Appendix IS-2

Geotechnical Investigation

Appendix IS-2.1

Geotechnical Investigation

GEOTECHNICAL INVESTIGATION

PROPOSED MIXED USE DEVELOPMENT
1000-1006 SEWARD STREET,
6565 WEST ROMAINE STREET, AND
1003, 1007, &1013 NORTH HUDSON AVE
LOS ANGELES, CALIFORNIA

TRACT: WHITE AND NEWBY'S HOLLYWOOD BOULEVARD
LOTS: 12-16
APNS: 5533-01-2011, 553-01-2012, 553-01-2013, & 5533-01-2025

GEOCON
WEST, INC.

GEOTECHNICAL
ENVIRONMENTAL
MATERIALS

PREPARED FOR

39 SOUTH LLC
LOS ANGELES, CALIFORNIA

PROJECT NO. W1084-06-01

REVISED APRIL 29, 2020

Project No. W1084-06-01
Revised April 29, 2020

39 South LLC
c/o Plus Development
8920 W. Sunset Boulevard, #200A
West Hollywood, California

Subject: GEOTECHNICAL INVESTIGATION
PROPOSED MIXED-USE DEVELOPMENT
1000-1006 SEWARD STREET, 6565 WEST ROMAINE STREET,
AND 1003, 1007, & 1013 NORTH HUDSON AVENUE
LOS ANGELES, CALIFORNIA
TRACT: WHITE AND NEWBY'S HOLLYWOOD BOULEVARD
LOTS: 12-16
APNS: 5533-012-011, 553-012-012, 553-012-013 & 5533-012-025

Ladies and Gentlemen:

In accordance with our professional services agreement dated October 17, 2019, we have prepared this geotechnical investigation report for the proposed mixed-use development located at the subject addresses in the City of Los Angeles, California. The accompanying report presents the findings of our study and our conclusions and recommendations pertaining to the geotechnical aspects of the 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,

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GEOTECHNICAL INVESTIGATION

1. PURPOSE AND SCOPE

This report presents the results of a geotechnical investigation for the proposed development located at 1000-1006 Seward Street, 6565 West Romaine Street, and 1003, 1007 & 1013 North Hudson Avenue in the City of Los Angeles, California (see Vicinity Map, Figure 1). The purpose of the investigation was to evaluate subsurface soil and geologic conditions underlying the site and, based on conditions encountered, to provide conclusions and recommendations pertaining to the geotechnical aspects of design and construction.

The scope of this investigation included a site reconnaissance, field exploration, laboratory testing, engineering analysis, and the preparation of this report. The site was explored on November 11, 2019, by excavating two 8-inch-diameter borings utilizing a truck-mounted hollow-stem auger drilling machine. The borings were excavated to depths 60½ feet below the existing ground surface. The approximate locations of the exploratory borings are depicted on the Site Plan (see Figure 2). A detailed discussion of the field investigation, including boring logs, is presented in Appendix A.

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

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

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

2. SITE AND PROJECT DESCRIPTION

The subject site is located at 1000-1006 Seward Street, 6565 West Romaine Street, and 1003, 1007 & 1013 North Hudson Avenue in the City of Los Angeles, California. The property is currently occupied by three single story commercial structures and an adjacent on grade asphalt parking lot. The site is bounded by West Romaine Street to the South, by North Hudson Avenue to the East, Seward Street to the West, and two-story residential structures to the North. The site is relatively level, with no pronounced highs or lows. Surface water drainage at the site appears to be by sheet flow along the existing ground contours to West Romaine Street and Hudson Avenue. Vegetation onsite consists of trees and shrubs limited to planter areas.

Based on our review of the provided plans, we understand that the proposed development will consist of a maximum ten stories of above-ground construction underlain by four levels of subterranean parking extending to depths of approximately 40 feet below the ground surface. Due to the preliminary nature of the project, formal plans depicting the proposed development are not available for inclusion in this report. The existing conditions are depicted on the Site Plan (see Figure 2).

Based on the information provided by the project structural engineer both a concrete and steel design scheme are being considered based on the following information.

The range of column loads and overall building loads for the **Concrete Scheme** assuming 4 levels of basement are as follows:

- Typical Single Column Dead Load (allowable stress unfactored load): 1600 to 1900 kips
- Typical Single Column Live Load (allowable stress unfactored load): 350 to 500 kips
- Approximate entire building dead weight (primary structure plus partitions, floor fill, facades, ceilings, and MEP is 70,700 kips based on 350,000 GSF.

The range of column loads and overall building loads for the **Steel Scheme** assuming 4 levels of basement are as follows:

- Typical Single Column Dead Load (allowable stress unfactored load) : 900 to 1200 kips
- Typical Single Column Live Load (allowable stress unfactored load) : 350 to 500 kips
- Approximate entire building dead weight (primary structure plus partitions, floor fill, facades, ceilings, and MEP is 45,000 kips based on 350,000 GSF.

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

3. GEOLOGIC SETTING

The site is located in the central portion of the Los Angeles Basin, a coastal plain bounded by the Santa Monica Mountains on the north, the Elysian Hills and Repetto Hills on the northeast, the Puente Hills and Whittier Fault on the east, the Palos Verdes Peninsula and Pacific Ocean on the west and south, and the Santa Ana Mountains and San Joaquin Hills on the southeast. The basin is underlain by a deep structural depression which has been filled by both marine and continental sedimentary deposits underlain by a basement complex of igneous and metamorphic composition. Regionally, the site is located within the northern portion of the Peninsular Ranges geomorphic province. This province is characterized by northwest-trending physiographic and geologic features such as the nearby Newport-Inglewood Fault Zone.

4. SOIL AND GEOLOGIC CONDITIONS

Based on our field investigation and published geologic maps of the area, the site is underlain by artificial fill and Quaternary age alluvial deposits consisting primarily of sand, silt, and clay (CGS, 2012). Detailed stratigraphic profiles of the materials encountered at the site are provided on the boring logs in Appendix A.

4.1 Artificial Fill

Artificial fill was encountered in our field explorations to a maximum depth of 4 feet below existing ground surface. The artificial fill generally consists of dark brown clay which can be characterized as moist and firm. The fill is likely the result of past grading or construction activities at the site. Deeper fill may exist between excavations and in other portions of the site that were not directly explored.

4.2 Alluvium

Quaternary age alluvium was encountered beneath the fill. The alluvium generally consists of dark brown to brown and reddish-brown interbedded clay, silt, and sand of varying composition. The alluvial soils are characterized as slightly moist to very moist, firm to hard or medium dense to very dense.

5. GROUDWATER

A review of the Seismic Hazard Zone Report for the Hollywood Quadrangle (California Division of Mines and Geology [CDMG], 1998) shows the property having a historical high groundwater level of approximately 18 feet below the existing ground surface. Groundwater information presented in this document is generated from data collected in the early 1900's to the late 1990s. Based on current groundwater basin management practices, it is unlikely that groundwater levels will ever exceed the historic high levels.

Groundwater was encountered in the borings at depths of approximately 18 and 27 feet below existing ground surface in borings B1 and B2 respectively. Considering the historic high groundwater level (CDMG, 1998), the depth to groundwater encountered in the borings, and the depth of the proposed construction, it is likely that groundwater will be encountered during construction. It is not uncommon for groundwater levels to vary seasonally or for groundwater seepage conditions to develop where none previously existed, especially in impermeable fine-grained soils which are heavily irrigated or after seasonal rainfall. In addition, recent requirements for stormwater infiltration could result in shallower seepage conditions in the immediate site vicinity. Proper surface drainage of irrigation and precipitation will be critical for future performance of the project. Recommendations for drainage are provided in the S*urface Drainage* section of this report (see Section 7.25).

6. GEOLOGIC HAZARDS

6.1 Surface Fault Rupture

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

The site is not located within a state-designated Alquist-Priolo Earthquake Fault Zone (CGS, 2019b; CGS, 2014) 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 active fault to the site is the Hollywood Fault located approximately 1.0 mile to the north. Other nearby active faults are the Newport-Inglewood Fault Zone, the Santa Monica Fault, the Raymond Fault, the Verdugo Fault, and the Northridge Hills Fault located approximately 3.8 miles southwest, 3.8 miles west, 5.8 miles northeast, 7.0 miles northeast, and 13.0 miles northwest of the site, respectively (Ziony and Jones, 1989). The active San Andreas Fault Zone is located approximately 35 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 Los Angeles area are not exposed at the surface and do not present a potential surface fault rupture hazard at the site; however, these deep thrust faults are considered active features capable of generating future earthquakes that could result in moderate to significant ground shaking at the site.

6.2 Seismicity

As with all of Southern California, the site has experienced historic earthquakes from various regional faults. The seismicity of the region surrounding the site was formulated based on research of an electronic database of earthquake data. The epicenters of recorded earthquakes with magnitudes equal to or greater than 5.0 in the site vicinity are depicted on Figure 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	62	E
Long Beach	March 10, 1933	6.4	39	SE
Tehachapi	July 21, 1952	7.5	74	NW
San Fernando	February 9, 1971	6.6	23	NNW
Whittier Narrows	October 1, 1987	5.9	15	E
Sierra Madre	June 28, 1991	5.8	22	ENE
Landers	June 28, 1992	7.3	109	E
Big Bear	June 28, 1992	6.4	86	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. However, this hazard is common in Southern California and the effects of ground shaking can be mitigated if the proposed structures are designed and constructed in conformance with current building codes and engineering practices.

6.3 Site-Specific Ground Motion Hazard Analysis

A site-specific ground motion hazard analyses was performed in accordance with ASCE 7-16 Chapter 21 and Section 1613A of the 2019 CBC using the online applications developed by USGS.

6.3.1 Site-Specific 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 USGS Unified Hazard Tool (UHT). The Dynamic U.S. 2014 (v4.2.0) edition was used within the analysis, which is based on the UCERF-3 fault model. The soil underlying the site was modeled as a Site Class “D” with a corresponding average shear wave velocity (V_S30) of 259 meters per second. The site class definition is based on Standard Penetration Test blow count data.

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 (Z2.5) 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 (Z1.0) be defined. The values of Z2.5 and Z1.0 are internally calculated by the Uniform Hazard Tool.

The MCE uniform hazard response spectra was 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.

6.3.2 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.

The characteristics of the deterministic scenario events were defined using the closest distance (Rrup) from the Uniform Hazard Tool deaggregation results and supplemented by the fault source parameters specified in the BSSC2014 Scenario Catalog. The values of Z2.5 and Z1.0 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 (v1.4.0).

The controlling deterministic scenario event was evaluated as a magnitude 6.7 event occurring on the Hollywood fault at a closest distance of 3.07 km.

The deterministic median and standard deviation (sigma) for the scenario event was 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 84th 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 resulting 84th percentile maximum rotated component deterministic response spectra 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).

6.3.3 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 Fa and Fv 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 as Table 1.

6.3.4 Mapped Acceleration Parameters

The following table summarizes the mapped acceleration parameters obtained from the 2019 California Building Code (CBC; Based on the 2018 International Building Code [IBC] and ASCE 7-16), Chapter 16A Structural Design, Section 1613A Earthquake Loads. The data was calculated using the computer program U.S. Seismic Design Maps, provided by the USGS. The short spectral response uses a period of 0.2 second.

MAPPED SPECTRAL ACCELERATIONS

Parameter	Value	2019 CBC Reference
MCE_R Ground Motion Spectral Response Acceleration – Class B (short), S_S	2.088g	Figure 1613A.3.1(1)
MCE_R Ground Motion Spectral Response Acceleration – Class B (1 sec), S_1	0.748g	Figure 1613A.3.1(2)

6.3.5 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}	2.040g
Site Class Modified MCE_R Spectral Response Acceleration – (1 sec), S_{M1}	1.561g
5% Damped Design Spectral Response Acceleration (short), S_{DS}	1.360g
5% Damped Design Spectral Response Acceleration (1 sec), S_{D1}	1.041g

6.3.6 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 84^{th} 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	0.787g	Section 21.5

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

6.4 Liquefaction Potential

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

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

The State of California Seismic Hazard Zone Map for the Hollywood Quadrangle (CDMG, 1999: CGS, 2014) indicates that the site is not located within an area designated as having a potential for liquefaction. In addition, a review of the County of Los Angeles Seismic Safety Element (Leighton, 1990) indicates that the site is not located within an area identified as having a potential for liquefaction. Based on these considerations, it is our opinion that the potential for liquefaction and associated ground deformations beneath the site is very low.

6.5 Slope Stability

The topography at the site is relatively level and the topography in the immediate site vicinity slopes gently to the south-southwest. The site is not located within a in a City of Los Angeles Hillside Grading Area or a Hillside Ordinance Area (City of Los Angeles, 2019). Also, the County of Los Angeles Safety Element (Leighton, 1990) indicates the site is not located within an area identified as a “Hillside Area” or an area having a potential for slope instability. The site is not located within an area identified as having a potential for seismic slope instability (CDMG, 1999; CGS, 2014). There are no known landslides near the site, nor is the site in the path of any known or potential landslides. Therefore, the potential for slope stability hazards to adversely affect the proposed development is considered low.

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

6.7 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 resulting from a seismically induced seiche is considered unlikely.

The site is within an area of minimal flooding (Zone X) as defined by the Federal Emergency Management Agency (LACDPW, 2019b; FEMA, 2019).

6.8 Oil Fields & Methane Potential

Based on a review of the California Division of Oil, Gas and Geothermal Resources (DOGGR) Well Finder website, the project property is not located within the limits of an oilfield (DOGGR, 2019). Additionally, there are no documented oil or gas wells within the immediate vicinity of the property (DOGGR, 2019). However, due to the voluntary nature of record reporting by the oil well drilling companies, wells may be improperly located or not shown on the location map and undocumented wells could be encountered during construction. Any wells encountered during construction will need to be properly abandoned in accordance with the current requirements of the DOGGR.

The site is located within the boundaries of a city-designated Methane Buffer Zone (City of Los Angeles, 2019). It is our understanding that a methane study has been performed by and has concluded that mitigation is not necessary (GeoScience Analytical, Inc.). Geocon is not a methane consultant and we have not reviewed the referenced report for technical accuracy.

6.9 Subsidence

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

7. CONCLUSIONS AND RECOMMENDATIONS

7.1 General

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

7.1.2 Up to 4 feet of existing artificial fill was encountered during the site exploration. 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. Future demolition of the existing structures which occupy the site will likely disturb the upper few feet of site soils. It is our opinion that the existing artificial fill, in its present condition, is not considered suitable for direct support of proposed new foundations or slabs; however, the existing site soils are suitable for re-use as engineered fill provided the recommendations in the *Grading* section of this report are followed (see Section 7.5).

7.1.3 The upper alluvial soils below a depth of 20 feet consist primarily of medium dense to firm silty sand, clayey sand, clayey silt, and sandy clay, which become very stiff and dense below a depth of approximately 40 feet. Based on laboratory testing (see Figures B9 to B17), the alluvium between 20 and 40 feet is moderately to highly compressible. Excavation for the subterranean portion of the structure is anticipated to penetrate through the existing artificial fill and compressible alluvial soils.

7.1.4 Based on these considerations, it is recommended that the proposed structure be supported on a reinforced concrete mat foundation system deriving support in the undisturbed alluvial soils found at or below a depth of 40 feet. In order to minimize differential settlement, it is recommended that the ramp and ramp walls for the subterranean parking garage be structurally supported on the mat foundation. A mat foundation is more accommodating to subgrade stabilization, waterproofing, and hydrostatic design. Any soils unintentionally disturbed should be properly compacted prior to placing construction materials. All foundation excavations must be observed and approved in writing by the Geotechnical Engineer prior to placement of steel or concrete. Recommendations for the design of a mat foundation system are provided in Section 7.7 of this report.

7.1.5 Groundwater was encountered at a depth of 18 and 27 feet below the existing ground surface. Excavation for the proposed subterranean parking levels is anticipated to extend to depths up to approximately 45 feet below the ground surface, including foundation construction and dewatering elements. Due to the depth of the proposed excavation and the potential for seasonal fluctuation in the groundwater level, temporary dewatering measures will likely be required to mitigate groundwater during excavation and construction. Recommendations for temporary dewatering are provided in Section 7.4 of this report. Furthermore, groundwater will likely be encountered during deep drilled excavations, such as shoring piles and/or elevator pistons.

7.1.6 The historically high groundwater level beneath the site is reported to be approximately 18 feet below the existing ground surface, and the proposed structure should be designed for hydrostatic pressure based on this groundwater level. The hydrostatic design will result in uplift forces on the structure that must be resisted by counterweight or structural design measures. The recommended floor slab uplift pressure to be used in design would be 62.4(H) in units of pounds per square foot (psf), where "H" is the height of the water above the bottom of the foundation in feet.

7.1.7 Excavations up to 45 feet in vertical height are anticipated for construction of the subterranean parking levels, including foundation depths and dewatering systems. Due to the depth of the excavation and the proximity to the property lines, city streets and adjacent offsite structures, excavation of the subterranean parking level will likely require sloping and/or shoring measures in order to provide a stable excavation. Where shoring is required it is recommended that a soldier pile shoring system be utilized. In addition, where the proposed excavation will be deeper than and adjacent to a structure, the proposed shoring should be designed to resist the surcharge imposed by the adjacent structure. Recommendations for *Temporary Excavations* are provided in Section 7.18 of this report.

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

7.1.9 Where new surface paving is to be placed, it is recommended that all existing uncertified fill and soft alluvial soils be excavated and properly compacted for paving support. The client should be aware that excavation and compaction of all existing uncertified fill and soft alluvial soils in the area of new paving is not required; however, paving constructed over existing uncertified fill or unsuitable alluvial 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 12 inches of subgrade soil should be scarified and properly compacted for paving support. Paving recommendations are provided in *Preliminary Pavement Recommendations* section of this report (see Section 7.12).

7.1.10 Based on the depth of the proposed structure, the relatively shallow groundwater level, and the predominantly fine-grained nature of the soil layers, a stormwater infiltration system is not recommended for this project. It is recommended that stormwater be retained, filtered, and discharged in accordance with the requirements of the local governing agency.

7.1.11 Once the design and foundation loading configuration proceeds to a more finalized plan, the recommendations within this report should be reviewed and revised, if necessary. If the proposed building loads will exceed those presented herein, the potential for settlement should be reevaluated by this office.

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

7.2 Soil and Excavation Characteristics

7.2.1 The in-situ soils can be excavated with moderate effort using conventional excavation equipment. Caving should be anticipated in unshored excavations, especially where granular or saturated soils are encountered. The contractor should be aware that excessive caving could occur during drilled excavations in the poorly-graded sand layers below the water.

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

7.2.3 All onsite excavations must be conducted in such a manner that potential surcharges from existing structures, construction equipment, and existing foundation supports 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. Excavation recommendations are provided in the *Temporary Excavations* section of this report (see Section 7.18).

7.2.4 Due to the presence of groundwater and the depth of the excavation, expansive soils are not expected to be pose a significant hazard to the foundations at the proposed depth.

7.3 Minimum Resistivity, pH, and Water-Soluble Sulfate

7.3.1 Potential of Hydrogen (pH) and resistivity testing as well as chloride content testing were performed on representative samples of soil to generally evaluate the corrosion potential to surface utilities. The tests were performed in accordance with California Test Method Nos. 643 and 422 and indicate that the soils are considered “moderately corrosive” with respect to corrosion of buried ferrous metals on site. The results are presented in Appendix B (Figure B19) and should be considered for design of underground structures. Due to the corrosive potential of the soils, it is recommended that PVC, ABS or equivalent plastic piping be considered in lieu of cast-iron for sewer pipes, subdrains and retaining wall drains in direct contact with the site soils.

7.3.2 Laboratory tests were performed on representative samples of the site materials to measure the percentage of water-soluble sulfate content. Results from the laboratory water-soluble sulfate tests are presented in Appendix B (Figure B19) and indicate that the on-site materials possess a sulfate exposure class of “S0” to concrete structures as defined by 2019 CBC Section 1904 and ACI 318-14 Table 19.3.1.1.

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

7.4 Temporary Dewatering

7.4.1 Groundwater was encountered during site exploration at depths ranging between 18 and 27 feet below existing ground surface. Based on the conditions encountered at the time of exploration, groundwater is anticipated to be encountered during construction activities. The depth to groundwater at the time of construction can be further verified during the installation of the initial dewatering well or shoring piles. If groundwater is present above the depth of the proposed foundation excavation, temporary dewatering will be necessary to maintain a safe working environment during excavation and construction activities.

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

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

7.5 Grading

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

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

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

7.5.4 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.) and the City of Los Angeles Inspector.

7.5.5 The proposed structure may be supported on a reinforced concrete mat foundation system deriving support in undisturbed alluvial soils found at and below a depth of 40 feet. The soils exposed excavation bottom, if disturbed, should be removed or compacted to a dense state prior to placing construction materials. Foundations should be deepened as necessary to extend into satisfactory soils and must be observed and approved in writing by the Geotechnical Engineer (a representative of Geocon West, Inc.).

