# GEOTECHNICAL INVESTIGATION

## PROPOSED RESIDENTIAL DEVELOPMENT
## TENTATIVE TRACT 20443
## 61650 ALTA LOMA DRIVE
## JOSHUA TREE, CALIFORNIA
## APN: 0602-361-04

GEOCON
WEST, INC.

GEOTECHNICAL
ENVIRONMENTAL
MATERIALS

**PREPARED FOR**

**LOVEMORE INVESTMENTS, LLC**
**LOS ANGELES, CALIFORNIA**

**PROJECT NO. W1558-99-01**

**JULY 7, 2022**

GEOCON
WEST, INC.
GEOTECHNICAL ■ ENVIRONMENTAL ■ MATERIALS

Project No. W1558-99-01
July 7, 2022

Mr. Axel Cramer
LoveMore Investments, LLC
2745 Tesla Avenue
Los Angeles, California 90039

Subject: GEOTECHNICAL INVESTIGATION
PROPOSED RESIDENTIAL DEVELOPMENT
TENTATIVE TRACT 20443
61650 ALTA LOMA DRIVE, JOSHUA TREE, CALIFORNIA
APN: 0602-361-04

Dear Mr. Cramer:

In accordance with our proposal dated March 21, 2022, we have prepared this geotechnical investigation report for the proposed residential tract development located at 61650 Alta Loma drive in the community of Joshua Tree, County of San Bernardino, 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.**

John Stapleton
Staff Engineer

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# GEOTECHNICAL INVESTIGATION

## 1. PURPOSE AND SCOPE

This report presents the results of a geotechnical investigation for the proposed residential tract development located at 61650 Alta Loma Drive in the community of Joshua Tree, San Bernardino County, California (see Vicinity Map, Figure 1). The purpose of the investigation was to evaluate subsurface soil and geologic conditions underlying the site and, based on conditions encountered, to provide conclusions and recommendations pertaining to the geotechnical aspects of design and construction.

The scope of this investigation included a site reconnaissance, field exploration, laboratory testing, engineering analysis, and the preparation of this report. The site was explored on May 26, 2022, and May 31, 2022, by excavating twelve 8-inch diameter borings to a maximum depth of 15½ feet below the existing ground surface utilizing a combination of truck-mounted and limited access track-mounted hollow-stem auger drilling machines. The approximate locations of the exploratory borings are depicted on the Site Plan (see Figure 2). A detailed discussion of the field investigation, including the boring logs, is presented in Appendix A.

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

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

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

## 2. SITE AND PROJECT DESCRIPTION

The subject site is located at 61650 Alta Loma Drive in the community of Joshua Tree in unincorporated San Bernardino County, California. The site consists of an approximate 33.4-acre vacant undeveloped lot with various protected plant species. The site is bounded by single story residential structures and Alta Vista Drive to the north, by Sunset Road to the east, by Alta Loma Drive to the south, and by Hillview Road to the west. The site gently slopes to the north with approximately 54 feet of elevation change with a slope gradient of approximately 5½ percent. The center portion of the site was graded to achieve a flat surface for storage and/or trailer parking. Surface water drainage at the site appears to be by sheet flow along the existing ground contours and vegetation consists of trees and brushes all throughout the site.

Based on the information provided by the Client, it is our understanding that the proposed development will consist of a 75-unit single-family tract home development constructed at or near present grade. Plans showing the proposed finished building pad elevations are not available at the time of this report. Plans depicting the existing site conditions and layout of the proposed development are indicated 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 structure will be up to 100 kips, and wall loads will be up to 1.5 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 site is located at the Mojave Desert and Transverse Ranges Geomorphic Provinces boundary. The Mojave Desert Geomorphic Province (MDGP) is characterized by a broad interior region of isolated mountain ranges separated by vast expanses of desert plains. The Province is wedged between the Garlock fault that separates the Basin and Range Province to the north and the San Andreas fault that separates the Transverse Ranges Province to the southwest. There are two main fault trends within the MDGP, the prominent northwest-southeast trend that includes the Helendale-South Lockhart, Lenwood Lockhart, Calico-Hidalgo faults among others; and a secondary east-west trend that includes the Pinto Mountain, Garlock, Cady, and Coyote Lake faults among others. The Transverse Ranges Geomorphic Province is characterized by east-west trending step mountains ranges and valleys. North-south compression of the province makes it one of the most rapidly rising regions on the planet. Specifically, the site is located on alluvial fan deposits emanating from the San Bernardino and Little San Bernardino mountains to the west and southwest of the site, respectively, as well as numerous isolated mountains north, east, and south of the site. Thick deposits of sand, gravel, cobble, and boulder sediments underlie the site.

## 4. SOIL AND GEOLOGIC CONDITIONS

Based on our field investigation and published geologic maps of the area, the site is underlain by Pleistocene age alluvial fan deposits. Detailed stratigraphic profiles of the materials encountered at the site are provided on the boring log in Appendix A.

### 4.1 Alluvium

The site is underlain by alluvial deposits consisting of light reddish brown to brown and olive brown interbeds of poorly-graded sand and well-graded sand with varying amounts of fine- to coarse-sized gravel and minimal amounts of silt. An increase of well-graded sand was encountered in borings located in the southern portion of the site where elevation was higher. The soils are characterized as dry to slightly moist and loose to very dense.

The results of the laboratory testing indicate that the upper alluvial soils are subject to excessive hydro-consolidation upon saturation (see Figures B11 though B26). Hydro-consolidation is the tendency of a soil structure to collapse upon saturation, resulting in the overall settlement of the effected soils and any overlying soils or foundations supported therein. The approximate depth to competent alluvial soils is summarized in the table below.

**DEPTH TO COMPETENT ALLUVIUM**

| Boring Number | Depth to Competent Alluvium (feet) |
|---|---|
| B1 | 4 |
| B2 | 5 |
| B3 | 6 |
| B4 | 7 |
| B5 | 4 |
| B6 | 10 |
| B7 | 8 |
| B8 | 6 |
| B9 | 6 |
| B10 | 6 |
| B11 | 11 |
| B12 | 9 |

## 5. GROUNDWATER

Review of groundwater monitoring well data provided by the U.S. Geologic Survey (USGS, 2022) indicates closest monitoring well to the site is Local Well No. JTUZ-1 (State Well No. 341325N1163184W001), located 0.75 mile to the north of the site. Monitoring data from this well is available for the period from June 2008 through March 2018. During this time, the depth to groundwater has ranged from 517 to 523 feet beneath the ground surface. The most recent groundwater level measurement was recorded on March 24, 2018, and the depth to groundwater was approximately greater than 520 feet below the ground surface.

Groundwater was not encountered in our field explorations, drilled to a maximum depth of 15½ feet below the ground surface. Based on the reported historic groundwater levels in the site vicinity (USGS, 2022), the lack of groundwater encountered in our borings, and the depth of proposed construction, static groundwater is neither expected to be encountered during construction, nor have a detrimental effect on the project. However, groundwater seepage may be encountered during construction. 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 7.17).

