



GEOTECHNICAL INVESTIGATION

CHAFFEY COLLEGE
IN-TECH WELDING CENTER
9400 CHERRY AVENUE
FONTANA, CALIFORNIA

DECEMBER 23, 2024
PROJECT NO. W1145-99-10

PREPARED FOR:
Chaffey College
Rancho Cucamonga, CA



Project No. W1145-99-10
December 23, 2024

Chaffey College
5885 Haven Avenue
Rancho Cucamonga, CA 91739

Subject: GEOTECHNICAL INVESTIGATION
CHAFFEY COLLEGE – IN-TECH WELDING CENTER
9400 CHERRY AVENUE
FONTANA, CALIFORNIA

Ladies and Gentlemen:

In accordance with your authorization of our proposal dated June 20, 2024, we have performed a geotechnical investigation for proposed new in-tech welding center located at 9400 Cherry 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 the 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 in this report are followed and implemented during design and construction.

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

Very truly yours,

GEOCON WEST, INC.

A handwritten signature in blue ink, appearing to read 'Celine', with a long horizontal stroke extending to the right.

Celine Treyes
Staff Engineer



Neal Berliner
GE 2576

A handwritten signature in blue ink, appearing to read 'Lisa Battiato', with a long horizontal stroke extending to the right.



Lisa Battiato
CEG 2316

(email) Addressee

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GEOTECHNICAL INVESTIGATION

1. PURPOSE AND SCOPE

This report presents the results of a geotechnical investigation for the proposed In-Tech Welding Center planned for 9400 Cherry 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 proposed design and construction.

The scope of this investigation included a site reconnaissance, field exploration, laboratory testing, engineering analysis, and the preparation of this report. The site was explored on September 17, 2024, by excavating three 8-inch-diameter borings using a truck-mounted hollow-stem auger drilling machine to depths of 25½ feet beneath the ground surface. Additionally, percolation testing was performed in one of the borings (B1) to determine the infiltration rate of site soils. The approximate locations of the exploratory borings are depicted on the Site Plan (see Figure 2). A detailed discussion of the field investigation, including boring logs, is presented in Appendix A.

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

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

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

2. SITE AND PROJECT DESCRIPTION

The subject site is located at 9400 Cherry Avenue in the City of Fontana, California. The site for the proposed In-Tech Welding Center structure is currently occupied by a grass field that is bounded by trees and an asphalt parking lot to the north, by landscaping and California Steel Way to the south, by landscaping and Cherry Avenue to the east and by a parking lot to the west, and by the existing In-Tech Building to the southwest. The site is relatively level and surface drainage appears to flow into existing v-ditch drains in lawn area.

Based on the information provided by the Client, it is our understanding that the project consists of the design and construction of a 5,000 square foot building to be constructed at or near existing grade. Plans depicting the proposed improvements are provided on the Site Plan (see Figure 2).

Based on the preliminary nature of the design at this time, wall and column loads were not available. It is anticipated that column loads for the proposed structures will be up to 350 kips, and wall loads will be up to 6 kips per linear foot.

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

3. GEOLOGIC SETTING

The site is located in the San Bernardino Valley in San Bernardino County, California. The Chino Basin encompasses a broad area of coalescing alluvial fans that extend southward from the San Gabriel Mountains and overlie a down-dropped structural block which is bounded by the Elsinore Fault and the Chino Fault to the southwest, by the Red Hill Fault and the San Jose Fault to the northwest, by the San Gabriel Mountains and Sierra Madre Fault Zone to the north, by the Rialto-Colton Fault to the northeast, and the La Sierra Hills and Jurupa Hills to the south and southeast. The alluvial deposits within the Chino Basin consist of Holocene age (last 11,700 years old) and Pleistocene age (11,700 to 2 million years old) alluvial sediments. Locally, a thin veneer of eolian sand locally covers areas of the basin.

4. SOIL AND GEOLOGIC CONDITIONS

Based on our field investigation and published geologic maps, the soils underlying the site consist of Holocene age alluvial fan deposits consisting of cobbles, gravel, sand and silt (CGS, 2010). Depiction of soils and surrounding soils are indicated in Regional Geologic Map (see Figure 3). Detailed stratigraphic profiles are provided in the boring logs in Appendix A.

4.1 Younger Alluvium

Holocene-age alluvium was encountered in each boring. The alluvium generally consists of light brown to grayish-brown interbedded silt, silty sand, and poorly to well-graded sand with various amounts of gravel and cobbles. The alluvial soils are characterized as slightly moist to moist and medium dense to dense, or stiff.

5. GROUNDWATER

The site is located in the Chino Basin of the Upper Santa Ana Valley Groundwater Basin. Historic groundwater contour maps indicate the depth to groundwater in the site vicinity was approximately 100 to 200 feet beneath the ground surface in 1904 (Mendenhall, 1907).

Review of groundwater monitoring well data provided by the U.S. Geologic Survey (USGS, 2024a) indicates closest monitoring well to the site is Local Well No. CHINO-1002215 (State Well No. 340935N1174885W001), located approximately 0.8 mile northeast of the site. Monitoring data from this well is available for the period from October 1925 through April 2017. During this time, the depth to groundwater has been approximately greater 300 feet beneath the ground surface. The most recent groundwater level measurement was recorded on April 1, 2017 and the depth to groundwater was approximately greater than 450 feet below the ground surface.

Groundwater was not encountered in our borings, drilled to a maximum depth of approximately 25½ feet below the existing ground surface. Based on groundwater not encountered in our borings and the depth of proposed construction, static groundwater is neither expected to be encountered during construction, nor have a detrimental effect on the project. However, it is not uncommon for groundwater levels to vary seasonally or for groundwater seepage conditions to develop where none previously existed (especially in impermeable fine-grained soils which are heavily irrigated or after seasonal rainfall), groundwater seepage levels encountered during construction may be actually higher than those encountered during our investigation. In addition, recent requirements for stormwater infiltration could result in shallower seepage conditions in the region. Proper surface drainage of irrigation and precipitation will be critical for future performance of the project. Recommendations for drainage are provided in the Surface Drainage section of this report (see Section 7.15).

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 subject to any known volcanic hazards. The nearest Quaternary age volcanic field is located about 150 miles to the north near Little Lake and Coso Mountains (USGS, 2024b). Also, the site is not within an area known to have radon 222 potential (CGS, 2024).

The San Bernardino County Land Use Plan (2010) indicates that the site is not located with an Earthquake Fault Zone. No Holocene-active or pre-Holocene faults with the potential for surface fault rupture are known to pass directly beneath the site. Therefore, the potential for surface rupture due to faulting occurring beneath the site during the design life of the proposed development is considered low. However, the site is located in the seismically active Southern California region and could be subjected to moderate to strong ground shaking in the event of an earthquake on one of the many active Southern California faults. The faults in the vicinity of the site are shown in Figure 4, Regional Fault Map.

The closest surface trace of a Holocene-active fault to the site is the Red Hill Fault, located approximately 5.1 miles to the northwest (USGS, 2006; City of Rancho Cucamonga, 2008). Other nearby active faults include the Cucamonga segment of the Sierra Madre Fault Zone, the San Jacinto Fault, and the Chino Fault located approximately 5.4 miles north-northeast, 7.7 miles northeast, and 14½ miles southwest of the site, respectively (USGS, 2006; Ziony and Jones, 1989). The active San Andreas Fault Zone is located approximately 11 miles northeast of the site (USGS, 2006).

Several buried thrust faults, commonly referred to as blind thrusts, underlie the Southern California area at depth. These faults are not exposed at the ground surface and are typically identified at depths greater than 3.0 kilometers. The October 1, 1987 Mw 5.9 Whittier Narrows earthquake and the January 17, 1994 Mw 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 Southern California 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 5, 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
Long Beach	March 10, 1933	6.4	42	SW
Tehachapi	July 21, 1952	7.5	107	NW
San Fernando	February 9, 1971	6.6	57	WNW
Whittier Narrows	October 1, 1987	5.9	34	W
Sierra Madre	June 28, 1991	5.8	32	WNW
Landers	June 28, 1992	7.3	61	E
Big Bear	June 28, 1992	6.4	39	ENE
Northridge	January 17, 1994	6.7	60	W
Hector Mine	October 16, 1999	7.1	78	ENE
Ridgecrest	July 5, 2019	7.1	116	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 minimized if the proposed structures are designed and constructed in conformance with current building codes and engineering practices.

6.3 Site-Specific Ground Motion Hazard Analysis

A site-specific ground motion hazard analysis was performed in accordance with ASCE 7-16 Chapter 21 and Section 1613A of the 2022 CBC.

6.3.1 Site-Specific Shear Wave Velocity

On November 11, 2024, we performed a 1-D refraction microtremor (ReMi) seismic survey at the site. The methodologies used for data acquisition and analysis are presented in the report dated November 25, 2024. A copy of the report is provided as Appendix C.

Based on the results of the ReMi survey, the site-specific soil shear wave velocity for the upper 30 meters of soil (V_{s30}) is estimated as 354 meters per second. In accordance with Section 1613A.3.2 of the 2022 California Building Code and Table 20.3-1 of ASCE 7-16, the estimated soil shear wave velocity falls within the boundaries of a Site Class “D”.

6.3.2 Probabilistic Seismic Hazard Analysis

The risk-targeted Maximum Considered Earthquake (MCE_R) probabilistic response spectrum consists of the spectral response accelerations which are expected to achieve a 1 percent probability of collapse within a 50-year period, evaluated at 5 percent damping.

The median spectral response accelerations having a 2 percent chance of exceedance in 50 years were evaluated at 5 percent damping using the USGS Hazard Curves (dynamic) tool. The NSHM Conterminous U.S. 2018 edition was used within the analysis, which is based on the UCERF3 fault model. The soil underlying the site was modeled with the measured shear wave velocity.

The web application uses the ground motion prediction equations (GMPEs) from the NGA-West 2 project: Abrahamson-et al. (2014) NGA West 2, Boore et al. (2014) NGA West 2, Campbell-Bozorgnia (2014) NGA West 2, and Chiou-Youngs (2014) NGA West 2. Each GMPE was assigned an equal weight and the median value of the four GMPEs was evaluated. The median spectral accelerations were rotated to maximum direction using the period specific ratios from Shahi et al. (2013 & 2014).

The GMPE of Campbell and Bozorgnia requires that the depth to where the shear wave velocity reaches 2.5 kilometers per second ($Z_{2.5}$) be defined. Additionally, the GMPEs of Abrahamson-et al., Boore et al. and Chiou-Youngs require that the depth to where the shear wave velocity reaches 1 kilometer per second ($Z_{1.0}$) be defined. The values of $Z_{2.5}$ and $Z_{1.0}$ are internally calculated by the Hazard Curves (dynamic) tool.

The MCE uniform hazard response spectrum was adjusted to risk-targeted spectral accelerations corresponding to a 1 percent chance of collapse in 50 years by using the values of C_{RS} and C_{R1} obtained from Figures 22-18 and 22-19 of ASCE 7-16 Chapter 22.

The risk-targeted Maximum Considered Earthquake (MCE_R) probabilistic response spectrum is provided on Figure 6.

6.3.3 Deterministic Seismic Hazard Analysis

In order to define the deterministic scenario events, disaggregation of the uniform hazard probabilistic response spectrum was performed using the USGS Disaggregation tool. The inversion approach used by UCERF-3 allows for a large number of variations for each source scenario, including multi-fault ruptures. Therefore, disaggregation of UCERF-3 consists of the contributions from multi-fault ruptures rather than individual source contributions. To address this, the USGS Disaggregation tool aggregates the contributions on a per-fault-section basis, with rupture contributions only ever counted once. The Hazard Disaggregation tool contributor list shows the fault sections which contribute most to hazard at a site and report a mean earthquake magnitude for each section identified by a 'parent' fault name and section index. Based on the disaggregation, we have considered scenario events with the greatest contribution to the deterministic ground motions.

The input values used to evaluate the deterministic scenario(s) are provided in Figure 8. The deterministic median and standard deviation (σ) for the scenario events were evaluated using the USGS Response Spectra tool. The deterministic analysis used the same four GMPEs, equally weighted, to generate the median and standard deviation of the ground motion which were then used to calculate the 84th percentile at 5% damping. The median spectral accelerations were rotated to maximum direction using the period specific ratios from Shahi et al. (2013 & 2014).

The deterministic scenarios were compared, and a combination of events controls the deterministic spectrum. The fault source resulting in the highest spectral accelerations from 0 to 3 seconds would be a magnitude 6.75 event on the Fontana fault; and from 4 to 5 seconds would be a magnitude 8.18 event on the Southern San Andreas fault.

The 84th percentile maximum rotated component deterministic response spectrum is provided on Figure 7.

6.3.4 Site-Specific Response Spectrum

The lesser of the probabilistic and deterministic MCE_R response spectra is the Site-Specific MCE_R . Two thirds of the Site-Specific MCE_R is the Design Earthquake (DE) Response Spectrum, provided the results are not less than 80 percent of the modified General Design Response Spectrum determined by ASCE 7-16 Section 11.4.6 with F_a and F_v determined as specified in Section 21.3.

Graphical representations of the analyses are presented on Figures 6 and 7. The Site-Specific Design Earthquake response spectrum at 5 percent damping is presented on Figure 7 and in tabular form on Figure 8.

6.3.5 Mapped Acceleration Parameters

The following table summarizes the site-specific design criteria obtained from the 2022 California Building Code (CBC; Based on the 2021 International Building Code [IBC] and ASCE 7-16), Chapter 16A Structural Design, Section 1613 Earthquake Loads. The data was calculated using the online application *U.S. Seismic Design Maps*, provided by the Structural Engineers Association of California (SEAOC). 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 2022 CBC and Table 20.3-1 of ASCE 7-16. The values presented on the following page are for the risk-targeted maximum considered earthquake (MCE_R).

MAPPED SPECTRAL ACCELERATIONS

Parameter	Value	2022 CBC Reference
Site Class	D	Section 1613A.2.2
MCE _R Ground Motion Spectral Response Acceleration – Class B (short), S_s	1.822	Figure 1613A.2.1(1)
MCE _R Ground Motion Spectral Response Acceleration – Class B (1 sec), S_1	0.682g	Figure 1613A.2.1(3)
Site Coefficient, F_A	1	Table 1613A.2.3(1)
Site Coefficient, F_V	1.7*	Table 1613A.2.3(2)
Site Class Modified MCE _R Spectral Response Acceleration (short), S_{MS}	1.822g	Section 1613A.2.3 (Eqn 16-20)
Site Class Modified MCE _R Spectral Response Acceleration – (1 sec), S_{M1}	1.16g*	Section 1613A.2.3 (Eqn 16-21)
5% Damped Design Spectral Response Acceleration (short), S_{DS}	1.215g	Section 1613A.2.4 (Eqn 16-22)
5% Damped Design Spectral Response Acceleration (1 sec), S_{D1}	0.773g*	Section 1613A.2.4 (Eqn 16-23)
T_s	0.64 sec	ASCE 7-16 Chapter 11
Site Latitude	34.082013	--
Site Longitude	-117.489677	--
Note: *Per Section 11.4.8 of ASCE/SEI 7-16, a ground motion hazard analysis shall be performed for projects for Site Class “E” sites with S_s greater than or equal to 1.0g and for Site Class “D” and “E” sites with S_1 greater than 0.2g. Section 11.4.8 also provides exceptions which indicates that the ground motion hazard analysis may be waived provided the exceptions are followed. Using the code based values presented in the table above, in lieu of a performing a ground motion hazard analysis, requires the exceptions outlined in ASCE 7-16 Section 11.4.8 be followed.		

6.3.6 Site-Specific Seismic Design Criteria

Based on the site-specific ground motion hazard analysis performed, and in accordance with the ASCE 7-16 Section 21.4, site-specific design acceleration parameters shall be derived using the results of the site-specific ground motion hazard analysis.

The parameter S_{D5} shall be taken as equal to 90 percent of the maximum spectral acceleration obtained from the site-specific analysis at any period within the range from 0.2 to 5 seconds, inclusive. The parameter S_{D1} shall be taken as the maximum value of the product of the spectral acceleration and period for periods 1 to 5 seconds, inclusive. The values of S_{M5} and S_{M1} shall be taken as 1.5 times the site-specific values of S_{D5} and S_{D1} . The site-specific design acceleration parameters shall not be less than 80 percent of the general seismic design values determined by ASCE 7-16 Section 11.4.

The following table presents the site-specific seismic design parameters based on the site-specific ground motion hazard analysis.

SITE-SPECIFIC DESIGN ACCELERATION PARAMETERS

Parameter	Value
Site Class Modified MCE_R Spectral Response Acceleration (short), S_{M5}	2.303g
Site Class Modified MCE_R Spectral Response Acceleration – (1 sec), S_{M1}	1.386g
5% Damped Design Spectral Response Acceleration (short), S_{D5}	1.535g
5% Damped Design Spectral Response Acceleration (1 sec), S_{D1}	0.924g

6.3.7 Site-Specific Peak Ground Acceleration

The site-specific Maximum Considered Earthquake (MCE_G) peak ground acceleration was evaluated in accordance with ASCE 7-16 Section 21.5. The significant difference between the MCE_G peak ground acceleration and the analysis presented above is that the MCE_G is calculated without the risk-targeted adjustment factors.

The probabilistic and deterministic 84th percentile peak ground accelerations were analyzed using the same approaches as described above. The analysis used the same Site Class and scenario earthquake. However, within the probabilistic calculation, the risk-targeted adjustment factor was not applied.

The deterministic MCE_G shall not be less than $0.5F_{PGA}$, where F_{PGA} is determined from ASCE 7-16 Table 11.8-1 with the value of PGA taken as 0.5g. The site-specific MCE_G peak ground acceleration is taken as the lesser of the probabilistic and deterministic MCE_G , provided the value is not less than 80 percent of the value of PGA_M as determined by ASCE 7-16 Equation 11.8.1.

ASCE 7-16 SITE-SPECIFIC PEAK GROUND ACCELERATION

Parameter	Value	ASCE 7-16 Reference
Site-Specific MCE _G Peak Ground Acceleration, PGA _M	0.857g	Section 21.5

6.4 Disaggregation of Seismic Source Parameters

Disaggregation of the MCE peak ground acceleration was performed using the USGS online Earthquake Hazard Toolbox, NSHM Conterminous U.S. 2018 model. The result of the disaggregation analysis indicates that the predominant earthquake contributing to the MCE peak ground acceleration is characterized as a 7.02 magnitude event occurring at a hypocentral distance of 11.82 kilometers from the site.

Disaggregation 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 6.91 magnitude occurring at a hypocentral distance of 14.39 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.

6.5 Liquefaction Potential

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

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

According to the City of Fontana Local Hazard Mitigation Plan (2017) and the San Bernardino Countywide Plan (2010), the site is not located in an area designated as having a potential for liquefaction. Based on the historic high groundwater levels in the site vicinity (greater than 100 to 200 feet beneath the ground surface, the lack of groundwater encountered in our borings, and depth to groundwater recorded in nearby water wells in the vicinity, it is our opinion that the potential for liquefaction of the soils underlying the site is very low.

6.6 Slope Stability

The topography of the site is generally level and the topography in the site vicinity gently slopes to the southeast at a gradient of less than 5%. According to the City of Fontana Local Hazard Mitigation Plan (2017) and the San Bernardino Countywide Plan (2010), the site is not located within an area identified as having a potential for slope instability (USGS, 2024c). There are no known landslides near the site, nor is the site in the path of any known or potential landslides. The potential for slope instability or landslides adversely affecting the proposed project is considered low.

6.7 Earthquake-Induced Flooding

Earthquake-induced flooding is inundation caused by failure of dams or other water-retaining structures due to earthquakes. A review of the City of Fontana Local Hazard Mitigation Plan (2017) and the San Bernardino Countywide Plan (2010), indicates that 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.

6.8 Tsunamis, Seiches, and Flooding

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

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

The site is not within a 100-year flood zone or a 500-year flood zone. The potential for flooding to adversely impact the site is considered low (City of Fontana Local Hazard Mitigation Plan, 2017).

6.9 Oil Fields & Methane Potential

Based on a review of the California Geologic Energy Management Division (CalGEM) Well Finder Website, the site is not located within the limits of an oilfield and there are no active or inactive oil or gas wells documented within the immediate site vicinity (CalGEM, 2024). However, due to the voluntary nature of record reporting by the oil well drilling companies, wells may be improperly located or not shown on the location map and undocumented wells could be encountered during construction. Any wells encountered will need to be properly abandoned in accordance with the current requirements of the CalGEM.

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.

6.10 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 located within an area of known ground subsidence due to groundwater pumping (USGS, 2024d). Ground subsidence typically occurs over a region and may not necessarily result in differential settlement within relatively small areas such as the subject 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 Artificial fill was not encountered during the site investigation; however, fill may exist between borings or in other areas of the site that were not directly explored. The existing 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 Based on these considerations, is recommended that the proposed structure be supported on a conventional foundation system deriving support in a blanket of newly placed engineered fill or competent alluvial soils after completion of the recommended grading. All foundation excavations must be observed and approved in writing by the Geotechnical Engineer prior to placement of steel or concrete. Recommendations for the *Conventional Foundation Design* are provided in section 7.6.
- 7.1.4 As a minimum, it is recommended that the upper 3 feet of existing earth materials within proposed building footprint area be excavated and properly compacted for foundation and slab support. Deeper excavations should be conducted as needed to remove any existing 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 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 All excavations must be observed and approved in writing by the Geotechnical Engineer (a representative of Geocon) prior to the placement of bedding materials, fill, gravel, steel, or concrete.