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

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

7.5.8 The mat foundation at the subterranean level may bear directly on the competent undisturbed alluvial deposits at the excavation bottom. It is recommended that the exposed soils be proof rolled prior to placing construction materials. Any disturbed soils should be removed or properly compacted for foundation/slab support, as necessary.

7.5.9 The City of Los Angeles Department of Building and Safety requires a minimum compactive effort of 95 percent of the laboratory maximum dry density in accordance with ASTM D 1557 (latest edition) where the soils placed as fill have less than 15 percent finer than 0.005 millimeters. Soils with more than 15 percent finer than 0.005 millimeters may be compacted to 90 percent of the laboratory maximum dry density in accordance with ASTM D 1557 (latest edition). All fill and backfill soils should be placed in horizontal loose layers approximately 6 to 8 inches thick, moisture conditioned to at least 2 percent above optimum moisture content, and properly compacted to the required degree of compaction in accordance with ASTM D 1557 (latest edition).

7.5.10 Where new paving is to be placed, it is recommended that all existing fill and disturbed alluvium be excavated and properly compacted for paving support. The client should be aware that excavation and compaction of all existing fill and soft soils in the area of new paving is not required; however, paving constructed over existing fill or unsuitable alluvium may experience increased settlement and/or cracking, and may, therefore, have a shorter design life and increased maintenance costs. As a minimum, the upper 12 inches of soil should be scarified, moisture conditioned to near optimum moisture content, and compacted to at least 95 percent relative compaction for paving support. Paving recommendations are provided in *Preliminary Pavement Recommendations* section of this report (see Section 7.12).

7.5.11 It is suggested that flexible utility connections be considered for all rigid utilities tied into the supported structure in order to minimize or prevent damage to utilities from minor differential soil movements, or potentially larger movements caused by an earthquake event. Utility trenches should be properly backfilled in accordance with the requirements of the Green Book (latest edition). The pipe should be bedded with clean sands (Sand Equivalent greater than 30) to a depth of at least 1 foot over the pipe, and the bedding material must be observed and approved in writing by the Geotechnical Engineer (a representative of Geocon). The use of gravel is not acceptable unless used in conjunction with filter fabric to prevent the gravel from having direct contact with soil. The remainder of the trench backfill may be derived from onsite soil or approved import soil, compacted as necessary, until the required compaction is obtained. The use of minimum 2-sack slurry is also acceptable as backfill (see Section 7.6). Prior to placing any bedding materials or pipes, the excavation bottom must be observed and approved in writing by the Geotechnical Engineer (a representative of Geocon).

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

7.6 Controlled Low Strength Material (CLSM)

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

Standard Requirements

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

Requirements for CLSM that will be used for support of footings

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

7.7 Mat Foundation Design

7.7.1 It is recommended that a reinforced concrete mat foundation be utilized for support of the proposed structure. The reinforced concrete mat foundation may derive support in the undisturbed alluvial soils found below a depth of 40 feet below ground surface. Foundations excavations should be deepened as necessary to extend into satisfactory soils and must be observed and approved in writing by the Geotechnical Engineer (a representative of Geocon West, Inc.).

7.7.2 The recommended maximum allowable bearing value is 6,500 pounds per square foot (psf). The allowable bearing pressure may be increased by up to one-third for transient loads due to wind or seismic forces.

7.7.3 It is recommended that a modulus of subgrade reaction of 230 pounds per cubic inch (pci) be utilized for the design of the mat foundation bearing in the undisturbed alluvial soils found at and below a depth of 40 feet. These values are unit values 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)

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

7.7.5 The proposed structure must be designed for hydrostatic pressure based on the groundwater level. The hydrostatic design will result in uplift forces on the structure that must be resisted by counterweight or structural design measures. The recommended floor slab uplift pressure to be used in design would be 62.4(H) in units of pounds per square foot (psf), where "H" is the height of the water above the bottom of the foundation in feet. For design purposes the groundwater table should be assumed to be at a depth of 18 feet below the ground surface. Considerations for uplift resistance are provided in Section 7.9 of this report.

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

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

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

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

7.8 Foundation Settlement

7.8.1 The maximum settlement for a reinforced concrete mat foundation with a maximum allowable bearing pressure of 6,500 psf deriving support in the recommended bearing materials is expected to be less than 2 inches and occur below the heaviest loaded structural element. Settlement of the foundation system is expected to occur on initial application of loading. Differential settlement is expected to be less than 1 inch between the center and corner of the mat foundation. These static settlements should be further verified once the design phase proceeds to a more finalized plan.

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

7.9 Uplift Resistance

7.9.1 Foundation uplift may be resisted by the weight of the structure, as well as friction along the sides of foundations. It is our understanding that the building will be designed to be heavy enough so that the dead load will exceed the potential buoyancy.

7.9.2 If additional uplift resistance is required, the perimeter shoring piles may be utilized provided the toes of the piles are poured with structural concrete and are designed as permanent piles. Recommendations for the design of shoring piles are provided in Section 7.20.

7.9.3 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 40 psf (⅔ the downward capacity, adjusted for buoyancy).

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

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

7.10 Lateral Design

7.10.1 Resistance to lateral loading may be provided by friction acting at the base of foundations, slabs and by passive earth pressure. An allowable coefficient of friction of 0.35 may be used with the dead load forces in the competent alluvial soils or stabilized subgrade, and 0.15 for slabs underlain by a moisture barrier.

7.10.2 Passive earth pressure for the sides of foundations and slabs poured against the alluvial soils above the groundwater table may be computed as an equivalent fluid having a density of 270 pcf with a maximum earth pressure of 2,700 pcf. 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 pcf with a maximum earth pressure of 1,300 pcf (these values have been adjusted for buoyant forces). When combining passive and friction for lateral resistance, the passive component should be reduced by one-third.

7.11 Exterior Concrete Slabs-on-Grade

7.11.1 Slabs-on-grade at the ground surface that may receive moisture-sensitive floor coverings or may be used to store moisture-sensitive materials should be underlain by a vapor retarder placed directly beneath the slab. The vapor retarder and acceptable permeance should be specified by the project architect or developer based on the type of floor covering that will be installed. The vapor retarder selection and design should be consistent with the guidelines presented in Section 9.3 of the American Concrete Institute's (ACI) Guide for Concrete Slabs that Receive Moisture-Sensitive Flooring Materials (ACI 302.2R-06) as well as ASTM E1745 and should be installed in general conformance with ASTM E 1643 (latest edition) and the manufacturer's recommendations. A minimum thickness of 15 mils extruded polyolefin plastic is recommended; vapor retarders which contain recycled content or woven materials are not recommended. The vapor retarder should have a permeance of less than 0.01 perms demonstrated by testing before and after mandatory conditioning is recommended. The vapor retarder should be installed in direct contact with the concrete slab with proper perimeter seal. If the California Green Building Code requirements apply to this project, the vapor retarder should be underlain by 4 inches of clean aggregate. It is important that the vapor retarder be puncture resistant since it will be in direct contact with angular gravel. As an alternative to the clean aggregate suggested in the Green Building Code, it is our opinion that the concrete slab-on-grade may be underlain by a vapor retarder over 4-inches of clean sand (sand equivalent greater than 30), since the sand will serve a capillary break and will minimize the potential for punctures and damage to the vapor barrier.

7.11.2 For seismic design purposes, an allowable coefficient of friction of 0.40 may be utilized between concrete slabs and subgrade soils; and 0.15 for slabs underlain by a vapor retarder.

7.11.3 Exterior slabs, not subject to traffic loads, should be at least 4 inches thick and reinforced with No. 3 steel reinforcing bars placed 18 inches on center in both horizontal directions, positioned near the slab midpoint. Prior to construction of slabs, the upper 12 inches of subgrade should be moisture conditioned to at least 2 percent above optimum moisture content and properly compacted to at least 95 percent relative compaction, as determined by ASTM Test Method D 1557 (latest edition). Crack control joints should be spaced at intervals not greater than 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.

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

7.12 Preliminary Pavement Recommendations

7.12.1 Where new paving is to be placed, it is recommended that all existing uncertified fill and soft or unsuitable alluvial materials be excavated and properly compacted for paving support. The client should be aware that excavation and compaction of all existing uncertified fill and soft alluvium in the area of new paving is not required; however, paving constructed over existing unsuitable 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 12 inches of paving subgrade should be scarified, moisture conditioned to at least 2 percent above optimum moisture content, and properly compacted to at least 95 percent relative compaction, as determined by ASTM Test Method D 1557 (latest edition).

7.12.2 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.

7.12.3 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

7.12.4 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 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).

7.12.5 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 thick and be 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 12 inches properly compacted subgrade soil that is compacted to 95 percent relative compaction, as determined by ASTM Test Method D 1557 (latest edition).

7.12.6 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.

7.13 Retaining Wall Design

7.13.1 The recommendations presented below are generally applicable to the design of rigid concrete or masonry retaining walls having a maximum height of 40 feet. In the event that walls significantly higher than 40 feet are planned, Geocon should be contacted for additional recommendations.

7.13.2 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. Calculation for the recommended retaining wall pressures are presented in Figure 9.

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 40	46	63

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

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

7.13.5 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.

7.13.6 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

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

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

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

7.14 Dynamic (Seismic) Lateral Forces

7.14.1 The structural engineer should determine the seismic design category for the project in accordance with Section 1613 of the CBC. If the project possesses a seismic design category of D, E, or F, proposed retaining walls in excess of 6 feet in height should be designed with seismic lateral pressure (Section 1803.5.12 of the 2016 CBC).

7.14.2 A seismic load of 10 pcf should be used for design of walls that support more than 6 feet of backfill in accordance with Section 1803.5.12 of the 2019 CBC. The seismic load is applied as an equivalent fluid pressure along the height of the wall and the calculated loads result in a maximum load exerted at the base of the wall and zero at the top of the wall. This seismic load should be applied in addition to the active earth pressure, not the at-rest pressure. The earth pressure is based on half of two thirds of PGA_M calculated from ASCE 7-10 Section 11.8.3.

7.15 Retaining Wall Drainage

7.15.1 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 7). The clean bottom and subdrain pipe, behind a retaining wall, should be observed by the Geotechnical Engineer (a representative of Geocon), prior to placement of gravel or compacting backfill.

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

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

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

7.16 Elevator Pit Design

7.16.1 The elevator pit slab and retaining wall should be designed by the project structural engineer. Elevator pit walls may be designed in accordance with the recommendations in the *Foundation Design* and *Retaining Wall Design* sections of this report (see Sections 7.7 through 7.15).

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

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

7.17 Elevator Piston

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

7.17.2 Casing will be required since caving is expected in the drilled excavation and the contractor should be prepared to use casing and should have it readily available at the commencement of drilling activities. Continuous observation of the drilling and installation of the elevator piston by the Geotechnical Engineer (a representative of Geocon West, Inc.) is required.

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

7.18 Temporary Excavations

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

7.18.2 Vertical excavations, greater than 5 feet or where surcharged by existing structures, will require sloping or shoring measures in order to provide a stable excavation. Where sufficient space is available, temporary unsurcharged embankments could be sloped back at a uniform 1:1 slope gradient or flatter up to maximum height of 12 feet. A uniform slope does not have a vertical portion. Excavations greater than 12 feet in height will require special excavations measures such as shoring. Recommendations for *Shoring* are provided in Section 7.29.

7.18.3 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. Geocon personnel should inspect the soils exposed in the cut slopes during excavation so that modifications of the slopes can be made if variations in the soil conditions occur. All excavations should be stabilized within 30 days of initial excavation.

7.19 Shoring – Soldier Pile Design and Installation

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

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

7.19.3 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 subgrade stabilization activities, foundations and/or adjacent drainage systems.

7.19.4 The proposed soldier piles may be utilized to provide a component of uplift resistance. If required to provide uplift resistance, the shoring piles must be designed as permanent piles. The uplift capacity may be taken as ⅔ of the downward frictional capacity.

7.19.5 The proposed soldier piles may also be designed as permanent piles. The required pile depths, dimensions, 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 7.13).

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

7.19.7 Groundwater was encountered during site exploration at depths ranging between 18 and 27 feet below existing ground surface. Should groundwater or seepage be encountered, 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 ensure that the tip of the tremie tube is never raised above the surface of the concrete.

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

7.19.9 Casing will likely be required since caving is expected to occur, especially where granular soils are encountered. The contractor should have casing available prior to commencement of drilling activities. When casing is used, extreme care should be employed so that the pile is not pulled apart as the casing is withdrawn. At no time should the distance between the surface of the concrete and the bottom of the casing be less than 5 feet. As an alternative, piles may be vibrated into place; however, there is always a risk that excessive vibrations in sandy soils could induce settlements and distress to adjacent offsite improvements. Continuous observation of the drilling and pouring of the piles by the Geotechnical Engineer (a representative of Geocon West, Inc.), is required.

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

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

7.19.12 If a vibratory method of solider pile installation is utilized, predrilling may be performed prior to installation of the steel beams. If predrilling is performed, the bore diameter should be no greater than 75 percent of the largest dimension of the pile to prevent excessive loss in the frictional component of the pile capacity. Predrilling should not be conducted below the proposed excavation bottom, and the auger should be backspun out of the pilot holes, leaving the soil in place.

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

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

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

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

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

7.19.18 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 0.35 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 330 psf per foot (value has been reduced for buoyant forces).

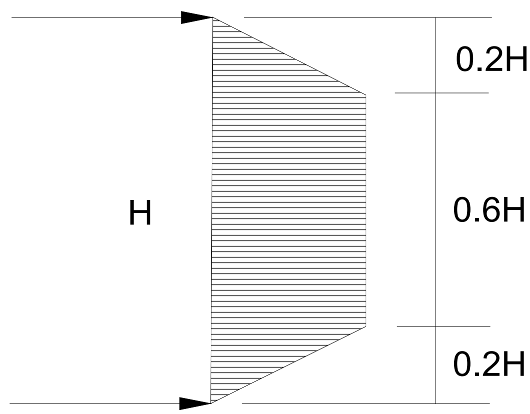
7.19.19 Due to the nature of the site soils, it is expected that continuous lagging between soldier piles will be required. However, it is recommended that the exposed soils be observed by the Geotechnical Engineer (a representative of Geocon West, Inc.), to verify the presence of any cohesive soils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

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

7.19.21 For the design of unbraced shoring, it is recommended that an equivalent fluid pressure be utilized for design. A trapezoidal distribution of lateral earth pressure may be used where shoring will be restrained by bracing or tiebacks. The recommended active and trapezoidal pressure are provided in the following table. A diagram depicting the trapezoidal pressure distribution of lateral earth pressure is provided below the table. Calculation of the recommended shoring wall pressure as provided as Figure 10.

HEIGHT OF SHORING (FEET)	EQUIVALENT FLUID PRESSURE (Pounds Per Cubic Foot) (ACTIVE PRESSURE)	EQUIVALENT FLUID PRESSURE (Pounds Per Square Foot per Foot) Active Trapezoidal (Where H is the height of the shoring in feet)
Up to 45 feet	38	24H

Trapezoidal Distribution of Pressure



7.19.22 Where a combination of sloped embankment and shoring is utilized, the pressure will be greater and must be determined for each combination. Additional active pressure should be added for a surcharge condition due to slopes, vehicular traffic or adjacent structures and should be designed for each condition.

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

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

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

and

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

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

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

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

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

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

and

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

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

then

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

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

7.19.25 In addition to the recommended earth pressure, the upper 10 feet of the shoring adjacent to the street or driveway areas should be designed to resist a uniform lateral pressure of 100 psf, acting as a result of an assumed 300 psf surcharge behind the shoring due to normal street traffic. If the traffic is kept back at least 10 feet from the shoring, the traffic surcharge may be neglected.

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

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

7.19.28 Due to the depth of the depth of the excavation and proximity to adjacent structures, it is suggested that prior to excavation the existing improvements be inspected and their present condition be documented. 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 observed, an investigation should be performed and corrective measures taken so that continued or worsened distress or settlement is mitigated. Documentation and monitoring of the offsite structures and improvements is not the responsibility of the geotechnical engineer.

7.20 Temporary Tieback Anchors

7.20.1 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.

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

- 10 feet below the top of the excavation – 700 pounds per square foot*
- 22 feet below the top of the excavation – 900 pounds per square foot*
- 34 feet below the top of the excavation – 1,100 pounds per square foot*

  *reduced for buoyancy

7.20.3 Depending on the techniques utilized, and the experience of the contractor performing the installation, a maximum allowable friction capacity of 3.0 kips per linear foot for post-grouted anchors (for a 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.

7.21 Anchor Installation

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

7.22 Anchor Testing

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

7.22.2 At least 10 percent of the anchors should be selected for "quick" 200 percent tests and three additional anchors should be selected for 24-hour 200 percent tests. The purpose of the 200 percent tests is to verify the friction value assumed in design. The anchors should be tested to develop twice the assumed friction value. These tests should be performed prior to installation of additional tiebacks. Where satisfactory tests are not achieved on the initial anchors, the anchor diameter and/or length should be increased until satisfactory test results are obtained.

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

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

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

7.23 Internal Bracing

7.23.1 Rakers may be utilized to brace the soldier piles in lieu of tieback anchors. The raker bracing could be supported laterally by temporary concrete footings (deadmen) or by the permanent, interior footings. For design of such temporary footings or deadmen, poured with the bearing surface normal to rakers inclined at 45 degrees, a bearing value of 2,500 psf in competent alluvial deposits may be used, provided the shallowest point of the footing is at least 18 inches 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. In addition, the raker footing plan should be checked by the project structural engineer to verify if there are any conflicts with the proposed structural foundations, and resolve any issues prior to commencement of construction activities.

7.24 Surcharge from Adjacent Structures and Improvements

7.24.1 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.

7.24.2 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.

7.24.3 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.

7.24.4 In addition to the recommended earth pressure, the upper 10 feet of the shoring adjacent to the street or driveway areas should be designed to resist a uniform lateral pressure of 100 psf, acting as a result of an assumed 300 psf surcharge behind the shoring due to normal street traffic. If the traffic is kept back at least 10 feet from the shoring, the traffic surcharge may be neglected.

7.25 Surface Drainage

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

7.25.2 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 2016 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 5 feet of the building perimeter. Planters which are located adjacent to foundations should be sealed to prevent moisture intrusion into the soils providing foundation support. Landscape irrigation is not recommended within 5 feet of the building perimeter footings except when enclosed in protected planters.

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

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

7.26 Plan Review

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

LIMITATIONS AND UNIFORMITY OF CONDITIONS

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

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

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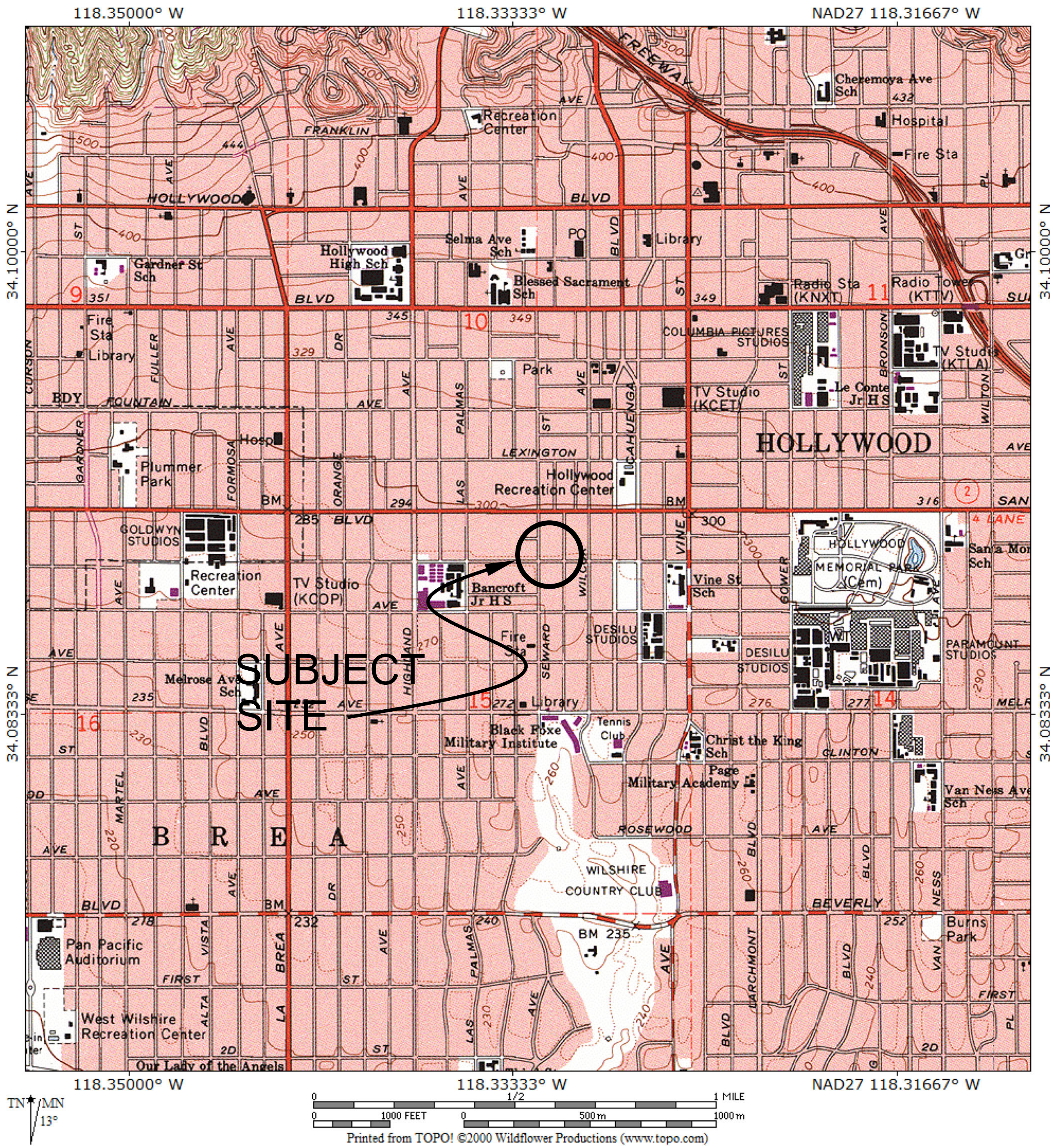
Ziony, J. I., and Jones, L. M., 1989, *Map Showing Late Quaternary Faults and 1978–1984 Seismicity of the Los Angeles Region, California*, U.S. Geological Survey Miscellaneous Field Studies Map MF-1964.