## 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.

According to the San Bernardino Countywide Plan (2007) the site is not within a state-designated Alquist-Priolo Earthquake Fault Zone for surface fault rupture hazards. No Holocene-active or pre-Holocene faults with the potential for surface fault rupture are known to pass directly beneath the site. Therefore, the potential for surface rupture due to faulting occurring beneath the site during the design life of the proposed development is considered low. However, the site is located in the seismically active Southern California region and could be subjected to moderate to strong ground shaking in the event of an earthquake on one of the many Holocene-active Southern California faults. The faults in the vicinity of the site are shown in Figure 3, Regional Fault Map.

The closest surface trace of an active fault to the site is the Pinto Mountain Fault Zone located approximately 1.2 miles to the north (USGS, 2006). Other nearby active faults include the Eureka Peak Fault, the Burnt Mountain Fault Zone, the Camp Rock Fault Zone, and the Long Canyon Fault located approximately 3.4 miles to the southwest, 5.3 miles to the southwest, 7.2 miles to the northeast, and 8.0 miles to the southwest of the site, respectively (Ziony and Jones, 1989). The San Andreas Fault is located 15½ miles to the southwest.

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

## 6.2 Seismicity

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

**LIST OF HISTORIC EARTHQUAKES**

| Earthquake (Oldest to Youngest) | Date of Earthquake | Magnitude | Distance to Epicenter (Miles) | Direction to Epicenter |
|---|---|---|---|---|
| Near Redlands | July 23, 1923 | 6.3 | 54 | W |
| Long Beach | March 10, 1933 | 6.4 | 101 | WSW |
| Tehachapi | July 21, 1952 | 7.5 | 165 | WNW |
| San Fernando | February 9, 1971 | 6.6 | 120 | W |
| Whittier Narrows | October 1, 1987 | 5.9 | 101 | W |
| Sierra Madre | June 28, 1991 | 5.8 | 97 | W |
| Landers | June 28, 1992 | 7.3 | 9 | NW |
| Big Bear | June 28, 1992 | 6.4 | 30 | W |
| Northridge | January 17, 1994 | 6.7 | 127 | W |
| Hector Mine | October 16, 1999 | 7.1 | 33 | N |
| Ridgecrest | July 5, 2019 | 7.1 | 135 | NW |

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

## 6.3 Seismic Design Criteria

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

**2019 CBC SEISMIC DESIGN PARAMETERS**

| Parameter | Value | 2019 CBC Reference |
|---|---|---|
| Site Class | D | Section 1613.2.2 |
| $MCE_R$ Ground Motion Spectral Response Acceleration – Class B (short), $S_S$ | 2.044g | Figure 1613.2.1(1) |
| $MCE_R$ Ground Motion Spectral Response Acceleration – Class B (1 sec), $S_1$ | 0.732g | Figure 1613.2.1(2) |
| Site Coefficient, $F_A$ | 1.0 | Table 1613.2.3(1) |
| Site Coefficient, $F_V$ | 1.7* | Table 1613.2.3(2) |
| Site Class Modified $MCE_R$ Spectral Response Acceleration (short), $S_{MS}$ | 2.044g | Section 1613.2.3 (Eqn 16-36) |
| Site Class Modified $MCE_R$ Spectral Response Acceleration – (1 sec), $S_{M1}$ | 1.245g* | Section 1613.2.3 (Eqn 16-37) |
| 5% Damped Design Spectral Response Acceleration (short), $S_{DS}$ | 1.362g | Section 1613.2.4 (Eqn 16-38) |
| 5% Damped Design Spectral Response Acceleration (1 sec), $S_{D1}$ | 0.83g* | Section 1613.2.4 (Eqn 16-39) |

**Note:**

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

The table below presents the mapped maximum considered geometric mean ($MCE_G$) seismic design parameters for projects located in Seismic Design Categories of D through F in accordance with ASCE 7-16. 12

**ASCE 7-16 PEAK GROUND ACCELERATION**

| Parameter | Value | ASCE 7-16 Reference |
|---|---|---|
| Mapped $MCE_G$ Peak Ground Acceleration, PGA | 0.87g | Figure 22-7 |
| Site Coefficient, $F_{PGA}$ | 1.1 | Table 11.8-1 |
| Site Class Modified $MCE_G$ Peak Ground Acceleration, $PGA_M$ | 0.956g | Section 11.8.3 (Eqn 11.8-1) |

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

Deaggregation of the MCE peak ground acceleration was performed using the USGS online Unified Hazard Tool, 2014 Conterminous U.S. Dynamic edition (v4.2.0). The result of the deaggregation analysis indicates that the predominant earthquake contributing to the MCE peak ground acceleration is characterized as a 6.71 magnitude event occurring at a hypocentral distance of 7.2 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 6.6 magnitude occurring at a hypocentral distance of 10.34 kilometers from the site.

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

## 6.4 Liquefaction Potential

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

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

According to the San Bernardino Countywide Plan (2007), the site is not located in an area designated as having a potential for liquefaction. Based on the historic high groundwater levels in the site vicinity, the lack of groundwater encountered in our borings, and depth to groundwater recorded in onsite water wells, it is our opinion that the potential for liquefaction of the soils underlying the site is very low.

## 6.5 Slope Stability

The topography of the site generally slopes to the north with a 5% slope gradient. The topography in the vicinity of the site slopes gently to the northeast. The San Bernardino Countywide Plan (2007) indicates that the site is not located within an area identified as having a potential for slope instability. There are no known landslides near the site, nor is the site in the path of any known or potential landslides. The potential for slope instability or landslides adversely affecting the proposed project is considered low.

## 6.6 Earthquake-Induced Flooding

Earthquake-induced flooding is inundation caused by failure of dams or other water-retaining structures due to earthquakes. A review of the San Bernardino Countywide Plan Policy Plan (2020) indicates that the site is not located within a potential inundation area for an earthquake-induced dam failure. Therefore, the probability of earthquake-induced flooding is considered very low.

## 6.7 Tsunamis, Seiches and Flooding

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

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

The site is not within a 100-year flood zone or a 500-year flood zone. The potential for flooding to adversely impact the site is considered low (San Bernardino Countywide Plan, 2022).

## 6.8 Oil Fields and Methane

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

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

## 6.9 Subsidence

Subsidence occurs when a large portion of land is displaced vertically, usually due to the withdrawal of groundwater, oil, or natural gas. Soils that are particularly subject to subsidence include those with high silt or clay content. The site is not located within an area of known ground subsidence. No large-scale extraction of groundwater, gas, oil, or geothermal energy is occurring or planned at the site. There appears to be little or no potential for ground subsidence due to withdrawal of fluid or gas 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 site improvements provided the recommendations presented herein are followed and implemented during design and construction.

7.1.2 Existing artificial fill was not encountered during the site investigation. However, areas of the site appear to have been previously graded into a roughly level pad for prior site use. Deeper fill may exist in other areas of the site that were not directly explored. 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 At this time, plans depicting the proposed grading and finish pad elevations are not available. It is anticipated that both cuts and fills will be required to achieve proposed finish grade elevations, and in order to provide minimum drainage requirements across proposed building lot lines.