- 7.1.6 It is anticipated that stable excavations for the recommended grading associated with the proposed structures 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 7.11).
- 7.1.7 Foundations for small outlying structures, such as block walls up to 6 feet high, planter walls or trash enclosures, which will not be tied-in to the proposed structure, may be supported on conventional foundations bearing on a minimum of 12 inches of newly placed engineered fill which extends laterally at least 12 inches beyond the foundation area. Where excavation and proper compaction cannot be performed, foundations may derive support directly in the undisturbed alluvial soils at or below a depth of 2½ feet and should be deepened as necessary to maintain a minimum of 12-inch embedment into recommended bearing materials. If the soils exposed in the excavation bottom are soft or loose, 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 in writing by a Geocon representative.
- 7.1.8 Where new paving is to be placed, it is recommended that all existing fill soils and soft alluvial soils be excavated and properly compacted for paving support. The client should be aware that excavation and compaction of all unsuitable soils in the area of new paving is not required; however, paving constructed over existing uncertified fill or unsuitable soils may experience increased settlement and/or cracking, and may therefore have a shorter design life and increased maintenance costs. As a minimum, the upper 12 inches of soil should be scarified and properly compacted for paving support. Paving recommendations are provided in the *Preliminary Pavement Recommendations* section of this report (see Section 7.10).
- 7.1.9 Based on the results of percolation testing performed at the site, a stormwater infiltration system is considered feasible for this project. Recommendations for infiltration are provided in the *Stormwater Infiltration* section of this report (see Section 7.14).
- 7.1.10 Once the design and foundation loading configuration for the proposed structures 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 re-evaluated 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. Caving should be anticipated in unshored excavations, especially where granular soils are encountered.

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

7.2.3 All onsite excavations must be conducted in such a manner that potential surcharges from existing structures, construction equipment, and vehicle loads are resisted. The surcharge area may be defined by a 1:1 projection down and away from the bottom of an existing foundation or vehicle load. Penetrations below this 1:1 projection will require special excavation measures such as sloping or shoring. Excavation recommendations are provided in the *Temporary Excavations* section of this report (see Section 7.11).

7.2.4 The upper five 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 2022 California Building Code (CBC) Section 1803.5.3. The recommendations presented herein assume that foundations and slabs as well as new paving and hardscape 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 “mildly corrosive” with respect to corrosion of buried ferrous metals on site. The results are presented in Appendix B (Figure B15) and should be considered for design of underground structures. Due to the corrosive potential of the soils, it is recommended that PVC, ABS or other approved plastic piping be utilized in lieu of cast-iron when in direct contact with the site soils.

- 7.3.2 Laboratory tests were performed on representative samples of the on-site soil to measure the percentage of water-soluble sulfate content. Results from the laboratory water-soluble sulfate tests are presented in Appendix B (Figure B15) and indicate that the on-site soil possess a sulfate exposure class of “S0” to concrete structures as defined by 2022 CBC Section 1904 and ACI 318-19 Chapter 19.
- 7.3.3 Geocon West, Inc. does not practice in the field of corrosion engineering and mitigation. If corrosion sensitive improvements are planned, it is recommended that a corrosion engineer be retained to evaluate corrosion test results and incorporate the necessary precautions to avoid premature corrosion of buried metal pipes and concrete structures in direct contact with the soils.

7.4 Grading

- 7.4.1 Grading is anticipated to include preparation of building pads, excavation of site soils for the foundations and utility trenches, placement of backfill for trenches, and preparation of paving and hardscape subgrade.
- 7.4.2 A preconstruction conference should be held at the site prior to the beginning of excavation 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 Earthwork should be observed, and compacted fill tested by representatives of Geocon West, Inc. The existing fill and alluvial soil encountered during exploration are suitable for re-use as an engineered fill, provided any encountered oversize material (greater than 6 inches) and any encountered deleterious debris are removed.
- 7.4.4 Grading should commence with the removal of all existing vegetation and existing improvements from the area to be graded. Deleterious debris such as wood and root structures should be exported from the site and should not be mixed with the fill soils. Asphalt and concrete should not be mixed with the fill soils unless approved by the Geotechnical Engineer. All existing underground improvements planned for removal should be completely excavated and the resulting depressions properly backfilled in accordance with the procedures described herein. Once a clean excavation bottom has been established it must be observed and approved in writing by the Geotechnical Engineer (a representative of Geocon West, Inc.).

- 7.4.5 As a minimum, it is recommended that the upper 3 feet of existing earth materials within proposed building footprint area be excavated and properly compacted for foundation and slab support. Deeper excavations should be conducted as needed to remove any existing 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. All excavations must be observed and approved in writing by the Geotechnical Engineer (a representative of Geocon).
- 7.4.6 All fill and backfill soils should be placed in horizontal loose layers approximately 6 to 8 inches thick, moisture conditioned to near to slightly above optimum moisture content, and properly compacted to a minimum 90 percent of the maximum dry density in accordance with ASTM D 1557 (latest edition).
- 7.4.7 Although not anticipated for this project, all imported fill shall be observed, tested, and approved by Geocon West, Inc. prior to bringing soil to the site. Import fill should consist of the characteristics presented in the table below.

SUMMARY OF IMPORT FILL RECOMMENDATIONS

Soil Characteristic	Values
Expansion Potential	“Very Low” (Expansion Index of 20 or less)
Particle Size	Maximum Dimension Less Than 6 Inches
	Free of Debris
Corrosivity	Less Detrimental Than Existing Onsite Soils

- 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 to slightly above optimum moisture content, and compacted to at least 95 percent relative compaction, as determined by ASTM Test Method D 1557 (latest edition). Paving recommendations are provided in *Preliminary Pavement Recommendations* section of this report (see Section 7.10).

- 7.4.9 Foundations for small outlying structures, such as block walls up to 6 feet high, planter walls or trash enclosures, which will not be tied to the proposed building, may be supported on conventional foundations deriving support on a minimum of 12 inches of newly placed engineered fill which extends laterally at least 12 inches beyond the foundation area. Where excavation and proper compaction cannot be performed, foundations may derive support directly in the undisturbed alluvial soils at and below a depth of 2½ feet below the existing ground surface 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.
- 7.4.10 Utility trenches should be properly backfilled in accordance with the following requirements. 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).
- 7.4.11 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 of between 9 and 13 percent should be anticipated when excavating and compacting the upper few feet of existing earth materials on the site to an average relative compaction of 92 percent.

- 7.5.2 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 Conventional Foundation Design

- 7.6.1 Subsequent to the recommended grading a conventional shallow spread foundation system may be utilized for support of the proposed structure provided foundations derive support in a blanket of newly placed engineered fill or underlying competent alluvial soils. All foundation excavations must be observed and approved in writing by the Geotechnical Engineer (a representative of Geocon), prior to placing steel or concrete.
- 7.6.2 Conventional shallow spread foundations for proposed structures should consist of continuous strip footings and/or isolated spread footings and should be designed using the parameters in the following table.

SUMMARY OF FOUNDATION RECOMMENDATIONS

Parameter	Value
Minimum Continuous Foundation Width	12 Inches
Minimum Isolated Foundation Width	24 Inches
Minimum Foundation Depth	18 Inches Below Lowest Adjacent Grade & 12 Inches into Recommended Bearing Material
Minimum Steel Reinforcement	4 No. 4 Bars, 2 Top and 2 Bottom
Allowable Bearing Capacity – Continuous Foundation	3,600 psf
Allowable Bearing Capacity – Isolated Foundation	4,000 psf
Bearing Capacity Increase	400 psf per Foot of Width
	600 psf per Foot of Depth
Maximum Allowable Bearing Capacity	4,500 psf
Estimated Total Settlement	1 Inch
Estimated Differential Settlement	¼ Inch over 20 Feet

- 7.6.3 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.
- 7.6.4 The allowable bearing pressure may be increased by up to one-third for transient loads due to wind or seismic forces.

- 7.6.5 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.
- 7.6.6 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.
- 7.6.7 Foundation excavations should be observed and approved in writing by the Geotechnical Engineer (a representative of Geocon West, Inc.), prior to the placement of reinforcing steel and concrete to verify that the excavations and exposed soil conditions are consistent with those anticipated. If unanticipated soil conditions are encountered, foundation modifications may be required.
- 7.6.8 This office should be provided with a copy of the final construction plans so that the excavation recommendations presented herein could be properly reviewed and revised if necessary.

7.7 Miscellaneous Foundations

- 7.7.1 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, foundations may derive support directly in the competent undisturbed alluvial soils at and below a depth of 2½ feet and should be deepened as necessary to maintain a minimum 12-inch embedment into the recommended bearing materials.
- 7.7.2 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.

- 7.7.3 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.

7.8 Lateral Design

- 7.8.1 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 newly placed engineered fill or alluvial soils.
- 7.8.2 Passive earth pressure for the sides of foundations and slabs poured against newly placed engineered fill or undisturbed alluvial soils may be computed as an equivalent fluid having a density of 300 pounds per cubic foot (pcf) with a maximum earth pressure of 3,000 psf. When combining passive and friction for lateral resistance, the passive component should be reduced by one-third. A one-third increase in the passive value may be used for wind or seismic loads.

7.9 Concrete Slabs-on-Grade

- 7.9.1 Concrete slabs-on-grade subject to vehicle loading should be designed in accordance with the recommendations in the Preliminary Pavement Recommendations section of this report (Section 7.10).
- 7.9.2 Subsequent to the recommended grading, concrete slabs-on-grade for structures, not subject to vehicle loading, should be a minimum of 4 inches thick and minimum slab reinforcement should consist of No. 3 steel reinforcing bars placed 18 inches on center in both horizontal directions. Steel reinforcing should be positioned vertically near the slab midpoint.

- 7.9.3 Slabs-on-grade at the ground surface that may receive moisture-sensitive floor coverings or may be used to store moisture-sensitive materials should be underlain by a vapor retarder placed directly beneath the slab. The vapor retarder and acceptable permeance should be specified by the project architect or developer based on the type of floor covering that will be installed. The vapor retarder selection and design should be consistent with the guidelines presented in Section 9.3 of the American Concrete Institute's (ACI) Guide for Concrete Slabs that Receive Moisture-Sensitive Flooring Materials (ACI 302.2R-06) as well as ASTM E1745 and should be installed in general conformance with ASTM E 1643 (latest edition) and the manufacturer's recommendations. A minimum thickness of 15 mils extruded polyolefin plastic is recommended; vapor retarders which contain recycled content or woven materials are not recommended. The vapor retarder should have a permeance of less than 0.01 perms demonstrated by testing before and after mandatory conditioning is recommended. The vapor retarder should be installed in direct contact with the concrete slab with proper perimeter seal. If the California Green Building Code requirements apply to this project, the vapor retarder should be underlain by 4 inches of clean aggregate. It is important that the vapor retarder be puncture resistant since it will be in direct contact with angular gravel. As an alternative to the clean aggregate suggested in the Green Building Code, it is our opinion that the concrete slab-on-grade may be underlain by a vapor retarder over 4-inches of clean sand (sand equivalent greater than 30), since the sand will serve a capillary break and will minimize the potential for punctures and damage to the vapor barrier.
- 7.9.4 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.
- 7.9.5 Exterior slabs for walkways and 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 to slightly above optimum moisture content and properly compacted to at least 95 percent relative compaction, as determined by ASTM Test Method D 1557 (latest edition). Crack control joints should be spaced at intervals not greater than 10 feet and should be constructed using saw-cuts or other methods as soon as practical following concrete placement. Crack control joints should extend a minimum depth of one-fourth the slab thickness. Construction joints should be designed by the project structural engineer.

- 7.9.6 The moisture content of the slab subgrade should be maintained and sprinkled as necessary to maintain a moist condition as would be expected in any concrete placement.
- 7.9.7 The recommendations of this report are intended to reduce the potential for cracking of slabs due to settlement. However, even with the incorporation of the recommendations presented herein, foundations, stucco walls, and slabs-on-grade may exhibit some cracking due to minor soil movement and/or concrete shrinkage. The occurrence of concrete shrinkage cracks is independent of the supporting soil characteristics. Their occurrence may be reduced and/or controlled by limiting the slump of the concrete, proper concrete placement and curing, and by the placement of crack control joints at periodic intervals, in particular, where re-entrant slab corners occur.

7.10 Preliminary Pavement Recommendations

- 7.10.1 Where new paving is to be placed, it is recommended that all existing fill or soft alluvium be excavated and properly compacted for paving support. The client should be aware that excavation and compaction of all existing unsuitable soils in the area of new paving is not required; however, paving constructed over existing fill or unsuitable alluvium material may experience increased settlement and/or cracking, and may therefore have a shorter design life and increased maintenance costs. As a minimum, the upper 12 inches of paving subgrade should be scarified, moisture conditioned to near to slightly above optimum moisture content, and properly compacted to at least 95 percent relative compaction, as determined by ASTM Test Method D 1557 (latest edition).
- 7.10.2 The following pavement sections are based on an assumed R-Value of 35. Once site grading activities are complete an R-Value should be obtained by laboratory testing to confirm the properties of the soils serving as paving subgrade, prior to placing pavement.
- 7.10.3 The Traffic Indices listed below are estimates. Geocon does not practice in the field of traffic engineering. The actual Traffic Index for each area should be determined by the project civil engineer. If pavement sections for Traffic Indices other than those listed below are required, Geocon should be contacted to provide additional recommendations. Pavement thicknesses were determined following procedures outlined in the *California Highway Design Manual* (Caltrans). It is anticipated that the majority of traffic will consist of automobile and large truck traffic.

PRELIMINARY PAVEMENT DESIGN SECTIONS

Location	Estimated Traffic Index (TI)	Asphalt Concrete (inches)	Class 2 Aggregate Base (inches)
Automobile Parking and Driveways	4.0	4.0	4.0
Trash Truck & Fire Lanes	7.0	4.0	9.0

- 7.10.4 Asphalt concrete should conform to Section 203-6 of the *“Standard Specifications for Public Works Construction”* (Green Book). Class 2 aggregate base materials should conform to Section 26-1.02A of the *“Standard Specifications of the State of California, Department of Transportation”* (Caltrans). The use of Crushed Miscellaneous Base (CMB) in lieu of Class 2 aggregate base is acceptable. Crushed Miscellaneous Base should conform to Section 200-2.4 of the *“Standard Specifications for Public Works Construction”* (Green Book).
- 7.10.5 Unless specifically designed and evaluated by the project structural engineer, where exterior concrete paving will be utilized for support of vehicles, it is recommended that the concrete be a minimum of 6 inches of concrete reinforced with No. 3 steel reinforcing bars placed 18 inches on center in both horizontal directions. Concrete paving supporting vehicular traffic should be underlain by a minimum of 4 inches of aggregate base and a properly compacted subgrade. The subgrade and base material should be compacted to 95 percent relative compaction as determined by ASTM Test Method D 1557 (latest edition).
- 7.10.6 The performance of pavements is highly dependent upon providing positive surface drainage away from the edge of pavements. Ponding of water on or adjacent to the pavement will likely result in saturation of the subgrade materials and subsequent cracking, subsidence and pavement distress. If planters are planned adjacent to paving, it is recommended that the perimeter curb be extended at least 12 inches below the bottom of the aggregate base to minimize the introduction of water beneath the paving.

7.11 Temporary Excavations

- 7.11.1 Excavations less than 5 feet in height are anticipated for grading and construction activities. The excavations are expected to expose artificial fill and alluvial soils, which may be subject to caving where granular soils are exposed. Temporary vertical excavations up to 5 feet in height may be attempted where loose soils or caving sands are not present, and where excavations are not surcharged by adjacent traffic or structures.

- 7.11.2 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 a maximum of 7 feet in height. A uniform slope does not have a vertical portion.
- 7.11.3 Where temporary construction slopes 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 slopes 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. The soils exposed in the cut slopes should be inspected during excavation by our personnel and the contractor's competent person 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.

7.12 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 *Conventional Foundation Design* (see Section 7.6).
- 7.12.3 Retaining walls with a level backfill surface that are not restrained at the top should be designed utilizing a triangular distribution of pressure (active pressure). Restrained walls are those that are not allowed to rotate more than $0.001H$ (where H equals the height of the retaining portion of the wall in feet) at the top of the wall. Where walls are restrained from movement at the top, walls may be designed utilizing a triangular distribution of pressure (at-rest pressure). The table below presents recommended pressures to be used in retaining wall design, assuming that proper drainage will be maintained.

RETAINING WALL WITH LEVEL BACKFILL SURFACE

HEIGHT OF RETAINING WALL (Feet)	ACTIVE PRESSURE EQUIVALENT FLUID PRESSURE (Pounds Per Cubic Foot)	AT-REST PRESSURE EQUIVALENT FLUID PRESSURE (Pounds Per Cubic Foot)
Up to 5	30	60

- 7.12.4 The wall pressures provided above assume that the proposed retaining walls will support relatively undisturbed alluvium. If sloping techniques are to be utilized for construction of proposed walls, which would result in a wedge of engineered fill behind the retaining walls, revised earth pressures may be required to account for the expansive potential of the soil placed as engineered fill. This should be evaluated once the use of sloping measures is established and once the geotechnical characteristics of the engineered backfill soils can be further evaluated.
- 7.12.5 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 Additional active pressure should be added for a surcharge condition due to sloping ground, vehicular traffic or adjacent structures and should be designed for each condition as the project progresses.

7.13 Retaining Wall Drainage

- 7.13.1 Retaining walls not designed for hydrostatic pressures 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 10). 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 11). 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. Drainage should not be allowed to flow uncontrolled over descending slopes.
- 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 Stormwater Infiltration

- 7.14.1 During the site exploration, boring B1 was used to perform percolation testing. The boring was excavated to a depth of 25½ feet below ground surface, as indicated in the table below. Slotted casing was placed in the borings and the borings were then filled with water to pre-saturate the soils. The casing was refilled with water and percolation test readings were performed after repeated flooding of the cased excavation.
- 7.14.2 The field-measured percolation rate has been adjusted to infiltration rates in accordance with the San Bernardino County's *Technical Guidance Document for Water Quality Management Plans*, which references the *Riverside County Flood Control and Water Conservation District, LID BMP Manual, Appendix A* for infiltration basins. Additional correction factors may be required and should be applied by the engineer in responsible charge of the design of the stormwater infiltration system and based on applicable guidelines. Percolation test results are provided as Figure 12.

Boring	Soil Type	Infiltration Depth (ft)	Measured Percolation Rate (in/hour)
B1	Sand (SP) & Silty Sand (SM)	7 – 16	6.13

- 7.14.3 The results of the percolation testing indicate that the soils at B1 are conducive to infiltration. It is our opinion that the soil zones encountered at the depths and location of boring B1 as listed in the table above are suitable for infiltration of stormwater.
- 7.14.4 It is our opinion that the introduction of stormwater at the depth and location indicated above will not induce excessive hydro-consolidation (see Figures B6 through B10), will not create a perched groundwater condition, will not affect soil structure interaction of existing or proposed foundations due to expansive soils, will not saturate soils supported by existing or proposed retaining walls, and will not increase the potential for liquefaction. Resulting settlements are anticipated to be less than ¼ inch, if any.
- 7.14.5 The infiltration system must be located such that the closest distance between an adjacent foundation is at least 10 feet in all directions from the zone of saturation. The zone of saturation may be assumed to project downward from the discharge of the infiltration facility at a gradient of 1:1. Additional property line or foundation setbacks may be required by the governing jurisdiction and should be incorporated into the stormwater infiltration system design as necessary.
- 7.14.6 Where the 10-foot horizontal setback cannot be maintained between the infiltration system and an adjacent footing, and the infiltration system penetrates below the foundation influence line, the proposed stormwater infiltration system must be designed to resist the surcharge from the adjacent foundation. The foundation surcharge line may be assumed to project down away from the bottom of the foundation at a 1:1 gradient. The stormwater infiltration system must still be sufficiently deep to maintain the 10-foot vertical offset between the bottom of the footing and the zone of saturation.
- 7.14.7 Subsequent to the placement of the infiltration system, it is acceptable to backfill the resulting void space between the excavation sidewalls and the infiltration system with minimum two-sack slurry provided the slurry is not placed in the infiltration zone. It is recommended that pea gravel be utilized adjacent to the infiltration zone so communication of water to the soil is not hindered.

- 7.14.8 Due to the preliminary nature of the project at this time, the type of stormwater infiltration system and location of the stormwater infiltration systems has not yet been determined. The design drawings should be reviewed and approved by the Geotechnical Engineer. The installation of the stormwater infiltration system should be observed and approved by the Geotechnical Engineer (a representative of Geocon).

7.15 Surface Drainage

- 7.15.1 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.
- 7.15.2 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 2022 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.
- 7.15.3 Positive site drainage should be provided away from structures, pavement, and the tops of slopes to swales or other controlled drainage structures.
- 7.15.4 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.

7.16 Plan Review

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

LIMITATIONS AND UNIFORMITY OF CONDITIONS

1. The 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.
2. 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 Incorporated 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 Incorporated.
3. This report is issued with the understanding that it is the responsibility of the owner or 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.
4. The findings of this report are valid as of the present date. However, changes in the conditions of a property can occur with the passage of time, whether they be 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.