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

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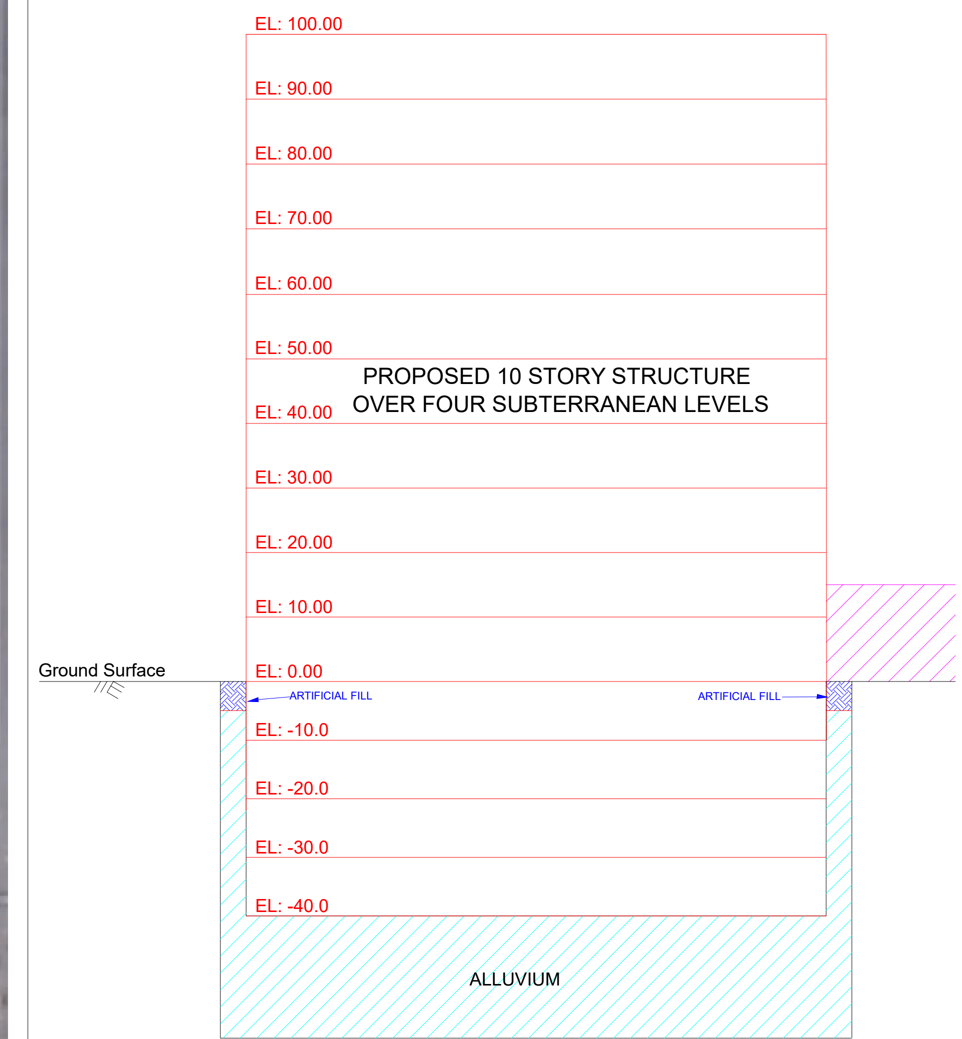
PHONE (818) 841-8388 - FAX (818) 841-1704

DRAFTED BY: RMA | CHECKED BY: SFK

VICINITY MAP

1000-1006 SEWARD STREET
6565 WEST ROMAINE STREET
1003-1013 NORTH HUDSON AVENUE
LOS ANGELES, CALIFORNIA

APRIL 2020 | PROJECT NO. W1084-06-01 | FIG. 1



CROSS SECTION A-A

NOTE: CROSS SECTION IS FOR ILLUSTRATION PURPOSE ONLY. NOT FOR CONSTRUCTION

LEGEND

- B2 Approximate Location of Boring
- Approximate Location of Property Line and Limits of Proposed Development
- Location of Existing Adjacent Structure
- Location of Existing Structure To Be Demolished

0 32' 64'

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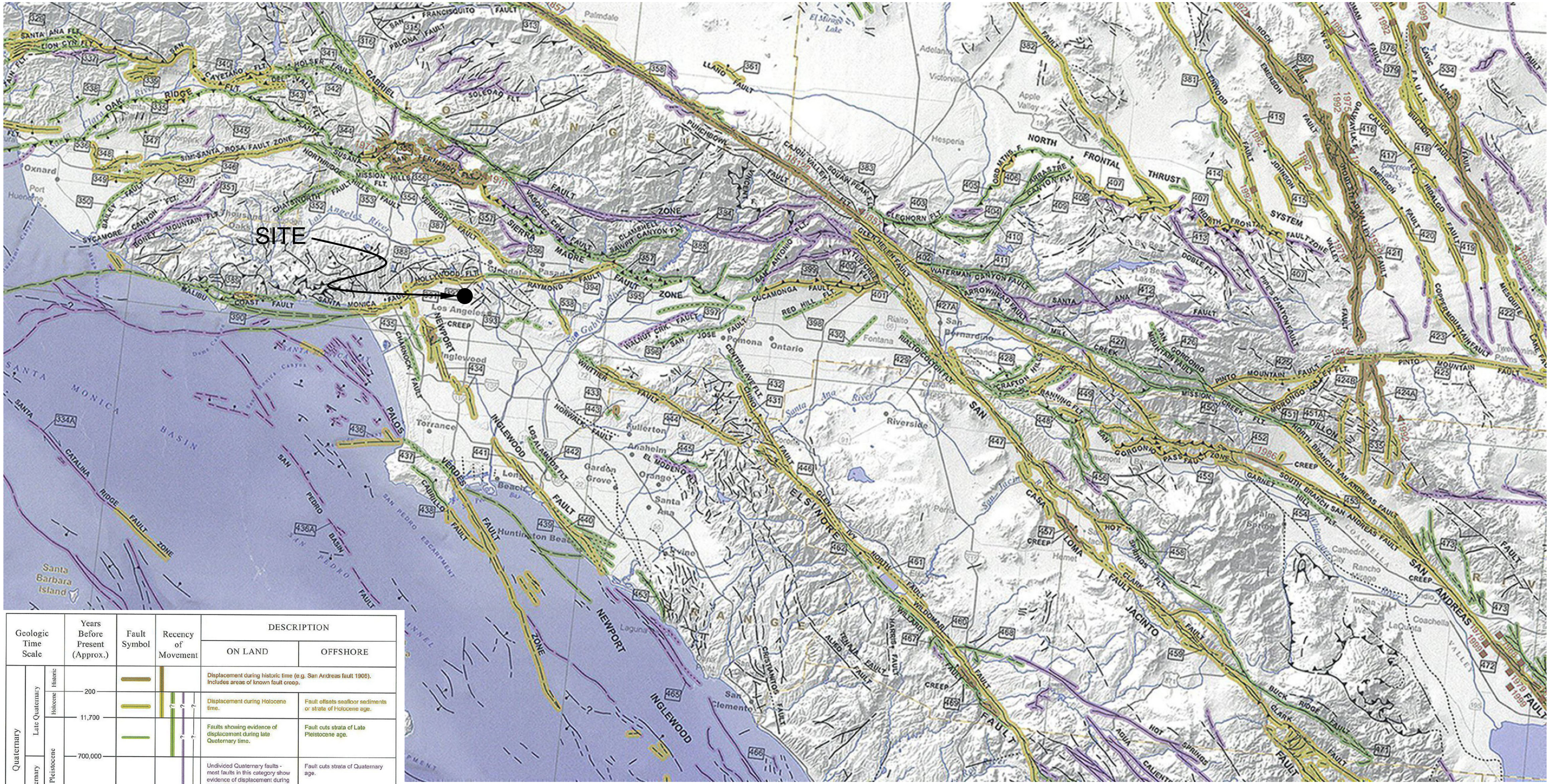
SITE PLAN

1000-1006 SEWARD STREET
6565 WEST ROMAINE STREET
1003-1013 NORTH HUDSON AVENUE
LOS ANGELES, CALIFORNIA

APRIL 2020	PROJECT NO. W1084-06-01	FIG. 2

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



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

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

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REGIONAL FAULT MAP

1000-1006 SEWARD STREET
6565 WEST ROMAINE STREET
1003-1013 NORTH HUDSON AVENUE
LOS ANGELES, CALIFORNIA

APRIL 2020	PROJECT NO. W1084-06-01	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.

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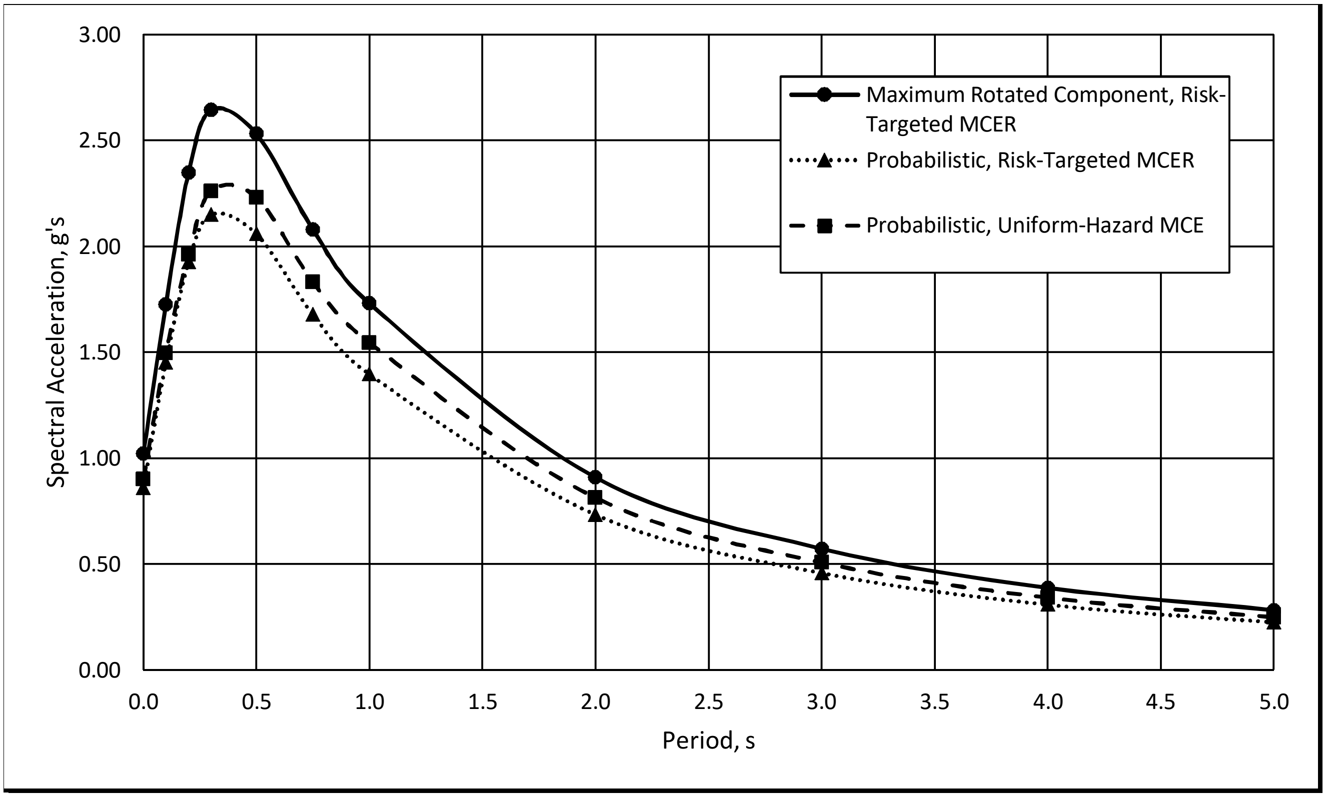
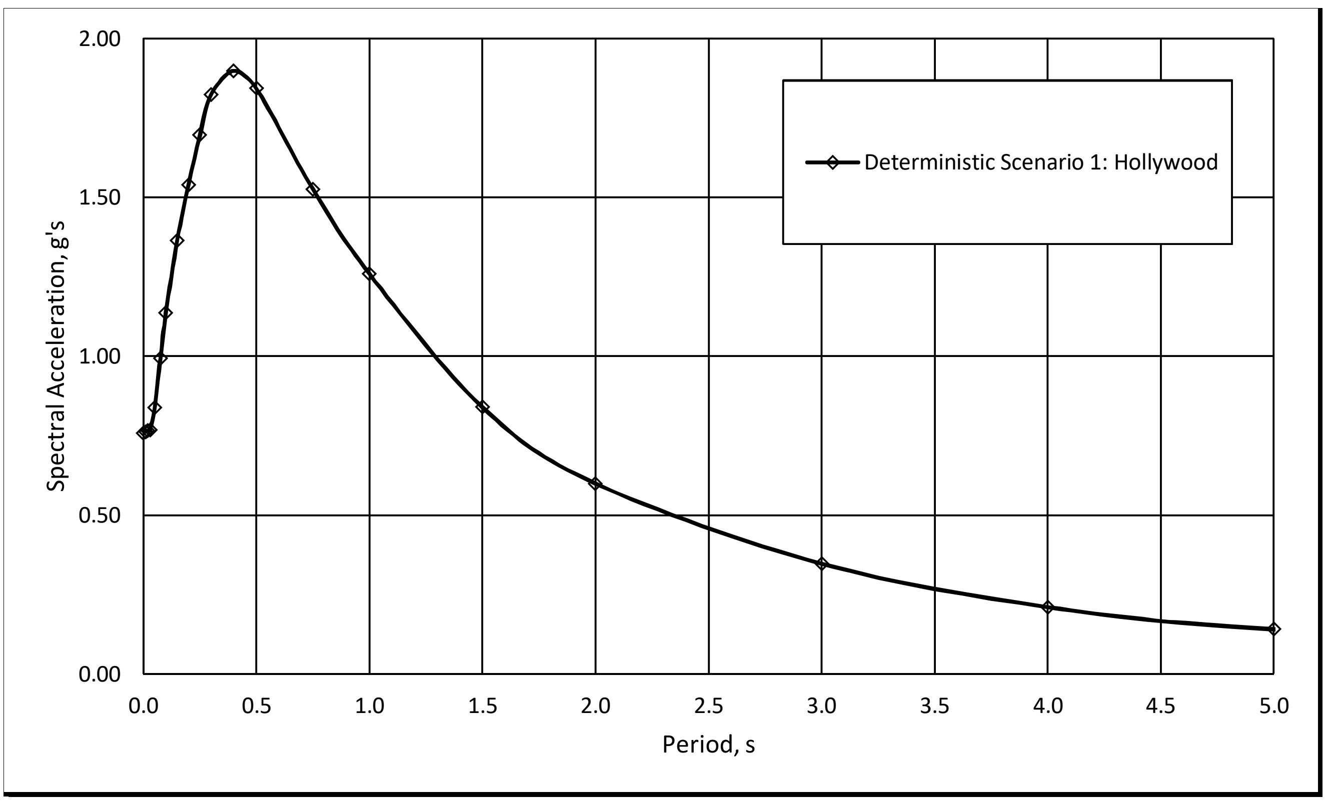
REGIONAL SEISMICITY MAP

1000-1006 SEWARD STREET
6565 WEST ROMAINE STREET
1003-1013 NORTH HUDSON AVENUE
LOS ANGELES, CALIFORNIA

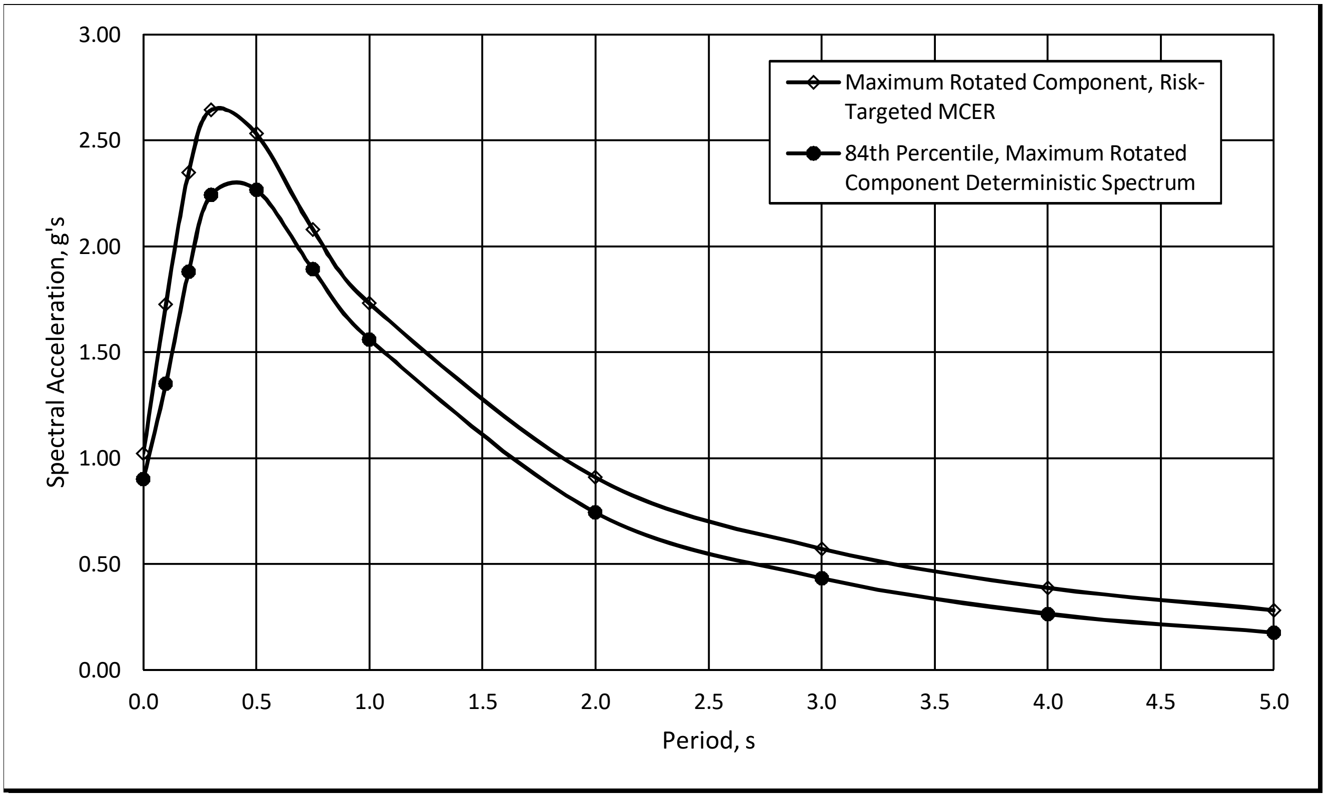
DRAFTED BY: RMA	CHECKED BY: SFK

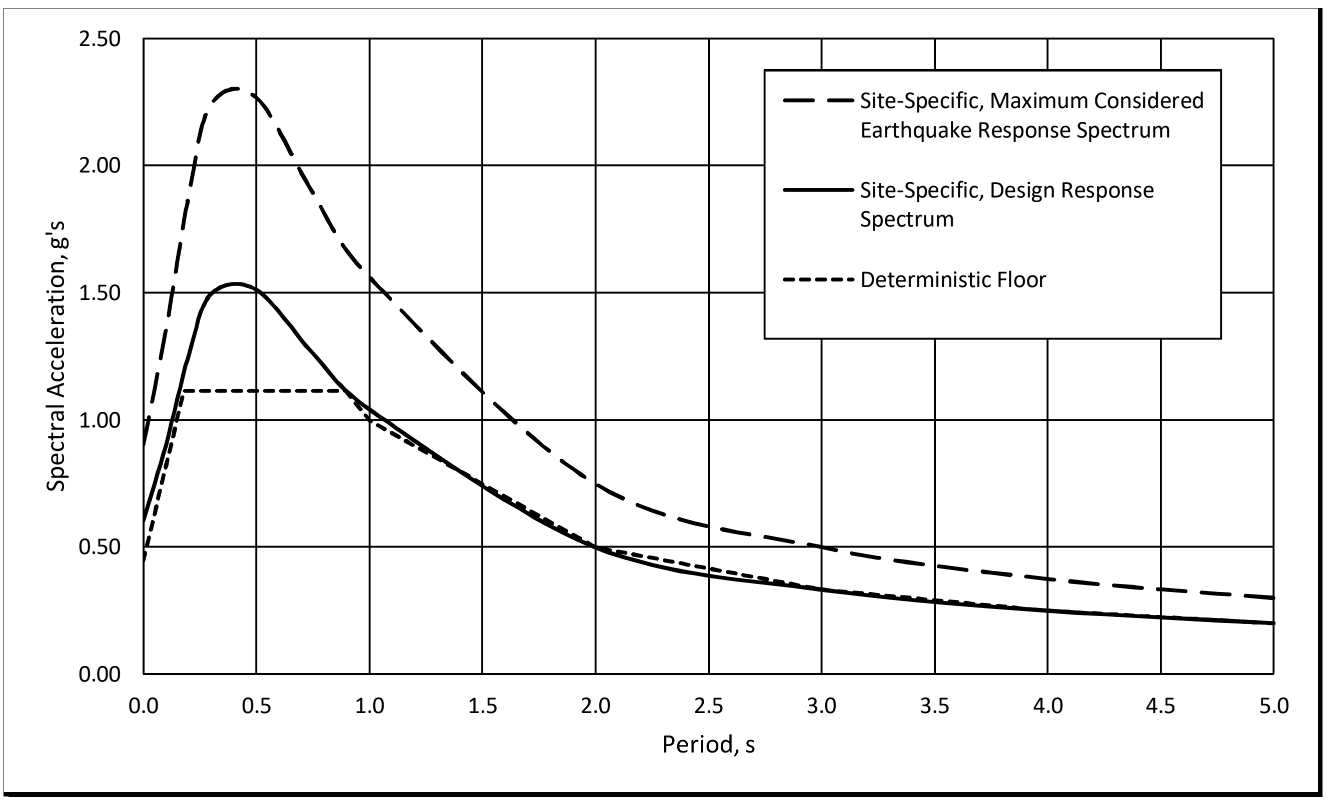
APRIL 2020	PROJECT NO. W1084-06-01	FIG. 4



