

Appendix IS-2

Geotechnical Investigation

GEOTECHNICAL INVESTIGATION

PROPOSED MIXED-USE DEVELOPMENT 1225-1235 NORTH VINE STREET 6311-6333 WEST LA MIRADA AVENUE LOS ANGELES, CALIFORNIA

TRACT: COLEGROVE, BLOCK: 13, LOT: FR 2, ARB: 3
TRACT: TR 4622, LOTS: 1-3 AND 7-12



GEOCON
WEST, INC.

GEOTECHNICAL
ENVIRONMENTAL
MATERIALS

PREPARED FOR

39 SOUTH, LLC
LOS ANGELES, CALIFORNIA

PROJECT NO. W1135-06-01

REVISED JULY 1, 2020



Project No. W1135-06-01

Revised July 1, 2020

39 South, LLC
c/o Plus Development Group
4525 Wilshire Boulevard, Ste #150
Los Angeles, CA 90010

Attention: Mr. Mick Unwin

Subject: GEOTECHNICAL INVESTIGATION
PROPOSED MIXED-USE DEVELOPMENT
1225-1235 NORTH VINE STREET
6311-6323 WEST LA MIRADA AVENUE LOS ANGELES, CALIFORNIA
TRACT: COLEGROVE, BLOCK: 13, LOT: FR 2, ARB: 3
TRACT: TR 4622; LOTS: 1-3 AND 7-12




Dear Mr. Unwin:

In accordance with your authorization of our proposal dated January 20, 2020, we have prepared a geotechnical investigation for the proposed mixed-use development to be located at 1225-1235 North Vine Street and 6311-6333 West La Mirada Avenue in the City of Los Angeles, California. The accompanying report presents the findings of our study and our conclusions and recommendations pertaining to the geotechnical aspects of proposed design and construction. Based on the results of our investigation, it is our opinion that the proposed project can proceed as proposed, provided the recommendations in this report are followed and implemented during design and construction.

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

Very truly yours,

GEOCON WEST, INC.



Petrina Zen
PE 87489

Susan F. Kirkgard
CEG 1754

Jelisa Thomas Adams
GE 3092

(EMAIL) Addressee

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GEOTECHNICAL INVESTIGATION

1. PURPOSE AND SCOPE

This report presents the results of a geotechnical investigation for the proposed mixed-use development to be located at 1225-1235 North Vine Street and 6311-6333 West La Mirada Avenue in the City of Los Angeles, California (see Vicinity Map, Figure 1). The purpose of the investigation was to evaluate subsurface soil and geologic conditions underlying the property and based on conditions encountered to provide conclusions and recommendations pertaining to the geotechnical aspects of the proposed 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 April 1, 2020 by excavating two 8-inch diameter hollow-stem auger borings using a truck-mounted hollow-stem auger drilling machine. The borings were advanced to depths of approximately 80½ feet below the existing ground surface. The approximate locations of the borings are depicted on the Site Plan (see Figure 2A). A detailed discussion of the field investigation, including boring logs, is presented in Appendix A.

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

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

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

2. SITE AND PROJECT DESCRIPTION

The subject site is located at 1225-1235 North Vine Street and 6311-6333 West La Mirada Avenue in the City of Los Angeles, California. The northern half of the site is currently occupied by a two-story commercial structure and the southern half is occupied by a single-story commercial structure, five single-family residential structures, and associated driveways and asphalt-paved parking areas. The southern half of the site also includes an approximately 1,012 square foot alley that is parallel to Vine Street and accessible from La Mirada Avenue. The alley is located between Lots 1-3 and 7 (Tract: TR4622). The site is bounded by a four-story mixed-use structure to the north, by La Mirada Street to the south, by North Vine Street to the east, and by single-family residences to the west. The site is relatively level, with no pronounced highs or lows. Surface water drainage at the site appears to flow to the city streets. Vegetation onsite consists of typical residential landscaping within the single-family residential lots, and the remainder of the site is devoid of vegetation.

Based on information provided by the Client, it is our understanding that the proposed development will consist of an 8-story, mixed-use structure over four subterranean levels. It is assumed that the proposed subterranean levels will extend approximately 45 feet below the existing ground surface, including foundation depths. The proposed development is depicted on the Site Plan and Cross Section (see Figures 2A and 2B).

Based on the preliminary nature of the design at this time, wall and column loads were not available. It is anticipated that column loads for the proposed structure will be up to 1,400 kips, and wall loads will be up to 16 kips per linear foot.

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

3. GEOLOGIC SETTING

The site is located in the northeastern portion of the Los Angeles Basin, a coastal plain bounded by the Santa Monica Mountains to the north, the Elysian Hills and Repetto Hills to the northeast, the Puente Hills and Whittier Fault to the east, the Palos Verdes Peninsula and Pacific Ocean to the west and south, and the Santa Ana Mountains and San Joaquin Hills to the southeast. The basin is underlain by a deep structural depression which has been filled by both marine and continental sediments which overlie a basement complex of igneous and metamorphic composition. The basement surface within the central portion of the basin extends to a maximum depth of 32,000 feet below sea level. Regionally, the site is located within the northern portion of the Peninsular Ranges geomorphic province characterized by northwest-trending physiographic and geologic features such as the Newport-Inglewood Fault Zone, located approximately 4.2 miles to the southwest.

4. SOIL AND GEOLOGIC CONDITIONS

Based on our field investigation and published geologic maps of the area, the earth materials underlying the site consist of artificial fill underlain by Holocene age alluvium that is in turn underlain by late Pleistocene age alluvial fan deposits at depth (California Geological Survey [CGS], 2012). Detailed stratigraphic profiles of the soils encountered in our explorations are provided on the boring logs in Appendix A.

4.1 Artificial Fill

Artificial fill was encountered in our explorations to a maximum depth of 8½ feet beneath the existing ground surface. The fill generally consists of dark brown to dark olive brown sandy clay and clay. The sand is fine- to medium grained and the fill is characterized as moist and firm to stiff. Deeper fill may exist between excavations and in other portions of the site that were not directly explored.

4.2 Alluvium

The fill materials are underlain by Holocene and Pleistocene age alluvial fan deposits. The alluvium consists of grayish brown, dark brown, or yellowish brown interbedded poorly graded and well-graded sand, silty sand, clayey sand, sandy clay and with varying amounts of fine to coarse gravel. The alluvial soils are moist to wet and loose to very dense or firm to hard.

5. GROUNDWATER

Review of the Seismic Hazard Evaluation Report for the Hollywood Quadrangle (California Division of Mines & Geology, 1998) indicates the historically highest groundwater in the area is approximately 40 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 available groundwater monitoring well data from State and local agencies, groundwater levels in the Los Angeles Coastal Plain have steeply declined from historic highs recorded in the early 1900s. The decline is related to an increasing demand for potable water over time that exceed the natural replenishment of groundwater in local aquifers. In the 1950s and early 1960s, formation of the Water Replenishment District of Southern California (and other local agencies) have kept water levels relatively stable since the 1960s. Based on internal discussions with County of Los Angeles, the Water Replenishment District (WRD) and other local agencies, there is a high demand for groundwater in the Los Angeles Coastal Plain area and there is an overdraft every year which means pumping exceeds natural groundwater replenishment. The overdraft is made up by purchasing artificial replenishment (imported and recycled water) to help make up the difference. According to a 2002 report by DWR, historical groundwater extractions and the use of imported water within the Central Basin for the years 1957-58 to 2002-03 indicate that the amount of groundwater extracted has remained essentially the same since 1962-63 and the imported water usage has been consistent since 1983-84. In conjunction with the WRD, the DWR systematically and continuously monitors and manages the groundwater use and changes in water levels within the Basin. Over the past 45 years, the withdrawal of groundwater by pumping has remained essentially constant, and every year there is an overdraft, because the amount pumped exceeds the amount of natural replenishment. The amount pumped also does not meet the total annual demand for water. As a result, the WRD purchases imported water and reclaimed water to meet the high demand as well as to maintain a balance between the water extracted and the natural replenishment that occurs. This approach does not significantly change the annual amount of water stored in the Basin. Going forward, if the management plan used for the past 45 years is maintained, there should not be any significant change in the depth to groundwater compared to the range of current levels. In fact, as the population increases the demand will certainly continue to increase.

Based on current groundwater basin management practices, it is unlikely that the groundwater levels will ever exceed the historic high levels.

Groundwater was not encountered in our borings, drilled to a maximum depth of approximately 80½ feet below the existing ground surface. Based on the lack of groundwater in our borings and the depth of proposed construction, groundwater is neither expected to be encountered during construction, nor have a detrimental effect on the project. 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), groundwater seepage levels encountered during construction may be actually higher than those encountered during our investigation. In addition, recent requirements for stormwater infiltration could result in shallower seepage conditions in the region. Proper surface drainage of irrigation and precipitation will be critical for future performance of the project. Recommendations for drainage are provided in the *Surface Drainage* section of this report (see Section 7.24).

6. GEOLOGIC HAZARDS

6.1 Surface Fault Rupture

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

The site is not within a state-designated Alquist-Priolo Earthquake Fault Zone (CGS, 2020a; 2020b; 2014) nor a city-designated Preliminary Fault Rupture Study Area (City of Los Angeles, 2020) for surface fault rupture hazards. No Holocene-active or pre-Holocene faults with the potential for surface fault rupture are known to pass directly beneath the site. Therefore, the potential for surface rupture due to faulting occurring beneath the site during the design life of the proposed development is considered low. However, the site is located in the seismically active Southern California region, and could be subjected to moderate to strong ground shaking in the event of an earthquake on one of the many active Southern California faults. The faults in the vicinity of the site are shown in Figure 3, Regional Fault Map.

The closest Holocene-active fault with the potential for surface fault rupture is the Hollywood Fault located approximately 0.7 mile (¾ mile) north of the site (CGS, 2014). Other nearby active faults are the Santa Monica Fault Zone, the Newport-Inglewood Fault Zone, and the Raymond Fault located approximately 4.1 miles west-southwest, 4.2 miles southwest, and 5.3 miles east-northeast of the site, respectively. The Holocene-active San Andreas Fault Zone is located approximately 36 miles to the northeast.

Several buried thrust faults, commonly referred to as blind thrusts, underlie the Los Angeles Coastal Plain 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 blind thrust faults and others in the Los Angeles area do not present a potential surface fault rupture hazard at the site because these blind thrust faults are not exposed at the ground surface. However, these deep thrust faults are considered active features capable of generating future earthquakes that could result in moderate to significant ground shaking at the site.

6.2 Seismicity

As with all of Southern California, the site has experienced historic earthquakes from various regional faults. The seismicity of the region surrounding the site was formulated based on research of an electronic database of earthquake data. The epicenters of recorded earthquakes with magnitudes equal to or greater than 5.0 in the site vicinity are depicted on Figure 4, Regional Seismicity Map. A partial list of moderate to major magnitude earthquakes that have occurred in the Southern California area within the last 100 years is included in the following table.

LIST OF HISTORIC EARTHQUAKES

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

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

6.3 Seismic Design Criteria

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

2019 CBC SEISMIC DESIGN PARAMETERS

Parameter	Value	2019 CBC Reference
Site Class	D	Table 1613.2.2
MCE_R Ground Motion Spectral Response Acceleration – Class D (short), S_S	2.098	Figure 1613.2.1(1)
MCE_R Ground Motion Spectral Response Acceleration – Class D (1 sec), S_1	0.748g	Figure 1613.2.1(2)
Site Coefficient, F_A	1.0	Table 1613.2.3(1)
Site Coefficient, F_V	1.7*	Table 1613.2.3(2)
Site Class Modified MCE_R Spectral Response Acceleration (short), S_{MS}	2.098g	Section 1613.2.3 (Eqn 16-36)
Site Class Modified MCE_R Spectral Response Acceleration – (1 sec), S_{M1}	1.271g*	Section 1613.2.3 (Eqn 16-37)
5% Damped Design Spectral Response Acceleration (short), S_{DS}	1.398g	Section 1613.2.4 (Eqn 16-38)
5% Damped Design Spectral Response Acceleration (1 sec), S_{D1}	0.848g*	Section 1613.2.4 (Eqn 16-39)
<p>Note: *Per Section 11.4.8 of ASCE/SEI 7-16, a ground motion hazard analysis shall be performed for projects for Site Class “E” sites with S_s greater than or equal to 1.0g and for Site Class “D” and “E” sites with S_1 greater than 0.2g. Section 11.4.8 also provides exceptions which indicates that the ground motion hazard analysis may be waived provided the exceptions are followed. Using the code based values presented in the table above, in lieu of a performing a ground motion hazard analysis, requires the exceptions outlined in ASCE 7-16 Section 11.4.8 be followed.</p>		

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

ASCE 7-16 PEAK GROUND ACCELERATION

Parameter	Value	ASCE 7-16 Reference
Mapped MCE_G Peak Ground Acceleration, PGA	0.899g	Figure 22-7
Site Coefficient, F_{PGA}	1.1	Table 11.8-1
Site Class Modified MCE_G Peak Ground Acceleration, PGA_M	0.989g	Section 11.8.3 (Eqn 11.8-1)

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

Probabilistic seismic hazard deaggregation is used to identify the distribution of earthquake scenarios that contribute to exceedance of a given spectral acceleration level. Deaggregation of the MCE peak ground acceleration was performed using the USGS online Unified Hazard Tool, 2014 Conterminous U.S. Dynamic edition (v4.20). The result of the deaggregation analysis indicates that the predominant earthquake contributing to the MCE peak ground acceleration is characterized as a 6.78 magnitude event occurring at a hypocentral distance of 7.61 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 6.68 magnitude occurring at a hypocentral distance of 11.42 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.

A review of the State of California Seismic Hazard Zone Map for the Hollywood Quadrangle (CDMG, 1999; CGS, 2014; CGS, 2020b) indicates that the site is not located in an area designated as having a potential for liquefaction. Also, the historic high groundwater level is reported to be at a depth of approximately 40 feet beneath the existing ground surface (CDMG, 1998). Based on these considerations, the potential for liquefaction and associated ground deformations beneath the site is considered low. Further, based on the low potential for liquefaction, no surface manifestations of liquefaction are expected at the subject site.

6.5 Slope Stability

The site and the surrounding area are relatively level and the topography in the general area slopes gently to the south-southeast. The site is not located within a City of Los Angeles Hillside Grading Area or a Hillside Ordinance Area (City of Los Angeles, 2020). There are no known landslides near the site, nor is the site in the path of any known or potential landslides. Also, according to State of California Seismic Hazard Zone Map for the Hollywood Quadrangle (CDMG, 1999; CGS, 2020b), the site is not within an area identified as having a potential for seismic slope instability. 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. Based on a review of the Los Angeles County Safety Element (Leighton, 1990), the site is located within a potential inundation area for an earthquake-induced dam failure from Mulholland Dam. However, these reservoirs, as well as others in California, are continually monitored by various governmental agencies (such as the State of California Division of Safety of Dams and the U.S. Army Corps of Engineers) to guard against the threat of dam failure. Current design and construction practices and ongoing programs of review, modification, or total reconstruction of existing dams are intended to ensure that all dams are capable of withstanding the maximum considered earthquake (MCE) for the dam site. Therefore, the potential for inundation at the site as a result of an earthquake-induced dam failure is considered low.

6.7 Tsunamis, Seiches, and Flooding

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

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

The site is within an area of designated as Flood Zone X (0.2%). Flood Zone X (0.2%) is defined by the Federal Emergency Management Agency as an area with a 0.2% chance of annual flooding (FEMA, 2020; Los Angeles County, 2020b). Therefore, the potential for flooding at the site is considered low.

6.8 Oil Fields & Methane Potential

Based on a review of the California Geologic Energy Management Division (CalGEM) Well Finder Website, the site is not located within the limits of an oil field and there are no active or inactive oil or gas wells documented within the immediate vicinity of the site (CalGEM, 2020). However, due to the voluntary nature of record reporting by the oil well drilling companies, wells may be improperly located or not shown on the location map and undocumented wells could be encountered during construction. Any wells encountered during construction will need to be properly abandoned in accordance with the current requirements of the CalGEM.

The site is not located within a City of Los Angeles Methane Zone or Methane Buffer Zone (City of Los Angeles, 2020). 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 to occur 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. 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 8½ feet of existing artificial fill was encountered during site exploration. The existing fill encountered is believed to be the result of past grading and construction activities at the site. Deeper fill may exist in other areas of the site that were not directly explored. Demolition of the existing structures which occupy the site are anticipated to disturb the upper few feet of existing site soils. It is our opinion that the existing fill, in its present condition, is not suitable for direct support of proposed foundations or slabs. The existing fill and site soils are suitable for re-use as engineered fill provided the recommendation in the *Grading* section of this report are followed (see Section 7.5).
- 7.1.3 Excavations for the subterranean levels are anticipated to penetrate through the existing artificial fill and expose undisturbed alluvial soils throughout the excavation bottom.
- 7.1.4 Based on these considerations, the proposed structure may be supported on conventional foundation system deriving support in the competent alluvium found at and below a depth of 30 feet. Foundations should be deepened as necessary to penetrate through soft or unsuitable alluvium 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. Recommendations for the design of a conventional foundation system are provided in Section 7.7.
- 7.1.5 Groundwater was not encountered during site exploration and the current groundwater table is sufficiently deep that it not expected to be encountered during construction. However, local seepage could be encountered during excavation of the subterranean level, especially if conducted during the rainy season. If groundwater is encountered during excavation, temporary dewatering measures will be required to mitigate groundwater during excavation and construction. Recommendations for temporary dewatering are discussed in Section 7.4 of this report.

- 7.1.6 The subterranean portion of the structure which extends to depths greater than 40 feet below existing ground surface should be designed for full hydrostatic pressure. The historic high groundwater may be assumed at a depth of 40 feet for design. The recommended floor slab uplift pressure to be used in design would be $62.4(H)$ in units of pounds per square foot (psf), where “H” is the height of the water above the bottom of the foundation in feet. If the proposed structure does not provide sufficient dead load to resist the buoyant forces then uplift mitigation will be required. Recommendations for uplift resistance can be provided under separate cover, if needed.
- 7.1.7 Based on the depth of proposed construction and potential hydrostatic pressures, the proposed structure may also be supported on a reinforced concrete mat foundation system. A mat foundation system could be a very cost-effective foundation system for this project since the pad can remain relatively flat which allows for more efficient construction of waterproofing, saving time and labor. If desired, recommendations for a reinforced concrete mat foundation system can be provided under separate cover.
- 7.1.8 Excavations up to 45 feet in vertical height are anticipated for construction of the subterranean levels, including foundation depths. Due to the depth of the excavation and the proximity to the property lines, city streets and adjacent offsite structures, excavation of the proposed subterranean level will likely require sloping and shoring measures in order to provide a stable excavation. Where shoring is required it is recommended that a soldier pile shoring system be utilized. In addition, where the proposed excavation will be deeper than and adjacent to an offsite structure, the proposed shoring should be designed to resist the surcharge imposed by the adjacent offsite structure. Recommendations for shoring are provided in Section 7.19 of this report.
- 7.1.9 Due to the nature of the proposed design and intent for a subterranean level, waterproofing of subterranean walls and slabs is suggested. Particular care should be taken in the design and installation of waterproofing to avoid moisture problems, or actual water seepage into the structure through any normal shrinkage cracks which may develop in the concrete walls, floor slab, foundations and/or construction joints. The design and inspection of the waterproofing is not the responsibility of the geotechnical engineer. A waterproofing consultant should be retained in order to recommend a product or method, which would provide protection to subterranean walls, floor slabs and foundations.

- 7.1.10 Foundations for small outlying structures, such as block walls up to 6 feet in height, planter walls or trash enclosures, which will not be tied to the proposed structure, may be supported on conventional foundations deriving support on a minimum of 12 inches of newly placed engineered fill which extends laterally at least 12 inches beyond the foundation area. Where excavation and compaction cannot be performed or is undesirable, foundations may derive support directly in the competent undisturbed alluvial soils, and should be deepened as necessary to maintain a minimum 12-inch embedment into the recommended bearing materials. If the soils exposed in the excavation bottom are soft or loose, compaction of the soils will be required prior to placing steel or concrete. Compaction of the foundation excavation bottom is typically accomplished with a compaction wheel or mechanical whacker and must be observed and approved by a Geocon representative.
- 7.1.11 Once the design and foundation loading configuration for the proposed structure proceeds to a more finalized plan, the recommendations within this report should be reviewed and revised, if necessary. Based on the final foundation loading configurations, the potential for settlement should be reevaluated by this office.
- 7.1.12 Any changes in the design, location or elevation, as outlined in this report, should be reviewed by this office. Geocon should be contacted to determine the necessity for review and possible revision of this report.

7.2 Soil and Excavation Characteristics

- 7.2.1 The in-situ soils can be excavated with moderate effort using conventional excavation equipment. Caving should be anticipated in vertical excavations where granular soils are encountered.
- 7.2.2 It is the responsibility of the contractor to ensure that all excavations and trenches are properly shored and maintained in accordance with applicable OSHA rules and regulations to maintain safety and maintain the stability of 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 7.18).

7.2.4 The existing site soils encountered during the field investigation at the subterranean levels are considered to have a “low” (EI = 43) expansive potential and are classified as “expansive” in accordance with the 2019 California Building Code (CBC) Section 1803.5.3. The recommendations presented in this report assume that 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 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 soils are considered “corrosive” with respect to corrosion of buried ferrous metal on site. The results are presented in Appendix B (Figure B26) and should be considered for design of underground structures. Due to the corrosive potential of the soils, it is suggested that ABS pipes be considered in lieu of cast-iron for retaining wall drains and subdrains beneath the structure.

7.3.2 Laboratory tests were performed on representative samples of the on-site materials to measure the percentage of water-soluble sulfate content. Results from the laboratory water-soluble sulfate tests are presented in Appendix B (Figure B26) and indicate that the on-site materials possess “S0” sulfate exposure to concrete structures as defined by 2019 CBC Section 1904 and ACI 318-14 Table 19.3.1.1.

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

7.4 Temporary Dewatering

7.4.1 Groundwater was not encountered during site exploration and the current groundwater table is sufficiently deep that it not expected to be encountered during construction. However, local seepage could be encountered during excavation of the subterranean level, especially if conducted during the rainy season. The depth to groundwater at the time of construction can be verified during initial dewatering well or shoring pile installation. If groundwater is present above the depth of the subterranean level, temporary dewatering will be necessary to maintain a safe working environment during excavation and construction activities.

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

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

7.5 Grading

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

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

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

7.5.4 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.

- 7.5.5 The foundation system for the proposed structure may derive support in the competent undisturbed alluvial soils found at and below a depth of 30 feet. Foundations should be deepened as necessary to extend into satisfactory soils and must be observed and approved in writing by the Geotechnical Engineer (a representative of Geocon West, Inc.).
- 7.5.6 If construction is performed during the rainy season and the excavation bottom becomes saturated, stabilization measures may have to be implemented to prevent excessive disturbance the excavation bottom. Should this condition exist, rubber tire equipment should not be allowed in the excavation bottom until it is stabilized or extensive soil disturbance could result. Track mounted equipment should be considered to minimize disturbance to the soils.
- 7.5.7 One method of subgrade stabilization would consist of introducing a thin lift of 3- to 6-inch diameter crushed angular rock into the soft excavation bottom. The use of crushed concrete will also be acceptable. The crushed rock should be spread thinly across the excavation bottom and pressed into the soils by track rolling or wheel rolling with heavy equipment. It is very important that voids between the rock fragments are not created so the rock must be thoroughly pressed or blended into the soils. All subgrade soils must be properly compacted and proof-rolled in the presence of the Geotechnical Engineer (a representative of Geocon West, Inc.).
- 7.5.8 The City of Los Angeles Department of Building and Safety requires a minimum compactive effort of 95 percent of the laboratory maximum dry density in accordance with ASTM D 1557 (latest edition) where the soils to be utilized in the fill have less than 15 percent finer than 0.005 millimeters. Soils with more than 15 percent finer than 0.005 millimeters may be compacted to 90 percent of the laboratory maximum dry density in accordance with ASTM D 1557 (latest edition). All fill and backfill soils should be placed in horizontal loose layers approximately 6 to 8 inches thick, moisture conditioned to optimum moisture content, and properly compacted to the required degree of compaction in accordance with ASTM D 1557 (latest edition).
- 7.5.9 Foundations for small outlying structures, such as block walls up to 6 feet high, planter walls or trash enclosures, which will not be tied to 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 proper compaction cannot be performed or is undesirable, foundations may derive support directly in the undisturbed alluvial soils, and should be deepened as necessary to maintain a minimum 12-inch embedment into the recommended bearing materials. If the soils exposed in the excavation bottom are soft or loose, compaction of the soils will be required prior to placing steel or concrete. Compaction of the foundation excavation bottom is typically accomplished with a compaction wheel or mechanical whacker and must be observed and approved by a Geocon representative.

- 7.5.10 Although not anticipated for this project, all imported fill shall be observed, tested, and approved by Geocon West, Inc. prior to bringing soil to the site. Rocks larger than 6 inches in diameter shall not be used in the fill. If necessary, import soils used as structural fill should have an expansion index less than 30 and corrosivity properties that are equally or less detrimental to that of the existing onsite soils (see Figure B26).
- 7.5.11 Utility trenches should be properly backfilled in accordance with the requirements of the Green Book (latest edition). The pipe should be bedded with clean sands (Sand Equivalent greater than 30) to a depth of at least 1 foot over the pipe, and the bedding material must be inspected and approved in writing by the Geotechnical Engineer (a representative of Geocon). The use of gravel is not acceptable unless used in conjunction with filter fabric to prevent the gravel from having direct contact with soil. The remainder of the trench backfill may be derived from onsite soil or approved import soil, compacted as necessary, until the required compaction is obtained. The use of minimum 2-sack slurry as backfill is also acceptable (see Section 7.5). Prior to placing any bedding materials or pipes, the excavation bottom must be observed and approved in writing by the Geotechnical Engineer (a representative of Geocon)
- 7.5.12 All trench and foundation excavation bottoms must be observed and approved in writing by the Geotechnical Engineer (a representative of Geocon), prior to placing bedding materials, fill, steel, gravel, or concrete.

