GEOTECHNICAL INVESTIGATION

PROPOSED MULTI-FAMILY RESIDENTIAL DEVELOPMENT SOUTH HILGARD AVENUE LOS ANGELES, CALIFORNIA

TRACT: TR 10690 LOTS: 6-8

GEOCON
WEST, INC.

GEOTECHNICAL
ENVIRONMENTAL
MATERIALS

PREPARED FOR

UCLA CAPITAL PROGRAMS
LOS ANGELES, CALIFORNIA

PROJECT NO. A9060-06-18

AUGUST 15, 2019

Project No. A9060-06-18
August 15, 2019

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Subject: GEOTECHNICAL INVESTIGATION
PROPOSED MULTI-FAMILY RESIDENTIAL DEVELOPMENT
SOUTH HILGARD AVENUE, LOS ANGELES, CALIFORNIA
TRACT: TR 10690 LOTS: 6-8

Dear Mr. Voltz:

In accordance with your authorization of our proposal dated June 25, 2019, we have performed a geotechnical investigation for the proposed multi-family residential development located at South Hilgard Avenue (Tract: TR 10690, Lots: 6-8) 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 multi-family residential development located at South Hilgard Avenue (Tract: TR 10690, Lots: 6-8) 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 July 16, 2019, by excavating four 8-inch diameter boring to depths of approximately 30½ to 55½ feet below the existing ground surface utilizing a truck-mounted hollow-stem auger drilling machine. The approximate locations of the exploratory borings are depicted on the Site Plan (see Figure 2). A detailed discussion of the field investigation, including boring logs, is presented in Appendix A.

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

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

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

2. SITE AND PROJECT DESCRIPTION

The subject site is located at South Hilgard Avenue (Tract: TR 10690, Lots: 6-8) in the City of Los Angeles, California. The site is an irregularly-shaped parcel that is currently a vacant lot. The site is bounded by Lindbrook Drive to the south, by South Hilgard Avenue to the west, by a two-story residential structure to the east, and by a single-story university structure to the north. The site is relatively level to gently sloping to the south with no pronounced highs or lows. Surface water drainage at the site appears to be by sheet flow along the existing ground contours to the city streets. There is no vegetation at the site.

Based on the information provided by the Client, it is our understanding that the proposed development will consist of four to eight-story multi-family residential structures to be constructed over two subterranean parking levels. Due to the preliminary nature of the project, formal plans depicting the proposed development are not available for inclusion in this report. The proposed structures are depicted on the Site Plan (see Figure 2).

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 eight-story residential structures will be up to 1,200 kips, and wall loads will be up to 12 kips per linear foot, and column loads for the proposed four-story residential structures will be up to 800 kips, and wall loads will be up to 8 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 on an older alluvial fan surface along the southern flank of the Santa Monica Mountains, approximately 0.9 mile south of the mouth of Stone Canyon. The Santa Monica Mountains, formed during regional uplift, trend east-west on the north side of the Los Angeles Basin and constitute the southern boundary of the Transvers Ranges geomorphic province. Structurally, the Santa Monica Mountains are a broad west-plunging anticline. The south flank of the anticline is truncated by the Hollywood-Santa Monica Fault Zone which separates the mountain range from the Los Angeles Basin to the south. Rock types exposed in the eastern Santa Monica Mountains consist of metasedimentary rocks associated with the Jurassic age Santa Monica Slate, Cretaceous age igneous rocks, and Cretaceous age and Miocene age sedimentary rocks.

Regionally, the site is located within the Transverse Ranges geomorphic province, near the boundary of the Peninsular Ranges geomorphic province. The Transverse Ranges is characterized by east-west geologic structures in contrast to the Peninsular Ranges, characterized by northwest-trending geologic structures. The Santa Monica Fault Zone, located approximately 0.6 mile to the south of the site, forms the boundary between the Peninsular Ranges and the Transverse Ranges geomorphic provinces in the immediate site vicinity.

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 alluvial fan deposits consisting of sand, silt and gravel derived from the Santa Monica Mountains to the north (Dibblee, 1991; 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 of 15 feet below the existing ground surface. The artificial fill generally consists of brown to dark brown silty sand, sand with silt, and clay with sand, with varying amounts of gravel. The artificial fill is characterized as dry to slightly moist and very loose to loose or soft with varying amounts of gravel and concrete fragments. The fill is likely the result of past grading or construction activities at the site. Deeper fill may exist between excavations and in other portions of the site that were not directly explored.

4.2 Older Alluvium

Pleistocene age older alluvium was encountered beneath the fill. The alluvium generally consists of brown to reddish brown poorly graded and well-graded sand, silty sand, sand with silt, and silt with varying amounts of gravel. The alluvial soils are characterized as fine- to coarse-grained, slightly moist to saturated and loose to very dense or hard.

5. GROUNDWATER

Review of the Seismic Hazard Zone Report for the Beverly Hills Quadrangle (California Division of Mines and Geology [CDMG], 1998) indicates the historically highest groundwater level in the area is approximately 25 feet beneath the ground surface. Groundwater information presented in this document is generated from data collected in the early 1900's to the late 1990s. Based on current groundwater basin management practices, it is unlikely that groundwater levels will ever exceed the historic high levels.

Groundwater was encountered in borings B1 and B2 at depths of 52.3 and 53.0 feet below the existing ground surface, respectively. Considering the depth to groundwater encountered in our borings, groundwater is not anticipated to be encountered during construction. However, it is not uncommon for groundwater levels to vary seasonally or for groundwater seepage conditions to develop where none previously existed, especially in impermeable fine-grained soils which are heavily irrigated or after seasonal rainfall. In addition, recent requirements for stormwater infiltration could result in shallower seepage conditions in the immediate site vicinity. Proper surface drainage of irrigation and precipitation will be critical for future performance of the project. Recommendations for drainage are provided in the S*urface Drainage* section of this report (see Section 7.23).

6. GEOLOGIC HAZARDS

6.1 Surface Fault Rupture

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

The site is not within a state-designated Alquist-Priolo Earthquake Fault Zone (CGS, 2019a and 2019b; CGS, 2018b) nor a city-designated Preliminary Fault Rupture Study Area (City of Los Angeles, 2019) for surface fault rupture hazards. No active or potentially active faults with the potential for surface fault rupture are known to pass directly beneath the site. Therefore, the potential for surface rupture due to faulting occurring beneath the site during the design life of the proposed development is considered low. However, the site is located in the seismically active Southern California region, and could be subjected to moderate to strong ground shaking in the event of an earthquake on one of the many active Southern California faults. The faults in the vicinity of the site are shown in Figure 3, Regional Fault Map.

The nearest surface trace of an active fault to the site is the Santa Monica Fault Zone located approximately 0.6 mile to the south (CGS, 2018b). Other nearby active faults are the Newport-Inglewood Fault Zone, the Hollywood Fault, the Raymond Fault, the Malibu Coast Fault, and the Northridge Fault located approximately 1.5 miles east, 2.3 miles northeast, 11.6 miles east-northeast, 12 miles west, and 12.3 miles north of the site, respectively (Ziony and Jones, 1989). The active San Andreas Fault Zone is located approximately 39 miles northeast of the site (Ziony and Jones, 1989).

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

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	68	E
Long Beach	March 10, 1933	6.4	41	SE
Tehachapi	July 21, 1952	7.5	73	NW
San Fernando	February 9, 1971	6.6	24	N
Whittier Narrows	October 1, 1987	5.9	21	E
Sierra Madre	June 28, 1991	5.8	29	ENE
Landers	June 28, 1992	7.3	115	E
Big Bear	June 28, 1992	6.4	93	E
Northridge	January 17, 1994	6.7	12	NW
Hector Mine	October 16, 1999	7.1	129	ENE
Ridgecrest	July 5, 2019	7.1	127	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 summarizes site-specific design criteria obtained from the 2016 California Building Code (CBC; Based on the 2015 International Building Code [IBC] and ASCE 7-10), Chapter 16 Structural Design, Section 1613 Earthquake Loads. The data was calculated using the computer program *U.S. Seismic Design Maps*, provided by the USGS. The short spectral response uses a period of 0.2 second. We evaluated the Site Class based on the discussion in Section 1613.3.2 of the 2016 CBC and Table 20.3-1 of ASCE 7-10. The values presented on the following page are for the risk-targeted maximum considered earthquake (MCE_R).

2016 CBC SEISMIC DESIGN PARAMETERS

Parameter	Value	2016 CBC Reference
Site Class	D	Section 1613.3.2
MCE_R Ground Motion Spectral Response Acceleration – Class B (short), S_S	2.241g	Figure 1613.3.1(1)
MCE_R Ground Motion Spectral Response Acceleration – Class B (1 sec), S_1	0.824g	Figure 1613.3.1(2)
Site Coefficient, F_A	1.0	Table 1613.3.3(1)
Site Coefficient, F_V	1.5	Table 1613.3.3(2)
Site Class Modified MCE_R Spectral Response Acceleration (short), S_{MS}	2.241g	Section 1613.3.3 (Eqn 16-37)
Site Class Modified MCE_R Spectral Response Acceleration – (1 sec), S_{M1}	1.236g	Section 1613.3.3 (Eqn 16-38)
5% Damped Design Spectral Response Acceleration (short), S_{DS}	1.494g	Section 1613.3.4 (Eqn 16-39)
5% Damped Design Spectral Response Acceleration (1 sec), S_{D1}	0.824g	Section 1613.3.4 (Eqn 16-40)

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

ASCE 7-10 PEAK GROUND ACCELERATION

Parameter	Value	ASCE 7-10 Reference
Mapped MCE_G Peak Ground Acceleration, PGA	0.859g	Figure 22-7
Site Coefficient, F_{PGA}	1.0	Table 11.8-1
Site Class Modified MCE_G Peak Ground Acceleration, PGA_M	0.859g	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 2016 California Building Code and ASCE 7-10, the MCE is to be utilized for the evaluation of liquefaction, lateral spreading, seismic settlements, and it is our understanding that the intent of the Building code is to maintain "Life Safety" during a MCE event. The Design Earthquake Ground Motion (DE) is the level of ground motion that has a 10 percent chance of exceedance in 50 years, with a statistical return period of 475 years.

Deaggregation of the MCE peak ground acceleration was performed using the USGS online Unified Hazard Tool, 2008 Conterminous U.S. Dynamic edition. The result of the deaggregation analysis indicates that the predominant earthquake contributing to the MCE peak ground acceleration is characterized as a 6.81 magnitude event occurring at a hypocentral distance of 4.72 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 6.74 magnitude occurring at a hypocentral distance of 11.03 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 Beverly Hills Quadrangle (1999) indicates that the site is not located in 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. As stated previously, the soils encountered at the site are Pleistocene age older alluvial fan deposits (CGS, 2012; Dibblee, 1991) which are typically dense and not prone to liquefaction. Based on these considerations, it is our opinion that the potential for liquefaction and associated ground deformations at the site is considered very low.

6.5 Slope Stability

The topography at the site is relatively level to sloping gently to the south and is not considered susceptible to slope stability hazards. The site is located within a City of Los Angeles Hillside Grading Area but is not located within a city-designated Hillside Ordinance Area (City of Los Angeles, 2019). Additionally, the site is not within an area identified as having a potential for seismic slope instability (CDMG, 1999). There are no known landslides near the site, nor is the site in the path of any known or potential landslides. Therefore, the potential for slope stability hazards to adversely affect the proposed development is considered low.

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 not located within a potential inundation area for upstream dams or reservoirs. 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 a Flood Zone X as defined by the Federal Emergency Management Agency (FEMA, 2019; LACDPW, 2019b). Zone X is defined as an area of minimal risk of flooding (LACDPW, 2019b).

6.8 Oil Fields & Methane Potential

Based on a review of the California Division of Oil, Gas and Geothermal Resources (DOGGR) Well Finder Website, the site is not located within the limits of an oilfield and oil or gas wells are not located in the immediate site vicinity (DOGGR, 2019). However, due to the voluntary nature of record reporting by the oil well drilling companies, wells may be improperly located or not shown on the location map and undocumented wells could be encountered during construction. Any wells encountered during construction will need to be properly abandoned in accordance with the current requirements of the DOGGR.

The site is not located within the boundaries of a city-designated Methane Zone or Methane Buffer Zone (City of Los Angeles, 2019). Since the site is not located within the boundaries of a known oil field, the potential for the presence of methane or other volatile gases at the site is considered low.

However, should it be determined that a methane study is required for the proposed development it is recommended that a qualified methane consultant be retained to perform the study and provide mitigation measures as necessary.

6.9 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 15 feet of existing artificial fill was encountered during site exploration. The existing fill encountered is believed to be the result of past grading, construction, and demolition activities at the site. Deeper fill may exist in other areas of the site that were not directly explored. Excavation for the proposed subterranean parking level is anticipated to penetrate through the existing fill and expose competent alluvium throughout the excavation bottom. If needed, the existing artificial fill and site soils are suitable for re-use as an engineered fill provided the procedures outlined in the *Grading* section of this report are followed (see Section 7.4).

7.1.2 Based on these considerations, is recommended that the proposed structure be supported on a conventional spread foundation system deriving support in the competent older alluvium found below a depth of 20 feet below the street level. Foundations should be deepened as necessary to penetrate through any unsuitable or disturbed soils as necessary. It is recommended that the subgrade exposed at the excavation bottom be proof-rolled with heavy equipment prior to construction. All foundation excavations must be observed and approved in writing by the Geotechnical Engineer prior to placement of steel or concrete. Recommendations for *Conventional Foundation Design* are provided in Section 7.5.

7.1.3 Where a proposed foundation will be deeper than an existing adjacent offsite foundation, the proposed foundation must be designed to resist the surcharge imposed by the existing foundation. The surcharge area may be defined by a 1:1 projection down and away from the bottom of the foundation.

7.1.4 Due to the presence of some granular soil layers at the proposed foundation level there is a high potential for caving. The contractor should be prepared for caving conditions and should be prepared to form foundation excavations as necessary should they extend into granular soils.

7.1.5 The concrete slab-on-grade and ramp for the subterranean level may bear directly on the undisturbed older alluvium at the excavation bottom. Any soils that are disturbed should be properly compacted for slab and ramp support. Where necessary, the existing artificial fill and older alluvium are suitable for re-use as an engineered fill beneath the building slab provided the procedures outlined in the *Grading* section of this report are followed (see Section 7.4).

7.1.6 It is anticipated that excavations on the order of 24 feet in vertical height may be required for construction of the subterranean level, including foundation depths. Due to the depth of the excavation and the proximity to the property lines, city streets and adjacent offsite structures, excavations will require shoring measures in order to provide a stable excavation. Where shoring is required it is recommended that a soldier pile shoring system by 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 7.17 of this report.

7.1.7 Due to the nature of the proposed design and intent for subterranean levels, waterproofing of subterranean walls and slabs is recommended, and likely required by the building official. 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.

7.1.8 Foundations for small outlying structures, such as block walls up to 6 feet high, planter walls or trash enclosures, which will not be tied to the proposed structure, may be supported on conventional foundations bearing on a minimum of 12 inches of newly placed engineered fill which extends laterally at least 12 inches beyond the foundation area. Where excavation and proper compaction cannot be performed or is undesirable, foundations may derive support directly in the undisturbed older alluvium at or 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.

7.1.9 Based on the results of percolation testing performed at the site, a stormwater infiltration system is not considered feasible for this project. The results of the percolation testing are further discussed in the *Stormwater Infiltration* section of this report (see Section 7.22).

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

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

7.2 Soil and Excavation Characteristics

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

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

7.2.3 All onsite excavations must be conducted in such a manner that potential surcharges from existing structures, construction equipment, and vehicle loads are resisted. The surcharge area may be defined by a 1:1 projection down and away from the bottom of an existing foundation or vehicle load. Penetrations below this 1:1 projection will require special excavation measures such as sloping and shoring. Excavation recommendations are provided in the *Temporary Excavations* section of this report (see Section 7.16).

7.2.4 Due to the nature of the existing site soils encountered during the investigation at the proposed foundation elevations, the soils are considered to have a "very low" to "low" expansive potential and are classified as "non-expansive" and "expansive" in accordance with the 2016 California Building Code (CBC) Section 1803.5.3. The recommendations presented herein assume that the building foundations and slabs will derive support in these materials.

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 on-site soil to generally evaluate the corrosion potential to surface utilities. The tests were performed in accordance with California Test Method Nos. 643 and 422 and indicate that the soils are considered "corrosive" with respect to corrosion of buried ferrous metals on site. The results are presented in Appendix B (Figure B23) and should be considered for design of underground structures. Due to the corrosive potential of the soils, it is recommended that corrosion-resistant ABS pipes (or equivalent) be utilized in lieu of cast-iron for subdrains and retaining wall drains beneath the structure.

7.3.2 Laboratory tests were performed on representative samples of the on-site soil to measure the percentage of water-soluble sulfate content. Results from the laboratory water-soluble sulfate tests are presented in Appendix B (Figure B23) and indicate that the on-site materials possess a sulfate exposure class of "S0" to concrete structures as defined by 2016 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 prevent premature corrosion of buried metal pipes and concrete structures in direct contact with the soils.

7.4 Grading

7.4.1 Earthwork is anticipated to include excavation of site soils for the subterranean levels, foundations, elevator pit, and utility trenches, as well as placement of backfill for walls, ramps, and trenches.

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

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

7.4.4 Grading should commence with the removal of all existing improvements from the area to be graded. Deleterious debris such as 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.

7.4.5 The foundation system for the subterranean portion of the proposed structure may derive support in the competent older alluvium found below a depth of 20 feet below the street level. Foundations should be deepened as necessary to penetrate through any unsuitable or disturbed soils as necessary at the direction of the Geotechnical Engineer (a representative of Geocon West, Inc.).

7.4.6 The concrete slab-on-grade and ramp for the subterranean portion of the proposed structure may bear directly on the competent older alluvium at the excavation bottom and or newly placed engineered fill. It is recommended that the subgrade exposed at the excavation bottom be proof-rolled with heavy equipment prior to construction. Depending on the season, the soils at the excavation bottom may be moist and may require stabilization measures.

7.4.7 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.

7.4.8 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.).

7.4.9 All fill and backfill soils should be placed in horizontal loose layers approximately 6 to 8 inches thick, moisture conditioned to at least two percent over optimum moisture content and properly compacted in accordance with ASTM D 1557 (latest edition). All man-made fill shall be compacted to a minimum 90 percent of the maximum dry density per ASTM D 1557 (latest edition).

7.4.10 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 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 7.10).

7.4.11 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. If necessary, import soils used as structural fill should have an expansion index less than 20 and corrosivity properties that are equally or less detrimental to that of the existing onsite soils (see Figure B23).

7.4.12 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. 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).

7.4.13 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.

7.5 Conventional Foundation Design

7.5.1 Once the subterranean design and foundation loading configuration for the proposed structure proceeds to a more finalized plan, the recommendations within this report should be reviewed and revised, if necessary.

7.5.2 A conventional shallow spread foundation system may be utilized for support of the proposed structure provided foundations derive support in the competent older alluvium found below a depth of 20 feet below the existing street elevation. Foundations should be deepened as necessary to penetrate through soft or disturbed 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.

7.5.3 The contractor should be aware that formwork will likely be required to prevent caving of shallow spread foundation excavations in granular soils.

7.5.4 Continuous footings may be designed for an allowable bearing capacity of 3,200 pounds per square foot (psf), and should be a minimum of 12 inches in width, 18 inches in depth below the lowest adjacent grade, and 12 inches into the recommended bearing material.

7.5.5 Isolated spread foundations may be designed for an allowable bearing capacity of 3,500 psf, and should be a minimum of 24 inches in width, 18 inches in depth below the lowest adjacent grade, and 12 inches into the recommended bearing material.

7.5.6 The allowable soil bearing pressure above may be increased by 300 psf and 600 psf for each additional foot of foundation width and depth, respectively, up to a maximum allowable soil bearing pressure of 5,900 psf.

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

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

7.5.9 Continuous footings should be reinforced with a minimum of four No. 4 steel reinforcing bars, two placed near the top of the footing and two near the bottom. The reinforcement for isolated spread footings should be designed by the project structural engineer.

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

7.5.11 No special subgrade pre-saturation is required prior to placement of concrete. However, the moisture in the foundation subgrade should be sprinkled as necessary to maintain a moist condition at the time of concrete placement.

7.5.12 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. If unanticipated soil conditions are encountered, foundation modifications may be required.

