Initial Study/Mitigated Negative Declaration

Alta Cuvee Mixed Use Project

APPENDIX E
GEOTECHNICAL INVESTIGATION

GEOCON
WEST, INC.

GEOTECHNICAL
ENVIRONMENTAL
MATERIALS

GEOTECHNICAL INVESTIGATION

PROPOSED MULTI-FAMILY RESIDENTIAL DEVELOPMENT
12939 FOOTHILL BOULEVARD
RANCHO CUCAMONGA, CALIFORNIA
APN: 0229-311-15

PREPARED FOR

CRP/WP ALTA CUVEE VENTURE, L.L.C.
LOS ANGELES, CA

PROJECT NO. W1211-99-01

AUGUST 21, 2020

GEOCON
WEST, INC.
GEOTECHNICAL ■ ENVIRONMENTAL ■ MATERIALS

Project No. W1211-99-01
August 21, 2020

Mr. Joe Gambill
CRP/WP ALTA CUVEE VENTURE, L.L.C.
11849 West Olympic Blvd., Suite 204
Los Angeles, California 900645

Subject: GEOTECHNICAL INVESTIGATION
PROPOSED MULTI-FAMILY RESIDENTIAL DEVELOPMENT
12939 FOOTHILL BOULEVARD, RANCHO CUCAMONGA, CALIFORNIA
APN: 0229-311-15

Dear Mr. Gambill:

In accordance with your authorization of our proposal dated March 10, 2020, we have prepared this geotechnical investigation report for the proposed multi-family residential development located at 12939 Foothill Boulevard in the City of Rancho Cucamonga, California. The accompanying report presents the findings of our study, and our conclusions and recommendations pertaining to the geotechnical aspects of proposed design and construction. Based on the results of our investigation, it is our opinion that the site can be developed as proposed, provided the recommendations of this report are followed and implemented during design and construction.

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

Very truly yours,

GEOCON WEST, INC.

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GEOTECHNICAL INVESTIGATION

1. PURPOSE AND SCOPE

This report presents the results of a geotechnical investigation for the proposed multi-family residential development located at 12939 Foothill Boulevard in the City of Rancho Cucamonga, California (see Vicinity Map, Figure 1). The purpose of the investigation was to evaluate subsurface soil and geologic conditions underlying the site and, based on conditions encountered, to provide conclusions and recommendations pertaining to the geotechnical aspects of design and construction.

The scope of this investigation included a review of published documents for the site, a site reconnaissance, field exploration, laboratory testing, engineering analysis, and the preparation of this report. The site was explored on August 4 and 5, 2020, by excavating six borings and eleven backhoe pits. The borings were 8-inches in diameter and were drilled to depths between 21½ and 71 feet below the existing ground surface using a truck-mounted hollow-stem auger drilling machine. The backhoe pits were excavated to depths between 5 and 10 feet below the existing ground surface using a rubber-tire backhoe equipped with a 3-foot bucket. The approximate locations of the exploratory excavations are depicted on the Site Plan (see Figure 2). A detailed discussion of the field investigation, including the boring and test pit 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 12939 Foothill Boulevard, in the City of Rancho Cucamonga, California. The site is roughly level, undeveloped, contains sparse vegetation, and appears to have been previously plowed for weed abatement. The site is bounded by Foothill Boulevard to the north, Etiwanda Avenue to the west, and existing residential developments to the south and east. The site topography is roughly level to gently sloping to the east. Surface water drainage at the site appears to be by sheet flow towards an existing earthen swale that bisects the property in a north-south direction in the eastern third of the site. An easement for an existing water line was observed along the northern site boundary. Based on a review of aerial photographs the prior site use appears to have been agricultural.

Based on the information provided by the Client, it is our understanding that the proposed development will consist of a four-story multi-family residential development to be constructed partially on-grade (western portion) and partially over a single level of subterranean parking (eastern portion). Additional site improvements are expected to include utility connections, CMU perimeter walls, lighting, landscaping, and pavement. The proposed development is depicted on the Site Plan (see Figure 2).

Based on the preliminary nature of the design at this time, wall and column loads were not available. It is anticipated that column loads will range between 150 and 550 kips, and wall loads will range between 2 and 5 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 northern portion of the Chino Basin in San Bernardino County, California. The Chino Basin encompasses a broad area of coalescing alluvial fans that extend southward from the San Gabriel Mountains and overlies a down-dropped structural block which is bounded by the Elsinore Fault and the Chino Fault to the southwest, the Red Hill-Etiwanda Avenue Fault to the north, the San Gabriel Mountains and Sierra Madre Fault to the north, by the Rialto-Colton Fault to the east, and the Jurupa Hills to the south. The alluvial deposits within the Chino Basin consist of Holocene age (last 11,700 years old) and Pleistocene age (11,000 to 2 million years old) alluvial sediments. A thin veneer of eolian sand mantles portions of the Chino Basin.

Locally, the site is located on one of the alluvial fans that extends southward from the San Gabriel Mountains. Regionally, the Chino Basin is located within the Peninsular Ranges geomorphic province. This province comprises the northwesterly-trending mountains, valleys, and geologic structures extending from the southern Baja Peninsula to the Transverse Ranges, which are located just to the north of the site.

4. SOIL AND GEOLOGIC CONDITIONS

Based on our field investigation and published geologic maps of the area, the site is underlain by undocumented fill on the eastern portion of the site and Holocene age alluvial fan deposits consisting predominately of silty sand (CGS, 2010). Detailed stratigraphic profiles of the materials encountered at the site are provided on the excavation logs in Appendix A.

4.1 Artificial Fill

Artificial fill was encountered in our field explorations to a maximum depth of 3 feet below grade. The fill generally consists of yellowish brown to dark brown silty sand. The fill is characterized as dry to moist and medium dense. The fill is likely the result of past farming activities at the site. Deeper fill may exist between excavations and in other portions of the site that were not directly explored.

4.2 Alluvial Fan Deposits (Qal)

Holocene age alluvial deposits were encountered beneath the fill. The alluvium was predominately reddish brown silty sand. Lesser amounts of sandy silt and clay, and poorly graded sand with varying amounts of gravel and cobble were also observed. Although not directly observed in our excavations, some cobbles and boulders are common in this geologic environment. The alluvium is characterized as fine- to coarse-grained, moist, and medium dense to very dense.

5. GROUNDWATER

The site is located in the Chino Basin of the Upper Santa Ana Valley Groundwater Basin (Chino Basin Water Master [CBWM] 2017). A review of groundwater contour maps published by the California Division of Mines and Geology (CDMG, 1976) and the U. S. Geological Survey (Mendenhall, 1904) indicate that the groundwater level in the immediate site vicinity has historically been greater than 250 feet beneath the ground surface since 1904.

Review of the California Department of Water Resources Data Library (CDWR, 2019) indicates the closest groundwater monitoring well to the site is Well Number 341217N1175119W001, located approximately 1.5 miles north of the site. The highest groundwater level recorded for this well for the monitoring period between 1966 and 2017, was in 2012 when groundwater was at a depth of approximately 574 feet beneath the existing ground surface. The most recent groundwater level measurement indicates the depth to water was approximately 586 feet below the surface on March 11, 2020. Based on current groundwater basin management practices, it is unlikely that groundwater levels will ever exceed the historic high levels.

Groundwater was not encountered in the borings drilled to a maximum depth of 71 feet beneath the existing ground surface. Based on the lack of groundwater observed in our borings, the depth to groundwater as recorded in nearby wells (CDWR, 2019; CBWM, 2017), and the depth of the proposed construction, groundwater is neither expected to be encountered during construction, nor have a detrimental effect on the Project. However, it is common for groundwater levels to vary seasonally or for perched groundwater conditions to develop where none previously existed, especially in impermeable fine-grained soils which are subjected to irrigation or precipitation. 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 to future performance of the Project. Recommendations for drainage are provided in the S*urface Drainage* section of this report (see Section 7.19).

6. GEOLOGIC HAZARDS

6.1 Surface Fault Rupture

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

The site is not within a state-designated Alquist-Priolo Earthquake Fault Zone (CGS, 2020a; CGS, 2020b) for surface fault rupture hazards. No active or potentially active faults with the potential for surface fault rupture are known to pass directly beneath the site. Therefore, the potential for surface rupture due to faulting occurring beneath the site during the design life of the proposed development is considered low. However, the site is located in the seismically active Southern California region, and could be subjected to moderate to strong ground shaking in the event of an earthquake on one of the many active Southern California faults. The faults in the vicinity of the site are shown in Figure 3, Regional Fault Map.

The closest active fault to the site is the Red Hill Fault located approximately 2.4 miles to the northwest (Ziony and Jones, 1989; USGS, 2006; CDMG, 1995). Other nearby active faults are the Cucamonga Fault, the San Jacinto Fault Zone, the San Andreas Fault Zone, and the Chino Fault located approximately 4.2 miles north, 8.4 miles northeast, 11.3 miles northeast, and 13.9 miles southwest of the site, respectively (Ziony and Jones, 1989).

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

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
San Jacinto-Hemet area	April 21, 1918	6.8	39	SE
Near Redlands	July 23, 1923	6.3	17	ESE
Long Beach	March 10, 1933	6.4	42	SW
Tehachapi	July 21, 1952	7.5	105	NW
San Fernando	February 9, 1971	6.6	54	WNW
Whittier Narrows	October 1, 1987	5.9	32	W
Sierra Madre	June 28, 1991	5.8	29	WNW
Landers	June 28, 1992	7.3	63	E
Big Bear	June 28, 1992	6.4	40	E
Northridge	January 17, 1994	6.7	58	W
Hector Mine	October 16, 1999	7.1	79	ENE
Ridgecrest China Lake	July 5, 2019	7.1	115	N

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

6.3 Seismic Design Criteria

The following table summarizes summarizes site-specific design criteria obtained from the 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 USGS. The short spectral response uses a period of 0.2 second. We evaluated the Site Class based on the discussion in Section 1613.3.2 of the 2019 CBC and Table 20.3-1 of ASCE 7-16. The values presented on the following page are for the risk-targeted maximum considered earthquake (MCE_R).

2019 CBC SEISMIC DESIGN PARAMETERS

Parameter	Value	2016 CBC Reference
Site Class	D	Table 1613.3.2
MCE_R Ground Motion Spectral Response Acceleration – Class B (short), S_S	1.821g	Figure 1613.3.1(1)
MCE_R Ground Motion Spectral Response Acceleration – Class B (1 sec), S_1	0..609g	Figure 1613.3.1(2)
Site Coefficient, F_A	1	Table 1613.3.3(1)
Site Coefficient, F_V	1.7	Table 1613.3.3(2)
Site Class Modified MCE_R Spectral Response Acceleration (short), S_{MS}	1.821g	Section 1613.3.3 (Eqn 16-37)
Site Class Modified MCE_R Spectral Response Acceleration – (1 sec), S_{M1}	1.035g	Section 1613.3.3 (Eqn 16-38)
5% Damped Design Spectral Response Acceleration (short), S_{DS}	1.214g	Section 1613.3.4 (Eqn 16-39)
5% Damped Design Spectral Response Acceleration (1 sec), S_{D1}	0.69g	Section 1613.3.4 (Eqn 16-40)

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

ASCE 7-10 PEAK GROUND ACCELERATION

Parameter	Value	ASCE 7-10 Reference
Mapped MCE_G Peak Ground Acceleration, PGA	0.764g	Figure 22-7
Site Coefficient, F_{PGA}	1.1	Table 11.8-1
Site Class Modified MCE_G Peak Ground Acceleration, PGA_M	0.841g	Section 11.8.3 (Eqn 11.8-1)

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

Deaggregation of the MCE peak ground acceleration was performed using the USGS online Unified Hazard Tool, 2014 Conterminous U.S. Dynamic edition (v4.2.0). The result of the deaggregation analysis indicates that the predominant earthquake contributing to the MCE peak ground acceleration is characterized as a 6.99 magnitude event occurring at a hypocentral distance of 11.97 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 6.89 magnitude occurring at a hypocentral distance of 14.51 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.

According to the County of San Bernardino (2007) and the City of Rancho Cucamonga Safety Element of the General Plan (2010), the site is not located within an area identified as having a potential for liquefaction. Also, the groundwater level in the immediate site vicinity has been greater than 250 feet since 1904 and is currently greater than 500 feet beneath the site. Based on these considerations, it is our opinion that the potential for liquefaction and seismically induced settlements to occur beneath the site is considered low.

6.5 Slope Stability

The topography at the site and surrounding is relatively level to sloping gently to the south. According to the City of Rancho Cucamonga Safety Element (2010), the site is not within an area identified as having a potential for slope instability. There are no known landslides near the site, nor is the site in the path of any known or potential landslides. Therefore, the potential for slope stability hazards to adversely affect the proposed development is considered low.

6.6 Earthquake-Induced Flooding

Earthquake-induced flooding is inundation caused by failure of dams or other water-retaining structures due to earthquakes. The City of Rancho Cucamonga Safety Element (City of Rancho Cucamonga, 2010) and the County of San Bernardino (2007) indicate that the site is not located within a dam or debris basin inundation area or flood boundary from any such reservoirs. Therefore, the potential for inundation at the site as a result of an earthquake-induced dam failure is considered low.

6.7 Tsunamis, Seiches, and Flooding

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

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

The site is within an area of minimal flooding (Zone X) as defined by the Federal Emergency Management Agency (FEMA, 2020; City of Rancho Cucamonga, 2010).

6.8 Oil Fields & Methane Potential

Based on a review of the California Department of Conservation, Geologic Energy Management Division (CalGEM, formally known as DOGGR) Well Finder Website, the site is not located within the limits of an oilfield and oil or gas wells are not located in the immediate site vicinity (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.

Since the site is not located within the boundaries of a known oil field, the potential for the presence of methane or other volatile gases at the site is considered low. However, should it be determined that a methane study is required for the proposed development it is recommended that a qualified methane consultant be retained to perform the study and provide mitigation measures as necessary.

6.9 Subsidence

Subsidence occurs when a large portion of land is displaced vertically, usually due to the withdrawal of groundwater, oil, or natural gas. Soils that are particularly subject to subsidence include those with high silt or clay content. The City of Rancho Cucamonga (2010) indicates that regional subsidence is possible within the general area of the site due to the low density of the subsurface soils. However, in the 1970s, the County of San Bernardino initiated a groundwater recharge program that has minimized subsidence in the area. No known large-scale extraction of groundwater, gas, oil, or geothermal energy is occurring or planned at the site or in the general site vicinity. As long as the County maintains the groundwater recharge program, the potential for ground subsidence due to withdrawal of fluids or gases at the site is considered low.

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 construction of the proposed development provided the recommendations presented herein are followed and implemented during design and construction.

7.1.2 Up to 3 feet of existing undocumented fill was encountered during the site investigation. The fill is likely the result of past agricultural activities at the site. Deeper fill may exist between excavations and in other portions of the site that were not directly explored. It is our opinion that the existing fill, in its present condition, is not suitable for direct support of proposed foundations or slabs. The existing fill and site soils are suitable for re-use as engineered fill provided the recommendations in the *Grading* section of this report are followed (see Section 7.4).

7.1.3 Where structures will be constructed at or near present site grade, as a minimum it is recommended that the upper 5 feet of existing earth materials within the building footprint areas be excavated and properly compacted for foundation and slab support. Deeper excavations should be conducted as needed to remove any encountered fill or soft soils as necessary at the direction of the Geotechnical Engineer (a representative of Geocon). The excavation should extend laterally a minimum distance of 5 feet beyond the building footprint areas, including building appurtenances, or a distance equal to the depth of fill below the foundation, whichever is greater. Subsequent to grading the proposed structures may be supported on a conventional foundation system deriving support in newly placed engineered fill. It is the intent of Geocon that all foundations for on-grade structures be underlain by a minimum of three feet of engineered fill. The limits of existing fill and/or soft soil removal will be verified by the Geocon representative during site grading activities. Recommendations for earthwork are provided in the *Grading* section of this report (see Section 7.4). Recommendations for the design of a conventional foundation system are provided in Section 7.6.

7.1.4 Where proposed structures will be constructed with a subterranean level the structure may be supported on conventional foundation system deriving support in the competent alluvium found below a depth of 9 feet below the existing ground surface. Foundations should be deepened as necessary to penetrate through and encountered soft or unsuitable alluvium at the direction of the Geotechnical Engineer.

7.1.5 All excavations must be observed and approved in writing by the Geotechnical Engineer (a representative of Geocon). Prior to placing any fill, the excavation bottom must be scarified, moistened, and proof-rolled with heavy equipment in the presence of the Geotechnical Engineer (a representative of Geocon West, Inc.).

7.1.6 Where sufficient space is available it is anticipated that stable excavations for the recommended excavations and grading can be achieved with sloping measures. Recommendations for temporary excavations are provided in Section 7.17 of this report.

7.1.7 It is anticipated that excavations on the order of 12 feet in vertical height may be required for construction of the subterranean level, including foundation depths. Due to the depth of the excavation and the proximity to the property lines, city streets and adjacent offsite improvements, excavations may require shoring measures in order to provide a stable excavation. Where shoring is required it is recommended that a soldier pile shoring system by utilized. In addition, where the proposed excavation will be deeper than and adjacent to an offsite structure, the proposed shoring should be designed to resist the surcharge imposed by the adjacent offsite structure. Recommendations for shoring are provided in Section 7.18 of this report.

7.1.8 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, found at or below a depth of 24 inches, and should be deepened as necessary to maintain a minimum 12-inch embedment into the recommended bearing materials. If the soils exposed in the excavation bottom are soft or loose, compaction of the soils will be required prior to placing steel or concrete. Compaction of the foundation excavation bottom is typically accomplished with a compaction wheel or mechanical whacker and must be observed and approved in writing by a Geocon representative.

7.1.9 Where new paving is to be placed, it is recommended that all existing fill soils and soft alluvial soils be excavated and properly compacted for paving support. The client should be aware that excavation and compaction of all existing fill in the area of new paving is not required; however, paving constructed over existing uncertified fill or unsuitable soils may experience increased settlement and/or cracking, and may therefore have a shorter design life and increased maintenance costs. As a minimum, the upper 12 inches of soil should be scarified and properly compacted. Paving recommendations are provided in the *Preliminary Pavement Recommendations* section of this report (see Section 7.11).

7.1.10 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.

7.1.11 Any changes in the design, location or elevation, as outlined in this report, should be reviewed by this office. Once the design and foundation loading configuration proceeds to a more finalized plan, the recommendations within this report should be reviewed and revised, as necessary. 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. 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.

7.2.2 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 7.17).

7.2.3 The existing site soils encountered during the investigation are considered to have a “very low” expansive potential (EI < 20) and are classified as “non-expansive” in accordance with the 2019 California Building Code (CBC) Section 1803.5.3. The recommendations presented herein assume that the building foundations and slabs will derive support in these materials.

7.3 Minimum Resistivity, pH, and Water-Soluble Sulfate

7.3.1 Potential of Hydrogen (pH) and resistivity testing as well as chloride content testing were performed on representative samples of soil to generally evaluate the corrosion potential to surface utilities. The tests were performed in accordance with California Test Method Nos. 643 and 422 and indicate that the soils are considered “moderately corrosive” with respect to corrosion of buried ferrous metals on site. The results are presented in Appendix B (Figure B31) and should be considered for design of underground structures. Due to the corrosive potential of the soils, it is recommended that PVC, ABS or other approved plastic piping be utilized in lieu of cast-iron when in direct contact with the site soils.

7.3.2 Laboratory tests were performed on representative samples of the site materials to measure the percentage of water-soluble sulfate content. Results from the laboratory water-soluble sulfate tests are presented in Appendix B (Figure B31) and indicate that the on-site materials possess a sulfate exposure class of "S0" to concrete structures as defined by 2019 CBC Section 1904 and ACI 318-14 Table 19.3.1.1.

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

7.4 Grading

7.4.1 Grading is anticipated to include preparation of the building pad, excavation for the proposed subterranean level, excavation for proposed foundations and utility trenches, as well as placement of backfill for walls, ramps, and trenches.

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

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

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

7.4.5 Where structures will be constructed at or near present site grade, as a minimum it is recommended that the upper 5 feet of existing earth materials within the building footprint areas be excavated and properly compacted for foundation and slab support. Deeper excavations should be conducted as needed to remove any encountered fill or soft soils as necessary at the direction of the Geotechnical Engineer (a representative of Geocon). The excavation should extend laterally a minimum distance of 5 feet beyond the building footprint areas, including building appurtenances, or a distance equal to the depth of fill below the foundation, whichever is greater. Subsequent to grading the proposed structures may be supported on a conventional foundation system deriving support in newly placed engineered fill. It is the intent of Geocon that all foundations for on-grade structures be underlain by a minimum of three feet of engineered fill. The limits of existing fill and/or soft soil removal will be verified by the Geocon representative during site grading activities. Recommendations for the design of a conventional foundation system are provided in Section 7.6.

7.4.6 If the proposed grade of the building pad is to be raised above the current ground surface elevation, the recommended grading of the upper 5 feet of site soils must be completed prior to placing additional soils to raise the grade.

7.4.7 Where proposed structures will be constructed with a subterranean level the structure may be supported on conventional foundation system deriving support in the competent alluvium found below a depth of 9 feet below the existing ground surface. Foundations should be deepened as necessary to penetrate through and encountered soft or unsuitable alluvium at the direction of the Geotechnical Engineer.

7.4.8 All excavations must be observed and approved in writing by the Geotechnical Engineer (a representative of Geocon). Prior to placing any fill, the excavation bottom must be scarified, moistened, and proof-rolled with heavy equipment in the presence of the Geotechnical Engineer (a representative of Geocon West, Inc.).

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

7.4.10 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, found at or below a depth of 24 inches, and should be deepened as necessary to maintain a minimum 12-inch embedment into the recommended bearing materials. If the soils exposed in the excavation bottom are soft or loose, compaction of the soils will be required prior to placing steel or concrete. Compaction of the foundation excavation bottom is typically accomplished with a compaction wheel or mechanical whacker and must be observed and approved in writing by a Geocon representative.

7.4.11 Although not anticipated for this Project, all imported fill shall be observed, tested, and approved by Geocon West, Inc. prior to bringing soil to the site. Rocks larger than 6 inches in diameter shall not be used in the fill. If necessary, import soils used as structural fill should have an expansion index less than 20 and corrosivity properties that are equally or less detrimental to that of the existing onsite soils (see Figure B31). If import soils will be utilized in the building pad, the soils must be placed uniformly and at equal thickness at the direction of the Geotechnical Engineer (a representative of Geocon West, Inc.). For efficiency purposes soils can be borrowed from non-building pad areas and later replaced with imported soils with lesser soil property restrictions for non-building areas.

7.4.12 It is anticipated that stable excavations for the recommended grading associated with the proposed on-grade portion of the structure can be achieved with sloping measures. However, if excavations in close proximity to an adjacent property line and/or structure are required, special excavation measures may be necessary in order to maintain lateral support of offsite improvements. Excavation recommendations are provided in the *Temporary Excavations* section of this report (see Section 7.17).

7.4.13. Where new paving is to be placed, it is recommended that all existing fill and soft alluvium be excavated and properly compacted for paving support. As a minimum, the upper 12 inches of soil should be scarified, moisture conditioned to optimum moisture content, and compacted to at least 95 percent relative compaction, as determined by ASTM Test Method D 1557 (latest edition). Paving recommendations are provided in *Preliminary Pavement Recommendations* section of this report (see Section 7.11).

7.4.14 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 for pipe bedding and shading is not acceptable unless used in conjunction with filter fabric to prevent the gravel from having direct contact with the excavation earth walls and overlying backfill. 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.

7.4.15 All utility 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, sand, fill, gravel, reinforcing steel or concrete.

7.5 Shrinkage

7.5.1 Shrinkage results when a volume of material removed at one density is compacted to a higher density. A shrinkage factor of between 5 and 10 percent should be anticipated when excavating and compacting the upper 5 feet of existing earth materials on the site to an average relative compaction of 92 percent.

7.6 Conventional Foundation Design

7.6.1 Subsequent to the recommended grading, a conventional shallow spread foundation system may be utilized for support of the proposed structure. Foundations may derive support in the newly placed engineered fill, and or in the competent alluvial soils found at or below a depth of 9 feet.

7.6.2 Continuous footings may be designed for an allowable bearing capacity of 2,500 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.

7.6.3 Isolated spread foundations may be designed for an allowable bearing capacity of 2,750 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.

7.6.4 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 4,000 psf.

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

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

7.6.7 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.

7.6.8 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.

7.6.9 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.

7.6.10 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.

7.6.11 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 Foundation Settlement

7.7.1 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 4,000 psf is estimated to be less than 1 inch and occur below the heaviest loaded structural element. Settlement of the foundation system is expected to occur on initial application of loading. Differential settlement is not expected to exceed ¾ inch over a distance of 20 feet. Due to the partial on-grade and subterranean design of the proposed structure it is recommended that the project structural engineer add additional reinforcing at the area of transition where stresses are likely to greatest.

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

7.8 Miscellaneous Foundations

7.8.1 Foundations for small outlying structures, such as block walls up to 6 feet in height, planter walls or trash enclosures, which will not be 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, found at or below a depth of 24 inches, and should be deepened as necessary to maintain a minimum 12-inch embedment into the recommended bearing materials. If the soils exposed in the excavation bottom are soft or loose, compaction of the soils will be required prior to placing steel or concrete. Compaction of the foundation excavation bottom is typically accomplished with a compaction wheel or mechanical whacker and must be observed and approved in writing by a Geocon representative.