7.6 Controlled Low Strength Material (CLSM)

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

Standard Requirements

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

Requirements for CLSM that will be used for support of footings

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

7.7 Conventional Foundation Design

- 7.7.1 The proposed structure may be supported on a conventional spread foundation system deriving support in the competent alluvium found at and below a depth of 30 feet. Foundations should be deepened as necessary to penetrate through soft or unsuitable alluvium at the direction of the Geotechnical Engineer. All foundation excavations must be observed and approved in writing by the Geotechnical Engineer (a representative of Geocon), prior to placing steel or concrete.
- 7.7.2 Continuous footings may be designed for an allowable bearing capacity of 3,000 pounds per square foot (psf), and should be a minimum of 12 inches in width, 18 inches in depth below the lowest adjacent grade, and 12 inches into the recommended bearing material.
- 7.7.3 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.
- 7.7.4 The allowable soil bearing pressure above may be increased by 250 psf and 500 psf for each additional foot of foundation width and depth, respectively, up to a maximum allowable soil bearing pressure of 6,000 psf.
- 7.7.5 The allowable bearing pressures may be increased by one-third for transient loads due to wind or seismic forces.

- 7.7.6 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.
- 7.7.7 Continuous footings should be reinforced with four No. 4 steel reinforcing bars, two placed near the top of the footing and two near the bottom. Reinforcement for spread footings should be designed by the project structural engineer.
- 7.7.8 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.
- 7.7.9 No special subgrade presaturation is required prior to placement of concrete. However, the slab and foundation subgrade should be sprinkled as necessary; to maintain a moist condition as would be expected in any concrete placement.
- 7.7.10 If the portion of the proposed structure which extends below the historic high groundwater table is to be designed for full hydrostatic pressure, the recommended floor slab uplift pressure to be used in design would be $62.4(H)$ in units of psf, where “H” is the height of the water above the bottom of the foundation in feet. The historic high groundwater may be assumed at a depth of 40 feet for design.
- 7.7.11 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.
- 7.7.12 Waterproofing of subterranean walls and slabs is recommended for this project. Particular care should be taken in the design and installation of waterproofing to avoid moisture problems, or actual water seepage into the structure through any normal shrinkage cracks which may develop in the concrete walls, floor slab, foundations and/or construction joints. The design and inspection of the waterproofing is not the responsibility of the geotechnical engineer. A waterproofing consultant should be retained in order to recommend a product or method which would provide protection to subterranean walls, floor slabs and foundations.
- 7.7.13 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.

7.8 Foundation Settlement

- 7.8.1 The maximum expected static settlement for a structure supported on a conventional foundation system deriving support in the recommended bearing materials and designed with a maximum bearing pressure of 6,000 psf is estimated to be less than 1 inch and occur below the heaviest loaded structural element. A majority of the settlement of the foundation system is expected to occur on initial application of loading; however, minor additional settlements are expected within the first 12 months. Differential settlement is not expected to exceed ½ inch over a distance of 20 feet.
- 7.8.2 Once the design and foundation loading configurations for the proposed structures 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.

7.9 Miscellaneous Foundations

- 7.9.1 Foundations for small outlying structures, such as block walls up to 6 feet in height, planter walls or trash enclosures which will not be tied to the proposed structure may be supported on conventional foundations bearing on a minimum of 12 inches of newly placed engineered fill which extends laterally at least 12 inches beyond the foundation area. Where excavation and compaction cannot be performed or is undesirable, such as adjacent to property lines, foundations may derive support in the undisturbed alluvial soils, and should be deepened as necessary to maintain a minimum 12-inch embedment into the recommended bearing materials. The client should be aware that the depth to alluvium, as encountered in our borings, is on the order of 8½ feet below the ground surface; alternate foundation recommendations may be necessary and can be provided once the location of any miscellaneous structures are known.
- 7.9.2 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.
- 7.9.3 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.

7.10 Lateral Design

- 7.10.1 Resistance to lateral loading may be provided by friction acting at the base of foundations, slabs and by passive earth pressure. An allowable coefficient of friction of 0.35 may be used with the dead load forces in the competent alluvial soils or in properly compacted engineered fill.
- 7.10.2 Passive earth pressure for the sides of foundations and slabs poured against the alluvial soils above the groundwater table may be computed as an equivalent fluid having a density of 230 pcf with a maximum earth pressure of 2,300 pcf. Passive earth pressure for the sides of foundations and slabs poured against the alluvial soils below the groundwater table may be computed as an equivalent fluid having a density of 110 pcf with a maximum earth pressure of 1,100 pcf (these values have been adjusted for buoyant forces). When combining passive and friction for lateral resistance, the passive component should be reduced by one-third.

7.11 Concrete Slabs-on-Grade

- 7.11.1 Unless specifically evaluated and designed by a qualified structural engineer, the slab-on-grade subject to vehicle loading should be a minimum of 5 inches of concrete reinforced with No. 4 steel reinforcing bars placed 16 inches on center in both horizontal directions and positioned vertically near the slab midpoint. The concrete slab-on-grade and ramp may derive support directly on the undisturbed alluvial soils at the excavation bottom as well as compacted soils, if necessary. Any disturbed soils should be properly compacted for slab support. Soil placed and compacted for ramp and slab support should be moisture conditioned to optimum moisture content and properly compacted to at least 95 percent relative compaction, as determined by ASTM Test Method D 1557 (latest edition) for ramp support.

- 7.11.2 Slabs-on-grade at the ground surface that may receive moisture-sensitive floor coverings or may be used to store moisture-sensitive materials should be underlain by a vapor retarder placed directly beneath the slab. The vapor retarder and acceptable permeance should be specified by the project architect or developer based on the type of floor covering that will be installed. The vapor retarder selection and design should be consistent with the guidelines presented in Section 9.3 of the American Concrete Institute's (ACI) Guide for Concrete Slabs that Receive Moisture-Sensitive Flooring Materials (ACI 302.2R-06) as well as ASTM E1745 and should be installed in general conformance with ASTM E 1643 (latest edition) and the manufacturer's recommendations. A minimum thickness of 15 mils extruded polyolefin plastic is recommended; vapor retarders which contain recycled content or woven materials are not recommended. The vapor retarder should have a permeance of less than 0.01 perms demonstrated by testing before and after mandatory conditioning is recommended. The vapor retarder should be installed in direct contact with the concrete slab with proper perimeter seal. If the California Green Building Code requirements apply to this project, the vapor retarder should be underlain by 4 inches of clean aggregate. It is important that the vapor retarder be puncture resistant since it will be in direct contact with angular gravel. As an alternative to the clean aggregate suggested in the Green Building Code, it is our opinion that the concrete slab-on-grade may be underlain by a vapor retarder over 4-inches of clean sand (sand equivalent greater than 30), since the sand will serve a capillary break and will minimize the potential for punctures and damage to the vapor barrier.
- 7.11.3 Due to the nature of the proposed design and intent for a subterranean level, waterproofing of subterranean walls and slabs is suggested. Particular care should be taken in the design and installation of waterproofing to avoid moisture problems, or actual water seepage into the structure through any normal shrinkage cracks which may develop in the concrete walls, floor slab, foundations and/or construction joints. The design and inspection of the waterproofing is not the responsibility of the geotechnical engineer. A waterproofing consultant should be retained in order to recommend a product or method, which would provide protection to subterranean walls, floor slabs and foundations.
- 7.11.4 For seismic design purposes, a coefficient of friction of 0.35 may be utilized between concrete slabs and subgrade soils without a moisture barrier, and 0.15 for slabs underlain by a moisture barrier.

- 7.11.5 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). Crack control joints should be spaced at intervals not greater than 10 feet and should be constructed using saw-cuts or other methods as soon as practical following concrete placement. Crack control joints should extend a minimum depth of one-fourth the slab thickness. The project structural engineer should design construction joints as necessary.
- 7.11.6 The recommendations of this report are intended to reduce the potential for cracking of slabs due to settlement. However, even with the incorporation of the recommendations presented herein, foundations, stucco walls, and slabs-on-grade may exhibit some cracking due to minor soil movement and/or concrete shrinkage. The occurrence of concrete shrinkage cracks is independent of the supporting soil characteristics. Their occurrence may be reduced and/or controlled by limiting the slump of the concrete, proper concrete placement and curing, and by the placement of crack control joints at periodic intervals, in particular, where re-entrant slab corners occur.

7.12 Preliminary Pavement Recommendations

- 7.12.1 Where new paving is to be placed, it is recommended that all existing fill and soft or unsuitable alluvial materials be excavated and properly compacted for paving support. The client should be aware that excavation and compaction of all existing artificial fill and soft alluvium in the area of new paving is not required; however, paving constructed over existing unsuitable material may experience increased settlement and/or cracking, and may therefore have a shorter design life and increased maintenance costs. As a minimum, the upper 12 inches of paving subgrade should be scarified, moisture conditioned to optimum moisture content, and properly compacted to at least 95 percent relative compaction, as determined by ASTM Test Method D 1557 (latest edition).
- 7.12.2 The following pavement sections are based on an assumed R-Value of 20. Once site grading activities are complete an R-Value should be obtained by laboratory testing to confirm the properties of the soils serving as paving subgrade, prior to placing pavement.
- 7.12.3 The Traffic Indices listed below are estimates. Geocon does not practice in the field of traffic engineering. The actual Traffic Index for each area should be determined by the project civil engineer. If pavement sections for Traffic Indices other than those listed below are required, Geocon should be contacted to provide additional recommendations. Pavement thicknesses were determined following procedures outlined in the *California Highway Design Manual* (Caltrans). It is anticipated that the majority of traffic will consist of automobile and large truck traffic.

PRELIMINARY PAVEMENT DESIGN SECTIONS

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

- 7.12.4 Asphalt concrete should conform to Section 203-6 of the “*Standard Specifications for Public Works Construction*” (Green Book). Class 2 aggregate base materials should conform to Section 26-1.02A of the “*Standard Specifications of the State of California, Department of Transportation*” (Caltrans). The use of Crushed Miscellaneous Base (CMB) in place of Class 2 aggregate base is acceptable. Crushed Miscellaneous Base should conform to Section 200-2.4 of the “*Standard Specifications for Public Works Construction*” (Green Book).
- 7.12.5 Unless specifically designed and evaluated by the project structural engineer, where exterior concrete paving will be utilized for support of vehicles, it is recommended that the concrete be a minimum of 6 inches 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).
- 7.12.6 The performance of pavements is highly dependent upon providing positive surface drainage away from the edge of pavements. Ponding of water on or adjacent to the pavement will likely result in saturation of the subgrade materials and subsequent cracking, subsidence and pavement distress. If planters are planned adjacent to paving, it is recommended that the perimeter curb be extended at least 12 inches below the bottom of the aggregate base to minimize the introduction of water beneath the paving.

7.13 Retaining Wall Design

- 7.13.1 The recommendations presented below are generally applicable to the design of rigid concrete or masonry retaining walls having a maximum height of 45 feet. In the event that walls higher than 45 feet are planned, Geocon should be contacted for additional recommendations.
- 7.13.2 Retaining wall foundations may be designed in accordance with the recommendations provided in the *Foundation Design* section of this report (see Section 7.7).
- 7.13.3 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) of 56 pcf. Calculation of the recommended retaining wall pressures is provided on Figure 5.

- 7.13.4 Restrained walls are those that are not allowed to rotate more than 0.001H (where H equals the height of the retaining portion of the wall in feet) at the top of the wall. Assuming that proper drainage is maintained, where walls are restrained from movement at the top, walls may be designed utilizing a triangular distribution of pressure (at-rest pressure) of 70 pcf.
- 7.13.5 The wall pressures provided above assume that the retaining wall will be properly drained preventing the buildup of hydrostatic pressure. If retaining wall drainage is not implemented, the equivalent fluid pressure to be used in design of undrained walls is 105 pcf. The value includes hydrostatic pressures plus buoyant lateral earth pressures.
- 7.13.6 The wall pressures provided above assume that the proposed retaining walls will support relatively undisturbed alluvial soils. If sloping techniques are to be utilized for construction of proposed walls, which would result in a wedge of engineered fill behind the retaining walls, revised earth pressures may be required, especially if the wall backfill does not consist of the existing onsite soils. This should be evaluated once the use of sloping measures is established and once the geotechnical characteristics of the engineered backfill soils can be further evaluated.
- 7.13.7 Additional active pressure should be added for a surcharge condition due to sloping ground, vehicular traffic or adjacent structures and should be designed for each condition as the project progresses.
- 7.13.8 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:

$$\text{For } x/H \leq 0.4$$

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

and

$$\text{For } x/H > 0.4$$

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

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

- 7.13.9 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:

$$\text{For } x/H \leq 0.4$$

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

and

$$\text{For } x/H > 0.4$$

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

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 .

- 7.13.10 In addition to the recommended earth pressure, the upper 10 feet of the subterranean wall adjacent to the street and parking lot should be designed to resist a uniform lateral pressure of 100 psf, acting as a result of an assumed 300 psf surcharge behind the walls due to normal street traffic. If the traffic is kept back at least 10 feet from the subterranean walls, the traffic surcharge may be neglected.
- 7.13.11 Seismic lateral forces should be incorporated into the design as necessary, and recommendations for seismic lateral forces are presented below.

7.14 Dynamic (Seismic) Lateral Forces

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

- 7.14.2 A seismic load of 10 pcf should be used for design of walls that support more than 6 feet of backfill in accordance with Section 1803.5.12 of the 2019 CBC. The seismic load is applied as an equivalent fluid pressure along the height of the wall and the calculated loads result in a maximum load exerted at the base of the wall and zero at the top of the wall. This seismic load should be applied in addition to the active earth pressure, not the at-rest pressure. The earth pressure is based on half of two-thirds of PGA_M calculated from ASCE 7-16 Section 11.8.3.

7.15 Retaining Wall Drainage

- 7.15.1 Retaining walls not designed for hydrostatic pressures should be provided with a drainage system. At the base of the drain system, a subdrain covered with a minimum of 12 inches of gravel should be installed, and a compacted fill blanket or other seal placed at the surface (see Figure 6). The clean bottom and subdrain pipe, behind a retaining wall, should be observed by the Geotechnical Engineer (a representative of Geocon), prior to placement of gravel or compacting backfill.
- 7.15.2 As an alternative, a plastic drainage composite such as Miradrain or equivalent may be installed in continuous, 4-foot wide columns along the entire back face of the wall, at 8 feet on center. The top of these drainage composite columns should terminate approximately 18 inches below the ground surface, where either hardscape or a minimum of 18 inches of relatively cohesive material should be placed as a cap (see Figure 7). These vertical columns of drainage material would then be connected at the bottom of the wall to a collection panel or a one-cubic-foot rock pocket drained by a 4-inch subdrain pipe.
- 7.15.3 Subdrainage pipes at the base of the retaining wall drainage system should outlet to an acceptable location via controlled drainage structures.
- 7.15.4 Moisture affecting below grade walls is one of the most common post-construction complaints. Poorly applied or omitted waterproofing can lead to efflorescence or standing water. Particular care should be taken in the design and installation of waterproofing to avoid moisture problems, or actual water seepage into the structure through any normal shrinkage cracks which may develop in the concrete walls, floor slab, foundations and/or construction joints. The design and inspection of the waterproofing is not the responsibility of the geotechnical engineer. A waterproofing consultant should be retained in order to recommend a product or method, which would provide protection to subterranean walls, floor slabs and foundations.

7.16 Elevator Pit Design

- 7.16.1 The elevator pit slab and retaining wall should be designed by the project structural engineer. Elevator pits may be designed in accordance with the recommendations in the *Foundation Design and Retaining Wall Design* section of this report (see Sections 7.7 and 7.13).

- 7.16.2 Additional active pressure should be added for a surcharge condition due to sloping ground, vehicular traffic, or adjacent foundations and should be designed for each condition as the project progresses.
- 7.16.3 If retaining wall drainage is to be provided, the drainage system should be designed in accordance with the *Retaining Wall Drainage* section of this report (see Section 7.15).
- 7.16.4 It is suggested that the exterior walls and slab be waterproofed to prevent excessive moisture inside of the elevator pit. Waterproofing design and installation is not the responsibility of the geotechnical engineer.

7.17 Elevator Piston

- 7.17.1 If a plunger-type elevator piston is installed for this project, a deep drilled excavation will be required. It is important to verify that the drilled excavation is not situated immediately adjacent to a foundation or shoring pile, or the drilled excavation could compromise the existing foundation or pile support, especially if the drilling is performed subsequent to the foundation or pile construction.
- 7.17.2 Casing will likely be required since caving is expected in the drilled excavation. The contractor should be prepared to use casing and should have it readily available at the commencement of drilling activities. Continuous observation of the drilling and installation of the elevator piston by the Geotechnical Engineer (a representative of Geocon West, Inc.) is required.
- 7.17.3 The annular space between the piston casing and drilled excavation wall should be filled with a minimum of 1½-sack slurry pumped from the bottom up. As an alternative, pea gravel may be utilized. The use of soil to backfill the annular space is not acceptable.

7.18 Temporary Excavations

- 7.18.1 Excavations up to 45 feet in height are anticipated for excavation and construction of the proposed subterranean levels and foundation system. The excavations are expected to expose alluvial soils, which are suitable for vertical excavations up to 5 feet where loose soils or caving sands are not present or where not surcharged by adjacent traffic or structures.
- 7.18.2 Vertical excavations greater than 5 feet will require sloping and/or shoring measures in order to provide a stable excavation. Where sufficient space is available, temporary unsurcharged embankments could be sloped back at a uniform 1:1 slope gradient or flatter, up to a maximum of 8 feet in height. A uniform slope does not have a vertical portion. Where space is limited, shoring measures will be required. *Shoring* data is provided in Section 7.18 of this report.

7.18.3 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.

7.19 Shoring – Soldier Pile Design and Installation

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

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

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

7.19.4 The proposed soldier piles may also be designed as permanent piles. The required pile depth, dimension, and spacing should be determined and designed by the project structural and shoring engineers. All piles utilized for shoring can also be incorporated into a permanent retaining wall system (shotcrete wall) and should be designed in accordance with the earth pressure provided in the *Retaining Wall Design* section of this report (see Section 7.13).

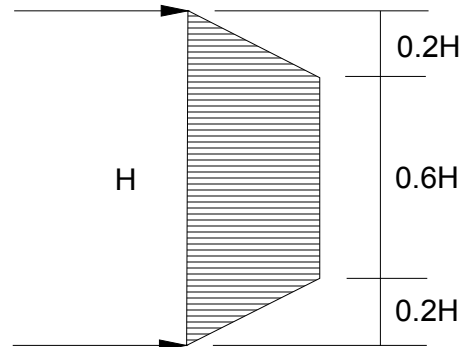
- 7.19.5 Drilled cast-in-place soldier piles should be placed no closer than three diameters on center. The minimum diameter of the piles is 18 inches. Structural concrete should be used for the soldier piles below the excavation; lean-mix concrete may be employed above that level. As an alternative, 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 260 psf per foot. Where piles are installed by vibration techniques, the passive pressure may be assumed to mobilize across a width equal to the two times the dimension of the beam flange. The allowable passive value may be doubled for isolated piles, spaced a minimum of two 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.
- 7.19.6 Groundwater was not encountered during site exploration, and groundwater is not expected to be encountered during construction. However, should groundwater or local seepage be encountered during pile installation, the contractor should be prepared. Piles placed below the water level require the use of a tremie to place the concrete into the bottom of the hole. A tremie should consist of a rigid, water-tight tube having a diameter of not less than 6 inches with a hopper at the top. The tube should be equipped with a device that will close the discharge end and prevent water from entering the tube while it is being charged with concrete. The tremie should be supported so as to permit free movement of the discharge end over the entire top surface of the work and to permit rapid lowering when necessary to retard or stop the flow of concrete. The discharge end should be closed at the start of the work to prevent water entering the tube and should be entirely sealed at all times, except when the concrete is being placed. The tremie tube should be kept full of concrete. The flow should be continuous until the work is completed and the resulting concrete seal should be monolithic and homogeneous. The tip of the tremie tube should always be kept about 5 feet below the surface of the concrete and definite steps and safeguards should be taken to insure that the tip of the tremie tube is never raised above the surface of the concrete.
- 7.19.7 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.

- 7.19.8 Casing will likely be required since caving is expected in the granular soils, and the contractor should have casing available prior to commencement of pile excavation. When casing is used, extreme care should be employed so that the pile is not pulled apart as the casing is withdrawn. At no time should the distance between the surface of the concrete and the bottom of the casing be less than 5 feet. As an alternative, piles may be vibrated into place; however, there is always a risk that excessive vibrations in sandy soils could induce settlements and distress to adjacent offsite improvements. Continuous observation of the drilling and pouring of the piles by the Geotechnical Engineer (a representative of Geocon West, Inc.), is required.
- 7.19.9 As an alternative, piles may be vibrated into place; however, there is always a risk that excessive vibrations in sandy soils could induce settlements and distress to adjacent offsite improvements. Continuous observation of the drilling and pouring of the piles by the Geotechnical Engineer (a representative of Geocon West, Inc.), is required.
- 7.19.10 If a vibratory method of soldier pile installation is utilized, predrilling may be performed prior to installation of the steel beams. If predrilling is performed, the bore diameter should be no greater than 75 percent of the largest dimension of the pile to prevent excessive loss in the frictional component of the pile capacity. Predrilling should not be conducted below the proposed excavation bottom, and the auger should be backspun out of the pilot holes, leaving the soil in place.
- 7.19.11 If a vibratory method is utilized, the owner should be aware of the potential risks associated with vibratory efforts, which typically involve inducing settlement within the vicinity of the pile which could result in a potential for damage to existing improvements in the area.
- 7.19.12 The level of vibration that results from the installation of the piles should not exceed a threshold where occupants of nearby structures are disturbed, despite higher vibration tolerances that a building may endure without deformation or damage. The main parameter used for vibration assessment is peak particle velocity in units of inch per second (in/sec). The acceptable range of peak particle velocity should be evaluated based on the age and condition of adjacent structures, as well as the tolerance of human response to vibration.
- 7.19.13 Based on Table 19 of the *Transportation and Construction Induced Vibration Guidance Manual* (Caltrans 2013), a continuous source of vibrations (ex. vibratory pile driving) which generates a maximum peak particle velocity of 0.5 in/sec is considered tolerable for modern industrial/commercial buildings and new residential structures. The Client should be aware that a lower value may be necessary if older or fragile structures are in the immediate vicinity of the site.

- 7.19.14 Vibrations should be monitored and record with seismographs during pile installation to detect the magnitude of vibration and oscillation experienced by adjacent structures. If the vibrations exceed the acceptable range during installation, the shoring contractor should modify the installation procedure to reduce the values to within the acceptable range. Vibration monitoring is not the responsibility of the Geotechnical Engineer.
- 7.19.15 Geocon does not practice in the field of vibration monitoring. If construction techniques will be implemented, it is recommended that qualified consultant be retained to provide site specific recommendations for vibration thresholds and monitoring.
- 7.19.16 The frictional resistance between the soldier piles and retained soil may be used to resist the vertical component of the anchor load. The coefficient of friction may be taken as 0.4 based on uniform contact between the steel beam and lean-mix concrete and retained earth. The portion of soldier piles below the plane of excavation may also be employed to resist the downward loads. The downward capacity may be determined using a frictional resistance of 800 psf per foot.
- 7.19.17 Due to the nature of the site soils, it is expected that continuous lagging between soldier piles will be required. However, it is recommended that the exposed soils be observed by the Geotechnical Engineer (a representative of Geocon West, Inc.), to verify the presence of any competent, cohesive soils and the areas where lagging may be omitted.
- 7.19.18 The time between lagging excavation and lagging placement should be as short as possible. Soldier piles should be designed for the full-anticipated pressures. Due to arching in the soils, the pressure on the lagging will be less. It is recommended that the lagging be designed for the full design pressure but be limited to a maximum of 400 psf. The 400 psf surcharge pressure on the lagging is due to the earth pressure only. Any potential surcharges should be evaluated in addition to earth pressures provided herein. We are relying on a qualified and licensed shoring engineer to design the shoring and lagging for this project. The proper strength lagging board should be selected and incorporated into the shoring design by a qualified shoring engineer.
- 7.19.19 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. Calculation of the recommended shoring pressures is provided in Figure 8.

HEIGHT OF SHORING (FEET)	EQUIVALENT FLUID PRESSURE (Pounds Per Cubic Foot) (ACTIVE PRESSURE)	EQUIVALENT FLUID PRESSURE Trapezoidal (Where H is the height of the shoring in feet)
Up to 45	47	30H

Trapezoidal Distribution of Pressure



- 7.19.20 Where a combination of sloped temporary excavations above 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.
- 7.19.21 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.

7.19.22 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:

$$\text{For } x/H \leq 0.4$$

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

and

$$\text{For } x/H > 0.4$$

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

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 .

7.19.23 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:

$$\text{For } x/H \leq 0.4$$

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

and

$$\text{For } x/H > 0.4$$

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

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 .

