Appendix H Geotechnical Feasibility Investigation

Appendices

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GEOCON
WEST, INC.

GEOTECHNICAL
ENVIRONMENTAL
MATERIALS

GEOTECHNICAL FEASIBILITY INVESTIGATION

CHAFFEY COLLEGE CAMPUS EXTENSION
11016-11098 SIERRA AVENUE
FONTANA, CALIFORNIA

APN'S: 0255-101-05-0000; -06-0000; -07-0000; -08-0000 & -09-0000

PREPARED FOR

CAFFEY COMMUNITY COLLEGE DISTRICT
RANCHO CUCAMONGA, CALIFORNIA

PROJECT NO. T2746-99-10A

FEBRUARY 28, 2020

Project No. T2746-99-10A
February 28, 2020

Chaffey Community College District
5885 Haven Avenue
Rancho Cucamonga, CA 91737

Subject: GEOTECHNICAL FEASIBILITY INVESTIGATION
CHAFFEY COLLEGE CAMPUS EXTENSION
11016-11098 SIERRA AVENUE, FONTANA, CALIFORNIA
APN's: 0255-101-05-0000; -06-0000; -07-0000; -08-0000 & -09-0000

Dear Mr. Shah:

In accordance with your authorization of our proposal dated December 17, 2019, we have performed a geotechnical feasibility investigation for the proposed Chaffey College campus extension located at 11016-11098 Sierra Avenue in the City of Fontana, 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 feasibility investigation for the proposed Chaffey College campus extension located at 11016-11098 Sierra Avenue in the City of Fontana, California (see Vicinity Map, Figure 1). The purpose of the investigation was to evaluate subsurface soil and geologic conditions underlying the site and, based on conditions encountered, to provide conclusions and recommendations pertaining to the geotechnical aspects of design and construction.

The scope of this investigation included a site reconnaissance, field exploration, laboratory testing, engineering analysis, and the preparation of this report. The site was explored on January 23, 2020, by excavating eight 8-inch diameter borings to depths between 10½ and 20½ feet below the existing ground surface using a truck-mounted hollow-stem auger drilling machine. The approximate locations of the exploratory borings are depicted on the Site Plan (see Figure 2). A detailed discussion of the field investigation, including the boring logs, is presented in Appendix A.

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

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

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

2. SITE AND PROJECT DESCRIPTION

The subject site is located at 11016-11098 Sierra Avenue in the City of Fontana, California. The site consists of five adjacent rectangular parcels. The two northernmost parcels are occupied by three single family residences and the remaining parcels are vacant. The subject site is bounded by a small commercial development consisting of three commercial structures and paved surface parking lots to the north, by single family residential lots to the west, by Sierra Avenue to the east, and by an infiltration basin to the south. The site is relatively level, with no pronounced highs or lows. Surface water drainage at the site appears to follow no discernable pattern. Vegetation onsite is grasses and weeds that have sprung up from the winter rains and a few trees in the vacant parcels. The two northern parcels have numerous trees and shrubs that are growing around the existing residences.

Based on the information provided by the Client, it is our understanding that the design of proposed development is currently unknown at this preliminary stage of development, Based on the preliminary nature of the design at this time, wall and column loads were not available. It is anticipated that column loads for the proposed structures will be up to 400 kips, and wall loads will be up to 4.0 kip 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 Chino Basin approximately 0.5 mile north of the Jurupa Hills. The Chino Basin is bounded on the north and northwest by the San Jose Fault and the Red Hill Fault, on the northeast by the Rialto-Colton Basin and the Rialto-Colton Fault, on the southeast and south by the Riverside Basin, and on the southwest by the Chino Hills. Regionally, the site is located within the northern portion of the Peninsular Ranges geomorphic province characterized by northwest-trending physiographic and geologic features such as the Elsinore Fault and the Chino Fault, located approximately 15½ miles to the southwest (USGS, 2006).

4. SOIL AND GEOLOGIC CONDITIONS

Based on our field investigation and published geologic maps of the area, the site is underlain by artificial fill and Holocene age alluvium consisting of interbedded sand and silt (Dibblee, 2004; CGS, 2010). Detailed stratigraphic profiles of the materials encountered at the site are provided on the boring logs in Appendix A.

4.1 Artificial Fill

Artificial fill was encountered in our borings to a maximum depth of 2 feet below existing ground surface. The artificial fill generally consists of light brown to brown silty sand with varying amounts of gravel. The artificial fill is characterized as slightly moist to moist and loose. The fill is likely the result of past grading or construction activities at the site. Deeper fill may exist between excavations and in other portions of the site that were not directly explored.

4.2 Alluvium

The fill soils are underlain by Holocene age alluvium consisting of light brown to brown, yellowish brown, and light grayish brown to grayish brown, interbedded silt, sandy silt, silty sand, sand with silt and poorly graded sand. These soils are characterized as fine-grained to coarse-grained with varying amounts of fine gravel, dry to moist, and loose to very dense or firm to hard.

5. GROUNDWATER

Historic groundwater water level information for the site and the immediate site vicinity was obtained from Mendenhall (1907), the California Geological Survey (CGS, formerly the California Division of Mines and Geology [CDMG], 1976), and the Chino Basin Watermaster (2017).

The depth to groundwater in 1904 (Mendenhall, 1907) and 1960 (CDMG, 1976) was reported to be approximately 250 feet and 275 feet beneath the existing ground surface, respectively. More recently, the groundwater is also reported to be at a depth of approximately 275 feet beneath the ground surface in 2000, 2012, and 2016 (Chino Basin Watermaster, 2017). Based on the information from Mendenhall (1907), CDMG (1976), and the Chino Basin Watermaster (2017), groundwater levels have remained at depths greater than 200 feet beneath the ground surface for over 100 years. Current groundwater basin management practices are designed to keep groundwater levels consistent and it is highly unlikely that groundwater levels will ever rise significantly above the current groundwater levels (Chino Basin Watermaster, 2017).

Groundwater was not encountered in our field explorations, drilled to a maximum depth of 20½ feet below the existing ground surface. Based on the lack of groundwater in our borings and the historic groundwater levels in the immediate site vicinity, groundwater is neither expected to be encountered during future construction nor have a detrimental effect on future developments at the site. However, it is not uncommon for groundwater levels to vary seasonally or for groundwater seepage conditions to develop where none previously existed, especially in impermeable fine-grained soils which are heavily irrigated or after seasonal rainfall. In addition, recent requirements for stormwater infiltration could result in shallower seepage conditions in the immediate site vicinity. Proper surface drainage of irrigation and precipitation will be critical for future performance of the project. Recommendations for drainage are provided in the S*urface Drainage* section of this report (see Section 7.18).

6. GEOLOGIC HAZARDS

6.1 Surface Fault Rupture

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

The site is not located within a state-designated Alquist-Priolo Earthquake Fault Zone for surface fault rupture hazards (CGS, 2020a; CGS, 2020b). 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 on Figure 3, Regional Fault Map.

The closest surface trace of an active fault to the site is the San Jacinto Fault Zone, located approximately 4.8 mile to the northeast (USGS, 2006; Ziony and Jones, 1989). Other nearby active faults include the Sierra Madre Fault Zone, the Red Hill Fault, the Chino Fault, and the Elsinore Fault located approximately 8.1 miles north-northeast, 8.6 miles northwest, 15.5 miles southwest, and 15.5 miles southwest of the site, respectively. The active San Andreas Fault Zone is located approximately 12.3 miles northeast of the site (USGS, 2006; Ziony and Jones, 1989).

Several buried thrust faults, commonly referred to as blind thrusts, underlie the Los Angeles Basin at depth. These faults are not exposed at the ground surface and are typically identified at depths greater than 3.0 kilometers. The October 1, 1987, M_w 5.9 Whittier Narrows earthquake and the January 17, 1994, M_w 6.7 Northridge earthquake were a result of movement on the Puente Hills Blind Thrust and the Northridge Thrust, respectively. These thrust faults and others in the 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
Near Redlands	July 23, 1923	6.3	11	ESE
Long Beach	March 10, 1933	6.4	43	SW
Tehachapi	July 21, 1952	7.5	111	NW
San Fernando	February 9, 1971	6.6	60	WNW
Whittier Narrows	October 1, 1987	5.9	37	W
Sierra Madre	June 28, 1991	5.8	35	WNW
Landers	June 28, 1992	7.3	58	E
Big Bear	June 28, 1992	6.4	36	ENE
Northridge	January 17, 1994	6.7	64	W
Hector Mine	October 16, 1999	7.1	77	ENE
Ridgecrest	July 5, 2019	7.1	119	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 online application *Seismic Design Maps*, provided by OSHPD. The short spectral response uses a period of 0.2 second. We evaluated the Site Class based on the discussion in Section 1613.2.2 of the 2019 CBC and Table 20.3-1 of ASCE 7-16. The values presented below are for the risk-targeted maximum considered earthquake (MCE_R).

2019 CBC SEISMIC DESIGN PARAMETERS

Parameter	Value	2019 CBC Reference
Site Class	D	Section 1613.2.2
MCE_R Ground Motion Spectral Response Acceleration – Class B (short), S_S	1.593g	Figure 1613.2.1(1)
MCE_R Ground Motion Spectral Response Acceleration – Class B (1 sec), S_1	0.6g	Figure 1613.2.1(2)
Site Coefficient, F_A	1.0	Table 1613.2.3(1)
Site Coefficient, F_V	1.7*	Table 1613.2.3(2)
Site Class Modified MCE_R Spectral Response Acceleration (short), S_{MS}	1.593g	Section 1613.2.3 (Eqn 16-36)
Site Class Modified MCE_R Spectral Response Acceleration – (1 sec), S_{M1}	1.02g*	Section 1613.2.3 (Eqn 16-37)
5% Damped Design Spectral Response Acceleration (short), S_{DS}	1.062g	Section 1613.2.4 (Eqn 16-38)
5% Damped Design Spectral Response Acceleration (1 sec), S_{D1}	0.68g*	Section 1613.2.4 (Eqn 16-39)

Note:

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

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

ASCE 7-16 PEAK GROUND ACCELERATION

Parameter	Value	ASCE 7-16 Reference
Mapped MCE_G Peak Ground Acceleration, PGA	0.648g	Figure 22-7
Site Coefficient, F_{PGA}	1.1	Table 11.8-1
Site Class Modified MCE_G Peak Ground Acceleration, PGA_M	0.712g	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 7.05 magnitude event occurring at a hypocentral distance of 12.66 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 6.87 magnitude occurring at a hypocentral distance of 14.63 kilometers from the site.

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

6.4 Liquefaction Potential

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

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

A review of the San Bernardino County Countywide Plan, Hazards Element (2019) indicates that the site is not located within an area identified as having a potential for liquefaction. Also, the groundwater level in the immediate area has been at a depth greater than 200 feet beneath the existing ground surface for over 100 years (Mendenhall, 1097; CDMG, 1976; Chino Basin Watermaster, 2017). Based on these considerations, it is our opinion that the potential for liquefaction and associated ground deformations beneath the site is very low. In addition, appreciable seismically induced settlements are not anticipated subsequent to the recommended site grading.

6.5 Slope Stability

The site is located on the valley floor and topography at the site is relatively level to gently sloping to the southwest. The San Bernardino County Countywide Plan, Hazards Element (2019) indicates 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. Based on a review of the San Bernardino County Countywide Plan, Hazards Element (2019), the site is not located within a potential inundation area for an earthquake-induced dam failure. Therefore, the probability of earthquake-induced flooding is considered very low.

6.7 Tsunamis, Seiches, and Flooding

The project property is not located in a coastal area. Therefore, tsunamis are not considered a significant hazard at the property.

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 not within an area of 100-year or 500-year flooding hazard (San Bernardino County, 2019).

6.8 Oil Fields & Methane Potential

Information on the California Geologic Energy Management Division (CalGEM) Well Finder Website indicates 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 DOGGR.

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

6.9 Subsidence

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

7. CONCLUSIONS AND RECOMMENDATIONS

7.1 General

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

7.1.2 Up to 2 feet of existing artificial fill was encountered during the site investigation. The existing fill encountered is believed to be the result of past grading and construction activities at the site. Deeper fill may exist in other areas of the site that were not directly explored. Future demolition of the existing structures and improvements which occupy the site will likely disturb the upper few feet of existing site soils. It is our opinion that the existing fill, in its present condition, is not suitable for direct support of proposed foundations or slabs. The existing fill and site soils are suitable for re-use as engineered fill provided the recommendations in the Grading section of this report are followed (see Section 7.4).

7.1.3 The results of our laboratory testing indicate that the existing upper alluvial soils are subject to excessive hydro-consolidation upon saturation. Hydro-consolidation is the tendency of a soil structure to collapse upon saturation, resulting in the overall settlement of the effected soils and any overlying soils, foundations, or improvements supported therein. The grading and foundation recommendations presented herein are intended to minimize the potential for settlement as a result of hydro-consolidation.

7.1.4 Based on these considerations, it is recommended that the upper 6 feet of existing earth materials in 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 3 feet beyond the building footprint areas, including building appurtenances, or a distance equal to the depth of fill below the foundation, whichever is greater. 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).

7.1.5 Subsequent to the recommended grading, the proposed structures may be supported on conventional shallow spread foundations deriving support in newly placed engineered fill. All foundations should be underlain by a minimum of three feet of newly placed engineered fill. Recommendations for the design of a conventional foundation system are provided in Section 7.6.

7.1.6 All excavations must be observed and approved in writing by the Geotechnical Engineer (a representative of Geocon).

7.1.7 It is anticipated that stable excavations for the recommended grading associated with the proposed 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 (Section 7.16).

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 at or below a depth of 18 inches, and should be deepened as necessary to maintain a minimum 12-inch embedment into the recommended bearing materials. If the soils exposed in the excavation bottom are soft or loose, compaction of the soils will be required prior to placing steel or concrete. Compaction of the foundation excavation bottom is typically accomplished with a compaction wheel or mechanical whacker and must be observed and approved by a Geocon representative.

7.1.9 Where new paving is to be placed in nonbuilding areas, it is recommended that all existing fill and soft alluvial soils be excavated and properly compacted for paving support. The client should be aware that excavation and compaction of all existing fill and soft alluvial soils in the area of new paving is not required; however, paving constructed over existing uncertified fill or unsuitable alluvial soil may experience increased settlement and/or cracking, and may therefore have a shorter design life and increased maintenance costs. As a minimum, the upper 12 inches of subgrade soil should be scarified and properly compacted for paving support. Paving recommendations are provided in *Preliminary Paving Design* 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. Geocon should be contacted to determine the necessity for review and possible revision of this report.

7.1.12 The most recent ASTM standards apply to this project and must be utilized, even if older ASTM standards are indicated in 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. Due to the granular nature of the soils, moderate caving should be anticipated in vertical excavations, especially where granular soils are encountered. In addition, the contractor should also be aware that formwork may be required to prevent caving of shallow spread foundation excavations.

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

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

7.2.4 The upper 5 feet of existing site soils encountered during the investigation are considered to have a "very low" expansive potential (EI = 0) 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 to mildly corrosive" with respect to corrosion of buried ferrous metals on site. The results are presented in Appendix B (Figure B23) and should be considered for design of underground structures.

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 B23) 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 Earthwork should be observed, and compacted fill tested by representatives of Geocon West, Inc. The existing fill and alluvial soils 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.

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

7.4.3 Grading should commence with the removal of 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. 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 approved in writing by the Geotechnical Engineer (a representative of Geocon West, Inc.).

7.4.4 As a minimum, it is recommended that the upper 6 feet of existing earth materials within the proposed building footprint areas be excavated and properly compacted for foundation and slab support. All foundations should be underlain by a minimum of three feet of newly placed engineered fill and deeper excavations should be conducted as necessary to maintain three feet of fill below foundations. Deeper excavations should also be conducted as necessary to remove deeper artificial fill or soft alluvial soil at the direction of the Geotechnical Engineer (a representative of Geocon). The excavation should extend laterally a minimum distance of 3 feet beyond the building footprint area, including building appurtenances, or a distance equal to the depth of fill below the foundation, whichever is greater. The limits of existing fill and/or soft alluvial soils removal will be verified by the Geocon representative during site grading activities.

7.4.5 All excavations must be observed and approved in writing by the Geotechnical Engineer (a representative of Geocon). Prior to placing any fill, the upper twelve inches of 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.).

7.4.6 Subsequent to the recommended grading, the proposed structures may be supported on conventional shallow spread foundations deriving support in newly placed engineered fill.

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

7.4.8. 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 near 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 Paving Design* section of this report (see Section 7.11).

7.4.9 It is anticipated that stable excavations for the recommended grading 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 the existing offsite improvements. Excavation recommendations are provided in the *Temporary Excavations* section of this report (Section 7.16).

7.4.10 Foundations for small outlying structures, such as block walls up to 6 feet high, planter walls or trash enclosures, which will not be tied to the proposed building, may be supported on conventional foundations deriving support on a minimum of 12 inches of newly placed engineered fill which extends laterally at least 12 inches beyond the foundation area. Where excavation and proper compaction cannot be performed or is undesirable, foundations may derive support directly in the undisturbed alluvial soils at or below a depth of 18 inches, and should be deepened as necessary to maintain a minimum 12-inch embedment into the recommended bearing materials. If the soils exposed in the excavation bottom are soft or loose, compaction of the soils will be required prior to placing steel or concrete. Compaction of the foundation excavation bottom is typically accomplished with a compaction wheel or mechanical whacker and must be observed and approved by a Geocon representative.

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

7.4.12 All imported fill shall be observed, tested, and approved by Geocon West, Inc. prior to bringing soil to the site. Rocks larger than 6 inches in diameter shall not be used in the fill. If necessary, import soils used as structural fill should have an expansion index less than 20 and corrosivity properties that are equally or less detrimental to that of the existing onsite soils (see Figure B23). Import soils placed in the building area should be placed uniformly across the building pad or in a manner that is approved by the Geotechnical Engineer (a representative of Geocon).

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

7.5 Shrinkage

7.5.1 Shrinkage results when a volume of material removed at one density is compacted to a higher density. A shrinkage factor between 5 and 10 percent should be anticipated when excavating and compacting the upper 6 feet of existing earth materials on the site to an average relative compaction of 92 percent.

7.4.2 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.). Soils can be borrowed from non-building pad areas and later replaced with imported soils.

7.6 Foundation Design

7.6.1 Subsequent to the recommended grading, the proposed structures may be supported on conventional shallow spread foundations deriving support in newly placed engineered fill.

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 materials.

7.6.3 Isolated spread foundations may be designed for an allowable bearing capacity of 3,000 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 materials.