7.8.2 Miscellaneous foundations may be designed for a bearing value of 1,500 psf and should be a minimum of 12 inches in width, 24 inches in depth below the lowest adjacent grade and 12 inches into the recommended bearing material. The allowable bearing pressure may be increased by up to one-third for transient loads due to wind or seismic forces.

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

7.9 Lateral Design

7.9.1 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 0.40 may be used with the dead load forces in the undisturbed alluvial soils or newly placed engineered fill.

7.9.2 Passive earth pressure for the sides of foundations and slabs poured against undisturbed alluvial soils or newly placed engineered fill soils may be computed as an equivalent fluid having a density of 280 pounds per cubic foot (pcf) with a maximum earth pressure of 2,800 psf. When combining passive and friction for lateral resistance, the passive component should be reduced by one-third.

7.10 Concrete Slabs-On-Grade

7.10.1 Unless specifically designed and evaluated by the project structural engineer, concrete slabs-on-grade at the basement level subject to vehicle loading should be a minimum of 6 inches of concrete reinforced with No. 3 steel reinforcing bars placed 18 inches on center in both horizontal directions. As a minimum, the upper 12 inches of subgrade at the basement level should be proof-rolled to a dense state, observed and approved in writing prior to placement of a vapor retarder, reinforcing steel, or concrete. If the slab thickness is reduced by the project structural engineer, it should not be reduced to less than 4 inches in thickness.

7.10.2 Subsequent to the recommended grading, concrete slabs-on-grade for structures, not subject to vehicle loading, should be a minimum of 4 inches thick and minimum slab reinforcement should consist of No. 3 steel reinforcing bars placed 18 inches on center in both horizontal directions. Steel reinforcing should be positioned vertically near the slab midpoint. The finished subgrade for the concrete slab-on-grade must be observed and approved in writing prior to placement of a vapor retarder, reinforcing steel, or concrete.

7.10.3 Slabs-on-grade at the ground surface that may receive moisture-sensitive floor coverings or may be used to store moisture-sensitive materials should be underlain by a vapor retarder placed directly beneath the slab. The vapor retarder and acceptable permeance should be specified by the project architect or developer based on the type of floor covering that will be installed. The vapor retarder 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.

7.10.4 For seismic design purposes, a coefficient of friction of 0.40 may be utilized between concrete slabs and subgrade soils without a moisture barrier, and 0.15 for slabs underlain by a moisture barrier.

7.10.5 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 near 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.

7.10.6 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.

7.11 Preliminary Pavement Recommendations

7.11.1 Where new paving is to be placed, it is recommended that all existing fill and soft alluvium materials be excavated and properly compacted for paving support. The client should be aware that excavation and compaction of all existing artificial fill and soft alluvium in the area of new paving is not required; however, paving constructed over existing uncertified fill or unsuitable alluvium material may experience increased settlement and/or cracking, and may therefore have a shorter design life and increased maintenance costs. As a minimum, the upper 12 inches of paving subgrade should be scarified, moisture conditioned to near optimum moisture content, and properly compacted to at least 95 percent relative compaction, as determined by ASTM Test Method D 1557 (latest edition).

7.11.2 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.

7.11.3 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

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

7.11.5 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. As a minimum, the upper 12 inches of paving subgrade should be scarified, moisture conditioned to near optimum moisture content, and properly compacted to at least 95 percent relative compaction, as determined by ASTM Test Method D 1557 (latest edition). The base material should be compacted to 95 percent relative compaction, as determined by ASTM Test Method D 1557 (latest edition).

7.11.6 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.

7.12 Retaining Wall Design

7.12.1 The recommendations presented below are generally applicable to the design of rigid concrete or masonry retaining walls having a maximum height of 10 feet. In the event that walls higher than 10 feet are planned, Geocon should be contacted for additional recommendations.

7.12.2 Retaining wall foundations may be designed in accordance with the recommendations provided in the *Foundation Design* sections of this report (see Section 7.6).

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

RETAINING WALL WITH LEVEL BACKFILL SURFACE

HEIGHT OF RETAINING WALL (Feet)	ACTIVE PRESSURE EQUIVALENT FLUID PRESSURE (Pounds Per Cubic Foot)	AT-REST PRESSURE EQUIVALENT FLUID PRESSURE (Pounds Per Cubic Foot)
Up to 10	32	57

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

7.12.5 The wall pressures provided above assume that the proposed retaining walls will support relatively undisturbed alluvium or engineered fill derived from site soils.

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

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

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

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

and

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

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

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

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

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

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

and

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

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

then

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

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

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

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

7.13 Dynamic (Seismic) Lateral Forces

7.13.1 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).

7.13.2 A seismic load of 10 pcf should be used for design of walls that support more than 6 feet of backfill in accordance with Section 1803.5.12 of the 2019 CBC. The seismic load is applied as an equivalent fluid pressure along the height of the wall and the calculated loads result in a maximum load exerted at the base of the wall and zero at the top of the wall. This seismic load should be applied in addition to the active earth pressure. The earth pressure is based on half of two-thirds of PGA_M calculated from ASCE 7-10 Section 11.8.3 and applied a pseudo-static coefficient of 0.27.

7.14 Retaining Wall Drainage

7.14.1 Retaining walls should be provided with a drainage system extended at least two-thirds the height of the wall. At the base of the drain system, a subdrain covered with a minimum of 12 inches of gravel should be installed, and a compacted fill blanket or other seal placed at the surface (see Figure 5). The clean bottom and subdrain pipe, behind a retaining wall, should be observed by the Geotechnical Engineer (a representative of Geocon), prior to placement of gravel or compacting backfill.

7.14.2 As an alternative, a plastic drainage composite such as Miradrain or equivalent may be installed in continuous, 4-foot-wide columns along the entire back face of the wall, at 8 feet on center. The top of these drainage composite columns should terminate approximately 18 inches below the ground surface, where either hardscape or a minimum of 18 inches of relatively cohesive material should be placed as a cap (see Figure 6). These vertical columns of drainage material would then be connected at the bottom of the wall to a collection panel or a 1-cubic-foot rock pocket drained by a 4-inch subdrain pipe.

7.14.3 Subdrainage pipes at the base of the retaining wall drainage system should outlet to an acceptable location via controlled drainage structures. Drainage should not be allowed to flow uncontrolled over descending slopes.

7.14.4 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.

7.15 Elevator Pit Design

7.15.1 The elevator pit slab and retaining wall should be designed by the project structural engineer. Elevator pit walls may be designed in accordance with the recommendations in the *Foundation Design* and *Retaining Wall Design* sections of this report (see Section 7.6 and 7.12).

7.15.2 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.

7.15.3 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 7.14).

7.15.4 Subdrainage pipes at the base of the retaining wall drainage system should outlet to a location acceptable to the building official.

7.15.5 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.

7.16 Elevator Piston

7.16.1 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.

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

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7.16.3 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.

7.17 Temporary Excavations

7.17.1 Excavations on the order of 12 feet in height may be required for excavation and construction of the subterranean level, including foundation depths. The excavations are expected to expose artificial fill and alluvial soils, which are considered suitable for temporary vertical excavations up to 5 feet in height where not surcharged by equipment, traffic or structures.

7.17.2 Vertical excavations greater than 5 feet or excavations that are surcharged will require sloping or shoring measures in order to provide a stable excavation. Where sufficient space is available, temporary unsurcharged embankments could be sloped back at a uniform 1:1 slope gradient or flatter up to a maximum of 12 feet in height. A uniform slope does not have a vertical portion. Where space is limited, shoring measures will be required. *Shoring* data is provided in Section 7.18 of this report.

7.17.3 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.

7.18 Shoring – Soldier Pile Design and Installation

7.18.1 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.

7.18.2 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.

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

7.18.4 The proposed soldier piles may also be designed as permanent piles. The required pile depths, dimensions, and spacing should be determined and designed by the project structural and shoring engineers. All piles utilized for shoring can also be incorporated into a permanent retaining wall system (shotcrete wall) and should be designed in accordance with the earth pressure provided in the *Retaining Wall Design* section of this report (see Section 7.12).

7.18.5 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 280 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 three times the pile diameter. To develop the full lateral value, provisions should be implemented to assure firm contact between the soldier piles and the undisturbed alluvium.

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

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

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

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

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

7.18.11 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.

7.18.12 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. 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.

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

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

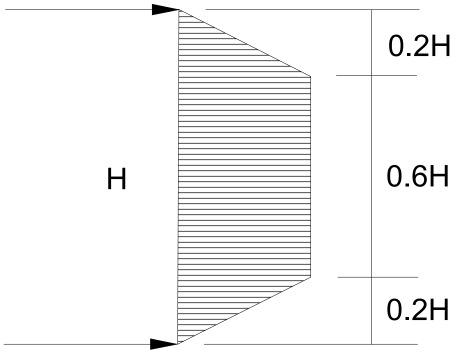
7.18.15 The frictional resistance between the soldier piles and retained soil may be used to resist the vertical component of the load. The coefficient of friction may be taken as 0.40 based on uniform contact between the steel beam and lean-mix concrete and alluvial soils. The portion of soldier piles below the plane of excavation may also be employed to resist the downward loads. The downward capacity may be determined using a frictional resistance of 500 psf.

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

7.18.17 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 at the top by bracing or tie backs. The recommended active and trapezoidal pressures are provided in the following table. A diagram depicting the trapezoidal pressure distribution of lateral earth pressure is provided below the table.

HEIGHT OF SHORING (FEET)	EQUIVALENT FLUID PRESSURE (Pounds Per Cubic Foot) (ACTIVE PRESSURE)	EQUIVALENT FLUID PRESSURE Active Trapezoidal (Where H is the height of the shoring in feet)
Up to 12	25	16H

Trapezoidal Distribution of Pressure



7.18.18 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.

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

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

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

and

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

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

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

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

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

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

and

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

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

then

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

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

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

7.18.22 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. 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.

7.18.23 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.

7.18.24 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.

7.19 Surface Drainage

7.19.1 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.

7.19.2 All site drainage should be collected and controlled in non-erosive drainage devices. Drainage should not be allowed to pond anywhere on the site, and especially not against any foundation or retaining wall. The site should be graded and maintained such that surface drainage is directed away from structures in accordance with 2016 CBC 1804.4 or other applicable standards. In addition, drainage should not be allowed to flow uncontrolled over any descending slope. Discharge from downspouts, roof drains and scuppers are not recommended onto unprotected soils within 5 feet of the building perimeter. Planters which are located adjacent to foundations should be sealed to prevent moisture intrusion into the soils providing foundation support. Landscape irrigation is not recommended within 5 feet of the building perimeter footings except when enclosed in protected planters.

7.19.3 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.

7.19.4 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.

7.20 Plan Review

7.20.1 Grading, shoring, and foundation 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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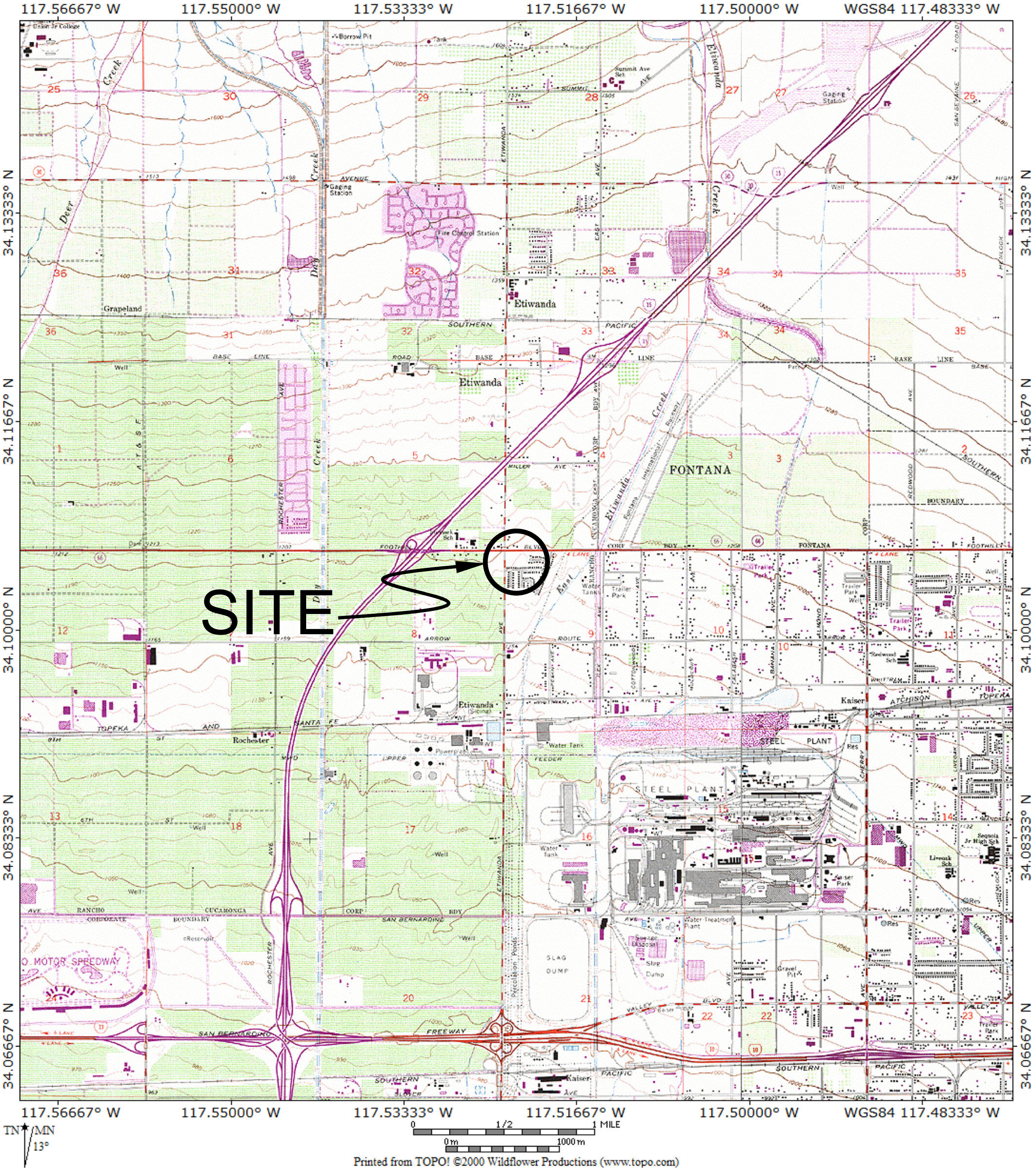

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REFERENCE: U.S.G.S. TOPOGRAPHIC MAPS, 7.5 MINUTE SERIES, GUASTI, CA QUADRANGLE

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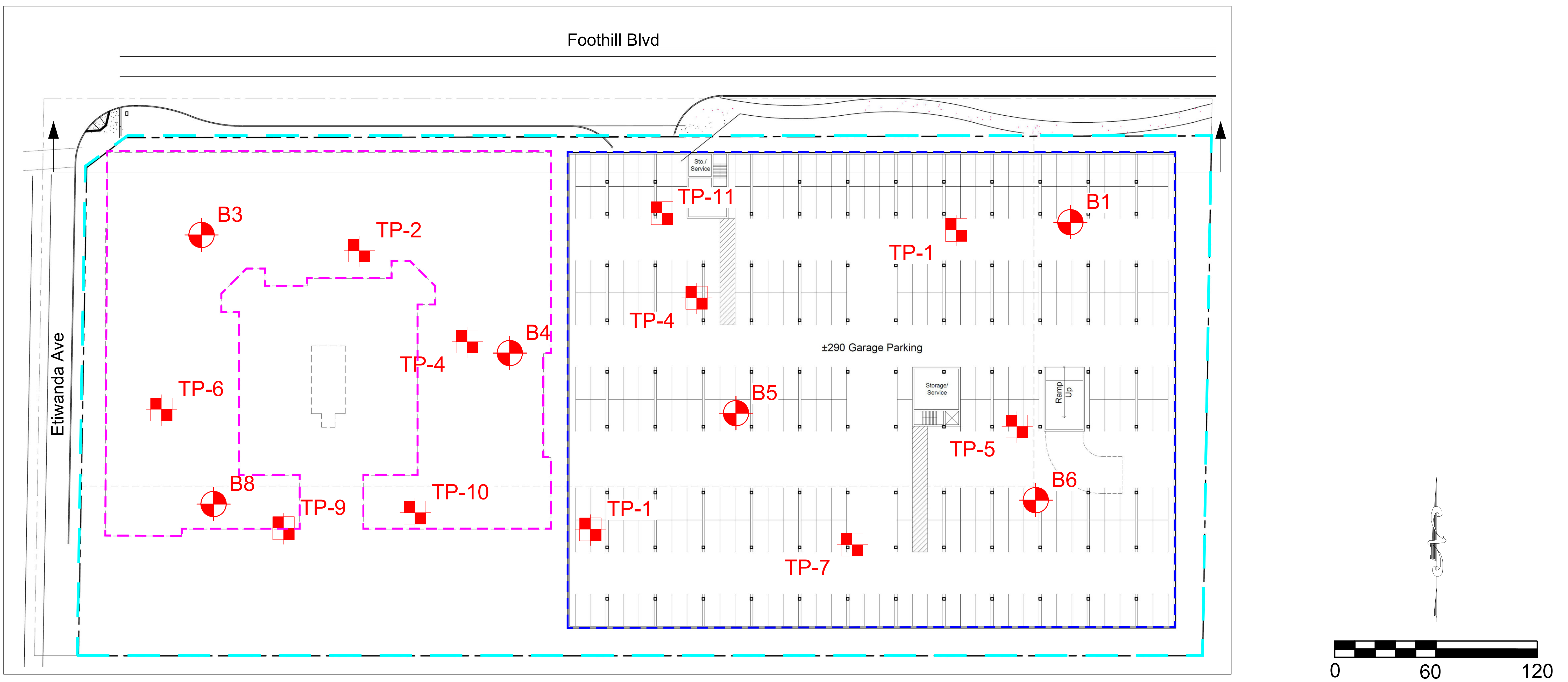
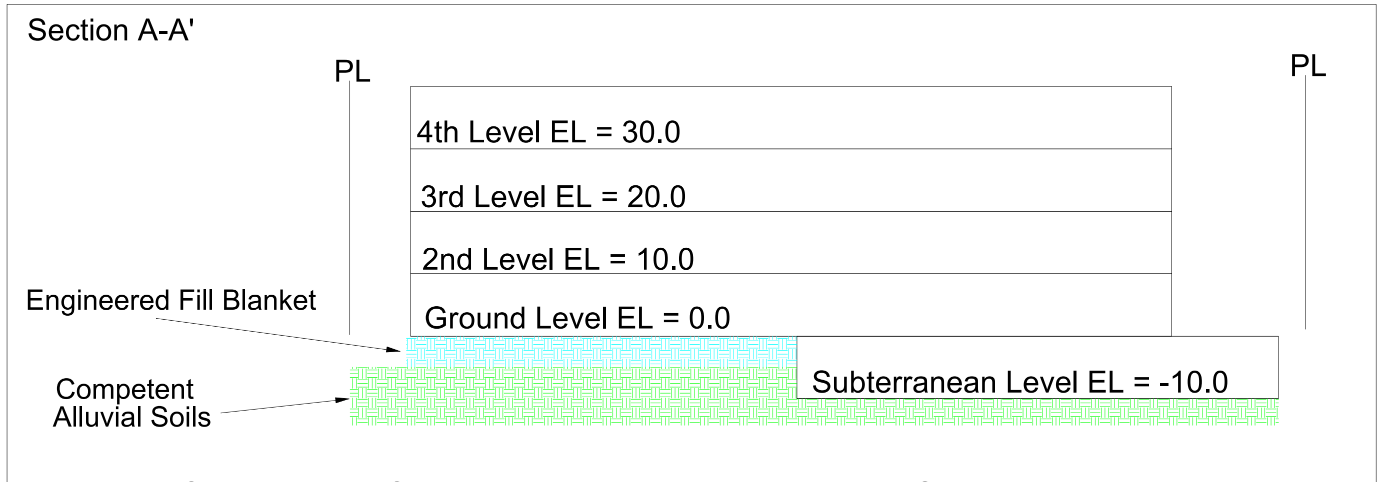
DRAFTED BY: JA	CHECKED BY: GAK

VICINITY MAP

12939 FOOTHILL BLVD
RANCHO CUCAMONGA, CALIFORNIA

AUGUST 2020	PROJECT NO. W1211-99-01	FIG. 1





Note: Not to Scale; Not For Construction; For Illustration Purposes Only

LEGEND

—	Approximate Property Line
⊕ B8	Approximate Location of Boring
🟥 TP-9	Approximate Location of Test Pit
▭	Limits of Subterranean Portion of Structure
▭	Limits of On-Grade Portion of Structure

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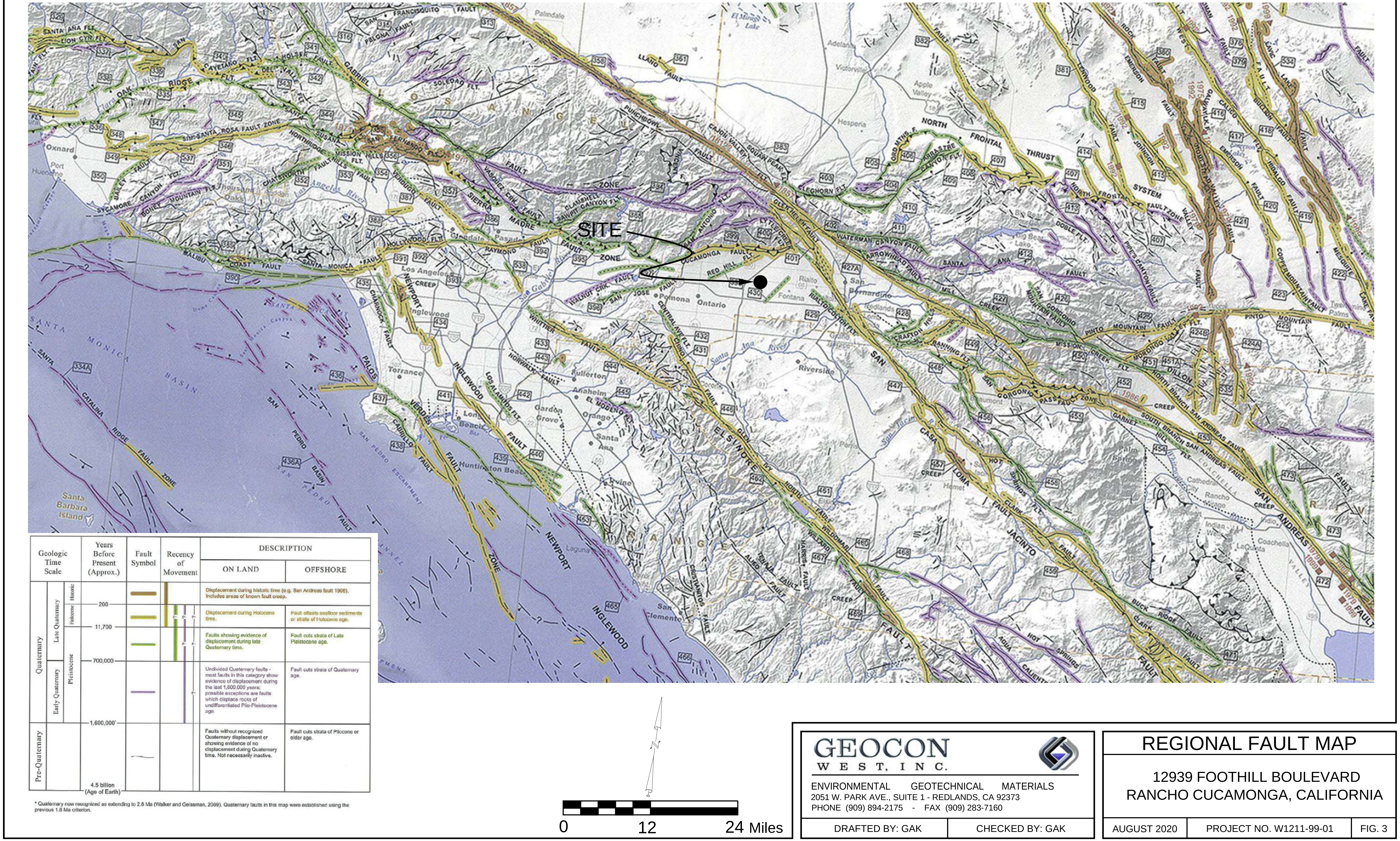
SITE PLAN

