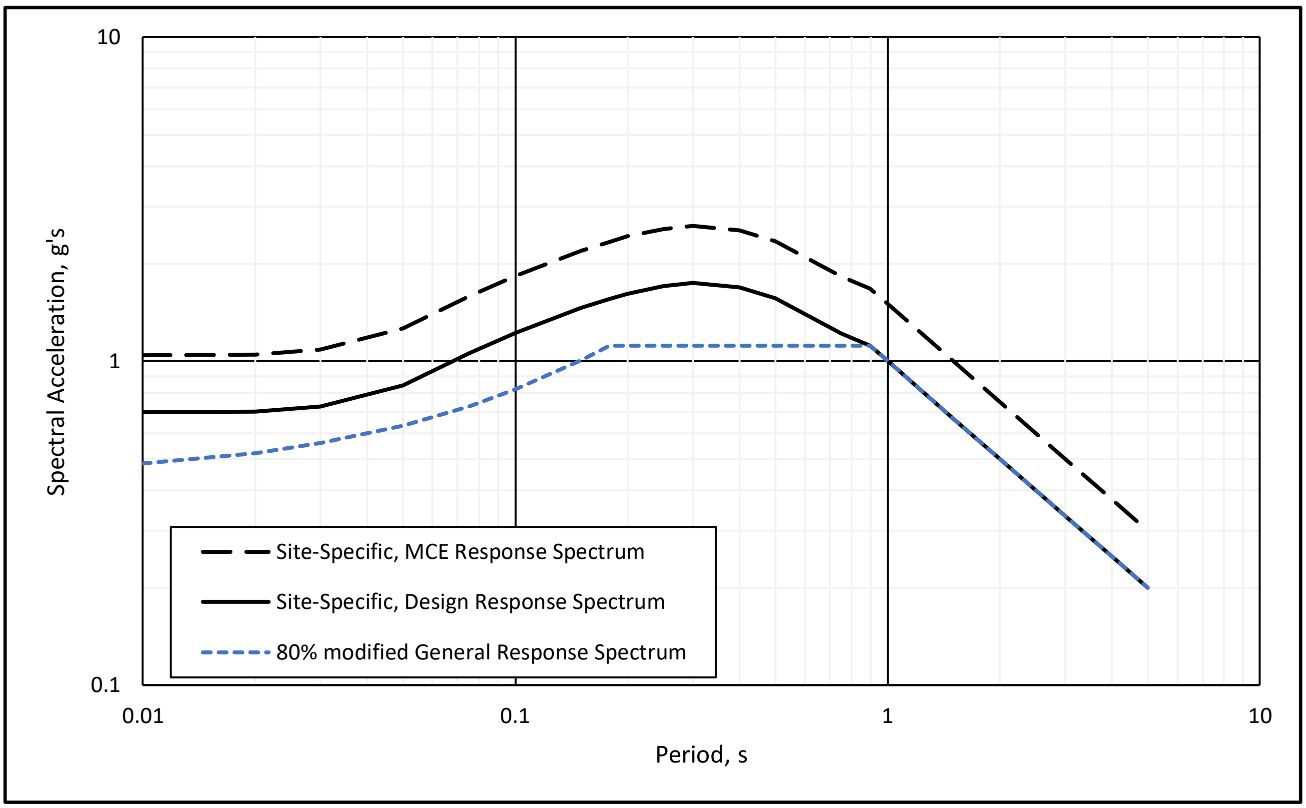
Project No.: W1718-06-01

1000, 1014 & 1020 NORTH LA BREA AVENUE
WEST HOLLYWOOD, CALIFORNIA

MAY 2023

Figure 5





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DESIGN RESPONSE SPECTRUM

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Project No.: W1718-06-01

1000, 1014 & 1020 NORTH LA BREA AVENUE
WEST HOLLYWOOD, CALIFORNIA

MAY 2023

Figure 6

Spectral Period (seconds)	Probabilistic Uniform-Hazard	Risk-Targeted, Probabilistic	Risk Factor, Cr	Maximum-Rotated Component Scale Factor	MRC, Risk-Targeted Probabilistic	84th Percentile, Deterministic	Site-Specific Design Earthquake	80% Modified General Response Spectrum	Site-Specific Maximum Considered Earthquake
0.01	0.977	0.877	0.929	1.19	1.043	1.214	0.695	0.484	1.043
0.02	0.983	0.882	0.929	1.19	1.049	1.232	0.699	0.521	1.049
0.03	1.020	0.915	0.934	1.19	1.089	1.277	0.726	0.559	1.089
0.05	1.183	1.061	0.942	1.19	1.263	1.462	0.842	0.633	1.263
0.08	1.486	1.333	0.938	1.19	1.586	1.746	1.057	0.727	1.586
0.10	1.719	1.542	0.950	1.19	1.835	1.981	1.223	0.820	1.835
0.15	2.038	1.828	0.949	1.20	2.193	2.363	1.462	1.007	2.193
0.18	--	--	--	--	--	--	1.557	1.116	2.336
0.20	2.238	2.007	0.938	1.21	2.429	2.661	1.619	1.116	2.429
0.25	2.342	2.101	0.933	1.22	2.563	2.890	1.708	1.116	2.563
0.30	2.389	2.143	0.926	1.22	2.614	3.031	1.743	1.116	2.614
0.40	2.301	2.064	0.914	1.23	2.538	3.065	1.692	1.116	2.538
0.50	2.124	1.905	0.907	1.23	2.344	2.860	1.562	1.116	2.344
0.75	1.643	1.474	0.905	1.24	1.828	2.283	1.219	1.116	1.828
0.90	--	--	--	--	--	--	1.116	1.116	1.674
1.0	1.294	1.161	0.901	1.24	1.440	1.782	1.000	1.000	1.500
1.5	0.833	0.748	0.901	1.24	0.927	1.118	0.667	0.667	1.000
2.0	0.589	0.528	0.900	1.24	0.655	0.769	0.500	0.500	0.750
3.0	0.357	0.320	0.903	1.25	0.400	0.451	0.333	0.333	0.500
4.0	0.241	0.216	0.904	1.26	0.272	0.283	0.250	0.250	0.375
5.0	0.178	0.160	0.900	1.26	0.202	0.192	0.200	0.200	0.300

SM_S = 2.353 g

SM_1 = 1.500 g

SD_S = 1.568 g

SD_1 = 1.000 g

Reference: ASCE 7-16 21.4 DESIGN ACCELERATION PARAMETERS

Where the site-specific procedure is used to determine the design ground motion in accordance with Section 21.3, the parameter S_{DS} shall be taken as 90% of the maximum spectral acceleration, S_a, obtained from the site-specific spectrum, at any period within the range from 0.2 to 5 s, inclusive. The parameter S_{D1} shall be taken as the maximum value of the product, TS_a, for periods from 1 to 2 s for sites with $v_{s,30}$ > 1,200 ft/s ($v_{s,30}$ > 365.76 m/s) and for periods from 1 to 5 s for sites with $v_{s,30}$ ≤ 1,200 ft/=s ($v_{s,30}$ ≤ 365.76 m/s). The parameters S_{MS} and S_{M1} shall be taken as 1.5 times S_{DS} and S_{D1}, respectively. The values so obtained shall not be less than 80% of the values determined in accordance with Section 11.4.3 for S_{MS} and S_{M1} and Section 11.4.5 for S_{DS} and S_{D1}.

Spectral acceleration values reported in units of "g". "--" Indicates that spectral period was not used at that calculation step

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DESIGN RESPONSE SPECTRUM

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Project No.: W1718-06-01

1000, 1014 & 1020 NORTH LA BREA AVENUE
WEST HOLLYWOOD, CALIFORNIA

MAY 2023

Figure 7

Parameter	Scenario 1	Scenario 2	Scenario 3	Scenario 4	Scenario 5	Reference
Parent Fault Name	Hollywood	Santa Monica	Compton	Newport Inglewood	S San Andreas	--
Scenario Name	Hollywood	Santa Monica Alt 2	Compton	Newport Inglewood Alt 2	S. San Andreas: PK+CH+CC+BB+NM+SM+NSB+SSB+BG+CO	BSSC Online Scenario Catalog
Earthquake Magnitude	6.7	6.78	7.45	7.15	8.18	BSSC Online Scenario Catalog
Fault Mechanism	Left Lateral Reverse	Left Reverse	Blind Thrust	Right Lateral Strike-Slip	--	--
Fault Dip (˚)	70	50	20	90	86.4	BSSC 2014 [1]
Fault Width	16.57	13.63	27.37	13.59	13.1	BSSC 2014 [1]
Rake (˚)	30	30	90	180	180	BSSC 2014 [1]
Z_{TOR} (km)	0	0	5.2	0	0	BSSC 2014 [1]
Rrup (km)	1.66	0.08	14.44	6.28	55.46	--
Rjb (km)	1.66	0	2.23	6.28	55.46	--
Rx (km)	1.66	0.1	27.94	6.28	55.46	--
V_{s30} (m/s)	351.7	351.7	351.7	351.7	351.7	Site-Specific Measurement
$Z_{1.0}$ (km)	0.35	0.35	0.35	0.35	0.35	SCEC Community Velocity Model Version 4, Iteration 26, Basin Depth
$Z_{2.5}$ (km)	2	2	2	2	2	SCEC Community Velocity Model Version 4, Iteration 26, Basin Depth

1 - BSSC 2014, aka. UCERF3_EventSet_All on GitHub

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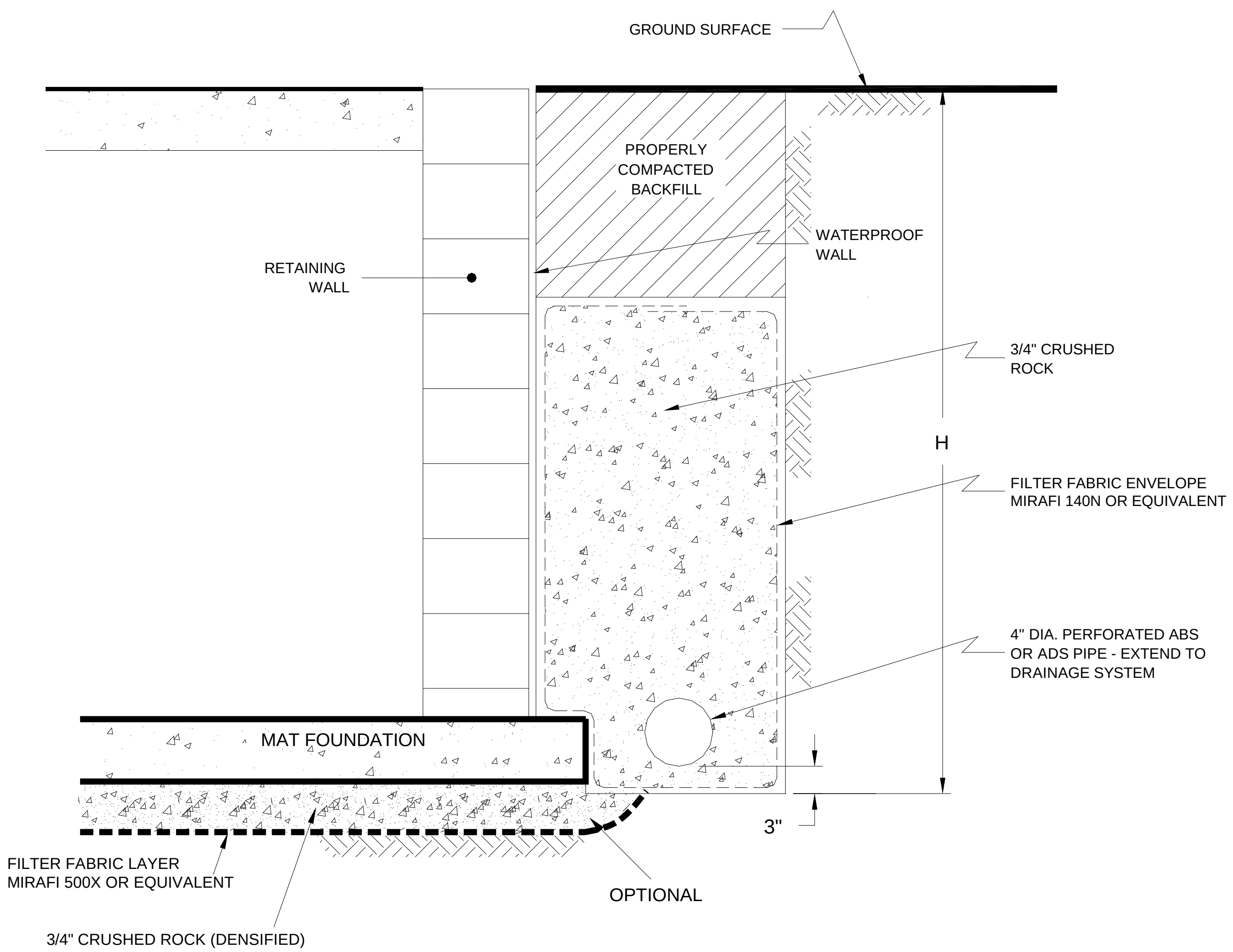
DETERMINISTIC SCENARIO EVENTS

Checked by: JJK

Project No.: W1718-06-01

1000, 1014 & 1020 NORTH LA BREA AVENUE
WEST HOLLYWOOD, CALIFORNIA

MAY 2023 Figure 8



NO SCALE

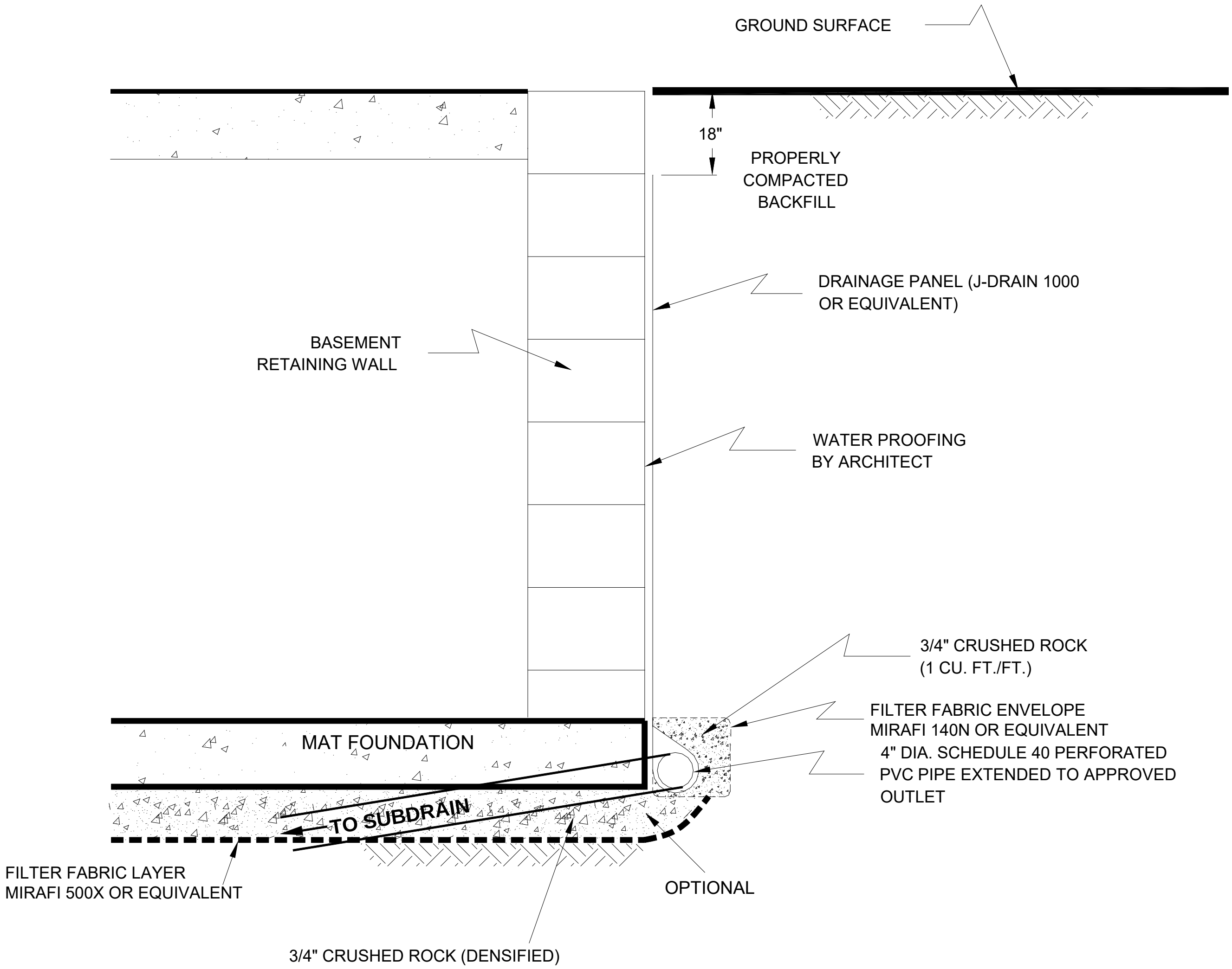
GEOCON
WEST, INC.

ENVIRONMENTAL GEOTECHNICAL MATERIALS
500 NORTH VICTORY BOULEVARD, BURBANK CA 91502
PHONE (818) 841-8388 - FAX (818) 841-1704

RETAINING WALL DRAIN DETAIL

1000, 1014 & 1020 NORTH LA BREA AVENUE
WEST HOLLYWOOD, CALIFORNIA

DRAFTED BY: JJK	CHECKED BY: PZ	MAY 2023	PROJECT NO. W1720-06-01	FIG. 9



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

NO SCALE

GEOCON
WEST, INC.
ENVIRONMENTAL GEOTECHNICAL MATERIALS
500 NORTH VICTORY BOULEVARD, BURBANK CA 91502
PHONE (818) 841-8388 - FAX (818) 841-1704

RETAINING WALL DRAIN DETAIL

1000, 1014 & 1020 NORTH LA BREA AVENUE
WEST HOLLYWOOD, CALIFORNIA

DRAFTED BY: JJK	CHECKED BY: NDB	MAY 2023	PROJECT NO. W1720-06-01	FIG. 10

APPENDIX A

APPENDIX A

FIELD INVESTIGATION

The site was explored on January 30, 2023, by excavating one 7-inch diameter boring to a depth of approximately 120½ feet below the existing ground surface using 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. 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.

The soil conditions encountered in the boring were visually examined, classified and logged in general accordance with the Unified Soil Classification System (USCS). The log of the boring is presented on Figure A1. The log depicts the soil and geologic conditions encountered and the depth at which samples were obtained. The log also includes our interpretation of the conditions between sampling intervals. Therefore, the log contains both observed and interpreted data. We determined the lines designating the interface between soil materials on the log using visual observations, penetration rates, excavation characteristics and other factors. The transition between materials may be abrupt or gradual. Where applicable, the log was revised based on subsequent laboratory testing. The location of the boring is shown on Figure 2A.

BORING 1

ELEV. (MSL.) - - DATE COMPLETED 01/30/23

EQUIPMENT HOLLOW STEM AUGER BY: JJK

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0					**CONCRETE: 8"** **ARTIFICIAL FILL** Clay, soft to firm, moist, dark brown to black.			
4					**ALLUVIUM** Sandy Clay, firm, moist, dark reddish brown, fine-grained, trace medium-grained.			
	B1@5'			CL	- stiff, dark brown	24	113.9	16.1
8					Clayey Sand, dense, slightly moist, reddish brown, fine-grained.			
10	B1@10'					64	107.1	19.0
				SC				
	B1@15'				- medium dense	40	101.9	22.3
			▼					
20	B1@20'				- loose, moist to very moist, reddish brown and olive gray	17	97.7	27.7
					Silty Sand, medium dense, very moist, reddish brown, fine-grained.			
	B1@22.5'					32	102.3	23.8
	B1@25'			SM	- wet, trace coarse-grained	42	103.5	20.6
					- no coarse-grained			
28	B1@27.5'					42	110.9	16.6

W1718-06-01 BORING LOG.GPJ

Figure A1,
Log of Boring 1, Page 1 of 5

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

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

GEOCON

BORING 1

ELEV. (MSL.) - - DATE COMPLETED 01/30/23

EQUIPMENT HOLLOW STEM AUGER BY: JJK

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 medium-grained	28	109.9	18.1
32	B1@32.5'				- dense, trace fine gravel	73	107.0	19.1
34								
36	B1@35'				- medium dense, fine-grained, trace fine to coarse gravel	48	107.3	19.1
38								
40	B1@40'				- very dense, some coarse-grained	50 (6")	122.3	14.3
42								
44	B1@45'			ML	Sandy Silt, hard, very moist, brown to reddish brown, fine- to medium-grained, trace fine- to coarse-grained.	50 (5.5")	99.3	21.3
46								
48								
50	B1@50'				- no recovery	53		
52								
54								
56	B1@55'			CL	Clay, hard, moist, brown.	65	99.9	25.1
58				SM	Silty Sand, very dense, wet, brown, fine-grained, some coarse-grained.			