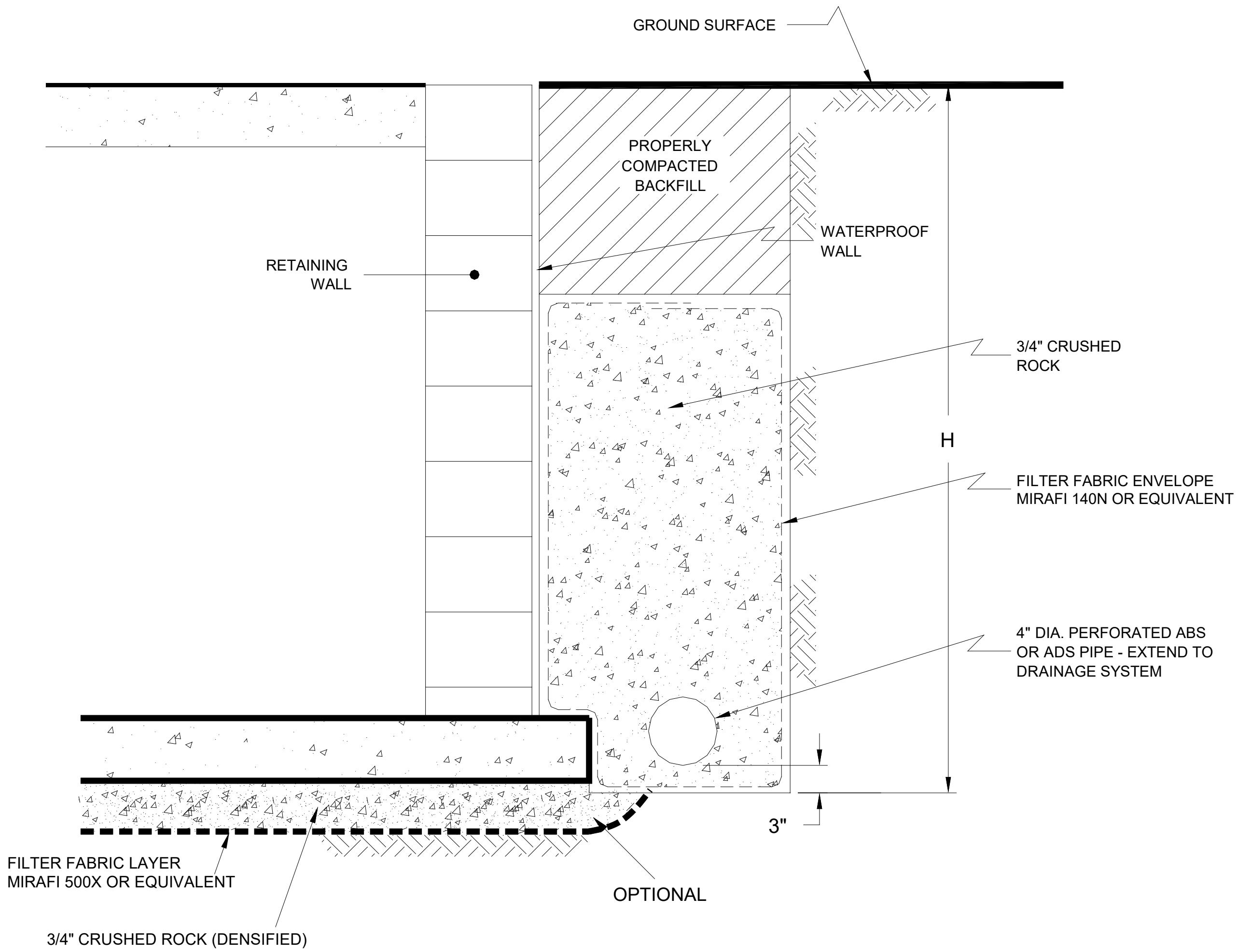


GEOCON	**DESIGN RESPONSE SPECTRUM**	Project No.: W1084-06-01
		1000 Seward Street Los Angeles, California
	Checked by: JJK	APRIL 2020 Figure 5





GEOCON	**DESIGN RESPONSE SPECTRUM**	Project No.: W1084-06-01
		1000 Seward Street Los Angeles, California
	Checked by: JJK	APRIL 2020 Figure 6



NO SCALE

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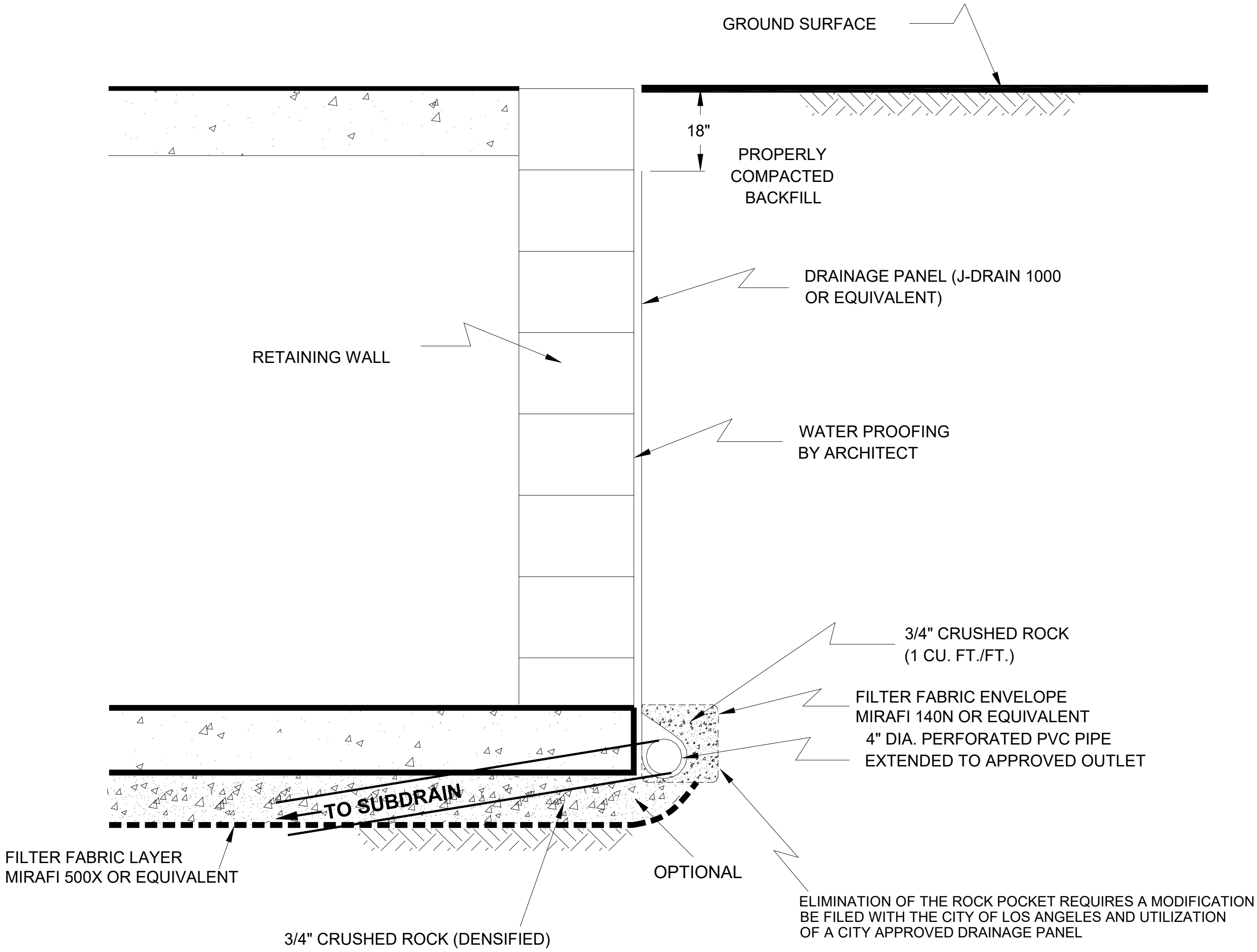
RETAINING WALL DRAIN DETAIL

1000 SEWARD STREET
LOS ANGELES, CALIFORNIA

DRAFTED BY: JMH	CHECKED BY: NDB	APRIL 2020	PROJECT NO. W1184-06-01	FIG. 7



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

NO SCALE

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RETAINING WALL DRAIN DETAIL

1000 SEWARD STREET
LOS ANGELES, CALIFORNIA

DRAFTED BY: JMH	CHECKED BY: NDB	APRIL 2020	PROJECT NO. W1084-06-01	FIG. 8

Spectral Period (seconds)	Probabilistic Uniform-Hazard	Risk-Targeted, Probabilistic	Risk Factor, Cr	Maximum-Rotated Componet Scale Factor	MRC, Risk-Targeted Probablistic	84th Percentile, Deterministic
0	0.901	0.860	0.955	1.190	1.024	0.902
0.1	1.496	1.452	0.970	1.190	1.727	1.353
0.2	1.963	1.924	0.981	1.220	2.348	1.879
0.3	2.261	2.149	0.950	1.230	2.644	2.242
0.5	2.231	2.058	0.922	1.230	2.532	2.267
0.75	1.832	1.678	0.916	1.240	2.081	1.892
1	1.544	1.396	0.904	1.240	1.731	1.561
2	0.814	0.733	0.901	1.240	0.909	0.744
3	0.508	0.458	0.901	1.250	0.573	0.434
4	0.342	0.308	0.902	1.260	0.389	0.266
5	0.250	0.225	0.900	1.260	0.284	0.178

Site-Specific Design Earthquake	Design Earthquake Floor	Site-Specific Maximum Considered Earthquake
0.602	0.445	0.902
0.902	0.818	1.353
1.253	1.114	1.879
1.495	1.114	2.242
1.511	1.114	2.267
1.262	1.114	1.892
1.041	0.997	1.561
0.499	0.499	0.748
0.332	0.332	0.499
0.249	0.249	0.374
0.199	0.199	0.299

SM_S = 2.040 g

SM_1 = 1.561 g

SD_S = 1.360 g

SD_1 = 1.041 g

GEOCON

DESIGN RESPONSE SPECTRUM

Checked by: JJK

Project No.: W1084-06-01

1000 Seward Street
Los Angeles, California

APRIL 2020

Table 1

APPENDIX A

APPENDIX A

FIELD INVESTIGATION

. The site was explored on November 11, 2019, by excavating two 8-inch-diameter borings utilizing a truck-mounted hollow-stem auger drilling machine. The borings were excavated to depths 60½ feet below the existing ground surface. Representative and relatively undisturbed samples were obtained by driving a 3-inch, O. D., California Modified Sampler into the "undisturbed" soil mass with blows from a 140-pound auto-hammer falling 30 inches. The California Modified Sampler was equipped with 1-inch high by 2³/₈-inch diameter brass sampler rings to facilitate soil removal and testing. Bulk samples were also obtained.

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

PROJECT NO. W1084-06-01

BORING 1

ELEV. (MSL.) -- DATE COMPLETED 11/12/19

EQUIPMENT HOLLOW STEM AUGER BY: JJK

DEPTH IN FEET	SAMPLE NO.	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0			**AC: 1.5" BASE: 2"** **ARTIFICIAL FILL** Clay, moist, dark brown, trace medium-grained sand.			
2			Clay, moist, dark brown, trace medium-grained sand.			
	B1@3'			19	111.5	19.2
4			**ALLUVIUM** Silty Clay, stiff, moist, brown, trace medium-grained sand.			
	B1@5'			40	112.6	18.5
6		CL				
8			Clay, firm, moist, dark brown, trace medium-grained sand.			
10	B1@10'			14	87.4	40.4
		CL				
12						
14			Clayey Sand, medium dense, moist, brown, fine- to coarse-grained sand, trace fine-grained sand.			
	B1@15'			35	115.4	17.9
16		SC				
18						
		SP	Sand with Clay, poorly graded, dense, very moist, brown, medium-grained sand, fine gravel.			
20	B1@20'			64	123.4	13.4
	BULK 20-25'		Sand, poorly graded, very dense, very moist, medium-grained sand, fine to coarse gravel, trace silt.			
22	B1@22'	SP		96	113.1	13.3
24			Clayey Silt, stiff, moist to very moist, trace fine-grained sand.			
	B1@25'	ML		26	98.3	29.3
26			Silty Sand, medium dense, moist to very moist, reddish brown, fine-grained sand, some clay.			
	B1@27'			16	105.0	23.8
28		SM				

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

W1084-06-01 BORING LOGS.GPJ

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

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

GEOCON

PROJECT NO. W1084-06-01

BORING 1

ELEV. (MSL.) -- DATE COMPLETED 11/12/19

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		23	114.1	19.6
32								
34				SC	Clayey Sand, medium dense, very moist, brown, fine- to medium-grained sand.			
36	B1@35'					25	111.3	20.1
38				ML	Sandy Silt, hard, moist, reddish brown and gray, fine-grained sand, trace clay.			
40	B1@40'					55	112.1	20.7
42								
44								
46	B1@45'				- dense, fine- to medium-grained	50 (5")	121.9	14.2
48				SP-SM	Silt with Sand, moist, reddish brown and gray, some clay, fine-grained sand.			
50	B1@50'					29	105.5	23.4
52								
54								
56	B1@55'			SP	- hard Sand, poorly graded, dense, moist to very moist, reddish grayish brown, fine- to medium-grained, trace clay.	54	115.6	19.3
58				ML	Clayey Silt, hard, slightly moist, reddish brown, trace medium-grained sand.			

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

W1084-06-01 BORING LOGS.GPJ

SAMPLE SYMBOLS

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

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

GEOCON

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	BORING 1 ELEV. (MSL.) -- DATE COMPLETED 11/12/19 EQUIPMENT HOLLOW STEM AUGER BY: JJK	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
					MATERIAL DESCRIPTION			
60	B1@60'			ML		50 (4")	113.7	19.1
					Total depth of boring: 60.5 feet Fill to 4 feet. Groundwater encountered at 18 feet. Backfilled with grout. *Penetration resistance for 140-pound hammer falling 30 inches by auto-hammer.			

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

W1084-06-01 BORING LOGS.GPJ

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

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

GEOCON

PROJECT NO. W1084-06-01

BORING 2

ELEV. (MSL.) -- DATE COMPLETED 11/12/19

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					**AC: 1.5" BASE: 3"**			
					ARTIFICIAL FILL Clay, moist, dark brown.			
2	B2@2'				**ALLUVIUM** Clay, firm, moist, dark brown, trace fine-grained sand.	17	108.3	19.0
4	B2@4'			CL	- stiff, fine- to medium-grained sand	24	105.6	20.8
6								
8								
				SC	Clayey Sand, loose, moist, brown, fine-grained sand.			
10	B2@10'			CL	Clay, firm, moist, dark brown, trace fine-grained sand.	16	105.1	18.1
12								
14				CL	Sandy Clay, stiff, moist, brown to olive gray, fine-grained sand.			
16	B2@15'					28	115.4	16.7
18				SC	Clayey Sand, medium dense, reddish brown with gray, fine-grained sand, trace medium-grained sand.			
20	B2@20'					32	117.7	16.6
				CL	Clay with Sand, stiff, moist, reddish brown and gray, fine-grained sand.			
22	B2@22'					31	108.3	22.7
24				ML	Clayey Silt, firm, moist, reddish brown with gray, trace fine-grained sand.			
	B2@25'					24	101.6	25.7
26				SM	Silty Sand, medium dense, moist to very moist, reddish brown and gray, fine-grained sand, trace clay.			
	B2@27'		▼			23	108.9	19.4
28								
				SC	Clayey Sand, medium dense, moist to very moist, reddish brown, fine- to			

W1084-06-01 BORING LOGS.GPJ

Figure A2,
Log of Boring 2, Page 1 of 3

SAMPLE SYMBOLS

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

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

GEOCON

PROJECT NO. W1084-06-01

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	BORING 2 ELEV. (MSL.) -- DATE COMPLETED 11/12/19 EQUIPMENT HOLLOW STEM AUGER BY: JJK MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
30	B2@30'			SC	medium-grained sand, trace silt.	23	80.8	66.4
32								
34				SM	Silty Sand, medium dense, moist, light gray, fine- to medium-grained sand, trace clay.			
36	B2@35'					37	115.5	18.9
38				ML	Clayey Silt, stiff, moist, reddish brown.			
40	B2@40'					37	110.3	21.1
42								
44				SM	Silty Sand, very dense, moist, reddish brown, fine-grained, trace clay, trace medium-grained sand.			
46	B2@45'					50 (6")	115.3	16.9
48				SC	Clayey Sand, medium dense, moist to very moist, reddish brown, fine-grained, trace medium-grained.			
50	B2@50'					40	110.3	20.7
52								
54				CL	Sandy Clay, hard, slightly moist to moist, reddish brown, fine-grained sand.			
56	B2@55'					50 (5")	113.8	18.2
58				SC	Clayey Sand, dense, moist, reddish brown, fine- to medium-grained, trace coarse-grained sand.			

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

W1084-06-01 BORING LOGS.GPJ

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

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

GEOCON

BORING 2

ELEV. (MSL.) -- DATE COMPLETED 11/12/19

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	B2@60'			SC		81	122.1	13.0
					Total depth of boring: 60.5 feet Fill to 1.5 feet. Groundwater encountered at 27 feet. Backfilled with grout. Patched with concrete. *Penetration resistance for 140-pound hammer falling 30 inches by auto-hammer.			