7.6.4 The soil bearing pressures above may be increased by 350 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 the exterior wall footings, this office should be provided a copy of the final construction plans so that the excavation recommendations presented herein could be properly reviewed and revised if necessary.

7.6.7 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 moisture in the foundation subgrade should be sprinkled as necessary to maintain a moist condition at the time of 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 of the proposed structures supported on conventional foundations deriving support in the recommended bearing material, 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 ½ inch over a distance of 20 feet.

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

7.8 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 at and below a depth of 18 inches, and should be deepened as necessary to maintain a minimum 12-inch embedment into the recommended bearing materials.

7.8.2 If the soils exposed in the excavation bottom are soft, compaction of the soft soils will be required prior to placing steel or concrete. Compaction of the foundation excavation bottom is typically accomplished with a compaction wheel or mechanical whacker and must be observed and approved by a Geocon representative. Miscellaneous foundations may be designed for a bearing value of 1,500 psf, and should be a minimum of 12 inches in width, 18 inches in depth below the lowest adjacent grade and 12 inches into the recommended bearing material. The allowable bearing pressure may be increased by up to one-third for transient loads due to wind or seismic forces.

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

7.9 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 competent alluvial soils or in properly compacted engineered fill.

7.9.2 Passive earth pressure for the sides of foundations and slabs poured against properly compacted engineered fill or competent alluvial soils may be computed as an equivalent fluid having a density of 260 pounds per cubic foot (pcf) with a maximum earth pressure of 2,600 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 Concrete slabs-on-grade subject to vehicle loading should be designed in accordance with the recommendations in the *Preliminary Paving Design* section of this report (Section 7.11).

7.10.2 Unless specifically evaluated and designed by a qualified structural engineer, 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 concrete slab-on-grade may derive support directly on the newly placed engineered fill subsequent the grading.

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

7.10.4 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 Paving Design

7.11.1 Where new paving is to be placed, it is recommended that all existing fill and soft or unsuitable alluvial materials be excavated and properly recompacted for paving support. The client should be aware that excavation and compaction of all existing artificial fill and soft alluvium in the area of new paving is not required; however, paving constructed over existing unsuitable material may experience increased settlement and/or cracking, and may therefore have a shorter design life and increased maintenance costs. As a minimum, the upper 12 inches of paving subgrade should be scarified, moisture conditioned to 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 site specific 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 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 5 inches of concrete reinforced with No. 3 steel reinforcing bars placed 18 inches on center in both horizontal directions. Concrete paving supporting vehicular traffic should be underlain by a minimum of 4 inches of aggregate base and a properly compacted subgrade. The subgrade and base material should be compacted to 95 percent relative compaction, respectively, 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 5 feet. In the event that walls higher than 5 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) of 30 pcf.

7.12.4 Restrained walls are those that are not allowed to rotate more than 0.001H (where H equals the height of the retaining portion of the wall in feet) at the top of the wall. Where walls are restrained from movement at the top, walls may be designed utilizing a triangular distribution of pressure (at-rest pressure) of 59 pcf.

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

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

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

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

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

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

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*.

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

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

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

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*.

7.12.10 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

7.13 Retaining Wall Drainage

7.13.1 Retaining walls not designed for hydrostatic pressure 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.

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

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

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

7.14 Elevator Pit Design

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

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

7.14.3 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 7.13).

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

7.15 Elevator Piston

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

7.15.2 Caving is anticipated especially where granular soils are encountered. The contractor should be prepared to use casing and should have it readily available at the commencement of drilling activities. Continuous observation of the drilling and installation of the elevator piston by the Geotechnical Engineer (a representative of Geocon West, Inc.) is required.

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

7.16 Temporary Excavations

7.16.1 Excavations on the order of 5 feet in height may be required during grading operations. The excavations are expected to expose artificial fill and alluvial soils, which are suitable for vertical excavations up to 5 feet in height where loose soils or caving sands are not present, and where not surcharged by adjacent traffic or structures.

7.16.2 Vertical excavations greater than 5 feet or where surcharged by existing structures will require sloping or shoring measures in order to provide a stable excavation. Where sufficient space is available, temporary unsurcharged embankments could be sloped back at a uniform 1:1 slope gradient or flatter up to maximum height of 7 feet. A uniform slope does not have a vertical portion.

7.16.3 If excavations in close proximity to an adjacent property line and/or structure are required, special excavation measures such as slot-cutting or shoring may be necessary in order to maintain lateral support of offsite improvements. Recommendations for special excavation measures can be provided under separate cover.

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

7.17 Surface Drainage

7.17.1 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.

7.17.2 All site drainage should be collected and controlled in non-erosive drainage devices. Drainage should not be allowed to pond anywhere on the site, and especially not against any foundation or retaining wall. The site should be graded and maintained such that surface drainage is directed away from structures in accordance with 2019 CBC 1804.4 or other applicable standards. In addition, drainage should not be allowed to flow uncontrolled over any descending slope. Discharge from downspouts, roof drains and scuppers are not recommended onto unprotected soils within 5 feet of the building perimeter. Planters which are located adjacent to foundations should be sealed to prevent moisture intrusion into the soils providing foundation support. Landscape irrigation is not recommended within 5 feet of the building perimeter footings except when enclosed in protected planters.

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

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

7.18 Plan Review

7.18.1 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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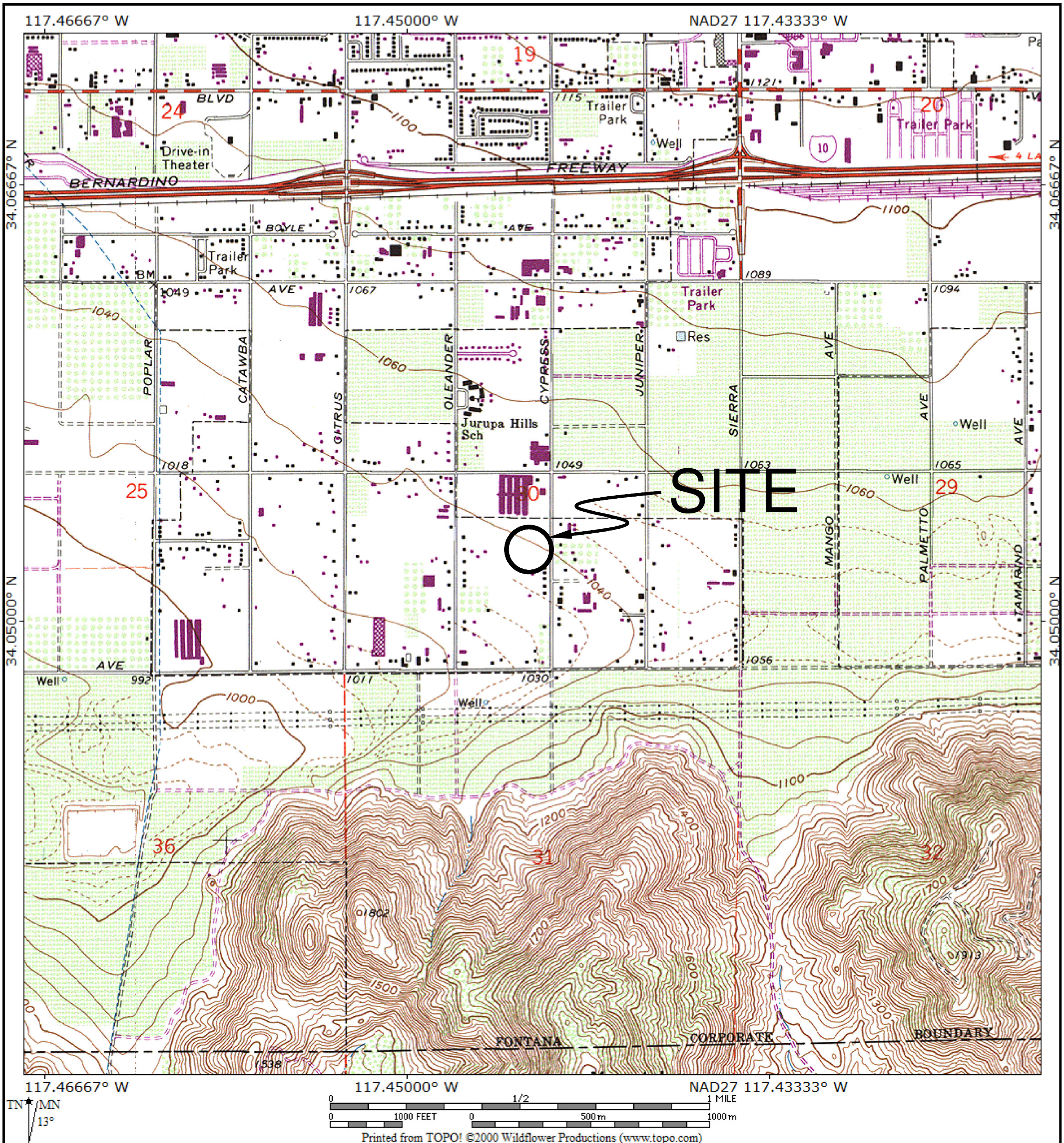
USGS, Unified Hazard Tool, https://earthquake.usgs.gov/hazards/interactive/, accessed January 2020.

Wildermuth Environmental, Inc., 2012, *Chino Basin Optimum Basin Management Program 2012 State of the Basin Atlas*. Figure 1. Depth to Groundwater Contours Chino Basin –Fall 2006.

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



U.S.G.S. TOPOGRAPHIC MAPS, 7.5 MINUTE SERIES, FONTANA, CA QUADRANGLE

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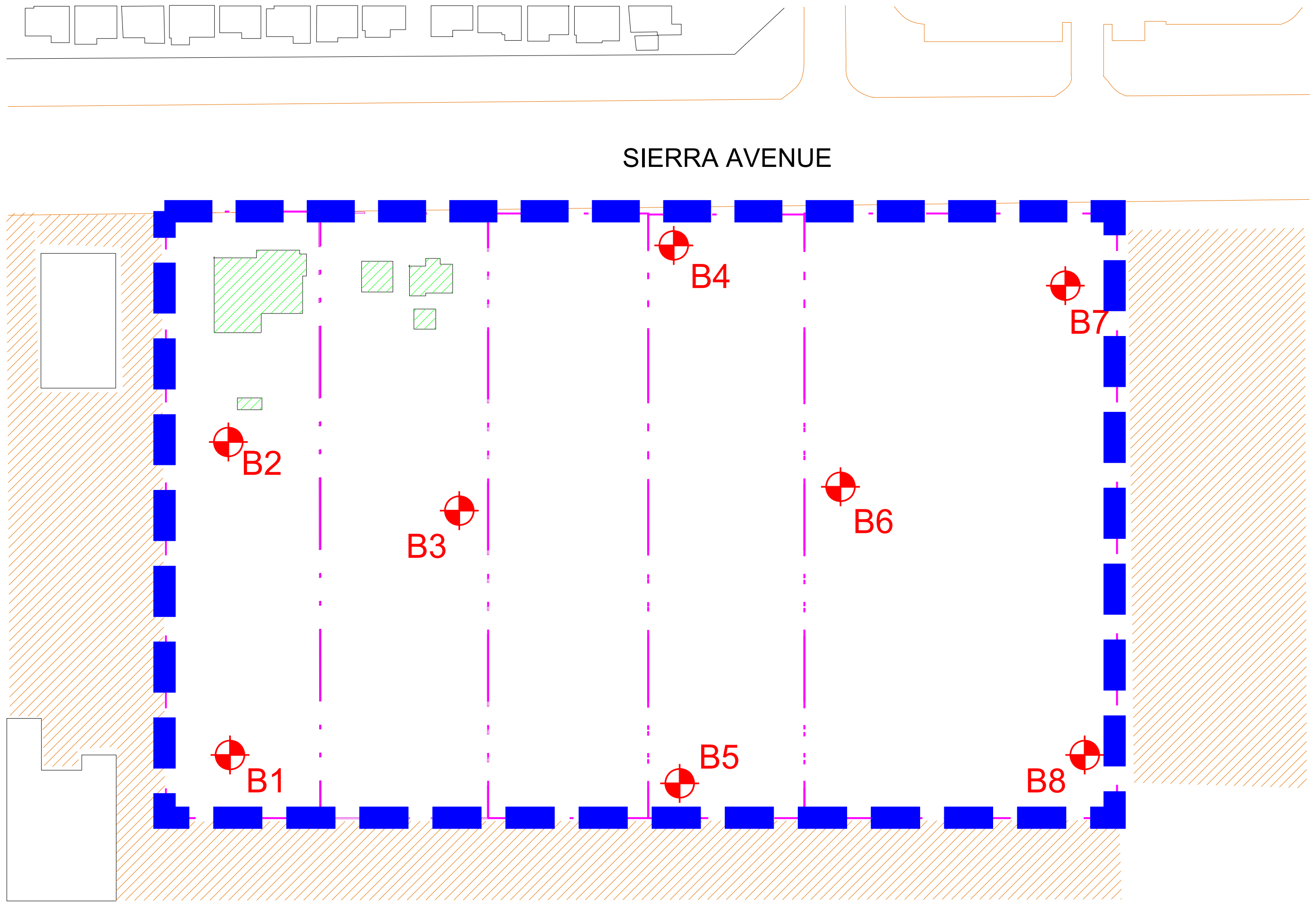
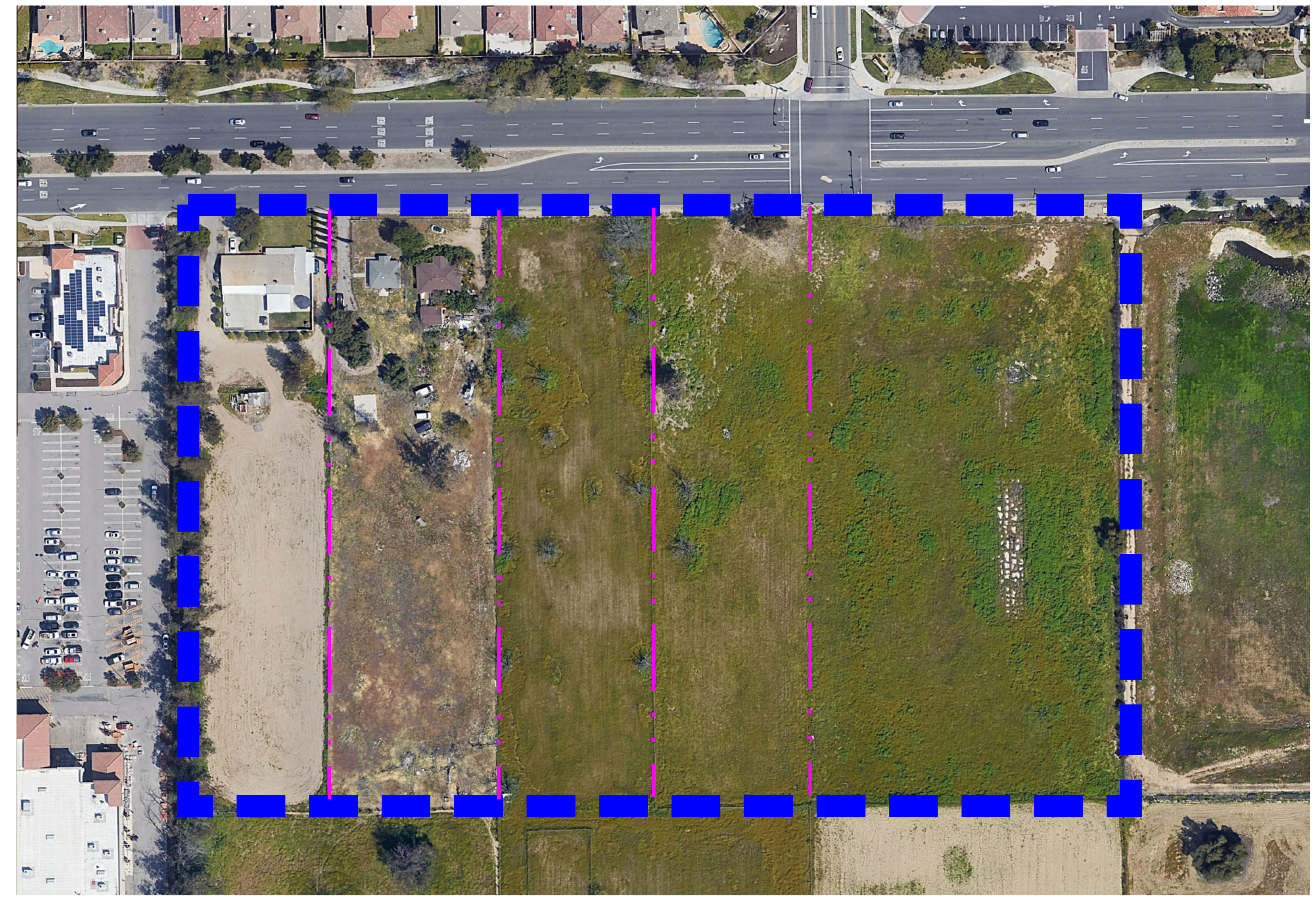
VICINITY MAP

11016 - 11098 SIERRA AVENUE
FONTANA, CALIFORNIA

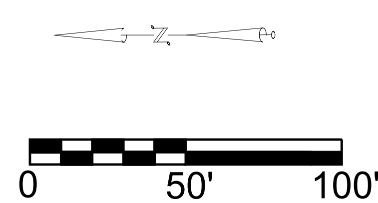
DRAFTED BY: JA	CHECKED BY: SFK	FEB. 2020	PROJECT NO. T2746-99-10A	FIG. 1





Legend

- Parcel Limits
- Existing Structures
- B1 Approximate Boring Locations
- Limit of Proposed Improvements