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

7.6 Foundation Settlement

7.6.1 The maximum expected total settlement for a four-story structure supported on a conventional foundation system designed with the maximum allowable bearing value of 5,900 psf and deriving support in the competent older alluvium at the excavation bottom is estimated to be less than 1 inch and occur below the heaviest loaded structural element. Settlement of the foundation system is expected to occur on initial application of loading. Differential settlement is expected to be less than ½ inch over a distance of 20 feet.

7.6.2 The maximum expected total settlement for an eight-story structure supported on a conventional foundation system designed with the maximum allowable bearing value of 5,900 psf and deriving support in the competent older alluvium at the excavation bottom is estimated to be less than 1¼ inch and occur below the heaviest loaded structural element. Settlement of the foundation system is expected to occur on initial application of loading. Differential settlement is expected to be less than ¾ inch over a distance of 20 feet.

7.6.3 Once the design and foundation loading configurations for the proposed structure proceeds 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.

7.7 Lateral Design

7.7.1 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 0.38 may be used with the dead load forces in the newly placed engineered fill and/or in competent older alluvium.

7.7.2 Passive earth pressure for the sides of foundations and slabs poured against engineered fill or competent older alluvium may be computed as an equivalent fluid having a density of 240 pcf with a maximum earth pressure of 2,400 pcf. When combining passive and friction for lateral resistance, the passive component should be reduced by one-third. A one-third increase in the passive value may be used for wind or seismic loads.

7.8 Miscellaneous Foundations

7.8.1 Foundations for small outlying structures, such as block walls up to 6 feet in height, planter walls or trash enclosures, which will not be structurally supported by the proposed building, 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, such as adjacent to property lines, foundations may bear in the undisturbed alluvial soils 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.

7.8.2 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, 18 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.

7.8.3 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.

7.9 Concrete Slabs-on-Grade

7.9.1 Unless specifically evaluated and designed by a qualified structural engineer, the slab-on-grade for the subterranean parking level should be a minimum of 5 inches of concrete reinforced with No. 3 steel reinforcing bars placed 18 inches on center in both horizontal directions and positioned vertically near the slab midpoint. The concrete slab-on-grade may bear directly on the older alluvium found at the excavation bottom. The ramp may derive support in the undisturbed older alluvium and/or engineered fill. Any disturbed soils should be properly compacted for slab and ramp support.

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

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

7.9.4 For seismic design purposes, a coefficient of friction of 0.38 may be utilized between concrete slabs and subgrade soils without a moisture barrier, and 0.15 for slabs underlain by a moisture barrier.

7.9.5 Exterior slabs for walkways or flatwork, 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 optimum moisture content and properly compacted to at least 95 percent relative compaction, as determined by ASTM Test Method D 1557 (latest edition).

7.9.6 Crack control joints should be spaced at intervals not greater than 12 feet and should be constructed using saw-cuts or other methods as soon as practical following concrete placement. Crack control joints should extend a minimum depth of one-fourth the slab thickness. The project structural engineer should design construction joints as necessary.

7.9.7 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.

7.10 Preliminary Pavement Recommendations

7.10.1 Where new paving is to be placed, it is recommended that all existing fill and soft alluvium 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 optimum moisture content, and properly compacted to at least 95 percent relative compaction, as determined by ASTM Test Method D 1557 (latest edition).

7.10.2 The following pavement sections are based on an assumed R-Value of 35. 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.

7.10.3 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	8.5

7.10.4 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).

7.10.5 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).

7.10.6 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.

7.11 Retaining Wall Design

7.11.1 The recommendations presented below are generally applicable to the design of rigid concrete or masonry retaining walls having a maximum height of 22 feet. In the event that walls higher than 22 feet are planned, Geocon should be contacted for additional recommendations.

7.11.2 Retaining wall foundations may be designed in accordance with the recommendations provided in the *Conventional Foundation Design* section of this report (see Section 7.5).

7.11.3 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). It is recommended that the equivalent fluid pressures provided in the following table be utilized for design.

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 22	41	65

7.11.4 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 undrained walls is 90 pcf. The value includes hydrostatic pressures plus buoyant lateral earth pressures.

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

7.11.6 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.

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

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

7.11.9 In addition to the recommended earth pressure, the upper 10 feet of the subterranean wall 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.

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

7.12 Dynamic (Seismic) Lateral Forces

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

7.12.2 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 2016 CBC. The seismic load is applied as an equivalent fluid pressure along the height of the wall and the calculated loads result in a maximum load exerted at the base of the wall and zero at the top of the wall. This seismic load should be applied in addition to the active earth pressure. The earth pressure is based on half of two-thirds of PGA_M calculated from ASCE 7-10 Section 11.8.3.

7.13 Retaining Wall Drainage

7.13.1 Retaining walls not designed for hydrostatic pressure 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 5). 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.

7.13.2 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 6). 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.

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

7.13.4 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.

7.14 Elevator Pit Design

7.14.1 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 *Foundation Design* and *Retaining Wall Design* sections of this report (see Sections 7.5 and 7.11).

7.14.2 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.

7.14.3 It is recommended that the elevator pit walls and slab be waterproofed to prevent water inside of the elevator pit. Waterproofing design and installation is not the responsibility of the geotechnical engineer.

7.15 Elevator Piston

7.15.1 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.

7.15.2 Groundwater seepage was encountered at a depth of 52.3 feet below the ground surface, and the contractor should be prepared for groundwater during deep excavation. Casing will be required since caving is expected in the drilled excavation and the contractor should be prepared to use casing and should have it readily available at the commencement of drilling activities. Continuous observation of the drilling and installation of the elevator piston by the Geotechnical Engineer (a representative of Geocon West, Inc.) is required.

7.15.3 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.

7.16 Temporary Excavations

7.16.1 Excavations on the order of 24 feet in vertical height may be required for excavation of the subterranean levels, including foundation depths. The excavations are expected to expose artificial fill and alluvial soils, which may be subject to caving where granular soils are exposed.

7.16.2 Temporary vertical excavations up to 5 feet in height may be attempted where not surcharged by adjacent traffic or structures. 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 8 feet in height. A uniform slope does not have a vertical portion. Where space is limited shoring measures will be required. Recommendations for shoring are provided in the following section of this report.

7.16.3 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.

7.17 Shoring – Soldier Pile Design and Installation

7.17.1 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.

7.17.2 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.

7.17.3 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 grading/stabilization activities, foundations and/or adjacent drainage systems.

7.17.4 The proposed soldier piles may also be designed as permanent piles. The required pile depths, dimensions, and spacing should be determined and designed by the project structural and shoring engineers. All piles utilized for shoring can also be incorporated into a permanent retaining wall system (shotcrete wall) and should be designed in accordance with the earth pressure provided in the *Retaining Wall Design* section of this report (see Section 7.11).

7.17.5 Drilled cast-in-place soldier piles should be placed no closer than three 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 240 psf per foot. 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.

7.17.6 Groundwater seepage was encountered at a depth of 52.3 feet below the ground surface, and the contractor should be prepared for groundwater during pile installation. Piles placed below the water level require the use of a tremie to place the concrete into the bottom of the hole. A tremie should consist of a rigid, water-tight tube having a diameter of not less than 6 inches with a hopper at the top. The tube should be equipped with a device that will close the discharge end and prevent water from entering the tube while it is being charged with concrete. The tremie should be supported so as to permit free movement of the discharge end over the entire top surface of the work and to permit rapid lowering when necessary to retard or stop the flow of concrete. The discharge end should be closed at the start of the work to prevent water entering the tube and should be entirely sealed at all times, except when the concrete is being placed. The tremie tube should be kept full of concrete. The flow should be continuous until the work is completed, and the resulting concrete seal should be monolithic and homogeneous. The tip of the tremie tube should always be kept about 5 feet below the surface of the concrete and definite steps and safeguards should be taken to ensure that the tip of the tremie tube is never raised above the surface of the concrete.

7.17.7 A special concrete mix should be used for concrete to be placed below water. The design should provide for concrete with an unconfined compressive strength psi of 1,000 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.

7.17.8 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.

7.17.9 The frictional resistance between the soldier piles and retained soil may be used to resist the vertical component of load. The coefficient of friction may be taken as 0.38 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 downward loads. The downward capacity may be determined using a frictional resistance of 700 psf per foot.

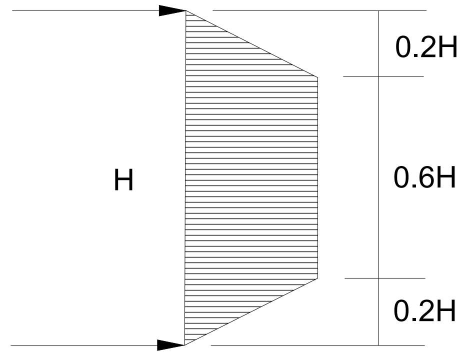
7.17.10 Due to the nature of the site soils, it is expected that continuous lagging between soldier piles will be required. However, it is recommended that the exposed soils be observed by the Geotechnical Engineer (a representative of Geocon West, Inc.), to verify the presence of any cohesive soils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

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

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

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 24	33	21H

Trapezoidal Distribution of Pressure



7.17.13 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.

7.17.14 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.

7.17.15 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.

7.17.16 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.

7.17.17 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 be minimized as much as possible 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.

7.17.18 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.

7.17.19 Due to the depth of the depth of the excavation and proximity to adjacent structures, it is suggested that prior to excavation the existing improvements be inspected to document the present condition. For documentation purposes, photographs should be taken of preconstruction distress conditions and level surveys of adjacent grade and pavement should be considered. During excavation activities, the adjacent structures and pavement should be periodically inspected for signs of distress. In the even that distress or settlement is noted, an investigation should be performed and corrective measures taken 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.

7.18 Temporary Tie-Back Anchors

7.18.1 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.

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

- 7 feet below the top of the excavation – 900 pounds per square foot
- 15 feet below the top of the excavation – 1,300 pounds per square foot

7.18.3 Depending on the techniques utilized, and the experience of the contractor performing the installation, a maximum allowable friction capacity of 3.0 kips per linear foot for post-grouted anchors (for a minimum 20 foot length beyond the active wedge) may be assumed for design purposes. Only the frictional resistance developed beyond the active wedge should be utilized in resisting lateral loads.

7.19 Anchor Installation

7.19.1 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.

7.20 Anchor Testing

7.20.1 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.

7.20.2 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.

7.20.3 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.

7.20.4 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.

7.20.5 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.

7.21 Internal Bracing

7.21.1 Rakers may be utilized to brace the soldier piles in lieu of tieback anchors. The raker bracing could be supported laterally by temporary concrete footings (deadmen) or by the permanent, interior footings. For design of such temporary footings or deadmen, poured with the bearing surface normal to rakers inclined at 45 degrees, a bearing value of 2,500 psf in competent older alluvial soil may be used, provided the shallowest point of the footing is at least 1 foot below the lowest adjacent grade. The client should be aware that the utilization of rakers could significantly impact the construction schedule due to their intrusion into the construction site and potential interference with equipment. The structural engineer should review the shoring plan to determine if the raker footings conflict with the structural foundation system.

7.22 Stormwater Infiltration

7.22.1 During the July 16, 2018, site exploration, borings B1 and B2 were utilized to perform percolation testing. The borings were advanced to the depths listed in the table below. Slotted casing was placed in the borings, and the annular space between the casings and excavations were filled with gravel. The borings were then filled with water to pre-saturate the soils. The casings were refilled with water and percolation test readings were performed after repeated flooding of the cased excavation. Based on the test results, the measured percolation rate and design infiltration rate, for the earth materials encountered, are provided in the following table. These values have been calculated in accordance with the Boring Percolation Test Procedure in the County of Los Angeles Department of Public Works GMED *Guidelines for Geotechnical Investigation and Reporting, Low Impact Development Stormwater Infiltration* (June 2017). Percolation test field data and calculation of the measured percolation rate and design infiltration rate are provided on Figures 7 and 8.

Boring	Soil Type	Infiltration Depth (ft)	Measured Percolation Rate (in / hour)	Design Infiltration Rate (in / hour)
B1	SP-SM	30-42	0.03	0.02
B2	SW	30-43	0.01	0.01

7.22.2 The results of the percolation testing indicated that the infiltration rate within the alluvial soils is less than the generally accepted minimally required infiltration rate of 0.3 inches per hour. Therefore, based on these considerations, a stormwater infiltration system is not recommended for this development. It is suggested that stormwater be retained, filtered and discharged in accordance with the requirements of the local governing agency.

7.23 Surface Drainage

7.23.1 Proper surface drainage is critical to the future performance of the project. Uncontrolled infiltration of irrigation excess and storm runoff into the supporting 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.

7.23.2 All site drainage should be collected and controlled in non-erosive drainage devices. Drainage should not be allowed to pond anywhere on the site, and especially not against any foundation or retaining wall. The site should be graded and maintained such that surface drainage is directed away from structures in accordance with 2016 CBC 1804.4 or other applicable standards. In addition, drainage should not be allowed to flow uncontrolled over any descending slope. The proposed structure should be provided with roof gutters. Discharge from downspouts, roof drains and scuppers 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 engineered fill providing foundation support. Landscape irrigation is not recommended within 5 feet of the building perimeter footings except when enclosed in protected planters.

7.23.3 Positive site drainage should be provided away from structures, pavement, and the tops of slopes to swales or other controlled drainage structures.

7.24 Plan Review

7.24.1 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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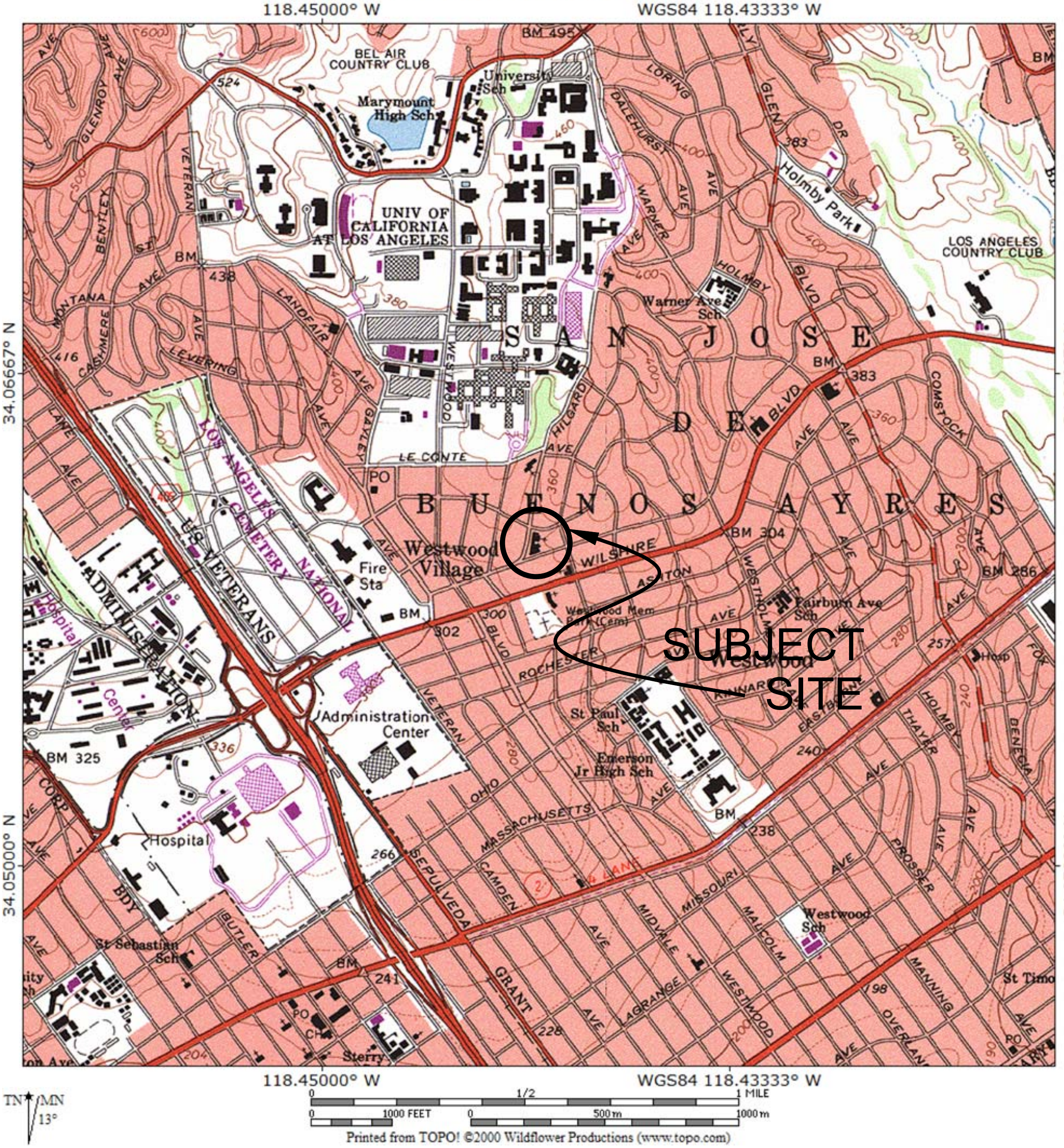