7.1.4 The results of the laboratory testing indicate that the upper alluvial soils are subject to excessive hydro-consolidation upon saturation (see Figures B11 though B26). Hydro-consolidation is the tendency of a soil structure to collapse upon saturation, resulting in the overall settlement of the effected soils and any overlying soils or foundations supported therein.

7.1.5 Based on the potential for hydro-consolidation, maintaining proper surface drainage will be vital for future performance of foundations and site improvements. Recommendations for drainage are provided in the surface drainage section of this report (see Section 7.17).

7.1.6 Based on these considerations, it is recommended that existing upper site soils in the proposed building footprint areas be excavated and properly compacted. The anticipated depths of the required removals in the areas explored are indicated on the Site Plan (see Figure 2). Deeper excavations should be conducted as necessary to remove any encountered soft alluvial soils as necessary at the direction of the Geotechnical Engineer (a representative of Geocon). Proposed building foundations should be underlain by a minimum of 3 feet of newly placed engineered fill. The excavation should extend laterally a minimum distance of 5 feet beyond the building footprint area, including building appurtenances, or a distance equal to the depth of fill below the foundation, whichever is greater. The limits of existing fill and/or soft alluvial soils removal will be verified by the Geocon representative during site grading activities.

7.1.7 Additional grading should be conducted as necessary to maintain the required 3 feet of newly placed engineered fill below foundations. The grading contractor should verify all bottom of footing elevations prior to commencement of grading activities to ensure that grading is conducted deep enough to provide the required three foot of engineering fill below foundations.

7.1.8 Subsequent to the recommended grading, the proposed structure may be supported on a conventional spread foundation or a post-tension foundation system deriving support in newly placed engineered fill. Recommendations for the design of conventional and post-tension foundations system are provided in Sections 7.6 and 7.7.

7.1.9 All excavations must be observed and approved in writing by the Geotechnical Engineer (a representative of Geocon). Prior to placing any fill, the upper 12 inches of the excavation bottom must be scarified, moistened, and proof-rolled with heavy equipment in the presence of the Geotechnical Engineer (a representative of Geocon West, Inc.).

7.1.10 It is anticipated that stable excavations for the recommended grading associated with the proposed structure can be achieved with sloping measures. However, if excavations in close proximity to an adjacent property line and/or structure are required, special excavation measures, such as slot-cutting, may be necessary in order to maintain lateral support of offsite improvements. Excavation recommendations are provided in the *Temporary Excavations* section of this report (Section 7.15).

7.1.11 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 structures, 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, such as adjacent to property lines, foundations may derive support directly in the competent undisturbed alluvial soils found at or below a depth of 24 inches below the existing ground surface, and should be deepened as necessary to maintain a minimum 12-inch embedment into the undisturbed alluvial soils. 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.

7.1.12 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 7.12).

7.1.13 Based on the results of the percolation testing performed at the site, a stormwater infiltration system is considered feasible for this project. Recommendations for infiltration are provided in *Stormwater Infiltration* section of this report (see Section 7.16).

7.1.14 Once finished building pad elevations are established, and 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.

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

7.1.16 The most recent ASTM standards apply to this project and must be utilized, even if older ASTM standards are referenced in this report.

## 7.2 Soil and Excavation Characteristics

7.2.1 The in-situ soils can be excavated with moderate effort using conventional excavation equipment. Caving should be anticipated in unshored vertical excavations since soils are primarily granular. The contractor should be aware that formwork may be required to prevent caving of shallow spread foundation excavations.

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

7.2.3 All onsite excavations must be conducted in such a manner that potential surcharges from existing structures, construction equipment, and vehicle loads are resisted. The surcharge area may be defined by a 1:1 projection down and away from the bottom of an existing foundation or vehicle load. Penetrations below this 1:1 projection will require special excavation measures such as sloping and possibly shoring. Excavation recommendations are provided in the *Temporary Excavations* section of this report (see Section 7.15).

7.2.4 The upper 10 feet of existing site soils encountered during the investigation near the ground surface are considered to have a “very low” (EI=0) expansive potential and are classified as “non-expansive” based on the 2019 California Building Code (CBC) Section 1803.5.3. Recommendations presented herein assume that the building foundations and slabs will derive support in these materials.

## 7.3 Minimum Resistivity, pH and Water-Soluble Sulfate

7.3.1 Potential of Hydrogen (pH) and resistivity testing as well as chloride content testing were performed on representative samples of soil to generally evaluate the corrosion potential to surface utilities. The tests were performed in accordance with California Test Method Nos. 643 and 422 and indicate that the soils are considered “mildly corrosive to moderately corrosive” with respect to corrosion of buried ferrous metals on site. The results are presented in Appendix B (Figure B38 and B39) and should be considered for design of underground structures. In order to prevent premature corrosion of buried metal pipes, plastic piping should be considered where possible.

7.3.2 Laboratory tests were performed on representative samples of the site soils to measure the percentage of water-soluble sulfate content. Results from the laboratory water-soluble sulfate tests are presented in Appendix B (Figure B38 and 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 preparation of the building pad and subgrade, excavation of site soils for proposed foundations, and utility trenches, and placement of backfill for utility trenches.

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

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

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

7.4.5 As a minimum, it is recommended that the upper existing earth materials within the proposed building footprint areas be excavated and properly compacted for foundation and slab support. The anticipated depths of the required removals in the areas explored are indicated on the Site Plan (see Figure 2). Deeper excavations should be conducted as necessary to remove all artificial fill or soft alluvial soil at the direction of the Geotechnical Engineer (a representative of Geocon). Proposed building foundations should be underlain by a minimum of three feet of newly placed engineered fill. The excavation should extend laterally a minimum distance of five feet beyond the building footprint area, including building appurtenances, or a distance equal to the depth of fill below the foundation, whichever is greater. The limits of existing fill and/or soft alluvial soils removal will be verified by the Geocon representative during site grading activities.

7.4.6 Additional grading should be conducted as necessary to maintain the required three feet of newly placed engineered fill below foundations. The grading contractor should verify all bottom of footing elevations prior to commencement of grading activities to ensure that grading is conducted deep enough to provide the required three foot of engineering fill below foundations.

7.4.7 All excavations must be observed and approved in writing by the Geotechnical Engineer (a representative of Geocon). Prior to placing any fill, the upper 12 inches of the excavation bottom must be scarified, moistened, and proof-rolled with heavy equipment in the presence of the Geotechnical Engineer (a representative of Geocon West, Inc.).

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

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. 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 7.12).

7.4.10 It is anticipated that stable excavations for the recommended grading can be achieved with sloping measures. However, if excavations adjacent to a property line and/or structure are required, special excavation measures may be necessary in order to maintain lateral support of the existing improvements. Excavation recommendations are provided in the *Temporary Excavations* section of this report (Section 7.15).

7.4.11 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 proper compaction cannot be performed, foundations may derive support directly in the undisturbed alluvial soils generally found at or below a depth of 24 inches and should be deepened as necessary to maintain a minimum 12-inch embedment into the recommended bearing materials. If the soils exposed in the excavation bottom are soft or loose, compaction of the soils will be required prior to placing steel or concrete. Compaction of the foundation excavation bottom is typically accomplished with a compaction wheel or mechanical whacker and must be observed and approved by a Geocon representative.