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

W1084-06-01 BORING LOGS.GPJ

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

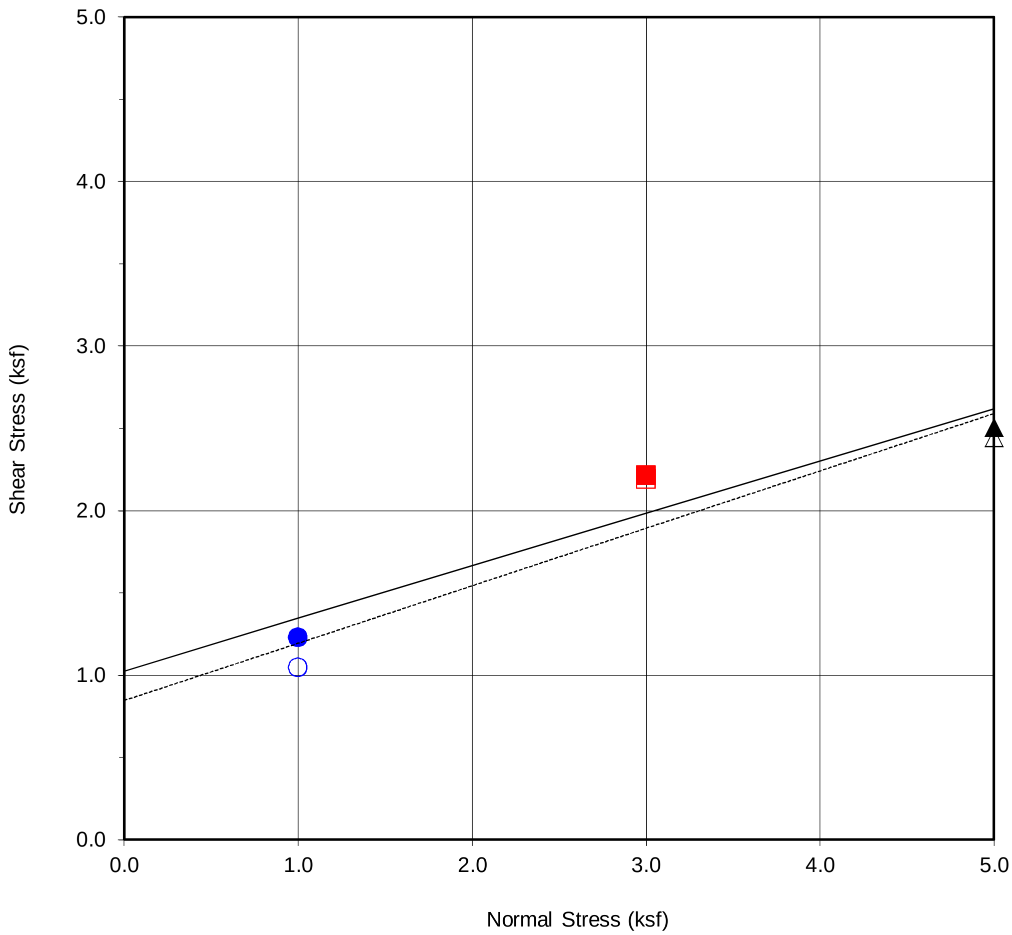
NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

APPENDIX B

APPENDIX B

LABORATORY TESTING

Laboratory tests were performed in accordance with generally accepted test methods of the International ASTM, or other suggested procedures. Selected samples were tested for direct shear strength, consolidation characteristics, expansion index, corrosivity, in-place dry density and moisture content. The results of the laboratory tests are summarized in Figures B1 through B19. The in-place dry density and moisture content of the samples tested are presented on the boring logs, Appendix A.



Boring No.	**B1**
Sample No.	**B1@3**
Depth (ft)	**3**
Sample Type:	Ring

Soil Identification:		
Dark Brown Clay (CL)		
Strength Parameters		
	C (psf)	ϕ (°)
Peak	1024	17.7
Ultimate	846	19.2

Normal Stress (kip/ft2)	1	3	5
Peak Shear Stress (kip/ft²)	● 1.23	■ 2.21	▲ 2.50
Shear Stress @ End of Test (ksf)	○ 1.05	□ 2.19	△ 2.44
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 (%)	20.1	19.2	19.7
Initial Dry Density (pcf)	107.0	110.6	109.7
Initial Degree of Saturation (%)	94.2	99.1	99.2
Soil Height Before Shearing (in.)	1.2	1.2	1.2
Final Moisture Content (%)	23.3	21.3	21.2

GEOCON

DIRECT SHEAR TEST RESULTS

Consolidated Drained ASTM D-3080

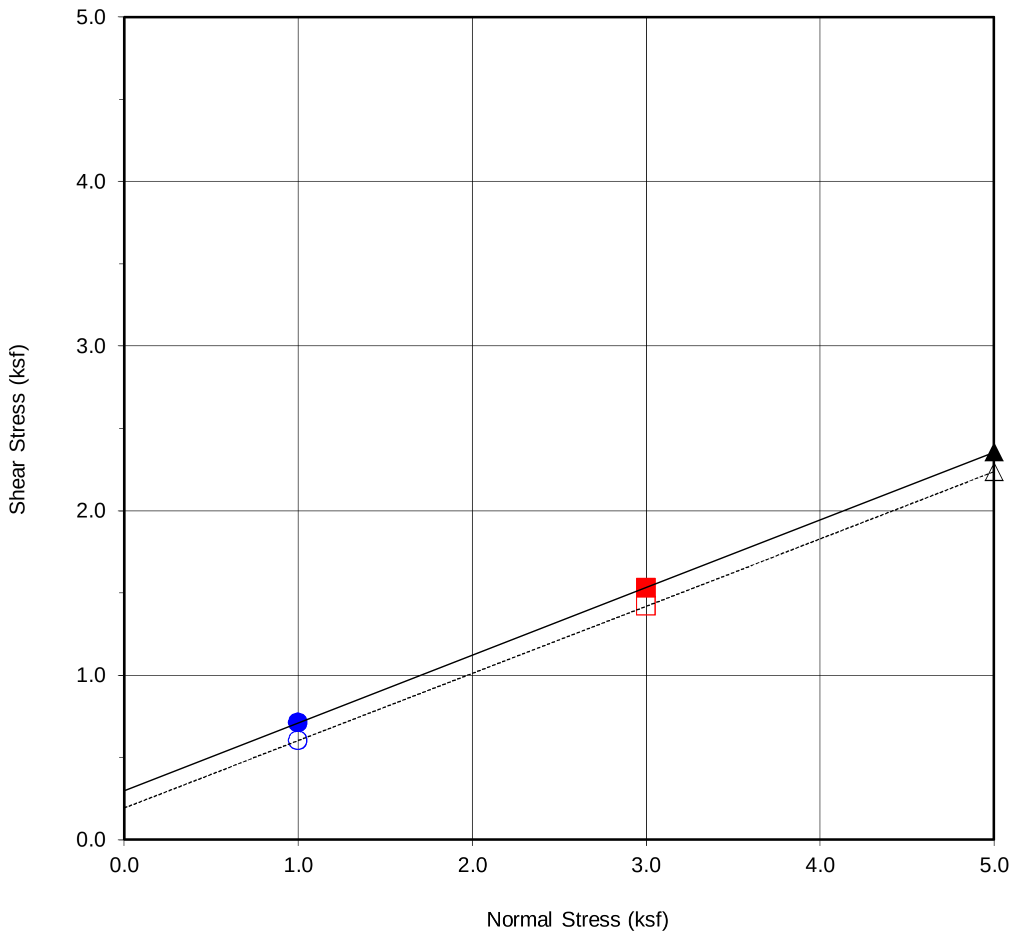
Checked by: JMH

Project No.: W1084-06-01

1000 Seward Street
Los Angeles, CA 90038

APRIL 2020

Figure B1



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

Soil Identification:		
Dark Brown Clay (CL)		
Strength Parameters		
	C (psf)	ϕ (°)
Peak	296	22.4
Ultimate	192	22.2

Normal Stress (kip/ft2)	1	3	5
Peak Shear Stress (kip/ft²)	● 0.71	■ 1.53	▲ 2.36
Shear Stress @ End of Test (ksf)	○ 0.60	□ 1.42	△ 2.24
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 (%)	40.4	37.4	37.0
Initial Dry Density (pcf)	87.0	86.3	87.9
Initial Degree of Saturation (%)	116.3	105.9	109.0
Soil Height Before Shearing (in.)	1.2	1.2	1.2
Final Moisture Content (%)	37.6	35.5	35.0

GEOCON

DIRECT SHEAR TEST RESULTS

Consolidated Drained ASTM D-3080

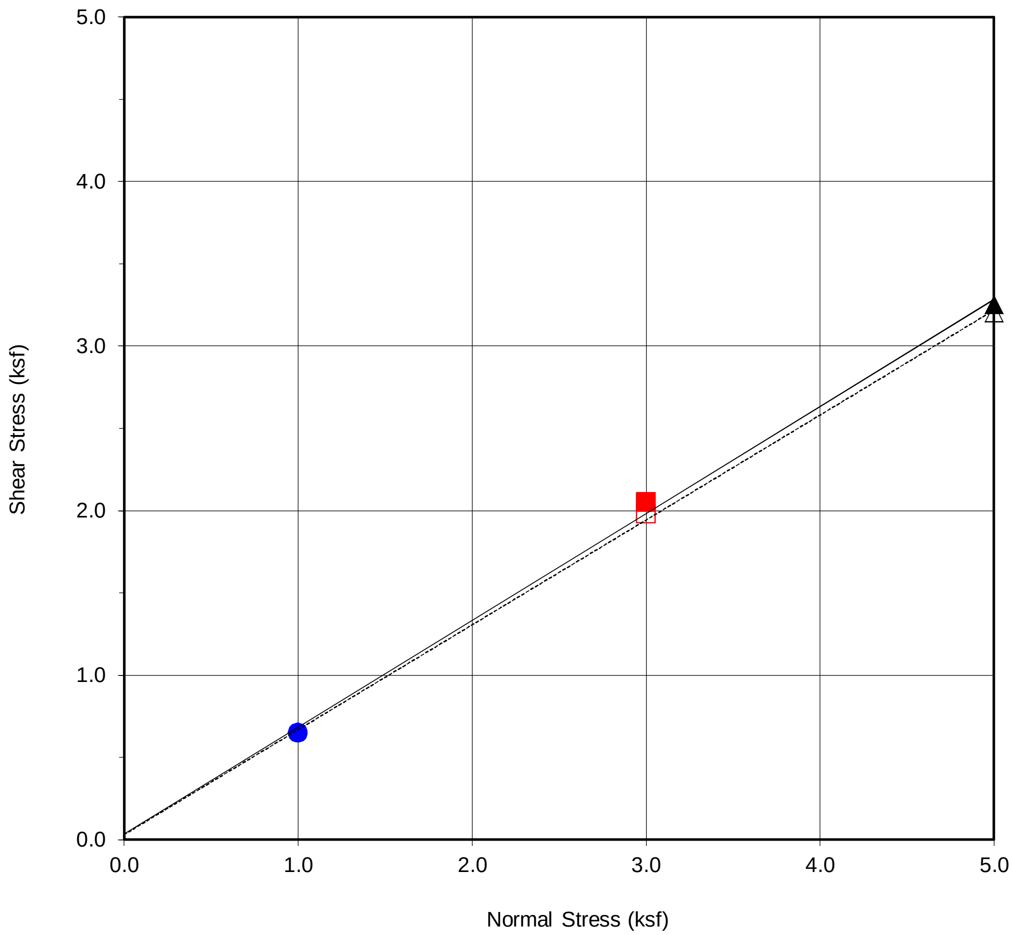
Checked by: JMH

Project No.: W1084-06-01

1000 Seward Street
Los Angeles, CA 90038

APRIL 2020

Figure B2



Boring No.	**B1**
Sample No.	**B1@22**
Depth (ft)	**22**
Sample Type:	Ring

Soil Identification:		
Olive Brown Poorly Graded Sand (SP)		
Strength Parameters		
	C (psf)	ϕ (°)
Peak	33	33.0
Ultimate	31	32.5

Normal Stress (kip/ft2)	1	3	5
Peak Shear Stress (kip/ft²)	● 0.65	■ 2.05	▲ 3.25
Shear Stress @ End of Test (ksf)	○ 0.65	□ 1.98	△ 3.20
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 (%)	13.3	15.2	16.4
Initial Dry Density (pcf)	115.1	108.1	115.3
Initial Degree of Saturation (%)	77.3	73.2	96.0
Soil Height Before Shearing (in.)	1.2	1.2	1.2
Final Moisture Content (%)	16.0	17.9	15.6

GEOCON

DIRECT SHEAR TEST RESULTS

Consolidated Drained ASTM D-3080

Checked by: JMH

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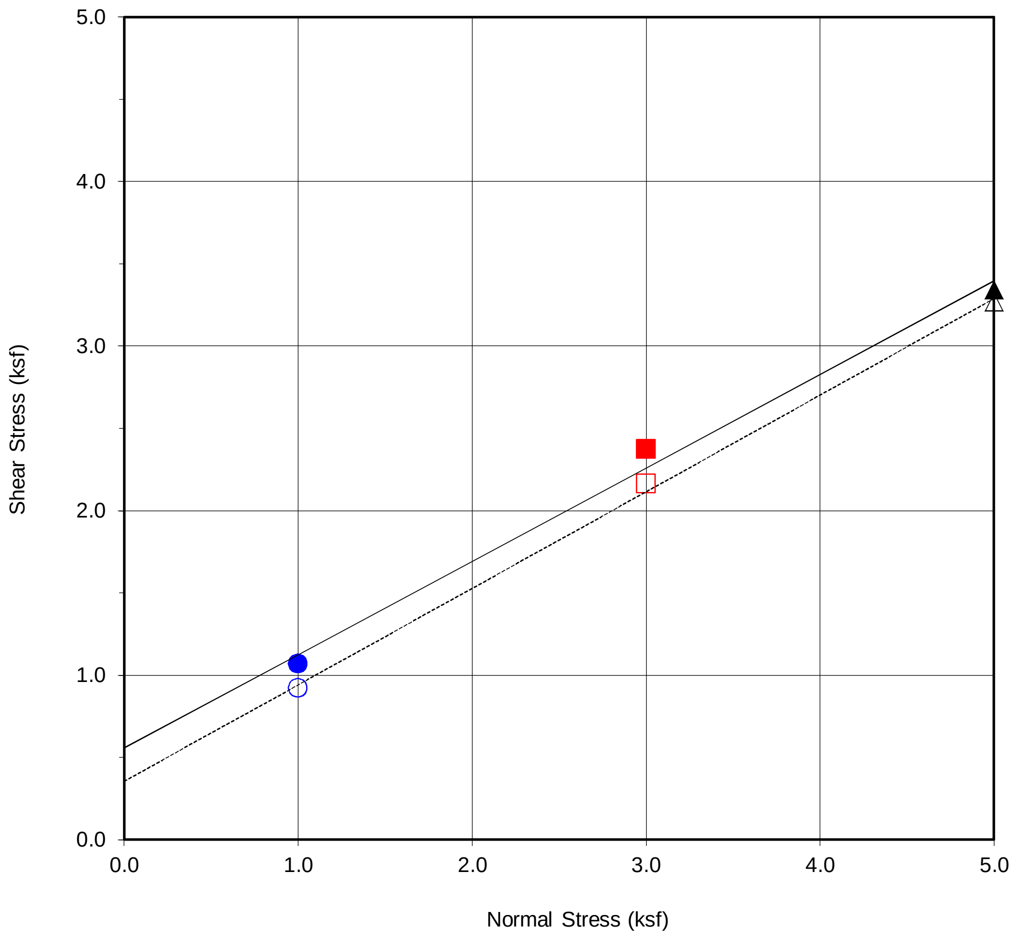
Figure B3



Boring No.	**B1**
Sample No.	**B1@35**
Depth (ft)	**35**
Sample Type:	Ring

Soil Identification:		
Brown Clay (CL)		
Strength Parameters		
	C (psf)	ϕ (°)
Peak	556	29.6
Ultimate	355	30.4

Normal Stress (kip/ft2)	1	3	5
Peak Shear Stress (kip/ft²)	● 1.07	■ 2.37	▲ 3.34
Shear Stress @ End of Test (ksf)	○ 0.92	□ 2.16	△ 3.27
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 (%)	22.8	22.8	22.2
Initial Dry Density (pcf)	106.1	104.3	105.5
Initial Degree of Saturation (%)	104.7	100.1	100.3
Soil Height Before Shearing (in.)	1.2	1.2	1.2
Final Moisture Content (%)	20.8	21.0	19.8

GEOCON

DIRECT SHEAR TEST RESULTS

Consolidated Drained ASTM D-3080

Checked by: JMH

Project No.: W1084-06-01

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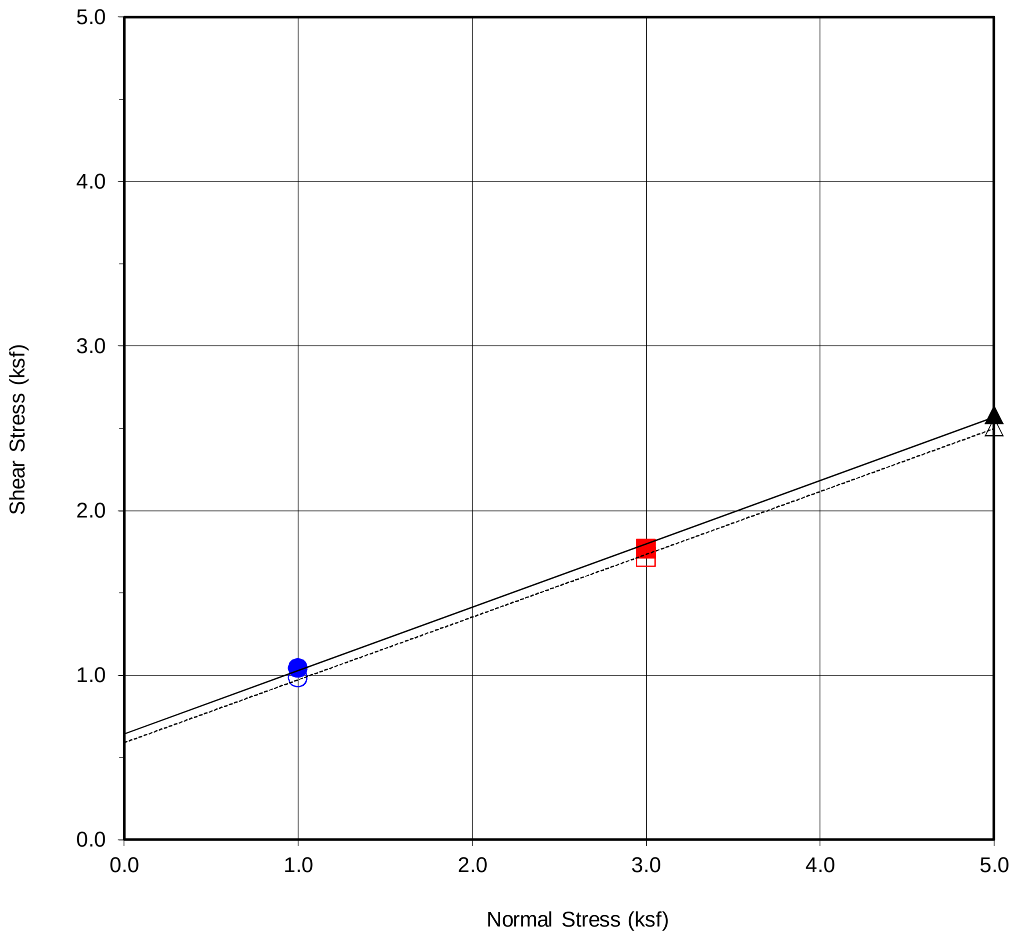
Figure B4



Boring No.	B2
Sample No.	**B2@4**
Depth (ft)	**4**
Sample Type:	Ring

Soil Identification:		
Brown Clayey Sand (SC)		
Strength Parameters		
	C (psf)	ϕ (°)
Peak	641	21.1
Ultimate	589	20.9

Normal Stress (kip/ft2)	1	3	5
Peak Shear Stress (kip/ft²)	● 1.04	■ 1.76	▲ 2.58
Shear Stress @ End of Test (ksf)	○ 0.98	□ 1.71	△ 2.51
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 (%)	20.8	20.5	20.3
Initial Dry Density (pcf)	105.9	105.6	106.8
Initial Degree of Saturation (%)	94.8	92.8	94.7
Soil Height Before Shearing (in.)	1.2	1.2	1.2
Final Moisture Content (%)	23.9	22.6	21.6