SOUTH REGION HIGH SCHOOL NO. 7
LOS ANGELES UNIFIED SCHOOL DISTRICT
6020 MILES AVENUE
HUNTINGTON PARK, CA 90255

JUNE 2020
PROJECT NO. W1169-06-01
FIG. 2

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



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

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

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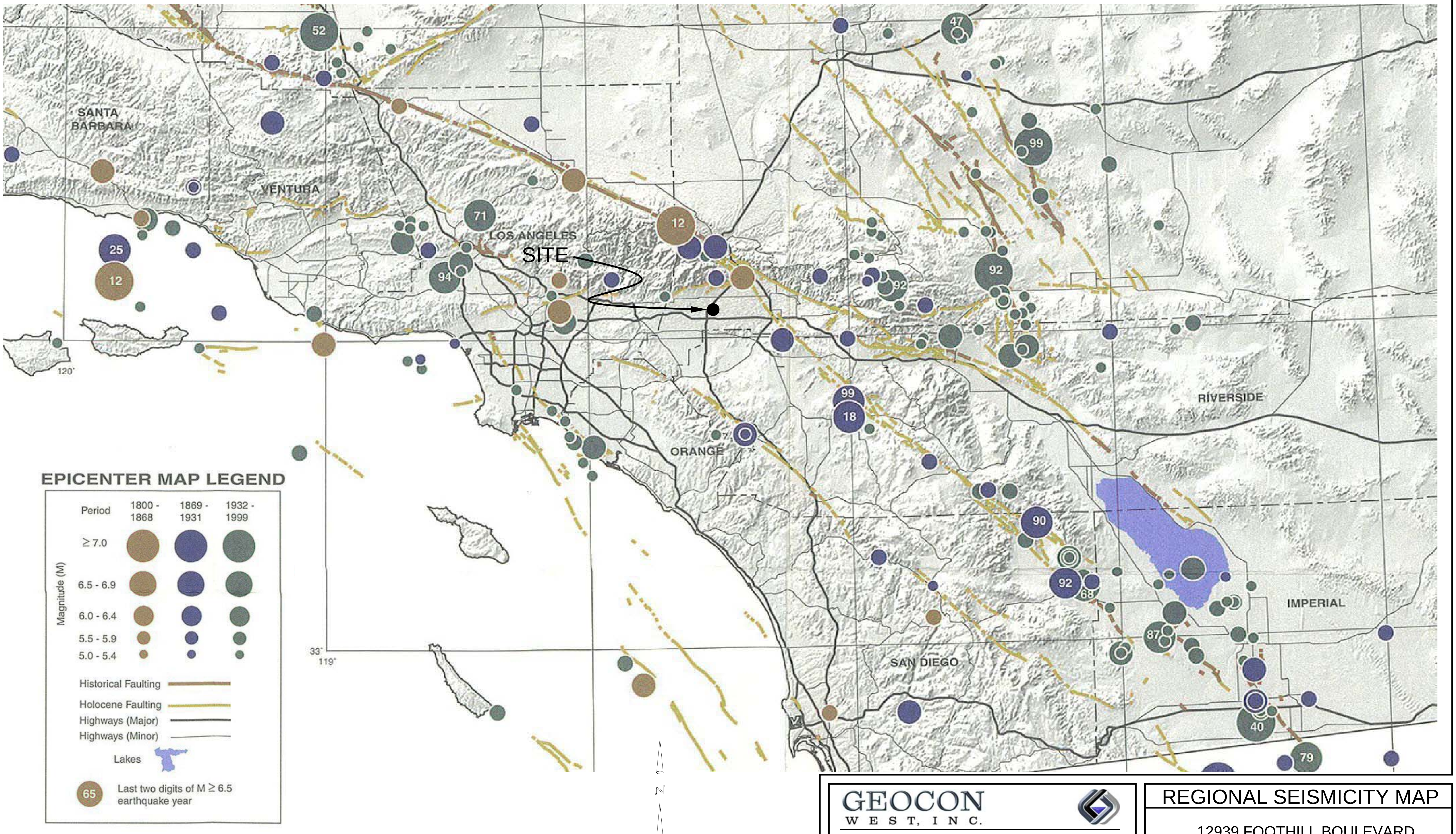
REGIONAL FAULT MAP

12939 FOOTHILL BOULEVARD
RANCHO CUCAMONGA, CALIFORNIA

AUGUST 2020 | PROJECT NO. W1211-99-01 | FIG. 3



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

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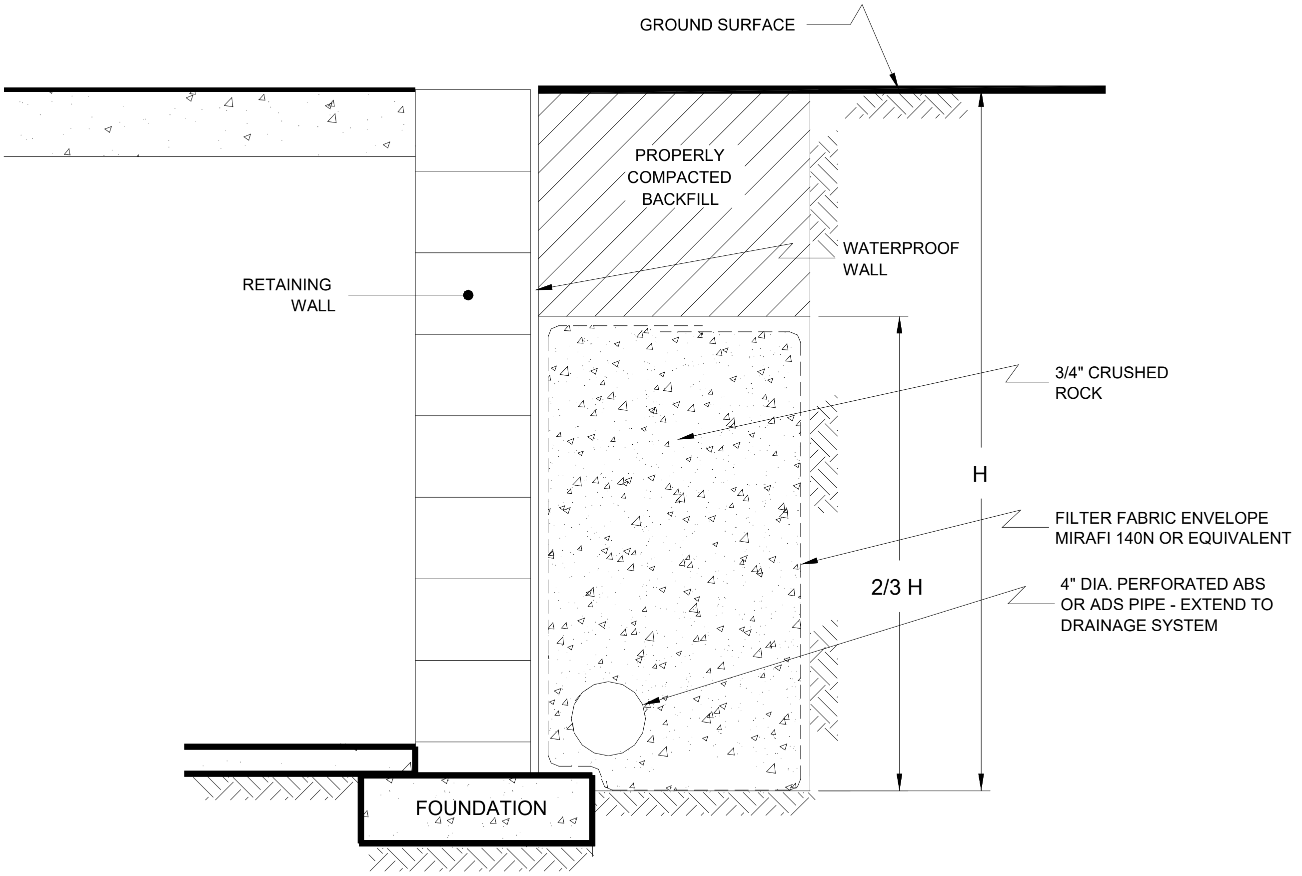
REGIONAL SEISMICITY MAP

12939 FOOTHILL BOULEVARD
RANCHO CUCAMONGA, CALIFORNIA

AUGUST 2020
PROJECT NO. W1211-99-01
FIG.4



NO SCALE

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RETAINING WALL DRAIN DETAIL

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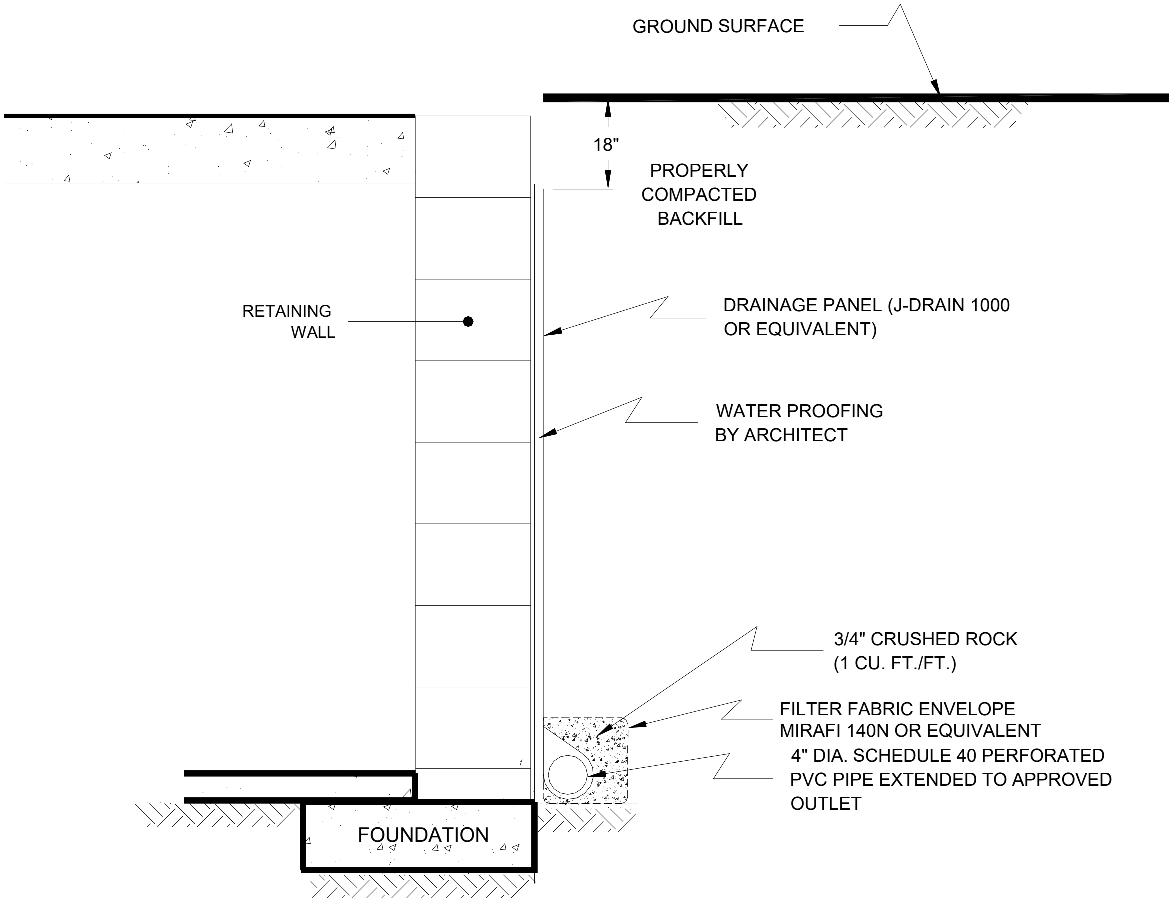
AUGUST 2020 | PROJECT NO. W1211-99-01 | FIG. 6



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RETAINING WALL DRAIN DETAIL

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AUGUST 2020	PROJECT NO. W1211-99-01	FIG. 7

APPENDIX A

APPENDIX A

FIELD INVESTIGATION

The site was explored on August 4 and 5, 2020, by excavating six borings and eleven backhoe pits. The borings were 8-inches in diameter and were drilled to depths between 21½ and 71 feet below the existing ground surface using a truck-mounted hollow-stem auger drilling machine. The backhoe pits were excavated to depths between 5 and 10 feet below the existing ground surface using a rubber-tire backhoe equipped with a 3-foot bucket. Representative and relatively undisturbed samples were obtained by driving a 3-inch O. D., California Modified Sampler into the "undisturbed" soil mass with blows from a 140-pound auto-hammer falling 30 inches. The California Modified Sampler was equipped with 1-inch high by 2³/₈-inch diameter brass rings to facilitate soil removal and testing. Bulk samples were also obtained.

The soil conditions encountered in the borings were visually examined, classified and logged in general accordance with the Unified Soil Classification System (USCS). The logs of the excavations are presented on Figures A1 though A17. The logs depict the soil and geologic conditions encountered and the depth at which samples were obtained. The logs also include our interpretation of the conditions between sampling intervals. Therefore, the logs contain both observed and interpreted data. We determined the lines designating the interface between soil materials on the logs using visual observations, penetration rates, excavation characteristics and other factors. The transition between materials may be abrupt or gradual. Where applicable, the logs were revised based on subsequent laboratory testing. The locations of the excavations are indicated on Figure 2.

PROJECT NO. W1211-99-01

BORING B1

ELEV. (MSL.) **1194** DATE COMPLETED **8/4/2020**

EQUIPMENT **HOLLOW STEM AUGER** BY: **PT**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0					**ARTIFICIAL FILL** Silty Sand, medium dense, dry, yellowish brown, fine- to medium-grained, some coarse-grained, micaceous, grass.			
2	B1@2.5'					32		
4				CL	**ALLUVIAL FAN DEPOSITS (Qal)** Sandy Clay, soft, moist, reddish brown, fine- to medium-grained, micaceous.			
	B1@5'			SM	Silty Sand, medium dense, moist, reddish brown, fine- to medium-grained, micaceous.	11	114.6	16.2
6					- trace coarse-grained and gravel			
8	B1@7.5'					22	123.5	11.1
10	B1@10' BULK 10-15'			SP	Sand, poorly graded, medium dense, moist, reddish brown, fine- to medium-grained, micaceous. - dense, fine- to coarse-grained, gravel	60	107.4	9.3
12				SM	Silty Sand, medium dense, moist, reddish brown, fine- to medium-grained, micaceous.			
14								
	B1@15'				- trace calcuim carbonate development	33	115.9	17.2
16								
18								
20	B1@20'				- very dense, fine- to medium-grained	50 (5")	110.2	13.3
22								
24								
	B1@25'					63	113.4	16.3
26				ML	Sandy Silt, very stiff, moist, reddish brown, fine-grained.			
28								
				SM	Silty Sand, medium dense, moist, reddish brown, fine- to medium-grained,			

Figure A1,
Log of Boring B1, Page 1 of 2

W1211-99-01 REV082120.GPJ

SAMPLE SYMBOLS

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

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

GEOCON

PROJECT NO. W1211-99-01

BORING B1

ELEV. (MSL.) **1194** DATE COMPLETED **8/4/2020**

EQUIPMENT **HOLLOW STEM AUGER** BY: **PT**

DEPTH IN FEET	SAMPLE NO.		LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
30	B1@30'	■			SM	trace coarse-grained.	49	120.2	15.5
						Total depth of boring: 31.5 feet Fill to 3 feet. No groundwater encountered. Backfilled with soil cuttings and tamped. *Penetration resistance for 140-lb hammer falling 30" by auto-hammer.			

Figure A1,
Log of Boring B1, Page 2 of 2

W1211-99-01 REV082120.GPJ

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

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

GEOCON

PROJECT NO. W1211-99-01

BORING B3

ELEV. (MSL.) **1198** DATE COMPLETED **8/4/2020**

EQUIPMENT **HOLLOW STEM AUGER** BY: **PT**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0					**ARTIFICIAL FILL** Silty Sand, medium dense, dry, yellowish brown, fine- to medium-grained, some coarse-grained, micaceous, grass.			
2								
	B3@2.5' ■				**ALLUVIAL FAN DEPOSITS (Qal)** Sand, poorly graded, medium dense, moist, reddish brown, fine- to coarse-grained, trace gravel, micaceous.	27	120.5	8.1
4								
	B3@5' ■					22	101.1	13.8
6								
				SP				
8	B3@7.5' ■					31	111.3	6.8
10	B3@10' ■				- very dense; decrease in silt	50(6")	126.3	3.4
12								
14								
	B3@15' ■				Silty Sand, medium dense, moist, reddish brown, fine- to medium-grained.	44	120.5	8.1
16								
18				SM			123.3	3.9
20	B3@20' ■				- dense, some coarse-grained, trace gravel	68	115.8	12.9

Total depth of boring: 31.5 feet
Fill to 3 feet.
No groundwater encountered.
Backfilled with soil cuttings and tamped.

*Penetration resistance for 140-lb hammer falling 30" by auto-hammer.

W1211-99-01 REV082120.GPJ

Figure A2,
Log of Boring B3, Page 1 of 1

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

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GEOCON

PROJECT NO. W1211-99-01

BORING B4

ELEV. (MSL.) **1196** DATE COMPLETED **8/4/2020**

EQUIPMENT **HOLLOW STEM AUGER** BY: **PT**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0	BULK 0-5'				**ARTIFICIAL FILL** Clayey Sand, medium dense, dry, yellowish brown, fine- to medium-grained, some coarse-grained, micaceous, grass. - moist, reddish brown			
2	B4@2.5'			SC	**ALLUVIAL FAN DEPOSITS (Qal)** Clayey Sand, loose, moist, reddish brown, fine- to medium-grained, trace coarse-grained, micaceous.	11		
4	B4@5'			CL	Sandy Clay, very stiff, moist, reddish brown, fine- to medium-grained, micaceous.	43	116.4	13.8
6								
8	B4@7.5'			SM	Silty Sand, medium dense, moist, reddish brown, fine- to medium-grained, trace clay, micaceous.	21	111.1	18.3
10	B4@10'					26	114.3	8.2
12								
14	B4@15'			SW	Sand, well-graded, medium dense, moist, light brown, fine- to coarse-grained.	45	124.4	5.7
16								
18				SM	Silty Sand, dense, moist, reddish brown, fine- to medium-grained, some coarse-grained.			
20	B4@20'					66	122.8	10.5

Total depth of boring: 21.5 feet
Fill to 2 feet.
No groundwater encountered.
Backfilled with soil cuttings and tamped.

*Penetration resistance for 140-lb hammer falling 30" by auto-hammer.

Figure A3,
Log of Boring B4, Page 1 of 1

W1211-99-01 REV082120.GPJ

SAMPLE SYMBOLS

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

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GEOCON

PROJECT NO. W1211-99-01

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	BORING B5 ELEV. (MSL.) **1194** DATE COMPLETED **8/4/2020** EQUIPMENT **HOLLOW STEM AUGER** BY: **PT**	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
					MATERIAL DESCRIPTION			
0	B5@0-5'				**ARTIFICIAL FILL** Silty Sand, loose, dry, yellowish brown, fine- to medium-grained, some coarse-grained, micaceous, grass.			
2	B5@2.5'			SM	**ALLUVIAL FAN DEPOSITS (Qal)** Silty Sand, loose, moist, reddish brown, fine- to medium-grained, micaceous.	9	104.8	16.8
4	B5@5'				- brown	8	101.9	14.7
6								
8	B5@7.5'			SC	Clayey Sand, medium dense, moist, brown, fine- to medium-grained, some coarse-grained.	46	125.9	11.4
10	B5@10'			SM	Silty Sand, medium dense, moist, reddish brown, fine- to medium-grained, micaceous, trace gravel.	42	110.9	10.0
12								
14								
16	B5@15'				- reddish yellowish brown	39	117.6	11.3
18								
20	B5@20'				- trace clay	21		
22								
24	B5@25'			SP	Sand, poorly graded, very dense, moist, light yellowish brown, fine- to medium-grained, some silt.	50 (6")	88.8	7.8
26								
28				SM	Silty Sand, dense, moist, yellowish brown, fine- to coarse-grained, trace gravel, micaceous.			

Figure A4,
Log of Boring B5, Page 1 of 3

W1211-99-01 REV082120.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.

PROJECT NO. W1211-99-01

BORING B5

ELEV. (MSL.) **1194** DATE COMPLETED **8/4/2020**

EQUIPMENT **HOLLOW STEM AUGER** BY: **PT**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
30	B5@30'			SM		50		
35	B5@35'				- very dense	50 (5")	124.8	6.8
38				SP	Sand, poorly graded, very dense, moist, yellowish brown, medium- to coarse-grained, trace gravel.			
40	B5@40'					59		
42				SM	Silty Sand, very dense, moist, reddish brown, fine- to medium-grained, some coarse-grained, micaceous.			
45	B5@45'					79	128.5	11.9
50	B5@50'				- medium dense	21		
55	B5@55'				- very dense, some silt, trace coarse-grained	50 (4")	109.3	8.6

Figure A4,
Log of Boring B5, Page 2 of 3

W1211-99-01 REV082120.GPJ

SAMPLE SYMBOLS

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

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GEOCON

PROJECT NO. W1211-99-01

BORING B5

ELEV. (MSL.) **1194** DATE COMPLETED **8/4/2020**

EQUIPMENT **HOLLOW STEM AUGER** BY: **PT**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
60	B5@60' ■			SM		40		
				SW	Sand, poorly graded, dense, moist, grayish brown, fine- to coarse-grained, trave gravel.			
62								
64					Silty Sand, very dense, moist, reddish brown; fine- to medium-grained, trace coarse-grained, micaceous.			
	B5@65' ■					96	122.7	7.2
66				SM				
68								
70	B5@70' ■					50 (5")		

Total depth of boring: 71 feet
Fill to 2 feet.
No groundwater encountered.
Backfilled with soil cuttings and tamped.

*Penetration resistance for 140-lb hammer falling 30" by auto-hammer.

Figure A4,
Log of Boring B5, Page 3 of 3

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

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GEOCON

PROJECT NO. W1211-99-01

BORING B6

ELEV. (MSL.) **1191** DATE COMPLETED **8/4/2020**

EQUIPMENT **HOLLOW STEM AUGER** BY: **PT**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0					**ARTIFICIAL FILL** Silty Sand, medium dense, dry, yellowish brown, fine- to medium-grained, some coarse-grained, micaceous, grass.			
2 4	B6@2.5' ■			SM	**ALLUVIAL FAN DEPOSITS (Qal)** Silty Sand, medium dense, moist, dark reddish brown, fine- to medium-grained, trace coarse-grained, micaeous.	14	117.9	9.7
6	B6@5' ■			CL	Sandy Clay, stiff, moist, reddish brown; fine- to medium-grained, micaceous.	23	108.9	14.4
8	B6@7.5' ■			SM	Silty Sand, medium dense, moist, reddish brown, fine- to medium-grained.	22	117.4	12.8
10 12	B6@10' ■			ML	Sandy Silt, stiff, moist, reddish brown, fine-grained, trace medium-grained.	35	114.2	16.9
14 16 18 20	B6@15' ■ B6@20' ■			SM	Silty Sand, dense, moist, reddish brown, fine- to medium-grained, micaceous.	51 59	122.2 123.5	11.2 10.9

Total depth of boring: 21.5 feet
Fill to 2 feet.
No groundwater encountered.
Backfilled with soil cuttings and tamped.

*Penetration resistance for 140-lb hammer falling 30" by auto-hammer.

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Figure A5,
Log of Boring B6, Page 1 of 1

SAMPLE SYMBOLS

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

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GEOCON

BORING B8

ELEV. (MSL.) **1195** DATE COMPLETED **8/4/2020**

EQUIPMENT **HOLLOW STEM AUGER** BY: **PT**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0					**ARTIFICIAL FILL** Silty Sand, medium dense, dry, yellowish brown; fine- to medium-grained, some coarse-grained, micaceous, grass.			
2					**ALLUVIAL FAN DEPOSITS (Qal)** Silty Sand, medium dense, moist, reddish brown, fine- to medium-grained, trace coarse-grained, micaceous.			
	B8@2.5'	■				16	119.2	11.4
4								
	B8@5' BULK 5-10'	■ ▨			- trace gravel	26	105.3	11.7
6								
					- decrease in silt, increase in fine-grained			
	B8@7.5'	■				9	103.3	8.6
8								
10	B8@10'	■				35	115.6	10.6
12								
14				SM				
	B8@15'	■			- dense	53	125.2	7.8
16								
18								
20	B8@20'	■				79	122.1	7.7
22								
24					- very dense, yellowish brown, fine- to coarse-grained			
	B8@25'	■				50 (5")	113.8	4.3
26								
28								

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Figure A6,
Log of Boring B8, Page 1 of 2

SAMPLE SYMBOLS

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

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BORING B8

ELEV. (MSL.) 1195 DATE COMPLETED 8/4/2020

EQUIPMENT HOLLOW STEM AUGER BY: PT

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
30	B8@30' ■			SM	- dense, reddish brown, fine- to medium-grained	50	109.6	4.6
					Total depth of boring: 31.5 feet Fill to 1 foot. No groundwater encountered. Backfilled with soil cuttings and tamped. *Penetration resistance for 140-lb hammer falling 30" by auto-hammer.			

Figure A6,
Log of Boring B8, Page 2 of 2

W1211-99-01 REV082120.GPJ

SAMPLE SYMBOLS

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

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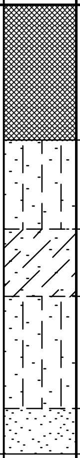
PROJECT NO. W1211-99-01

TEST PIT TP1

ELEV. (MSL.) **1193** DATE COMPLETED **8/5/2020**

EQUIPMENT **BACKHOE** BY: **PT**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0 2					**ARTIFICIAL FILL** Silty Sand, medium dense, dry, yellowish brown, fine- to medium-grained, some coarse-grained, micaceous, grass.			
4				SM	**ALLUVIAL FAN DEPOSITS (Qal)** Silty Sand, medium dense, moist, reddish brown, fine- to coarse-grained, micaceous.			
6				SC	Clayey Sand, medium dense, moist, reddish brown, fine- to medium-grained.			
8				SM	Silty Sand, medium dense, moist, reddish brown, fine- to medium-grained, some coarse-grained, trace gravel.			
10				SW	Sand, well-graded, medium dense, moist, light reddish brown, fine- to coarse-grained, some gravel.			