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

7.4.13 All imported soils intended for use as engineered fill shall be observed, tested, and approved by Geocon West, Inc. prior to bringing soil to the site. Rocks larger than 6 inches in diameter shall not be used in the fill. If necessary, import soils used as structural fill should have an expansion index less than 20 and corrosivity properties that are equally or less detrimental to that of the existing onsite soils (see Figure B38 and B39). Import soils placed in the building area should be placed uniformly across the building pad or in a manner that is approved by the Geotechnical Engineer (a representative of Geocon).

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

## 7.5 Shrinkage

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

7.4.2 If import soils will be utilized in the building pad, the soils must be placed uniformly and at equal thickness at the direction of the Geotechnical Engineer (a representative of Geocon West, Inc.). Soils can be borrowed from non-building pad areas and later replaced with imported soils.

## 7.6 Foundation Design

7.6.1 Subsequent to the recommended grading, a conventional shallow spread foundation system may be utilized for support of the proposed structures provided foundations derive support in newly placed engineered fill. Foundations should be underlain by a minimum of 3 feet of newly placed engineered fill. All foundation excavations must be observed and approved by the Geotechnical Engineer (a representative of Geocon), prior to placing steel or concrete.

7.6.2 Continuous footings may be designed for an allowable bearing capacity of 2,250 pounds per square foot (psf), and should be a minimum of 12 inches in width, 18 inches in depth below the lowest adjacent grade, and 12 inches into the recommended bearing material.

7.6.3 Isolated spread foundations may be designed for an allowable bearing capacity of 2,500 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.

7.6.4 The allowable soil bearing pressure may be increased by 600 psf and 1,000 psf for each additional foot of foundation width and depth, respectively, up to a maximum allowable soil bearing pressure of 4,000 psf.

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

7.6.6 If depth increases are utilized for the exterior wall footings, this office should be provided a copy of the final construction plans so that the excavation recommendations presented herein could be properly reviewed and revised if necessary. Additional grading should be conducted as necessary in order to maintain the required 3-foot-thick blanket of engineered fill below foundations.

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

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

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

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

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

## 7.7 Post-Tensioned Foundation Recommendations

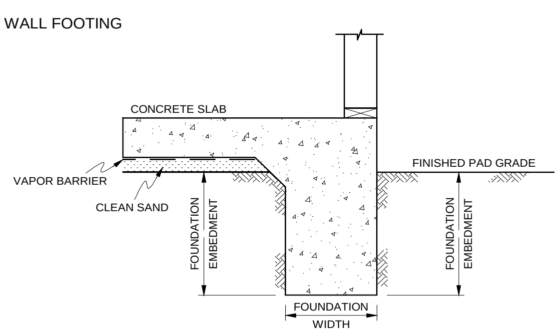
7.7.1 As an alternative, the proposed structures may also be supported on a post-tensioned foundation. Proposed post-tensioned foundations should be underlain by at least 3 feet of newly placed engineered fill. Additional grading should be conducted as necessary in order to maintain the required 3-foot-thick blanket of newly placed engineered fill below foundations.

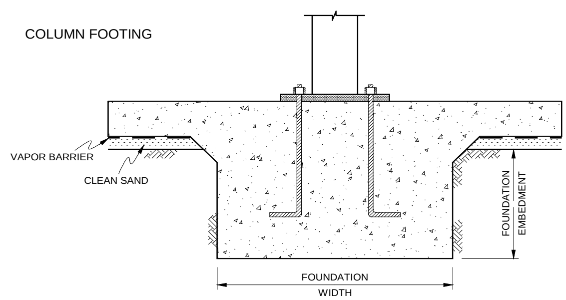
7.7.2 The post-tensioned system should be designed by a structural engineer experienced in post-tensioned slab design and design criteria of the Post-Tensioning Institute (PTI) DC 10.5-12 *Standard Requirements for Design and Analysis of Shallow Post-Tensioned Concrete Foundations on Expansive Soils* or *WRI/CRSI Design of Slab-on-Ground Foundations,* as required by the 2019 California Building Code (CBC Section 1808.6.2). Although this procedure was developed for expansive soil conditions, we understand it can also be used to reduce the potential for foundation distress due to differential settlement. The post-tensioned design should incorporate the geotechnical parameters presented in the following table, which are based on the guidelines presented in the PTI DC 10.5 design manual.

**POST-TENSIONED FOUNDATION SYSTEM DESIGN PARAMETERS**

| Post-Tensioning Institute (PTI) DC 10.5-12 Design Parameters | Value |
|---|---|
| Thornthwaite Index | -20 |
| Equilibrium Suction | 3.9 |
| Edge Lift Moisture Variation Distance, $e_M$ (Feet) | 5.2 |
| Edge Lift, $y_M$ (Inches) | 0.61 |
| Center Lift Moisture Variation Distance, $e_M$ (Feet) | 9.0 |
| Center Lift, $y_M$ (Inches) | 0.19 |

7.7.3 The foundations for the post-tensioned slabs should be embedded in accordance with the recommendations of the structural engineer. If a post-tensioned mat foundation system is proposed, the slab should possess a thickened edge with a minimum width of 12 inches and extend below the clean sand or crushed rock layer. A graphic depicting the foundation embedment is provided below.



7.7.4 If the structural engineer proposes a post-tensioned foundation design method other than PTI DC 10.5:

- The criteria presented in the above table are still applicable.
- Interior stiffener beams should be used.
- The width of the perimeter foundations should be at least 12 inches.
- The perimeter footing embedment depths should be at least 30 inches. The embedment depths should be measured from the lowest adjacent pad grade.

7.7.5 During the construction of the post-tension foundation system, the concrete should be placed monolithically. Under no circumstances should cold joints form between the footings/grade beams and the slab during the construction of the post-tension foundation system unless specifically designed by the structural engineer.

7.7.6 Post-tensioned foundations for support of the structure may be designed for an allowable soil bearing pressure of 4,000 psf (dead plus live load). This bearing pressure may be increased by one-third for transient loads due to wind or seismic forces. We estimate the total static settlements under the imposed allowable loads to be less than 1 inch with differential settlements on the order of ½ inch over a horizontal distance of 20 feet. A majority of the settlement of the foundation system is expected to occur on initial application of loading; however, additional settlements are expected within the first twelve months.

7.7.7 Isolated footings, if present, should have a minimum embedment depth and width of 30 inches. The use of isolated footings, which are located beyond the perimeter of the building and support structural elements connected to the building, are not recommended. If this condition cannot be avoided, the isolated footings should be connected to the building foundation system with grade beams. In addition, consideration should be given to connecting patio slabs, which exceed 5 feet in width, to the building foundation to reduce the potential for future separation to occur.

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

7.7.9 Interior stiffening beams should be incorporated into the design of the foundation system in accordance with the PTI design procedures.

7.7.10 Our experience indicates post-tensioned slabs may be susceptible to excessive edge lift, regardless of the underlying soil conditions. Placing reinforcing steel at the bottom of the perimeter footings and the interior stiffener beams may mitigate this potential. The structural engineer should design the foundation system to reduce the potential of edge lift occurring for the proposed structures.

