INITIAL STUDY

APPENDIX D.1: GEOTECHNICAL REPORT

GEOTECHNICAL INVESTIGATION

PROPOSED COMMERCIAL DEVELOPMENT
6103 WEST MELROSE AVENUE
LOS ANGELES, CALIFORNIA

TRACT: 4427, LOT: 21-23, ARB: 1 & 2

GEOCON
WEST, INC.

GEOTECHNICAL
ENVIRONMENTAL
MATERIALS

PREPARED FOR

BARDAS INVESTMENT GROUP
WEST HOLLYWOOD, CALIFORNIA

PROJECT NO. W1153-06-01

APRIL 28, 2020

GEOCON
WEST, INC.
GEOTECHNICAL ■ ENVIRONMENTAL ■ MATERIALS

Project No. W1153-06-01
April 28, 2020

Bardas Investment Group
c/o Mr. David Stafford
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690 E. Green Street, Suite 201
Pasadena, CA 91101

Subject: GEOTECHNICAL INVESTIGATION
PROPOSED COMMERCIAL DEVELOPMENT
6103 WEST MELROSE AVENUE
LOS ANGELES, CALIFORNIA
TRACT: 4427, LOTS: 21-23, ARB 1 & 2

Dear Mr. Stafford:

In accordance with your authorization of our proposal dated February 26, 2020, we have performed a geotechnical investigation for the proposed commercial development located at 6103 West Melrose Avenue 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 proposed design and construction. Based on the results of our investigation, it is our opinion that the site can be developed as proposed, provided the recommendations of this report are followed and implemented during design and construction.

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

Very truly yours,

GEOCON WEST, INC.

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GEOTECHNICAL INVESTIGATION

1. PURPOSE AND SCOPE

This report presents the results of a geotechnical investigation for the proposed commercial development located at 6103 West Melrose 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 March 16, 2020 by excavating two 8-inch diameter borings to depths of approximately 45½ feet and 50½ feet below the existing ground surface using a truck-mounted hollow-stem auger drilling machine. The approximate locations of the exploratory borings are depicted on the Site Plan (see Figure 2A). 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 6103 West Melrose Avenue in the City of Los Angeles, California. The subject property is currently occupied by a single-story commercial structure and paved surface parking areas north of the existing structure. The site is bounded by a public library and single-family residential structures to the west, by single-story commercial structures to the north, by Seward Street to the east, and by Melrose Avenue to the south. The site is relatively level with no pronounced highs or low. Surface water drainage at the site appears to be by sheet flow along the existing ground contours to the city streets.

It is our understanding that the proposed development will consist of a three-story office building over two levels of subterranean parking (see Site Plan and Cross Section, Figures 2A and 2B). Due to the preliminary nature of the project, formal plans depicting the proposed development are not available for inclusion in this report. It is assumed that the proposed subterranean parking levels will extend approximately 33 feet below the existing ground surface, including foundation depths.

Based on the preliminary nature of the design at this time, wall and column loads were not available. It is anticipated that column loads for the proposed structure will be up to 1,300 kips, and wall loads will be up to 13 kips per linear foot.

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 Pleistocene age old alluvial fan deposits consisting of interbedded sand and silt with lesser amounts of clay (California Geological Survey [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 2½ feet below existing ground surface. The artificial fill generally consists of dark brown clay with varying amounts of gravel and is characterized as slightly 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 Older Alluvium

The fill soils are underlain by Pleistocene age old alluvial fan deposits consisting of brown to dark brown, reddish brown, yellowish-brown or olive brown interbedded silty sand, clayey sand, sandy silt, sandy clay, and clay. The alluvium is characterized as primarily fine to medium-grained, slightly moist to moist, and medium dense to dense or stiff to hard.

5. GROUNDWATER

Based on a review of the Seismic Hazard Zone Report for the Hollywood Quadrangle (California Division of Mines and Geology [CDMG], 1998), the historically highest groundwater level in the area is approximately 15 feet beneath the ground surface. Groundwater information presented in this document is generated from data collected in the early 1900's to the late 1990s.

Perched groundwater was encountered in borings B1 and B2 at a depth of approximately 32 feet below the existing ground surface. Based on the historic high groundwater level in the site vicinity and the depth to groundwater encountered in our borings, groundwater may be encountered during construction. Also, it is not uncommon for groundwater levels to vary seasonally or for groundwater seepage conditions to develop where none previously existed, especially in impermeable fine-grained soils which are heavily irrigated or after seasonal rainfall. In addition, recent requirements for stormwater infiltration could result in shallower seepage conditions in the region. 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 within a state-designated Alquist-Priolo Earthquake Fault Zone (CGS, 2020a; 2020b; 2014) nor a city-designated Preliminary Fault Rupture Study Area (City of Los Angeles, 2020) for surface fault rupture hazards. No Holocene-active or pre-Holocene faults with the potential for surface fault rupture are known to pass directly beneath the site. Therefore, the potential for surface rupture due to faulting occurring beneath the site during the design life of the proposed development is considered low. However, the site is located in the seismically active Southern California region, and could be subjected to moderate to strong ground shaking in the event of an earthquake on one of the many active Southern California faults. The faults in the vicinity of the site are shown in Figure 3, Regional Fault Map.

The closest active fault to the site is the Hollywood Fault located approximately 1.3 miles to the north (CGS, 2014). Other nearby active faults are the Newport-Inglewood Fault Zone, the Santa Monica Fault, the Raymond Fault, and the Verdugo Fault located approximately 3.3 miles southwest, 3.7 miles west, 6.2 miles northeast, and 7.4 miles northeast of the site, respectively. The active San Andreas Fault Zone is located approximately 33 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	38	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	23	ENE
Landers	June 28, 1992	7.3	109	E
Big Bear	June 28, 1992	6.4	86	E
Northridge	January 17, 1994	6.7	15	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 Seismic Design Criteria

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

2019 CBC SEISMIC DESIGN PARAMETERS

Parameter	Value	2019 CBC Reference
Site Class	D	Section 1613.2.2
MCE_R Ground Motion Spectral Response Acceleration – Class B (short), S_S	2.073g	Figure 1613.2.1(1)
MCE_R Ground Motion Spectral Response Acceleration – Class B (1 sec), S_1	0.742g	Figure 1613.2.1(2)
Site Coefficient, F_A	1	Table 1613.2.3(1)
Site Coefficient, F_V	1.7*	Table 1613.2.3(2)
Site Class Modified MCE_R Spectral Response Acceleration (short), S_{MS}	2.073g	Section 1613.2.3 (Eqn 16-36)
Site Class Modified MCE_R Spectral Response Acceleration – (1 sec), S_{M1}	1.261g*	Section 1613.2.3 (Eqn 16-37)
5% Damped Design Spectral Response Acceleration (short), S_{DS}	1.382g	Section 1613.2.4 (Eqn 16-38)
5% Damped Design Spectral Response Acceleration (1 sec), S_{D1}	0.841g*	Section 1613.2.4 (Eqn 16-39)

Note:

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

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

ASCE 7-16 PEAK GROUND ACCELERATION

Parameter	Value	ASCE 7-16 Reference
Mapped MCE_G Peak Ground Acceleration, PGA	0.888g	Figure 22-7
Site Coefficient, F_{PGA}	1.1	Table 11.8-1
Site Class Modified MCE_G Peak Ground Acceleration, PGA_M	0.976g	Section 11.8.3 (Eqn 11.8-1)

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

Deaggregation of the MCE peak ground acceleration was performed using the USGS online Unified Hazard Tool, 2014 Conterminous U.S. Dynamic edition (v4.2.0). The result of the deaggregation analysis indicates that the predominant earthquake contributing to the MCE peak ground acceleration is characterized as a 6.79 magnitude event occurring at a hypocentral distance of 8.03 kilometers from the site.

Deaggregation was also performed for the Design Earthquake (DE) peak ground acceleration, and the result of the analysis indicates that the predominant earthquake contributing to the DE peak ground acceleration is characterized as a 6.69 magnitude occurring at a hypocentral distance of 11.7 kilometers from the site.

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

6.4 Liquefaction Potential

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

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

The State of California Seismic Hazard Zone Map for the Hollywood Quadrangle (CDMG, 1999) 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 Safety Element (Leighton, 1990) indicates that the site is not located within an area identified as having a potential for liquefaction. The site is underlain by Pleistocene age alluvial sediments that are considered stiff to hard or medium dense to dense and are not prone to liquefaction. Based on these considerations, it is our opinion that the potential for liquefaction and associated ground deformations beneath the site is very low.

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 City of Los Angeles Hillside Grading Area or a Hillside Ordinance Area (City of Los Angeles, 2020). Also, the site is not located within an area identified as having a potential for seismic slope instability (CDMG, 1999). There are no known landslides near the site, nor is the site in the path of any known or potential landslides. Therefore, the potential for slope stability hazards to adversely affect the proposed development is considered low.

6.6 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, 2020; FEMA, 2020).

6.8 Oil Fields & Methane Potential

Based on a review of the California Geologic Energy Management Division (CalGEM) Well Finder website, the site is located immediately north of the northern border of the Salt Lake Oil Field. The nearest active well to the site is Well S-76 operated by E&B Natural Resources Management, located approximately 2.6 miles to the west-southwest (CalGEM, 2020). However, due to the voluntary nature of record reporting by the oil well drilling companies, wells may be improperly located or not shown on the location map and undocumented wells could be encountered during construction. Any wells encountered will need to be properly abandoned in accordance with the current requirements of the CalGEM.

The site is located within the boundaries of a city-designated Methane Zone (City of Los Angeles, 2020). Prior to approval of the proposed project, the City of Los Angeles will require a site-specific methane study be performed to evaluate the potential for methane and other volatile gases to impact the proposed development. We recommend that a qualified methane consultant be retained to perform the study and provide mitigation measures as necessary.

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 2½ feet of existing artificial fill was encountered during the site investigation. The existing fill encountered is believed to be the result of past grading and construction activities at the site. Deeper fill may exist in other areas of the site that were not directly explored. Demolition of the existing structures that occupy the site is anticipated to disturb the upper few feet of existing site soils. It is our opinion that the existing fill, in its present condition, is not suitable for direct support of proposed foundations or slabs. The existing fill and site soils are suitable for re-use as engineered fill provided the recommendations in the Grading section of this report are followed (see Section 7.5).

7.1.3 Excavations for the subterranean levels is anticipated to penetrate through the existing artificial fill and expose competent alluvium throughout the excavation bottom. Depending on the season, the soils at the excavation bottom may be moist and may require stabilization measures. Recommendations for stabilization procedures are provided in the *Grading* section of this report (see Section 7.5).

7.1.4 Groundwater was encountered at a depth of 32 feet below existing ground surface. Excavation for the construction of the lowest subterranean level is anticipated to extend to a depth of 35 feet below ground surface, including foundation excavations and dewatering systems. Based on these considerations, groundwater may be encountered near the excavation bottom. Due to the depth of the proposed excavation and the potential for seasonal fluctuation in the groundwater level, temporary dewatering measures may be required to mitigate groundwater during excavation and construction. Recommendations for temporary dewatering are discussed in Section 7.4 of this report.

7.1.5 Based on a historic high groundwater depth of 15 feet below the existing ground surface, the proposed structure must be designed for hydrostatic pressure for any portion of the structure below a depth of 15 feet. The hydrostatic design will result in uplift forces on the structure that must be resisted by counterweight or structural design measures. The recommended floor slab uplift pressure to be used in design would be 62.4(H) in units of pounds per square foot (psf), where "H" is the height of the water above the bottom of the foundation in feet. If the proposed structure does not provide sufficient dead load to resist the buoyant forces then uplift mitigation will be required.

7.1.6 Based on these considerations, it is recommended that the proposed structure be supported on a reinforced concrete mat foundation system deriving support in undisturbed alluvial soils found at or below a depth of 30 feet. A mat foundation is more accommodating to subgrade stabilization, waterproofing, hydrostatic design, and allows for more efficient construction when performed in conjunction with a methane mitigation system. In addition, the bottom of the mat can be shaped to channel the methane simplifying the passive mitigation system. A qualified methane consultant should be retained for the design of the mitigation system. Recommendations for the design of a mat foundation system are provided in Section 7.7 of this report.

7.1.7 The concrete ramp for the subterranean level may bear directly on the undisturbed alluvium at the excavation bottom. Any soils that are disturbed should be properly compacted for ramp support. Where necessary, the existing artificial fill and undisturbed alluvium are suitable for re-use as an engineered fill beneath the ramp provided the procedures outlined in the *Grading* section of this report are followed (see Section 7.6).

7.1.8 Excavations up to 35 feet in vertical height are anticipated for construction of the subterranean 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 proposed subterranean level will likely require sloping and shoring measures in order to provide a stable excavation. Where shoring is required, it is recommended that a soldier pile shoring system be utilized. In addition, where the proposed excavation will be deeper than and adjacent to an offsite structure, the proposed shoring should be designed to resist the surcharge imposed by the adjacent offsite structure. Recommendations for shoring are provided in Section 7.20 of this report.

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

7.1.10 Foundations for small outlying structures, such as block walls up to 6 feet in height, planter walls or trash enclosures, which will not be tied to the proposed structure, may be supported on conventional foundations deriving support on a minimum of 12 inches of newly placed engineered fill which extends laterally at least 12 inches beyond the foundation area. Where excavation and compaction cannot be performed or is undesirable, foundations may derive support directly in the competent undisturbed alluvium at and below a depth of 24 inches. If the soils exposed in the excavation bottom are soft or loose, compaction of the soils will be required prior to placing steel or concrete. Compaction of the foundation excavation bottom is typically accomplished with a compaction wheel or mechanical whacker and must be observed and approved by a Geocon representative.

7.1.11 Where new paving is to be placed, it is recommended that all existing fill soils and soft soils be excavated and properly compacted for paving support. The client should be aware that excavation and compaction of all existing fill in the area of new paving is not required, however, paving constructed over existing uncertified fill or unsuitable soils may experience increased settlement and/or cracking, and may therefore have a shorter design life and increased maintenance costs. As a minimum, the upper 12 inches of soil should be scarified and properly compacted. Paving recommendations are provided in the *Preliminary Pavement Recommendations* section of this report (see Section 7.13).

7.1.12 Based on the current groundwater levels, as well as the fine-grained nature of the underlying site soils, stormwater infiltration is not recommended for this project. It is suggested that stormwater be retained, filtered and discharged in accordance with the requirements of the local governing agency.

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

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

7.2 Soil and Excavation Characteristics

7.2.1 The in-situ soils can be excavated with moderate effort using conventional excavation equipment. Some caving should be anticipated in unshored excavations, especially where granular soils are encountered.

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 existing adjacent improvements.

7.2.3 All onsite excavations must be conducted in such a manner that potential surcharges from existing structures, construction equipment, and vehicle loads are resisted. The surcharge area may be defined by a 1:1 projection down and away from the bottom of an existing foundation or vehicle load. Penetrations below this 1:1 projection will require special excavation measures such as sloping or shoring. Excavation recommendations are provided in the *Temporary Excavations* section of this report (see Section 7.19).

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

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 encountered at foundation depths are considered “corrosive” with respect to corrosion of buried ferrous metals on site. The results are presented in Appendix B (Figure B15) and should be considered for design of underground structures. Due to the corrosive potential of the soils, it is recommended that PVC, ABS or other approve plastic piping be utilized in lieu of cast-iron when in direct contact with the site soils.

7.3.2 Laboratory tests were performed on representative samples of the site soils to measure the percentage of water-soluble sulfate content. Results from the laboratory water-soluble sulfate tests are presented in Appendix B (Figure B15) and indicate that the on-site materials possess “S0” sulfate exposure 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 and mitigation. If corrosion sensitive improvements are planned, it is recommended that a corrosion engineer be retained to evaluate corrosion test results and incorporate the necessary precautions to avoid premature corrosion of buried metal pipes and concrete structures in direct contact with the soils.