Total depth of boring: 10 feet
Fill to 3 feet.
No groundwater encountered.
Backfilled with soil cuttings and tamped.

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Figure A7,
Log of Test Pit TP1, Page 1 of 1

SAMPLE SYMBOLS

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

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

GEOCON

PROJECT NO. W1211-99-01

TEST PIT TP2

ELEV. (MSL.) 1195 DATE COMPLETED 8/4/2020

EQUIPMENT BACKHOE BY: PT

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0					**ARTIFICIAL FILL** Silty Sand, medium dense, dry, yellowish brown fine- to medium-grained, some coarse-grained, micaceous, grass.			
2				SM	**ALLUVIAL FAN DEPOSITS (Qal)** Silty Sand, medium dense, moist, reddish brown, fine- to medium-grained, micaceous.			
4					Sand, poorly graded, medium dense, moist, reddish brown, medium- to coarse-grained, some gravel, trace cobbles.			
6								
8				SP				
	TP2@9' ■							
10					Total depth of boring: 10 feet Fill to 1 foot. No groundwater encountered. Backfilled with soil cuttings and tamped.			

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Figure A8,
Log of Test Pit TP2, Page 1 of 1

SAMPLE SYMBOLS

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

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

GEOCON

PROJECT NO. W1211-99-01

TEST PIT TP3

ELEV. (MSL.) **1195** DATE COMPLETED **8/4/2020**

EQUIPMENT **BACKHOE** BY: **PT**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0 2					**ARTIFICIAL FILL** Silty Sand, medium dense, dry, yellowish brown, fine- to medium-grained, some coarse-grained, micaceous, grass. - moist, brown			
4 6 8				SM	**ALLUVIAL FAN DEPOSITS (Qal)** Silty Sand, medium dense, moist, reddish brown, fine- to medium-grained, some coarse-grained, trace gravel, micaceous. - some porosity (to 1/16")			
10				SP	Sand, poorly graded, medium dense, moist, light reddish brown; medium- to coarse-grained, some gravel, trace cobbles.			

Total depth of boring: 10 feet
Fill to 3 feet.
No groundwater encountered.
Backfilled with soil cuttings and tamped.

Figure A9, Log of Test Pit TP3, Page 1 of 1

W1211-99-01 REV082120.GPJ

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

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GEOCON

PROJECT NO. W1211-99-01

TEST PIT TP4

ELEV. (MSL.) **1196** DATE COMPLETED **8/4/2020**

EQUIPMENT **BACKHOE** BY: **PT**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0					**ARTIFICIAL FILL** Silty Sand, medium dense, dry, yellowish brown; fine- to medium-grained, some coarse-grained, micaceous, grass.			
2 4				SM	**ALLUVIAL FAN DEPOSITS (Qal)** Silty Sand, medium dense, moist, brown, fine- to medium-grained, some coarse-grained, trace gravel, micaceous. - reddish brown, some clay - trace porosity (to 1/16"), no clay			

Total depth of boring: 5 feet
Fill to 1 foot.
No groundwater encountered.
Backfilled with soil cuttings and tamped.

Figure A10,
Log of Test Pit TP4, Page 1 of 1

W1211-99-01 REV082120.GPJ

SAMPLE SYMBOLS

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

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

GEOCON

PROJECT NO. W1211-99-01

TEST PIT TP5

ELEV. (MSL.) **1193** DATE COMPLETED **8/4/2020**

EQUIPMENT **BACKHOE** BY: **PT**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0		▨			**ARTIFICIAL FILL** Silty Sand, medium dense, dry, brown, fine- to medium-grained, some coarse-grained, micaceous. grass.			
	TP5@1' ■				**ALLUVIAL FAN DEPOSITS (Qal)** Silty Sand, medium dense, moist, reddish brown, fine- to medium-grained, micaceous.			
2								
4					- brown, some coarse-grained			
				SM	- trace clay			
6					- no clay			
8								
					- light reddish brown, trace coarse-grained			
10					Total depth of boring: 10 feet Fill to 1 foot. No groundwater encountered. Backfilled with soil cuttings and tamped.			

Figure A11,
Log of Test Pit TP5, Page 1 of 1

W1211-99-01 REV082120.GPJ

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

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

GEOCON

PROJECT NO. W1211-99-01

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	**TEST PIT TP6** ELEV. (MSL.) **1197** DATE COMPLETED **8/4/2020** EQUIPMENT **BACKHOE** BY: **PT**	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
					MATERIAL DESCRIPTION			
0		▨			**ARTIFICIAL FILL** Silty Sand, medium dense, dry, yellowish brown, fine- to medium-grained, some coarse-grained, micaceous, grass.			
2				SM	**ALLUVIAL FAN DEPOSITS (Qal)** Silty Sand, medium dense, moist, reddish brown, fine- to medium-grained, micaceous.			
4					- trace pinhole porosity, root hairs			

Total depth of boring: 5 feet
Fill to 0.5 foot.
No groundwater encountered.
Backfilled with soil cuttings and tamped.

W1211-99-01 REV082120.GPJ

Figure A12,
Log of Test Pit TP6, Page 1 of 1

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

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

PROJECT NO. W1211-99-01

TEST PIT TP7

ELEV. (MSL.) **1192** DATE COMPLETED **8/4/2020**

EQUIPMENT **BACKHOE** BY: **PT**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0 2		▩			**ARTIFICIAL FILL** Silty Sand, medium dense, dry, yellowish brown, fine- to medium-grained, some coarse-grained, micaceous, grass. - moist, reddish brown - dark brown			
4 6 8		┆		SM	**ALLUVIAL FAN DEPOSITS (Qal)** Silty Sand, medium dense, moist, reddish brown, fine- to medium-grained, micaceous, trace porosity (to 1/16"). - trace gravel, decrease in porosity			
10					Total depth of boring: 10 feet Fill to 3 feet. No groundwater encountered. Backfilled with soil cuttings and tamped.			

W1211-99-01 REV082120.GPJ

Figure A13,
Log of Test Pit TP7, Page 1 of 1

SAMPLE SYMBOLS

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

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

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PROJECT NO. W1211-99-01

TEST PIT TP8

ELEV. (MSL.) **1192** DATE COMPLETED **8/4/2020**

EQUIPMENT **BACKHOE** BY: **PT**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0 2		▩			**ARTIFICIAL FILL** Silty Sand, medium dense, dry, yellowish brown, fine- to medium-grained, some coarse-grained sand micaceous, grass. - moist, dark brown			
4		▤		SM	**ALLUVIAL FAN DEPOSITS (Qal)** Silty Sand, medium dense, moist, reddish brown, fine- to medium-grained, trace coarse-grained and clay, micaceous.			
					Total depth of boring: 5 feet Fill to 3 feet. No groundwater encountered. Backfilled with soil cuttings and tamped.			

Figure A14,
Log of Test Pit TP8, Page 1 of 1

W1211-99-01 REV082120.GPJ

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

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

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PROJECT NO. W1211-99-01

TEST PIT TP9

ELEV. (MSL.) **1194** DATE COMPLETED **8/4/2020**

EQUIPMENT **BACKHOE** BY: **PT**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0					**ARTIFICIAL FILL** Silty Sand, medium dense, dry, yellowish brown, fine- to medium-grained, some coarse-grained, micaceous, grass. - moist, brown			
2 4				SM	**ALLUVUIAL FAN DEPOSITS (Qal)** Silty Sand, medium dense, moist, reddish brown, fine- to medium-grained, micaceous. - trace pinhole porosity			

Total depth of boring: 5 feet
Fill to 1.5 feet.
No groundwater encountered.
Backfilled with soil cuttings and tamped.

W1211-99-01 REV082120.GPJ

Figure A15,
Log of Test Pit TP9, Page 1 of 1

SAMPLE SYMBOLS

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

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

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PROJECT NO. W1211-99-01

TEST PIT TP10

ELEV. (MSL.) **1193** DATE COMPLETED **8/4/2020**

EQUIPMENT **BACKHOE** BY: **PT**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0		▨			**ARTIFICIAL FILL** Silty Sand, medium dense, dry, yellowish brown, fine- to medium-grained, some coarse-grained, micaceous, grass.			
2					**ALLUVIAL FAN DEPOSITS (Qal)** Silty Sand, medium dense, moist, reddish brown, fine- to coarse-grained, trace cobbles, micaceous.			
4					- some gravel, reddish yellowish brown			
6				SM				
8	TP10@7' ■				- trace cobbles			
10								

Total depth of boring: 10 feet
Fill to 1 foot.
No groundwater encountered.
Backfilled with soil cuttings and tamped.

Figure A16,
Log of Test Pit TP10, Page 1 of 1

W1211-99-01 REV082120.GPJ

SAMPLE SYMBOLS

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

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

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PROJECT NO. W1211-99-01

TEST PIT TP11

ELEV. (MSL.) **1195** DATE COMPLETED **8/4/2020**

EQUIPMENT **BACKHOE** BY: **PT**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0					**ARTIFICIAL FILL** Silty Sand, medium dense, dry, yellowish brown, fine- to medium-grained, some coarse-grained, micaceous, grass.			
2					- moist, brown			
4				SM	**ALLUVIAL FAN DEPOSITS (Qal)** Silty Sand, medium dense, moist, reddish brown, fine- to coarse-grained, micaceous.			
6				SC	Clayey Sand, medium dense, moist, reddish brown, fine- to medium-grained, trace coars-grained, micaceous.			
8				SM	Silty Sand, medium dense, moist, reddish brown, fine- to medium-grained, trace pinhole porosity.			
	TP11@9' ■				- trace coarse-grained, root hairs			
10					Total depth of boring: 10 feet Fill to 3 feet. No groundwater encountered. Backfilled with soil cuttings and tamped.			

Figure A17,
Log of Test Pit TP11, Page 1 of 1

W1211-99-01 REV082120.GPJ

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

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

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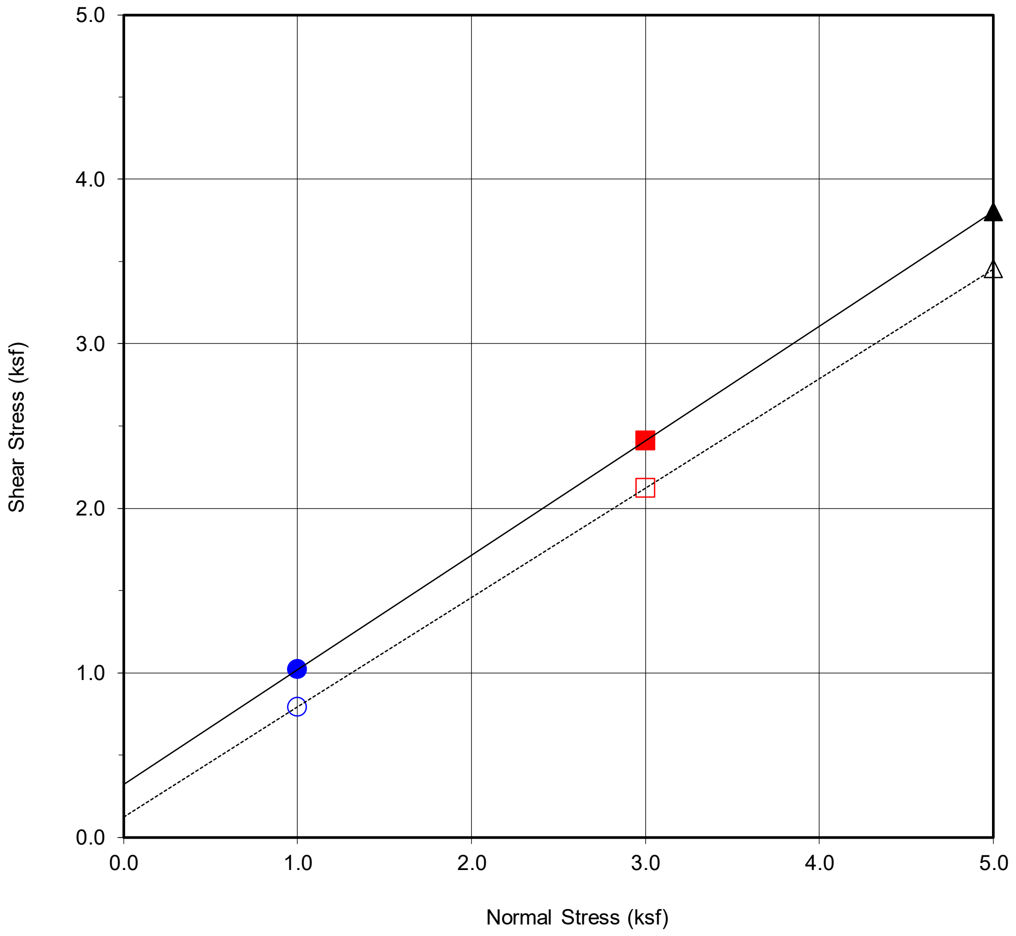
APPENDIX B

APPENDIX B

LABORATORY TESTING

Laboratory tests were performed in accordance with generally accepted test methods of the "American Society for Testing and Materials (ASTM)", or other suggested procedures. Selected samples were tested for direct shear strength, moisture density relationship, corrosivity and in-place dry density and moisture content. The results of the laboratory tests are summarized in Figures B1 through B31. The in-place dry density and moisture content of the samples tested are presented on the boring logs, Appendix A.



Boring No.	**B1**
Sample No.	**B1@5'**
Depth (ft)	**5**
Sample Type:	Ring

Normal Stress (kip/ft2)	1	3	5
Peak Shear Stress (kip/ft²)	● 1.02	■ 2.41	▲ 3.80
Shear Stress @ End of Test (ksf)	○ 0.79	□ 2.12	△ 3.46
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 (%)	14.3	15.6	16.2
Initial Dry Density (pcf)	116.2	114.8	115.8
Initial Degree of Saturation (%)	85.7	90.1	96.2
Soil Height Before Shearing (in.)	1.2	1.2	1.2
Final Moisture Content (%)	19.9	21.1	21.1

Soil Identification:

Reddish Brown Silty Sand (SM)

Strength Parameters

	C (psf)	ϕ (^{o})
Peak	323	34.8
Ultimate	126	33.7

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DIRECT SHEAR TEST RESULTS

Consolidated Drained ASTM D-3080

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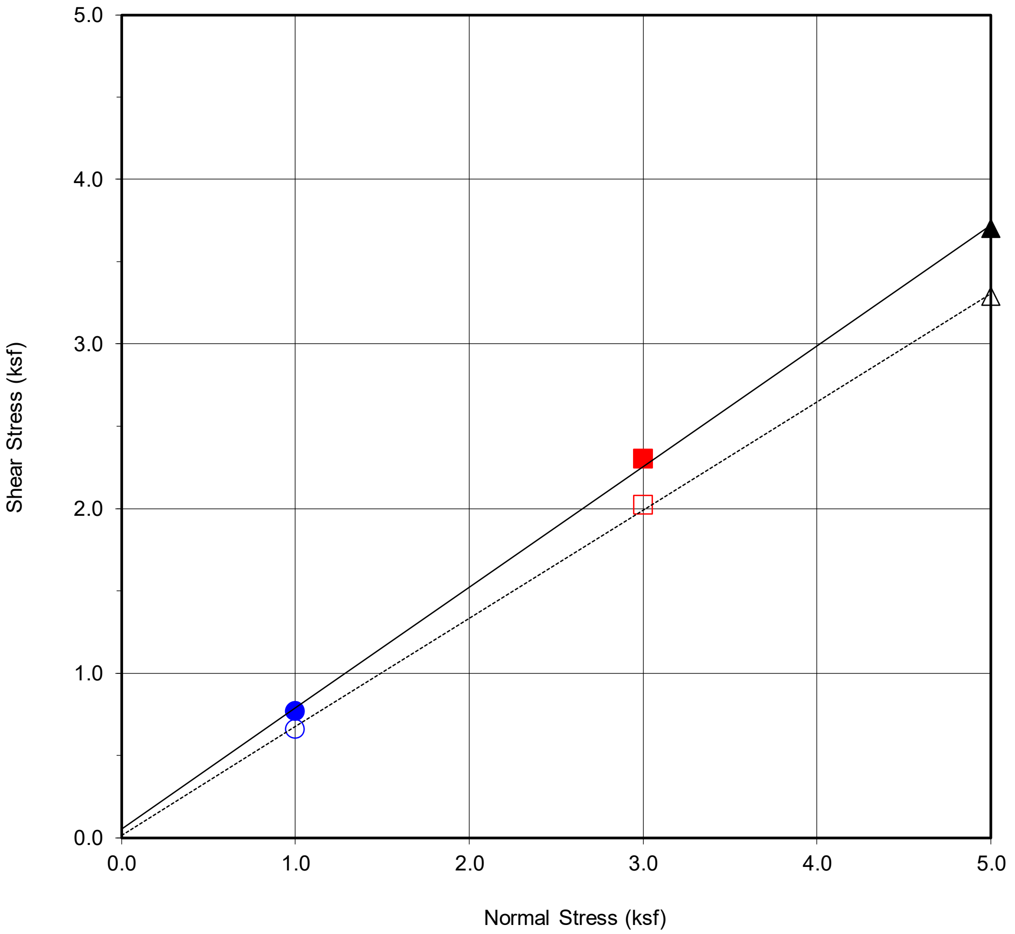
Figure B1



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

Soil Identification:		
Dark Yellowish Brown Poorly Graded Sand (SP)		
Strength Parameters		
	C (psf)	ϕ (°)
Peak	57	36.2
Ultimate	17	33.3

Normal Stress (kip/ft2)	1	3	5
Peak Shear Stress (kip/ft²)	● 0.77	■ 2.30	▲ 3.70
Shear Stress @ End of Test (ksf)	○ 0.66	□ 2.02	△ 3.29
Deformation Rate (in./min.)	0.05	0.05	0.05
Initial Sample Height (in.)	1.0	1.0	1.0
Ring Inside Diameter (in.)	2.375	2.375	2.375
Initial Moisture Content (%)	9.3	10.6	12.1
Initial Dry Density (pcf)	111.8	105.7	100.2
Initial Degree of Saturation (%)	49.3	48.1	47.8
Soil Height Before Shearing (in.)	1.2	1.2	1.2
Final Moisture Content (%)	16.7	15.4	23.1

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DIRECT SHEAR TEST RESULTS

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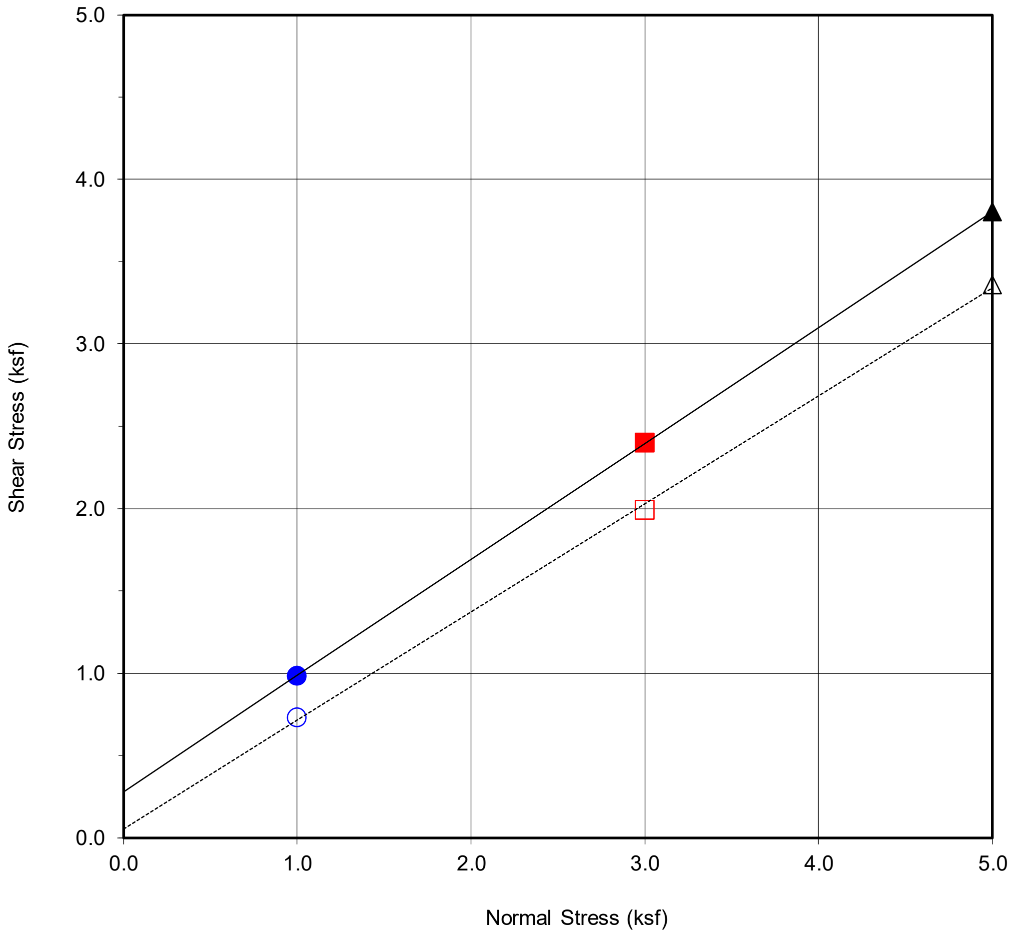
Figure B2



Boring No.	**B1**
Sample No.	**B1@20'**
Depth (ft)	**20**
Sample Type:	Ring

Normal Stress (kip/ft2)	1	3	5
Peak Shear Stress (kip/ft²)	● 0.98	■ 2.40	▲ 3.80
Shear Stress @ End of Test (ksf)	○ 0.73	□ 1.99	△ 3.36
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 (%)	11.3	13.1	13.3
Initial Dry Density (pcf)	111.7	110.8	110.9
Initial Degree of Saturation (%)	60.0	67.6	68.8
Soil Height Before Shearing (in.)	1.2	1.2	1.2
Final Moisture Content (%)	26.2	19.2	19.7

Soil Identification:

Grayish Brown Silty Sand (SM)

Strength Parameters

	C (psf)	ϕ (°)
Peak	281	35.2
Ultimate	56	33.3

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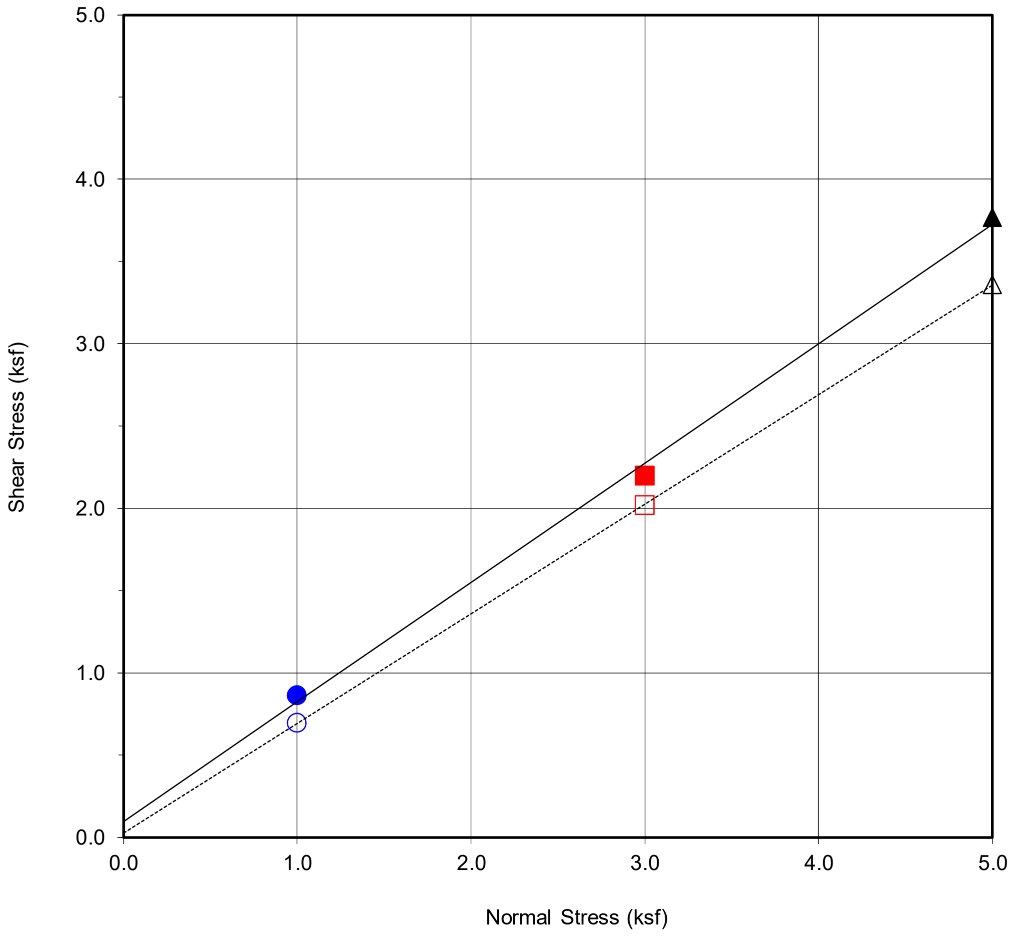
Figure B3