Figure A1,
Log of Boring 1, Page 2 of 5

W1718-06-01 BORING LOG.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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BORING 1

ELEV. (MSL.) - - DATE COMPLETED 01/30/23

EQUIPMENT HOLLOW STEM AUGER BY: JJK

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
60	B1@60'			SM		50 (6")	115.0	17.5
62				SP	Sand, poorly graded, very dense, wet, grayish brown, fine-grained, some medium-grained.			
64								
66	B1@65'					50 (5")	126.8	8.4
68				SM	Silty Sand, dense, wet, yellowish brown, fine-grained, some coarse gravel.			
70	B1@70'					65	109.6	19.3
72								
74								
76				CL	Sandy Clay, stiff, moist, light reddish brown and light gray, fine-grained.			
78								
80	B1@80'					42	105.8	20.9
82								
84								
86								
88								

Figure A1,
Log of Boring 1, Page 3 of 5

W1718-06-01 BORING LOG.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 01/30/23

EQUIPMENT HOLLOW STEM AUGER BY: JJK

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
90	B1@90'				- hard, olive gray and light olive brown with reddish brown mottles	50 (6")	101.7	30.3
					Silty Sand, very dense, wet, brown, fine-grained.			
92								
94								
96								
98								
100	B1@100'				- no recovery			
102								
104								
	B1@105'			SM	- no recovery	50 (3")		
106								
108								
110								
112								
114								
116								
118								

Figure A1,
Log of Boring 1, Page 4 of 5

W1718-06-01 BORING LOG.GPJ

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

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

GEOCON

BORING 1

ELEV. (MSL.) - - DATE COMPLETED 01/30/23

EQUIPMENT HOLLOW STEM AUGER BY: JJK

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
120	B1@120'			SM	- no recovery	50 (2")		

Total depth of boring: 120.5 feet
Fill to 3 feet.
Groundwater encountered at 17 feet.
Backfilled with grout.
Concrete patched.

*Penetration resistance for 140-pound hammer falling 30 inches by auto-hammer.
NOTE: The stratification lines presented herein represent the approximate boundary between earth types; the transitions may be gradual.

Figure A1,
Log of Boring 1, Page 5 of 5

W1718-06-01 BORING LOG.GPJ

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

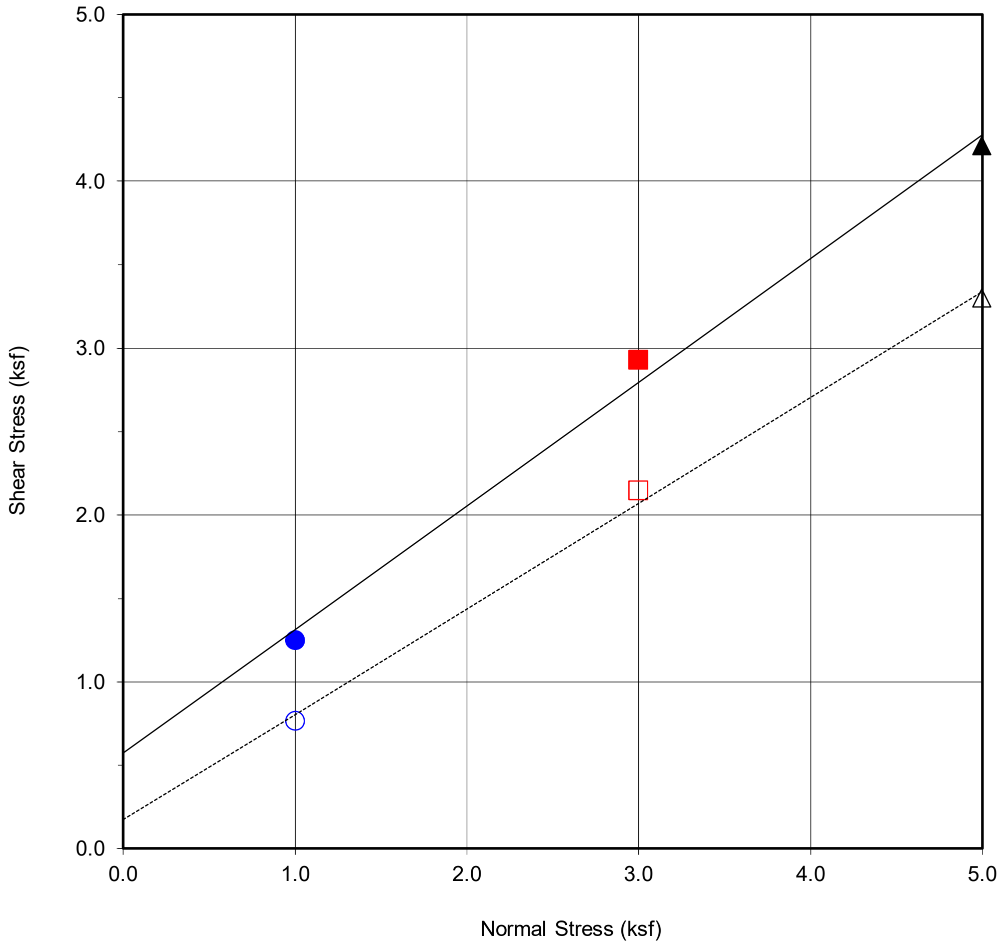
NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

APPENDIX B

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, and in-place dry density and moisture content. The results of the laboratory tests are summarized in Figures B1 through B13. The in-place dry density and moisture content of the samples tested are presented on the boring log, Appendix A.



Boring No.	**B1**
Sample No.	**B1@10'**
Depth (ft)	**10**
Sample Type:	Ring

Soil Identification:		
Clayey Sand (SC)		
Strength Parameters		
	C (psf)	ϕ (°)
Peak	573	37
Ultimate	173	32

Normal Strest (kip/ft2)	1	3	5
Peak Shear Stress (kip/ft²)	● 1.25	■ 2.93	▲ 4.21
Shear Stress @ End of Test (ksf)	○ 0.77	□ 2.15	△ 3.30
Deformation Rate (in./min.)	0.05	0.05	0.05
Initial Sample Height (in.)	1.0	1.0	1.0
Ring Inside Diameter (in.)	2.375	2.375	2.375
Initial Moisture Content (%)	18.4	19.3	19.0
Initial Dry Density (pcf)	105.5	103.4	108.5
Initial Degree of Saturation (%)	83.1	83.0	92.8
Soil Height Before Shearing (in.)	1.2	1.2	1.2
Final Moisture Content (%)	23.6	23.4	22.4

GEOCON

DIRECT SHEAR TEST RESULTS

Consolidated Drained ASTM D-3080

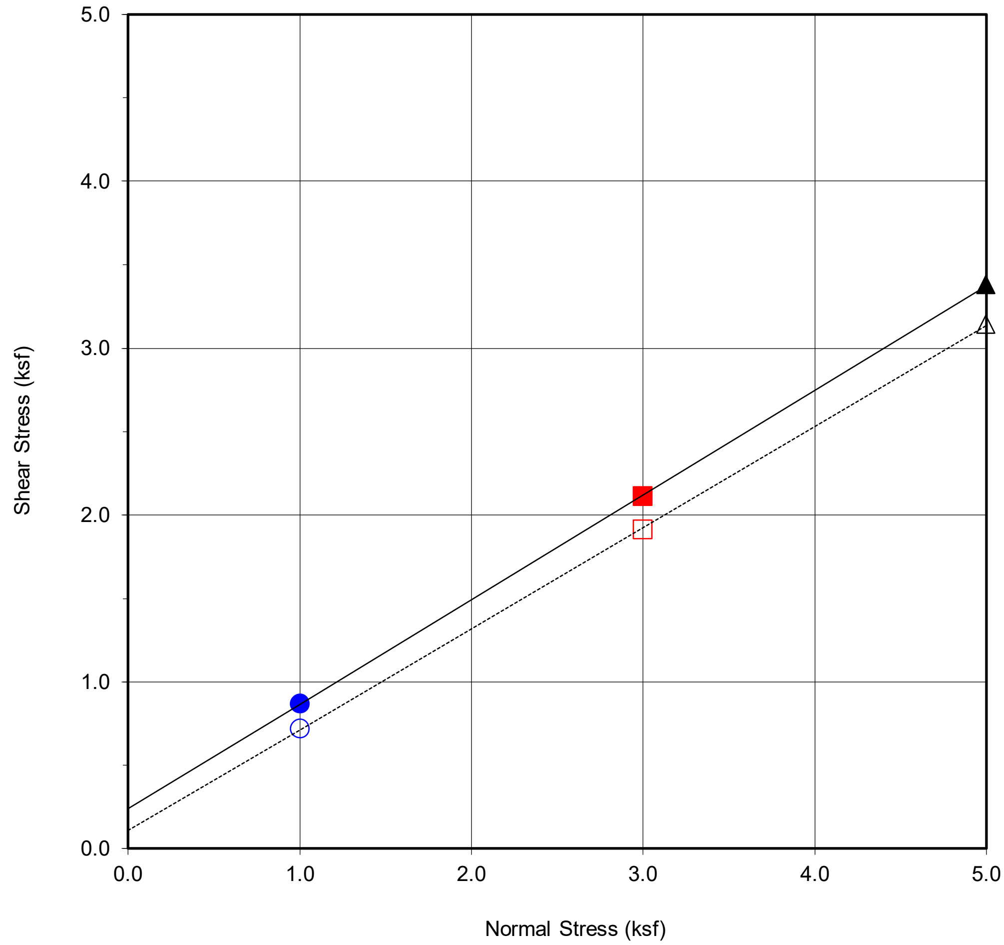
Checked by: JJK

Project No.: W1718-06-01

1000, 1014 & 1020 NORTH LA BREA AVENUE
WEST HOLLYWOOD, CALIFORNIA

MAY 2023

Figure B1



Boring No.	**B1**
Sample No.	**B1@22.5'**
Depth (ft)	**22.5**
Sample Type:	Ring

Soil Identification:		
Silty Sand (SM)		
Strength Parameters		
	C (psf)	ϕ (°)
Peak	238	32
Ultimate	108	31

Normal Stress (kip/ft2)	1	3	5
Peak Shear Stress (kip/ft²)	● 0.87	■ 2.11	▲ 3.38
Shear Stress @ End of Test (ksf)	○ 0.72	□ 1.91	△ 3.14
Deformation Rate (in./min.)	0.05	0.05	0.05
Initial Sample Height (in.)	1.0	1.0	1.0
Ring Inside Diameter (in.)	2.375	2.375	2.375
Initial Moisture Content (%)	25.2	27.2	23.8
Initial Dry Density (pcf)	99.7	96.8	102.4
Initial Degree of Saturation (%)	98.7	99.2	99.7
Soil Height Before Shearing (in.)	1.2	1.2	1.2
Final Moisture Content (%)	23.6	24.7	20.9

GEOCON

DIRECT SHEAR TEST RESULTS

Consolidated Drained ASTM D-3080

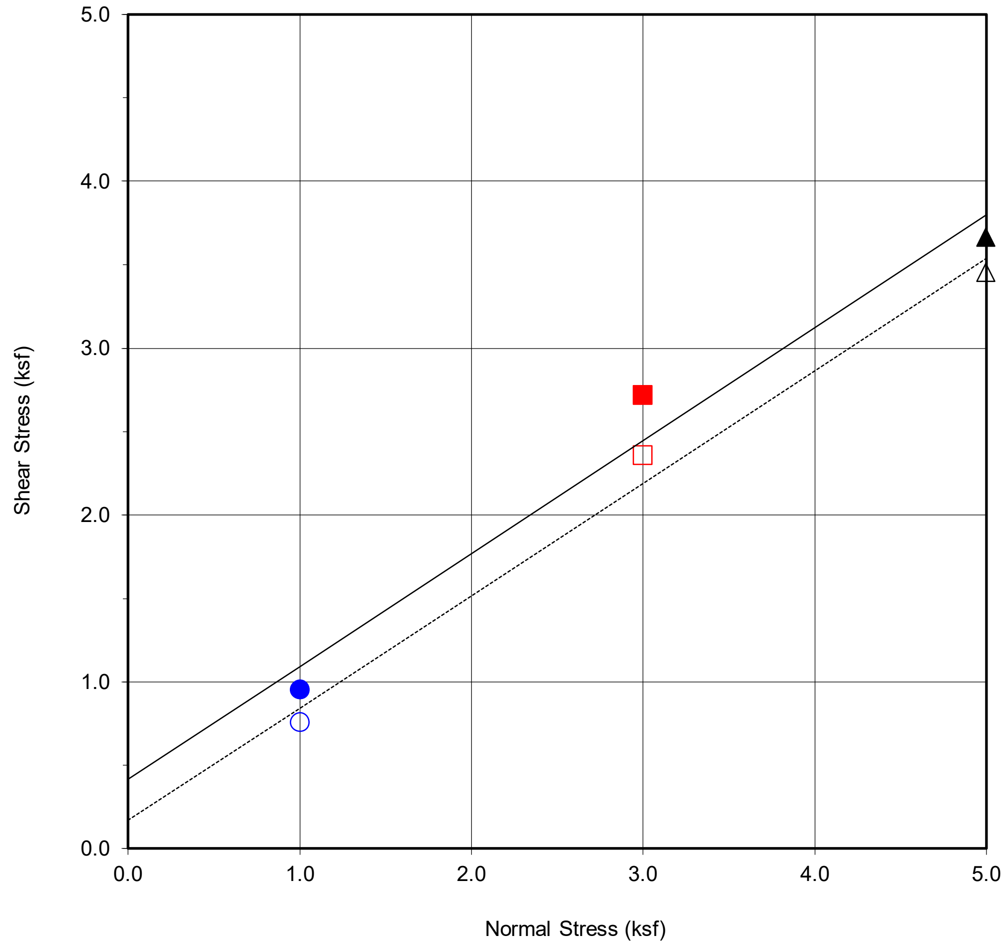
Checked by: JJK

Project No.: W1718-06-01

1000, 1014 & 1020 NORTH LA BREA AVENUE
WEST HOLLYWOOD, CALIFORNIA

MAY 2023

Figure B2



Boring No.	**B1**
Sample No.	**B1@30**
Depth (ft)	**30**
Sample Type:	Ring

Soil Identification:		
Silty Sand (SM)		
Strength Parameters		
	C (psf)	ϕ (°)
Peak	416	34
Ultimate	168	34

Normal Stress (kip/ft2)	1	3	5
Peak Shear Stress (kip/ft²)	● 0.96	■ 2.72	▲ 3.66
Shear Stress @ End of Test (ksf)	○ 0.76	□ 2.36	△ 3.45
Deformation Rate (in./min.)	0.05	0.05	0.05
Initial Sample Height (in.)	1.0	1.0	1.0
Ring Inside Diameter (in.)	2.375	2.375	2.375
Initial Moisture Content (%)	18.1	19.8	21.9
Initial Dry Density (pcf)	113.8	109.0	105.0
Initial Degree of Saturation (%)	101.6	97.7	97.7
Soil Height Before Shearing (in.)	1.2	1.2	1.2
Final Moisture Content (%)	17.5	19.8	20.5

GEOCON

DIRECT SHEAR TEST RESULTS

Consolidated Drained ASTM D-3080

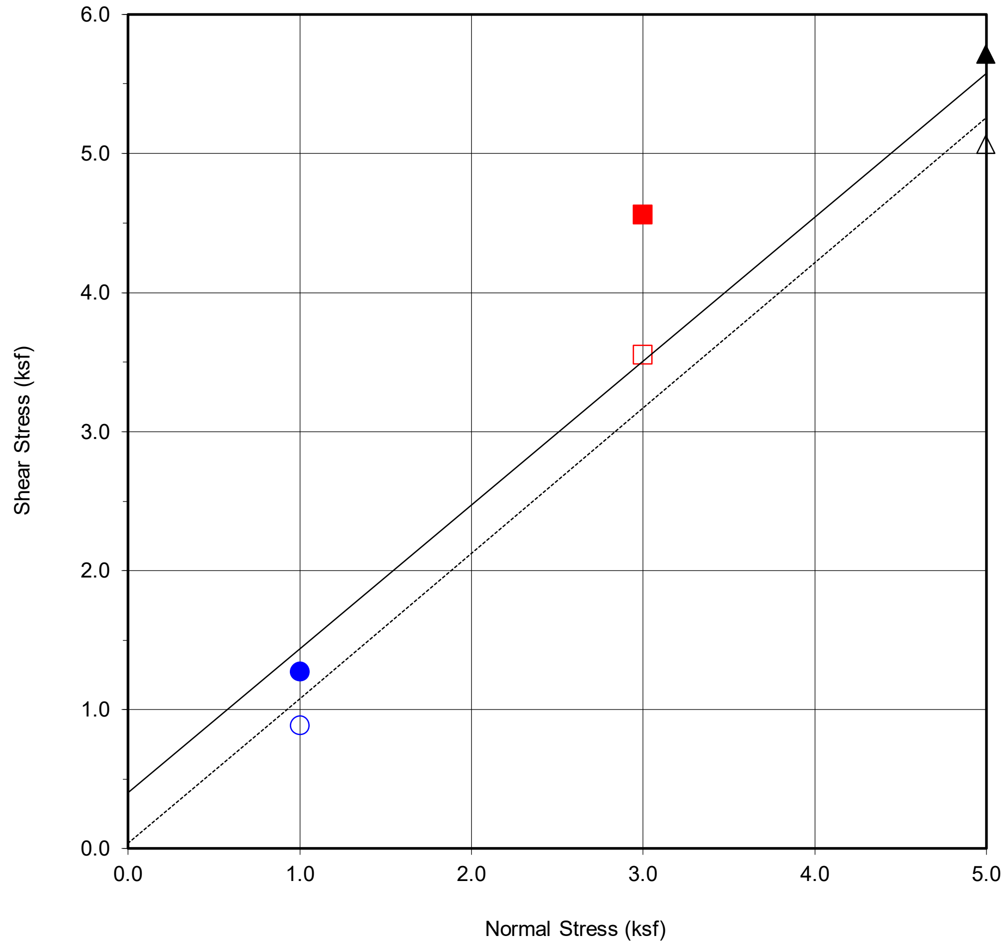
Checked by: JJK

Project No.: W1718-06-01

1000, 1014 & 1020 NORTH LA BREA AVENUE
WEST HOLLYWOOD, CALIFORNIA

MAY 2023

Figure B3



Boring No.	**B1**
Sample No.	**B1@60'**
Depth (ft)	**60**
Sample Type:	Ring

Soil Identification:		
Silty Sand (SM)		
Strength Parameters		
	C (psf)	ϕ (°)
Peak	403	46
Ultimate	36	46

Normal Strest (kip/ft2)	1	3	5
Peak Shear Stress (kip/ft²)	● 1.27	■ 4.56	▲ 5.71
Shear Stress @ End of Test (ksf)	○ 0.89	□ 3.55	△ 5.06
Deformation Rate (in./min.)	0.05	0.05	0.05
Initial Sample Height (in.)	1.0	1.0	1.0
Ring Inside Diameter (in.)	2.375	2.375	2.375
Initial Moisture Content (%)	20.0	17.5	17.0
Initial Dry Density (pcf)	112.3	116.7	116.1
Initial Degree of Saturation (%)	107.7	106.2	101.9
Soil Height Before Shearing (in.)	1.2	1.2	1.2
Final Moisture Content (%)	16.2	14.4	14.3

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DIRECT SHEAR TEST RESULTS

Consolidated Drained ASTM D-3080

Checked by: JJK

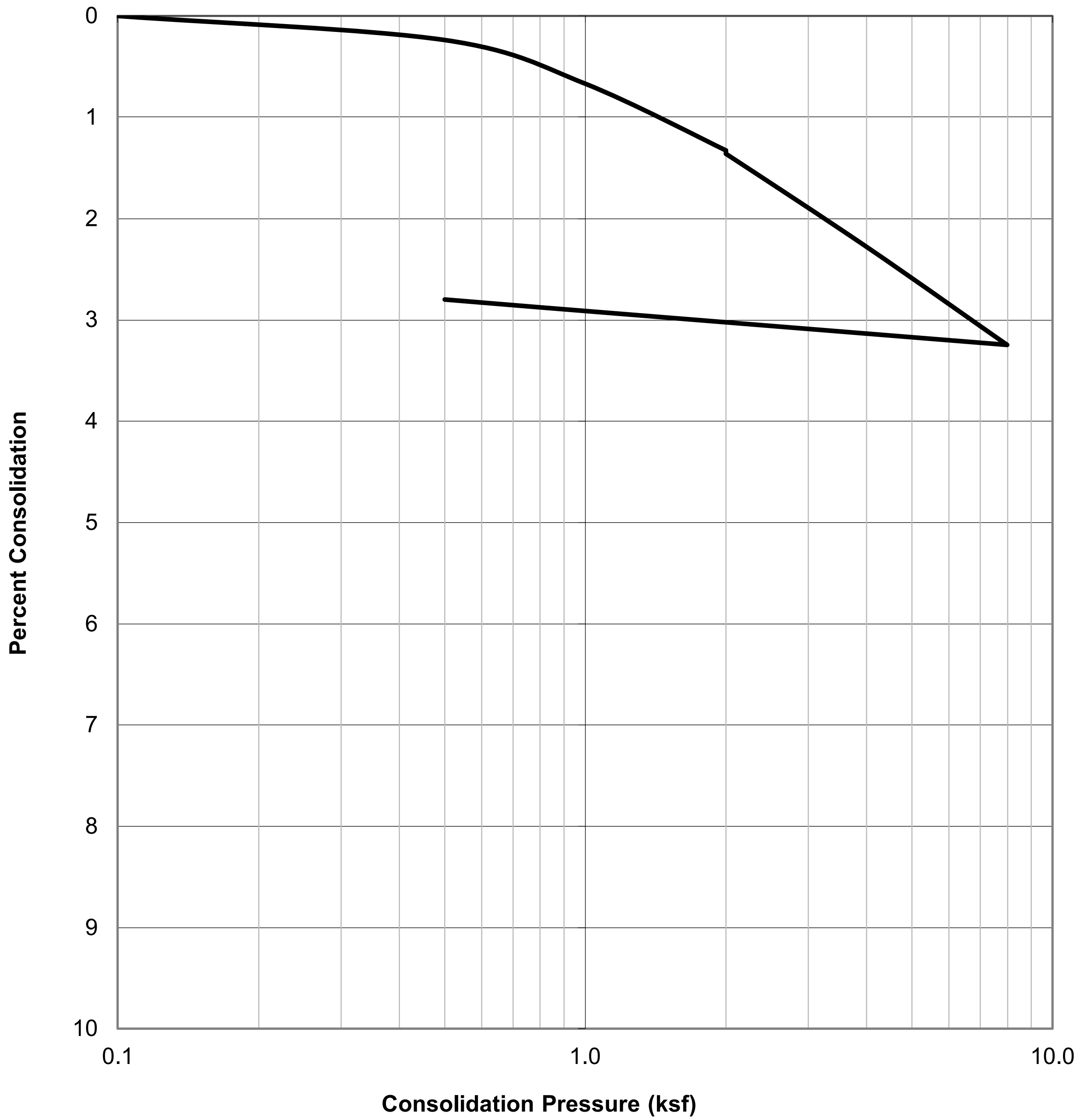
Project No.: W1718-06-01

1000 NORTH LA BREA AVENUE
WEST HOLLYWOOD, CALIFORNIA

MAY 2023

Figure B4

WATER ADDED AT 2.0 KSF



SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B1@25	Silty Sand (SM)	103.5	20.6	19.1