7.4 Temporary Dewatering

7.4.1 Groundwater was encountered during site exploration at a depth of approximately 32 feet below ground surface. Based on the conditions encountered at the time of exploration, groundwater nay be encountered during construction activities. The depth to groundwater at the time of construction can be further verified with a test well or during initial shoring pile installation. If groundwater is present above the depth of the subterranean level, temporary dewatering will be necessary to maintain a safe working environment during excavation and construction activities.

7.4.2 It is recommended that a qualified dewatering consultant be retained to design the dewatering system. Recommendations for design flow rates for the temporary dewatering system should be determined by a qualified contractor or dewatering consultant. 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 consider 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 excavation of site soils for the subterranean levels, foundations, and utility trenches, as well as placement of backfill for walls 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 and alluvium encountered during exploration are suitable for re-use as engineered fill, provided any encountered oversize material (greater than 6 inches) and any encountered deleterious debris are 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 30 feet.

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

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

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

7.5.9 Foundations for small outlying structures, such as block walls up to 6 feet in height, planter walls or trash enclosures, which will not be tied to the proposed structure, may be supported on conventional foundations deriving support on a minimum of 12 inches of newly placed engineered fill which extends laterally at least 12 inches beyond the foundation area. Where excavation and compaction cannot be performed or is undesirable, foundations may derive support directly in the competent undisturbed alluvium at and below a depth of 24 inches, and should be deepened as necessary to maintain a minimum 12-inch embedment into the recommended bearing materials. If the soils exposed in the excavation bottom are soft or loose, compaction of the soils will be required prior to placing steel or concrete. Compaction of the foundation excavation bottom is typically accomplished with a compaction wheel or mechanical whacker and must be observed and approved by a Geocon representative.

7.5.10 Where new paving is to be placed, it is recommended that all existing 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 fill and soft soils in the area of new paving is not required; however, paving constructed over existing uncertified fill or unsuitable alluvial soil 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 two percent above 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 7.13).

7.5.11 Although not anticipated for this project, all imported fill shall be observed, tested, and approved by Geocon West, Inc. prior to bringing soil to the site. Rocks larger than 6 inches in diameter shall not be used in the fill. Import soils should have any expansion index of less than 50 and corrosivity properties that are less than or equal to that of existing site soils, see Appendix B (Figure B15).

7.5.12 Utility trenches should be properly backfilled in accordance with the requirements of the Green Book (latest edition). The pipe should be bedded with clean sands (Sand Equivalent greater than 30) to a depth of at least 1 foot over the pipe, and the bedding material must be inspected and approved in writing by the Geotechnical Engineer (a representative of Geocon). The use of gravel is not acceptable unless used in conjunction with filter fabric to prevent the gravel from having direct contact with soil. The remainder of the trench backfill may be derived from onsite soil or approved import soil, compacted as necessary, until the required compaction is obtained. The use of minimum 2-sack slurry as backfill is also acceptable (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).

7.5.13 All trench and foundation excavation bottoms must be observed and approved in writing by the Geotechnical Engineer (a representative of Geocon), prior to placing bedding 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:

<u>Standard Requirements</u>

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.

<u>Requirements for CLSM that will be used for support of footings</u>

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 psi when tested on the 28^{th} 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 The proposed structure may be supported on a reinforced concrete mat foundation system deriving support in undisturbed alluvial soils found at or below a depth of 30 feet. Foundations should be deepened as necessary to penetrate through soft or unsuitable soils at the direction of the Geotechnical Engineer. All foundation excavations must be observed and approved in writing by the Geotechnical Engineer (a representative of Geocon), prior to placing steel or concrete.

7.7.2 The recommended maximum allowable bearing value is 4,000 psf. The allowable bearing pressure may be increased by up to one-third for transient loads due to wind or seismic forces.

7.7.3 It is recommended that a modulus of subgrade reaction of 150 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 30 feet. If the subgrade is stabilized in accordance with the recommendation of this report a modulus of subgrade reaction of 250 pci may be utilized. These values are unit values for use with a one-foot square footing. The modulus should be reduced in accordance with the following equation when used with larger foundations:

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

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 City of Los Angeles Building Code requires that the structure be designed for the historically high groundwater level, which is approximately 15 feet below the existing ground surface. Any portion of the structure below a depth of 15 feet must be designed for hydrostatic pressure based on the historic high groundwater level of 15 feet below the ground surface. The hydrostatic design will result in uplift forces on the slab that that must be resisted by structural design. The recommended floor slab uplift pressure to be used in design would be 62.4(H) in units of pounds per square foot (psf), where “H” is the height of the water above the bottom of the foundation in feet.

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

7.7.9 The client should be aware that if a methane barrier is installed to envelope the structure, the installation of a waterproofing barrier should not be necessary since it is a redundant system. 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.

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

7.8 Foundation Settlement

7.8.1 The maximum expected settlement for the structure supported on a mat foundation system deriving support in alluvium with a maximum allowable bearing pressure of 4,000 psf is estimated to be less than ¾ inch and occur below the heaviest loaded structural element. Differential settlement is not expected to exceed ½ inch between the center and corner of the mat.

7.8.2 Once the design and foundation loading configurations for the proposed structure proceed to a more finalized plan, the estimated settlements presented in this report should be reviewed and revised, if necessary. If the final foundation loading configurations are greater than the assumed loading conditions, the potential for settlement should be reevaluated by this office.

7.9 Uplift Resistance

7.9.1 Foundation uplift may be resisted by the weight of structure, as well as friction along the sides of foundations. If additional uplift resistance is required, the perimeter shoring piles may be utilized provided the toes of the piles are poured with structural concrete and are designed as permanent piles. Recommendations for the design of shoring piles are provided in Section 7.20.

7.9.2 Uplift resistance may also be generated by additional piles constructed within the interior of the structure. It is recommended that post-grouted friction piles be utilized. The uplift capacity may be determined using a frictional resistance of 240 psf (⅔ the downward capacity, adjusted for buoyancy).

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

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

7.10 Miscellaneous Foundations

7.10.1 Foundations for small outlying structures, such as block walls up to 6 feet in height, planter walls or trash enclosures which will not be tied to the proposed structure may be supported on conventional foundations bearing on a minimum of 12 inches of newly placed engineered fill which extends laterally at least 12 inches beyond the foundation area. Where excavation and compaction cannot be performed or is undesirable, such as adjacent to property lines, foundations may derive support in the undisturbed alluvium at and below a depth of 24 inches, and should be deepened as necessary to maintain a 12 inch embedment in to the recommended bearing materials.

7.10.2 If the soils exposed in the excavation bottom are soft, compaction of the soft soils will be required prior to placing steel or concrete. Compaction of the foundation excavation bottom is typically accomplished with a compaction wheel or mechanical whacker and must be observed and approved by a Geocon representative. Miscellaneous foundations may be designed for a bearing value of 1,500 psf and should be a minimum of 12 inches in width, 24 inches in depth below the lowest adjacent grade and 12 inches into the recommended bearing material. The allowable bearing pressure may be increased by up to one-third for transient loads due to wind or seismic forces.

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

7.11 Lateral Design

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

7.11.2 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 250 pcf with a maximum earth pressure of 2,500 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 125 pcf with a maximum earth pressure of 1,250 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.

7.12 Exterior Concrete Slabs-on-Grade

7.12.1 Exterior concrete slabs-on-grade at the ground surface subject to vehicle loading should be designed in accordance with the recommendations in the *Preliminary Pavement Recommendations* section of this report (Section 7.13).

7.12.2 Exterior slabs, not subject to traffic loads, should be at least 4 inches thick and reinforced with No. 3 steel reinforcing bars placed 18 inches on center in both horizontal directions, positioned near the slab midpoint. Prior to construction of slabs, the upper 12 inches of subgrade should be moistened to two 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 10 feet and should be constructed using saw-cuts or other methods as soon as practical following concrete placement. Crack control joints should extend a minimum depth of 1/4 the slab thickness. The project structural engineer should design construction joints as necessary.

7.12.3 The moisture content of the slab subgrade should be maintained and sprinkled as necessary to maintain a moist condition as would be expected in any concrete placement.

7.12.4 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.

7.13 Preliminary Pavement Recommendations

7.13.1 Where new paving is to be placed, it is recommended that all existing fill and soft alluvium materials be excavated and properly compacted for paving support. The client should be aware that excavation and compaction of all existing artificial fill and soft alluvium in the area of new paving is not required; however, paving constructed over existing uncertified fill or unsuitable alluvium material may experience increased settlement and/or cracking, and may therefore have a shorter design life and increased maintenance costs. As a minimum, the upper 12 inches of paving subgrade should be scarified, moisture conditioned to two 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).

7.13.2 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.

7.13.3 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

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

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

7.13.6 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.

7.14 Retaining Wall Design

7.14.1 The recommendations presented below are generally applicable to the design of rigid concrete or masonry retaining walls having a maximum height of 30 feet. In the event that walls higher than 30 feet are planned, Geocon should be contacted for additional recommendations.

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

7.14.3 Retaining walls with a level backfill surface that are not restrained at the top should be designed utilizing a triangular distribution of pressure (active pressure). Restrained walls are those that are not allowed to rotate more than 0.001H (where H equals the height of the retaining portion of the wall in feet) at the top of the wall. Where walls are restrained from movement at the top, walls may be designed utilizing a triangular distribution of pressure (at-rest pressure). The table below presents recommended pressures to be used in retaining wall design, assuming that proper drainage will be maintained. A retaining wall calculation is provided on Figure 5.

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 30	45	63

7.14.4 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 cantilever and restrained undrained walls is 95 pcf. The value includes hydrostatic pressures plus buoyant lateral earth pressures.

7.14.5 The wall pressures provided above assume that the proposed retaining walls will support relatively undisturbed soils. If sloping techniques are to be utilized for construction of proposed walls, which would result in a wedge of engineered fill behind the retaining walls, revised earth pressures may be required, especially if the wall backfill does not consist of the existing onsite soils. This should be evaluated once the use of sloping measures is established and once the geotechnical characteristics of the engineered backfill soils can be further evaluated.

7.14.6 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.

7.14.7 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.

7.14.8 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.

7.14.9 In addition to the recommended earth pressure, the upper 10 feet of the subterranean wall adjacent to the street and parking lot 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.

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

7.15 Dynamic (Seismic) Lateral Forces

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

7.15.2 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. The earth pressure is based on half of two-thirds of PGA_M calculated from ASCE 7-16 Section 11.8.3.

7.16 Retaining Wall Drainage

7.16.1 Retaining walls not designed for hydrostatic pressures 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 6). 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.

7.16.2 As an alternative, a plastic drainage composite such as Miradrain or equivalent may be installed in continuous, 4-foot wide columns along the entire back face of the wall, at 8 feet on center. The top of these drainage composite columns should terminate approximately 18 inches below the ground surface, where either hardscape or a minimum of 18 inches of relatively cohesive material should be placed as a cap (see Figure 7). 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.

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

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

7.17 Elevator Pit Design

7.17.1 The elevator pit slab and retaining wall should be designed by the project structural engineer. Elevator pits may be designed in accordance with the recommendations in the *Mat Foundation Design* and *Retaining Wall Design* sections of this report (see Sections 7.7 and 7.14).

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

7.17.3 The City of Los Angeles Building Code requires that the structure be designed for the historically high groundwater level, which is approximately 15 feet below the existing ground surface. Any portion of the structure below a depth of 15 feet must be designed for hydrostatic pressure based on the historic high groundwater level of 15 feet below the ground surface. The hydrostatic design will result in uplift forces on the slab that that must be resisted by structural design. The recommended floor slab uplift pressure to be used in design would be 62.4(H) in units of pounds per square foot (psf), where "H" is the height of the water above the bottom of the foundation in feet.

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

7.18 Elevator Piston

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

7.18.2 Casing may be required if caving is experienced in the drilled excavation. 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.

7.18.3 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.

7.19 Temporary Excavations

7.19.1 Excavations up to 35 feet in height are anticipated for excavation and construction of the proposed subterranean level 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.

7.19.2 Vertical excavations greater than 5 feet will require sloping and/or shoring measures in order to provide a stable excavation. Where sufficient space is available, temporary unsurcharged embankments could be sloped back at a uniform 1:1 slope gradient or flatter, up to a maximum of 12 feet in height. A uniform slope does not have a vertical portion. Where space is limited, shoring measures will be required. *Shoring* data is provided in Section 7.20 of this report.

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

7.20 Shoring – Soldier Pile Design and Installation

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

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

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

7.20.4 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. 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 7.14).

7.20.5 Drilled cast-in-place soldier piles should be placed no closer than 3 diameters on center. The minimum diameter of the piles is 18 inches. Structural concrete should be used for the soldier piles below the excavation; lean-mix concrete may be employed above that level. As an alternative, lean-mix concrete may be used throughout the pile where the reinforcing consists of a wideflange section. The slurry must be of sufficient strength to impart the lateral bearing pressure developed by the wideflange section to the soil. For design purposes, an allowable passive value for the soils below the bottom plane of excavation may be assumed to be 250 psf per foot above the groundwater elevation and 125 psf per foot (value reduced for buoyant forces) below the groundwater elevation. Where piles are installed by vibration techniques, the passive pressure may be assumed to mobilize across a width equal to the two times the dimension of the beam flange. The allowable passive value may be doubled for isolated piles spaced a minimum of three times the pile diameter. To develop the full lateral value, provisions should be implemented to assure firm contact between the soldier piles and the undisturbed alluvium.

7.20.6 Groundwater was encountered at a depth of 32 feet below the existing ground surface. If more than 6 inches of water is present in the bottom of the excavation, a tremie is required 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.

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

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

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

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

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

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

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

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

7.20.15 The frictional resistance between the soldier piles and retained soil may be used to resist the vertical component of the load. The coefficient of friction may be taken as 0.35 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 360 psf per foot (value has been reduced for buoyant forces).

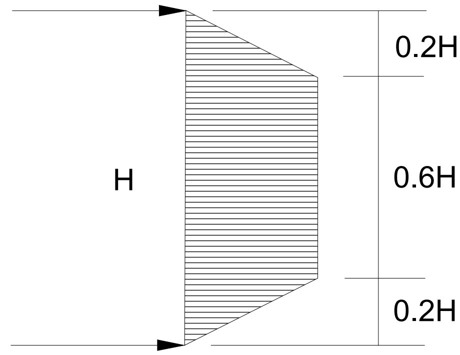
7.20.16 Due to the nature of the site soils, it is expected that continuous lagging between soldier piles will be required. However, it is recommended that the exposed soils be observed by the Geotechnical Engineer (a representative of Geocon West, Inc.), to verify the presence of any competent, cohesive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

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

7.20.18 For the design of shoring, it is recommended that an equivalent fluid pressure based on the following table, be utilized for design. A trapezoidal distribution of lateral earth pressure may be used where shoring will be restrained by bracing or tie-backs. The recommended active and trapezoidal pressure are provided in the following table. A diagram depicting the trapezoidal pressure distribution of lateral earth pressure is provided below the table and a calculation of shoring pressure is provided on Figure 8.

HEIGHT OF SHORING (FEET)	**EQUIVALENT FLUID PRESSURE (Pounds Per Cubic Foot) (ACTIVE PRESSURE)**	**EQUIVALENT FLUID PRESSURE Trapezoidal (Where H is the height of the shoring in feet)**
Up to 35	38	24H

Trapezoidal Distribution of Pressure



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

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

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

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

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.

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

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

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

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.

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

7.20.23 It is difficult to accurately predict the amount of deflection of a shored embankment. It should be realized that some deflection will occur. It is recommended that the deflection be minimized to prevent damage to existing structures and adjacent improvements. Where public rights-of-way 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.

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

7.20.25 Due to 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.

7.21 Temporary Tie-Back Anchors

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

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

- 7 feet below the top of the excavation – 900 pounds per square foot
- 15 feet below the top of the excavation – 750 pounds per square foot*
- 25 feet below the top of the excavation – 1,000 pounds per square foot*

7.21.3 Depending on the techniques utilized, and the experience of the contractor performing the installation, a maximum allowable friction capacity of 2.4 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.

7.22 Anchor Installation

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

7.23 Anchor Testing

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

7.23.2 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.

