GEOTECHNICAL INVESTIGATION

465-577 SOUTH ARROYO PARKWAY
PASADENA, CALIFORNIA

GEOCON
WEST, INC.

GEOTECHNICAL
ENVIRONMENTAL
MATERIALS

PROJECT NO.
W1111-06-01
July 13, 2021

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GEOTECHNICAL ■ ENVIRONMENTAL ■ MATERIALS

Project No. W1111-06-01
July 13, 2021

Mr. Peter Kutzer
The Arroyo Parkway. LLC
716 Mission Street
South Pasadena, California 91030

Subject: GEOTECHNCIAL INVESTIGATION
PROPOSED MEDICAL OFFICE/COMMERCIAL STRUCTURE
(OR MULTI-FAMILY RESIDENTIAL/COMMERCIAL STRUCTURE)
465-577 SOUTH ARROYO PARKWAY
PASADENA, CALIFORNIA

Dear Mr. Kutzer,

In accordance with your authorization of our proposal dated October 23, 2019, we have performed a geotechnical investigation for the proposed medical office structure with ground-floor commercial uses (or multi-family residential structure with ground-floor commercial uses) and assisted living structure located at 465-577 South Arroyo Parkway in the City of Pasadena, California. The accompanying report presents the findings of our study and our conclusions and recommendations pertaining to the geotechnical aspects of proposed design and construction. Based on the results of our investigation, it is our opinion that the site can be developed as proposed, provided the recommendations of this report are followed and implemented during design and construction.

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

Very truly yours,

GEOCON WEST, INC.

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TABLE OF CONTENTS

TABLE OF CONTENTS (Continued)

GEOTECHNICAL INVESTIGATION

1. PURPOSE AND SCOPE

This report presents the results of a geotechnical investigation for the proposed medical office with ground-floor commercial uses (or multi-family residential structure with ground-floor commercial uses) and assisted living structures located at 465-577 South Arroyo Parkway in the City of Pasadena, 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 January 13, 2020 and June 11, 2020 by excavating five 8-inch-diameter borings to depths of 30½ and 91 feet below the existing ground surface using a truck-mounted hollow-stem auger drilling machine. The approximate locations of the exploratory borings are depicted on the Site Plan (see Figure 2). A detailed discussion of the field investigation, including 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 465-577 South Arroyo Parkway in the City of Pasadena, California. The site consists of multiple lots that are currently occupied by several one- to two-story structures and associated asphalt paved parking lots with a total area of approximately 3.3 acres. It is our understanding that the existing on-site structures will be demolished with the exception of the grocery store, located in the northernmost portion of the site and two historic buildings that front South Arroyo Parkway. The site is bounded Bellevue Drive to the north, by California Boulevard to the south, by Metro Gold Line light rail tracks to the west, and by South Arroyo Parkway to the east. The site gently slopes to the south and surface water drainage at the site appears to flow to the city streets. Vegetation onsite consists of shrubs and trees in localized planter areas.

Based on the information provided by the Client, it is our understanding that the proposed development will consist of a seven-story medical office building with ground-floor commercial uses (or a seven-story multi-family residential building with ground-floor commercial uses) and a seven-story assisted living building, both to be constructed over five subterranean levels. The proposed design plan provides the flexibility to exchange the medical office uses for residential uses. The proposed site improvements are depicted on the Site Plan (see Figure 2).

Based on the preliminary nature of the design at this time, wall and column loads were not available. It is anticipated that column loads for the proposed seven-story medical office building with ground-floor commercial uses (or a seven-story multi-family residential building with ground-floor commercial uses) will be up to 1350 kips, and wall loads will be up to 13.5 kips per linear foot. The anticipated column loads for the proposed assisted living facility will be up to 1,450 kips and wall loads will be up to 14.5 kips per linear foot.

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

3. GEOLOGIC SETTING

The project property is located in the southwestern portion of the Raymond Basin, an alluvial-filled structural basin bounded on the north by the Sierra Madre Fault Zone and on the south by the Raymond Fault. The site is situated in an early Holocene age alluvial channel that has dissected the older Altadena fan (see Figure 3, Local Geologic Map).

Regionally, the property is located in the Transverse Ranges geomorphic province. The province is bounded by the Big Pine Fault on the north, the San Andreas Fault Zone on the east, the Pacific Ocean on the west, and the Santa Monica-Raymond-Sierra Madre-Cucamonga fault system on the south. The province is characterized by east-west trending mountain ranges that extends approximately 325 miles and vary in width from 10 to 50 miles. These mountain ranges include the Santa Ynez, San Gabriel, San Bernardino, and Santa Monica Mountains, and associated valleys. The narrowest points are at situated along the western extreme in the Santa Ynez Mountains and at the Cajon Pass which separates the San Gabriel and San Bernardino Mountains. The province's broadest point extends from the Santa Monica Mountains to the Tehachapi Mountains.

4. SOIL AND GEOLOGIC CONDITIONS

Based on our field investigation and published geologic maps of the area, the site is underlain by artificial fill and Quaternary age alluvium consisting primarily of sand and lesser amounts of silt (Crook et al., 1987). Detailed stratigraphic profiles of the materials encountered at the site are provided on the boring logs in Appendix A.

4.1 Artificial Fill

Artificial fill was encountered in our explorations to a maximum depth of 2 feet below existing ground surface. The artificial fill generally consists of dark brown to grayish brown poorly graded sand and silty sand. The artificial fill is characterized as primarily fine to medium grained, slightly moist, and loose to medium dense. The fill is likely the result of past grading or construction activities at the site. Deeper fill may exist between excavations and in other portions of the site that were not directly explored.

4.2 Alluvium

Quaternary age alluvial deposits were encountered beneath the fill. The alluvium generally consists of light to dark yellowish brown, poorly graded and well-graded sand with varying amounts of coarse gravel. In borings B2 and B3, silt and sandy silt were encountered above the sand to a maximum depth of 6 feet beneath the existing ground surface. The alluvial soils are characterized as medium dense to very dense or stiff to hard, and slightly moist.

5. GROUDWATER

Based on a review of the Seismic Hazard Zone Report for the Pasadena Quadrangle (California Division of Mines and Geology [CDMG], 1998), the historically highest groundwater level in the area is between 50 and 100 feet beneath the ground surface. Groundwater information presented in this document is generated from data collected in the early 1900's to the late 1990s. Based on current groundwater basin management practices, it is unlikely that groundwater levels will ever exceed the historic high levels.

Groundwater was not encountered in our borings drilled to a maximum depth of approximately 91 feet beneath the existing ground surface. Considering the lack of groundwater in our borings, the reported depth of the historic high groundwater level (CDMG, 1998), and the depth of the proposed subterranean levels, 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. In addition, recent requirements for stormwater infiltration could result in shallower seepage conditions in the immediate site vicinity. Proper surface drainage of irrigation and precipitation will be critical for future performance of the project. Recommendations for drainage are provided in the S*urface Drainage* section of this report (see Section 7.23).

6. GEOLOGIC HAZARDS

6.1 Surface Fault Rupture

The numerous faults in Southern California include 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.

As shown on Figure 4, Seismic Hazard Zone Map, the site is not within a state-designated Alquist-Priolo Earthquake Fault Zone for surface fault rupture hazards (CGS, 2020a; CGS, 2020b; CDMG, 1999). No active or potentially active faults with the potential for surface fault rupture are known to pass directly beneath the site. Therefore, the potential for surface rupture due to faulting occurring beneath the site during the design life of the proposed development is considered low. However, the site is located in the seismically active Southern California region, and could be subjected to moderate to strong ground shaking in the event of an earthquake on one of the many active Southern California faults. The faults in the vicinity of the site are shown on Figure 4 and Figure 5, Regional Fault Map.

The closest surface trace of an active fault to the site is the Raymond Fault located approximately 1.2 miles to the south (CGS, 2017). Other nearby active faults are the Sierra Madre Fault Zone, the Verdugo Fault, and the Hollywood Fault located approximately 4.0 miles northeast,4.4 miles west-northwest, and 5.6 miles west-southwest of the site, respectively (Ziony and Jones, 1989). The active San Andreas Fault Zone is located approximately 28 miles northeast of the site.

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

6.2 Seismicity

As with all of Southern California, the site has experienced historic earthquakes from various regional faults. The seismicity of the region surrounding the site was formulated based on research of an electronic database of earthquake data. The epicenters of recorded earthquakes with magnitudes equal to or greater than 5.0 in the site vicinity are depicted on Figure 6, Regional Seismicity Map. A partial list of moderate to major magnitude earthquakes that have occurred in the Southern California area within the last 100 years is included in the following table.

LIST OF HISTORIC EARTHQUAKES

Earthquake (Oldest to Youngest)	Date of Earthquake	Magnitude	Distance to Epicenter (Miles)	Direction to Epicenter
Near Redlands	July 23, 1923	6.3	52	E
Long Beach	March 10, 1933	6.4	37	SSE
Tehachapi	July 21, 1952	7.5	77	NW
San Fernando	February 9, 1971	6.6	24	NW
Whittier Narrows	October 1, 1987	5.9	7	SE
Sierra Madre	June 28, 1991	5.8	12	NE
Landers	June 28, 1992	7.3	98	E
Big Bear	June 28, 1992	6.4	76	E
Northridge	January 17, 1994	6.7	23	WNW
Hector Mine	October 16, 1999	7.1	112	ENE
Ridgecrest	July 5, 2019	7.1	116	NNE

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

6.3 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 1613 of the 2019 CBC using the online applications developed by USGS.

6.3.1 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 mean spectral response accelerations having a 2 percent chance of exceedance in 50 years were evaluated at 5 percent damping using the USGS Unified Hazard Tool (UHT). The Dynamic U.S. 2014 (v4.2.0) edition was used within the analysis, which is based on the UCERF-3 fault model. The soil underlying the site was modeled as a Site Class "C/D" with a corresponding average shear wave velocity (V_S30) of 360 meters per second. The site class definition is based on Standard Penetration Test blow count data and site information (V_S30) provided by the OpenSha, Site Data Application, Version 1.5.0.

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 mean value of the four GMPEs was evaluated. The mean spectral accelerations were rotated to maximum direction using the period specific ratios from Shahi et al. (2013 & 2014).

The GMPE of Campbell and Borzorgnia requires that the depth to where the shear wave velocity reaches 2.5 kilometers per second (Z2.5) 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 (Z1.0) be defined. The values of Z2.5 and Z1.0 are internally calculated by the Uniform Hazard Tool.

The MCE uniform hazard response spectra was adjusted to risk-targeted spectral accelerations corresponding to a 1 percent chance of collapse in 50 years by using the USGS Risk-Targeted Ground Motion Calculator and following ASCE 7-16 Section 21.2.1.2 Method 2.

The risk-targeted Maximum Considered Earthquake (MCE_R) probabilistic response spectrum is provided on Figure 7.

6.3.2 Deterministic Seismic Hazard Analysis

In order to define the deterministic scenario events, deaggregation of the uniform hazard probabilistic response spectra was performed using the USGS Uniform Hazard Tool. The inversion approach used by UCERF-3 allows for a large number of variations for each source scenario, including multi-fault ruptures. Therefore, deaggregation of UCERF-3 consists of the contributions from multi-fault ruptures rather than individual source contributions. To address this, the USGS Unified Hazard Tool aggregates the contributions on a per-fault-section basis, with rupture contributions only ever counted once. The Unified Hazard Tool deaggregation contributor list shows the fault sections which contribute most to the 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 deaggregation, we have considered scenario events with the greatest contribution to the deterministic ground motions.

The earthquake magnitudes of the deterministic scenario events were based on the BSSC 2014 Scenario Event which includes the parent fault identified in the deaggregation and has the largest earthquake magnitude. The closest distance (R_{rup}) from the fault to the site was taken from the Uniform Hazard Tool deaggregation results. Other fault source parameters were defined by the values in the BSSC2014 Scenario Catalog. The values of Z2.5 and Z1.0 were estimated using data from the Community Velocity Model (CVM) Version 11.9.x, Basin Depth developed by Southern California Earthquake Data Center (SCEDC) accessed by the OpenSHA Site Data Application (v1.5.0).

Two deterministic scenario events were considered for this analysis and consisted of a magnitude 7.08 event occurring on the Puente Hills fault at a closest distance of 11.33 km and a magnitude 6.71 event occurring on the Raymond fault at a closest distance of 2.48 km.

The deterministic median and standard deviation (sigma) for the scenario events were evaluated using the USGS NSHMP-HAZ-WS Response Spectra online application. 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 geometric 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 0.5 second would be a magnitude 7.08 event on the Puente Hills fault and from 0.75 to 5 seconds would be a magnitude 6.71 event on the Raymond fault. The 84th percentile maximum rotated component deterministic response spectrum is provided on Figure 8.

6.3.3 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 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 7 and 8. The Site-Specific Design Earthquake response spectrum at 5 percent damping is presented on Figure 8 and in tabular form on Figure 9.

6.3.4 Mapped Acceleration Parameters

The following table summarizes the mapped acceleration parameters obtained from the 2019 California Building Code (CBC; Based on the 2018 International Building Code [IBC] and ASCE 7-16), Chapter 16 Structural Design, Section 1613A Earthquake Loads. The data was calculated using the computer program U.S. Seismic Design Maps, provided by the USGS. The short spectral response uses a period of 0.2 second.

MAPPED SPECTRAL ACCELERATIONS

Parameter	Value	2019 CBC Reference
MCE_R Ground Motion Spectral Response Acceleration – Class B (short), S_S	2.093g	Figure 1613.2.1(1)
MCE_R Ground Motion Spectral Response Acceleration – Class B (1 sec), S_1	0.761g	Figure 1613.2.1(2)

6.3.5 Site-Specific Seismic Design Criteria

Based 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_{DS} 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 from 1 to 5 seconds, inclusive. The values of S_{MS} and S_{M1} shall be taken as 1.5 times the site-specific values of S_{DS} 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_{MS}	2.515g
Site Class Modified MCE_R Spectral Response Acceleration – (1 sec), S_{M1}	1.522g
5% Damped Design Spectral Response Acceleration (short), S_{DS}	1.677g
5% Damped Design Spectral Response Acceleration (1 sec), S_{D1}	1.015g

6.3.6 Site-Specific Peak Ground Acceleration

The site-specific Maximum Considered Earthquake (MCE_G) geometric mean peak ground acceleration was evaluated in accordance with ASCE 7-16 Section 21.5.

The probabilistic geometric mean peak ground acceleration and the deterministic 84th percentile geometric mean peak ground acceleration were analyzed using the same approaches as described above. The analysis used the same site class and earthquake scenario.

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	0.97g	Section 21.5

6.4 Deaggregated Seismic Source Parameters

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

Deaggregation of the MCE peak ground acceleration was performed using the USGS online Unified Hazard Tool, 2014 Conterminous U.S. Dynamic edition (v4.2.0). The result of the deaggregation analysis indicates that the predominant earthquake contributing to the MCE peak ground acceleration is characterized as a 6.97 magnitude event occurring at a hypocentral distance of 8.04 kilometers from the site.

Deaggregation was also performed for the Design Earthquake (DE) peak ground acceleration, and the result of the analysis indicates that the predominant earthquake contributing to the DE peak ground acceleration is characterized as a 6.88 magnitude occurring at a hypocentral distance of 12.75 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.

The State of California Seismic Hazard Zone Map for the Pasadena Quadrangle (CDMG, 1999) indicates that the site is not located within an area designated as having a potential for liquefaction. Groundwater was not encountered in our borings drilled to a maximum depth of approximately 91 feet beneath the existing ground surface. Additionally, the historic high groundwater level in the site vicinity is documented by CDMG (1998) as approximately between 50 and 100 feet below ground surface. Based on these considerations, it is our opinion that the potential for liquefaction to occur beneath the site is considered low.

6.6 Slope Stability

The topography at the site is gently sloping to the south to southeast. The site is not located within an area identified as a "Hillside" area or an area identified as having a potential for slope stability hazards (Leighton, 1990; City of Pasadena, 2002). Additionally, the site is not located within an area identified as having a potential for seismic slope instability (CDMG, 1999). There are no known landslides near the site, nor is the site in the path of any known or potential landslides. Therefore, the potential for slope stability hazards to adversely affect the site 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. According to the County of Los Angeles Safety Element (Leighton, 1990) and the City of Pasadena Safety Element (Earth Consultants International, 2002), 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 seismic-induced seiche is considered unlikely.

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

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 vicinity of the site (CalGEM, 2020). However, due to the voluntary nature of record reporting by the oil well drilling companies, wells may be improperly located or not shown on the location map and undocumented wells could be encountered during construction. Any wells encountered will need to be properly abandoned in accordance with the current requirements of the DOGGR.

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

6.10 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 not located within an area of known ground subsidence. No large-scale extraction of groundwater, gas, oil, or geothermal energy is occurring or planned at the site or in the general site vicinity. There appears to be little or no potential for ground subsidence due to withdrawal of fluids or gases at the site.

7. CONCLUSIONS AND RECOMMENDATIONS

7.1 General

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

7.1.2 Up to 2 feet of existing artificial fill was encountered during the site investigation. The existing fill encountered is believed to be the result of past grading and construction activities at the site. Deeper fill may exist in other areas of the site that were not directly explored. Excavations for the subterranean portions of the structure are anticipated to penetrate through the existing fill and expose undisturbed granular alluvial soils throughout the excavation bottom. It is our opinion that the existing fill, in its present condition, is not suitable for direct support of proposed foundations or slabs. The existing fill and site soils are suitable for re-use as engineered fill provided the recommendations in the *Grading* section of this report are followed (see Section 7.4).

7.1.3 Based on these considerations, the proposed structure may be supported on a conventional shallow spread foundation system deriving support in competent undisturbed alluvium at the bottom of the subterranean level. Foundations should be deepened as necessary to extend into satisfactory soils and must be observed and approved in writing by the Geotechnical Engineer (a representative of Geocon West, Inc.).

7.1.4 Due to the granular nature of the soils and potential for excessive caving, the contractor should be prepared to use shoring and casing as well as to form foundation excavations into granular alluvial soils at the excavation bottom, as necessary.

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

7.1.6 Due to the depth of the excavation and the proximity to the property lines, city streets and adjacent offsite structures, excavations for the structure will require sloping and shoring measures in order to provide a stable excavation. Where shoring is required it is recommended that a soldier pile shoring system be utilized. In addition, where the proposed excavation will be deeper than and adjacent to an offsite structure, the proposed shoring should be designed to resist the surcharge imposed by the adjacent offsite structures. Recommendations for *Temporary Excavations* are provided in Section 7.16 of this report.

7.1.7 The bottom of the subterranean level is in close proximity to the historic high groundwater level. However, groundwater was not encountered in our borings drilled to a maximum depth to 91 feet below ground surface. Based on these considerations it is our opinion that a hydrostatic design of the basement level to offset potential buoyancy is not required. Due to the nature of the design, waterproofing of subterranean walls and slabs is suggested. Particular care should be taken in the design and installation of waterproofing to avoid moisture problems, or actual water seepage into the structure through any normal shrinkage cracks which may develop in the concrete walls, floor slab, foundations and/or construction joints. The design and inspection of the waterproofing is not the responsibility of the geotechnical engineer. A waterproofing consultant should be retained in order to recommend a product or method, which would provide protection to subterranean walls, floor slabs and foundations.

7.1.8 Where new foundations are constructed immediately adjacent to existing foundations, the new foundation should be deepened to match or exceed the depth of the existing foundation to prevent a surcharge on the existing foundation. Where a proposed foundation will be deeper than an existing adjacent foundation, the proposed foundation must be designed to resist the surcharge imposed by the existing foundation. The surcharge area may be defined by a 1:1 projection down and away from the bottom of an existing foundation.

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

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

7.1.11 Where new paving is to be placed, it is recommended that all existing fill and soft alluvial soils be excavated and properly compacted for paving support. The client should be aware that excavation and compaction of all existing fill and soft alluvial soils in the area of new paving is not required; however, paving constructed over existing uncertified fill or unsuitable alluvial soil may experience increased settlement and/or cracking, and may therefore have a shorter design life and increased maintenance costs. As a minimum, the upper 12 inches of subgrade soil should be scarified and properly compacted for paving support. *Preliminary Pavement Recommendations* section of this report (see Section 7.10).

7.1.12 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 7.22).

7.1.13 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 reevaluated by this office.

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

7.2 Soil and Excavation Characteristics

7.2.1 The in-situ soils can be excavated with moderate effort using conventional excavation equipment. Due to the granular nature of the soils, excessive caving should be anticipated in vertical excavations. The contractor should also be aware that formwork will likely be required to prevent caving of shallow spread foundation excavations.

7.2.2 It is the responsibility of the contractor to ensure that all excavations and trenches are properly shored and maintained in accordance with applicable OSHA rules and regulations to maintain safety and maintain the stability of 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 7.16).

7.2.4 Based on the depth of the foundation level and the granular nature of the soils encountered at that depth, the existing site soils are considered to be "non-expansive" and to have a "very low" (EI < 20) expansive potential in accordance with the 2019 California Building Code (CBC) Section 1803.5.3. Based on the depth of the proposed subterranean level and granular nature of the site soils, the proposed structure would not be prone to the effects of expansive soil.

7.3 Minimum Resistivity, pH, and Water-Soluble Sulfate

7.3.1 Potential of Hydrogen (pH) and resistivity testing as well as chloride content testing were performed on representative samples of soil to generally evaluate the corrosion potential to surface utilities. The tests were performed in accordance with California Test Method Nos. 643 and 422 and indicate that the soils are considered "moderately corrosive" with respect to corrosion of buried ferrous metals on site. The results are presented in Appendix B (Figure B39) and should be considered for design of underground structures. Due to the corrosive potential of the soils, it is recommended that PVC, ABS or other approved plastic piping be utilized in lieu of cast-iron when in direct contact with the site soils.

7.3.2 Laboratory tests were performed on representative samples of the site materials to measure the percentage of water-soluble sulfate content. Results from the laboratory water-soluble sulfate tests are presented in Appendix B (Figure B39) and indicate that the on-site materials possess a sulfate exposure class of "S0" to concrete structures as defined by 2019 CBC Section 1904 and ACI 318-14 Table 19.3.1.1.

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

7.4 Grading

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

7.4.2 A preconstruction conference should be held at the site prior to the beginning of grading operations with the owner, contractor, civil engineer, geotechnical engineer, and, if applicable, 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 encountered during exploration is suitable for re-use as an engineered fill, provided any encountered oversize material (greater than 6 inches) and any encountered deleterious debris is removed.

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

7.4.5 All foundations should derive support in the competent undisturbed alluvial soils. Foundations should be deepened as necessary to extend into satisfactory soils and must be observed and approved in writing by the Geotechnical Engineer (a representative of Geocon West, Inc.).

7.4.6 The concrete slab-on-grade and ramp for the subterranean portion of the proposed structure may bear directly on the competent undisturbed alluvial soil at the excavation bottom or newly placed engineered fill. It is recommended that the exposed soils be proof rolled prior to placing construction materials. Any disturbed soils should be properly compacted for slab and ramp support, as necessary.

7.4.7 All fill and backfill soils should be placed in horizontal loose layers approximately 6 to 8 inches thick, moisture conditioned to optimum moisture content, and properly compacted to at least 90 percent relative compaction, as determined by ASTM Test Method D 1557 (latest edition).

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

7.4.9 Where new paving is to be placed, it is recommended that all existing fill and soft alluvial soils be excavated and properly compacted for paving support. The client should be aware that excavation and compaction of all existing fill and soft alluvial soils in the area of new paving is not required; however, paving constructed over existing uncertified fill or unsuitable alluvial soil may experience increased settlement and/or cracking, and may therefore have a shorter design life and increased maintenance costs. As a minimum, the upper 12 inches of subgrade soil should be scarified and properly compacted to a minimum of 95 percent of the laboratory maximum dry density in accordance with ASTM D 1557 (latest edition). *Preliminary Pavement Recommendations* section of this report (see Section 7.10).

7.4.10 Although not anticipated for this project, all imported fill shall be observed, tested, and approved by Geocon West, Inc. prior to bringing soil to the site. Rocks larger than 6 inches in diameter shall not be used in the fill. If necessary, import soils used as structural fill should have an expansion index less than or equal to 20 and soil corrosivity properties that are equally or less detrimental to that of the existing onsite soils (see Figure B39). Imported soil placed in building pad areas must be placed uniformly across the pad at the direction of the Geotechnical Engineer (a representative of Geocon).

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

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

7.5 Conventional Foundation Design

7.5.1 A conventional foundation system may be utilized for support of the proposed structures provided foundations derive support in undisturbed competent alluvium at the proposed subterranean level. Foundations should be deepened as necessary to penetrate through existing fill and/or soft or disturbed alluvium at the direction of the Geotechnical Engineer. All foundation excavations must be observed and approved in writing by the Geotechnical Engineer (a representative of Geocon), prior to placing steel or concrete.

7.5.2 Due to the granular nature of soils and potential for caving, the contractor should be prepared to form foundation excavations into granular alluvial soils at the excavation bottom, as necessary.

7.5.3 Continuous footings may be designed for an allowable bearing capacity of 4,000 pounds per square foot (psf), and should be a minimum of 12 inches in width, 18 inches in depth below the lowest adjacent grade, and 12 inches into the recommended bearing material.

7.5.4 Isolated spread foundations may be designed for an allowable bearing capacity of 4,500 psf, and should be a minimum of 24 inches in width, 18 inches in depth below the lowest adjacent grade, and 12 inches into the recommended bearing material.

7.5.5 The allowable soil bearing pressure may be increased by 500 psf and 1,000 psf for each additional foot of foundation width and depth, respectively, up to a maximum allowable soil bearing pressure of 8,000 psf.

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

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

7.5.8 Continuous footings should be reinforced with four No. 4 steel reinforcing bars, two placed near the top of the footing and two near the bottom. Reinforcement for spread footings should be designed by the project structural engineer.

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

7.5.10 No special subgrade presaturation is required prior to placement of concrete. However, the slab and foundation subgrade should be sprinkled as necessary; to maintain a moist condition as would be expected in any concrete placement.

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

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

7.6 Foundation Settlement

7.6.1 The maximum expected static settlement for an on-grade structure supported on a conventional foundation system or deepened foundation system deriving support in the recommended bearing materials and designed with a maximum bearing pressure of 8,000 psf is estimated to be less than 1 inch and occur below the heaviest loaded structural element. Settlement of the foundation system is expected to occur on initial application of loading. Differential settlement is not expected to exceed ¾ inch over a distance of 20 feet.

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

7.7 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 or is undesirable, foundations may derive support directly in the competent undisturbed alluvial soils, and should be deepened as necessary to maintain a minimum 12-inch embedment into the recommended bearing materials.

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, 24 inches in depth below the lowest adjacent grade and 12 inches into the recommended bearing material. The allowable bearing pressure may be increased by up to one-third for transient loads due to wind or seismic forces.

7.7.3 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 0.4 may be used with the dead load forces in the undisturbed alluvial soils or engineered fill.

7.8.2 Passive earth pressure for the sides of foundations and slabs poured against the alluvial soils or properly compacted engineered fill may be computed as an equivalent fluid having a density of 300 pcf with a maximum earth pressure of 3,000 pcf. When combining passive and friction for lateral resistance, the passive component should be reduced by one-third.

7.9 Concrete Slabs-on-Grade

7.9.1 Unless specifically evaluated and designed by a qualified structural engineer, the concrete slab-on-grade and ramp for the subterranean parking garage should be a minimum of 6 inches thick and reinforced with No. 3 steel reinforcing bars placed 18 inches on center in both horizontal directions positioned vertically near the slab midpoint. The concrete slab-on-grade for the parking garage may bear directly on undisturbed alluvium at the excavation bottom. Any disturbed soils should be properly compacted for slab support.

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

7.9.3 For seismic design purposes, a coefficient of friction of 0.4 may be utilized between concrete slabs and subgrade soils without a moisture barrier, and 0.15 for slabs underlain by a moisture barrier.

7.9.4 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.

7.9.5 Exterior slabs for walkways or flatwork, not subject to traffic loads, should be at least 4 inches thick and reinforced with No. 3 steel reinforcing bars placed 18 inches on center in both horizontal directions, positioned near the slab midpoint. Prior to construction of slabs, the upper 12 inches of subgrade should be approximately moistened to optimum moisture content and properly compacted to at least 95 percent relative compaction, as determined by ASTM Test Method D 1557 (latest edition). Crack control joints should be spaced at intervals not greater than 10 feet and should be constructed using saw-cuts or other methods as soon as practical following concrete placement. Crack control joints should extend a minimum depth of one-fourth the slab thickness. The project structural engineer should design construction joints as necessary.

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

7.10 Preliminary Pavement Recommendations

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

7.10.2 The following pavement sections are based on an assumed R-Value of 35. Once site grading activities are complete an R-Value should be obtained by laboratory testing to confirm the properties of the soils serving as paving subgrade, prior to placing pavement.

7.10.3 The Traffic Indices listed below are estimates. Geocon does not practice in the field of traffic engineering. The actual Traffic Index for each area should be determined by the project civil engineer. If pavement sections for Traffic Indices other than those listed below are required, Geocon should be contacted to provide additional recommendations. Pavement thicknesses were determined following procedures outlined in the *California Highway Design Manual* (Caltrans). It is anticipated that the majority of traffic will consist of automobile and large truck traffic.

PRELIMINARY PAVEMENT DESIGN SECTIONS

Location	Estimated Traffic Index (TI)	Asphalt Concrete (inches)	Class 2 Aggregate Base (inches)
Automobile Parking And Driveways	4.0	3.0	4.0
Trash Truck & Fire Lanes	7.0	4.0	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 in lieu of Class 2 aggregate base is acceptable. Crushed Miscellaneous Base should conform to Section 200-2.4 of the "*Standard Specifications for Public Works Construction*" (Green Book).

7.10.5 Unless specifically designed and evaluated by the project structural engineer, where exterior concrete paving will be utilized for support of vehicles, it is recommended that the concrete be a minimum of 6 inches of concrete reinforced with No. 3 steel reinforcing bars placed 18 inches on center in both horizontal directions. Concrete paving supporting vehicular traffic should be underlain by a minimum of 4 inches of aggregate base and a properly compacted subgrade. The subgrade and base material should be compacted to 95 percent relative compaction as determined by ASTM Test Method D 1557 (latest edition).

7.10.6 The performance of pavements is highly dependent upon providing positive surface drainage away from the edge of pavements. Ponding of water on or adjacent to the pavement will likely result in saturation of the subgrade materials and subsequent cracking, subsidence and pavement distress. If planters are planned adjacent to paving, it is recommended that the perimeter curb be extended at least 12 inches below the bottom of the aggregate base to minimize the introduction of water beneath the paving.

7.11 Retaining Wall Design

7.11.1 The recommendations presented below are generally applicable to the design of rigid concrete or masonry retaining walls having a maximum height of 60 feet. In the event that walls significantly higher than 60 feet are planned, Geocon should be contacted for additional recommendations.

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

7.11.3 Retaining walls with a level backfill surface that are not restrained at the top should be designed utilizing a triangular distribution of pressure (active pressure). Restrained walls are those that are not allowed to rotate more than 0.001H (where H equals the height of the retaining portion of the wall in feet) at the top of the wall. Where walls are restrained from movement at the top, walls may be designed utilizing a triangular distribution of pressure (at-rest pressure). 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 60	44	52

7.11.4 The wall pressures provided above assume that the proposed retaining walls will support relatively undisturbed alluvial soils or engineered fill derived from onsite soil. If import soil is used to backfill proposed walls, revised earth pressures may be required to account for the geotechnical properties of the soil placed as engineered fill. This should be evaluated once the use of import soil is established. All imported fill shall be observed, tested, and approved by Geocon West, Inc. prior to bringing soil to the site.

7.11.5 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.

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

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

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

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

and

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

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

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

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

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

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

and

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

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

then

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

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

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

7.11.10 Seismic lateral forces will be required for any retaining walls in excess of 6 feet. Recommendations for seismic lateral forces are provided in the following section.

7.12 Dynamic (Seismic) Lateral Forces

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

7.12.2 A seismic load of 22 pcf should be used for design of walls that support more than 6 feet of backfill in accordance with Section 1803.5.12 of the 2019 CBC. The seismic load is applied as an equivalent fluid pressure along the height of the wall and the calculated loads result in a maximum load exerted at the base of the wall and zero at the top of the wall. This seismic load should be applied in addition to the active earth pressure. We used the peak site acceleration. PGA_M, of 0.971 calculated from ASCE 7-16 Section 11.8.3 and applied a pseudo-static coefficient of 0.33.

7.13 Retaining Wall Drainage

7.13.1 Retaining walls should be provided with a drainage system extended at least two-thirds the height of the wall. At the base of the drain system, a subdrain covered with a minimum of 12 inches of gravel should be installed, and a compacted fill blanket or other seal placed at the surface. 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 10). These vertical columns of drainage material would then be connected at the bottom of the wall to 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 Elevator Pit Design

7.14.1 The elevator pit slab and retaining wall should be designed by the project structural engineer. Elevator pits may be designed in accordance with the recommendations in the *Conventional Foundation Design and Retaining Wall Design* sections of this report (see Section 7.5 and Section 7.11).

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

7.14.3 If retaining wall drainage is to be provided, the drainage system should be designed in accordance with the *Retaining Wall Drainage* section of this report (see Section 7.13).

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

7.15 Elevator Piston

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

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

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

7.16 Temporary Excavations

7.16.1 Excavations up to 60 feet in height may be required during basement and foundation excavations. The excavations are expected to expose artificial fill and alluvial soils, which are subject to excessive caving. Excavations up to 5 feet in height may be attempted where loose soils or caving sands are not present, and where not surcharged by adjacent traffic or structures.

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

7.16.3 If excavations in close proximity to an adjacent property line and/or structure are required, shoring will be necessary in order to maintain lateral support of offsite improvements. Recommendation for shoring are presented in the following section of this report.