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DIRECT SHEAR TEST RESULTS

Consolidated Drained ASTM D-3080

Checked by: JMH

Project No.: W1084-06-01

1000 Seward Street
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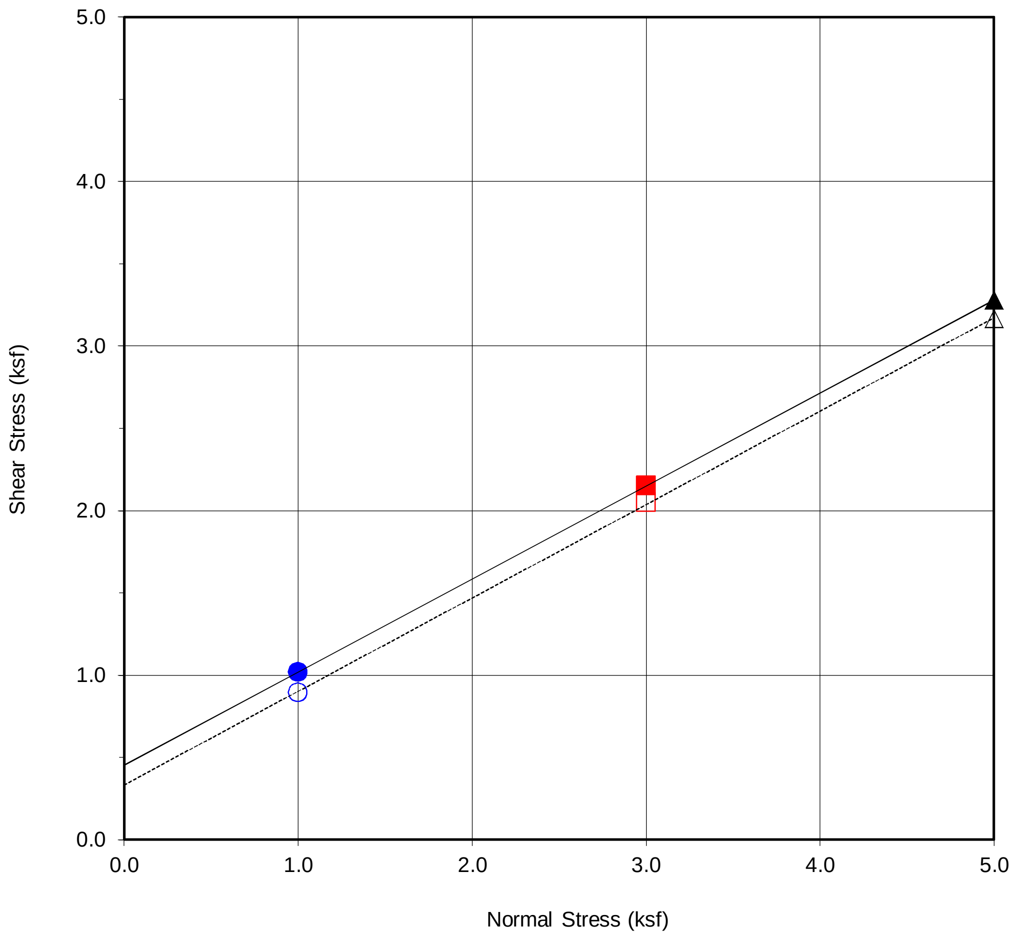
Figure B5



Boring No.	**B2**
Sample No.	**B2@10**
Depth (ft)	**10**
Sample Type:	Ring

Soil Identification:		
Brown Clayey Sand (SC)		
Strength Parameters		
	C (psf)	ϕ (^{o})
Peak	453	29.5
Ultimate	332	29.6

Normal Stress (kip/ft2)	1	3	5
Peak Shear Stress (kip/ft²)	● 1.02	■ 2.15	▲ 3.28
Shear Stress @ End of Test (ksf)	○ 0.89	□ 2.05	△ 3.17
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	17.2	21.3
Initial Dry Density (pcf)	105.2	101.5	101.4
Initial Degree of Saturation (%)	81.1	70.5	86.7
Soil Height Before Shearing (in.)	1.2	1.2	1.2
Final Moisture Content (%)	21.0	20.1	20.4

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DIRECT SHEAR TEST RESULTS

Consolidated Drained ASTM D-3080

Checked by: JMH

Project No.: W1084-06-01

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Los Angeles, CA 90038

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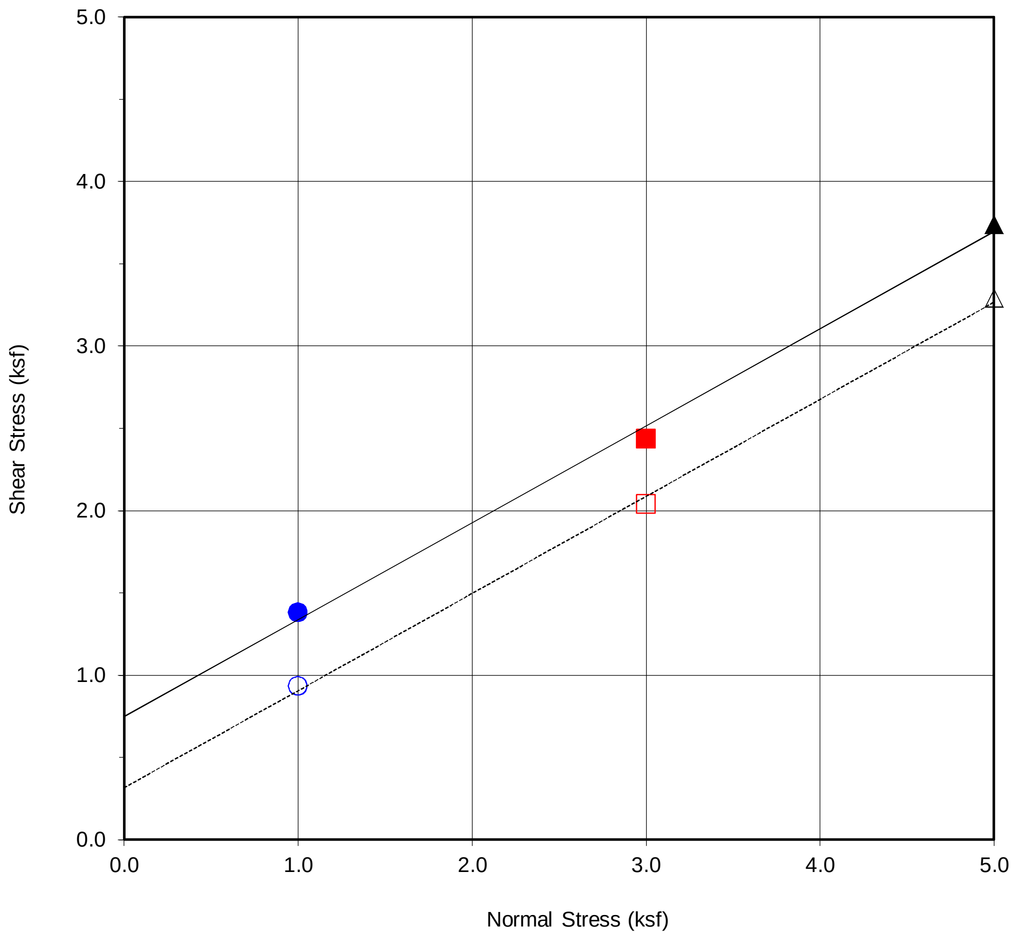
Figure B6



Boring No.	**B2**
Sample No.	**B2@22**
Depth (ft)	**22**
Sample Type:	Ring

Soil Identification:		
Brown Clay w/Sand (CL)s		
Strength Parameters		
	C (psf)	ϕ (°)
Peak	747	30.5
Ultimate	316	30.5

Normal Stress (kip/ft2)	1	3	5
Peak Shear Stress (kip/ft²)	● 1.38	■ 2.43	▲ 3.74
Shear Stress @ End of Test (ksf)	○ 0.93	□ 2.04	△ 3.29
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 (%)	22.3	22.7	22.8
Initial Dry Density (pcf)	104.2	106.3	104.2
Initial Degree of Saturation (%)	97.6	104.3	99.7
Soil Height Before Shearing (in.)	1.2	1.2	1.2
Final Moisture Content (%)	24.8	23.7	23.1

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DIRECT SHEAR TEST RESULTS

Consolidated Drained ASTM D-3080

Checked by: JMH

Project No.: W1084-06-01

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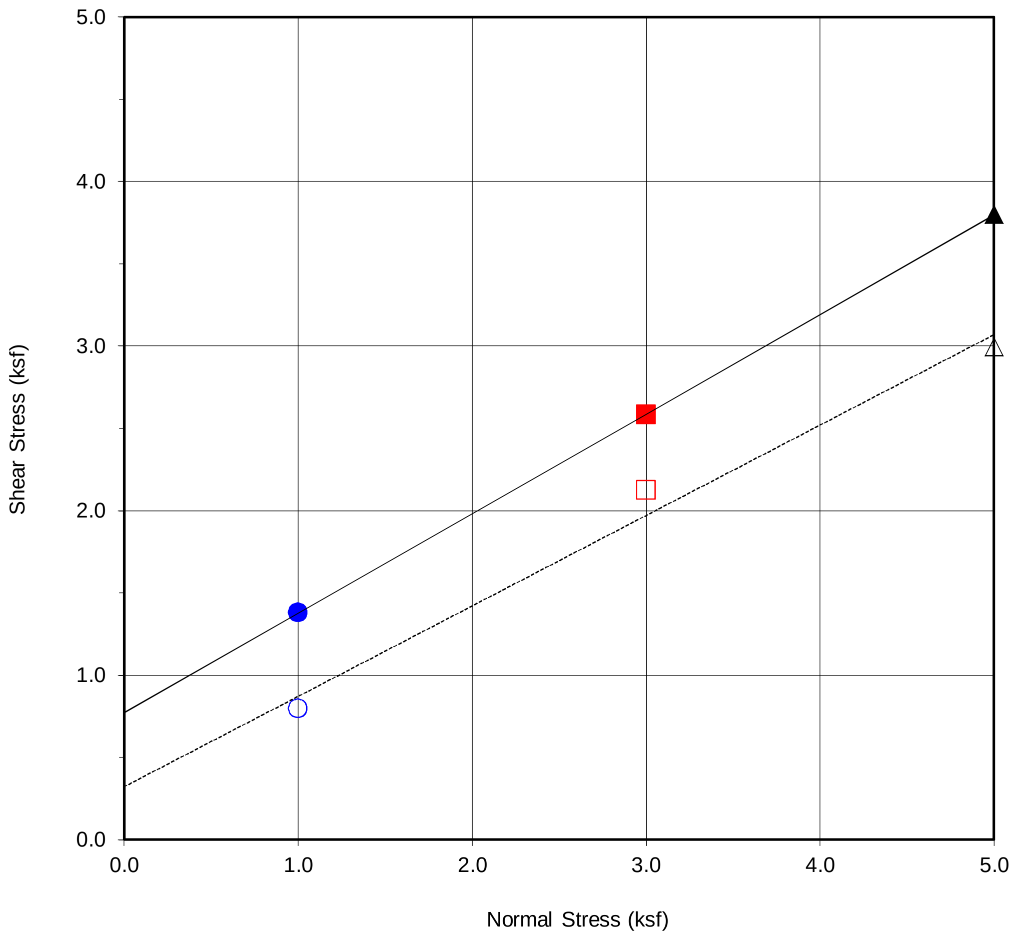
Figure B7



Boring No.	**B2**
Sample No.	**B2@40**
Depth (ft)	**40**
Sample Type:	Ring

Soil Identification:		
Brown Silt (ML)		
Strength Parameters		
	C (psf)	ϕ (°)
Peak	770	31.2
Ultimate	322	28.8

Normal Stress (kip/ft2)	1	3	5
Peak Shear Stress (kip/ft²)	● 1.38	■ 2.58	▲ 3.80
Shear Stress @ End of Test (ksf)	○ 0.80	□ 2.12	△ 2.99
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 (%)	22.9	22.8	23.0
Initial Dry Density (pcf)	105.6	103.9	105.4
Initial Degree of Saturation (%)	103.6	99.0	103.9
Soil Height Before Shearing (in.)	1.2	1.2	1.2
Final Moisture Content (%)	24.8	23.8	22.8

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DIRECT SHEAR TEST RESULTS

Consolidated Drained ASTM D-3080

Checked by: JMH

Project No.: W1084-06-01

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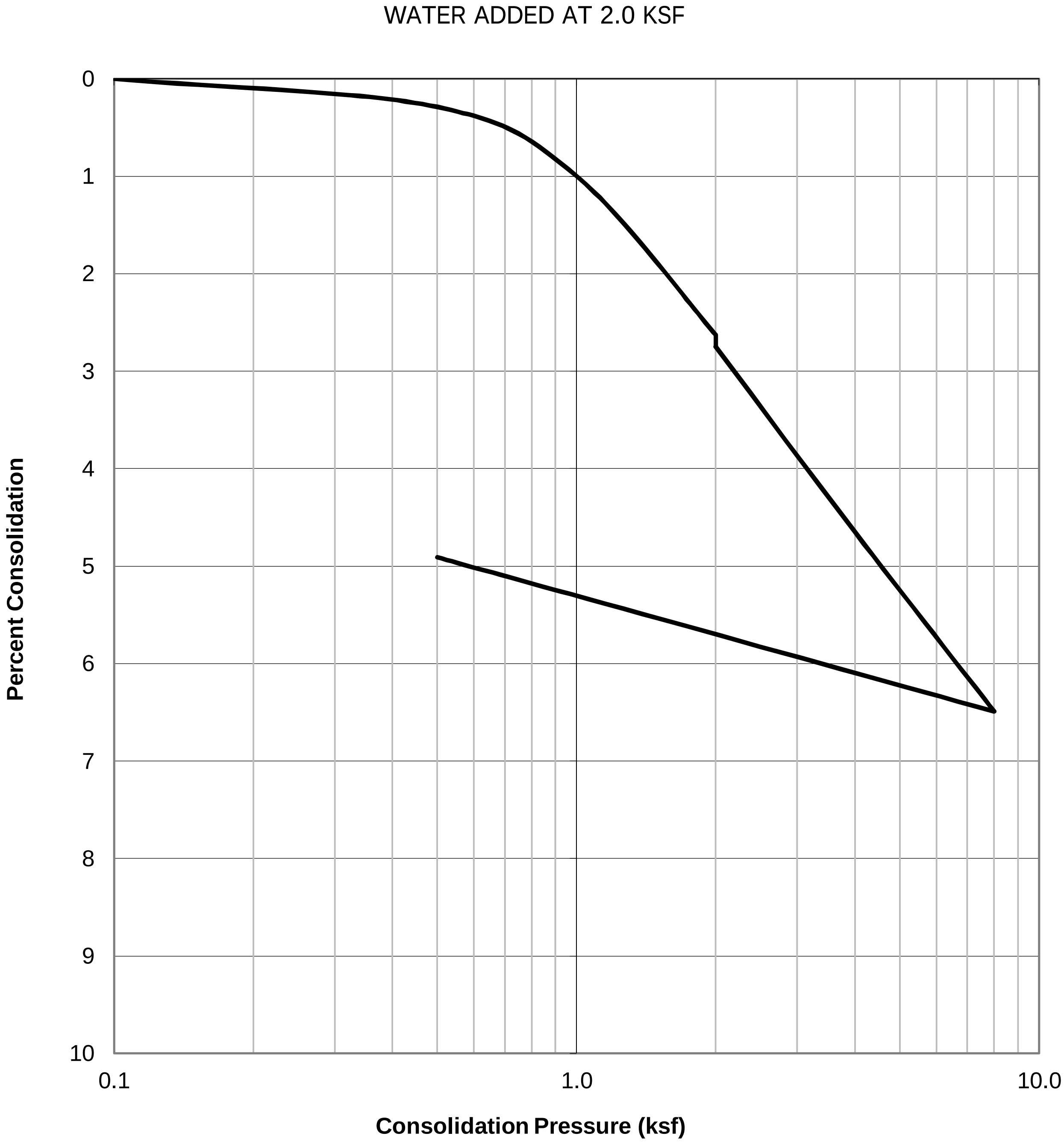
Los Angeles, CA 90038

APRIL 2020

Figure B8



SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B1@25	Brown Clayey Silt (ML)	90.0	29.3	29.5

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CONSOLIDATION TEST RESULTS

ASTM D-2435

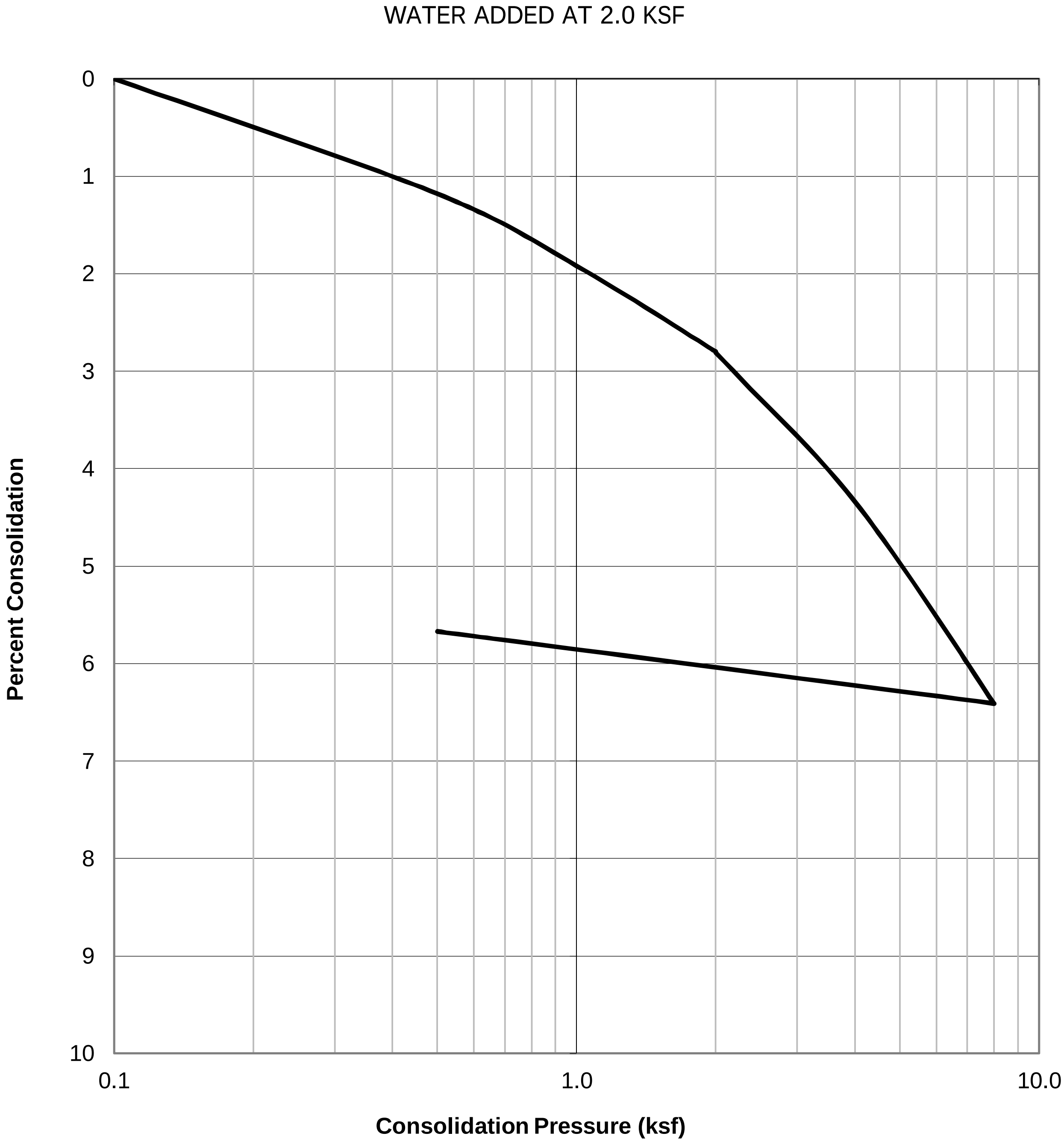
Checked by: JMH

Project No.: W1084-06-01

1000 Seward Street
Los Angeles, CA 90038

APRIL 2020

Figure B9



SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B1@27	Brown Silty Sand (SM)	101.8	21.9	19.4