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

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

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

7.24 Internal Bracing

7.24.1 Rakers may be utilized to brace the soldier piles in lieu of tieback anchors. The raker bracing could be supported laterally by temporary concrete footings (deadmen) or by the permanent, interior footings. For design of such temporary footings or deadmen, poured with the bearing surface normal to rakers inclined at 45 degrees, a bearing value of 2,000 psf may be used, provided the shallowest point of the footing is at least one foot below the lowest adjacent grade. The structural engineer should review the shoring plans to determine if raker footings conflict with the structural foundation system. The client should be aware that the utilization of rakers could significantly impact the construction schedule due to their intrusion into the construction site and potential interference with equipment.

7.25 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 2019 CBC 1804.4 or other applicable standards. In addition, drainage should not be allowed to flow uncontrolled over any descending slope. Discharge from downspouts, roof drains and scuppers are not recommended onto unprotected soils within 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 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 (a representative of Geocon West, Inc.), prior to finalization to verify that the plans have been prepared in substantial conformance with the recommendations of this report and to provide additional analyses or recommendations.

LIMITATIONS AND UNIFORMITY OF CONDITIONS

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

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

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

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

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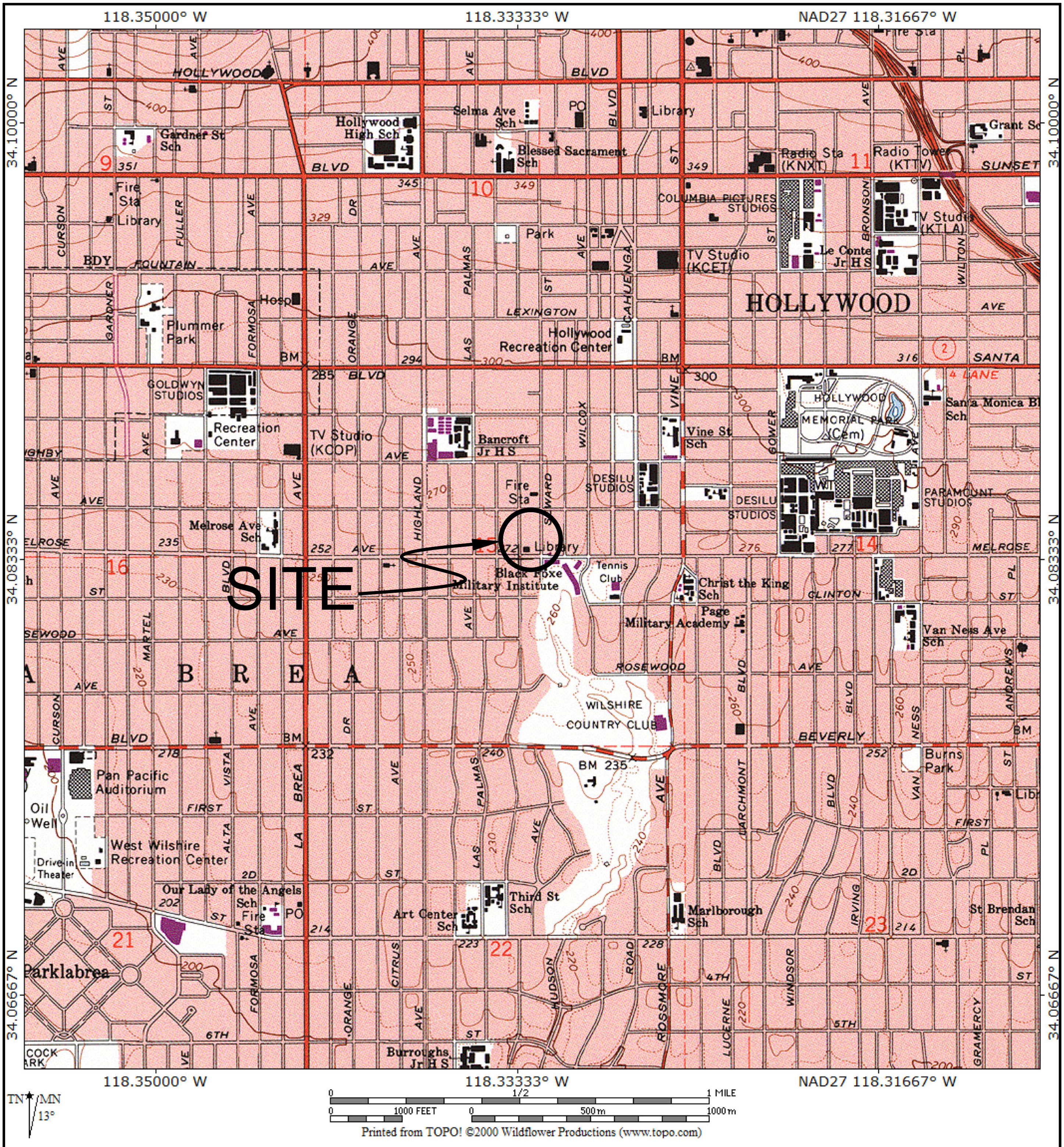

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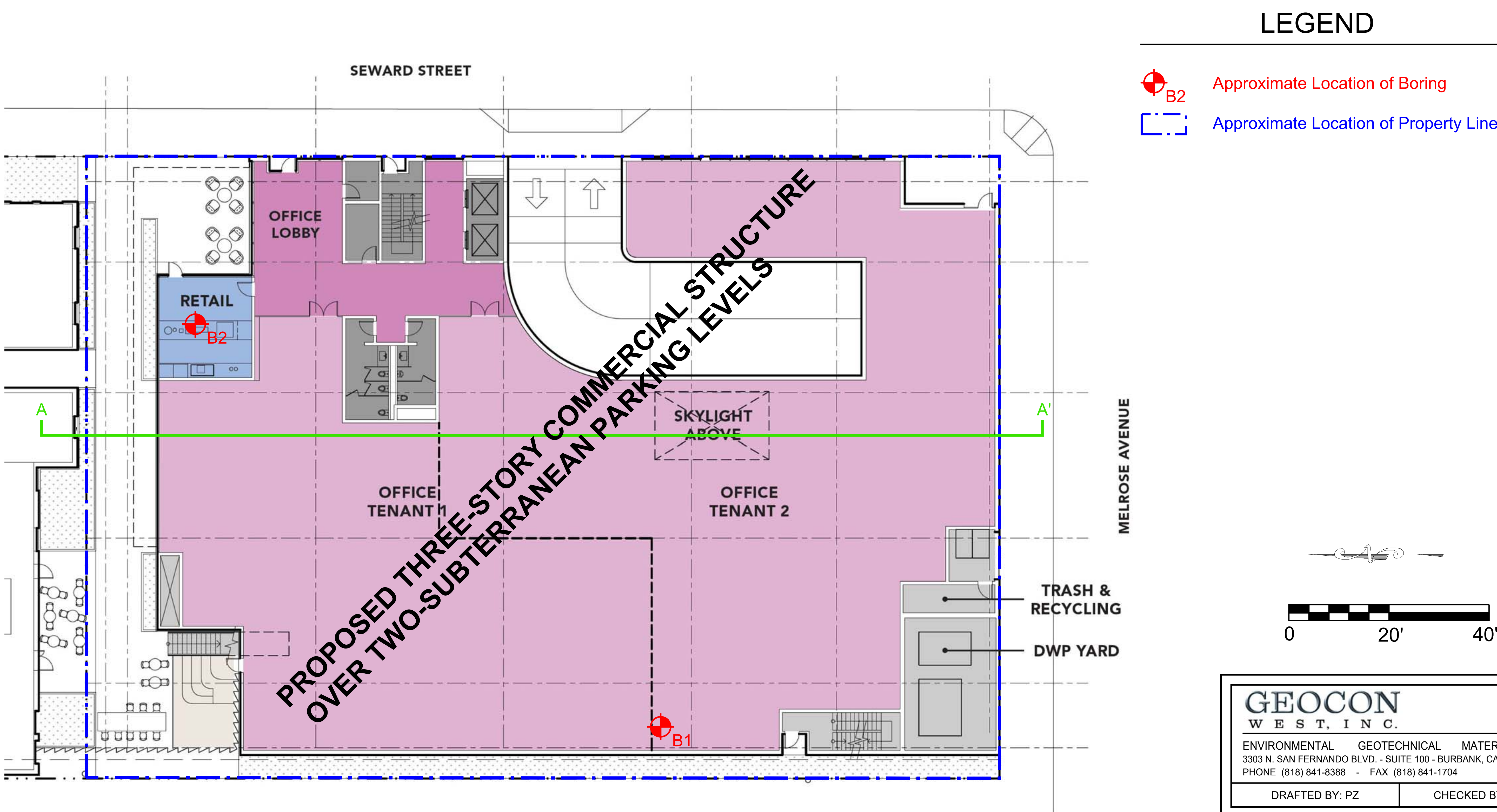
VICINITY MAP

6103 WEST MELROSE AVENUE
LOS ANGELES, CALIFORNIA

APR. 2020 | PROJECT NO. W1153-06-01 | FIG. 1



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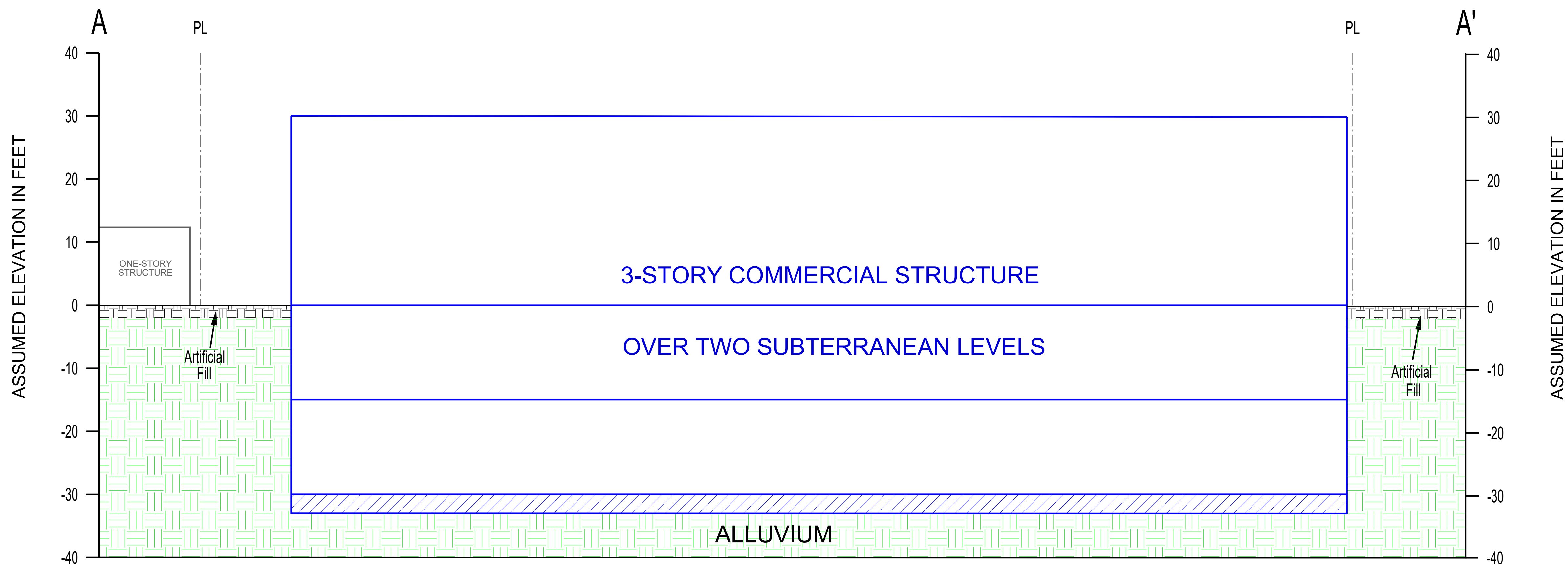
SITE PLAN

6103 WEST MELROSE AVENUE
LOS ANGELES, CALIFORNIA

APRIL 2020	PROJECT NO. W1153-06-01	FIG. 2A



SECTION A - A'

0 20' 40'

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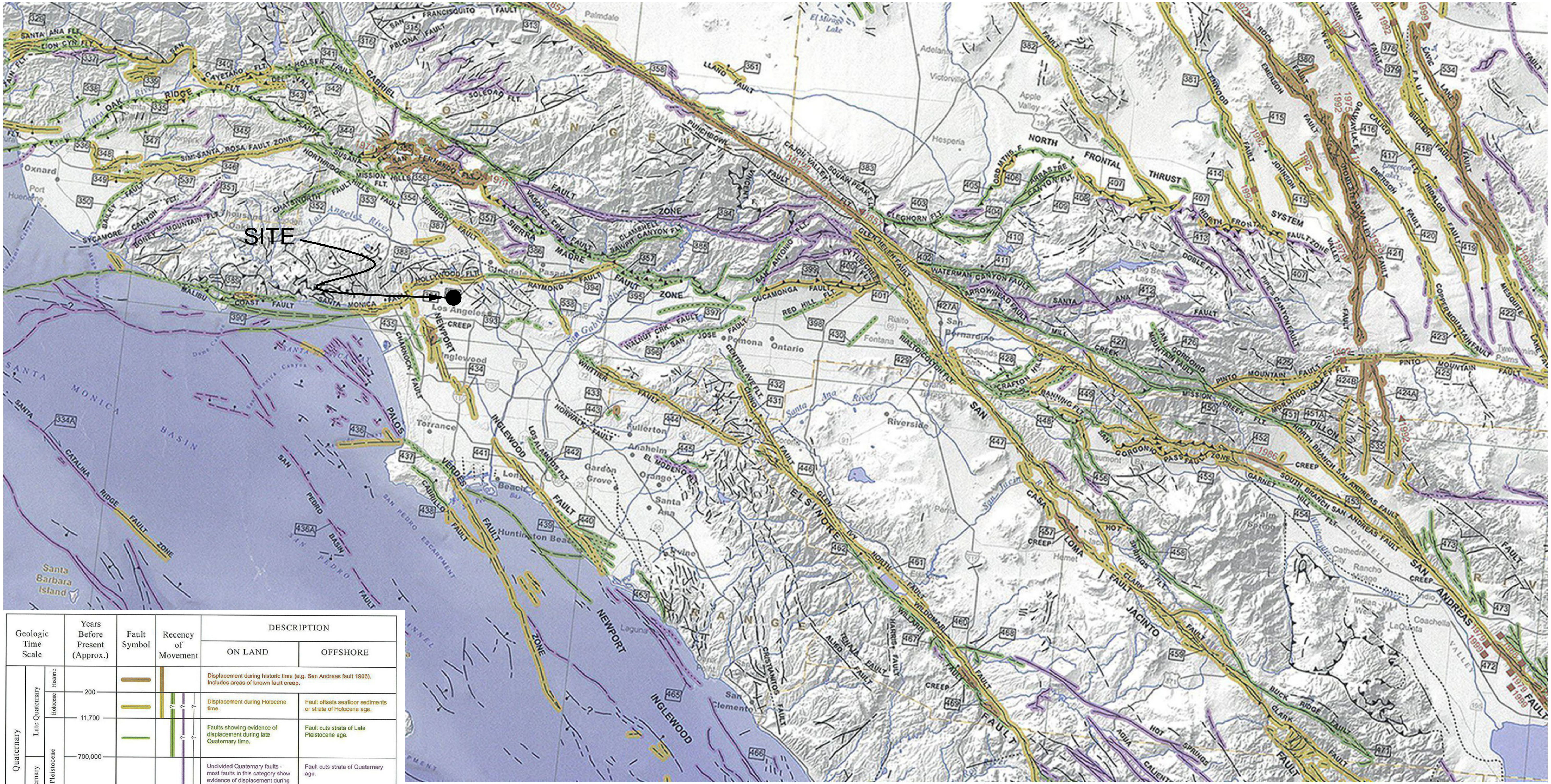
CROSS SECTION