Boring No.	**B3**
Sample No.	**B3@5'**
Depth (ft)	**5**
Sample Type:	Ring

Normal Stress (kip/ft2)	1	3	5
Peak Shear Stress (kip/ft²)	● 0.86	■ 2.20	▲ 3.77
Shear Stress @ End of Test (ksf)	○ 0.70	□ 2.02	△ 3.36
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 (%)	10.6	11.2	13.8
Initial Dry Density (pcf)	100.4	97.8	105.4
Initial Degree of Saturation (%)	42.1	41.9	62.4
Soil Height Before Shearing (in.)	1.2	1.2	1.2
Final Moisture Content (%)	20.0	25.9	18.2

Soil Identification:

Yellowish Brown Poorly Graded Sand (SP)

Strength Parameters

	C (psf)	ϕ (°)
Peak	98	36.0
Ultimate	26	33.7

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DIRECT SHEAR TEST RESULTS

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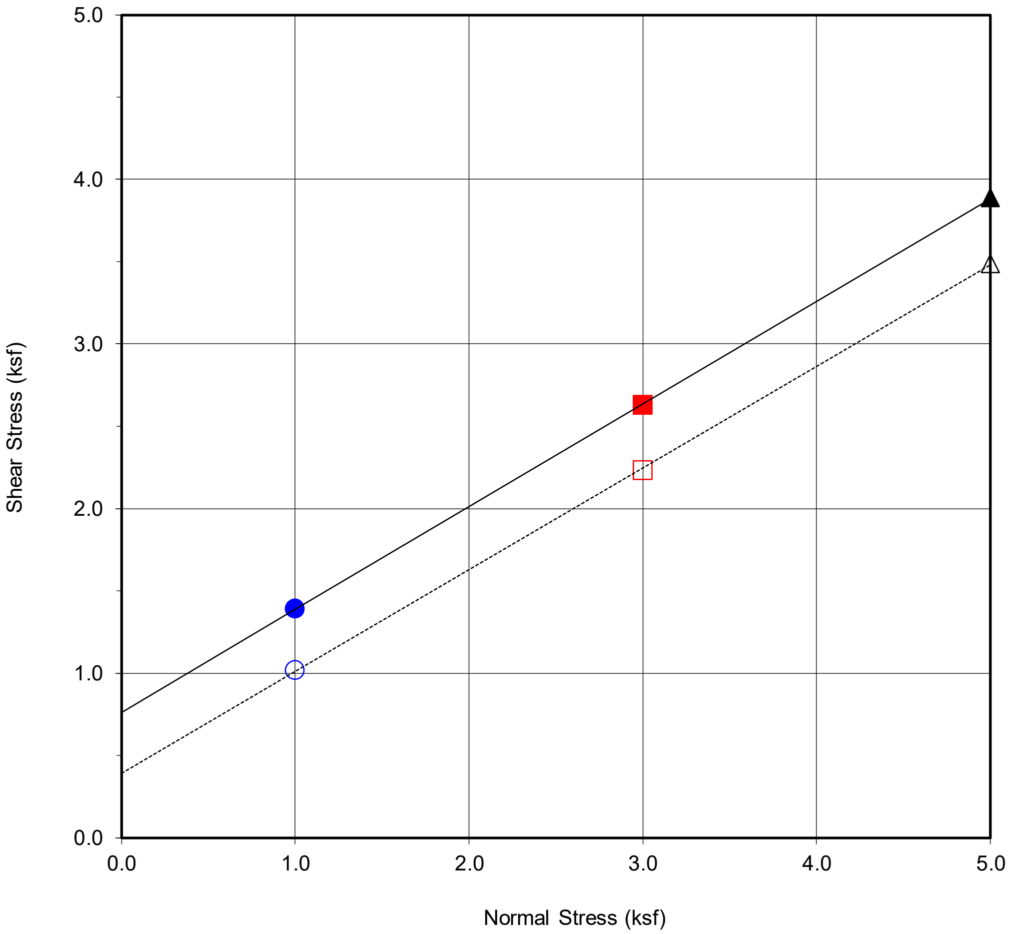
Figure B4



Boring No.	**B4**
Sample No.	**B4@5'**
Depth (ft)	**5**
Sample Type:	Ring

Soil Identification:

Brown Clay (CL)

Strength Parameters

	C (psf)	ϕ (°)
Peak	764	32.0
Ultimate	392	31.7

Normal Stress (kip/ft2)	1	3	5
Peak Shear Stress (kip/ft²)	● 1.39	■ 2.63	▲ 3.89
Shear Stress @ End of Test (ksf)	○ 1.02	□ 2.23	△ 3.49
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 (%)	13.8	13.2	16.2
Initial Dry Density (pcf)	117.4	112.5	114.1
Initial Degree of Saturation (%)	85.5	71.4	91.7
Soil Height Before Shearing (in.)	1.2	1.2	1.2
Final Moisture Content (%)	16.4	15.7	17.1

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DIRECT SHEAR TEST RESULTS

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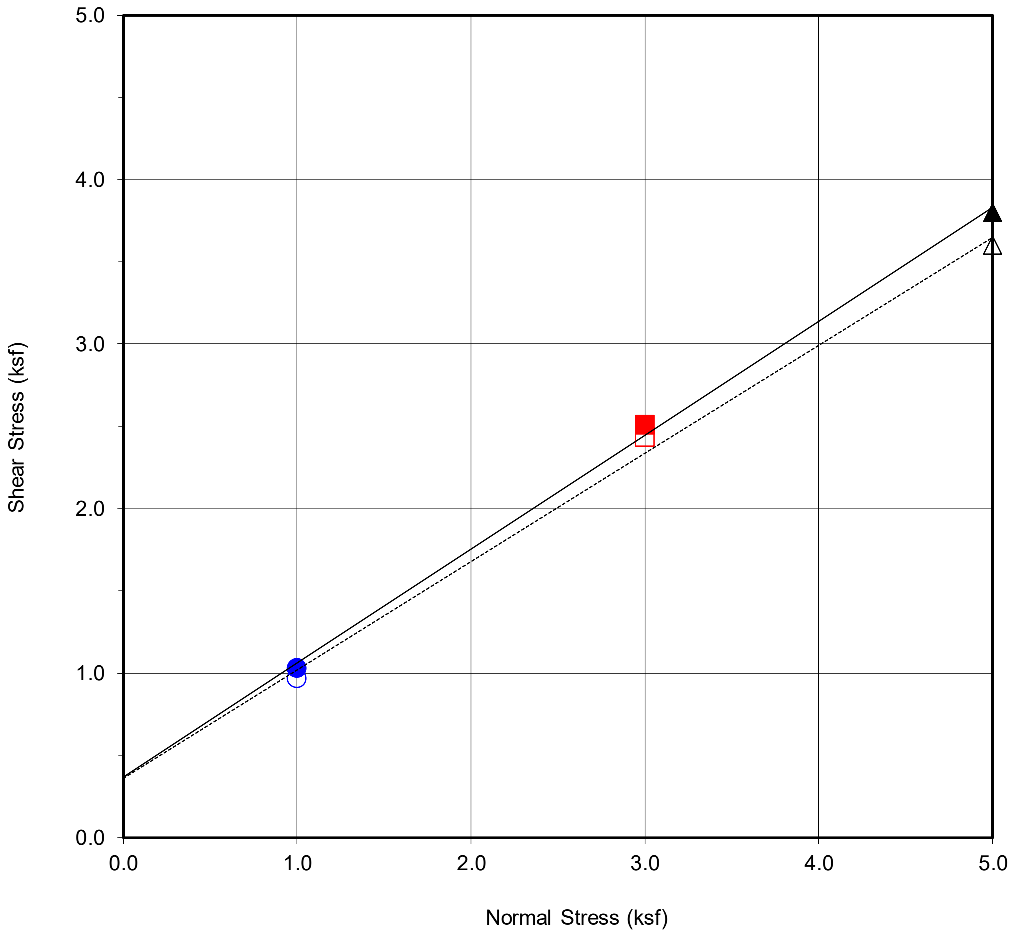
Figure B5



Boring No.	**B5**
Sample No.	**B5@2.5'**
Depth (ft)	**2.5**
<u>Sample Type:</u>	Ring

<u>Soil Identification:</u>		
Brown Silty Sand (SM)		
<u>Strength Parameters</u>		
	C (psf)	ϕ (^{o})
Peak	370	34.7
Ultimate	361	33.3

Normal Stress (kip/ft2)	1	3	5
Peak Shear Stress (kip/ft²)	● 1.03	■ 2.51	▲ 3.80
Shear Stress @ End of Test (ksf)	○ 0.97	□ 2.43	△ 3.60
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	11.1	16.6
Initial Dry Density (pcf)	106.1	107.8	102.3
Initial Degree of Saturation (%)	76.8	53.3	69.1
Soil Height Before Shearing (in.)	1.2	1.2	1.2
Final Moisture Content (%)	17.1	16.5	16.8

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DIRECT SHEAR TEST RESULTS

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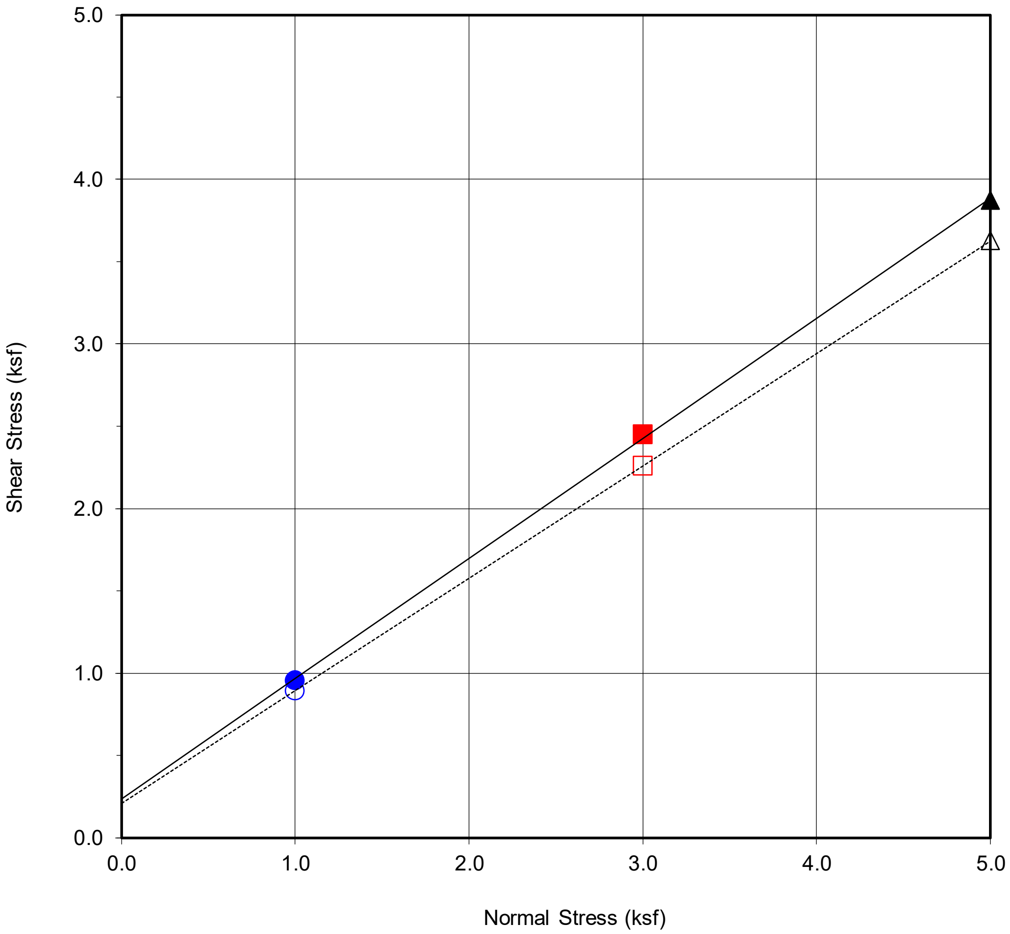
Figure B6



Boring No.	**B5**
Sample No.	**B5@10'**
Depth (ft)	**10**
Sample Type:	Ring

Soil Identification:		
Brown Silty Sand (SM)		
Strength Parameters		
	C (psf)	ϕ (^{o})
Peak	238	36.1
Ultimate	211	34.3

Normal Stress (kip/ft2)	1	3	5
Peak Shear Stress (kip/ft²)	● 0.96	■ 2.45	▲ 3.87
Shear Stress @ End of Test (ksf)	○ 0.89	□ 2.26	△ 3.63
Deformation Rate (in./min.)	0.05	0.05	0.05
Initial Sample Height (in.)	1.0	1.0	1.0
Ring Inside Diameter (in.)	2.375	2.375	2.375
Initial Moisture Content (%)	10.2	10.0	9.7
Initial Dry Density (pcf)	105.9	114.8	108.0
Initial Degree of Saturation (%)	46.8	57.5	46.7
Soil Height Before Shearing (in.)	1.2	1.2	1.2
Final Moisture Content (%)	17.1	15.4	15.4

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DIRECT SHEAR TEST RESULTS

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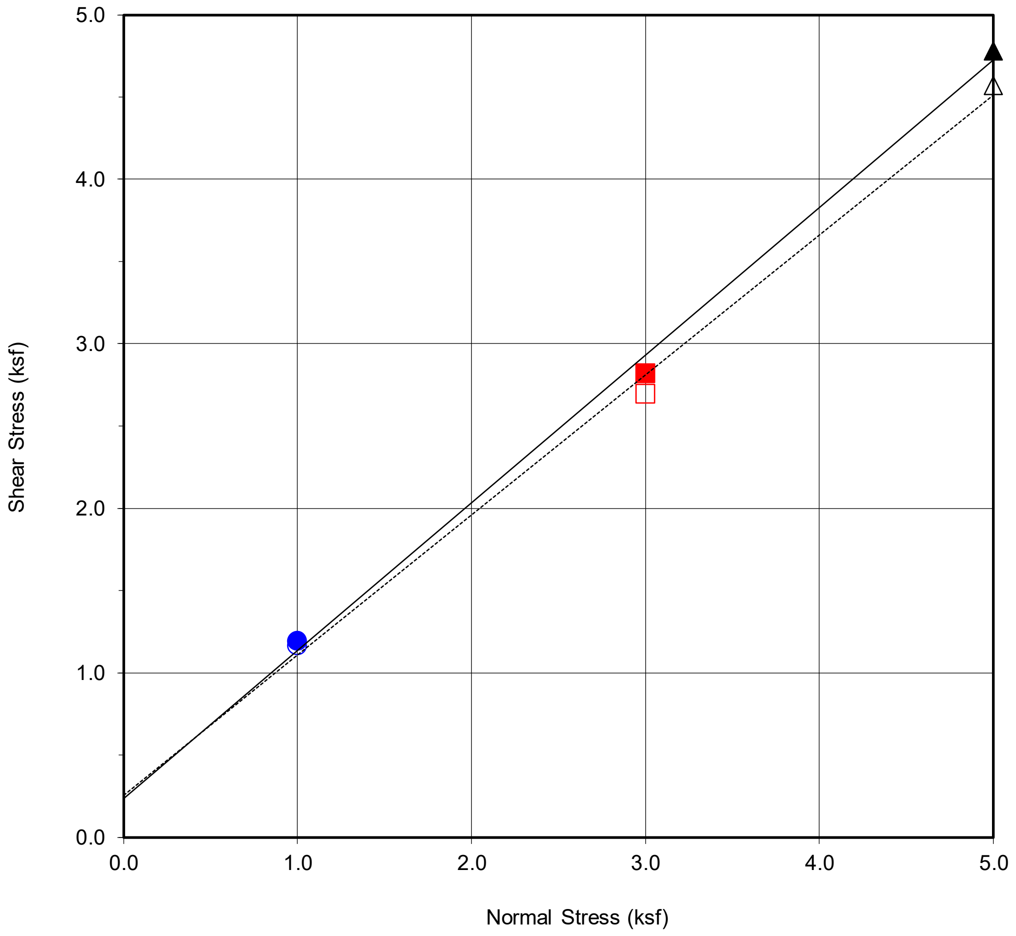

Boring No.	**B5**
Sample No.	**B5@25'**
Depth (ft)	**25'**
Sample Type:	Ring

Soil Identification:
Sand with Some Silt and Trace Gravel (SM)

Strength Parameters		
	C (psf)	ϕ (°)
Peak	239	41.9
Ultimate	258	40.4

Normal Stress (kip/ft2)	1	3	5
Peak Shear Stress (kip/ft²)	● 1.19	■ 2.82	▲ 4.78
Shear Stress @ End of Test (ksf)	○ 1.17	□ 2.70	△ 4.57
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 (%)	7.3	6.3	7.8
Initial Dry Density (pcf)	88.4	89.5	89.7
Initial Degree of Saturation (%)	21.8	19.1	24.1
Soil Height Before Shearing (in.)	1.2	1.2	1.2
Final Moisture Content (%)	17.4	15.7	16.9

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DIRECT SHEAR TEST RESULTS

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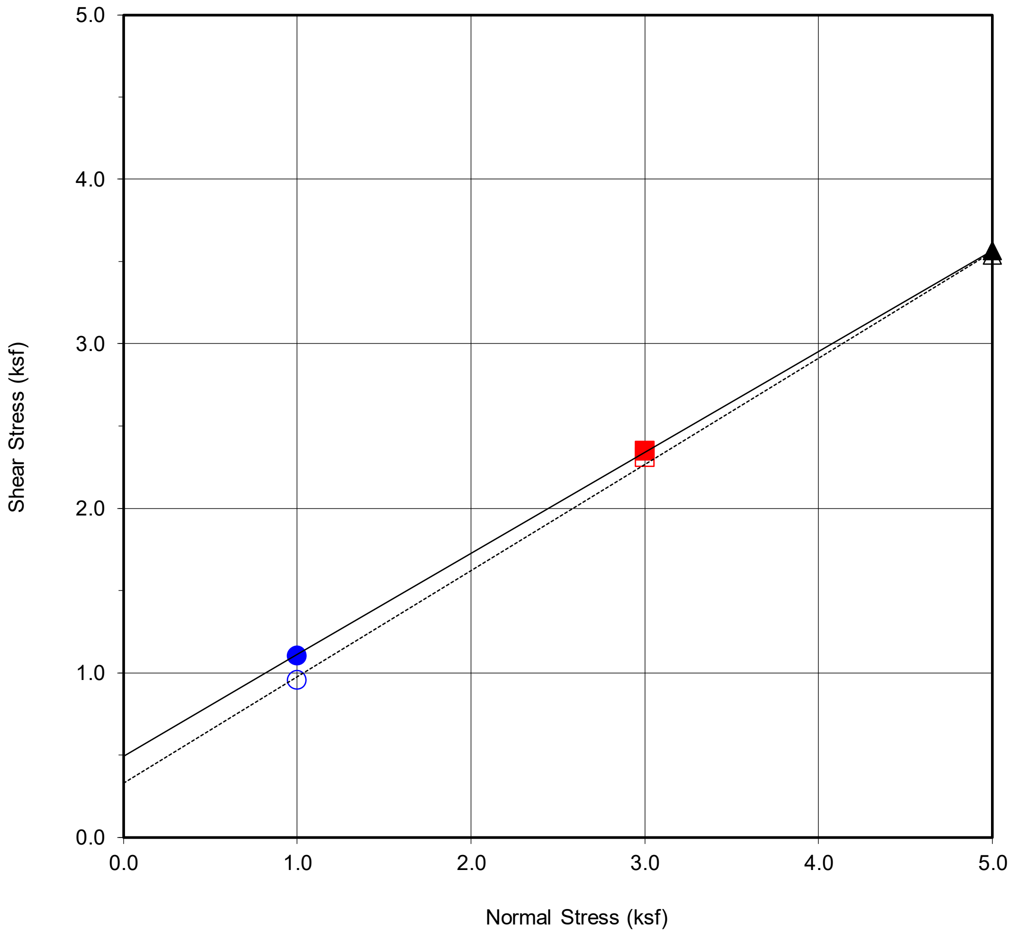
Figure B8



Boring No.	**B6**
Sample No.	**B6@5'**
Depth (ft)	**5**
Sample Type:	Ring

Normal Stress (kip/ft2)	1	3	5
Peak Shear Stress (kip/ft²)	● 1.11	■ 2.35	▲ 3.56
Shear Stress @ End of Test (ksf)	○ 0.96	□ 2.31	△ 3.54
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 (%)	14.4	14.3	5.1
Initial Dry Density (pcf)	110.3	112.7	115.2
Initial Degree of Saturation (%)	73.4	77.8	29.7
Soil Height Before Shearing (in.)	1.2	1.2	1.2
Final Moisture Content (%)	18.4	16.0	7.4

Soil Identification:

Brown Clay (CL)

Strength Parameters

	C (psf)	ϕ (°)
Peak	495	31.6
Ultimate	331	32.9

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DIRECT SHEAR TEST RESULTS

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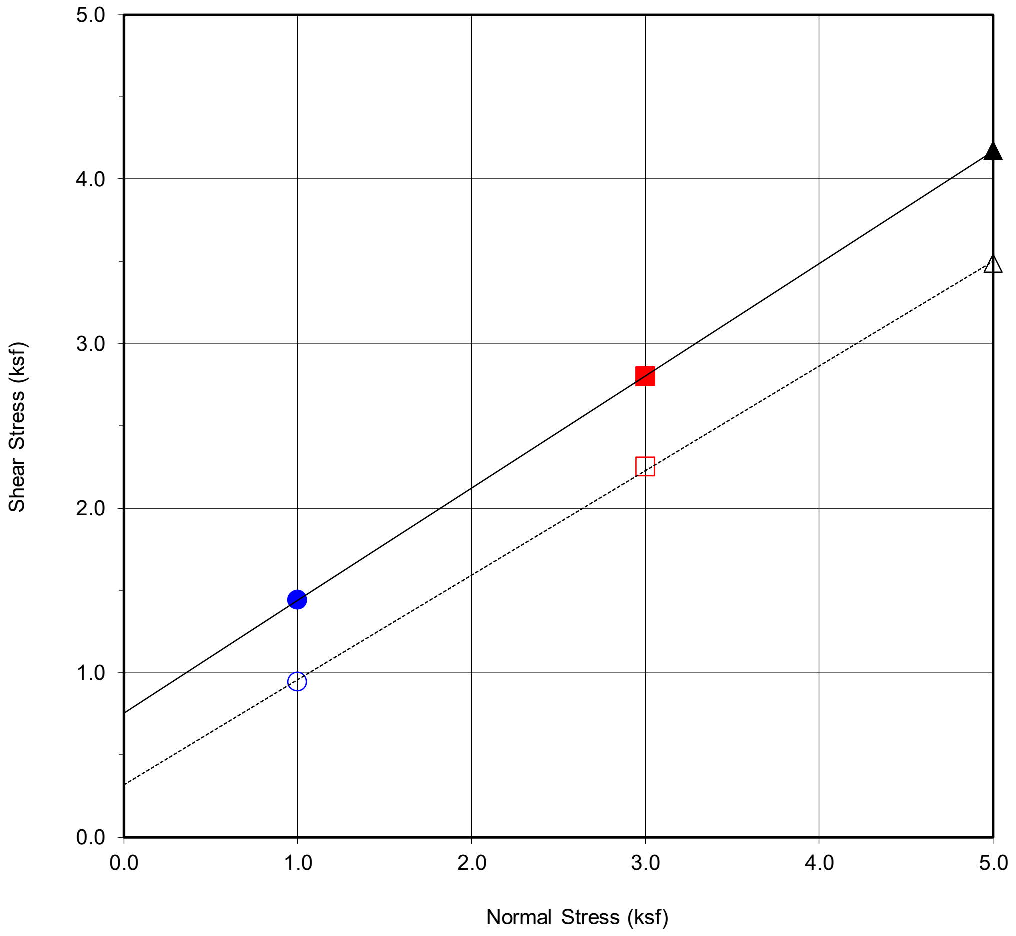
Figure B9



Boring No.	**B6**
Sample No.	**B6@10'**
Depth (ft)	**10**
Sample Type:	Ring

Soil Identification:		
Brown Silt (ML)		
Strength Parameters		
	C (psf)	ϕ (°)
Peak	755	34.3
Ultimate	319	32.5

Normal Stress (kip/ft2)	1	3	5
Peak Shear Stress (kip/ft²)	● 1.44	■ 2.80	▲ 4.17
Shear Stress @ End of Test (ksf)	○ 0.94	□ 2.25	△ 3.49
Deformation Rate (in./min.)	0.05	0.05	0.05
Initial Sample Height (in.)	1.0	1.0	1.0
Ring Inside Diameter (in.)	2.375	2.375	2.375
Initial Moisture Content (%)	16.9	16.1	17.1
Initial Dry Density (pcf)	113.8	114.3	111.5
Initial Degree of Saturation (%)	95.0	91.4	90.5
Soil Height Before Shearing (in.)	1.2	1.2	1.2
Final Moisture Content (%)	17.5	16.6	16.9