- 7.19.24 In addition to the recommended earth pressure, the upper 10 feet of the shoring adjacent to the street or driveway areas should be designed to resist a uniform lateral pressure of 100 psf, acting as a result of an assumed 300 psf surcharge behind the shoring due to normal street traffic. If the traffic is kept back at least 10 feet from the shoring, the traffic surcharge may be neglected.
- 7.19.25 It is difficult to accurately predict the amount of deflection of a shored embankment. It should be realized that some deflection will occur. It is recommended that the deflection be minimized to prevent damage to existing structures and adjacent improvements. Where public right-of-ways are present or adjacent offsite structures do not surcharge the shoring excavation, the shoring deflection should be limited to less than 1 inch at the top of the shored embankment. Where offsite structures are within the shoring surcharge area it is recommended that the beam deflection be limited to less than ½ inch at the elevation of the adjacent offsite foundation, and no deflection at all if deflections will damage existing structures. The allowable deflection is dependent on many factors, such as the presence of structures and utilities near the top of the embankment, and will be assessed and designed by the project shoring engineer.
- 7.19.26 Because of the depth of the excavation, some means of monitoring the performance of the shoring system is suggested. The monitoring should consist of periodic surveying of the lateral and vertical locations of the tops of all soldier piles and the lateral movement along the entire lengths of selected soldier piles.
- 7.19.27 Due to the depth of the excavation and proximity to adjacent structures, it is suggested that prior to excavation the existing improvements be inspected to document the present condition. For documentation purposes, photographs should be taken of preconstruction distress conditions and level surveys of adjacent grade and pavement should be considered. During excavation activities, the adjacent structures and pavement should be periodically inspected for signs of distress. In the event that distress or settlement is noted, an investigation should be performed and corrective measures taken so that continued or worsened distress or settlement is mitigated. Documentation and monitoring of the offsite structures and improvements is not the responsibility of the geotechnical engineer.

7.20 Temporary Tie-Back Anchors

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

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

- 7 feet below the top of the excavation – 950 pounds per square foot
- 15 feet below the top of the excavation – 1,250 pounds per square foot
- 25 feet below the top of the excavation – 1,550 pounds per square foot
- 35 feet below the top of the excavation – 2,500 pounds per square foot

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

7.21 Anchor Installation

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

7.22 Anchor Testing

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

- 7.22.2 At least 10 percent of the anchors should be selected for "quick" 200 percent tests and three additional anchors should be selected for 24-hour 200 percent tests. The purpose of the 200 percent tests is to verify the friction value assumed in design. The anchors should be tested to develop twice the assumed friction value. These tests should be performed prior to installation of additional tiebacks. Where satisfactory tests are not achieved on the initial anchors, the anchor diameter and/or length should be increased until satisfactory test results are obtained.
- 7.22.3 The total deflection during the 24-hour 200 percent test should not exceed 12 inches. During the 24-hour tests, the anchor deflection should not exceed 0.75 inches measured after the 200 percent test load is applied.
- 7.22.4 For the "quick" 200 percent tests, the 200 percent test load should be maintained for 30 minutes. The total deflection of the anchor during the 200 percent quick tests should not exceed 12 inches; the deflection after the 200 percent load has been applied should not exceed 0.25 inch during the 30-minute period.
- 7.22.5 After a satisfactory test, each anchor should be locked-off at the design load. This should be verified by rechecking the load in the anchor. The load should be within 10 percent of the design load. A representative of this firm should observe the installation and testing of the anchors.

7.23 Internal Bracing

- 7.23.1 Rakers may be utilized to brace the soldier piles in lieu of tieback anchors. The raker bracing could be supported laterally by temporary concrete footings (deadmen) or by the permanent, interior footings. For design of such temporary footings or deadmen, poured with the bearing surface normal to rakers inclined at 45 degrees, a bearing value of 3,000 psf may be used, provided the shallowest point of the footing is at least 1 foot below the lowest adjacent grade. The structural engineer should review the shoring plans to determine if raker footings conflict with the structural foundation system. The client should be aware that the utilization of rakers could significantly impact the construction schedule due to their intrusion into the construction site and potential interference with equipment.

7.24 Surface Drainage

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

- 7.24.2 All site drainage should be collected and controlled in non-erosive drainage devices. Drainage should not be allowed to pond anywhere on the site, and especially not against any foundation or retaining wall. The site should be graded and maintained such that surface drainage is directed away from structures in accordance with 2019 CBC 1804.4 or other applicable standards. In addition, drainage should not be allowed to flow uncontrolled over any descending slope. Discharge from downspouts, roof drains and scuppers are not recommended onto unprotected soils within five feet of the building perimeter. Planters which are located adjacent to foundations should be sealed to prevent moisture intrusion into the soils providing foundation support. Landscape irrigation is not recommended within 5 feet of the building perimeter footings except when enclosed in protected planters.
- 7.24.3 Positive site drainage should be provided away from structures, pavement, and the tops of slopes to swales or other controlled drainage structures. The building pad and pavement areas should be fine graded such that water is not allowed to pond.
- 7.24.4 Landscaping planters immediately adjacent to paved areas are not recommended due to the potential for surface or irrigation water to infiltrate the pavement's subgrade and base course. Either a subdrain, which collects excess irrigation water and transmits it to drainage structures, or an impervious above-grade planter boxes should be used. In addition, where landscaping is planned adjacent to the pavement, it is recommended that consideration be given to providing a cutoff wall along the edge of the pavement that extends at least 12 inches below the base material.

7.25 Plan Review

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

LIMITATIONS AND UNIFORMITY OF CONDITIONS

1. The recommendations of this report pertain only to the site investigated and are based upon the assumption that the soil conditions do not deviate from those disclosed in the investigation. If any variations or undesirable conditions are encountered during construction, or if the proposed construction will differ from that anticipated herein, Geocon West, Inc. should be notified so that supplemental recommendations can be given. The evaluation or identification of the potential presence of hazardous or corrosive materials was not part of the scope of services provided by Geocon West, Inc.
2. This report is issued with the understanding that it is the responsibility of the owner, or of his representative, to ensure that the information and recommendations contained herein are brought to the attention of the architect and engineer for the project and incorporated into the plans, and the necessary steps are taken to see that the contractor and subcontractors carry out such recommendations in the field.
3. The findings of this report are valid as of the date of this report. However, changes in the conditions of a property can occur with the passage of time, whether they are due to natural processes or the works of man on this or adjacent properties. In addition, changes in applicable or appropriate standards may occur, whether they result from legislation or the broadening of knowledge. Accordingly, the findings of this report may be invalidated wholly or partially by changes outside our control. Therefore, this report is subject to review and should not be relied upon after a period of three years.
4. The firm that performed the geotechnical investigation for the project should be retained to provide testing and observation services during construction to provide continuity of geotechnical interpretation and to check that the recommendations presented for geotechnical aspects of site development are incorporated during site grading, construction of improvements, and excavation of foundations. If another geotechnical firm is selected to perform the testing and observation services during construction operations, that firm should prepare a letter indicating their intent to assume the responsibilities of project geotechnical engineer of record. A copy of the letter should be provided to the regulatory agency for their records. In addition, that firm should provide revised recommendations concerning the geotechnical aspects of the proposed development, or a written acknowledgement of their concurrence with the recommendations presented in our report. They should also perform additional analyses deemed necessary to assume the role of Geotechnical Engineer of Record.

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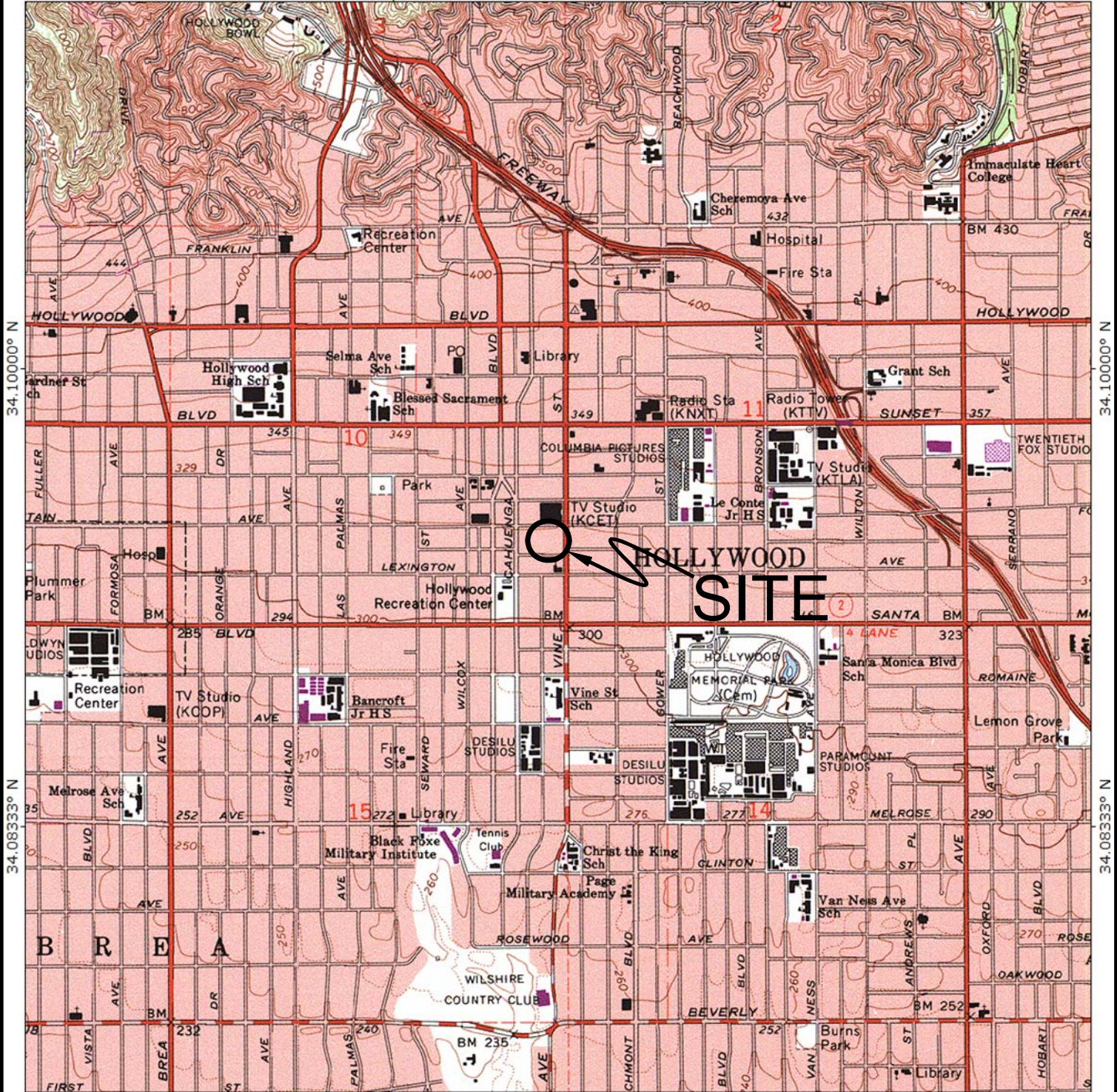
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- Ziony, J. I., and Jones, L. M., 1989, *Map Showing Late Quaternary Faults and 1978–1984 Seismicity of the Los Angeles Region, California*, U.S. Geological Survey Miscellaneous Field Studies Map MF-1964.

118.33333° W

NAD27 118.31667° W



34.10000° N

34.10000° N

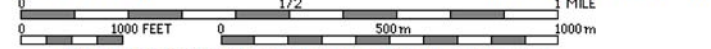
34.08333° N

34.08333° N



118.33333° W

NAD27 118.31667° W



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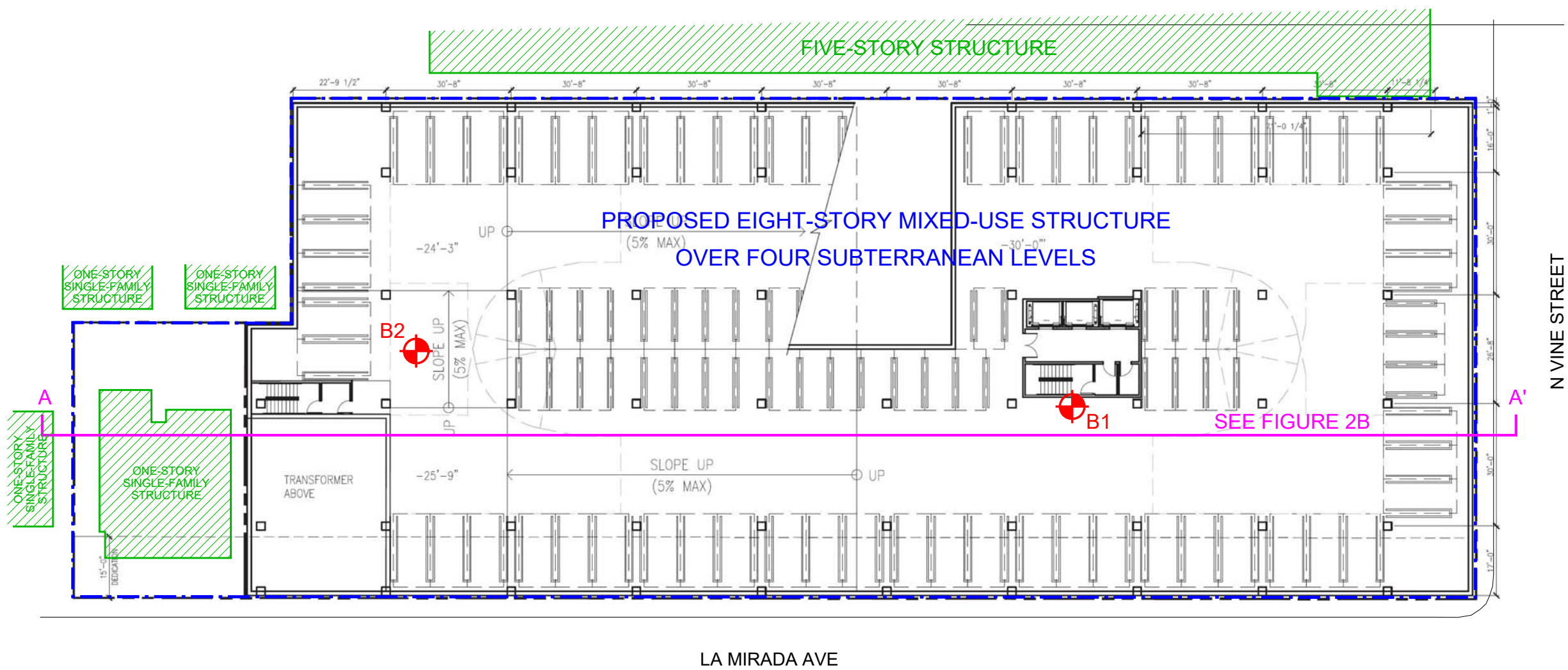
VICINITY MAP

1235 VINE STREET
LOS ANGELES, CALIFORNIA



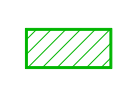
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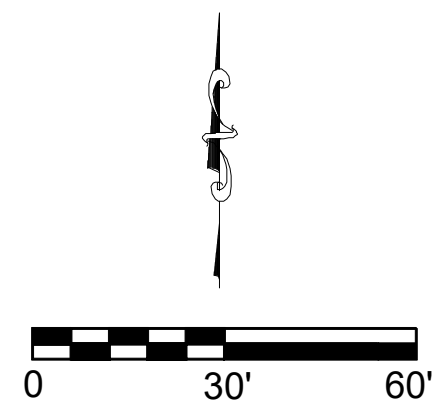
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FIG. 1



LEGEND

-  Approximate Location of Boring
-  Approximate Location of Property Line
-  Location of Existing Offsite Structure



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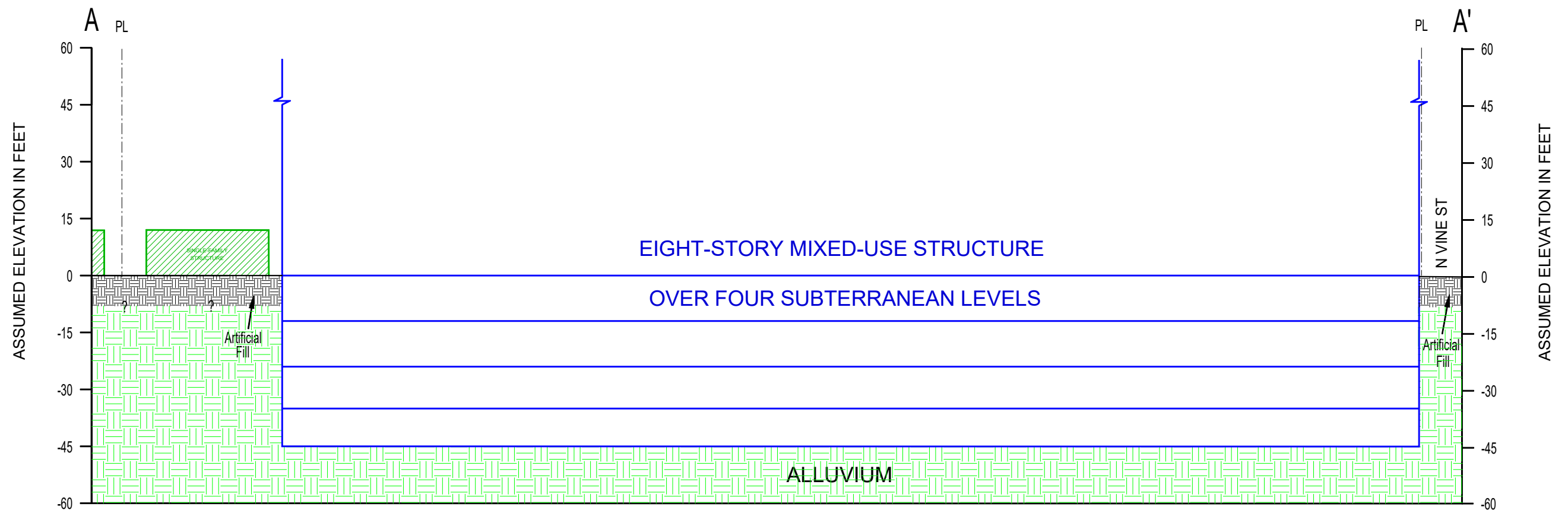
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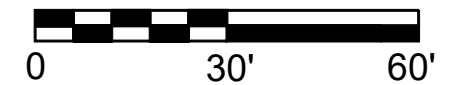
SITE PLAN

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JULY 2020 PROJECT NO. W1135-06-01 FIG. 2A



SECTION A - A'



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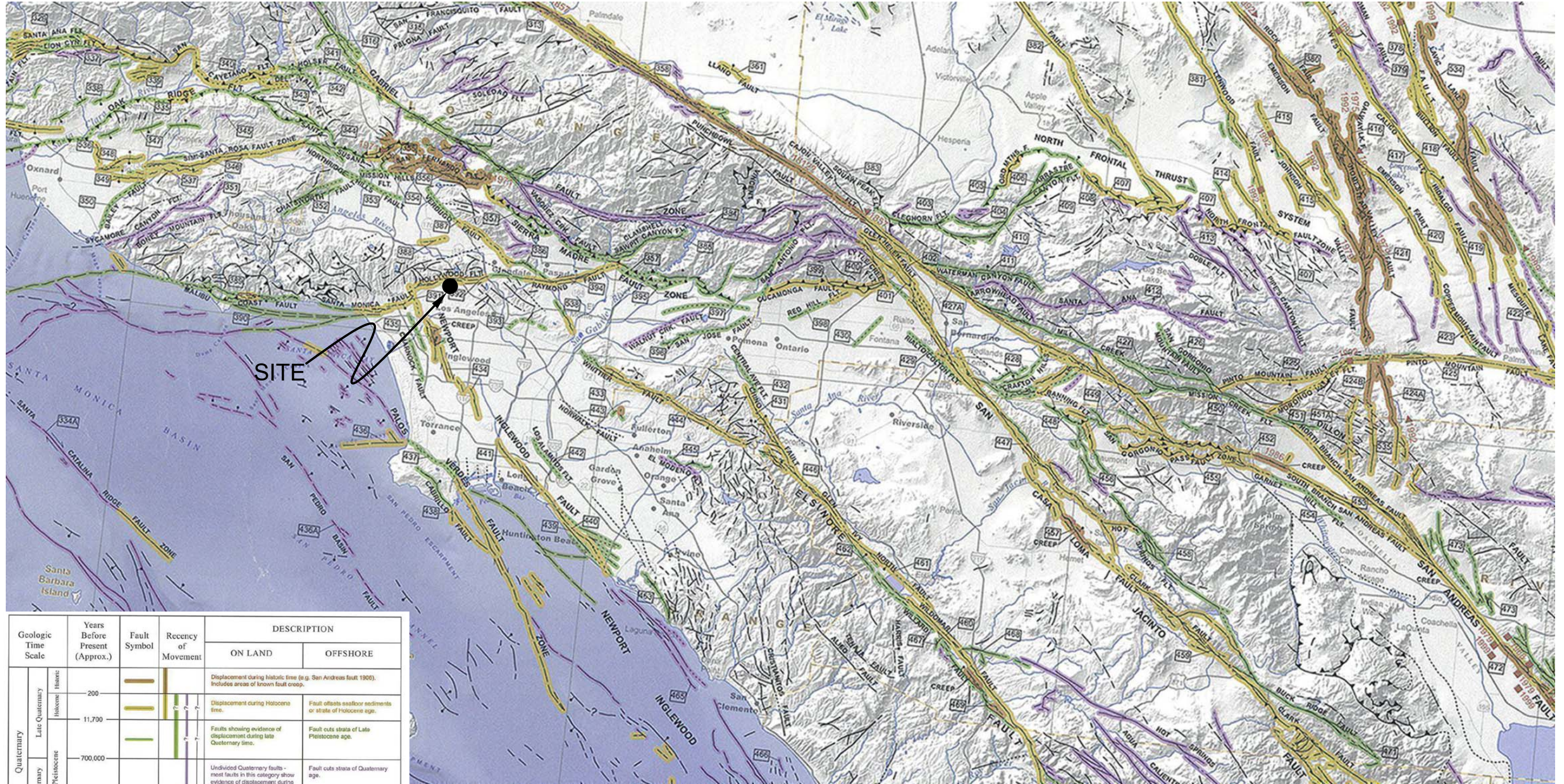
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CROSS SECTION

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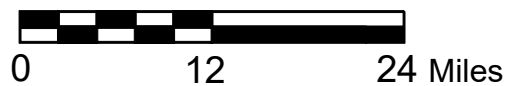
JULY 2020 PROJECT NO. W1135-06-01 FIG. 2B

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



Geologic Time Scale	Years Before Present (Approx.)	Fault Symbol	Recency of Movement	DESCRIPTION	
				ON LAND	OFFSHORE
Quaternary	Late Quaternary Holocene Recent	[Symbol]	[Symbol]	Displacement during historic time (e.g. San Andreas fault 1906). Includes areas of known fault creep.	Fault offsets onshore sediments or strata of Holocene age.
				Faults showing evidence of displacement during late Quaternary time.	Fault cuts strata of Late Pleistocene age.
	Early Quaternary Pleistocene	[Symbol]	[Symbol]	Undivided Quaternary faults - most faults in this category show evidence of displacement during the last 1,800,000 years; possible exceptions are faults which displace rocks of undifferentiated Plio-Pleistocene age.	Fault cuts strata of Quaternary age.
Pre-Quaternary	1,800,000 4.5 billion (Age of Earth)	[Symbol]	[Symbol]	Faults without recognized Quaternary displacement or showing evidence of no displacement during Quaternary time. Not necessarily inactive.	Fault cuts strata of Pliocene or older age.

* Quaternary now recognized as extending to 2.6 Ma (Walker and Gessman, 2009). Quaternary faults in this map were established using the previous 1.8 Ma criterion.