6103 WEST MELROSE AVENUE
LOS ANGELES, CALIFORNIA

APRIL 2020	PROJECT NO. W1153-06-01	FIG. 2B

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



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

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

0 12 24 Miles

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REGIONAL FAULT MAP

6103 WEST MELROSE AVENUE
LOS ANGELES, CALIFORNIA

APR. 2020 | PROJECT NO. W1153-06-01 | 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

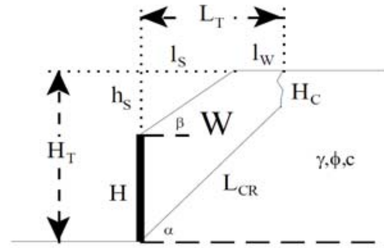
6103 WEST MELROSE AVENUE
LOS ANGELES, CALIFORNIA

APR. 2020 | PROJECT NO. W1153-06-01 | FIG.4

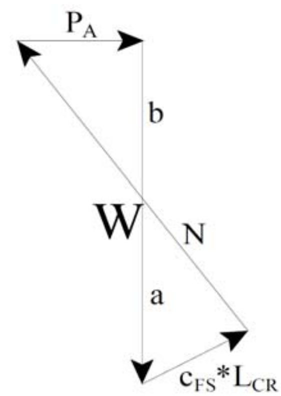
Retaining Wall Design with Transitioned Backfill (Vector Analysis)

Input:

Retaining Wall Height	(H)	30.00	feet
Slope Angle of Backfill	(b)	0.0	degrees
Height of Slope above Wall	(h_s)	0.0	feet
Horizontal Length of Slope	(l_s)	0.0	feet
Total Height (Wall + Slope)	(H_T)	30.0	feet
Unit Weight of Retained Soils	(g)	125.0	pcf
Friction Angle of Retained Soils	(f)	30.0	degrees
Cohesion of Retained Soils	(c)	240.0	psf
Factor of Safety	(FS)	1.50	
Factored Parameters:	(f_{FS})	21.1	degrees
	(c_{FS})	160.0	psf



Failure Angle (a) degrees	Height of Tension Crack (H_C) feet	Area of Wedge (A) $feet^2$	Weight of Wedge (W) lbs/lineal foot	Length of Failure Plane (L_{CR}) feet	a lbs/lineal foot	b lbs/lineal foot	Active Pressure (P_A) lbs/lineal foot
45	4.2	441	55167.4	36.5	13442.0	41725.4	18532.2
46	4.1	427	53316.8	36.0	12757.5	40559.3	18868.5
47	4.0	412	51520.0	35.5	12130.5	39389.5	19167.6
48	3.9	398	49774.4	35.1	11554.6	38219.8	19430.5
49	3.9	385	48077.3	34.6	11024.5	37052.8	19658.5
50	3.8	371	46426.2	34.2	10535.4	35890.9	19852.3
51	3.8	359	44818.6	33.7	10083.0	34735.6	20012.8
52	3.8	346	43252.2	33.3	9663.9	33588.3	20140.6
53	3.8	334	41724.7	32.9	9274.7	32450.0	20236.3
54	3.7	322	40234.0	32.5	8912.7	31321.3	20300.1
55	3.7	310	38778.0	32.1	8575.2	30202.8	20332.4
56	3.7	299	37354.8	31.7	8260.2	29094.7	20333.2
57	3.7	288	35962.6	31.3	7965.4	27997.2	20302.6
58	3.8	277	34599.6	31.0	7689.2	26910.4	20240.4
59	3.8	266	33264.2	30.6	7430.0	25834.2	20146.4
60	3.8	256	31954.7	30.3	7186.1	24768.6	20020.2
61	3.8	245	30669.7	29.9	6956.4	23713.4	19861.4
62	3.9	235	29407.7	29.6	6739.5	22668.2	19669.3
63	3.9	225	28167.4	29.3	6534.3	21633.0	19443.2
64	4.0	216	26947.3	28.9	6339.9	20607.5	19182.0
65	4.1	206	25746.4	28.6	6155.1	19591.3	18884.9
66	4.2	197	24563.2	28.3	5979.1	18584.1	18550.6
67	4.3	187	23396.7	28.0	5811.0	17585.6	18177.6
68	4.4	178	22245.6	27.6	5650.0	16595.6	17764.5
69	4.5	169	21108.9	27.3	5495.1	15613.8	17309.4
70	4.6	160	19985.4	27.0	5345.5	14639.8	16810.5



Design Equations (Vector Analysis):

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

$b = W-a$

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

$EFP = 2*P_A/H^2$

Maximum Active Pressure Resultant

$P_{A,\ max}$ 20333.2 lbs/lineal foot

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

$EFP = 2*P_A/H^2$

EFP 45.2 pcf 62.5 pcf

Design Wall for an Equivalent Fluid Pressure: 45 pcf 63 pcf

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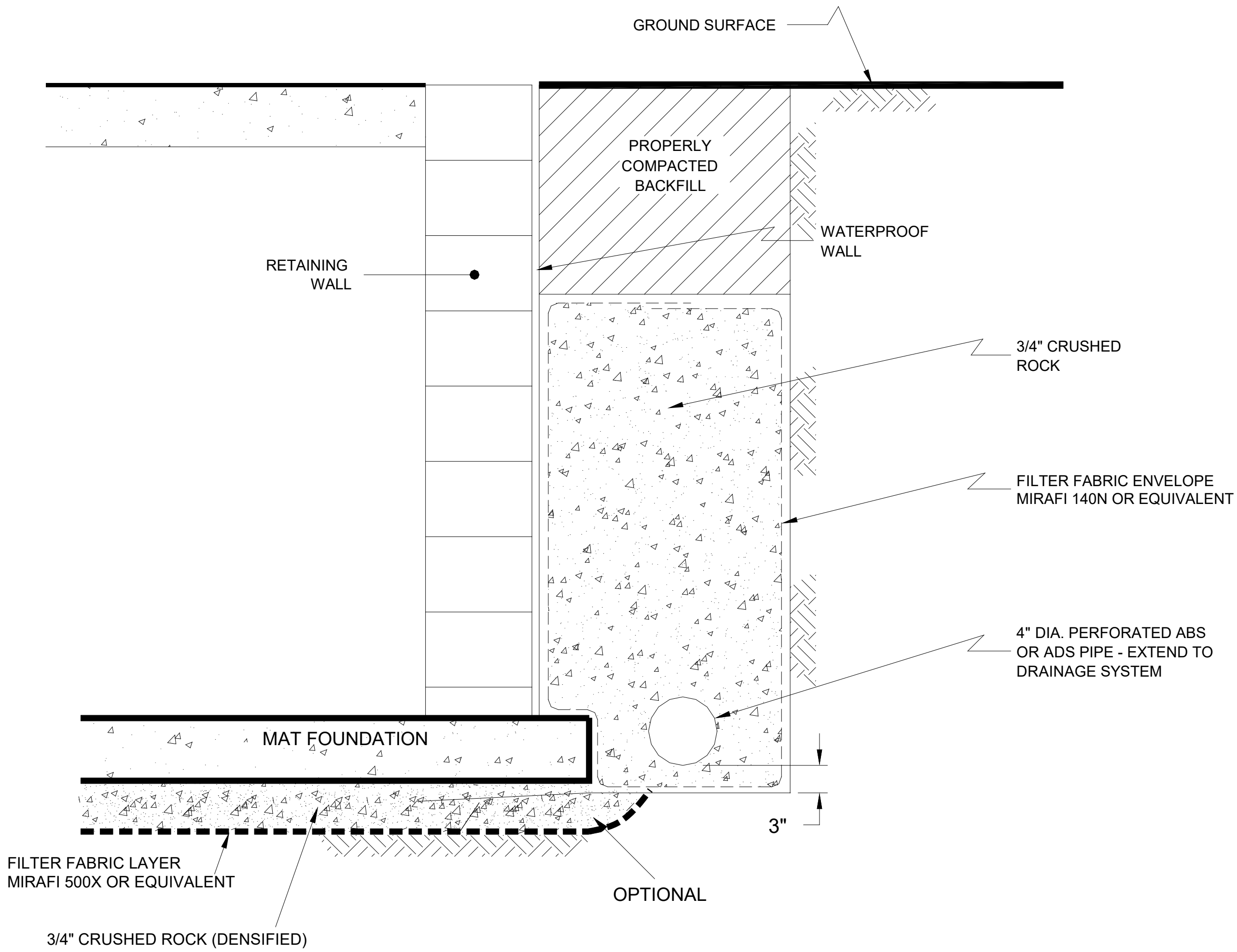
RETAINING WALL PRESSURE CALCULATION

6103 W MELROSE AVE
LOS ANGELES, CALIFORNIA

DRAFTED BY: PZ	CHECKED BY: HHD	APRIL 2020	PROJECT NO. W1153-06-01	FIG. 5



NO SCALE

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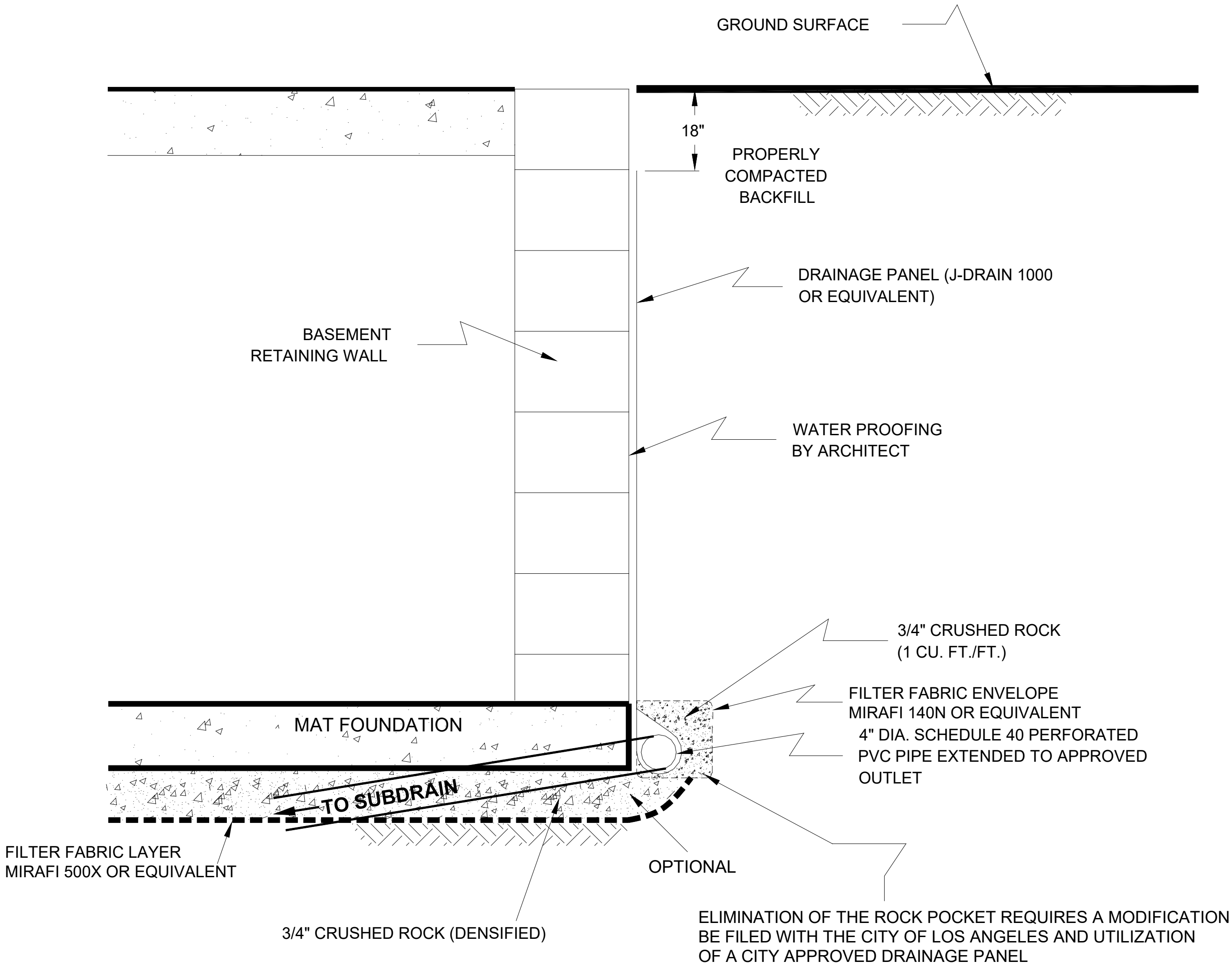
RETAINING WALL DRAIN DETAIL

6103 W MELROSE AVE
LOS ANGELES, CALIFORNIA

DRAFTED BY: PZ	CHECKED BY: HHD	APRIL 2020	PROJECT NO. W1153-06-01	FIG. 6



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

NO SCALE

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RETAINING WALL DRAIN DETAIL

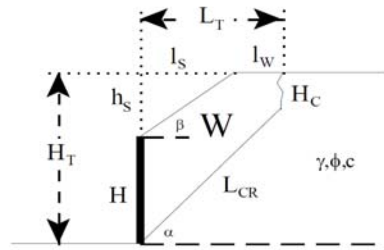
6103 W MELROSE AVE
LOS ANGELES, CALIFORNIA

DRAFTED BY: PZ	CHECKED BY: HHD	APRIL 2020	PROJECT NO. W1153-06-01	FIG. 7

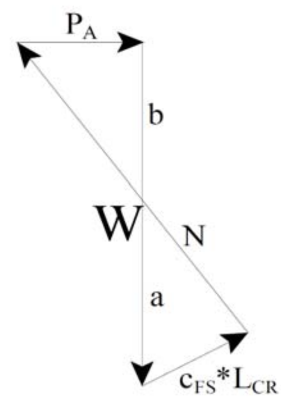
Shoring Design with Transitioned Backfill
(Vector Analysis)

Input:

Shoring Height	(H)	35.00	feet
Slope Angle of Backfill	(b)	0.0	degrees
Height of Slope above Shoring	(h_s)	0.0	feet
Horizontal Length of Slope	(l_s)	0.0	feet
Total Height (Shoring + Slope)	(H_T)	35.0	feet
Unit Weight of Retained Soils	(g)	125.0	pcf
Friction Angle of Retained Soils	(f)	30.0	degrees
Cohesion of Retained Soils	(c)	240.0	psf
Factor of Safety	(FS)	1.25	
Factored Parameters:	(f_{FS})	24.8	degrees
	(c_{FS})	192.0	psf



Failure Angle (a) degrees	Height of Tension Crack (H_C) feet	Area of Wedge (A) feet^2	Weight of Wedge (W) lbs/lineal foot	Length of Failure Plane (L_{CR}) feet	a lbs/lineal foot	b lbs/lineal foot	Active Pressure (P_A) lbs/lineal foot
45	5.7	596	74525.6	41.4	20902.1	53623.5	19738.9
46	5.5	577	72077.2	40.9	19726.6	52350.7	20314.7
47	5.4	558	69690.3	40.5	18658.1	51032.2	20834.9
48	5.3	539	67363.5	40.0	17684.0	49679.5	21301.6
49	5.2	521	65095.1	39.5	16793.5	48301.6	21716.4
50	5.1	503	62883.0	39.0	15977.1	46905.9	22081.0
51	5.0	486	60725.1	38.6	15226.9	45498.2	22396.5
52	5.0	469	58619.0	38.1	14535.6	44083.4	22664.2
53	4.9	452	56562.3	37.7	13897.3	42665.0	22885.1
54	4.9	436	54552.7	37.3	13306.4	41246.3	23060.0
55	4.8	421	52587.9	36.8	12758.4	39829.5	23189.4
56	4.8	405	50665.7	36.4	12249.0	38416.6	23273.9
57	4.8	390	48783.7	36.0	11774.6	37009.2	23313.8
58	4.8	376	46940.0	35.6	11331.7	35608.3	23309.2
59	4.8	361	45132.5	35.2	10917.6	34214.8	23260.0
60	4.8	347	43359.1	34.8	10529.6	32829.5	23166.1
61	4.9	333	41618.0	34.5	10165.2	31452.7	23027.3
62	4.9	319	39907.3	34.1	9822.4	30084.9	22842.9
63	5.0	306	38225.2	33.7	9499.2	28726.1	22612.3
64	5.0	293	36570.2	33.3	9193.7	27376.5	22334.6
65	5.1	280	34940.4	33.0	8904.3	26036.0	22009.0
66	5.2	267	33334.2	32.6	8629.5	24704.8	21634.0
67	5.3	254	31750.2	32.3	8367.7	23382.6	21208.5
68	5.4	241	30186.8	31.9	8117.5	22069.3	20730.8
69	5.6	229	28642.5	31.5	7877.6	20764.9	20199.1
70	5.7	217	27115.7	31.1	7646.5	19469.1	19611.5