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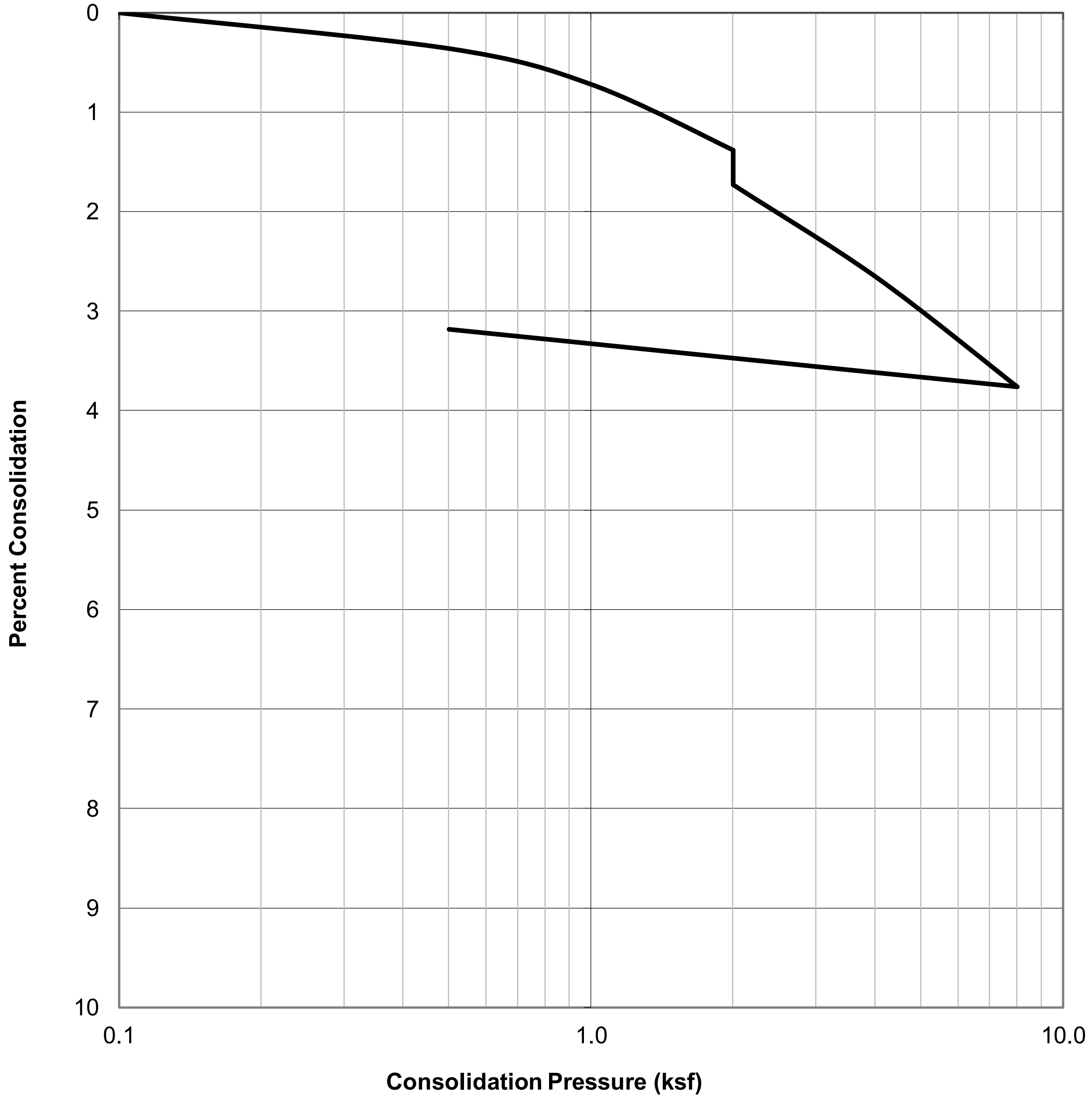
Figure B10

WATER ADDED AT 2.0 KSF



SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B3@5	Reddish Brown Silty Sand (SM)	107.3	12.8	17.3

GEOCON

CONSOLIDATION TEST RESULTS

ASTM D-2435

Checked by: JMH

Project No.: W1211-99-01

12939 FOOTHILL BLVD
RANCHO CUCAMONGA

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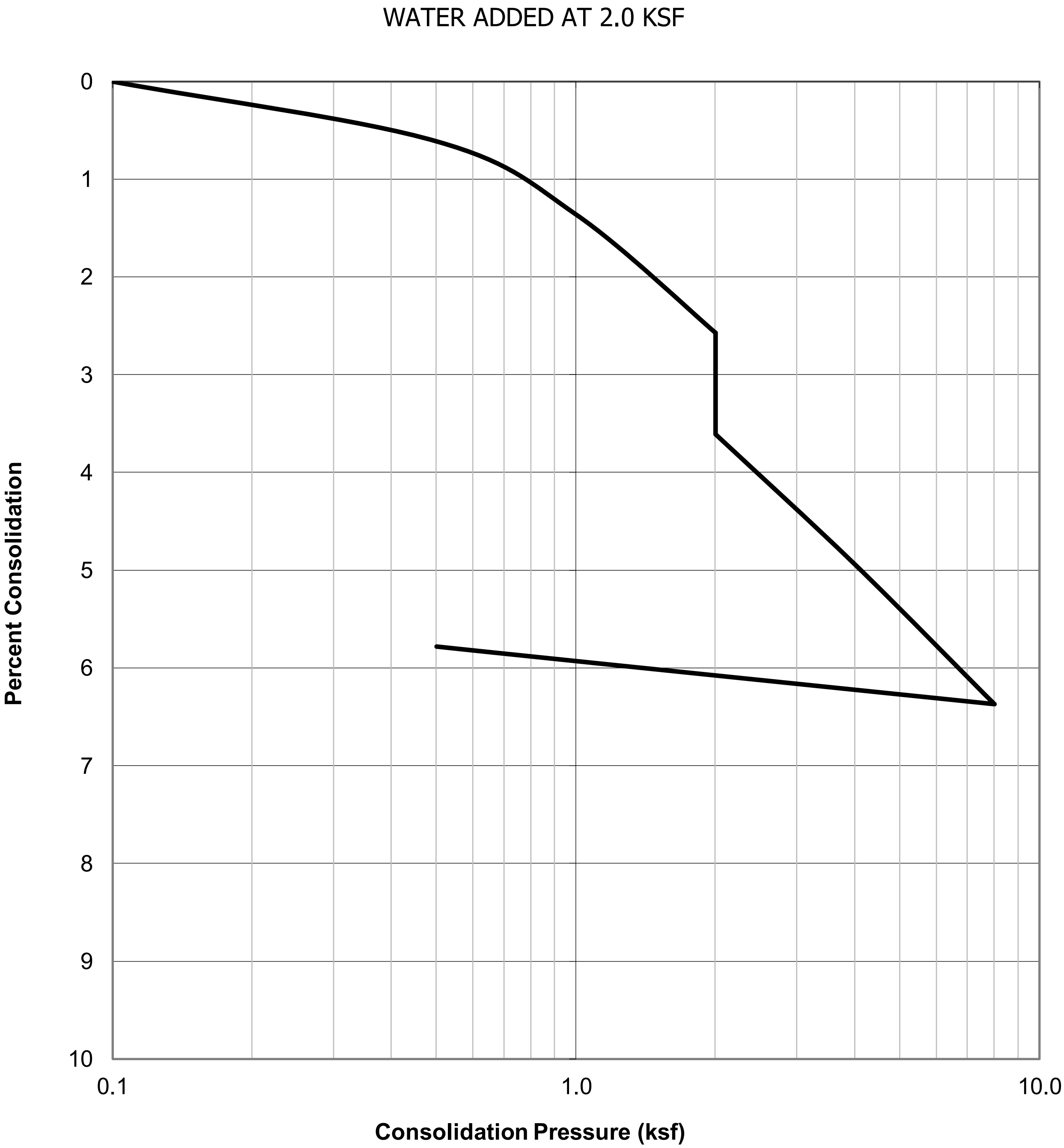
Figure B11



WATER ADDED AT 2.0 KSF

SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B3@7.5	Reddish Brown Silty Sand (SM)	99.7	7.6	20.1

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CONSOLIDATION TEST RESULTS

ASTM D-2435

Checked by: JMH

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12939 FOOTHILL BLVD
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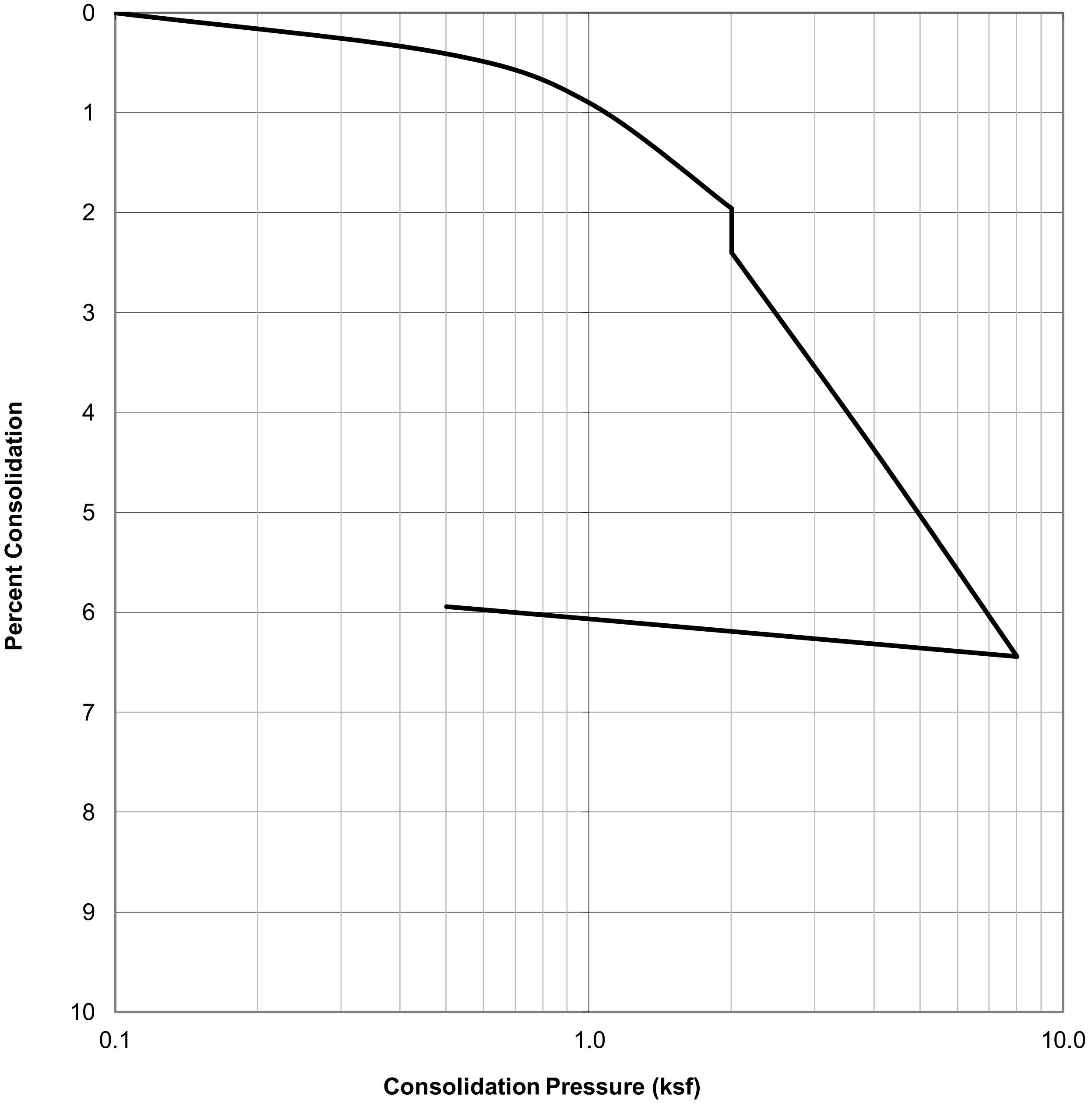
WATER ADDED AT 2.0 KSF



SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B3@10	Reddish Brown Silty Sand (SM)	96.4	5.8	14.6

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CONSOLIDATION TEST RESULTS

ASTM D-2435

Checked by: JMH

Project No.: W1211-99-01

12939 FOOTHILL BLVD
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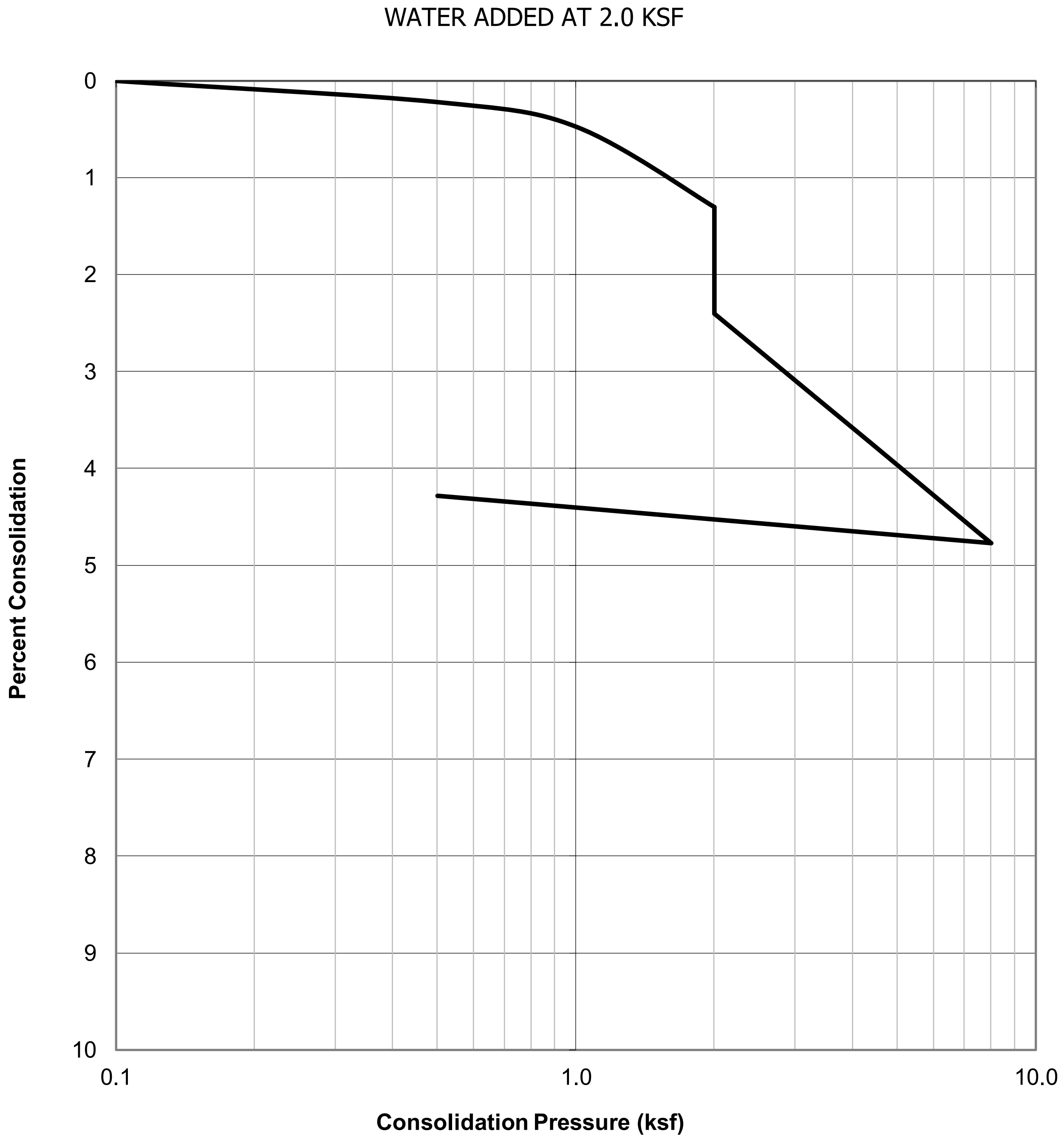
Figure B13



SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B3@20	Reddish Brown Silty Sand (SM)	111.6	4.8	14.4

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CONSOLIDATION TEST RESULTS

ASTM D-2435

Checked by: JMH

Project No.: W1211-99-01

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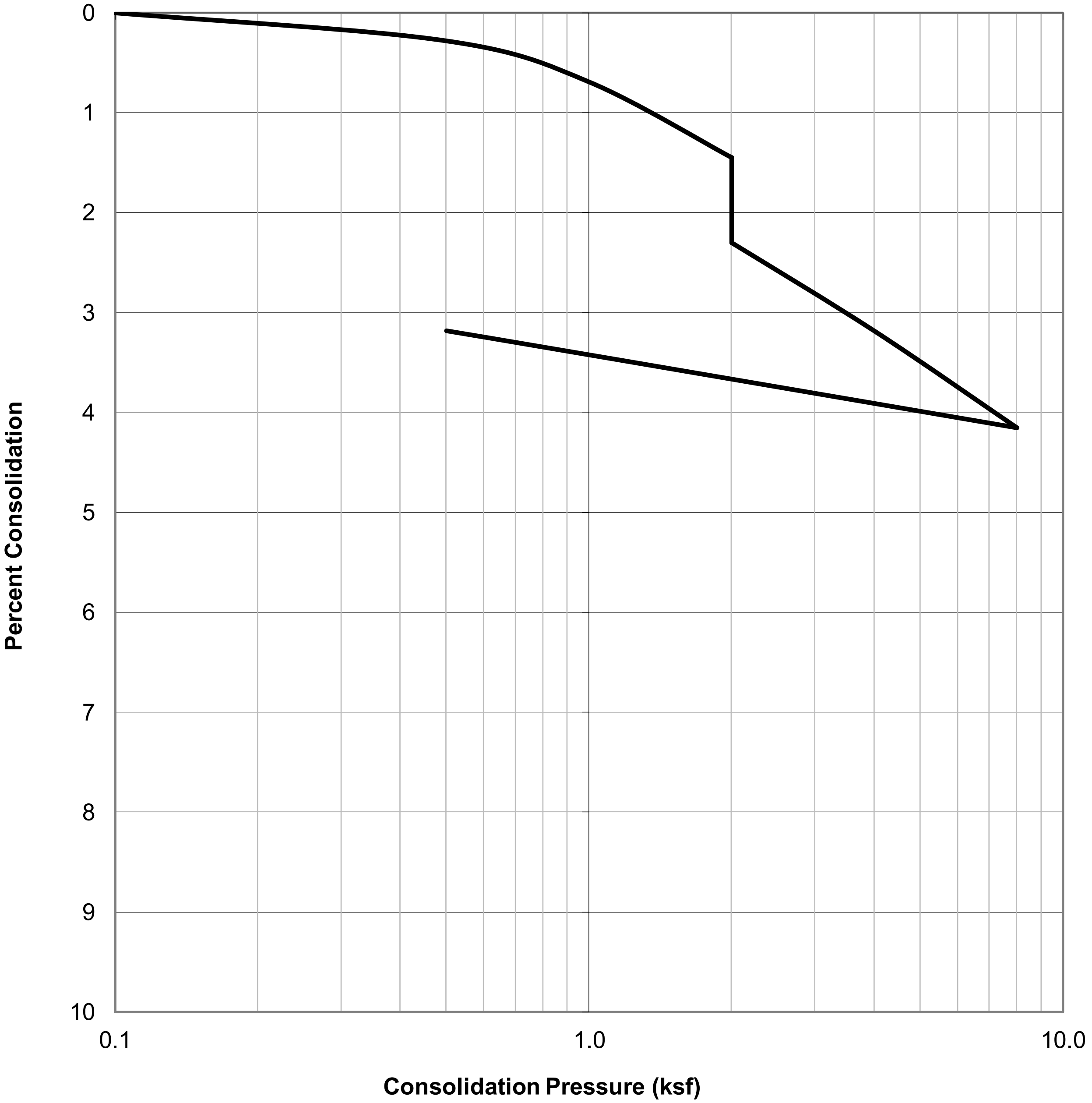
Figure B14

WATER ADDED AT 2.0 KSF



SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B4@5	Brown Clay (CL)	113.3	14.7	16.1

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CONSOLIDATION TEST RESULTS

ASTM D-2435

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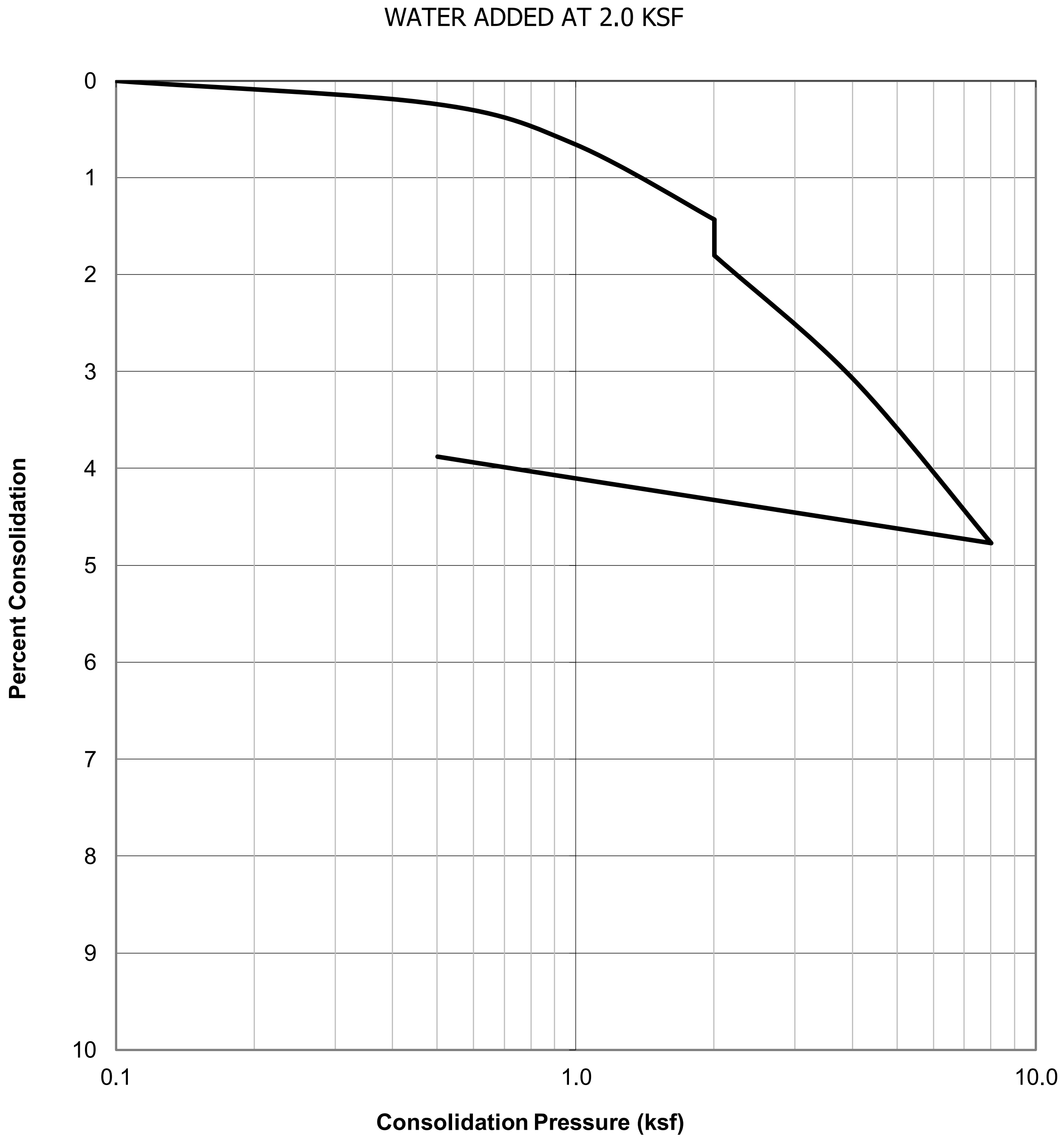

SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B4@7.5	Reddish Brown Silty Sand (SM)	107.7	18.3	18.6