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CONSOLIDATION TEST RESULTS

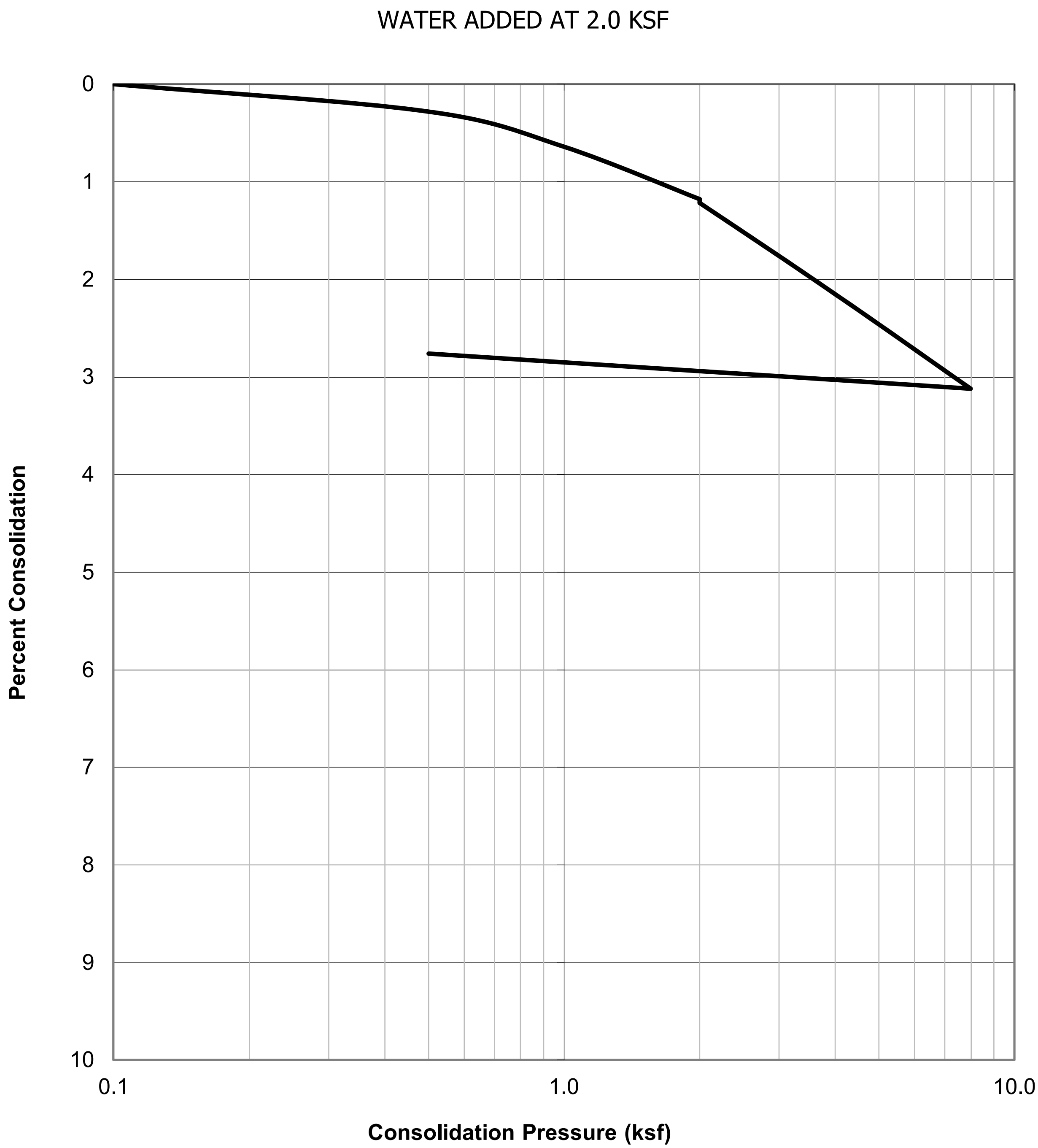
ASTM D-2435

Checked by: JJK

Project No.: W1718-06-01

1000, 1014 & 1020 NORTH LA BREA AVENUE
WEST HOLLYWOOD, CALIFORNIA

MAY 2023



SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B1@27.5	Silty Sand (SM)	110.9	16.6	16.1

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CONSOLIDATION TEST RESULTS

ASTM D-2435

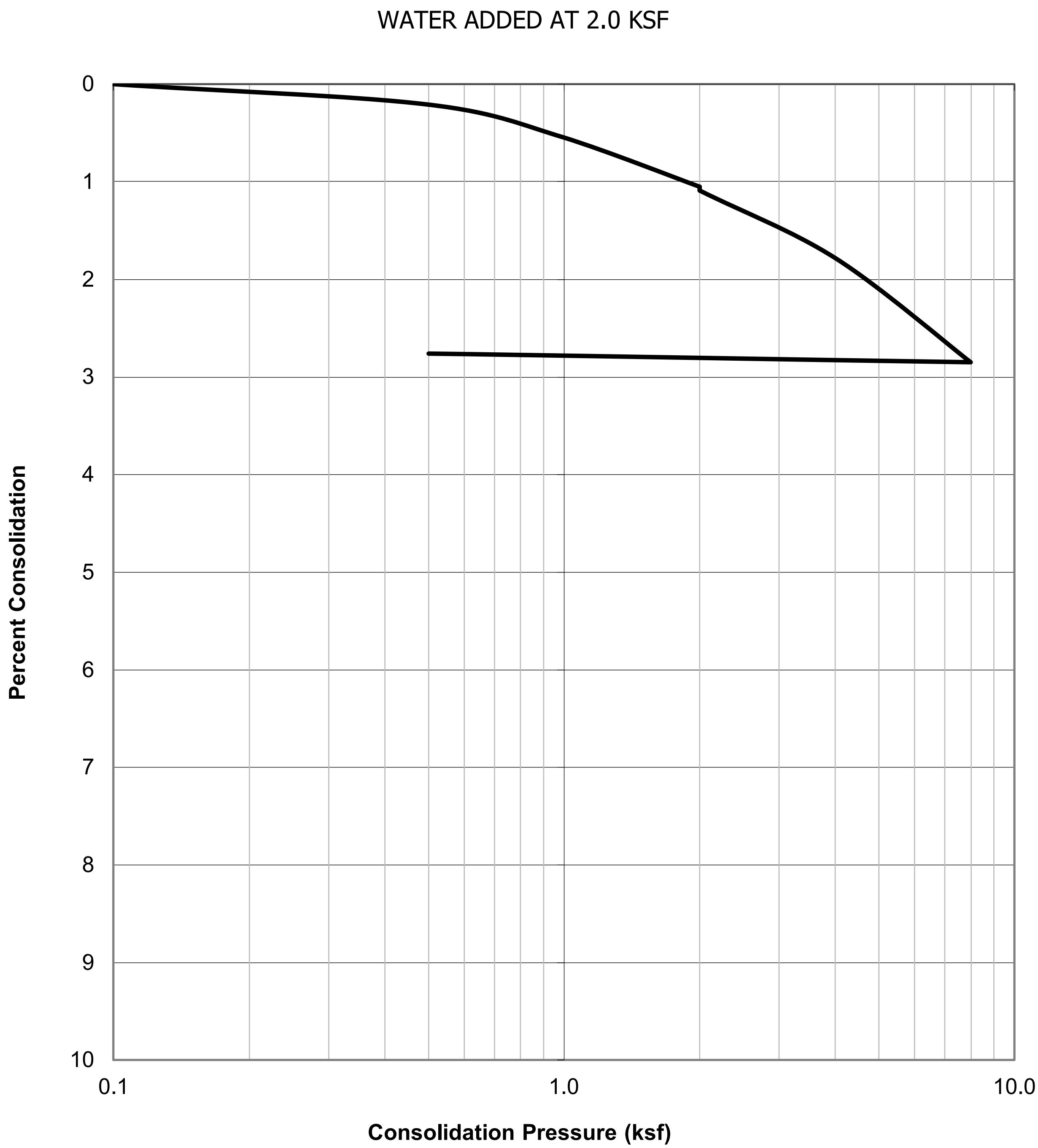
Checked by: JJK

Project No.: W1718-06-01

1000, 1014 & 1020 NORTH LA BREA AVENUE
WEST HOLLYWOOD, CALIFORNIA

MAY 2023

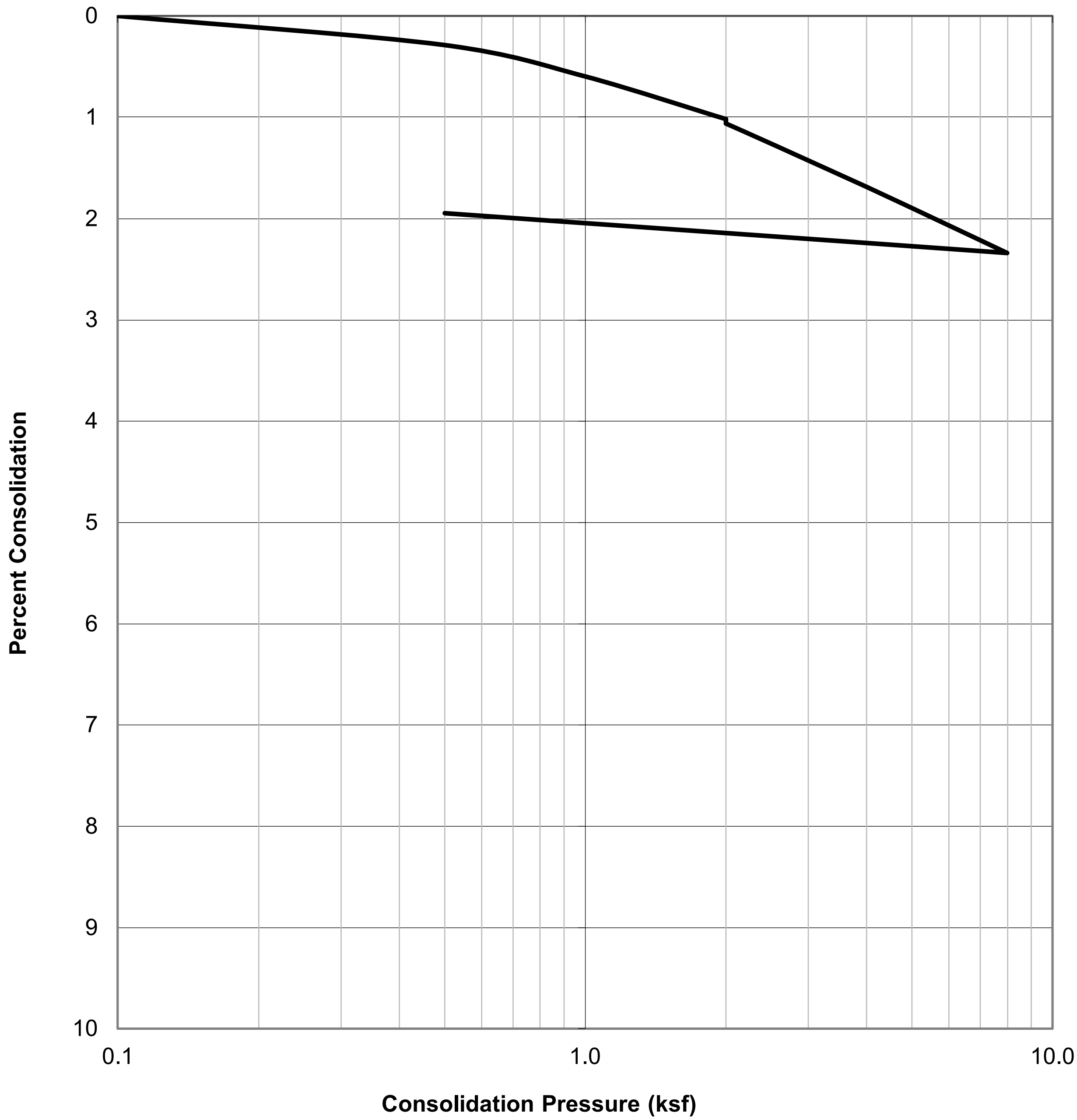
Figure B6



SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B1@32.5	Silty Sand (SM)	107.0	19.1	17.8

GEOCON	CONSOLIDATION TEST RESULTS ASTM D-2435	Project No.: W1718-06-01
		1000, 1014 & 1020 NORTH LA BREA AVENUE WEST HOLLYWOOD, CALIFORNIA
	Checked by: JJK	MAY 2023 Figure B7

WATER ADDED AT 2.0 KSF



SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B1@35	Silty Sand (SM)	107.3	19.1	17.8

GEOCON

CONSOLIDATION TEST RESULTS

ASTM D-2435

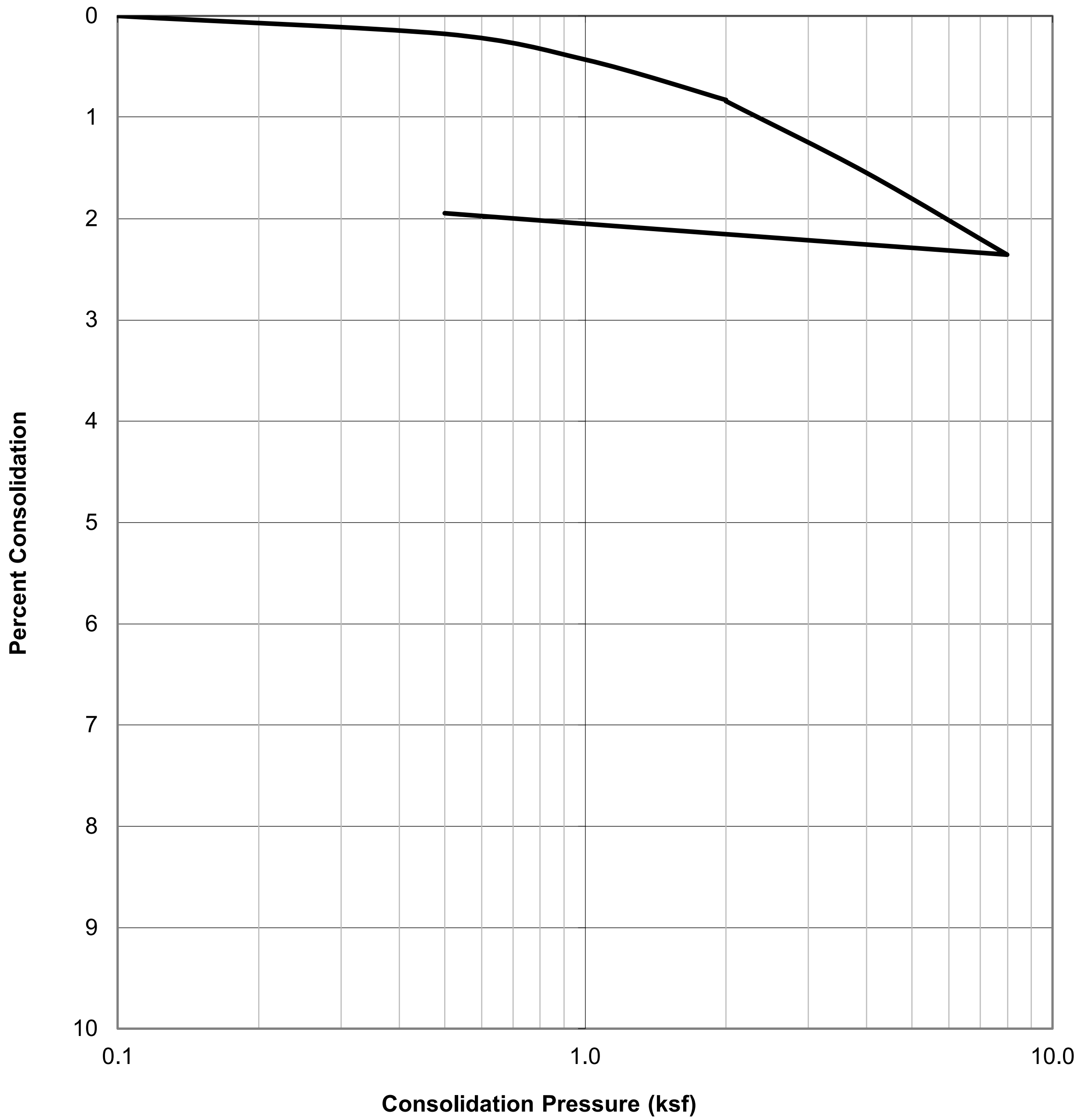
Checked by: JJK

Project No.: W1718-06-01

1000, 1014 & 1020 NORTH LA BREA AVENUE
WEST HOLLYWOOD, CALIFORNIA

MAY 2023

WATER ADDED AT 2.0 KSF



SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B1@40	Silty Sand (SM)	122.3	14.3	14.3

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CONSOLIDATION TEST RESULTS

ASTM D-2435

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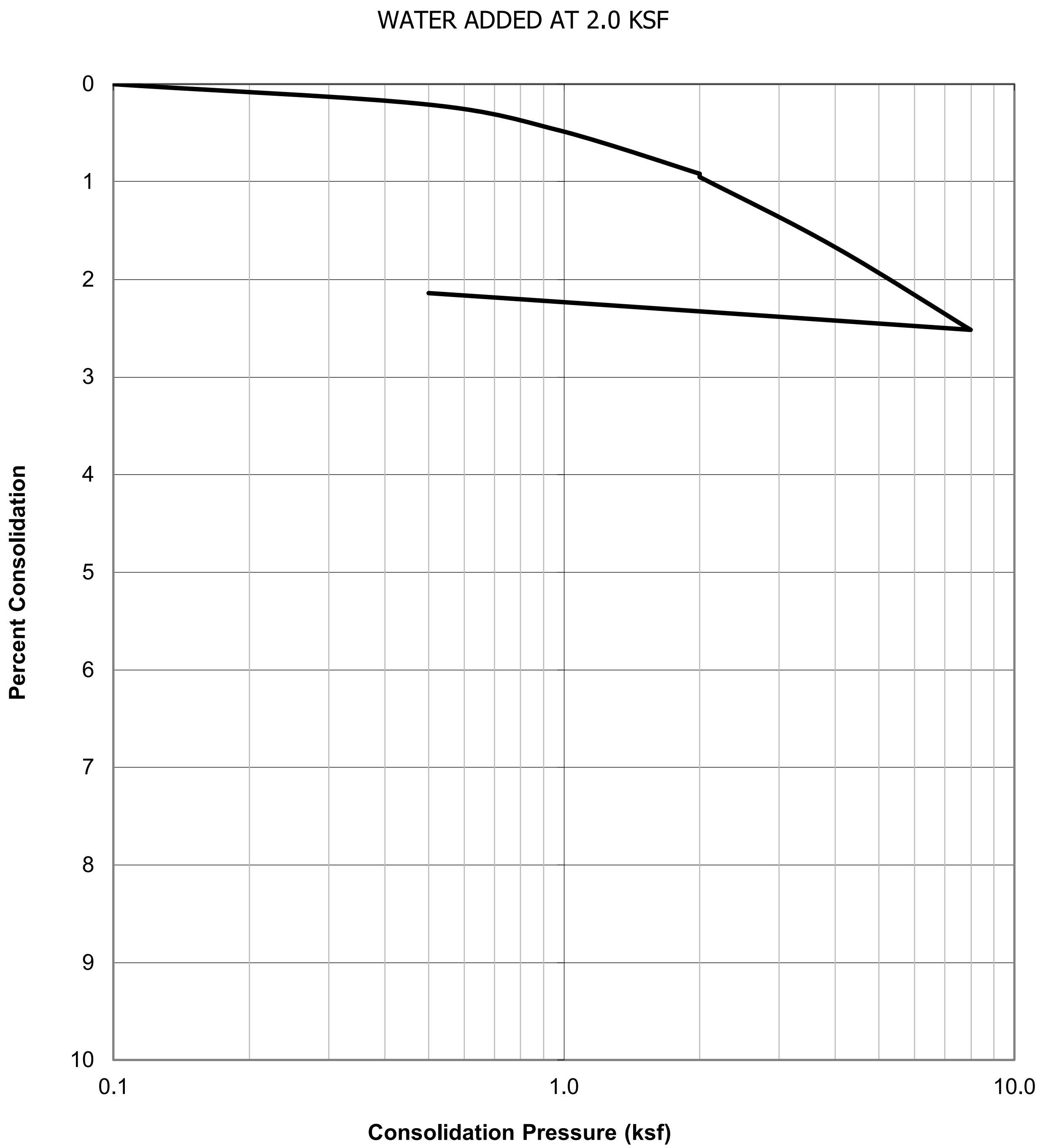
Project No.: W1718-06-01