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

7.17 Shoring – Soldier Pile Design and Installation

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

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

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

7.17.4 The proposed soldier piles may also be designed as permanent piles and may be utilized to underpin the existing offsite structures. The required pile depth, dimension, spacing and underpinning connection to existing offsite foundation should be determined and designed by the project structural and shoring engineers. All piles utilized for shoring can also be incorporated into a permanent retaining wall system (shotcrete wall) provided they are designed in accordance with the earth pressure provided in the *Retaining Wall Design* section of this report (see Section 7.11).

7.17.5 Drilled cast-in-place soldier piles should be placed no closer than 2 diameters on center. The minimum diameter of the piles is 18 inches. Structural concrete should be used for the soldier piles below the excavation; lean-mix concrete may be employed above that level. As an alternative, lean-mix concrete may be used throughout the pile where the reinforcing consists of a wideflange section. The slurry must be of sufficient strength to impart the lateral bearing pressure developed by the wideflange section to the soil. For design purposes, an allowable passive value for the soils below the bottom plane of excavation may be assumed to be 300 psf per foot. Where piles are installed by vibration techniques, the passive pressure may be assumed to mobilize across a width equal to the two times the dimension of the beam flange. The allowable passive value may be doubled for isolated piles spaced a minimum of three times the pile diameter. To develop the full lateral value, provisions should be implemented to assure firm contact between the soldier piles and the undisturbed soils.

7.17.6 Groundwater was not encountered in our borings, drilled to a maximum depth of 91 feet below ground surface during the site exploration. However, groundwater may be encountered during excavations for the proposed soldier piles. If more than 6 inches of water is present in the bottom of the excavation, a tremie is required to place the concrete into the bottom of the hole. A tremie should consist of a rigid, water-tight tube having a diameter of not less than 6 inches with a hopper at the top. The tube should be equipped with a device that will close the discharge end and prevent water from entering the tube while it is being charged with concrete. The tremie should be supported so as to permit free movement of the discharge end over the entire top surface of the work and to permit rapid lowering when necessary to retard or stop the flow of concrete. The discharge end should be closed at the start of the work to prevent water entering the tube and should be entirely sealed at all times, except when the concrete is being placed. The tremie tube should be kept full of concrete. The flow should be continuous until the work is completed and the resulting concrete seal should be monolithic and homogeneous. The tip of the tremie tube should always be kept about 5 feet below the surface of the concrete and definite steps and safeguards should be taken to insure that the tip of the tremie tube is never raised above the surface of the concrete.

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

7.17.8 Casing will likely be required since excessive caving is anticipated in the drilled excavations. The contractor should have casing available prior to commencement of pile excavation. When casing is used, extreme care should be employed so that the pile is not pulled apart as the casing is withdrawn. At no time should the distance between the surface of the concrete and the bottom of the casing be less than 5 feet. Continuous observation of the drilling and pouring of the piles by the Geotechnical Engineer (a representative of Geocon West, Inc.), is required.

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

7.17.10 If a vibratory method is utilized, the owner should be aware of the potential risks associated with vibratory efforts, which typically involve inducing settlement within the vicinity of the pile which could result in a potential for damage to existing improvements in the area.

7.17.11 The level of vibration that results from the installation of the piles should not exceed a threshold where occupants of nearby structures are disturbed, despite higher vibration tolerances that a building may endure without deformation or damage. The main parameter used for vibration assessment is peak particle velocity in units of inch per second (in/sec). The acceptable range of peak particle velocity should be evaluated based on the age and condition of adjacent structures, as well as the tolerance of human response to vibration.

7.17.12 Based on Table 19 of the *Transportation and Construction Induced Vibration Guidance Manual* (Caltrans 2013), a continuous source of vibrations (ex. vibratory pile driving) which generates a maximum peak particle velocity of 0.5 in/sec is considered tolerable for modern industrial/commercial buildings and new residential structures. The Client should be aware that a lower value may be necessary if older or fragile structures are in the immediate vicinity of the site.

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

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

7.17.15 The frictional resistance between the soldier piles and retained soil may be used to resist any vertical component of load on the soldier pile. The coefficient of friction may be taken as 0.4 based on uniform contact between the steel beam and lean-mix concrete and retained earth. The portion of soldier piles below the plane of excavation may also be employed to resist the downward loads. The downward capacity may be determined using a frictional resistance of 500 psf.

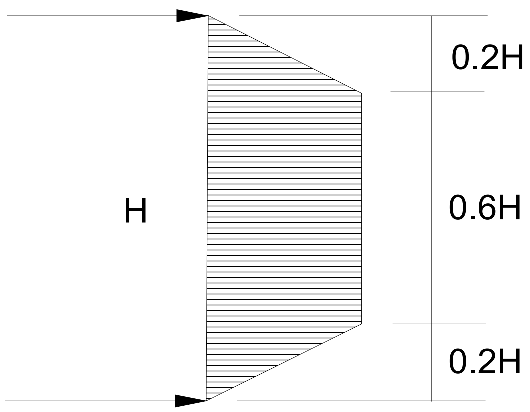
7.17.16 Due to the nature of the site soils, it is expected that continuous lagging between soldier piles will be required. However, it is recommended that the exposed soils be observed by the Geotechnical Engineer (a representative of Geocon West, Inc.), to verify the presence of any competent, cohesive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

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

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

HEIGHT OF SHORING (FEET)	**EQUIVALENT FLUID PRESSURE (Pounds Per Cubic Foot) (ACTIVE PRESSURE)**	**EQUIVALENT FLUID PRESSURE (Pounds Per Square Foot per Foot) Active Trapezoidal (Where H is the height of the shoring in feet)**
Up to 60	36	23H

Trapezoidal Distribution of Pressure



7.17.19 It is very important to note that active pressures can only be achieved when movement in the soil (earth wall) occurs. If movement in the soil is not acceptable, such as adjacent to an existing structure, an at-rest pressure of 52 pcf should be considered for design purposes.

7.17.20 Where a combination of sloped embankment and shoring is utilized, the pressure will be greater and must be determined for each combination. Additional active pressure should be added for a surcharge condition due to slopes, vehicular traffic or adjacent structures and should be designed for each condition.

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

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

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

and

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

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

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

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

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

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

and

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

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

then

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

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

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

7.17.24 It is difficult to accurately predict the amount of deflection of a shored embankment. It should be realized that some deflection will occur. It is recommended that the deflection be minimized to prevent damage to existing structures and adjacent improvements. Where public right-of-ways are present or adjacent offsite structures do not surcharge the shoring excavation, the shoring deflection should be limited to less than 1 inch at the top of the shored embankment. Where offsite structures are within the shoring surcharge area it is recommended that the beam deflection be limited to less than ½ inch at the elevation of the adjacent offsite foundation, and no deflection at all if deflections will damage existing structures. The allowable deflection is dependent on many factors, such as the presence of structures and utilities near the top of the embankment, and will be assessed and designed by the project shoring engineer.

7.17.25 Because of the depth of the excavation, some means of monitoring the performance of the shoring system is suggested. The monitoring should consist of periodic surveying of the lateral and vertical locations of the tops of all soldier piles and the lateral movement along the entire lengths of selected soldier piles.

7.17.26 Due to the depth of the excavation and proximity to adjacent structures, it is suggested that prior to excavation the existing improvements be inspected to document the present condition. For documentation purposes, photographs should be taken of preconstruction distress conditions and level surveys of adjacent grade and pavement should be considered. During excavation activities, the adjacent structures and pavement should be periodically inspected for signs of distress. In the even that distress or settlement is noted, an investigation should be performed and corrective measures taken so that continued or worsened distress or settlement is mitigated. Documentation and monitoring of the offsite structures and improvements is not the responsibility of the geotechnical engineer.

7.18 Tie-Back Anchors

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

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

- 5 feet below the top of the excavation – 1,200 pounds per square foot
- 15 feet below the top of the excavation – 1,400 pounds per square foot
- 25 feet below the top of the excavation – 1,700 pounds per square foot
- 35 feet below the top of the excavation – 1,850 pounds per square foot
- 45 feet below the top of the excavation – 2,200 pounds per square foot

7.18.3 Depending on the techniques utilized, and the experience of the contractor performing the installation, a maximum allowable friction capacity of 3.5 kips per linear foot for post-grouted anchors (for a minimum 20-foot length beyond the active wedge) may be assumed for design purposes. Only the frictional resistance developed beyond the active wedge should be utilized in resisting lateral loads. Higher capacity assumptions may be acceptable but must be verified by testing.

7.19 Anchor Installation

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

7.20 Anchor Testing

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

7.20.2 At least 10 percent of the anchors should be selected for "quick" 200 percent tests and three additional anchors should be selected for 24-hour 200 percent tests. The purpose of the 200 percent tests is to verify the friction value assumed in design. The anchors should be tested to develop twice the assumed friction value. These tests should be performed prior to installation of additional tiebacks. Where satisfactory tests are not achieved on the initial anchors, the anchor diameter and/or length should be increased until satisfactory test results are obtained.

7.20.3 The total deflection during the 24-hour 200 percent test should not exceed 12 inches. During the 24-hour tests, the anchor deflection should not exceed 0.75 inches measured after the 200 percent test load is applied.

7.20.4 For the "quick" 200 percent tests, the 200 percent test load should be maintained for 30 minutes. The total deflection of the anchor during the 200 percent quick tests should not exceed 12 inches; the deflection after the 200 percent load has been applied should not exceed 0.25 inch during the 30-minute period.

7.20.5 After a satisfactory test, each anchor should be locked-off at the design load. This should be verified by rechecking the load in the anchor. The load should be within 10 percent of the design load. A representative of this firm should observe the installation and testing of the anchors.

7.21 Internal Bracing

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

7.22 Stormwater Infiltration

7.22.1 During the June 11 and 12, 2020, site exploration, borings B4 and B5 were utilized to perform percolation testing. The borings were advanced to the depths listed in the table below. Slotted casings were placed in the borings, and the annular spaces between the casings and excavations were filled with gravel. The borings were then filled with water to pre-saturate the soils. After the saturation period was completed, the casings were refilled with water and percolation test readings were performed after repeated flooding of the cased excavations. Based on the test results, the measured percolation rates and design infiltration rates, for the earth materials encountered, are provided in the following table. These values have been calculated in accordance with the Boring Percolation Test Procedure in the County of Los Angeles Department of Public Works GMED *Guidelines for Geotechnical Investigation and Reporting, Low Impact Development Stormwater Infiltration* (June 2017). Percolation test field datum and calculations of the measured percolation rate and design infiltration rate are provided as Figures 11 and 12.

Boring	Soil Type	Infiltration Depth (ft)	Measured Percolation Rate (in / hour)	Reduction Factor (Rf)	Design Infiltration Rate (in / hour)
B4	Sand (SP)/Silty Sand (SM)	75-90	0.37	2	0.18
B5	Sand (SP)/Silty Sand (SM)	40-50	1.52	2	0.76

7.22.2 Based on the test method utilized (Boring Percolation Test), the reduction factor RF_t may be taken as 2.0 in the infiltration system design. Based on the number of tests performed and consistency of the soils throughout the site, it is suggested that the reduction factor RF_v be taken as 1.0. In addition, provided proper maintenance is performed to minimize long-term siltation and plugging, the reduction factor RF_s may be taken as 1.0. Additional reduction 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.

7.22.3 The results of the percolation testing indicate that the soils at depths between 40 – 50 feet in the above table are conductive to infiltration. It is our opinion that the soil zone encountered at the depth and location as listed in the table above are suitable for infiltration of stormwater.

7.22.4 The results of the percolation testing indicate that soils at depths between 75 – 90 feet listed in the table above are minimally conductive to infiltration. These infiltration rates are considered to be slow and the project civil engineer should evaluate the results and suitability for design.

7.22.5 The infiltration of stormwater and will not induce excessive hydro-consolidation, 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.

7.22.6 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.

7.22.7 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.

7.22.8 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).

7.23 Surface Drainage

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

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

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

7.23.4 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.

7.24 Plan Review

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

LIMITATIONS AND UNIFORMITY OF CONDITIONS

1. The recommendations of this report pertain only to the site investigated and are based upon the assumption that the soil conditions do not deviate from those disclosed in the investigation. If any variations or undesirable conditions are encountered during construction, or if the proposed construction will differ from that anticipated herein, Geocon West, Inc. should be notified so that supplemental recommendations can be given. The evaluation or identification of the potential presence of hazardous or corrosive materials was not part of the scope of services provided by Geocon West, Inc.

2. This report is issued with the understanding that it is the responsibility of the owner, or of his representative, to ensure that the information and recommendations contained herein are brought to the attention of the architect and engineer for the project and incorporated into the plans, and the necessary steps are taken to see that the contractor and subcontractors carry out such recommendations in the field.

3. The findings of this report are valid as of the date of this report. However, changes in the conditions of a property can occur with the passage of time, whether they are due to natural processes or the works of man on this or adjacent properties. In addition, changes in applicable or appropriate standards may occur, whether they result from legislation or the broadening of knowledge. Accordingly, the findings of this report may be invalidated wholly or partially by changes outside our control. Therefore, this report is subject to review and should not be relied upon after a period of three years.

4. The firm that performed the geotechnical investigation for the project should be retained to provide testing and observation services during construction to provide continuity of geotechnical interpretation and to check that the recommendations presented for geotechnical aspects of site development are incorporated during site grading, construction of improvements, and excavation of foundations. If another geotechnical firm is selected to perform the testing and observation services during construction operations, that firm should prepare a letter indicating their intent to assume the responsibilities of project geotechnical engineer of record. A copy of the letter should be provided to the regulatory agency for their records. In addition, that firm should provide revised recommendations concerning the geotechnical aspects of the proposed development, or a written acknowledgement of their concurrence with the recommendations presented in our report. They should also perform additional analyses deemed necessary to assume the role of Geotechnical Engineer of Record.

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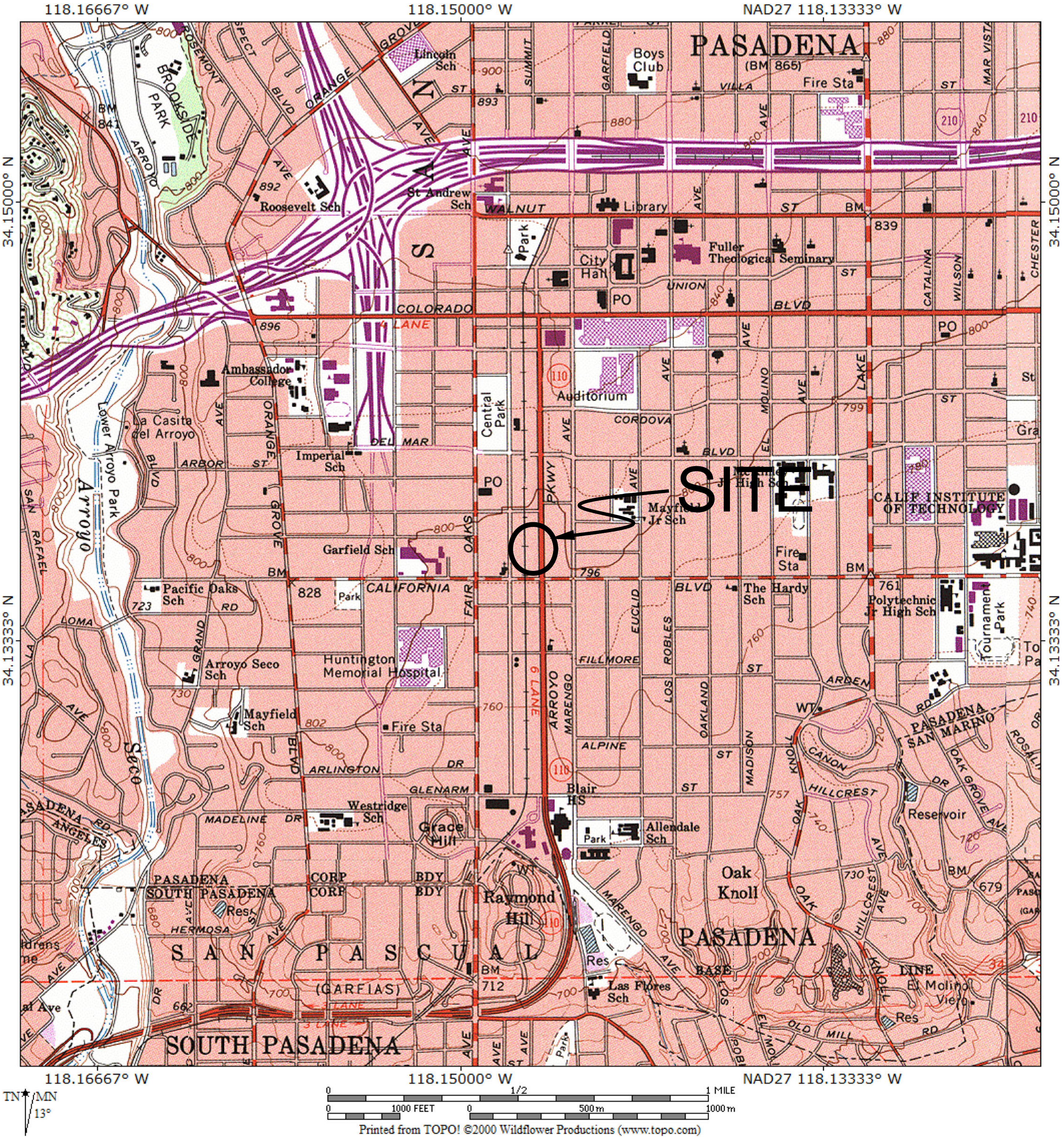

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LATITUDE = 34.137037
LONGITUDE = -118.147802

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

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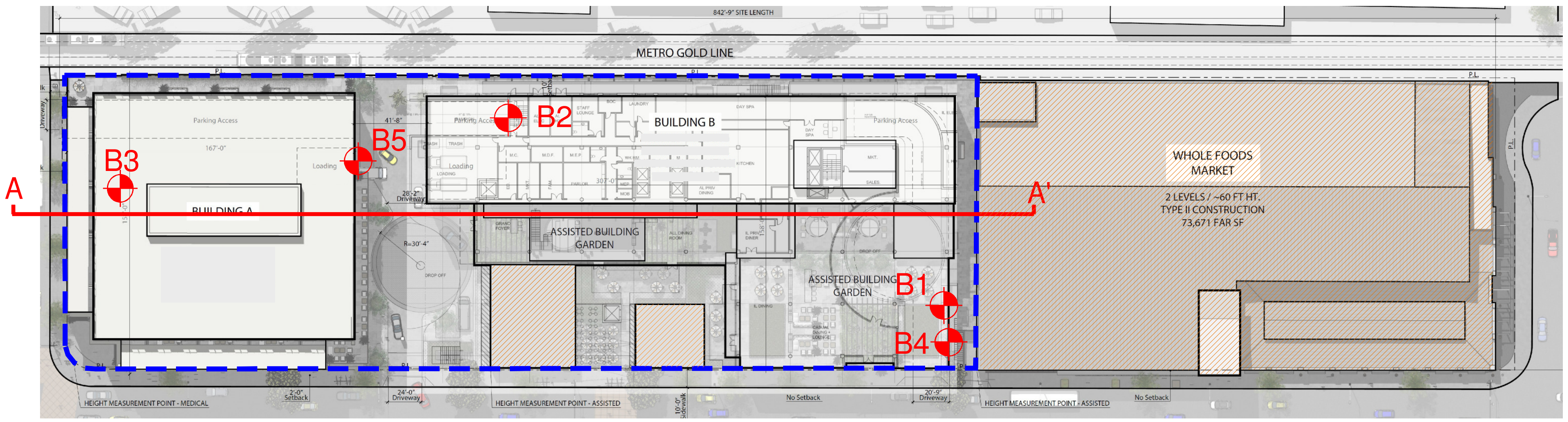
VICINITY MAP

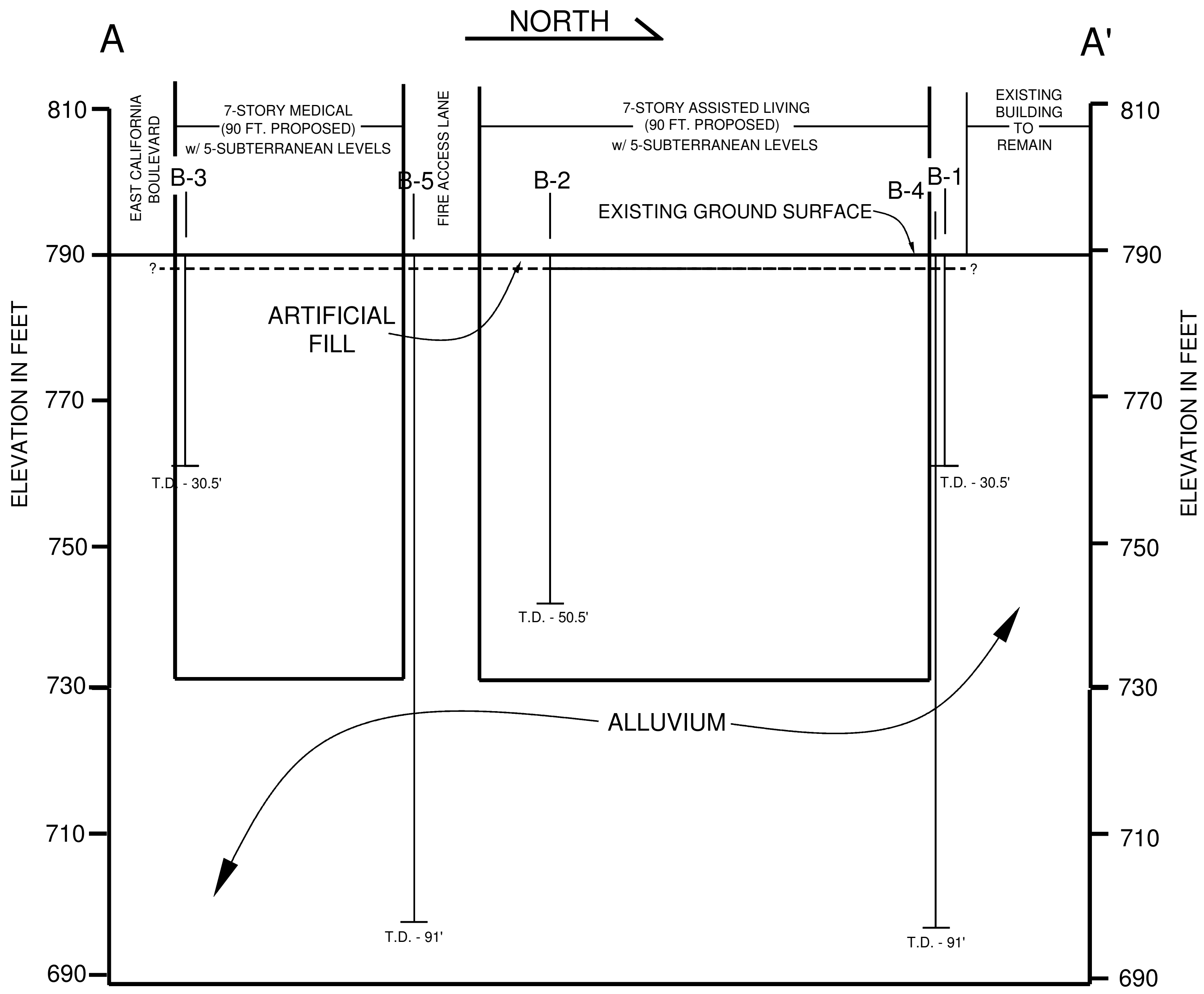
465-577 SOUTH ARROYO PARKWAY
PASADENA, CALIFORNIA

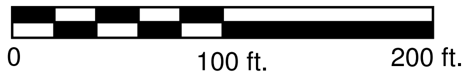
JULY 2021
PROJECT NO. W1111-06-01
FIG. 1

Legend

⊕ B1 Approximate Boring Locations
- - - Limit of Proposed Development
— Geologic Cross Section







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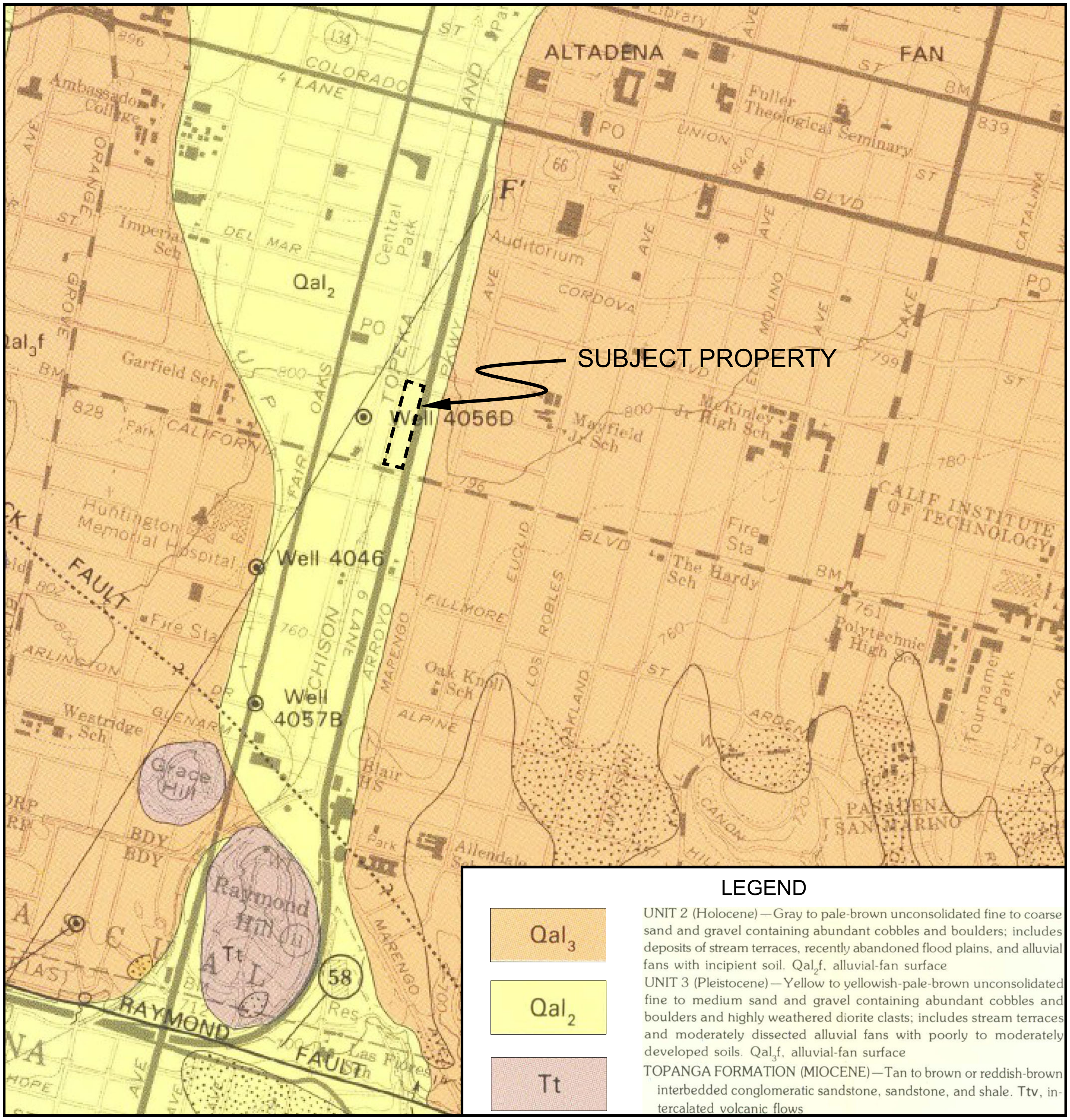
SITE PLAN

465-577 SOUTH ARROYO PARKWAY
PASADENA, CALIFORNIA

JULY 2021	PROJECT NO. W1111-06-01	FIG. 2



LEGEND

Qal$_3$	UNIT 2 (Holocene)—Gray to pale-brown unconsolidated fine to coarse sand and gravel containing abundant cobbles and boulders; includes deposits of stream terraces, recently abandoned flood plains, and alluvial fans with incipient soil. Qal$_2$f, alluvial-fan surface
Qal$_2$	UNIT 3 (Pleistocene)—Yellow to yellowish-pale-brown unconsolidated fine to medium sand and gravel containing abundant cobbles and boulders and highly weathered diorite clasts; includes stream terraces and moderately dissected alluvial fans with poorly to moderately developed soils. Qal$_3$f, alluvial-fan surface
Tt	TOPANGA FORMATION (MIOCENE)—Tan to brown or reddish-brown interbedded conglomeratic sandstone, sandstone, and shale. Ttv, intercalated volcanic flows

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LONGITUDE = -118.147802

0 1500 ft 3000 ft

NORTH

MAP REFERENCE: CROOK ET AL., 1987, PLATE 2.4

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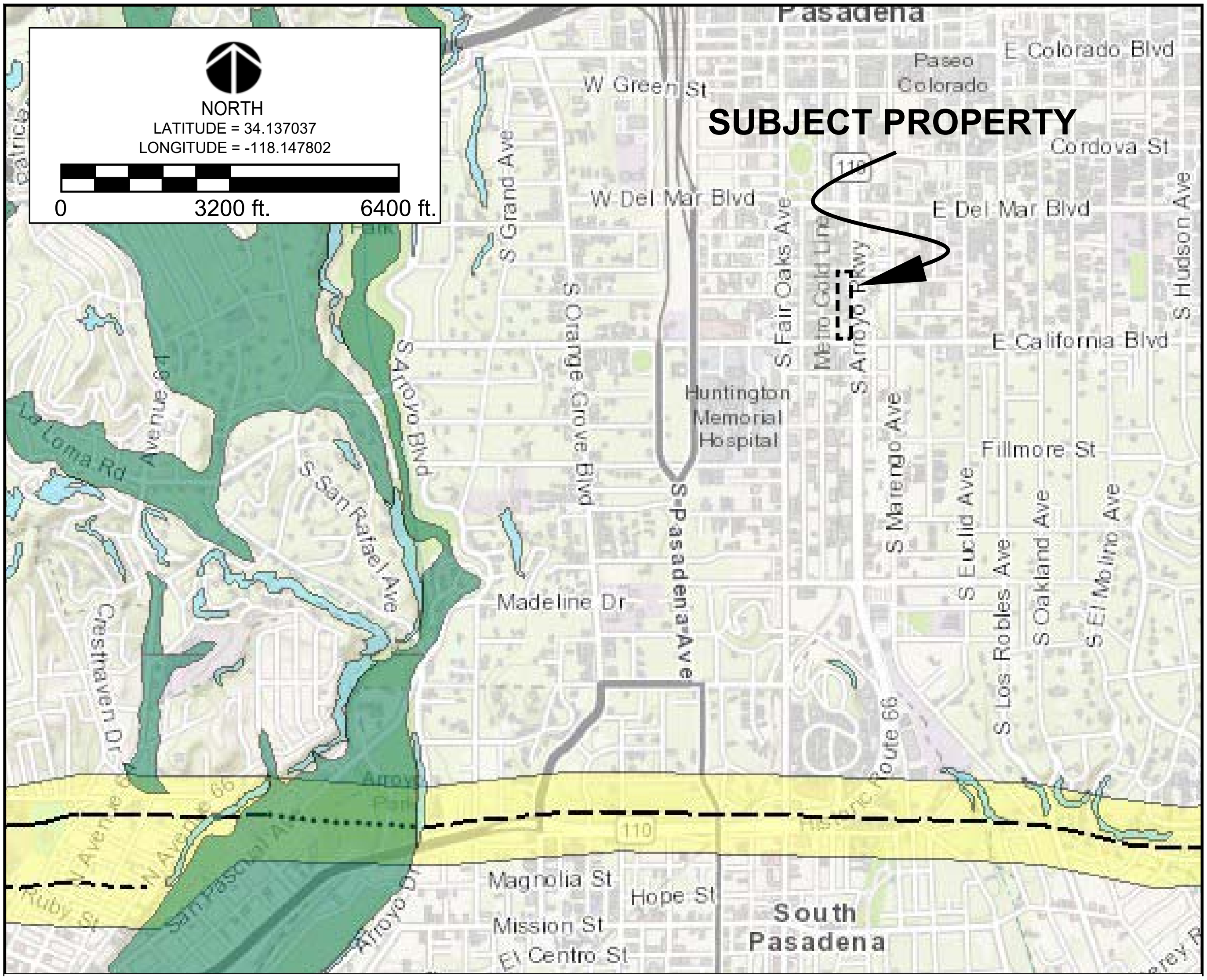
LOCAL GEOLOGIC MAP

465-577 SOUTH ARROYO PARKWAY
PASADENA, CALIFORNIA

JULY 2021
PROJECT NO. W1111-06-01
FIG. 3



MAP REFERENCE: C.G.S., 2020b, EARTHQUAKE ZONES OF REQUIRED INVESTIGATION

ALQUIST-PRIOLO EARTHQUAKE FAULT ZONES

🟨 **Earthquake Fault Zones**
Zone boundaries are delineated by straight-line segments that connect encircled turning points; the boundaries define the zone encompassing active faults that constitute a potential hazard to structures from surface faulting or fault creep such that avoidance as described in Public Resources Code Section 2621.5(a) would be required.

Active Fault Traces
Faults considered to have been active during Holocene time and to have potential for surface rupture; solid line where accurately located, long dash where approximately located, short dash where inferred, dotted where concealed; query (?) indicates additional uncertainty. Evidence of historic offset indicated by year of earthquake-associated event or C for displacement caused by fault creep.

Note: Mitigation methods differ for each zone – AP Act only allows avoidance; Seismic Hazard Mapping Act allows mitigation by engineering/geotechnical design as well as avoidance.