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CONSOLIDATION TEST RESULTS

ASTM D-2435

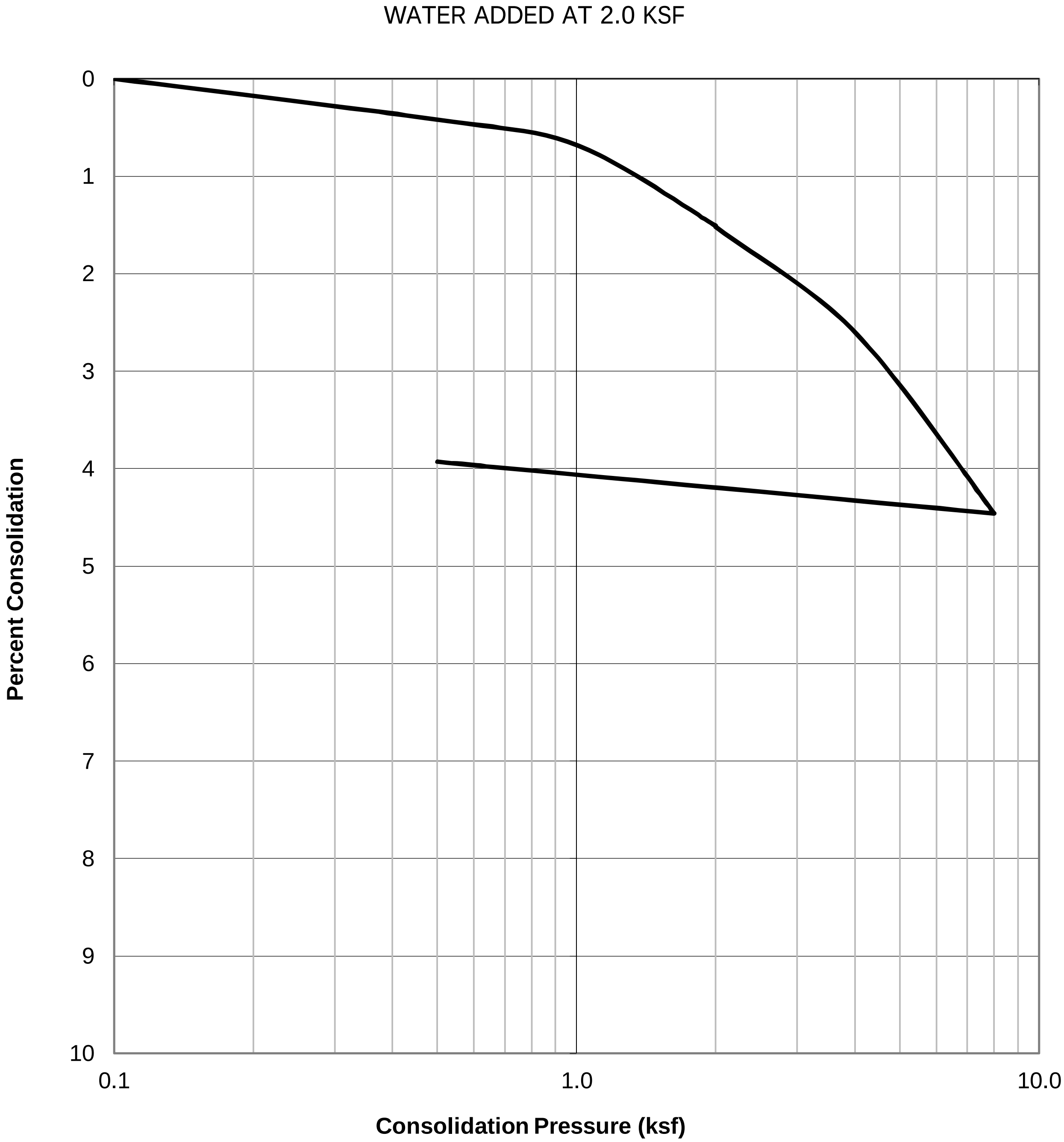
Checked by: JMH

Project No.: W1084-06-01

1000 Seward Street
Los Angeles, CA 90038

APRIL 2020

Figure B10



SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B1@35	Brown Clayey Sand (SC)	107.1	20.1	18.6

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CONSOLIDATION TEST RESULTS

ASTM D-2435

Checked by: JMH

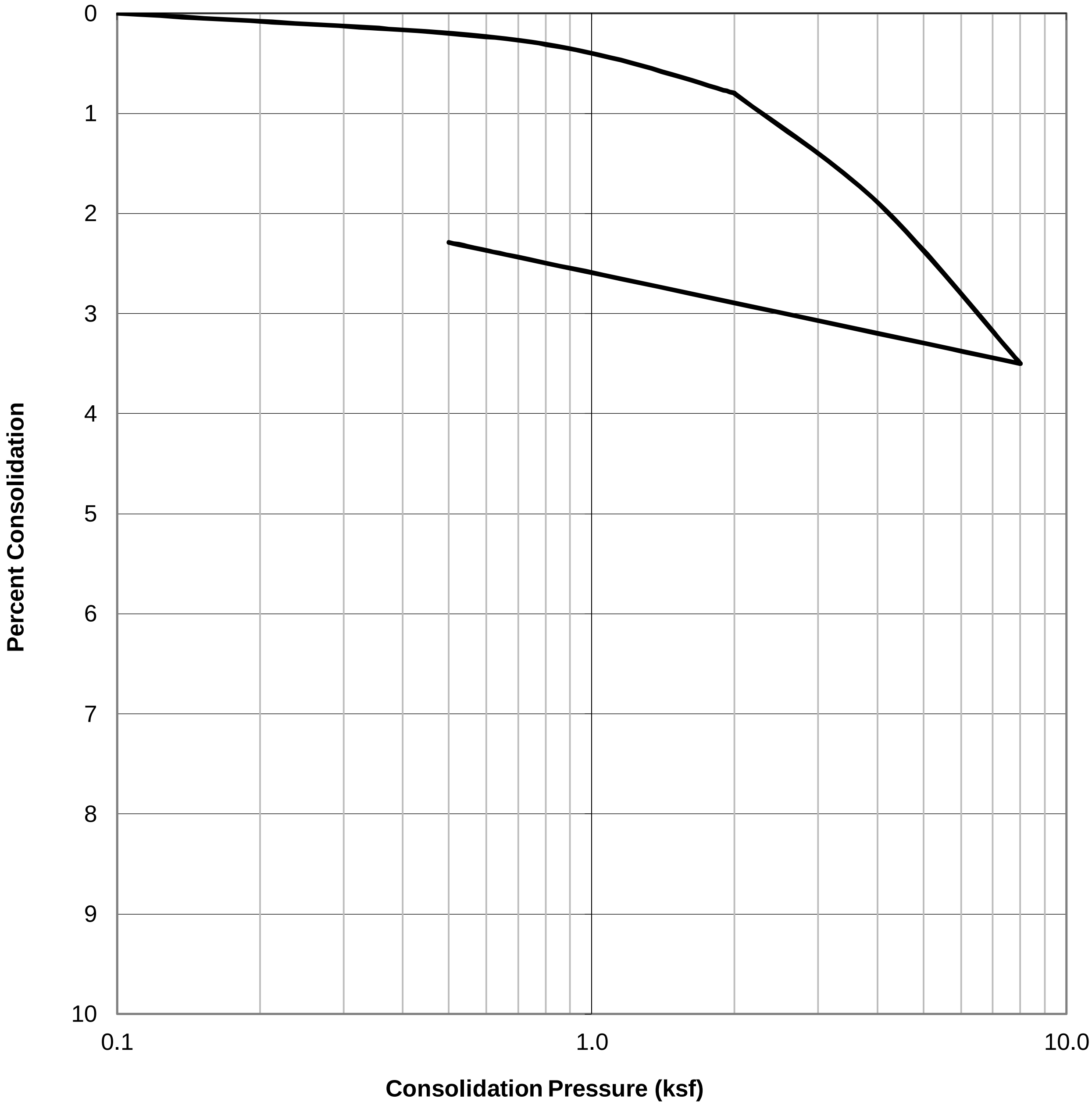
Project No.: W1084-06-01

1000 Seward Street
Los Angeles, CA 90038

APRIL 2020

Figure B11

WATER ADDED AT 2.0 KSF



SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B1@50	Brown Silt w Sand (ML)	102.9	23.4	22.8

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CONSOLIDATION TEST RESULTS

ASTM D-2435

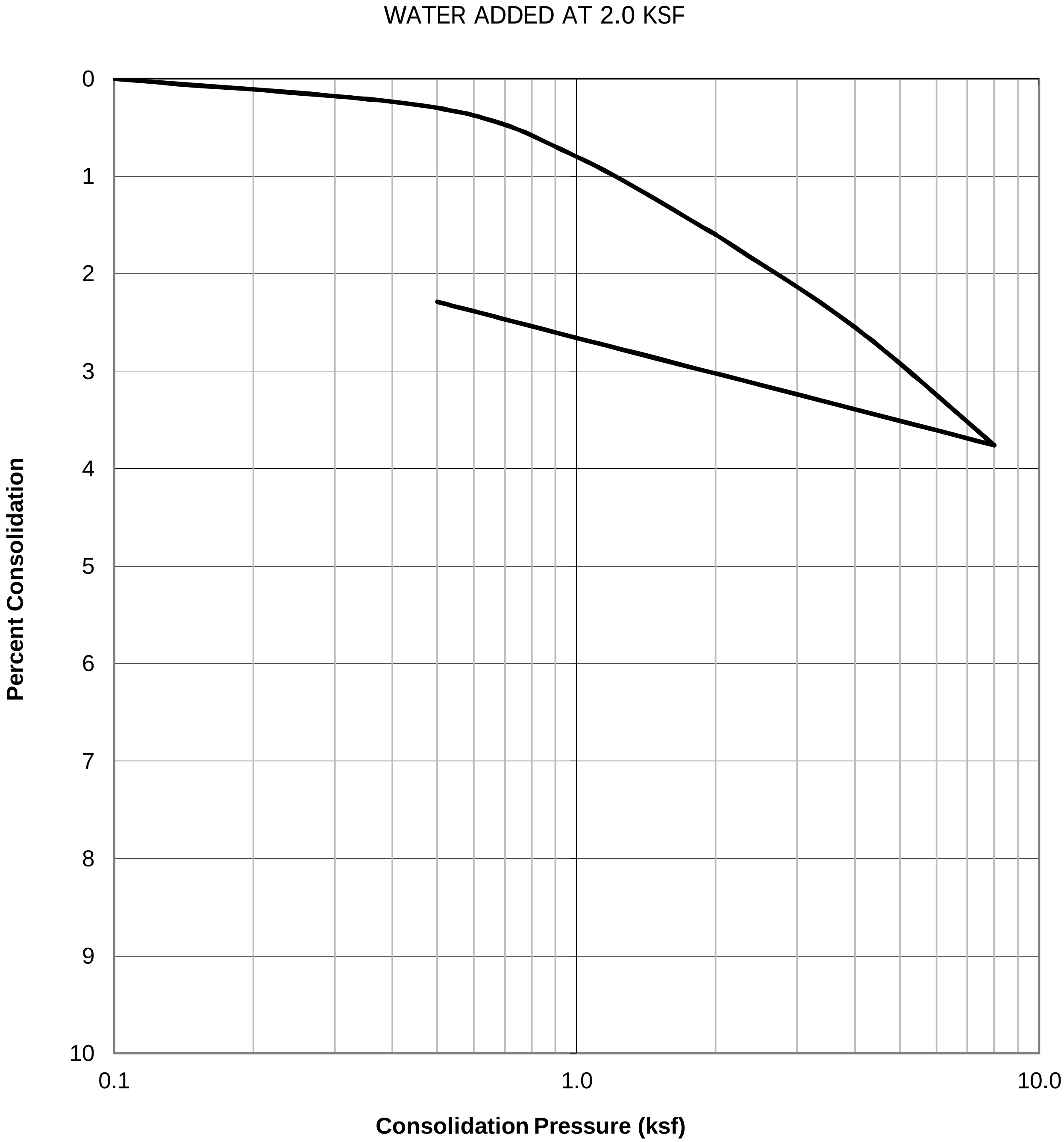
Checked by: JMH

Project No.: W1084-06-01

1000 Seward Street
Los Angeles, CA 90038

APRIL 2020

Figure B12



SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B2@22	Brown Clay (CL)	101.9	22.2	23.2

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CONSOLIDATION TEST RESULTS

ASTM D-2435

Checked by: JMH

Project No.: W1084-06-01

1000 Seward Street
Los Angeles, CA 90038

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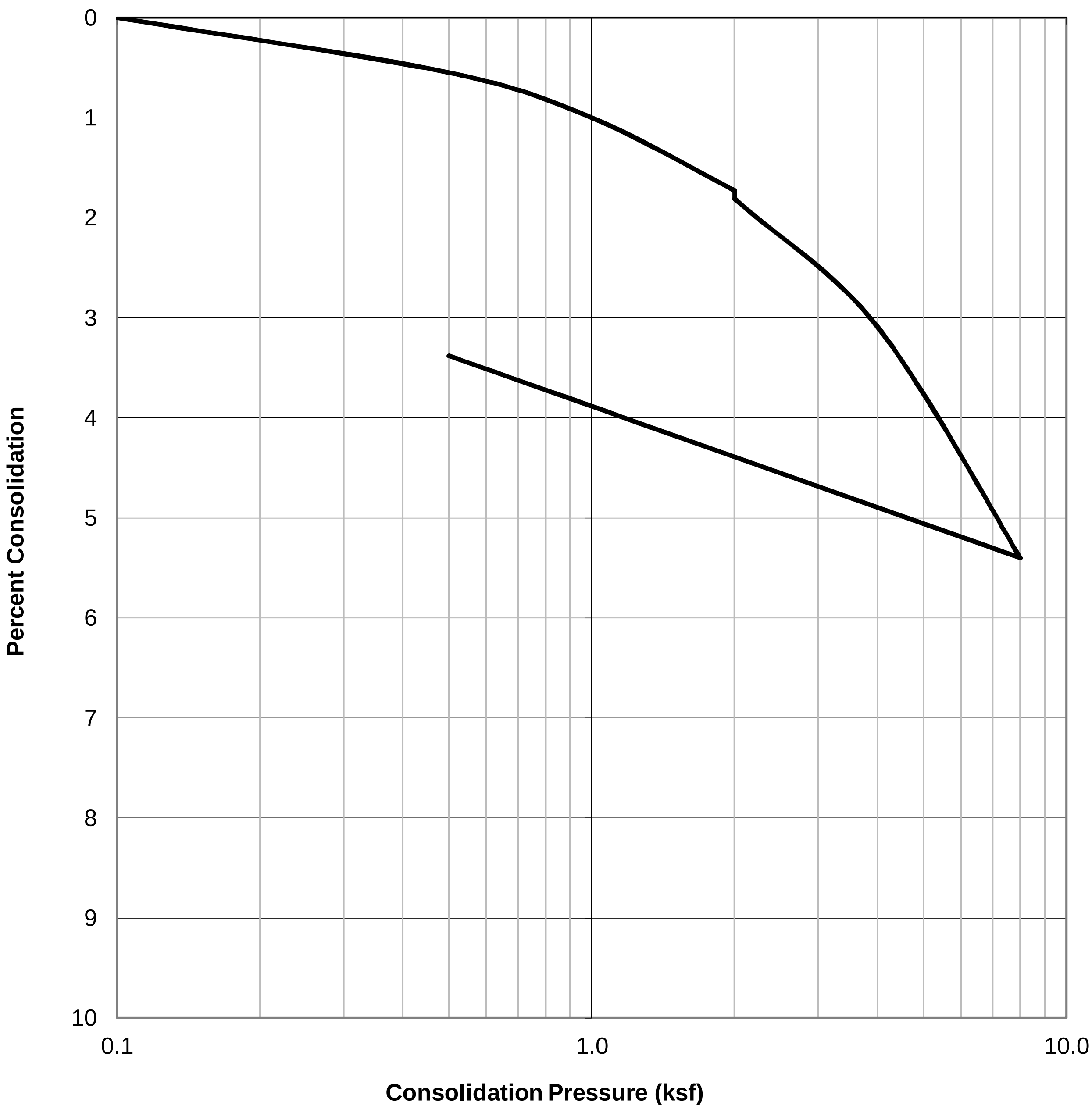
WATER ADDED AT 2.0 KSF



SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B2@25	Brown Clayey Silt (ML)	96.8	25.7	25.8

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CONSOLIDATION TEST RESULTS

ASTM D-2435

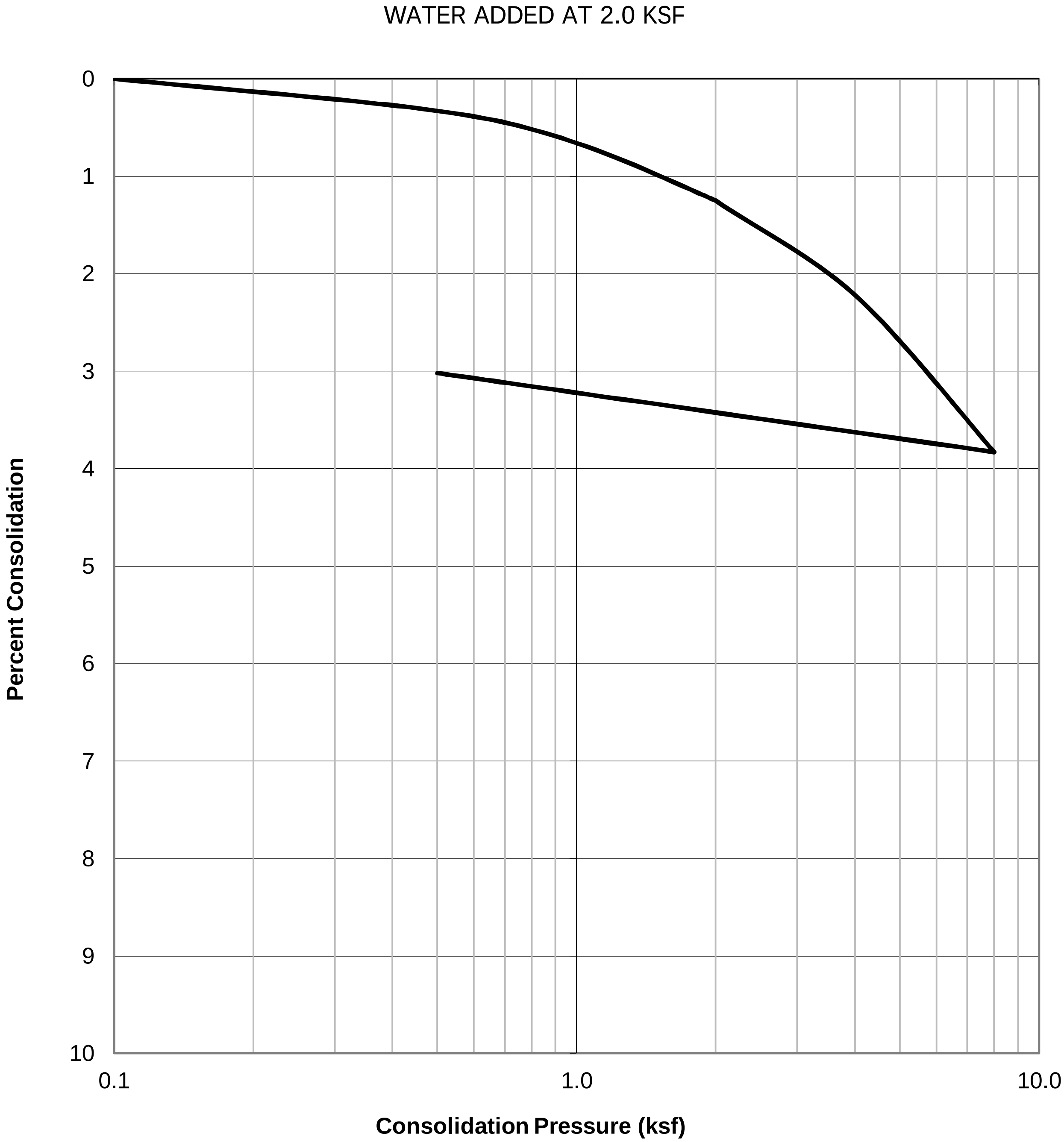
Checked by: JMH

Project No.: W1084-06-01

1000 Seward Street
Los Angeles, CA 90038

APRIL 2020

Figure B14



SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B2@30	Brown Clayey Sand (SC)	111.0	18.1	17.4

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CONSOLIDATION TEST RESULTS

ASTM D-2435

Checked by: JMH

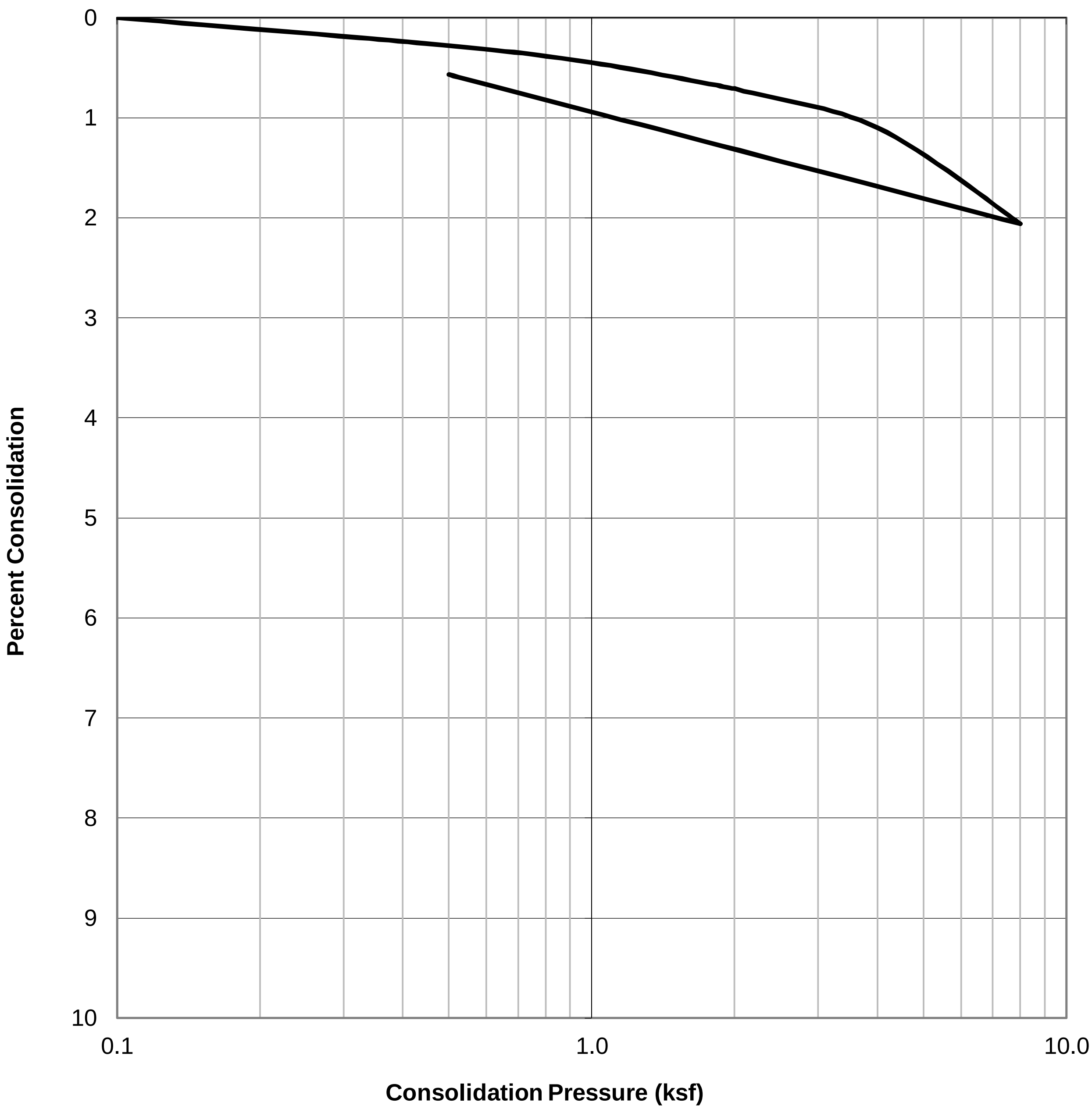
Project No.: W1084-06-01

1000 Seward Street
Los Angeles, CA 90038

APRIL 2020

Figure B15

WATER ADDED AT 2.0 KSF



SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B2@40	Reddish Brown Clayey Silt (ML)	106.9	21.1	22.1