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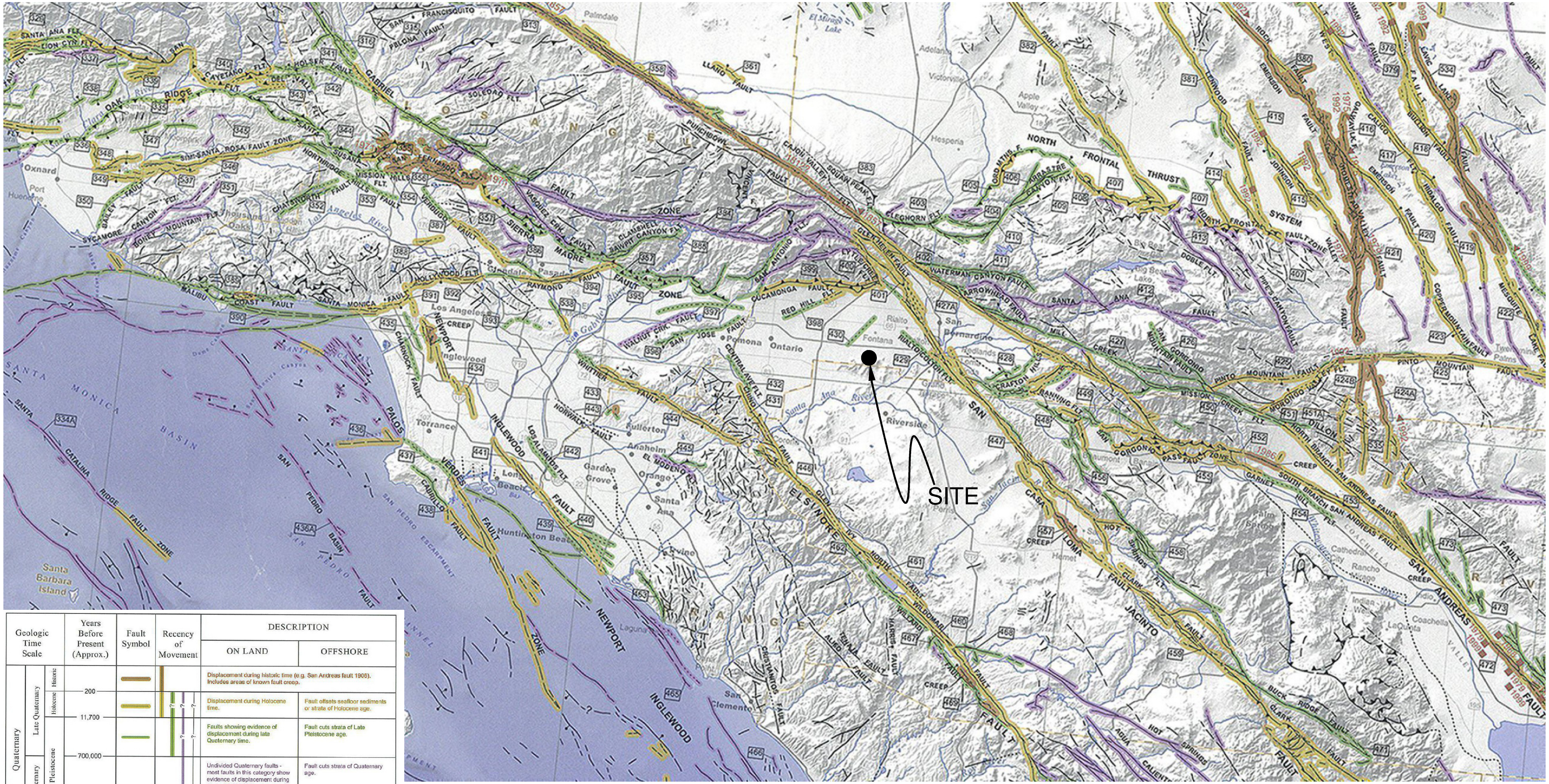
SITE PLAN

11016-11098 SIERRA AVENUE
FONTANA, CALIFORNIA

FEB. 2020 | PROJECT NO: T2746-99-10A | FIG. 2

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



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

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

N

0 12 24 Miles

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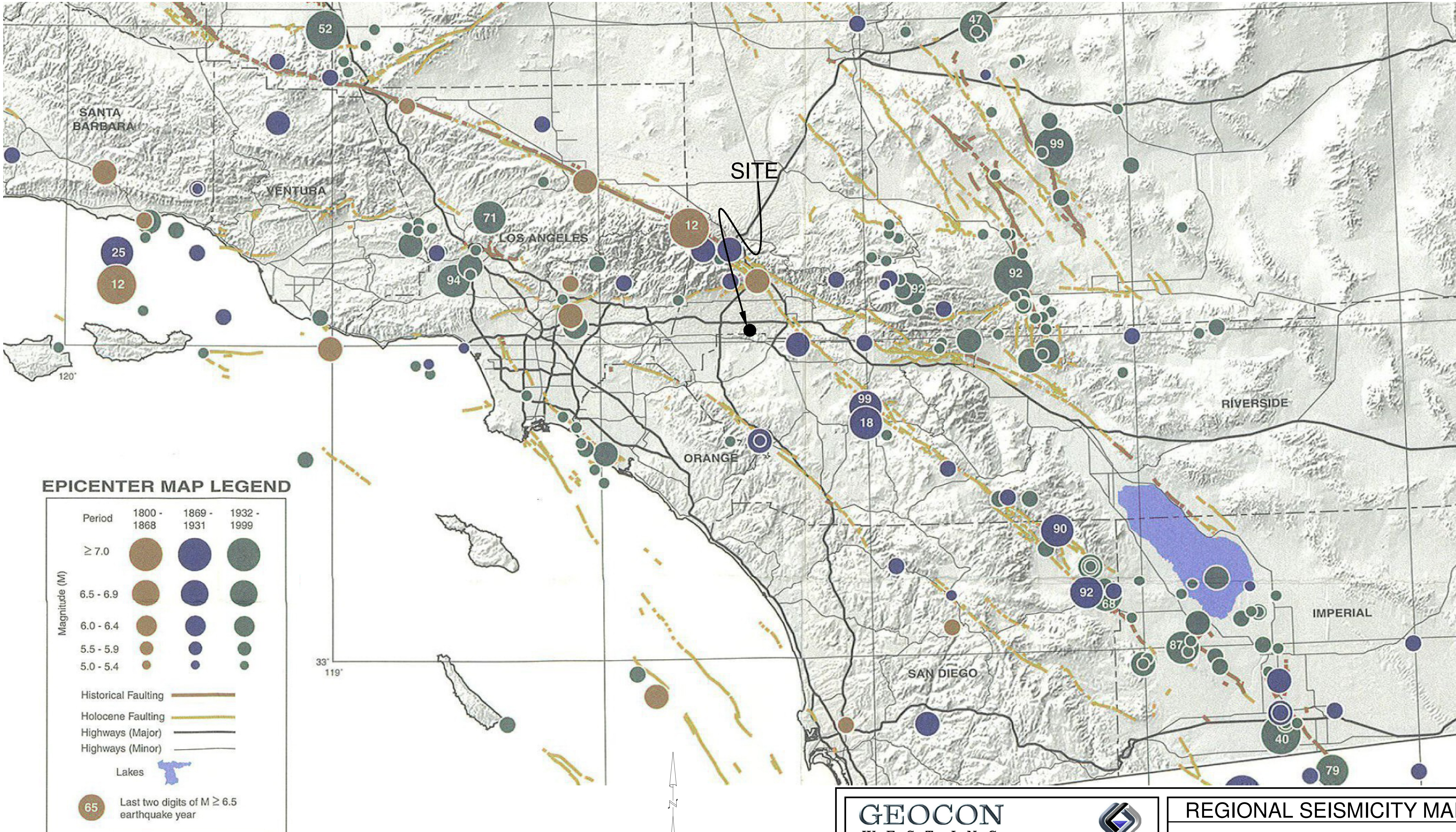
REGIONAL FAULT MAP

11016 - 11098 SIERRA AVENUE
FONTANA, CALIFORNIA

FEB. 2020 | PROJECT NO. T2746-99-10A | 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.

N

0 20 40 Miles

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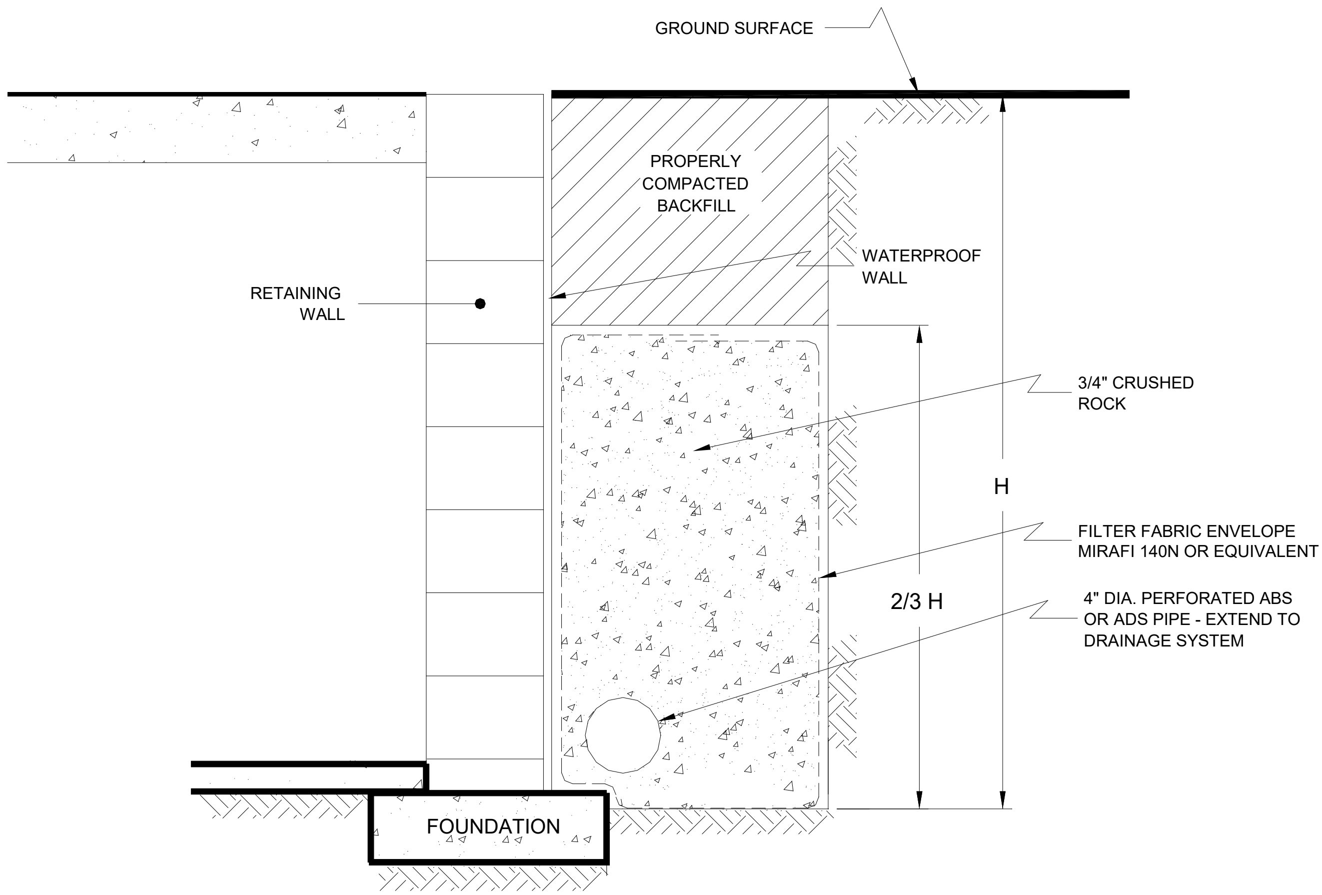
REGIONAL SEISMICITY MAP

11016 - 11098 SIERRA AVENUE
FONTANA, CALIFORNIA

FEB. 2020 | PROJECT NO. T2746-99-10A | FIG.4



NO SCALE

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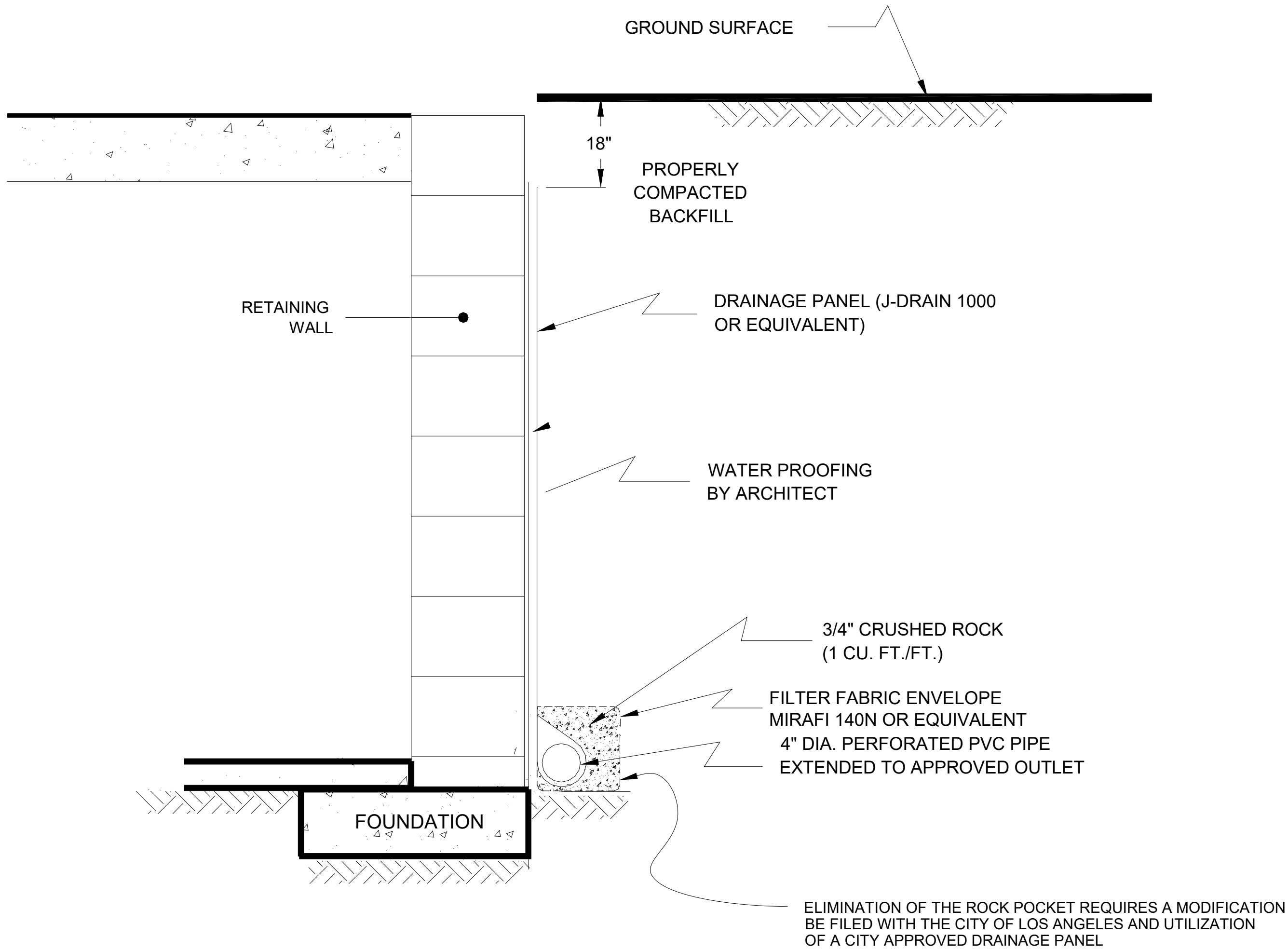
RETAINING WALL DRAIN DETAIL

11016-11098 SIERRA AVENUE

FONTANA, CALIFORNIA

FEB. 2020	PROJECT NO. T2746-99-10A	FIG. 5



NO SCALE

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RETAINING WALL DRAIN DETAIL

11016-11098 SIERRA AVENUE

FONTANA, CALIFORNIA

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APPENDIX A

APPENDIX A

FIELD INVESTIGATION

The site was explored on January 23, 2020, by excavating eight 8-inch diameter borings to depths between 10½ and 20½ feet below the existing ground surface using a truck-mounted hollow-stem auger drilling machine. Representative and relatively undisturbed samples were obtained by driving a 3-inch, O. D., California Modified Sampler into the "undisturbed" soil mass with blows from a 140-pound auto-hammer falling 30 inches. The California Modified Sampler was equipped with 1-inch by 2³/₈-inch diameter brass sampler rings to facilitate soil removal and testing. Bulk samples were also obtained.

The soil conditions encountered in the borings were visually examined, classified and logged in general accordance with the Unified Soil Classification System (USCS). The logs of the borings are presented on Figures A1 through A8. The logs depict the soil and geologic conditions encountered and the depth at which samples were obtained. The logs also include our interpretation of the conditions between sampling intervals. Therefore, the logs contain both observed and interpreted data. We determined the lines designating the interface between soil materials on the logs using visual observations, penetration rates, excavation characteristics and other factors. The transition between materials may be abrupt or gradual. Where applicable, the logs were revised based on subsequent laboratory testing. The location of the borings are shown on Figure 2.

BORING 1

ELEV. (MSL.) -- DATE COMPLETED 1/23/2020

EQUIPMENT HOLLOW STEM AUGER BY: JJK

DEPTH IN FEET	SAMPLE NO.	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0	BULK 0-5'		**ARTIFICIAL FILL** Silty Sand, loose, slightly moist, light brown, fine- to medium-grained, trace fine gravel.			
2	B1@2'	SM	**ALLUVIUM** Silty Sand, loose, slightly moist to moist, loose, yellowish brown, fine-grained.	12	110.8	4.6
	B1@5'		- medium dense, dry to slightly moist, light brown	38	115.2	2.2
6	B1@7'	ML	Sandy Silt, stiff, slightly moist, light brown.	28	107.5	5.3
10	B1@10'		- hard	53	107.5	5.3
12.5	B1@15'	SM	Silty Sand, dense, slightly moist, brown, fine-grained, some medium- to coarse-grained.	73	122.1	1.8
17.5	B1@20'	ML	Silt with Sand, hard, slightly moist, light brown.	60	103.8	4.5

Total depth of boring: 20.5 feet
Fill to 2 feet.
No groundwater encountered.
Backfilled with soil cuttings and tamped.

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

T2746-99-10A BORING LOGS.GPJ

Figure A1,
Log of Boring 1, 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. T2749-99-10A

BORING 2

ELEV. (MSL.) -- DATE COMPLETED 1/23/2020

EQUIPMENT HOLLOW STEM AUGER BY: JJK

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0					**ARTIFICIAL FILL** Silty Sand, loose, slightly moist, brown, fine-grained, trace fine gravel.			
2	B2@2'			SM	**ALLUVIUM** Silty Sand, medium dense, slightly moist, light brown to brown, fine-grained, trace coarse-grained.	37	105.0	10.8
4								
	B2@5'				- some medium- and coarse-grained	29	115.7	2.4
6				ML	Sandy Silt, hard, dry to slightly moist, light brown, fine-grained.			
	B2@7'					46	112.9	3.5
8								
				ML	Silt with Sand, firm, dry to slightly moist, light brown.			
10	B2@10'					28	105.4	3.7
12								
14								
	B2@15'				- hard, slightly moist	46	105.2	5.4
16								
18				SM	Silty Sand, medium dense, slightly moist, grayish brown, fine-grained, some medium-grained.			
20	B2@20'					24	123.5	2.2

Total depth of boring: 20.5 feet
Fill to 2 feet.
No groundwater encountered.
Backfilled with soil cuttings and tamped.