🟩 **Liquefaction Zones**
Areas where historical occurrence of liquefaction, or local geological, geotechnical and ground water conditions indicate a potential for permanent ground displacements such that mitigation as defined in Public Resources Code Section 2693(c) would be required.

🟦 **Earthquake-Induced Landslide Zones**
Areas where previous occurrence of landslide movement, or local topographic, geological, geotechnical and subsurface water conditions indicate a potential for permanent ground displacements such that mitigation as defined in Public Resources Code Section 2693(c) would be required.

Overlap of Earthquake Fault Zone and Liquefaction Zone
Areas that are covered by both Earthquake Fault Zone and Liquefaction Zone.

Overlap of Earthquake Fault Zone and Earthquake-Induced Landslide Zone
Areas that are covered by both Earthquake Fault Zone and Earthquake-Induced Landslide Zone.

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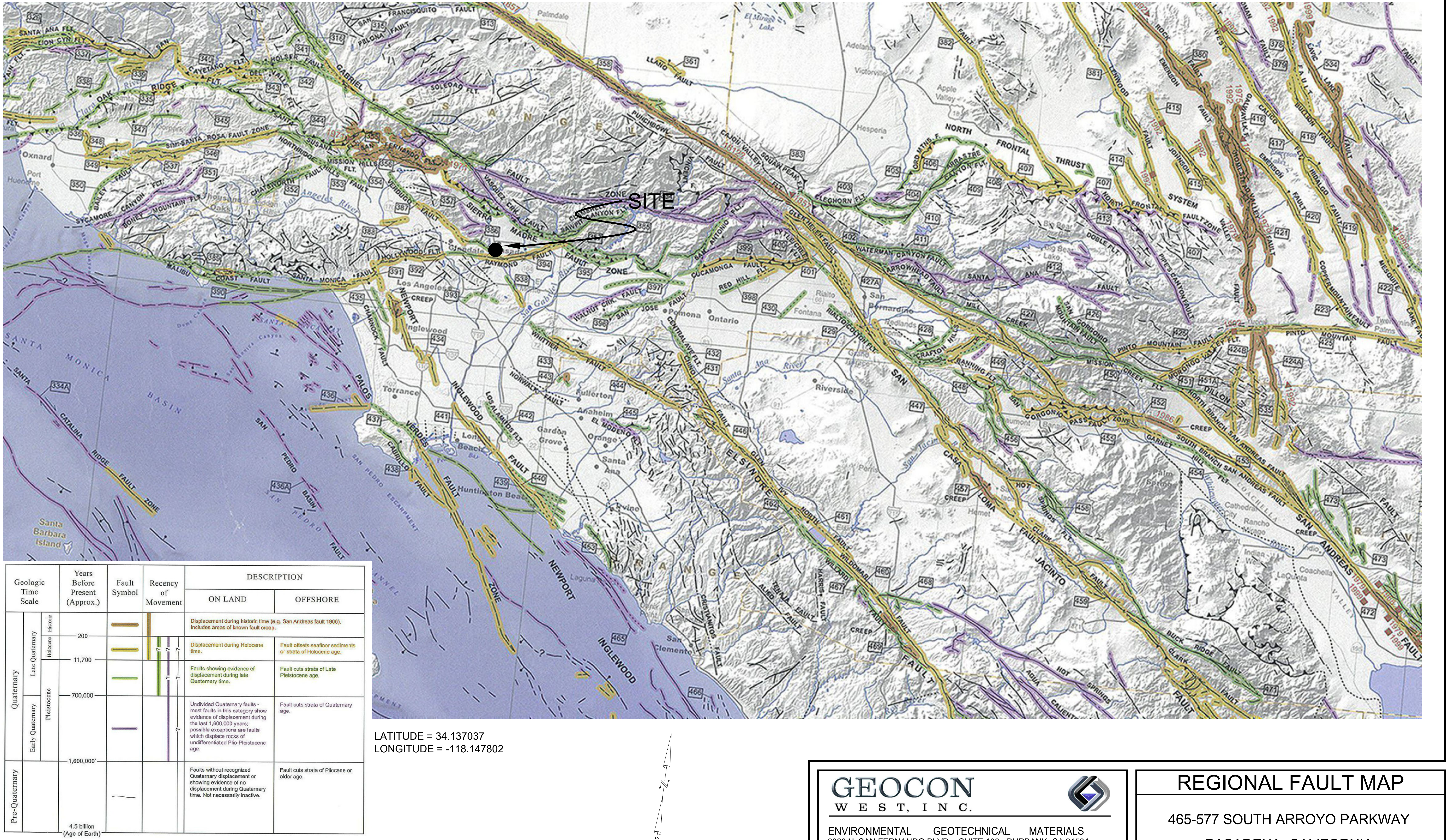
SEISMIC HAZARD ZONES MAP

465-577 SOUTH ARROYO PARKWAY
PASADENA, CALIFORNIA

DRAFTED BY: CB	CHECKED BY: SFK	JULY 2021	PROJECT NO. W1111-06-01	FIG. 4

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	Historic	200	🟫		Displacement during historic time (e.g. San Andreas fault 1906). Includes areas of known fault creep.	
		Holocene	11,700	🟨	? ? ?	Displacement during Holocene time.	Fault offsets seafloor sediments or strata of Holocene age.
		Pleistocene	700,000	🟩	? ?	Faults showing evidence of displacement during late Quaternary time.	Fault cuts strata of Late Pleistocene age.
	Early Quaternary		1,600,000*	🟪	?	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			4.5 billion (Age of Earth)	—		Faults without recognized Quaternary displacement or showing evidence of no displacement during Quaternary time. Not necessarily inactive.	Fault cuts strata of Pliocene or older age.

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

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LONGITUDE = -118.147802

0 12 24 Miles

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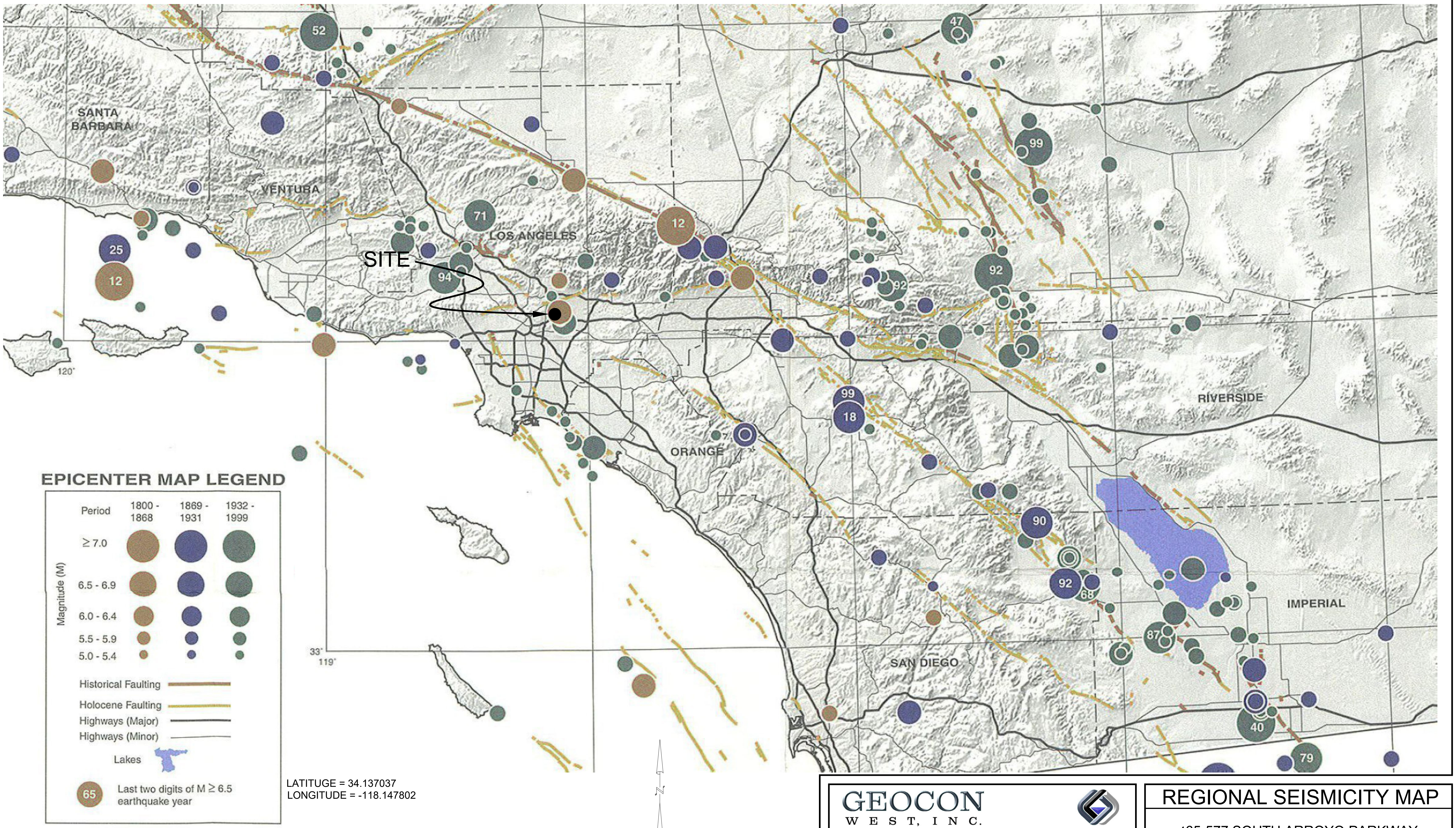
REGIONAL FAULT MAP

465-577 SOUTH ARROYO PARKWAY
PASADENA, CALIFORNIA

JULY 2021 | PROJECT NO. W1111-06-01 | FIG. 5



LATITUDE = 34.137037
LONGITUDE = -118.147802

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

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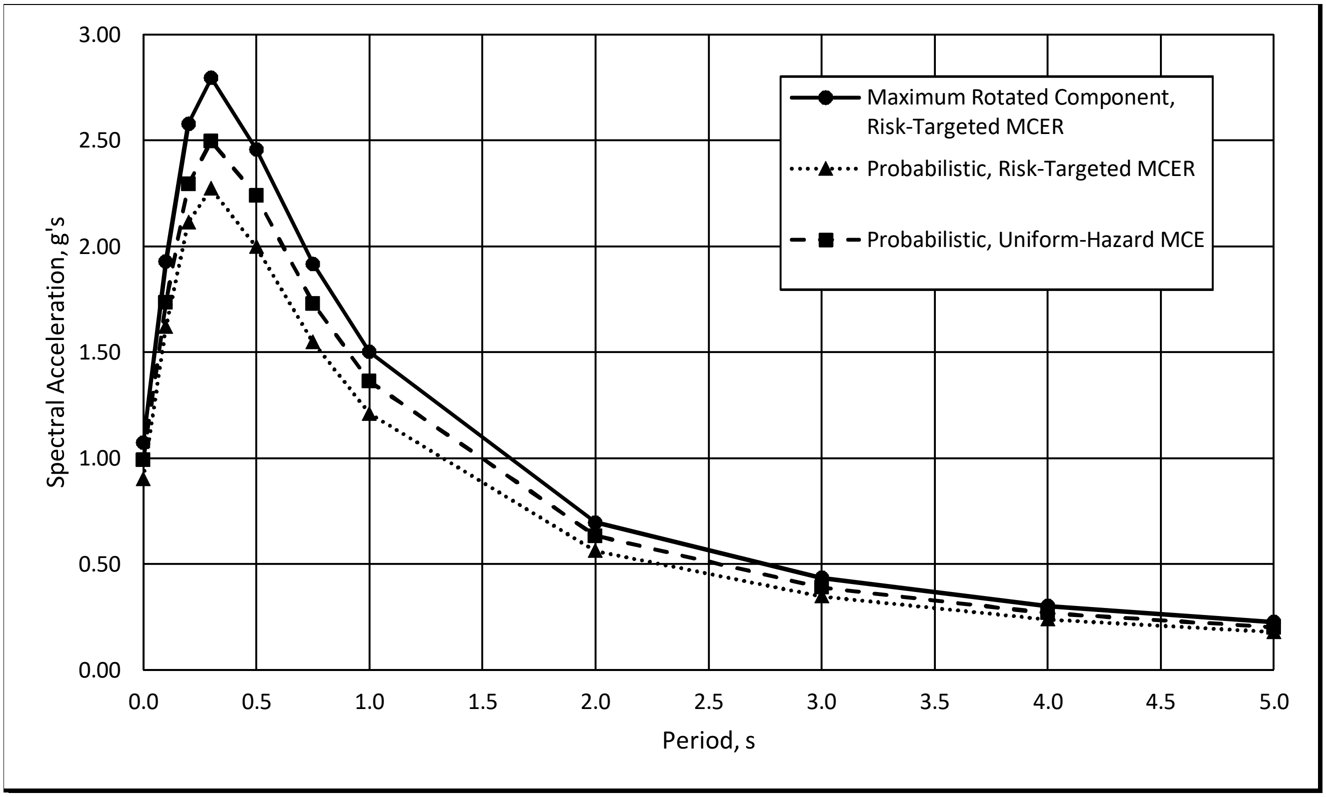
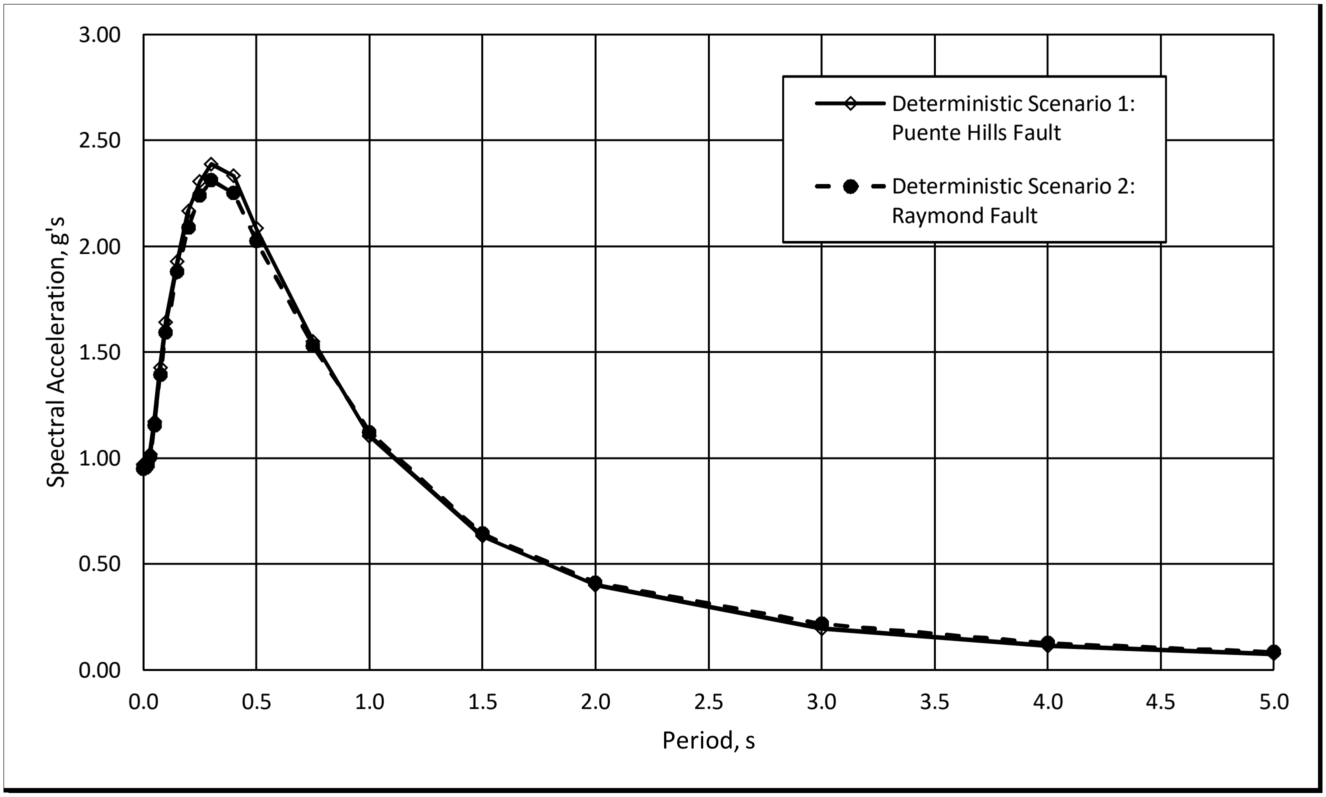
REGIONAL SEISMICITY MAP

465-577 SOUTH ARROYO PARKWAY
PASADENA, CALIFORNIA

JULY 2021	PROJECT NO. W1111-06-01	FIG.6





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DESIGN RESPONSE SPECTRUM

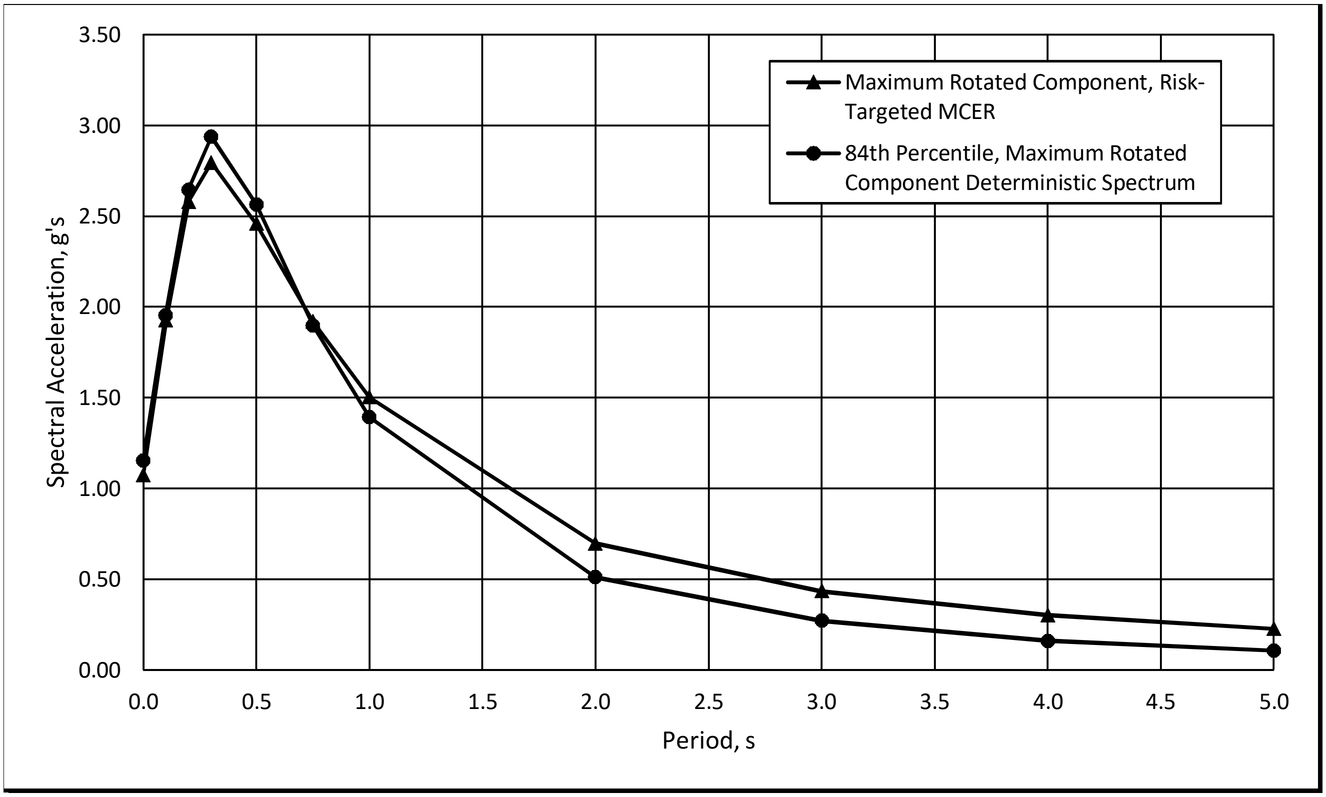
Checked by: JJK

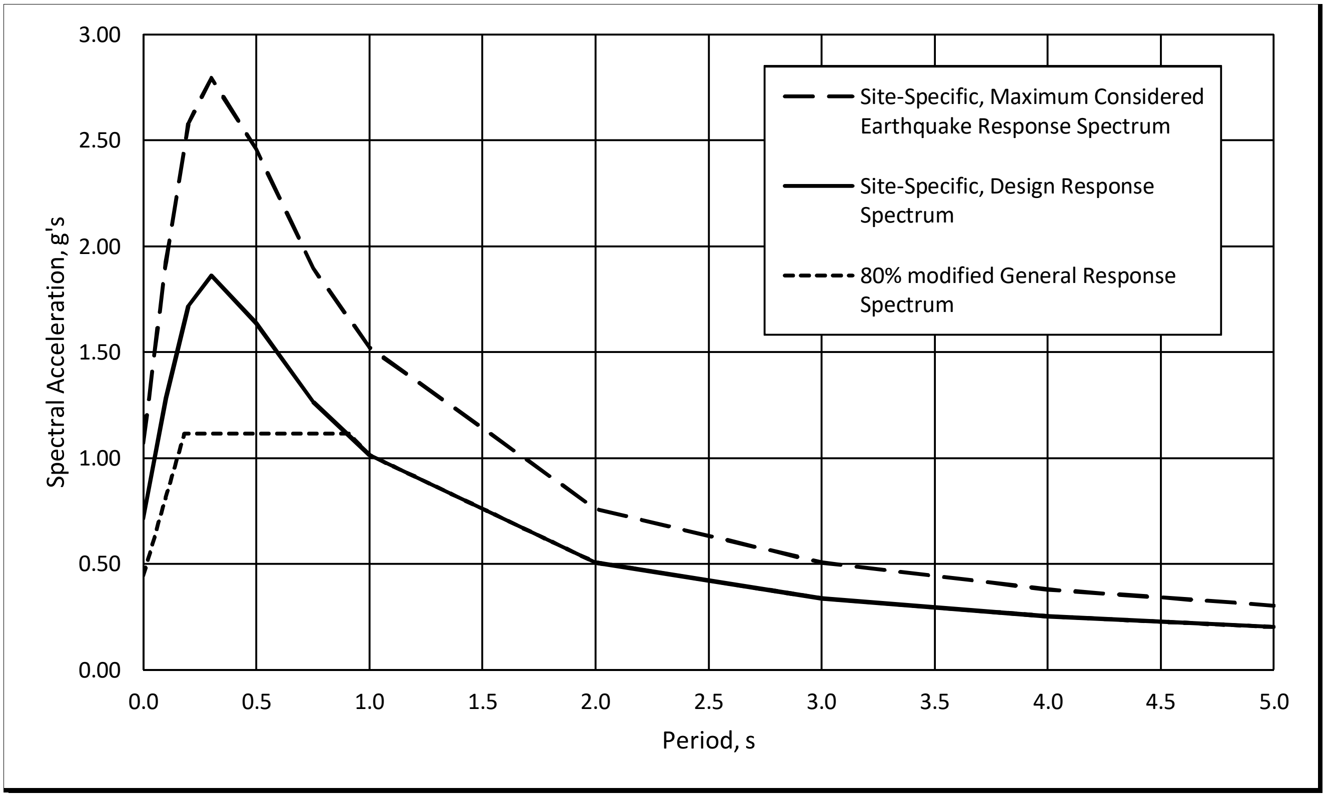
Project No.: W1111-06-01

465-577 ARROYO PARKWAY
PASADENA, CALIFORNIA

JULY 2021

Figure 7





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DESIGN RESPONSE SPECTRUM

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JULY 2021

Figure 8

Spectral Period (seconds)	Probabilistic Uniform-Hazard	Risk-Targeted, Probabilistic	Risk Factor, Cr	Maximum-Rotated Componet Scale Factor	MRC, Risk-Targeted Probablistic	84th Percentile, Deterministic	Site-Specific Design Earthquake	Design Earthquake Floor	Site-Specific Maximum Considered Earthquake
0	0.991	0.902	0.911	1.190	1.074	1.155	0.716	0.447	1.074
0.1	1.735	1.620	0.934	1.190	1.927	1.954	1.285	0.815	1.927
0.2	2.296	2.114	0.921	1.220	2.579	2.645	1.719	1.116	2.579
0.3	2.497	2.272	0.910	1.230	2.795	2.938	1.863	1.116	2.795
0.5	2.239	1.998	0.892	1.230	2.457	2.565	1.638	1.116	2.457
0.75	1.730	1.547	0.894	1.240	1.918	1.897	1.265	1.116	1.897
1	1.364	1.211	0.888	1.240	1.502	1.393	1.015	1.015	1.522
2	0.634	0.562	0.887	1.240	0.697	0.512	0.507	0.507	0.761
3	0.390	0.348	0.892	1.250	0.435	0.273	0.338	0.338	0.507
4	0.269	0.240	0.891	1.260	0.302	0.162	0.254	0.254	0.381
5	0.202	0.180	0.889	1.260	0.227	0.108	0.203	0.203	0.304

SM_S = 2.515 g
SM_1 = 1.522 g

SD_S = 1.677 g
SD_1 = 1.015 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}.

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DESIGN RESPONSE SPECTRUM

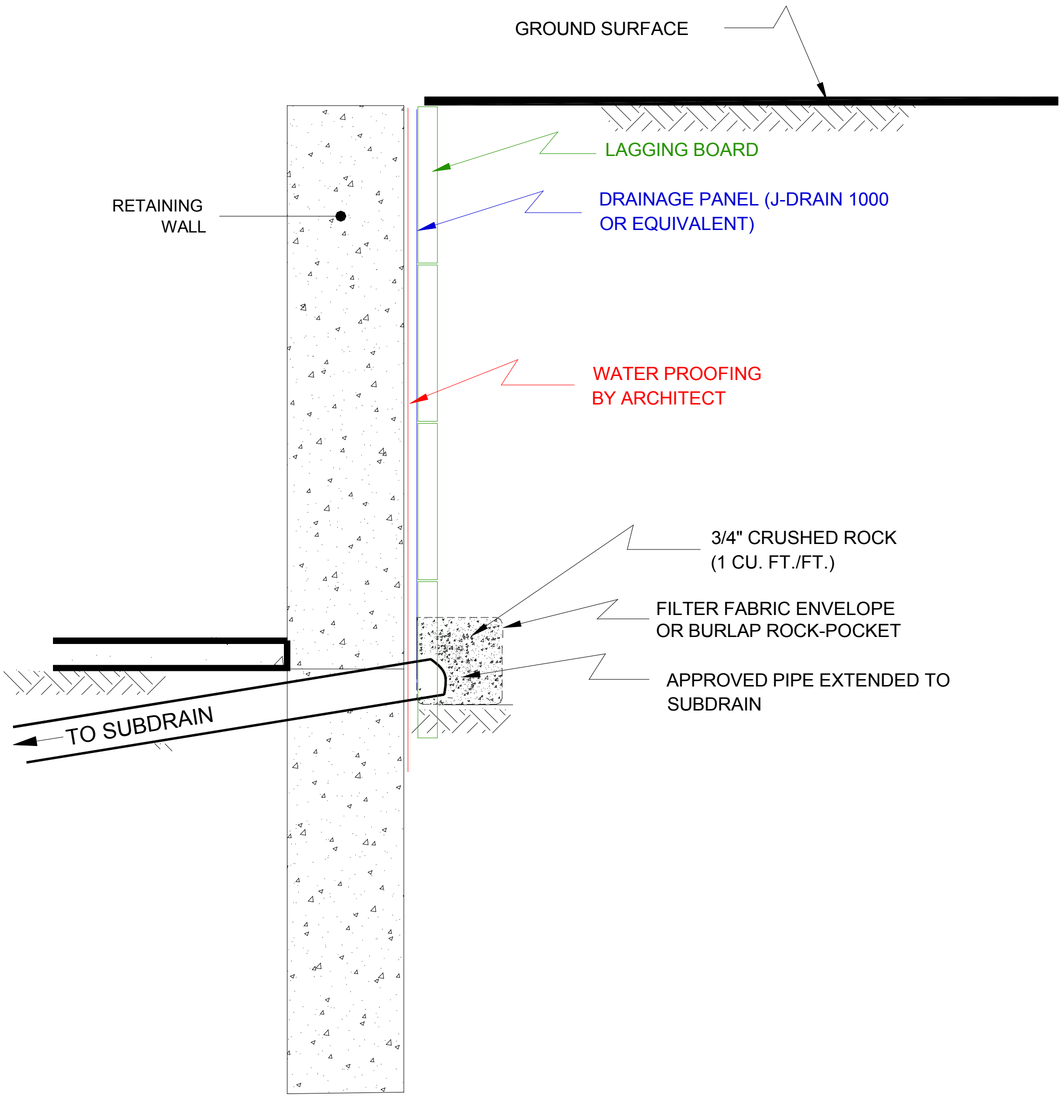
Checked by: JJK

Project No.: W1111-06-01

465-577 ARROYO PARKWAY
PASADENA, CALIFORNIA

JULY 2021

Figure 9



NO SCALE

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REVIEWED BY: NBD

RETAINING WALL DRAIN DETAIL

465-577 ARROYO PARKWAY
PASADENA, CALIFORNIA

JULY 2021
PROJECT NO. W1111-06-01
FIG. 10

BORING PERCOLATION TEST FIELD LOG

Date: Thursday, June 11, 2020
Project Number: W1111-06-01
Project Location: 465-577 S. Arroyo Parkway
Earth Description: SP/SM
Tested By: RP
Liquid Description: Water
Measurement Method: Sounder

Boring/Test Number: B4
Diameter of Boring: 8 inches
Diameter of Casing: 2 inches
Depth of Boring: 91 feet
Depth to Invert of BMP: 60 feet
Depth to Water Table: N/A feet
Depth to Initial Water Depth (d_1): 900 inches

Start Time for Pre-Soak: 10:30 AM
Start Time for Standard: 11:45 AM

Water Remaining in Boring (Y/N): Yes
Standard Time Interval Between Readings: 30

Reading Number	Time Start (hh:mm)	Time End (hh:mm)	Elapsed Time Δtime (min)	Water Drop During Standard Time Interval, Δd (in)	Soil Description Notes Comments
1	11:45 AM	12:15 PM	30	19.0	
2	12:17 PM	12:47 PM	30	19.2	
3	12:50 PM	1:20 PM	30	18.2	
4	1:24 PM	1:54 PM	30	18.1	
5	1:57 PM	2:27 PM	30	18.0	
6	2:30 PM	3:00 PM	30	18.0	Stabilized Readings
7	3:03 PM	3:33 PM	30	17.6	Achieved with Readings
8	3:35 PM	4:05 PM	30	17.5	6, 7, and 8

MEASURED PERCOLATION RATE & DESIGN INFILTRATION RATE CALCULATIONS*

* Calculations Below Based on Stabilized Readings Only

Boring Radius, r: 4 inches
Test Section Height, h: 192.0 inches

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

A = **4876** in^2

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

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

Reading 6	V =	**905**	in^3	Percolation Rate =	**0.37**	inches/hour
Reading 7	V =	**887**	in^3	Percolation Rate =	**0.36**	inches/hour
Reading 8	V =	**881**	in^3	Percolation Rate =	**0.36**	inches/hour

Measured Percolation Rate = **0.37** inches/hour

Reduction Factors

Boring Percolation Test, RF_t = 2
Site Variability, RF_v = 1
Long Term Siltation, RF_s = 1

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

Total Reduction Factor = 2

Design Infiltration Rate

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

Design Infiltration Rate = **0.18** inches/hour

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CHECKED BY: HHD

PERCOLATION TEST RESULTS

465-577 SOUTH ARROYO PARKWAY
PASADENA, CALIFORNIA

JULY 2021
PROJECT NO. W1111-06-01
FIG.11

BORING PERCOLATION TEST FIELD LOG

Date: Friday, June 12, 2020
Project Number: W1111-06-01
Project Location: 465-577 S. Arroyo Parkway
Earth Description: SP/SM
Tested By: RP
Liquid Description: Water
Measurement Method: Sounder

Start Time for Pre-Soak: 7:30 AM
Start Time for Standard: 8:30 AM

Boring/Test Number: B5
Diameter of Boring: 8 inches
Diameter of Casing: 2 inches
Depth of Boring: 50 feet
Depth to Invert of BMP: 10 feet
Depth to Water Table: N/A feet
Depth to Initial Water Depth (d_1): 480 inches

Water Remaining in Boring (Y/N): Yes
Standard Time Interval Between Readings: 30

Reading Number	Time Start (hh:mm)	Time End (hh:mm)	Elapsed Time Δtime (min)	Water Drop During Standard Time Interval, Δd (in)	Soil Description Notes Comments
1	8:56 AM	9:26 AM	30	50.8	
2	9:42 AM	10:12 AM	30	48.0	
3	10:19 AM	10:49 AM	30	48.2	
4	10:53 AM	11:23 AM	30	48.0	
5	11:28 AM	11:58 AM	30	48.4	
6	12:01 PM	12:31 PM	30	43.0	Stabilized Readings
7	12:37 PM	1:07 PM	30	48.1	Achieved with Readings
8	1:14 PM	1:44 PM	30	48.0	6, 7, and 8

MEASURED PERCOLATION RATE & DESIGN INFILTRATION RATE CALCULATIONS*

* Calculations Below Based on Stabilized Readings Only

Boring Radius, r: 4 inches
Test Section Height, h: 120.0 inches

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

A = **3066** in^2

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

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

Reading 6	V =	**2159**	in^3	Percolation Rate =	**1.41**	inches/hour
Reading 7	V =	**2419**	in^3	Percolation Rate =	**1.58**	inches/hour
Reading 8	V =	**2413**	in^3	Percolation Rate =	**1.57**	inches/hour

Measured Percolation Rate = **1.52** inches/hour

Reduction Factors

Boring Percolation Test, RF_t = 2
Site Variability, RF_v = 1
Long Term Siltation, RF_s = 1

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

Total Reduction Factor = 2

Design Infiltration Rate

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

Design Infiltration Rate = **0.76** inches/hour

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PERCOLATION TEST RESULTS

465-577 SOUTH ARROYO PARKWAY
PASADENA, CALIFORNIA

DRAFTED BY: JJK	CHECKED BY: HHD	JULY 2021	PROJECT NO. W1111-06-01	FIG.12

APPENDIX A

APPENDIX A

FIELD INVESTIGATION

The site was explored on January 13, 2020 and June 11, 2020 by excavating five 8-inch-diameter borings to depths of 30½ and 91 feet below the existing ground surface using a truck-mounted hollow-stem auger drilling machine. The borings were excavated to depths of 30½ and 91 feet below the existing ground surface. Representative and relatively undisturbed samples were obtained by driving a 3-inch, O. D., California Modified Sampler into the "undisturbed" soil mass with blows from a 140-pound 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 A5. The logs depict the soil and geologic conditions encountered and the depth at which samples were obtained. The logs also include our interpretation of the conditions between sampling intervals. Therefore, the logs contain both observed and interpreted data. We determined the lines designating the interface between soil materials on the logs using visual observations, penetration rates, excavation characteristics and other factors. The transition between materials may be abrupt or gradual. Where applicable, the logs were revised based on subsequent laboratory testing. The location of the borings are shown on Figure 2.