7.7.11 The recommendations of this report are intended to reduce the potential for cracking of slabs and foundations due to expansive soil, differential settlement of fill soil with varying thicknesses. However, even with the incorporation of the recommendations presented herein, foundations, stucco walls, and slabs-on-grade placed on such conditions may still exhibit some cracking due to soil movement and/or shrinkage. The occurrence of concrete shrinkage cracks is independent of the supporting soil characteristics. Their occurrence may be reduced 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.

7.7.12 Foundation excavations should be observed by the Geotechnical Engineer (a representative of Geocon West, Inc.) prior to the placement of reinforcing steel and concrete to check that the exposed soil conditions are consistent with those expected and have been extended to appropriate bearing strata. If unexpected soil conditions are encountered, foundation modifications may be required.

## 7.8 Foundation Settlement

7.8.1 The maximum expected static settlement for a structure supported on a conventional foundation system deriving support in the recommended bearing materials and designed with a maximum bearing pressure of 4,000 psf is estimated to be less than 1 inch and occur below the heaviest loaded structural element. Settlement of the foundation system is expected to occur on initial application of loading. Differential settlement is not expected to exceed ½ inch over a distance of 20 feet.

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

## 7.9 Lateral Design

7.9.1 Resistance to lateral loading may be provided by friction acting at the base of foundations, slabs and by passive earth pressure. An allowable coefficient of friction of 0.40 may be used with the dead load forces in the newly placed engineered fill or competent alluvium.

7.9.2 Passive earth pressure for the sides of foundations and slabs poured against properly compacted fill or competent alluvial soils may be computed as an equivalent fluid having a density of 260 pounds per cubic foot with a maximum earth pressure of 2,600 pounds per square foot. When combining passive and friction for lateral resistance, the passive component should be reduced by one-third.

## 7.10 Miscellaneous Foundations

7.10.1 Foundations for small outlying structures, such as block walls up to 6 feet in height, planter walls or trash enclosures which will not be tied to the proposed structure may be supported on conventional foundations bearing on a minimum of 12 inches of newly placed engineered fill which extends laterally at least 12 inches beyond the foundation area. Where excavation and compaction cannot be performed, such as adjacent to property lines, foundations may derive support in the undisturbed alluvial soils generally found at or below a depth of 24 inches and should be deepened as necessary to maintain a minimum 12-inch embedment into the recommended bearing materials.

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

7.10.3 Based on the potential for hydro-consolidation, maintaining proper surface drainage will be vital for future performance of foundations and site improvements. 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 foundations.

7.10.4 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.

## 7.11 Concrete Slabs-on-Grade

7.11.1 Concrete slabs-on-grade subject to vehicle loading should be designed in accordance with the recommendations in the *Pavement Recommendations* section of this report (Section 7.12).

7.11.2 Subsequent to the recommended grading, concrete slabs-on-grade for structures, not subject to vehicle loading, should be a minimum of 4-inches thick and minimum slab reinforcement should consist of No. 3 steel reinforcing bars placed 18 inches on center in both horizontal directions. Steel reinforcing should be positioned vertically near the slab midpoint.

7.11.3 Slabs-on-grade at the ground surface that may receive moisture-sensitive floor coverings or may be used to store moisture-sensitive materials should be underlain by a vapor retarder placed directly beneath the slab. The vapor retarder and acceptable permeance should be specified by the project architect or developer based on the type of floor covering that will be installed. The vapor retarder selection and design should be consistent with the guidelines presented in Section 9.3 of the American Concrete Institute's (ACI) Guide for Concrete Slabs that Receive Moisture-Sensitive Flooring Materials (ACI 302.2R-06) as well as ASTM E1745 and should be installed in general conformance with ASTM E 1643 (latest edition) and the manufacturer's recommendations. A minimum thickness of 15 mils extruded polyolefin plastic is recommended; vapor retarders which contain recycled content or woven materials are not recommended. The vapor retarder should have a permeance of less than 0.01 perms demonstrated by testing before and after mandatory conditioning is recommended. The vapor retarder should be installed in direct contact with the concrete slab with proper perimeter seal. If the California Green Building Code requirements apply to this project, the vapor retarder should be underlain by 4 inches of clean aggregate. It is important that the vapor retarder be puncture resistant since it will be in direct contact with angular gravel. As an alternative to the clean aggregate suggested in the Green Building Code, it is our opinion that the concrete slab-on-grade may be underlain by a vapor retarder over 4-inches of clean sand (sand equivalent greater than 30), since the sand will serve a capillary break and will minimize the potential for punctures and damage to the vapor barrier.

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

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

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

## 7.12 Preliminary Pavement Recommendations

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

7.12.2 The following pavement sections are based on an assumed R-Value of 40 and the assumption that all site drainage will be collected and controlled and not allowed to pond on the paving per paragraph 7.12.6 below. 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. Offsite improvements, including sidewalks, curbs/gutters, and roadways, are not included in this report. If required, offsite improvements will be addressed in an addendum letter.

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

**PRELIMINARY PAVEMENT DESIGN SECTIONS**

| Location | Estimated Traffic Index (TI) | Asphalt Concrete (inches) | Class 2 Aggregate Base (inches) |
|---|---|---|---|
| Automobile Parking And Driveways | 4.0 | 3.0 | 4.0 |
| Trash Truck & Fire Lanes | 7.0 | 4.0 | 7.5 |

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

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

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

## 7.13 Retaining Walls Design

7.13.1 The recommendations presented below are generally applicable to the design of rigid concrete or masonry retaining walls having a maximum height of 5 feet. In the event that walls significantly higher than 5 feet are planned, Geocon should be contacted for additional recommendations.

7.13.2 Retaining wall foundations may be designed in accordance with the recommendations provided in the *Foundation Design* sections of this report (see Sections 7.6 and 7.7).

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

**RETAINING WALL WITH LEVEL BACKFILL SURFACE**

| HEIGHT OF RETAINING WALL (Feet) | ACTIVE PRESSURE EQUIVALENT FLUID PRESSURE (Pounds Per Cubic Foot) | AT-REST PRESSURE EQUIVALENT FLUID PRESSURE (Pounds Per Cubic Foot) |
|---|---|---|
| Up to 5 | 40 | 60 |

7.13.4 The wall pressures provided above assume that the proposed retaining walls will support newly placed engineered fill soils. 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.

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

7.13.6 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 $z$.

7.13.7 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 $z$, $\theta$ is the angle between a line perpendicular to the excavation/wall and a line from the point-load to location on the excavation/wall where the surcharge is being evaluated, and $\sigma_H(z)$ is the horizontal pressure at depth $z$.

7.13.8 In addition to the recommended earth pressure, the upper ten 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 ten feet from the wall, the traffic surcharge may be neglected.

7.13.9 Seismic lateral forces will be required for any retaining walls in excess of 6 feet. Recommendations for seismic lateral forces will be provided under separate cover should they become necessary.