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CONSOLIDATION TEST RESULTS

ASTM D-2435

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Project No.: W1084-06-01

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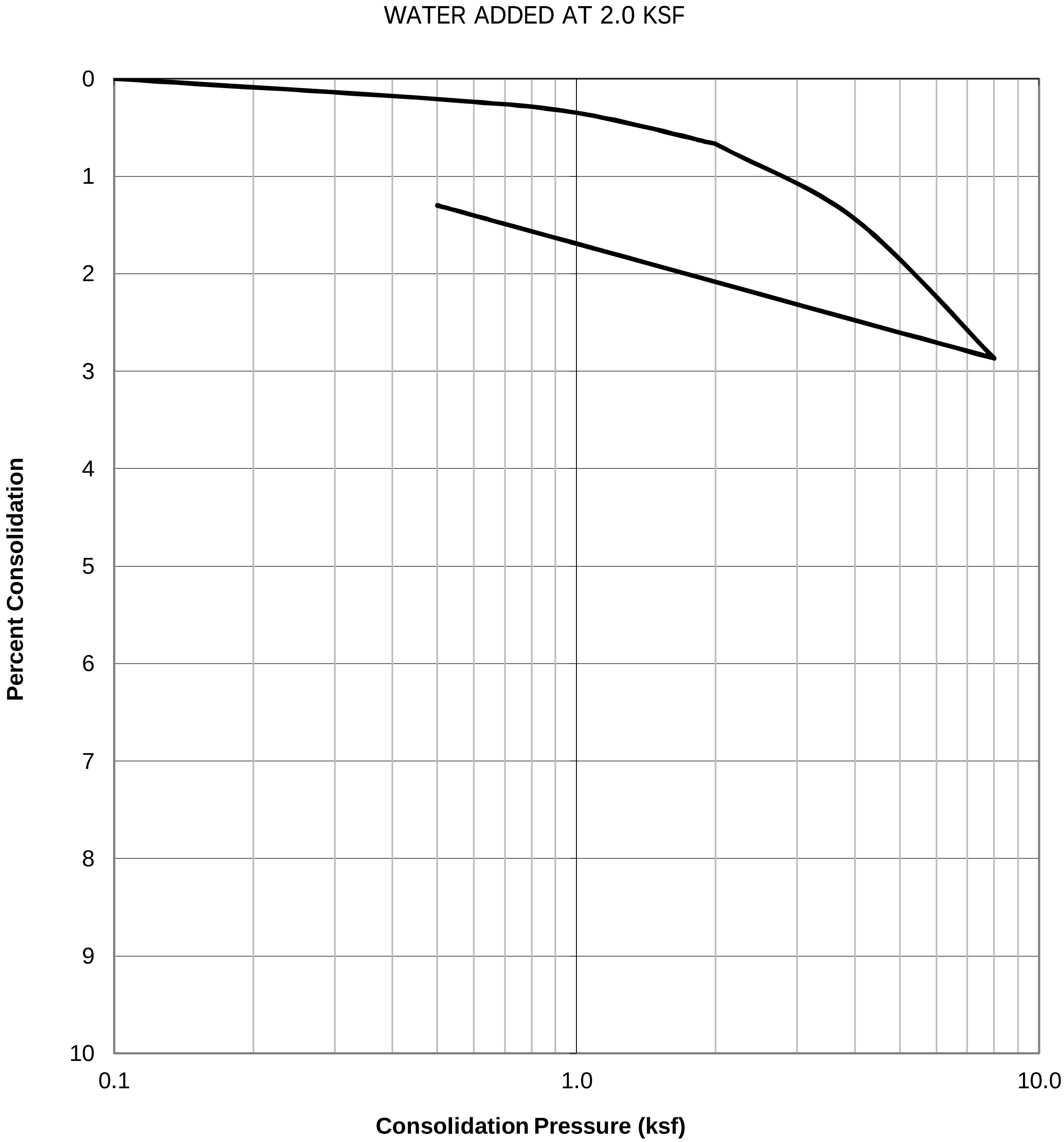
APRIL 2020

Figure B16



SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B2@50	Reddish Brown Silt (ML)	108.1	20.7	20.9

GEOCON

CONSOLIDATION TEST RESULTS

ASTM D-2435

Checked by: JMH

Project No.: W1084-06-01

1000 Seward Street
Los Angeles, CA 90038

APRIL 2020

Figure B17

B1@20-25

MOLDED SPECIMEN		BEFORE TEST	AFTER TEST
Specimen Diameter	(in.)	4.0	4.0
Specimen Height	(in.)	1.0	1.0
Wt. Comp. Soil + Mold	(gm)	770.1	805.6
Wt. of Mold	(gm)	367.7	367.7
Specific Gravity	(Assumed)	2.7	2.7
Wet Wt. of Soil + Cont.	(gm)	503.9	805.6
Dry Wt. of Soil + Cont.	(gm)	477.1	366.5
Wt. of Container	(gm)	203.9	367.7
Moisture Content	(%)	9.8	19.5
Wet Density	(pcf)	121.4	131.9
Dry Density	(pcf)	110.5	110.4
Void Ratio		0.5	0.6
Total Porosity		0.3	0.4
Pore Volume	(cc)	71.3	78.6
Degree of Saturation	(%) [S_{meas}]	50.8	90.8

Date	Time	Pressure (psi)	Elapsed Time (min)	Dial Readings (in.)
11/22/2019	10:00	1.0	0	0.296
11/22/2019	10:10	1.0	10	0.295
Add Distilled Water to the Specimen				
11/23/2019	10:00	1.0	1430	0.3305
11/23/2019	11:00	1.0	1490	0.3305

Expansion Index (EI meas) =	35.5
Expansion Index (Report) =	**36**

Expansion Index, EI_{50}	CBC CLASSIFICATION *	UBC CLASSIFICATION **
0-20	Non-Expansive	Very Low
21-50	Expansive	Low
51-90	Expansive	Medium
91-130	Expansive	High
>130	Expansive	Very High

* Reference: 2016 California Building Code, Section 1803.5.3

** Reference: 1997 Uniform Building Code, Table 18-I-B.

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EXPANSION INDEX TEST RESULTS

ASTM D-4829

Checked by: JMH

Project No.: W1084-06-01

1000 Seward Street
Los Angeles, CA 90038

APRIL 2020

Figure B18

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

Sample No.	**pH**	**Resistivity (ohm centimeters)**
B1 @ 20-25	7.8	1700 (Corrosive)
B1 @ 40'	7.3	1300 (Corrosive)

SUMMARY OF LABORATORY CHLORIDE CONTENT TEST RESULTS
EPA NO. 325.3

Sample No.	Chloride Ion Content (%)
B1@20-25	0.005
B1@40'	0.008

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

Sample No.	**Water Soluble Sulfate (% SQ_4)**	**Sulfate Exposure***
B1@20-25	0.000	S0
B1@40'	0.000	S0

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CORROSIVITY TEST RESULTS

Checked by: JMH

Project No.: W1084-06-01

1000 Seward Street
Los Angeles, CA 90038

April 2020

Figure B19

Appendix IS-2.2

Geology and Soils Report Approval Letter

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

OSAMA YOUNAN, P.E.
GENERAL MANAGER

GEOLOGY AND SOILS REPORT APPROVAL LETTER

June 18, 2020

LOG # 113348
SOILS/GEOLOGY FILE – 2

39 South, LLC
4525 Wilshire Boulevard, #150
Los Angeles, CA 90010

TRACT: White and Newby's Hollywood Boulevard Tract (MP 8 – 176)
LOTS: 12 - 16
LOCATION: 1000-1006 N. Seward St., 6565 W. Romain St., 1003, 1007 & 1013 N. Hudson Avenue

CURRENT REFERENCE REPORT/LETTER(S)	REPORT No.	DATE(S) OF DOCUMENT	PREPARED BY
Geology/Soils Report	W1084-06-01	05/22/2020	GeoCon West Inc.

The Grading Division of the Department of Building and Safety has reviewed the referenced report. The City Navigate LA Maps and aerial pictometry were utilized during review to view the proposed construction area and confirm the legal descriptions with addresses provided. Per the report and Navigate LA, the site is occupied currently by existing buildings and an asphalt-paved parking lot.

The northwest portion of the site is located adjacent to an existing, multi-level, off-site building (at least six levels) with associated perimeter structures. The northeast portion is located adjacent to off-site, residential apartments (two stories) and associated perimeter structures. The respective west, south and east sides of the property adjoin Seward Street, Romaine Street and Hudson Avenue.

As clarified (top of pg. 2 in the 05/22/2020 report), Figure 2 is the current site plan with cross section A-A. As shown and described, a new building is proposed. The proposed building will be fourteen (14) levels with four (4), subterranean levels. Excavations to depths of about forty-six (46) feet and retaining walls with heights of about forty (40) feet are currently anticipated.

Exploration provided to an approximate depth of sixty (60) feet, showed that Quaternary alluvium underlies artificial fill. The fill was observed to a maximum depth of four (4) feet. Groundwater was reported at approximate depths of eighteen (18) and twenty-six (26) feet below ground surface, with the historic high groundwater level cited at an approximate level/depth of eighteen (18) feet. Per the report, groundwater should be anticipated (pgs. 8 & 9) during excavations for shoring piles/elevator shafts and the building construction. Recommendations for temporary de-watering measures were included (pg. 15).

The City of Los Angeles has adopted the new 2020 Los Angeles Building Code (LABC). The 2020 LABC requirements apply to all projects where the permit application submittal date is after January 1, 2020.The referenced report is acceptable, provided the following conditions are complied with:

1. A stormwater infiltration system is not recommended for this site (pg. 10) and is therefore not approved. Site water (see recommendations provided on pgs. 39 and 40) shall be conveyed in non-

erosive devices to the street or other approved location in a manner acceptable to the LADBS and the Department of Public Works.

2. Obtain approval from the Dept. of Public Works, Bureau of Engineering, Development Services/Permits Program for the proposed construction and development.

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

4. Prior to excavation, an initial inspection shall be called at which time the sequence of construction, shoring pile and other excavation locations, grading work, protection fences and dust and traffic control will be scheduled. These shall be performed under the inspection/approval of the soils engineer and deputy grading inspector.

5. A grading permit shall be obtained.

6. As recommended, existing fill or loose, soft, disturbed alluvium shall not be used for support of foundations, structural slabs or future structural fill. The recommendations provided in the section section titled "Grading" on pgs. 15 – 18 and the sections titled "Exterior Concrete Slabs-on-Grade" and "Preliminary Pavement Recommendations", shall be incorporated into the final plans and implemented.

7. As concluded, recommended and specified (section 7.1.4 on pg. 8 and section 7.8 on pgs. 19 – 21), the proposed building shall be supported on a reinforced concrete mat foundation system that bears on competent alluvium, as approved by inspection by the geologist and soil engineer.

8. This letter approves exclusively the option in which the structure is designed to withstand hydrostatic pressures, as a measure to control groundwater under permanent conditions.

9. The proposed subterranean structure shall be supported on a mat foundation designed to resist uplift hydrostatic pressures that would develop due to the historic high groundwater level at a depth of 18 feet below the existing ground surface, as recommended on page 20 - 21 of the report.

10. The below-grade building walls shall be designed to resist the hydrostatic pressure that would develop if the groundwater level rose to the historic high groundwater level of 18 feet below the existing ground surface, as recommended. For that portion of the retaining walls above the historically-high groundwater level, a subdrain system may be installed. If the subdrain system is not installed above the historically-high groundwater level, then the walls shall be designed for the full hydrostatic pressure from top to bottom, as recommended.

11. The LABC Soil Site Class Type (pg. 10) underlying the site is D. All other seismic design parameters shall be reviewed by LADBS building plan check.

12. Retaining walls shall be designed for the minimum lateral earth pressures recommended and specified including the seismic loads (pgs. 24 – 28 in the 05/22/2020 report and Retaining Wall Drain Details Fig.'s 7 and 8). All surcharge loads shall be included into the design.

13. All retaining walls shall be provided with a standard surface back-drain system and all drainage conducted to the street or other approved location in non-erosive devices and in a manner acceptable to the LADBS and the Department of Public Works.

14. Basement and other walls in which horizontal movement is restricted at the top, shall be designed for at-rest pressures as specified (1610.1). All surcharge loads shall be included into the design.

15. With the exception of retaining walls designed for hydrostatic pressure, all retaining walls shall be provided with a subdrain system to prevent possible hydrostatic pressure behind the wall. Prior to issuance of any permit, the retaining wall subdrain system recommended in the soils report shall be incorporated into the foundation plan which shall be reviewed and approved by the soils engineer of record (1805.4).

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

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

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

19. All recommendations of the referenced report/s which are in addition to or more restrictive than the conditions contained herein shall be incorporated into the plans.

20. Final plans based on a recent licensed survey for proposed construction and grading purposes, shall include but not be limited to showing by labeling, the following:

    - existing off-site structures within 15 feet of the property boundaries near the construction/grading area;
    - the entire length of all retaining walls proposed;
    - areas where shoring as recommended will be utilized;
    - shoring pile locations proposed.

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

22. The geologist and soils engineer shall review and approve the detailed plans prior to issuance of any permits. This approval shall be by signature on the plans which clearly indicates that the geologist and soils engineer have reviewed the plans prepared by the design engineer and that the plans include the recommendations in their reports.

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

24. Temporary excavations shall be shored, supported, performed and monitored as recommended and specified (section 7.19 on pgs. 28 – 39 in the 05/22/2020 report). This shall include restricting the vertical height in <u>*un-surcharged*</u> excavations to five (5) feet, with portions exceeding this specified vertical height sloped to a horizontal to vertical slope gradient of 1:1 or flatter to a height of twelve (12) feet, or the excavation shall be shored. <u>*Surcharged*</u> excavations shall be shored.

    <u>Note</u>: Support shall be considered removed from existing adjacent off-site structures or property (i.e., they are <u>*surcharging*</u> the excavation), <u>*if*</u> they are located within a horizontal distance from the top of the excavation equal to the depth of the excavation.

25. Shoring shall be designed for the minimum lateral earth pressures as specified (starting on pg. 32 in the 05/22/2020 report); all surcharge loads shall be included into the design.

26. Prior to the issuance of the permits, the soils engineer and the structural designer shall evaluate the surcharge loads used in the report calculations for the design of shoring and retaining wall/s. If the surcharge loads used do not conform to the actual surcharge loads, the soil engineer shall submit a supplementary report with revised recommendations to the Department for approval.

27. The soils engineer shall review and approve the shoring plans prior to issuance of the permit (3307.3.2).

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

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

30. Prior to the placing of compacted fill, a representative of the consulting soils engineer shall inspect and approve the bottom excavations. The representative shall post a notice on the job site for the LADBS Grading Inspector and the Contractor stating that the soil inspected meets the conditions of the report, but that no fill shall be placed until the LADBS Grading Inspector has also inspected and approved the bottom excavations. A written certification to this effect shall be filed in the final compaction report filed with the Grading Division of the Department. All fill shall be placed under the inspection and approval of the soils engineer. A compaction report together with the approved soil report and Department approval letter shall be submitted to the Grading Division of the Department upon completion of the compaction. The engineer's certificate of compliance shall include the grading permit number and the legal description as described in the permit.

31. Prior to the pouring of concrete, a representative of the consulting soils engineer shall inspect and approve the foundation excavations. The representative shall post a notice on the job site for the LADBS Building Inspector and the Contractor stating that the work so inspected meets the conditions of the report, but that no concrete shall be poured until the LADBS Building Inspector has also inspected and approved the foundation excavations. A written certification to this effect shall be filed with the Department upon completion of the work.

FOR

STEPHEN DAWSON
Engineering Geologist II

DAN RYAN EVANGELISTA
Structural Engineering Associate III

SD/DRE:sd/dre
Log No. 113348
213-482-0480

cc: Yesenia Prieto – Plus Development Group (Applicant)
GeoCon West Inc.
LA District Office

CITY OF LOS ANGELES
DEPARTMENT OF BUILDING AND SAFETY
Grading Division

District	Log No.

APPLICATION FOR REVIEW OF TECHNICAL REPORTS

INSTRUCTIONS

A. Address all communications to the Grading Division, LADBS, 221 N. Figueroa St., 12th Fl., Los Angeles, CA 90012
Telephone No. (213)482-0480.
B. Submit two copies (three for subdivisions) of reports, one "pdf" copy of the report on a CD-Rom or flash drive, and one copy of application with items "1" through "10" completed.
C. Check should be made to the City of Los Angeles.

1. LEGAL DESCRIPTION
Tract: White and Newby's Hollywood Boulevard
Block: ____ Lots: 12-16

2. PROJECT ADDRESS:
1000-1006 Seward St, 6565 W Romain St, 1003, 1007, & 1013 N Hudson Ave

3. OWNER: 39 South LLC
Address: 4525 Wilshire Blvd #150
City: Los Angeles Zip: 90010
Phone (Daytime): 708-514-4756

4. APPLICANT Yesenia Prieto
Address: 4525 Wilshire Blvd #150
City: Los Angeles Zip: 90010
Phone (Daytime): 323-921-8372
E-mail address: yessie@plusdevelopmentgroup.com

5. Report(s) Prepared by: Geocon West Inc.
6. Report Date(s): 05/22/2020

7. Status of project: ☑ Proposed ☐ Under Construction ☐ Storm Damage
8. Previous site reports? ☐ YES if yes, give date(s) of report(s) and name of company who prepared report(s)

9. Previous Department actions? ☐ YES if yes, provide dates and attach a copy to expedite processing.
Dates:

10. Applicant Signature: [signature] Position: Agent

(DEPARTMENT USE ONLY)

REVIEW REQUESTED	FEES	REVIEW REQUESTED	FEES
☐ Soils Engineering		No. of Lots	
☐ Geology		No. of Acres	
☒ Combined Soils Engr. & Geol.	726.00	☐ Division of Land	
☐ Supplemental		Other	
☐ Combined Supplemental		☒ Expedite	363.00
☐ Import-Export Route		☐ Response to Correction	
Cubic Yards:		☐ Expedite ONLY	
		Sub-total	1089.00
		Surcharges	249.58
		TOTAL FEE	1338.58

Fee Due: 1338.58
Fee Verified By: ml Date: 6/2/20
(Cashier Use Only)

LA Department of Building and Safety
LA HANN 108011183 6/2/20
0 3:24:30 PM
GRADING REPORT $726.00
SYSTEMS DEV SURCH $43.56
GEN PLAN MAINT SURCH $50.82
DEV SERV CENTER SURCH $21.78
CITY PLAN SURCH $43.56
PLAN APPROVAL FEE $363.00
SYSTEMS DEV SURCH $21.78
GEN PLAN MAINT SURCH $25.41
DEV SERV CENTER SURCH $10.89
CITY PLAN SURCH $21.78
MISCELLANEOUS $10.00
Sub Total: $1,338.58
Receipt #: 0108177772
1000-1006 Seward St, 6565 W Romain St., 1003, 1007, & 1013 N Hud

ACTION BY:

THE REPORT IS: ☐ NOT APPROVED
☐ APPROVED WITH CONDITIONS ☐ BELOW ☐ ATTACHED

For Geology ____ Date ____

For Soils ____ Date ____