PROJECT NO. W1111-06-01

BORING 1

ELEV. (MSL.) **--** DATE COMPLETED **01/13/2020**

EQUIPMENT **HOLLOW STEM AUGER** BY: **RMA**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0	BULK 0-5'				**ASPHALT: 3" BASE: NONE** **ARTIFICIAL FILL** Sand, poorly graded, loose, slightly moist, dark brown, fine- to medium-grained.			
2				SP	**ALLUVIUM** Sand, poorly graded, dense, slightly moist, dark yellowish brown, fine- to medium-grained.			
4								
	B1@5'				- some coarse-grained, some gravel (to 3")	67	111.7	8.7
6								
8	B1@7.5'				- medium dense	41	103.8	22.8
10	B1@10'				- dense	67	102.5	22.1
12								
14				SW	Sand, well-graded, very dense, slightly moist, light yellowish brown.			
	B1@15'					50 (4")	113.3	3.8
16								
18								
20	B1@20'					50 (4")	104.3	2.5
22								
				SP	Sand, poorly graded, very dense, slightly moist, light yellowish brown, fine-to medium-grained, some gravel (to 3").			
24								
	B1@25'				- poorly graded, fine- to medium-grained, some gravel (to 4")	50 (5")	108.9	2.4
26								
28								

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

W1111-06-01 BORING LOGS.GPJ

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

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

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PROJECT NO. W1111-06-01

BORING 1

ELEV. (MSL.) **--** DATE COMPLETED **01/13/2020**

EQUIPMENT **HOLLOW STEM AUGER** BY: **RMA**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
30	B1@30'	░		SP	- no recovery	50 (5")		

Total depth of boring: 30.5 feet
Fill to 2 feet.
No groundwater encountered
Backfilled with soil cuttings and tamped.
Asphalt patched.

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

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

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

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

PROJECT NO. W1111-06-01

BORING 2

ELEV. (MSL.) **--** DATE COMPLETED **01/13/2020**

EQUIPMENT **HOLLOW STEM AUGER** BY: **RMA**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0	BULK 0-5'				**ASPHALT: 5" BASE: NONE** **ARTIFICIAL FILL** Silty Sand, poorly graded, medium dense, slightly moist, dark brown.			
2					**ALLUVIUM** Sandy Silt, hard, slightly moist, dark brown.			
4				ML				
	B2@5'					58	110.9	17.1
6					Sand, poorly graded, very dense, slightly moist, light yellowish brown, fine- to medium-grained, trace coarse-grained.			
	B2@7'					50 (5")	113.7	7.6
8								
10	B2@10'				- dense	68	121.1	9.2
12								
14								
	B2@15'				- very dense	50 (3")	109.3	8.8
16								
				SP				
18								
20	B2@20'				- trace gravel (to 3")	50 (6")	113.0	3.8
22								
24								
	B2@25'					50 (3")	120.5	8.8
26								
28								

W1111-06-01 BORING LOGS.GPJ

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

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

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

GEOCON

PROJECT NO. W1111-06-01

BORING 2

ELEV. (MSL.) --   DATE COMPLETED **01/13/2020**

EQUIPMENT **HOLLOW STEM AUGER**   BY: **RMA**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
30	B2@30' ■					50 (5")	116.7	13.3
32								
34								
	B2@35' ■					50 (5")	120.1	14.1
36								
38								
				SP				
40	B2@40' ■					50 (4")	101.7	7.7
42								
44								
	B2@45' ■					50 (6")	113.1	4.6
46								
48								
50	B2@50' ■					50 (3")	120.8	5.7

Total depth of boring: 50.5 feet
Fill to 2 feet.
No groundwater encountered
Backfilled with soil cuttings and tamped.

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

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

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

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

GEOCON

BORING 3

ELEV. (MSL.) -- DATE COMPLETED 01/13/2020

EQUIPMENT HOLLOW STEM AUGER BY: RMA

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0	BULK 0-5'	⊠			**ASPHALT: 3" BASE: NONE** **ARTIFICIAL FILL** Sand, poorly graded, medium dense, slightly moist, grayish brown, fine- to medium-grained, some coarse-grained.			
2								
					ALLUVIUM Silt, stiff, slightly moist, dark brown.			
4				ML				
	B3@5'	■				29	115.4	15.6
6					Sand, poorly graded, dense, slightly moist, light yellowish brown, fine- to medium-grained, some coarse-grained.			
8	B3@8'	■				76	108.0	7.1
10	B3@10'	■			- very dense	50 (5")	106.2	25.9
12								
14								
	B3@15'	■			- no coarse-grained	50 (5")	103.3	10.6
16				SP				
18								
20	B3@20'	■			- dense	61	97.0	20.1
22								
24								
	B3@25'	■			- very dense	50 (5")	112.7	11.1
26								
28								

Figure A3,
Log of Boring 3, Page 1 of 2

SAMPLE SYMBOLS

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

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PROJECT NO. W1111-06-01

BORING 3

ELEV. (MSL.) -- DATE COMPLETED 01/13/2020

EQUIPMENT **HOLLOW STEM AUGER** BY: **RMA**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
30	B3@30' ■	⁚⁚⁚		SP		76	116.3	10.0

Total depth of boring: 30.5 feet
Fill to 2 feet.
No groundwater encountered
Backfilled with soil cuttings and tamped.
Asphalt patched.

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

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

SAMPLE SYMBOLS

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

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

GEOCON

PROJECT NO. W1111-06-01

BORING 4

ELEV. (MSL.) **--** DATE COMPLETED **6/11/2020**

EQUIPMENT **HOLLOW STEM AUGER** BY: **JJK**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0					**ASPHALT: 2.5" BASE: NONE** **ARTIFICIAL FILL** Silty Sand, medium dense, slightly moist, reddish brown, fine- to medium-grained, fine to coarse gravel.			
2								
4				SM	**ALLUVIUM** Silty Sand, medium dense, slightly moist, reddish brown.			
6	B4@5'			SP-SM	Sand with Silt, very dense, slightly moist, brown, fine- to coarse-grained, with fine to coarse gravel.	50 (3")	126.1	4.4
8								
10	B4@10'			SM	Silty Sand, meidum dense, slightly moist, reddish brown and light gray, fine-grained.	50 (5")	108.2	8.8
12								
14								
16	B4@15'			SP	Sand, poorly graded, dense, slightly moist, brown, fine-grained, some coarse-grained, fine to coarse gravel, trace small cobbles (to 3").	77	109.9	6.9
18								
20	B1@20'				- medium- to coarse-grained sand, some coarse gravel, no cobbles	60	114.1	2.3
22								
24								
26	B4@25'				- very dense, some fine to coarse gravel	50 (6")	114.6	5.1
28								

W1111-06-01 BORING LOGS.GPJ

Figure A4,
Log of Boring 4, **Page 1 of 4**

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

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

GEOCON

PROJECT NO. W1111-06-01

BORING 4

ELEV. (MSL.) **--** DATE COMPLETED **6/11/2020**

EQUIPMENT **HOLLOW STEM AUGER** BY: **JJK**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
30	B4@30'			SP	- slightly moist to moist, fine-grained, trace fine to coarse gravel	50 (6")	103.8	3.6
32				SP-SM	Sand with Silt, very dense, slightly moist, brown, medium- to coarse-grained, trace coarse gravel and cobbles (to 4").			
34								
	B4@35'					50 (6")	114.6	2.4
36								
38				SP	Sand, poorly graded, very dense, slightly moist, reddish brown, fine- to medium-grained, some coarse-grained and fine gravel.			
40	B4@40'					50 (6")	111.2	3.7
42								
44								
	B4@45'				- fine- to coarse-grained, some fine gravel, trace coarse gravel	50 (4")	112.1	2.1
46								
48								
50	B4@50' BULK 50-55'				- fine-grained, trace fine gravel and small cobbles (4-5")	50 (5")	111.9	3.8
52								
54								
56								
58								

W1111-06-01 BORING LOGS.GPJ

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

SAMPLE SYMBOLS

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

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PROJECT NO. W1111-06-01

BORING 4

ELEV. (MSL.) -- DATE COMPLETED 6/11/2020

EQUIPMENT HOLLOW STEM AUGER BY: JJK

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
60	B4@60'			SP	- some medium- to coarse-grained and fine gravel, trace small cobbles	50 (4.5")	120.3	2.8
62								
64								
66								
				GM	Silty Gravel, very dense, slightly moist to moist, reddish brown, fine to medium gravel, trace small cobbles.			
68								
70	B4@70'					50 (3")	117.3	9.3
72								
74								
76								
78								
				SM	Silty Sand, very dense, moist, light reddish brown, fine-grained.			
80	B4@80'					50 (6")	101.3	16.9
82				SP	Sand, poorly graded, very dense, slightly moist, brown.			
84								
86								
88								

Figure A4,
Log of Boring 4, Page 3 of 4

SAMPLE SYMBOLS

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

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PROJECT NO. W1111-06-01

BORING 4

ELEV. (MSL.) **--** DATE COMPLETED **6/11/2020**

EQUIPMENT **HOLLOW STEM AUGER** BY: **JJK**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
90	B4@90' ■	░		SP	- refusal at 91'	50 (5")	114.4	10.5

Total depth of boring: 91 feet
Fill to 2 feet.
No groundwater encountered.
Backfilled with soil cuttings and tamped.
Patched with cold patch asphalt.

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

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

W1111-06-01 BORING LOGS.GPJ

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

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GEOCON

BORING 5

ELEV. (MSL.) -- DATE COMPLETED 6/11/2020

EQUIPMENT HOLLOW STEM AUGER BY: JJK

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0					**ASPHALT: 3" BASE: NONE** **ARTIFICIAL FILL** Silty Sand, medium dense, slightly moist to moist, dark brown.			
2				SM	**ALLUVIUM** Silty Sand, medium dense, slightly moist, reddish brown.			
4				GM	Silty Gravel, dense, slightly moist, brown, fine grave, some coarse gravel and small cobbles.			
	B5@5'					60	123.1	7.9
10	B5@10'				- very dense	50 (6")	121.0	6.7
12				SM	Silty Sand, dense, slightly moist, reddish brown and olive gray, fine-grained.			
	B5@15'					77	117.2	16.3
				SP	Sand, poorly graded, dense, dry to slightly moist, brown, fine-grained, some coarse-grained and fine to coarse gravel.			
20	B5@20'					86	120.2	2.6
	B5@25'				- reddish brown, fine-grained, no medium- to coarse-grained sand or fine gravel	75	104.6	6.0
28				SM	Silty Sand, very dense, moist, reddish brown and gray, fine-grained, some medium- to coarse-grained.			

Figure A5,
Log of Boring 5, Page 1 of 4

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

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GEOCON

BORING 5

ELEV. (MSL.) **--** DATE COMPLETED **6/11/2020**

EQUIPMENT **HOLLOW STEM AUGER** BY: **JJK**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
30	B5@30' ■			SM		50 (6")	126.4	13.3
32								
				SP	Sand, poorly graded, very dense, dry to slightly moist, brown, fine- to coarse-grained, some fine gravel.			
34								
	B5@35' ■					50 (5.5")	119.1	2.3
36								
38								
40	B5@40' ■				- trace small cobbles (3")	50 (5")	117.3	3.9
42								
44								
	B5@45' ■				- coarse-grained with fine gravel, some coarse gravel, trace silt	50 (6")	113.9	3.2
46								
48								
50	B5@50' ■				- fine- to coarse-grained, trace cobbles (3-4")	50 (6")	126.5	2.7
52								
54								
56								
58								

Figure A5,
Log of Boring5, Page 2 of 4

SAMPLE SYMBOLS

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

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PROJECT NO. W1111-06-01

BORING 5

ELEV. (MSL.) **--** DATE COMPLETED **6/11/2020**

EQUIPMENT **HOLLOW STEM AUGER** BY: **JJK**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
60	B5@60' ■				Sandy Silt, dense, slightly moist, reddish brown, some medium- to coarse-grained sand.	86	106.2	27.2
62				ML				
64					Sand, very dense, dry to slightly moist, reddish brown and gray, fine-grained.			
66								
68				SP				
70	B5@70' ■					50 (6")	99.6	6.0
72								
74								
					Silty Sand, very dense, moist, reddish brown and gray, fine-grained, some coarse-grained and fine gravel.			
76								
78				SM				
80	B5@80' ■					50 (6")	106.3	19.7
82								
					Sand, very dense, dry, brown, fine-grained with fine gravel, trace small cobbles.			
84								
86				SP				
88								

Figure A5,
Log of Boring 5, Page 3 of 4

W1111-06-01 BORING LOGS.GPJ

SAMPLE SYMBOLS

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

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GEOCON

PROJECT NO. W1111-06-01

BORING 5

ELEV. (MSL.) **--** DATE COMPLETED **6/11/2020**

EQUIPMENT **HOLLOW STEM AUGER** BY: **JJK**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
90	B5@90' ■	░		SP		50 (5")	111.2	4.9
					Total depth of boring: 90 feet Fill to 2 feet. No groundwater encountered. *Penetration resistance for 140-pound hammer falling 30 inches by auto-hammer.			

W1111-06-01 BORING LOGS.GPJ

Figure A5,
Log of Boring 5, Page 4 of 4

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

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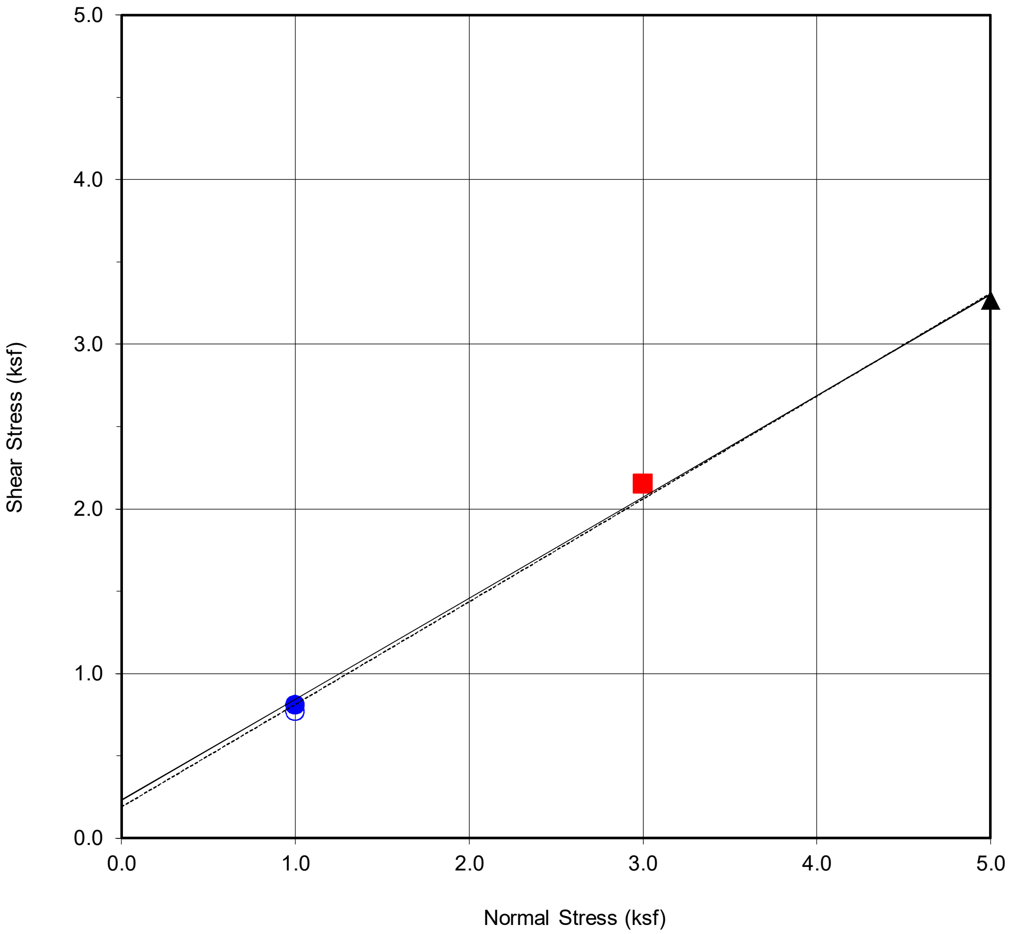
GEOCON

APPENDIX B

APPENDIX B

LABORATORY TESTING

Laboratory tests were performed in accordance with generally accepted test methods of the International ASTM, or other suggested procedures. Selected samples were tested for direct shear strength, consolidation and expansion characteristics, maximum dry density, corrosivity, in-place dry density and moisture content. The results of the laboratory tests are summarized in Figures B1 through B39. The in-place dry density and moisture content of the samples tested are presented on the boring logs, Appendix A.



Boring No.	**B1 + B2 + B3**
Sample No.	**Combined @ 0-5'**
Depth (ft)	**0-5'**
Sample Type:	Ring

Soil Identification:		
Brown Silty Sand (SM)		
Strength Parameters		
	C (psf)	ϕ (°)
Peak	229	31.6
Ultimate	189	32.0

Normal Stress (kip/ft2)	1	3	5
Peak Shear Stress (kip/ft²)	● 0.81	■ 2.15	▲ 3.27
Shear Stress @ End of Test (ksf)	○ 0.77	□ 2.15	△ 3.27
Deformation Rate (in./min.)	0.05	0.05	0.05
Initial Sample Height (in.)	1.0	1.0	1.0
Ring Inside Diameter (in.)	2.375	2.375	2.375
Initial Moisture Content (%)	8.3	8.4	8.4
Initial Dry Density (pcf)	117.0	117.0	117.0
Initial Degree of Saturation (%)	51.2	51.5	51.5
Soil Height Before Shearing (in.)	1.2	1.2	1.2
Final Moisture Content (%)	13.6	12.2	11.4

GEOCON

DIRECT SHEAR TEST RESULTS

Consolidated Drained ASTM D-3080

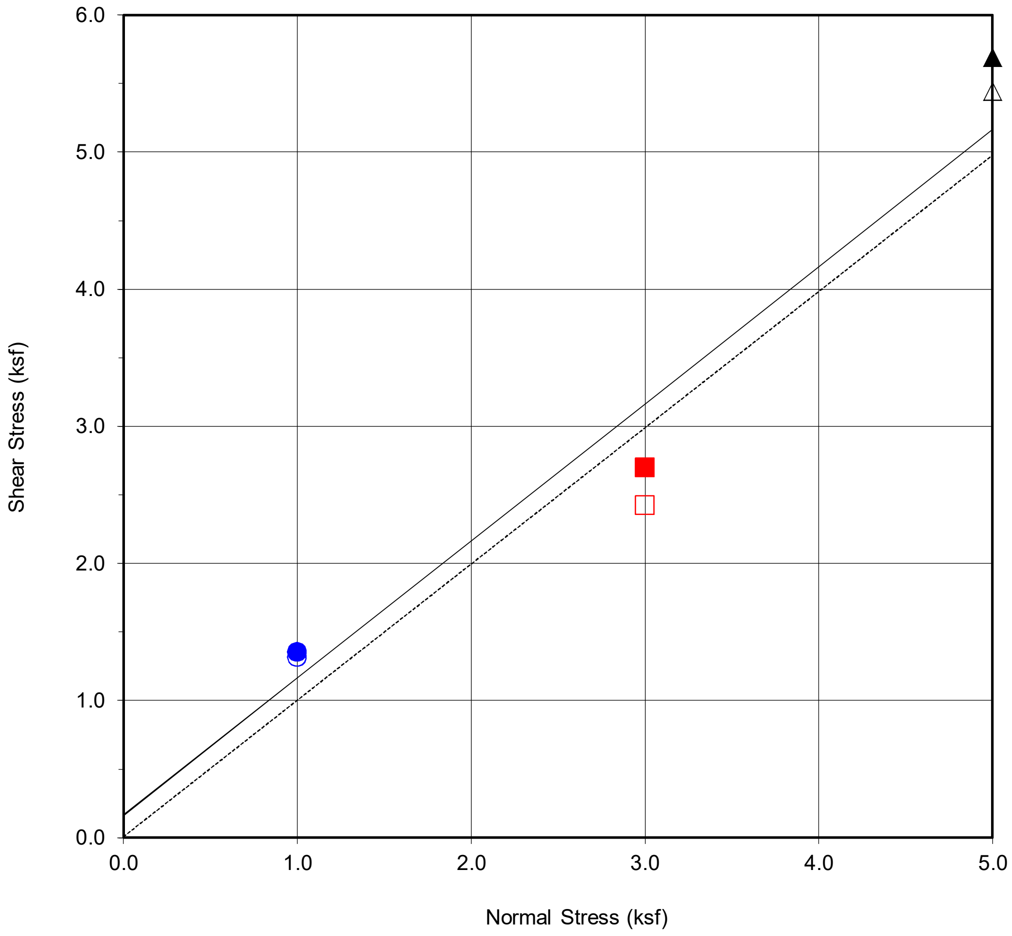
Checked by: JJK

Project No.: W1111-06-01

465-577 SOUTH ARROYO PARKWAY
PASADENA, CALIFORNIA

JULY 2021

Figure B1



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

Soil Identification:		
Light Brown Poorly Graded Sand (SP)		
Strength Parameters		
	C (psf)	ϕ (°)
Peak	165	45.0
Ultimate	7	44.8

Normal Stress (kip/ft2)	1	3	5
Peak Shear Stress (kip/ft²)	● 1.35	■ 2.70	▲ 5.69
Shear Stress @ End of Test (ksf)	○ 1.32	□ 2.42	△ 5.44
Deformation Rate (in./min.)	0.05	0.05	0.05
Initial Sample Height (in.)	1.0	1.0	1.0
Ring Inside Diameter (in.)	2.375	2.375	2.375
Initial Moisture Content (%)	10.4	9.9	8.7
Initial Dry Density (pcf)	107.0	109.1	112.7
Initial Degree of Saturation (%)	48.8	49.1	47.5
Soil Height Before Shearing (in.)	1.2	1.2	1.2
Final Moisture Content (%)	15.9	14.4	13.1

GEOCON

DIRECT SHEAR TEST RESULTS

Consolidated Drained ASTM D-3080

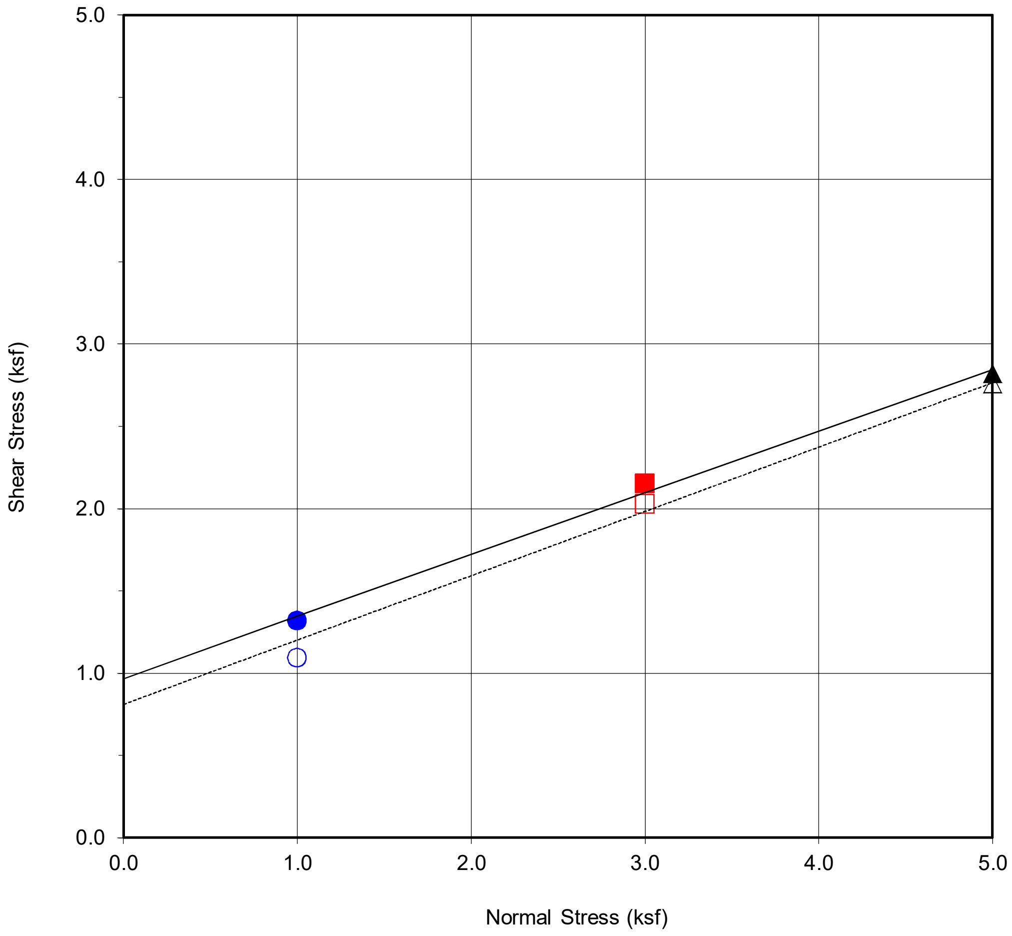
Checked by: JJK

Project No.: W1111-06-01

465-577 SOUTH ARROYO PARKWAY
PASADENA, CALIFORNIA

JULY 2021

Figure B2



Boring No.	**B2**
Sample No.	**B2@5'**
Depth (ft)	**5**
Sample Type:	Ring

Soil Identification:

Brown Sandy Silt (ML)

Strength Parameters

	C (psf)	ϕ (°)
Peak	967	20.6
Ultimate	810	21.4

Normal Stress (kip/ft2)	1	3	5
Peak Shear Stress (kip/ft²)	● 1.32	■ 2.15	▲ 2.82
Shear Stress @ End of Test (ksf)	○ 1.09	□ 2.02	△ 2.76
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 (%)	16.0	16.4	17.1
Initial Dry Density (pcf)	110.4	108.5	114.5
Initial Degree of Saturation (%)	82.1	80.3	97.6
Soil Height Before Shearing (in.)	1.2	1.2	1.2
Final Moisture Content (%)	19.3	18.7	17.6

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DIRECT SHEAR TEST RESULTS

Consolidated Drained ASTM D-3080

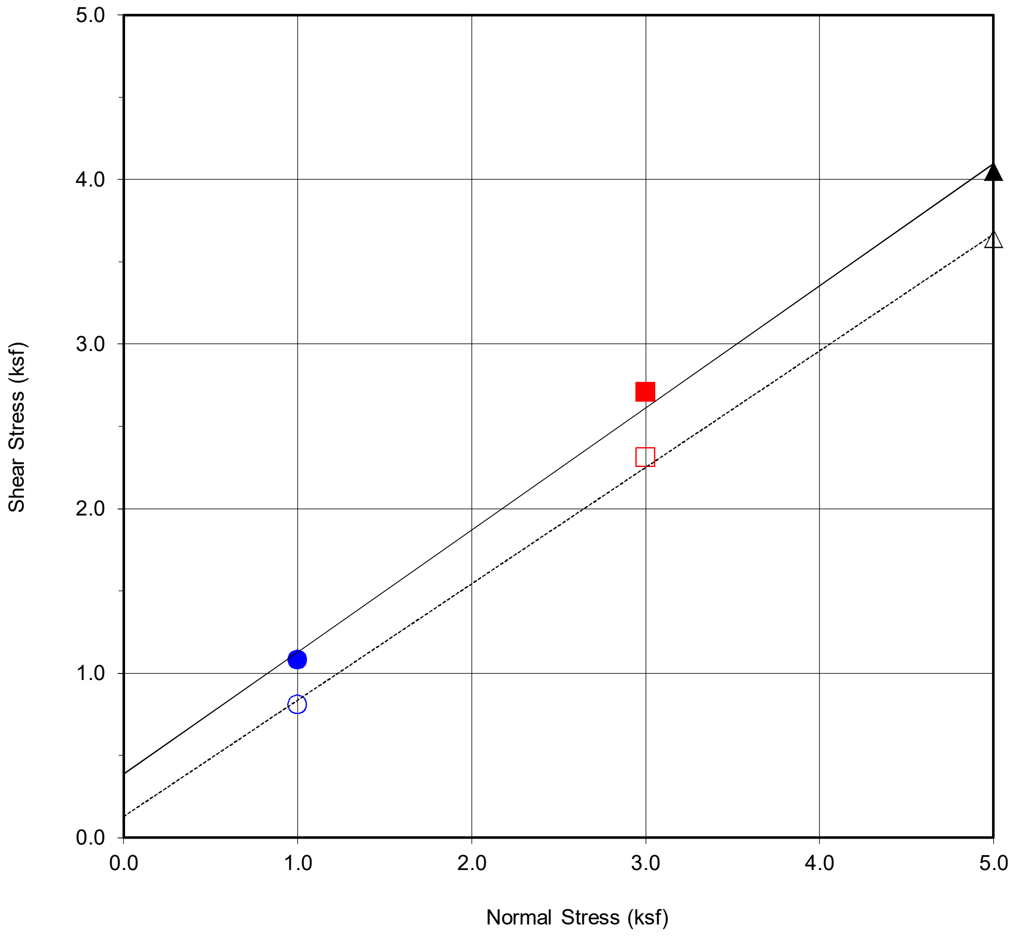
Checked by: JJK

Project No.: W1111-06-01

465-577 SOUTH ARROYO PARKWAY
PASADENA, CALIFORNIA

JULY 2021

Figure B3



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

Soil Identification:		
Brown Silty Sand (SM)		
Strength Parameters		
	C (psf)	ϕ (°)
Peak	386	36.6
Ultimate	128	35.3

Normal Stress (kip/ft2)	1	3	5
Peak Shear Stress (kip/ft²)	● 1.08	■ 2.71	▲ 4.05
Shear Stress @ End of Test (ksf)	○ 0.81	□ 2.31	△ 3.64
Deformation Rate (in./min.)	0.05	0.05	0.05
Initial Sample Height (in.)	1.0	1.0	1.0
Ring Inside Diameter (in.)	2.375	2.375	2.375
Initial Moisture Content (%)	15.6	15.8	15.9
Initial Dry Density (pcf)	114.1	113.9	115.0
Initial Degree of Saturation (%)	88.0	88.8	92.0
Soil Height Before Shearing (in.)	1.2	1.2	1.2
Final Moisture Content (%)	14.8	14.5	14.0

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DIRECT SHEAR TEST RESULTS

Consolidated Drained ASTM D-3080

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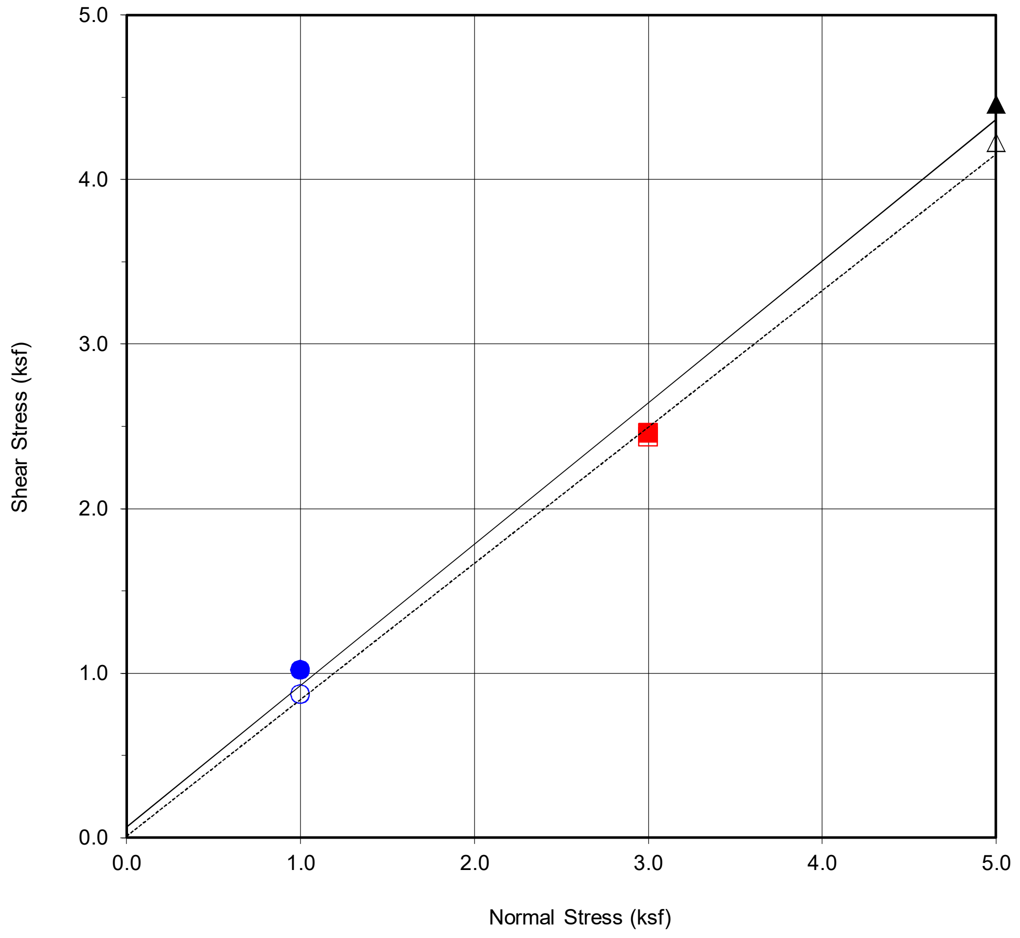
Project No.: W1111-06-01