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

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

T2746-99-10A BORING LOGS.GPJ

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

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

GEOCON

BORING 3

ELEV. (MSL.) -- DATE COMPLETED 1/23/2020

EQUIPMENT HOLLOW STEM AUGER BY: JJK

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0					**ARTIFICIAL FILL** Silty Sand, loose, slightly moist, brown, fine- to medium-grained.			
2	B3@2'			SM	**ALLUVIUM** Silty Sand, loose, slightly moist to moist, brown, trace fine gravel and fine-grained sand.	12	105.6	10.9
4								
6	B3@5'				- medium dense, dry to slighlty moist, light brown, fine-grained, some fine gravel	40	127.0	2.1
8	B3@7'			ML	Sandy Silt, hard, dry to slightly moist, light brown, trace fine gravel.	44	119.9	3.0
10	B3@10'			ML	Silt, hard, dry to slightly moist, light brown, trace fine-grained sand.	78	115.5	3.5

Total depth of boring: 10.5 feet
Fill to 2 feet.
No groundwater encountered.
Backfilled with soil cuttings and tamped.

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

T2746-99-10A BORING LOGS.GPJ

Figure A3,
Log of Boring 3, 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. T2749-99-10A

BORING 4

ELEV. (MSL.) -- DATE COMPLETED 1/23/2020

EQUIPMENT HOLLOW STEM AUGER BY: JJK

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0					**ARTIFICIAL FILL** Silty Sand, loose, slightly moist, light brown, fine- to medium-grained.			
2	B4@2'			SM	**ALLUVIUM** Silty Sand, medium dense, slightly moist, brown, fine-grained.	27	110.6	16.4
5	B4@5'				- dry, light grayish brown, some coarse-grained	31	115.4	1.2
7	B4@7'				- trace medium- to coarse-grained	28	115.2	2.9
9				ML	Sandy Silt, firm, slightly moist, brown, fine-grained, trace medium-grained sand.			
10	B4@10'					21	112.4	5.3
13				ML	Silt with Sand, stiff, slighlty moist, brown, trace fine-grained.			
15	B4@15'					40	111.0	6.7
18				SP-SM	Sand with Silt, dense, dry, grayish brown, fine-grained, some medium-grained.			
20	B4@20'					54	116.0	1.2

Total depth of boring: 20.5 feet
Fill to 2 feet.
No groundwater encountered.
Backfilled with soil cuttings and tamped.

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

T2746-99-10A BORING LOGS.GPJ

Figure A4,
Log of Boring 4, 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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DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	BORING 5 ELEV. (MSL.) -- DATE COMPLETED 1/23/2020 EQUIPMENT HOLLOW STEM AUGER BY: JJK	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
					MATERIAL DESCRIPTION			
0	BULK 0-5'				**ARTIFICIAL FILL** Silty Sand, loose, slightly moist to moist, brown, fine-grained sand.			
2	B5@2'			SM	**ALLUVIUM** Silty Sand, medium dense, slightly moist, brown, fine-grained sand.	19	119.4	7.4
4								
	B5@5'			ML	Sandy Silt, hard, dry, light brown, trace fine-grained. - dry light brown, trace fine gravel	67	116.5	2.4
6					Silt with Sand, hard, dry, light brown, trace fine-grained.			
	B5@7'			SP-SM		62	110.9	3.3
8					Silty Sand, medium dense, dry, grayish brown, fine- to medium-grained, trace coarse-grained and fine gravel.			
10	B5@10'					47	117.3	1.1
12								
14				SM				
	B5@15'				- light brown, fine-grained, no medium- or coarse-grained, no fine gravel	38	199.9	2.3
16								
18				SP	Sand, poorly graded, very dense, dry, brown, medium-grained, trace fine gravel.			
20	B5@20'					90		

Total depth of boring: 20.5 feet
Fill to 2 feet.
No groundwater encountered.
Backfilled with soil cuttings and tamped.

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

T2746-99-10A BORING LOGS.GPJ

Figure A5,
Log of Boring 5, 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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BORING 6

ELEV. (MSL.) -- DATE COMPLETED 1/23/2020

EQUIPMENT HOLLOW STEM AUGER BY: JJK

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0					**ARTIFICIAL FILL** Silty Sand, loose, slightly moist to moist, brown, fine-grained.			
2	B6@2'			SM	**ALLUVIUM** Silty Sand, medium dense, slightly moist, brown, fine-grained, trace medium- to coarse-grained.	20	113.4	10.4
4								
	B6@5'				- dry to slightly moist	38	124.0	5.3
6								
	B6@7'				- loose, trace medium-grained, no coarse-grained	18	115.8	2.4
8								
10	B6@10'				- medium dense, no medium-grained	27	113.7	3.9

Total depth of boring: 10.5 feet
Fill to 2 feet.
No groundwater encountered.
Backfilled with soil cuttings and tamped.

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

T2746-99-10A BORING LOGS.GPJ

Figure A6,
Log of Boring 6, 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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BORING 7

ELEV. (MSL.) -- DATE COMPLETED 1/23/2020

EQUIPMENT HOLLOW STEM AUGER BY: JJK

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0	BULK 0-5'				**ARTIFICIAL FILL** Silty Sand, loose, moist, brown, fine- to medium-grained, trace fine gravel.			
2	B7@2'			SM	**ALLUVIUM** Silty Sand, medium dense, moist, brown, fine-grained, trace medium-grained.	24	114.3	6.8
4								
	B7@5'				- loose, no medium-grained sand, decrease in silt	17	112.9	5.5
6								
	B7@7'				- no recovery, large 4" rock in bottom of sampler	37	120.6	4.0
8				ML	Silt with Sand, stiff, slightly moist to moist, brown, trace fine-grained.			
10	B7@10'					25	112.6	8.5
12								
				ML	Sandy Silt, stiff, slightly moist to moist, brown, fine-grained.			
14								
	B7@15'					27	109.5	10.7
16								
18								
20	B7@20'				- firm, trace medium-grained	20	113.8	7.6

Total depth of boring: 20.5 feet
Fill to 2 feet.
No groundwater encountered.
Backfilled with soil cuttings and tamped.

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

T2746-99-10A BORING LOGS.GPJ

Figure A7,
Log of Boring 7, 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. T2749-99-10A

BORING 8

ELEV. (MSL.) -- DATE COMPLETED 1/23/2020

EQUIPMENT HOLLOW STEM AUGER BY: JJK

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0					**ARTIFICIAL FILL** Silty Sand, loose, slightly moist, brown, fine-grained.			
2	B8@2'			SM	**ALLUVIUM** Silty Sand, loose, slightly moist to moist, light brown, fine-grained, trace coarse-grained.	16	101.9	9.9
4					Silt with Sand, firm, slightly moist, olive brown, trace fine-grained, trace plasticity.			
	B8@5'			ML		16	112.7	13.2
6								
	B8@7'				- stiff	36	105.7	10.7
8					Sandy Silt, stiff, slightly moist, light brown, fine-grained.			
10	B8@10'			ML		26	110.6	4.1
12								
14					Silty Sand, dense, dry to slightly moist, grayish brown, fine-grained, some fine gravel.			
	B8@15'					57	123.7	1.1
16								
				SM				
18								
20	B8@20'				- medium dense, fine-grained, no gravel	45	104.9	2.6

Total depth of boring: 20.5 feet
Fill to 2 feet.
No groundwater encountered.
Backfilled with soil cuttings and tamped.

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

T2746-99-10A BORING LOGS.GPJ

Figure A8,
Log of Boring 8, 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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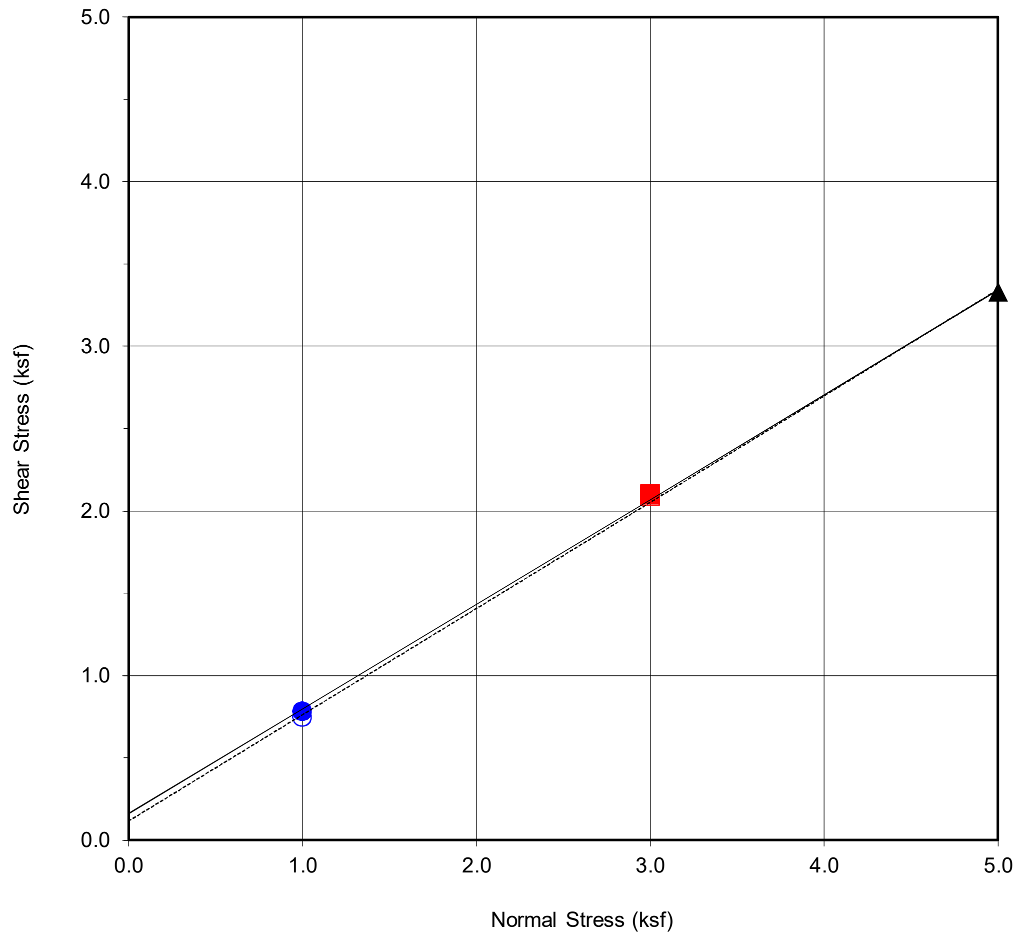
APPENDIX B

APPENDIX B

LABORATORY TESTING

Laboratory tests were performed in accordance with generally accepted test methods of the International ASTM, or other suggested procedures. Selected samples were tested for direct shear strength, compaction, consolidation characteristics, expansive index, corrosivity, grain-size distribution, in-place dry density and moisture content. The results of the laboratory tests are summarized in Figures B1 through B23. The in-place dry density and moisture content of the samples tested are presented on the boring logs, Appendix A.



Boring No.	**B1**
Sample No.	**B1@0-5'**
Depth (ft)	**0-5'**
Sample Type:	Ring

Soil Identification:		
Light Brown Silty Sand (SM)		
Strength Parameters		
	C (psf)	ϕ (°)
Peak	160	32.5
Ultimate	116	32.9

Normal Stress (kip/ft2)	1	3	5
Peak Shear Stress (kip/ft²)	● 0.78	■ 2.10	▲ 3.33
Shear Stress @ End of Test (ksf)	○ 0.75	□ 2.09	△ 3.33
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 (%)	9.4	9.4	9.5
Initial Dry Density (pcf)	106.0	106.0	106.0
Initial Degree of Saturation (%)	43.2	43.1	43.3
Soil Height Before Shearing (in.)	1.2	1.2	1.2
Final Moisture Content (%)	22.7	21.8	22.9

GEOCON

DIRECT SHEAR TEST RESULTS

Consolidated Drained ASTM D-3080

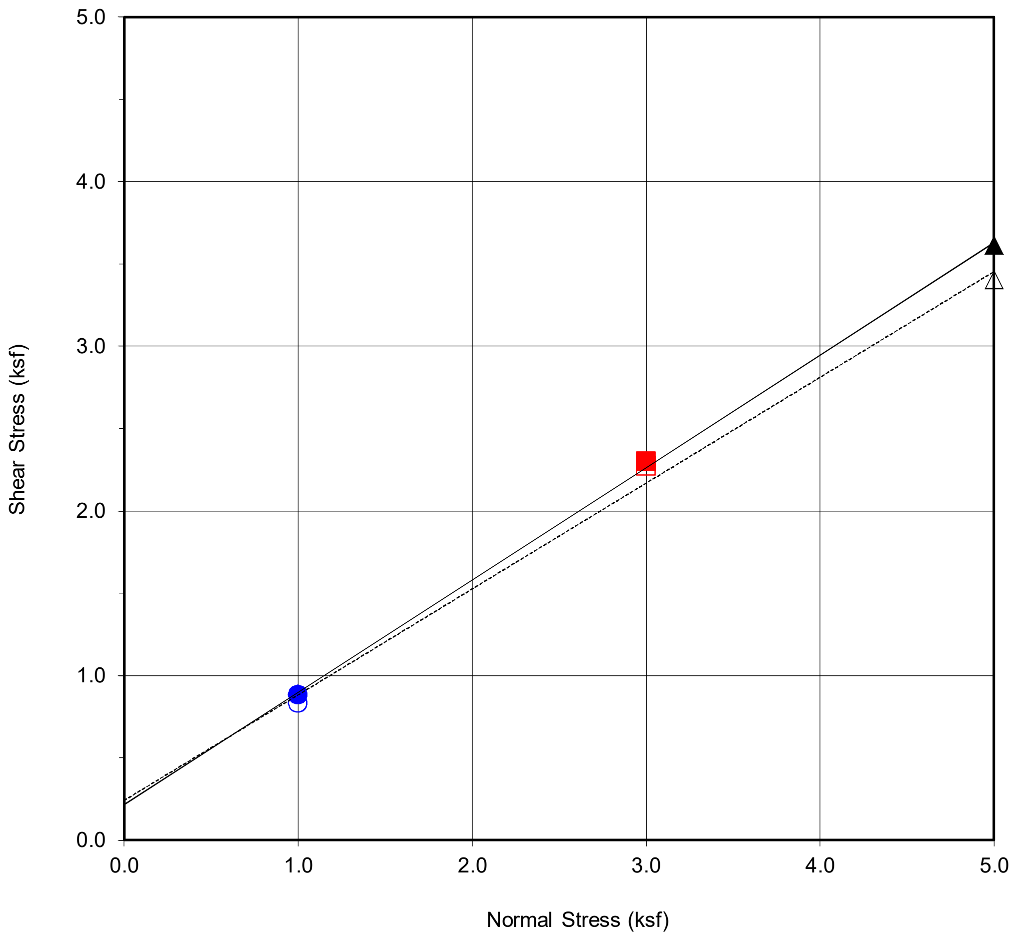
Checked by: JJK

Project No.: T2746-99-10A

11016-11098 SIERRA AVENUE
FONTANA, CALIFORNIA

FEB 2020

Figure B1



Boring No.	B2
Sample No.	B2@2'
Depth (ft)	2
Sample Type:	Ring

Soil Identification:		
Light Brown Silty Sand (SM)		
Strength Parameters		
	C (psf)	ϕ (°)
Peak	216	34.3
Ultimate	241	32.7

Normal Stress (kip/ft2)	1	3	5
Peak Shear Stress (kip/ft²)	● 0.88	■ 2.30	▲ 3.61
Shear Stress @ End of Test (ksf)	○ 0.83	□ 2.27	△ 3.40
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 (%)	9.6	10.1	10.8
Initial Dry Density (pcf)	101.0	106.5	107.3
Initial Degree of Saturation (%)	38.8	46.8	51.2
Soil Height Before Shearing (in.)	1.2	1.2	1.2
Final Moisture Content (%)	18.6	16.5	15.5

GEOCON

DIRECT SHEAR TEST RESULTS

Consolidated Drained ASTM D-3080

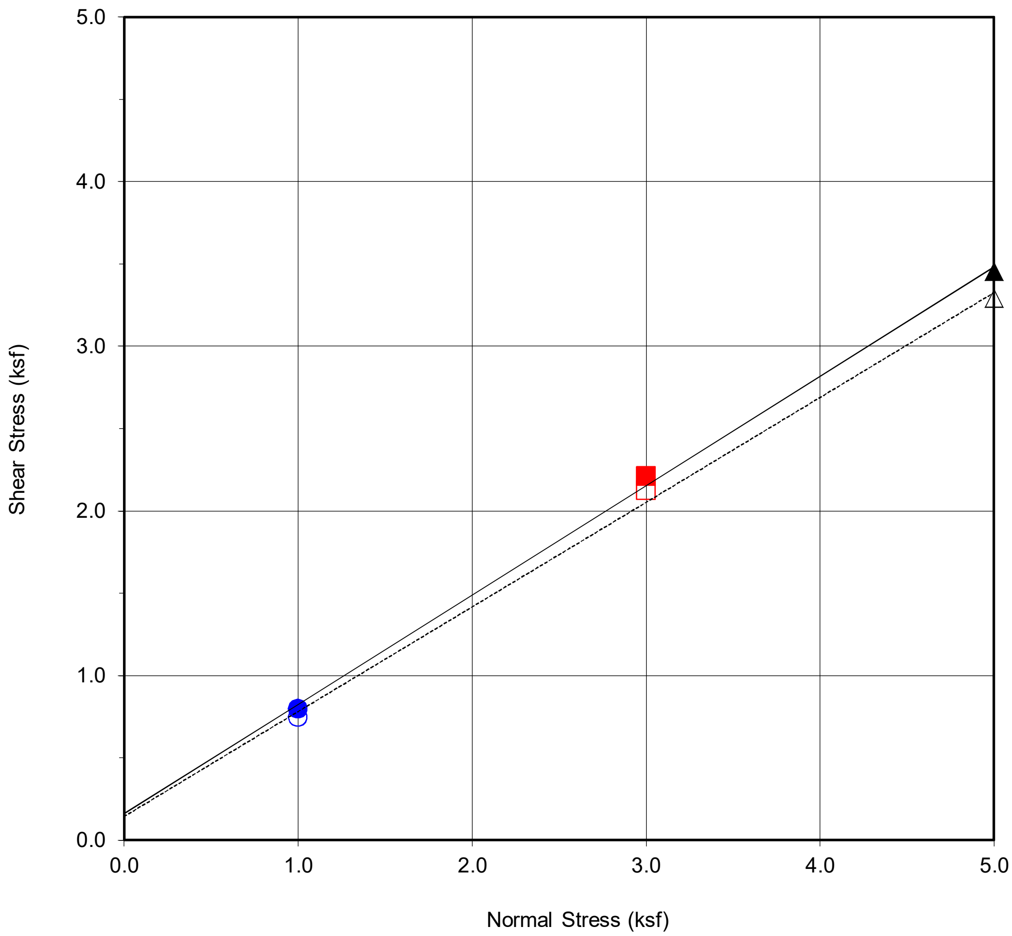
Checked by: JJK

Project No.: T2746-99-10A

11016-11098 SIERRA AVENUE
FONTANA, CALIFORNIA

FEB 2020

Figure B2



Boring No.	**B3**
Sample No.	**B3@2'**
Depth (ft)	**2**
Sample Type:	Ring

Soil Identification:		
Brown Silty Sand (SM)		
Strength Parameters		
	C (psf)	ϕ (°)
Peak	160	33.6
Ultimate	143	32.5