1000, 1014 & 1020 NORTH LA BREA AVENUE
WEST HOLLYWOOD, CALIFORNIA

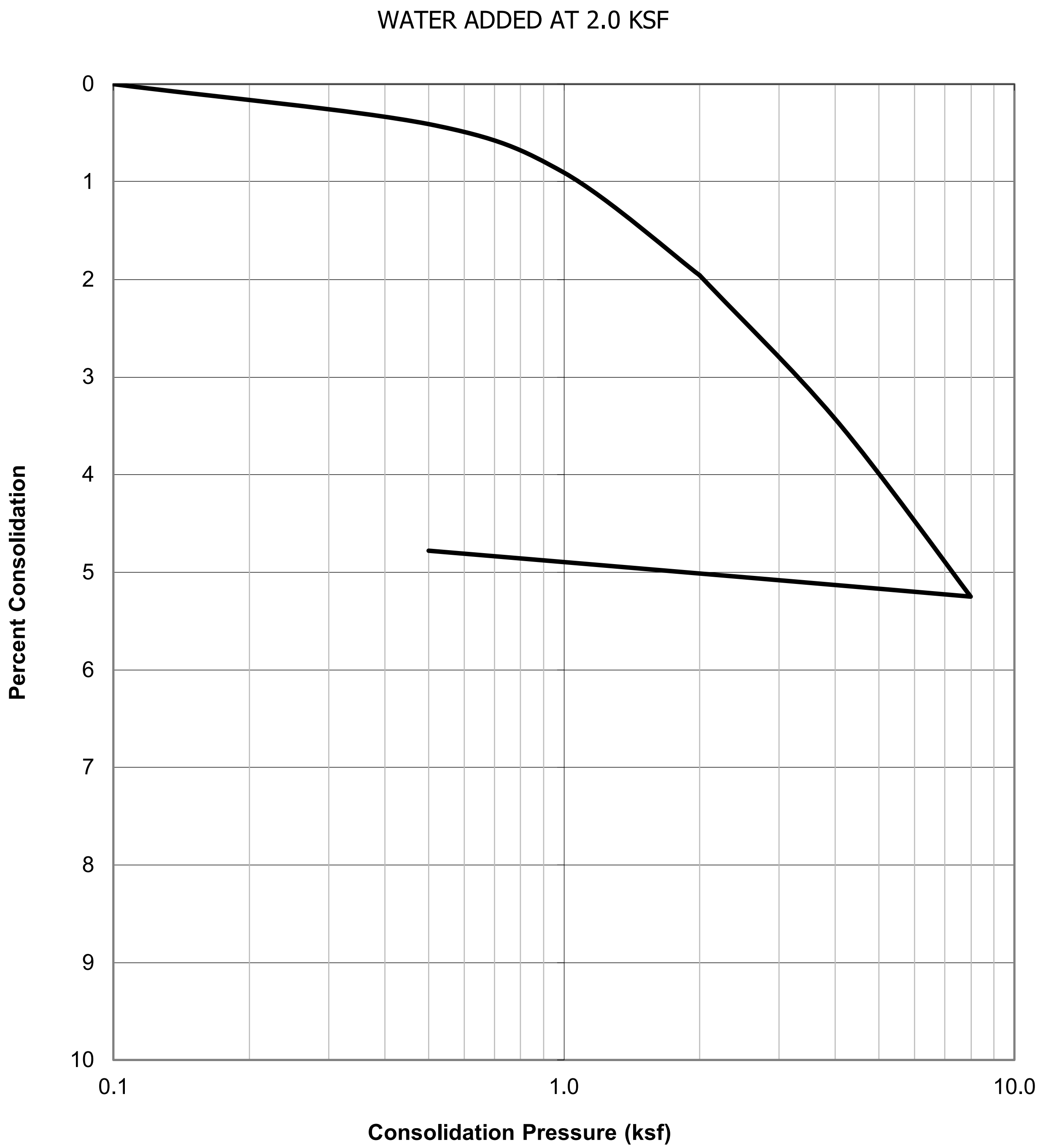
MAY 2023

Figure B9



SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B1@45	Sandy Silt (ML)	99.3	21.3	22.1

GEOCON	CONSOLIDATION TEST RESULTS ASTM D-2435	Project No.: W1718-06-01
		1000 NORTH LA BREA AVENUE WEST HOLLYWOOD, CALIFORNIA
	Checked by: JJK	MAY 2023 Figure B10



SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B1@55	Clay (CL)	99.9	25.1	22.0

GEOCON

CONSOLIDATION TEST RESULTS

ASTM D-2435

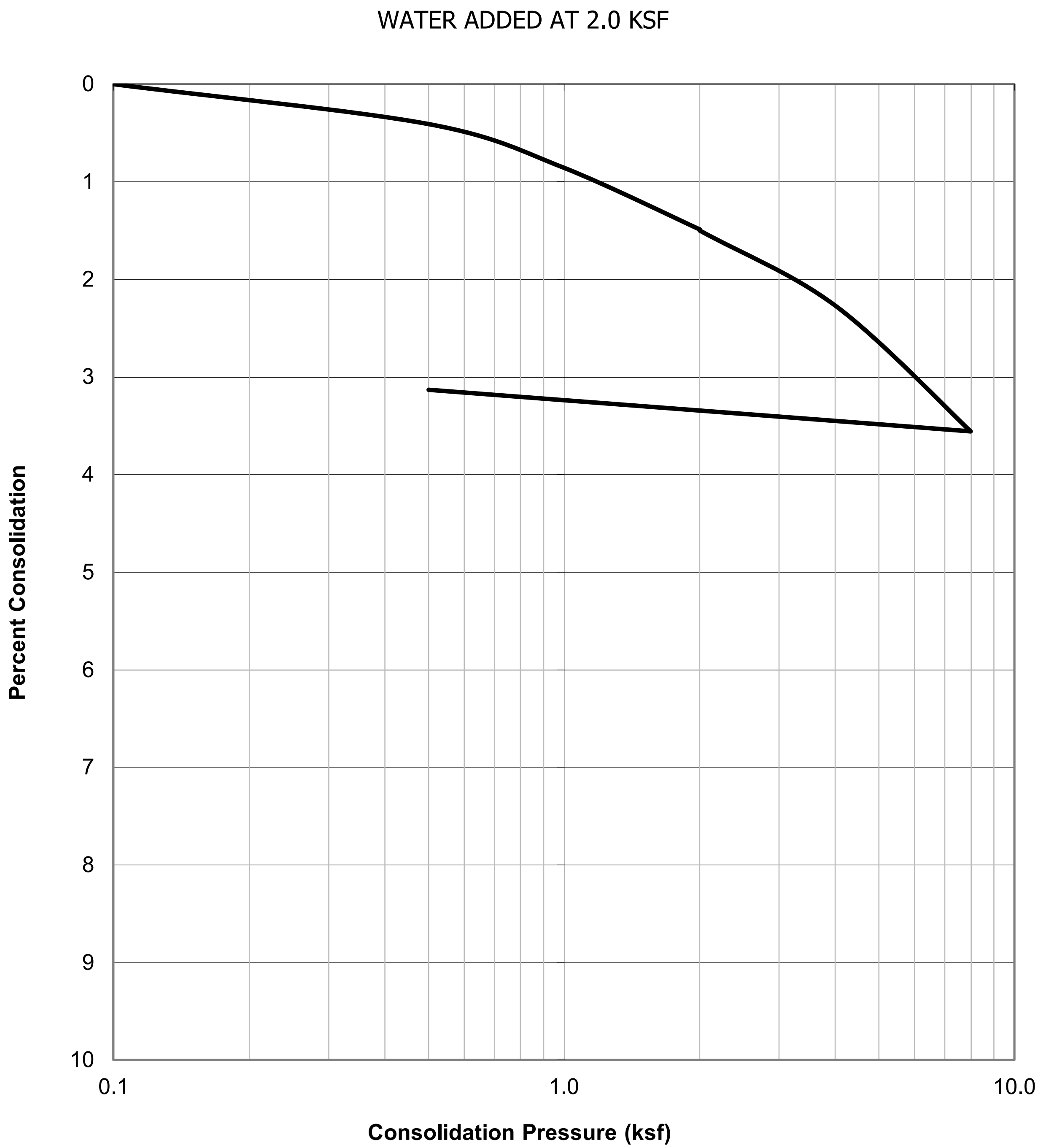
Checked by: JJK

Project No.: W1718-06-01

1000, 1014 & 1020 NORTH LA BREA AVENUE
WEST HOLLYWOOD, CALIFORNIA

MAY 2023

Figure B11



SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B1@65	Poorly Graded Sand (SP)	126.8	8.4	9.9

GEOCON

CONSOLIDATION TEST RESULTS

ASTM D-2435

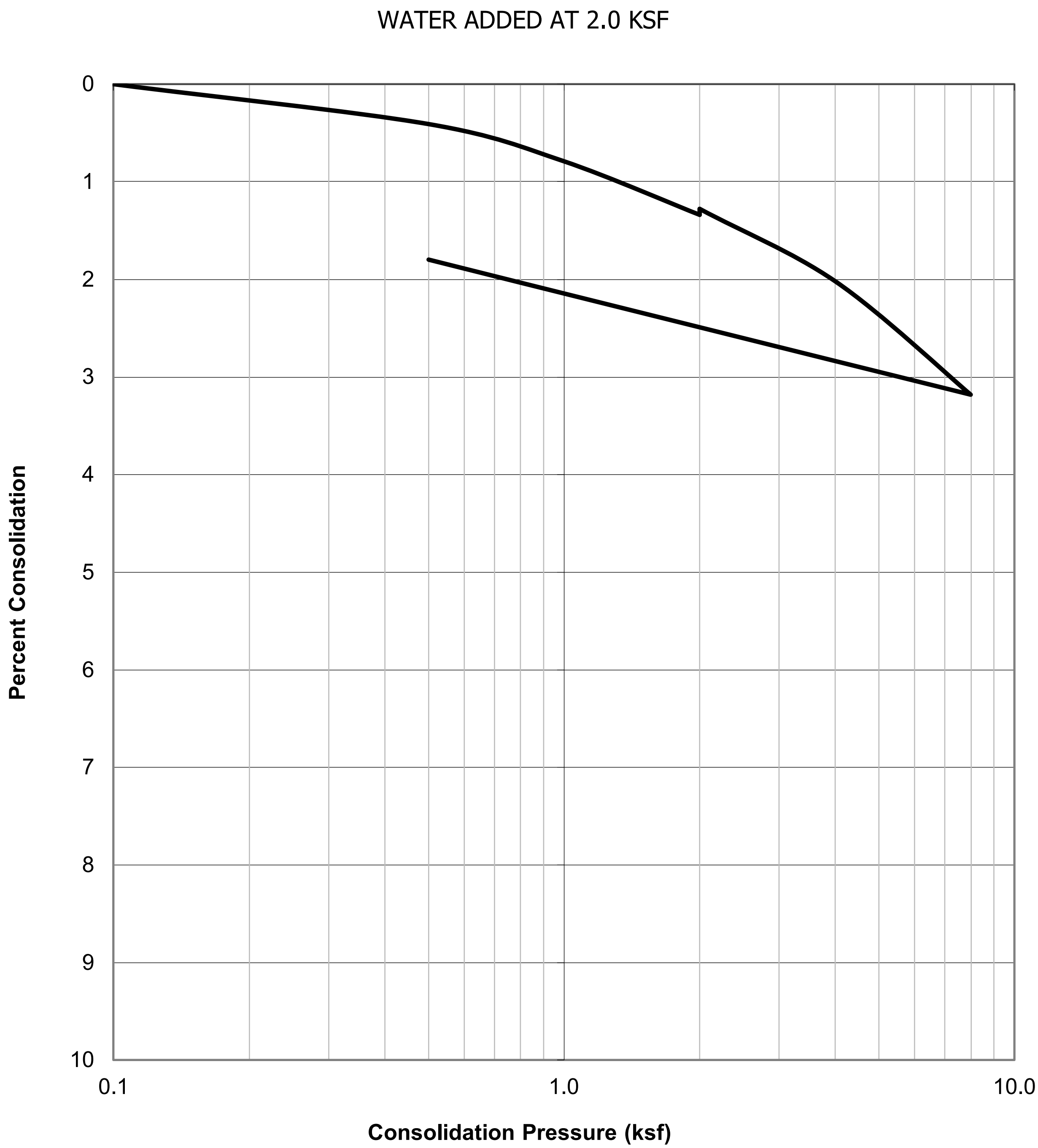
Checked by: JJK

Project No.: W1718-06-01

1000, 1014 & 1020 NORTH LA BREA AVENUE
WEST HOLLYWOOD, CALIFORNIA

MAY 2023

Figure B12



SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B1@80	Sandy Clay (CL)	105.8	20.9	21.6

GEOCON

CONSOLIDATION TEST RESULTS

ASTM D-2435

Checked by: JJK

Project No.: W1718-06-01

1000, 1014 & 1020 NORTH LA BREA AVENUE
WEST HOLLYWOOD, CALIFORNIA

MAY 2023

Figure B13

APPENDIX C

APPENDIX C

PRIOR GEOTECHNOLOGIES INVESTIGATION REPORTS

Geotechnologies, Inc.
Consulting Geotechnical Engineers

439 Western Avenue
Glendale, California 91201-2837
818.240.9600 • Fax 818.240.9675

October 25, 2019
File Number 21849

Faring
659 North Robertson Boulevard
West Hollywood, California 90069

Attention: Sarah Oliveira

Subject: Preliminary Geotechnical Engineering Investigation
Proposed Mixed-Use Development
1011 North Sycamore Avenue, Los Angeles, California

Dear Ms. Oliveira:

This letter transmits the Preliminary Geotechnical Engineering Investigation for the subject site prepared by Geotechnologies, Inc. This report provides preliminary geotechnical recommendations for the development of the site, including earthwork, seismic design, retaining walls, excavations, shoring and foundation design. Engineering for the proposed project should not begin until approval of the geotechnical investigation is granted by the local building official. Significant changes in the geotechnical recommendations may result due to the building department review process.

This report is preliminary in nature because the proposed project plan remains under development and is not well defined at this time. Due to its preliminary nature, this report is not intended for submission to the building official for building permit purposes. Once the proposed development plan achieves refinement, this firm should re-evaluate the recommendations presented herein, to ensure they are suitable for the proposed development. A final geotechnical engineering investigation, suitable for submission to the building official for building permit purposes, will be prepared at that time.

The validity of the recommendations presented herein is dependent upon review of the geotechnical aspects of the project during construction by this firm. The subsurface conditions described herein have been projected from limited subsurface exploration and laboratory testing. The exploration and testing presented in this report should in no way be construed to reflect any variations which may occur between the exploration locations or which may result from changes in subsurface conditions.

Should you have any questions please contact this office.

Respectfully submitted,
GEOTECHNOLOGIES, INC.

REGISTERED PROFESSIONAL ENGINEER
GREGORIO VARELA
No. 81201
Exp. 9/30/21
CIVIL
STATE OF CALIFORNIA

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PRELIMINARY GEOTECHNICAL ENGINEERING INVESTIGATION
PROPOSED MIXED-USE DEVELOPMENT
1011 NORTH SYCAMORE AVENUE
LOS ANGELES, CALIFORNIA

INTRODUCTION

This report presents the results of the preliminary geotechnical engineering investigation performed on the subject site. The purpose of this investigation was to identify the distribution and engineering properties of the geologic materials underlying the site, and to provide geotechnical recommendations for the design of the proposed development.

This report is preliminary in nature because the proposed project plan remains under development and is not well defined at this time. Due to its preliminary nature, this report is not intended for submission to the building official for building permit purposes. Once the proposed development plan achieves refinement, this firm should re-evaluate the recommendations presented herein, to ensure they are suitable for the proposed development. A final geotechnical engineering investigation, suitable for submission to the building official for building permit purposes, will be prepared at that time.

This investigation included two exploratory excavations, collection of representative samples, laboratory testing, engineering analysis, review of published geologic data, review of available geotechnical engineering information and the preparation of this report. The exploratory excavation locations are shown on the enclosed Plot Plan. The results of the exploration and the laboratory testing are presented in the Appendix of this report.

PROPOSED DEVELOPMENT

Preliminary information concerning the proposed development was furnished by the client. In addition, the preliminary drawings prepared by Gensler, dated September 19, 2019, were reviewed for the preparation of this report. The proposed project consists of the construction of a mixed-use development. It should be noted that the site of the proposed development is located within two different jurisdictions. The western portion of the site is located within the City of West Hollywood, while the eastern portion is located within the City of Los Angeles. At this time, it is unknown which jurisdiction would be on charge of reviewing the project. For the purpose of preparing this preliminary investigation, it has been assumed that the portion of the structure proposed within the City of Los Angeles will fall within its jurisdiction. This investigation is specific to the portion of the development located within the City of Los Angeles. A separate investigation is currently being prepared for the portion of the structure located within the City of West Hollywood jurisdiction.

It is anticipated that the structure proposed within the City of Los Angeles may be up to 10 stories in height. The structure will be built over a 3-level subterranean parking garage. Based on review of the enclosed Cross Section A-A', it is anticipated that the finished floor elevation of the lowest subterranean level may extend to a depth of 33 feet below the existing grade. Excavations up to a depth of 43 feet would be anticipated for construction of the proposed subterranean garage, including its mat foundation. The proposed location, alignment and depth of the structure are shown in the enclosed Plot Plan and Cross Sections A-A'.

Any changes in the design of the project or location of any structure, as outlined in this report, should be reviewed by this office. The recommendations contained in this report should not be considered valid until reviewed and modified or reaffirmed, in writing, subsequent to such review.

SITE CONDITIONS

The site is located at 1011 North Sycamore Avenue, in the City of Los Angeles, California. The site is rectangular in shape, and just over ½-acre in area. The site is bounded by commercial and office developments to the north and west, Sycamore Avenue to the east, and Romaine Street to the south. The site is shown relative to nearby topographic features in the enclosed Vicinity Map.

The site grade is relatively level, with no pronounced highs or lows. The site is currently developed with a concrete plant. Vegetation at the site is non-existent. Drainage across the site appears to be by sheetflow to the City streets.

GEOTECHNICAL EXPLORATION

FIELD EXPLORATION

The site was explored on August 1, 2, 5, 7 and 8, 2019, by drilling two borings. The borings were drilled with the aid of a truck-mounted drilling machine using 8-inch diameter hollowstem augers. Boring B1 was drilled to a depth of 130 feet, while Boring B2 was drilled to a depth of 180 feet below the existing grade. The exploration locations are shown on the Plot Plan and the geologic materials encountered are logged on Plates A-1 and A-2.

The location of exploratory excavations was determined from hardscape features shown in the enclosed Plot Plan. The location of the exploratory excavations should be considered accurate only to the degree implied by the method used.

Geologic Materials

Fill:

Fill materials were encountered in the exploratory borings to a depth 5 and 8 feet below the existing grades, respectively. The existing fill materials consist of a mixture of clay, silt and sand, which is dark brown and gray in color, moist, stiff or medium dense and fine grained.

Older Alluvium:

Older alluvial soils were observed to underlie the fill in the exploratory borings. The older alluvial soils consist of interlayered mixtures of silty and sandy clays, Sandy and clayey silts, silty and clayey sands, and sands, which are yellowish to dark brown to gray in color, moist to wet, medium dense to very dense, or stiff to very stiff, and fine to coarse grained with occasional gravel and cobbles.

Bedrock (Puente Formation):

Bedrock was encountered in the borings underlying the older alluvial soils. The bedrock was observed at a depth of 107½ and 115 feet below the existing grade, respectively. The bedrock underlying the site is comprised of upper Miocene-age Puente Formation, consisting of thin bedded siltstone and claystone. The bedrock is gray to dark gray in color, moist, and moderately hard to hard.

More detailed descriptions of the earth materials encountered may be obtained from individual logs of the subsurface excavations.

Groundwater

Groundwater was encountered during drilling of Boring 1 and Boring 2, at depths of 19 and 20 feet below the existing grade, respectively. According to groundwater data provided in the Seismic

Hazard Zone Report of the Hollywood 7½-Minute Quadrangle, the historically-highest groundwater level for the site was on the order of 10 feet below the ground surface (CDMG, 1998, Revised 2006). A copy of the historic high water map is appended.

Fluctuations in the level of groundwater may occur due to variations in rainfall, temperature, and other factors not evident at the time of the measurements reported herein. Fluctuations also may occur across the site. High groundwater levels can result in changed conditions.

Caving

Caving could not be directly observed during exploration due to the continuously cased design of the hollow stem auger Based on the experience of this firm, large diameter excavations, excavations that encounter granular, cohesionless soils and excavations below the groundwater table will likely experience caving.

SEISMIC EVALUATION

REGIONAL GEOLOGIC SETTING

The subject site is located in the Los Angeles Basin which is considered the northern portion of the Peninsular Ranges Geomorphic Province. The Peninsular Ranges are characterized by northwest-trending blocks of mountain ridges and sediment-floored valleys. The dominant geologic structural features are northwest trending fault zones that either die out to the northwest or terminate at east-trending reverse faults that form the southern margin of the Transverse Ranges.

The Los Angeles Basin is bounded by the east and southeast by the Santa Ana Mountains and San Joaquin Hills, to the northwest by the Santa Monica Mountains. Over 22 million years ago the Los Angeles basin was a deep marine basin formed by tectonic forces between the North American and Pacific plates. Since that time, over 5 miles of marine and non-marine sedimentary rock as well as intrusive and extrusive igneous rocks have filled the basin. During the last 2 million years, defined by the Pleistocene and Holocene epochs, the Los Angeles basin and surrounding mountain ranges have been uplifted to form the present day landscape. Erosion of the surrounding mountains has resulted in deposition of unconsolidated sediments in low-lying areas by rivers such as the Los Angeles River. Areas that have experienced subtle uplift have been eroded with gullies.

The site is underlain by deep, unconsolidated older alluvial sediments deposited by river and stream action.