465-577 SOUTH ARROYO PARKWAY
PASADENA, CALIFORNIA

JULY 2021

Figure B4



Boring No.	**B4**
Sample No.	**B4@15'**
Depth (ft)	**15'**
Sample Type:	Ring

Soil Identification:

Brown Poorly Graded Sand (SP)

Strength Parameters

	C (psf)	ϕ (°)
Peak	65	40.7
Ultimate	9	39.7

Normal Stress (kip/ft2)	1	3	5
Peak Shear Stress (kip/ft²)	● 1.02	■ 2.46	▲ 4.46
Shear Stress @ End of Test (ksf)	○ 0.87	□ 2.43	△ 4.22
Deformation Rate (in./min.)	0.05	0.05	0.05
Initial Sample Height (in.)	1.0	1.0	1.0
Ring Inside Diameter (in.)	2.375	2.375	2.375
Initial Moisture Content (%)	10.7	6.9	7.6
Initial Dry Density (pcf)	105.6	111.2	105.7
Initial Degree of Saturation (%)	48.4	36.1	34.7
Soil Height Before Shearing (in.)	1.2	1.2	1.2
Final Moisture Content (%)	22.4	15.9	16.1

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DIRECT SHEAR TEST RESULTS

Consolidated Drained ASTM D-3080

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465-577 SOUTH ARROYO PARKWAY
PASADENA, CALIFORNIA

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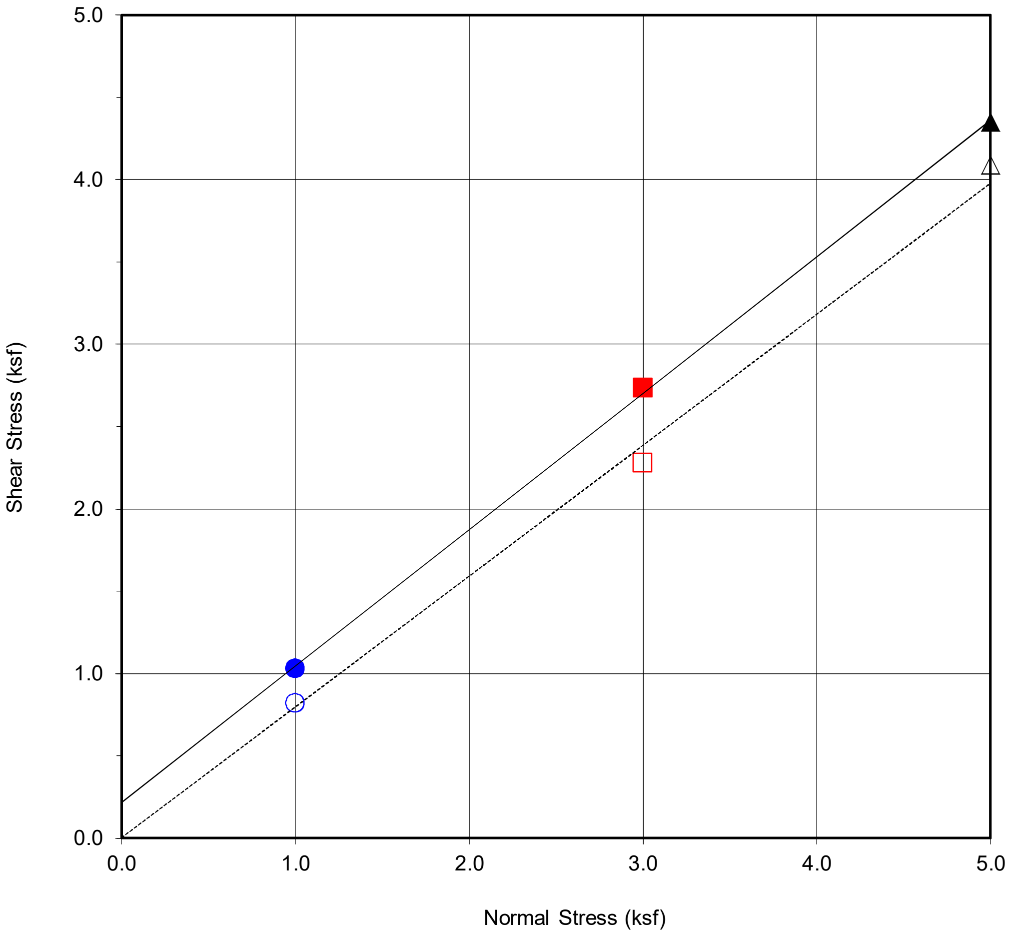
Figure B5



Boring No.	**B4**
Sample No.	**B4@30'**
Depth (ft)	**30'**
Sample Type:	Ring

Soil Identification:		
Brown Poorly Graded Sand (SP)		
Strength Parameters		
	C (psf)	ϕ (^{o})
Peak	217	39.7
Ultimate	3	38.5

Normal Stress (kip/ft2)	1	3	5
Peak Shear Stress (kip/ft²)	● 1.03	■ 2.73	▲ 4.35
Shear Stress @ End of Test (ksf)	○ 0.82	□ 2.17	△ 4.09
Deformation Rate (in./min.)	0.05	0.05	0.05
Initial Sample Height (in.)	1.0	1.0	1.0
Ring Inside Diameter (in.)	2.375	2.375	2.375
Initial Moisture Content (%)	8.2	7.3	7.7
Initial Dry Density (pcf)	100.2	103.8	103.0
Initial Degree of Saturation (%)	32.6	31.6	32.7
Soil Height Before Shearing (in.)	1.2	1.2	1.2
Final Moisture Content (%)	19.5	18.4	17.9

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DIRECT SHEAR TEST RESULTS

Consolidated Drained ASTM D-3080

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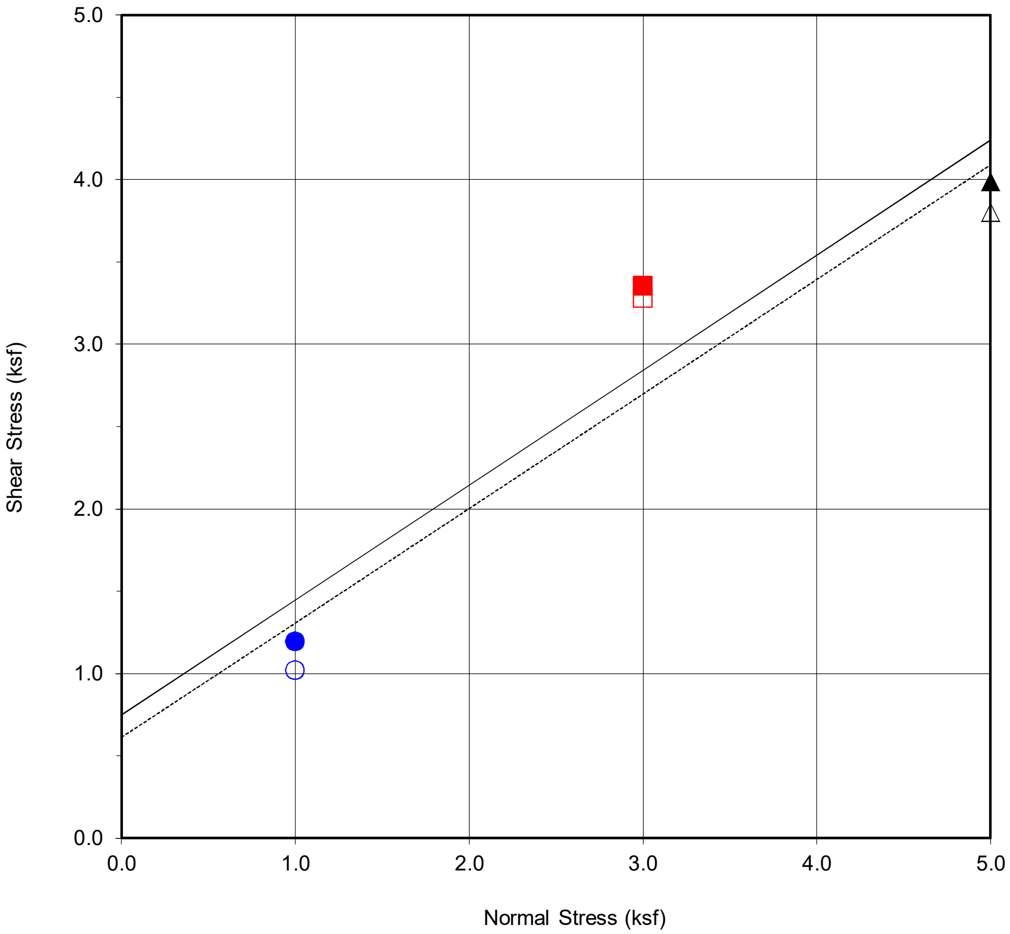
Project No.: W1111-06-01

465-577 SOUTH ARROYO PARKWAY
PASADENA, CALIFORNIA

JULY 2021

Figure B6



Boring No.	**B4**
Sample No.	**B4@40'**
Depth (ft)	**40'**
Sample Type:	Ring

Normal Stress (kip/ft2)	1	3	5
Peak Shear Stress (kip/ft²)	● 1.19	■ 3.35	▲ 3.99
Shear Stress @ End of Test (ksf)	○ 1.02	□ 3.28	△ 3.80
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 (%)	6.8	6.4	6.7
Initial Dry Density (pcf)	106.0	106.2	100.9
Initial Degree of Saturation (%)	31.3	29.5	26.9
Soil Height Before Shearing (in.)	1.2	1.2	1.2
Final Moisture Content (%)	15.6	14.7	16.7

Soil Identification:

Brown Poorly Graded Sand (SP)

Strength Parameters

	C (psf)	φ (°)
Peak	748	34.9
Ultimate	612	34.8

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DIRECT SHEAR TEST RESULTS

Consolidated Drained ASTM D-3080

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465-577 SOUTH ARROYO PARKWAY
PASADENA, CALIFORNIA

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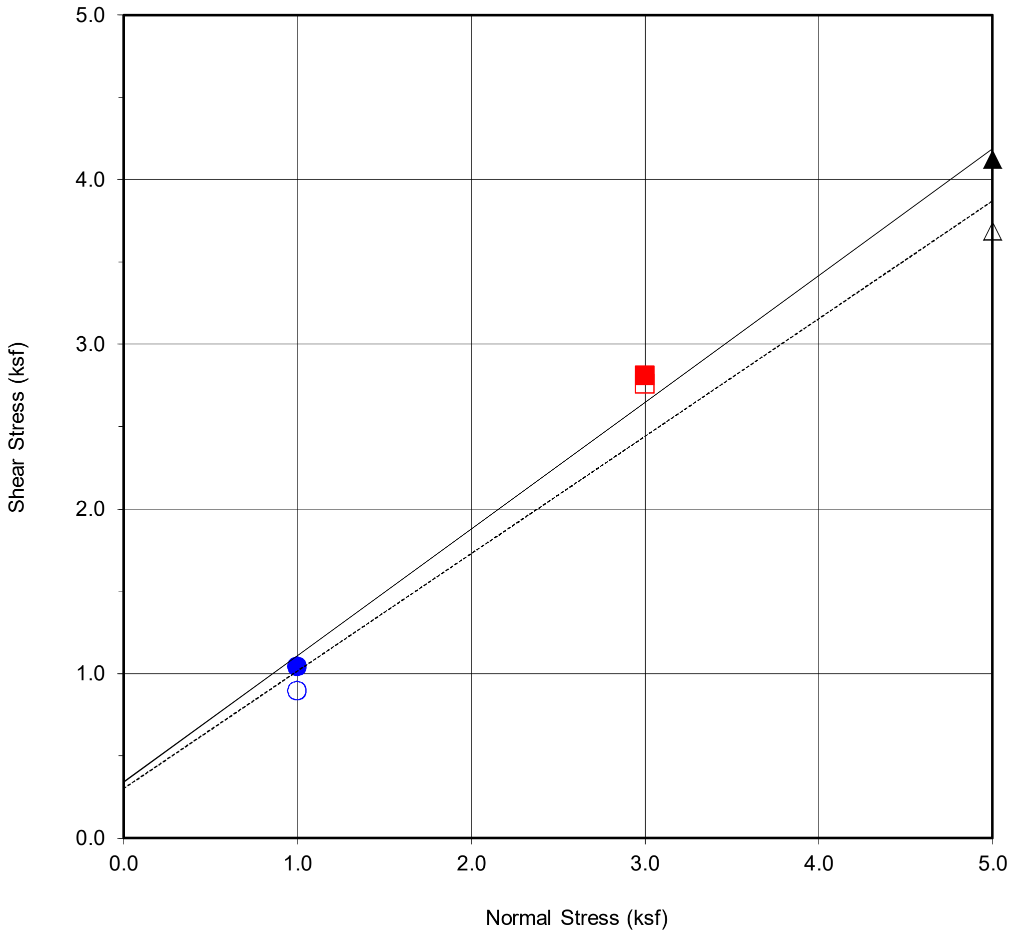
Figure B7



Boring No.	**B4**
Sample No.	**B4@50'**
Depth (ft)	**50**
Sample Type:	Ring

Soil Identification:

Brown Poorly Graded Sand (SP)

Strength Parameters

	C (psf)	ϕ (°)
Peak	339	37.6
Ultimate	300	35.5

Normal Stress (kip/ft2)	1	3	5
Peak Shear Stress (kip/ft²)	● 1.04	■ 2.81	▲ 4.12
Shear Stress @ End of Test (ksf)	○ 0.89	□ 2.76	△ 3.69
Deformation Rate (in./min.)	0.05	0.05	0.05
Initial Sample Height (in.)	1.0	1.0	1.0
Ring Inside Diameter (in.)	2.375	2.375	2.375
Initial Moisture Content (%)	7.2	7.8	8.2
Initial Dry Density (pcf)	106.2	99.9	95.8
Initial Degree of Saturation (%)	33.1	30.5	29.2
Soil Height Before Shearing (in.)	1.2	1.2	1.2
Final Moisture Content (%)	17.2	18.1	18.2

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DIRECT SHEAR TEST RESULTS

Consolidated Drained ASTM D-3080

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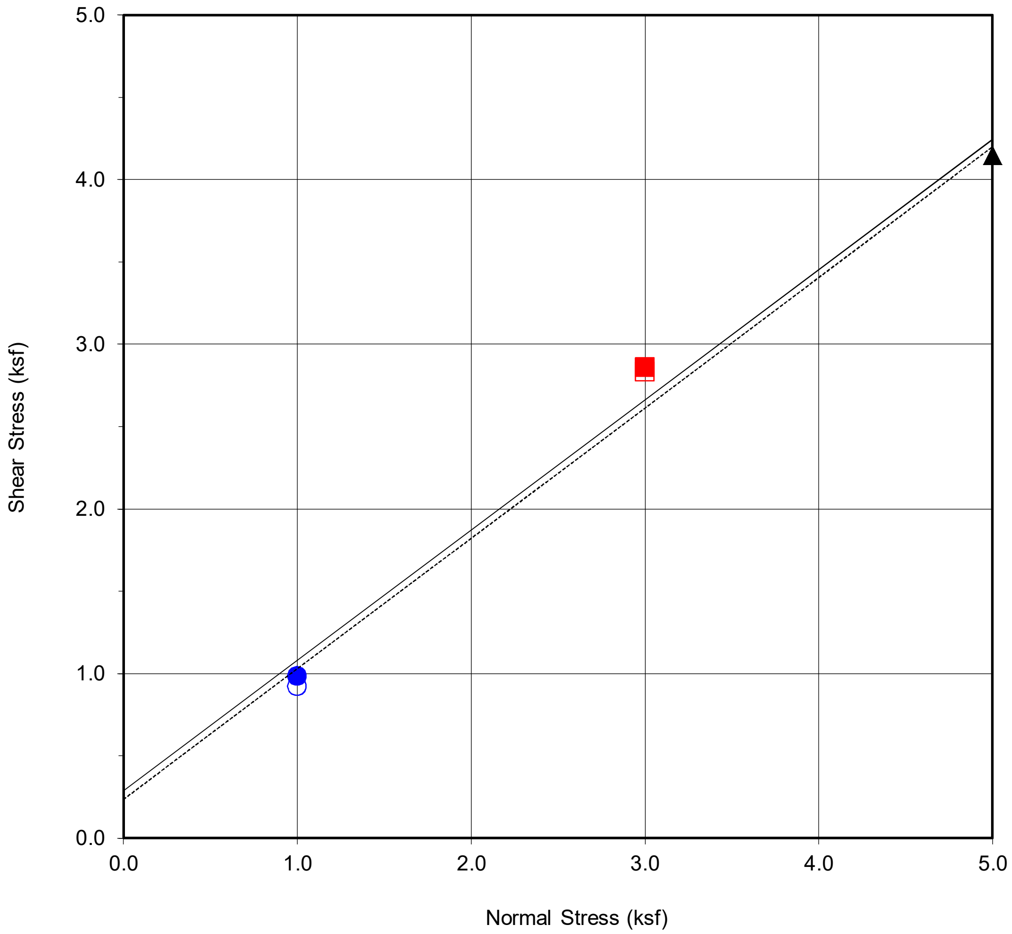
Project No.: W1111-06-01

465-577 SOUTH ARROYO PARKWAY
PASADENA, CALIFORNIA

JULY 2021

Figure B8



Boring No.	**B4**
Sample No.	**B4@70'**
Depth (ft)	**70'**
Sample Type:	Ring

Soil Identification:

Brown Silty Gravel (GM)

Strength Parameters

	C (psf)	ϕ (^{o})
Peak	287	38.4
Ultimate	235	38.4

Normal Stress (kip/ft2)	1	3	5
Peak Shear Stress (kip/ft²)	● 0.98	■ 2.86	▲ 4.15
Shear Stress @ End of Test (ksf)	○ 0.92	□ 2.83	△ 4.15
Deformation Rate (in./min.)	0.05	0.05	0.05
Initial Sample Height (in.)	1.0	1.0	1.0
Ring Inside Diameter (in.)	2.375	2.375	2.375
Initial Moisture Content (%)	10.9	11.4	13.0
Initial Dry Density (pcf)	107.8	107.0	102.3
Initial Degree of Saturation (%)	52.4	53.7	54.0
Soil Height Before Shearing (in.)	1.2	1.2	1.2
Final Moisture Content (%)	15.8	15.0	13.9

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DIRECT SHEAR TEST RESULTS

Consolidated Drained ASTM D-3080

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Project No.: W1111-06-01

465-577 SOUTH ARROYO PARKWAY
PASADENA, CALIFORNIA

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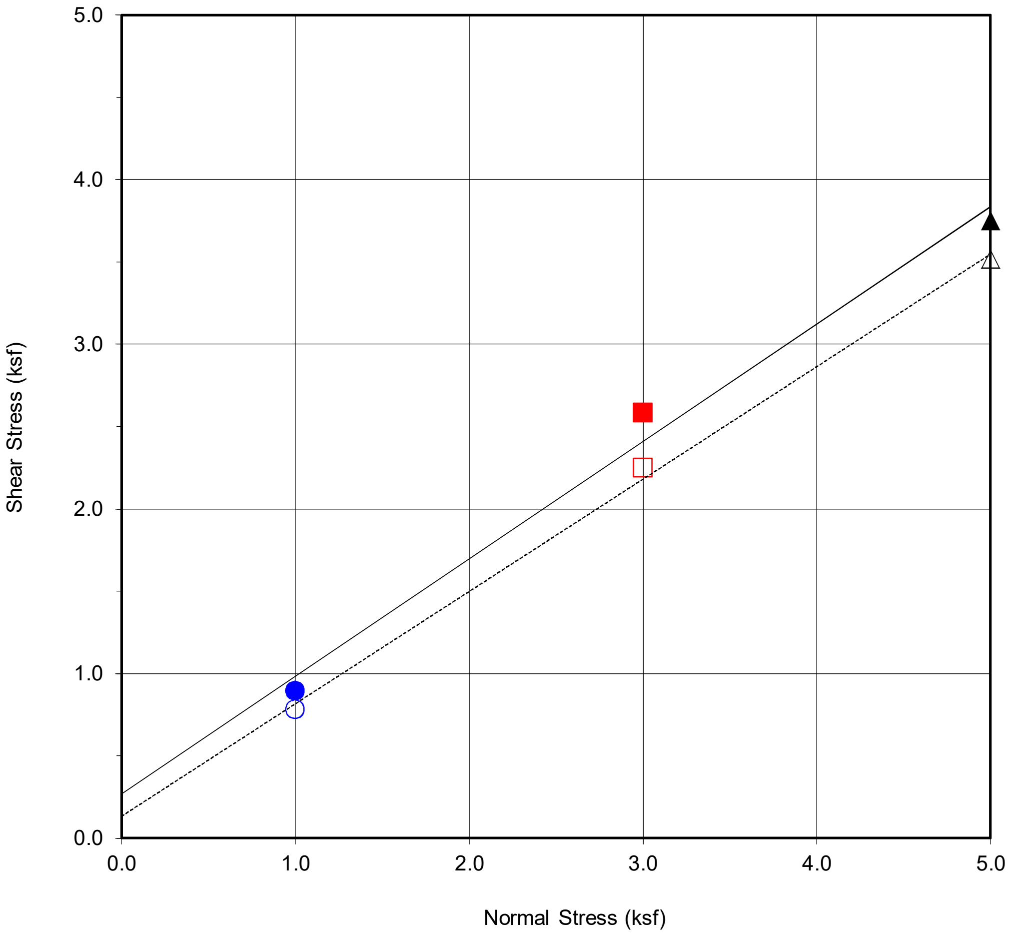
Figure B9



Boring No.	**B4**
Sample No.	**B4@90'**
Depth (ft)	**90'**
Sample Type:	Ring

Soil Identification:		
Brown Poorly Graded Sand (SP)		
Strength Parameters		
	C (psf)	ϕ (°)
Peak	267	35.5
Ultimate	132	34.3

Normal Stress (kip/ft2)	1	3	5
Peak Shear Stress (kip/ft²)	● 0.89	■ 2.58	▲ 3.75
Shear Stress @ End of Test (ksf)	○ 0.78	□ 2.25	△ 3.51
Deformation Rate (in./min.)	0.05	0.05	0.05
Initial Sample Height (in.)	1.0	1.0	1.0
Ring Inside Diameter (in.)	2.375	2.375	2.375
Initial Moisture Content (%)	13.8	14.2	12.8
Initial Dry Density (pcf)	103.5	103.8	107.0
Initial Degree of Saturation (%)	59.1	61.4	60.2
Soil Height Before Shearing (in.)	1.2	1.2	1.2
Final Moisture Content (%)	19.1	17.9	16.9

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DIRECT SHEAR TEST RESULTS

Consolidated Drained ASTM D-3080

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Project No.: W1111-06-01

465-577 SOUTH ARROYO PARKWAY
PASADENA, CALIFORNIA

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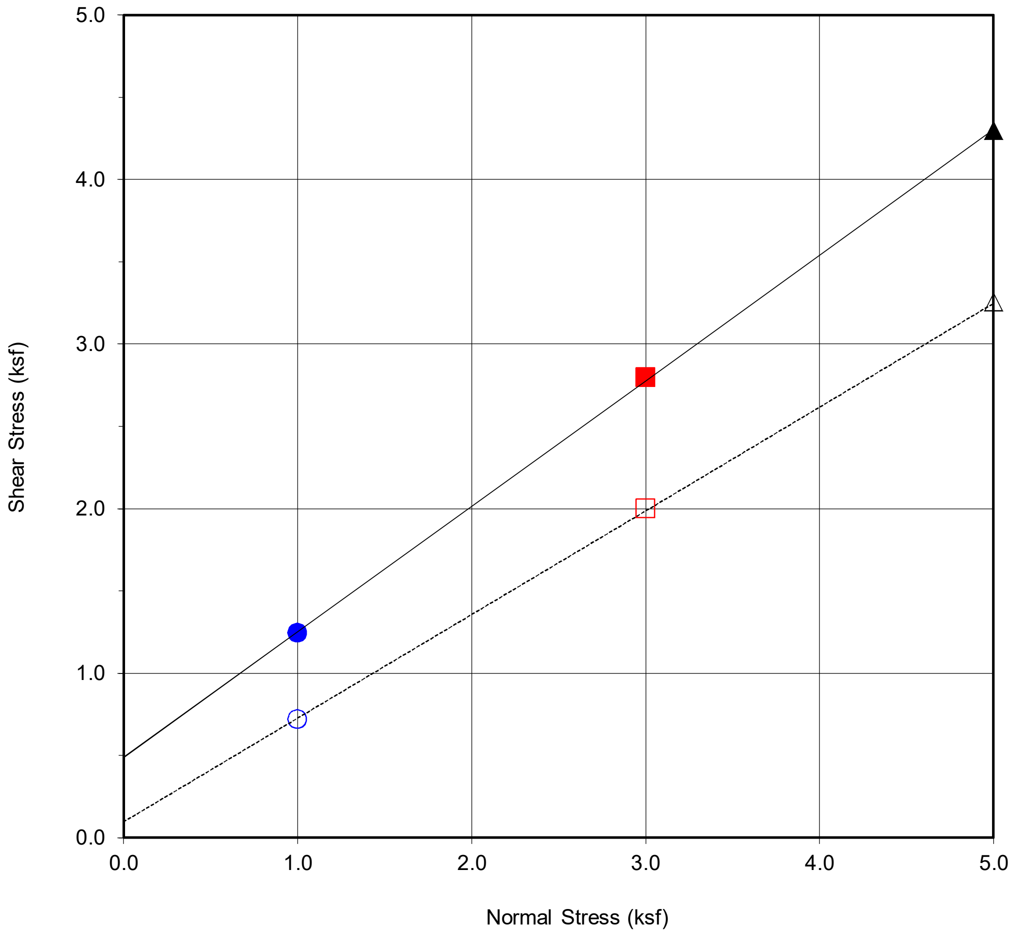
Figure B10



Boring No.	**B5**
Sample No.	**B5@15'**
Depth (ft)	**15**
Sample Type:	Ring

Soil Identification:

Reddish Brown Silty Sand (SM)

Strength Parameters

	C (psf)	ϕ (°)
Peak	487	37.4
Ultimate	96	32.2

Normal Stress (kip/ft2)	1	3	5
Peak Shear Stress (kip/ft²)	● 1.24	■ 2.79	▲ 4.30
Shear Stress @ End of Test (ksf)	○ 0.72	□ 2.00	△ 3.25
Deformation Rate (in./min.)	0.05	0.05	0.05
Initial Sample Height (in.)	1.0	1.0	1.0
Ring Inside Diameter (in.)	2.375	2.375	2.375
Initial Moisture Content (%)	15.6	16.8	16.3
Initial Dry Density (pcf)	116.2	114.6	116.0
Initial Degree of Saturation (%)	93.3	96.6	97.1
Soil Height Before Shearing (in.)	1.2	1.2	1.2
Final Moisture Content (%)	18.0	18.3	16.9

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DIRECT SHEAR TEST RESULTS

Consolidated Drained ASTM D-3080

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Project No.: W1111-06-01

465-577 SOUTH ARROYO PARKWAY
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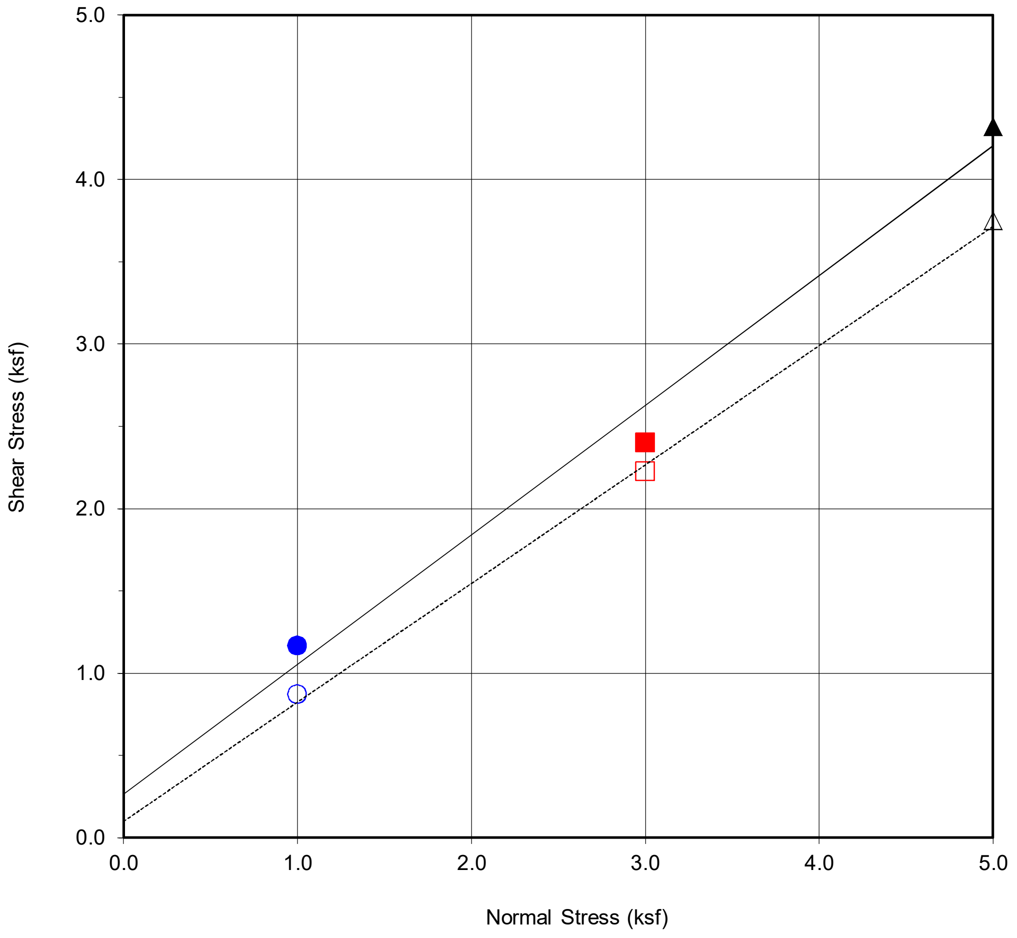
JULY 2021

Figure B11



Boring No.	**B5**
Sample No.	**B5@30'**
Depth (ft)	**30'**
Sample Type:	Ring

Soil Identification:

Reddish Brown Silty Sand (SM)

Strength Parameters

	C (psf)	ϕ (°)
Peak	263	38.3
Ultimate	99	35.9

Normal Stress (kip/ft2)	1	3	5
Peak Shear Stress (kip/ft²)	● 1.17	■ 2.40	▲ 4.32
Shear Stress @ End of Test (ksf)	○ 0.87	□ 2.22	△ 3.75
Deformation Rate (in./min.)	0.05	0.05	0.05
Initial Sample Height (in.)	1.0	1.0	1.0
Ring Inside Diameter (in.)	2.375	2.375	2.375
Initial Moisture Content (%)	13.6	15.0	14.8
Initial Dry Density (pcf)	120.1	115.9	119.7
Initial Degree of Saturation (%)	91.2	89.5	98.0
Soil Height Before Shearing (in.)	1.2	1.2	1.2
Final Moisture Content (%)	17.2	21.3	15.9