Normal Stress (kip/ft2)	1	3	5
Peak Shear Stress (kip/ft²)	● 0.80	■ 2.21	▲ 3.45
Shear Stress @ End of Test (ksf)	○ 0.75	□ 2.12	△ 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 (%)	9.7	11.7	10.9
Initial Dry Density (pcf)	101.5	103.4	107.4
Initial Degree of Saturation (%)	39.7	50.3	51.9
Soil Height Before Shearing (in.)	1.2	1.2	1.2
Final Moisture Content (%)	16.6	15.3	15.2

GEOCON

DIRECT SHEAR TEST RESULTS

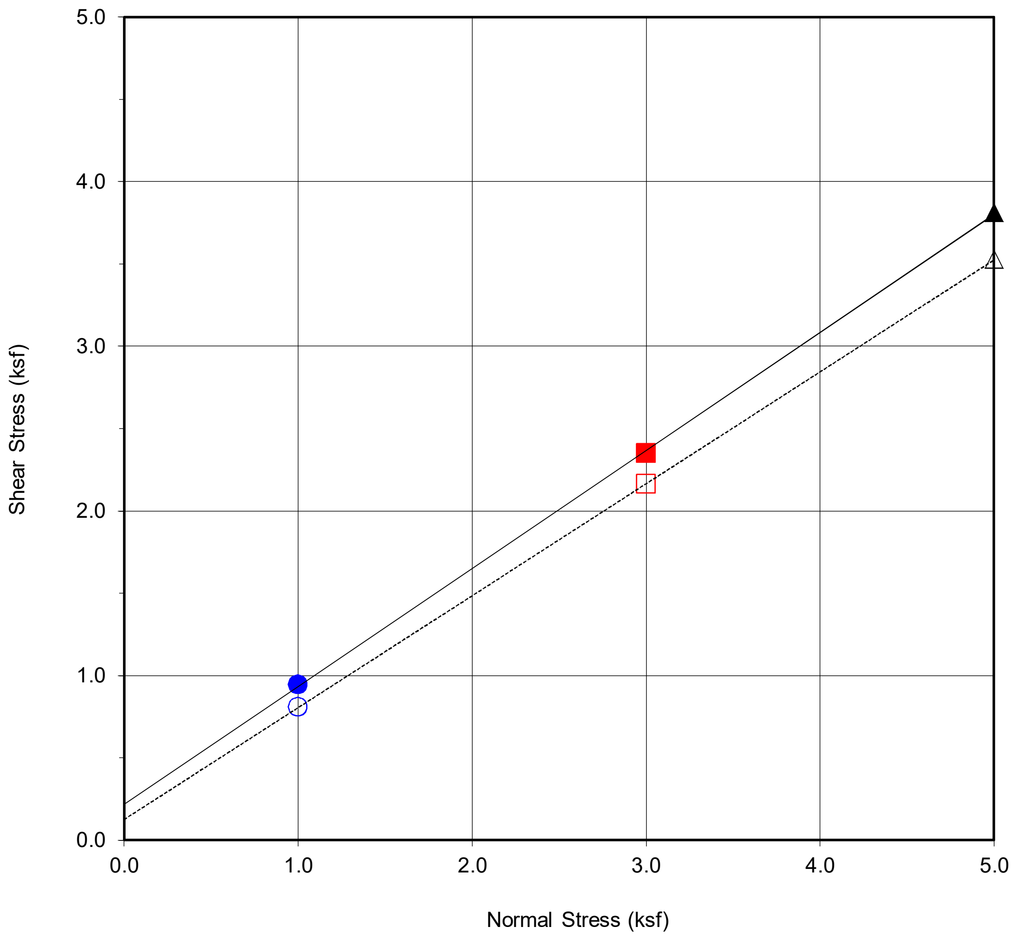
Consolidated Drained ASTM D-3080

Checked by: JJK

Project No.: T2746-99-10A

11016-11098 SIERRA AVENUE
FONTANA, CALIFORNIA

FEB 2020 Figure B3



Boring No.	B4
Sample No.	B4@2'
Depth (ft)	2
Sample Type:	Ring

Normal Stress (kip/ft2)	1	3	5
Peak Shear Stress (kip/ft²)	● 0.94	■ 2.35	▲ 3.81
Shear Stress @ End of Test (ksf)	○ 0.81	□ 2.16	△ 3.53
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 (%)	14.0	12.5	16.4
Initial Dry Density (pcf)	111.7	110.7	111.4
Initial Degree of Saturation (%)	74.4	64.4	86.4
Soil Height Before Shearing (in.)	1.2	1.2	1.2
Final Moisture Content (%)	16.4	15.4	14.8

Soil Identification:

Brown Sandy Silt (ML)

Strength Parameters

	C (psf)	ϕ (°)
Peak	217	35.6
Ultimate	125	34.2

GEOCON

DIRECT SHEAR TEST RESULTS

Consolidated Drained ASTM D-3080

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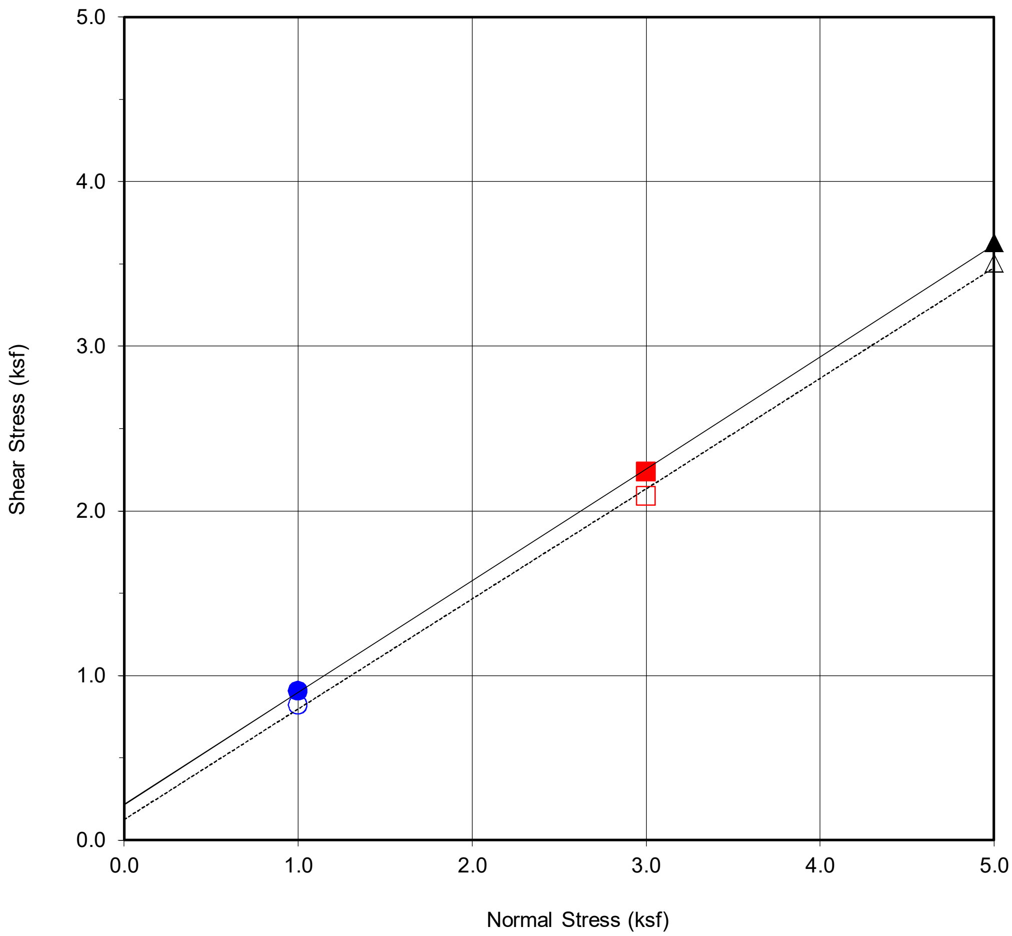
Project No.: T2746-99-10A

11016-11098 SIERRA AVENUE
FONTANA, CALIFORNIA

FEB 2020 Figure B4



Boring No.	B6
Sample No.	B6@2'
Depth (ft)	2
Sample Type:	Ring

Soil Identification:		
Brown Sandy Silt (ML)		
Strength Parameters		
	C (psf)	φ (°)
Peak	216	34.2
Ultimate	125	33.8

Normal Stress (kip/ft2)	1	3	5
Peak Shear Stress (kip/ft²)	● 0.91	■ 2.24	▲ 3.63
Shear Stress @ End of Test (ksf)	○ 0.82	□ 2.09	△ 3.50
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 (%)	10.4	11.8	10.6
Initial Dry Density (pcf)	112.4	111.5	115.0
Initial Degree of Saturation (%)	56.0	62.5	61.5
Soil Height Before Shearing (in.)	1.2	1.2	1.2
Final Moisture Content (%)	16.2	15.6	14.8

GEOCON

DIRECT SHEAR TEST RESULTS

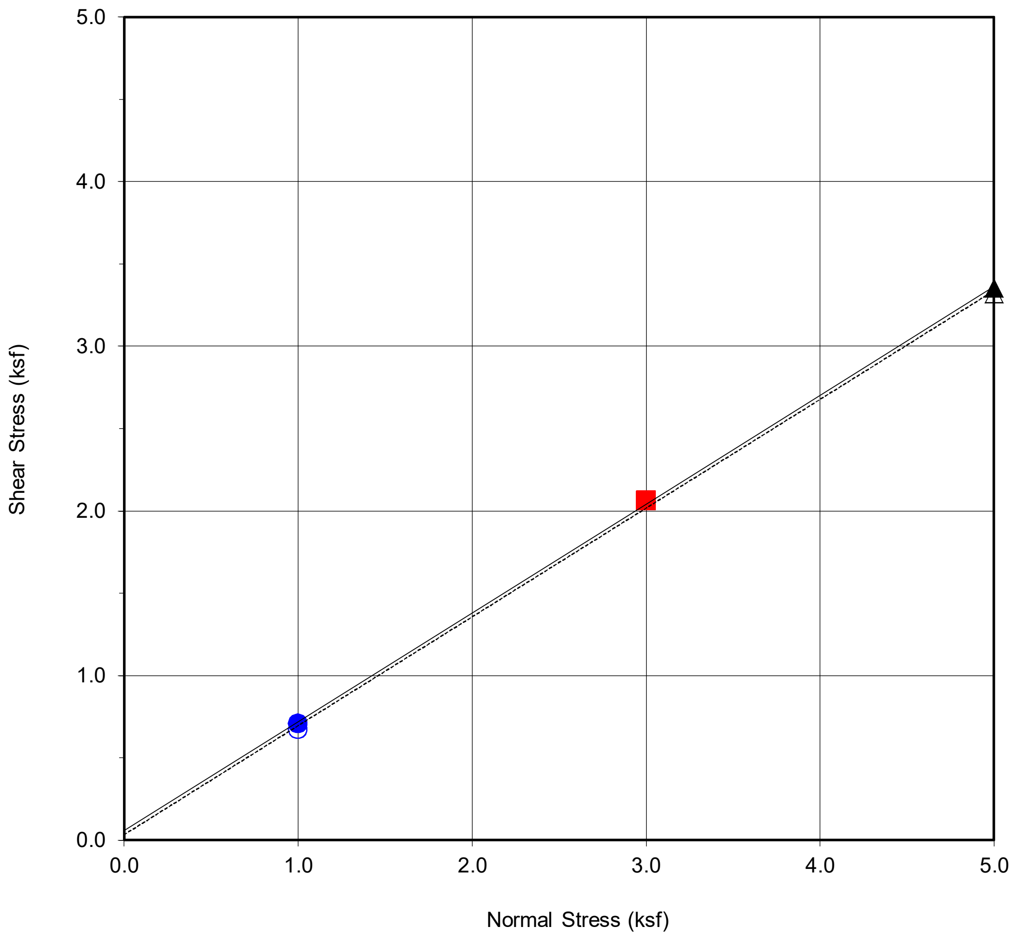
Consolidated Drained ASTM D-3080

Checked by: JJK

Project No.: T2746-99-10A

11016-11098 SIERRA AVENUE
FONTANA, CALIFORNIA

FEB 2020 Figure B5



Boring No.	**B7**
Sample No.	**B7@0-5'**
Depth (ft)	**0-5'**
Sample Type:	Ring

Soil Identification:		
Brown Silty Sand (SM)		
Strength Parameters		
	C (psf)	ϕ (°)
Peak	57	33.5
Ultimate	33	33.5

Normal Stress (kip/ft2)	1	3	5
Peak Shear Stress (kip/ft²)	● 0.71	■ 2.06	▲ 3.35
Shear Stress @ End of Test (ksf)	○ 0.67	□ 2.06	△ 3.32
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 (%)	8.0	8.4	8.5
Initial Dry Density (pcf)	113.0	113.0	113.0
Initial Degree of Saturation (%)	43.8	46.4	46.9
Soil Height Before Shearing (in.)	1.2	1.2	1.2
Final Moisture Content (%)	8.1	5.8	6.8

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DIRECT SHEAR TEST RESULTS

Consolidated Drained ASTM D-3080

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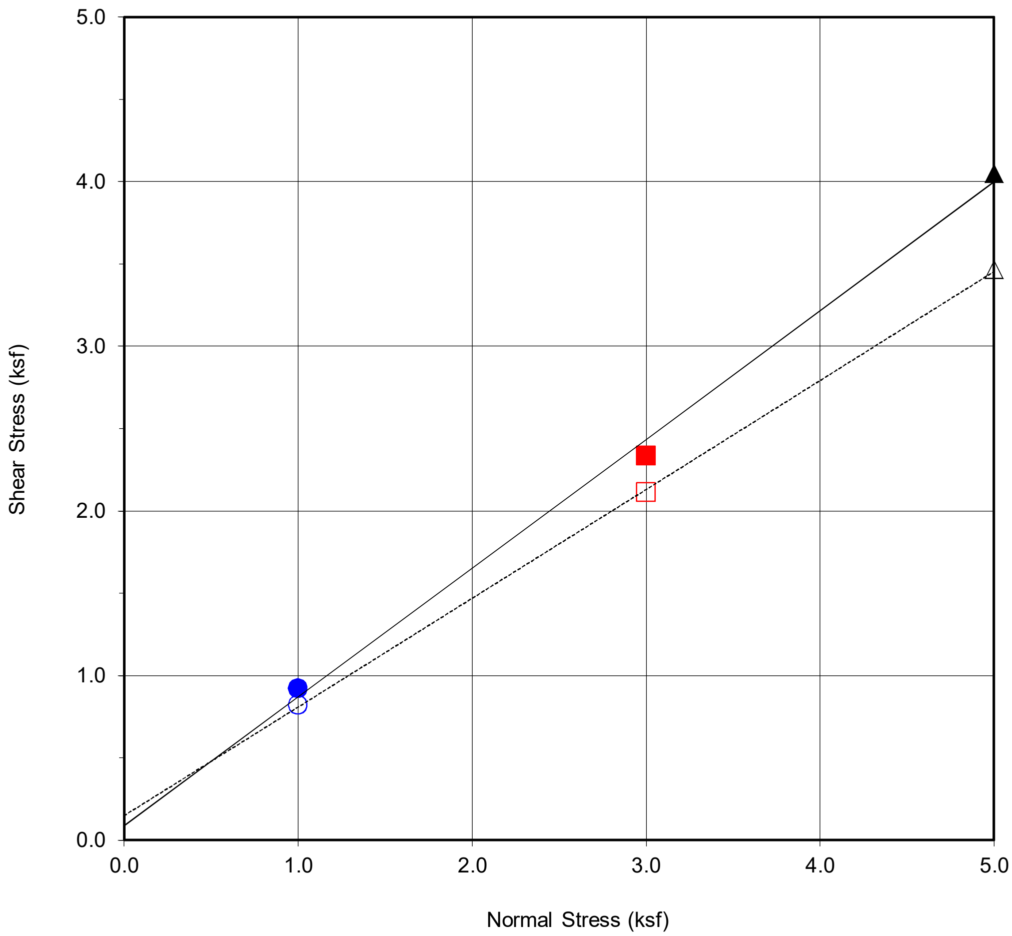
Project No.: T2746-99-10A

11016-11098 SIERRA AVENUE
FONTANA, CALIFORNIA

FEB 2020 Figure B6



Boring No.	B8
Sample No.	B8@2'
Depth (ft)	2
Sample Type:	Ring

Soil Identification:		
Brown Silty Sand (SM)		
Strength Parameters		
	C (psf)	ϕ (°)
Peak	87	38.0
Ultimate	149	33.5

Normal Stress (kip/ft2)	1	3	5
Peak Shear Stress (kip/ft²)	● 0.92	■ 2.33	▲ 4.05
Shear Stress @ End of Test (ksf)	○ 0.82	□ 2.11	△ 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 (%)	8.9	9.9	12.7
Initial Dry Density (pcf)	99.4	100.3	101.0
Initial Degree of Saturation (%)	34.6	39.4	51.1
Soil Height Before Shearing (in.)	1.2	1.2	1.2
Final Moisture Content (%)	17.5	16.8	16.2