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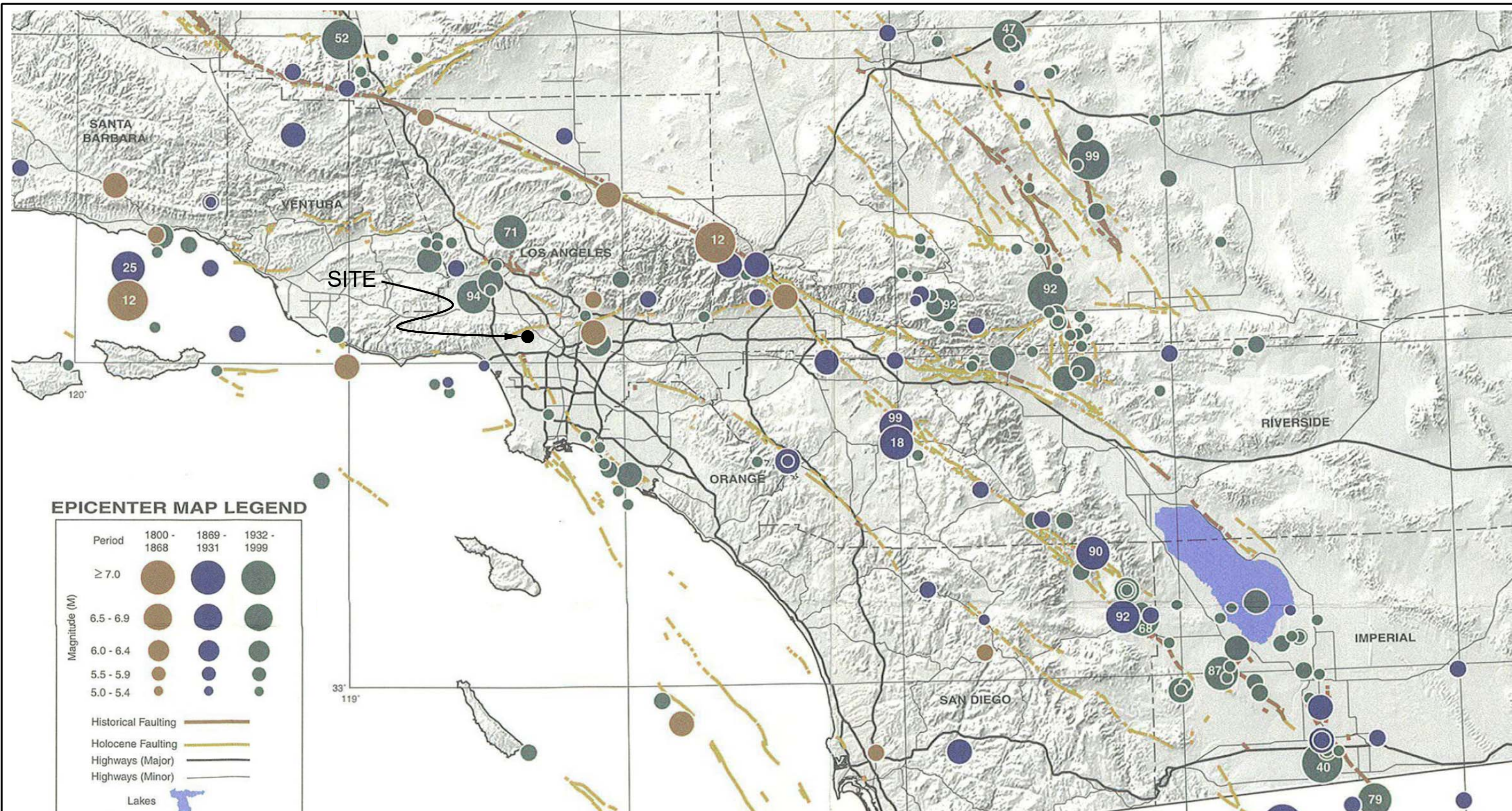
REGIONAL FAULT MAP

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JULY 2020

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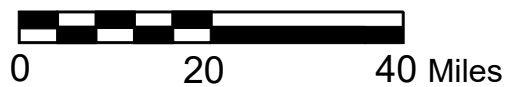
FIG. 3



EPICENTER MAP LEGEND

Period	1800 - 1868	1869 - 1931	1932 - 1999
Magnitude (M)			
≥ 7.0			
6.5 - 6.9			
6.0 - 6.4			
5.5 - 5.9			
5.0 - 5.4			
Historical Faulting			
Holocene Faulting			
Highways (Major)			
Highways (Minor)			
Lakes			
	Last two digits of M ≥ 6.5 earthquake year		

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



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REGIONAL SEISMICITY MAP

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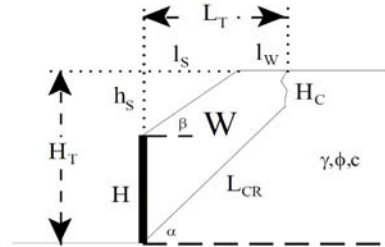
FIG.4

Retaining Wall Design with Transitioned Backfill (Vector Analysis)

Input:

Retaining Wall Height	(H)	45.00 feet
Slope Angle of Backfill	(b)	0.0 degrees
Height of Slope above Wall	(h _s)	0.0 feet
Horizontal Length of Slope	(l _s)	0.0 feet
Total Height (Wall + Slope)	(H _T)	45.0 feet
Unit Weight of Retained Soils	(g)	125.0 pcf
Friction Angle of Retained Soils	(f)	26.0 degrees
Cohesion of Retained Soils	(c)	240.0 psf
Factor of Safety	(F _S)	1.50

Factored Parameters:	(f _{FS})	18.0 degrees
	(c _{FS})	160.0 psf



Failure Angle (a) degrees	Height of Tension Crack (H _c) feet	Area of Wedge (A) feet ²	Weight of Wedge (W) lbs/lineal foot	Length of Failure Plane (L _{CR}) feet	a lbs/lineal foot	b lbs/lineal foot	Active Pressure (P _A) lbs/lineal foot
45	3.8	1005	125663.1	58.3	19539.4	106123.7	54044.3
46	3.7	971	121378.4	57.4	18600.2	102778.2	54620.1
47	3.7	938	117230.9	56.5	17737.5	99493.3	55122.4
48	3.6	906	113211.9	55.7	16943.2	96268.6	55553.4
49	3.6	875	109313.5	54.9	16210.3	93103.2	55915.1
50	3.6	844	105528.3	54.1	15532.6	89995.7	56208.9
51	3.6	815	101849.5	53.3	14904.7	86944.8	56436.3
52	3.5	786	98270.6	52.6	14321.9	83948.7	56598.1
53	3.5	758	94785.7	51.9	13780.0	81005.7	56695.1
54	3.5	731	91389.0	51.3	13275.2	78113.8	56727.6
55	3.5	705	88075.5	50.6	12804.2	75271.3	56695.9
56	3.5	679	84840.2	50.0	12364.1	72476.1	56599.7
57	3.6	653	81678.5	49.4	11952.2	69726.3	56438.7
58	3.6	629	78586.0	48.8	11566.1	67019.9	56212.1
59	3.6	604	75558.8	48.3	11203.6	64355.2	55919.1
60	3.6	581	72593.0	47.8	10862.9	61730.1	55558.2
61	3.7	557	69685.0	47.2	10542.1	59142.8	55128.1
62	3.7	535	66831.3	46.7	10239.6	56591.6	54626.6
63	3.8	512	64028.7	46.2	9954.0	54074.7	54051.7
64	3.9	490	61274.2	45.8	9683.8	51590.4	53400.7
65	3.9	469	58564.8	45.3	9427.7	49137.1	52670.6
66	4.0	447	55897.8	44.8	9184.7	46713.1	51858.0
67	4.1	426	53270.4	44.4	8953.4	44316.9	50958.9
68	4.2	405	50680.0	44.0	8732.9	41947.1	49969.0
69	4.4	385	48124.3	43.5	8522.1	39602.2	48883.4
70	4.5	365	45600.8	43.1	8319.9	37280.8	47696.4

Design Equations (Vector Analysis):
 $a = c_{FS} \cdot L_{CR} \cdot \sin(90 + f_{FS}) / \sin(a - f_{FS})$
 $b = W - a$
 $P_A = b \cdot \tan(a - f_{FS})$
 $EFP = 2 \cdot P_A / H^2$

Maximum Active Pressure Resultant

$P_{A, max}$

56727.6 lbs/lineal foot

Equivalent Fluid Pressure (per lineal foot of wall)

$EFP = 2 \cdot P_A / H^2$

At-Rest = $\gamma \cdot (1 - \sin(\phi))$

EFP

56.0 pcf

70.2 pcf

Design Wall for an Equivalent Fluid Pressure:

56 pcf

70 pcf

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RETAINING WALL PRESSURE CALCULATION

1235 NORTH VINE STREET
LOS ANGELES, CALIFORNIA

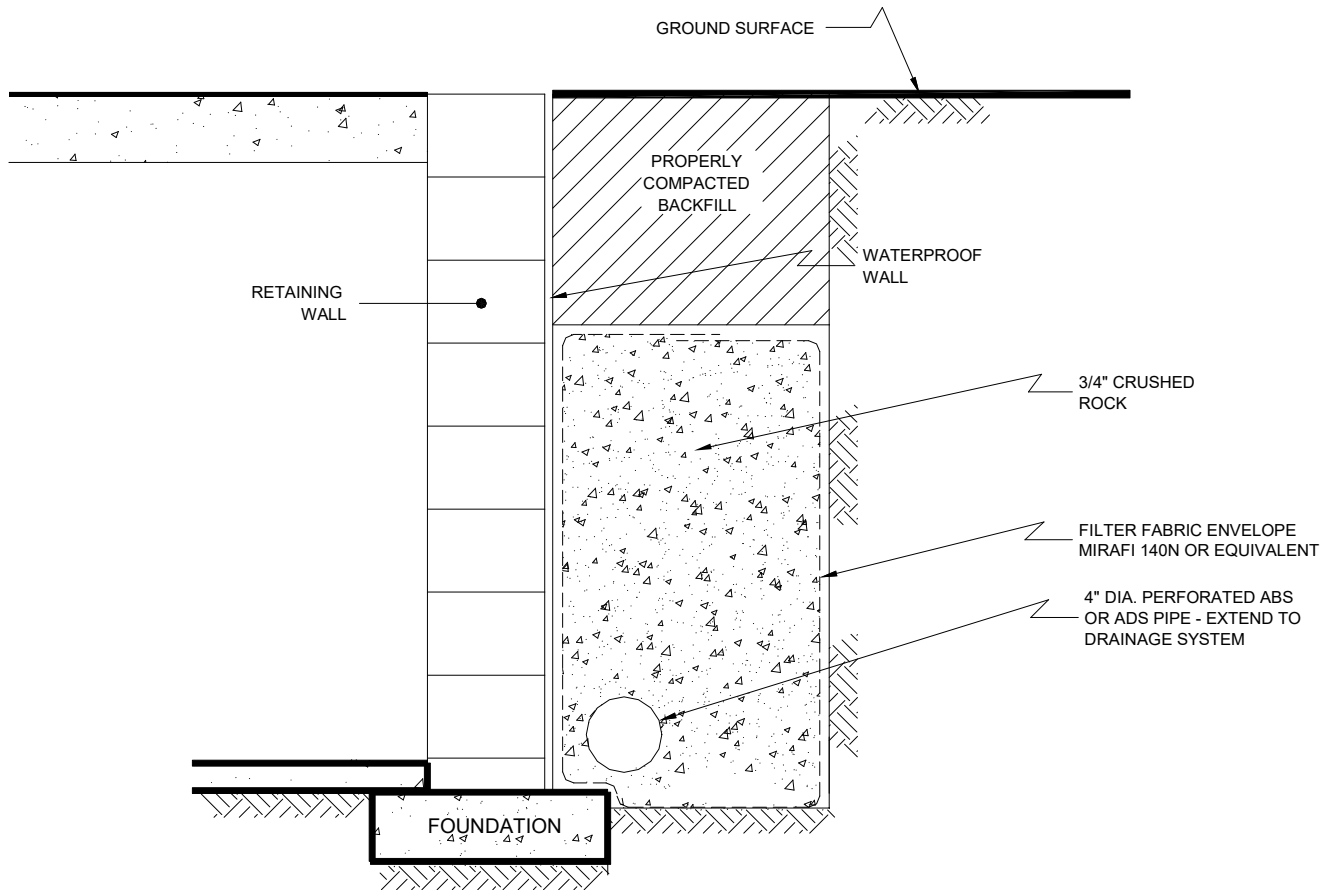
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PROJECT NO. W1135-06-01

FIG. 5



NO SCALE

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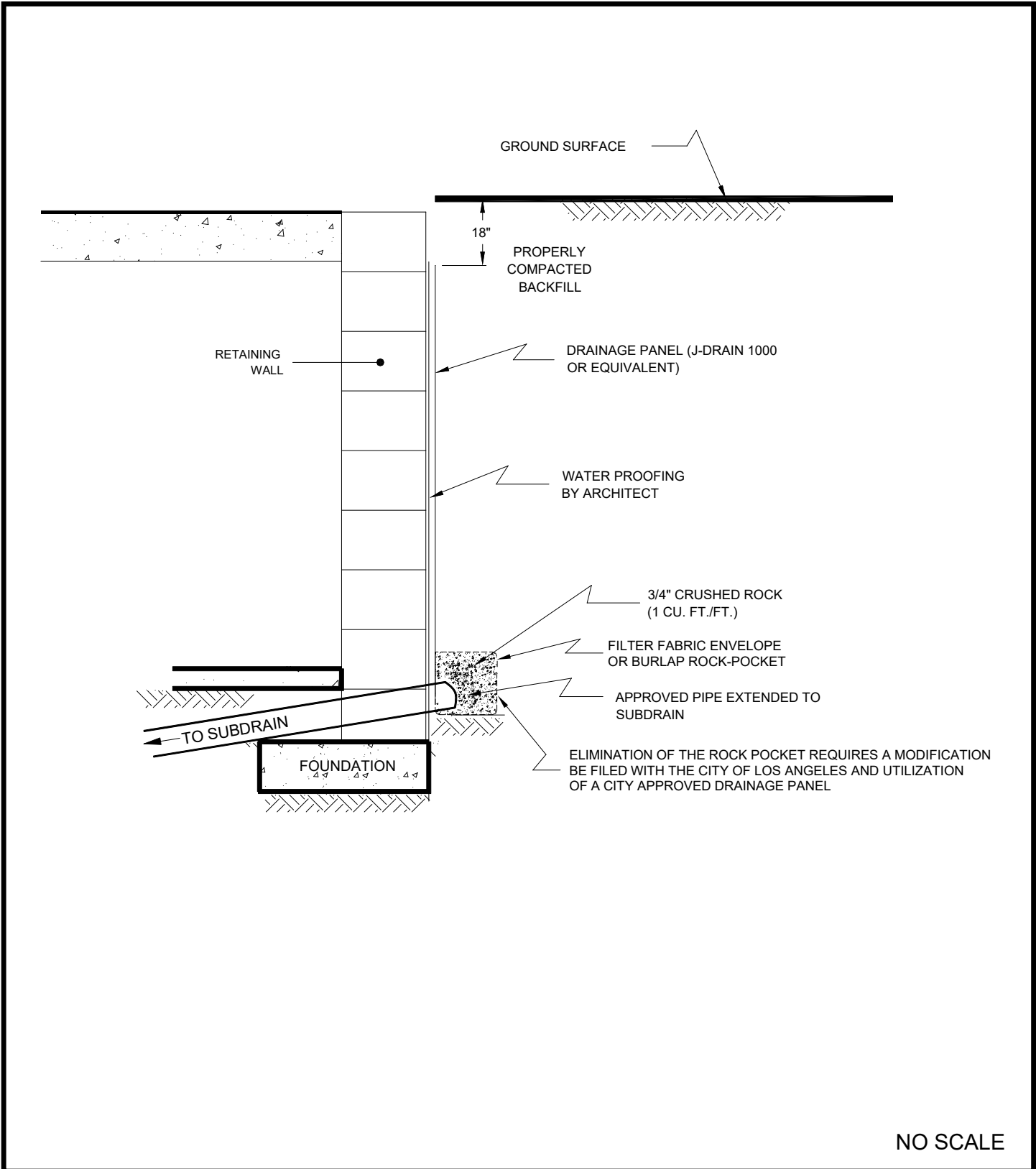
RETAINING WALL DRAIN DETAIL

1235 NORTH VINE STREET
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FIG. 6



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RETAINING WALL DRAIN DETAIL

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JULY 2020 PROJECT NO. W1135-06-01 FIG. 7

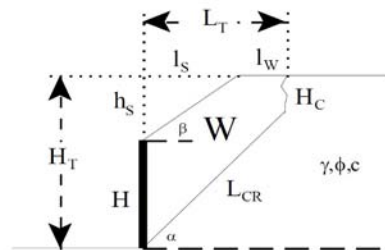
Shoring Design with Transitioned Backfill (Vector Analysis)

Input:

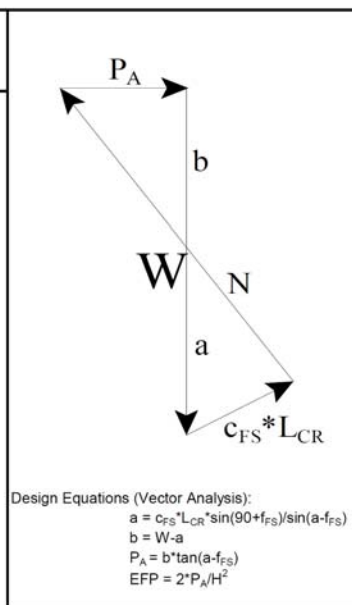
Shoring Height (H) 45.00 feet
 Slope Angle of Backfill (b) 0.0 degrees
 Height of Slope above Shoring (h_s) 0.0 feet
 Horizontal Length of Slope (l_s) 0.0 feet
 Total Height (Shoring + Slope) (H_T) 45.0 feet

Unit Weight of Retained Soils (g) 125.0 pcf
 Friction Angle of Retained Soils (f) 26.0 degrees
 Cohesion of Retained Soils (c) 240.0 psf
 Factor of Safety (F_S) 1.25

Factored Parameters: (f_{FS}) 21.3 degrees
 (c_{FS}) 192.0 psf



Failure Angle (a) degrees	Height of Tension Crack (H _c) feet	Area of Wedge (A) feet ²	Weight of Wedge (W) lbs/lineal foot	Length of Failure Plane (L _{CR}) feet	a lbs/lineal foot	b lbs/lineal foot	Active Pressure (P _A) lbs/lineal foot
45	5.0	1000	124976.4	56.5	25164.3	99812.1	43783.2
46	4.9	966	120751.6	55.7	23856.0	96895.6	44536.2
47	4.8	933	116655.7	54.9	22660.7	93994.9	45206.3
48	4.8	901	112681.3	54.1	21565.8	91115.6	45796.4
49	4.7	871	108821.8	53.4	20560.1	88261.7	46308.9
50	4.6	841	105070.5	52.7	19634.2	85436.3	46745.9
51	4.6	811	101421.4	52.0	18779.9	82641.5	47109.1
52	4.6	783	97868.6	51.3	17989.8	79878.7	47400.2
53	4.5	755	94406.6	50.7	17257.7	77148.9	47620.1
54	4.5	728	91030.2	50.1	16577.9	74452.3	47769.9
55	4.5	702	87734.6	49.4	15945.6	71789.0	47850.1
56	4.5	676	84515.1	48.9	15356.2	69158.9	47861.0
57	4.5	651	81367.3	48.3	14806.0	66561.3	47802.7
58	4.5	626	78287.2	47.7	14291.4	63995.8	47674.9
59	4.5	602	75270.8	47.2	13809.2	61461.5	47477.1
60	4.6	579	72314.4	46.7	13356.8	58957.6	47208.6
61	4.6	555	69414.6	46.2	12931.5	56483.1	46868.2
62	4.7	533	66568.0	45.7	12530.9	54037.1	46454.6
63	4.7	510	63771.5	45.2	12153.0	51618.5	45966.1
64	4.8	488	61022.0	44.7	11795.8	49226.2	45400.7
65	4.9	467	58316.7	44.2	11457.5	46859.2	44756.1
66	5.0	445	55652.8	43.8	11136.4	44516.4	44029.6
67	5.1	424	53027.6	43.3	10830.8	42196.8	43218.0
68	5.2	404	50438.6	42.9	10539.3	39899.3	42317.8
69	5.4	383	47883.2	42.4	10260.4	37622.9	41325.2
70	5.6	363	45359.2	42.0	9992.5	35366.7	40235.7



Maximum Active Pressure Resultant

$$P_{A, \max}$$

47861.0 lbs/lineal foot

Equivalent Fluid Pressure (per lineal foot of shoring)

$$EFP = 2 * P_A / H^2$$

EFP

47.3 pcf

Design Shoring for an Equivalent Fluid Pressure:

47 pcf

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SHORING PRESSURE CALCULATION

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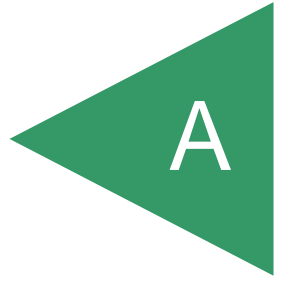
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FIG. 8

APPENDIX

A



APPENDIX A

FIELD INVESTIGATION







The site was explored on April 1, 2020 by excavating two 8-inch diameter hollow-stem auger borings using a truck-mounted hollow-stem auger drilling machine. The borings were advanced to depths of approximately 80½ feet below the existing ground surface. Representative and relatively undisturbed samples were obtained by driving a 3-inch O.D., California Modified Sampler into the “undisturbed” soil mass with blows from a 140-pound automatic hammer falling 30 inches. The California Modified Sampler was equipped with 1-inch high by 2³/₈-inch diameter brass sampler rings to facilitate removal and testing. Bulk samples were also obtained.

The soil conditions encountered in the explorations were visually examined, classified and logged in general accordance with the Unified Soil Classification System (USCS). The logs of the borings are presented on Figures A1 and A2. The logs depict the soil and geologic conditions encountered and the depth at which samples were obtained. The logs also include our interpretation of the conditions between sampling intervals. Therefore, the logs contain both observed and interpreted data. We determined the lines designating the interface between soil materials on the logs using visual observations, penetration rates, excavation characteristics and other factors. The transition between materials may be abrupt or gradual. Where applicable, the logs were revised based on subsequent laboratory testing. The location of the borings are shown on Figure 2A.

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	BORING 1		PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
					ELEV. (MSL.) --	DATE COMPLETED <u>4/1/2020</u>			
					EQUIPMENT <u>HOLLOW STEM AUGER</u> BY: <u>JS</u>				
MATERIAL DESCRIPTION									
0	BULK 0-5'				CONCRETE: 6" BASE: NONE ARTIFICIAL FILL Clay, firm, moist, dark olive brown, trace fine-grained sand.				
2									
4									
6	B1@5'				- stiff, dark brown, increase in sand, fine-grained, trace medium-grained		24		
8									
10	B1@10'			SP-SC	ALLUVIUM Sand with Clay, medium dense, moist to wet, yellowish brown, fine- to medium-grained, some fine gravel.		22		
12									
14	B1@15'			CL	Sandy Clay / Clayey Sand, firm to loose, moist, yellowish brown, fine- to medium-grained.		14		
16									
18					Sandy Clay, firm, moist, dark brown, fine- to medium-grained.				
20	B1@20'			CL			13	97.3	26.0
22									
24	B1@25'			SC	Clayey Sand, medium dense, moist to wet, grayish brown, fine- to medium-grained, trace coarse-grained, some oxidation.		31	117.5	12.8
26									
28				CL	Sandy Clay, stiff, moist, dark yellowish brown, fine- to medium-grained,				

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

W1135-06-01 BORING LOGS.GPJ

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

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

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	BORING 1			PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)		
					ELEV. (MSL.)	--	DATE COMPLETED					
					ELEV. (MSL.)	--	DATE COMPLETED					
					EQUIPMENT	HOLLOW STEM AUGER	BY:	JS				
					MATERIAL DESCRIPTION							
30	B1@30'				some oxidation.			27				
	BULK 30-35'											
32	B1@32.5'			CL				35				
34												
36	B1@35'				- moist to wet, grayish brown, clayey sand interbeds			26	111.6	17.3		
38	B1@37.5'			SC	Clayey Sand, medium dense, moist to wet, dark yellowish brown, fine- to medium-grained.			22	110.4	18.7		
40	B1@40'					- decrease in clay content		20	104.7	24.1		
42	B1@42.5'				Sand, well-graded, dense, moist to wet, yellowish brown, fine- to coarse-grained.			84				
44	B1@45'			SW	- medium dense, trace clay			46				
46	B1@47.5'					- dense		55				
50	B1@50'			CL	Sandy Clay, stiff, moist, dark yellowish brown, fine-grained, trace medium-grained.			32	119.0	16.7		
52					Clayey Sand, medium dense, moist, yellowish brown, fine- to medium-grained.							
54												
56	B1@55'			SC				35				
58												

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

W1135-06-01 BORING LOGS.GPJ

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

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

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	BORING 1			PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
					ELEV. (MSL.) --	DATE COMPLETED				
					ELEV. (MSL.)	--	DATE COMPLETED	4/1/2020		
					EQUIPMENT	HOLLOW STEM AUGER		BY:	JS	
MATERIAL DESCRIPTION										
60	B1@60'			SC	- dense			65	121.8	15.8
62										
64										
66	B1@65'			SM	Silty Sand, dense, moist, yellowish brown, fine- to medium-grained.			47	115.6	17.3
68										
70	B1@70'			SW	Sand, well-graded, dense, moist to wet, yellowish brown.			60	114.6	21.4
72										
74										
76	B1@75'			SC	Clayey Sand, very dense, moist, yellowish brown, well-graded.			50(4")		
78										
80	B1@80'				- sandy clay interbeds, medium dense			47		
					Total depth of boring: 80.5 feet Fill to 8.5 feet. No groundwater encountered. Backfilled with grout. Surface restored. *Penetration resistance for 140-pound hammer falling 30 inches by auto-hammer.					

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

W1135-06-01 BORING LOGS.GPJ







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

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

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	BORING 2		PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
					ELEV. (MSL.) --	DATE COMPLETED <u>4/1/2020</u>			
					EQUIPMENT <u>HOLLOW STEM AUGER</u> BY: <u>JS</u>				
MATERIAL DESCRIPTION									
0	BULK 0-5'				CONCRETE: 3.5" BASE: NONE ARTIFICIAL FILL Sandy Clay, firm, moist, dark brown, fine-grained.				
2									
4									
6	B2@5'						21	111.7	18.3
8					ALLUVIUM Clayey Sand, medium dense, moist, dark yellowish brown, fine-grained, trace medium-grained.				
10	B2@10'						24		
12				SC					
14									
16	B2@15'						25		
18									
20	B2@20'				Sandy Clay, stiff, moist, dark brown, some oxidation, fine-grained.		24		
22									
24				CL					
26	B2@25'				- moist to wet		23	115.5	13.7
28									

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

W1135-06-01 BORING LOGS.GPJ







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

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

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	BORING 2		PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
					ELEV. (MSL.) --	DATE COMPLETED <u>4/1/2020</u>			
					EQUIPMENT <u>HOLLOW STEM AUGER</u> BY: <u>JS</u>				
MATERIAL DESCRIPTION									
30	B2@30'				- firm, moist, dark brown with grayish brown mottles		15	113.1	16.6
32									
34	B2@33'				- stiff		33	120.7	16.2
36	B2@35'			CL	- hard		39	113.6	16.1
38	B2@37'				- stiff, dark yellowish brown		28	114.6	17.1
40	B2@40'				- increase in sand		34		
42	B2@42'			SP	Sand, dense, moist to wet, dark yellowish brown, medium- to coarse-grained.		80	119.9	16.8
44									
46	B2@45'				Sandy Clay, stiff, moist, dark yellowish brown, fine- to medium-grained, trace coarse-grained.		26	105.9	21.9
48									
50	B2@50'			CL	- hard		41		
52									
54									
56	B2@55'				- stiff, fine-grained, trace medium-grained		24	119.9	17.1
58				SC	Clayey Sand, dense, moist, yellowish brown, fine- to medium-grained.				