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DIRECT SHEAR TEST RESULTS

Consolidated Drained ASTM D-3080

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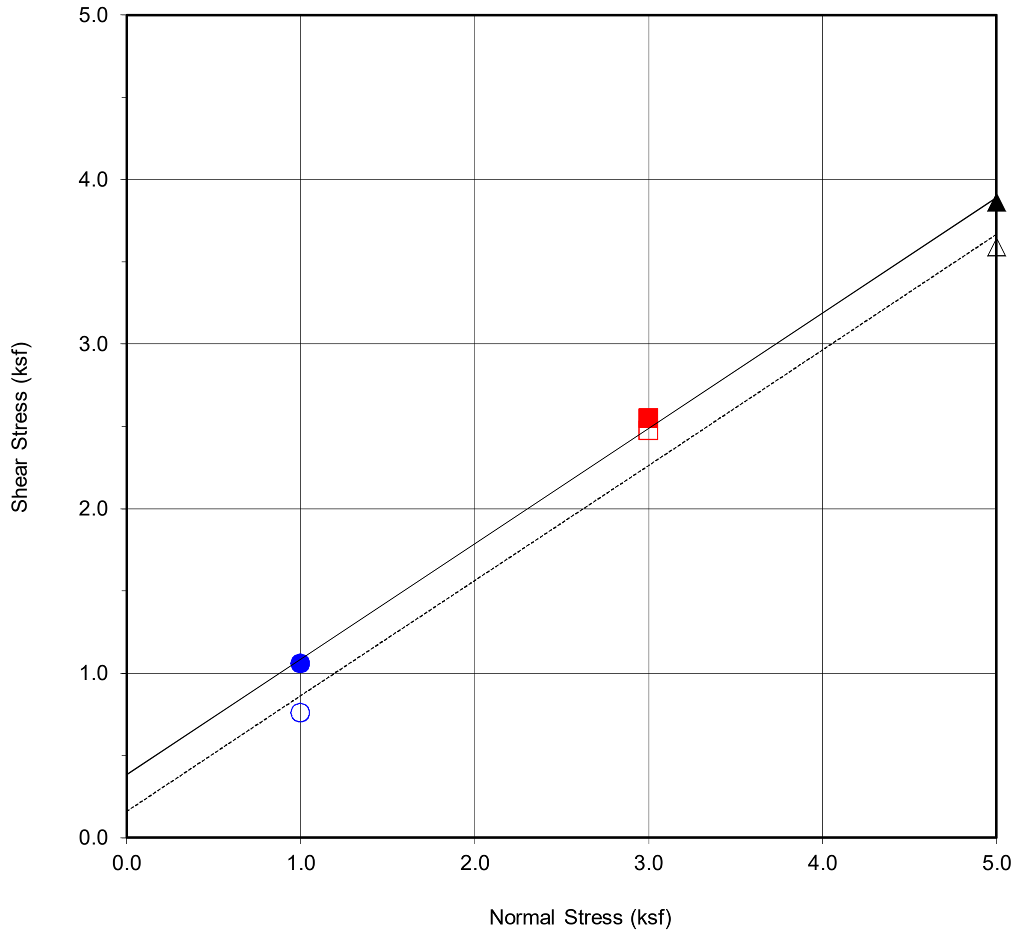
Project No.: W1111-06-01

465-577 SOUTH ARROYO PARKWAY
PASADENA, CALIFORNIA

JULY 2021

Figure B12



Boring No.	**B5**
Sample No.	**B5@80**
Depth (ft)	**80**
Sample Type:	Ring

Soil Identification:		
Reddish Brown Silty Sand (SM)		
Strength Parameters		
	C (psf)	ϕ (°)
Peak	382	35.1
Ultimate	159	35.0

Normal Stress (kip/ft2)	1	3	5
Peak Shear Stress (kip/ft²)	● 1.06	■ 2.55	▲ 3.86
Shear Stress @ End of Test (ksf)	○ 0.76	□ 2.47	△ 3.59
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 (%)	19.7	15.2	14.1
Initial Dry Density (pcf)	106.1	109.6	108.7
Initial Degree of Saturation (%)	90.3	76.2	69.0
Soil Height Before Shearing (in.)	1.2	1.2	1.2
Final Moisture Content (%)	22.4	18.6	18.4

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DIRECT SHEAR TEST RESULTS

Consolidated Drained ASTM D-3080

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Project No.: W1111-06-01

465-577 SOUTH ARROYO PARKWAY
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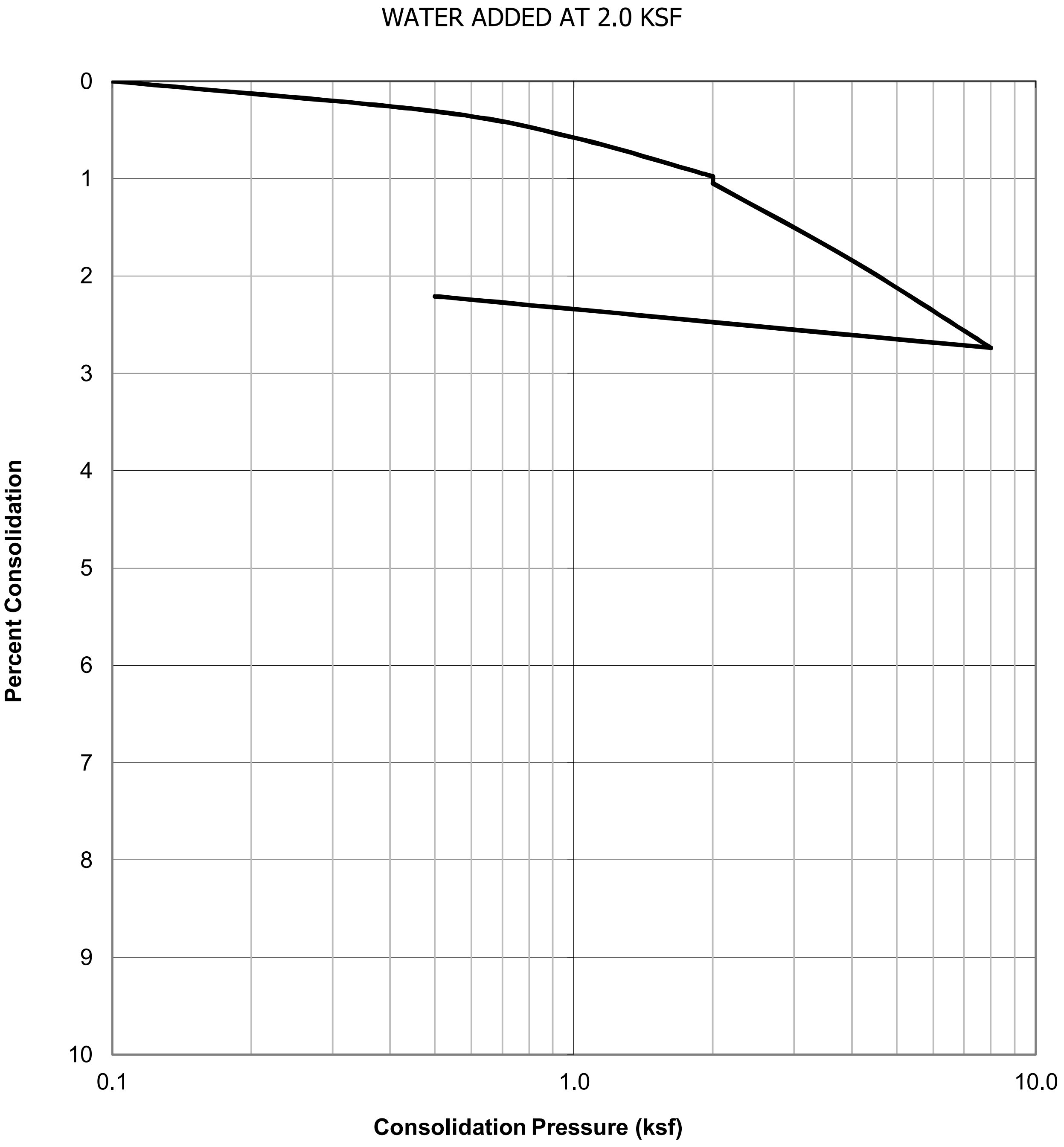

SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B1@5	Light Brown Poorly Graded Sand (SP)	113.3	5.9	12.6

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CONSOLIDATION TEST RESULTS

ASTM D-2435

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Project No.: W1111-06-01

465-577 SOUTH ARROYO PARKWAY
PASADENA, CALIFORNIA

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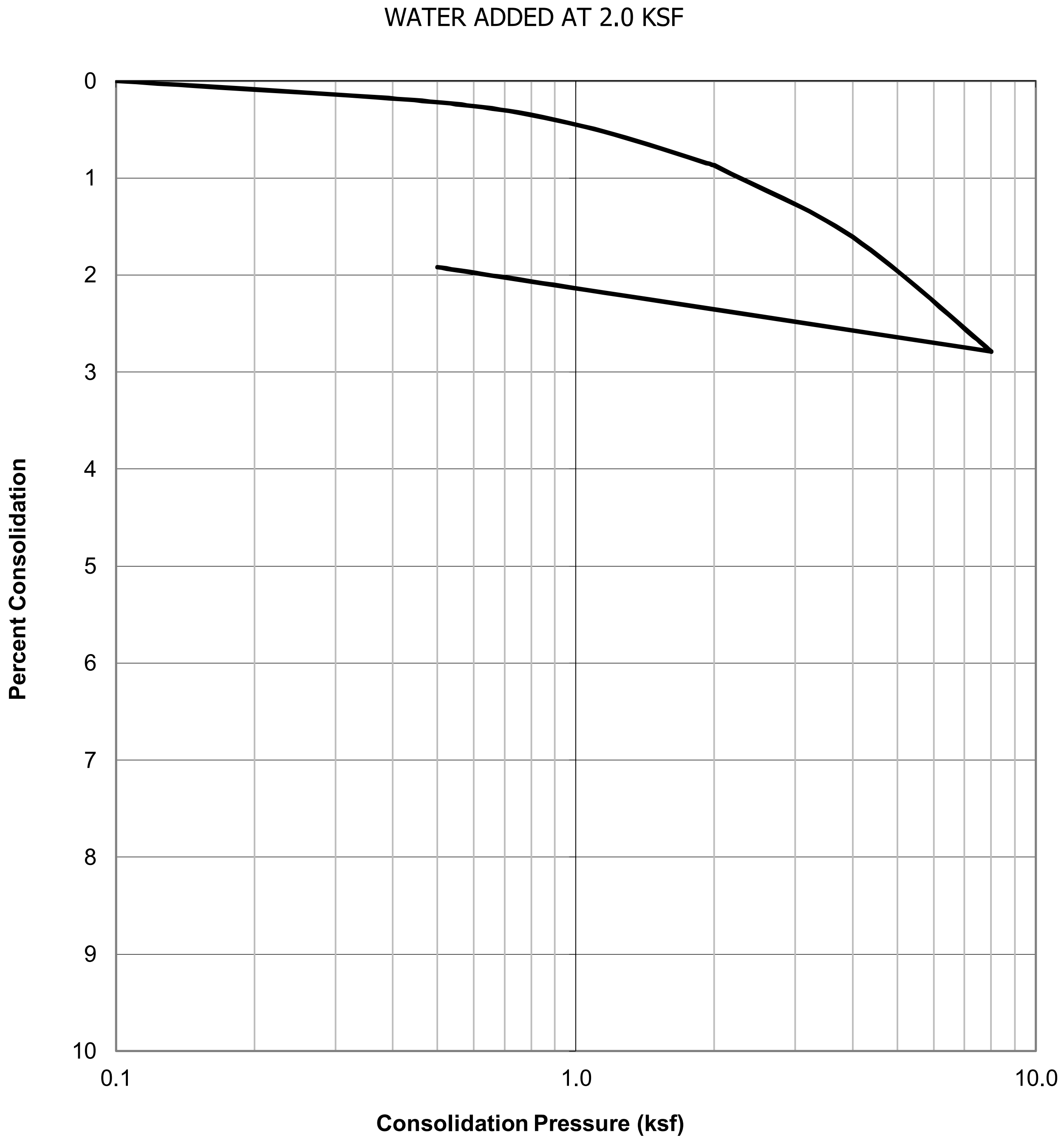
Figure B14



SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B1@7.5	Brown Poorly Graded Sand (SP)	99.6	22.8	23.6

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465-577 SOUTH ARROYO PARKWAY
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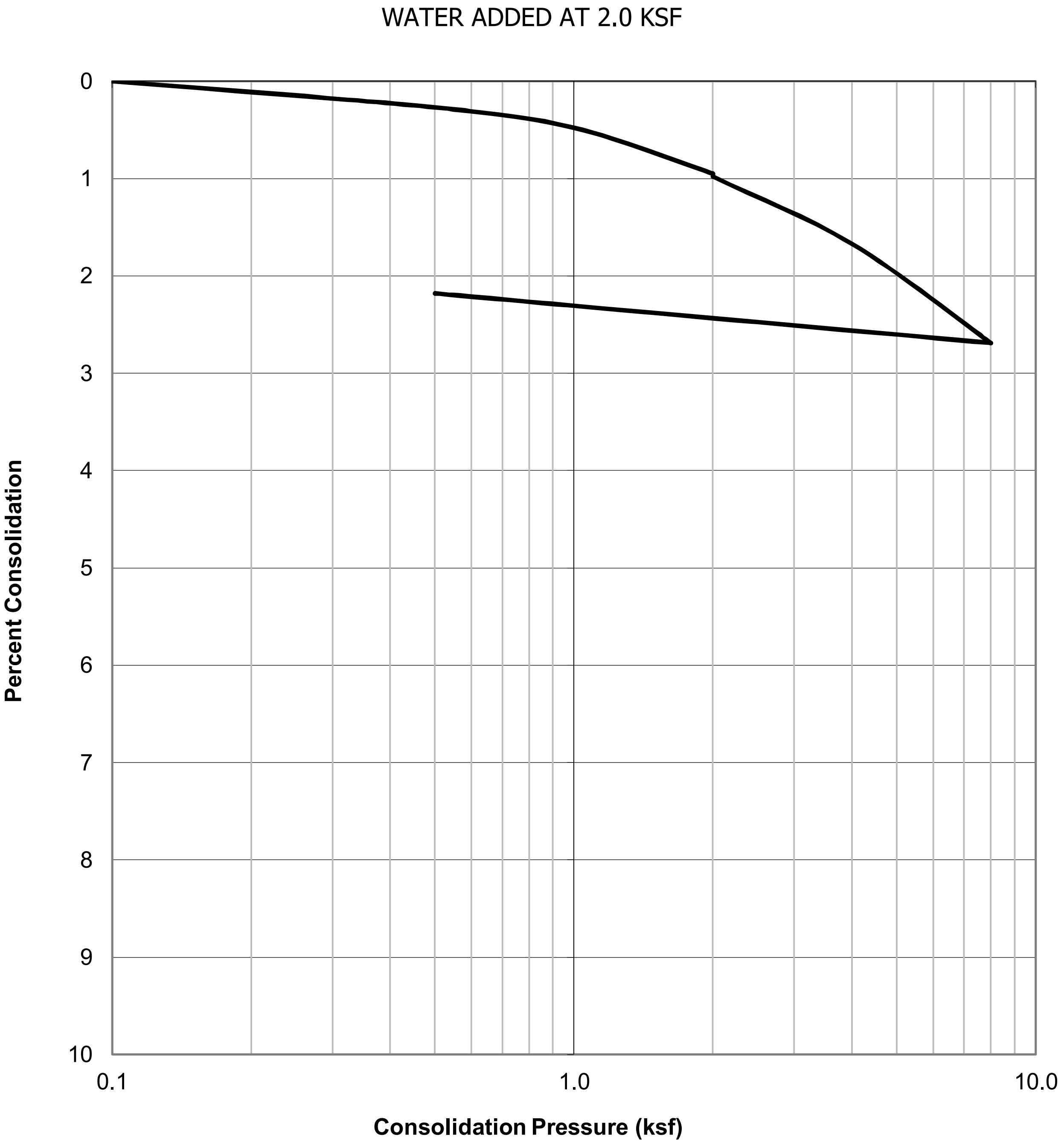
JULY 2021

Figure B15



SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B1@10	Brown Poorly Graded Sand (SP)	106.4	22.1	21.4

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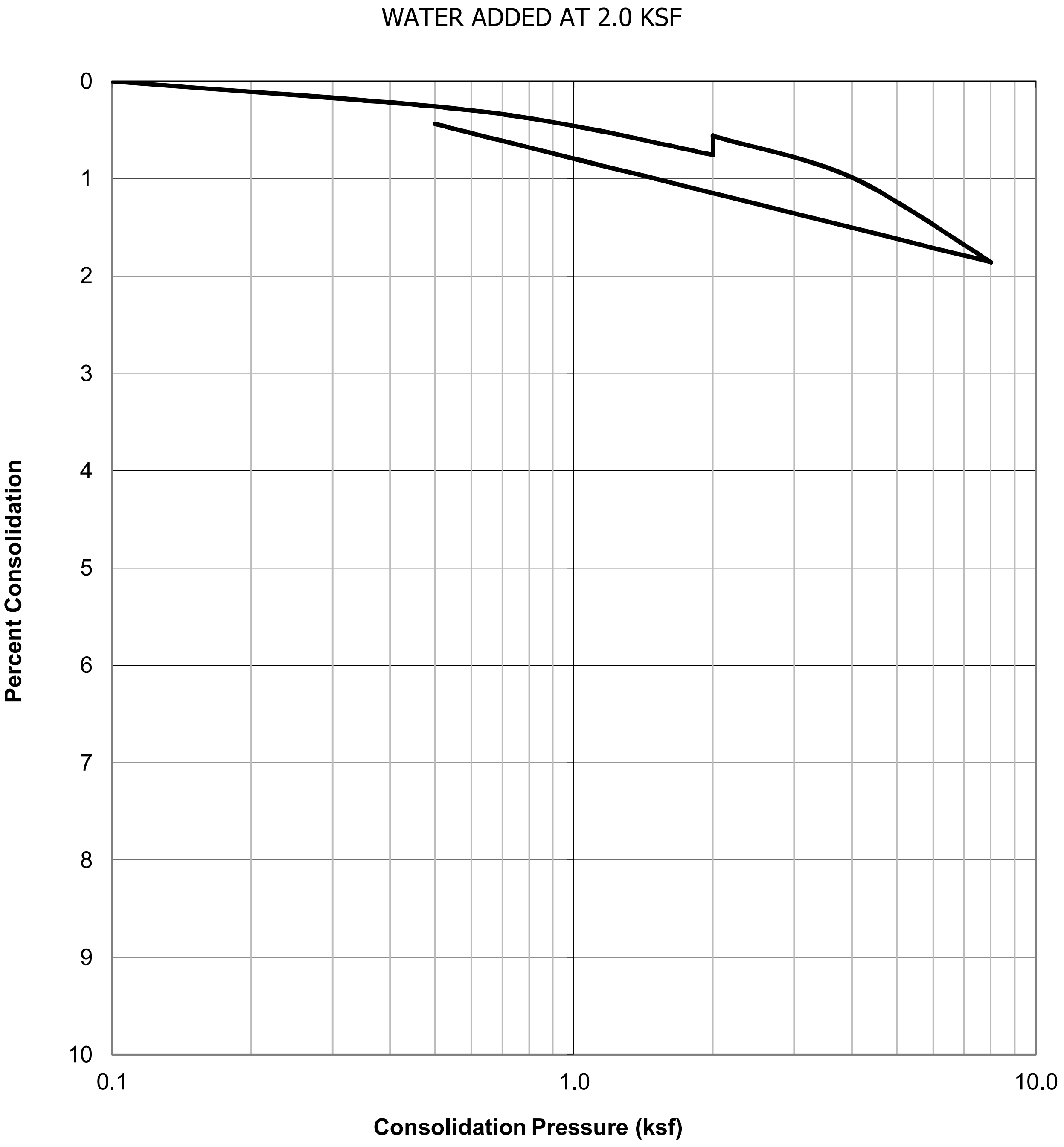
Project No.: W1111-06-01

465-577 SOUTH ARROYO PARKWAY
PASADENA, CALIFORNIA

JULY 2021



SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B2@5	Dark Brown Sandy Silt (ML)	114.1	9.5	17.6

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ASTM D-2435

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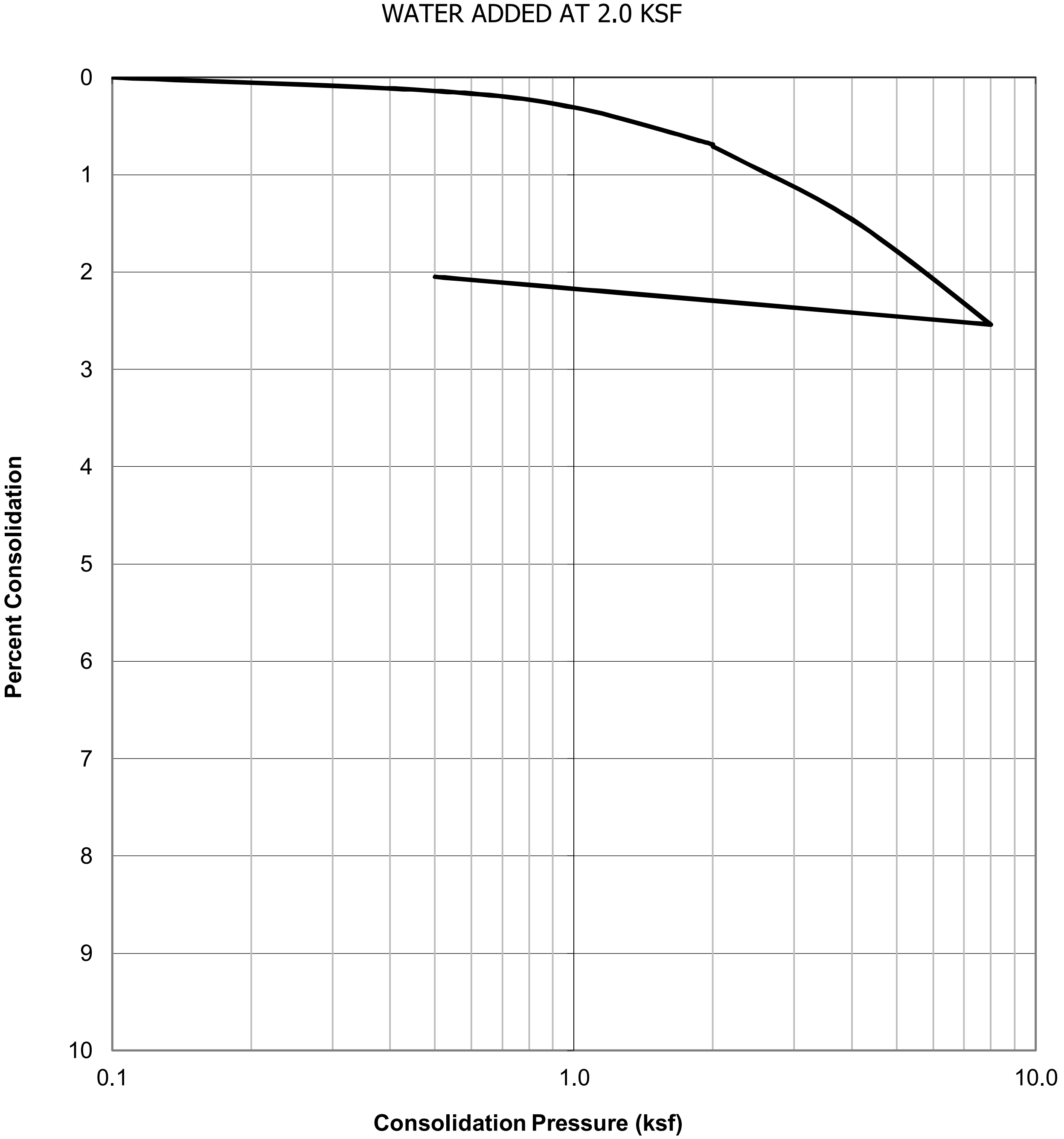
Project No.: W1111-06-01

465-577 SOUTH ARROYO PARKWAY
PASADENA, CALIFORNIA

JULY 2021



SAMPLE ID.	**SOIL TYPE**	**DRY DENSITY (PCF)**	**INITIAL MOISTURE (%)**	**FINAL MOISTURE (%)**
B2@7	Brown Poorly Graded Sand (SP)	112.5	7.6	13.4

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ASTM D-2435

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465-577 SOUTH ARROYO PARKWAY
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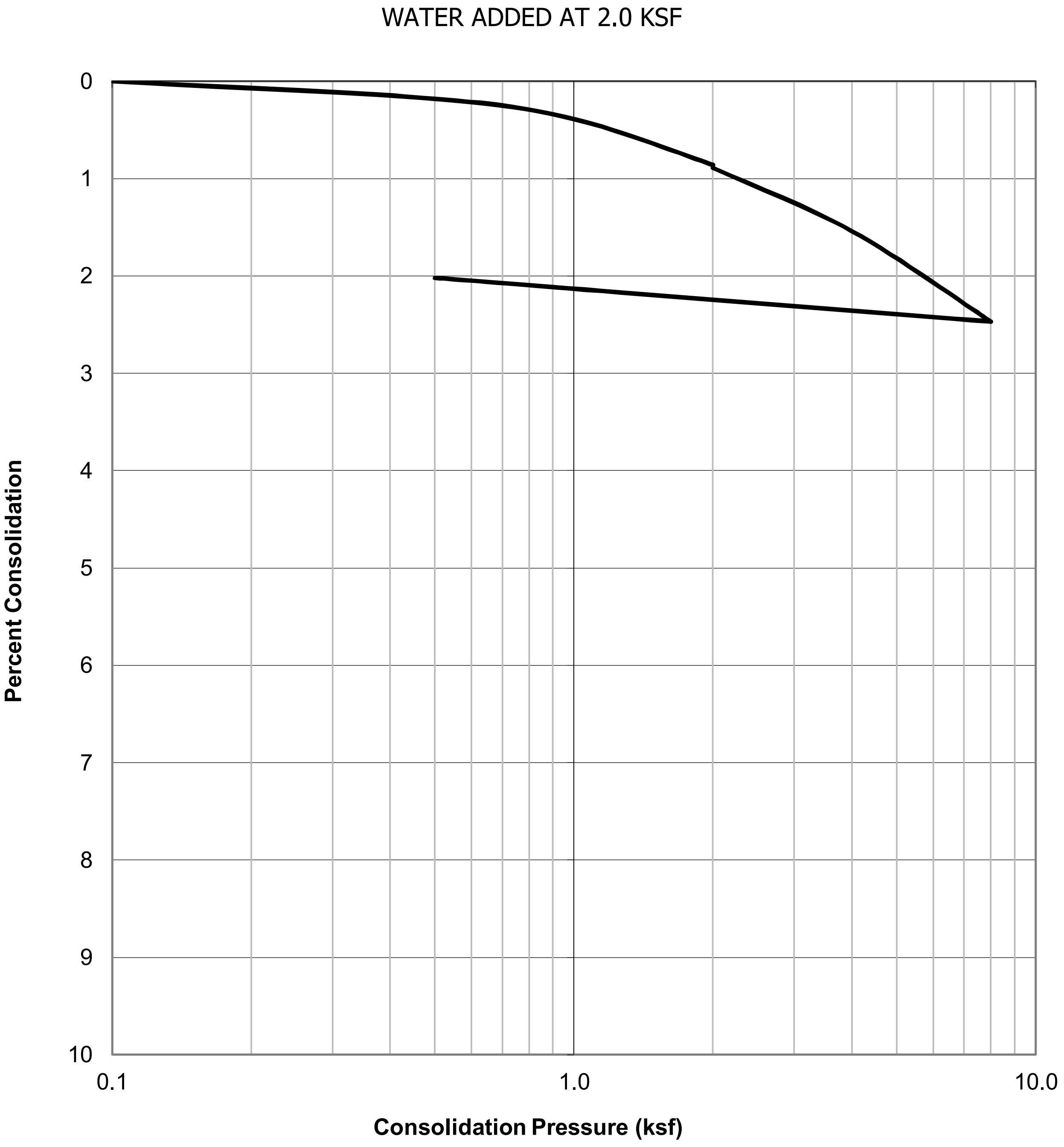
Figure B18



SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B2@10	Brown Poorly Graded Sand (SP)	112.2	9.2	14.0

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ASTM D-2435

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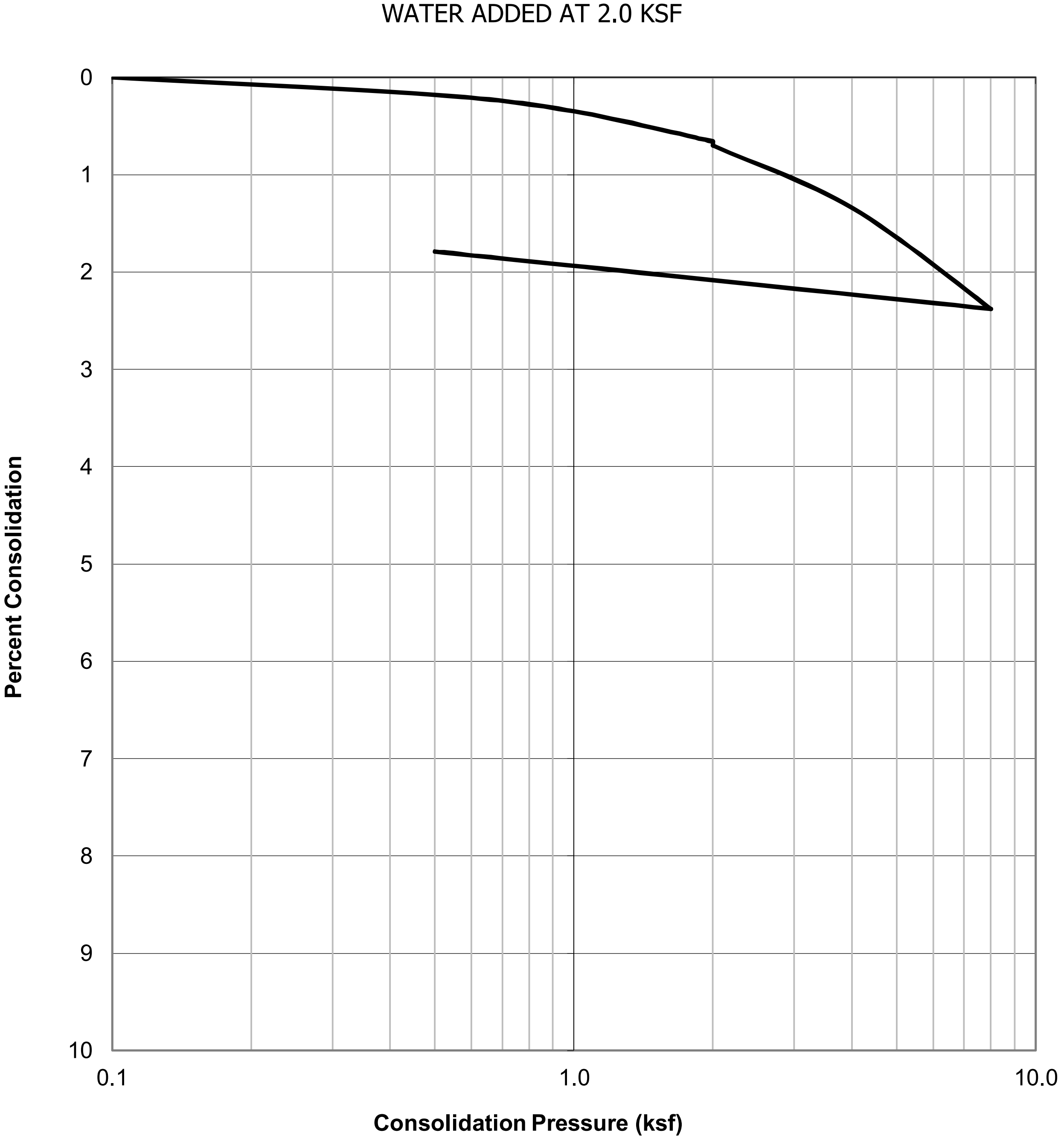
Project No.: W1111-06-01

465-577 SOUTH ARROYO PARKWAY
PASADENA, CALIFORNIA

JULY 2021

Figure B19



SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B2@15	Brown Poorly Graded Sand (SP)	104.2	8.8	16.0

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ASTM D-2435

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465-577 SOUTH ARROYO PARKWAY
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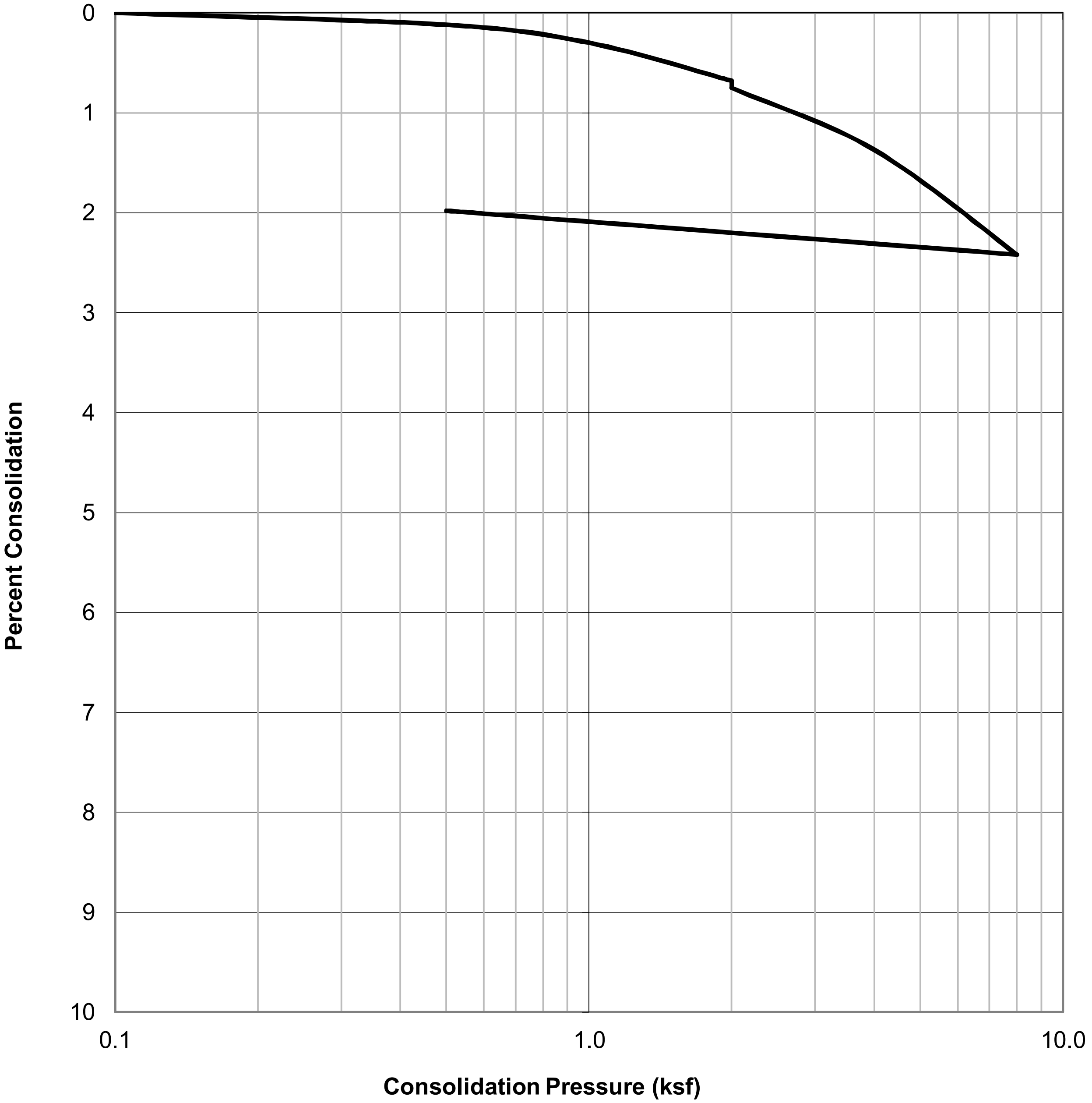
JULY 2021