REFERENCE:U.S.G.S. TOPOGRAPHIC MAPS, 7.5 MINUTE SERIES, BEVERLY HILLS, CA QUADRANGLE

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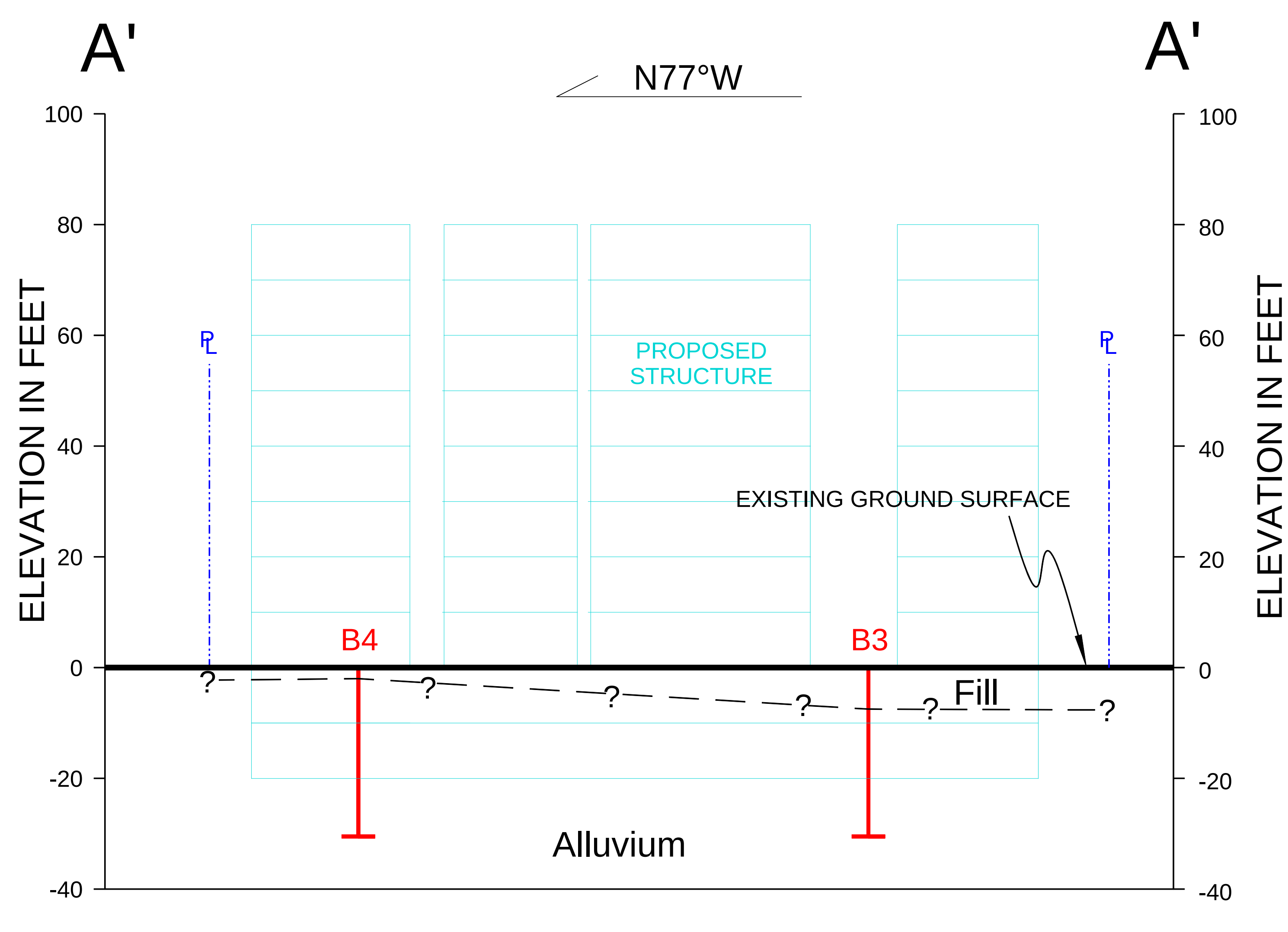
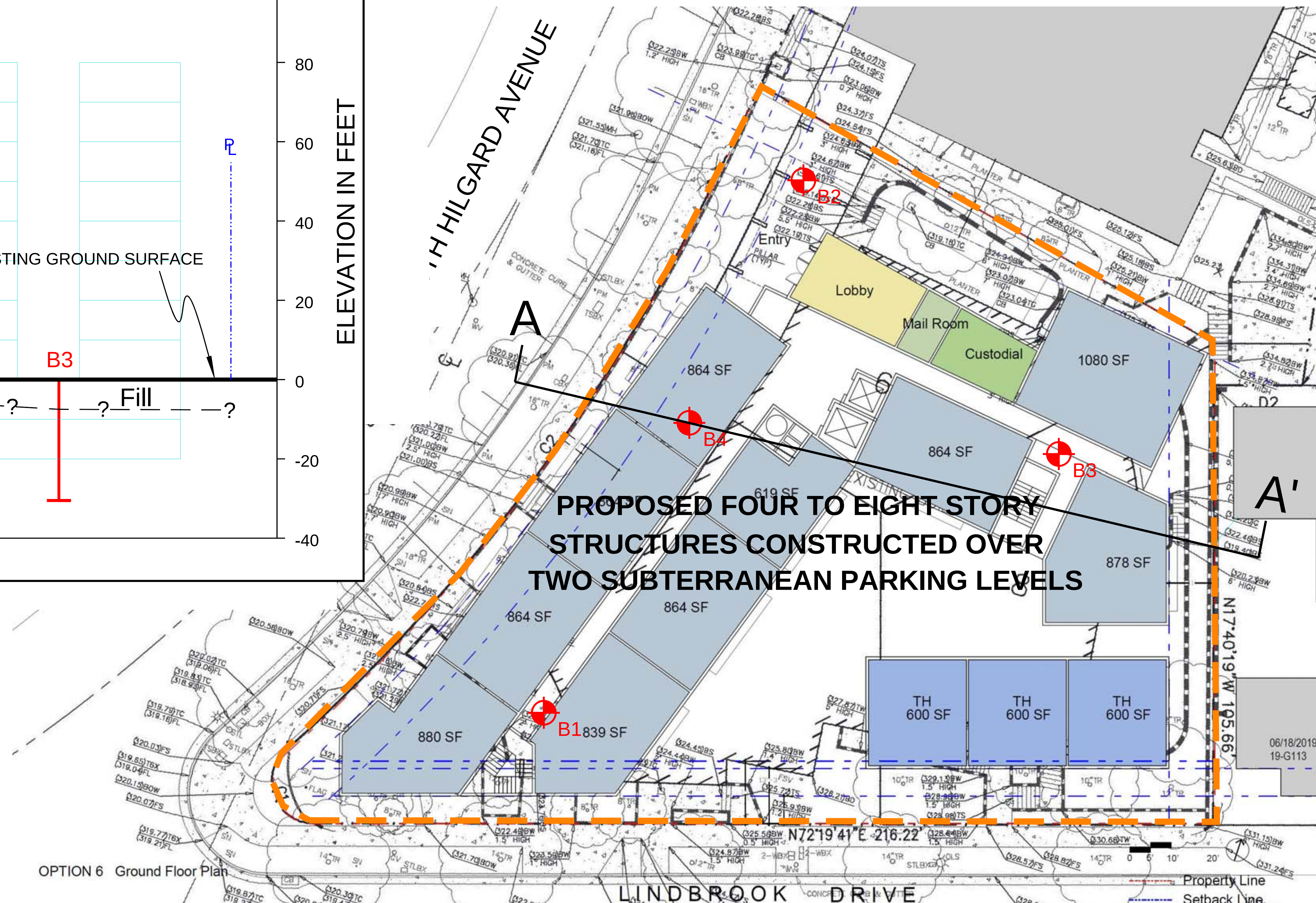
VICINITY MAP

SOUTH HILGARD AVENUE
LOS ANGELES, CALIFORNIA

DRAFTED BY: RA	CHECKED BY: SFK	AUGUST 2019	PROJECT NO. A9060-06-18	FIG. 1





LEGEND

- B4 — Location of Boring
- Location of Property Line
- A A' — Location of Cross Section

0 40' 80'

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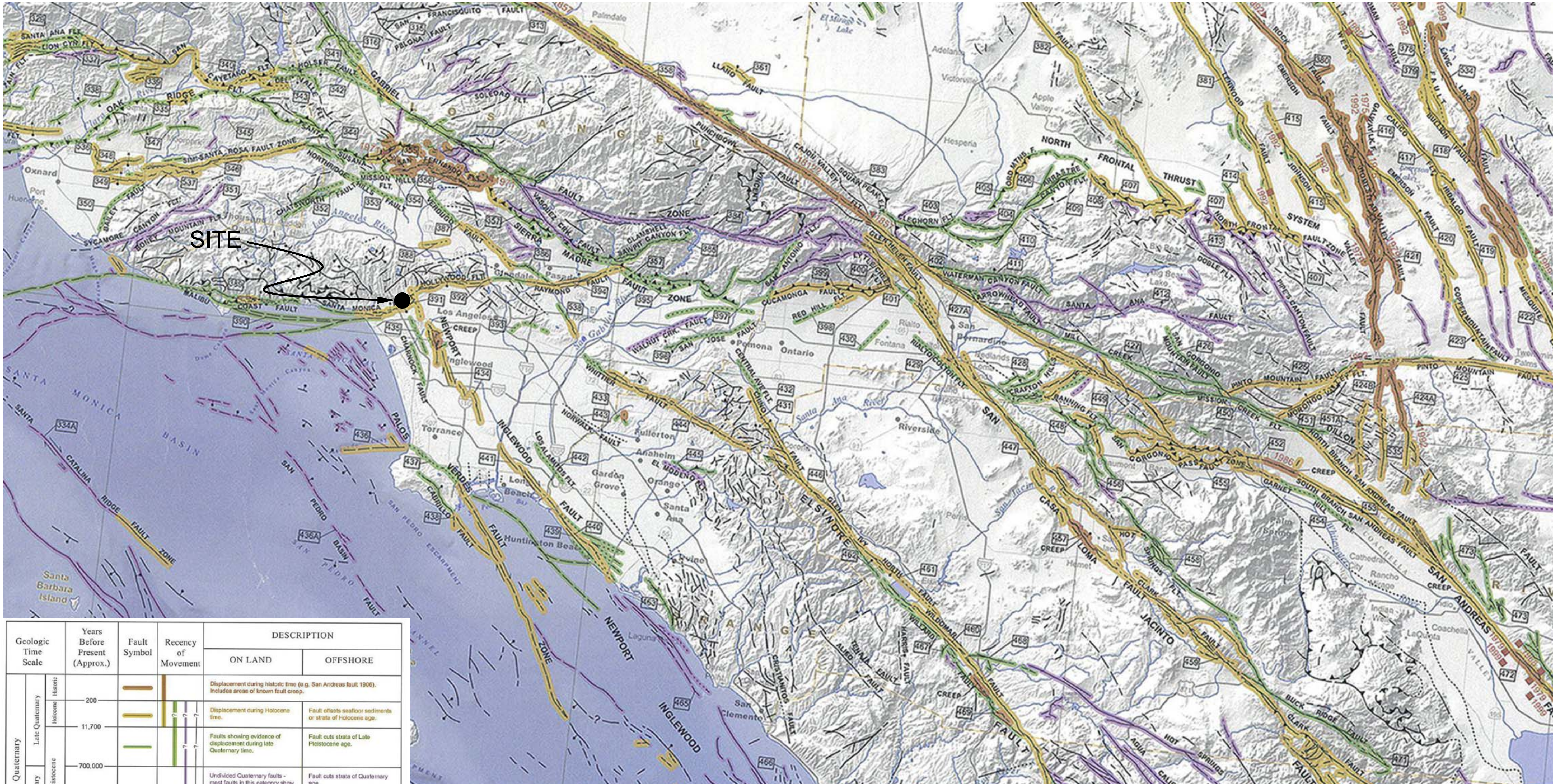
SITE PLAN

SOUTH HILGARD AVENUE
LOS ANGELES, CALIFORNIA

AUGUST 2019	PROJECT NO. A9060-06-18	FIG. 2

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



Geologic Time Scale			Years Before Present (Approx.)	Fault Symbol	Recency of Movement	DESCRIPTION: ON LAND	DESCRIPTION: OFFSHORE
Quaternary	Late Quaternary	Historic				Displacement during historic time (e.g. San Andreas fault 1906). Includes areas of known fault creep.	
			200				
		Holocene				Displacement during Holocene time.	Fault offsets seafloor sediments or strata of Holocene age.
			11,700				
		Pleistocene				Faults showing evidence of displacement during late Quaternary time.	Fault cuts strata of Late Pleistocene age.
			700,000				
	Early Quaternary					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.
			1,600,000*				
Pre-Quaternary						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.
			4.5 billion (Age of Earth)				

* 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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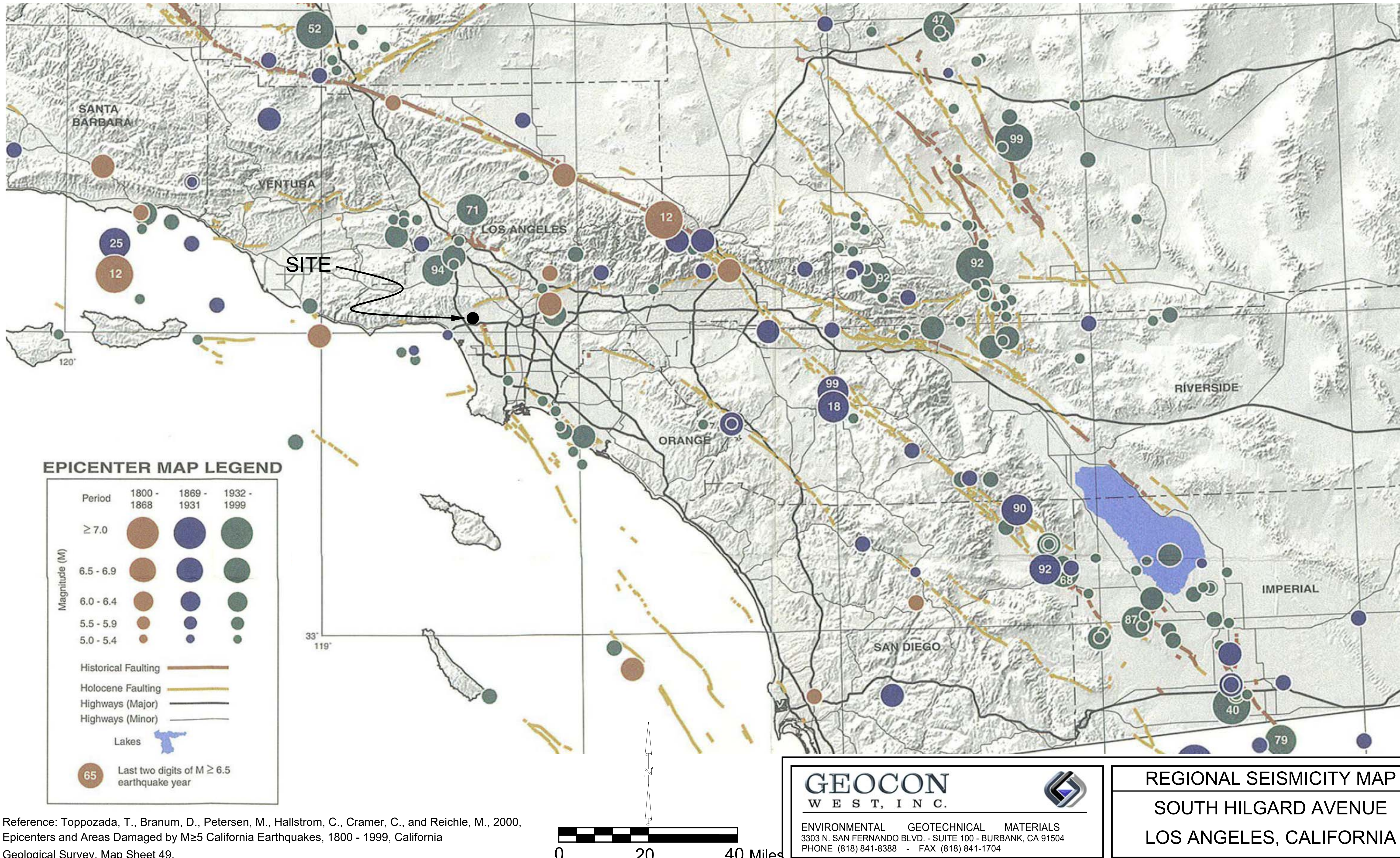
REGIONAL FAULT MAP

SOUTH HILGARD AVENUE
LOS ANGELES, CALIFORNIA

AUGUST 2019	PROJECT NO. A9060-06-18	FIG. 3



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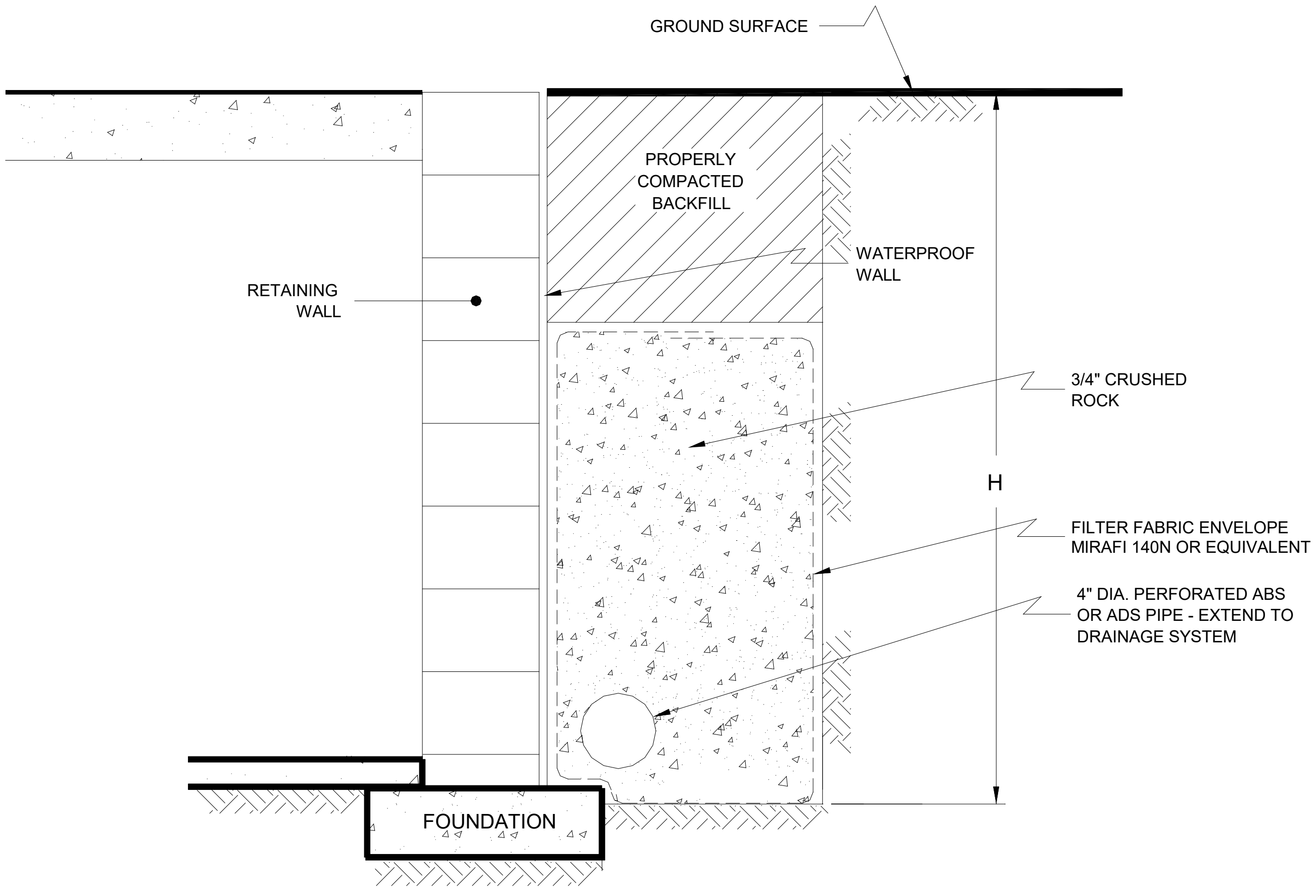
REGIONAL SEISMICITY MAP
SOUTH HILGARD AVENUE
LOS ANGELES, CALIFORNIA

AUGUST 2019 | PROJECT NO. A9060-06-18 | FIG. 4



NO SCALE

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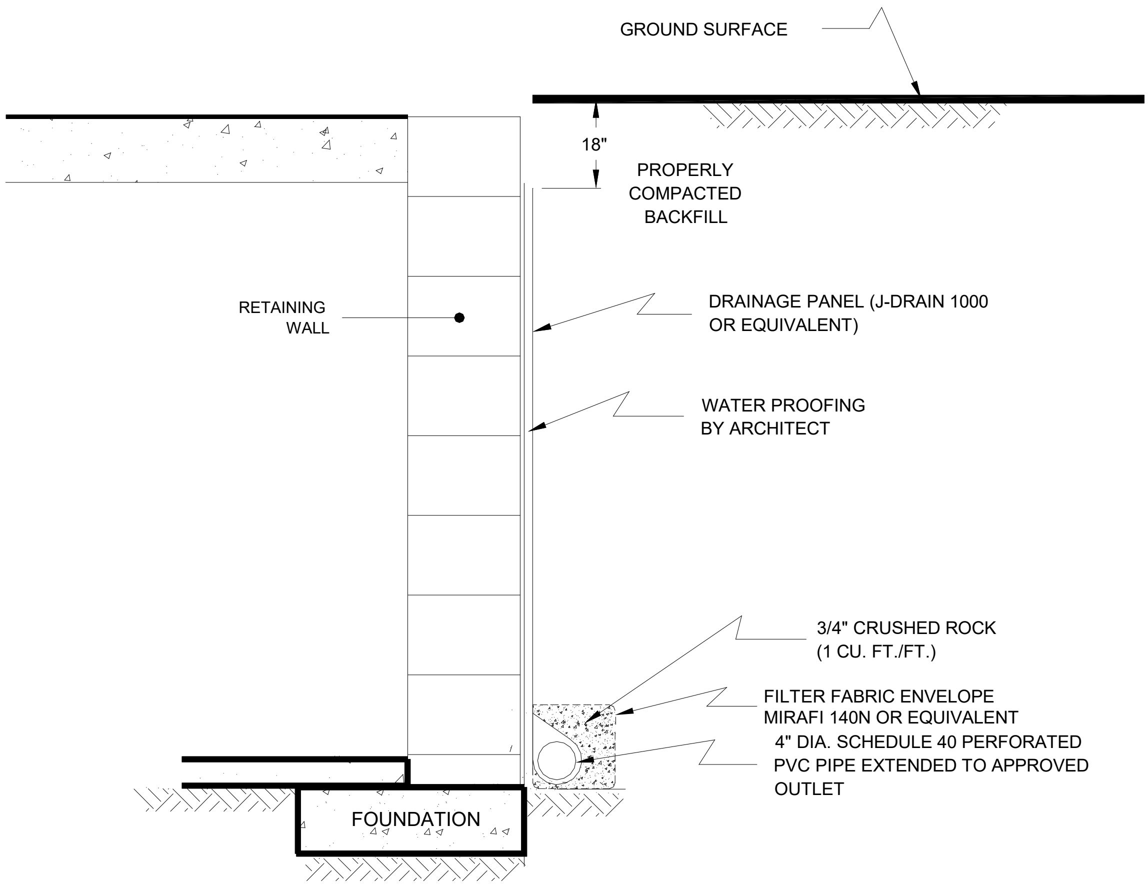
RETAINING WALL DRAIN DETAIL