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

W1135-06-01 BORING LOGS.GPJ

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

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








DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	BORING 2			PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
					ELEV. (MSL.) --	DATE COMPLETED				
					ELEV. (MSL.)	--	DATE COMPLETED	4/1/2020		
					EQUIPMENT	HOLLOW STEM AUGER		BY:	JS	
MATERIAL DESCRIPTION										
60	B2@60'			SC	- some grayish brown mottles		70	117.9	14.6	
62										
64										
66	B2@65'					64				
68										
70	B2@70'					65				
72										
74					- decrease in clay content		89			
76	B2@75'									
78										
80	B2@80'				- increase in clay, medium dense		27			
					Total depth of boring: 80.5 feet Fill to 7.5 feet. No groundwater encountered. Backfilled with grout. Surface restored.					
					*Penetration resistance for 140-pound hammer falling 30 inches by auto-hammer.					

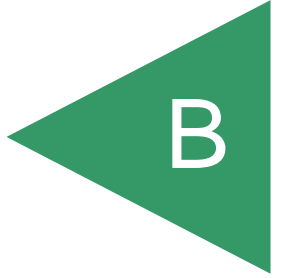
Figure A2,
Log of Boring 2, Page 3 of 3

W1135-06-01 BORING LOGS.GPJ

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

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

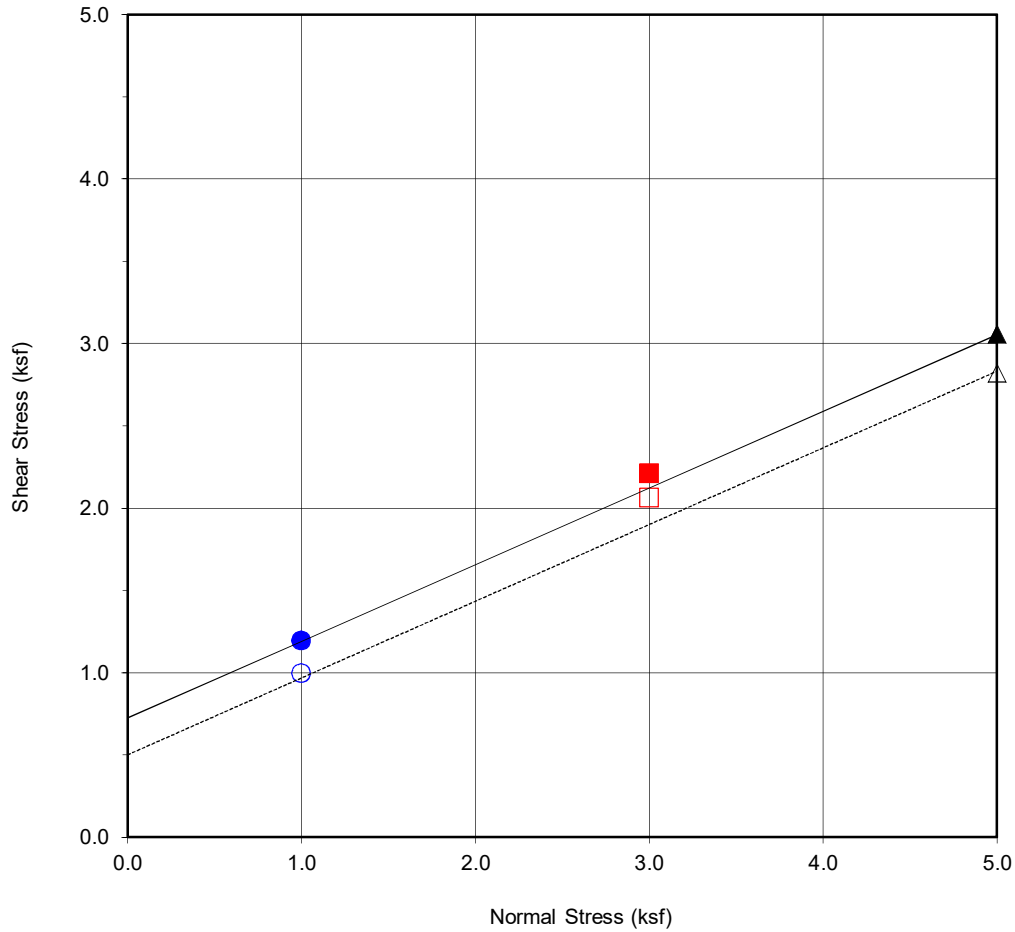
APPENDIX



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, corrosivity, in-place dry density and moisture content. The results of the laboratory tests are summarized in Figures B1 through B26. The in-place dry density and moisture content of the samples tested are presented in the boring logs, Appendix A.



Boring No.	B2
Sample No.	B2@5'
Depth (ft)	5
<u>Sample Type:</u>	Ring

<u>Soil Identification:</u>		
Dark Brown Sandy Clay (CL)		
Strength Parameters		
	C (psf)	ϕ ($^{\circ}$)
Peak	725	25.0
Ultimate	500	25.0

Normal Stress (kip/ft ²)	1	3	5
Peak Shear Stress (kip/ft ²)	● 1.19	■ 2.21	▲ 3.06
Shear Stress @ End of Test (ksf)	○ 0.99	□ 2.06	△ 2.82
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 (%)	18.3	18.3	18.2
Initial Dry Density (pcf)	109.9	110.8	110.1
Initial Degree of Saturation (%)	92.4	94.9	92.2
Soil Height Before Shearing (in.)	1.2	1.2	1.2
Final Moisture Content (%)	20.9	19.5	19.2



DIRECT SHEAR TEST RESULTS

Consolidated Drained ASTM D-3080

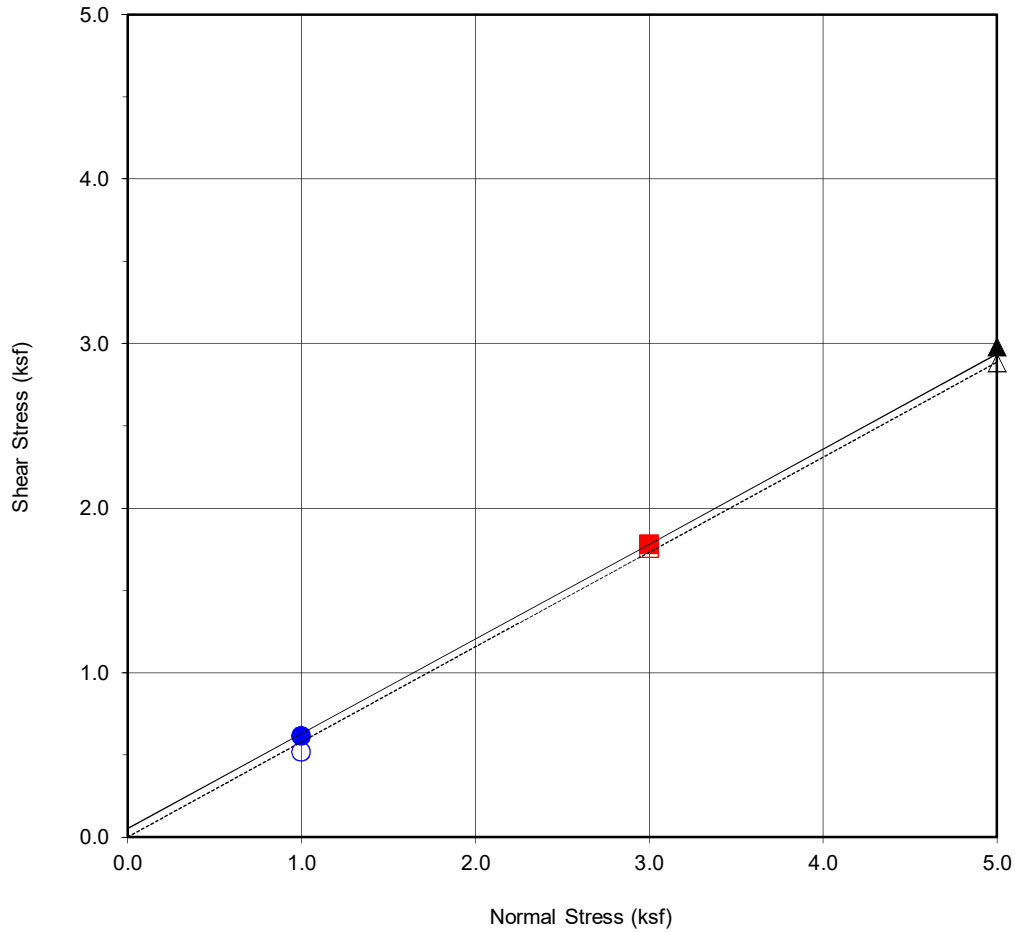
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Figure B1



Boring No.	B1
Sample No.	B1@10
Depth (ft)	10
<u>Sample Type:</u>	Ring

<u>Soil Identification:</u>		
Yellowish Brown Sand with Clay (SP-SC)		
Strength Parameters		
	C (psf)	ϕ (°)
Peak	50	30.0
Ultimate	0	30.0

Normal Stress (kip/ft ²)	1	3	5
Peak Shear Stress (kip/ft ²)	● 0.61	■ 1.78	▲ 2.98
Shear Stress @ End of Test (ksf)	○ 0.52	□ 1.75	△ 2.88
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 (%)	30.9	24.8	24.2
Initial Dry Density (pcf)	96.1	95.8	96.3
Initial Degree of Saturation (%)	110.8	88.3	87.1
Soil Height Before Shearing (in.)	1.2	1.2	1.2
Final Moisture Content (%)	29.6	20.0	17.1



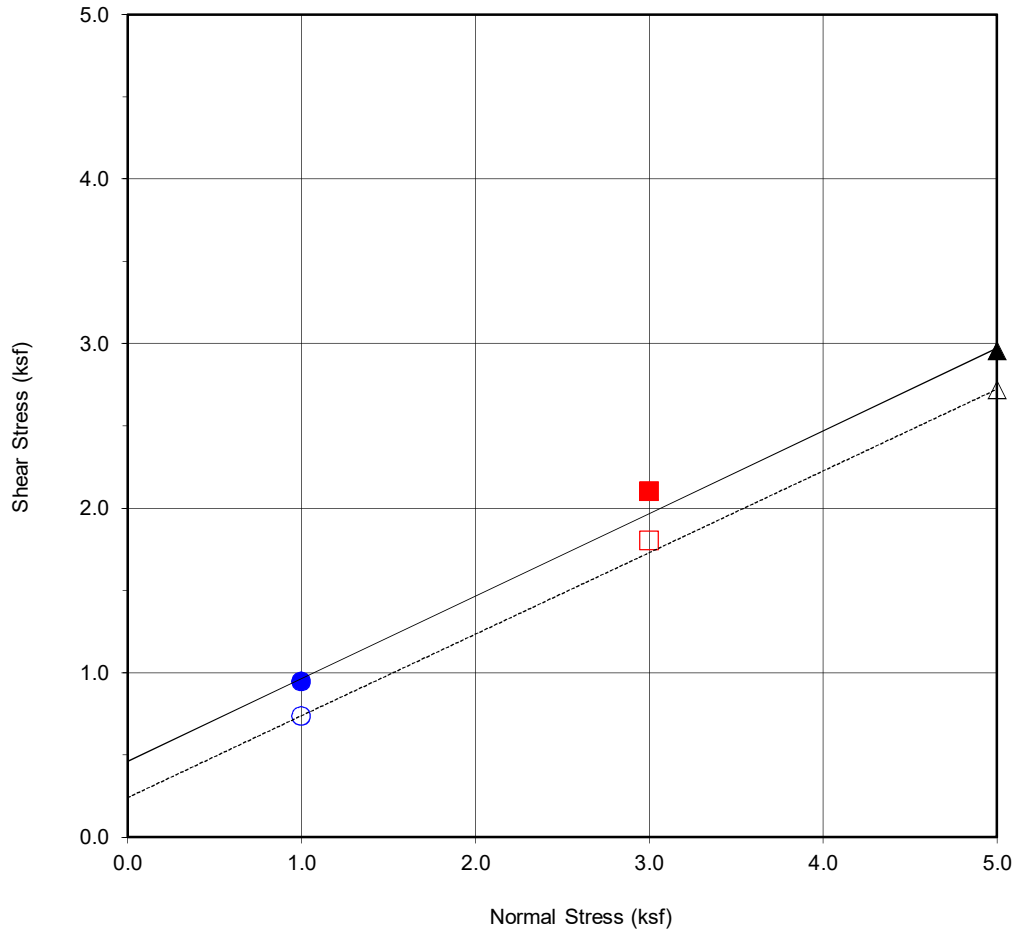
DIRECT SHEAR TEST RESULTS
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July 2020 Figure B2



Boring No.	B1
Sample No.	B1@20'
Depth (ft)	20
<u>Sample Type:</u>	Ring

<u>Soil Identification:</u>		
Dark Brown Sandy Clay (CL)		
Strength Parameters		
	C (psf)	ϕ ($^{\circ}$)
Peak	460	26.7
Ultimate	240	26.4

Normal Stress (kip/ft ²)	1	3	5
Peak Shear Stress (kip/ft ²)	● 0.94	■ 2.10	▲ 2.96
Shear Stress @ End of Test (ksf)	○ 0.73	□ 1.80	△ 2.72
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 (%)	31.9	27.9	26.0
Initial Dry Density (pcf)	89.7	94.9	97.1
Initial Degree of Saturation (%)	97.9	97.2	95.3
Soil Height Before Shearing (in.)	1.2	1.2	1.2
Final Moisture Content (%)	34.5	29.0	26.5



DIRECT SHEAR TEST RESULTS
Consolidated Drained ASTM D-3080

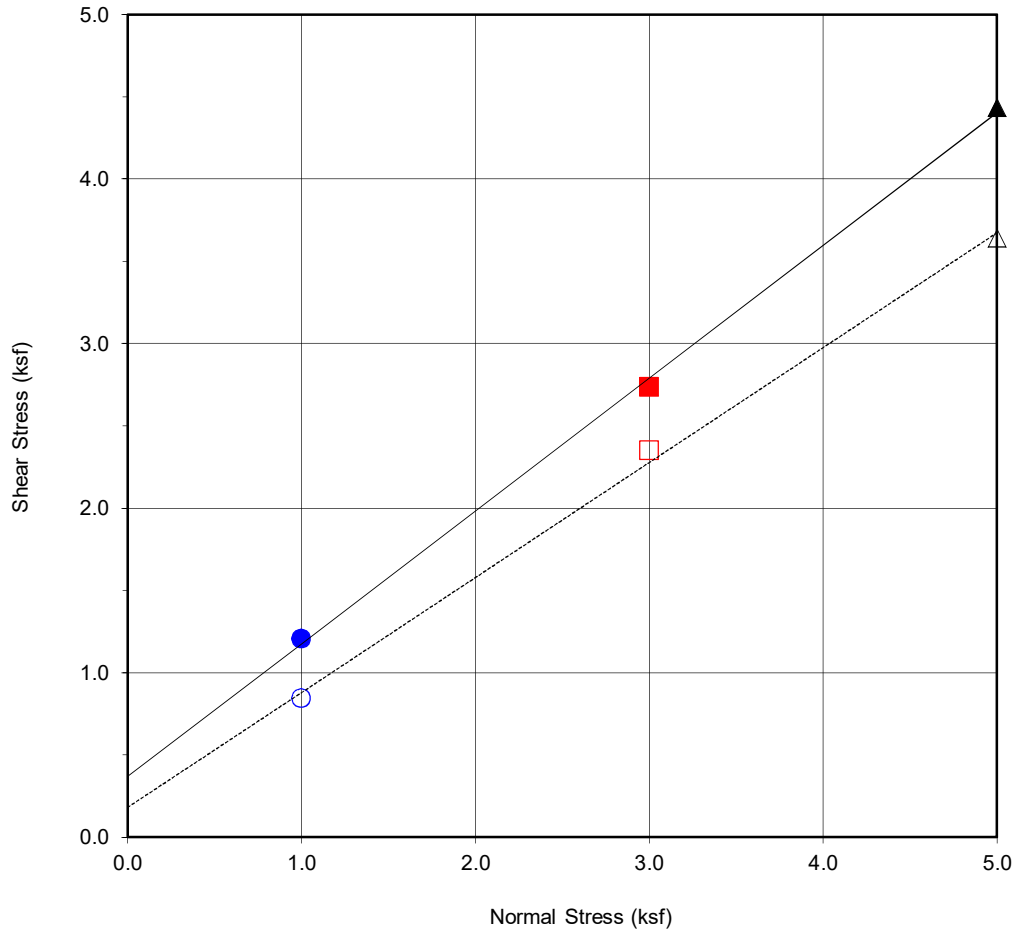
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Figure B3



Boring No.	B1
Sample No.	B1@25'
Depth (ft)	25
<u>Sample Type:</u>	Ring

<u>Soil Identification:</u>		
Grayish Brown Clayey Sand (SC)		
Strength Parameters		
	C (psf)	ϕ ($^{\circ}$)
Peak	368	38.9
Ultimate	181	34.9

Normal Stress (kip/ft ²)	1	3	5
Peak Shear Stress (kip/ft ²)	● 1.20	■ 2.73	▲ 4.43
Shear Stress @ End of Test (ksf)	○ 0.84	□ 2.35	△ 3.64
Deformation Rate (in./min.)	0.05	0.05	0.05
Initial Sample Height (in.)	1.0	1.0	1.0
Ring Inside Diameter (in.)	2.375	2.375	2.375
Initial Moisture Content (%)	13.7	13.5	12.8
Initial Dry Density (pcf)	118.1	116.4	119.3
Initial Degree of Saturation (%)	86.8	81.5	83.5
Soil Height Before Shearing (in.)	1.2	1.2	1.2
Final Moisture Content (%)	14.6	14.2	13.4



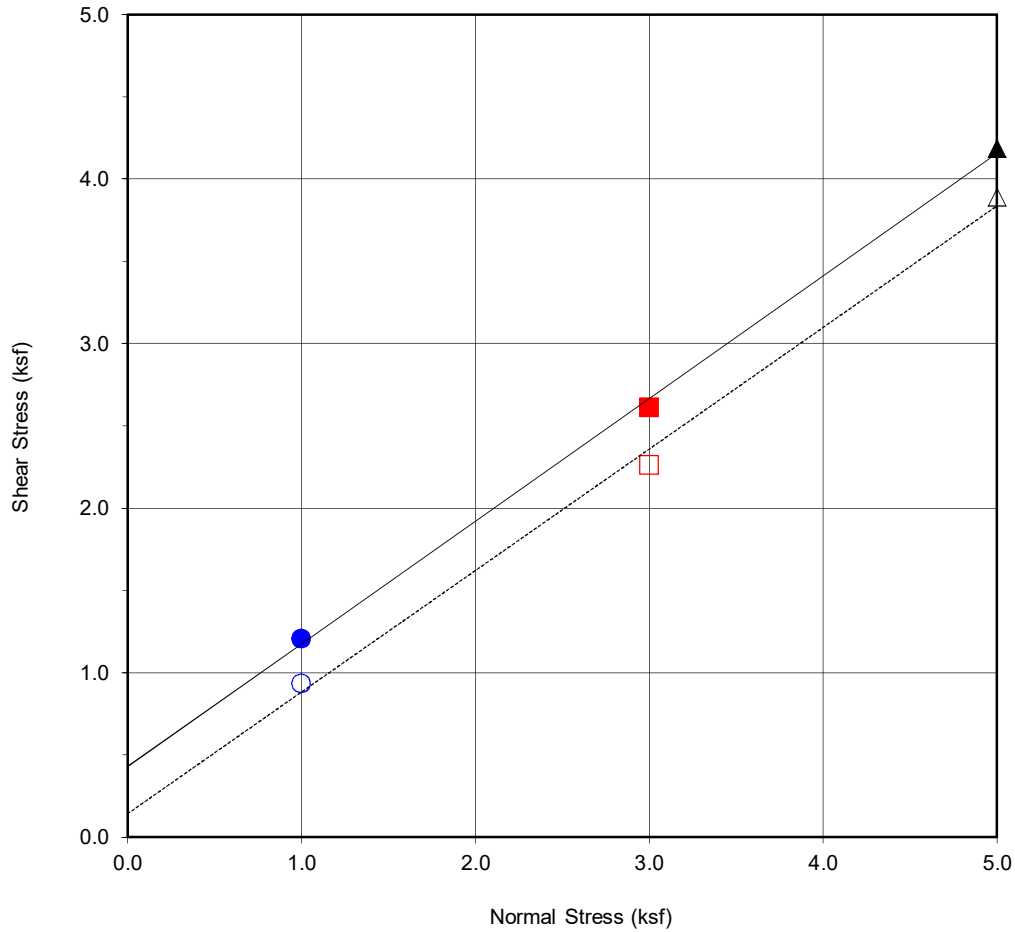
DIRECT SHEAR TEST RESULTS
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July 2020 Figure B4



Boring No.	B2
Sample No.	B2@25'
Depth (ft)	25
<u>Sample Type:</u>	Ring

<u>Soil Identification:</u>		
Dark Brown Sandy Clay (CL)		
Strength Parameters		
	C (psf)	ϕ ($^{\circ}$)
Peak	430	36.7
Ultimate	142	36.5

Normal Stress (kip/ft ²)	1	3	5
Peak Shear Stress (kip/ft ²)	● 1.20	■ 2.61	▲ 4.19
Shear Stress @ End of Test (ksf)	○ 0.93	□ 2.26	△ 3.89
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	12.4	13.7
Initial Dry Density (pcf)	113.5	117.4	115.6
Initial Degree of Saturation (%)	71.6	76.7	80.5
Soil Height Before Shearing (in.)	1.2	1.2	1.2
Final Moisture Content (%)	15.2	13.9	14.1



DIRECT SHEAR TEST RESULTS
Consolidated Drained ASTM D-3080

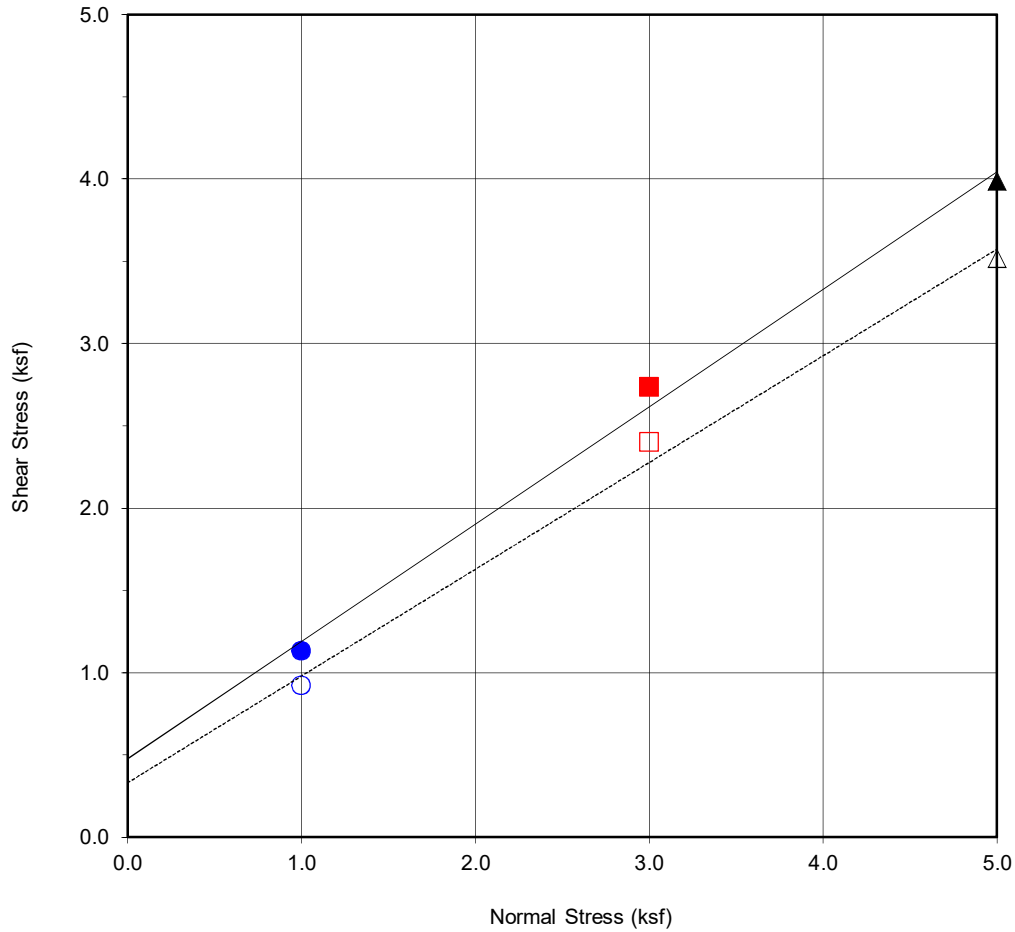
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Figure B5



Boring No.	B2
Sample No.	B2@30'
Depth (ft)	30
<u>Sample Type:</u>	Ring

<u>Soil Identification:</u>		
Dark Brown Sandy Clay (CL)		
Strength Parameters		
	C (psf)	ϕ ($^{\circ}$)
Peak	474	35.5
Ultimate	330	33.0