WATER ADDED AT 2.0 KSF



SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B2@25	Brown Poorly Graded Sand (SP)	115.1	8.8	12.9

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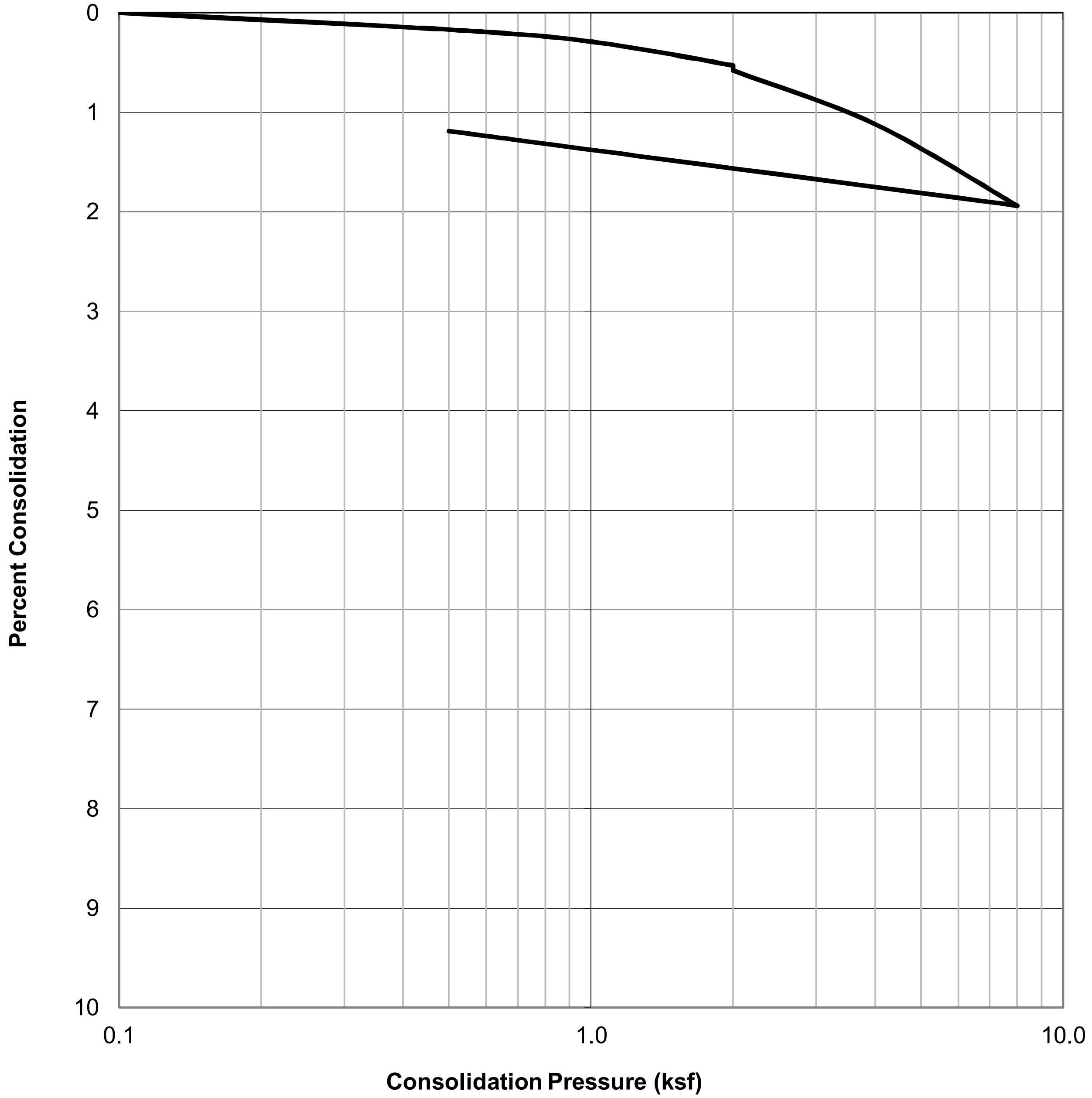
Figure B21

WATER ADDED AT 2.0 KSF



SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B2@30	Brown Poorly Graded Sand (SP)	116.7	13.3	15.1

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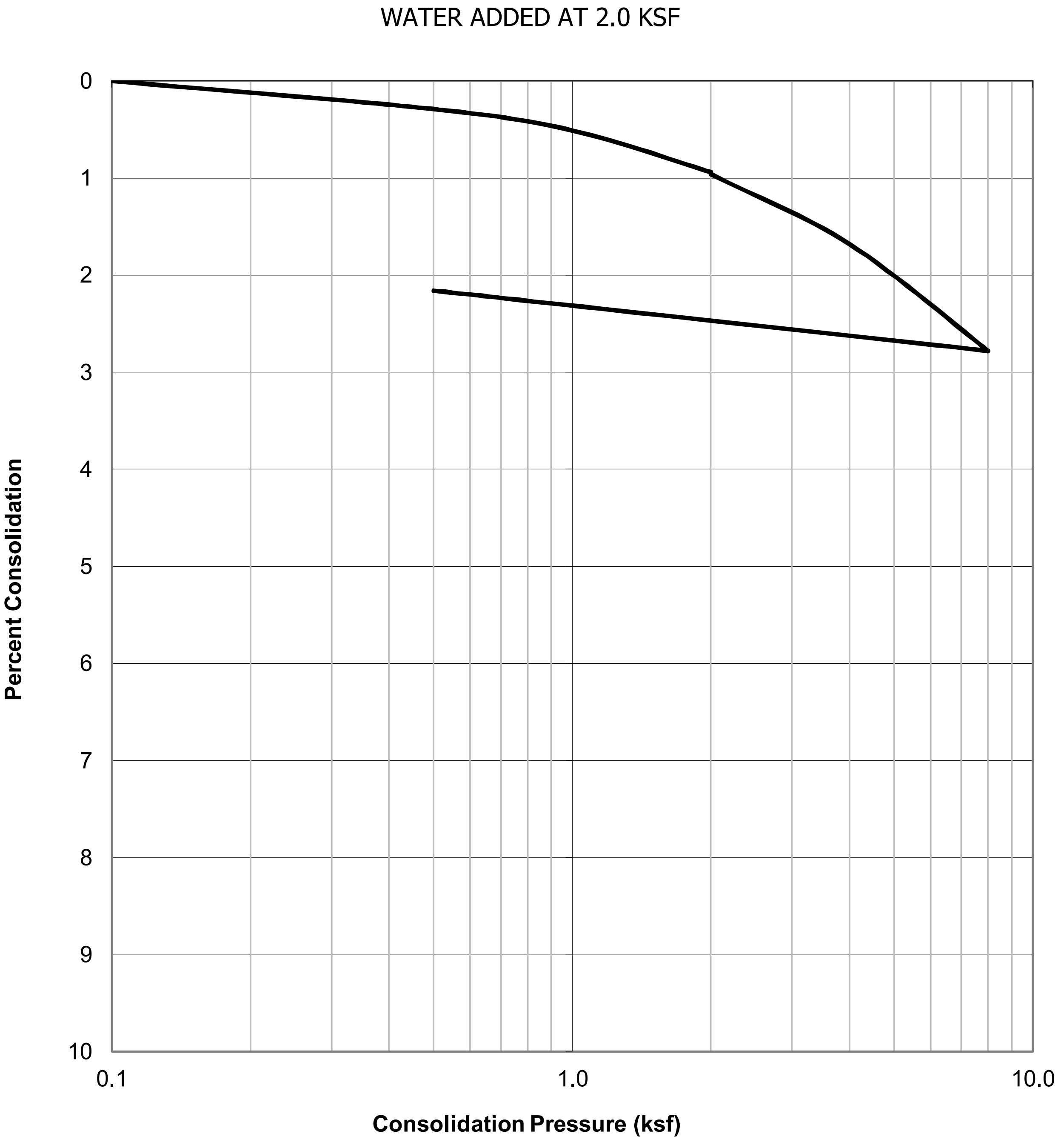
JULY 2021



SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B3@5	Brown Sandy Silt (ML)	117.2	14.2	13.0

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ASTM D-2435

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465-577 SOUTH ARROYO PARKWAY
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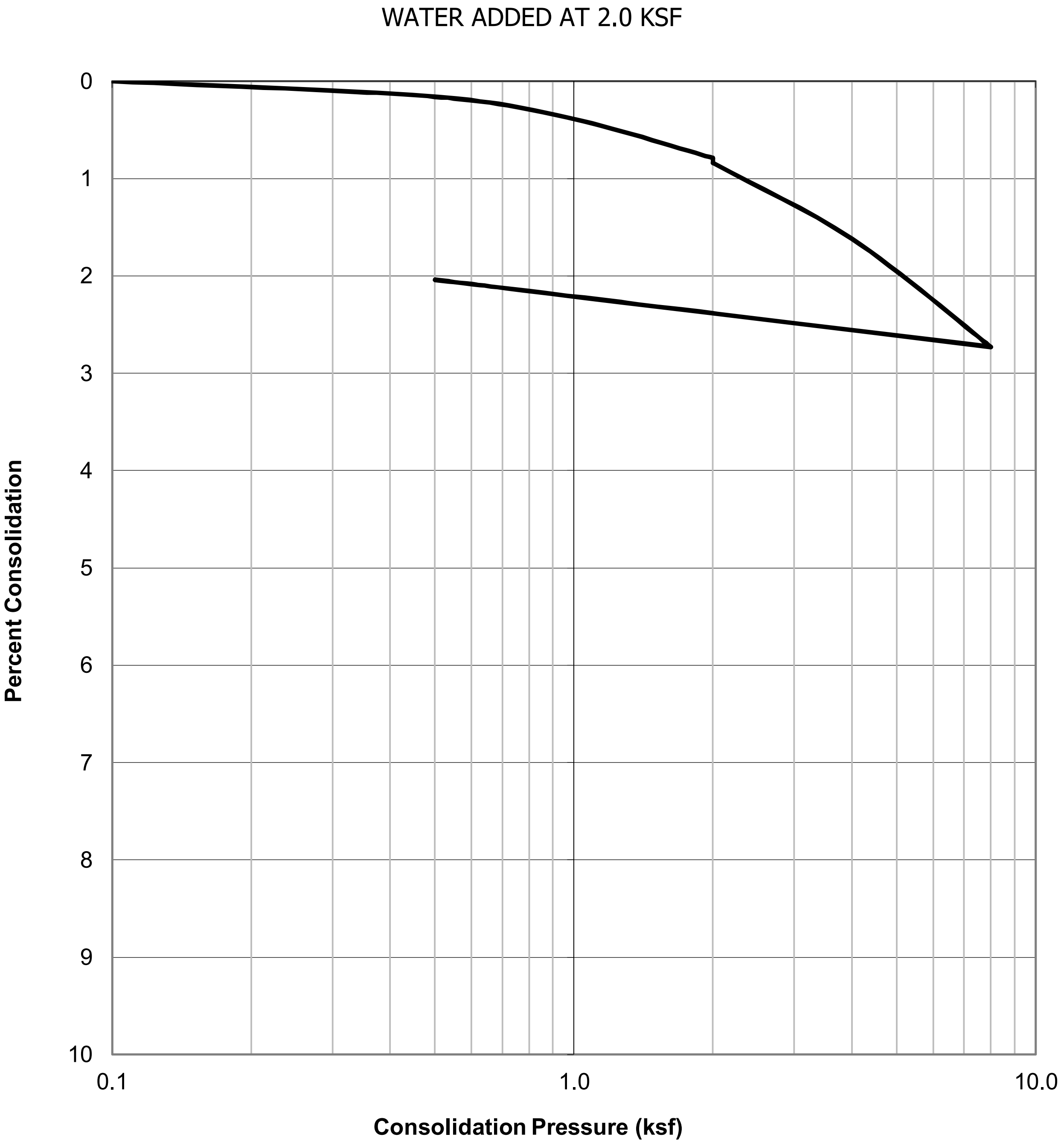
JULY 2021

Figure B23



SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B3@20	Brown Poorly Graded Sand (SP)	100.0	20.1	22.6

GEOCON

CONSOLIDATION TEST RESULTS

ASTM D-2435

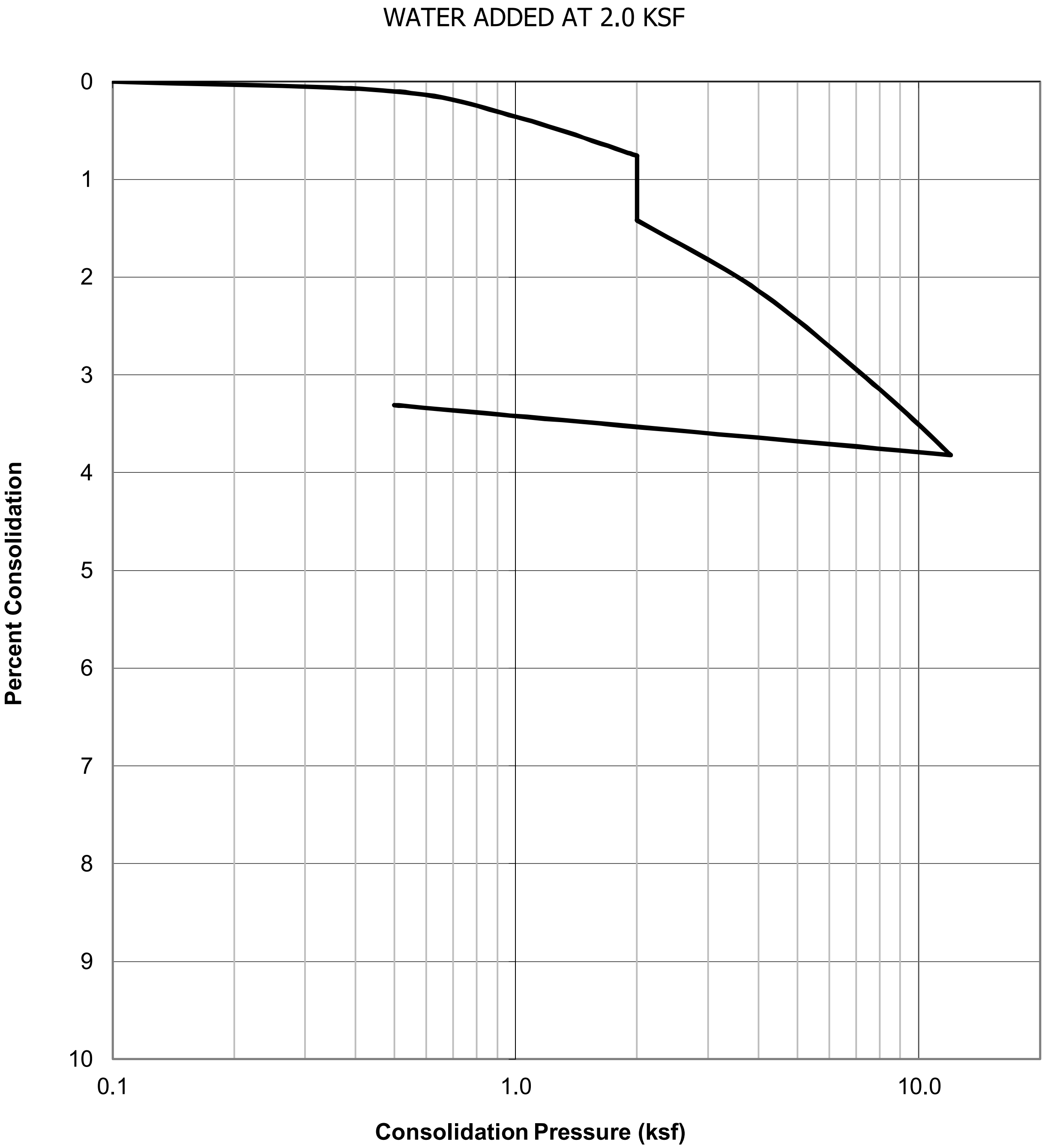
Checked by: JJK

Project No.: W1111-06-01

465-577 SOUTH ARROYO PARKWAY
PASADENA, CALIFORNIA

JULY 2021

Figure B24



SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B4@30	Brown Poorly Graded Sand (SP)	109.6	3.6	16.2

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CONSOLIDATION TEST RESULTS

ASTM D-2435

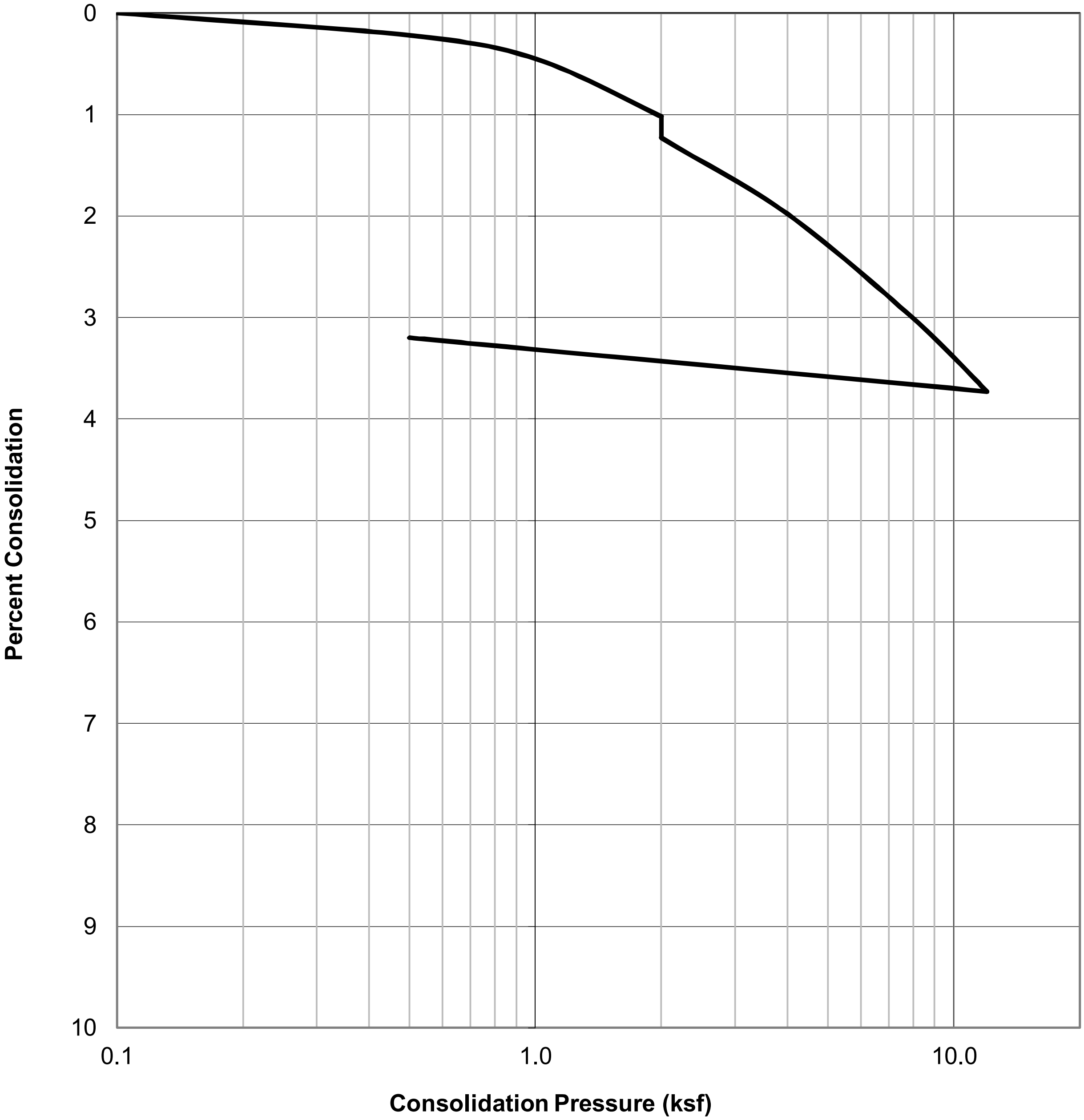
Checked by: JJK

Project No.: W1111-06-01

465-577 SOUTH ARROYO PARKWAY
PASADENA, CALIFORNIA

JULY 2021

WATER ADDED AT 2.0 KSF



SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B4@40	Brown Poorly Graded Sand (SP)	103.8	3.7	15.6

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CONSOLIDATION TEST RESULTS

ASTM D-2435

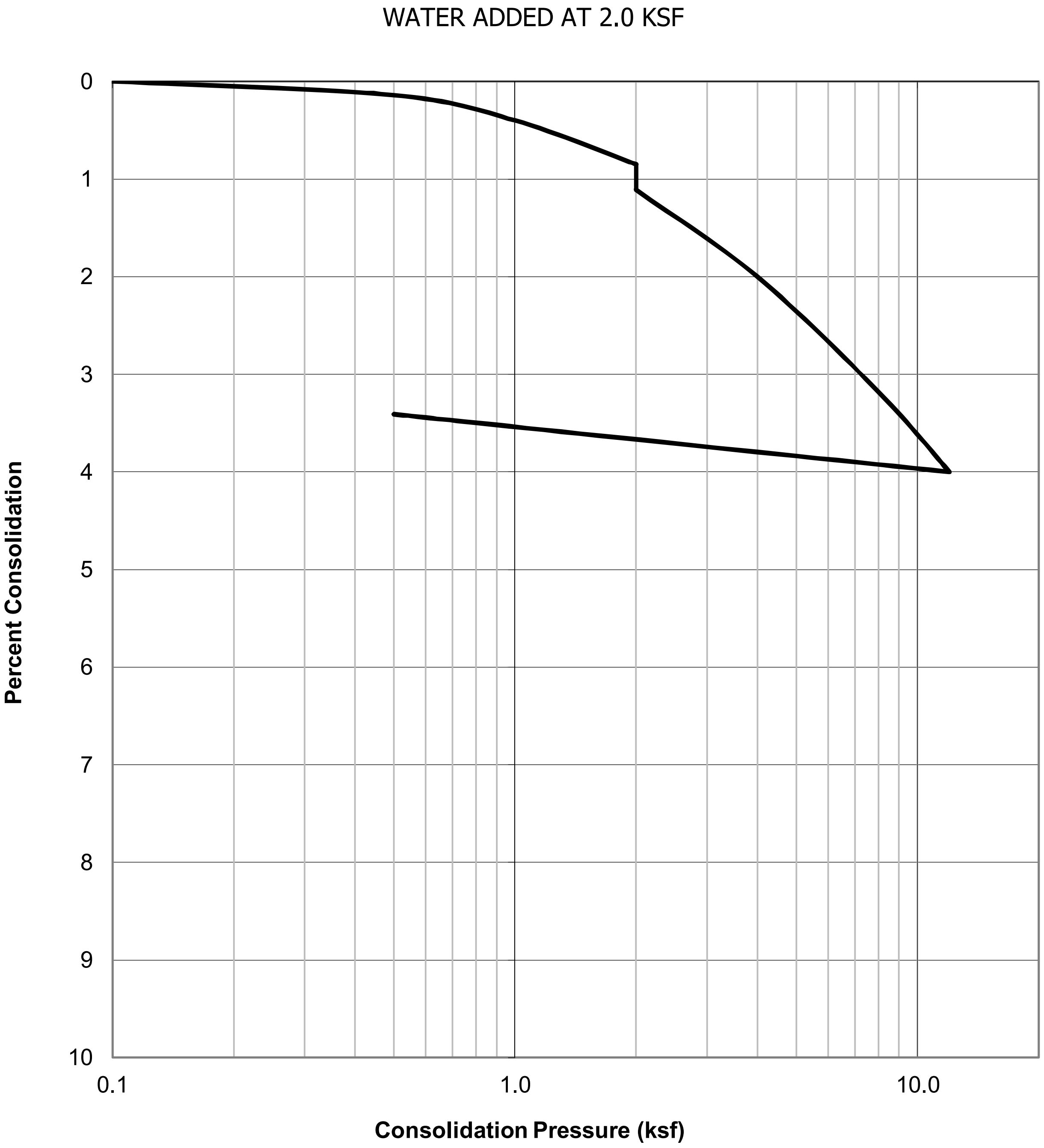
Checked by: JJK

Project No.: W1111-06-01

465-577 SOUTH ARROYO PARKWAY
PASADENA, CALIFORNIA

JULY 2021

Figure B26



SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B4@50	Brown Poorly Graded Sand (SP)	110.4	3.8	16.3

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CONSOLIDATION TEST RESULTS

ASTM D-2435

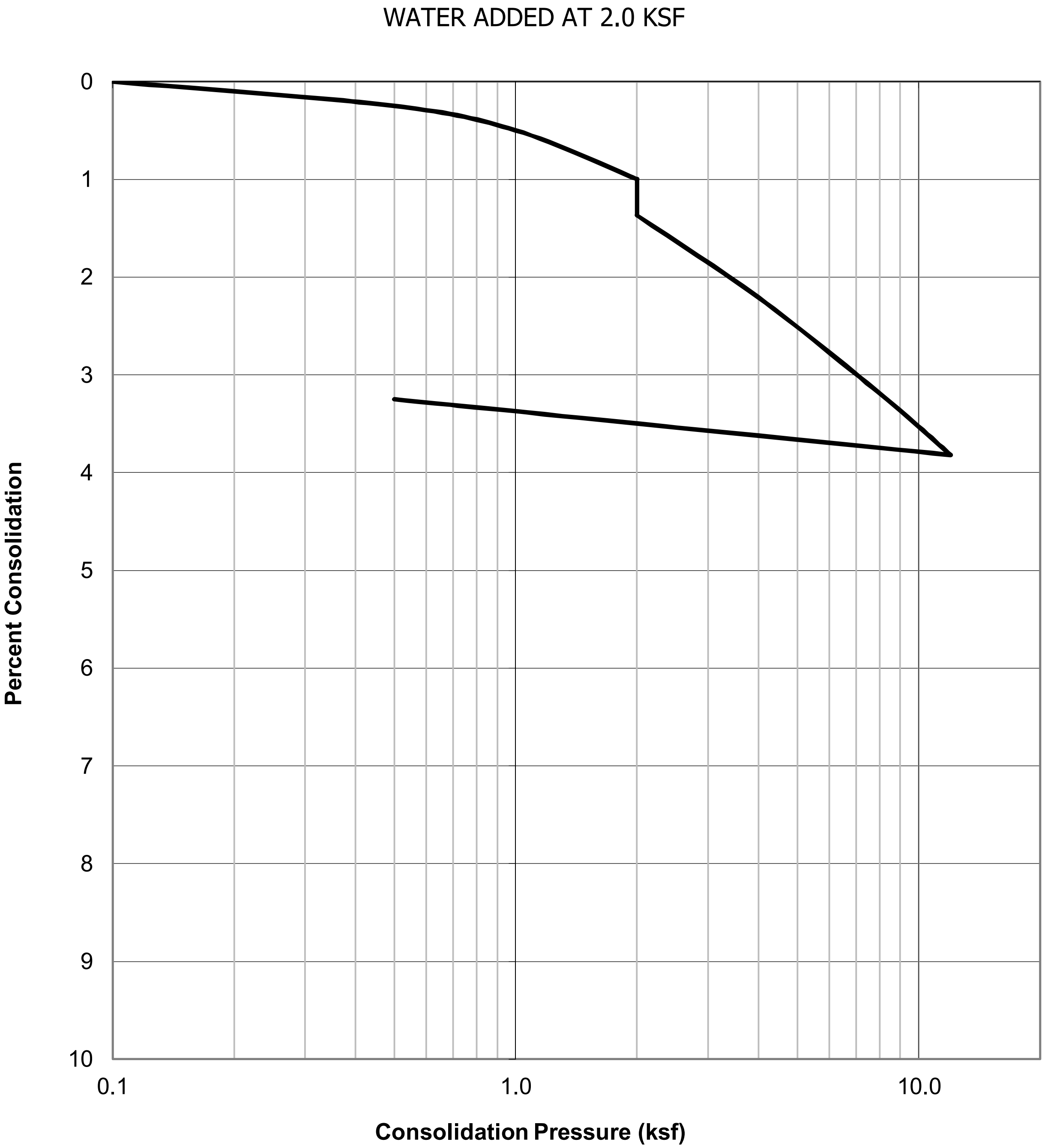
Checked by: JJK

Project No.: W1111-06-01

465-577 SOUTH ARROYO PARKWAY
PASADENA, CALIFORNIA

JULY 2021

Figure B27



SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B4@60	Brown Poorly Graded Sand (SP)	107.5	2.8	16.7

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CONSOLIDATION TEST RESULTS

ASTM D-2435

Checked by: JJK

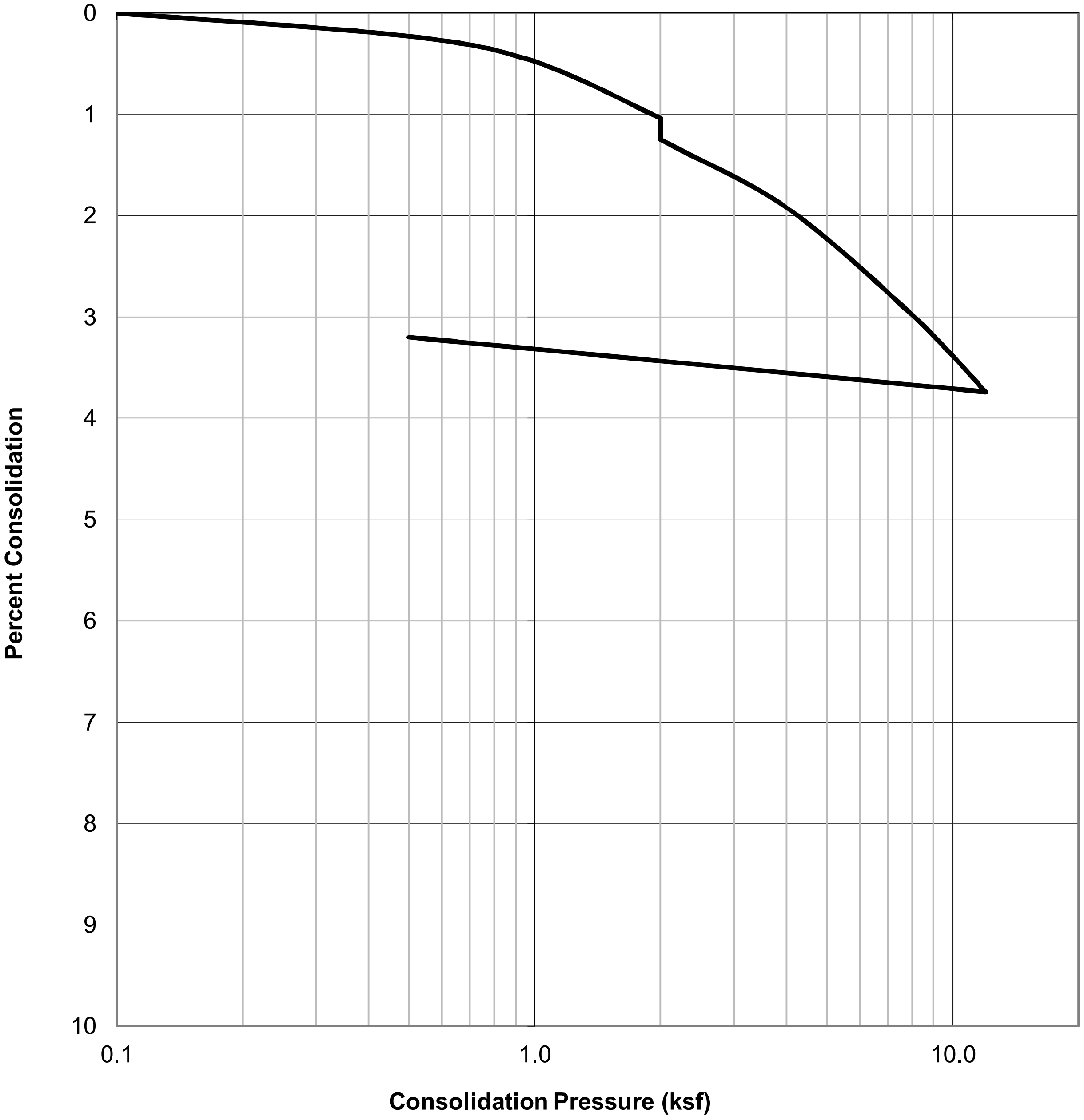
Project No.: W1111-06-01

465-577 SOUTH ARROYO PARKWAY
PASADENA, CALIFORNIA

JULY 2021

Figure B28

WATER ADDED AT 2.0 KSF



SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B4@70	Reddish Brown Silty Gravel (GM)	113.3	9.3	14.0