SOUTH HILGARD AVENUE
LOS ANGELES, CALIFORNIA

AUGUST, 2019 | PROJECT NO. A9060-06-18 | FIG. 5



NO SCALE

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RETAINING WALL DRAIN DETAIL

SOUTH HILGARD AVENUE
LOS ANGELES, CALIFORNIA

DRAFTED BY: HHD	CHECKED BY: HHD	AUGUST, 2019	PROJECT NO. A9060-06-18	FIG. 6

BORING PERCOLATION TEST FIELD LOG

Date:	Wednesday, July 17, 2019	**Boring/Test Number:**	B1
Project Number:	A9060-06-18	**Diameter of Boring:**	8 inches
Project Location:	UCLA - Hilgard Faculty Housing	**Diameter of Casing:**	2 inches
Earth Description:	Sand with Silt (SP-SM)	**Depth of Boring:**	42 feet
Tested By:	CB	**Depth to Invert of BMP:**	30 feet
Liquid Description:	Water	**Depth to Water Table:**	-- feet
Measurement Method:	Sounder	**Depth to Initial Water Depth (d_1):**	361.44 inches
Start Time for Pre-Soak:	8:00 AM	**Water Remaining in Boring (Y/N):**	Yes
Start Time for Standard:	9:00 AM	**Standard Time Interval Between Readings:**	30

Reading Number	Time Start (hh:mm)	Time End (hh:mm)	Elapsed Time Δtime (min)	Water Drop During Standard Time Interval, Δd (in)	Soil Description Notes Comments
1	8:00 AM	8:30 AM	30	9.0	
2	8:30 AM	9:00 AM	30	1.1	
3	9:00 AM	9:30 AM	30	1.2	
4	9:30 AM	10:00 AM	30	5.8	
5	10:00 AM	10:30 AM	30	3.6	
6	10:30 AM	11:00 AM	30	1.3	Stabilized Readings
7	11:00 AM	11:30 AM	30	1.1	Achieved with Readings
8	11:30 AM	12:00 PM	30	1.0	6, 7, and 8

MEASURED PERCOLATION RATE & DESIGN INFILTRATION RATE CALCULATIONS*

* Calculations Below Based on Stabilized Readings Only

Boring Radius, r: 4 inches

Test Section Height, h: 142.6 inches

$Test\ Section\ Surface\ Area, A = 2\pi rh + \pi r^2$

A = **3633** in^2

$Discharged\ Water\ Volume, V = \pi r^2 \Delta d$

$Percolation\ Rate = \left(\frac{V/A}{\Delta T}\right)$

Reading 6	V = **66** in^3	Percolation Rate =	**0.04**	inches/hour
Reading 7	V = **54** in^3	Percolation Rate =	**0.03**	inches/hour
Reading 8	V = **48** in^3	Percolation Rate =	**0.03**	inches/hour

Measured Percolation Rate = **0.03** inches/hour

Reduction Factors

Boring Percolation Test, RF_t = 2

Site Variability, RF_v = 1

Long Term Siltation, RF_s = 1

$Total\ Reduction\ Factor, RF = RF_t \times RF_v \times RF_s$

Total Reduction Factor = 2

Design Infiltration Rate

$Design\ Infiltration\ Rate = Measured\ Percolation\ Rate\ /RF$

Design Infiltration Rate = **0.02** inches/hour

FIGURE 7

BORING PERCOLATION TEST FIELD LOG

Date:	Wednesday, July 17, 2019	**Boring/Test Number:**	B2
Project Number:	A9060-06-18	**Diameter of Boring:**	8 inches
Project Location:	UCLA - Hilgard Faculty Housing	**Diameter of Casing:**	2 inches
Earth Description:	Sand (SW)	**Depth of Boring:**	43 feet
Tested By:	CB	**Depth to Invert of BMP:**	30 feet
Liquid Description:	Water	**Depth to Water Table:**	-- feet
Measurement Method:	Sounder	**Depth to Initial Water Depth (d_1):**	360.12 inches
Start Time for Pre-Soak:	8:30 AM	**Water Remaining in Boring (Y/N):**	Yes
Start Time for Standard:	9:30 AM	**Standard Time Interval Between Readings:**	30

Reading Number	Time Start (hh:mm)	Time End (hh:mm)	Elapsed Time Δtime (min)	Water Drop During Standard Time Interval, Δd (in)	Soil Description Notes Comments
1	8:30 AM	9:00 AM	30	16.1	
2	9:00 AM	9:30 AM	30	5.8	
3	9:30 AM	10:00 AM	30	6.2	
4	10:00 AM	10:30 AM	30	2.4	
5	10:30 AM	11:00 AM	30	1.0	
6	11:00 AM	11:30 AM	30	0.6	Stabilized Readings
7	11:30 AM	12:00 PM	30	0.5	Achieved with Readings
8	12:00 PM	12:30 PM	30	0.2	6, 7, and 8

MEASURED PERCOLATION RATE & DESIGN INFILTRATION RATE CALCULATIONS*

* Calculations Below Based on Stabilized Readings Only

Boring Radius, r: 4 inches

Test Section Height, h: 155.9 inches

$$Test\ Section\ Surface\ Area, A = 2\pi rh + \pi r^2$$

A = **3968** in^2

$$Discharged\ Water\ Volume, V = \pi r^2 \Delta d$$

$$Percolation\ Rate = \left(\frac{V/A}{\Delta T}\right)$$

Reading 6	V =	**30**	in^3	Percolation Rate =	**0.02**	inches/hour
Reading 7	V =	**24**	in^3	Percolation Rate =	**0.01**	inches/hour
Reading 8	V =	**8**	in^3	Percolation Rate =	**0.00**	inches/hour

Measured Percolation Rate = **0.01** inches/hour

Reduction Factors

Boring Percolation Test, RF_t = 2

Site Variability, RF_v = 1

Long Term Siltation, RF_s = 1

$$Total\ Reduction\ Factor, RF = RF_t \times RF_v \times RF_s$$

Total Reduction Factor = 2

Design Infiltration Rate

$$Design\ Infiltration\ Rate = Measured\ Percolation\ Rate\ /RF$$

Design Infiltration Rate = **0.01** inches/hour

APPENDIX A

APPENDIX A

FIELD INVESTIGATION

The site was explored on July 16, 2019, by excavating four 8-inch diameter boring to depths of approximately 30½ to 55½ feet below the existing ground surface utilizing a truck-mounted hollow-stem auger drilling machine. Representative and relatively undisturbed samples were obtained by driving a 3 inch O. D. California Modified Sampler into the "undisturbed" soil mass with blows from a 140-pound auto-hammer falling 30 inches. The California Modified Sampler was equipped with 1-inch high by 2⅜-inch diameter brass sampler rings to facilitate soil removal and testing. Bulk samples were also obtained.

The soil conditions encountered in the borings were visually examined, classified and logged in general accordance with the Unified Soil Classification System (USCS). The logs of the borings are presented on Figures A1 through A4. The logs depict the soil and geologic conditions encountered and the depth at which samples were obtained. The logs also include our interpretation of the conditions between sampling intervals. Therefore, the logs contain both observed and interpreted data. We determined the lines designating the interface between soil materials on the logs using visual observations, penetration rates, excavation characteristics and other factors. The transition between materials may be abrupt or gradual. Where applicable, the logs were revised based on subsequent laboratory testing. The locations of the borings are shown on Figure 2.

BORING 1

ELEV. (MSL.) -- DATE COMPLETED 7/16/19

EQUIPMENT HOLLOW STEM AUGER BY: CB

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0					**ARTIFICIAL FILL** Clay with Sand, soft, slightly moist, dark brown, trace silt and fine gravel.			
2	B1@2'			SM	**ALLUVIUM** Silty Sand, poorly graded, loose, slightly moist, brown, fine-grained.	10		
6	B1@6'				- trace gravel (to 1"), some oxidation mottles, grading coarser	14	104.0	17.2
10	B1@10'			SP-SM	Sand with Silt, well-graded, medium dense, moist, brown with orange oxidation mottles, trace fine gravel.	26	120.7	8.3
14	B1@14'			SM	Silty Sand, poorly graded, medium dense, moist, reddish brown, fine-grained, trace medium-grained.	42	111.4	18.2
18	B1@18'					28	107.9	16.7
20				SW	Sand, well-graded, medium dense, moist, reddish brown with orange oxidation mottles, some silt.			
22	B1@22'			SM	Silty Sand, poorly graded, medium dense, moist, brown, fine-grained, trace medium-grained.	24	108.1	20.2
24	B1@24'			SP	Sand, poorly graded, medium dense, moist, brown, fine- to medium-grained, some coarse-grained, some silt.	42	106.4	17.4
26	B1@26'			SW	Sand, well-graded, medium dense, moist, brown, fine- to coarse-grained, trace silt.	41	111.0	8.2
28				SM	Sand with Silt, poorly graded, dense, slighlty moist, brown, fine- to medium-grained, trace coarse-grained.			

A9060-06-18 BORING LOGS.GPJ

Figure A1,
Log of Boring 1, 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.

GEOCON

PROJECT NO. A9060-06-18

BORING 1

ELEV. (MSL.) -- DATE COMPLETED 7/16/19

EQUIPMENT HOLLOW STEM AUGER BY: CB

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
30	B1@30'			SM		54	116.0	17.5
32 – 40	B1@35'			SP-SM	Silty Sand, dense, slightly moist, brown, fine- to medium-grained, trace coarse-grained.	73	116.3	13.3
40 – 42	B1@40'			SW-SM	Sand with Silt, well-graded, very dense, slightly moist, brown, trace clay.	50 (4")	114.4	16.4
42 – 48	B1@45'			SM	Silty Sand, poorly graded, medium dense, slightly moist, brown, fine- to medium-grained.	43	110.8	19.1
48 – 50				ML	Silt with Clay, hard, slightly moist, brown.			
50 – 54	B1@50'		▼ (52.3)	SP	Sand, poorly graded, dense, wet, fine- to medium-grained, brown. - saturated	51	105.4	23.1
54.5 – 55.5	B1@55'			ML	Silt, hard, wet, brown, some fine-grained sand and clay.	50 (5")	108.3	20.0

Total depth of boring: 55.5 feet
Fill to 1.5 feet.
Groundwater encountered at 52.3 feet.
Set perc well on 7/16/19.

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

A9060-06-18 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.

GEOCON

PROJECT NO. A9060-06-18

BORING 2

ELEV. (MSL.) -- DATE COMPLETED 7/16/19

EQUIPMENT HOLLOW STEM AUGER BY: CB

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'				**ARTIFICIAL FILL** Silty Sand, very loose, dry, brown, fine-grained, trace fine gravel, concrete fragments.			
2	B2@2'					2		
6	B2@6'					10	110.0	16.9
					Sand with Silt, poorly graded, loose, moist, brown with orange oxidation mottles, fine- to medium-grained, trace fine gravel.			
10	B2@10'					9	112.1	15.1
14	B2@14'				Sand with Silt, well graded, very loose, saturated, dark brown, trace fine gravel and concrete.	5	118.8	17.3
					ALLUVIUM Silty Sand, well graded, medium dense, moist, brown, trace fine gravel.			
18	B2@18'			SM		52	121.6	12.6
22	B2@22'			SW-SM	Sand with Silt, well-graded, medium dense, slightly moist, brown with black and reddish brown mottles, some gravel.	53	123.7	11.1
24	B2@24'				Sand with Gravel, well-graded, dense, slightly moist, brown with redddish brown, orange, and dark gray mottles, gravel (to 1.5"), some silt.	58	117.2	16.9
26	B2@26'			SW	- very dense	50 (6")	119.8	8.5
28								

A9060-06-18 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.

GEOCON

BORING 2

ELEV. (MSL.) -- DATE COMPLETED 7/16/19

EQUIPMENT HOLLOW STEM AUGER BY: CB

DEPTH IN FEET	SAMPLE NO.	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
30	B2@30'		SW		44	113.3	14.7
30–34			SP-SM	Sand with Silt, poorly graded, medium dense, slightly moist, reddish brown, fine- to medium-grained. - grades coarser			
34–36.5	B2@35'		SW	Sand with Gravel, well-graded, very dense, moist, brown, gravel (to 1").	50 (6")	126.8	8.0
36.5–43	B2@40'		SW	Sand, well-graded, dense, moist, brown.	59	119.8	13.1
43–48.5	B2@45'		SM	Silty Sand, poorly graded, dense, slightly moist, brown, fine-grained, trace medium-grained. - grades coarser	55	114.2	18.4
48.5–50			SW	Sand, well-graded, dense, wet, brown, some silt.			
50–53	B2@50'	▼ (53 ft)	SM	Silty Sand, poorly graded, medium dense, slightly moist, reddish brown, fine- to medium-grained.	50	113.5	15.9
53–55.5	B2@55'		SP-SM	Sand with Silt, poorly graded, medium dense, moist, brown, fine-grained, trace medium-grained.	38	109.2	16.8

Total depth of boring: 55.5 feet
Fill to 15 feet.
Groundwater encountered at 53 feet.
Set perc well on 7/16/19.

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

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

A9060-06-18 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 3

ELEV. (MSL.) -- DATE COMPLETED 7/16/19

EQUIPMENT HOLLOW STEM AUGER BY: CB

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0					**ARTIFICIAL FILL** Silty Sand, well-graded, very loose, dry to slightly moist, brown, some gravel (to 1.5").			
2	B3@2'					6	116.3	7.3
4								
6	B3@6'				- loose	9	88.2	14.7
8					**ALLUVIUM** Silty Sand, poorly graded, slightly moist, medium dense, fine- to medium-grained, brown.			
10	B3@10'			SM		31	117.2	11.5
12								
14	B3@14'			SP	Sand, poorly graded, medium dense, slightly moist, brown, fine- to medium-grained, some silt.	30	111.5	13.2
16								
18	B3@18'			SW	Sand, well graded, dense, wet, brown, trace gravel (to 1").	66	126.1	9.9
20								
22	B3@22'			SW	Sand with Gravel, well-graded, very dense, moist, brown.	50 (5")	101.3	6.5
24	B3@24'			SM	Silty Sand, medium dense, stiff, slightly moist, brown, fine-grained.	34	99.1	24.2
26	B3@26'			SP	Sand, poorly graded, very dense, slightly moist, brown, fine- to medium-grained, trace silt. - trace gravel (to 1.5")	50 (5")	122.3	10.9
28								

A9060-06-18 BORING LOGS.GPJ

Figure A3,
Log of Boring 3, 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.

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	BORING 3 ELEV. (MSL.) -- DATE COMPLETED 7/16/19 EQUIPMENT HOLLOW STEM AUGER BY: CB MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
30	B3@30'			SP	- medium dense, some silt	52	117.0	9.4
					Total depth of boring: 30.5 feet Fill to 7.5 feet. No groundwater encountered. Backfilled with soil cuttings and tamped. *Penetration resistance for 140-pound hammer falling 30 inches by auto-hammer.			

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

A9060-06-18 BORING LOGS.GPJ

SAMPLE SYMBOLS

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

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

GEOCON

PROJECT NO. A9060-06-18

BORING 4

ELEV. (MSL.) -- DATE COMPLETED 7/16/19

EQUIPMENT HOLLOW STEM AUGER BY: CB

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0					**ARTIFICIAL FILL** Silty Sand, well-graded, loose, dry, brown, some gravel, concrete fragments.			
2	B4@2'			SP-SM	**ALLUVIUM** Sand with Silt, poorly graded, very loose, slightly moist, brown, fine- to medium-grained, trace coarse-grained.	6	120.1	6.6
6	B4@6'			ML	Sandy Silt, soft, slightly moist, brown, fine- to medium-grained, trace coarse-grained, fine gravel.	5	111.4	9.0
10	B4@10'			SM	Silty Sand, poorly graded, very loose, brown with reddish brown mottles, some dark brown, fine- to medium-grained, trace fine gravel.	4	98.2	10.1
14	B4@14'				- medium dense, slightly moist, brown wiht grayish brown mottles	44	126.5	8.4
18	B4@18'			SP-SM	Sand with Silt, poorly graded, medium dense, slightly moist, brown, fine-grained, trace clay and medium-grained sand.	29	119.2	8.6
22	B4@22'			SP	Sand, poorly graded, medium dense, moist, fine- to medium-grained, dark reddish brown, some silt.	19	118.8	11.5
24	B4@24'				- loose	11	117.5	11.2
26	B4@26'					13	119.3	9.2
28				SW	Sand with Gravel, well-graded, medium dense, wet, brown, gravel (to 1.5"), some silt.			

A9060-06-18 BORING LOGS.GPJ

Figure A4,
Log of Boring 4, 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.

GEOCON

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	BORING 4 ELEV. (MSL.) -- DATE COMPLETED 7/16/19 EQUIPMENT HOLLOW STEM AUGER BY: CB	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
					MATERIAL DESCRIPTION			
30	B4@30'			SW		35	124.8	8.6
					Total depth of boring: 30.5 feet Fill to 2 feet. No groundwater encountered. Backfilled with soil cuttings and tamped. *Penetration resistance for 140-pound hammer falling 30 inches by auto-hammer.			

Figure A4,
Log of Boring 4, Page 2 of 2

A9060-06-18 BORING LOGS.GPJ

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

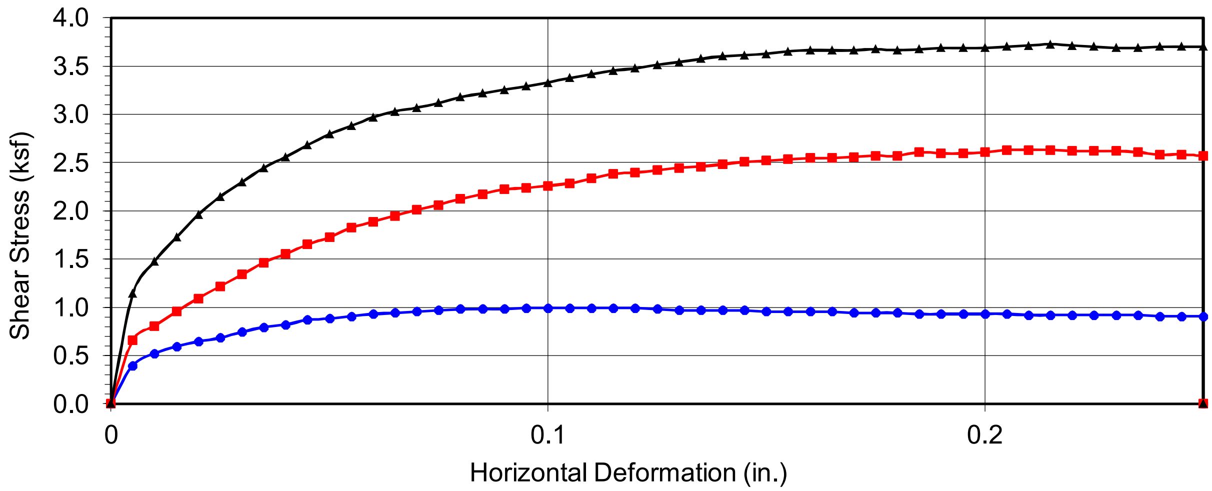
NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

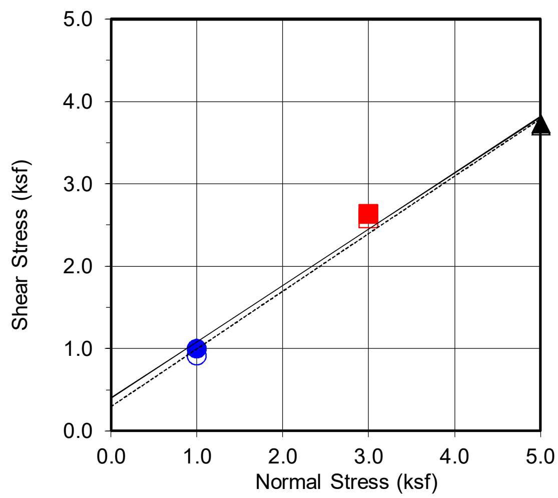
APPENDIX B

APPENDIX B

LABORATORY TESTING

Laboratory tests were performed in accordance with generally accepted test methods of the "American Society for Testing and Materials (ASTM)", or other suggested procedures. Selected samples were tested for direct shear strength, consolidation and expansion characteristics, corrosivity, corrosivity, in-place dry density and moisture content. The results of the laboratory tests are summarized in Figures B1 through B23. 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@6**
Depth (ft)	**6**
Sample Type:	Ring

Soil Identification:		
Brown Silty Sand (SM)		
Strength Parameters		
	C (psf)	ϕ (°)
Peak	401	34.3
Ultimate	296	34.9