REGIONAL FAULTING

Based on criteria established by the California Division of Mines and Geology (CDMG) now called California Geologic Survey (CGS), Faults may be categorized as Holocene-active, Pre-Holocene faults, and Age-undetermined faults. Holocene-active faults are those which show evidence of surface displacement within the last 11,700 years. Pre-Holocene faults are those that have not moved in the past 11,700 years. Age-undetermined faults are faults where the recency of fault movement has not been determined.

Buried thrust faults are faults without a surface expression but are a significant source of seismic activity. They are typically broadly defined based on the analysis of seismic wave recordings of hundreds of small and large earthquakes in the southern California area. Due to the buried nature of these thrust faults, their existence is usually not known until they produce an earthquake. The risk for surface rupture potential of these buried thrust faults is inferred to be low (Leighton, 1990). However, the seismic risk of these buried structures in terms of recurrence and maximum potential magnitude is not well established. Therefore, the potential for surface rupture on these surface-verging splays at magnitudes higher than 6.0 cannot be precluded.

The enclosed Regional Fault Location Map shows faults located in the region. This map is based on the 2010 Fault Activity Map, prepared by the California Department of Conservation. Some of the Holocene-active and Blind Thrusts faults located closest to the site are addressed in the following sections.

Holocene-Active Faults

Hollywood Fault

The Hollywood fault is part of the Transverse Ranges Southern Boundary fault system. The Hollywood fault is located approximately 1 mile north of the site. This fault trends east-west along the base of the Santa Monica Mountains from the West Beverly Hills Lineament in the West Hollywood–Beverly Hills area to the Los Feliz area of Los Angeles. The Hollywood fault is the eastern segment of the reverse oblique Santa Monica–Hollywood fault. Based on geomorphic evidence, stratigraphic correlation between exploratory borings, and fault trenching studies, this fault is classified as active.

Until recently, the approximately 9.3-mile long Hollywood fault was considered to be expressed as a series of linear ground-surface geomorphic expressions and south-facing ridges along the south margin of the eastern Santa Monica Mountains and the Hollywood Hills. Multiple recent fault rupture hazard investigations have shown that the Hollywood fault is located south of the ridges and bedrock outcroppings along portions of Sunset Boulevard. The Hollywood fault has not produced any damaging earthquakes during the historical period and has had relatively minor micro-seismic activity. It is estimated that the Hollywood fault is capable of producing a maximum 6.7 magnitude earthquake. In 2014, the California Geological Survey established an Earthquake Fault Zone for the Hollywood Fault. A copy of this map may be found in the Appendix.

Santa Monica Fault

In 2018, the California Geological Survey established an Earthquake Fault Zone for the Santa Monica Fault. The nearest segment of the active portion of the Santa Monica fault is located approximately 4¼-miles to the west of the site. The Santa Monica fault is a part of the Transverse Ranges Southern Boundary fault system, extending east from the coastline in Pacific Palisades through Santa Monica and West Los Angeles and merges with the Hollywood fault at the West Beverly Hills Lineament in Beverly Hills where its strike is northeast. It is believed that at least six surface ruptures have occurred in the past 50 thousand years. In addition, a well-documented surface rupture occurred between 10 and 17 thousand years ago, although a more recent earthquake probably occurred 1 to 3 thousand years ago. This leads to an average earthquake recurrence interval of 7 to 8 thousand years.[a] It is thought that the Santa Monica fault system may produce earthquakes with a maximum magnitude of 7.4.

Newport-Inglewood Fault System

The Newport-Inglewood fault system is located 4 miles to the southwest of the site. The Newport-Inglewood fault zone is a broad zone of discontinuous north to northwestern echelon faults and northwest to west trending folds. The fault zone extends southeastward from West Los Angeles, across the Los Angeles Basin, to Newport Beach and possibly offshore beyond San Diego (Barrows, 1974; Weber, 1982; Ziony, 1985).

The onshore segment of the Newport-Inglewood fault zone extends for about 37 miles from the Santa Ana River to the Santa Monica Mountains. Here it is overridden by, or merges with, the east-west trending Santa Monica zone of reverse faults.

The surface expression of the Newport-Inglewood fault zone is made up of a strikingly linear alignment of domal hills and mesas that rise on the order of 400 feet above the surrounding plains. From the northern end to its southernmost onshore expression, the Newport-Inglewood fault zone is made up of: Cheviot Hills, Baldwin Hills, Rosecrans Hills, Dominguez Hills, Signal Hill-

[a] *Southern California Earthquake Center, a National Science Foundation and U.S. Geological Survey Center. Active Faults in the Los Angeles Metropolitan Region, www.scec.org/research/special/SCEC001activefaultsLA.pdf; accessed May 24, 2012.*

Reservoir Hill, Alamitos Heights, Landing Hill, Bolsa Chica Mesa, Huntington Beach Mesa, and Newport Mesa. Several single and multiple fault strands, arranged in a roughly left stepping en echelon arrangement, make up the fault zone and account for the uplifted mesas.

The most significant earthquake associated with the Newport-Inglewood fault system was the Long Beach earthquake of 1933 with a magnitude of 6.3 on the Richter scale. It is believed that the Newport-Inglewood fault zone is capable of producing a 7.5 magnitude earthquake.

Raymond Fault

The Raymond fault is located approximately 7 miles to the northeast of the site. The Raymond fault is an effective groundwater barrier which divides the San Gabriel Valley into groundwater sub-basins. Much of the geomorphic evidence for the Raymond fault has been obliterated by urbanization of the San Gabriel Valley. However, a discontinuous escarpment can be traced from Monrovia to the Arroyo Seco in South Pasadena. The very bold, "knife edge" escarpment in Monrovia parallel to Scenic Drive is believed to be a fault scarp of the Raymond fault. Trenching of the Raymond fault is reported to have revealed Holocene movement (Weaver and Dolan, 1997).

The recurrence interval for the Raymond fault is probably slightly less than 3,000 years, with the most recent documented event occurring approximately 1,600 years ago (Crook, et al, 1978). However, historical accounts of an earthquake that occurred in July 1855 as reported by Toppozada and others, 1981, places the epicenter of a Richter Magnitude 6 earthquake within the Raymond fault. It is believed that the Raymond fault is capable of producing a 6.8 magnitude earthquake. The Raymond Fault is considered active by the California Geological Survey.

Verdugo Fault

The Verdugo Fault is located approximately 7½ miles to the north of the site. The Verdugo Fault runs along the southwest edge of the Verdugo Mountains. The fault displays a reverse motion.

According to Weber, et. al., (1980) 2 to 3 meter high scarps were identified in alluvial fan deposits in the Burbank and Glendale areas. Further to the northeast, in Sun Valley, a fault was reportedly identified at a depth of 40 feet in a sand and gravel pit. Although considered active by the County of Los Angeles, Department of Public Works (Leighton, 1990), and the United States Geological Survey, the fault is not designated with an Earthquake Fault Zone by the California Geological Survey. It is estimated that the Verdugo Fault is capable of producing a maximum 6.9 magnitude earthquake.

Malibu Coast Fault

The Malibu Coast fault is part of the Transverse Ranges Southern Boundary fault system, a west-trending system of reverse, oblique-slip, and strike-slip faults that extends for more than approximately 124 miles along the southern edge of the Transverse Ranges and includes the Hollywood, Raymond, Anacapa–Dume, Malibu Coast, Santa Cruz Island, and Santa Rosa Island faults.

The Malibu Coast fault zone runs in an east-west orientation onshore subparallel to and along the shoreline for a linear distance of about 17 miles through the Malibu City limits, but also extends offshore to the east and west for a total length of approximately 37.5 miles. The onshore Malibu Coast fault zone involves a broad, wide zone of faulting and shearing as much as 1 mile in width. While the Malibu Coast Fault Zone has not been officially designated as an active fault zone by the State of California and no Special Studies Zones have been delineated along any part of the fault zone under the Alquist-Priolo Act of 1972, evidence for Holocene activity (movement in the last 11,000 years) has been established in several locations along individual fault splays within the fault zone. Due to such evidence, several fault splays within the onshore portion of the fault zone are identified as active.[b]

[b] *City of Malibu Planning Department, Malibu General Plan, Chapter 5.0, Safety and Health Element, http://qcode.us/codes/malibu-general-plan/; accessed October 25, 2012.*

Large historic earthquakes along the Malibu Coast fault include the 1979, 5.2 magnitude earthquake and the 1989, 5.0 magnitude earthquake.[c] The Malibu Coast fault zone is approximately 11¼-miles northwest of the site and is believed to be capable of producing a maximum 7.0 magnitude earthquake.

Sierra Madre Fault System

The Sierra Madre fault alone forms the southern tectonic boundary of the San Gabriel Mountains in the northern San Fernando Valley. It consists of a system of faults approximately 75 miles in length. The individual segments of the Sierra Madre fault system range up to 16 miles in length and display a reverse sense of displacement and dip to the north. The most recently active portions of the zone include the Mission Hills, Sylmar and Lakeview segments, which produced an earthquake in 1971 of magnitude 6.4. Tectonic rupture along the Lakeview Segment during the San Fernando Earthquake of 1971 produced displacements of approximately 2½ to 4 feet upward and southwestward.

It is believed that the Sierra Madre fault zone is capable of producing an earthquake of magnitude 7.3. The closest trace of the fault is located approximately 12 miles northeast of the site.

Palos Verdes Fault

Studies indicate that there are several active on-shore extensions of the strike-slip Palos Verdes fault, which is located approximately 14½-miles southwest of the site. Geophysical data also indicate the off-shore extensions of the fault are active, offsetting Holocene age deposits. No historic large magnitude earthquakes are associated with this fault. However, the fault is considered active by the California Geological Survey. It is estimated that the Palos Verdes fault is capable of producing a maximum 7.7 magnitude earthquake.

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[c] *California Institute of Technology, Southern California Data Center. Chronological Earthquake Index, www.data.scec.org/significant/malibu1979.html; accessed October 25, 2012.*

San Gabriel Fault System

The San Gabriel fault system is located approximately 16 miles northeast of the site. The San Gabriel fault system comprises a series of subparallel, steeply north-dipping faults trending approximately north 40 degrees west with a right-lateral sense of displacement. There is also a small component of vertical dip-slip separation. The fault system exhibits a strong topographic expression and extends approximately 90 miles from San Antonio Canyon on the southeast to Frazier Mountain on the northwest. The estimated right lateral displacement on the fault varies from 34 miles (Crowell, 1982) to 40 miles (Ehlig, 1986), to 10 miles (Weber, 1982). Most scholars accept the larger displacement values and place the majority of activity between the Late Miocene and Late Pliocene Epochs of the Tertiary Era (65 to 1.8 million years before present).

Portions of the San Gabriel fault system are considered active by California Geological Survey. Recent seismic exploration in the Valencia area (Cotton and others, 1983; Cotton, 1985) has established Holocene offset. Radiocarbon data acquired by Cotton (1985) indicate that faulting in the Valencia area occurred between 3,500 and 1,500 years before present.

It is hypothesized by Ehlig (1986) and Stitt (1986) that the Holocene offset on the San Gabriel fault system is due to sympathetic (passive) movement as a result of north-south compression of the upper Santa Susana thrust sheet. Seismic evidence indicates that the San Gabriel fault system is truncated at depth by the younger, north-dipping Santa Susana-Sierra Madre faults (Oakeshott, 1975; Namson and Davis, 1988).

Whittier-Elsinore Fault System

The Whittier fault is located approximately 18 miles to the southeast of the site. The Whittier fault together with the Chino fault comprises the northernmost extension of the northwest trending Elsinore fault system. The mapped surface of the Whittier fault extends in a west-northwest direction for a distance of 20 miles from the Santa Ana River to the terminus of the Puente Hills.

The Whittier fault is essentially a strike-slip, northeast dipping fault zone which also exhibits evidence of reverse movement along with en echelon[d] fault segments, en echelon folds and anatomizing (braided) fault segments. Right lateral offsets of stream drainages of up to 8800 feet (Durham and Yerkes, 1964) and vertical separation of the basement complex of 6,000 to 12,000 feet (Yerkes, 1972), have been documented. It is believed that the Whittier fault is capable of producing a 7.8 magnitude earthquake.

The Whittier Narrows earthquakes of October 1, 1987, and October 4, 1987, occurred in the area between the westernmost terminus of the mapped trace of the Whittier fault and the frontal fault system. The main 5.9 magnitude shock of October 1, 1987 was not caused by slip on the Whittier fault. The quake ruptured a gently dipping thrust fault with an east-west strike (Haukson, Jones, Davis and others, 1988). In contrast, the earthquake of October 4, 1987, is assumed to have occurred on the Whittier fault as focal mechanisms show mostly strike-slip movement with a small reverse component on a steeply dipping northwest striking plane (Haukson, Jones, Davis and others, 1988).

Santa Susana Fault

The Santa Susana fault extends approximately 17 miles west-northwest from the northwest edge of the San Fernando Valley into Ventura County and is at the surface high on the south flank of the Santa Susana Mountains. The fault ends near the point where it overrides the south-side-up South strand of the Oak Ridge fault. The Santa Susana fault strikes northeast at the Fernando lateral ramp and turns east at the northern margin of the Sylmar Basin to become the Sierra Madre fault. This fault is exposed near the base of the San Gabriel Mountains for approximately 46 miles from the San Fernando Pass at the Fernando lateral ramp east to its intersection with the San Antonio Canyon fault in the eastern San Gabriel Mountains, east of which the range front is formed by the Cucamonga fault. The Santa Susana fault has not experienced any recent major ruptures

[d] *En echelon refers to closely-spaced, parallel or subparallel, overlapping or step-like minor structural features*

except for a slight rupture during the 6.5 magnitude 1971 Sylmar earthquake.[e] The Santa Susana Fault is considered to be active by the County of Los Angeles. It is believed that the Santa Susana fault has the potential to produce a 6.9 magnitude earthquake. The closest trace of the fault is located approximately 18 miles north of the site.

San Andreas Fault System

The San Andreas Fault system forms a major plate tectonic boundary along the western portion of North America. The system is predominantly a series of northwest trending faults characterized by a predominant right lateral sense of movement. At its closest point the San Andreas Fault system is located approximately 34 miles to the northeast of the site.

The San Andreas and associated faults have had a long history of inferred and historic earthquakes. Cumulative displacement along the system exceeds 150 miles in the past 25 million years (Jahns, 1973). Large historic earthquakes have occurred at Fort Tejon in 1857, at Point Reyes in 1906, and at Loma Prieta in 1989. Based on single-event rupture length, the maximum Richter magnitude earthquake is expected to be approximately 8.25 (Allen, 1968). The recurrence interval for large earthquakes on the southern portion of the fault system is on the order of 100 to 200 years.

Blind Thrusts Faults

Blind or buried thrust faults are faults without a surface expression but are a significant source of seismic activity. By definition, these faults have no surface trace, therefore the potential for ground surface rupture is considered remote. They are typically broadly defined based on the analysis of seismic wave recordings of hundreds of small and large earthquakes in the Southern California area. Due to the buried nature of these thrust faults, their existence is sometimes not known until they produce an earthquake. Two blind thrust faults in the Los Angeles metropolitan area are the

[e] *California Institute of Technology, Southern California Data Center. Chronological Earthquake Index, www.data.scec.org/significant/santasusana.html; accessed May 24, 2012.*

Puente Hills blind thrust and the Elysian Park blind thrust. Another blind thrust fault of note is the Northridge fault located in the northwestern portion of the San Fernando Valley.

The Elysian Park anticline is thought to overlie the Elysian Park blind thrust. This fault has been estimated to cause an earthquake every 500 to 1,300 years in the magnitude range 6.2 to 6.7. The Elysian Park anticline is approximately 3 miles to the southeast of the site.

The Puente Hills blind thrust fault extends eastward from Downtown Los Angeles to the City of Brea in northern Orange County. The Puente Hills blind thrust fault includes three north-dipping segments, named from east to west as the Coyote Hills segment, the Santa Fe Springs segment, and the Los Angeles segment. These segments are overlain by folds expressed at the surface as the Coyote Hills, Santa Fe Springs Anticline, and the Montebello Hills.

The Los Angeles segment of the Puente Hills blind thrust is located approximately 4 miles to the southeast of the site.

The Santa Fe Springs segment of the Puente Hills blind thrust fault is believed to be the cause of the October 1, 1987, Whittier Narrows Earthquake. Based on deformation of late Quaternary age sediments above this fault system and the occurrence of the Whittier Narrows earthquake, the Puente Hills blind thrust fault is considered an active fault capable of generating future earthquakes beneath the Los Angeles Basin. A maximum moment magnitude of 7.0 is estimated by researchers for the Puente Hills blind thrust fault.

The Mw 6.7 Northridge earthquake was caused by the sudden rupture of a previously unknown, blind thrust fault. This fault has since been named the Northridge Thrust, however it is also known in some of the literature as the Pico Thrust. It has been assigned a maximum magnitude of 6.9 and a 1,500 to 1,800 year recurrence interval. The Northridge thrust is located 15¼-miles to the northwest of the site.

SEISMIC HAZARDS AND DESIGN CONSIDERATIONS

The primary geologic hazard at the site is moderate to strong ground motion (acceleration) caused by an earthquake on any of the local or regional faults. The potential for other earthquake-induced hazards was also evaluated including surface rupture, liquefaction, dynamic settlement, inundation and landsliding.

Surface Rupture

In 1972, the Alquist-Priolo Special Studies Zones Act (now known as the Alquist-Priolo Earthquake Fault Zoning Act) was passed into law. As revised in 2018, The Act defines "Holocene-active" Faults utilizing the same aging criteria as that used by California Geological Survey (CGS). However, established state policy has been to zone only those faults which have direct evidence of movement within the last 11,700 years. It is this recency of fault movement that the CGS considers as a characteristic for faults that have a relatively high potential for ground rupture in the future.

CGS policy is to delineate a boundary from 200 to 500 feet wide on each side of the Holocene-Active fault trace based on the location precision, the complexity, or the regional significance of the fault. If a site lies within an Earthquake Fault Zone, a geologic fault rupture investigation must be performed that demonstrates that the proposed building site is not threatened by surface displacement from the fault before development permits may be issued.

Review of the Earthquake Zones of Required Investigation Map of the Hollywood Quadrangle (CGS, 2014) indicates that the subject site is not located within an Alquist-Priolo Earthquake Fault Zone. The closest zone is the Hollywood Fault Zone, which is located approximately one mile to the north of the subject site. A copy of this map is enclosed herein.

Ground rupture is defined as surface displacement which occurs along the surface trace of the causative fault during an earthquake. Based on research of available literature and results of site reconnaissance, no known active or potentially active faults underlie the subject site. In addition, the subject site is not located within an Alquist-Priolo Earthquake Fault Zone. Based on these considerations, the potential for surface ground rupture at the subject site is considered low.

Liquefaction

Liquefaction is a phenomenon in which saturated silty to cohesionless soils below the groundwater table are subject to a temporary loss of strength due to the buildup of excess pore pressure during cyclic loading conditions such as those induced by an earthquake. Liquefaction-related effects include loss of bearing strength, amplified ground oscillations, lateral spreading, and flow failures.

Review of the California Seismic Hazards Zones Map for the Hollywood Quadrangle (CDMG 1999), indicates that the subject site is not located within a "Liquefiable" area. This determination is based on groundwater records, soil type and distance to a fault capable of producing a substantial earthquake. A copy of this map has been enclosed to this report.

Two site-specific liquefaction analyses were performed following the Recommended Procedures for Implementation of the California Geologic Survey Special Publication 117A, Guidelines for Analyzing and Mitigating Seismic Hazards in California (CGS, 2008), and the EERI Monograph (MNO-12) by Idriss and Boulanger (2008). This semi-empirical method is based on a correlation between measured values of Standard Penetration Test (SPT) resistance and field performance data.

Groundwater was encountered during exploration, at depths ranging between 19 and 20 feet below the existing site grade. According to the Seismic Hazard Zone Report of the Hollywood 7½-Minute Quadrangle (CDMG, 2006), the historically highest groundwater level for the site was approximately 10 feet below the existing ground surface. The enclosed liquefaction analysis takes into consideration the historically highest and current groundwater levels.

Section 11.8.3 of ASCE 7-10 indicates that the potential for liquefaction shall be evaluated utilizing an acceleration consistent with the MCE_G PGA. Utilizing the USGS U.S. Seismic Design Maps tool, this corresponds to a PGA_M of 0.99g. The USGS Probabilistic Seismic Hazard Deaggregation program (USGS, 2014) indicates a PGA of 0.90g (2 percent in 50 years ground motion) and a modal magnitude of 6.9 for the site. The liquefaction potential evaluation was performed by utilizing a magnitude 6.9 earthquake, and a peak horizontal acceleration of 0.99g.

The enclosed "Empirical Estimations of Liquefaction Potential" are based on the results obtained from Borings B1 and B2, which were prosecuted to depths of 130 and 180 feet below grade, respectively. Standard Penetration Test (SPT) data were collected at 5 and 10-foot intervals. Samples of the collected materials were conveyed to the laboratory for testing and analysis. The percent passing a Number 200 sieve, Atterberg Limits, and the plasticity index (PI) of representative samples of the soils encountered in the exploratory borings are presented on the enclosed E-Plates and F-Plates.

Based on CGS Special Publication 117A (CDMG, 2008) and (Bray and Sancio, 2006), the vast majority of liquefaction hazards are associated with sandy soils and silty soils of low plasticity. Furthermore, soils having a PI greater than 18 exhibit clay-like behavior, and the liquefaction potential of these soils are considered to be low. The results of Atterberg Limits testing (shown on Plate F) indicate that some of soil layers below the subject site have PI greater than 18. Therefore, these soils are not considered prone to liquefaction, and the analysis of these soil layers was turned off in the liquefaction susceptibility columns.

The site-specific liquefaction analyses included in the Appendix indicates that the site soils would not be prone to liquefaction during the ground motion expected during the design-based seismic event.