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CONSOLIDATION TEST RESULTS

ASTM D-2435

Checked by: JJK

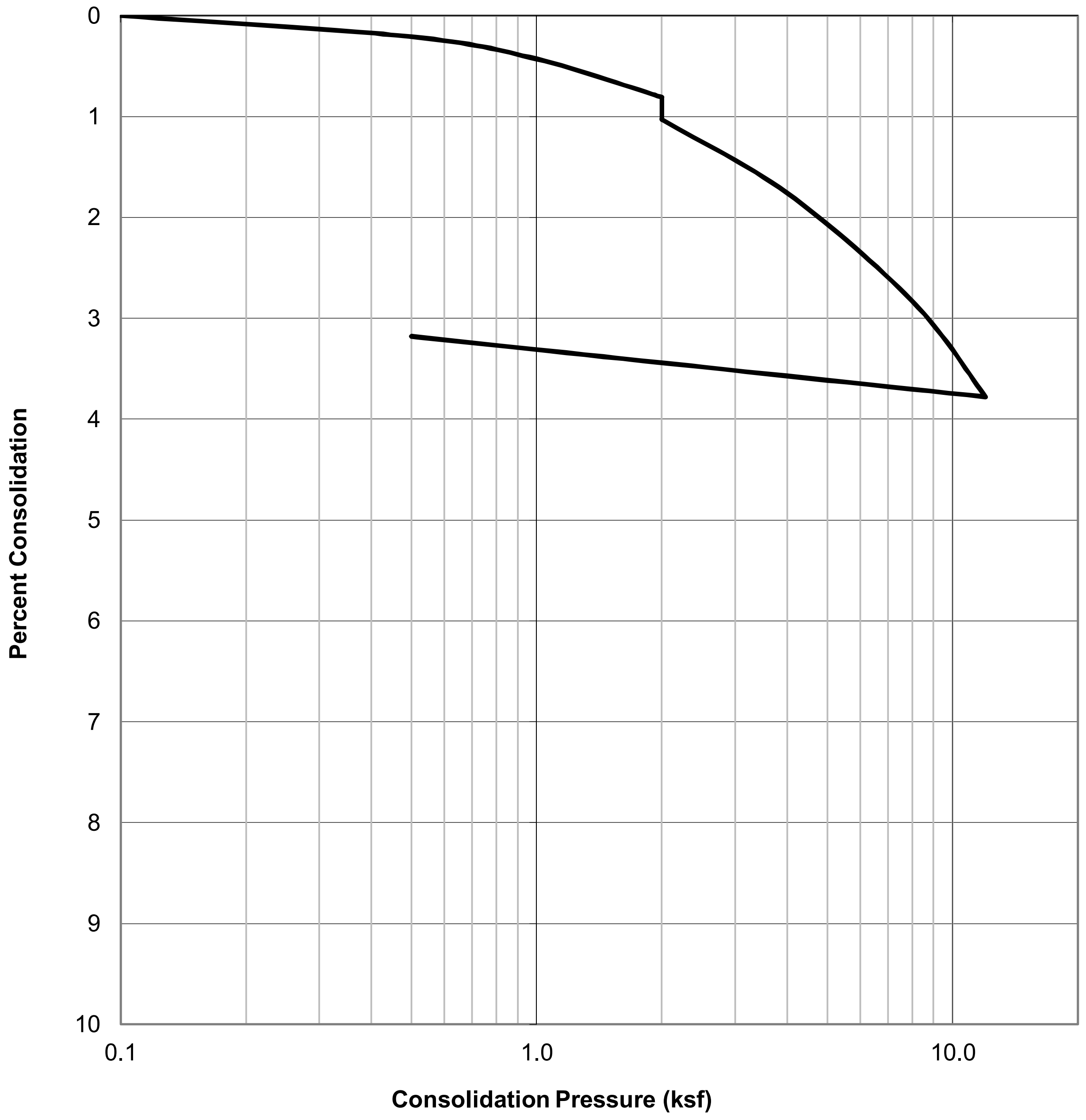
Project No.: W1111-06-01

465-577 SOUTH ARROYO PARKWAY
PASADENA, CALIFORNIA

JULY 2021

Figure B29

WATER ADDED AT 2.0 KSF



SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B4@80	Reddish Brown Silty Sand (SM)	98.4	16.9	23.8

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CONSOLIDATION TEST RESULTS

ASTM D-2435

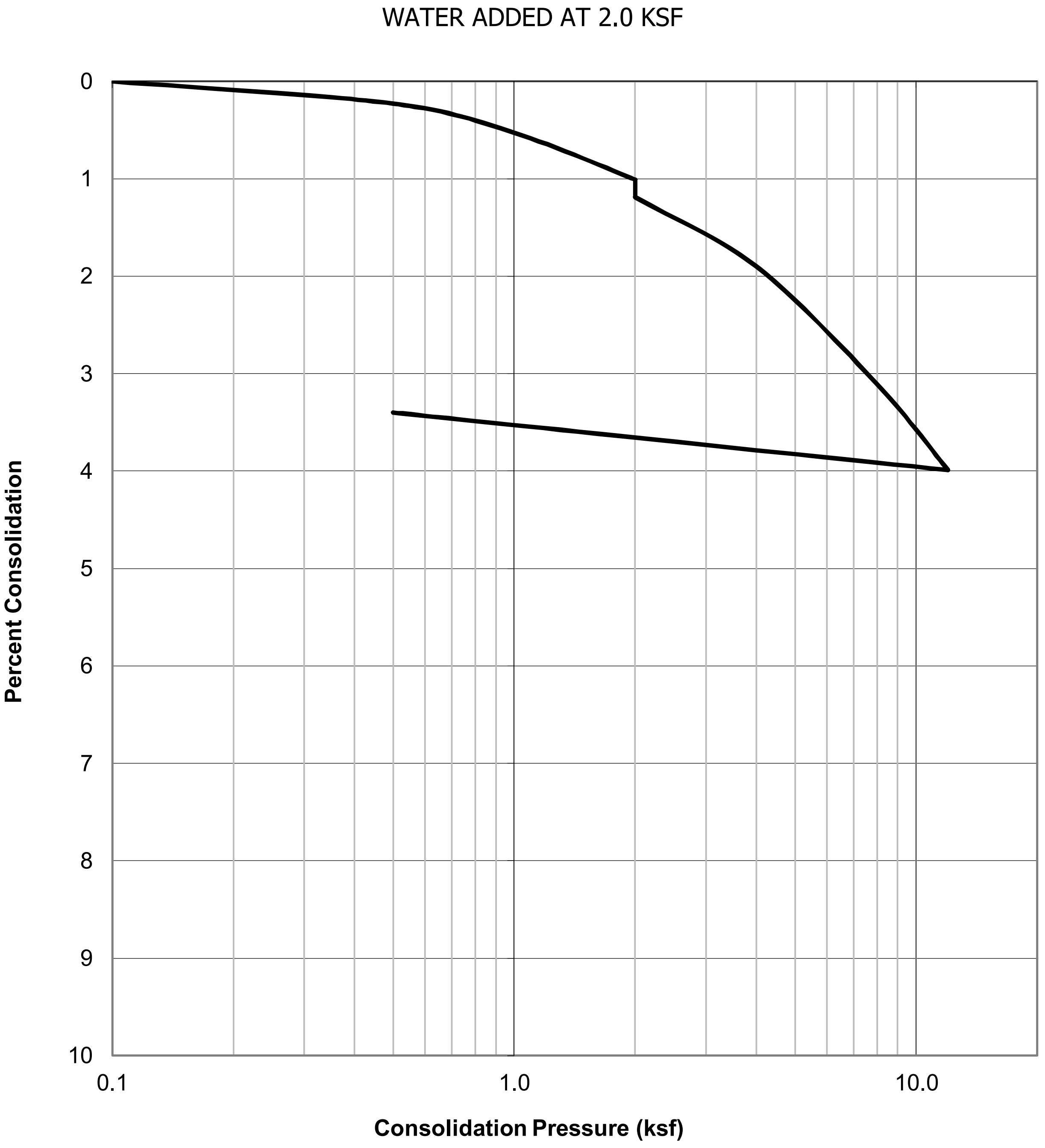
Checked by: JJK

Project No.: W1111-06-01

465-577 SOUTH ARROYO PARKWAY
PASADENA, CALIFORNIA

JULY 2021

Figure B30



SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B4@90	Brown Poorly Graded Sand (SP)	112.1	10.5	14.1

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CONSOLIDATION TEST RESULTS

ASTM D-2435

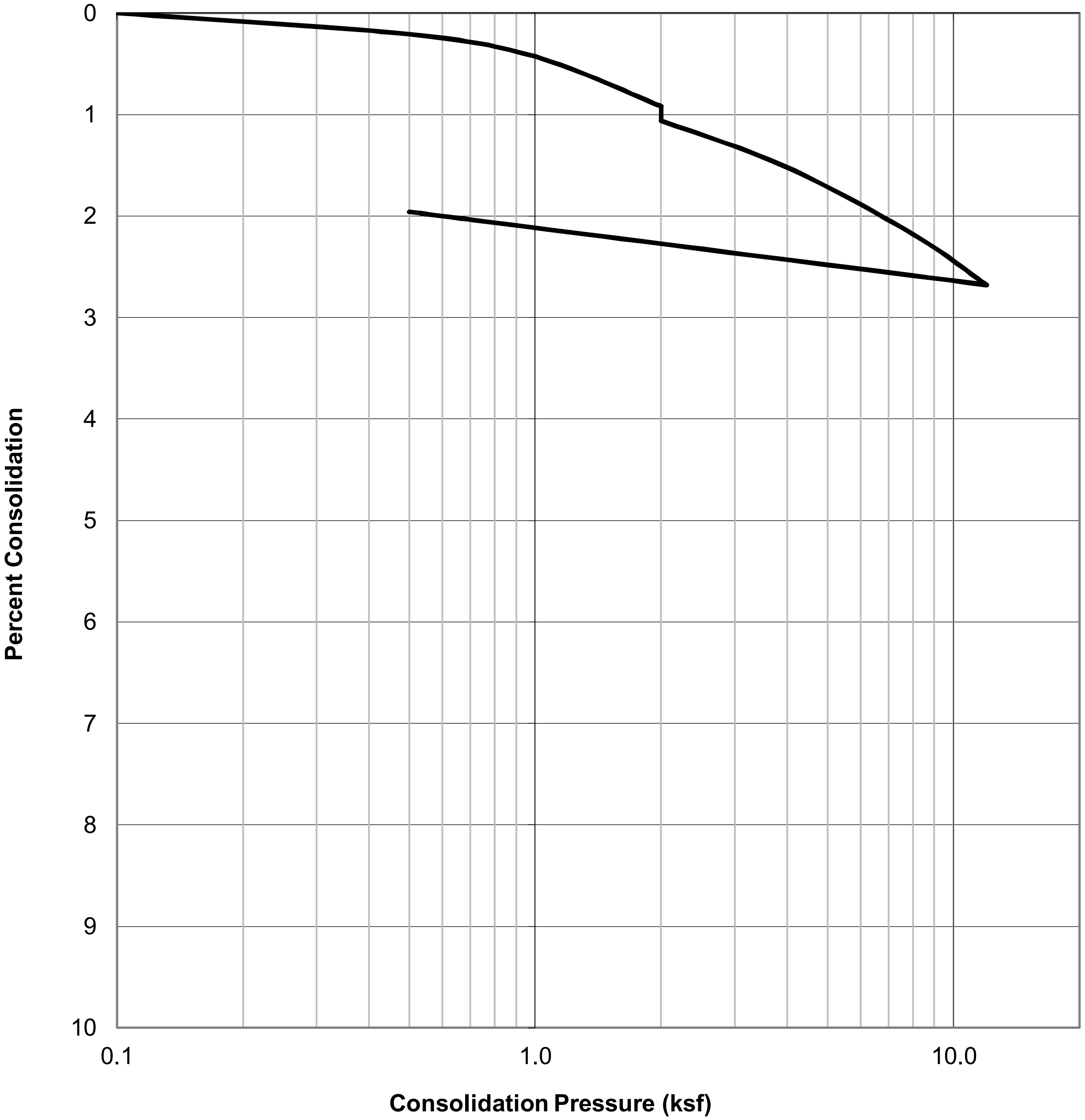
Checked by: JJK

Project No.: W1111-06-01

465-577 SOUTH ARROYO PARKWAY
PASADENA, CALIFORNIA

JULY 2021

WATER ADDED AT 2.0 KSF



SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B5@30	Reddish Brown Silty Sand (SM)	119.2	13.3	12.2

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CONSOLIDATION TEST RESULTS

ASTM D-2435

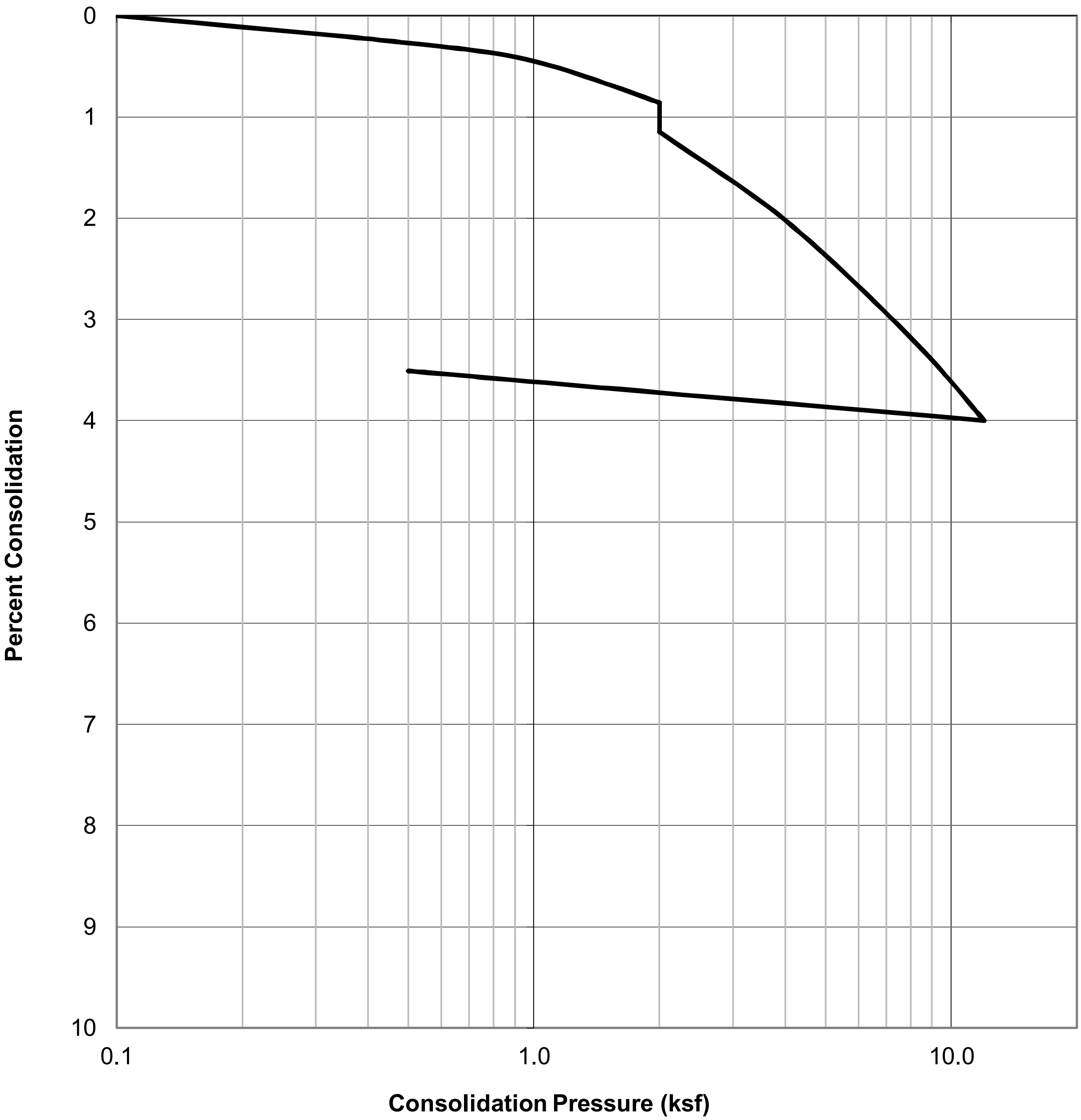
Checked by: JJK

Project No.: W1111-06-01

465-577 SOUTH ARROYO PARKWAY
PASADENA, CALIFORNIA

JULY 2021

Figure B32


WATER ADDED AT 2.0 KSF

SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B5@40	Brown Poorly Graded Sand (SP)	114.5	3.9	13.1

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CONSOLIDATION TEST RESULTS

ASTM D-2435

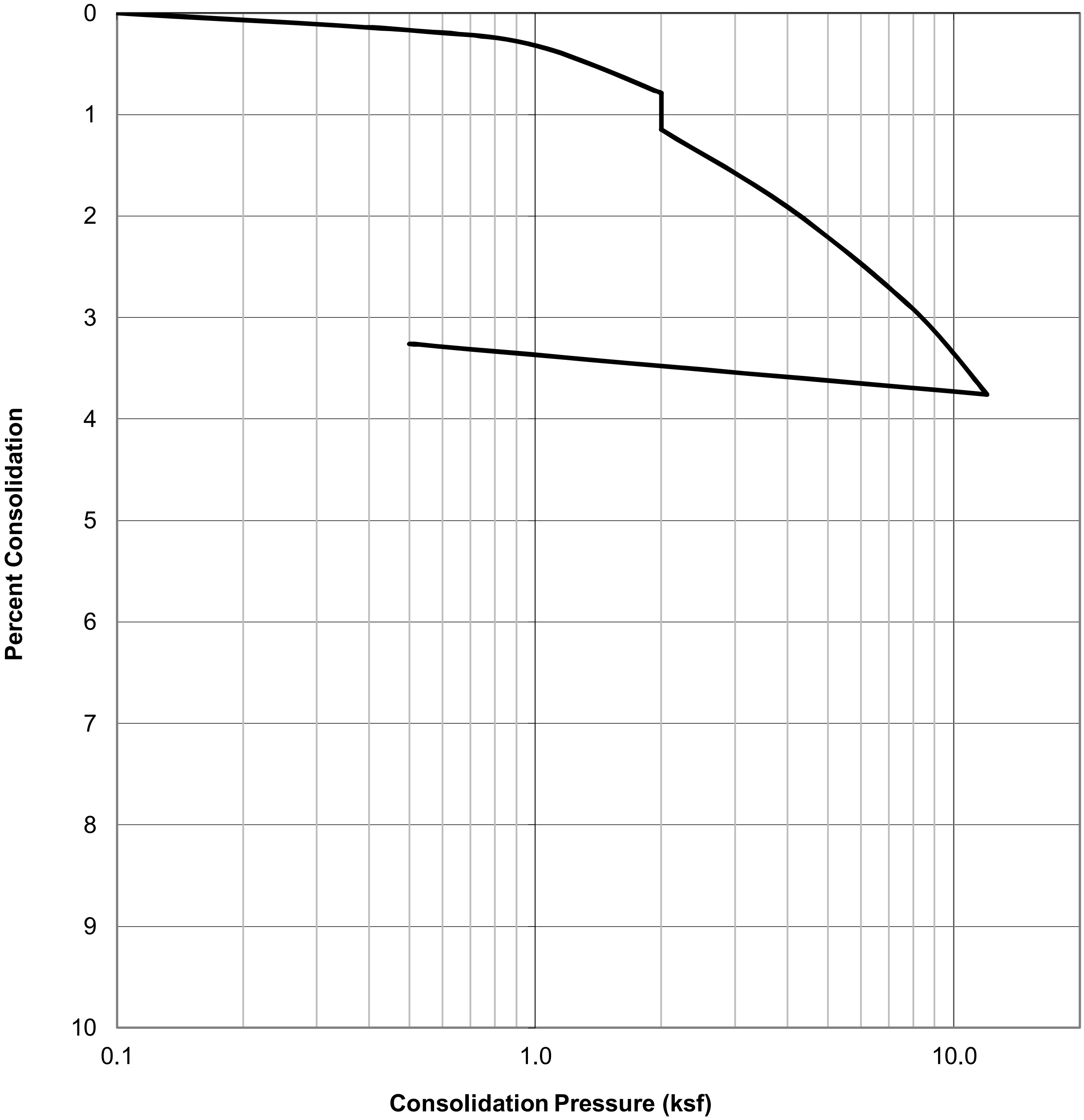
Checked by: JJK

Project No.: W1111-06-01

465-577 SOUTH ARROYO PARKWAY
PASADENA, CALIFORNIA

JULY 2021

WATER ADDED AT 2.0 KSF



SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B5@45	Brown Poorly Graded Sand (SP)	109.1	3.2	13.9

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CONSOLIDATION TEST RESULTS

ASTM D-2435

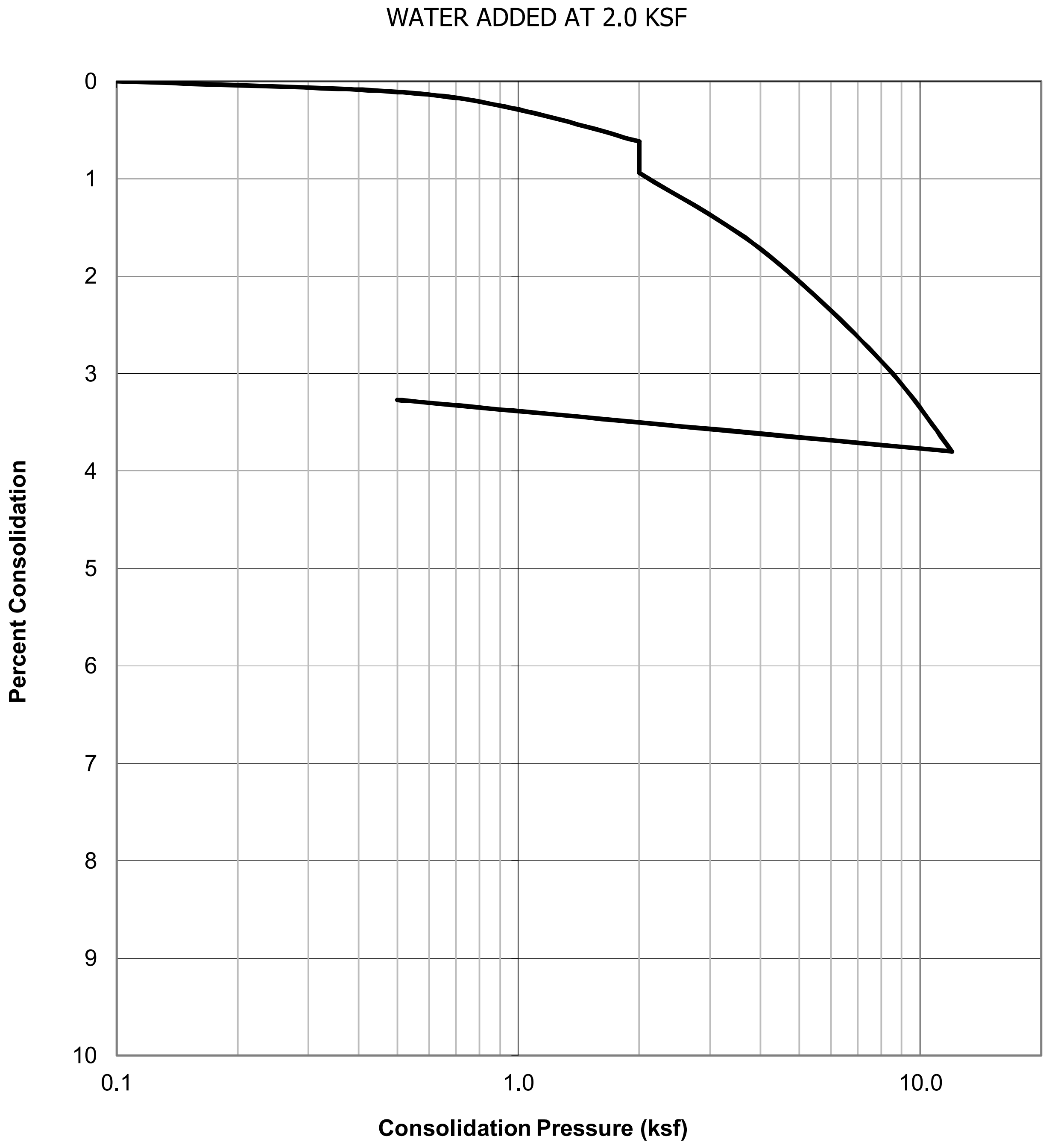
Checked by: JJK

Project No.: W1111-06-01

465-577 SOUTH ARROYO PARKWAY
PASADENA, CALIFORNIA

JULY 2021

Figure B34



SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B5@50	Brown Poorly Graded Sand (SP)	106.9	2.7	14.0

GEOCON

CONSOLIDATION TEST RESULTS

ASTM D-2435

Checked by: JJK

Project No.: W1111-06-01

465-577 SOUTH ARROYO PARKWAY
PASADENA, CALIFORNIA

JULY 2021

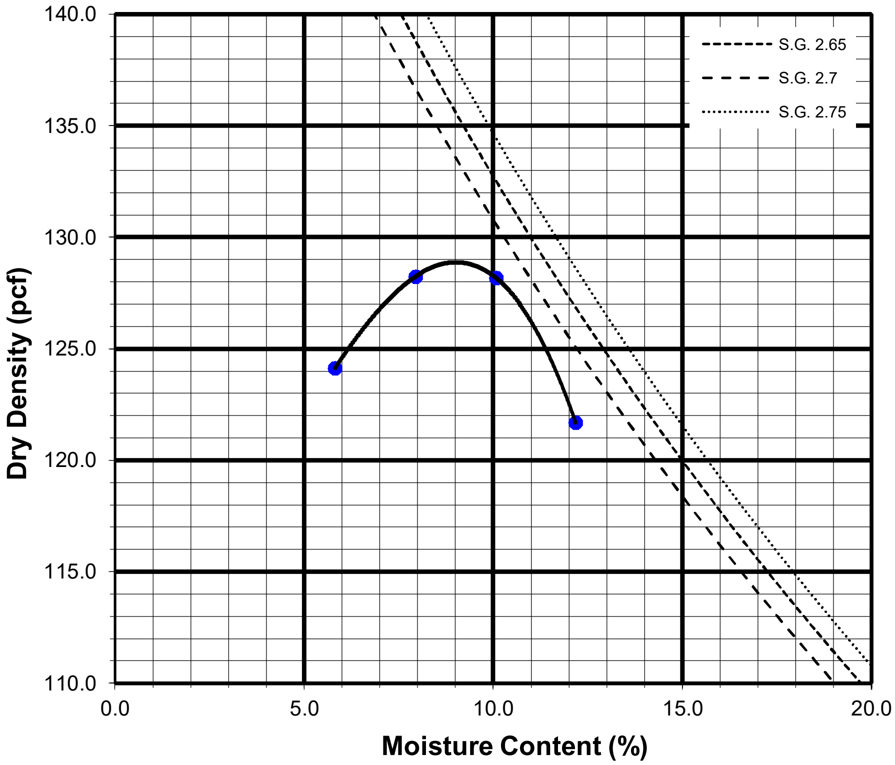
Sample No:

B1+B2+B3@0-5	Brown Silty Sand (SM)

TEST NO.		1	2	3	4	5	6
Wt. Compacted Soil + Mold	(g)	6132	6239	6279	6210		
Weight of Mold	(g)	4148	4148	4148	4148		
Net Weight of Soil	(g)	1984	2091	2131	2062		
Wet Weight of Soil + Cont.	(g)	718.3	754.7	737.3	660.6		
Dry Weight of Soil + Cont.	(g)	686.8	709.9	682.0	602.4		
Weight of Container	(g)	145.5	146.0	132.9	124.3		
Moisture Content	(%)	5.8	7.9	10.1	12.2		
Wet Density	(pcf)	131.3	138.4	141.1	136.5		
Dry Density	(pcf)	124.1	128.2	128.2	121.7		

Maximum Dry Density (pcf) **129.0**

Optimum Moisture Content (%) **9.0**



Preparation Method: A

MODIFIED COMPACTION TEST OF SOILS

ASTM D-1557

Checked by: JJK

Project No.: W1111-06-01

465-577 SOUTH ARROYO PARKWAY
PASADENA, CALIFORNIA

JULY 2021

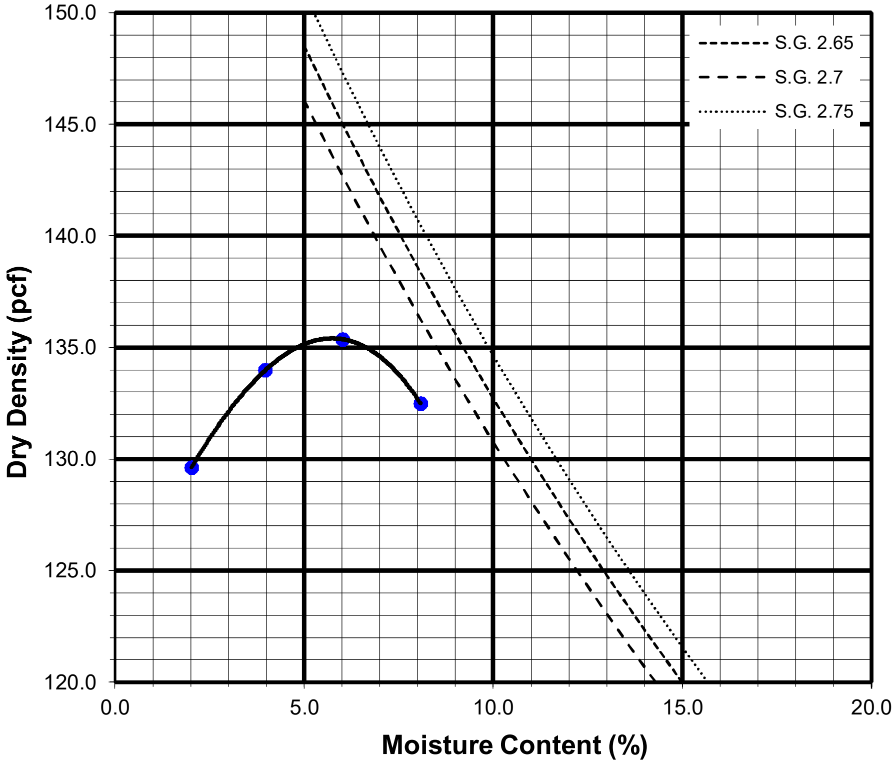
Sample No:

B4@35-45'	Brown Sand (SP)

TEST NO.		1	2	3	4	5	6
Wt. Compacted Soil + Mold	(g)	6141	6248	6312	6307		
Weight of Mold	(g)	4142	4142	4142	4142		
Net Weight of Soil	(g)	1999	2106	2170	2165		
Wet Weight of Soil + Cont.	(g)	686.4	662.2	737.2	683.6		
Dry Weight of Soil + Cont.	(g)	675.7	642.5	703.6	641.9		
Weight of Container	(g)	146.3	146.5	145.5	126.0		
Moisture Content	(%)	2.0	4.0	6.0	8.1		
Wet Density	(pcf)	132.2	139.3	143.5	143.2		
Dry Density	(pcf)	129.6	134.0	135.4	132.5		

Maximum Dry Density (pcf)	**135.5**
Bulk Specific Gravity (dry)	**2.65**
Corrected Maximum Dry Density (pcf)	**139.0**

Optimum Moisture Content (%)	**5.5**
Oversized Fraction (%)	**13.0**
Corrected Moisture Content (%)	**5.0**



Preparation Method: B

GEOCON

MODIFIED COMPACTION TEST OF SOILS

ASTM D-1557

Checked by: JJK

Project No.: W1111-06-01

465-577 SOUTH ARROYO PARKWAY
PASADENA, CALIFORNIA

JULY 2021

Figure B37

B1+B2+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)	783.0	801.3
Wt. of Mold	(gm)	368.4	368.4
Specific Gravity	(Assumed)	2.7	2.7
Wet Wt. of Soil + Cont.	(gm)	484.5	801.3
Dry Wt. of Soil + Cont.	(gm)	460.2	381.0
Wt. of Container	(gm)	184.5	368.4
Moisture Content	(%)	8.8	13.6
Wet Density	(pcf)	125.1	130.4
Dry Density	(pcf)	114.9	114.8
Void Ratio		0.5	0.5
Total Porosity		0.3	0.3
Pore Volume	(cc)	65.9	66.8
Degree of Saturation	(%) [S_{meas}]	51.3	77.7

Date	Time	Pressure (psi)	Elapsed Time (min)	Dial Readings (in.)
1/23/2020	10:00	1.0	0	0.258
1/23/2020	10:10	1.0	10	0.2575
Add Distilled Water to the Specimen				
1/24/2020	10:00	1.0	1430	0.262
1/24/2020	11:00	1.0	1490	0.262

Expansion Index (EI meas) =	4.5
Expansion Index (Report) =	**5**

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

* Reference: 2016 California Building Code, Section 1803.5.3

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

GEOCON

EXPANSION INDEX TEST RESULTS

ASTM D-4829

Checked by: JJK

Project No.: W1111-06-01

465-577 SOUTH ARROYO PARKWAY
PASADENA, CALIFORNIA

JULY 2020

Figure B38

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

Sample No.	**pH**	**Resistivity (ohm centimeters)**
B1+B2+B3 @ 0-5'	7.9	3400 (Moderately Corrosive)

SUMMARY OF LABORATORY CHLORIDE CONTENT TEST RESULTS
EPA NO. 325.3

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

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

Sample No.	**Water Soluble Sulfate (% SQ_4)**	**Sulfate Exposure***
B1+B2+B3 @ 0-5'	0.000	S0
B4 @ 50-55'	0.000	S0

GEOCON

CORROSIVITY TEST RESULTS

Checked by: JJK

Project No.: W1111-06-01

555 SOUTH ARROYO PARKWAY
PASADENA, CALIFORNIA

JULY 2021

Figure B39