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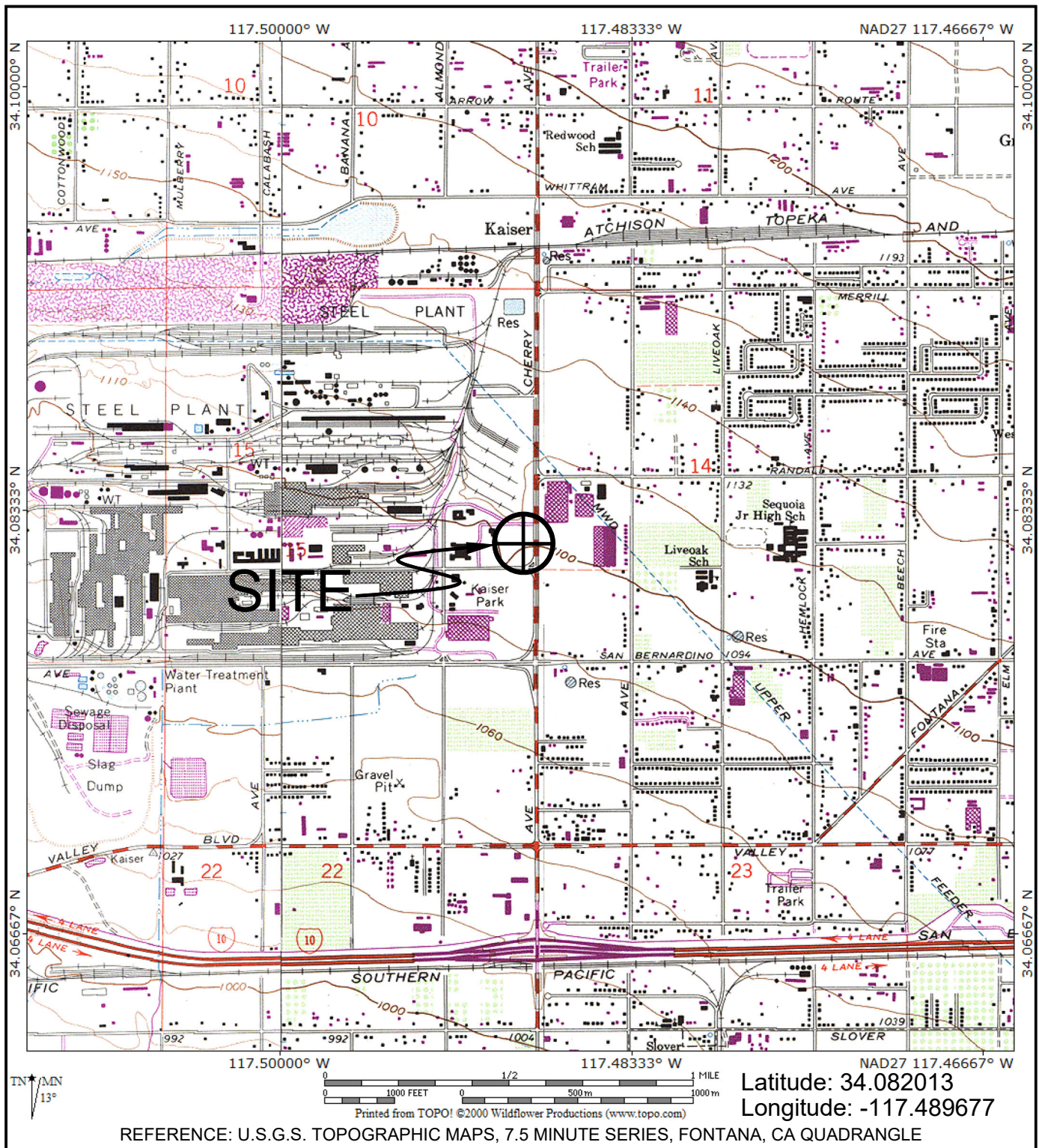
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VICINITY MAP

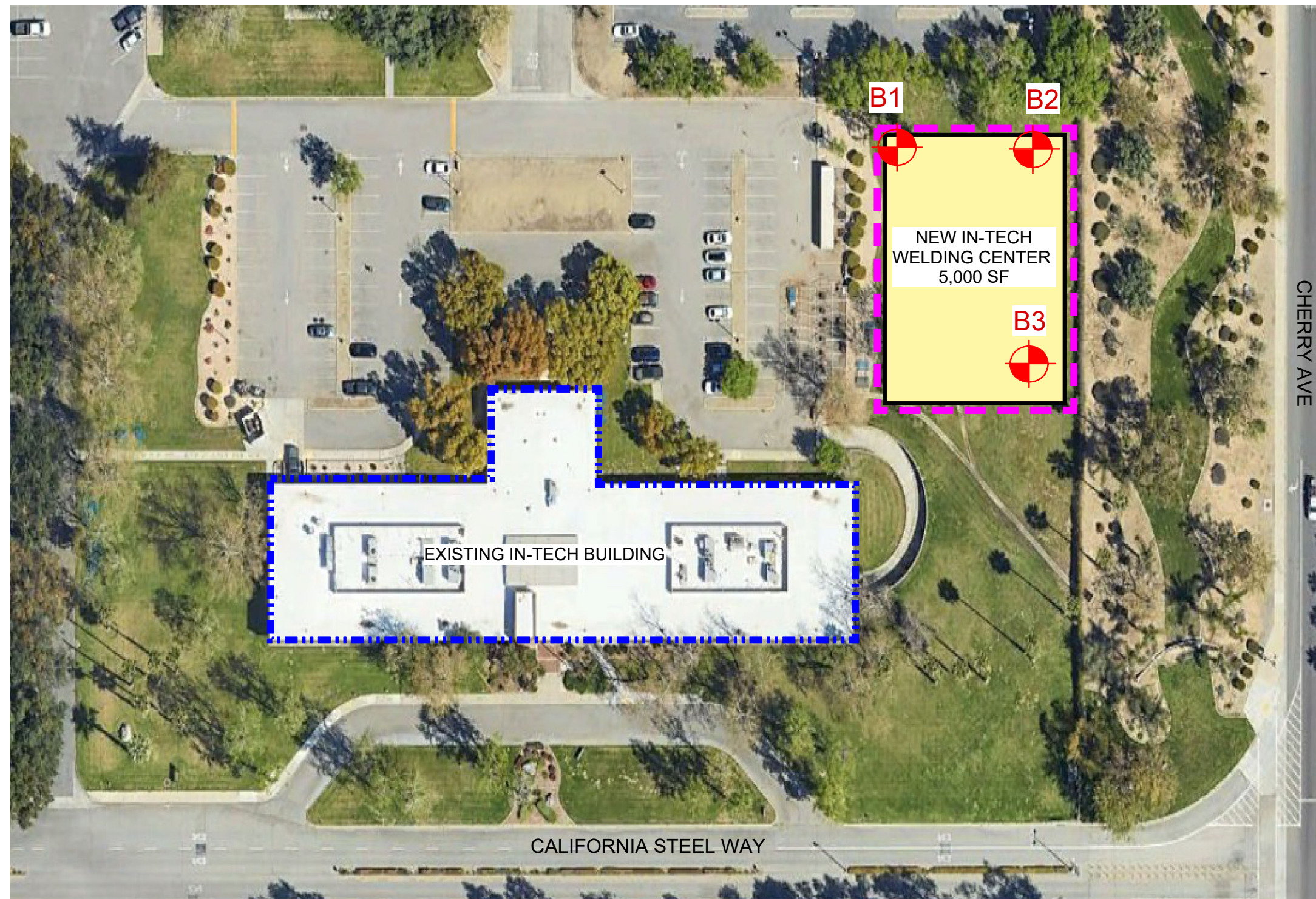
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FONTANA, CALIFORNIA

DEC 2024

PROJECT NO. W1145-99-10

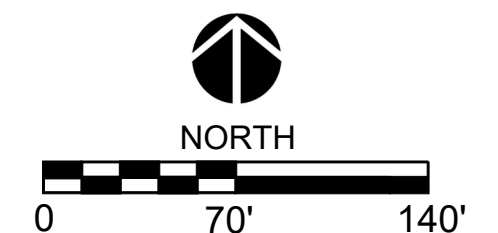
FIG. 1


LEGEND



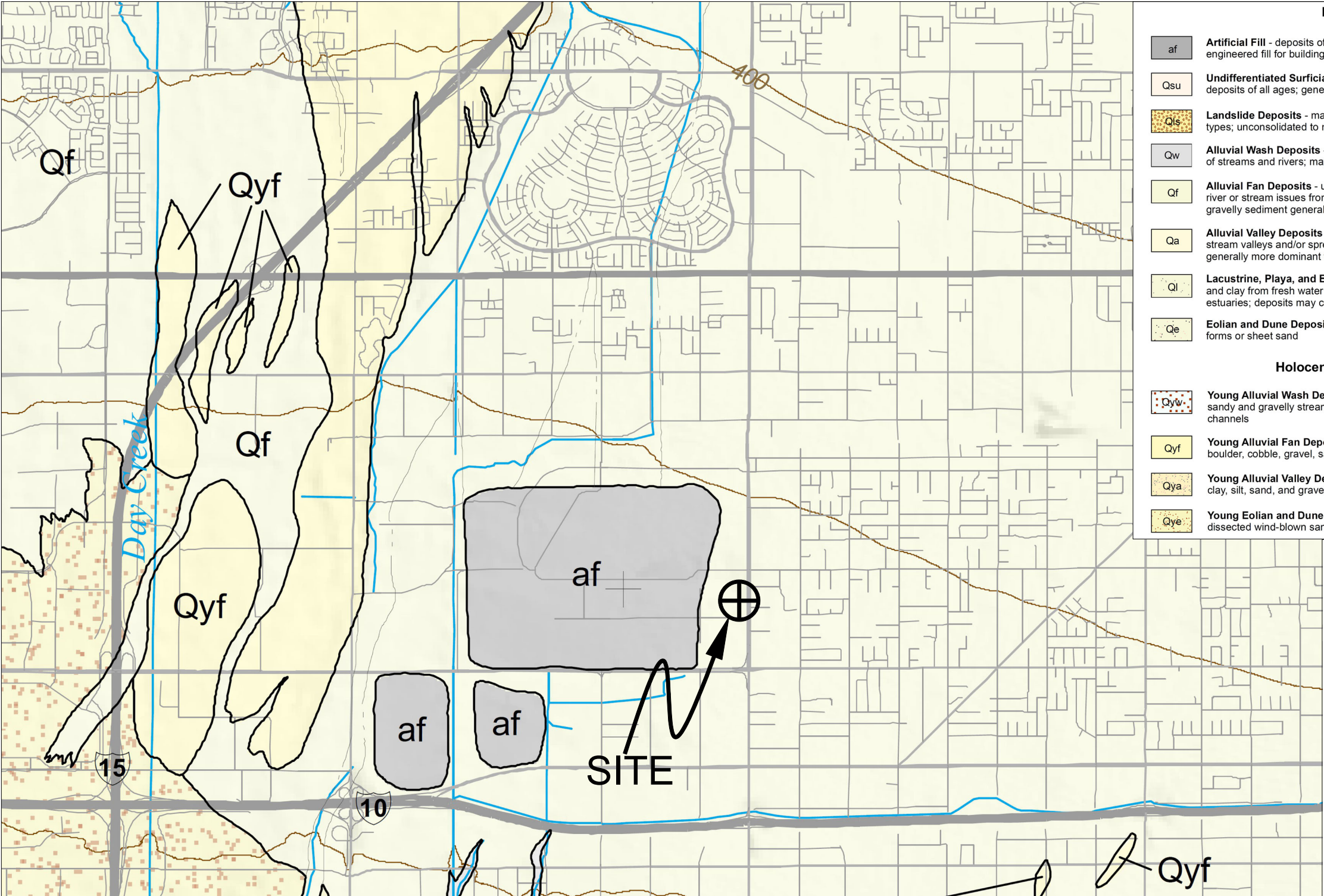
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- ■ ■ Proposed New In-Tech Welding Center
- Existing In-Tech Building
-  Approximate Boring Locations
-  Alluvial Fan Deposits - unconsolidated boulders, cobbles, gravel, sand, and silt recently deposited where a river or stream issues from a confined valley or canyon; sediment typically deposited in a fan-shaped cone; gravelly sediment generally more dominant than sandy sediment



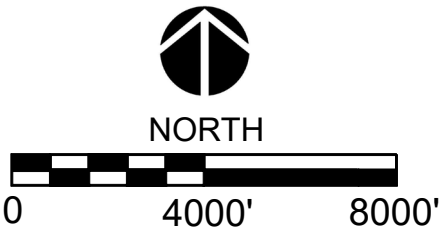
<h1 style="margin: 0;">GEOCON</h1> <h2 style="margin: 0;">W E S T, I N C.</h2>		
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DRAFTED BY: CT	CHECKED BY: NDB	

<h1 style="margin: 0;">SITE PLAN</h1>			
<h2 style="margin: 0;">Chaffey College - In-Tech Welding Center</h2> <h3 style="margin: 0;">9400 Cherry Avenue</h3> <h3 style="margin: 0;">Fontana, CA</h3>			
<table border="1" style="width: 100%; border-collapse: collapse;"> <tr> <td style="width: 33%; padding: 5px;">DEC 2024</td> <td style="width: 33%; padding: 5px;">PROJECT NO. W1145-99-10</td> <td style="width: 33%; padding: 5px;">FIG. 2</td> </tr> </table>	DEC 2024	PROJECT NO. W1145-99-10	FIG. 2
DEC 2024	PROJECT NO. W1145-99-10	FIG. 2	



Late Holocene (Surficial Deposits)	
af	Artificial Fill - deposits of fill resulting from human construction, mining, or quarrying activities; includes engineered fill for buildings, roads, dams, airport runways, harbor facilities, and waste landfills
Qsu	Undifferentiated Surficial Deposits - includes colluvium, slope wash, talus deposits, and other surface deposits of all ages; generally unconsolidated but locally may contain consolidated layers
Qls	Landslide Deposits - may include debris flows and older landslides of various earth material and movement types; unconsolidated to moderately well-consolidated
Qw	Alluvial Wash Deposits - unconsolidated sandy and gravelly sediment deposited in recently active channels of streams and rivers; may contain loose to moderately loose sand and silty sand
Qf	Alluvial Fan Deposits - unconsolidated boulders, cobbles, gravel, sand, and silt recently deposited where a river or stream issues from a confined valley or canyon; sediment typically deposited in a fan-shaped cone; gravelly sediment generally more dominant than sandy sediment
Qa	Alluvial Valley Deposits - unconsolidated clay, silt, sand, and gravel recently deposited parallel to localized stream valleys and/or spread more regionally onto alluvial flats of larger river valleys; sandy sediment generally more dominant than gravelly sediment
Ql	Lacustrine, Playa, and Estuarine (Paralic) Deposits - mostly unconsolidated fine-grained sand, silt, mud, and clay from fresh water (lacustrine) lakes, saline (playa) dry lakes that are periodically flooded, and estuaries; deposits may contain salt and other evaporites
Qe	Eolian and Dune Deposits - unconsolidated, generally well-sorted wind-blown sand; may occur as dune forms or sheet sand
Holocene to Late Pleistocene (Surficial Deposits)	
Qyw	Young Alluvial Wash Deposits - unconsolidated to slightly consolidated, undissected to slightly dissected sandy and gravelly stream bed sediments in marginal parts of active and recently active washes and river channels
Qyf	Young Alluvial Fan Deposits - unconsolidated to slightly consolidated, undissected to slightly dissected boulder, cobble, gravel, sand, and silt deposits issued from a confined valley or canyon
Qya	Young Alluvial Valley Deposits - unconsolidated to slightly consolidated, undissected to slightly dissected clay, silt, sand, and gravel along stream valleys and alluvial flats of larger rivers
Qye	Young Eolian and Dune Deposits - unconsolidated to slightly consolidated, undissected to slightly dissected wind-blown sands

Latitude: 34.082013
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GEOLOGIC MAP

Chaffey College - In-Tech Welding Center

9400 Cherry Avenue

Fontana, CA

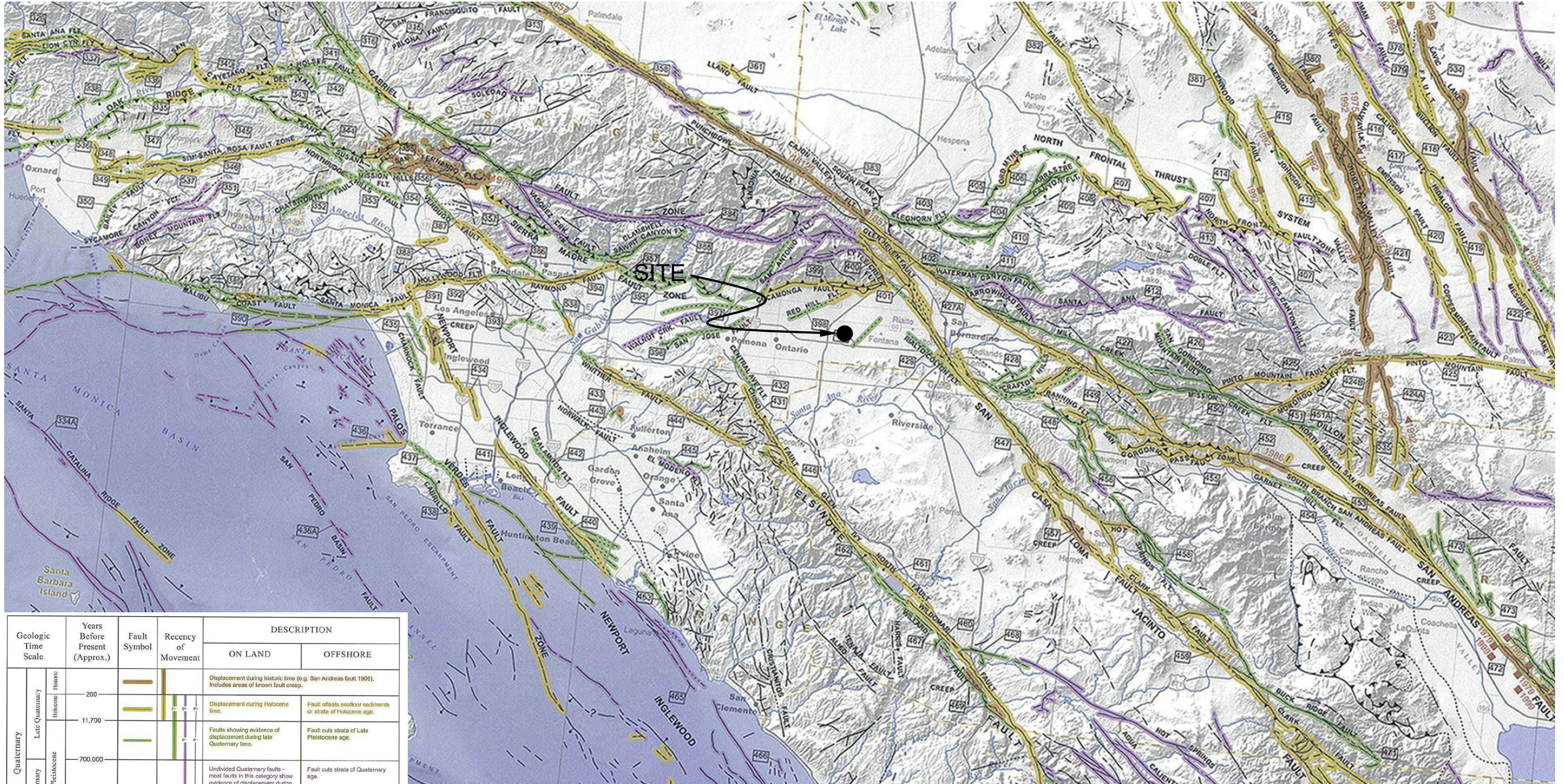
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PROJECT NO. W1145-99-10

FIG. 3

Reference: California Geological Survey, 2018, Earthquake Fault Zones, A Guide for Government Agencies, Property Owners/Developers, and Geoscience Practitioners for Assessing Fault Rupture Hazards in California, Special Publication 42, Revised 2018.

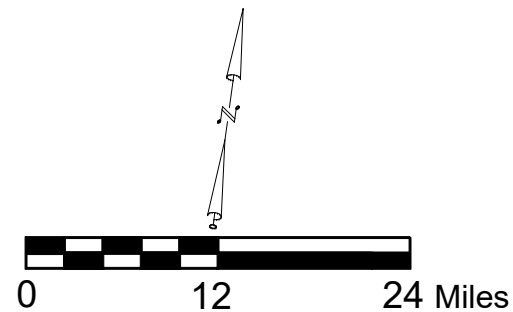
Reference: Jennings, C.W. and Bryant, W. A., 2010, Fault Activity Map of California, California Geological Survey Geologic Data Map No. 6.




Geologic Time Scale			Years Before Present (Approx.)	Fault Symbol	Recency of Movement	DESCRIPTION		
						ON LAND	OFFSHORE	
Quaternary	Late Quaternary	Holocene				Displacement during historic time (e.g. San Andreas fault 1906). Includes areas of known fault creep.		
			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.	
			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					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.

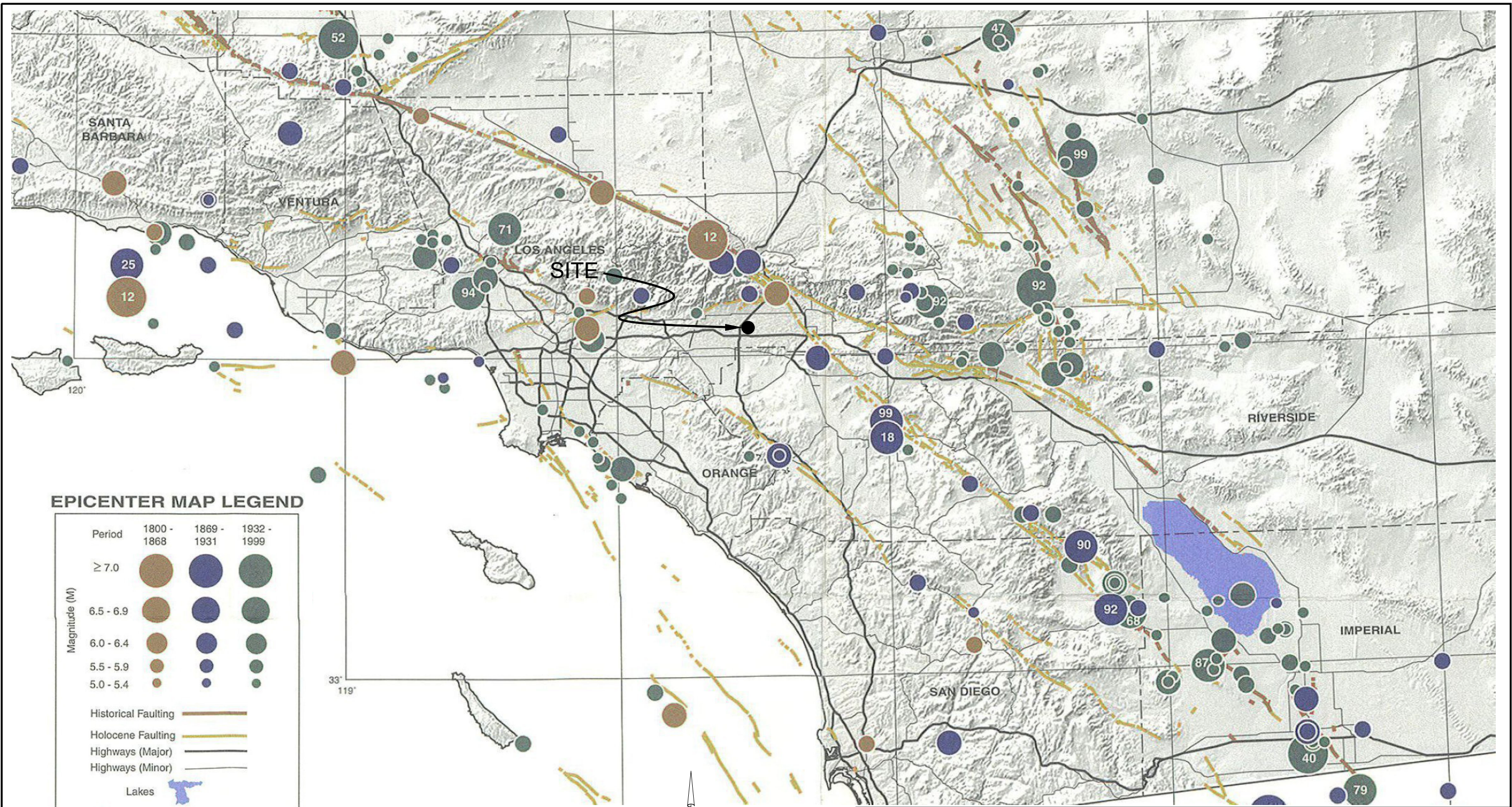
* Quaternary now recognized as extending to 2.6 Ma (Walker and Geissman, 2009). Quaternary faults in this map were established using the previous 1.6 Ma criterion.