Dynamic Settlement

As explained in the previous section, the site soils are not considered prone to liquefaction. Therefore, it is the opinion of this firm that the anticipated liquefaction settlement at the site may be considered to be negligible.

The proposed structure will extend below the current and historically highest groundwater levels. Therefore, dynamic dry-sand settlement is not expected below the proposed structure.

Tsunamis, Seiches and Flooding

Tsunamis are large ocean waves generated by sudden water displacement caused by a submarine earthquake, landslide, or volcanic eruption. Review of the County of Los Angeles Flood and Inundation Hazards Map, Leighton (1990), indicates the site does not lie within the mapped tsunami inundation boundaries.

Seiches are oscillations generated in enclosed bodies of water which can be caused by ground shaking associated with an earthquake. Review of the County of Los Angeles Flood and Inundation Hazards Map, Leighton (1990), indicates the site is located within mapped inundation boundaries if the Mulholland Reservoir should breach. However, review of the applicable Flood Insurance Rate Map (06037C1605F) indicates the site lies within an area of minimal flood hazard.

A determination of whether a higher site elevation would remove the site from the potential inundation zones is beyond the scope of this investigation.

Landsliding

The probability of seismically-induced landslides occurring on the site is considered to be low due to the general lack of elevation difference across or adjacent to the site.

CONCLUSIONS AND RECOMMENDATIONS

Based upon the exploration, laboratory testing, and research, it is the preliminary finding of Geotechnologies, Inc. that construction of the proposed high-rise structure is considered feasible from a geotechnical engineering standpoint provided the advice and recommendations presented herein are followed and implemented during construction.

This report is preliminary in nature because the proposed project plan remains under development and is not well defined at this time. Due to its preliminary nature, this report is not intended for submission to the building official for building permit purposes. Once the proposed development plan achieves refinement, this firm should re-evaluate the recommendations presented herein, to ensure they are suitable for the proposed development. A final geotechnical engineering investigation, suitable for submission to the building official for building permit purposes, will be prepared at that time.

Due to the preliminary nature of the project, the structural loads are currently not available. Detailed structural loads shall be provided to this firm for analyses when the project achieves more definition. Foundation recommendations presented herein shall be considered preliminary and are subject to be confirmed or modified subsequent to review of foundation loads.

Between 5 and 8 feet of existing fill materials was encountered during exploration at the site. The existing fill materials are considered to be unsuitable for support of new foundations, floor slabs, or additional fill. It is however anticipated that the existing fill materials will be removed during excavation of the proposed subterranean levels.

The lowest subterranean level of the proposed structure is expected to extend to an approximate depth of 33 feet below the existing ground surface, with foundations expected to extend between 5 and 10 feet below this depth. Preliminarily, it is anticipated that the proposed structure may be supported on a mat foundation bearing in the older alluvial soils present near the subterranean subgrade. Detailed structural loads, size and dimensions of the mat footing shall be provided to this firm for analyses when the project achieves more definition. The design of the foundation system for support of the tower will be an iterative process between the structural engineer and the geotechnical engineer.

Groundwater was encountered at depths ranging between 19 and 20 feet below the existing site grade during exploration. Therefore, excavation of the proposed subterranean levels will require dewatering measures to provide a dry excavation. It is expected that a formal pre-construction temporary dewatering program consisting of wells or well-points will be required to lower the groundwater table prior to excavation of the subterranean levels. The expected number and depths of well-points, expected flow rates, and expected pre-pumping time frames should be determined during a dewatering test program conducted by a qualified dewatering consultant.

Once the temporary construction dewatering is discontinued, the water table will likely return to its current elevation. Since the elevation of the water table is higher than the proposed bottom of structure, hydrostatic forces on the walls and floor will result. It is recommended the proposed development be designed to resist hydrostatic forces in lieu of installation of a permanent dewatering system. This will eliminate the need for maintenance of a permanent dewatering system and continuous handling of waters pumped from the system. Hydrostatic forces are addressed in the “Retaining Wall Design” and “Foundation Design” sections of this report.

It is recommended that the mat foundation system and retaining walls be completely watertight in order to prevent water seepage through normal shrinkage cracks or construction joints. It is recommended care be taken in the design and installation of waterproofing to avoid moisture problems, and to prevent water seepage into the structure. The design and inspection of 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, floors, and foundations.

Although temporary dewatering will lower the groundwater elevation prior to construction, the soils at the proposed subgrade level should be expected to be well above their optimum moisture level. These soils could be wet and soft. The placement of a mat of gravel over the bottom excavation will most likely be necessary to protect the subgrade soils from disturbance, create a firm working surface, and provide a firm bottom that is suitable for support of the proposed mat foundation. Placement of gravel and wet subgrade soils are discussed in a following section.

Due to the depth of the proposed subterranean levels, and the proximity of the property lines, excavations around the perimeter of the proposed structure will require shoring in order to provide a stable excavation. Shoring recommendations are provided in the "Excavations" section of this report.

The validity of the conclusions and design recommendations presented herein is dependent upon review of the geotechnical aspects of the proposed construction by this firm. The subsurface conditions described herein have been projected from excavations on the site as indicated and should in no way be construed to reflect any variations which may occur between these excavations or which may result from changes in subsurface conditions. Any changes in the design, as outlined in this report, should be reviewed by this office. The recommendations contained herein should not be considered valid until reviewed and modified or reaffirmed subsequent to such review.

SEISMIC DESIGN CONSIDERATIONS

Seismic Shearwave Velocity Measurements

Downhole seismic velocity measurements were performed by GeoPentech within Boring Number 2, which was excavated to a depth of 180 feet below the existing site grade. However, the survey was conducted to a maximum depth of 149 feet. Results of the seismic velocity measurements are presented in the Downhole Seismic Tests Results report by GeoPentech, dated October 22, 2019.

The following table presents the average shear wave velocities of the underlying earth materials measured within Boring Number 2. A copy of the GeoPentech's report is enclosed at the end of the Appendix.

Depth Range (feet)	Average Shear Wave Velocity (feet/second)
0 to 5	863
5 to 35	1,166
35 to 50	934
50 to 75	1,256
75 to 95	1,010
95 to 149	1,459

2019 California Building Code Seismic Parameters

Based on information derived from the subsurface investigation, the subject site is classified as Site Class D, which corresponds to a "Stiff Soil" Profile, according to Table 20.3-1 of ASCE 7-10, and ASCE 7-16. This information and the site coordinates were input into the OSHPD seismic utility program at https://seismicmaps.org in order to calculate ground motion parameters for the site.

2019 CALIFORNIA BUILDING CODE SEISMIC PARAMETERS	
Site Class	D
Mapped Spectral Acceleration at Short Periods (S_S)	2.092g
Site Coefficient (F_a)	1.0
Maximum Considered Earthquake Spectral Response for Short Periods (S_{MS})	2.092g
Five-Percent Damped Design Spectral Response Acceleration at Short Periods (S_{DS})	1.395g
Mapped Spectral Acceleration at One-Second Period (S_1)	0.750g
Site Coefficient (F_v)	1.7
Maximum Considered Earthquake Spectral Response for One-Second Period (S_{M1})	1.275g
Five-Percent Damped Design Spectral Response Acceleration for One-Second Period (S_{D1})	0.850g

* *According to ASCE 7-16, a Long Period Site Coefficient (F_v) of 1.7 may be utilized provided that the value of the Seismic Response Coefficient (C_s) is determined by Equation 12.8-2 for values of $T \leq 1.5T_s$ and taken as equal to 1.5 times the value computed in accordance with either Equation 12.8-3 for $T_L \geq T > 1.5T_s$ or equation 12.8-4 for $T > T_L$. Alternatively, a site-specific ground motion hazard analysis may be performed in accordance with ASCE 7-16 Section 21.1 and/or a ground motion hazard analysis in accordance with ASCE 7-16 Section 21.2 to determine ground motions for any structure.*

EXPANSIVE SOILS

The onsite geologic materials are in the Moderate to High expansion range. The Expansion Index was found to be 82 and 128 for representative bulk samples. Recommended reinforcing is provided in the “Foundation Design” and “Slabs on Grade” sections of this report.

SOIL CORROSION POTENTIAL

The results of the soil corrosivity testing performed on four samples representative of the onsite soils by Project X Corrosion Engineering indicate that the electrical resistivities of the soils are severely corrosive to general metals when saturated. The soil pH value of the samples was between 7.8 and 8.2. The pH was determined to be at levels not detrimental to copper or aluminum alloys but can allow corrosion of steel and iron in moist environments. Chloride levels in the samples are low and may cause insignificant corrosion of metals. Ammonia and Nitrates concentrations are not high enough to cause accelerated corrosion of copper and copper alloys.

Sulfate content in the samples are considered negligible for corrosion of metals and cement. Special cement types need not be utilized for concrete structures in contact with the soils, since the sulfate content of the soils is negligible.

Detailed results, discussion of results and recommended mitigating measures are provided within the enclosed Corrosion Evaluation Report prepared by Project X Corrosion Engineering, dated October 9, 2019.

TEMPORARY DEWATERING

Groundwater was encountered during exploration, to depths ranging between 19 and 20 feet below grade. It is anticipated that the lowest subterranean level will extend to a depth of 33 feet below grade, and the mat foundation system may extend 5 to 10 feet below that depth.

Since the proposed subterranean level will extend well below the current groundwater level, it is recommended that a qualified dewatering consultant should be retained during the design phase of the project. The expected number and depths of well-points, expected flow rates, and expected pre-pumping time frames should be determined during a dewatering test program conducted by a qualified dewatering consultant.

It is anticipated that the well points will collect the majority of the water, however, even after pre-pumping, some free water may be encountered during excavation due to entrapment within cohesive lenses. Such water may be collected within the excavation through the use of French drains and sump pumps.

Wet Subgrade Soils

Soils at the proposed subgrade level should be expected to be well above their optimum moisture level. A representative of this office should observe the subgrade as it becomes exposed so that the recommendations provided herein may be revised or reaffirmed as necessary. At this time, pumping, rutting, and disturbance of the high-moisture content soils should be expected to occur during operation of heavy equipment. In order to minimize disturbance of the subgrade bearing soils, provide a firm working surface, and provide a subgrade suitable for support of the proposed mat foundation, it is recommended the subgrade be protected and/or stabilized as it becomes exposed.

Protection or stabilization of the subgrade may be accomplished by placement of a layer of angular ¾-inch gravel. This layer should be a minimum of 1 foot in thickness; however, the exact thickness of the gravel would be a trial and error procedure and would be determined in the field. The gravel should be placed and vibrated to a dense state as the subgrade becomes exposed. The elevation at the bottom of excavation will require adjustment to provide space for the gravel mat. It is not recommended that rubber tire construction equipment attempt to operate directly on the subgrade soils prior to placing the gravel. Direct operation of rubber tire equipment on soft subgrade soils will likely result in excessive disturbance to the soils, which in turn could result in a delay to the construction schedule. Extreme care should be utilized to place gravel as the subgrade becomes exposed.

METHANE ZONES

This office has reviewed the City of Los Angeles Methane Zone and Methane Buffer Zones map. Based on this review it appears that the subject site is not located within a Methane Zone or a Buffer Zone as designated by the City.

GRADING GUIDELINES

The following recommendations are provided for any miscellaneous grading that may be required, such as trench backfill or subgrade preparation.

Site Preparation

- A thorough search should be made for possible underground utilities and/or structures. Any existing or abandoned utilities or structures located within the footprint of the proposed grading should be removed or relocated as appropriate.
- All vegetation, existing fill, and soft or disturbed geologic materials should be removed from the areas to receive controlled fill. All existing fill materials and any disturbed geologic materials resulting from grading operations shall be completely removed and properly recompacted prior to foundation excavation.
- Any vegetation or associated root system located within the footprint of the proposed structures should be removed during grading.
- Subsequent to the indicated removals, the exposed grade shall be scarified to a depth of six inches, moistened to optimum moisture content, and recompacted in excess of the minimum required comparative density.
- The excavated areas shall be observed by the geotechnical engineer prior to placing compacted fill.

Compaction

The City of Los Angeles Department of Building and Safety requires a minimum comparative compaction of 95 percent of the laboratory maximum density where the soils to be utilized in the fill have less than 15 percent finer than 0.005 millimeters. Fill materials having more than 15 percent finer than 0.005 millimeters may be compacted to a minimum of 90 percent of the maximum density.

All fill should be mechanically compacted in layers not more than 8 inches thick. Based on the moderate to high expansion index of the site soils, it is recommended that fill materials are moisture conditioned to approximately 3 to 5 percent over optimum moisture content before recompaction.

Field observation and testing shall be performed by a representative of the geotechnical engineer during grading to assist the contractor in obtaining the required degree of compaction and the proper moisture content. Where compaction is less than required, additional compactive effort shall be made with adjustment of the moisture content, as necessary, until a minimum of 90 percent (or 95 percent for cohesionless soils having less than 15 percent finer than 0.005 millimeters) compaction is obtained.

Acceptable Materials

The excavated onsite materials are considered satisfactory for reuse in the controlled fills as long as any debris and/or organic matter is removed. Any imported materials shall be observed and tested by the representative of the geotechnical engineer prior to use in fill areas. Imported materials should contain sufficient fines so as to be relatively impermeable and result in a stable subgrade when compacted. Any required import materials should consist of geologic materials with an expansion index of less than 50. The water-soluble sulfate content of the import materials should be less than 0.1% percentage by weight.

Imported materials should be free from chemical or organic substances which could affect the proposed development. A competent professional should be retained in order to test imported materials and address environmental issues and organic substances which might affect the proposed development.

Utility Trench Backfill

Utility trenches should be backfilled with controlled fill. The utility should be bedded with clean sands at least one foot over the crown. The remainder of the backfill may be onsite soil compacted to 90 percent (or 95 percent for cohesionless soils having less than 15 percent finer than 0.005 millimeters) of the laboratory maximum density. Utility trench backfill should be tested by representatives of this firm in accordance with the most recent revision of ASTM D-1557.

Shrinkage

Shrinkage results when a volume of soil removed at one density is compacted to a higher density. A shrinkage factor between 5 and 15 percent should be anticipated when excavating and recompacting the existing fill and underlying native geologic materials on the site to an average comparative compaction of 92 percent.

Weather Related Grading Considerations

When rain is forecast all fill that has been spread and awaits compaction shall be properly compacted prior to stopping work for the day or prior to stopping due to inclement weather. These fills, once compacted, shall have the surface sloped to drain to an area where water can be removed.

Temporary drainage devices should be installed to collect and transfer excess water to the street in non-erosive drainage devices. Drainage should not be allowed to pond anywhere on the site, and especially not against any foundation or retaining wall. Drainage should not be allowed to flow uncontrolled over any descending slope.

Work may start again, after a period of rainfall, once the site has been reviewed by a representative of this office. Any soils saturated by the rain shall be removed and aerated so that the moisture content will fall within three percent of the optimum moisture content.

Surface materials previously compacted before the rain shall be scarified, brought to the proper moisture content and recompacted prior to placing additional fill, if considered necessary by a representative of this firm.

Geotechnical Observations and Testing During Grading

Geotechnical observations and testing during grading are considered to be a continuation of the geotechnical investigation. It is critical that the geotechnical aspects of the project be reviewed by representatives of Geotechnologies, Inc. during the construction process. Compliance with the design concepts, specifications or recommendations during construction requires review by this firm during the course of construction. Any fill which is placed should be observed, tested, and verified if used for engineered purposes. Please advise this office at least twenty-four hours prior to any required site visit.

Proper compaction is necessary to reduce settlement of overlying improvements. Some settlement of compacted fill should be anticipated. Any utilities supported therein should be designed to accept differential settlement. Differential settlement should also be considered at the points of entry to the structure.

FOUNDATION DESIGN

MAT FOUNDATION

Due to the preliminary nature of the project, the structural loads are currently not available. Foundation recommendations presented herein shall be considered preliminary and are subject to be confirmed or modified subsequent to review of foundation loads. Detailed structural loads, size and dimensions of the mat footing shall be provided to this firm for analyses when the project achieves more definition. The design of the foundation system for support of the tower will be an iterative process between the structural engineer and the geotechnical engineer.

Preliminarily, it is anticipated that the proposed structure may be supported on a mat foundation system, bearing in the dense older alluvial soils expected at the subterranean subgrade. It is estimated that the proposed structure will have an average bearing pressure between 2,000 and 4,000 pounds per square foot. Foundation bearing pressure will vary across the mat footings, with the highest concentrated loads located at the central cores of the mat foundations.

The proposed mat foundation shall bear in the older alluvial soils expected at the subterranean subgrade. For preliminary design purposes, an allowable bearing pressure of up to 5,000 pounds per square foot, with locally higher pressures up to 7,000 pounds per square foot may be utilized in the mat foundation design.

The mat foundation may be designed utilizing a modulus of subgrade reaction of 250 pounds per cubic inch. This value is a unit value for use with a one-foot square footing. The modulus should be reduced in accordance with the following equation when used with larger foundations.

$$K = K_1 * [(B + 1) / (2 * B)]^2$$

where K = Reduced Subgrade Modulus
K_1 = Unit Subgrade Modulus
B = Foundation Width (feet)

The bearing values indicated above are for the total of dead and frequently applied live loads and may be increased by one third for short duration loading, which includes the effects of wind or seismic forces. Since the recommended bearing value is a net value, the weight of concrete in the foundations may be taken as 50 pounds per cubic foot and the weight of the soil backfill may be neglected when determining the downward load on the foundations.

Hydrostatic Considerations for Mat Foundations

Where constructed below the groundwater level, mat foundations shall be waterproofed and designed to withstand the hydrostatic uplift pressure based on the historically highest groundwater level. As discussed in the "Groundwater" Section of this report, the historically highest groundwater level on the site may be considered to correspond to a depth of 10 feet below grade. The uplift pressure to be used in design should be 62.4(H) pounds per square foot, where "H" is the height of the height of the historically highest groundwater level above the bottom of the mat foundation in feet.

If necessary, uplift anchors may be designed to provide resistance against the anticipated hydrostatic uplift pressures acting on the recommended mat foundations. Uplift anchors should be a minimum of 12 inches in diameter and should be embedded a minimum of 20 feet into the underlying native soils. Preliminarily, it is assumed that pressure grouted anchors will be utilized. Uplift anchors may be designed using a frictional capacity of 3 kips per lineal foot. Based on communication with the structural engineer, uplift anchors are not anticipated. In the event that uplift anchors will be required, please contact this office so installation and testing guidelines are provided.

Lateral Mat Foundation Design

Resistance to lateral loading may be provided by friction acting at the base of the mat and by passive earth pressure. An allowable coefficient of friction of 0.35 may be used with the dead load forces.

Passive geologic pressure for the sides of foundations poured against undisturbed soil may be computed as an equivalent fluid having a density of 300 pounds per cubic foot with a maximum earth pressure of 3,000 pounds per square foot. The passive and friction components may be combined for lateral resistance without reduction. A one-third increase in the passive value may be used for short duration loading such as wind or seismic forces.

Mat Foundation Settlement

Settlement of the mat foundation will be analyzed once structural loads are available.

RETAINING WALL DESIGN

Retaining walls on the order of 33 feet in height are anticipated for the proposed subterranean parking levels. As a precautionary measure, recommendations for retaining walls up to a height of 40 feet are provided herein. It is anticipated these walls will be restrained. Foundations for these walls may be designed in accordance with the previous "Foundation Design" section.

As previously discussed, it is recommended that the proposed structure be designed to resist hydrostatic forces in lieu of installing a permanent dewatering system at the base of the structure. Wall pressures are provided below for hydrostatic design.

Additional active pressure should be added to the retaining wall design for any additional surcharge conditions, such as adjacent traffic and structures. Additional surcharge pressures should be considered for all adjacent foundations located within a 1:1 (45 degree) surcharge plane. Based on review of the enclosed Plot Plan and Cross Section A-A', it is anticipated that the northern retaining walls will be surcharged by existing structures. Information regarding the loading of the adjacent foundations will be required to analyze the anticipated surcharge pressure.

Vehicular traffic from adjacent driveways and parking areas is expected in the vicinity of the proposed retaining walls. For traffic surcharge, the upper 10 feet of any retaining wall adjacent to streets, driveways or parking areas should be designed to resist a uniform lateral pressure of 100 pounds per square foot, acting as a result of an assumed 300 pounds per square foot traffic surcharge. If the traffic is more than 10 feet from the retaining walls, the traffic surcharge may be neglected.

Restrained Retaining Walls

Restrained subterranean retaining walls supporting a level back slope may be designed to resist a triangular distribution of earth pressure. It is recommended the walls be designed to resist the greater of the at-rest pressure, or the active pressure plus the seismic pressure, as discussed in the "Dynamic (Seismic) Earth Pressure" section below. The pressures provided in the following table are based on a full hydrostatic design. These pressures shall be applied over the entire length of the wall.

RESTRAINED BASEMENT WALLS ***(HYDROSTATIC DESIGN)***		
	AT-REST EARTH PRESSURE (Pounds per Cubic Foot) Includes Hydrostatic Pressure of 62.4 pcf	**ACTIVE EARTH PRESSURE *(To be Combined with Dynamic Seismic Earth Pressure) Includes Hydrostatic Pressure of 62.4 pcf**
Height of Wall (Feet)	Triangular Distribution of Pressure (Pounds per Cubic Foot)	Triangular Distribution of Pressure (Pounds per Cubic Foot)
Up to 40 feet	94	93*

Additional active pressure should be added for a surcharge condition due to sloping ground, vehicular traffic or adjacent structures.

Dynamic (Seismic) Earth Pressure

Retaining walls exceeding 6 feet in height shall be designed to resist the additional earth pressure caused by seismic ground shaking. A triangular pressure distribution should be utilized for the additional seismic loads, with an equivalent fluid pressure of 28 pounds per cubic foot. When using the code based loading equations, the seismic earth pressure should be combined with the lateral active earth pressure for analyses of restrained basement walls under seismic loading condition.