Normal Stress (kip/ft ²)	1	3	5
Peak Shear Stress (kip/ft ²)	● 1.13	■ 2.73	▲ 3.99
Shear Stress @ End of Test (ksf)	○ 0.92	□ 2.40	△ 3.51
Deformation Rate (in./min.)	0.05	0.05	0.05
Initial Sample Height (in.)	1.0	1.0	1.0
Ring Inside Diameter (in.)	2.375	2.375	2.375
Initial Moisture Content (%)	17.4	16.6	16.7
Initial Dry Density (pcf)	111.7	113.2	113.3
Initial Degree of Saturation (%)	92.1	91.7	92.3
Soil Height Before Shearing (in.)	1.2	1.2	1.2
Final Moisture Content (%)	17.1	15.7	15.3



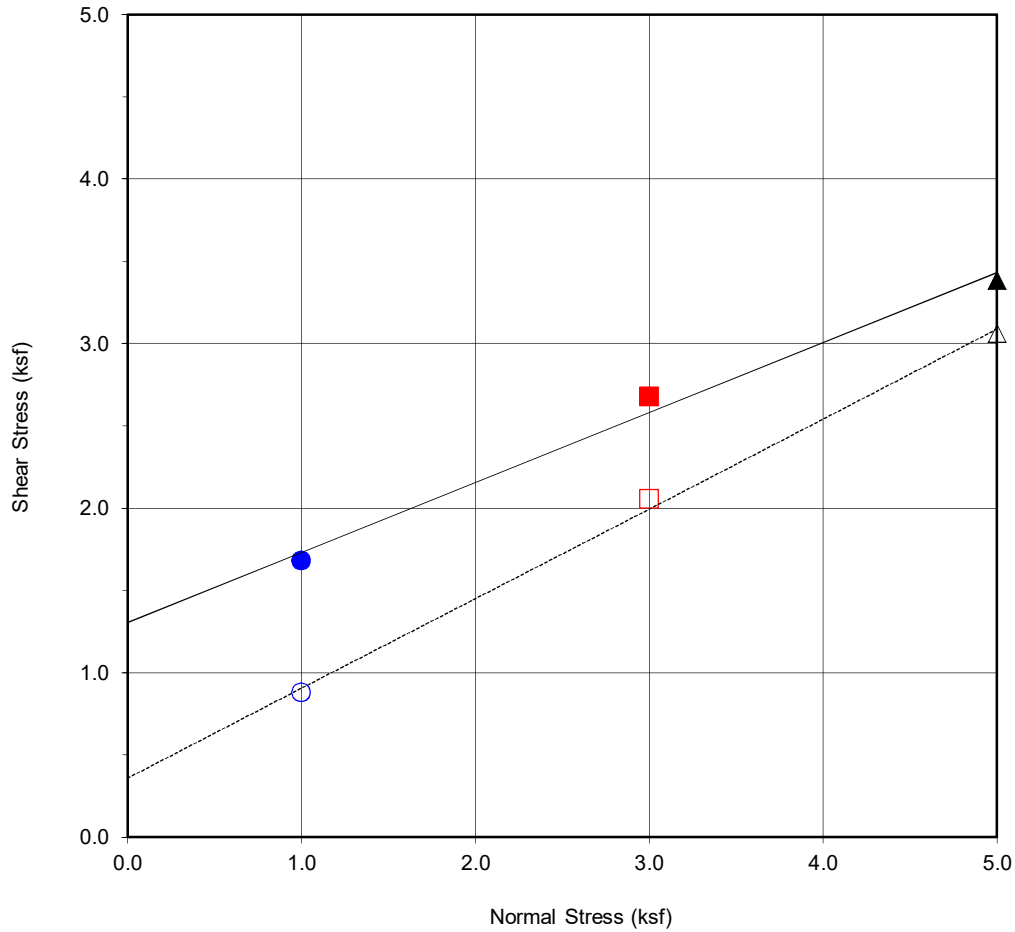
DIRECT SHEAR TEST RESULTS
Consolidated Drained ASTM D-3080

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July 2020 Figure B6



Boring No.	B1
Sample No.	B1@32.5
Depth (ft)	32.5
<u>Sample Type:</u>	Ring

<u>Soil Identification:</u>		
Dark Yellowish Brown Sandy Clay (CL)		
Strength Parameters		
	C (psf)	ϕ ($^{\circ}$)
Peak	1302	23.1
Ultimate	358	28.6

Normal Stress (kip/ft ²)	1	3	5
Peak Shear Stress (kip/ft ²)	● 1.68	■ 2.68	▲ 3.38
Shear Stress @ End of Test (ksf)	○ 0.88	□ 2.05	△ 3.06
Deformation Rate (in./min.)	0.05	0.05	0.05
Initial Sample Height (in.)	1.0	1.0	1.0
Ring Inside Diameter (in.)	2.375	2.375	2.375
Initial Moisture Content (%)	16.8	16.2	-0.5
Initial Dry Density (pcf)	114.5	116.6	136.7
Initial Degree of Saturation (%)	96.1	98.0	-5.8
Soil Height Before Shearing (in.)	1.2	1.2	1.2
Final Moisture Content (%)	18.9	17.2	35.5



DIRECT SHEAR TEST RESULTS
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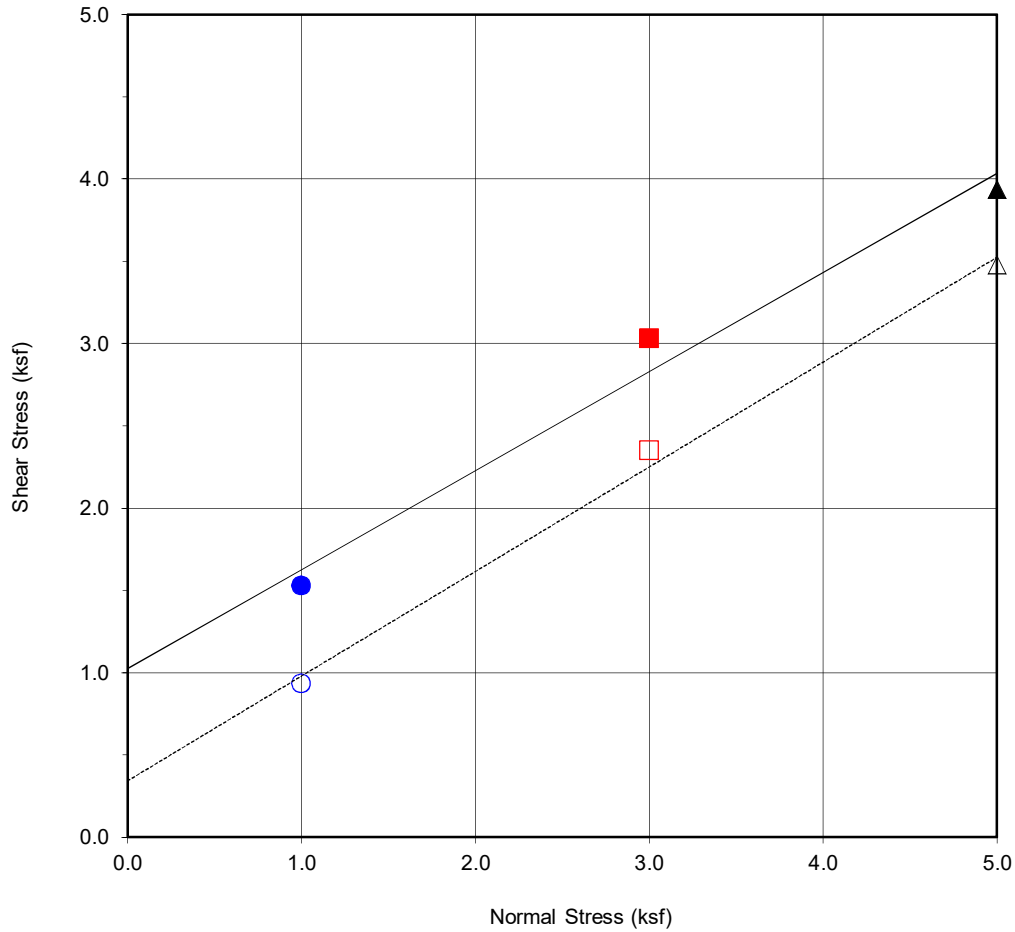
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Figure B7



Boring No.	B2
Sample No.	B2@35'
Depth (ft)	35
<u>Sample Type:</u>	Ring

<u>Soil Identification:</u>		
Dark Brown Sandy Clay (CL)		
Strength Parameters		
	C (psf)	ϕ ($^{\circ}$)
Peak	1024	31.1
Ultimate	342	32.5

Normal Stress (kip/ft ²)	1	3	5
Peak Shear Stress (kip/ft ²)	● 1.53	■ 3.03	▲ 3.94
Shear Stress @ End of Test (ksf)	○ 0.93	□ 2.35	△ 3.48
Deformation Rate (in./min.)	0.05	0.05	0.05
Initial Sample Height (in.)	1.0	1.0	1.0
Ring Inside Diameter (in.)	2.375	2.375	2.375
Initial Moisture Content (%)	17.1	16.1	16.5
Initial Dry Density (pcf)	111.8	115.1	111.9
Initial Degree of Saturation (%)	91.1	93.8	87.9
Soil Height Before Shearing (in.)	1.2	1.2	1.2
Final Moisture Content (%)	19.1	17.3	17.4



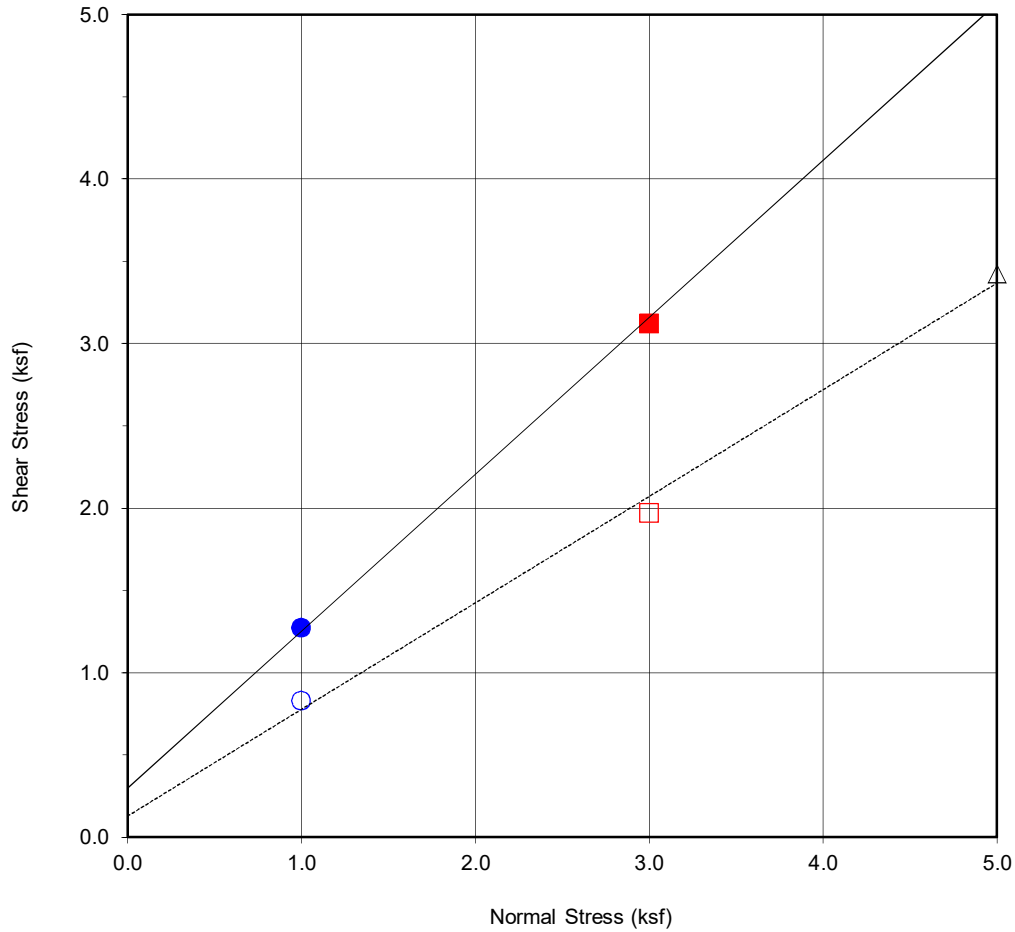
DIRECT SHEAR TEST RESULTS
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July 2020 Figure B8



Boring No.	B1
Sample No.	B1@42.5
Depth (ft)	42.5
<u>Sample Type:</u>	Ring

<u>Soil Identification:</u>		
Yellowish Brown Well-Graded Sand (SW)		
Strength Parameters		
	C (psf)	ϕ ($^{\circ}$)
Peak	298	43.7
Ultimate	128	32.9

Normal Stress (kip/ft ²)	1	3	5
Peak Shear Stress (kip/ft ²)	● 1.27	■ 3.12	▲ 5.09
Shear Stress @ End of Test (ksf)	○ 0.83	□ 1.97	△ 3.42
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 (%)	18.9	20.7	21.0
Initial Dry Density (pcf)	116.6	111.2	111.6
Initial Degree of Saturation (%)	114.1	108.3	111.2
Soil Height Before Shearing (in.)	1.2	1.2	1.2
Final Moisture Content (%)	18.0	19.8	19.7



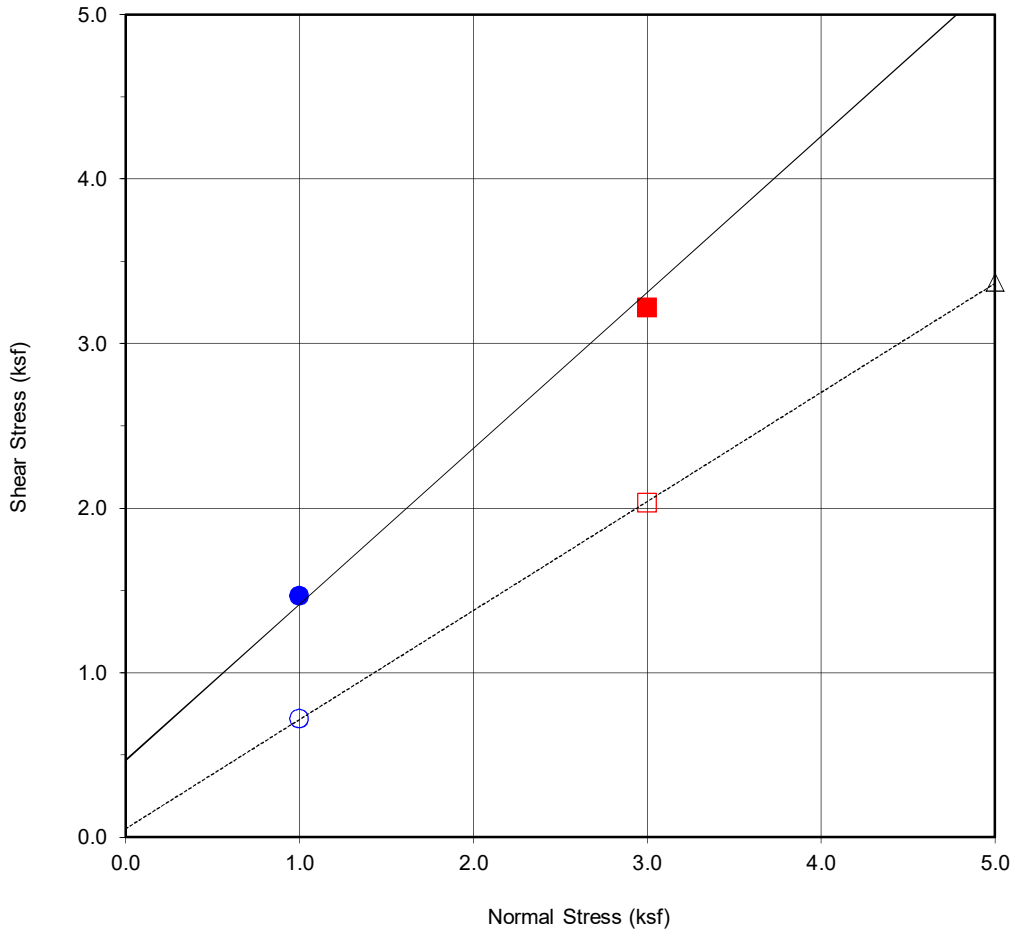
DIRECT SHEAR TEST RESULTS
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July 2020 Figure B9



Boring No.	B-1
Sample No.	B1@47.5
Depth (ft)	47.5'
<u>Sample Type:</u>	Ring

<u>Soil Identification:</u>		
Yellowish Brown Well-Graded Sand (SW)		
Strength Parameters		
	C (psf)	ϕ ($^{\circ}$)
Peak	468	43.5
Ultimate	51	33.5

Normal Stress (kip/ft ²)	1	3	5
Peak Shear Stress (kip/ft ²)	● 1.46	■ 3.22	▲ 5.26
Shear Stress @ End of Test (ksf)	○ 0.72	□ 2.03	△ 3.37
Deformation Rate (in./min.)	0.00	0.00	0.00
Initial Sample Height (in.)	1.0	1.0	1.0
Ring Inside Diameter (in.)	2.375	2.375	2.375
Initial Moisture Content (%)	15.2	15.6	15.4
Initial Dry Density (pcf)	121.4	116.6	121.0
Initial Degree of Saturation (%)	105.5	94.6	105.8
Soil Height Before Shearing (in.)	1.2	1.2	1.2
Final Moisture Content (%)	15.2	15.4	15.0



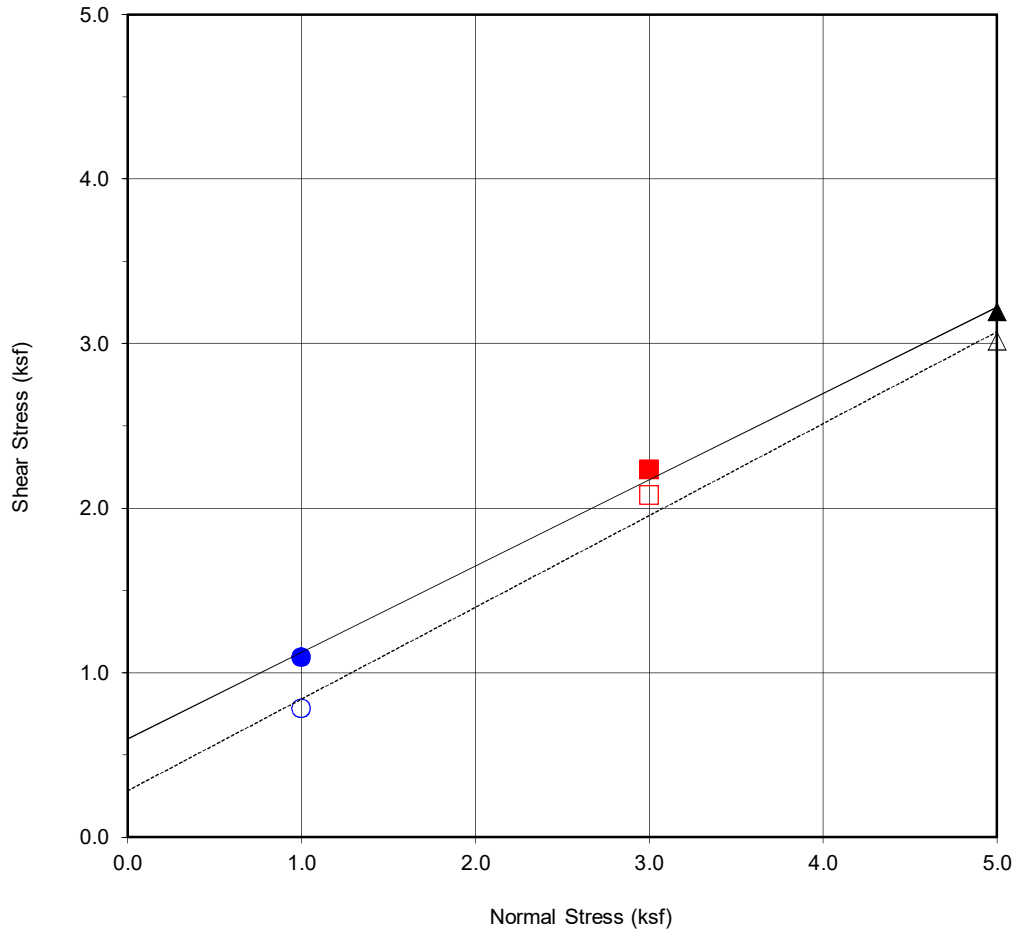
DIRECT SHEAR TEST RESULTS
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July 2020 Figure B10



Boring No.	B1
Sample No.	B1@55
Depth (ft)	55
<u>Sample Type:</u>	Ring

<u>Soil Identification:</u>		
Yellowish Brown Clayey Sand (SC)		
Strength Parameters		
	C (psf)	ϕ ($^{\circ}$)
Peak	597	27.7
Ultimate	282	29.2

Normal Stress (kip/ft ²)	1	3	5
Peak Shear Stress (kip/ft ²)	● 1.09	■ 2.23	▲ 3.19
Shear Stress @ End of Test (ksf)	○ 0.78	□ 2.08	△ 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 (%)	15.8	22.0	23.1
Initial Dry Density (pcf)	119.8	108.5	105.7
Initial Degree of Saturation (%)	104.9	107.3	104.9
Soil Height Before Shearing (in.)	1.2	1.2	1.2
Final Moisture Content (%)	16.0	19.9	20.4



DIRECT SHEAR TEST RESULTS
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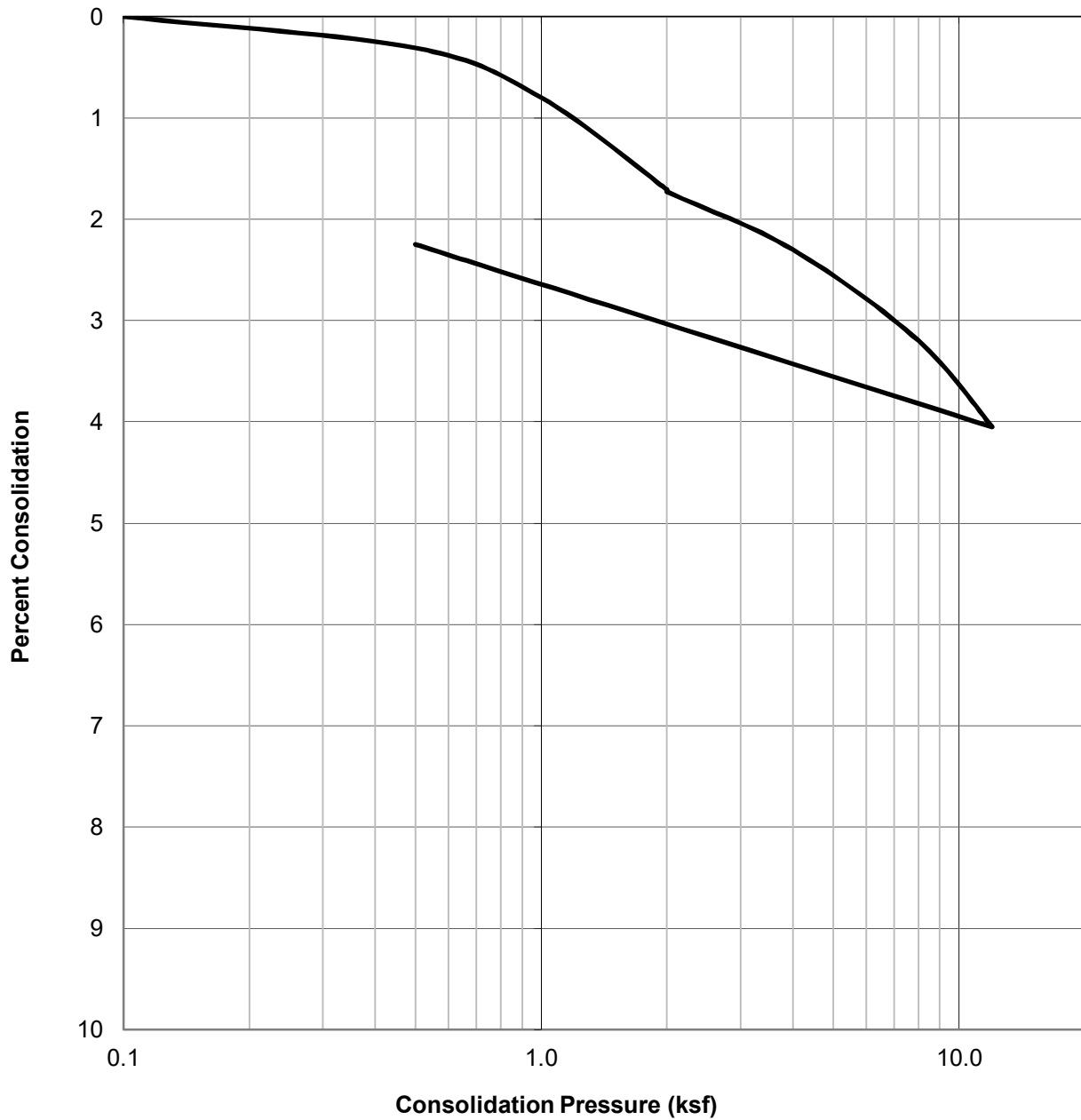
Project No.: W1135-06-01