Design Equations (Vector Analysis):

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

$b = W-a$

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

$EFP = 2*P_A/H^2$

Maximum Active Pressure Resultant

$P_{A,\ max}$ 23313.8 lbs/lineal foot

Equivalent Fluid Pressure (per lineal foot of shoring)

$EFP = 2*P_A/H^2$

EFP 38.1 pcf

Design Shoring for an Equivalent Fluid Pressure: 38 pcf

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SHORING PRESSURE CALCULATION

6103 W MELROSE AVE
LOS ANGELES, CALIFORNIA

DRAFTED BY: PZ	CHECKED BY: HHD	APRIL 2020	PROJECT NO. W1153-06-01	FIG. 8

APPENDIX A

APPENDIX A

FIELD INVESTIGATION

The site was explored on March 16, 2020 by excavating two 8-inch diameter borings to depths of approximately 45½ and 50½ feet below the existing ground surface using a truck-mounted hollow-stem auger drilling machine. Representative and relatively undisturbed samples were obtained by driving a 3-inch, O. D., California Modified Sampler into the "undisturbed" soil mass with blows from a 140-pound auto-hammer falling 30 inches. The California Modified Sampler was equipped with 1-inch high by 2⅜-inch diameter brass sampler rings to facilitate soil removal and testing. A bulk sample was 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 and A2. The logs depict the soil and geologic conditions encountered and the depth at which samples were obtained. The logs also include our interpretation of the conditions between sampling intervals. Therefore, the logs contain both observed and interpreted data. We determined the lines designating the interface between soil materials on the logs using visual observations, penetration rates, excavation characteristics and other factors. The transition between materials may be abrupt or gradual. Where applicable, the logs were revised based on subsequent laboratory testing. The locations of the borings are shown on Figure 2A.

BORING 1

ELEV. (MSL.) -- DATE COMPLETED 3/16/2020

EQUIPMENT HOLLOW STEM AUGER BY: RP

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0	BULK 0-5'				**ASP: 3.5" BASE: NONE**			
					ARTIFICIAL FILL			
					Clay, firm, slightly moist, dark brown, some coarse-grained sand, trace fine gravel.			
2					- brown with yellowish brown mottles			
4					**OLDER ALLUVIUM**			
					Silty Sand, dense, slightly moist, yellowish brown, fine- to medium-grained, some clay.			
5	B1@5'			SM	- fine-grained	50 (4")	108.6	14.6
9					Sandy Silt, hard, slightly moist, yellowish brown with brown mottles.			
10	B1@10'				- fine- to medium-grained, trace fine gravel (to 1/2"), some clay.	71	104.9	16.9
14				ML				
15	B1@15'				- brown to reddish brown, increase in sand	50 (4")	120.2	12.6
19					Sandy Clay, hard, slightly moist, reddish brown, fine- to medium-grained, trace fine gravel, some silt.			
20	B1@20' BULK 20-25'			CL		61	101.7	23.8
21.5					Silty Clay, hard, slightly moist, reddish brown, trace coarse-grained sand.			
22.5	B1@22.5'			CL		45	94.9	28.3
24					Silt, stiff, dark brown, fine-grained, some clay.			
25	B1@25'					19	90.1	31.2
26				ML				
27.5	B1@27.5'				- hard, olive with reddish brown mottles - oxidized	53	107.5	23.4

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

W1153-06-01 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.

BORING 1

ELEV. (MSL.) -- DATE COMPLETED 3/16/2020

EQUIPMENT HOLLOW STEM AUGER BY: RP

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'					72	99.7	27.6
32			▼		- perched groundwater			
34								
36	B1@35'				- trace gravel	50 (5")	110.9	17.1
				ML				
38								
40	B1@40'				- increase in silt and sand	42	96.4	29.7
42								
44								
	B1@45'				- trace interbedded clayey sand, medium- to coarse-grained	45	89.8	35.4

Total depth of boring: 45.5 feet
Fill to 2.5 feet.
Perched groundwater encountered at 32 feet.
Asphalt patched.

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

W1153-06-01 BORING LOGS.GPJ

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

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

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

PROJECT NO. W1153-06-01

BORING 2

ELEV. (MSL.) -- DATE COMPLETED 3/16/2020

EQUIPMENT HOLLOW STEM AUGER BY: RP

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0	BULK 0-5'				**ASP: 6" BASE: 2"** **ARTIFICIAL FILL** Clay, firm, slightly moist, dark brown, plastic, trace coarse-grained sand.			
2					**OLDER ALLUVIUM** Sandy Clay, hard, slightly moist, brown to reddish brown, fine- to medium-grained, some silt, trace fine gravel.			
4								
	B2@5'					50	111.4	20.4
6								
				CL				
8								
10	B2@10'				- increase in sand - no recovery	50 (6")		
12								
14					Sandy Silt, hard, slightly moist, reddish brown, fine-grained, trace medium-grained, some clay.			
	B2@15'					79	115.4	16.3
16								
18								
20	B2@20'			ML	- reddish brown with olive brown mottles, fine- to medium-grained, increase in clay, oxidized	45	108.5	21.7
22	B2@22.5'				- increase in silt	50 (5")	113.2	18.7
24								
	B2@25'				- increase in sand	50 (4")	117.7	17.7
26								
28	B2@27.5'				- some interbedded clayey sand (fine- to coarse-grained)	70	107.3	21.2
				SC	Clayey Sand, medium dense, moist, reddish brown, fine- to medium-grained.			

W1153-06-01 BORING LOGS.GPJ

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

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

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

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PROJECT NO. W1153-06-01

BORING 2

ELEV. (MSL.) **--** DATE COMPLETED **3/16/2020**

EQUIPMENT **HOLLOW STEM AUGER** BY: **RP**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
30	B2@30'					29	112.1	11.9
32			▼	SC	- perched groundwater			
34					Clay, stiff, slightly moist, dark brown, some silt.			
35	B2@35'					40	97.3	27.0
36								
38				CL				
40	B2@40'				- olive brown with reddish brown mottles, some interbedded sandy clay	47	107.9	18.4
42								
44					Sandy Silt, stiff, slightly moist, olive brown with reddish brown mottles, fine-grained.			
45	B2@45'					40	106.4	23.8
46								
48				ML				
50	B2@50'				- hard, oxidized, some carbon fragments, trace fine gravel	42	103.7	23.4

Total depth of boring: 50.5 feet
Fill to 2 feet.
Perched groundwater encountered at 32 feet.
Backfilled with soil cuttings and tamped.
Asphalt patched.

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

W1153-06-01 BORING LOGS.GPJ

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

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

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

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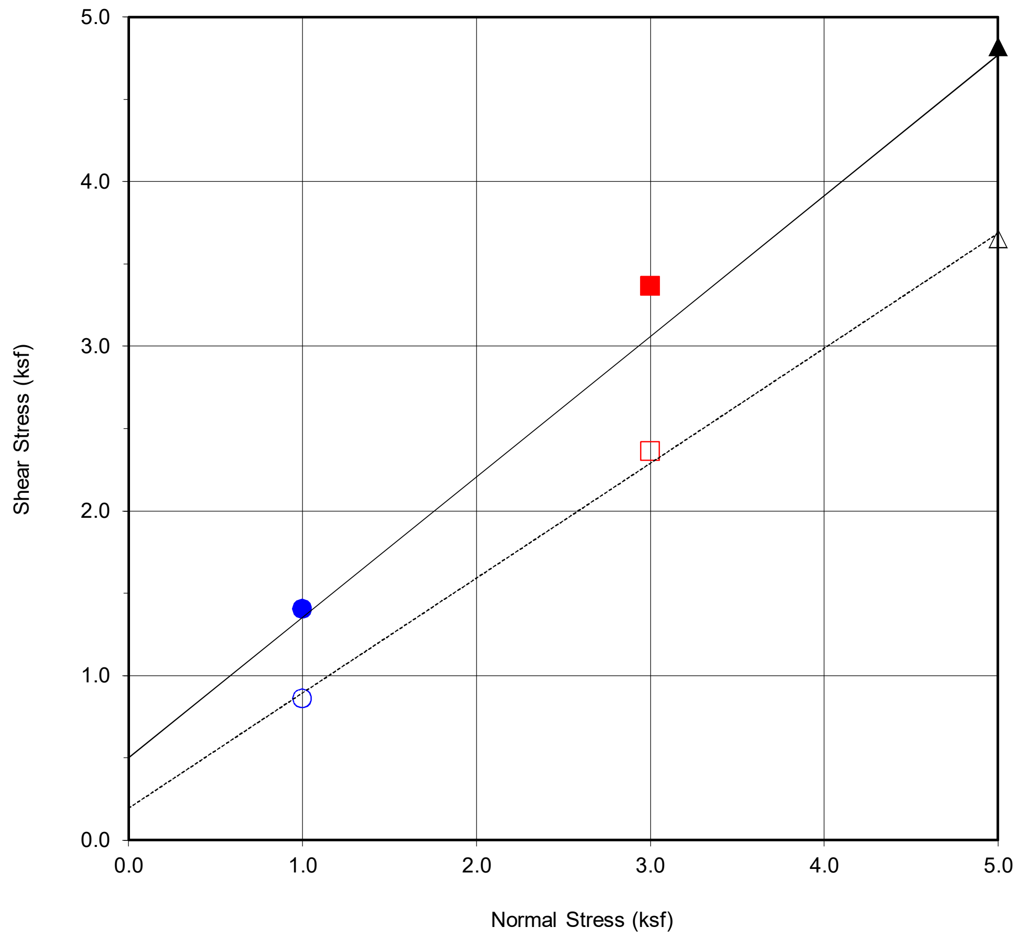
APPENDIX B

APPENDIX B

LABORATORY TESTING

Laboratory tests were performed in accordance with generally accepted test methods of the "American Society for Testing and Materials (ASTM)", or other suggested procedures. Selected samples were tested for direct shear strength, consolidation characteristics, corrosivity, and in-place dry density and moisture content. The results of the laboratory tests are summarized in Figures B1 through B15. 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@5'
Depth (ft)	5
Sample Type:	Ring

Soil Identification:		
Yellowish Brown Silty Sand (SM)		
Strength Parameters		
	C (psf)	ϕ (°)
Peak	500	40.5
Ultimate	193	34.9

Normal Stress (kip/ft2)	1	3	5
Peak Shear Stress (kip/ft²)	● 1.40	■ 3.37	▲ 4.82
Shear Stress @ End of Test (ksf)	○ 0.86	□ 2.36	△ 3.65
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 (%)	15.4	14.7	14.6
Initial Dry Density (pcf)	107.6	106.9	113.2
Initial Degree of Saturation (%)	73.2	68.8	80.5
Soil Height Before Shearing (in.)	1.2	1.2	1.2
Final Moisture Content (%)	19.3	17.8	16.8

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DIRECT SHEAR TEST RESULTS

Consolidated Drained ASTM D-3080

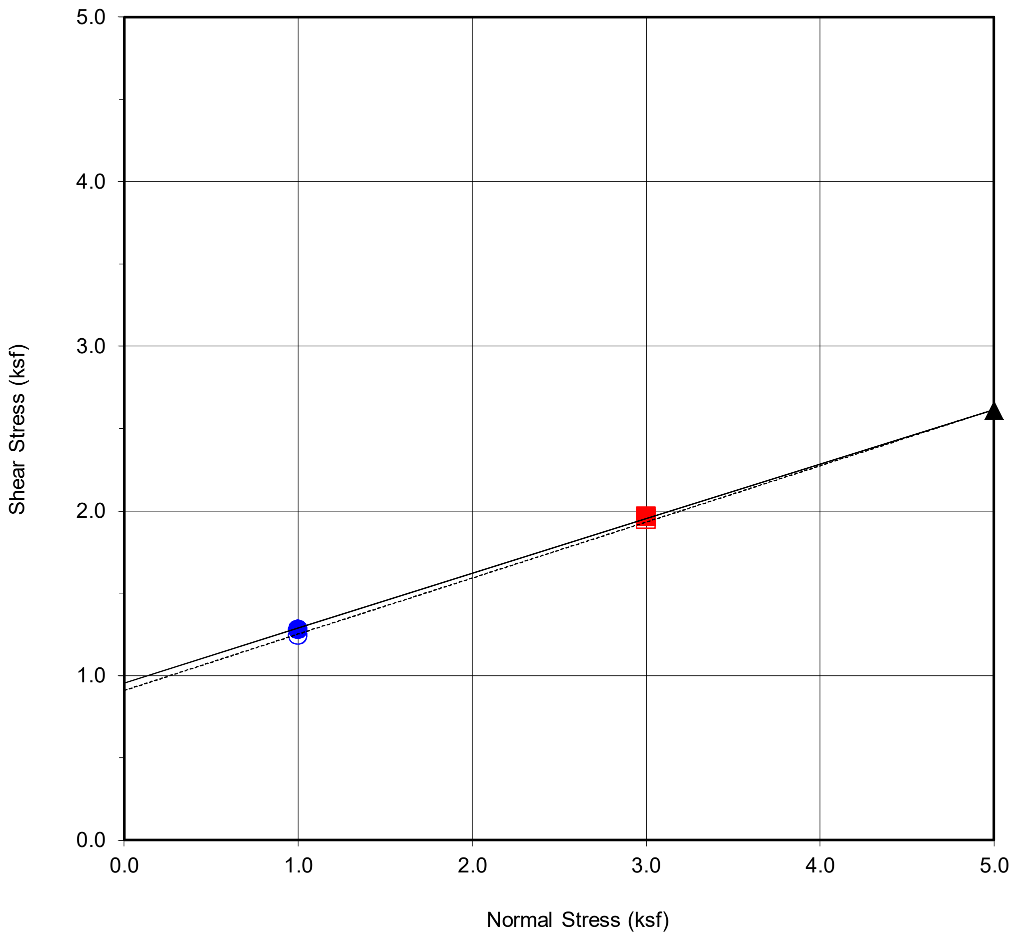
Checked by: PZ

Project No.: W1153-06-01

6103 W Melrose Ave
Los Angeles, California

April 2020

Figure B1



Boring No.	**B2**
Sample No.	**B2@5'**
Depth (ft)	**5**
Sample Type:	Ring

Soil Identification:		
Brown Sandy Clay (CL)		
Strength Parameters		
	C (psf)	ϕ (°)
Peak	953	18.4
Ultimate	909	18.9

Normal Stress (kip/ft2)	1	3	5
Peak Shear Stress (kip/ft²)	● 1.28	■ 1.96	▲ 2.61
Shear Stress @ End of Test (ksf)	○ 1.24	□ 1.95	△ 2.61
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 (%)	17.1	17.6	17.7
Initial Dry Density (pcf)	112.7	111.6	112.5
Initial Degree of Saturation (%)	92.8	93.2	95.9
Soil Height Before Shearing (in.)	1.2	1.2	1.2
Final Moisture Content (%)	20.4	20.2	19.5