### 7.14 Retaining Wall Drainage

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

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

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

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

## 7.15 Temporary Excavations

7.15.1 Excavations on the order of 11 feet in height may be required during grading operations. The excavations are expected to expose granular alluvial soils, which are subject to caving. Vertical excavations up to 5 feet in height may be attempted where not surcharged by adjacent foundations or traffic.

7.15.2 Vertical excavations situated along property lines, greater than 5 feet, or surcharged by existing structures or traffic loads 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 1.5:1 slope gradient or flatter up to a maximum of 12 feet in height. A uniform slope does not have a vertical portion. It is anticipated that sufficient space is available to complete the required earthwork for this project using sloping measures.

7.15.3 If excavations in close proximity to an adjacent property line and/or structure are required, special excavation measures such shoring may be necessary in order to maintain lateral support of offsite improvements. Recommendations for special excavation measures can be provided under separate cover once the project has proceeded to a more finalized design.

7.15.4 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.

### 7.16 Stormwater Infiltration

7.16.1 During the May 27, 2022, site exploration, borings B1 and B3 were utilized to perform percolation testing. Slotted casing was placed in each boring, and the annular space between the casing and excavation was filled with gravel. The boring was then filled with water to pre-saturate the soils. The casing was refilled with water and percolation test readings were performed after repeated flooding of the cased excavation. Based on the test results, the average infiltration rate (adjusted percolation rate), for the earth materials encountered, is provided in the following table. The field-measured percolation rate has been adjusted to infiltration rates in accordance with the County of San Bernardino Technical Guidance Document for Water Quality Management Plans (June 2013). Additional correction factors may be required and should be applied by the engineer in responsible charge of the design of the stormwater infiltration system and based on applicable guidelines. Percolation test field data and calculation of the measured percolation rate and design infiltration rate are provided on Figures 7 and 8.

| Boring | Soil Type | Infiltration Depth (ft) | Average Infiltration Rate (in / hour) |
|---|---|---|---|
| B1 | Silty Sand (SM), and Poorly Graded Sand (SP) | 10-15 | 0.95 |
| B3 | Poorly Graded Sand | 10-15 | 3.59 |

7.16.2 The results of the percolation testing in the table above indicate that the infiltration rate for soils encountered at the depths and locations indicated in the table above are considered conductive to infiltration, and it is our opinion that the site is suitable for infiltration of stormwater.

7.16.3 Due to the presence of hydro-collapsible soils, infiltration of stormwater within the upper 10 feet of site soils is not recommended for this project. Provided that infiltration of stormwater occurs below a depth of 10 feet, it is our opinion that there is a very low potential for infiltration-related soil settlement to adversely affect the proposed structures; however, some settlement may occur locally within the area of the infiltration system. The project civil engineer should evaluate the impact on surface drainage should some soil settlement occur locally within the area of the infiltration system. It is suggested that flexible connections be utilized between the storm drain pipes and infiltration chambers.

7.16.4 The project owner should understand that the recommendations herein are intended minimize, not prevent soil settlement as a result of stormwater infiltration, as doing so would be cost prohibitive to the project.

7.16.5 If infiltration is planned for any locations other than where the above testing was performed, additional testing may be required.

7.16.6 The infiltration system must be located such that the closest distance between an adjacent foundation is at least 15 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.

7.16.7 Where a 15-foot horizontal setback cannot be maintained between the infiltration system and an adjacent footing, and the infiltration system penetrates below the foundation influence line, the proposed stormwater infiltration system must be designed to resist the surcharge from the adjacent foundation. The foundation surcharge line may be assumed to project down away from the bottom of the foundation at a 1:1 gradient

7.16.8 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 2-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.

7.16.9 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).

## 7.17 Surface Drainage

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

7.17.2 Based on the potential for hydro-consolidation, maintaining proper surface drainage will be vital for future performance of foundations and site improvements. 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 1804A.4 or other applicable standards. In addition, drainage should not be allowed to flow uncontrolled over any descending slope. Discharge from downspouts, roof drains and scuppers are not recommended onto unprotected soils within 5 feet of the building perimeter. Planters which are located adjacent to foundations should be sealed to prevent moisture intrusion into the soils providing foundation support. Landscape irrigation is not recommended within 5 feet of the building perimeter footings except when enclosed in protected planters.

7.17.3 Positive site drainage should be provided away from structures, pavement, and the tops of slopes to swales or other controlled drainage structures. Pavement areas should be fine graded such that water is not allowed to pond.

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

## 7.18 Plan Review

7.18.1 Grading, foundation, and shoring plans (if required) 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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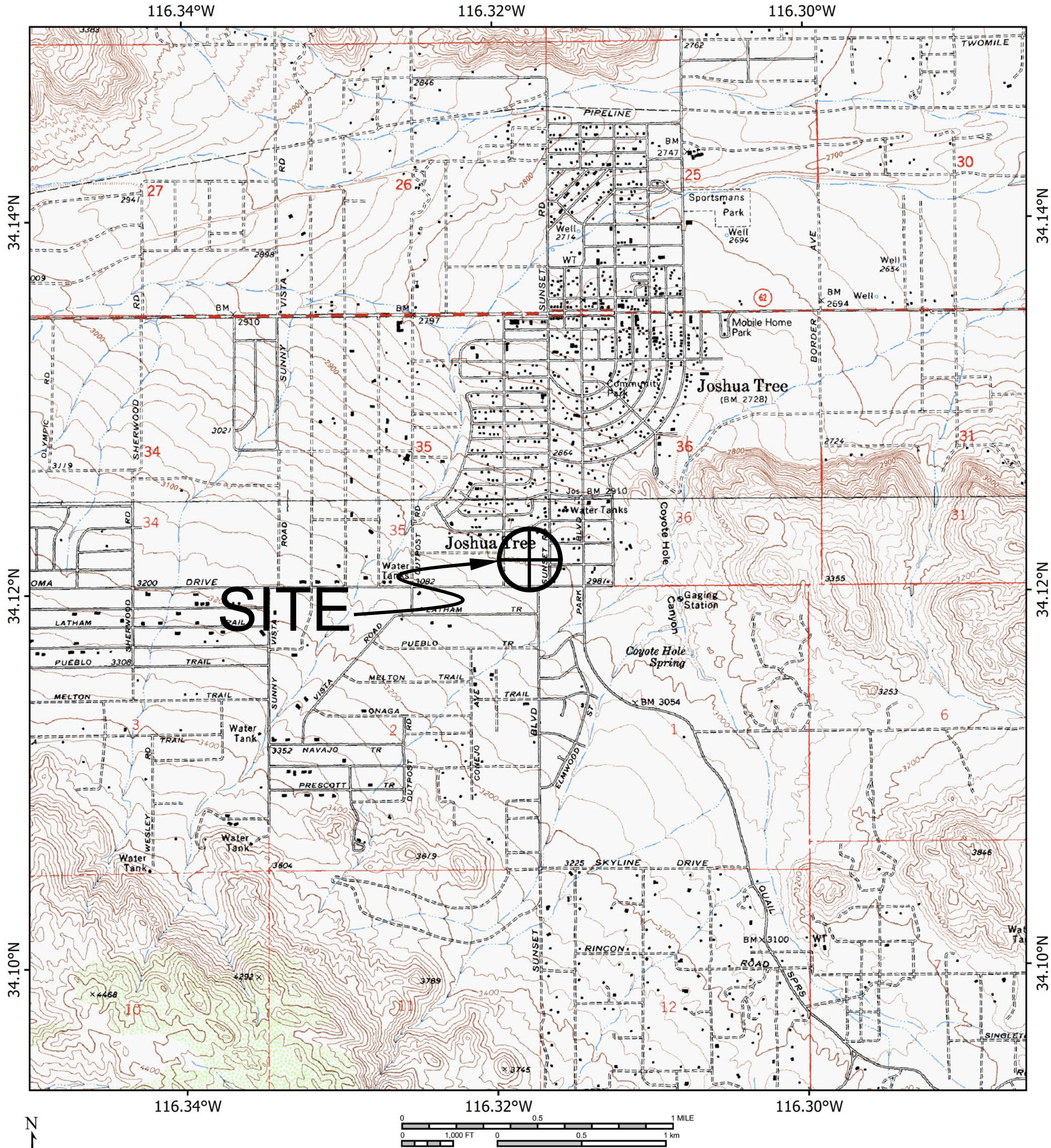