GEOCON

DIRECT SHEAR TEST RESULTS

Consolidated Drained ASTM D-3080

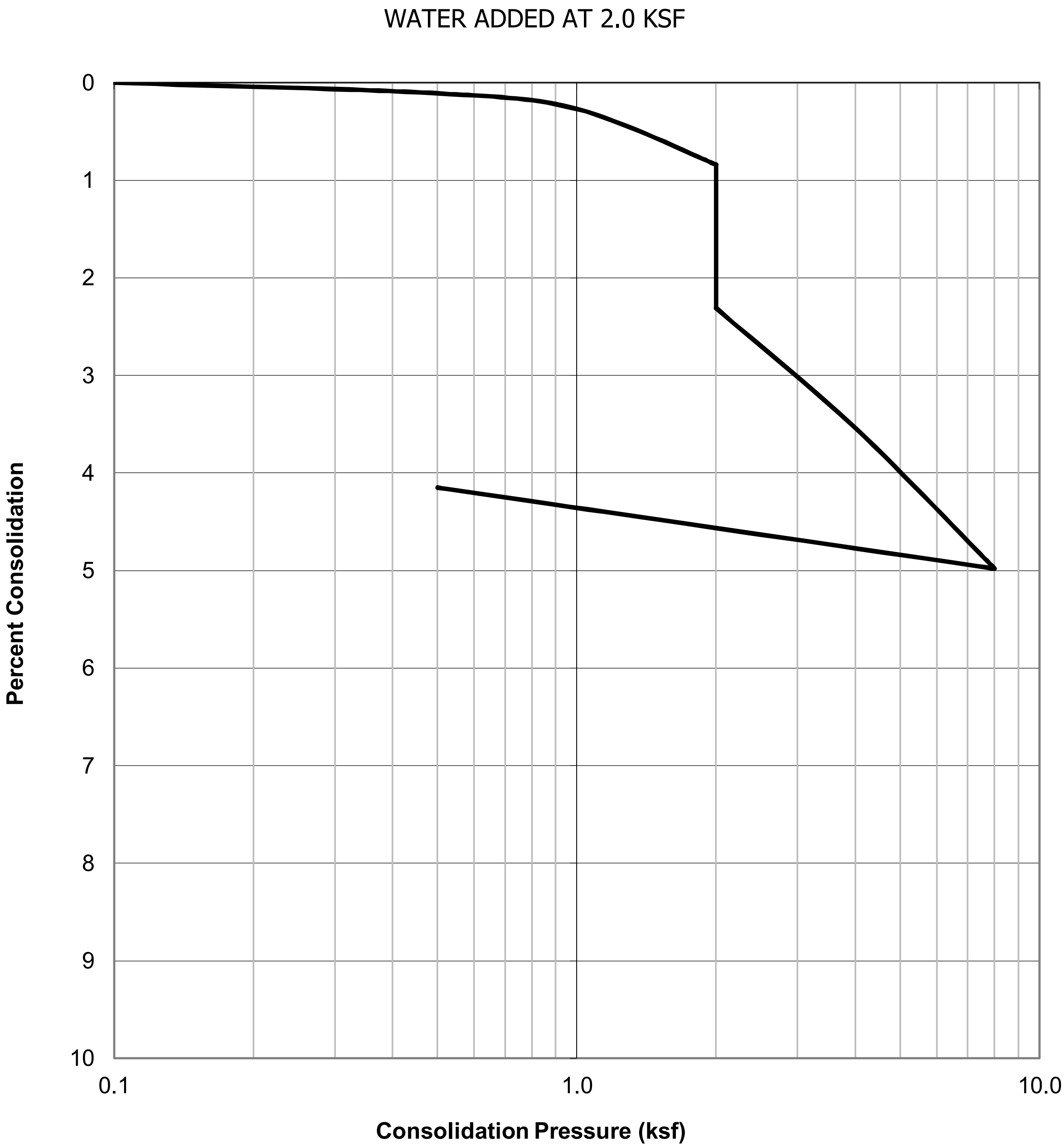
Checked by: JJK

Project No.: T2746-99-10A

11016-11098 SIERRA AVENUE
FONTANA, CALIFORNIA

FEB 2020

Figure B7



SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B2@7	Light Brown Sandy Silt (ML)	104.1	3.5	16.3

GEOCON

CONSOLIDATION TEST RESULTS

ASTM D-2435

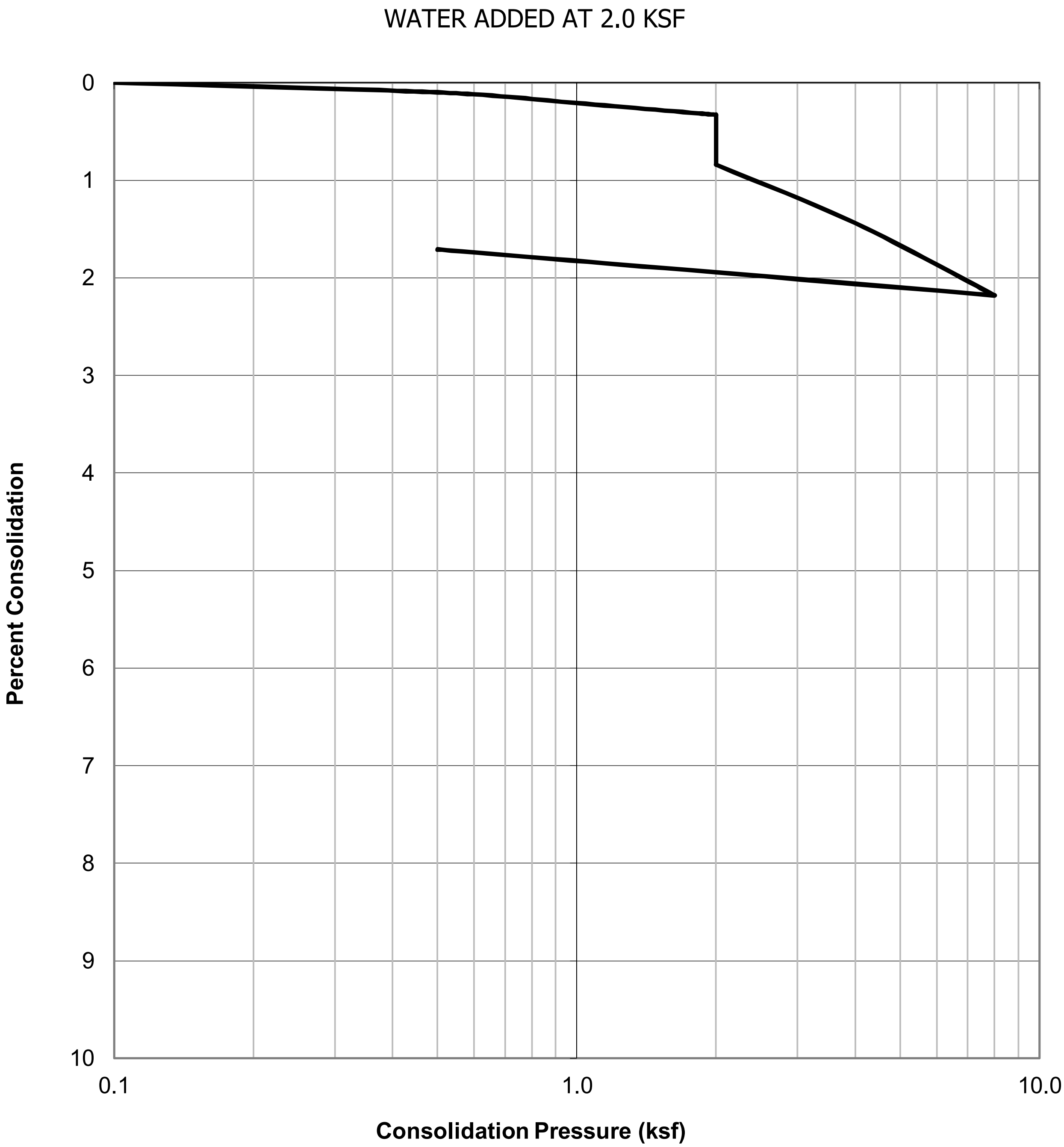
Checked by: JJK

Project No.: T2746-99-10A

11016-11098 SIERRA AVENUE
FONTANA, CALIFORNIA

FEB 2020

Figure B8



SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B2@5	Brown Silty Sand (SM)	109.5	2.4	15.1

GEOCON

CONSOLIDATION TEST RESULTS

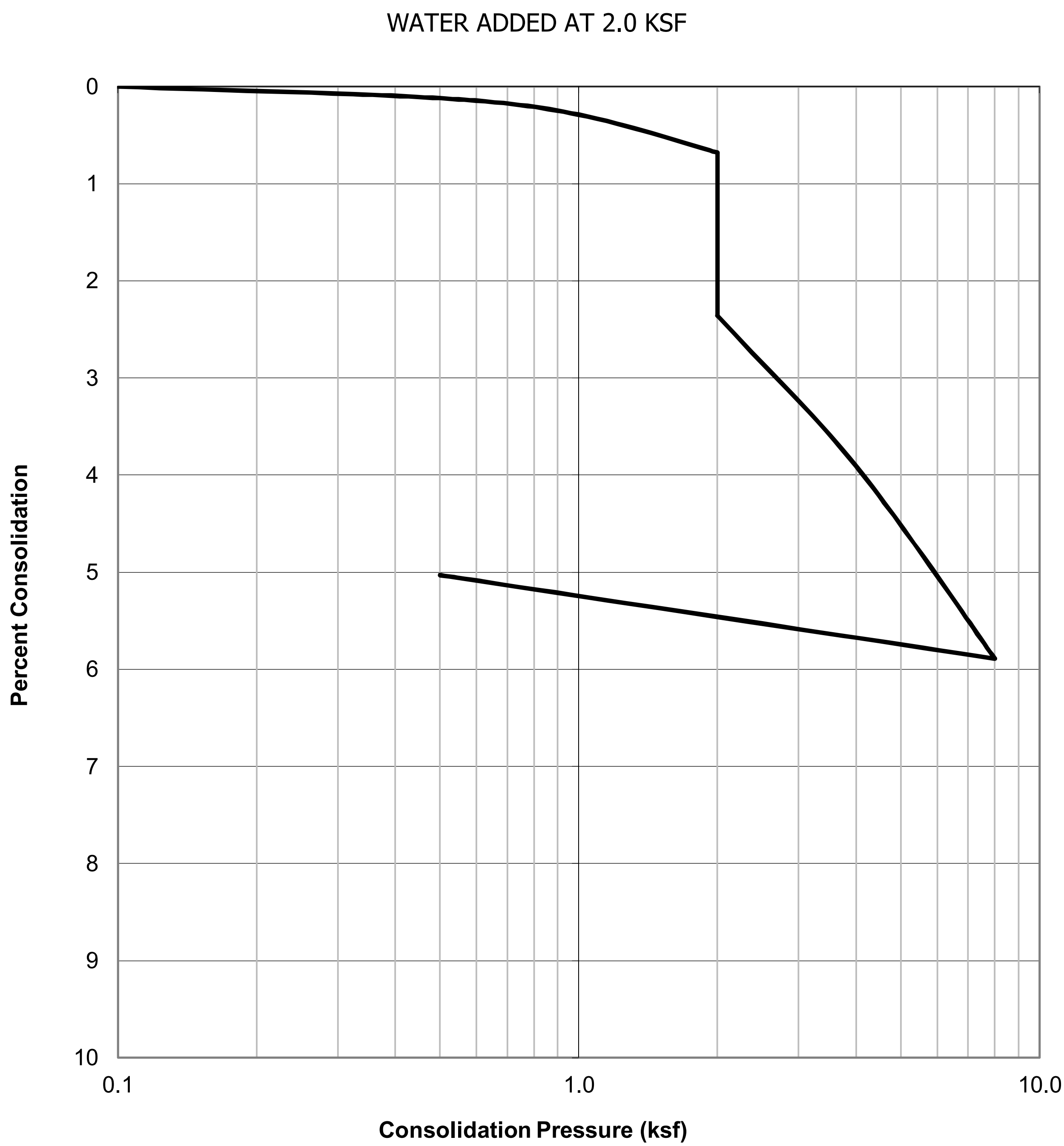
ASTM D-2435

Checked by: JJK

Project No.: T2746-99-10A

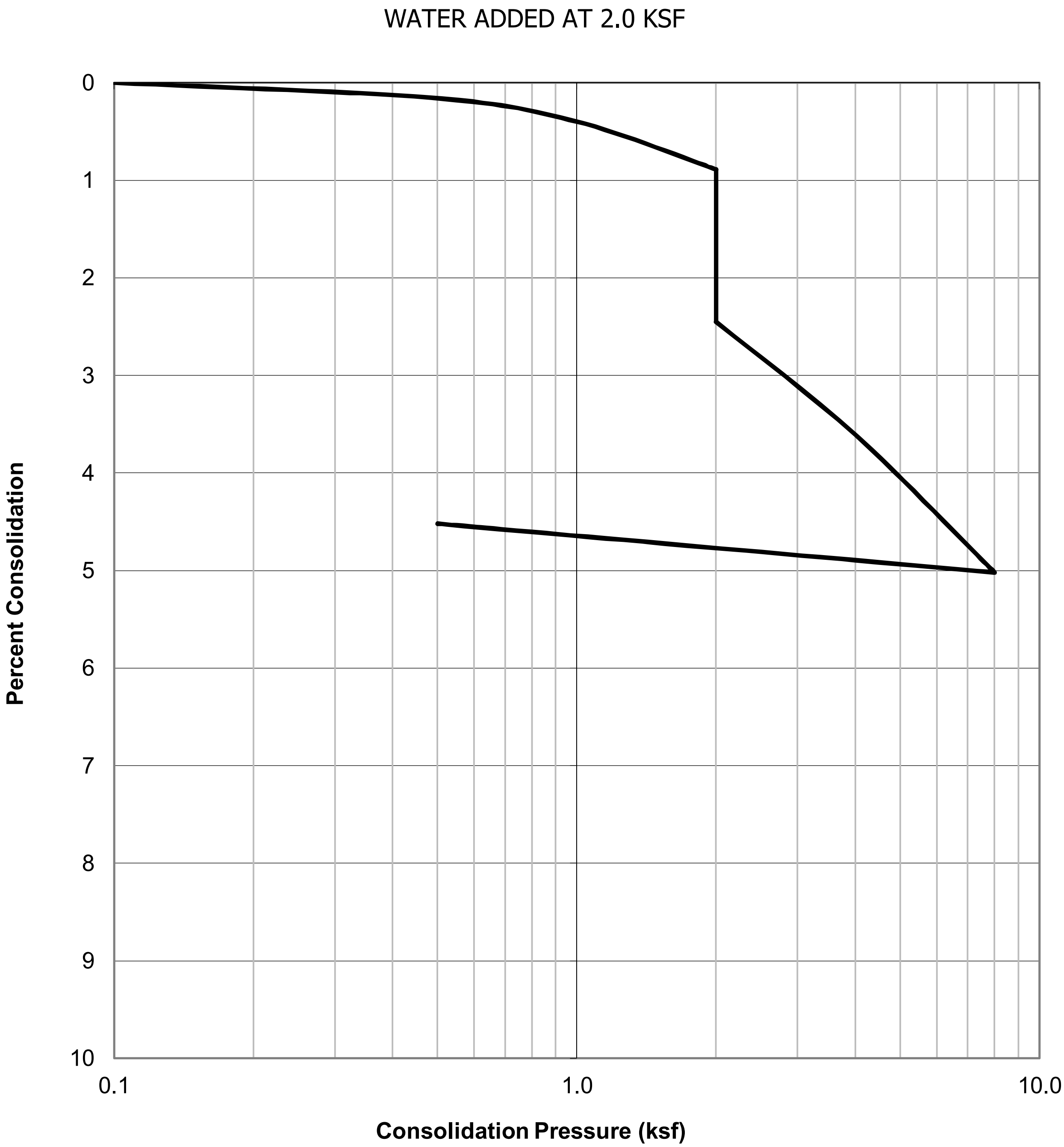
11016-11098 SIERRA AVENUE
FONTANA, CALIFORNIA

FEB 2020 Figure B9



SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B5@2	Brown Silt (ML)	107.8	7.4	16.9

GEOCON	**CONSOLIDATION TEST RESULTS** ASTM D-2435	Project No.: T2746-99-10A
		11016-11098 SIERRA AVENUE FONTANA, CALIFORNIA
	Checked by: JJK	FEB 2020 Figure B10



SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B5@5	Light Brown Silt (ML)	111.9	2.4	14.0

GEOCON

CONSOLIDATION TEST RESULTS

ASTM D-2435

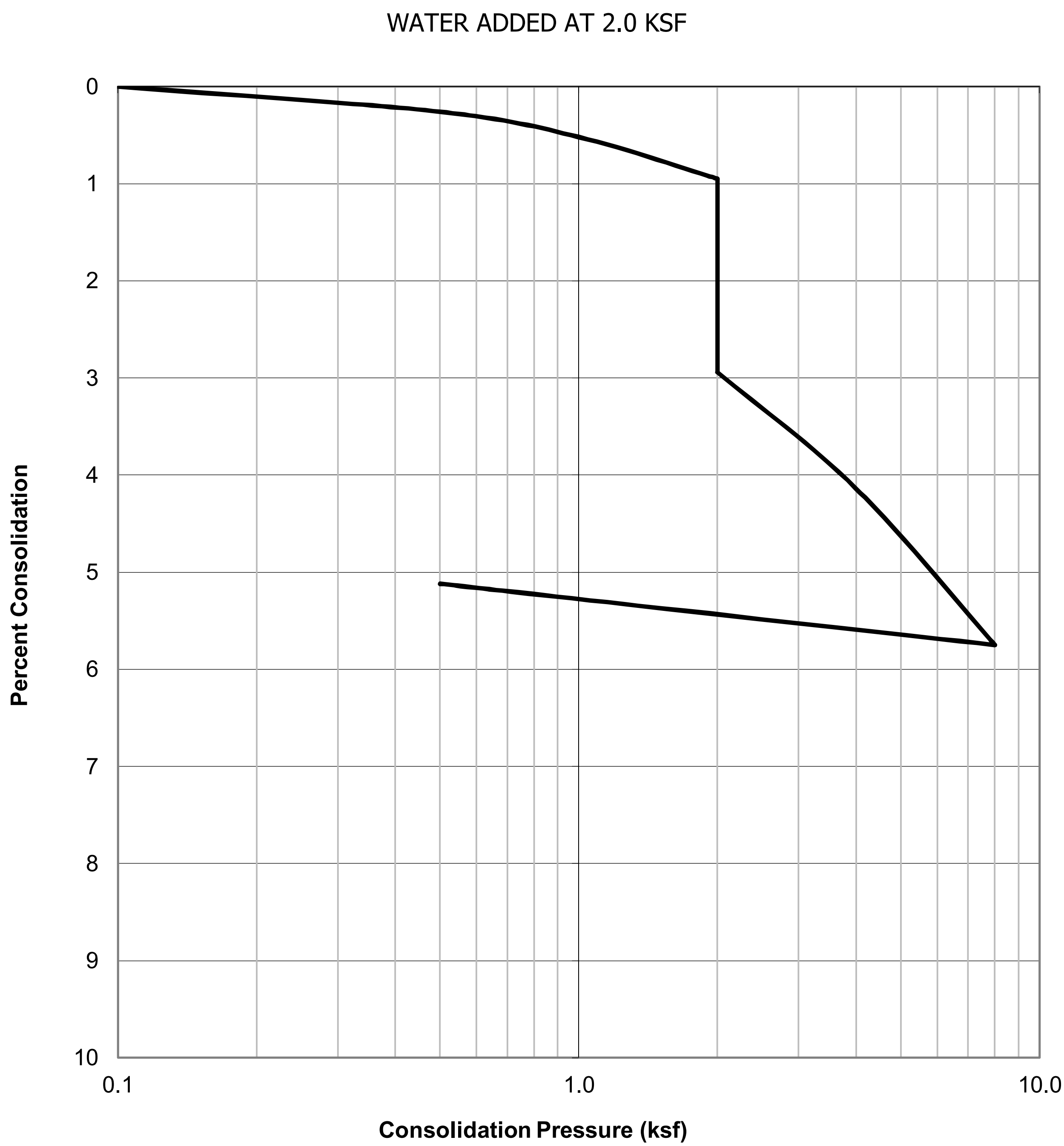
Checked by: JJK

Project No.: T2746-99-10A

11016-11098 SIERRA AVENUE
FONTANA, CALIFORNIA

FEB 2020

Figure B11



SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B5@7	Light Brown Silt (ML)	104.2	3.3	16.6