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Los Angeles, California


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Figure B11

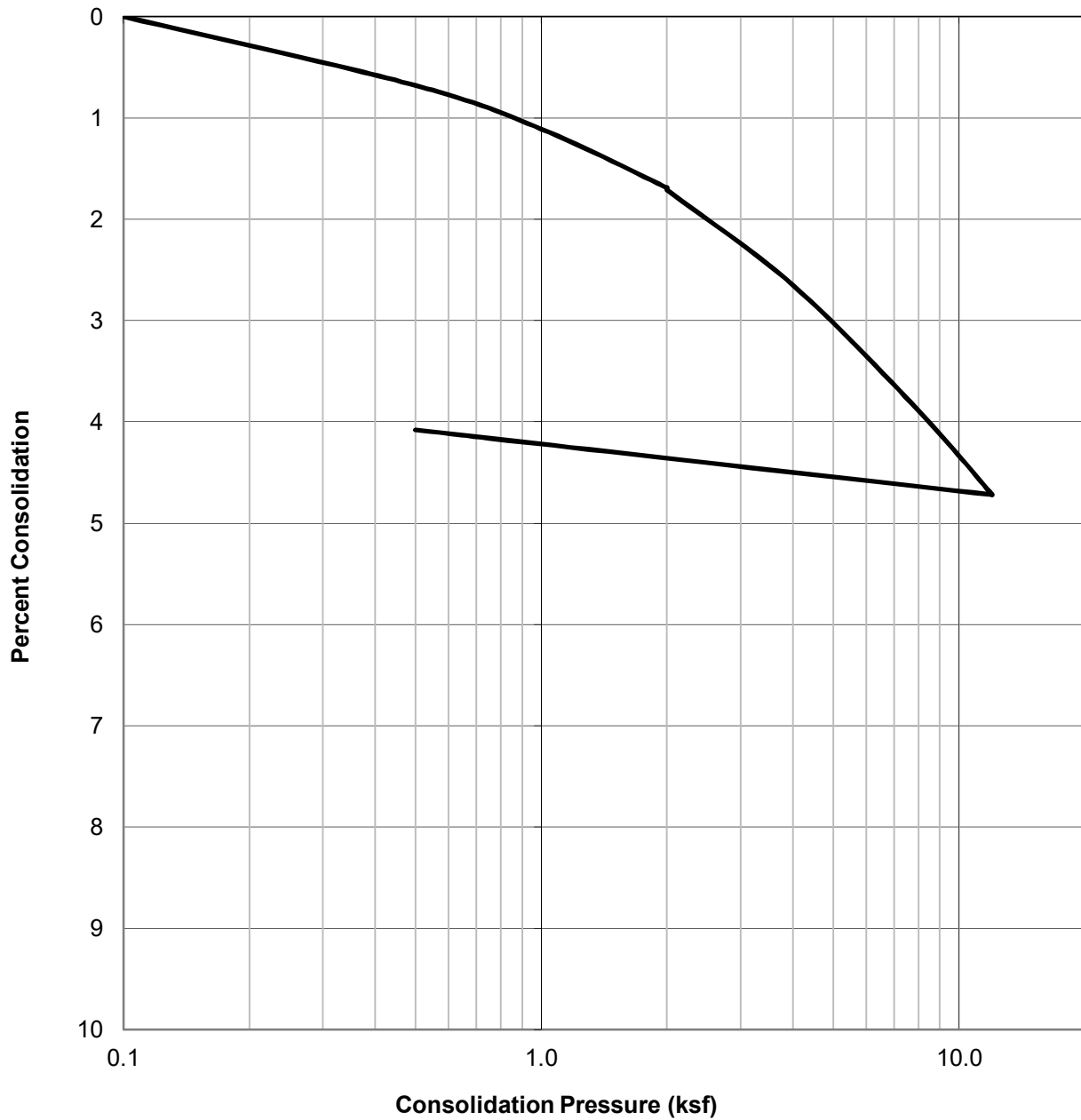
WATER ADDED AT 2.0 KSF




SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B2@33	Dark Brown Sandy Clay (CL)	112.0	16.2	17.4

 GEOCON	CONSOLIDATION TEST RESULTS		Project No.:	W1135-06-01
	ASTM D-2435		1235 Vine Street Los Angeles, California	
	Checked by:	PZ	July 2020	Figure B12

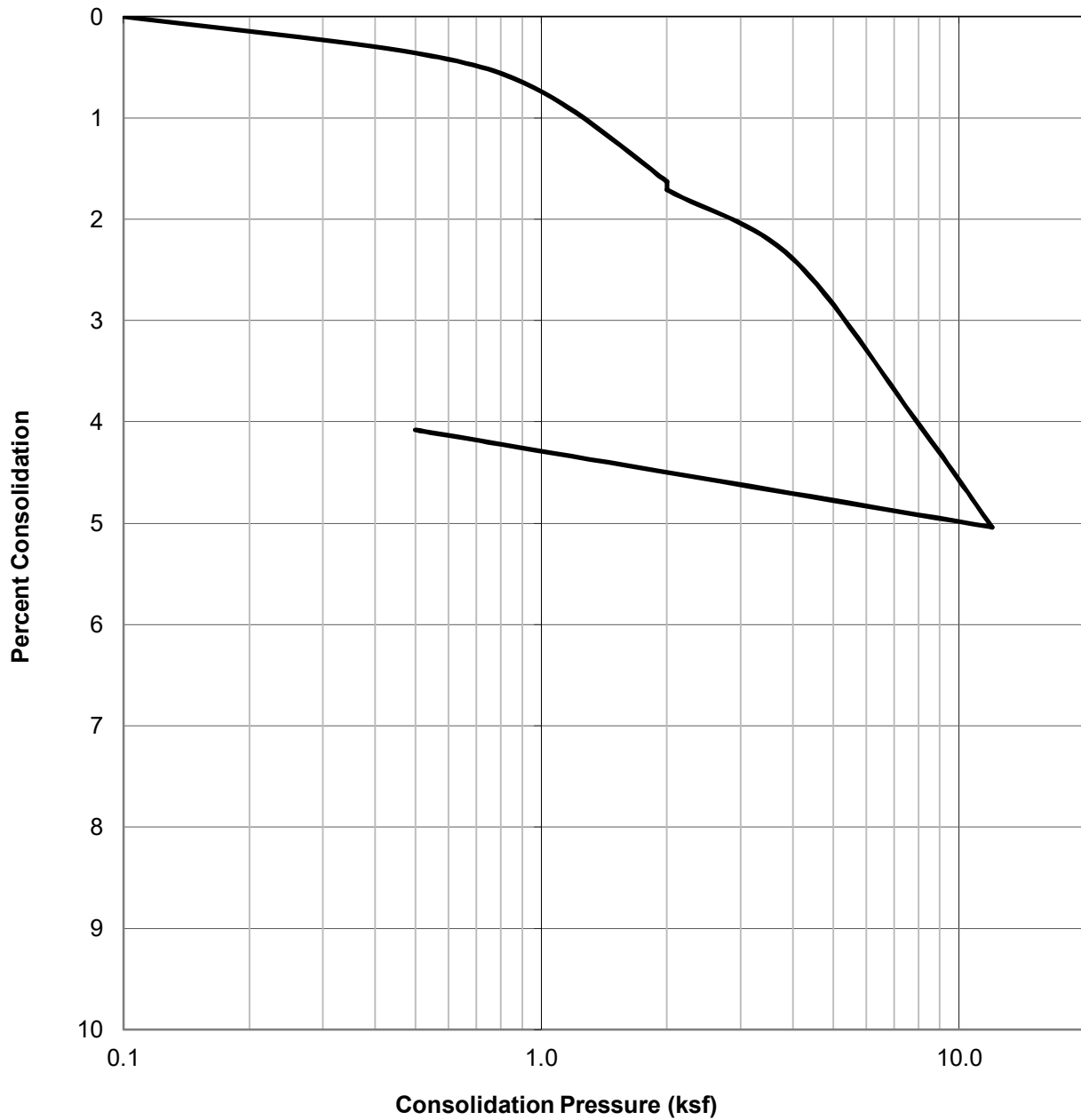
WATER ADDED AT 2.0 KSF



SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B1@35	Grayish Brown Sandy Clay (CL)	113.0	17.3	15.4

 GEOCON	CONSOLIDATION TEST RESULTS <small>ASTM D-2435</small>	Project No.: W1135-06-01
	Checked by: PZ	1235 Vine Street Los Angeles, California
	July 2020	Figure B13

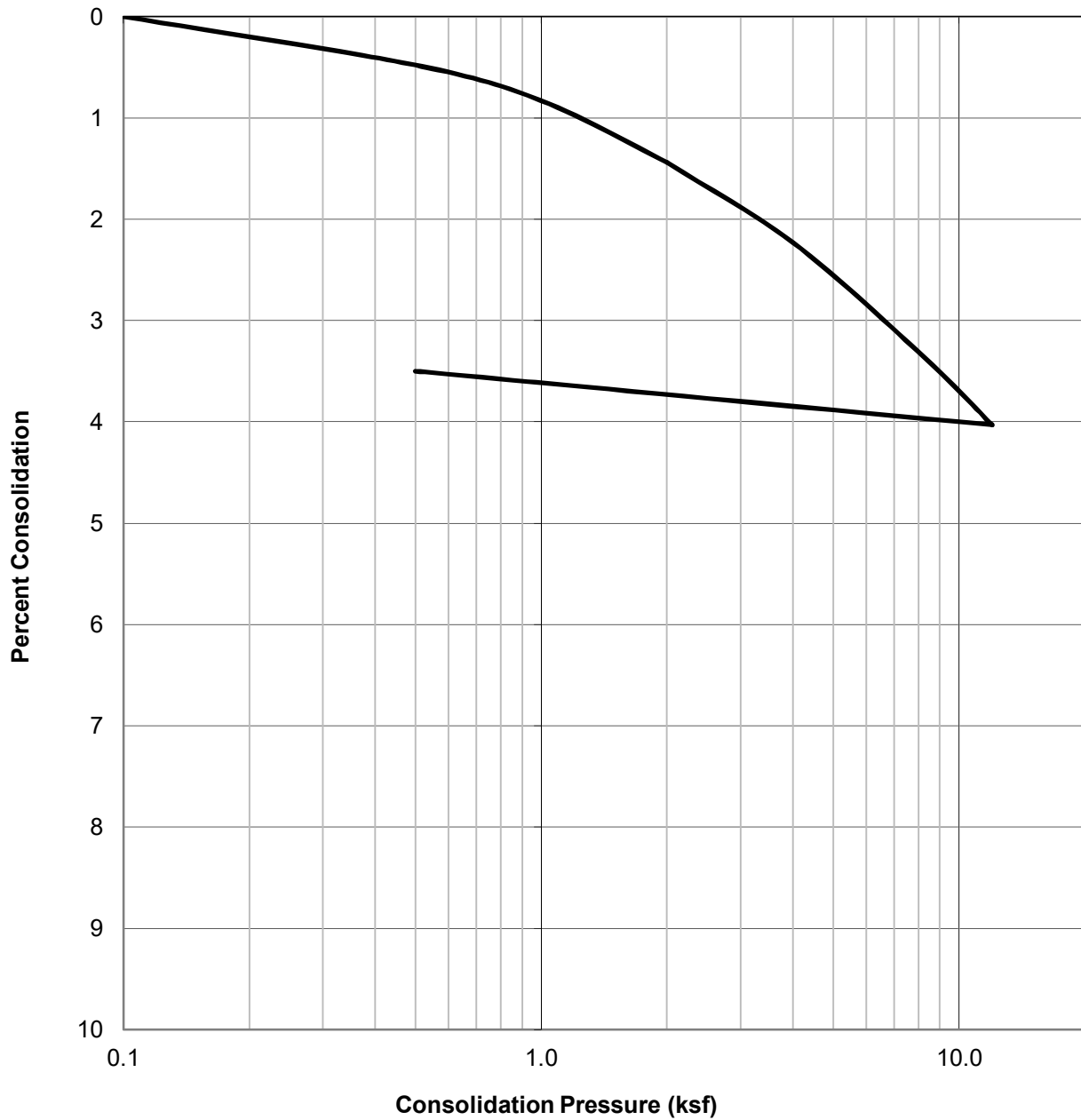
WATER ADDED AT 2.0 KSF




SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B2@37	Dark Yellowish Brown Sandy Clay (CL)	112.3	17.1	22.5

 GEOCON	CONSOLIDATION TEST RESULTS <small>ASTM D-2435</small>	Project No.: W1135-06-01
		1235 Vine Street Los Angeles, California
	Checked by: PZ	July 2020

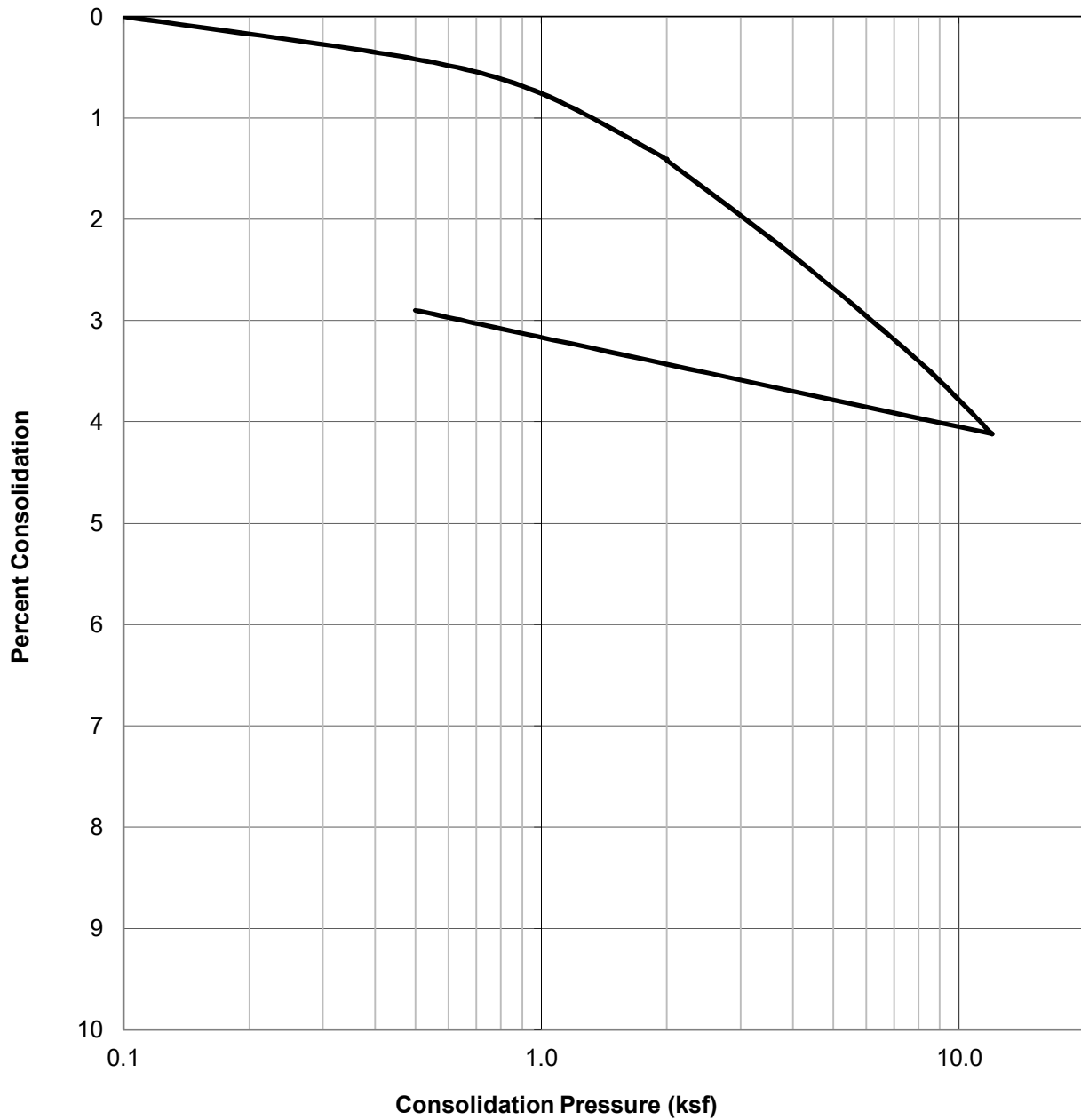
WATER ADDED AT 2.0 KSF




SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B1@37.5	Dark Yellowish Brown Clayey Sand (SC)	109.1	18.7	16.1

	CONSOLIDATION TEST RESULTS ASTM D-2435	Project No.: W1135-06-01
		1235 Vine Street Los Angeles, California
	Checked by: PZ	July 2020

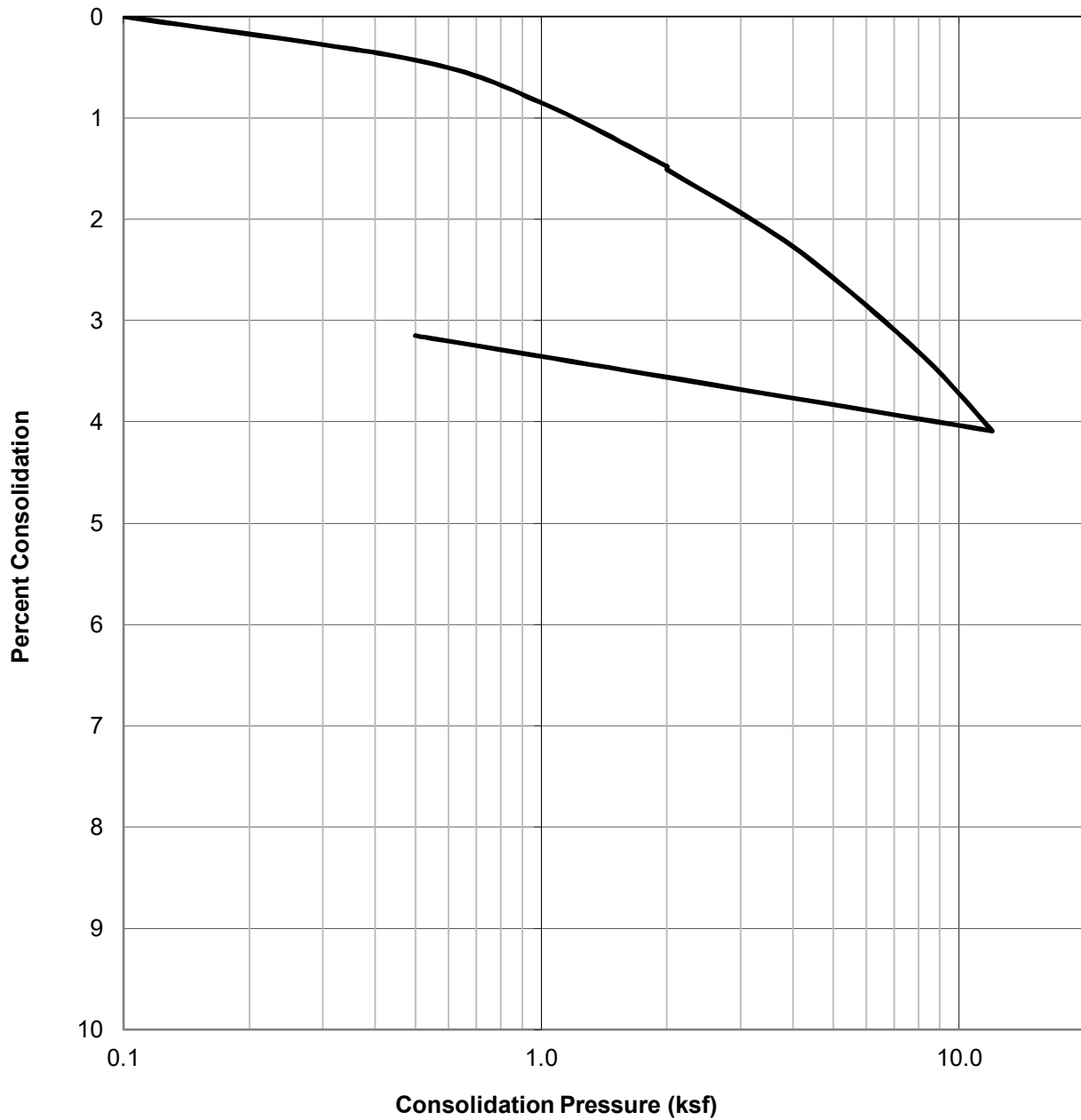
WATER ADDED AT 2.0 KSF




SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B1@40	Dark Yellowish Brown Clayey Sand (SC)	101.7	24.1	20.7

	CONSOLIDATION TEST RESULTS ASTM D-2435	Project No.: W1135-06-01
	Checked by: PZ	1235 Vine Street Los Angeles, California
	July 2020	Figure B16

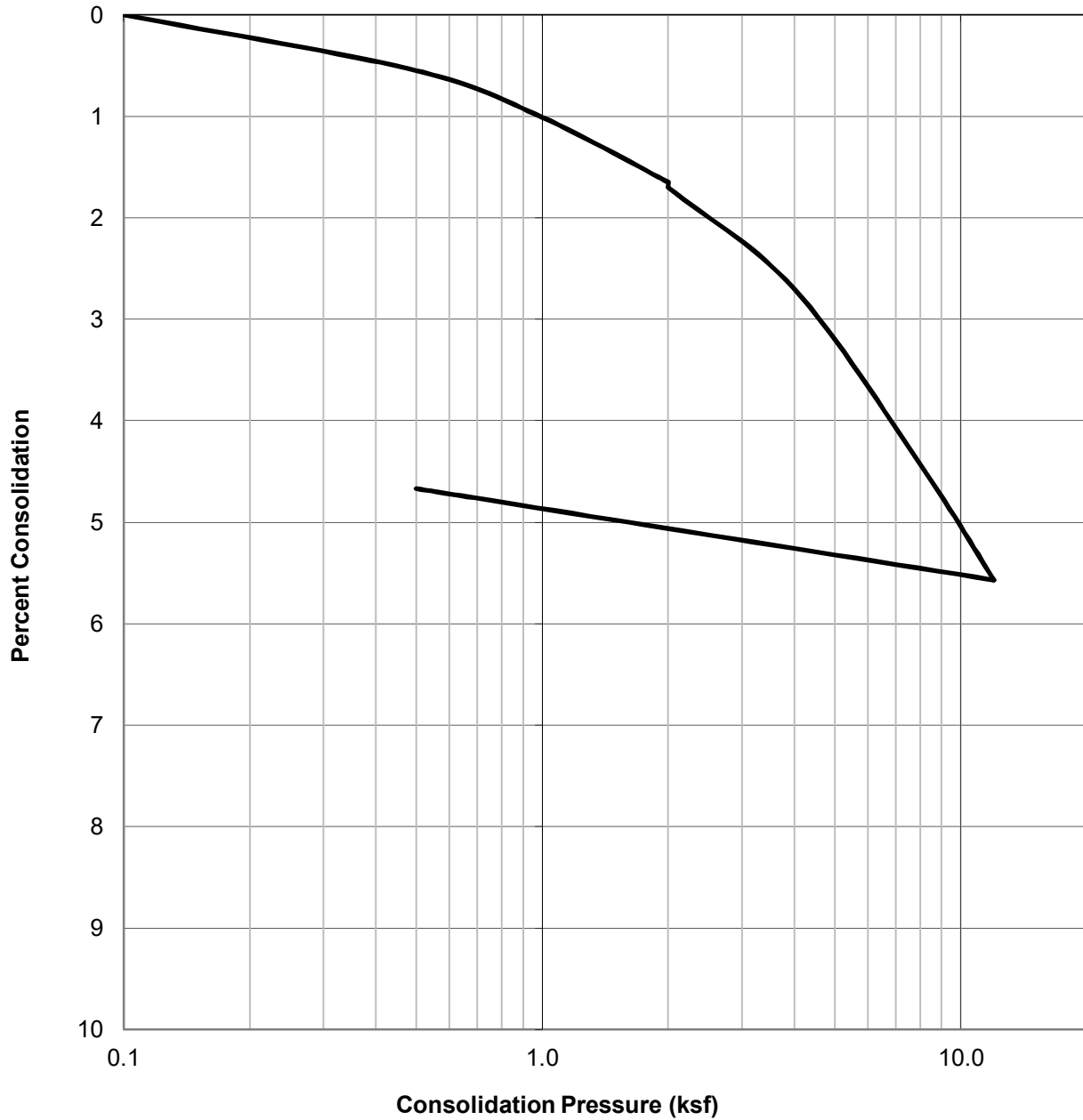
WATER ADDED AT 2.0 KSF




SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B2@42	Dark Yellowish Brown Poorly Graded Sand (SP)	111.3	16.8	15.9

 GEOCON	CONSOLIDATION TEST RESULTS		Project No.:	W1135-06-01
	ASTM D-2435		1235 Vine Street Los Angeles, California	
	Checked by:	PZ	July 2020	Figure B17