REFERENCE: U.S.G.S. TOPOGRAPHIC MAP, 7.5 MINUTE SERIES, JOSHUA TREE NORTH AND SOUTH, CA QUADRANGLES

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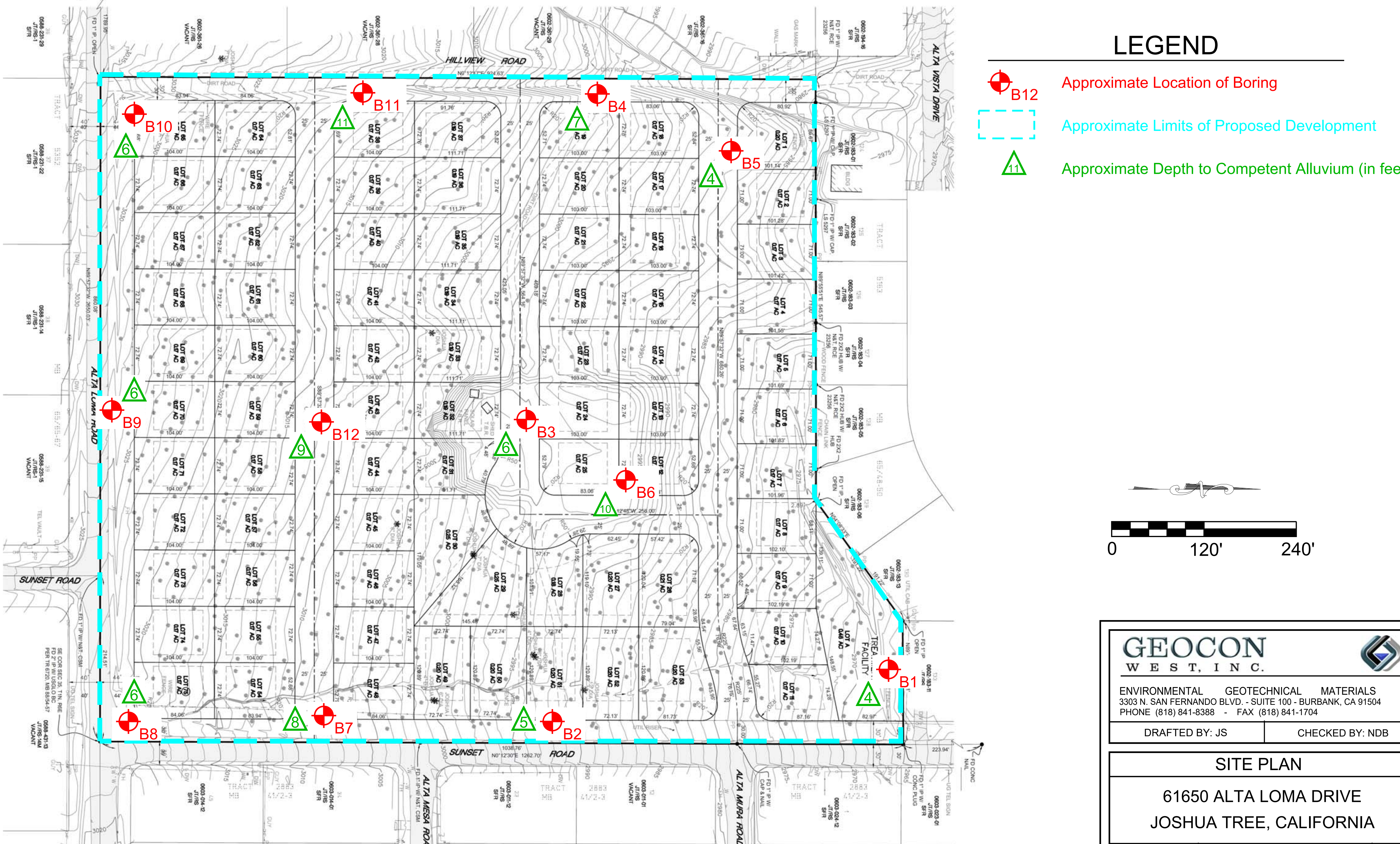
| | |
|---|---|
| DRAFTED BY: RA | CHECKED BY: SFK |