GEOCON

CONSOLIDATION TEST RESULTS

ASTM D-2435

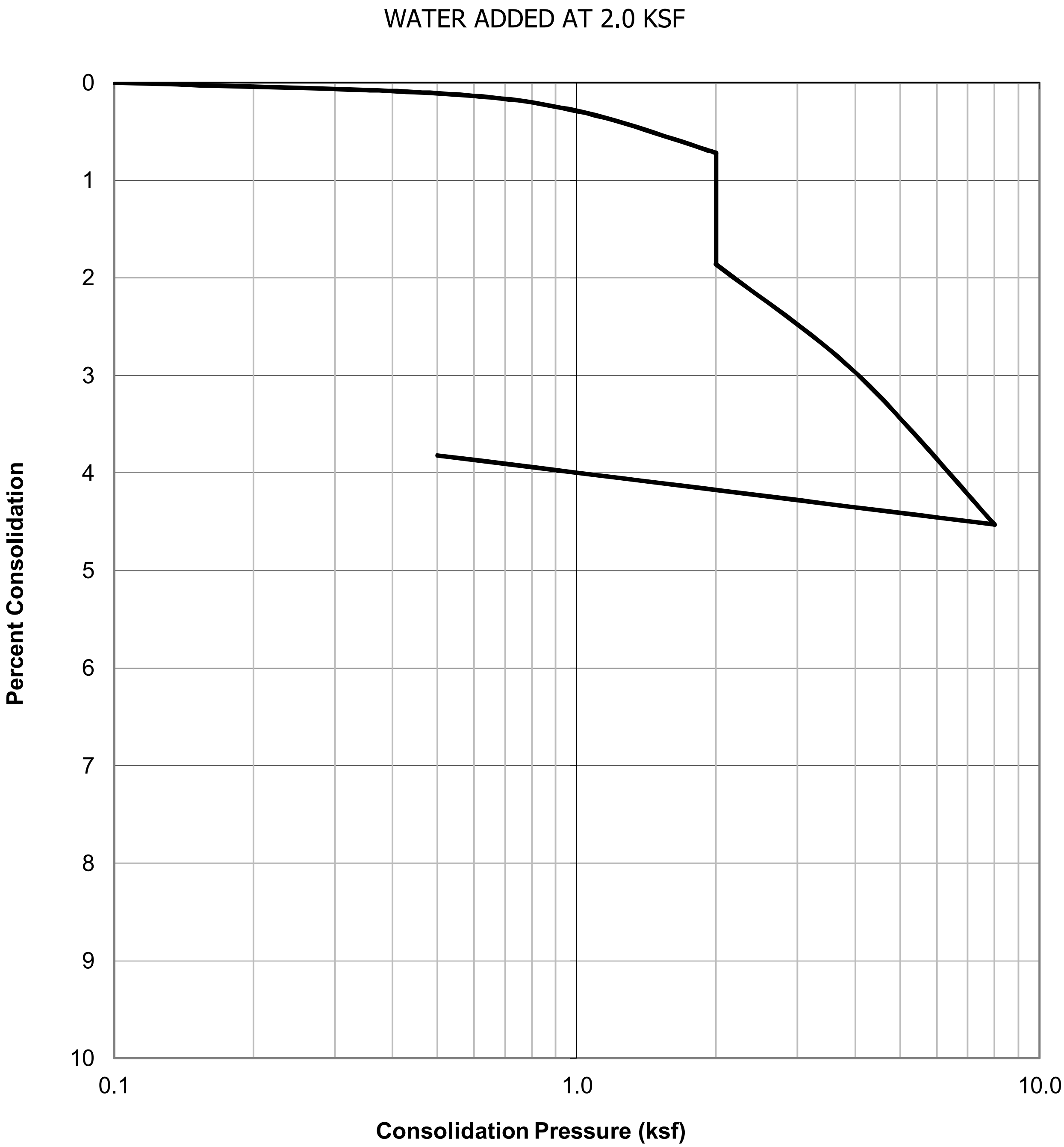
Checked by: JJK

Project No.: T2746-99-10A

11016-11098 SIERRA AVENUE
FONTANA, CALIFORNIA

FEB 2020

Figure B12



SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B7@10	Brown Silt (ML)	104.5	8.5	18.1

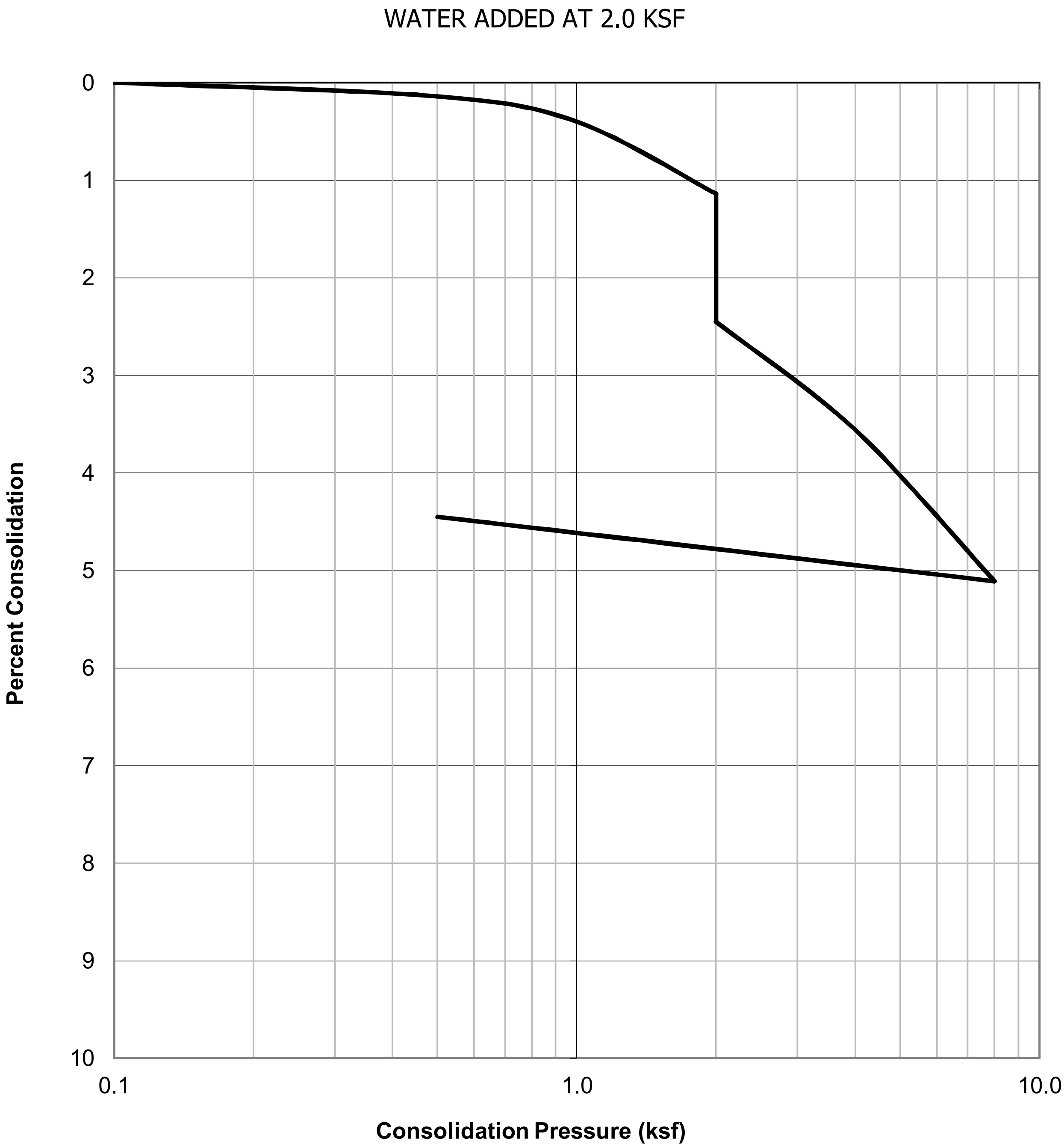
GEOCON

CONSOLIDATION TEST RESULTS

ASTM D-2435

Checked by: JJK

Project No.:	T2746-99-10A
11016-11098 SIERRA AVENUE FONTANA, CALIFORNIA	
FEB 2020	Figure B13



SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B8@7	Light Brown Silt (ML)	104.6	10.7	17.7

GEOCON

CONSOLIDATION TEST RESULTS

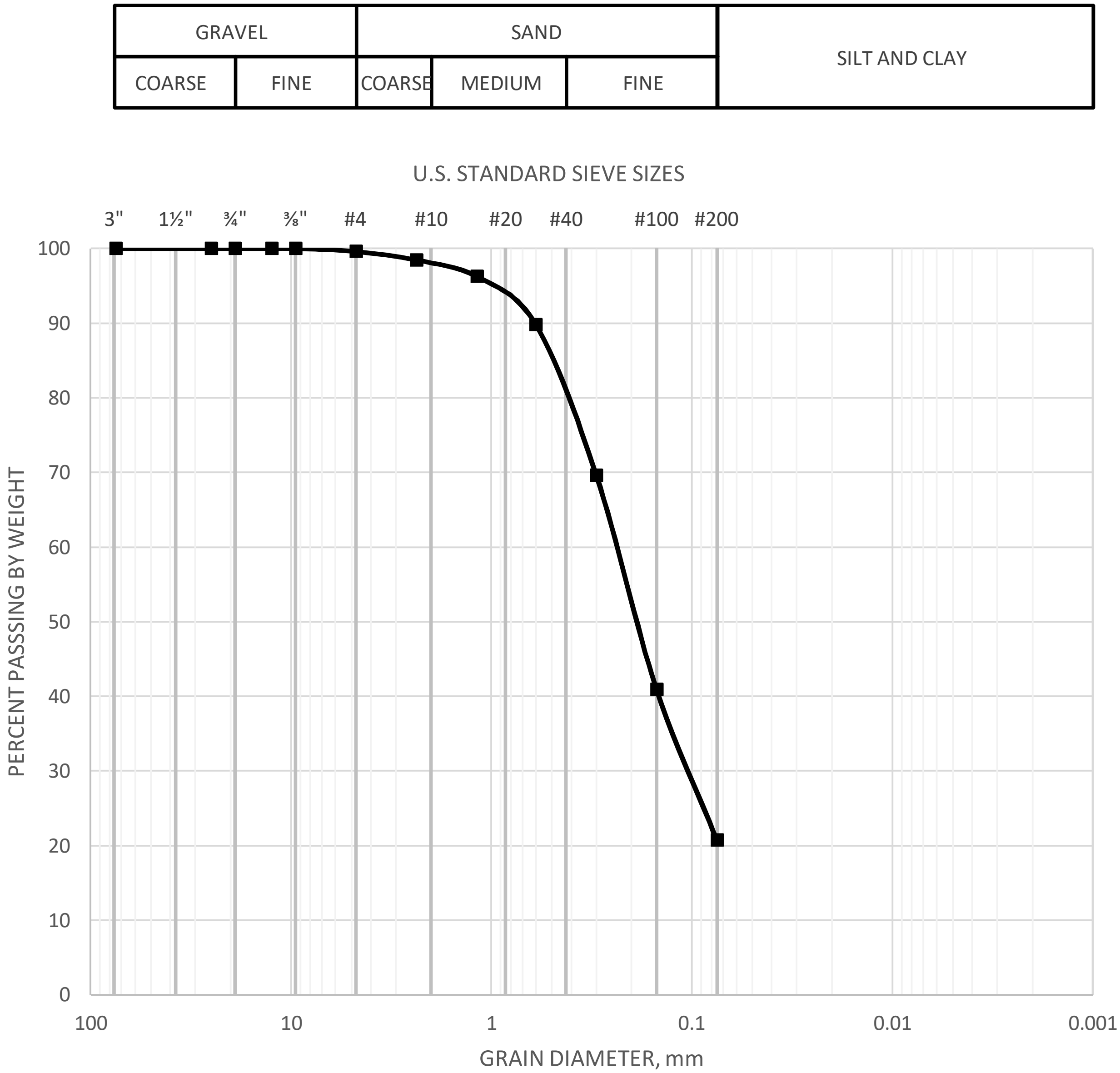
ASTM D-2435

Checked by: JJK

Project No.: T2746-99-10A

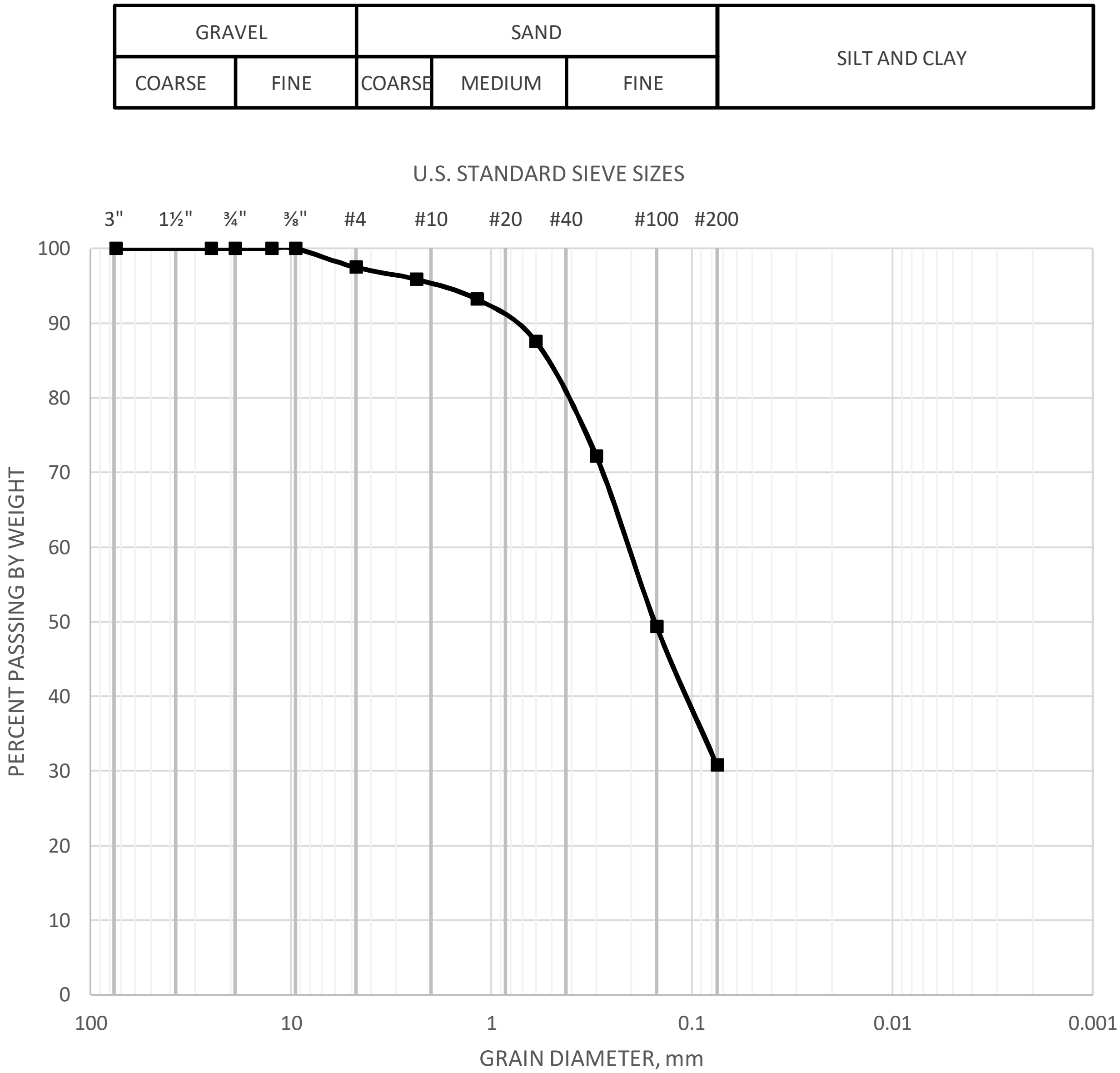
11016-11098 SIERRA AVENUE
FONTANA, CALIFORNIA

FEB 2020 Figure B14



SAMPLE	CLASSIFICATION	D60	D30	D10
B1@2'	Brown Silty Sand (SM)	2.36	0.68	0.25

GEOCON	**GRAIN SIZE DISTRIBUTION** ASTM D-422	Project No.: T2746-99-10A
	Checked by: JJK	11016-11098 SIERRA AVENUE FONTANA, CALIFORNIA
		FEB 2020 Figure B15