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<h1 style="text-align: center;">REGIONAL FAULT MAP</h1>		
<p style="text-align: center;">Chaffey College - In-Tech Welding Center 9400 Cherry Avenue Fontana, CA</p>		
DEC 2024	PROJECT NO. W1145-99-10	FIG. 4

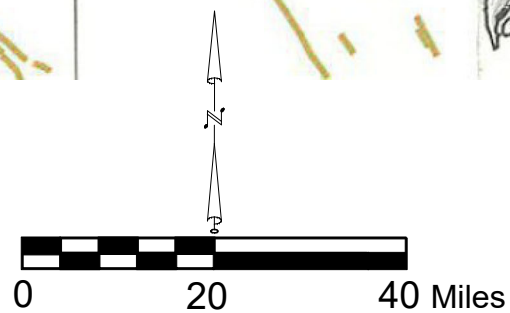


EPICENTER MAP LEGEND

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

Latitude: 34.082013
Longitude: -117.489677

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



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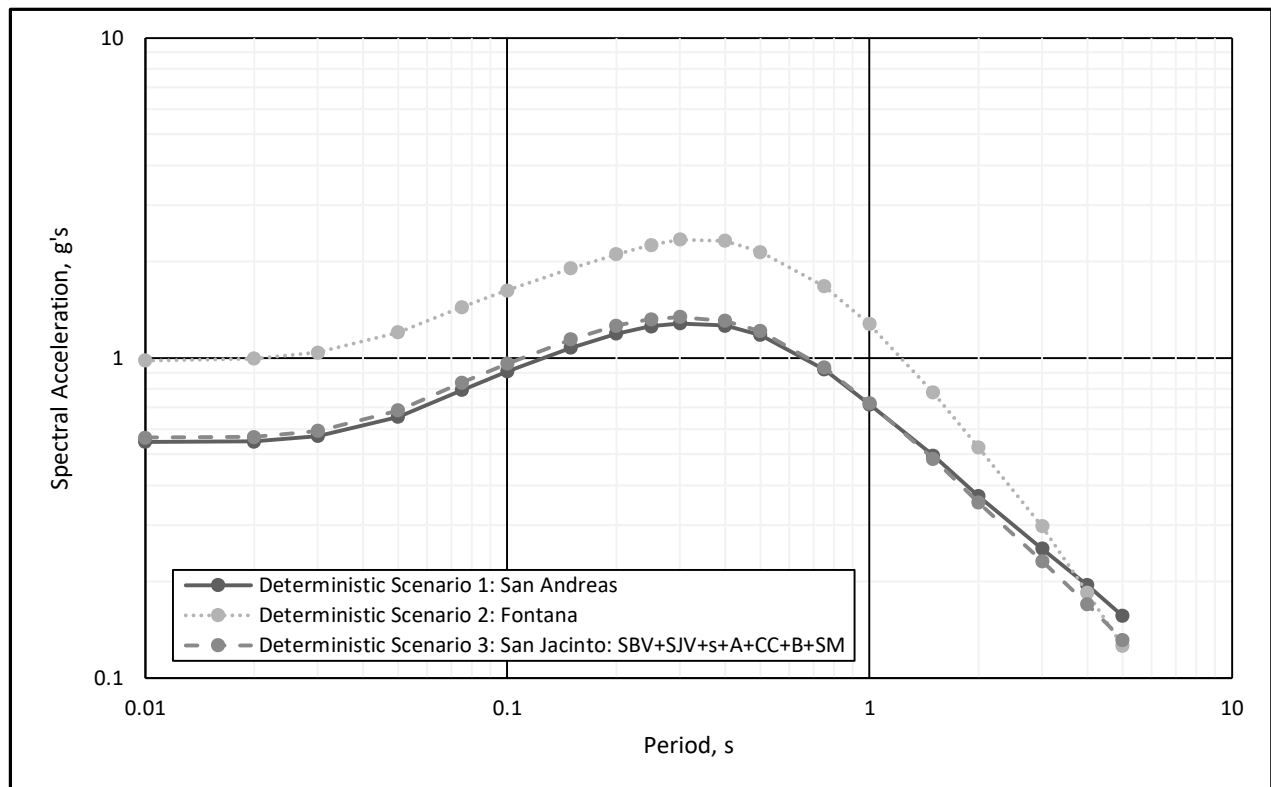
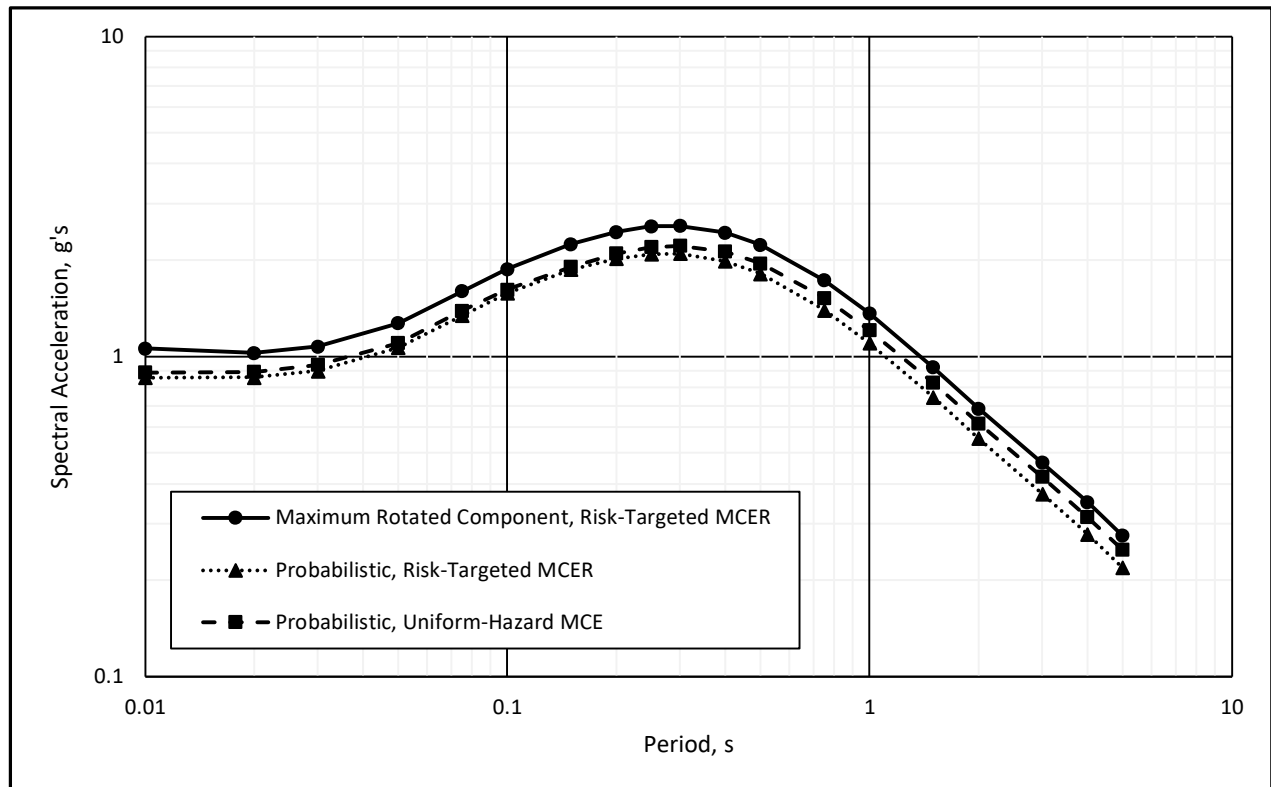
REGIONAL SEISMICITY MAP

Chaffey College - In-Tech Welding Center
9400 Cherry Avenue
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PROJECT NO. W1145-99-10

FIG.5



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DESIGN RESPONSE SPECTRUM

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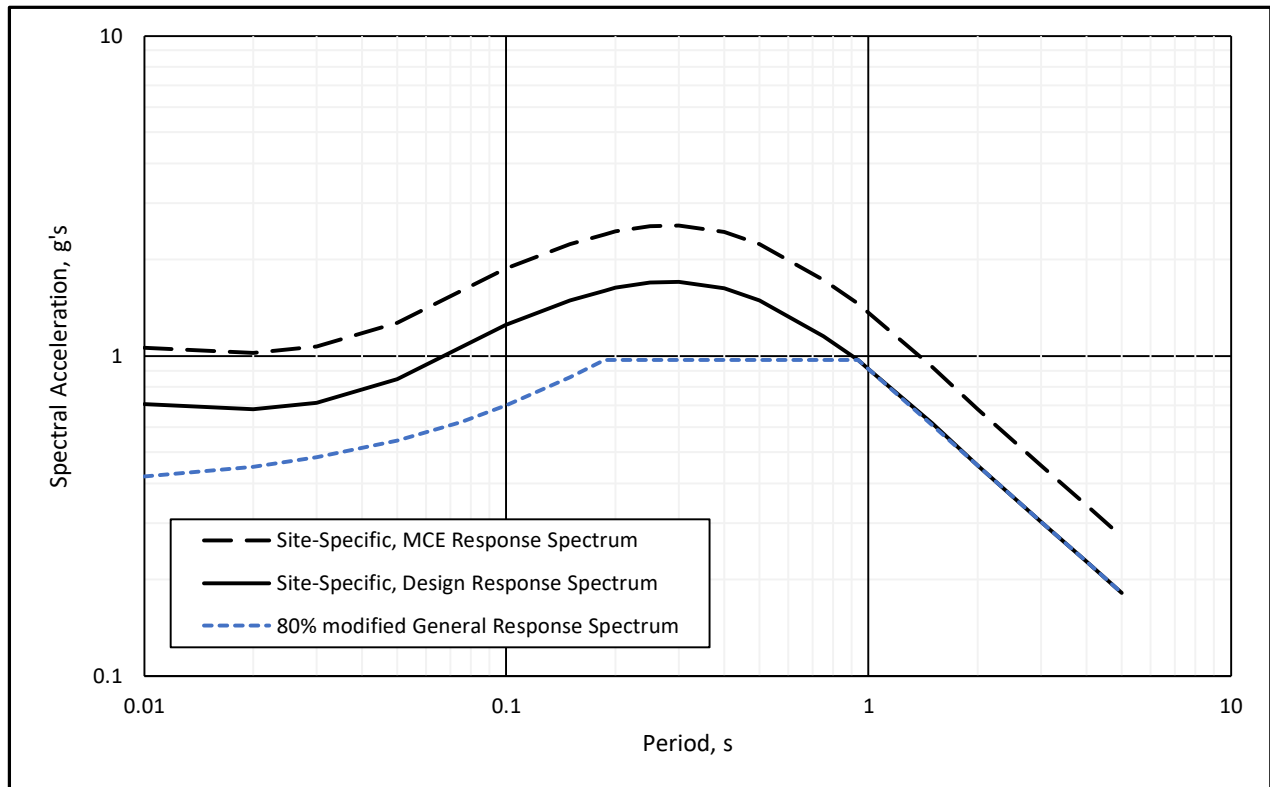
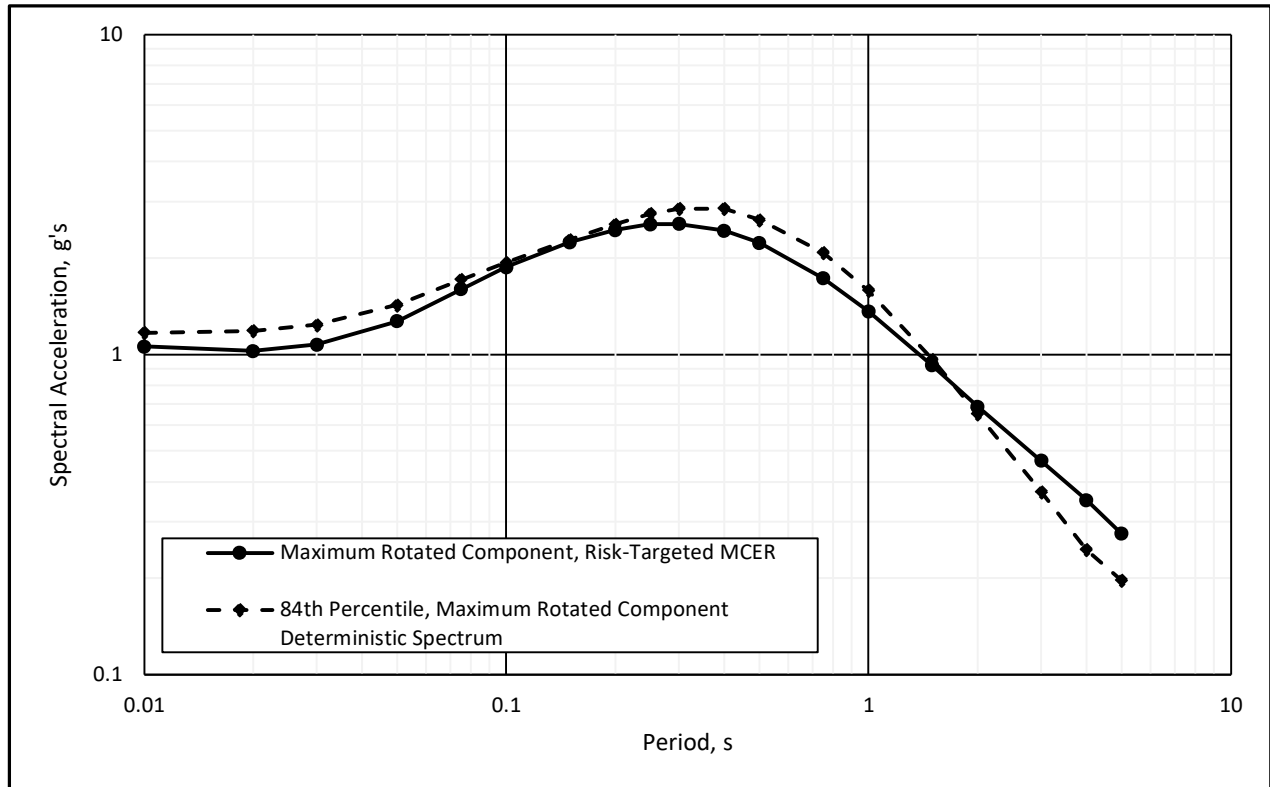
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Figure 6



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DESIGN RESPONSE SPECTRUM

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Project No.:

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DEC 2024

Figure 7

Spectral Period (seconds)	Probabilistic Uniform-Hazard	Risk-Targeted, Probabilistic	Risk Factor, Cr	Maximum-Rotated Component Scale Factor	MRC, Risk-Targeted Probabilistic	84th Percentile, Deterministic	Site-Specific Design Earthquake	80% Modified General Response Spectrum	Site-Specific Maximum Considered Earthquake
0.01	0.890	0.857	0.963	1.19	1.059	1.167	0.706	0.420	1.059
0.02	0.895	0.860	0.961	1.19	1.024	1.185	0.683	0.451	1.024
0.03	0.940	0.901	0.958	1.19	1.072	1.237	0.715	0.482	1.072
0.05	1.101	1.066	0.968	1.19	1.268	1.428	0.845	0.544	1.268
0.08	1.389	1.343	0.967	1.19	1.598	1.714	1.065	0.622	1.598
0.10	1.616	1.575	0.975	1.19	1.875	1.933	1.250	0.700	1.875
0.15	1.905	1.867	0.980	1.20	2.240	2.282	1.493	0.856	2.240
0.19	--	--	--	--	--	--	1.598	0.972	2.398
0.20	2.094	2.022	0.966	1.21	2.447	2.551	1.631	0.972	2.447
0.25	2.195	2.088	0.951	1.22	2.548	2.750	1.698	0.972	2.548
0.30	2.222	2.098	0.944	1.22	2.559	2.854	1.706	0.972	2.559
0.40	2.124	1.981	0.933	1.23	2.437	2.852	1.625	0.972	2.437
0.50	1.951	1.815	0.930	1.23	2.232	2.633	1.488	0.972	2.232
0.75	1.523	1.394	0.916	1.24	1.729	2.076	1.153	0.972	1.729
0.94	--	--	--	--	--	--	0.972	0.972	1.458
1.0	1.208	1.099	0.910	1.24	1.363	1.582	0.909	0.909	1.364
1.5	0.827	0.745	0.902	1.24	0.924	0.966	0.616	0.606	0.924
2.0	0.618	0.554	0.897	1.24	0.687	0.650	0.455	0.455	0.682
3.0	0.420	0.372	0.885	1.25	0.465	0.372	0.303	0.303	0.455
4.0	0.314	0.278	0.884	1.26	0.350	0.245	0.227	0.227	0.341
5.0	0.248	0.219	0.881	1.26	0.276	0.197	0.182	0.182	0.273

$$SM_S = \frac{2.303}{1.5} g$$

$$SM_1 = \frac{1.386}{1.5} g$$

$$SD_S = \frac{1.535}{1.5} g$$

$$SD_1 = \frac{0.924}{1.5} g$$


Reference: ASCE 7-16 21.4 DESIGN ACCELERATION PARAMETERS

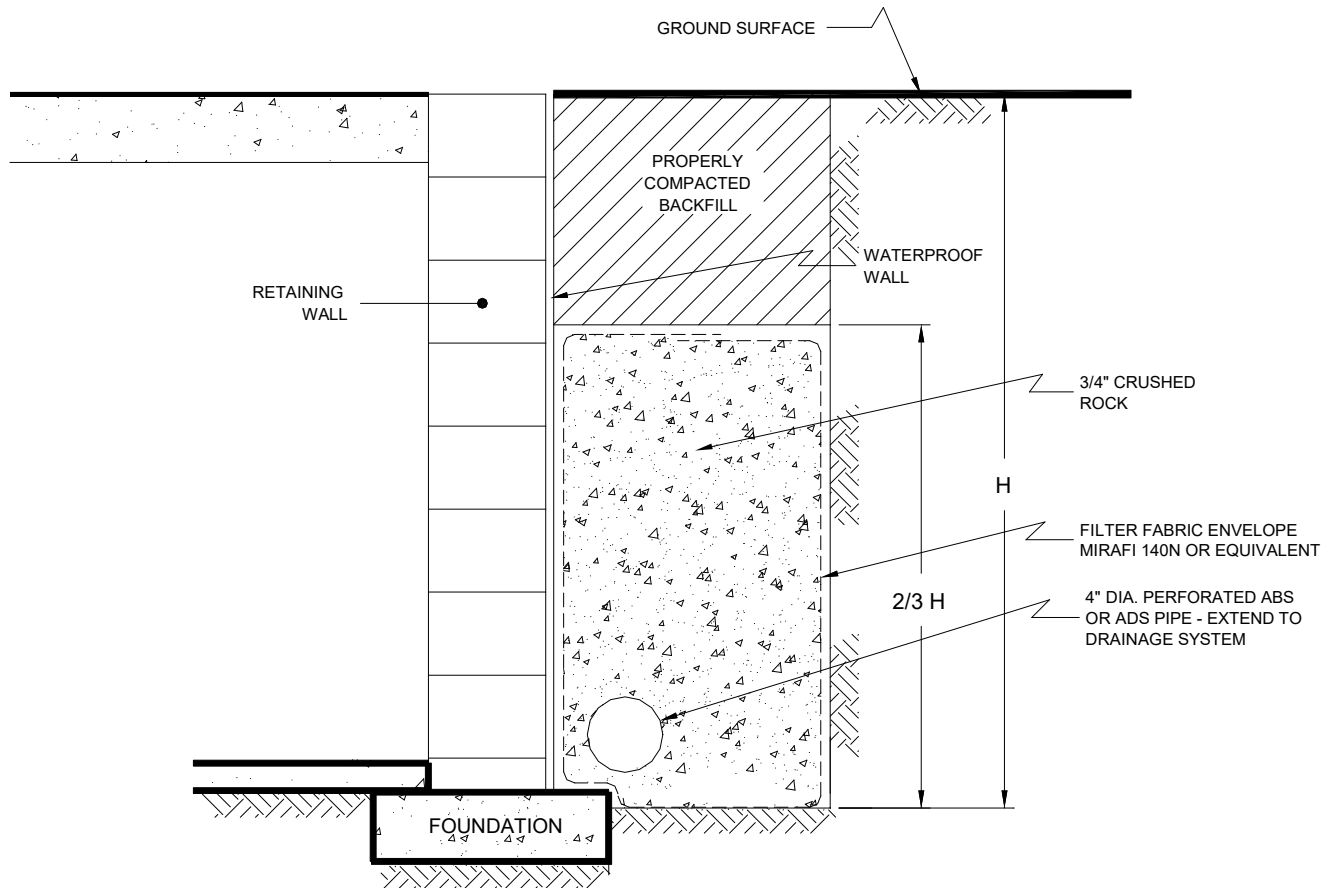
Where the site-specific procedure is used to determine the design ground motion in accordance with Section 21.3, the parameter S_{DS} shall be taken as 90% of the maximum spectral acceleration, S_a , obtained from the site-specific spectrum, at any period within the range from 0.2 to 5 s, inclusive. The parameter S_{D1} shall be taken as the maximum value of the product, TS_a , for periods from 1 to 2 s for sites with $V_{s,30} > 1,200$ ft/s ($V_{s,30} > 365.76$ m/s) and for periods from 1 to 5 s for sites with $V_{s,30} \leq 1,200$ ft/s ($V_{s,30} \leq 365.76$ m/s). The parameters S_{MS} and S_{M1} shall be taken as 1.5 times S_{DS} and S_{D1} , respectively. The values so obtained shall not be less than 80% of the values determined in accordance with Section 11.4.3 for S_{MS} and S_{M1} and Section 11.4.5 for S_{DS} and S_{D1} .