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CONSOLIDATION TEST RESULTS

ASTM D-2435

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Project No.: W1211-99-01

12939 FOOTHILL BLVD
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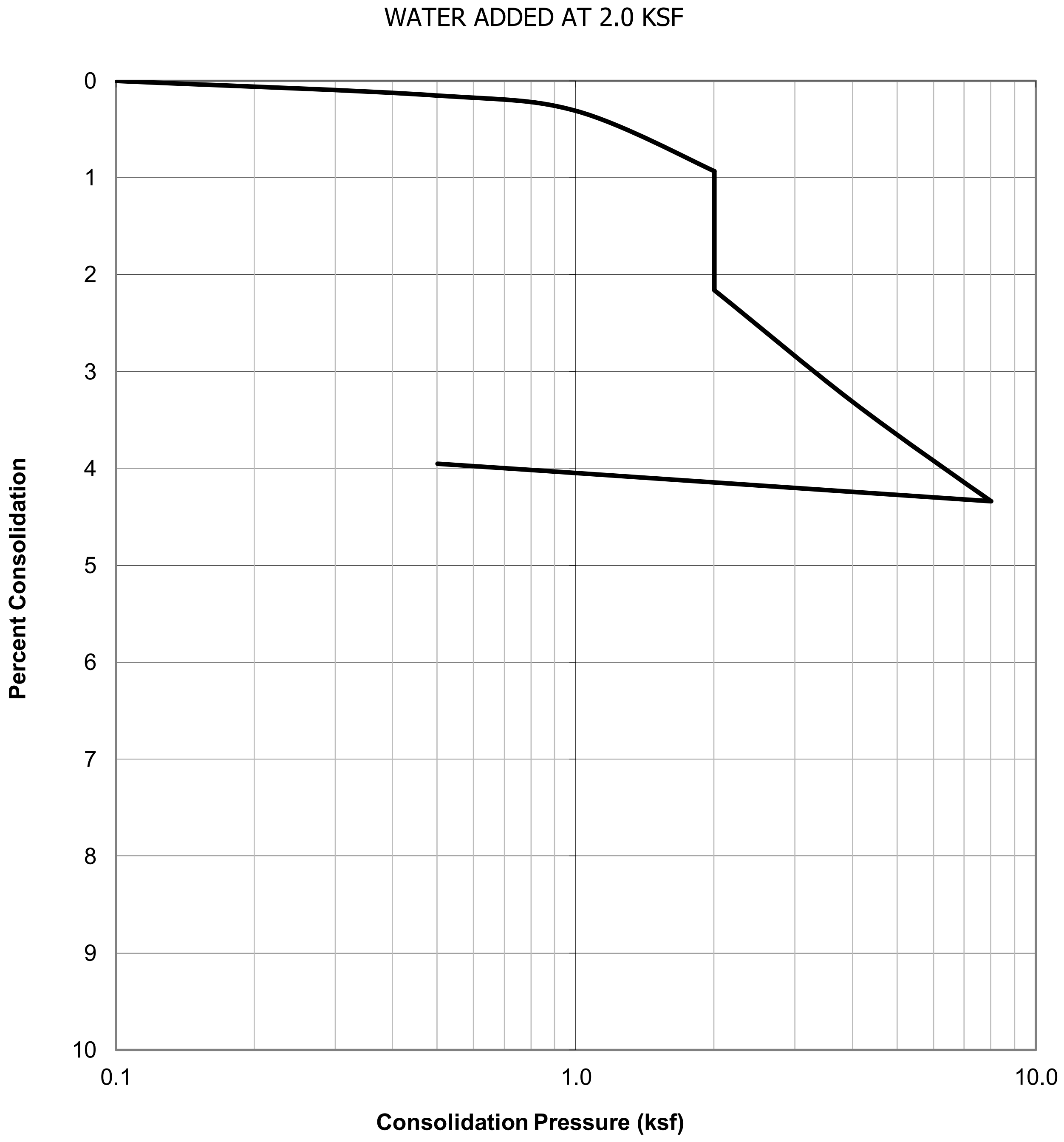

SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B4@15	Light Brown Well Graded Sand (SW)	108.8	5.7	15.0

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CONSOLIDATION TEST RESULTS

ASTM D-2435

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Project No.: W1211-99-01

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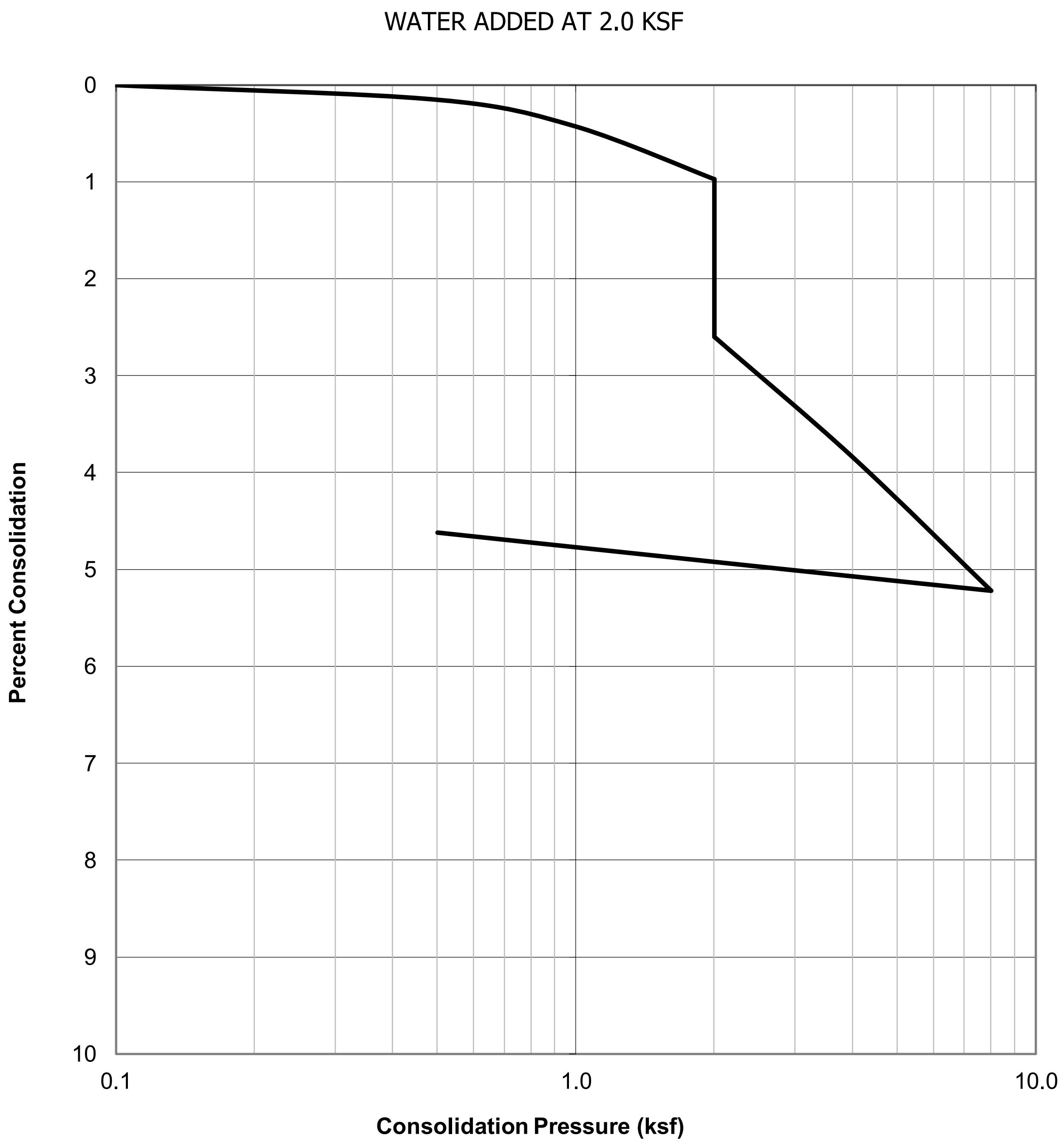
Figure B17



SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B5@10	Reddish Brown Silty Sand (SM)	114.4	6.5	13.5

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CONSOLIDATION TEST RESULTS

ASTM D-2435

Checked by: JMH

Project No.: W1211-99-01

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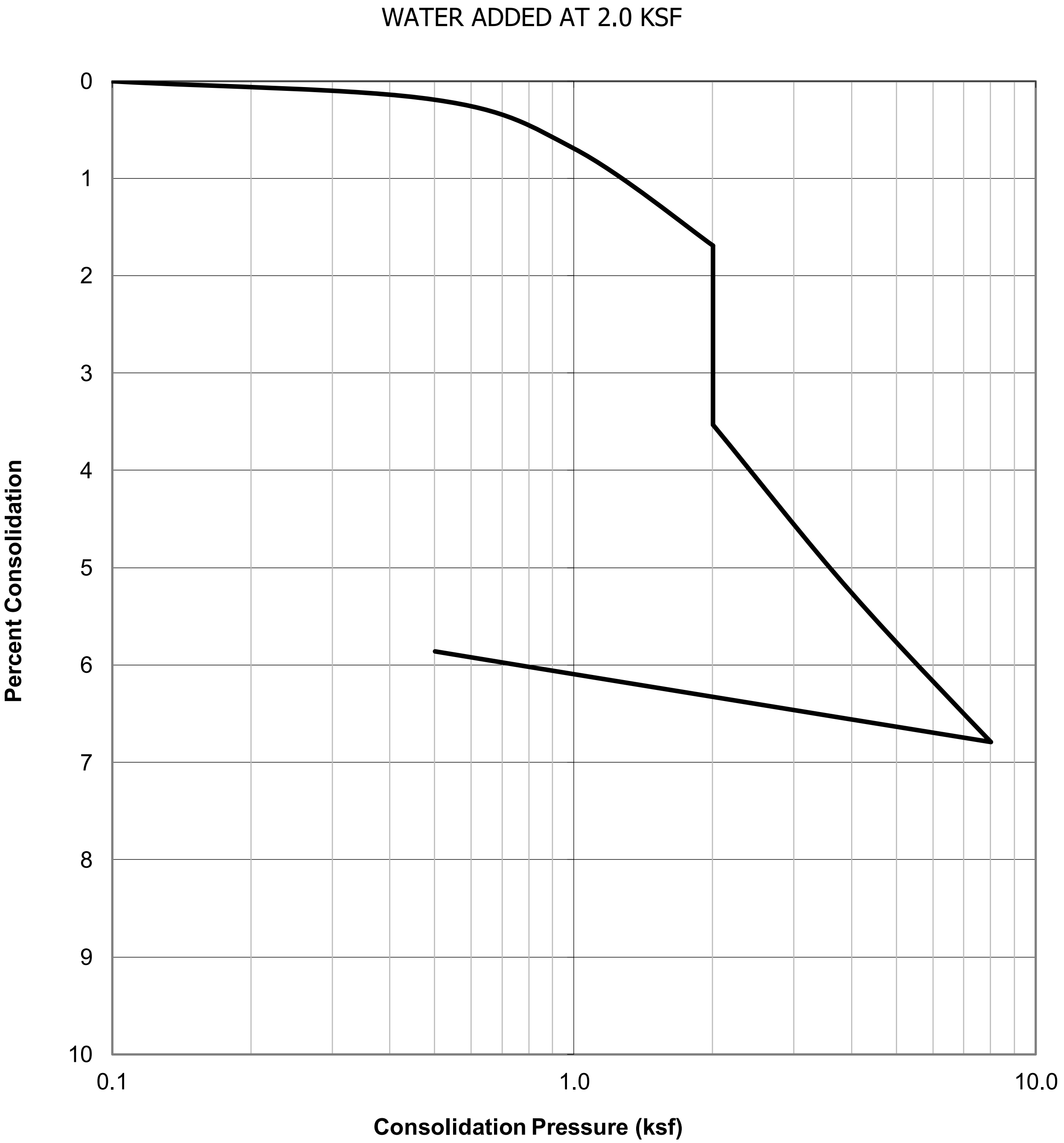
Figure B18



SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B5@15	Reddish Yellow Brown Silty Sand (SM)	113.9	11.3	14.7

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CONSOLIDATION TEST RESULTS

ASTM D-2435

Checked by: JMH

Project No.: W1211-99-01

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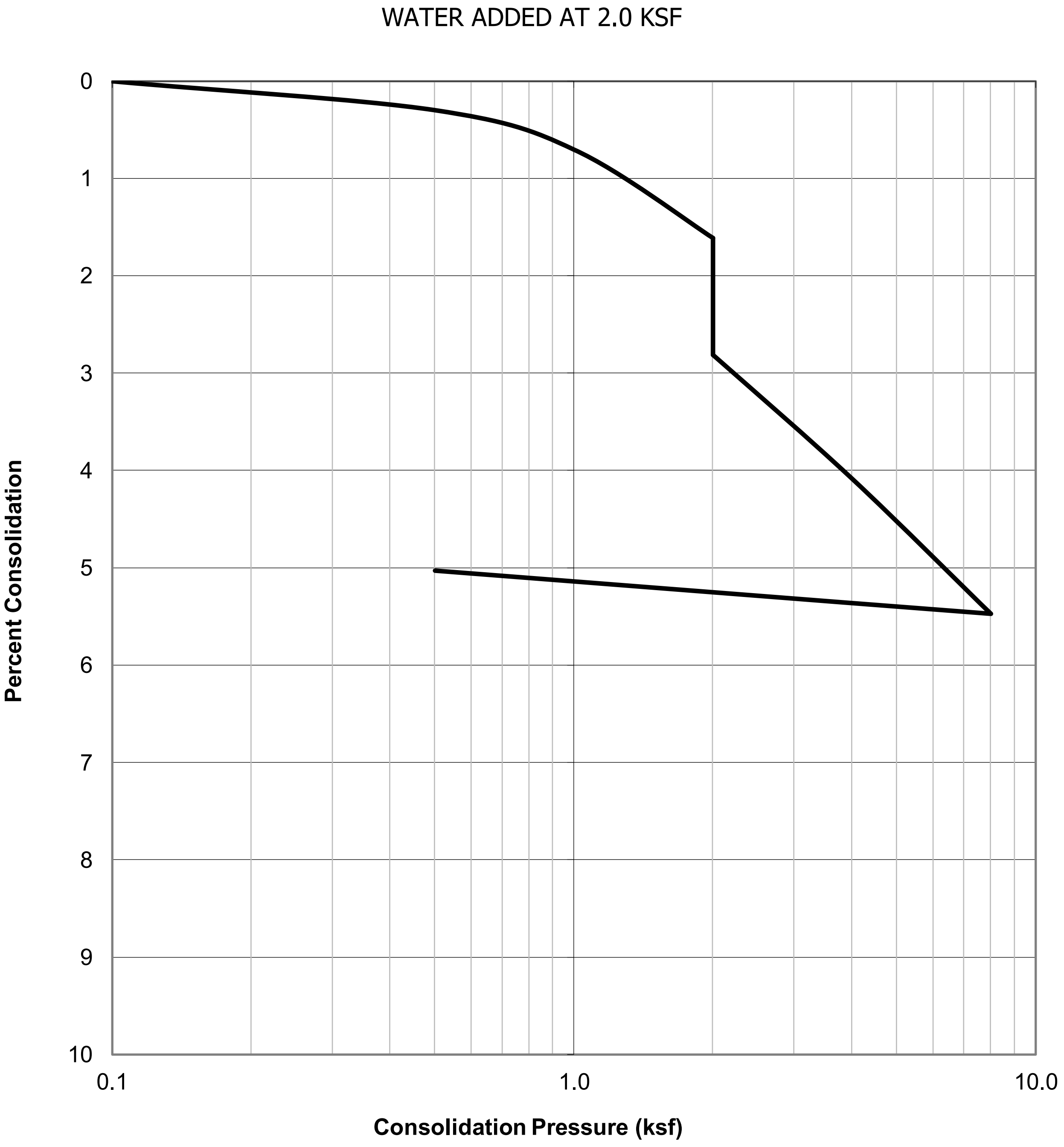
Figure B19



SAMPLE ID.	**SOIL TYPE**	**DRY DENSITY (PCF)**	**INITIAL MOISTURE (%)**	**FINAL MOISTURE (%)**
B5@25	Light Yellowish Brown Poorly Graded Sand (SP)	99.5	3.8	18.3

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CONSOLIDATION TEST RESULTS

ASTM D-2435

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Project No.: W1211-99-01

12939 FOOTHILL BLVD
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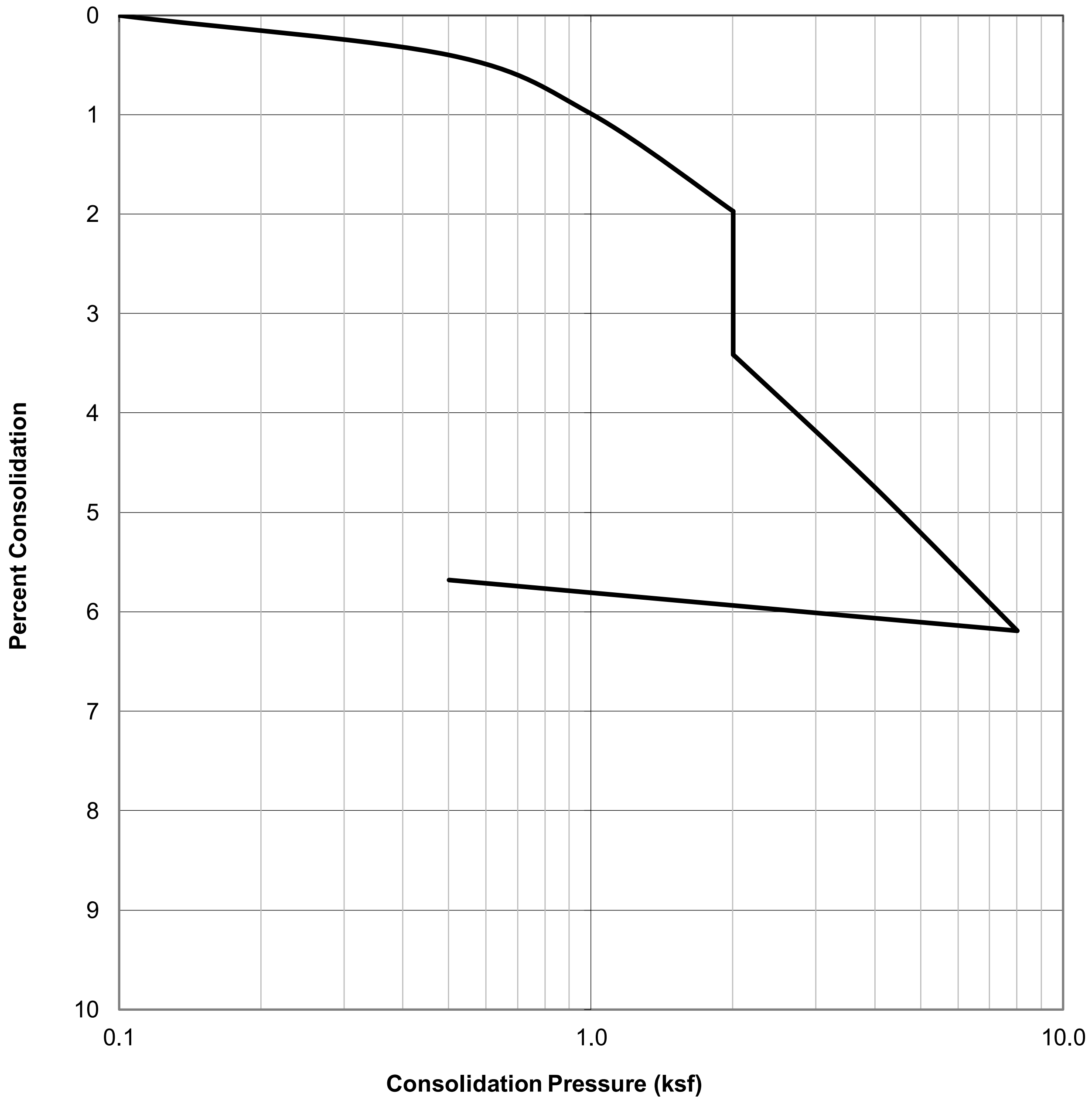
Figure B20

WATER ADDED AT 2.0 KSF



SAMPLE ID.	**SOIL TYPE**	**DRY DENSITY (PCF)**	**INITIAL MOISTURE (%)**	**FINAL MOISTURE (%)**
B5@35	Yellowish Brown Silty Sand (SM)	101.0	6.8	17.1

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CONSOLIDATION TEST RESULTS

ASTM D-2435

Checked by: JMH

Project No.: W1211-99-01

12939 FOOTHILL BLVD
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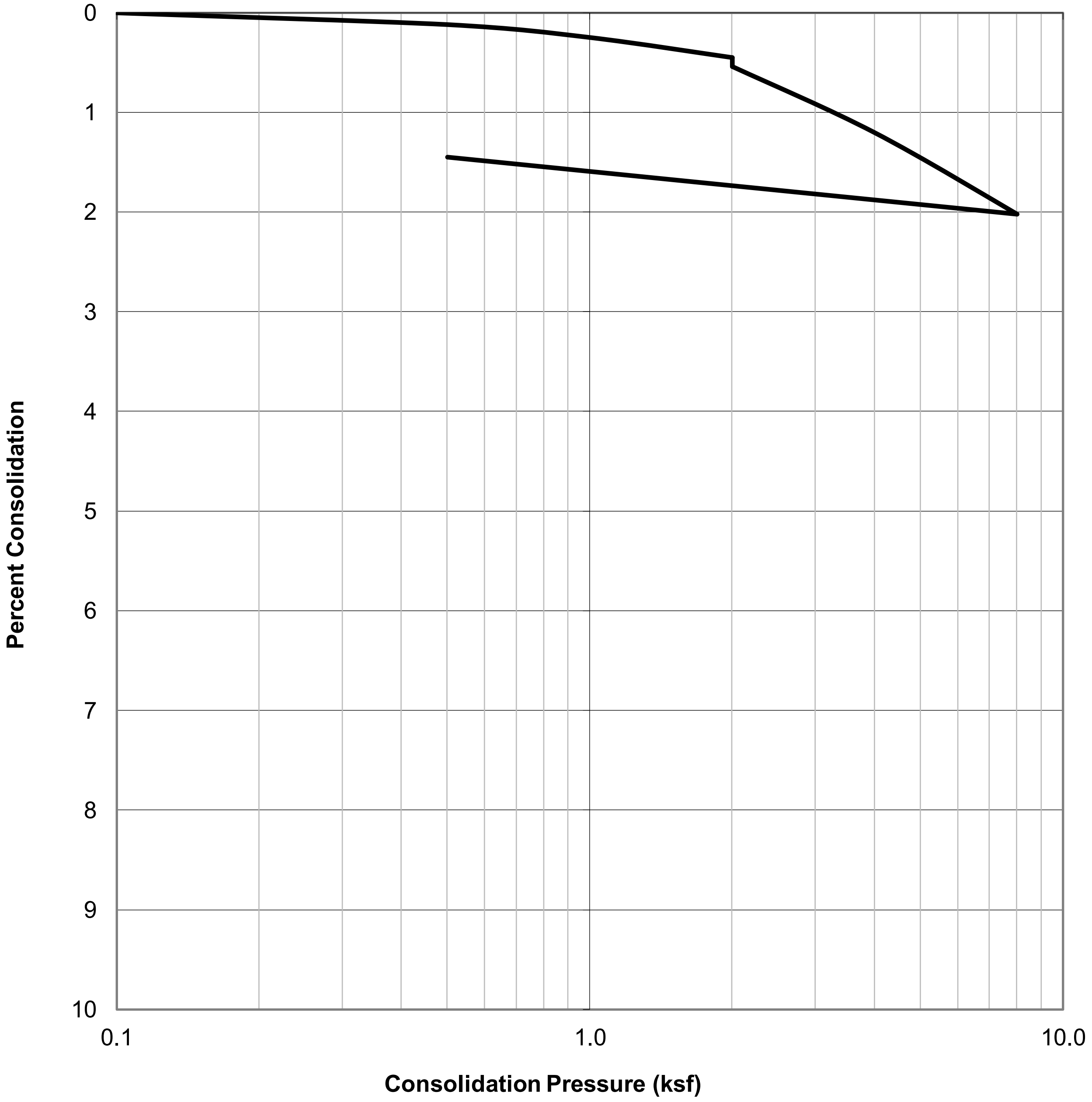
WATER ADDED AT 2.0 KSF



SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B5@45	Reddish Brown Silty Sand (SM)	120.7	11.9	13.9