GEOCON

DIRECT SHEAR TEST RESULTS

Consolidated Drained ASTM D-3080

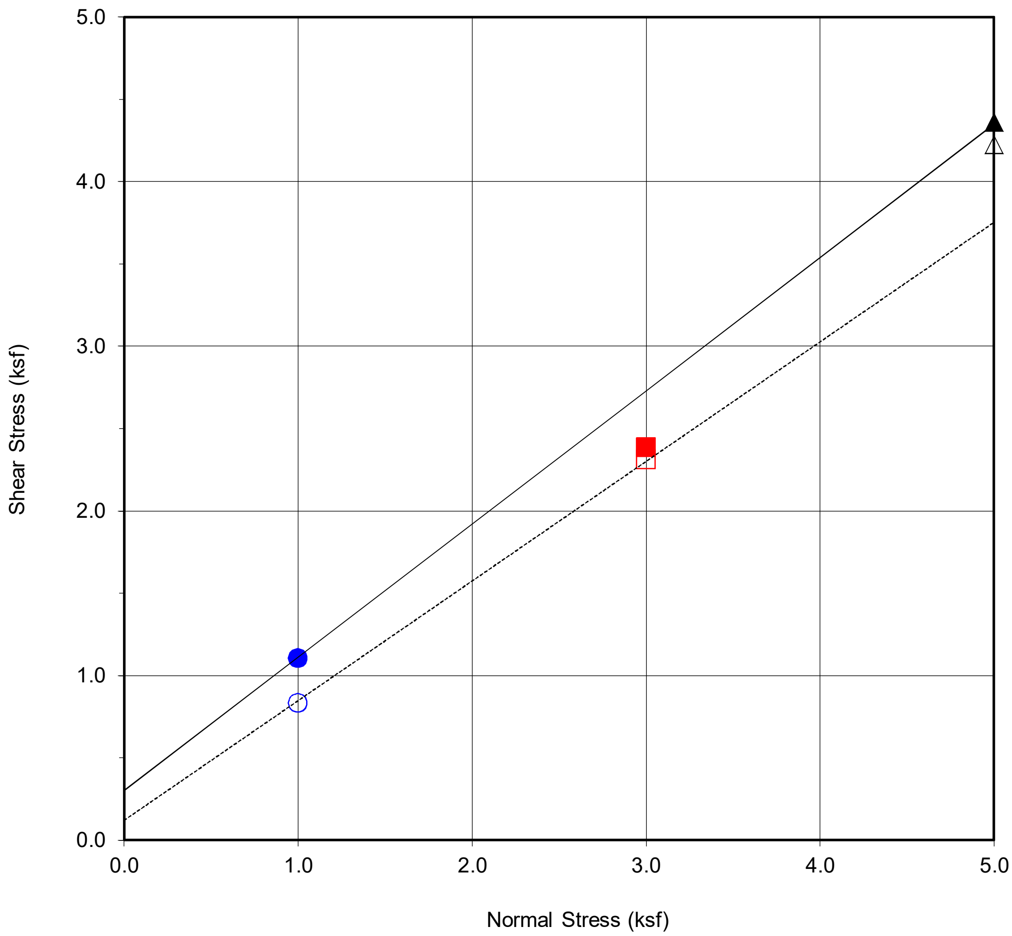
Checked by: PZ

Project No.: W1153-06-01

6103 W Melrose Ave
Los Angeles, California

April 2020

Figure B2



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

Soil Identification:		
Yellowish Brown Sandy Silt (ML)		
Strength Parameters		
	C (psf)	ϕ (^{o})
Peak	300	39.0
Ultimate	120	36.0

Normal Stress (kip/ft2)	1	3	5
Peak Shear Stress (kip/ft²)	● 1.11	■ 2.38	▲ 4.36
Shear Stress @ End of Test (ksf)	○ 0.83	□ 2.31	△ 4.22
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 (%)	16.9	17.5	17.5
Initial Dry Density (pcf)	107.2	100.6	109.4
Initial Degree of Saturation (%)	79.8	69.9	87.5
Soil Height Before Shearing (in.)	1.2	1.2	1.2
Final Moisture Content (%)	20.9	19.7	18.7

GEOCON

DIRECT SHEAR TEST RESULTS

Consolidated Drained ASTM D-3080

Checked by: PZ

Project No.: W1153-06-01

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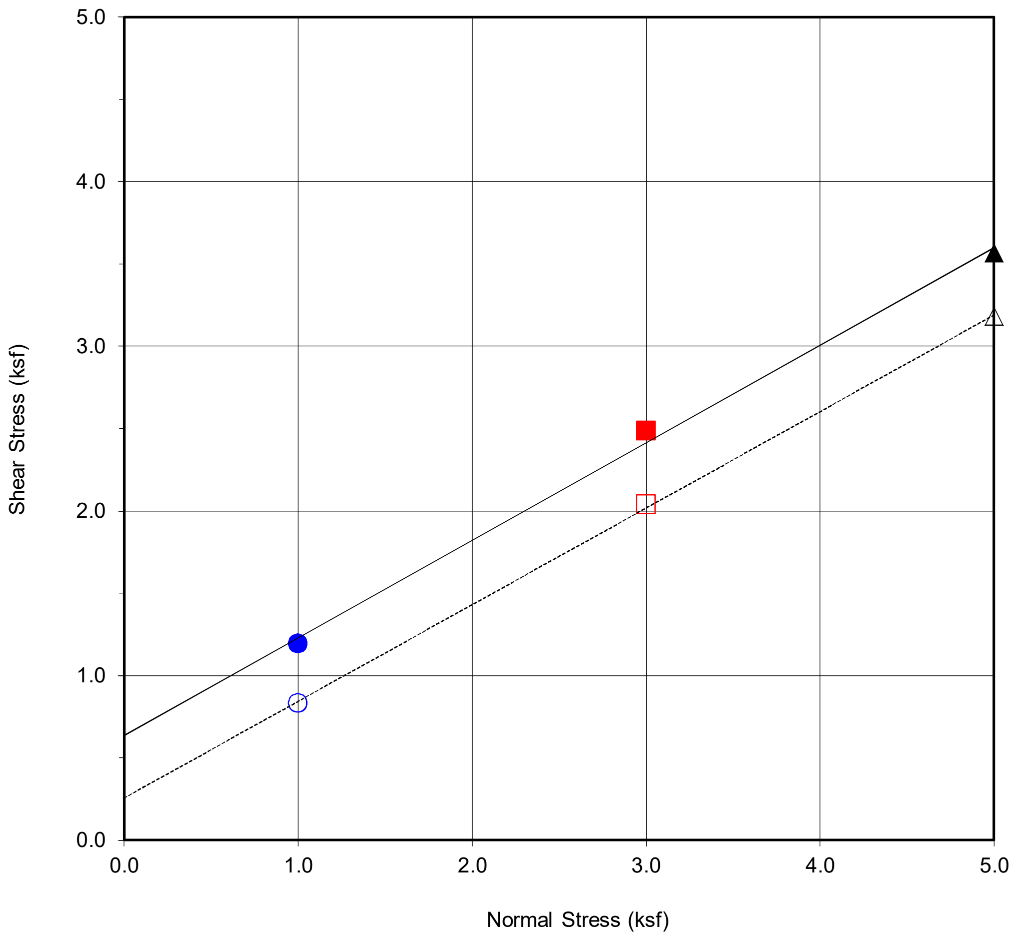
Figure B3



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

Normal Stress (kip/ft2)	1	3	5
Peak Shear Stress (kip/ft²)	● 1.19	■ 2.48	▲ 3.56
Shear Stress @ End of Test (ksf)	○ 0.83	□ 2.04	△ 3.18
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 (%)	27.8	28.3	28.7
Initial Dry Density (pcf)	95.2	95.7	95.4
Initial Degree of Saturation (%)	97.3	100.3	100.9
Soil Height Before Shearing (in.)	1.2	1.2	1.2
Final Moisture Content (%)	28.9	28.7	27.8

Soil Identification:

Reddish Brown Silty Clay (CL)

Strength Parameters

	C (psf)	ϕ (°)
Peak	634	30.7
Ultimate	255	30.4

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DIRECT SHEAR TEST RESULTS

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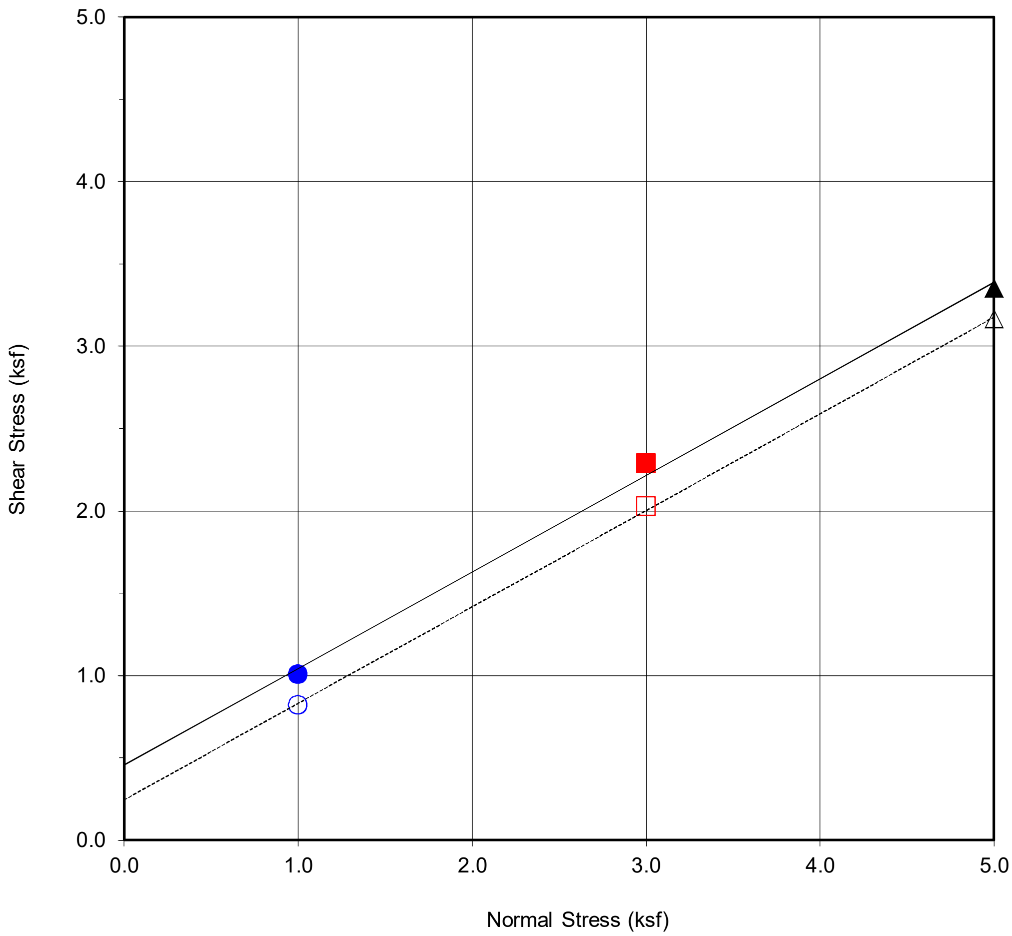
Figure B4



Boring No.	**B1**
Sample No.	**B1@25'**
Depth (ft)	**25**
Sample Type:	Ring

Soil Identification:		
Dark Brown Silt (ML)		
Strength Parameters		
	C (psf)	ϕ (^{o})
Peak	454	30.4
Ultimate	244	30.4

Normal Stress (kip/ft2)	1	3	5
Peak Shear Stress (kip/ft²)	● 1.01	■ 2.29	▲ 3.35
Shear Stress @ End of Test (ksf)	○ 0.82	□ 2.02	△ 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 (%)	34.6	34.1	34.9
Initial Dry Density (pcf)	86.6	88.6	86.8
Initial Degree of Saturation (%)	98.5	101.9	100.1
Soil Height Before Shearing (in.)	1.2	1.2	1.2
Final Moisture Content (%)	33.9	31.2	31.7

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DIRECT SHEAR TEST RESULTS

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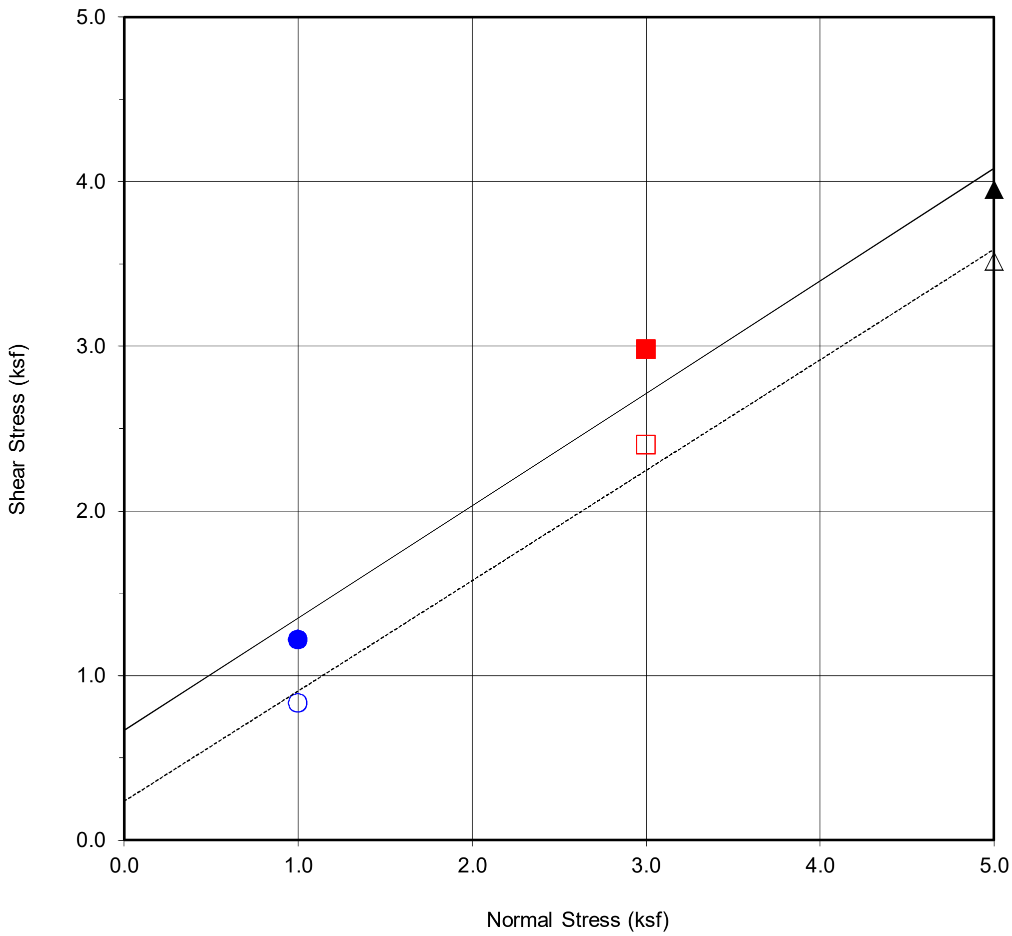
Figure B5



Boring No.	**B2**
Sample No.	**B2@30'**
Depth (ft)	**30**
Sample Type:	Ring

Soil Identification:		
Reddish Brown Clayey Sand (SC)		
Strength Parameters		
	C (psf)	ϕ (^{o})
Peak	666	34.3
Ultimate	236	33.8

Normal Stress (kip/ft2)	1	3	5
Peak Shear Stress (kip/ft²)	● 1.22	■ 2.98	▲ 3.95
Shear Stress @ End of Test (ksf)	○ 0.83	□ 2.40	△ 3.51
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 (%)	22.8	22.7	22.6
Initial Dry Density (pcf)	101.6	102.9	102.9
Initial Degree of Saturation (%)	93.4	96.2	95.4
Soil Height Before Shearing (in.)	1.2	1.2	1.2
Final Moisture Content (%)	22.4	21.2	20.5