SAMPLE	CLASSIFICATION	D60	D30	D10
B3@2	Brown Silty Sand (SM)	2.36	0.68	0.25

GEOCON

GRAIN SIZE DISTRIBUTION

ASTM D-422

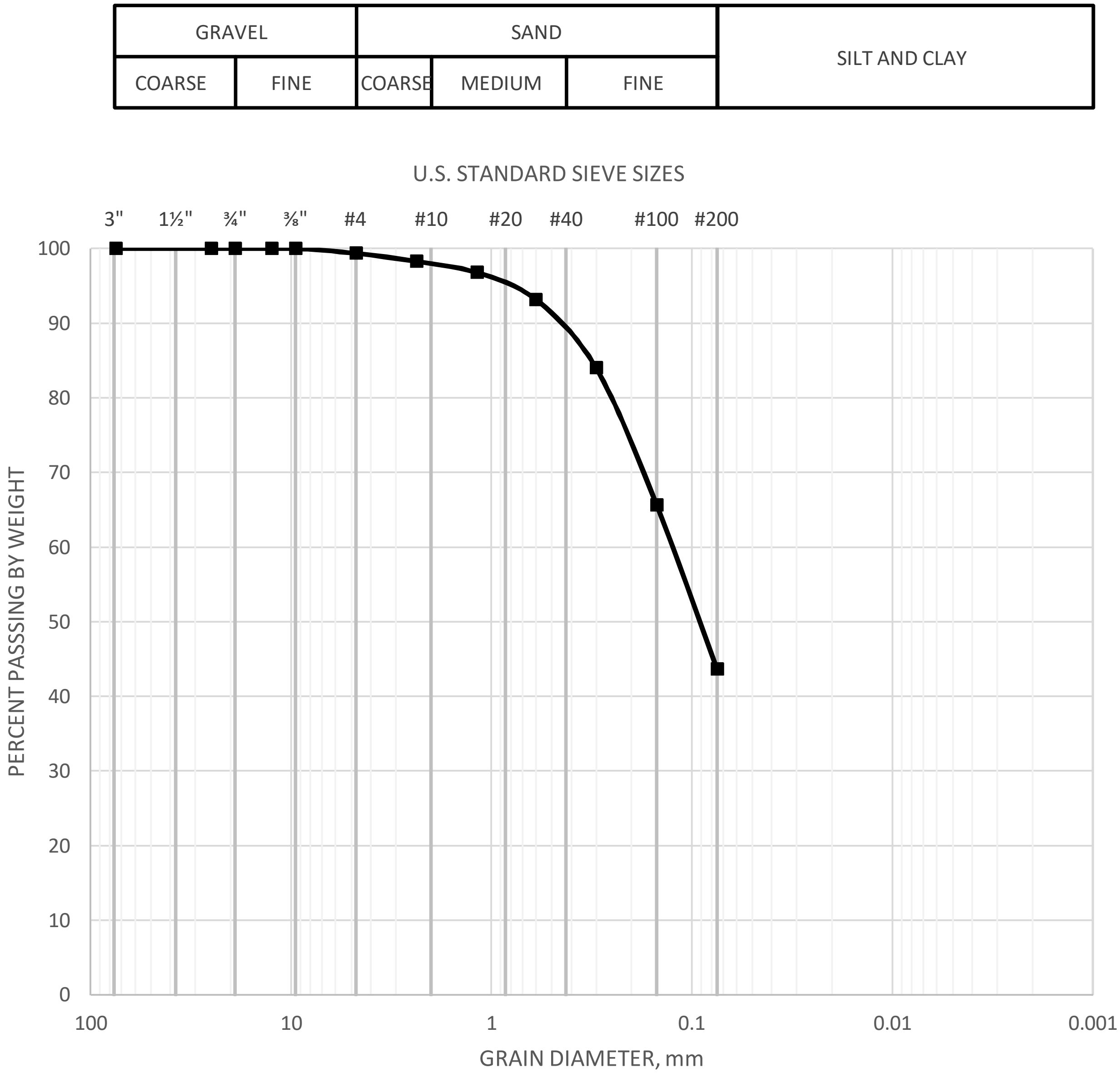
Checked by: JJK

Project No.: T2746-99-10A

11016-11098 SIERRA AVENUE
FONTANA, CALIFORNIA

FEB 2020

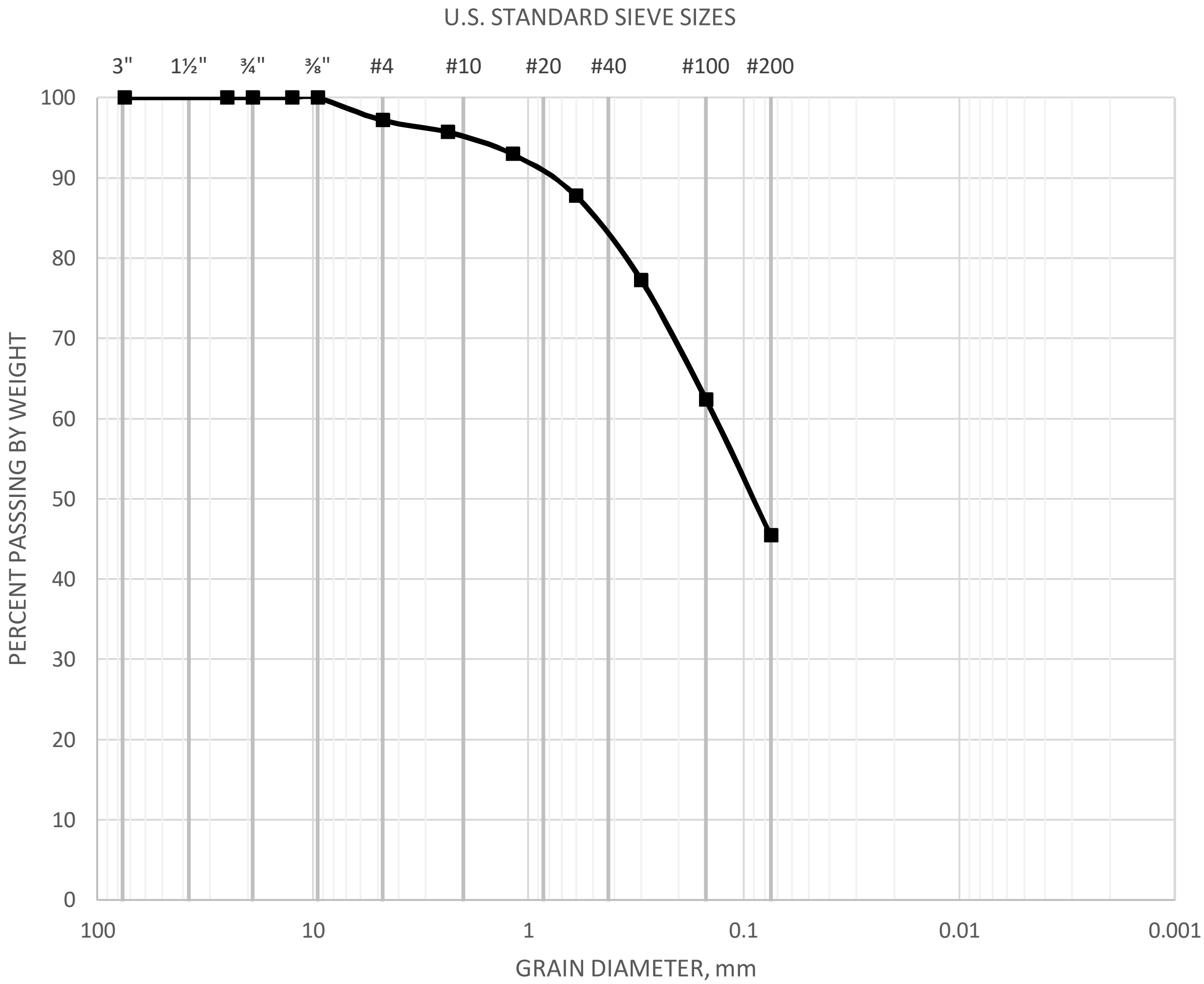
Figure B16



SAMPLE	CLASSIFICATION	D60	D30	D10
B5@5'	Light Brown Silty Sand (SM)	2.36	0.68	0.25

GEOCON	**GRAIN SIZE DISTRIBUTION** ASTM D-422	Project No.: T2746-99-10A
	Checked by: JJK	11016-11098 SIERRA AVENUE FONTANA, CALIFORNIA
		FEB 2020 Figure B17

GRAVEL		SAND			SILT AND CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	



SAMPLE	CLASSIFICATION	D60	D30	D10
B6@2'	Brown Silty Sand (SM)	2.36	0.68	0.25

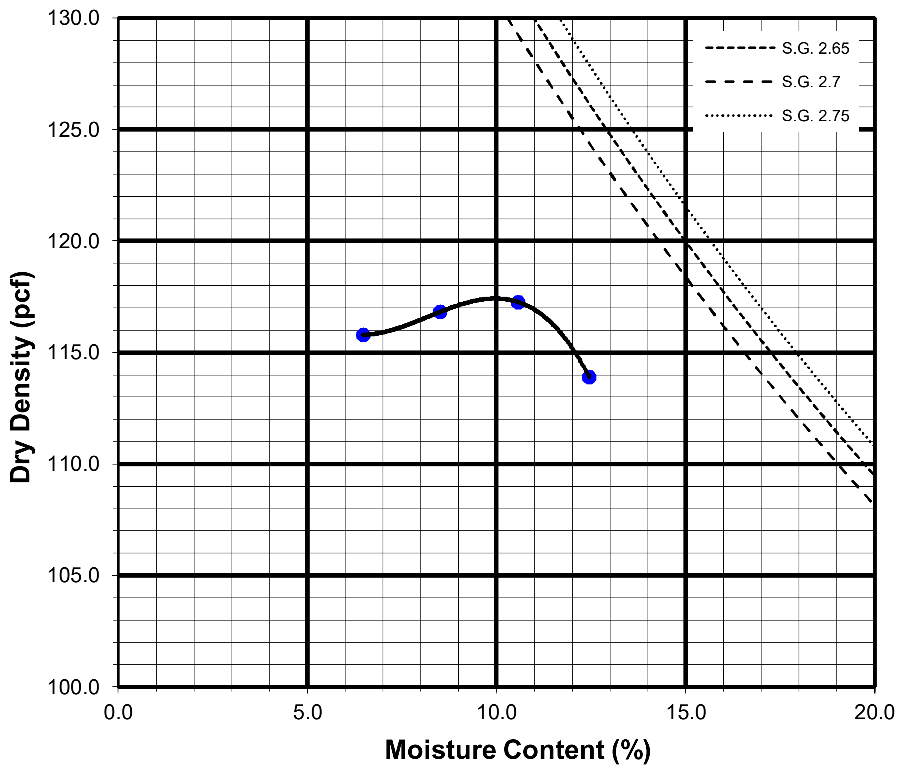
GEOCON	**GRAIN SIZE DISTRIBUTION** ASTM D-422	Project No.: T2746-99-10A
		11016-11098 SIERRA AVENUE FONTANA, CALIFORNIA
	Checked by: JJK	FEB 2020 Figure B18

Sample No:

B1@0-5'	Light Brown Silty Sand (SM)

TEST NO.		1	2	3	4	5	6
Wt. Compacted Soil + Mold	(g)	6011	6063	6107	6083		
Weight of Mold	(g)	4148	4148	4148	4148		
Net Weight of Soil	(g)	1862	1915	1958	1935		
Wet Weight of Soil + Cont.	(g)	676.6	602.3	629.7	690.4		
Dry Weight of Soil + Cont.	(g)	644.3	564.9	582.4	630.3		
Weight of Container	(g)	145.4	125.3	134.7	147.7		
Moisture Content	(%)	6.5	8.5	10.6	12.5		
Wet Density	(pcf)	123.3	126.8	129.7	128.1		
Dry Density	(pcf)	115.8	116.8	117.3	113.9		

Maximum Dry Density (pcf)	117.5	Optimum Moisture Content (%)	10.0



Preparation Method: A

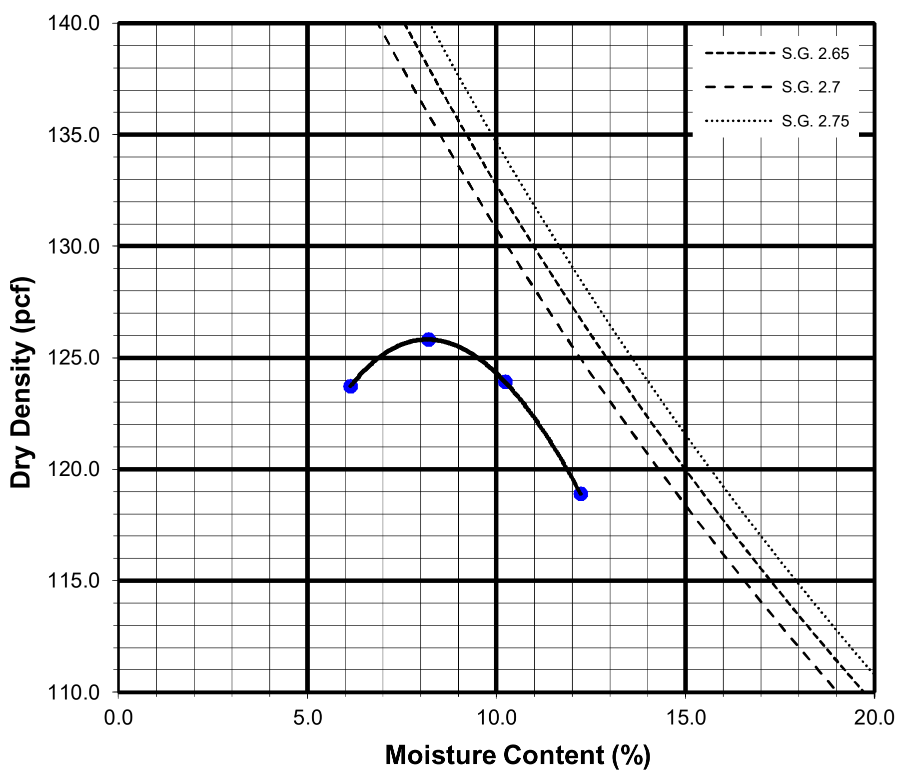
GEOCON	**MODIFIED COMPACTION TEST OF SOILS** ASTM D-1557	Project No.: T2746-99-10A
		11016-11098 SIERRA AVENUE FONTANA, CALIFORNIA
	Checked by: JJK	FEB 2020 Figure B19

Sample No:

B7@0-5'	Brown Silty Sand (SM)

TEST NO.		1	2	3	4	5	6
Wt. Compacted Soil + Mold	(g)	6132	6205	6212	6164		
Weight of Mold	(g)	4148	4148	4148	4148		
Net Weight of Soil	(g)	1984	2056	2063	2016		
Wet Weight of Soil + Cont.	(g)	721.4	691.6	675.8	668.7		
Dry Weight of Soil + Cont.	(g)	688.2	650.4	624.8	611.6		
Weight of Container	(g)	147.3	148.3	125.8	144.6		
Moisture Content	(%)	6.1	8.2	10.2	12.2		
Wet Density	(pcf)	131.3	136.1	136.6	133.4		
Dry Density	(pcf)	123.7	125.8	123.9	118.9		

Maximum Dry Density (pcf)	**126.0**	**Optimum Moisture Content (%)**	**8.5**



Preparation Method: A

GEOCON	**MODIFIED COMPACTION TEST OF SOILS** ASTM D-1557	Project No.: T2746-99-10A
		11016-11098 SIERRA AVENUE FONTANA, CALIFORNIA
	Checked by: JJK	FEB 2020 Figure B20

B1@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)	772.2	792.7
Wt. of Mold	(gm)	367.9	367.9
Specific Gravity	(Assumed)	2.7	2.7
Wet Wt. of Soil + Cont.	(gm)	487.5	792.7
Dry Wt. of Soil + Cont.	(gm)	462.7	370.9
Wt. of Container	(gm)	187.5	367.9
Moisture Content	(%)	9.0	14.5
Wet Density	(pcf)	122.0	128.0
Dry Density	(pcf)	111.9	111.7
Void Ratio		0.5	0.5
Total Porosity		0.3	0.3
Pore Volume	(cc)	69.6	69.6
Degree of Saturation	(%) [S_{meas}]	48.4	77.4

Date	Time	Pressure (psi)	Elapsed Time (min)	Dial Readings (in.)
1/28/2020	10:00	1.0	0	0.294
1/28/2020	10:10	1.0	10	0.2935
Add Distilled Water to the Specimen				
1/29/2020	10:00	1.0	1430	0.2935
1/29/2020	11:00	1.0	1490	0.2935

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: 2016 California Building Code, Section 1803.5.3
** Reference: 1997 Uniform Building Code, Table 18-I-B.

GEOCON

EXPANSION INDEX TEST RESULTS
ASTM D-4829

Checked by: JJK

Project No.: T2746-99-10A

11016-11098 SIERRA AVENUE
FONTANA, CALIFORNIA

FEB 2020 Figure B21

B7@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)	791.6	813.0
Wt. of Mold	(gm)	368.3	368.3
Specific Gravity	(Assumed)	2.7	2.7
Wet Wt. of Soil + Cont.	(gm)	487.5	813.0
Dry Wt. of Soil + Cont.	(gm)	466.6	393.8
Wt. of Container	(gm)	187.5	368.3
Moisture Content	(%)	7.5	12.9
Wet Density	(pcf)	127.7	134.0
Dry Density	(pcf)	118.8	118.6
Void Ratio		0.4	0.4
Total Porosity		0.3	0.3
Pore Volume	(cc)	61.2	61.2
Degree of Saturation	(%) [S_{meas}]	48.7	83.3

Date	Time	Pressure (psi)	Elapsed Time (min)	Dial Readings (in.)
1/28/2020	10:00	1.0	0	0.254
1/28/2020	10:10	1.0	10	0.254
Add Distilled Water to the Specimen				
1/29/2020	10:00	1.0	1430	0.254
1/29/2020	11:00	1.0	1490	0.254

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: 2016 California Building Code, Section 1803.5.3
** Reference: 1997 Uniform Building Code, Table 18-I-B.

GEOCON

EXPANSION INDEX TEST RESULTS
ASTM D-4829

Checked by: JJK

Project No.: T2746-99-10A

11016-11098 SIERRA AVENUE
FONTANA, CALIFORNIA

FEB 2020 Figure B22

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

Sample No.	**pH**	**Resistivity (ohm centimeters)**
B1 @ 0-5'	7.2	11000 (Mildly Corrosive)
B7 @ 0-5'	7.1	8500 (Moderately Corrosive)

SUMMARY OF LABORATORY CHLORIDE CONTENT TEST RESULTS
EPA NO. 325.3

Sample No.	Chloride Ion Content (%)
B1@0-5'	0.006
B7@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***
B1@0-5'	0.000	S0
B7@0-5'	0.000	S0

GEOCON	**CORROSIVITY TEST RESULTS**	Project No.: T2746-99-10A
		11016-11098 SIERRA AVENUE FONTANA, CALIFORNIA
	Checked by: JJK	FEB 2020 Figure B23