Spectral acceleration values reported in units of "g".

"--" Indicates that spectral period was not used at that calculation step

 GEOCON	DESIGN RESPONSE SPECTRUM				Project No.:	W1145-99-10
					CHAFFEY COLLEGE 9400 CHERRY AVENUE FONTANA, CALIFORNIA	
	Checked by: JJK				DEC 2024	Figure 8

Parameter	Scenario 1	Scenario 2	Scenario 3	--	--	Reference
Parent Fault Name	San Andreas	Fontana	San Jacinto	--	--	--
Scenario Name	San Andreas: PK+CH+CC+BB+NM+ SM+NSM+SSB+BG+C	Fontana	San Jacinto: SBV+SJV+s+A+CC+B+ SM	--	--	BSSC Online Scenario Catalog
Earthquake Magnitude	8.18	6.75	7.76	--	--	BSSC Online Scenario Catalog
Fault Mechanism	--	Reverse	RL Strike Slip	--	--	--
Fault Dip (°)	86.4	80	90	--	--	BSSC 2014 ¹
Fault Width	13.1	14.9	15.01	--	--	BSSC 2014 ¹
Rake (°)	180	0	180	--	--	BSSC 2014 ¹
Z _{TOR} (km)	0	0	0	--	--	BSSC 2014 ¹
Rrup (km)	19.12	0.75	14.16	--	--	--
Rjb (km)	19.12	0	14.16	--	--	--
Rx (km)	19.12	0.76	14.16	--	--	--
V _{s30} (m/s)	354	354	354	--	--	Site-Specific Measurement
Z _{1.0} (km)	0.2	0.2	0.2	--	--	SCEC Community Velocity Model Version 4, Iteration 26, Basin Depth
Z _{2.5} (km)	0.35	0.35	0.35	--	--	SCEC Community Velocity Model Version 4, Iteration 26, Basin Depth
1 - BSSC 2014, aka. UCERF3_EventSet_All on GitHub						
			DETERMINISTIC SCENARIO EVENTS		Project No.: W1145-99-10	
					CHAFFEY COLLEGE 9400 CHERRY AVENUE FONTANA, CALIFORNIA	
			Checked by: JJK		DEC 2024	Figure 9



NO SCALE

GEOCON
WEST, INC.



ENVIRONMENTAL GEOTECHNICAL MATERIALS
2807 MCGAW AVENUE - IRVINE, CA 92614
PHONE (949) 491-6570

DRAFTED BY: CT

CHECKED BY: NDB

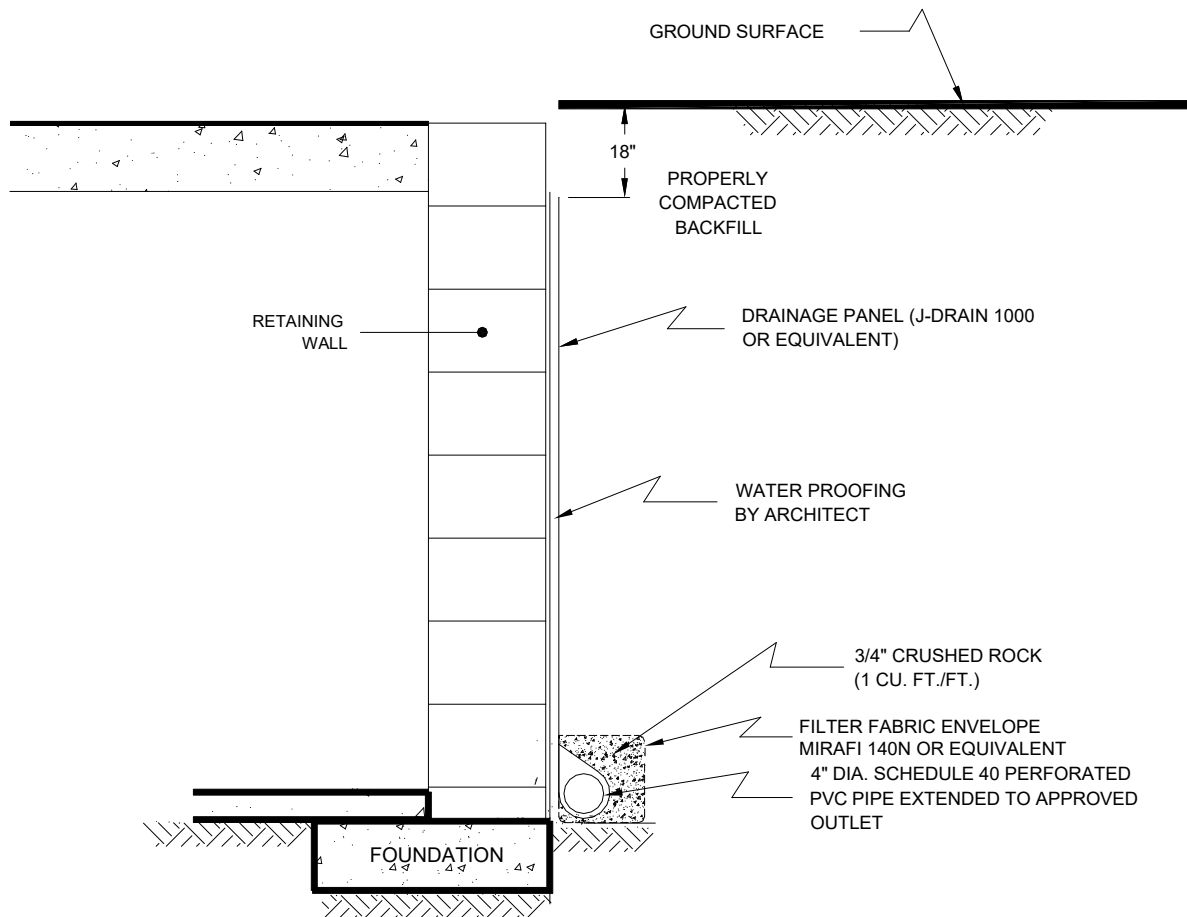
RETAINING WALL DRAIN DETAIL

Chaffey College - In-Tech Welding Center
9400 Cherry Avenue
Fontana, CA

DEC 2024

PROJECT NO. W1145-99-10

FIG. 10



NO SCALE

GEOCON
WEST, INC.



ENVIRONMENTAL GEOTECHNICAL MATERIALS
2807 MCGAW AVENUE - IRVINE, CA 92614
PHONE (949) 491-6570

DRAFTED BY: CT

CHECKED BY: NDB

RETAINING WALL DRAIN DETAIL

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9400 Cherry Avenue
Fontana, CA

DEC 2024

PROJECT NO. W1145-99-10

FIG. 11

PERCOLATION TEST DATA SHEET

Project:	Chaffey College - In-Tech Welding Center	Project No.:	W1145-99-10	Date:	Tuesday, September 17, 2024
Test Hole No.:	B1	Tested By:	CT		
Depth of Test Hole, DT:	25.5	USCS Soil Classification:	Sand (SP) & Silty Sand (SM)		
Test Hole Dimensions (inches)				Length	Width
Diameter (if round) =		8	Sides (if rectangular) =		--

Sandy Soil Criteria Test*

Trial No.	Start Time	End Time	Δt Time Interval (min)	D ₀ Initial Depth to Water (in)	D _f Final Depth to Water (in)	ΔD Change in Water Level (in)	Greater than or Equal to 6"? (y/n)
1	8:30 AM	8:55 AM	25	60.0	217.2	157.2	yes
2	9:30 AM	9:55 AM	25	60.0	213.6	153.6	yes

*If two consecutive measurements show that six inches of water seeps away in less than 25 minutes, the test shall be run for an additional hour with measurements, taken every 10 minutes. Otherwise, pre-soak (fill) overnight. Obtain at least twelve measurements per hole over at least six hours (approximately 30 minute intervals) with a precision of at least 0.25".

Trial No.	Start Time	End Time	Δt Time Interval (min)	D ₀ Initial Depth to Water (in)	D _f Final Depth to Water (in)	ΔD Change in Water Level (in)	Percolation Rate (min/in)
1	10:49 AM	10:59 AM	10.00	84.0	184.8	100.8	0.10
2	11:03 AM	11:13 AM	10.00	84.0	180.0	96.0	0.10
3	11:15 AM	11:25 AM	10.00	84.0	177.6	93.6	0.11
4	11:30 AM	11:40 AM	10.00	84.0	175.2	91.2	0.11
5	11:43 AM	11:53 AM	10.00	84.0	175.2	91.2	0.11
6	11:58 AM	12:08 PM	10.00	84.0	175.2	91.2	0.11
7							
8							

Infiltration Rate Calculation:

Time Interval, Δt =	10.00	minutes	Height of Gravel Pack =	1	feet
Final Depth to Water, D _f =	175.2	inches	Gravel Pack Material =	Sand	
Test Hole Radius, r =	4	inches	Gravel Pack Porosity =	0.3	
Initial Depth to Water, D ₀ =	84.0	inches	Casing Inner Diameter =	2.3	inches
Total Depth of Test Hole, D _t =	306	inches	Casing Outer Diameter =	2.3	inches
H ₀ =	222.0	inches	Adj. Cross Sectional Area =	18.0	inches ²
H _f =	130.8	inches	Adj. Volume of Water =	4584	inches ³
ΔH =	91.2	inches	$I_t = \frac{\text{Volume of Water Dissipated}}{\text{Surface Area of Sidewall}} = \frac{\Delta H \pi r^2 (60)}{\Delta t (\pi r^2 + 2 \pi r H_{avg})} = \frac{\Delta H (60r)}{\Delta t (r + 2 H_{avg})}$		
H _{avg} =	176.4	inches			
			Infiltration Rate, I _t =	6.13	inches/hour

GEOCON
WEST, INC.



ENVIRONMENTAL GEOTECHNICAL MATERIALS
2807 MCGAW AVENUE - IRVINE, CALIFORNIA, 92614
PHONE: 949-491-6570

PERCOLATION TEST RESULTS AND CALCULATIONS

Chaffey College - In-Tech Welding Center
9400 Cherry Avenue
Fontana, CA

DRAFTED BY: CT

CHECKED BY: NDB

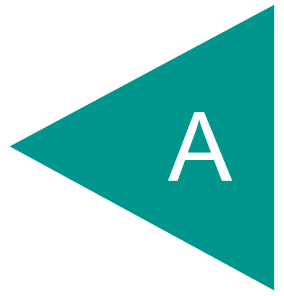
DEC 2024

PROJECT NO. W1145-99-10

FIG. 12

APPENDIX

A



APPENDIX A

FIELD INVESTIGATION

The site was explored on September 17, 2024, by excavating three 8-inch-diameter borings using a truck-mounted hollow-stem auger drilling machine to depths of 25½ feet beneath the ground surface. Representative and relatively undisturbed samples were obtained by driving a 3-inch, O. D., California Modified Sampler into the “undisturbed” soil mass with blows from a 140-pound 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 A3. 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 approximate locations of the borings are shown on Figure 2.



SOIL BORING NUMBER: B - 1

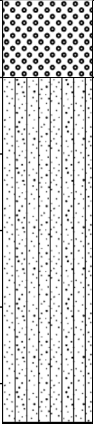

Page 1 of 2

PROJECT NAME	Chaffey Fontana – In-Tech Welding Center	LOGGED BY	CT
PROJECT NUMBER	W1145-99-10	LATITUDE / LONGITUDE	34.08213, -117.48990
DATE STARTED	09/17/2024	COMPLETED	09/17/2024
DEPTH	25.5'	SURFACE ELEVATION	N/A
LOCATIO	9400 Cherry Avenue, Fontana, CA		
DRILLING FIRM	2R Drilling	RIG TYPE	CME-75
METHOD	Hollow Stem Auger	BORING DIAMETER	8 in
		HAMMER TYPE	Auto
		HAMMER WEIGHT / DROP	- / -

Depth (ft)	Graphic Log	USCS	Water Levels	Material Description	Bulk Driven	Sample Number	Blow Counts/6"	Moisture Content (%)	Dry Density (pcf)
		SM		ALLUVIUM (GRASS, ORGANICS) Dense, damp, grayish brown, Silty SAND , fine to medium grained, with some sand, fine to coarse grained, some gravel; trace cobbles					
				Light brown, with trace sand, coarse grained, trace gravel fine to coarse grained		B1@2.5'	25 32 45	1.30	132.80
5				Brown; increase gravel, fine to coarse grained		B1@5'	40 50/4"	1.60	131.40
		SP		Very dense, moist, olive brown, Poorly Graded SAND , fine grained, with gravel, fine to coarse grained, trace sand fine to medium grained; oxidation staining		B1@7.5'	27 45 50	1.80	130.10
10				Increase in gravel, fine to coarse grained; trace cobbles		B1@10'	49 50/5"	3.00	112.70
				Fine to medium grained, with trace gravel, fine to coarse grained		B1@12.5'	30 50/5"	1.90	132.50
15						B1@15'	50/5"	4.90	128.30
		SM		Dense, moist, olive brown, Silty SAND , fine grained, with trace gravel, fine grained; trace cobbles					

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. THE STRATIFICATION LINES PRESENTED HEREIN REPRESENT THE APPROXIMATE BOUNDARY BETWEEN EARTH TYPES; THE TRANSITIONS MAY BE GRADUAL.

SOIL BORING: B-1

Depth (ft)	Graphic Log	USCS	Water Levels	Material Description	Bulk Driven	Sample Number	Blow Counts/6"	Moisture Content (%)	Dry Density (pcf)
		SW SM		Dense, damp, brown, Well-graded SAND , fine to coarse grained, with trace gravel, fine to coarse grained; trace cobbles Dense, damp, brown, Silty SAND , fine grained, weakly cemented; trace cobbles		B1@20'	30 50/5"	1.70	17.90
25				Fine to medium grained, with trace gravel, fine grained		B1@25'	32 50/5"	4.50	109.50
30				Total depth of boring: 25.5 feet No fill. No groundwater encountered. Backfilled with soil cuttings and tamped.					
35									

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. THE STRATIFICATION LINES PRESENTED HEREIN REPRESENT THE APPROXIMATE BOUNDARY BETWEEN EARTH TYPES; THE TRANSITIONS MAY BE GRADUAL.



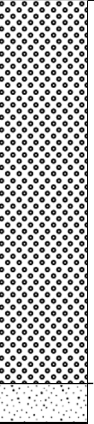

SOIL BORING NUMBER: B - 2

Page 1 of 2

PROJECT NAME	Chaffey Fontana – In-Tech Welding Center	LOGGED BY	CT
PROJECT NUMBER	W1145-99-10	LATITUDE / LONGITUDE	34.08213, -117.48957
DATE STARTED	09/18/2024	COMPLETED	09/18/2024
DEPTH	25.5'	SURFACE ELEVATION	1099.2'
LOCATION	9400 Cherry Avenue, Fontana, CA		
DRILLING FIRM	2R Drilling	RIG TYPE	CME-75
METHOD	Hollow Stem Auger	BORING DIAMETER	8 in
		HAMMER TYPE	Auto
		HAMMER WEIGHT / DROP	- / -

Depth (ft)	Graphic Log	USCS	Water Levels	Material Description	Bulk Driven	Sample Number	Blow Counts/6"	Moisture Content (%)	Dry Density (pcf)
		ML		ALLUVIUM (GRASS) Stiff, damp, light brown, SILT , fine grained; trace organics					
						B2@2.5'	10 16 22	1.50	113.10
5		SM		Dense, damp, light brown, Silty SAND , fine to medium grained, with trace gravel, fine to coarse grained		B2@5'	12 24 50/5"	1.00	119.50
				Trace cobbles		B2@7.5'	15	0.60	133.90
10				No cobbles Increase in gravel, fine to coarse grained		B2@10'	48 50/5"	1.10	117.90
				With Trace sand, coarse grained		B2@12.5'	40 50/5"	1.80	114.80
15				Very dense, moist, brown, fine grained		B2@15'	21 14 32	5.70	108.40
		SW		Very dense, moist, brown, Well-graded SAND , fine to coarse grained, with trace gravel, fine to coarse grained					

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. THE STRATIFICATION LINES PRESENTED HEREIN REPRESENT THE APPROXIMATE BOUNDARY BETWEEN EARTH TYPES; THE TRANSITIONS MAY BE GRADUAL.

Depth (ft)	Graphic Log	USCS	Water Levels	Material Description	Bulk Driven	Sample Number	Blow Counts/6"	Moisture Content (%)	Dry Density (pcf)
		SW		No recovery Rig chatter		B2@20'	50/5"		
25		SP		Dense, moist, Poorly Graded SAND , fine to medium grained, with trace sand, coarse grained, trace gravel coarse grained; trace cobbles Total depth of boring: 25.5 feet No fill. No groundwater encountered. Backfilled with soil cuttings and tamped.		B2@25'	50/5"	2.70	118.00
30									
35									

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. THE STRATIFICATION LINES PRESENTED HEREIN REPRESENT THE APPROXIMATE BOUNDARY BETWEEN EARTH TYPES; THE TRANSITIONS MAY BE GRADUAL.



SOIL BORING NUMBER: B - 3

Page 1 of 2

PROJECT NAME	Chaffey Fontana – In-Tech Welding Center	LOGGED BY	CT
PROJECT NUMBER	W1145-99-10	LATITUDE / LONGITUDE	34.08179, -117.48957
DATE STARTED	09/18/2024	COMPLETED	09/18/2024
DEPTH	25.5'	SURFACE ELEVATION	1099.2'
LOCATION	9400 Cherry Avenue, Fontana, CA		
DRILLING FIRM	2R Drilling	RIG TYPE	CME-75
METHOD	Hollow Stem Auger	BORING DIAMETER	8 in
		HAMMER TYPE	Auto
		HAMMER WEIGHT / DROP	- / -

Depth (ft)	Graphic Log	USCS	Water Levels	Material Description	Bulk Driven	Sample Number	Blow Counts/6"	Moisture Content (%)	Dry Density (pcf)
		SM		ALLUVIUM (GRASS) Medium dense, moist, brown, Silty SAND , fine grained, with trace gravel, fine grained					
		SP		Medium dense, damp, brown, Poorly Graded SAND , fine to medium grained, with trace sand, coarse grained, trace gravel fine to coarse grained		B3@2.5'	11 10 11	8.50	112.10
5						B3@5'	15 20 24	2.60	123.80
				Fine grained, with trace sand, medium grained Fine to coarse grained		B3@7.5'	17 15 23	4.30	119.30
10						B3@10'	15 27 37	2.10	128.60
		SM		With Some gravel, fine to coarse grained; trace cobbles Very dense, damp, gray, Silty SAND , fine to coarse grained, with trace gravel, fine grained		B3@12.5'	40 50/4"	3.50	131.70
15						B3@15'	40 50/5"	2.20	115.40
				Rig chatter					

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. THE STRATIFICATION LINES PRESENTED HEREIN REPRESENT THE APPROXIMATE BOUNDARY BETWEEN EARTH TYPES; THE TRANSITIONS MAY BE GRADUAL.

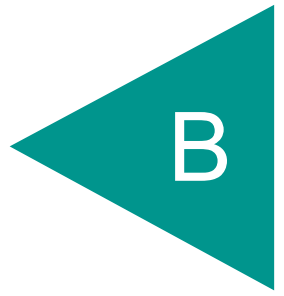
SOIL BORING: B-3

Depth (ft)	Graphic Log	USCS	Water Levels	Material Description	Bulk Driven	Sample Number	Blow Counts/6"	Moisture Content (%)	Dry Density (pcf)
		SM		Brown, with gravel, fine to coarse grained		B3@20'	50/5"	1.80	108.10
				Dense, fine to coarse grained, with some gravel, fine grained					
25		SW		Dense, damp, brown, Well-graded SAND , fine to coarse grained, with trace gravel, fine to coarse grained		B3@25'	50/5"	3.00	111.90
				Total depth of boring: 25.5 feet No fill. No groundwater encountered. Backfilled with soil cuttings and tamped.					
30									
35									

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. THE STRATIFICATION LINES PRESENTED HEREIN REPRESENT THE APPROXIMATE BOUNDARY BETWEEN EARTH TYPES; THE TRANSITIONS MAY BE GRADUAL.