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CONSOLIDATION TEST RESULTS

ASTM D-2435

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Project No.: W1211-99-01

12939 FOOTHILL BLVD
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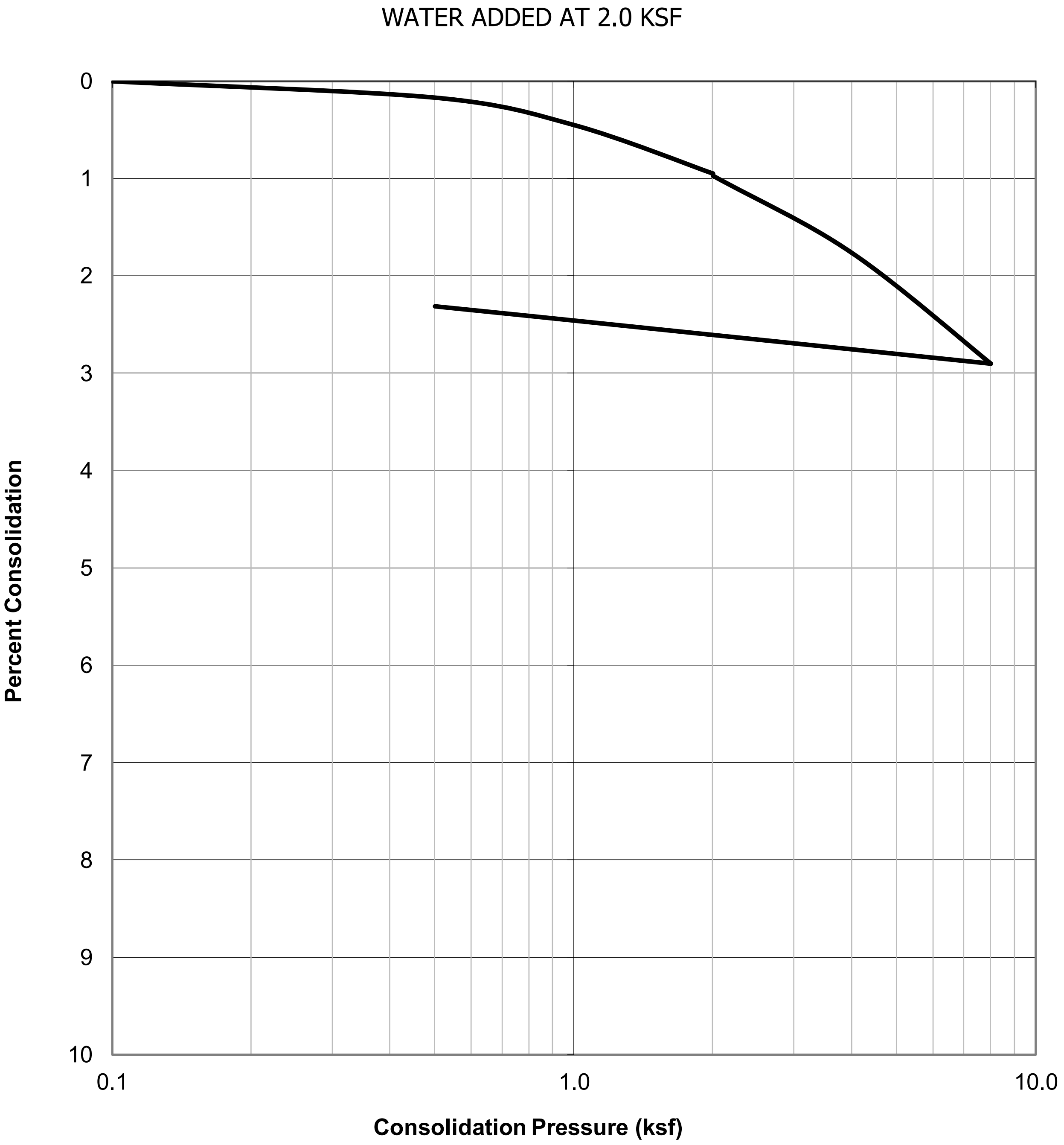
Figure B22



SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B6@10	Brown Silt (ML)	117.2	15.1	15.1

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CONSOLIDATION TEST RESULTS

ASTM D-2435

Checked by: JMH

Project No.: W1211-99-01

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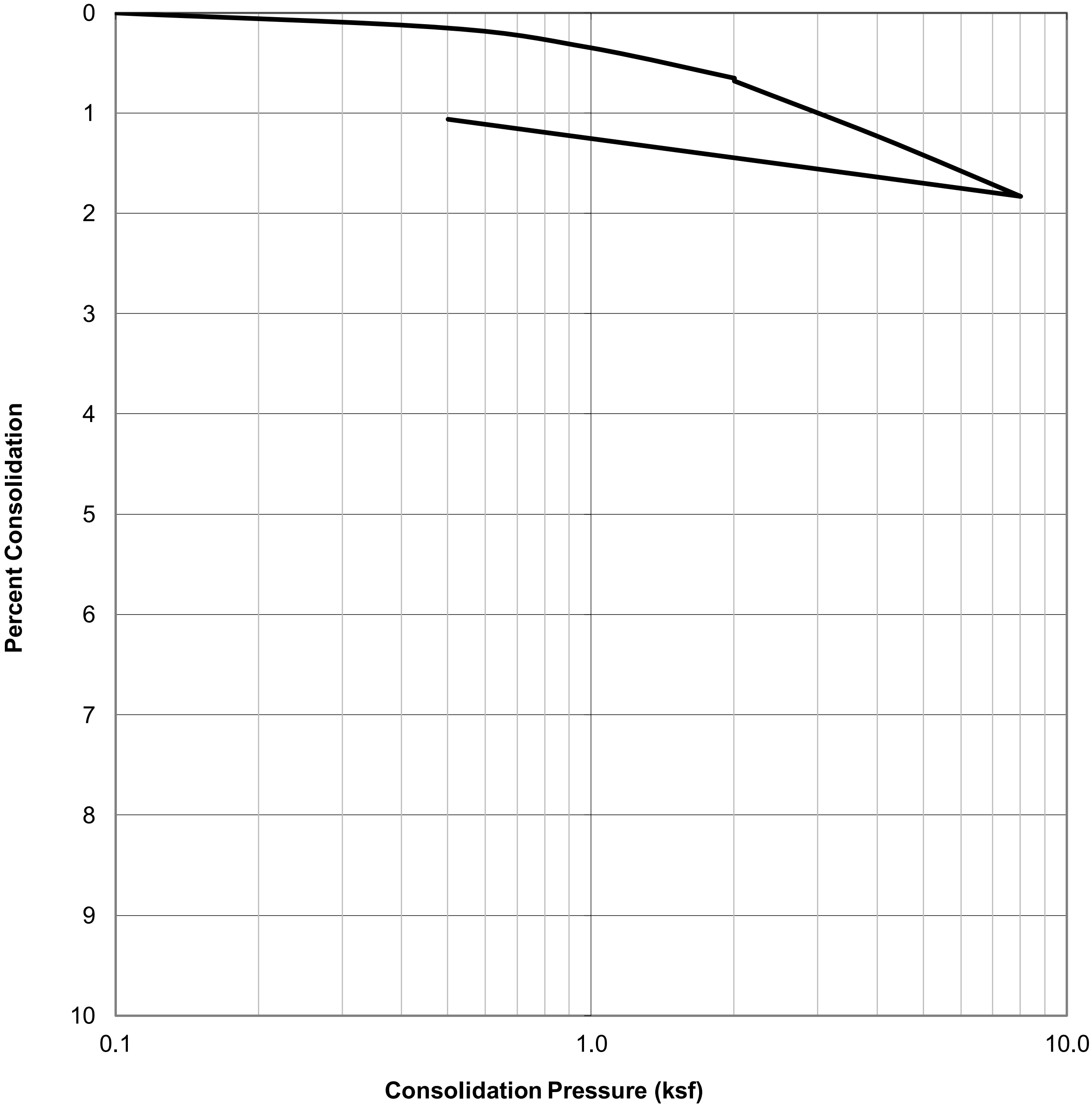
WATER ADDED AT 2.0 KSF



SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B6@15	Brown Silty Sand (SM)	119.8	11.2	13.9

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CONSOLIDATION TEST RESULTS

ASTM D-2435

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Project No.: W1211-99-01

12939 FOOTHILL BLVD
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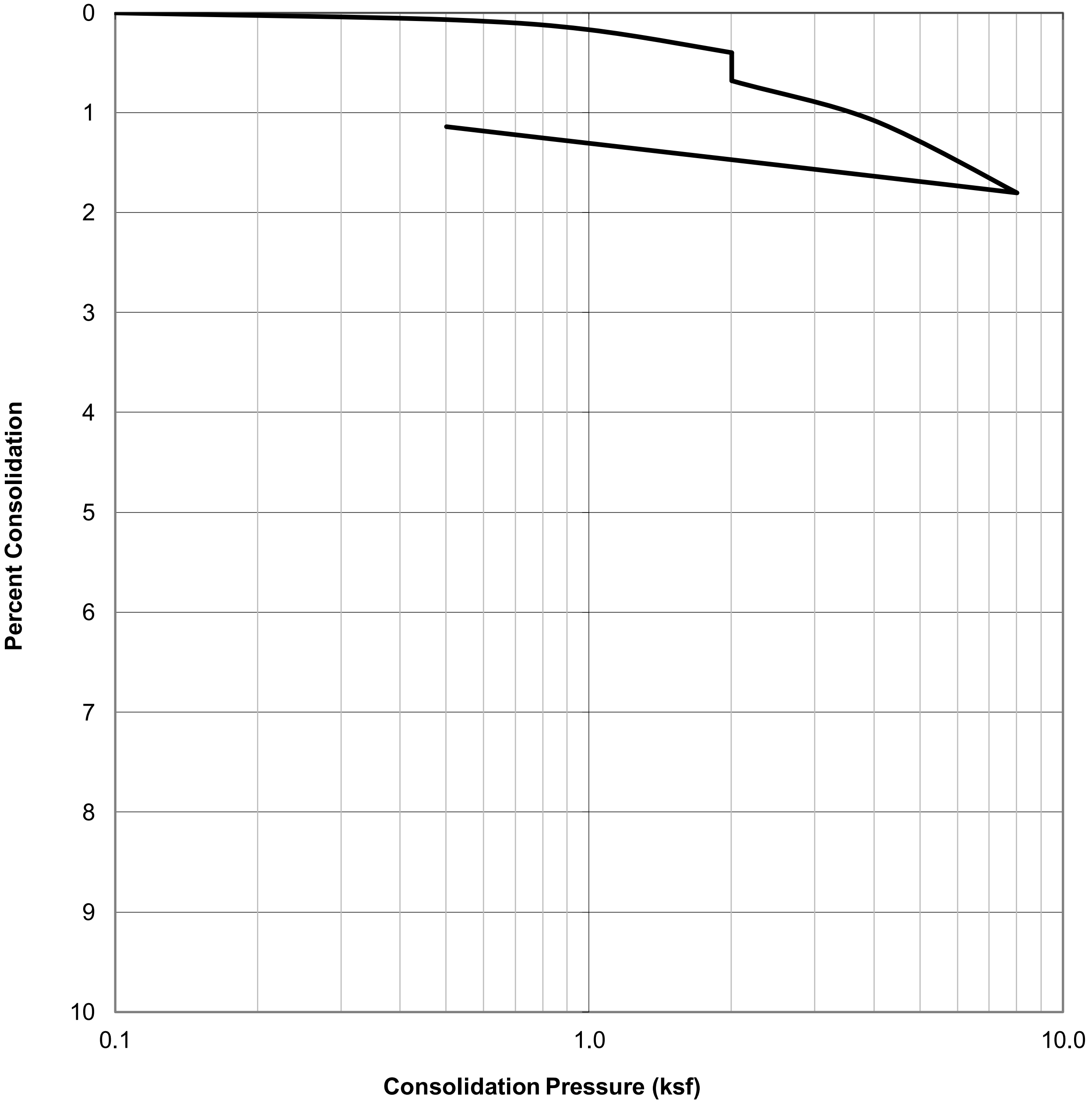
Figure B24

WATER ADDED AT 2.0 KSF



SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B6@20	Reddish Brown Silty Sand (SM)	120.4	10.9	13.9

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CONSOLIDATION TEST RESULTS

ASTM D-2435

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Project No.: W1211-99-01

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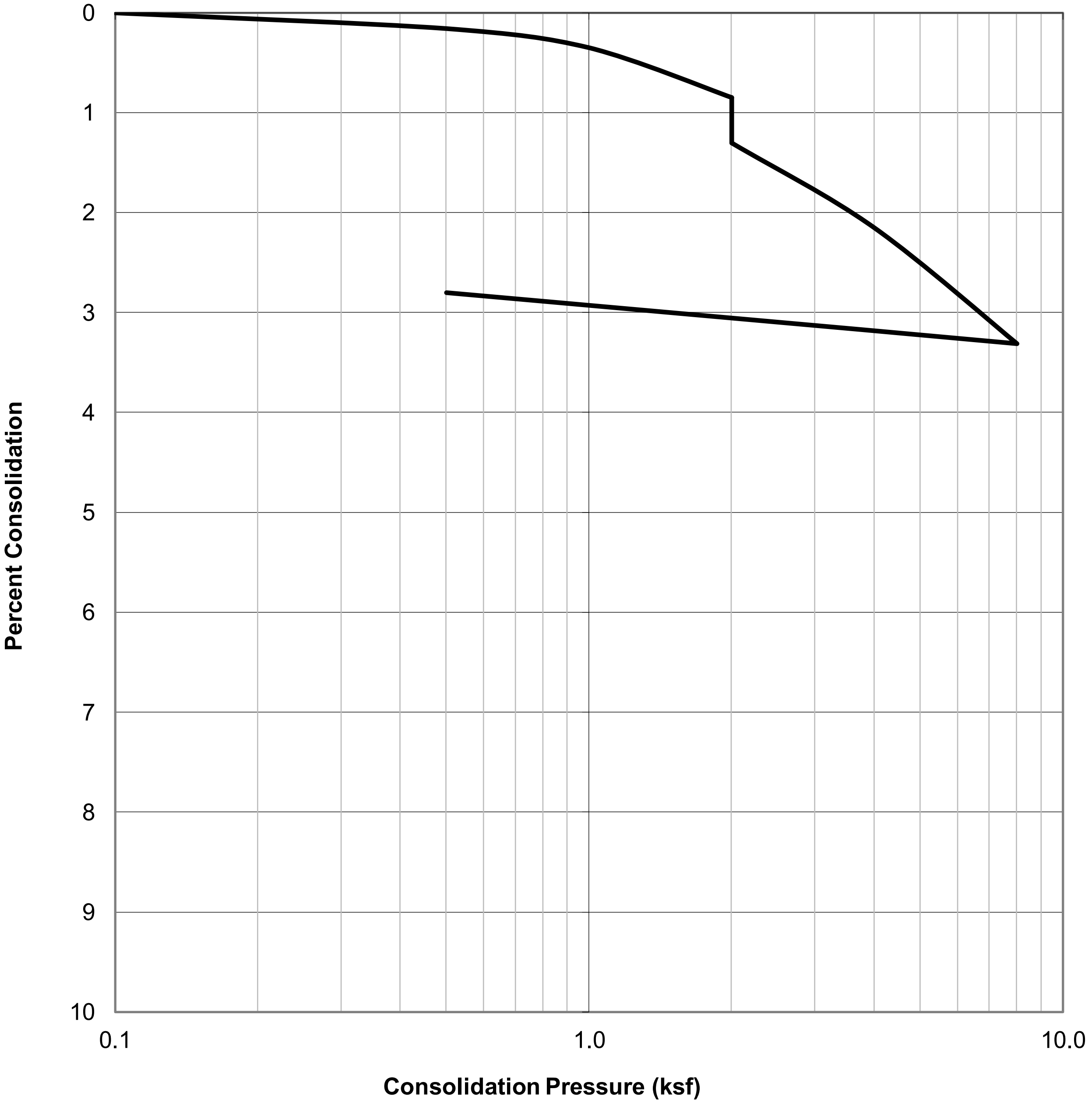
Figure B25

WATER ADDED AT 2.0 KSF



SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B8@7.5	Reddish Brown Silty Sand (SM)	103.1	8.6	19.6

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CONSOLIDATION TEST RESULTS

ASTM D-2435

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Project No.: W1211-99-01

12939 FOOTHILL BLVD
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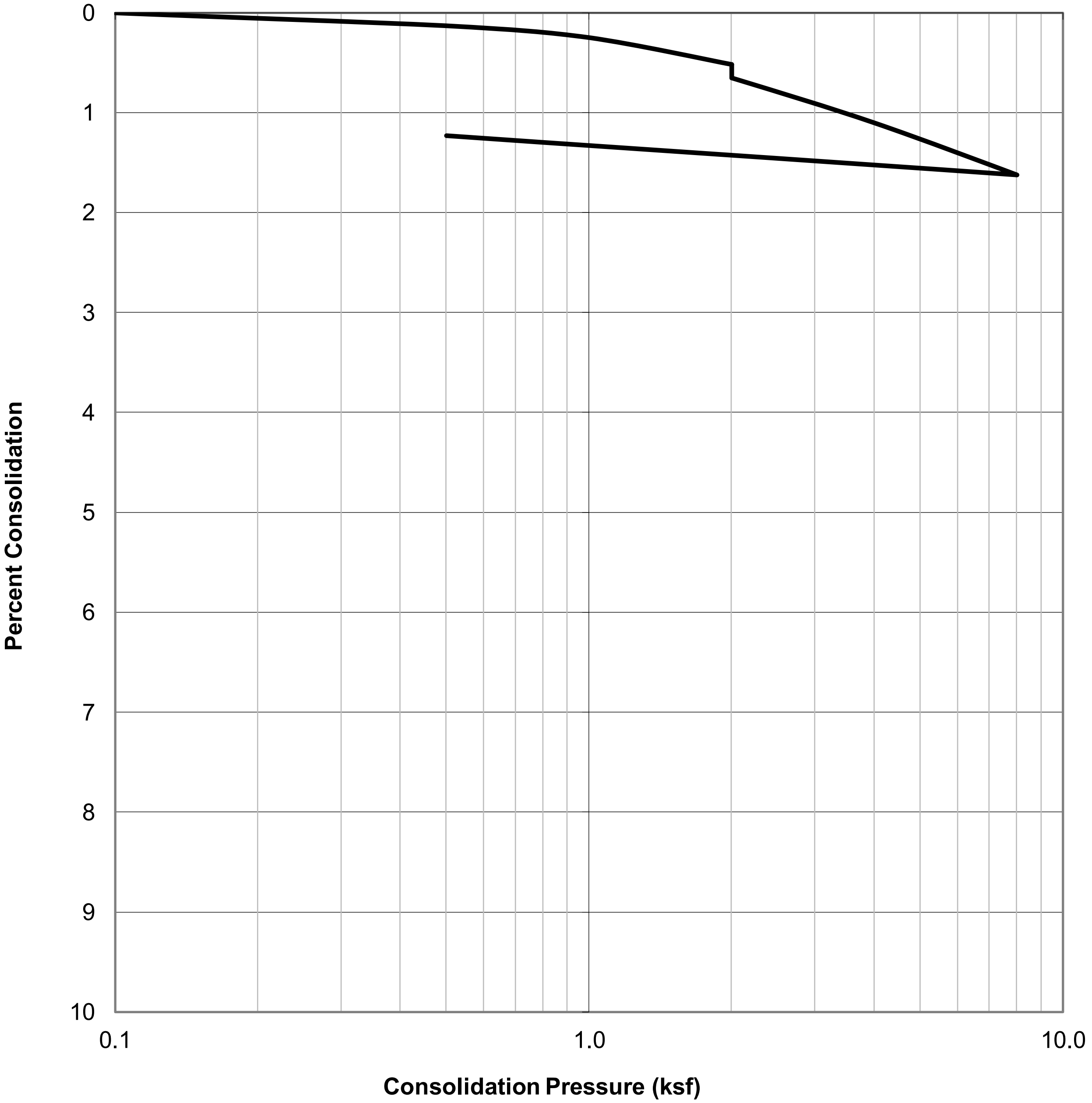
Figure B26

WATER ADDED AT 2.0 KSF



SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B8@15	Brown Silty Sand (SM)	118.5	7.8	9.9

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CONSOLIDATION TEST RESULTS

ASTM D-2435

Checked by: JMH

Project No.: W1211-99-01

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B4@0-5'

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)	600.3	615.5
Wt. of Mold	(gm)	196.8	196.8
Specific Gravity	(Assumed)	2.7	2.7
Wet Wt. of Soil + Cont.	(gm)	558.5	615.5
Dry Wt. of Soil + Cont.	(gm)	532.5	368.5
Wt. of Container	(gm)	258.5	196.8
Moisture Content	(%)	9.5	13.6
Wet Density	(pcf)	121.7	126.1
Dry Density	(pcf)	111.2	111.0
Void Ratio		0.5	0.5
Total Porosity		0.3	0.3
Pore Volume	(cc)	70.5	70.1
Degree of Saturation	(%) [S_{meas}]	50.0	71.6

Date	Time	Pressure (psi)	Elapsed Time (min)	Dial Readings (in.)
8/14/2020	10:00	1.0	0	0.2856
8/14/2020	10:10	1.0	10	0.2842
Add Distilled Water to the Specimen				
8/15/2020	10:00	1.0	1430	0.2823
8/15/2020	11:00	1.0	1490	0.2823

Expansion Index (EI meas) =	0
Expansion Index (Report) =	**0**

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.

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EXPANSION INDEX TEST RESULTS

ASTM D-4829

Checked by: JMH

Project No.: W1211-99-01

12939 FOOTHILL BLVD
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Figure B28

B1@5-10'

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)	614.8	637.7
Wt. of Mold	(gm)	198.3	198.3
Specific Gravity	(Assumed)	2.7	2.7
Wet Wt. of Soil + Cont.	(gm)	553.9	637.7
Dry Wt. of Soil + Cont.	(gm)	531.7	385.6
Wt. of Container	(gm)	253.9	198.3
Moisture Content	(%)	8.0	13.9
Wet Density	(pcf)	125.6	132.4
Dry Density	(pcf)	116.3	116.2
Void Ratio		0.4	0.5
Total Porosity		0.3	0.3
Pore Volume	(cc)	64.2	64.9
Degree of Saturation	(%) [S_{meas}]	48.5	82.9

Date	Time	Pressure (psi)	Elapsed Time (min)	Dial Readings (in.)
8/14/2020	10:00	1.0	0	0.3606
8/14/2020	10:10	1.0	10	0.3604
Add Distilled Water to the Specimen				
8/15/2020	10:00	1.0	1430	0.3638
8/15/2020	11:00	1.0	1490	0.3638

Expansion Index (EI meas) =	3.4
Expansion Index (Report) =	**3**

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.

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EXPANSION INDEX TEST RESULTS

ASTM D-4829

Checked by: JMH

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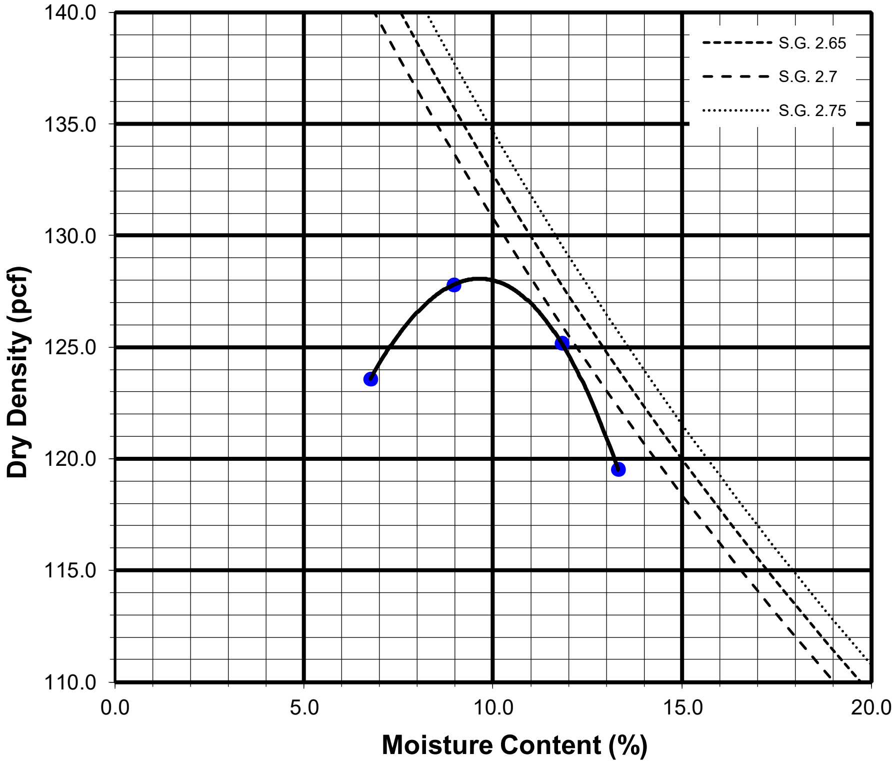
Figure B29

Sample No:

B4@0-5'	Reddish Brown Clayey Sand (SC)

TEST NO.		1	2	3	4	5	6
Wt. Compacted Soil + Mold	(g)	6270	6381	6391	6323		
Weight of Mold	(g)	4277	4277	4277	4277		
Net Weight of Soil	(g)	1993	2104	2114	2046		
Wet Weight of Soil + Cont.	(g)	591.8	593.7	626.2	664.0		
Dry Weight of Soil + Cont.	(g)	570.6	566.2	587.3	616.3		
Weight of Container	(g)	256.8	259.4	258.0	257.8		
Moisture Content	(%)	6.8	9.0	11.8	13.3		
Wet Density	(pcf)	131.9	139.3	140.0	135.4		
Dry Density	(pcf)	123.6	127.8	125.2	119.5		

Maximum Dry Density (pcf)	**129.0**	**Optimum Moisture Content (%)**	**10.0**



Preparation Method: A

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MODIFIED COMPACTION TEST OF SOILS

ASTM D-1557

Checked by: JMH

Project No.: W1211-99-01

12939 FOOTHILL BLVD
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Figure B30

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

Sample No.	**pH**	**Resistivity (ohm centimeters)**
B4 @ 0-5'	7.1	2800 (Moderately Corrosive)

SUMMARY OF LABORATORY CHLORIDE CONTENT TEST RESULTS
EPA NO. 325.3

Sample No.	Chloride Ion Content (%)
B4@0-5'	0.006

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

Sample No.	**Water Soluble Sulfate (% SQ_4)**	**Sulfate Exposure***
B4@0-5'	0.001	S0

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CORROSIVITY TEST RESULTS

Checked by: JMH

Project No.: W1211-99-01

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Figure B31