Normal Stress (kip/ft2)	1	3	5
Peak Shear Stress (kip/ft²)	● 0.99	■ 2.63	▲ 3.73
Shear Stress @ End of Test (ksf)	○ 0.91	□ 2.57	△ 3.70
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 (%)	17.2	16.3	13.2
Initial Dry Density (pcf)	110.5	110.6	107.0
Initial Degree of Saturation (%)	88.1	83.9	62.0
Soil Height Before Shearing (in.)	1.2	1.2	1.2
Final Moisture Content (%)	19.6	16.8	15.3

GEOCON

DIRECT SHEAR TEST RESULTS

Consolidated Drained ASTM D-3080

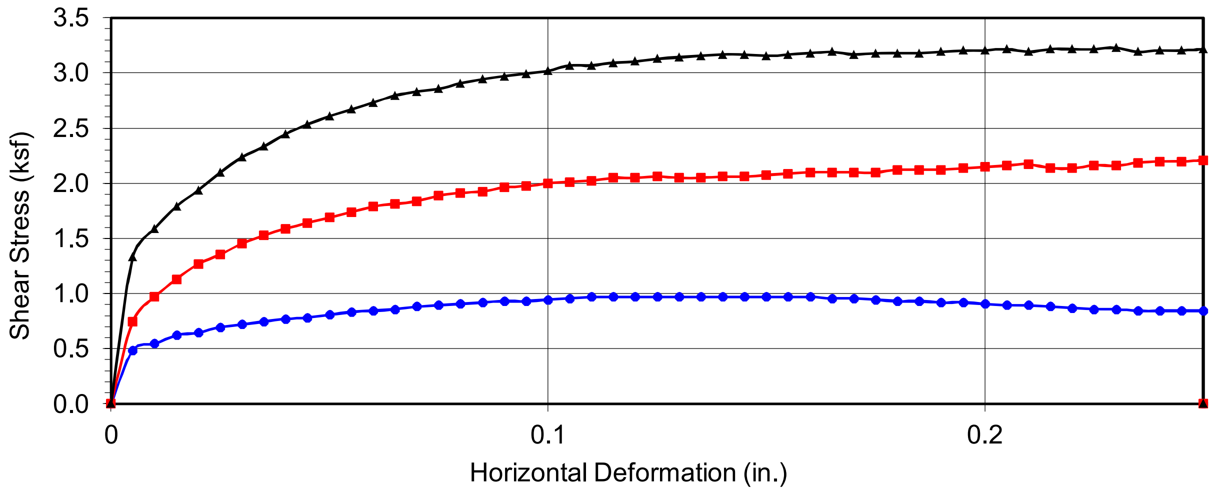
Checked by: JJK

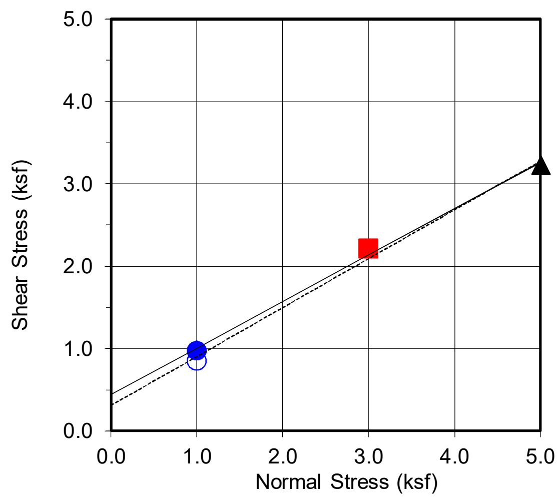
Project No.: A9060-06-18

SOUTH HILGARD AVENUE
LOS ANGELES, CALIFORNIA

August, 2019

Figure B1





Boring No.	**B2**
Sample No.	**B2@6**
Depth (ft)	**6**
Sample Type:	Ring

Soil Identification:		
Brown Silty Sand (SM)		
Strength Parameters		
	C (psf)	ϕ (^{o})
Peak	441	29.5
Ultimate	311	30.7

Normal Stress (kip/ft2)	1	3	5
Peak Shear Stress (kip/ft²)	● 0.97	■ 2.21	▲ 3.23
Shear Stress @ End of Test (ksf)	○ 0.84	□ 2.21	△ 3.22
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 (%)	17.6	16.9	18.1
Initial Dry Density (pcf)	109.2	109.9	107.6
Initial Degree of Saturation (%)	87.3	85.8	86.4
Soil Height Before Shearing (in.)	1.2	1.2	1.2
Final Moisture Content (%)	19.6	17.7	18.5

GEOCON

DIRECT SHEAR TEST RESULTS

Consolidated Drained ASTM D-3080

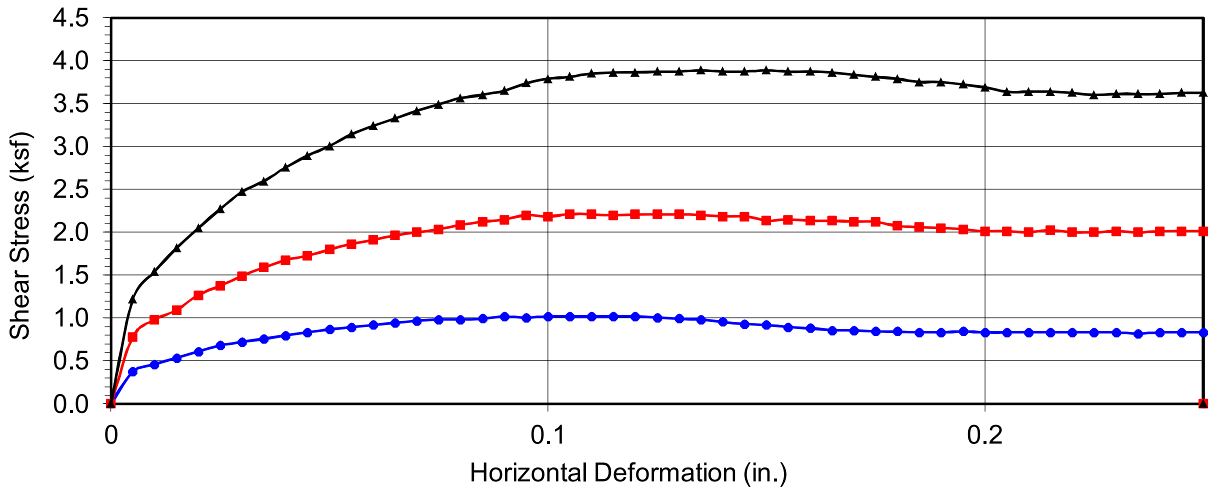
Checked by: JJK

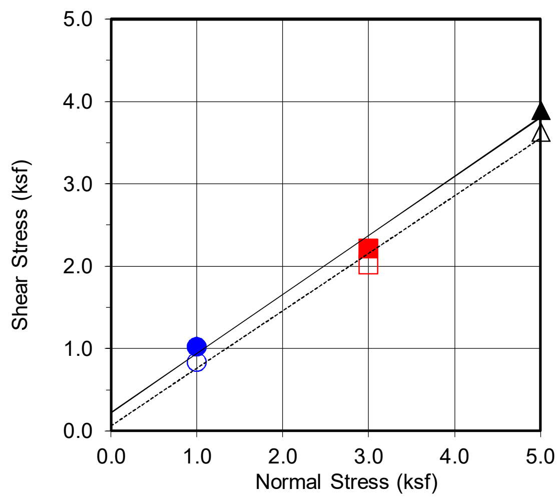
Project No.: A9060-06-18

South Hilgard Avenue
Los Angeles, California

August, 2019

Figure B2





Boring No.	**B2**
Sample No.	**B2@14**
Depth (ft)	**14**
Sample Type:	Ring

Soil Identification:		
Dark Olive Brown Sand w/ Silt (SP-SM)		
Strength Parameters		
	C (psf)	ϕ (°)
Peak	220	35.6
Ultimate	61	34.9

Normal Stress (kip/ft2)	1	3	5
Peak Shear Stress (kip/ft²)	● 1.02	■ 2.21	▲ 3.89
Shear Stress @ End of Test (ksf)	○ 0.83	□ 2.01	△ 3.63
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 (%)	17.4	17.0	17.3
Initial Dry Density (pcf)	117.1	118.0	117.6
Initial Degree of Saturation (%)	106.8	107.4	107.6
Soil Height Before Shearing (in.)	1.2	1.2	1.2
Final Moisture Content (%)	14.7	13.9	13.5

GEOCON

DIRECT SHEAR TEST RESULTS

Consolidated Drained ASTM D-3080

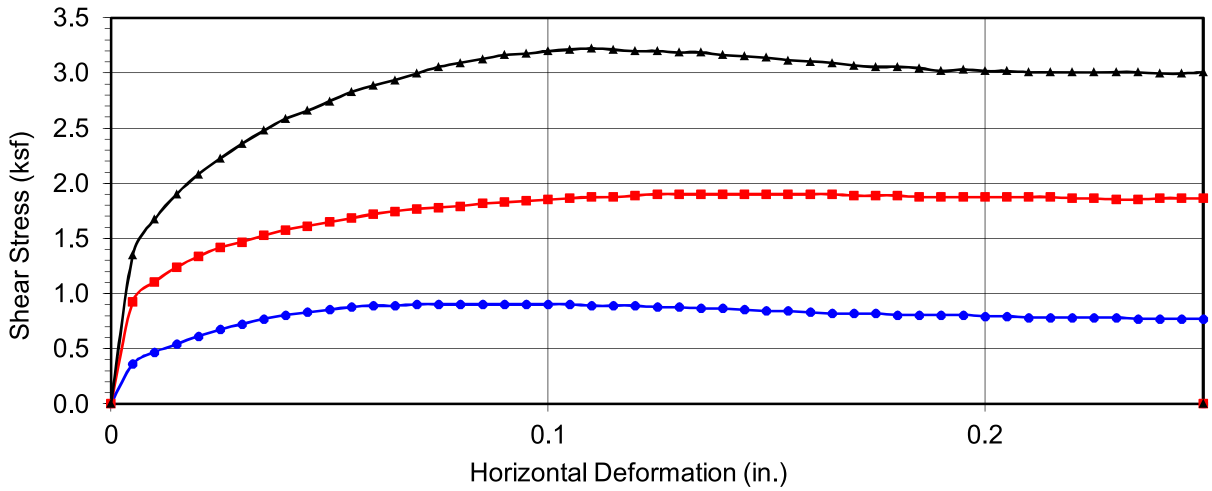
Checked by: JJK

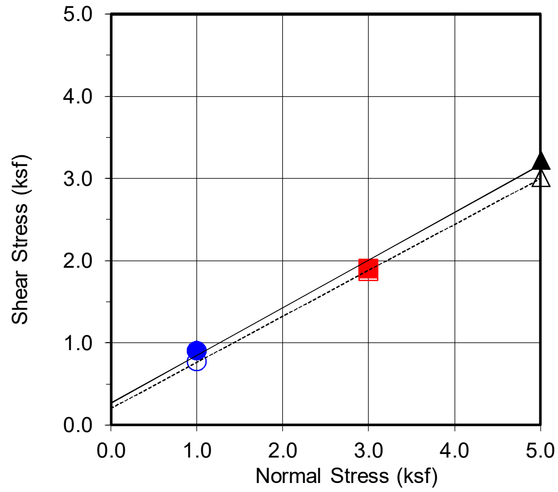
Project No.: A9060-06-18

South Hilgard Avenue
Los Angeles, California

August, 2019

Figure B3





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

Soil Identification:		
Olive Brown Silty Sand (SM)		
Strength Parameters		
	C (psf)	ϕ (^{o})
Peak	268	30.1
Ultimate	203	29.2

Normal Stress (kip/ft2)	1	3	5
Peak Shear Stress (kip/ft²)	● 0.90	■ 1.90	▲ 3.22
Shear Stress @ End of Test (ksf)	○ 0.77	□ 1.86	△ 3.01
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 (%)	23.5	22.9	20.2
Initial Dry Density (pcf)	102.4	102.7	109.3
Initial Degree of Saturation (%)	98.1	96.2	100.8
Soil Height Before Shearing (in.)	1.2	1.2	1.2
Final Moisture Content (%)	24.1	21.9	18.6

GEOCON

DIRECT SHEAR TEST RESULTS

Consolidated Drained ASTM D-3080

Checked by: JJK

Project No.: A9060-06-18

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Los Angeles, California

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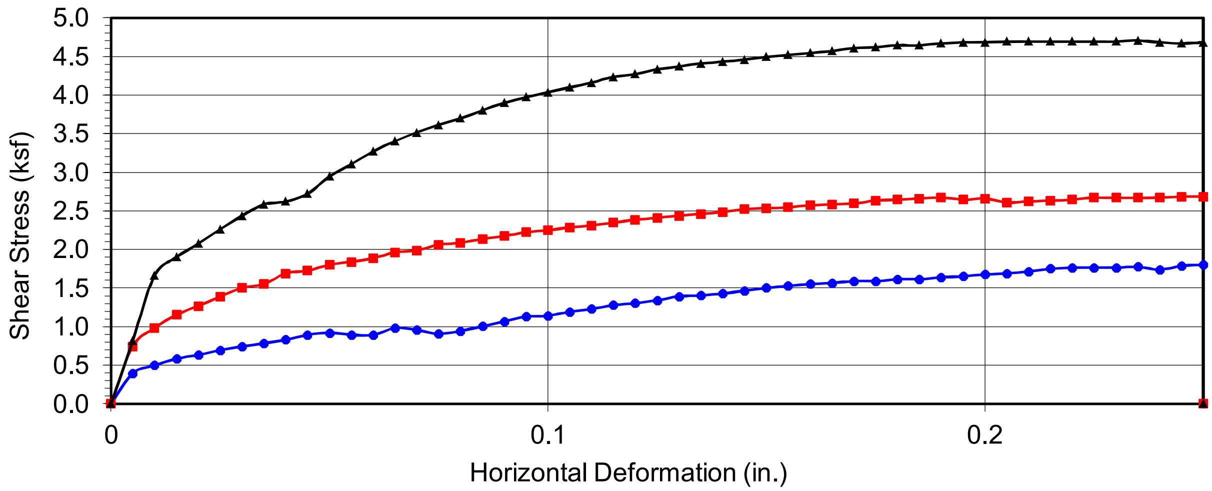
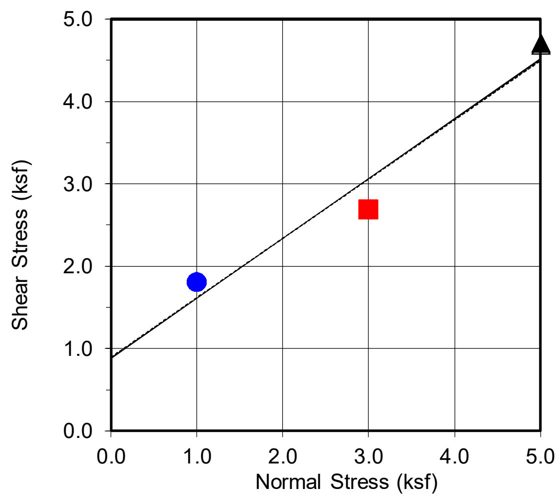
Figure B4





Boring No.	**B2**
Sample No.	**B2@24**
Depth (ft)	**24**
Sample Type:	Ring

Soil Identification:		
Dark Yellowish Brown Sand w/Gravel (SW)		
Strength Parameters		
	C (psf)	ϕ (°)
Peak	883	36.0
Ultimate	894	35.8

Normal Stress (kip/ft2)	1	3	5
Peak Shear Stress (kip/ft²)	● 1.80	■ 2.68	▲ 4.71
Shear Stress @ End of Test (ksf)	○ 1.80	□ 2.68	△ 4.68
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 (%)	12.9	16.9	14.7
Initial Dry Density (pcf)	120.9	116.7	116.7
Initial Degree of Saturation (%)	88.1	102.7	89.4
Soil Height Before Shearing (in.)	1.2	1.2	1.2
Final Moisture Content (%)	16.4	17.0	16.8

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DIRECT SHEAR TEST RESULTS

Consolidated Drained ASTM D-3080

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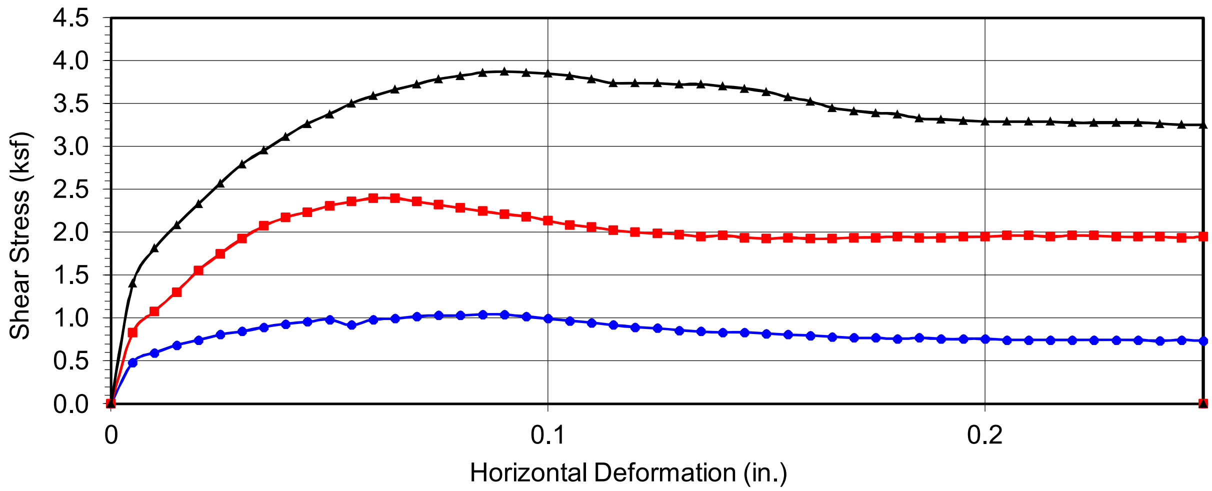
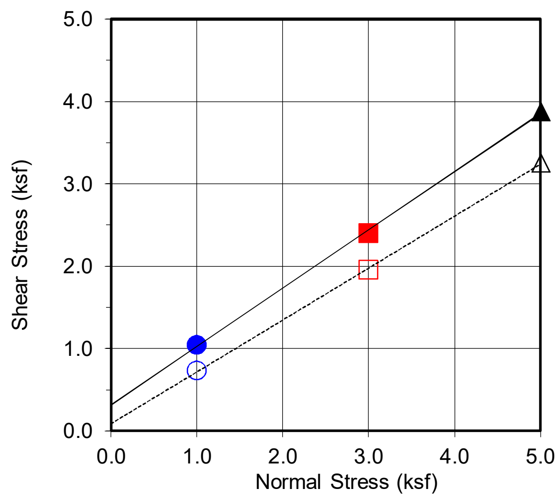
Figure B5





Boring No.	**B3**
Sample No.	**B3@24**
Depth (ft)	**24**
Sample Type:	Ring

Soil Identification:		
Brown Silty Sand (SM)		
Strength Parameters		
	C (psf)	ϕ (°)
Peak	315	35.3
Ultimate	88	32.2

Normal Stress (kip/ft2)	1	3	5
Peak Shear Stress (kip/ft²)	● 1.04	■ 2.40	▲ 3.87
Shear Stress @ End of Test (ksf)	○ 0.73	□ 1.95	△ 3.25
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 (%)	15.9	10.9	24.2
Initial Dry Density (pcf)	105.2	105.5	103.2
Initial Degree of Saturation (%)	71.4	49.0	103.0
Soil Height Before Shearing (in.)	1.2	1.2	1.2
Final Moisture Content (%)	20.3	21.3	19.7