APPENDIX

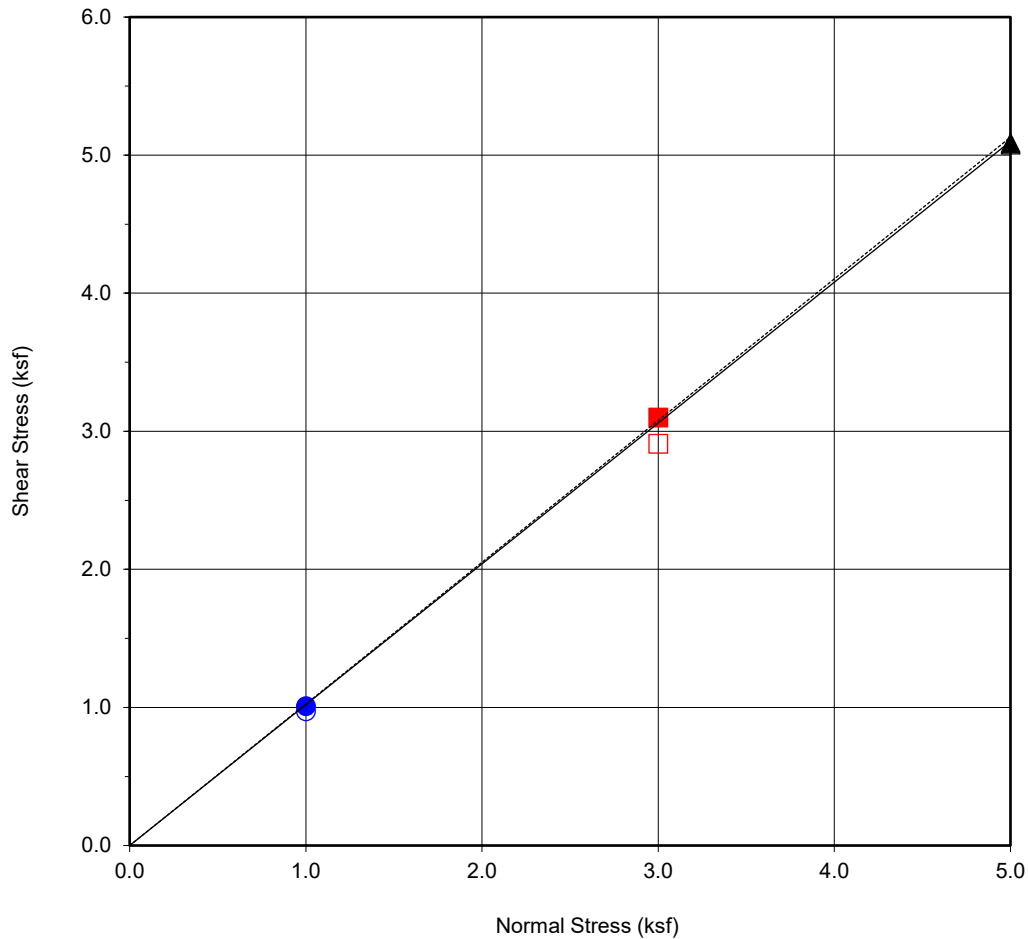
B



APPENDIX B

LABORATORY TESTING

We performed laboratory tests in accordance with generally accepted test methods of the American Society for Testing and Materials (ASTM) or other suggested procedures. We tested selected soil samples for in-place dry density/moisture content, expansion index, water-soluble sulfate, pH, resistivity, water-soluble chloride ion content, consolidation, and direct shear strength. The in-place dry density and moisture content of the samples tested are presented on the boring logs, Appendix A.



Boring No.	B3
Sample No.	B3@5'
Depth (ft)	5'
<u>Sample Type:</u>	RING

<u>Soil Identification:</u>		
Sand, poorly graded (SP)		
<u>Strength Parameters</u>		
	C (psf)	ϕ ($^{\circ}$)
Peak	0	46
Ultimate	0	46

Normal Stress (kip/ft ²)	1	3	5
Peak Shear Stress (kip/ft ²)	● 1.01	■ 3.10	▲ 5.09
Shear Stress @ End of Test (ksf)	○ 0.97	□ 2.91	△ 5.08
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 (%)	6.1	7.5	7.6
Initial Dry Density (pcf)	102.6	111.5	105.6
Initial Degree of Saturation (%)	25.8	39.4	34.6
Soil Height Before Shearing (in.)	1.2	1.2	1.2
Final Moisture Content (%)	14.1	13.3	13.9



DIRECT SHEAR TEST RESULTS

Consolidated Drained ASTM D-3080

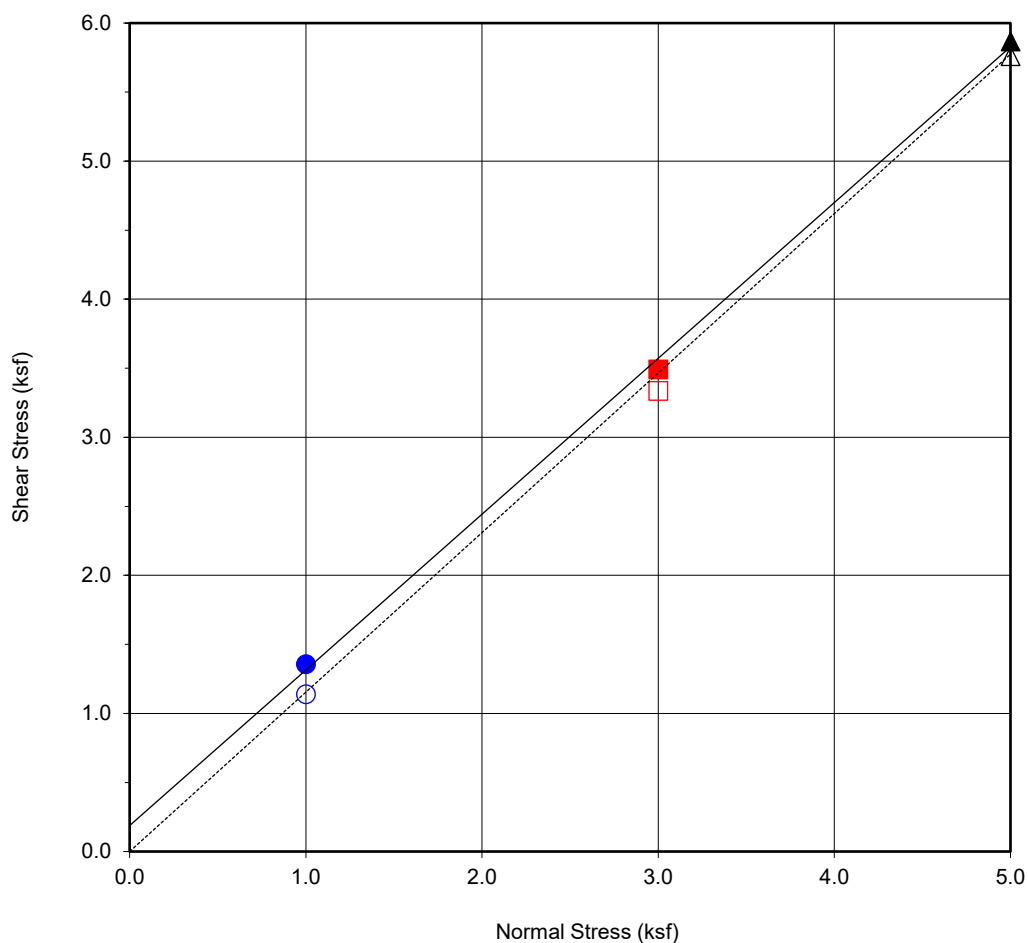
Checked by: CT

Project No.: W1145-99-10

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9400 Cherry Avenue
Fontana, CA

DEC 2024

Figure B1



Boring No.	B2
Sample No.	B2@7.5'
Depth (ft)	7.5'
<u>Sample Type:</u>	RING

<u>Soil Identification:</u>		
Silty Sand trace cobbles (SM)		
Strength Parameters		
	C (psf)	ϕ ($^{\circ}$)
Peak	188	48
Ultimate	0	49

Normal Stress (kip/ft ²)	1	3	5
Peak Shear Stress (kip/ft ²)	● 1.36	■ 3.49	▲ 5.87
Shear Stress @ End of Test (ksf)	○ 1.14	□ 3.34	△ 5.76
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 (%)	9.0	3.6	8.2
Initial Dry Density (pcf)	114.9	118.4	123.2
Initial Degree of Saturation (%)	52.3	22.9	60.3
Soil Height Before Shearing (in.)	1.2	1.2	1.2
Final Moisture Content (%)	13.0	12.9	10.6



DIRECT SHEAR TEST RESULTS

Consolidated Drained ASTM D-3080

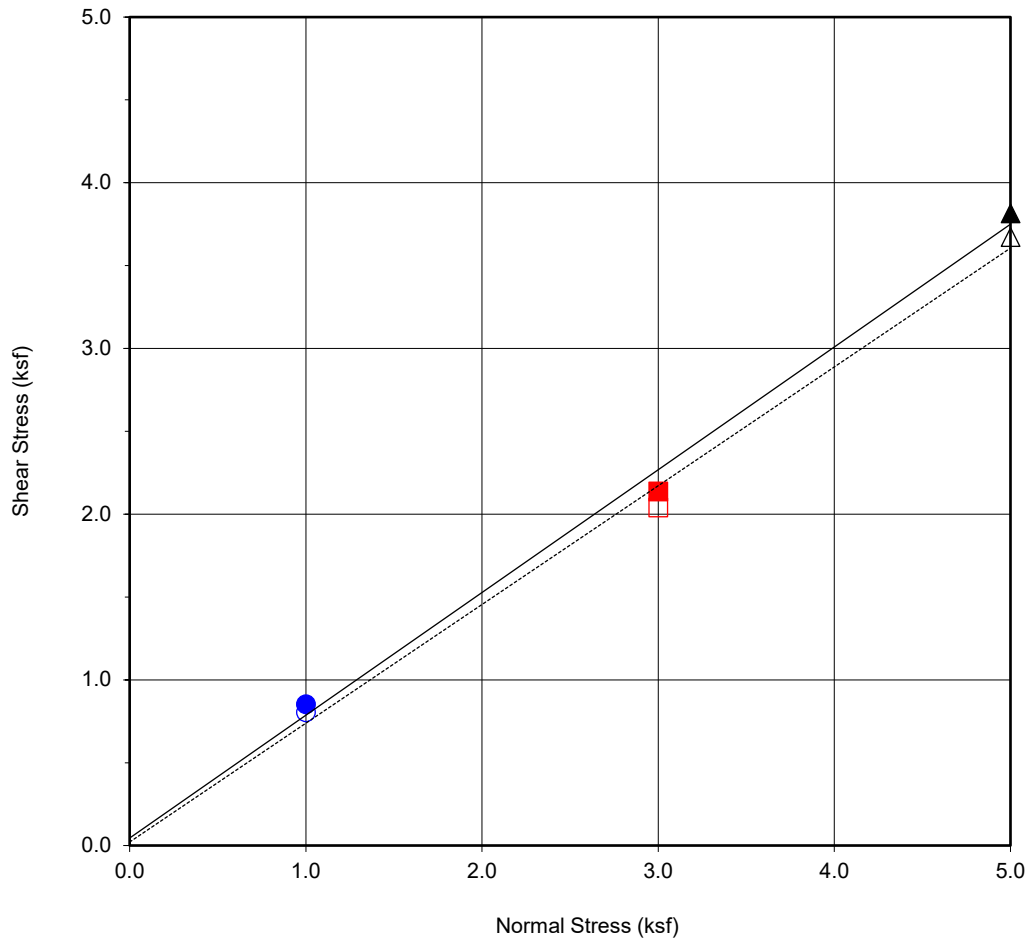
Checked by: CT

Project No.: W1145-99-10

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Figure B2



Boring No.	B1+B2
Sample No.	B1+B2@0-5'
Depth (ft)	0-5'
<u>Sample Type:</u>	REMOLD

<u>Soil Identification:</u>		
Silty Sand (SM) & Silt (ML)		
<u>Strength Parameters</u>		
	C (psf)	ϕ ($^{\circ}$)
Peak	45	37
Ultimate	21	36

Normal Stress (kip/ft ²)	1	3	5
Peak Shear Stress (kip/ft ²)	● 0.85	■ 2.14	▲ 3.82
Shear Stress @ End of Test (ksf)	○ 0.80	□ 2.04	△ 3.67
Deformation Rate (in./min.)	0.05	0.05	0.05
Initial Sample Height (in.)	1.0	1.0	1.0
Ring Inside Diameter (in.)	2.375	2.375	2.375
Initial Moisture Content (%)	11.5	13.5	14.1
Initial Dry Density (pcf)	108.2	106.5	105.7
Initial Degree of Saturation (%)	55.6	62.4	64.3
Soil Height Before Shearing (in.)	1.2	1.2	1.2
Final Moisture Content (%)	17.6	16.9	14.6



DIRECT SHEAR TEST RESULTS

Consolidated Drained ASTM D-3080

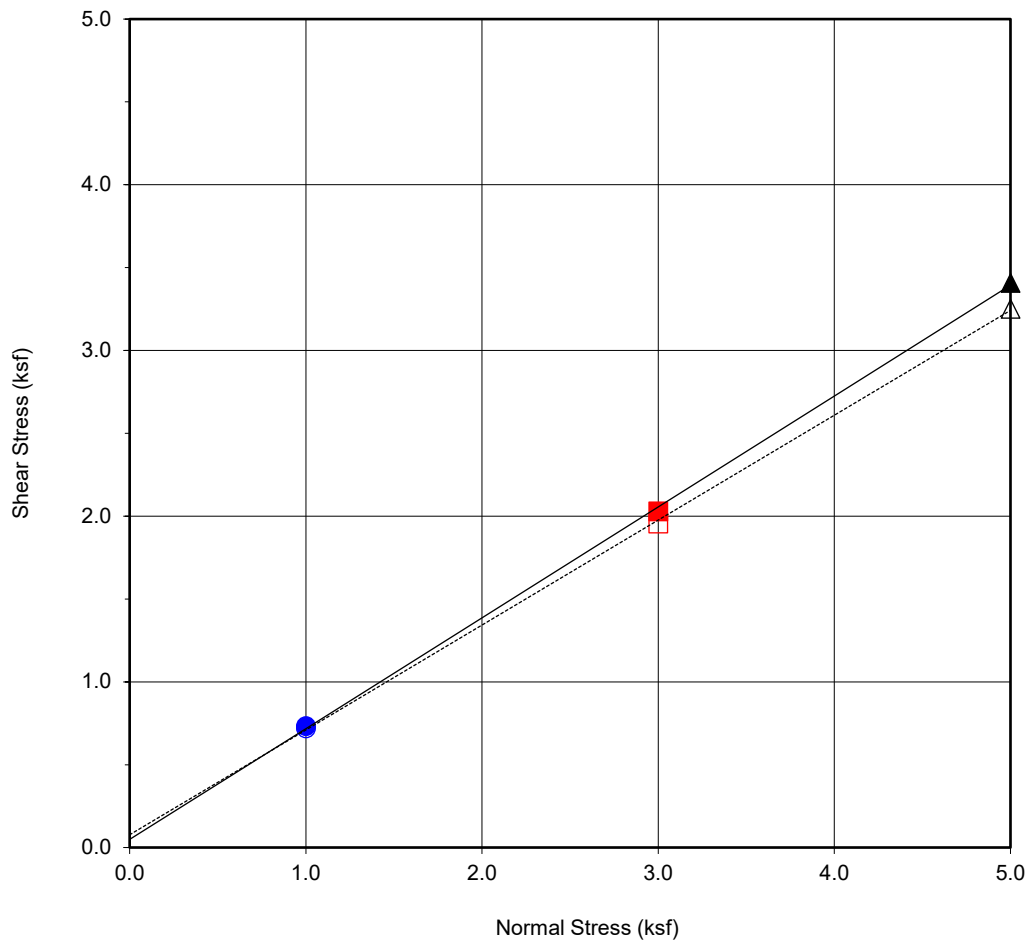
Checked by: CT

Project No.: W1145-99-10

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Fontana, CA

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Figure B3



Boring No.	B3
Sample No.	B3@0-5'
Depth (ft)	0-5'
<u>Sample Type:</u>	REMOLD

<u>Soil Identification:</u>		
Silty Sand (SM) & Sand (SP)		
<u>Strength Parameters</u>		
	C (psf)	ϕ ($^{\circ}$)
Peak	49	34
Ultimate	77	32

Normal Stress (kip/ft ²)	1	3	5
Peak Shear Stress (kip/ft ²)	● 0.73	■ 2.03	▲ 3.41
Shear Stress @ End of Test (ksf)	○ 0.72	□ 1.96	△ 3.25
Deformation Rate (in./min.)	0.05	0.05	0.05
Initial Sample Height (in.)	1.0	1.0	1.0
Ring Inside Diameter (in.)	2.375	2.375	2.375
Initial Moisture Content (%)	12.1	14.4	14.7
Initial Dry Density (pcf)	107.8	105.6	105.3
Initial Degree of Saturation (%)	57.8	65.3	66.2
Soil Height Before Shearing (in.)	1.2	1.2	1.2
Final Moisture Content (%)	16.9	16.4	16.3



DIRECT SHEAR TEST RESULTS

Consolidated Drained ASTM D-3080

Checked by: CT

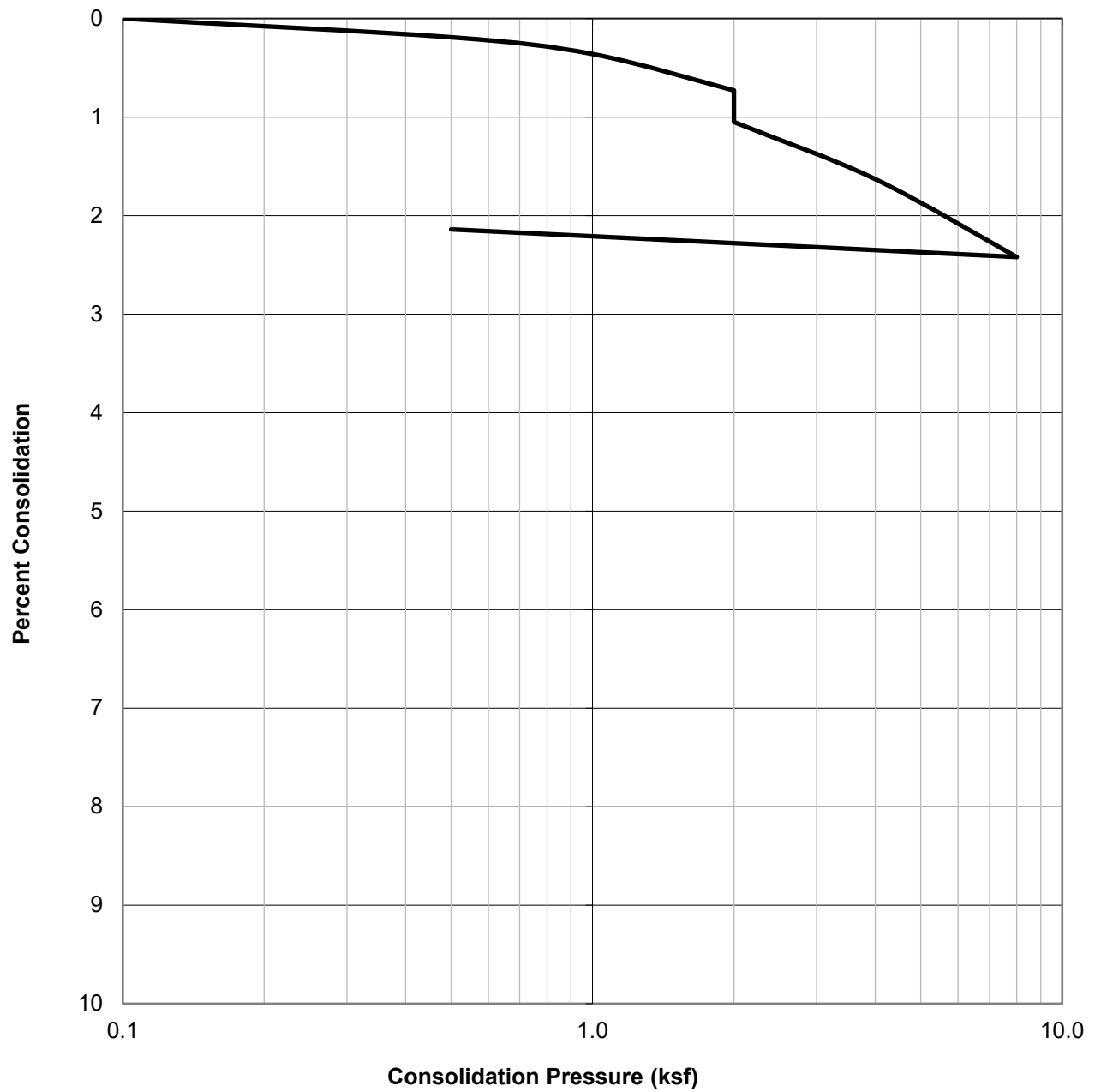
Project No.: W1145-99-10

Chaffey College - In-Tech Welding Center
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Fontana, CA

DEC 2024

Figure B4

WATER ADDED AT 2.0 KSF



SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B2@5'	Silty Sand (SM)	113.5	1.7	15.8