Miscellaneous Drained Cantilever Retaining Walls

Miscellaneous cantilever retaining walls to be built above the historically highest groundwater levels may be designed utilizing a triangular distribution of pressure. Cantilever retaining walls may be designed for 45 pounds per cubic foot for walls retaining up to 10 feet of earth. This pressure takes into account the moderate to high potential expansion of the site soils. This pressure assumes that the wall will be built above the historically highest groundwater level, and that a subdrain system will be installed behind the wall. In addition to this pressure, cantilever walls greater than 6 feet in height shall be designed to resist seismic earth pressure indicated in the "Dynamic (Seismic) Earth Pressure" section above.

For this equivalent fluid pressure to be valid, walls which are to be restrained at the top should be backfilled prior to the upper connection being made. Additional active pressure should be added for a surcharge condition due to sloping ground, vehicular traffic or adjacent structures.

Waterproofing

Moisture affecting retaining walls is one of the most common post- construction complaints. Poorly applied or omitted waterproofing can lead to efflorescence or standing water inside the building. Efflorescence is a process in which a powdery substance is produced on the surface of the concrete by the evaporation of water. The white powder usually consists of soluble salts such as gypsum, calcite, or common salt.

It is recommended that retaining walls be waterproofed. Waterproofing design and inspection of its installation 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 below grade walls.

Retaining Wall Drainage

This section is intended for miscellaneous drained retaining walls, to be built above the historically highest groundwater level. A drainage system is not anticipated for the subterranean parking garage retaining walls, because these will be designed to fully resist hydrostatic forces.

Where miscellaneous retaining walls are designed for a drained condition, these walls should be provided with a subdrain consisting of a perforated pipe, placed with perforations facing down, covered with a minimum of 12 inches of gravel, and a compacted fill blanket or other seal at the surface. The gravel shall be wrapped in filter fabric. The gravel may consist of three-quarter inch to one inch crushed rocks.

As an alternative to the standard perforated subdrain pipe and gravel drainage system, the use of gravel pockets and weepholes is an acceptable drainage method. Weepholes shall be a minimum of 4 inches in diameter, placed at 8 feet on center along the base of the wall. Gravel pockets shall be a minimum of 1 cubic foot in dimension and may consist of three-quarter inch to one inch crushed rocks, wrapped in filter fabric. A collector pipe shall be installed to direct collected water to a sump.

Certain types of subdrain pipe are not acceptable to the various municipal agencies. It is recommended that prior to purchasing subdrainage pipe, the type and brand is cleared with the proper municipal agencies.

It is recommended a qualified dewatering consultant be retained in order to establish design flow rates and ensure adequate sizing of subdrainage pipes and systems. Subdrainage pipes should outlet to an acceptable location.

Retaining Wall Backfill

Any required backfill should be mechanically compacted in layers not more than 8 inches thick, to at least 90 (or 95 percent for cohesionless soils having less than 15 percent finer than 0.005 millimeters) of the maximum density in general accordance with the most recent revision of ASTM D 1557 method of compaction. Flooding should not be permitted. Proper compaction of the backfill will be necessary to reduce settlement of the backfill and to reduce settlement of overlying walks and paving. Some settlement of required backfill should be anticipated, and any utilities supported therein should be designed to accept differential settlement.

TEMPORARY EXCAVATIONS

It is anticipated that excavations between 38 and 43 feet in vertical height will be required for construction of the proposed subterranean levels and mat foundation. The excavations are expected to expose fill and dense native alluvial soils, which are suitable for vertical excavations up to 5 feet where not surcharged by adjacent traffic or structures.

Due to the presence of groundwater, the depth of the excavation, and the proximity of adjacent structures and public ways, excavation of the proposed subterranean levels will require shoring and dewatering measures to provide a stable and dry excavation. Shoring recommendations are provided in the following section.

All excavations should be stabilized within 30 days of initial excavation. Water should not be allowed to pond on top of the excavation nor to flow towards it. No vehicular surcharge should be allowed within 5 feet of the top of an unshored cut.

SHORING DESIGN

The following information on the design and installation of the shoring is as complete as possible at this time. It is suggested that a review of the final shoring plans and specifications be made by this office prior to bidding or negotiating with a shoring contractor be made.

The recommended method of shoring consists of steel soldier piles, placed in drilled holes and backfilled with concrete. Due to the depth of the soldier piles, it is anticipated they will be laterally braced utilizing drilled tie-back anchors.

Soldier Piles

Drilled cast-in-place soldier piles should be placed no closer than 2½ 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 earth materials. For design purposes, an allowable passive value for the earth materials below the bottom plane of excavation may be assumed to be 500 pounds per square foot per foot of depth, up to a maximum of 5,000 pounds per square foot. To develop the full lateral value, provisions should be implemented to assure firm contact between the soldier piles and the undisturbed earth materials.

The frictional resistance between the soldier piles and retained earth material may be used to resist the vertical component of the anchor load. The coefficient of friction may be taken as 0.35 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 500 pounds per square foot. The minimum depth of embedment for shoring piles is 5 feet below the bottom of the footing excavation, or 7 feet below the bottom of excavated plane, whichever is deeper.

Casing or polymer drilling fluid may be required should caving be experienced in the saturated earth materials. 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 5 feet.

Groundwater was encountered during exploration at depths between 19 and 20 feet below the existing site grade. Depending on the draw down level associated with the future dewatering program, it is anticipated that the proposed piles will likely encounter water. Piles placed below the water level will require the use of a tremie to place the concrete into the bottom of the hole. A tremie shall consist of a water-tight tube having a diameter of not less than 6 inches with a hopper at the top. The tube shall 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 shall 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 shall be closed at the start of the work to prevent water entering the tube and shall be entirely sealed at all times, except when the concrete is being placed. The tremie tube shall be kept full of concrete. The flow shall be continuous until the work is completed and the resulting concrete seal shall be monolithic and homogeneous. The tip of the tremie tube shall always be kept about five feet below the surface of the concrete and definite steps and safeguards should be taken to ensure that the tip of the tremie tube is never raised above the surface of the concrete.

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

Lagging

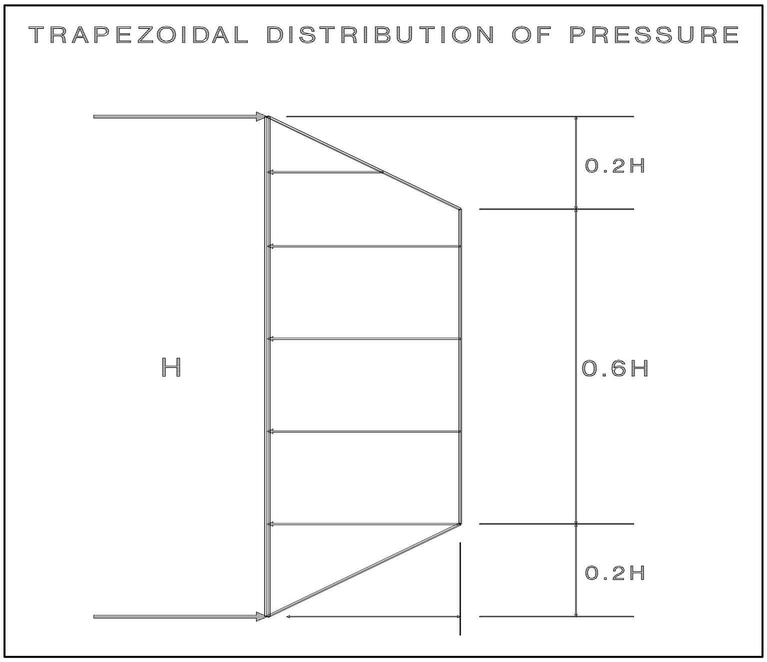
It is recommended that lagging be installed throughout the entire depth of the excavation. Soldier piles and anchors should be designed for the full anticipated pressures. Due to arching in the geologic materials, the pressure on the lagging will be less. It is recommended that the lagging should be designed for the full design pressure but be limited to a maximum of 400 pounds per square foot. It is recommended that a representative of this firm observe the installation of lagging to insure uniform support of the excavated embankment.

Lateral Pressures

A trapezoidal distribution of lateral earth pressure would be appropriate where shoring is to be restrained at the top by bracing or tie backs. The trapezoidal distribution of pressure is shown in the diagram below. The shoring wall pressure for design of restrained shoring is presented in the following table:

Height of Shoring (feet)	**Restrained Shoring System Lateral Earth Pressure (psf)* Trapezoidal Distribution of Pressure**
Up to 45 feet	25H psf

*Where H is the height of the shoring in feet.



Where a combination of sloped embankment and shoring is utilized, the pressure will be greater and must be determined for each combination. Additional active pressures should be applied where the shoring will be surcharged by adjacent traffic or structures.

Tieback Anchor Design and Installation

Tieback anchors may be used to resist lateral loads. Friction anchors are recommended. For design purposes, it may be assumed that the active wedge 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.

Tieback anchors may be installed between 20 and 45 degrees below the horizontal. Caving may occur within granular materials. Where caving occurs, the following provisions should be implemented in order to minimize such caving. 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.

Drilled friction anchors constructed without utilizing pressure-grouting techniques may be designed for a skin friction of 500 pounds per square foot. Depending on the techniques utilized, and the experience of the contractor performing the installation, it is anticipated that a skin friction of 2,500 pounds per square foot could be utilized for post-grouted anchors, provided the design does not rely on end-bearing plates to provide the necessary capacity. Only the frictional resistance developed beyond the active wedge should be utilized in resisting lateral loads. Anchors should be placed at least 6 feet on center to be considered isolated.

Tieback Anchor Testing

At least 10 percent of the anchors should be selected for “Quick”, 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. Where satisfactory tests are not achieved on these initial anchors, the anchor diameter and/or length should be increased until satisfactory test results are obtained.

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.

All of the remaining anchors should be tested to at least 150 percent of design load. The total deflection during the 150 percent 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.

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. Where satisfactory tests are not attained, the anchor diameter and/or length should be increased, or additional anchors installed until satisfactory test results are obtained. Where post-grouted anchors are utilized, additional post-grouting may be required. The installation and testing of the anchors should be observed by a representative of the soils engineer.

Raker Brace Foundations

An allowable bearing pressure of 4,000 pounds per square foot may be used for the design a raker foundations. This bearing pressure is based on a raker foundation a minimum of 24 inches in width and length as well as 24 inches in depth. The base of the raker foundations should be horizontal. Care should be employed in the positioning of raker foundations so that they do not interfere with the foundations for the proposed structure.

Deflection

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 shoring be designed for a maximum deflection of 1-inch at the top of the shored embankment. Embankments which are surcharged by adjacent structures should be designed for a maximum deflection of ½-inch at the top of the shored embankment. If greater deflection occurs during construction, additional bracing may be necessary to minimize settlement of adjacent buildings and streets. If desired to reduce the deflection, a greater active pressure could be used in the shoring design.

Monitoring

Because of the depth of the excavations, 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.

Shoring Observations

It is critical that the installation of shoring is observed by a representative of this office. Many local agencies require that shoring installation be performed under the continuous observation of the geotechnical engineer. The observations are made so that modifications of the recommendations can be made if variations in the earth material or groundwater conditions occur. Also, the observations will allow for a report to be prepared on the installation of shoring for the use of the local building official.

SLABS ON GRADE

Outdoor Concrete Slabs

Outdoor concrete flatwork should be a minimum of 5 inches in thickness. Outdoor concrete flatwork should be cast over properly controlled fill materials. Any geologic materials loosened or over-excavated should be wasted from the site or properly compacted to 90 percent (or 95 percent for cohesionless soils having less than 15 percent finer than 0.005 millimeters) of the maximum dry density.

Outdoor flatwork should be reinforced with a minimum of #3 steel bars on 18-inch centers each way.

Design of Slabs That Receive Moisture-Sensitive Floor Coverings

Geotechnologies, Inc. does not practice in the field of moisture vapor transmission evaluation and mitigation. Therefore, it is recommended that a qualified consultant be engaged to evaluate the general and specific moisture vapor transmission paths and any impact on the proposed construction. The qualified consultant should provide recommendations for mitigation of potential adverse impacts of moisture vapor transmission on various components of the structure.

Where dampness would be objectionable, it is recommended that the floor slabs should be waterproofed. A qualified waterproofing consultant should be retained in order to recommend a product or method which would provide protection for concrete slabs-on-grade.

All concrete slabs-on-grade should be supported on vapor retarder. The design of the slab and the installation of the vapor retarder should comply with the most recent revisions of ASTM E 1643 and ASTM E 1745. The vapor retarder should comply with ASTM E 1745 Class A requirements.

Where a vapor retarder is used, a low-slump concrete should be used to minimize possible curling of the slabs. The barrier can be covered with a layer of trimable, compactible, granular fill, where it is thought to be beneficial. See ACI 302.2R-32, Chapter 7 for information on the placement of vapor retarders and the use of a fill layer.

Concrete Crack Control

The recommendations presented in this report are intended to reduce the potential for cracking of concrete slabs-on-grade due to settlement. However even where these recommendations have been implemented, foundations, stucco walls and concrete slabs-on-grade may display some cracking due to minor soil movement and/or concrete shrinkage. The occurrence of concrete cracking may be reduced and/or controlled by limiting the slump of the concrete used, proper concrete placement and curing, and by placement of crack control joints at reasonable intervals, in particular, where re-entrant slab corners occur.

For standard control of concrete cracking, a maximum crack control joint spacing of 8 feet should not be exceeded. Lesser spacings would provide greater crack control. Joints at curves and angle points are recommended. The crack control joints should be installed as soon as practical following concrete placement. Crack control joints should extend a minimum depth of one-fourth the slab thickness. Construction joints should be designed by a structural engineer.

Complete removal of the existing fill soils beneath outdoor flatwork such as walkways or patio areas, is not required, however, due to the rigid nature of concrete, some cracking, a shorter design life and increased maintenance costs should be anticipated. In order to provide uniform support beneath the flatwork it is recommended that a minimum of 12 inches of the exposed subgrade beneath the flatwork be scarified and recompacted to 90 percent (or 95 percent for cohesionless soils having less than 15 percent finer than 0.005 millimeters) relative compaction.

PAVEMENTS

Prior to placing paving, the existing grade should be scarified to a depth of 12 inches, moistened as required to obtain optimum moisture content, and recompacted to 90 percent (or 95 percent for cohesionless soils having less than 15 percent finer than 0.005 millimeters) relative compaction as determined by the most recent revision of ASTM D 1557. The client should be aware that removal of all existing fill in the area of new paving is not required, however, pavement constructed in this manner will most likely have a shorter design life and increased maintenance costs. The following pavement sections are recommended:

Service	**Asphalt Pavement Thickness Inches**	**Base Course Inches**
Passenger Cars	4	6
Moderate Truck	5	8

Concrete paving may also be used on the project. For passenger cars and moderate truck traffic, concrete paving should be 6 inches of concrete over 4 inches of compacted base. For standard crack control maximum expansion joint spacing of 8 feet should not be exceeded. Lesser spacings would provide greater crack control. Joints at curves and angle points are recommended. Concrete paving should be reinforced with a minimum of #3 steel bars on 18-inch centers each way.

Aggregate base should be compacted to a minimum of 95 percent of the most recent revision of ASTM D 1557 laboratory maximum dry density. Base materials should conform to Sections 200-2.2 or 200-2.4 of the "Standard Specifications for Public Works Construction", (Green Book), latest edition.

The performance of pavement is highly dependent upon providing positive surface drainage away from the edges. Ponding of water on or adjacent to pavement can result in saturation of the subgrade materials and subsequent pavement distress.

SITE DRAINAGE

Proper surface drainage is critical to the future performance of the project. Saturation of a soil can cause it to lose internal shear strength and increase its compressibility, resulting in a change in the designed engineering properties. Proper site drainage should be maintained at all times.

All site drainage should be collected and transferred to the street in non-erosive drainage devices. The proposed structure should be provided with roof drainage. Discharge from downspouts, roof drains and scuppers should not be permitted on unprotected soils within five feet of the building perimeter. Drainage should not be allowed to pond anywhere on the site, and especially not against any foundation or retaining wall. Drainage should not be allowed to flow uncontrolled over any descending slope. Planters which are located within a distance equal to the depth of a retaining wall should be sealed to prevent moisture adversely affecting the wall. Planters which are located within five feet of a foundation should be sealed to prevent moisture affecting the earth materials supporting the foundation.

STORMWATER DISPOSAL

Recently regulatory agencies have been requiring the disposal of a certain amount of stormwater generated on a site by infiltration into the site soils. Increasing the moisture content of a soil can cause it to lose internal shear strength and increase its compressibility, resulting in a change in the designed engineering properties. This means that any overlying structure, including buildings, pavements and concrete flatwork, could sustain damage due to saturation of the subgrade soils. Structures serviced by subterranean levels could be adversely impacted by stormwater disposal by increasing the design fluid pressures on retaining walls and causing leaks in the walls. Proper site drainage is critical to the performance of any structure in the built environment.

Groundwater was encountered below the subject site at depths ranging between 19 and 20 feet below the ground surface during exploration, and the historically highest groundwater level was 10 feet below the ground surface. Based on the anticipated finished floor elevation of the proposed subterranean parking level, there is no potential for filtration of stormwater prior to its interaction with groundwater. Based on this consideration, stormwater infiltration is not recommended for the subject site.

Where infiltration of stormwater into the subgrade soils is not advisable, most Building Officials have allowed the stormwater to be filtered through soils in planter areas. Once the water has been filtered through a planter it may be released into the storm drain system. It is recommended that overflow pipes are incorporated into the design of the discharge system in the planters to prevent flooding. In addition, the planters shall be sealed and waterproofed to prevent leakage. Please be advised that adverse impact to landscaping and periodic maintenance may result due to excessive water and contaminants discharged into the planters.

It is recommended that the design team (including the structural engineer, waterproofing consultant, plumbing engineer, and landscape architect) be consulted in regards to the design and construction of filtration systems.

DESIGN REVIEW

Engineering of the proposed project should not begin until approval of the geotechnical report by the Building Official is obtained in writing. Significant changes in the geotechnical recommendations may result during the building department review process.

It is recommended that the geotechnical aspects of the project be reviewed by this firm during the design process. This review provides assistance to the design team by providing specific recommendations for particular cases, as well as review of the proposed construction to evaluate whether the intent of the recommendations presented herein are satisfied.

CONSTRUCTION MONITORING

Geotechnical observations and testing during construction are considered to be a continuation of the geotechnical investigation. It is critical that this firm review the geotechnical aspects of the project during the construction process. Compliance with the design concepts, specifications or recommendations during construction requires review by this firm during the course of construction. All foundations should be observed by a representative of this firm prior to placing concrete or steel. Any fill which is placed should be observed, tested, and verified if used for engineered purposes. Please advise Geotechnologies, Inc. at least twenty-four hours prior to any required site visit.

If conditions encountered during construction appear to differ from those disclosed herein, notify Geotechnologies, Inc. immediately so the need for modifications may be considered in a timely manner.

It is the responsibility of the contractor to ensure that all excavations and trenches are properly sloped or shored. All temporary excavations should be cut and maintained in accordance with applicable OSHA rules and regulations.

EXCAVATION CHARACTERISTICS

The exploration performed for this investigation is limited to the geotechnical excavations described. Direct exploration of the entire site would not be economically feasible. The owner, design team and contractor must understand that differing excavation and drilling conditions may be encountered based on boulders, gravel, oversize materials, groundwater and many other conditions. Fill materials, especially when they were placed without benefit of modern grading codes, regularly contain materials which could impede efficient grading and drilling. Southern California sedimentary bedrock is known to contain variable layers which reflect differences in depositional environment. Such layers may include abundant gravel, cobbles and boulders. Similarly bedrock can contain concretions. Concretions are typically lenticular and follow the bedding. They are formed by mineral deposits. Concretions can be very hard. Excavation and drilling in these areas may require full size equipment and coring capability. The contractor should be familiar with the site and the geologic materials in the vicinity.

CLOSURE AND LIMITATIONS

The purpose of this report is to aid in the design and completion of the described project. Implementation of the advice presented in this report is intended to reduce certain risks associated with construction projects. The professional opinions and geotechnical advice contained in this report are sought because of special skill in engineering and geology and were prepared in accordance with generally accepted geotechnical engineering practice. Geotechnologies, Inc. has a duty to exercise the ordinary skill and competence of members of the engineering profession. Those who hire Geotechnologies, Inc. are not justified in expecting infallibility, but can expect reasonable professional care and competence.

The recommendations of this report pertain only to the site investigated and are based upon the assumption that the geologic conditions do not deviate from those disclosed in the investigation. If any variations are encountered during construction, or if the proposed construction will differ from that anticipated herein, Geotechnologies, Inc. should be notified so that supplemental recommendations can be prepared.

This report is issued with the understanding that it is the responsibility of the owner, or the owner's representatives, to ensure that the information and recommendations contained herein are brought to the attention of the project architect and engineer and are incorporated into the plans. The owner is also responsible to see that the contractor and subcontractors carry out the geotechnical recommendations during construction.

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 control of this firm. Therefore, this report is subject to review and should not be relied upon after a period of three years.

Geotechnical observations and testing during construction is considered to be a continuation of the geotechnical investigation. It is, therefore, most prudent to employ the consultant performing the initial investigative work to provide observation and testing services during construction. This practice enables the project to flow smoothly from the planning stages through to completion.

Should another geotechnical firm be selected to provide the testing and observation services during construction, that firm should prepare a letter indicating their assumption of the responsibilities of geotechnical engineer of record. A copy of the letter should be provided to the regulatory agency for review. The letter should acknowledge the concurrence of the new geotechnical engineer with the recommendations presented in this report.