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DIRECT SHEAR TEST RESULTS

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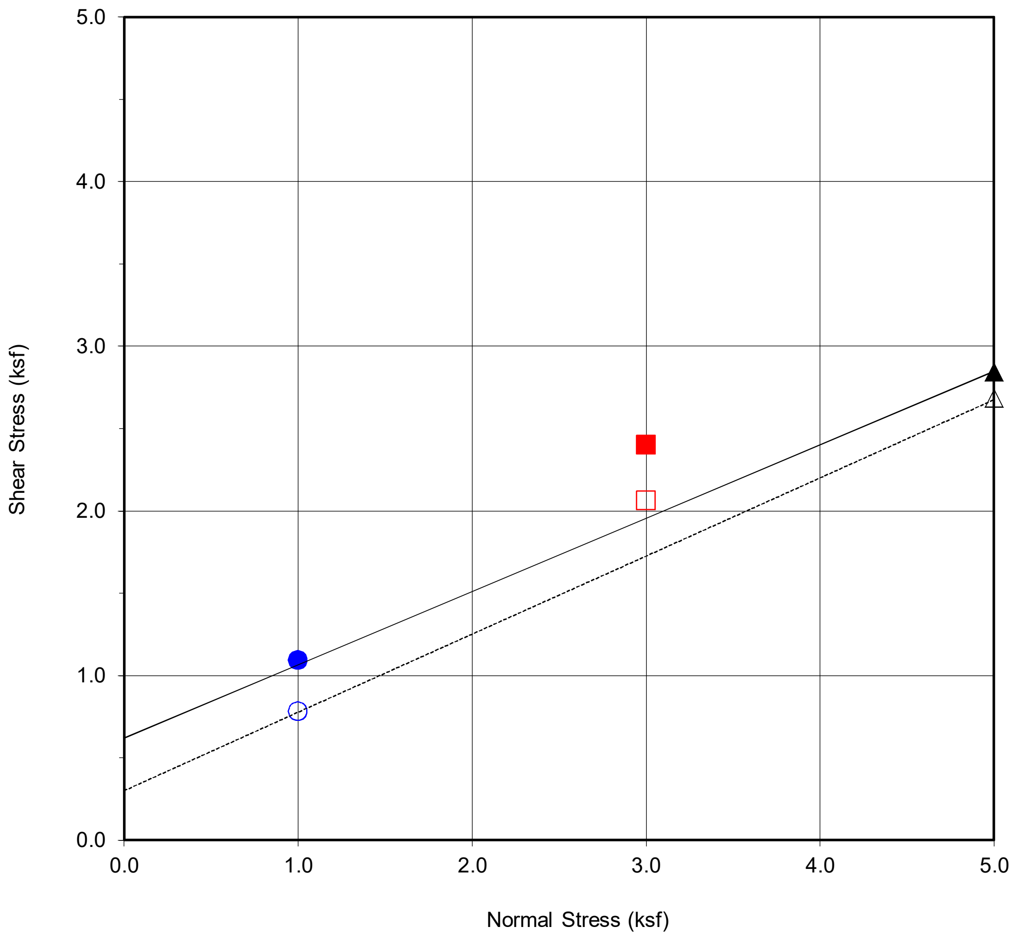
Figure B6



Boring No.	B2
Sample No.	B2@35'
Depth (ft)	35
Sample Type:	Ring

Soil Identification:		
Dark Brown Clay (CL)		
Strength Parameters		
	C (psf)	ϕ (°)
Peak	620	24.0
Ultimate	300	25.4

Normal Stress (kip/ft2)	1	3	5
Peak Shear Stress (kip/ft²)	● 1.09	■ 2.40	▲ 2.84
Shear Stress @ End of Test (ksf)	○ 0.78	□ 2.06	△ 2.68
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 (%)	27.0	27.5	32.8
Initial Dry Density (pcf)	91.6	94.6	91.2
Initial Degree of Saturation (%)	86.6	95.1	104.2
Soil Height Before Shearing (in.)	1.2	1.2	1.2
Final Moisture Content (%)	32.5	30.5	30.5

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DIRECT SHEAR TEST RESULTS

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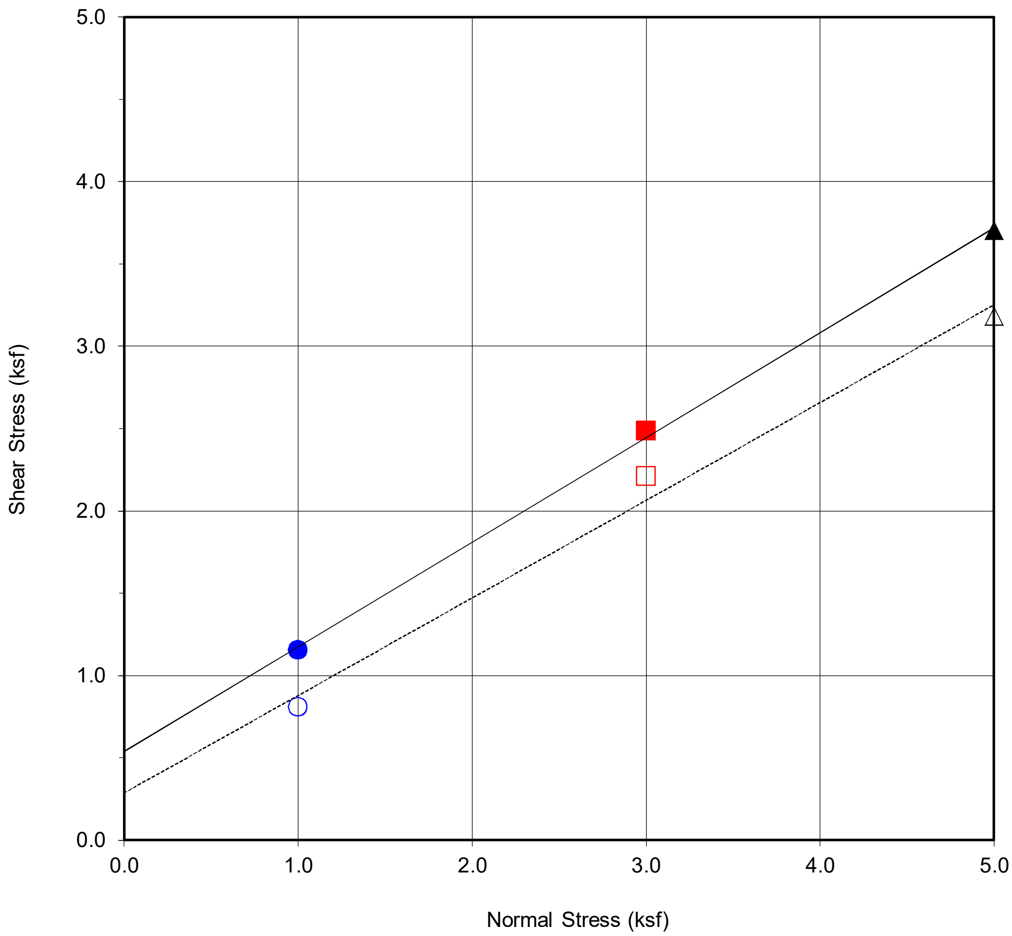
Figure B7



Boring No.	**B2**
Sample No.	**B2@50'**
Depth (ft)	**50**
Sample Type:	Ring

Soil Identification:		
Olive Brown Sandy Silt (ML)		
Strength Parameters		
	C (psf)	ϕ (^{o})
Peak	537	32.5
Ultimate	286	30.7

Normal Stress (kip/ft2)	1	3	5
Peak Shear Stress (kip/ft²)	● 1.16	■ 2.48	▲ 3.70
Shear Stress @ End of Test (ksf)	○ 0.81	□ 2.21	△ 3.18
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 (%)	21.7	22.8	23.4
Initial Dry Density (pcf)	103.1	101.5	102.8
Initial Degree of Saturation (%)	92.1	93.3	99.0
Soil Height Before Shearing (in.)	1.2	1.2	1.2
Final Moisture Content (%)	23.1	24.0	23.4

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DIRECT SHEAR TEST RESULTS

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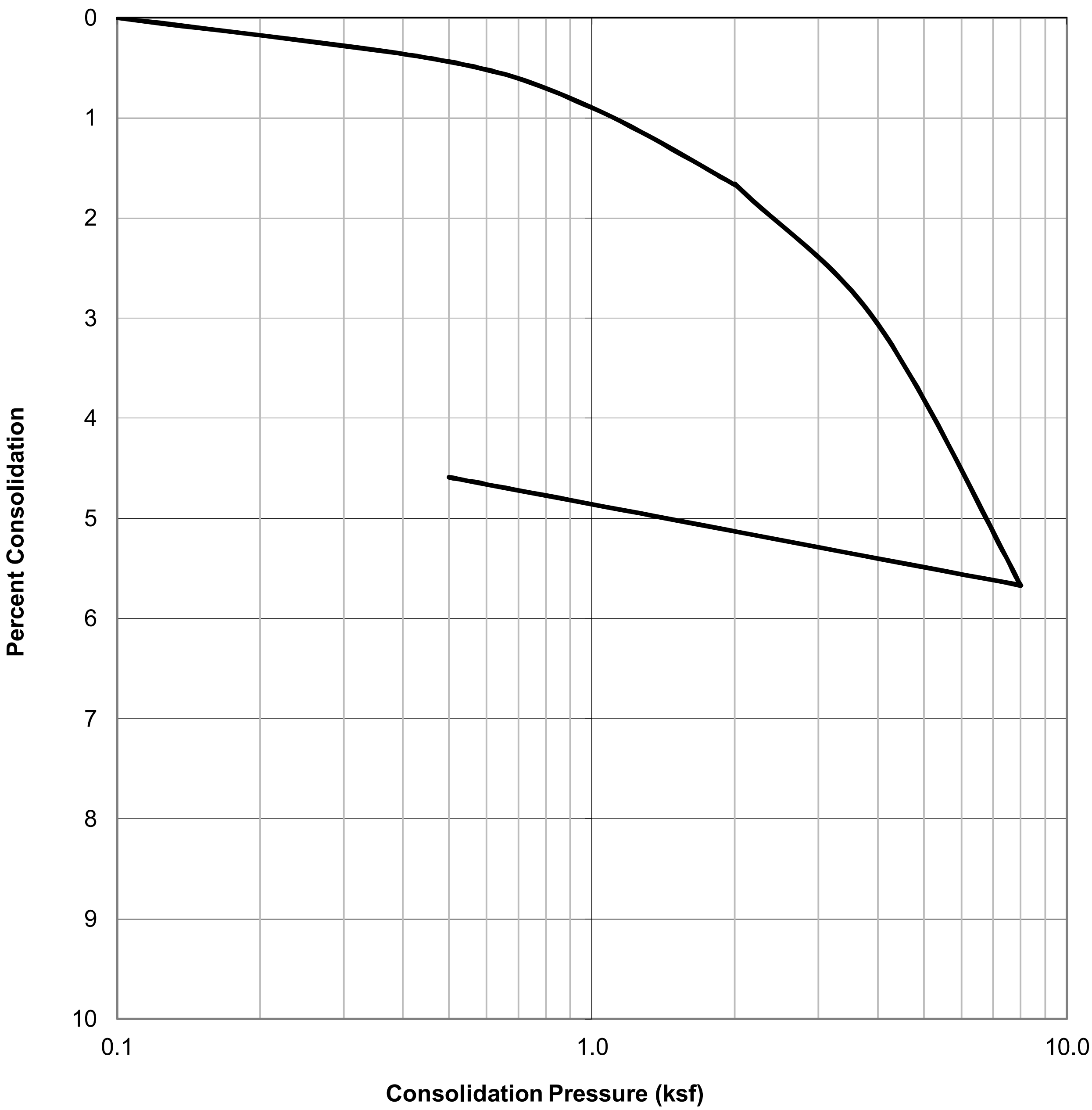
Figure B8

WATER ADDED AT 2.0 KSF



SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B1@25	Dark Brown Silt (ML)	87.7	34.1	31.5

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CONSOLIDATION TEST RESULTS

ASTM D-2435

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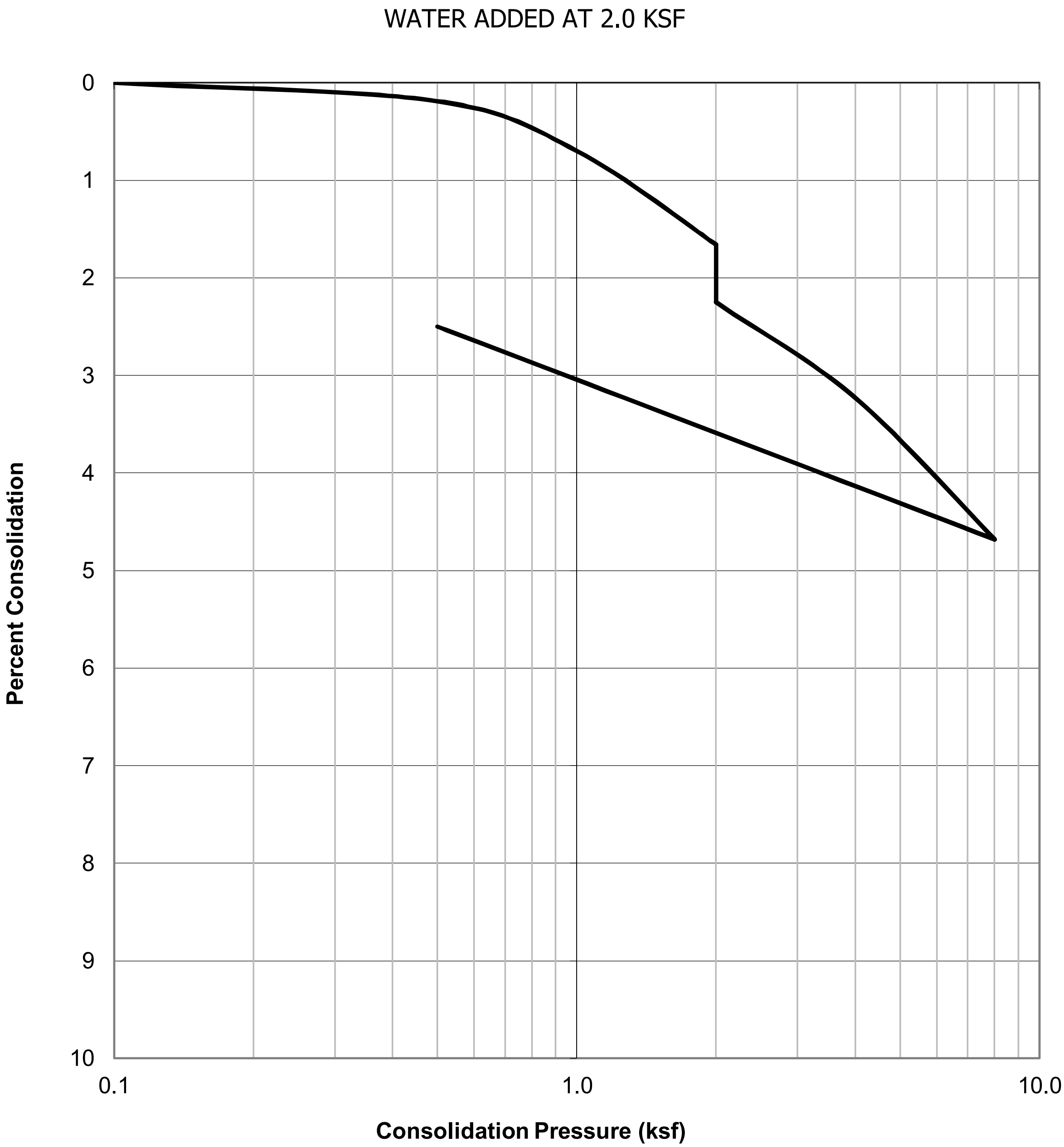

SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B2@25	Redish Brown Sandy Silt (ML)	108.0	17.7	20.0