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DIRECT SHEAR TEST RESULTS

Consolidated Drained ASTM D-3080

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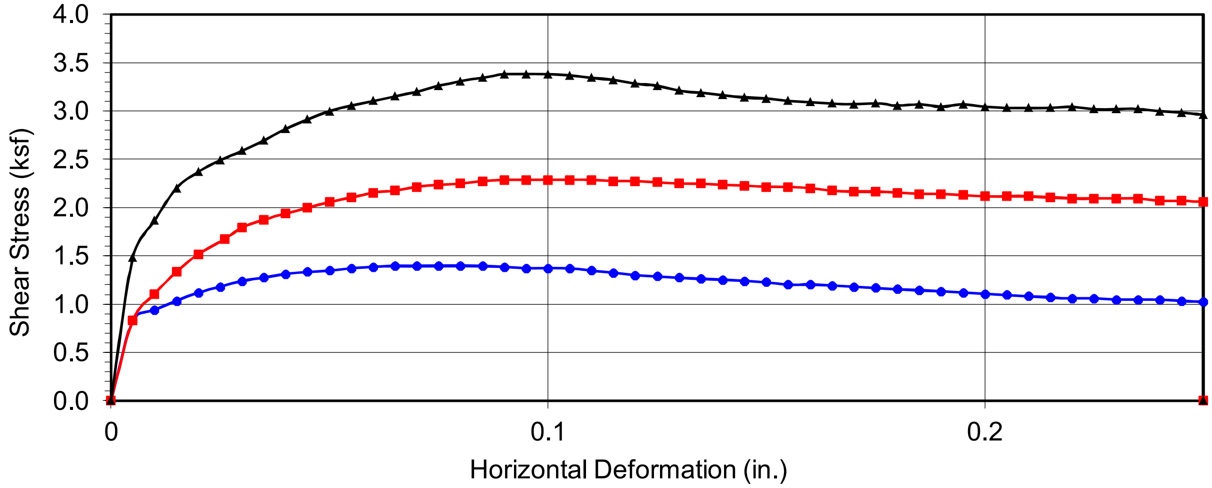
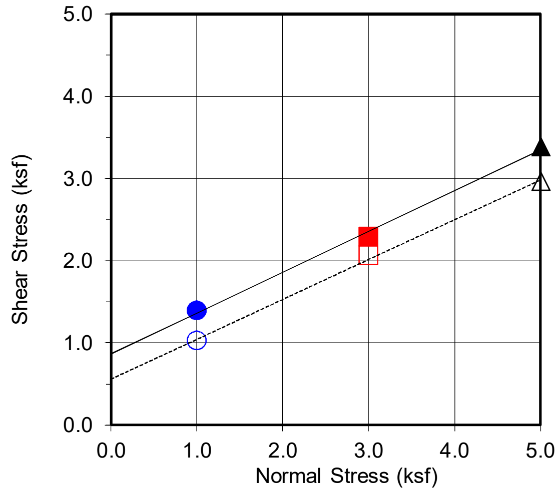
Figure B6





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

Soil Identification:		
Olive Brown Silty Sand (SM)		
Strength Parameters		
	C (psf)	ϕ (°)
Peak	865	26.4
Ultimate	560	25.8

Normal Stress (kip/ft2)	1	3	5
Peak Shear Stress (kip/ft²)	● 1.40	■ 2.29	▲ 3.38
Shear Stress @ End of Test (ksf)	○ 1.02	□ 2.06	△ 2.96
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 (%)	17.5	17.9	17.0
Initial Dry Density (pcf)	115.4	115.1	114.2
Initial Degree of Saturation (%)	102.5	104.0	96.1
Soil Height Before Shearing (in.)	1.2	1.2	1.2
Final Moisture Content (%)	19.9	18.9	17.7

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DIRECT SHEAR TEST RESULTS

Consolidated Drained ASTM D-3080

Checked by: JJK

Project No.: A9060-06-18

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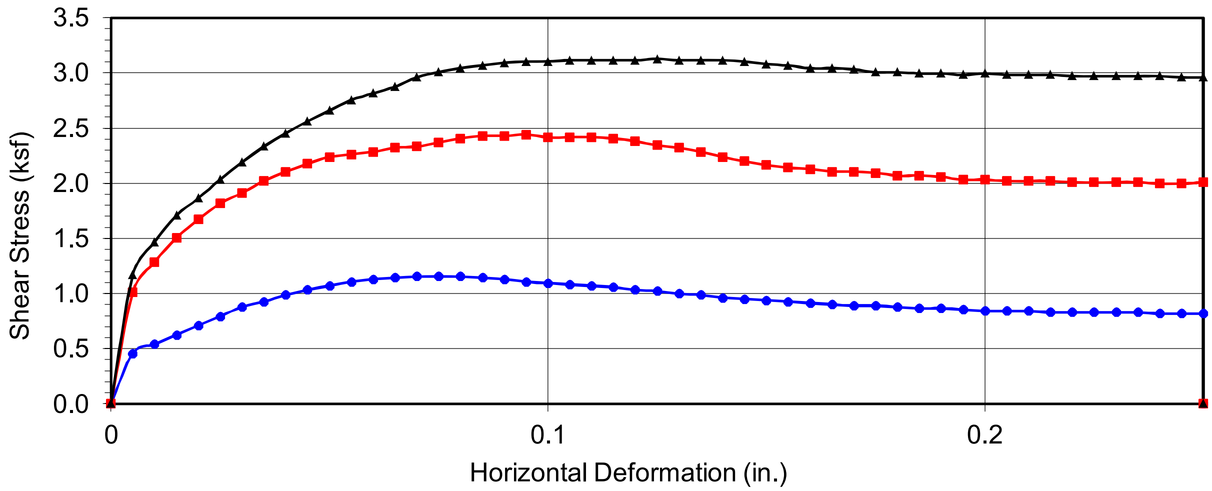
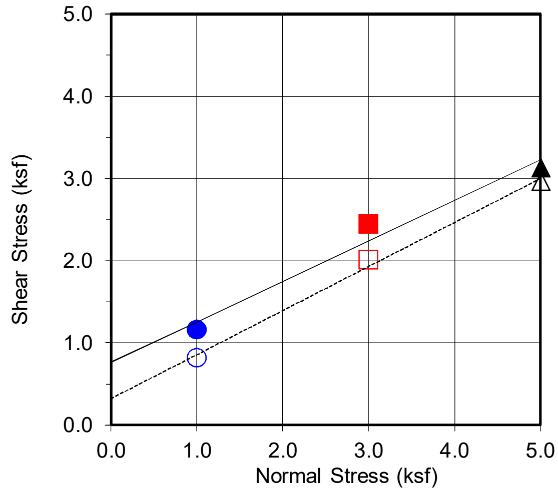
Figure B7





Boring No.	**B1**
Sample No.	**B1@45**
Depth (ft)	**45**
Sample Type:	Ring

Soil Identification:		
Olive Brown Silty Sand (SM)		
Strength Parameters		
	C (psf)	ϕ (^{o})
Peak	762	26.2
Ultimate	323	28.2

Normal Stress (kip/ft2)	1	3	5
Peak Shear Stress (kip/ft²)	● 1.16	■ 2.44	▲ 3.13
Shear Stress @ End of Test (ksf)	○ 0.82	□ 2.01	△ 2.96
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 (%)	17.6	19.1	22.1
Initial Dry Density (pcf)	108.5	112.3	108.1
Initial Degree of Saturation (%)	85.7	103.2	107.0
Soil Height Before Shearing (in.)	1.2	1.2	1.2
Final Moisture Content (%)	21.4	19.7	20.0

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DIRECT SHEAR TEST RESULTS

Consolidated Drained ASTM D-3080

Checked by: JJK

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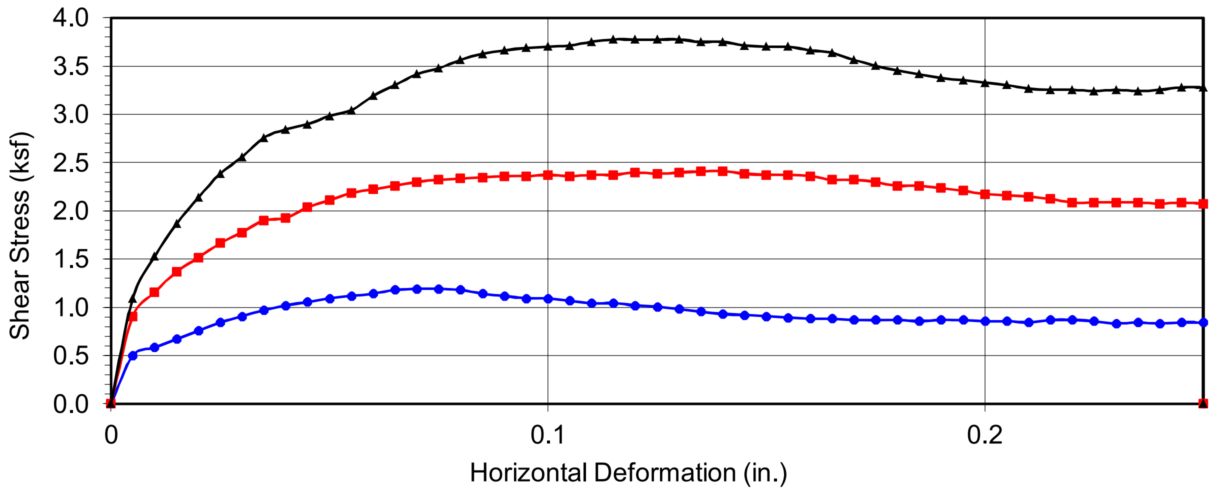
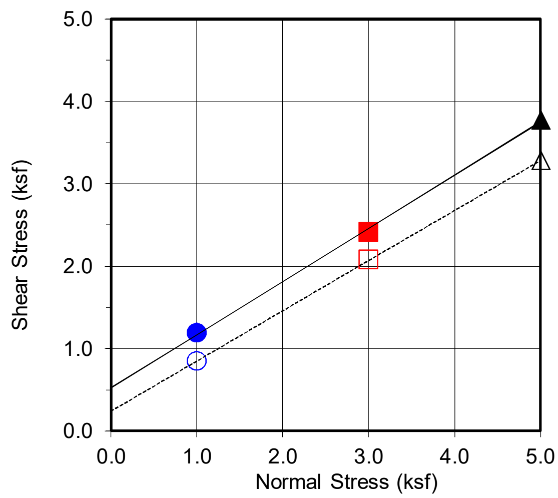
Figure B8





Boring No.	**B2**
Sample No.	**B2@45**
Depth (ft)	**45**
Sample Type:	Ring

Normal Stress (kip/ft2)	1	3	5
Peak Shear Stress (kip/ft²)	● 1.19	■ 2.41	▲ 3.78
Shear Stress @ End of Test (ksf)	○ 0.84	□ 2.07	△ 3.28
Deformation Rate (in./min.)	0.05	0.05	0.05
Initial Sample Height (in.)	1.0	1.0	1.0
Ring Inside Diameter (in.)	2.375	2.375	2.375
Initial Moisture Content (%)	18.2	18.4	18.5
Initial Dry Density (pcf)	113.3	113.6	113.7
Initial Degree of Saturation (%)	100.5	102.9	103.5
Soil Height Before Shearing (in.)	1.2	1.2	1.2
Final Moisture Content (%)	19.1	19.1	18.8

Soil Identification:

Brown Silty Sand (SM)

Strength Parameters

	C (psf)	ϕ (°)
Peak	521	32.9
Ultimate	240	31.3

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DIRECT SHEAR TEST RESULTS

Consolidated Drained ASTM D-3080

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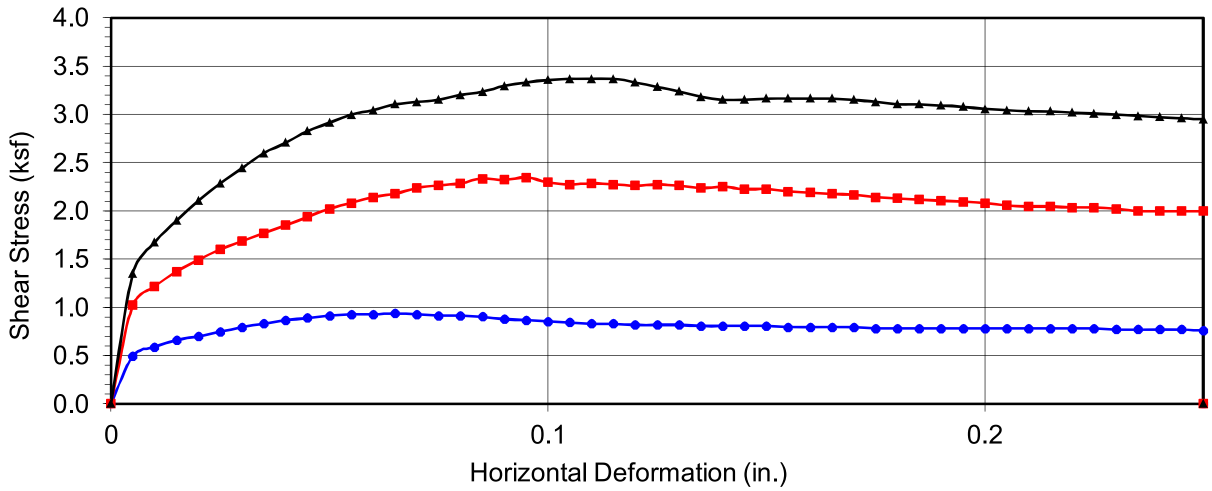
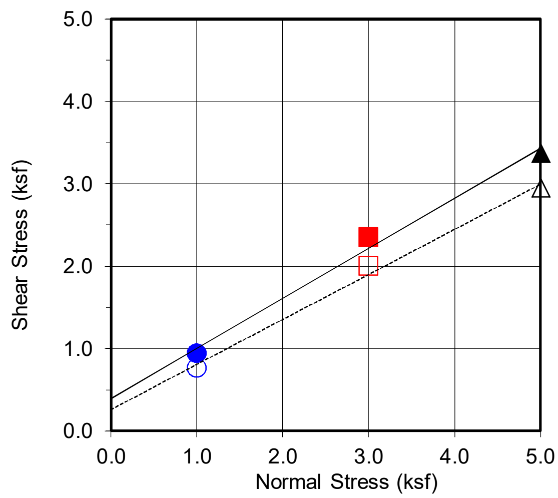
Figure B9





Boring No.	**B1**
Sample No.	**B1@55**
Depth (ft)	**55**
Sample Type:	Ring

Soil Identification:		
Olive Brown Sandy Silt (ML)		
Strength Parameters		
	C (psf)	ϕ (^{o})
Peak	394	31.3
Ultimate	259	28.7

Normal Stress (kip/ft2)	1	3	5
Peak Shear Stress (kip/ft²)	● 0.94	■ 2.35	▲ 3.37
Shear Stress @ End of Test (ksf)	○ 0.76	□ 2.00	△ 2.95
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 (%)	22.7	20.0	24.7
Initial Dry Density (pcf)	105.2	109.1	102.2
Initial Degree of Saturation (%)	101.8	99.0	102.7
Soil Height Before Shearing (in.)	1.2	1.2	1.2
Final Moisture Content (%)	26.6	21.9	23.9

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DIRECT SHEAR TEST RESULTS

Consolidated Drained ASTM D-3080

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Project No.: A9060-06-18

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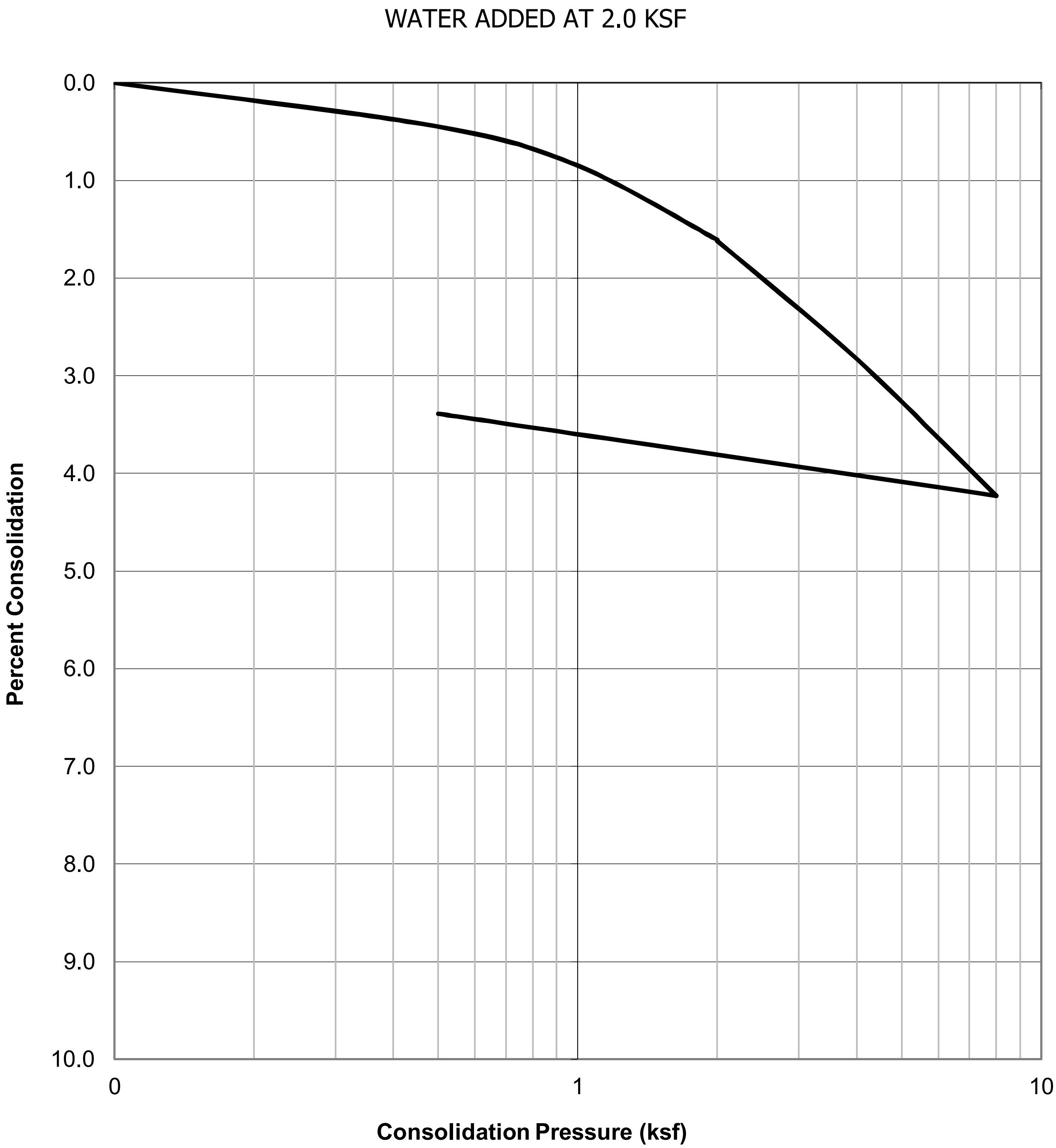
Figure B10



SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B1@24	Brown Silty Sand (SM)	110.5	17.4	16.9

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CONSOLIDATION TEST RESULTS

ASTM D-2435

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Los Angeles, California

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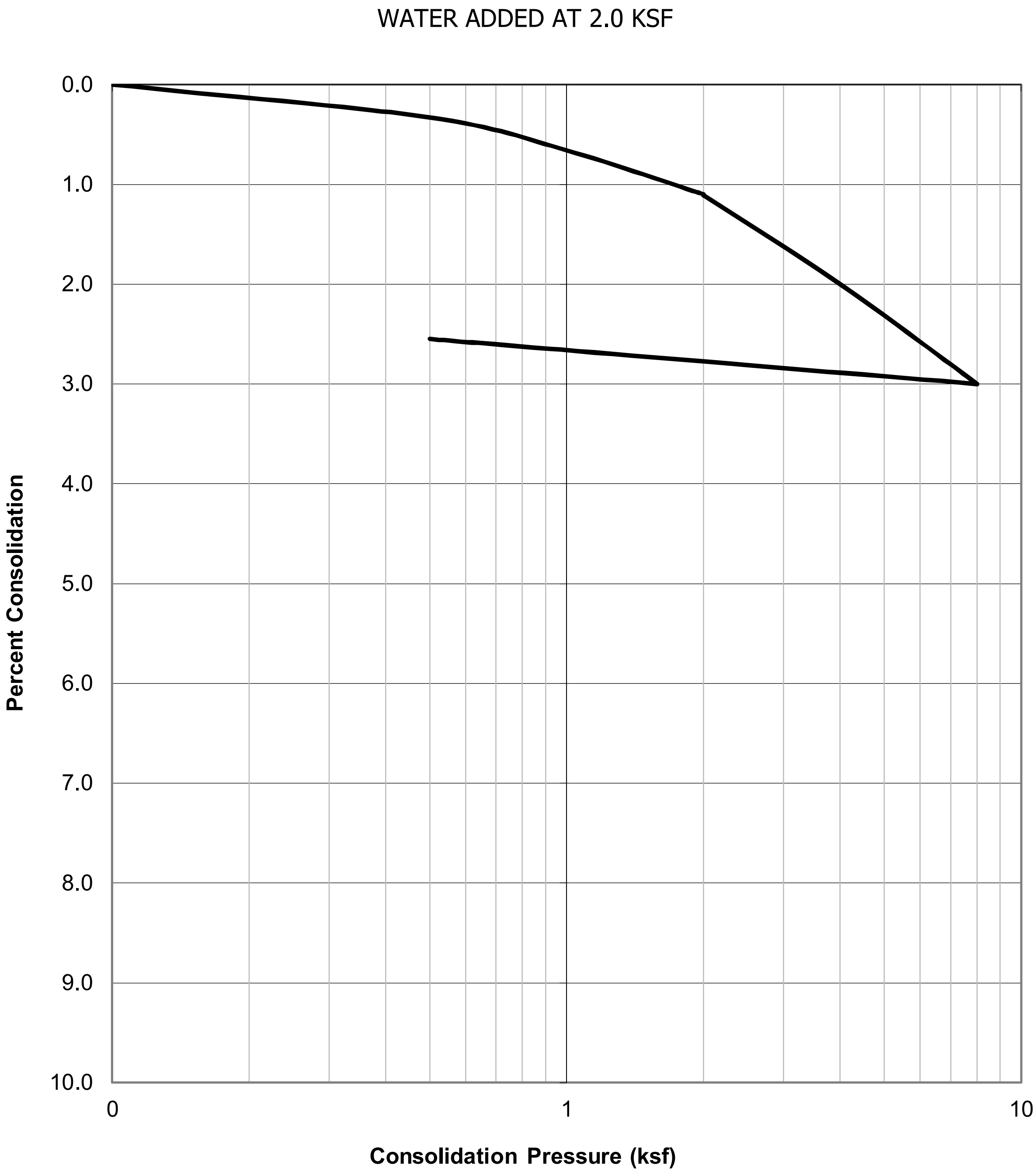
Figure B11



SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B2@24	Brown Silty Sand (SM)	117.4	9.7	13.2

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CONSOLIDATION TEST RESULTS

ASTM D-2435

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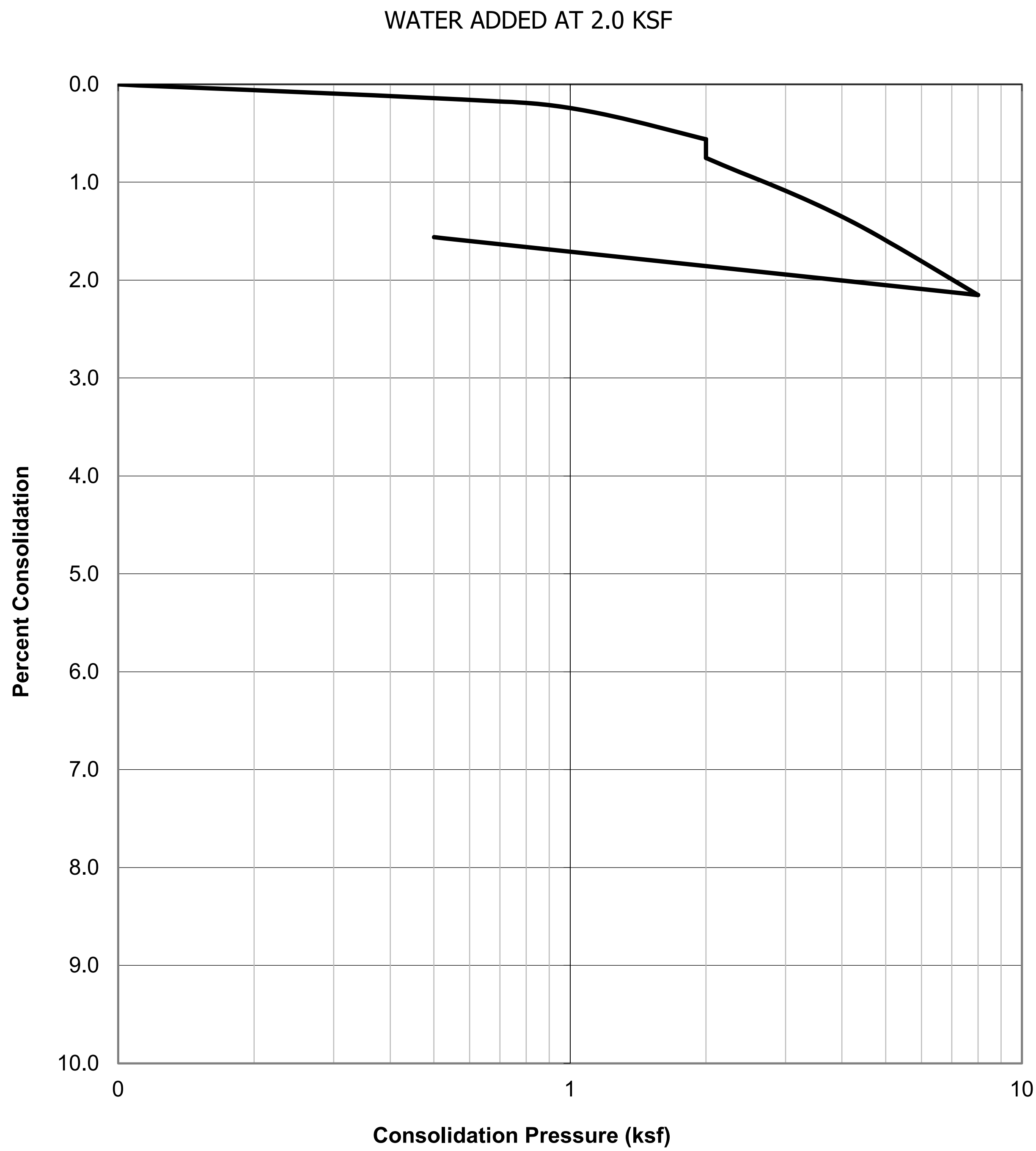
Figure B12



SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B1@26	Brown Silty Sand (SM)	109.1	8.2	14.7

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CONSOLIDATION TEST RESULTS

ASTM D-2435

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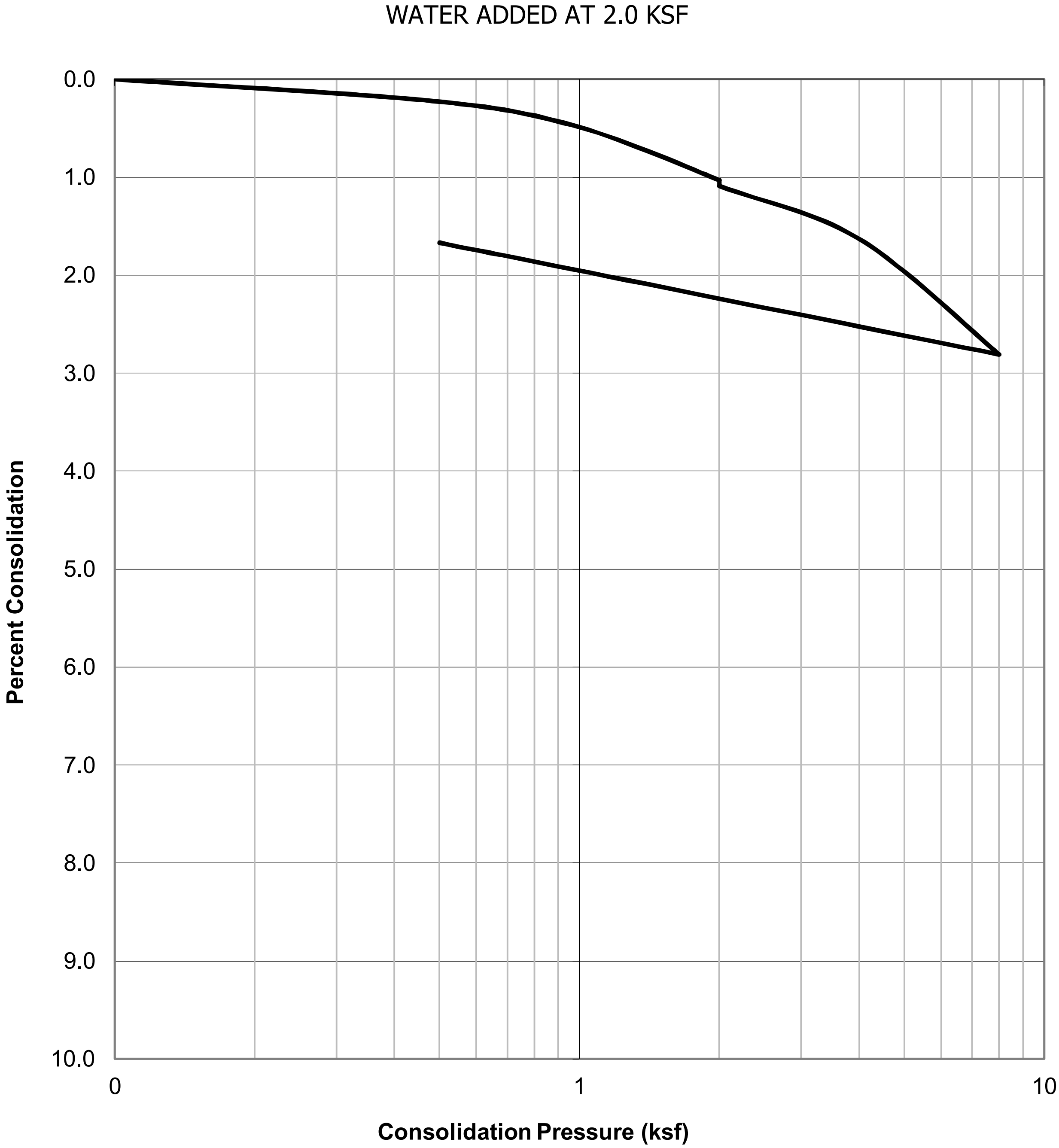
Figure B13



SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B1@30	Brown Silty Sand (SM)	114.8	15.6	16.4

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CONSOLIDATION TEST RESULTS

ASTM D-2435

Checked by: JJK

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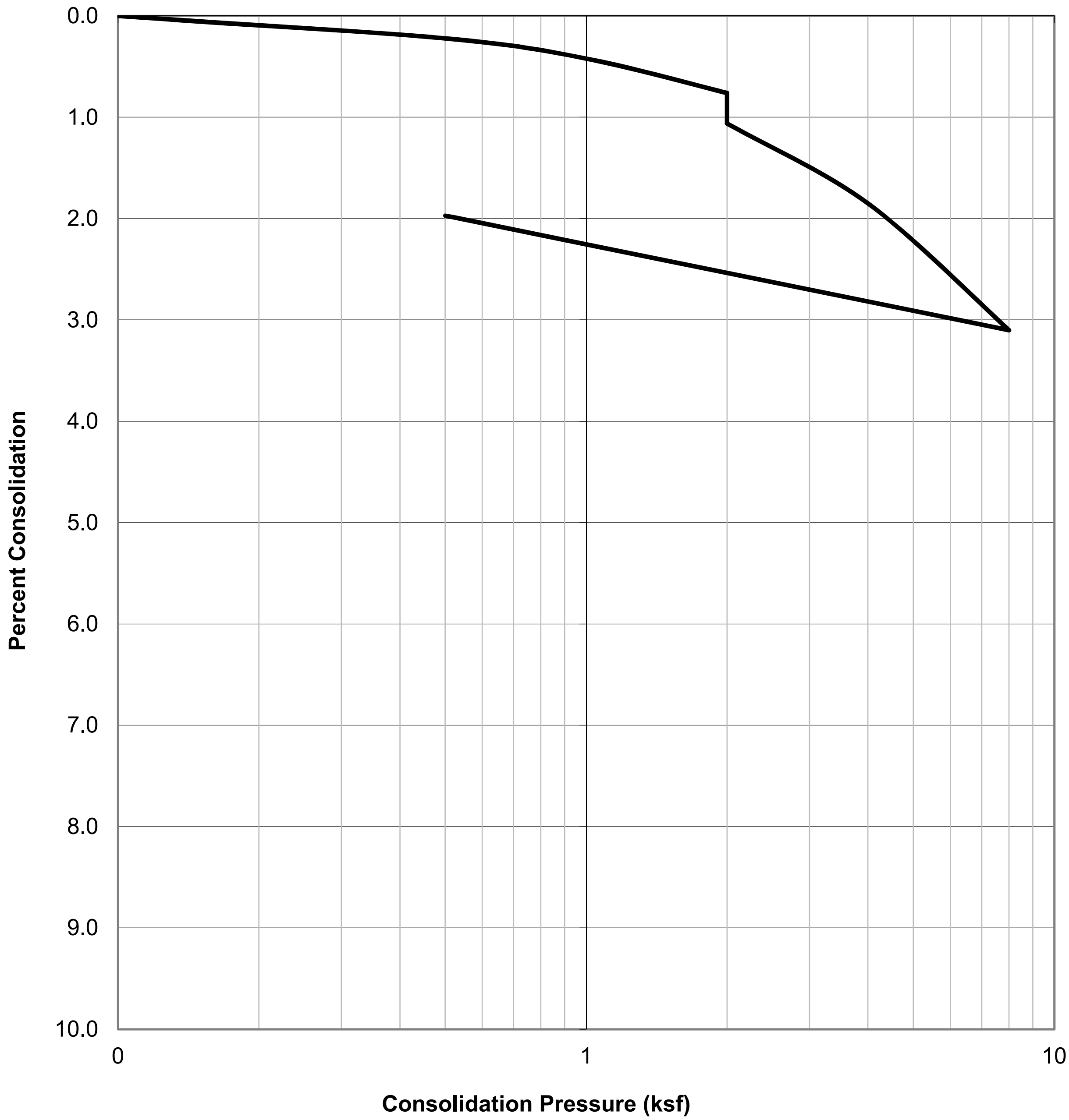
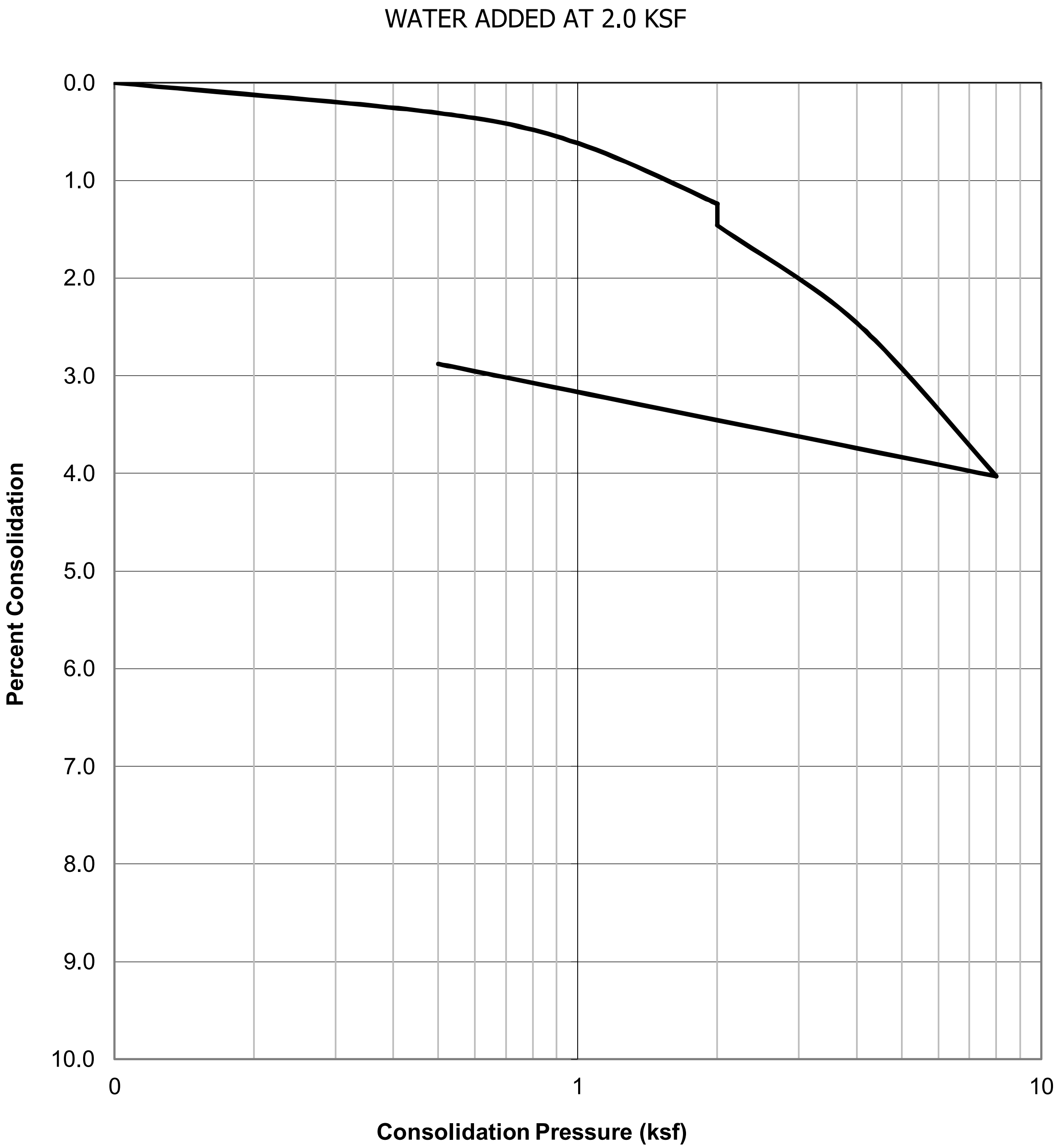
Figure B14

WATER ADDED AT 2.0 KSF



SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B2@30	Brown Silty Sand (SM)	110.0	14.7	17.2

GEOCON	**CONSOLIDATION TEST RESULTS** ASTM D-2435	Project No.: A9060-06-18
		South Hilgard Avenue Los Angeles, California
	Checked by: JJK	August, 2019 Figure B15



SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B1@40	Brown Silty Sand (SM)	112.8	16.4	17.5

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CONSOLIDATION TEST RESULTS

ASTM D-2435

Checked by: JJK

Project No.: A9060-06-18

South Hilgard Avenue
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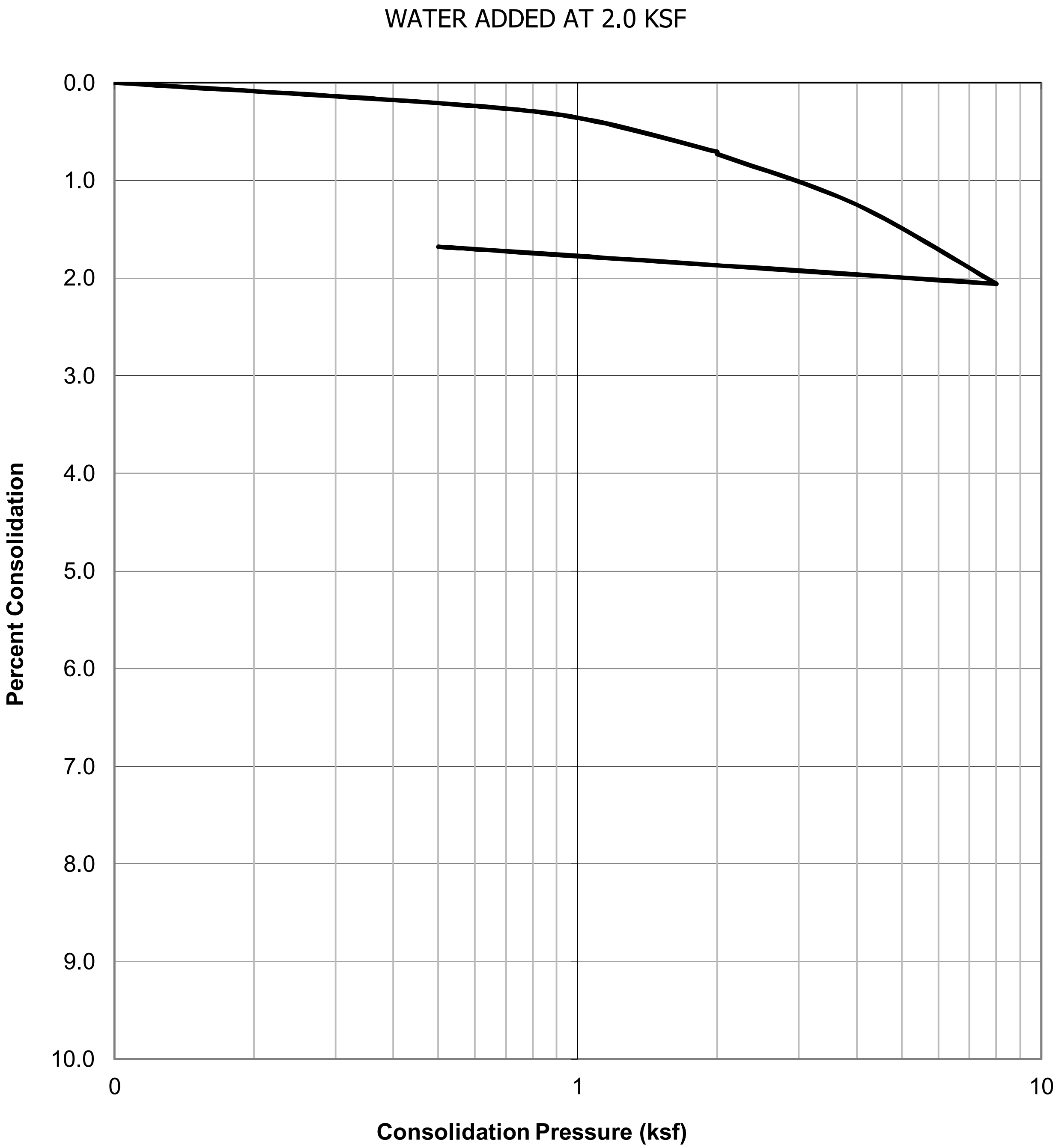
Figure B16



SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B2@40	Dark Brown Poorly Graded Sand (SP)	113.2	13.1	14.4

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CONSOLIDATION TEST RESULTS

ASTM D-2435

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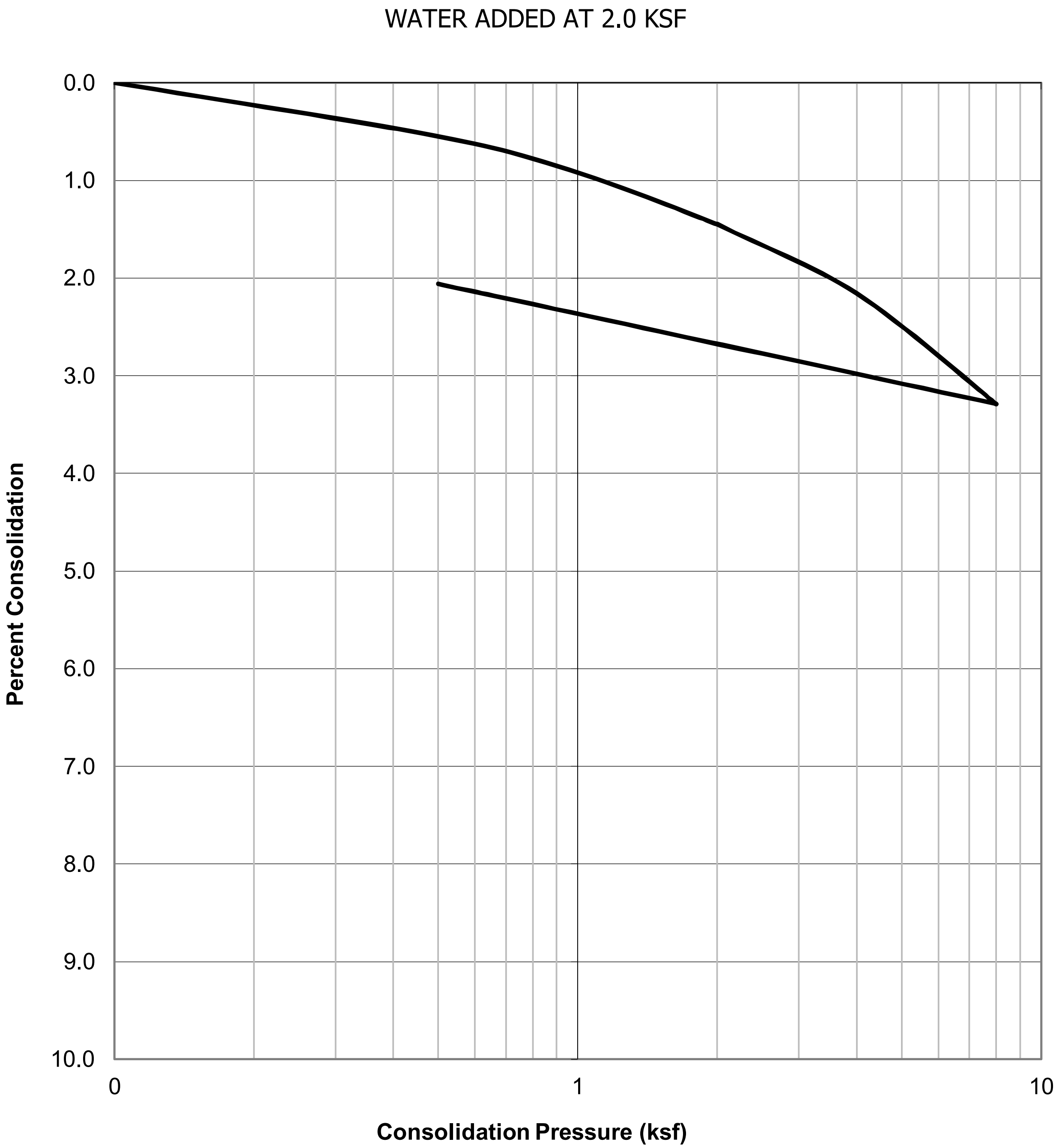
Figure B17



SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B1@45	Brown Silty Sand (SM)	111.2	18.3	18.4

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CONSOLIDATION TEST RESULTS

ASTM D-2435

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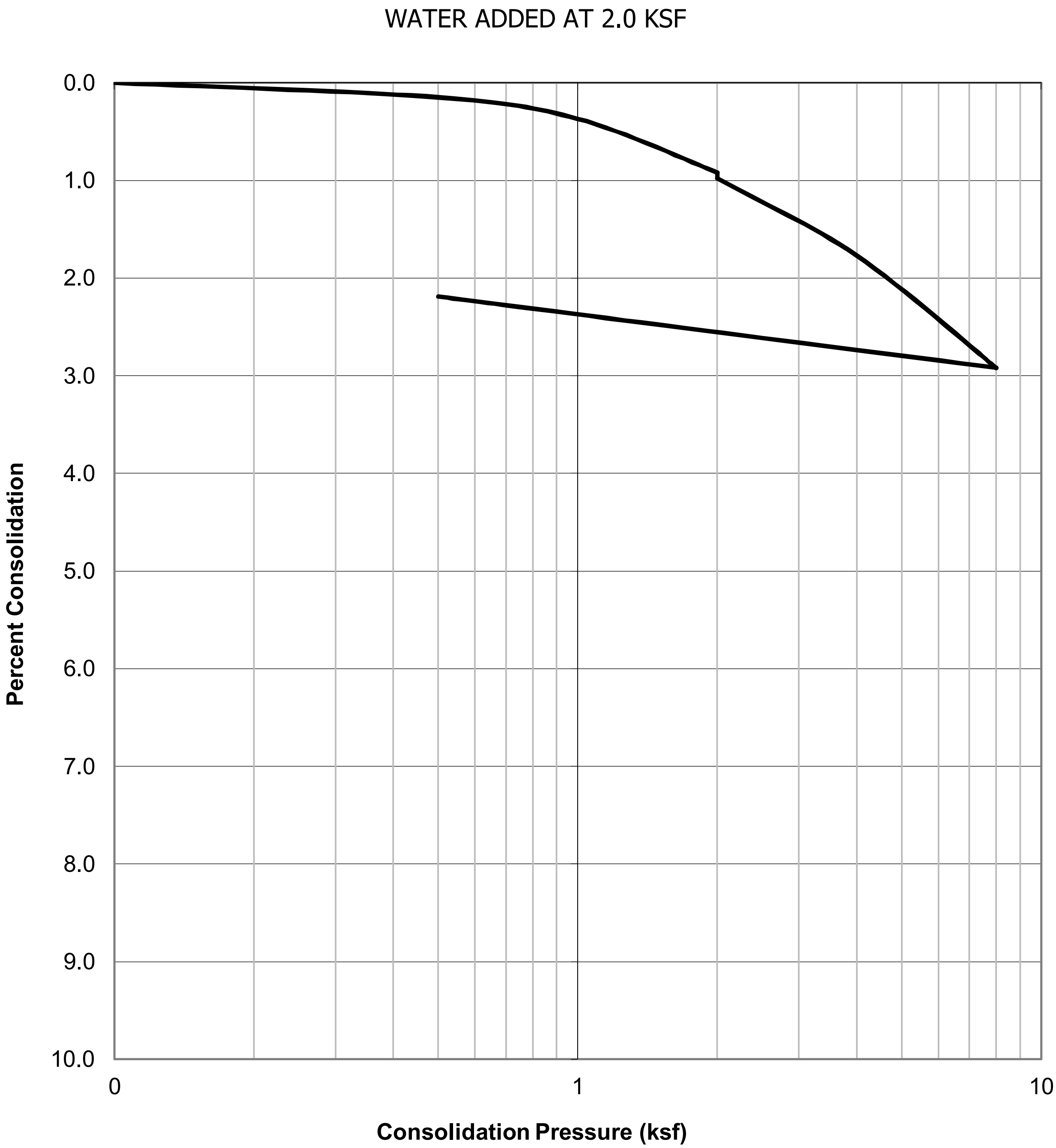
Figure B18



SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B2@50	Brown Silty Sand (SM)	113.0	15.9	16.3

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CONSOLIDATION TEST RESULTS

ASTM D-2435

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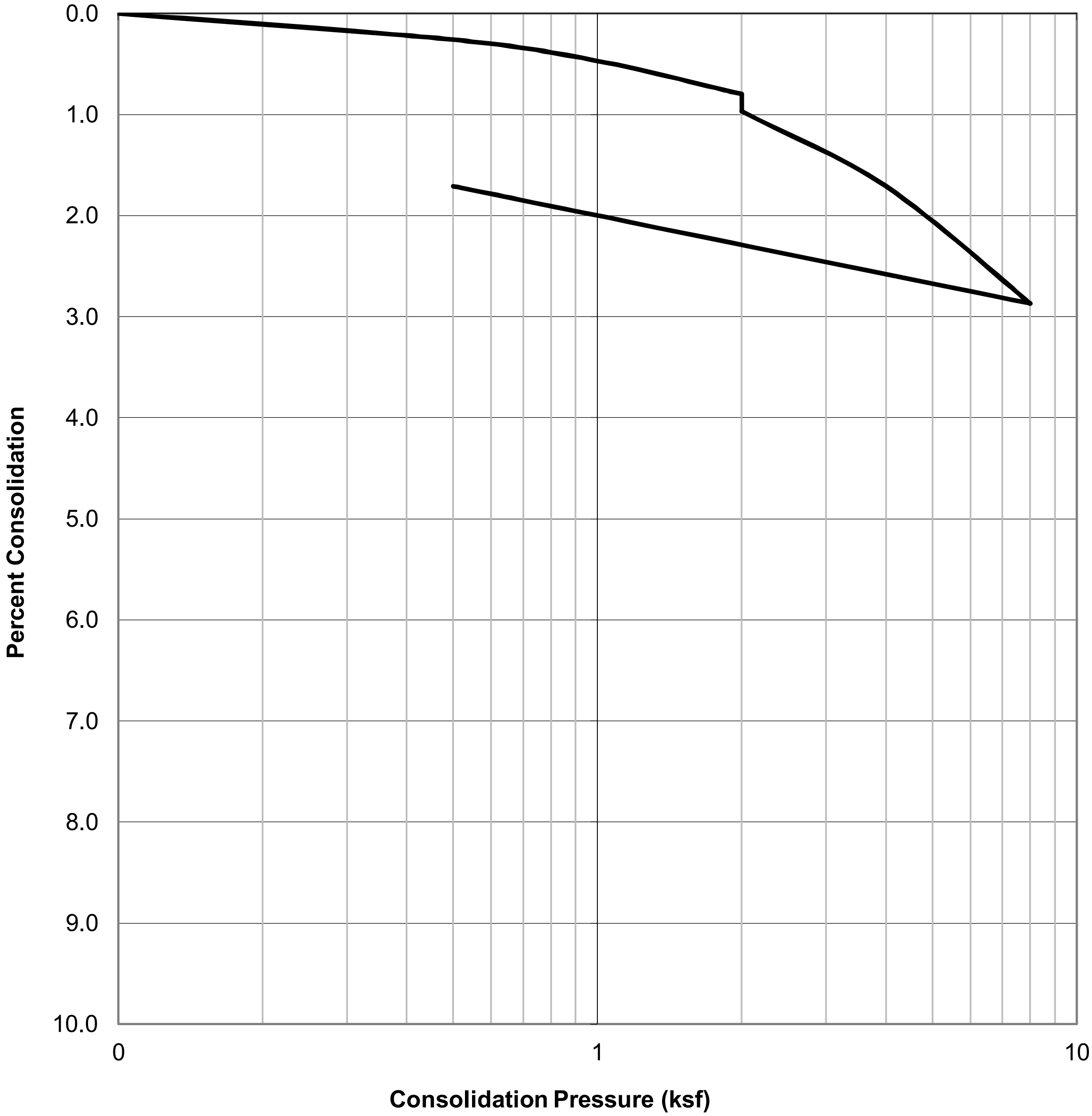
WATER ADDED AT 2.0 KSF



SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B1@55	Brown Silt (ML)	115.7	15.1	16.2

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CONSOLIDATION TEST RESULTS

ASTM D-2435

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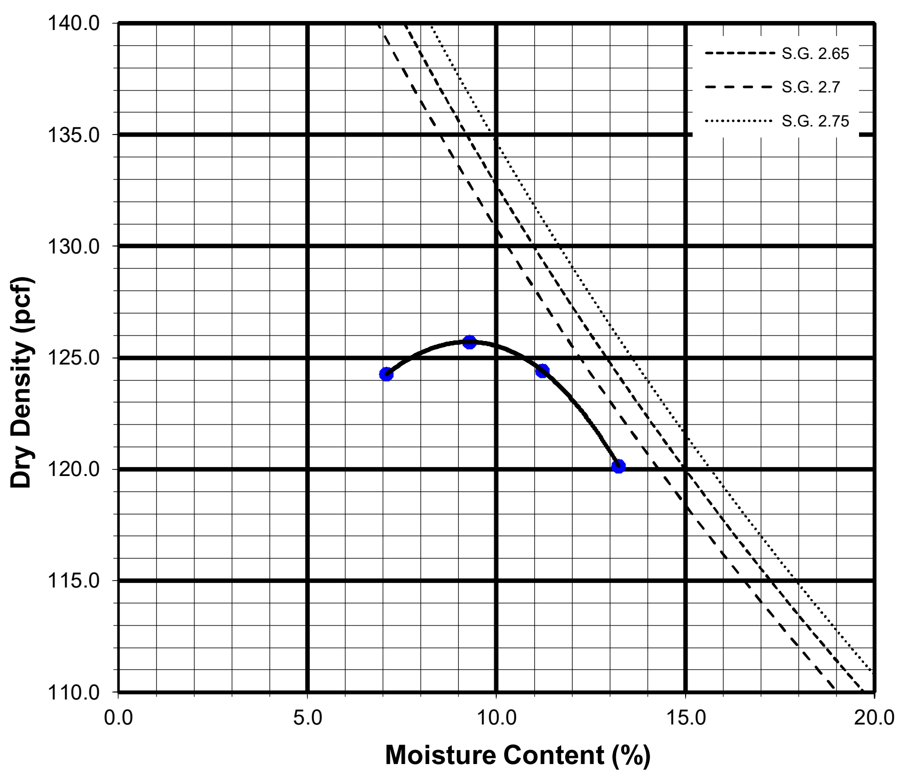
Figure B20

Sample No:

B2@0-5	Brown Silty Sand (SM)

TEST NO.		1	2	3	4	5	6
Wt. Compacted Soil + Mold	(g)	6248	6263	6228	6183		
Weight of Mold	(g)	4173	4173	4173	4173		
Net Weight of Soil	(g)	2075	2090	2055	2010		
Wet Weight of Soil + Cont.	(g)	707.0	778.2	688.4	808.8		
Dry Weight of Soil + Cont.	(g)	659.3	714.4	623.6	763.6		
Weight of Container	(g)	145.0	145.6	133.8	125.8		
Moisture Content	(%)	9.3	11.2	13.2	7.1		
Wet Density	(pcf)	137.4	138.4	136.0	133.1		
Dry Density	(pcf)	125.7	124.4	120.2	124.3		

Maximum Dry Density (pcf)	**126.0**	**Optimum Moisture Content (%)**	**10.0**



Preparation Method: A

GEOCON

MODIFIED COMPACTION TEST OF SOILS

ASTM D-1557

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Project No.: A9060-06-18

South Hilgard Avenue
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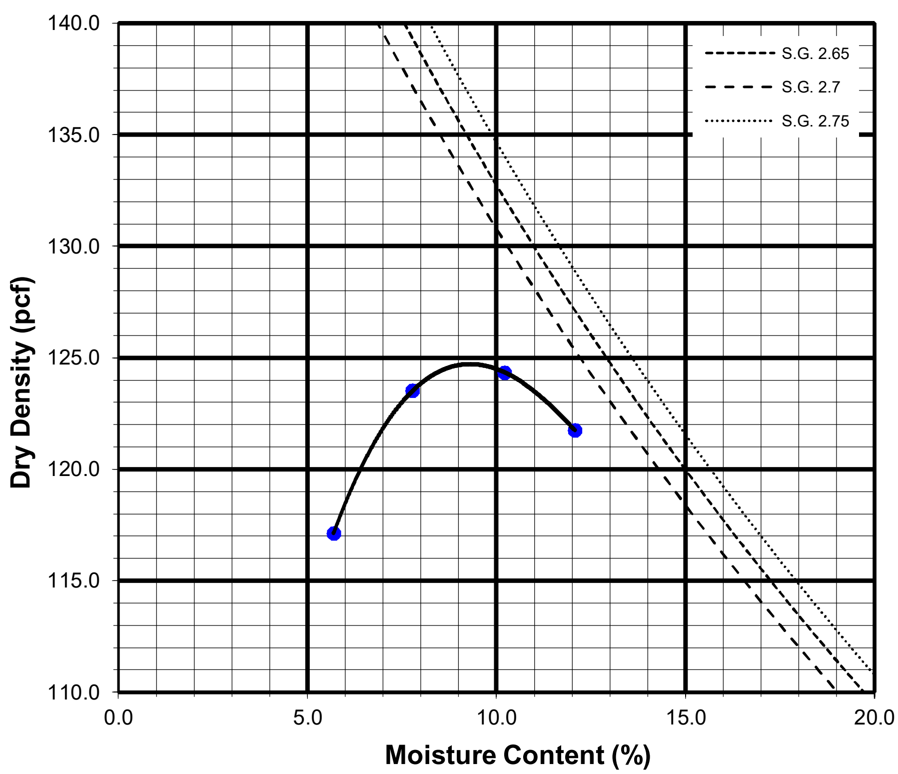
Figure B21

Sample No:

B2@20-25	Brown Silty Sand (SM)

TEST NO.	1	2	3	4	5	6
Wt. Compacted Soil + Mold (g)	6243	6234	6184	6043		
Weight of Mold (g)	4173	4173	4173	4173		
Net Weight of Soil (g)	2070	2061	2011	1870		
Wet Weight of Soil + Cont. (g)	696.4	649.8	725.4	691.8		
Dry Weight of Soil + Cont. (g)	645.5	594.1	683.7	662.4		
Weight of Container (g)	147.2	133.0	147.7	145.8		
Moisture Content (%)	10.2	12.1	7.8	5.7		
Wet Density (pcf)	137.0	136.4	133.1	123.8		
Dry Density (pcf)	124.3	121.7	123.5	117.1		

Maximum Dry Density (pcf)	125.0	Optimum Moisture Content (%)	9.0



Preparation Method: A

MODIFIED COMPACTION TEST OF SOILS

ASTM D-1557

Checked by: JJK

GEOCON

Project No.: A9060-06-18

South Hilgard Avenue
Los Angeles, California

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Figure B22

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

Sample No.	**pH**	**Resistivity (ohm centimeters)**
B2 @ 20-25	6.5	1030 (Corrosive)

SUMMARY OF LABORATORY CHLORIDE CONTENT TEST RESULTS
EPA NO. 325.3

Sample No.	Chloride Ion Content (%)
B2@20-25	0.012

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

Sample No.	**Water Soluble Sulfate (% SQ_4)**	**Sulfate Exposure***
B2@20-25	0.011	S0

GEOCON	**CORROSIVITY TEST RESULTS**	Project No.: A9060-06-18
		South Hilgard Avenue Los Angeles, California
	Checked by: JJK	August, 2019 Figure B23