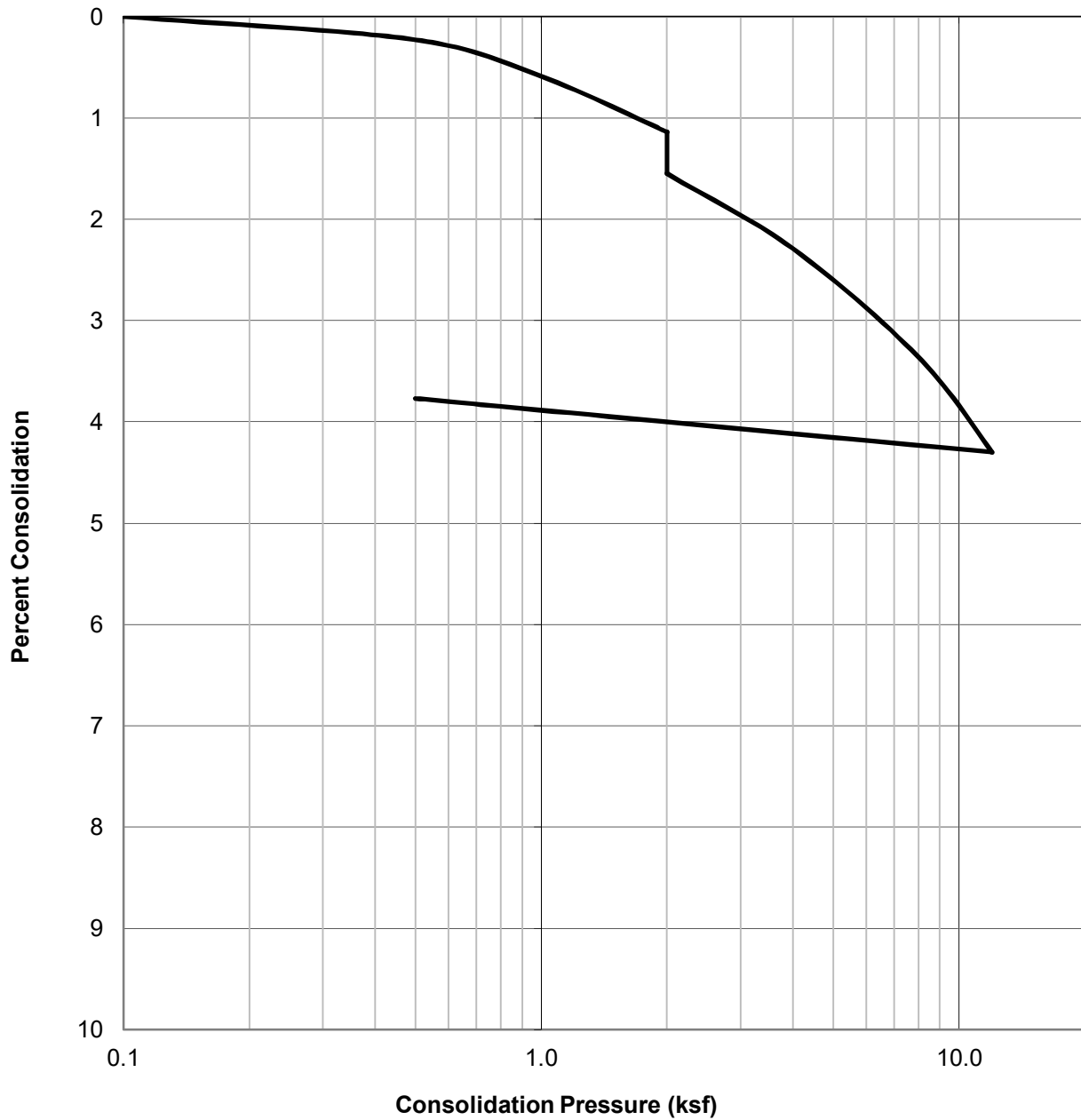
WATER ADDED AT 2.0 KSF




SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B2@45	Dark Yellowish Brown Sandy Clay (CL)	103.9	21.9	19.4

	CONSOLIDATION TEST RESULTS ASTM D-2435	Project No.: W1135-06-01
		1235 Vine Street Los Angeles, California
	Checked by: PZ	July 2020

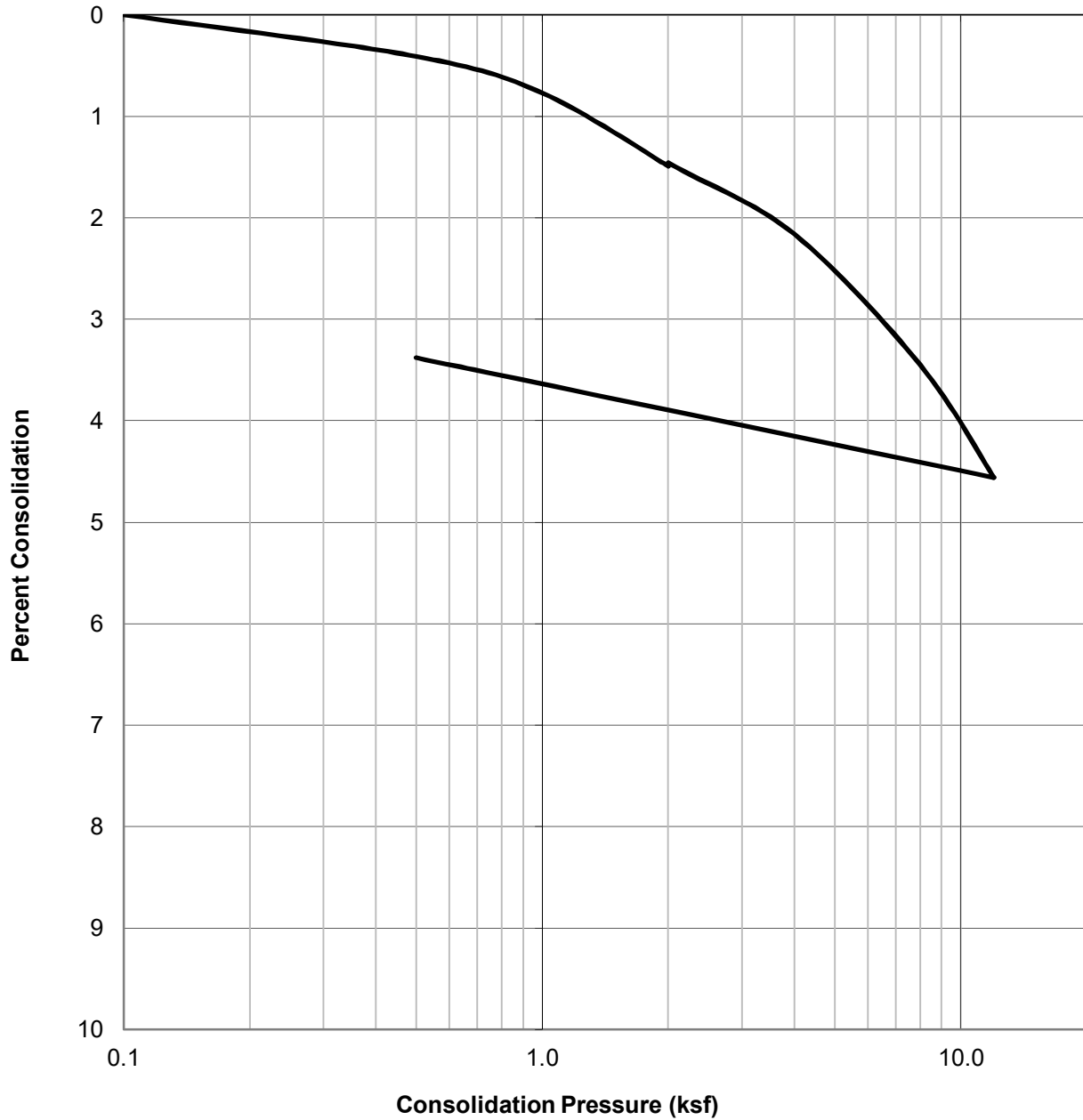
WATER ADDED AT 2.0 KSF




SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B1@50	Dark Yellowish Brown Sandy Clay (CL)	113.1	16.7	16.8

 GEOCON	CONSOLIDATION TEST RESULTS ASTM D-2435	Project No.: W1135-06-01
		1235 Vine Street Los Angeles, California
	Checked by: PZ	July 2020

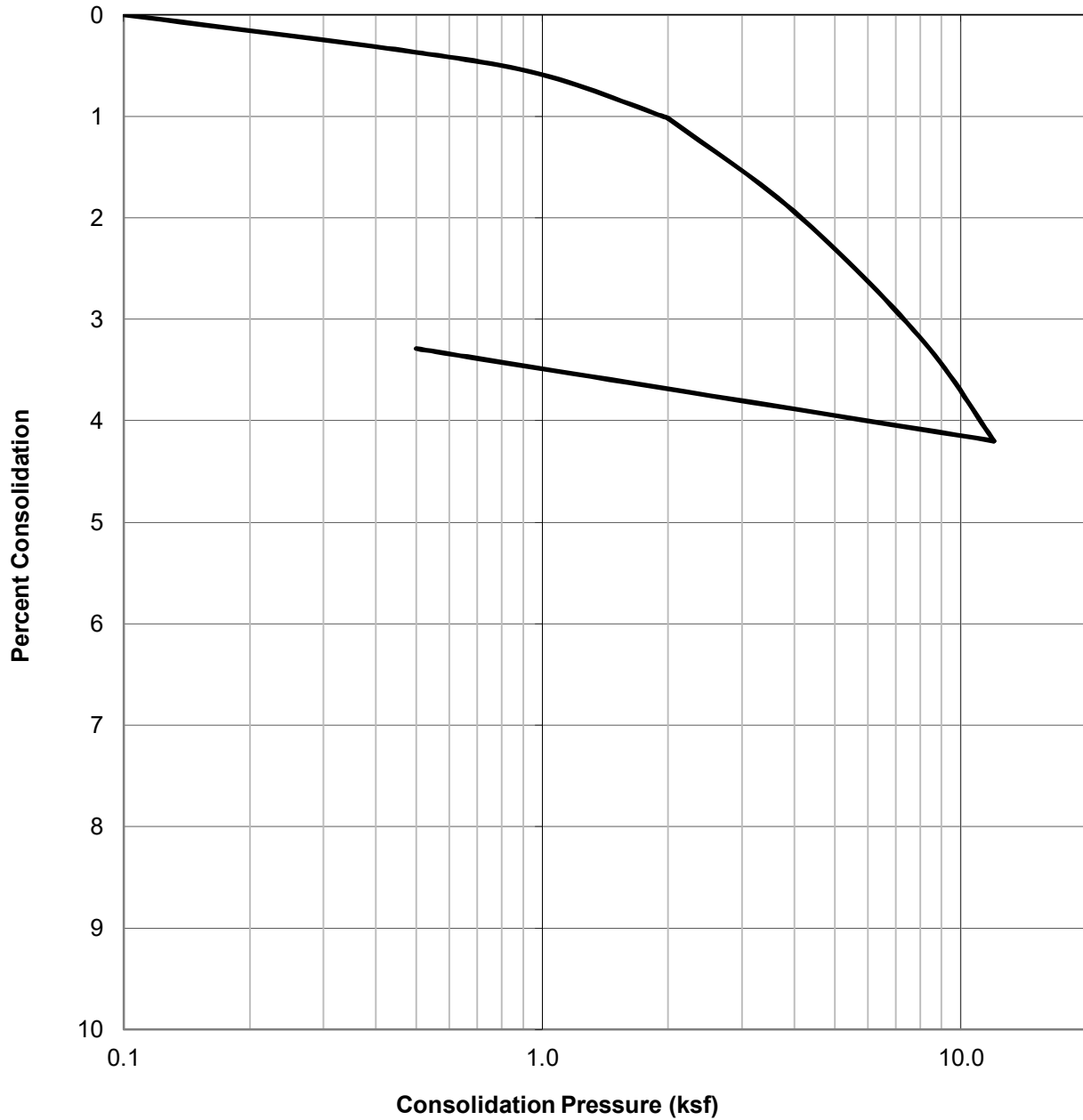
WATER ADDED AT 2.0 KSF



SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B2@55	Dark Yellowish Brown Sandy Clay (CL)	112.5	17.1	16.7

	CONSOLIDATION TEST RESULTS ASTM D-2435	Project No.: W1135-06-01
	Checked by: PZ	1235 Vine Street Los Angeles, California
	July 2020	Figure B20

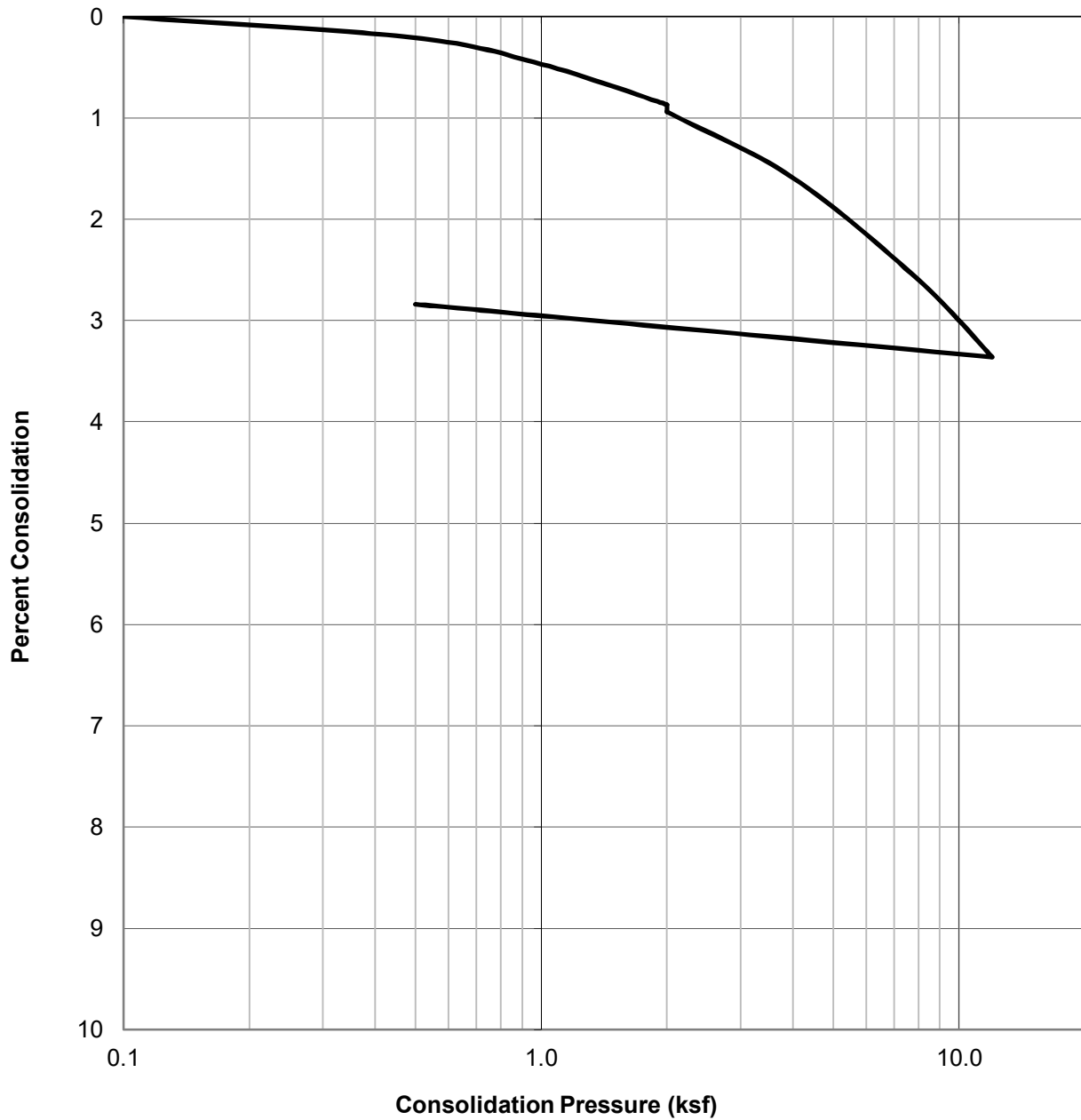
WATER ADDED AT 2.0 KSF



SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B1@60	Yellowish Brown Clayey Sand (SC)	117.2	15.8	14.8

	CONSOLIDATION TEST RESULTS ASTM D-2435	Project No.: W1135-06-01
		1235 Vine Street Los Angeles, California
	Checked by: PZ	July 2020

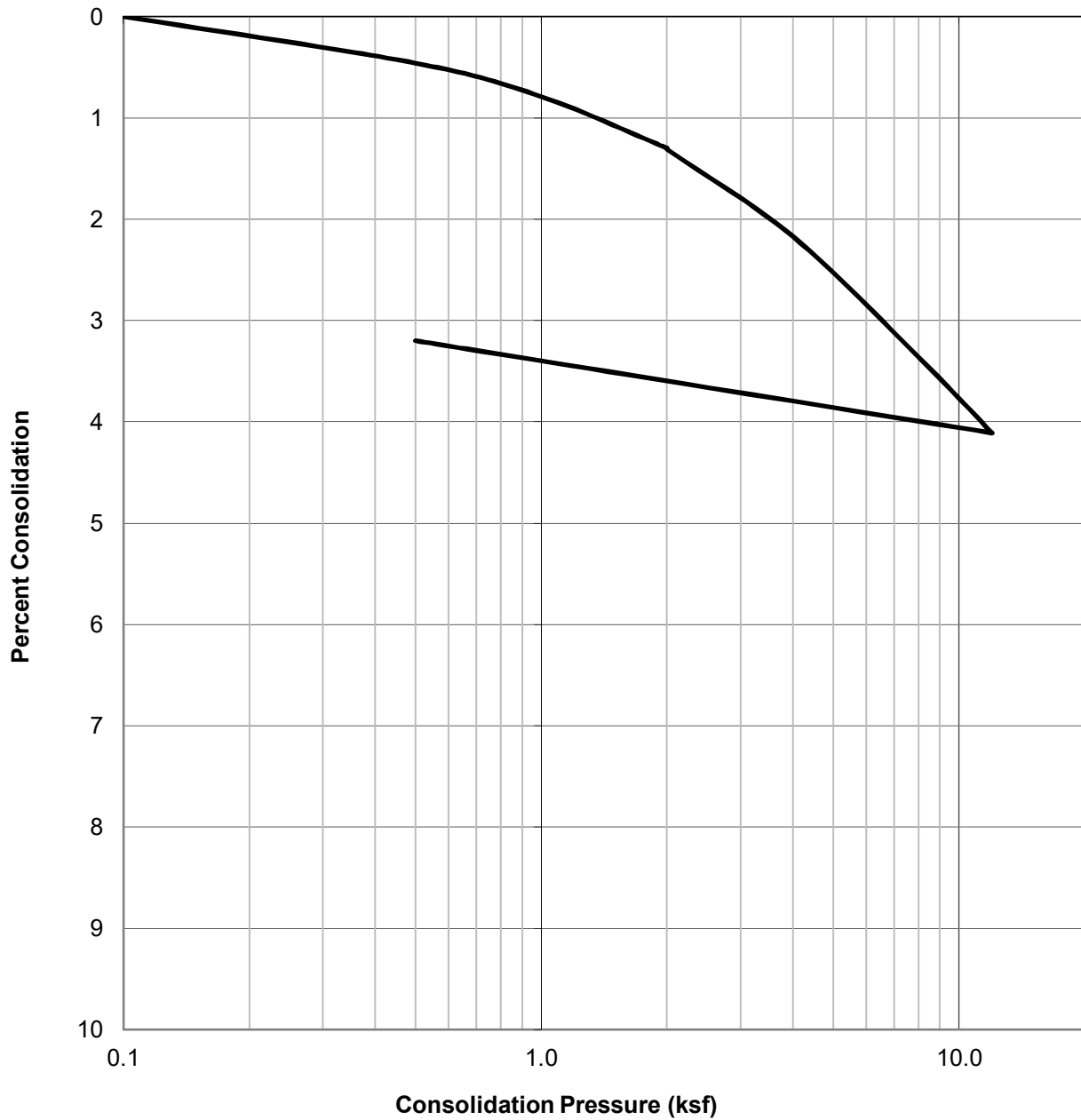
WATER ADDED AT 2.0 KSF



SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B2@60	Yellowish Brown Clayey Sand (SC)	112.4	14.6	15.8

	CONSOLIDATION TEST RESULTS <small>ASTM D-2435</small>	Project No.: W1135-06-01
	Checked by: PZ	1235 Vine Street Los Angeles, California
	July 2020	Figure B22

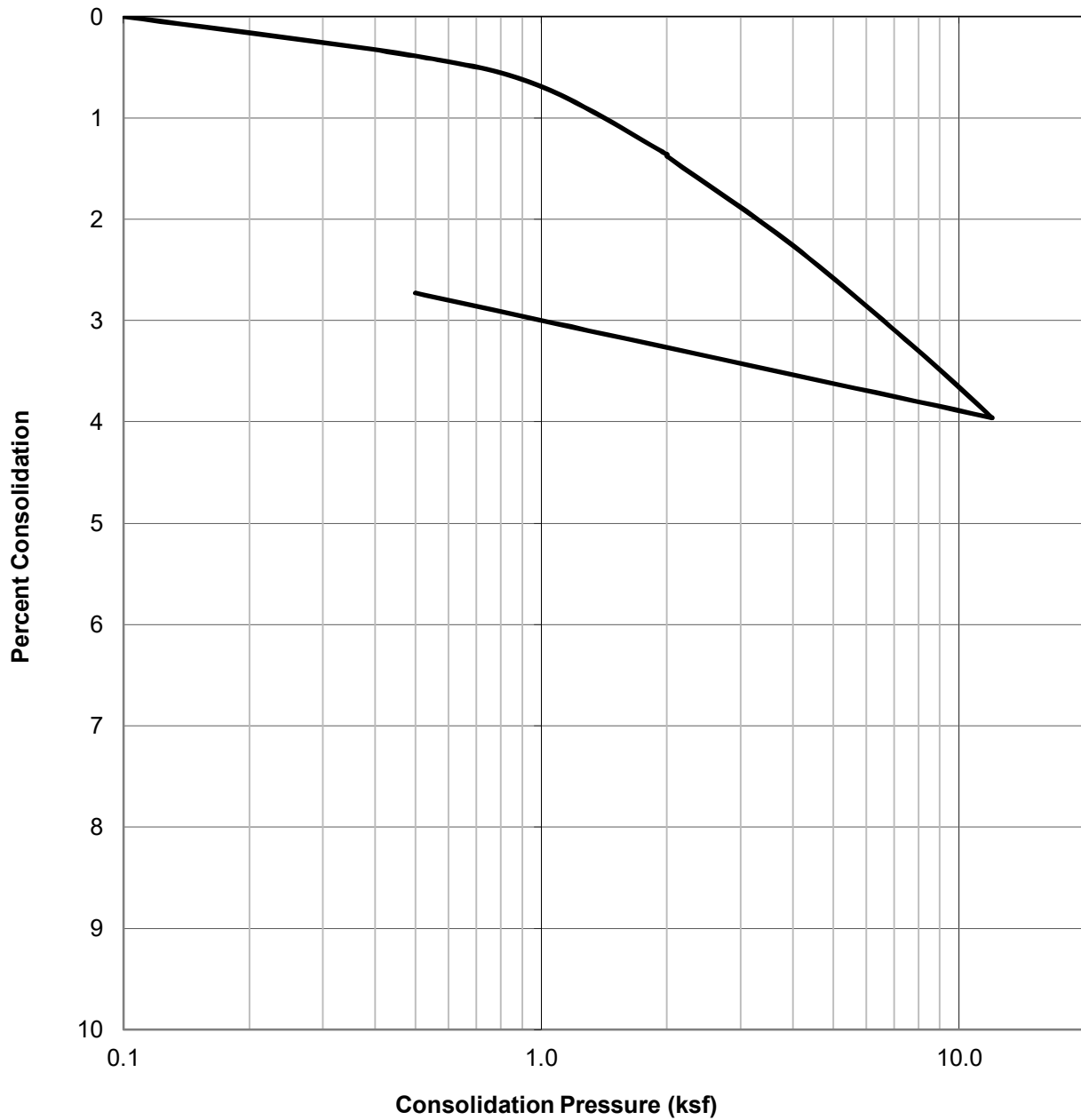
WATER ADDED AT 2.0 KSF




SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B1@65	Yellowish Brown Clayey Sand (SC)	113.3	17.3	16.4

	CONSOLIDATION TEST RESULTS ASTM D-2435	Project No.: W1135-06-01
	Checked by: PZ	1235 Vine Street Los Angeles, California
	July 2020	Figure B23

WATER ADDED AT 2.0 KSF



SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B1@70	Yellowish Brown Well Graded Sand (SW)	105.2	21.4	20.1

	CONSOLIDATION TEST RESULTS ASTM D-2435	Project No.: W1135-06-01
	Checked by: PZ	1235 Vine Street Los Angeles, California
	July 2020	Figure B24

B1@30-35'

MOLDED SPECIMEN		BEFORE TEST	AFTER TEST
Specimen Diameter	(in.)	4.0	4.0
Specimen Height	(in.)	1.0	1.0
Wt. Comp. Soil + Mold	(gm)	615.0	649.0
Wt. of Mold	(gm)	202.1	202.1
Specific Gravity	(Assumed)	2.7	2.7
Wet Wt. of Soil + Cont.	(gm)	396.5	649.0
Dry Wt. of Soil + Cont.	(gm)	371.7	378.8
Wt. of Container	(gm)	96.5	202.1
Moisture Content	(%)	9.0	18.0
Wet Density	(pcf)	124.6	134.6
Dry Density	(pcf)	114.3	114.1
Void Ratio		0.5	0.5
Total Porosity		0.3	0.3
Pore Volume	(cc)	66.7	75.5
Degree of Saturation	(%) [S_{meas}]	51.5	90.2

Date	Time	Pressure (psi)	Elapsed Time (min)	Dial Readings (in.)
4/20/2020	11:55	1.0	0	0.4167
4/20/2020	12:05	1.0	10	0.4162
Add Distilled Water to the Specimen				
4/21/2020	10:45	1.0	1360	0.4588
4/21/2020	11:45	1.0	1420	0.4588

Expansion Index (EI meas) =	42.6
Expansion Index (Report) =	43

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

* Reference: 2019 California Building Code, Section 1803.5.3

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

	EXPANSION INDEX TEST RESULTS	Project No.: W1135-06-01
	ASTM D-4829	1235 Vine Street Los Angeles, California
	Checked by: PZ	July 2020 Figure B25

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


Sample No.	pH	Resistivity (ohm centimeters)
B1 @ 30-35'	7.4	1500 (Corrosive)

SUMMARY OF LABORATORY CHLORIDE CONTENT TEST RESULTS
EPA NO. 325.3

Sample No.	Chloride Ion Content (%)
B1 @ 30-35'	0.003

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

Sample No.	Water Soluble Sulfate (% SQ ₄)	Sulfate Exposure*
B1 @ 30-35'	0.000	S0
B1 @ 45'	0.035	S0

	CORROSIVITY TEST RESULTS	Project No.: W1135-06-01
	Checked by: PZ	1235 Vine Street Los Angeles, California
		July 2020 Figure B26