EXCLUSIONS

Geotechnologies, Inc. does not practice in the fields of methane gas, radon gas, environmental engineering, waterproofing, dewatering organic substances or the presence of corrosive soils or wetlands which could affect the proposed development including mold and toxic mold. Nothing in this report is intended to address these issues and/or their potential effect on the proposed development. A competent professional consultant should be retained in order to address environmental issues, waterproofing, organic substances and wetlands which might affect the proposed development.

GEOTECHNICAL TESTING

Classification and Sampling

The soil is continuously logged by a representative of this firm and classified by visual examination in accordance with the Unified Soil Classification system. The field classification is verified in the laboratory, also in accordance with the Unified Soil Classification System. Laboratory classification may include visual examination, Atterberg Limit Tests and grain size distribution. The final classification is shown on the excavation logs.

Samples of the geologic materials encountered in the exploratory excavations were collected and transported to the laboratory. Undisturbed samples of soil are obtained at frequent intervals. Unless noted on the excavation logs as an SPT sample, samples acquired while utilizing a hollow-stem auger drill rig are obtained by driving a thin-walled, California Modified Sampler with successive

30-inch drops of an automatic-trip 140-pound hammer. The soil is retained in brass rings of 2.50 inches outside diameter and 1.00 inch in height. The central portion of the samples are stored in close fitting, waterproof containers for transportation to the laboratory. Samples noted on the excavation logs as SPT samples are obtained in general accordance with the most recent revision of ASTM D 1586. Samples are retained for 30 days after the date of the geotechnical report.

Moisture and Density Relationships

The field moisture content and dry unit weight are determined for each of the undisturbed soil samples, and the moisture content is determined for SPT samples in general accordance with the most recent revision of ASTM D 4959 or ASTM D 4643. This information is useful in providing a gross picture of the soil consistency between exploration locations and any local variations. The dry unit weight is determined in pounds per cubic foot and shown on the "Excavation Logs", A-Plates. The field moisture content is determined as a percentage of the dry unit weight.

Expansion Index Testing

The expansion tests performed on the remolded samples are in accordance with the Expansion Index testing procedures, as described in the most recent revision of ASTM D 4829. The soil sample is compacted into a metal ring at a saturation degree of 50 percent. The ring sample is then placed in a consolidometer, under a vertical confining pressure of 1 lbf/square inch and inundated with distilled water. The deformation of the specimen is recorded for a period of 24 hour or until the rate of deformation becomes less than 0.0002 inches/hour, whichever occurs first. The expansion index, EI, is determined by dividing the difference between final and initial height of the ring sample by the initial height, and multiplied by 1,000. Results are presented on Plate D of this report.

Laboratory Compaction Characteristics

The maximum dry unit weight and optimum moisture content of a soil are determined in general accordance with the most recent revision of ASTM D 1557. A soil at a selected moisture content is placed in five layers into a mold of given dimensions, with each layer compacted by 25 blows of a 10 pound hammer dropped from a distance of 18 inches subjecting the soil to a total compactive effort of about 56,000 pounds per cubic foot. The resulting dry unit weight is determined. The procedure is repeated for a sufficient number of moisture contents to establish a relationship between the dry unit weight and the water content of the soil. The data when plotted represent a curvilinear relationship known as the compaction curve. The values of optimum moisture content and modified maximum dry unit weight are determined from the compaction curve. Results are presented on Plate D of this report.

Grain Size Distribution

These tests cover the quantitative determination of the distribution of particle sizes in soils. Sieve analysis is used to determine the grain size distribution of the soil larger than the Number 200 sieve. The most recent revision of ASTM D 422 is used to determine particle sizes smaller than the Number 200 sieve. A hydrometer is used to determine the distribution of particle sizes by a sedimentation process. The grain size distributions are plotted on the E-Plates presented in the Appendix of this report.

Atterberg Limits

Depending on their moisture content, cohesive soils can be solid, plastic, or liquid. The water contents corresponding to the transitions from solid to plastic or plastic to liquid are known as the Atterberg Limits. The transitions are called the plastic limit and liquid limit. The difference between the liquid and plastic limits is known as the plasticity index. ASTM D 4318 is utilized to determine the Atterberg Limits. The results are shown on the enclosed F-Plates.

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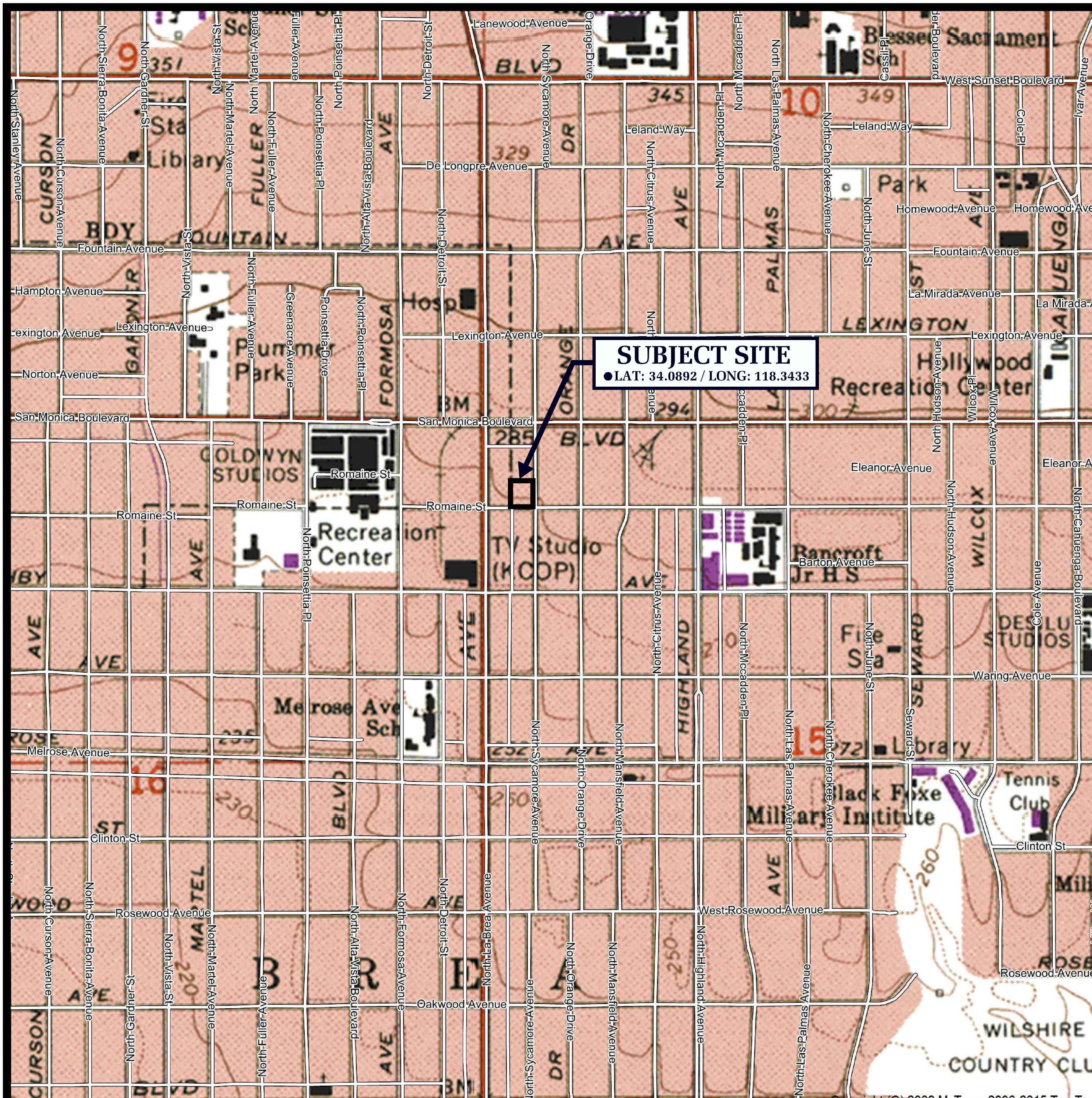

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MILES

0 1000 2000 3000
FEET

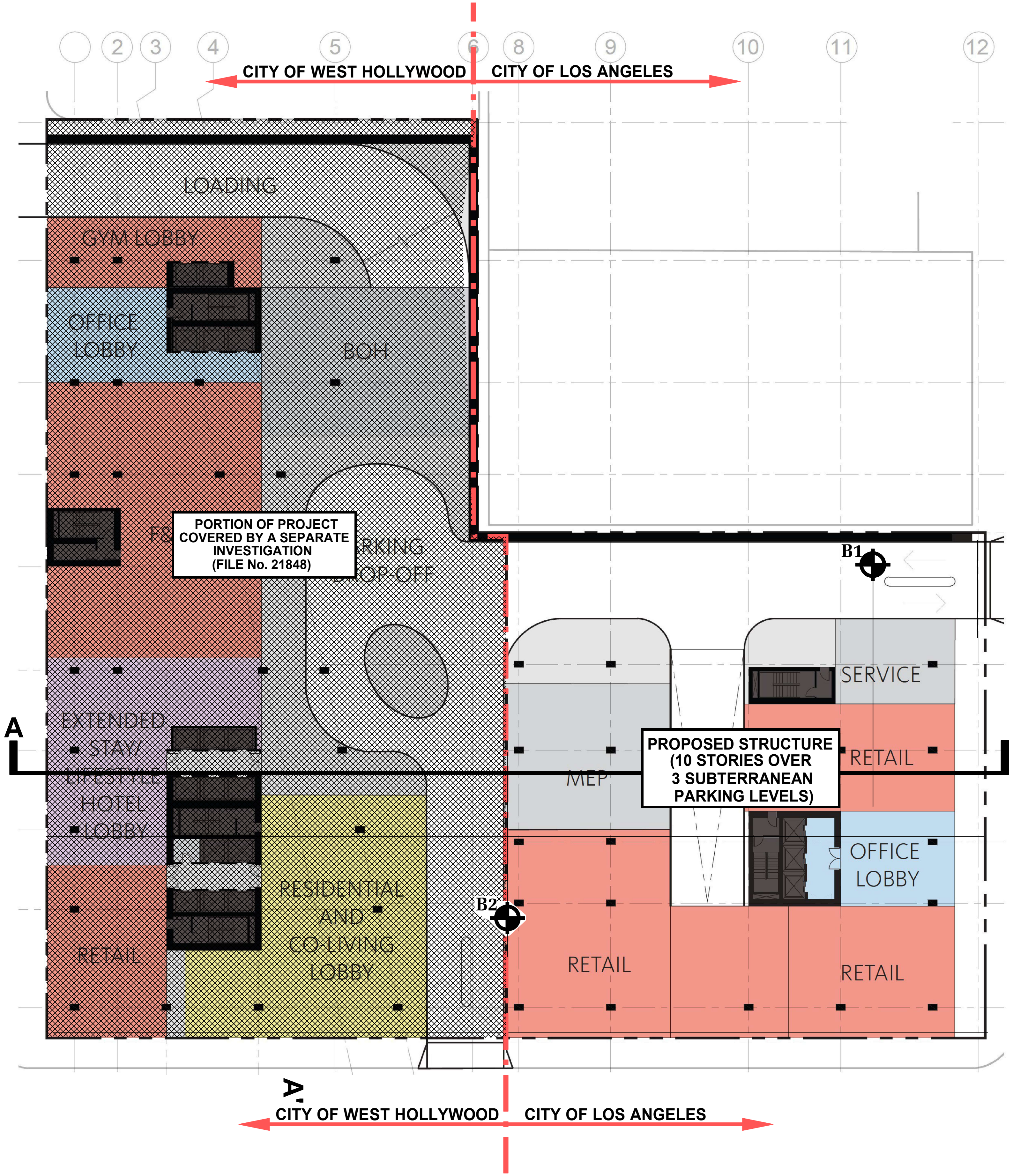
REFERENCE: U.S.G.S. TOPOGRAPHIC MAPS, 7.5 MINUTE SERIES,
HOLLYWOOD, CA QUADRANGLE

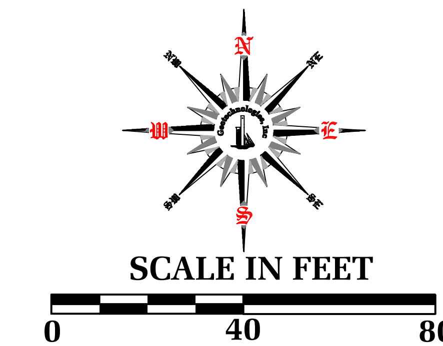
VICINITY MAP

Geotechnologies, Inc.
Consulting Geotechnical Engineers

FARING
1011 N. SYCAMORE AVE., LOS ANGELES

FILE NO. 21849





LEGEND

B2 — LOCATION & NUMBER OF PROPOSED BORING

A — A' — CROSS SECTION

REFERENCE: GROUND FLOOR PLANS BY GENSLER DATED 9/19/19

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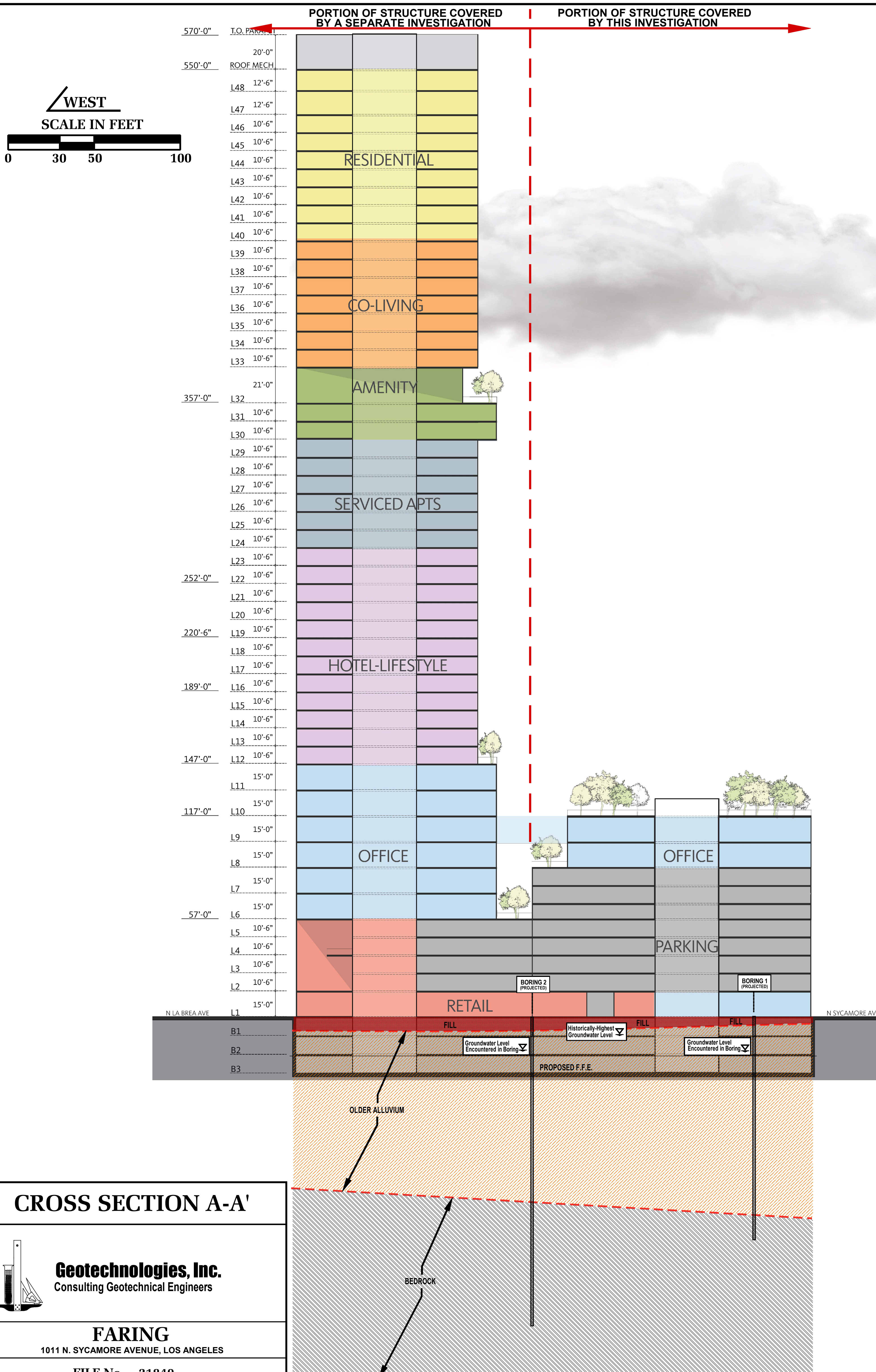
PLOT PLAN

FARING

1011 N. SYCAMORE AVENUE, LOS ANGELES

FILE No. 21849

Date: October '19



CROSS SECTION A-A'

Geotechnologies, Inc.
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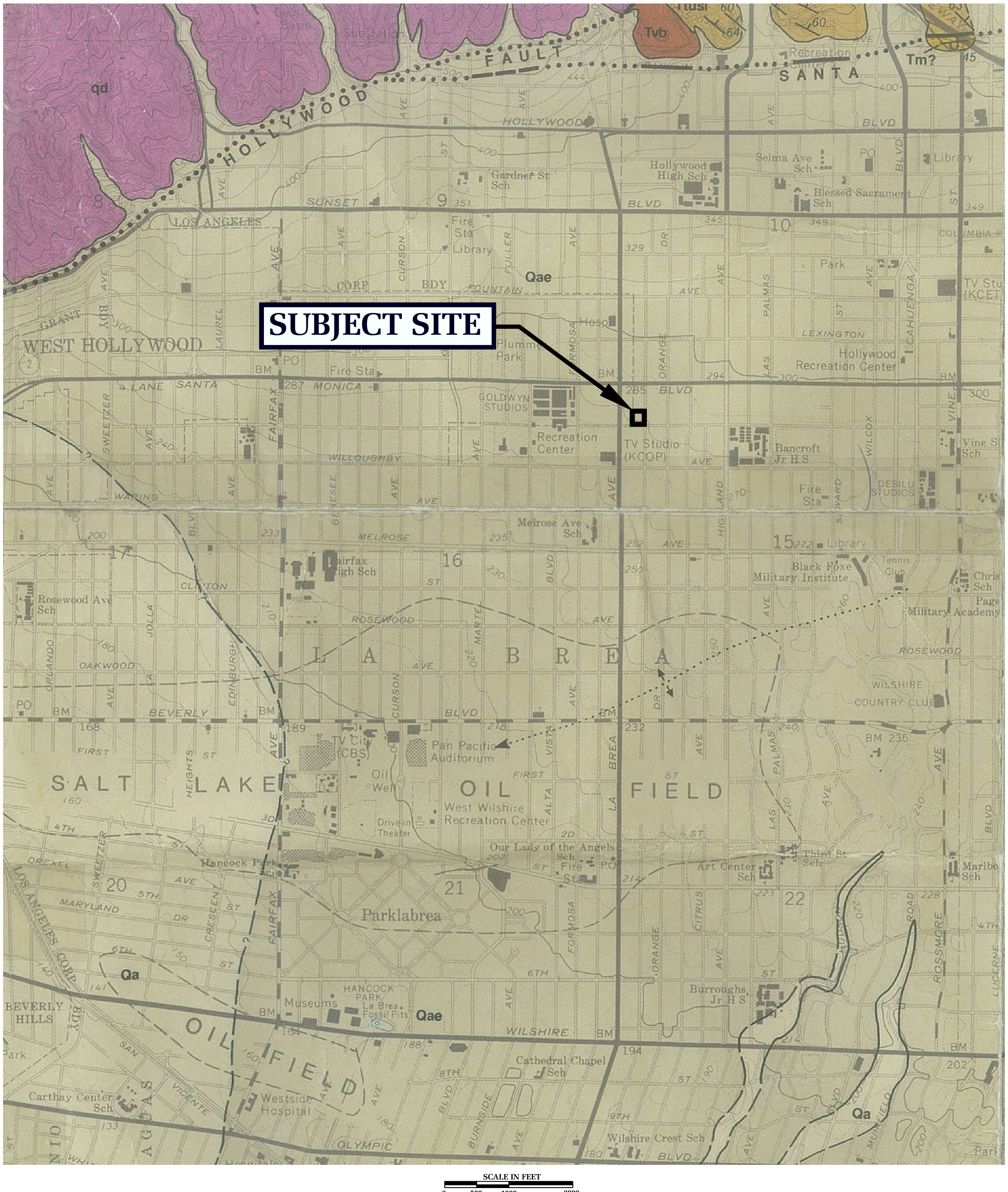
FARING

1011 N. SYCAMORE AVENUE, LOS ANGELES

FILE No. 21849

DATE: October '19

REFERENCE: NORTH-SOUTH SECTION BY GENSLER DATED 9/19/19



LEGEND

Qa: Surficial Sediments: alluvium - gravel, sand and clay
Qae: Older Surficial Sediments: similar to QA, slightly elevated and dissected
Tm: Monterey Formation: platy siliceous shale
Ttusi: Upper Topanga Formation: micaceous clay shale or claystone
Tvb: Middle Topanga Formation and Volcanic Rocks: basaltic volcanic rocks
Qd: Granitic Rocks: quartz diorite, mostly of plagioclase feldspar
Folds - arrow on axial trace of fold indicates direction of plunge
Fault - dashed where indefinite or inferred, dotted where concealed, queried where existence is doubtful

REFERENCE: DIBBLEE, T.W., (1991) GEOLOGIC MAP OF THE HOLLYWOOD-BURBANK (SOUTH 1/2) QUADRANGLES (#DF-30)

LOCAL GEOLOGIC MAP - DIBBLEE

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FARING
1011 N. SYCAMORE AVE., LOS ANGELES

FILE NO. 21849