CONSOLIDATION TEST RESULTS

ASTM D-2435

Checked by: CT

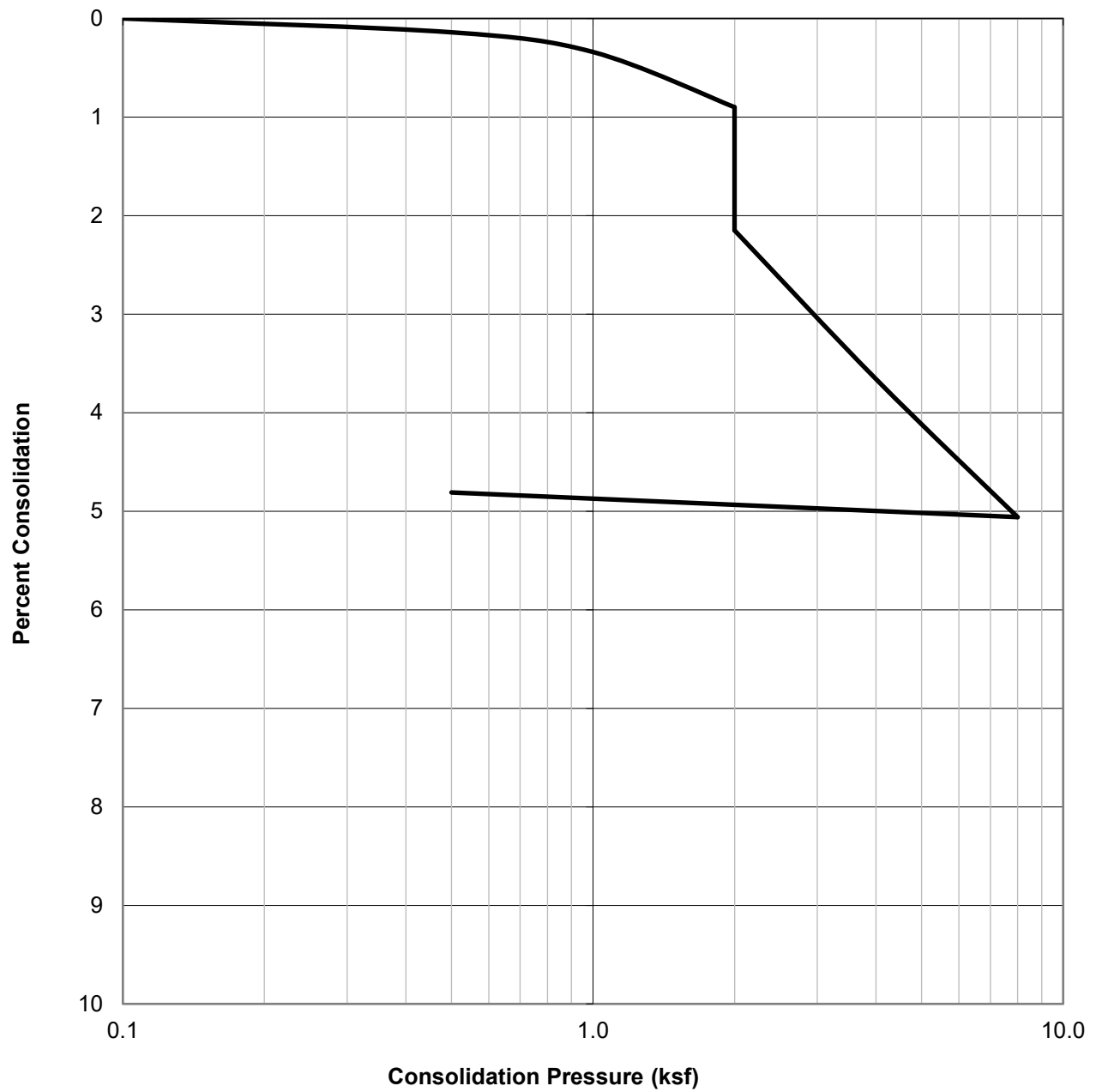
Project No.: W1145-99-10

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DEC 2024

Figure B5

WATER ADDED AT 2.0 KSF



SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B3@5'	Sand, poorly graded (SP)	108.5	3.9	17.9



CONSOLIDATION TEST RESULTS

ASTM D-2435

Checked by: CT

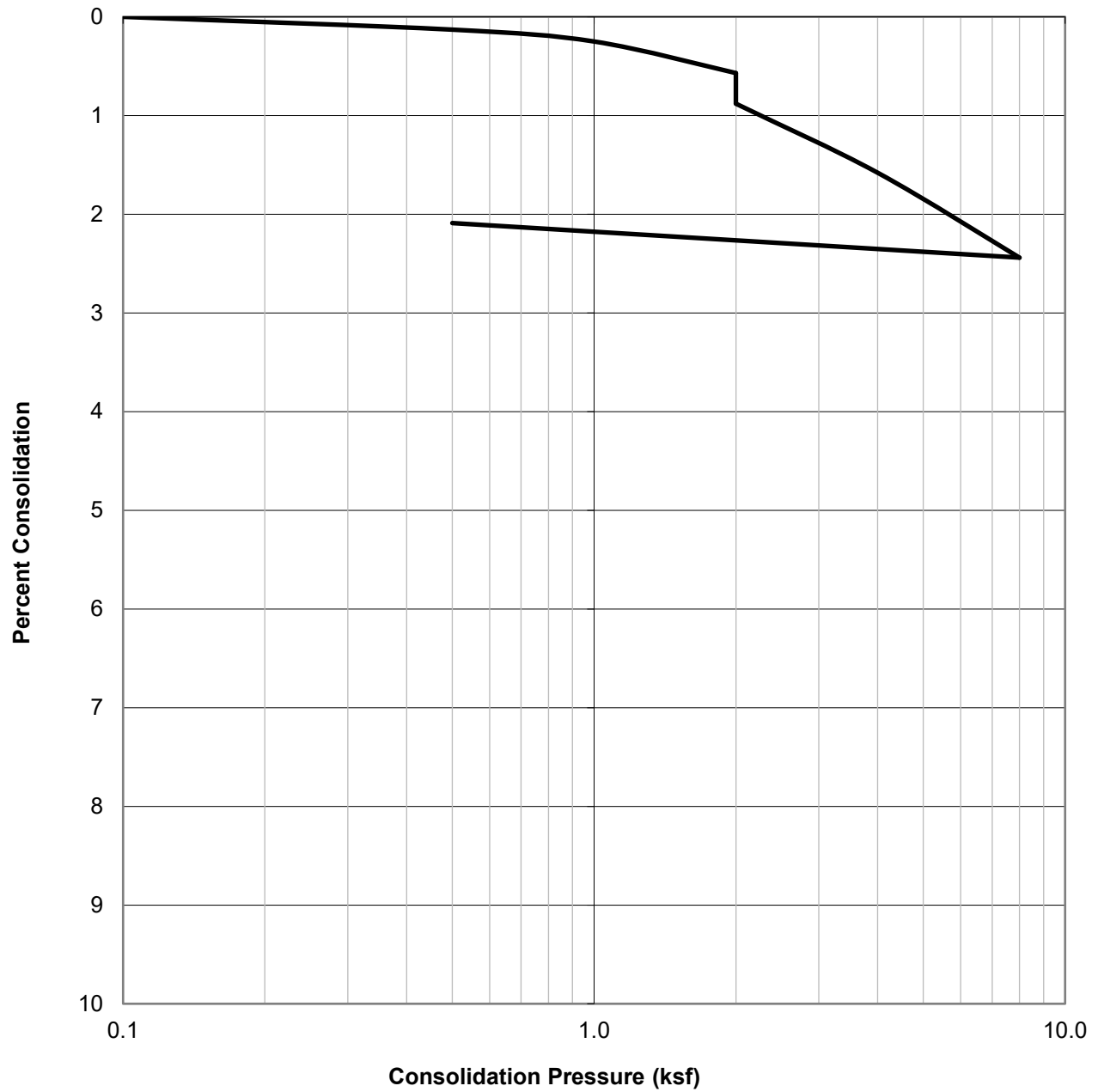
Project No.: W1145-99-10

Chaffey College - In-Tech Welding Center
9400 Cherry Avenue
Fontana, CA

DEC 2024

Figure B6

WATER ADDED AT 2.0 KSF



SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B1@7.5'	Sand, poorly graded (SP)	119.8	2.5	14.3



CONSOLIDATION TEST RESULTS

ASTM D-2435

Checked by: CT

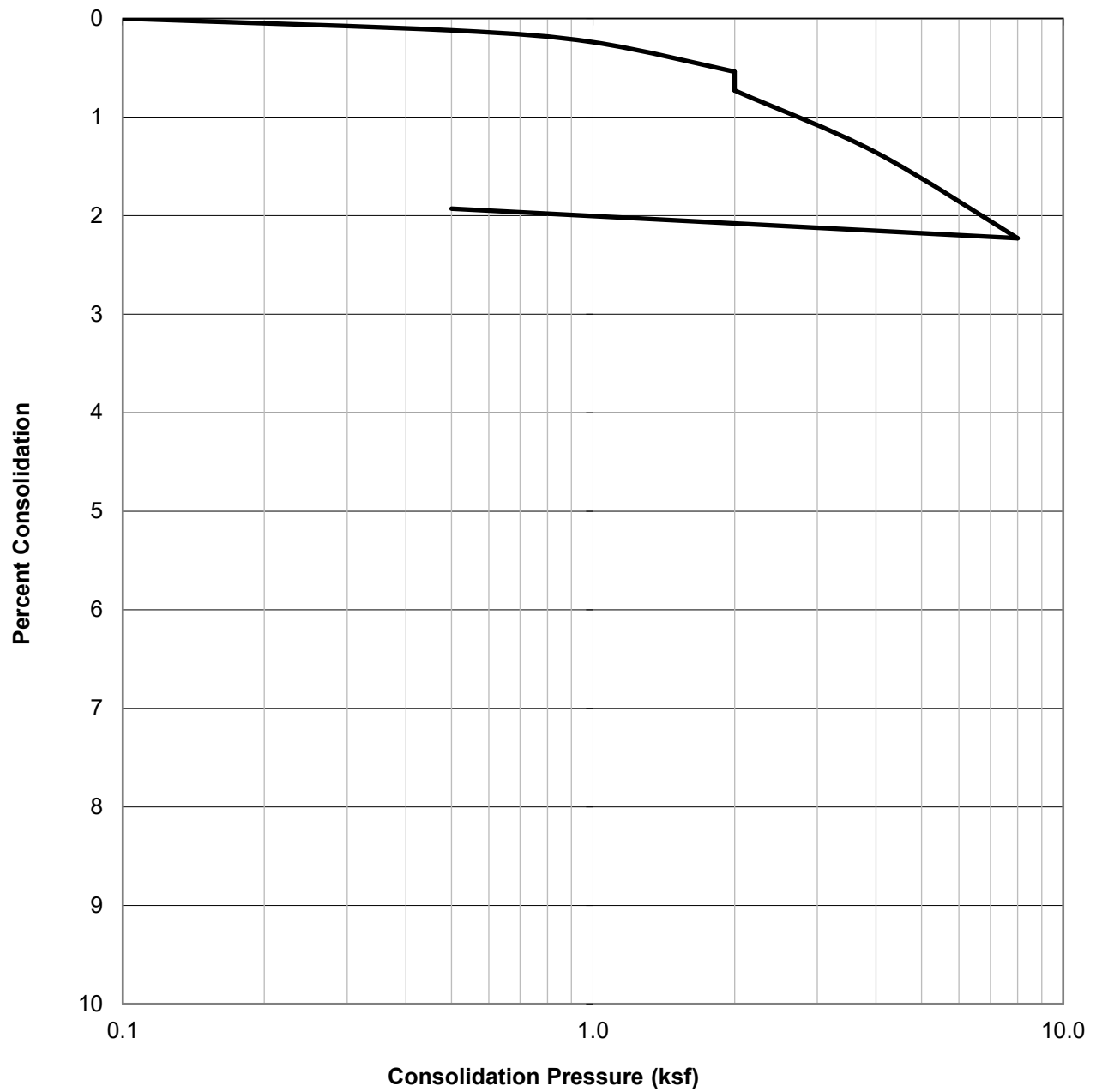
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Figure B7

WATER ADDED AT 2.0 KSF



SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B1@10'	Sand, poorly graded w/ gravel (SP)	110.7	4.3	15.8



CONSOLIDATION TEST RESULTS

ASTM D-2435

Checked by: CT

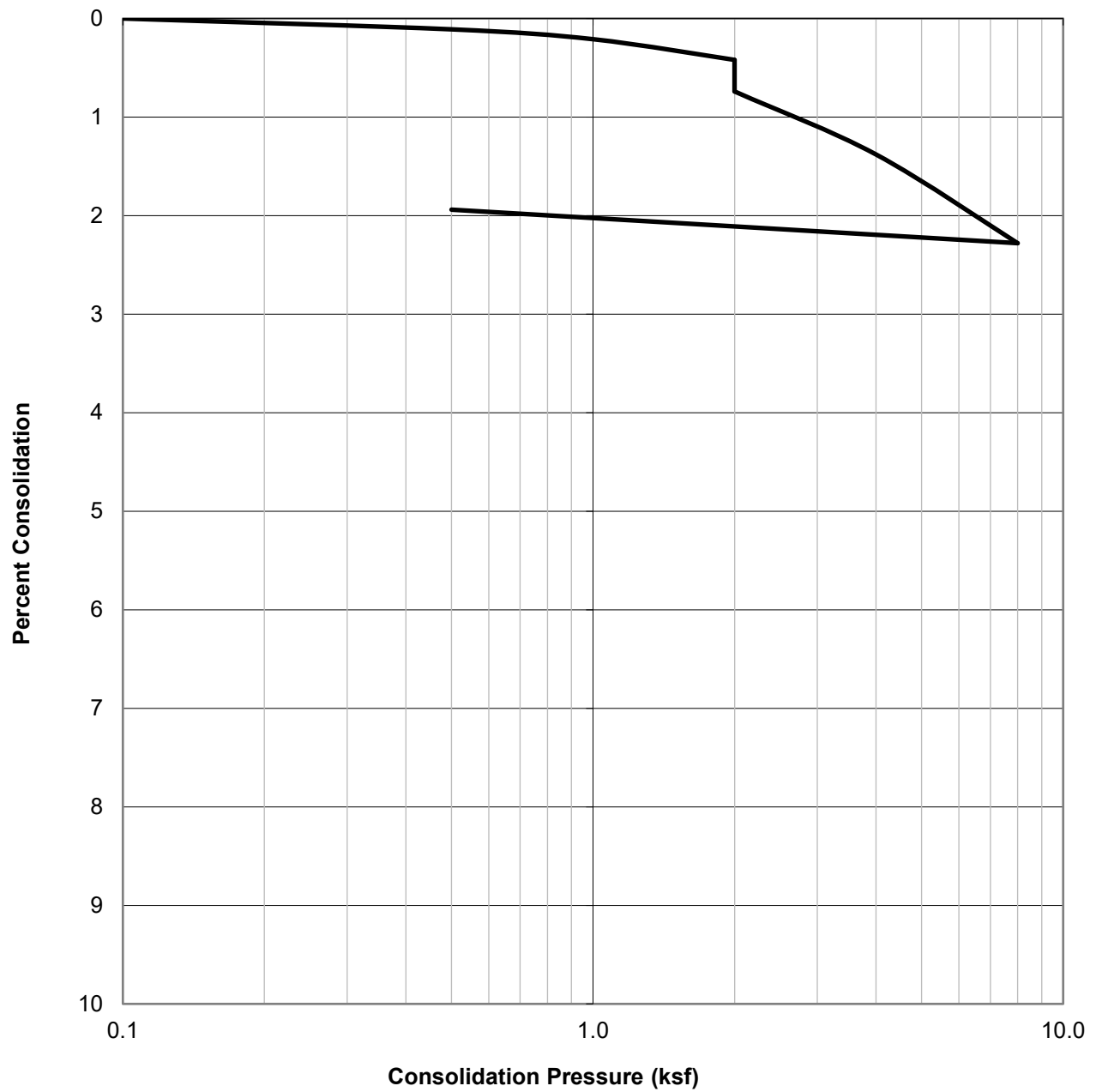
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Figure B8

WATER ADDED AT 2.0 KSF



SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B3@10'	Sand, poorly graded (SP)	117.4	4.3	13.1



CONSOLIDATION TEST RESULTS

ASTM D-2435

Checked by: CT

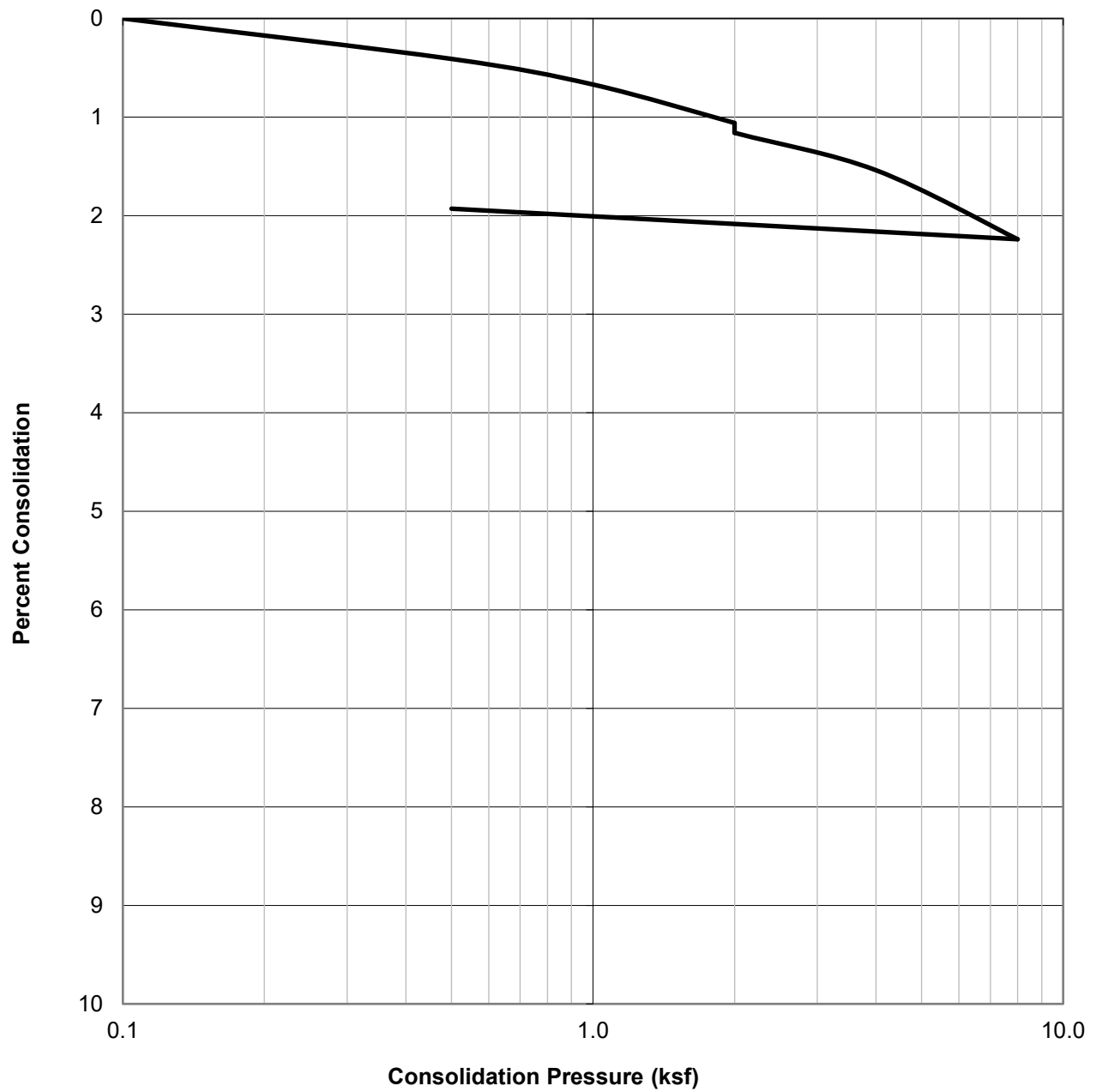
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Figure B9

WATER ADDED AT 2.0 KSF



SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B1@15'	Sand, poorly graded (SP)	123.8	11.3	13.2



CONSOLIDATION TEST RESULTS

ASTM D-2435

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Figure B10

Sample No:

B1+B2@0-5'

Silty Sand (SM) & Silt w/ Sand (ML)

TEST NO.		1	2	3	4	5	6
Wt. Compacted Soil + Mold	(g)	6208	6202	6270	6314		
Weight of Mold	(g)	4282	4282	4282	4282		
Net Weight of Soil	(g)	1926	1920	1988	2032		
Wet Weight of Soil + Cont.	(g)	2371.6	2298.8	2376.9	2393.9		
Dry Weight of Soil + Cont.	(g)	2167.3	2196.3	2237.9	2219.4		
Weight of Container	(g)	377.3	376.7	378.6	378.1		
Moisture Content	(%)	11.4	5.6	7.5	9.5		
Wet Density	(pcf)	127.5	127.1	131.6	134.5		
Dry Density	(pcf)	114.4	120.3	122.5	122.9		

Maximum Dry Density (pcf) 123.5

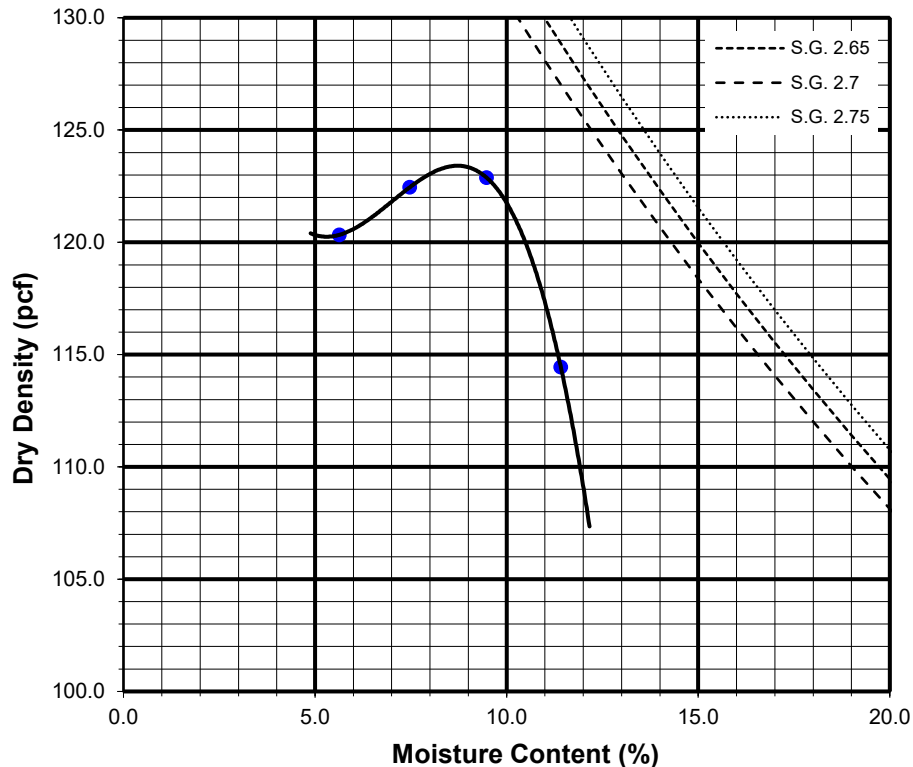
Bulk Specific Gravity (dry) 2.24

Corrected Maximum Dry Density (pcf) 127.9

Optimum Moisture Content (%) 8.5

Oversized Fraction (%) 29.2

Corrected Moisture Content (%) 6.0



Preparation Method: A



**COMPACTION CHARACTERISTICS USING
MODIFIED EFFORT TEST RESULTS**

ASTM D-1557

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Project No.: W1145-99-10

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Figure B11

Sample No:

B3@0-5'

Silty Sand (SM)

TEST NO.		1	2	3	4	5	6
Wt. Compacted Soil + Mold	(g)	6316	6296	6254	6174		
Weight of Mold	(g)	4282	4282	4282	4282		
Net Weight of Soil	(g)	2034	2014	1972	1892		
Wet Weight of Soil + Cont.	(g)	2422.1	2398.6	2352.4	2273.9		
Dry Weight of Soil + Cont.	(g)	2242.9	2186.9	2214.8	2176.7		
Weight of Container	(g)	378.5	377.2	377.9	378.2		
Moisture Content	(%)	9.6	11.7	7.5	5.4		
Wet Density	(pcf)	134.7	133.3	130.6	125.3		
Dry Density	(pcf)	122.9	119.4	121.5	118.8		

Maximum Dry Density (pcf) 123.0

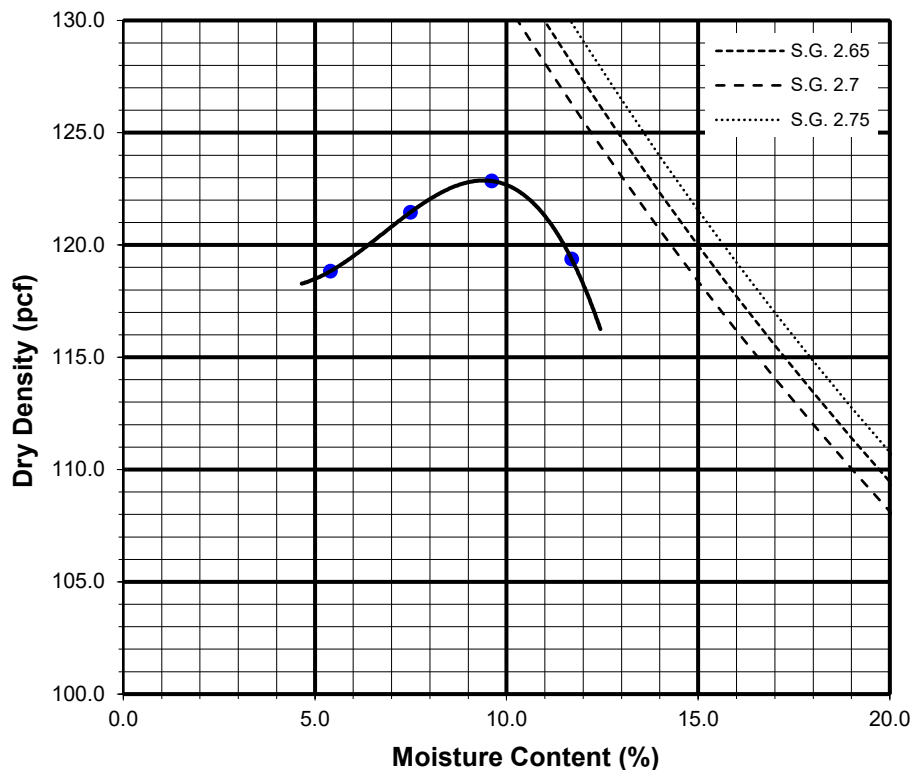
Bulk Specific Gravity (dry) 2.15

Corrected Maximum Dry Density (pcf) 125.2

Optimum Moisture Content (%) 9.0

Oversized Fraction (%) 20.7

Corrected Moisture Content (%) 7.1



Preparation Method: A



**COMPACTION CHARACTERISTICS USING
MODIFIED EFFORT TEST RESULTS**

ASTM D-1557

Checked by: CT

Project No.: W1145-99-10

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Figure B12

B1+B2@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)	618.4	634.6
Wt. of Mold	(gm)	200.8	200.8
Specific Gravity	(Assumed)	2.7	2.7
Wet Wt. of Soil + Cont.	(gm)	679.0	634.6
Dry Wt. of Soil + Cont.	(gm)	655.5	384.9
Wt. of Container	(gm)	379.0	200.8
Moisture Content	(%)	8.5	12.7
Wet Density	(pcf)	126.0	130.7
Dry Density	(pcf)	116.1	115.9
Void Ratio		0.5	0.4
Total Porosity		0.3	0.3
Pore Volume	(cc)	64.4	64.1
Degree of Saturation	(%) [S_{meas}]	51.2	76.3

Date	Time	Pressure (psi)	Elapsed Time (min)	Dial Readings (in.)
9/25/2024	10:00	1.0	0	0.4541
9/25/2024	10:10	1.0	10	0.4535
Add Distilled Water to the Specimen				
9/26/2024	10:00	1.0	1430	0.452
9/26/2024	11:00	1.0	1490	0.452

Expansion Index (EI meas) =	-1.5
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: 2022 California Building Code, Section 1803.5.3

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

**EXPANSION INDEX TEST RESULTS**

ASTM D-4829

Checked by: CT

Project No.: W1145-99-10

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Figure B13

B3@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)	620.0	628.6
Wt. of Mold	(gm)	201.7	201.7
Specific Gravity	(Assumed)	2.7	2.7
Wet Wt. of Soil + Cont.	(gm)	711.7	628.6
Dry Wt. of Soil + Cont.	(gm)	689.5	387.3
Wt. of Container	(gm)	411.7	201.7
Moisture Content	(%)	8.0	10.2
Wet Density	(pcf)	126.2	128.6
Dry Density	(pcf)	116.8	116.7
Void Ratio		0.4	0.4
Total Porosity		0.3	0.3
Pore Volume	(cc)	63.5	63.6
Degree of Saturation	(%) [S_{meas}]	49.2	62.3

Date	Time	Pressure (psi)	Elapsed Time (min)	Dial Readings (in.)
9/25/2024	10:00	1.0	0	0.453
9/25/2024	10:10	1.0	10	0.4524
Add Distilled Water to the Specimen				
9/26/2024	10:00	1.0	1430	0.4526
9/26/2024	11:00	1.0	1490	0.4526

Expansion Index (EI meas) =	0.2
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: 2022 California Building Code, Section 1803.5.3

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

**EXPANSION INDEX TEST RESULTS**

ASTM D-4829

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Figure B14

SUMMARY OF LABORATORY
POTENTIAL OF HYDROGEN (pH) AND RESISTIVITY TEST RESULTS
AASHTO T289 ASTM D4972 and AASHTO T288 ASTM G187


Sample No.	pH	Resistivity (ohm centimeters)
B1+B2@0-5'	8.3	12000 (Mildly Corrosive)
B3@0-5'	8.4	14000 (Mildly Corrosive)

SUMMARY OF LABORATORY CHLORIDE CONTENT TEST RESULTS
AASHTO T291 ASTM C1218

Sample No.	Chloride Ion Content (%)
B1+B2@0-5'	0.008
B3@0-5'	0.005

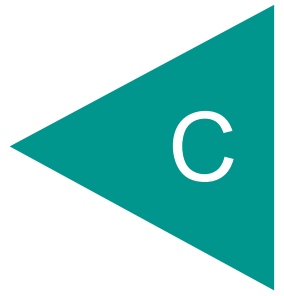
SUMMARY OF LABORATORY WATER SOLUBLE SULFATE TEST RESULTS
AASHTO T290 ASTM C1580

Sample No.	Water Soluble Sulfate (% SO ₄)	Sulfate Exposure
B1+B2@0-5'	0.002	S0
B3@0-5'	0.001	S0

 GEOCON	CORROSIVITY TEST RESULTS	Project No.: W1145-99-10
		Chaffey College - In-Tech Welding Center 9400 Cherry Avenue Fontana, CA
	Checked by: CT	DEC 2024 Figure B15

APPENDIX

C



APPENDIX C

S-WAVE SEISMIC SURVEY REPORT

APPENDIX C

S-WAVE SEISMIC SURVEY REPORT

INTRODUCTION

We performed a 1-D refraction microtremor (ReMi) seismic survey to help with site class determination and design efforts for construction at the project site located at 9400 Cherry Avenue in Fontana, California (**Figure 1**). The survey involved conducting two seismic traverses (SL-1 and SL-2) at designated areas within the project site, as shown in (**Figures 2 and 3**).



Figure 1: Vicinity Map

METHODOLOGY

S-Wave Refraction Microtremor (ReMi)

The ReMi method uses surface waves (Rayleigh waves) contained in background noise (such as ambient noise generated by nearby vehicle traffic and construction) to produce a shear (S)-wave velocity profile of the subsurface geologic conditions. Like the P-wave refraction method, the ReMi method uses a seismograph and vertical component geophones, which makes it very convenient to collect ReMi data along the same line where P-wave refraction data is collected. The effective depth of investigation for ReMi is related to the length of the geophone array and the frequency response of the measurement system. Multiple records per ReMi line record unfiltered data for each ReMi line and are then downloaded to a field computer. An S-wave dispersion curve is derived from the data and used to model subsurface 1-D S-wave velocity at depth. Unlike P-wave refraction, the ReMi method is capable of detecting velocity inversions (lower velocity layers underlying higher velocity layers). In addition, the ReMi method is not as sensitive to the presence of a water table as P-wave refraction method.

24 Vertical-component 4.5-Hertz geophones were positioned approximately 10 feet apart along lines SL-1 and SL-2 to capture the ReMi data. Several shots (signal generation points) were also performed by striking a high-density polyethylene (HDPE) plate with a 16-pound hammer at the ends of each line during ReMi data collection to help capture ambient seismic noise for S-wave analysis. ReMi data was recorded using a sample interval of 2 microseconds and a record length of 30 seconds. The ReMi data collected were analyzed by Geocon and the 1-D S-wave velocity model was producing using Terēan's 2ds and Disper software.

During processing, the software generated phase-velocity dispersion curves for each record, with an interactive dispersion modeling tool allowing us to refine and select the best-fitting model for the subsurface conditions. This approach provided detailed and accurate S-wave velocity profiles, enhancing our understanding of subsurface geologic layers, including any velocity inversions. The final models yield a robust interpretation of site conditions, crucial for site classification and other geotechnical assessments. The result is a 1-D shear-wave velocity model of the site which, based on published studies, is typically 85 to 95 percent of the velocity of shear waves, and results in a relatively conservative estimate of shear wave velocity using the ReMi surface wave data and analysis method.

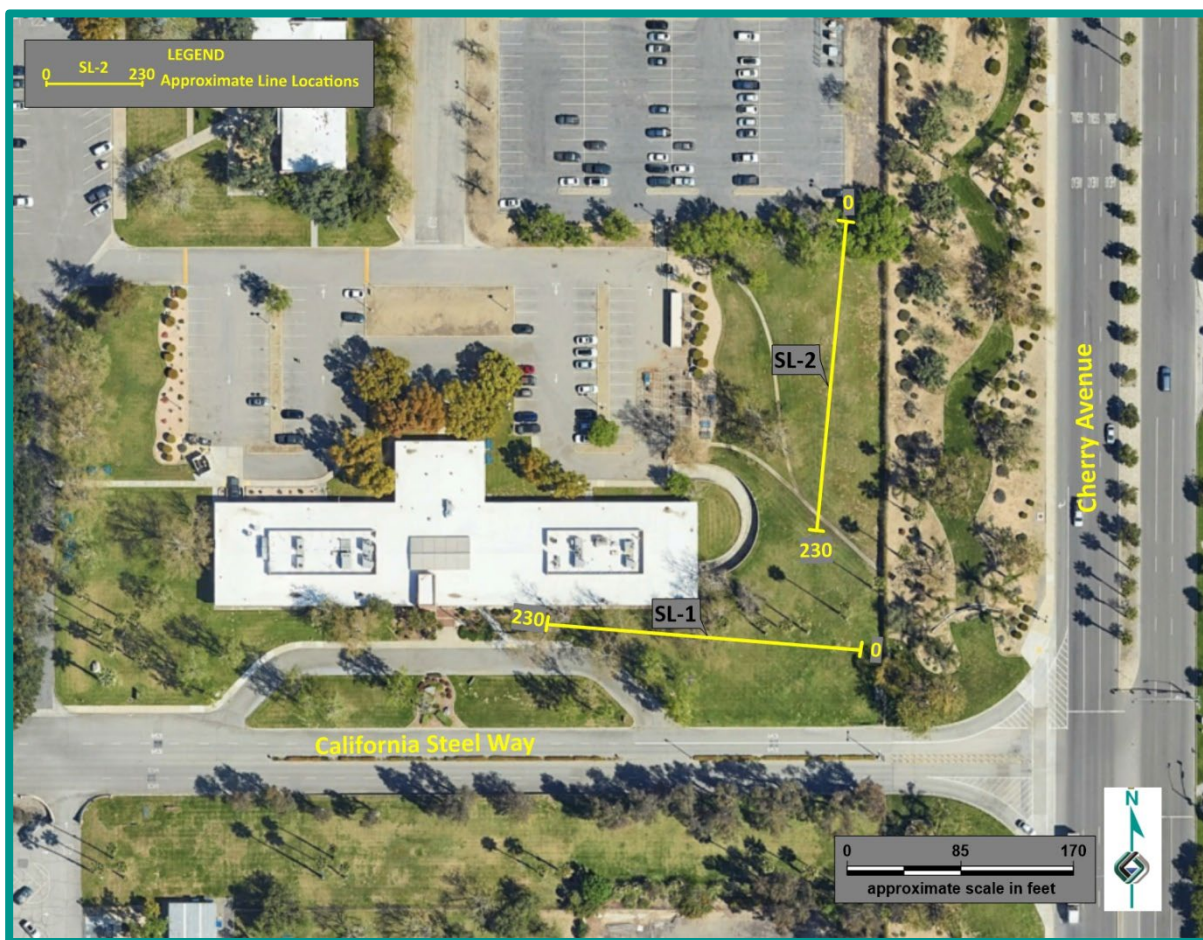


Figure 2: Line Location Map



Figure 3: Site Photographs

RESULTS

The ReMi 1-D S-wave velocity models for seismic lines SL-1 and SL-2 are shown in **Figures 4 and 5**. The Vs100 values calculated from the ReMi 1-D S-wave velocity models for lines SL-1 and SL-2 are provided in **Table 1**. The calculated **Vs values** for SL-1 and SL-2 lines were **1,167 ft/s, and 1,160 ft/s** respectively, classifying the site as **Site Class D** according to National Earthquake Hazards Reduction Program (NEHRP) seismic site classification standards and the 2021 International Building Code (IBC, 2021). It should be noted the ReMi results represent the average condition across the length of the line. When the 1-D ReMi surface wave velocity results (analogous to shear wave) show an IBC Vs100 velocity value that is close to the "border line" boundary between IBC Site Classes, the geotechnical data should be closely reviewed and the geotechnical engineer of record should also consider other existing available site information and whether obtaining additional new geotechnical evaluation data such as boreholes, the surface to downhole seismic (ASTM D7400), cross-hole seismic (ASTM D4428), and/or additional 1-D ReMi data collections would be needed concerning the site's subsurface geologic stratigraphy and structure, soil mechanics and soil modulus, along with the initial 1-D ReMi evaluation results when assessing the "borderline" IBC Vs100 Seismic Site Class.

TABLE 1: SUMMARY OF S-WAVE REMI RESULTS

Line No.	Depth (feet)	Shear Wave Velocity (feet/second)	Site Class	Vs 100 (ft/s)
SL-1 (E-W)	0-7.4	608	D	1,167
	7.4-48.3	1,201		
	48.3-100	1,310		
SL-2 (N-S)	0-7.4	792	D	1,160
	7.4-48.3	1,126		
	48.3-100	1,276		

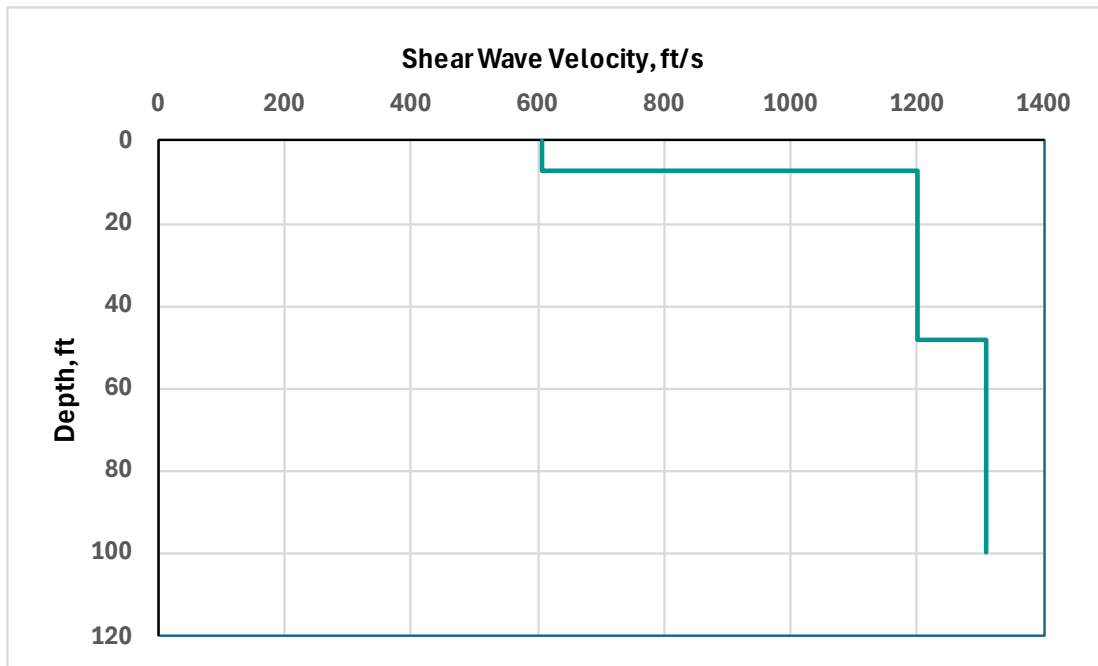


Figure 4: SL-1 ReMi Survey Results

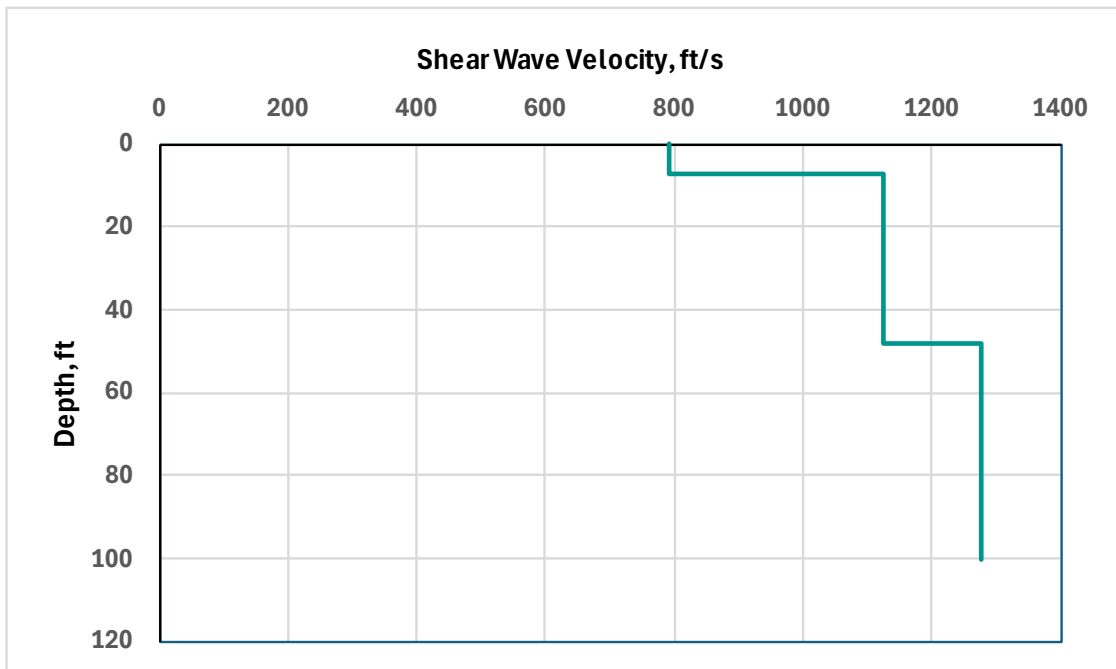


Figure 5: SL-2 ReMi Survey Results

LIMITATIONS

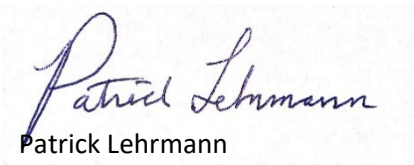
The geophysical field investigations and evaluation in this report have been conducted in general accordance with the current practice and standard of care exercised by consultants performing similar tasks in the project area. No evaluation is detailed enough to reveal every subsurface feature or condition, and no warranty (expressed or implied) is made regarding the conclusions and opinions in this report. Variations and conditions not observed or described in this report may be present.

Even though care was taken during the geophysical survey, results have been evaluated by computer modeling that may not result unique solutions for a given dataset. Therefore, results should be considered interpretive and approximate, and only provide information at the locations of geophysical data collection.

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

Very truly yours,

GEOCON INCORPORATED



Patrick Lehrmann
No. 1043

PL:OA:am

(e-mail) Addressee



Orion Adah
Geophysical Services Manager