GEOCON

CONSOLIDATION TEST RESULTS

ASTM D-2435

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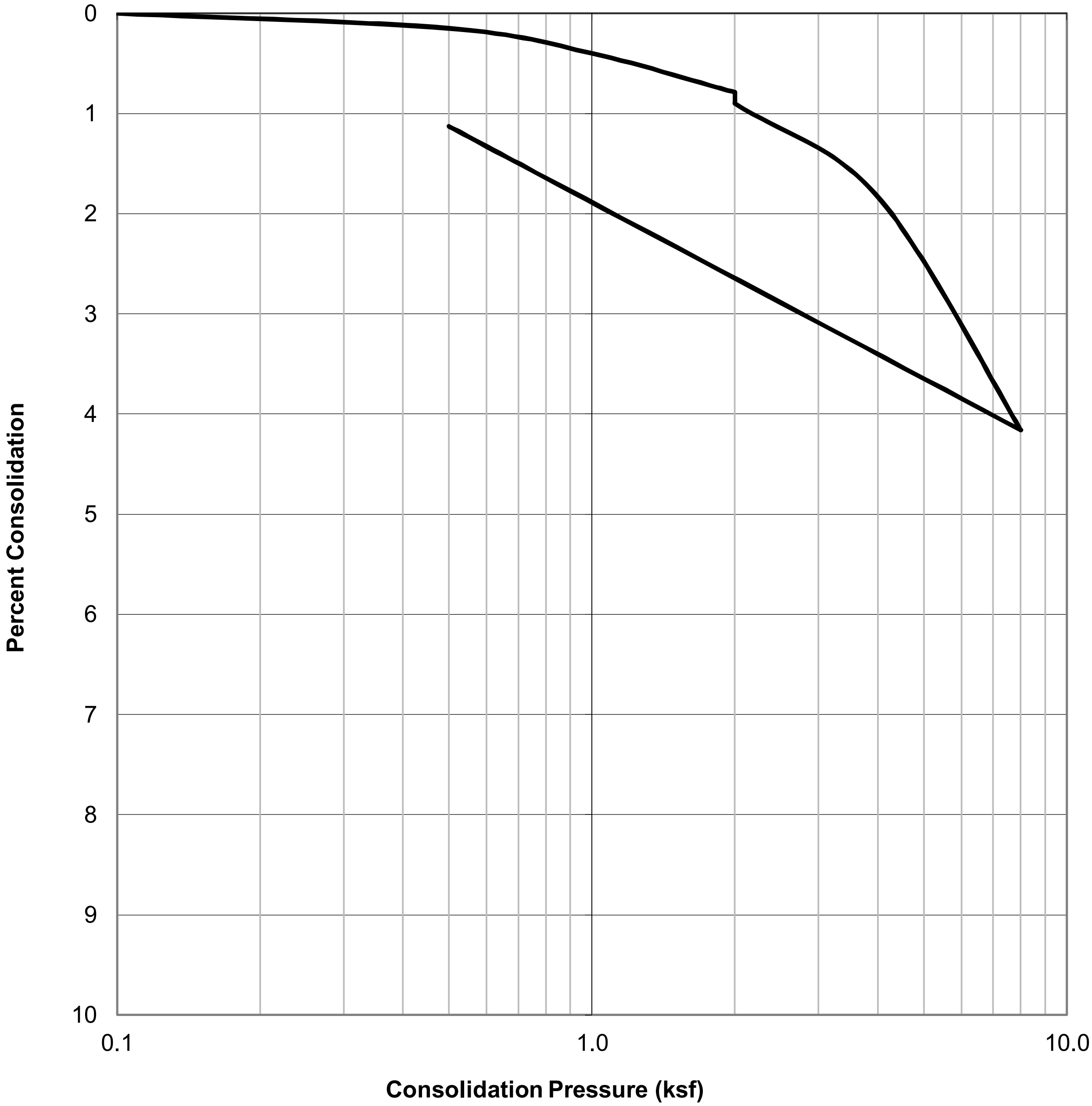
Figure B10

WATER ADDED AT 2.0 KSF



SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B1@30	Olive Brown Silt (ML)	92.7	27.6	31.1

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CONSOLIDATION TEST RESULTS

ASTM D-2435

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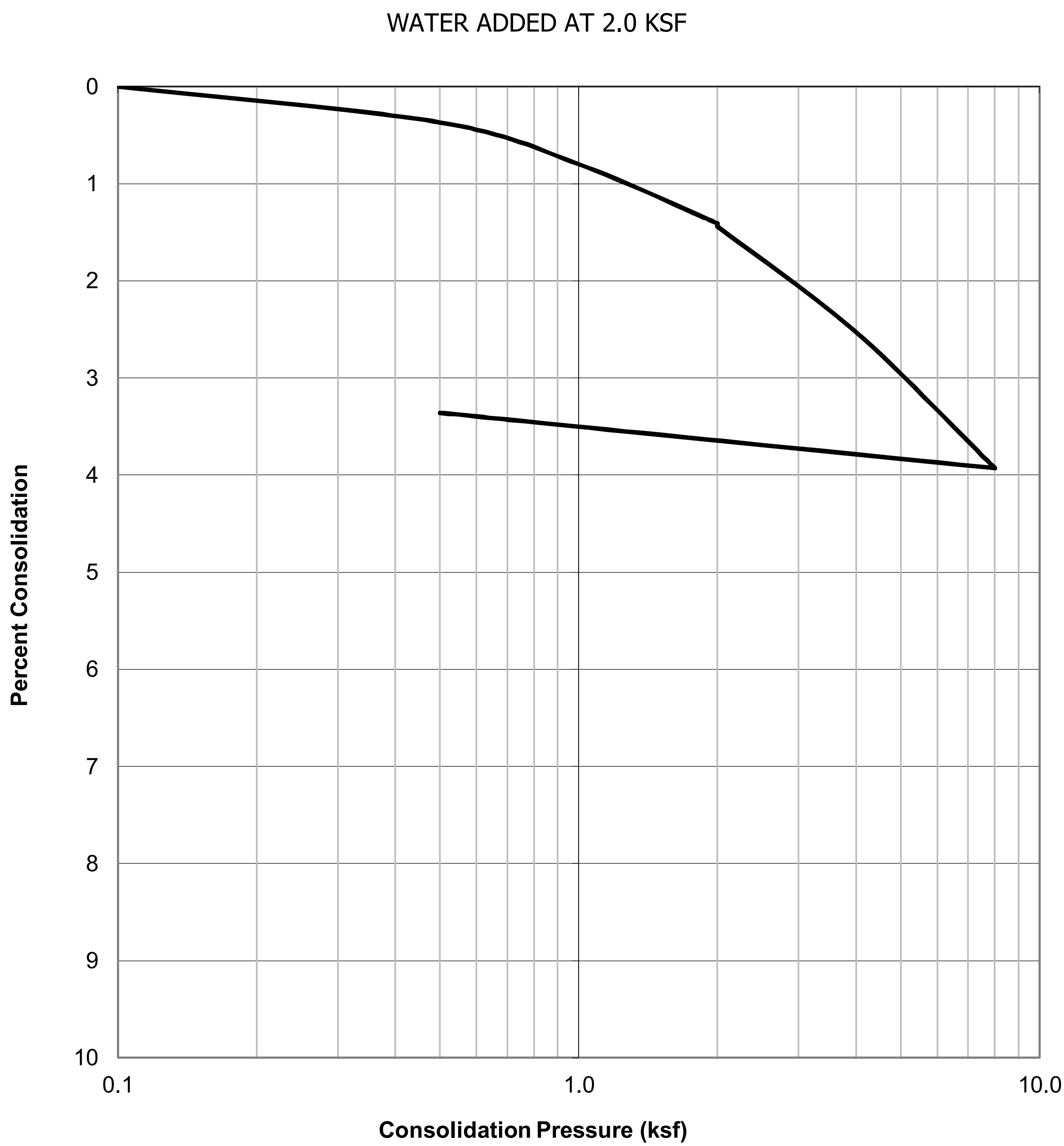
Figure B11



SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B2@30	Reddish Brown Clayey Sand (SC)	105.5	20.0	19.3

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CONSOLIDATION TEST RESULTS

ASTM D-2435

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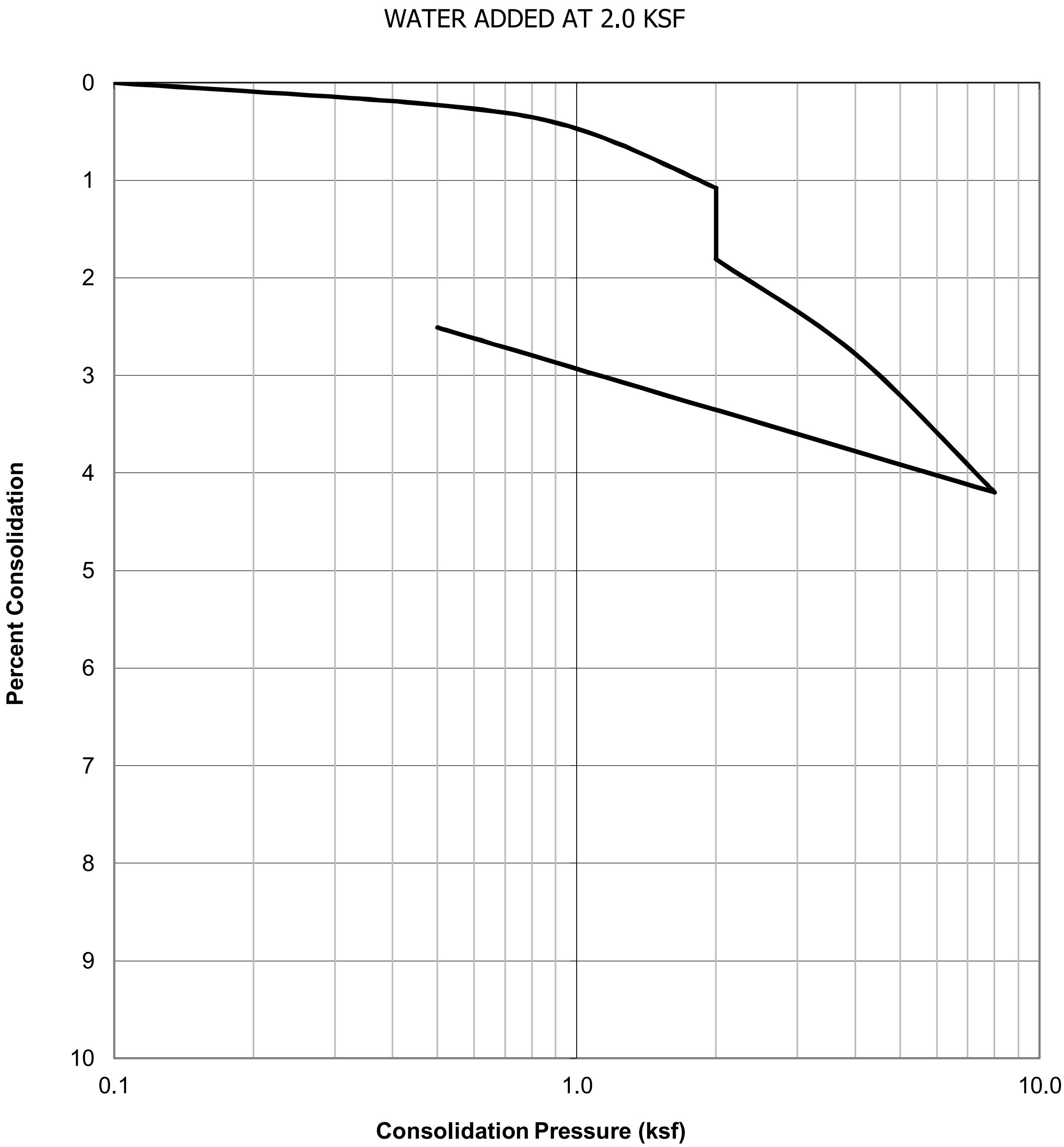
Figure B12



SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B1@35	Brown Silt (ML)	108.1	17.1	19.5

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CONSOLIDATION TEST RESULTS

ASTM D-2435

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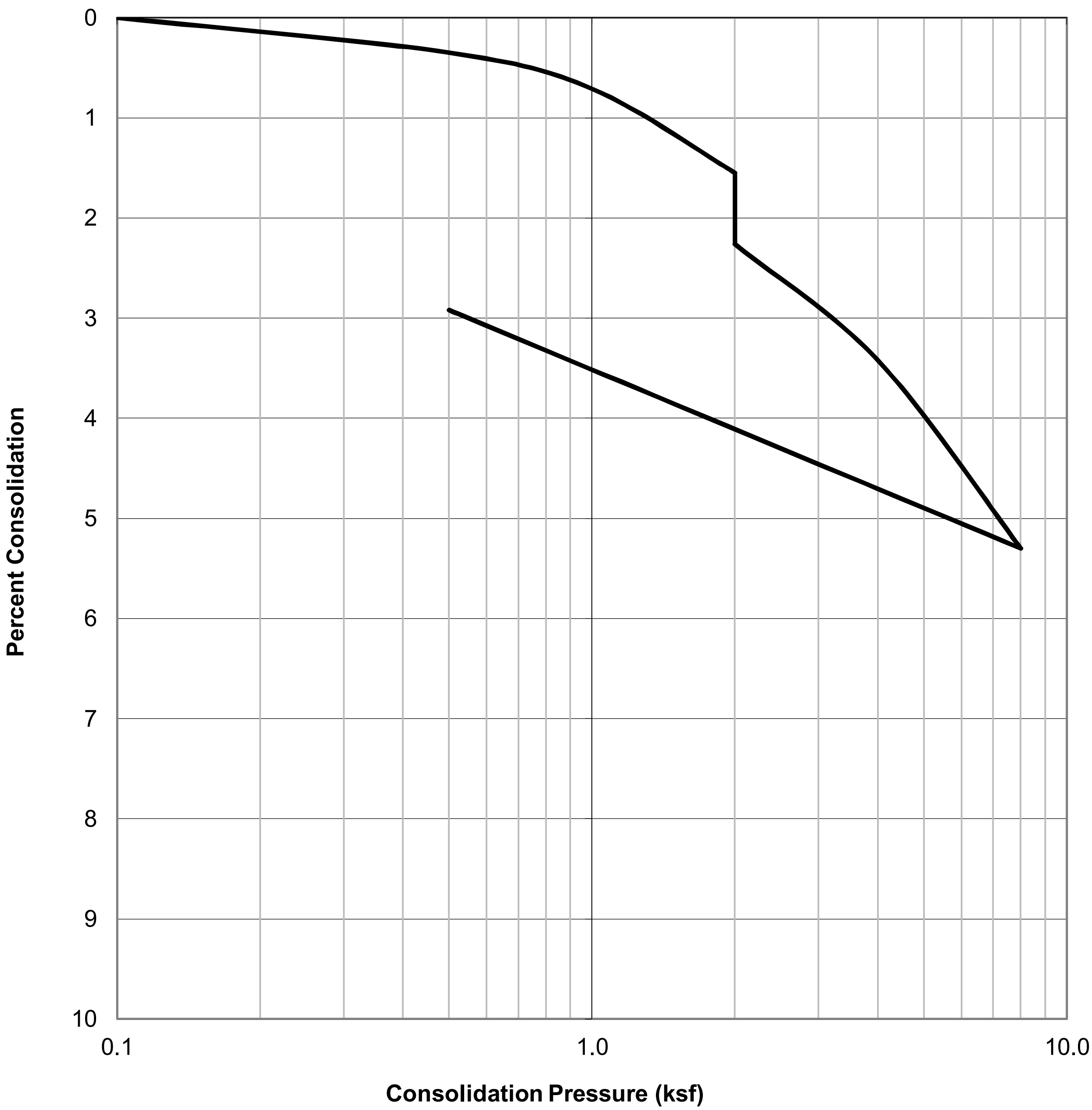
Figure B13

WATER ADDED AT 2.0 KSF



SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B2@35	Dark Brown Clay (CL)	91.8	27.0	30.0

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CONSOLIDATION TEST RESULTS

ASTM D-2435

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Figure B14

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	1200 (Corrosive)

SUMMARY OF LABORATORY CHLORIDE CONTENT TEST RESULTS
EPA NO. 325.3

Sample No.	Chloride Ion Content (%)
B1@20-25'	0.007

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

GEOCON	**CORROSIVITY TEST RESULTS**	Project No.: W1153-06-01
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