# VICINITY MAP

61650 ALTA LOMA DRIVE
JOSHUA TREE, CALIFORNIA

| | | |
|---|---|---|
| JULY 2022 | PROJECT NO. W1558-99-01 | FIG. 1 |

Proposed 75-Home Tract Development

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| --- | --- |

SITE PLAN

61650 ALTA LOMA DRIVE
JOSHUA TREE, CALIFORNIA

| JULY 2022 | PROJECT NO. W1558-99-01 | FIG. 2 |
| --- | --- | --- |

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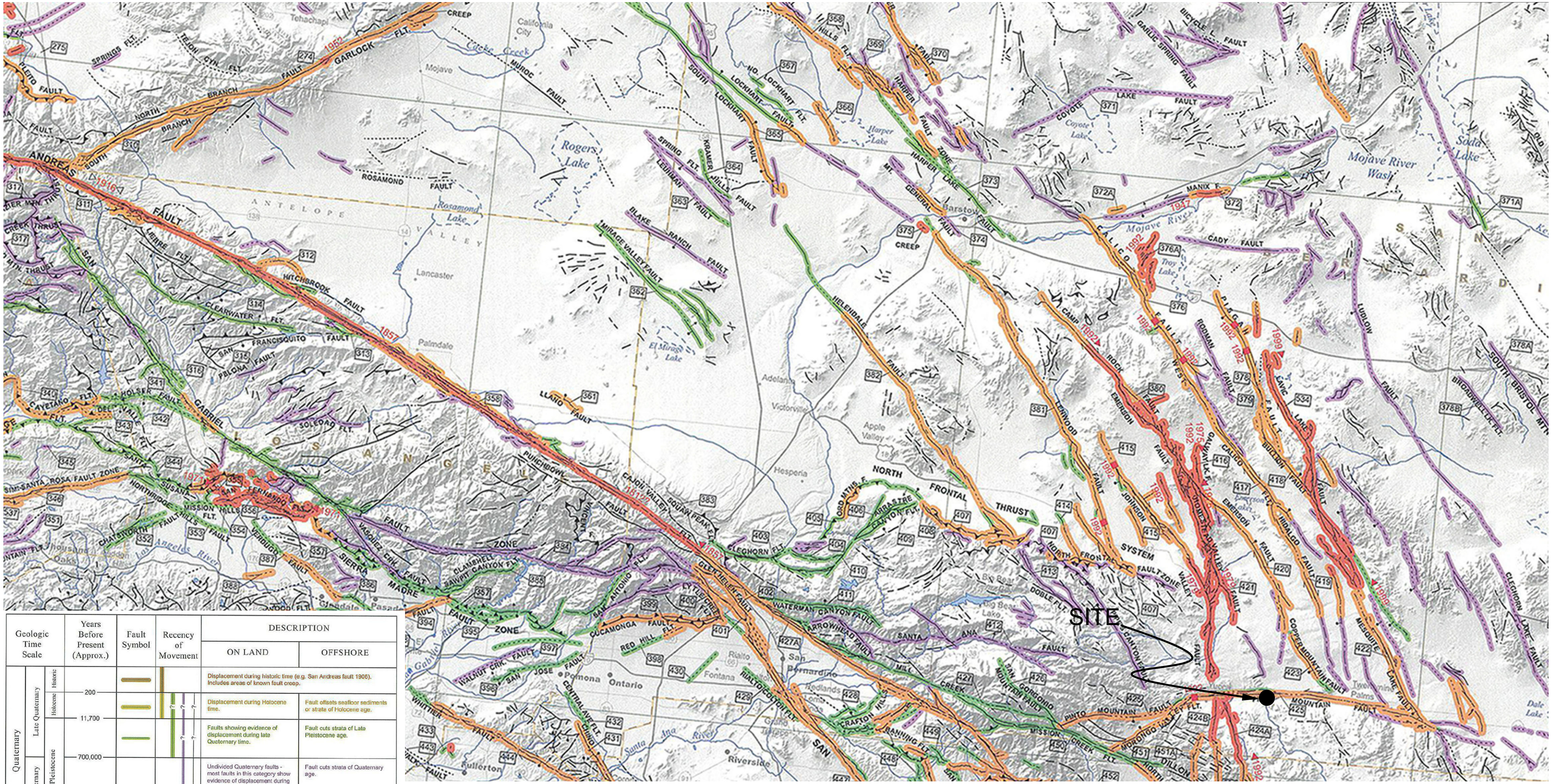


| Geologic Time Scale | | | Years Before Present (Approx.) | Fault Symbol | Recency of Movement | DESCRIPTION: ON LAND | DESCRIPTION: OFFSHORE |
|---|---|---|---|---|---|---|---|
| Quaternary | Late Quaternary | Historic | | | | Displacement during historic time (e.g. San Andreas fault 1906). Includes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

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

0 12 24 Miles

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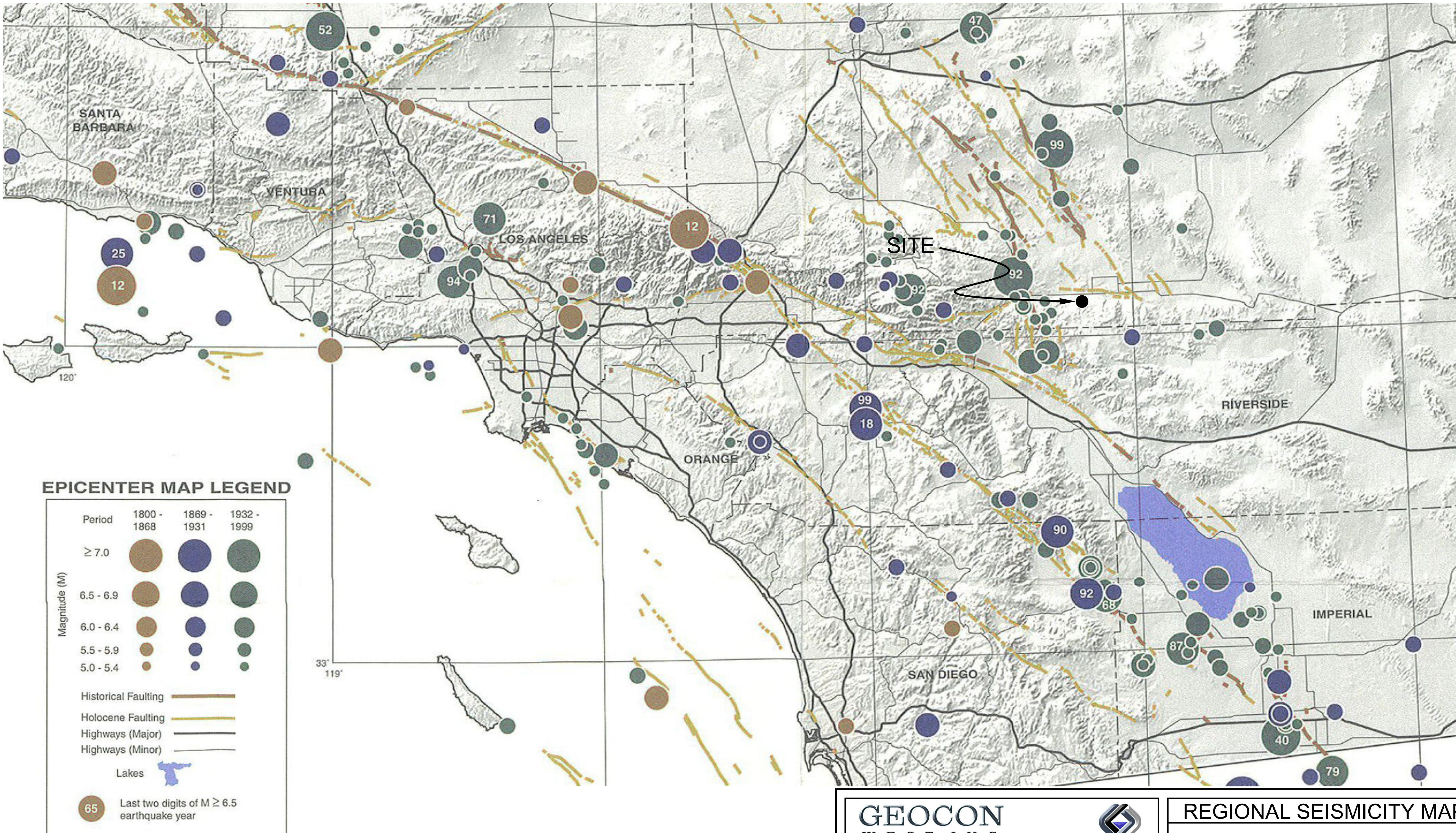
# REGIONAL FAULT MAP

61650 ALTA LOMA DRIVE
JOSHUA TREE, CALIFORNIA

JULY 2022 | PROJECT NO. W1558-99-01 | FIG. 3




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

0 20 40 Miles

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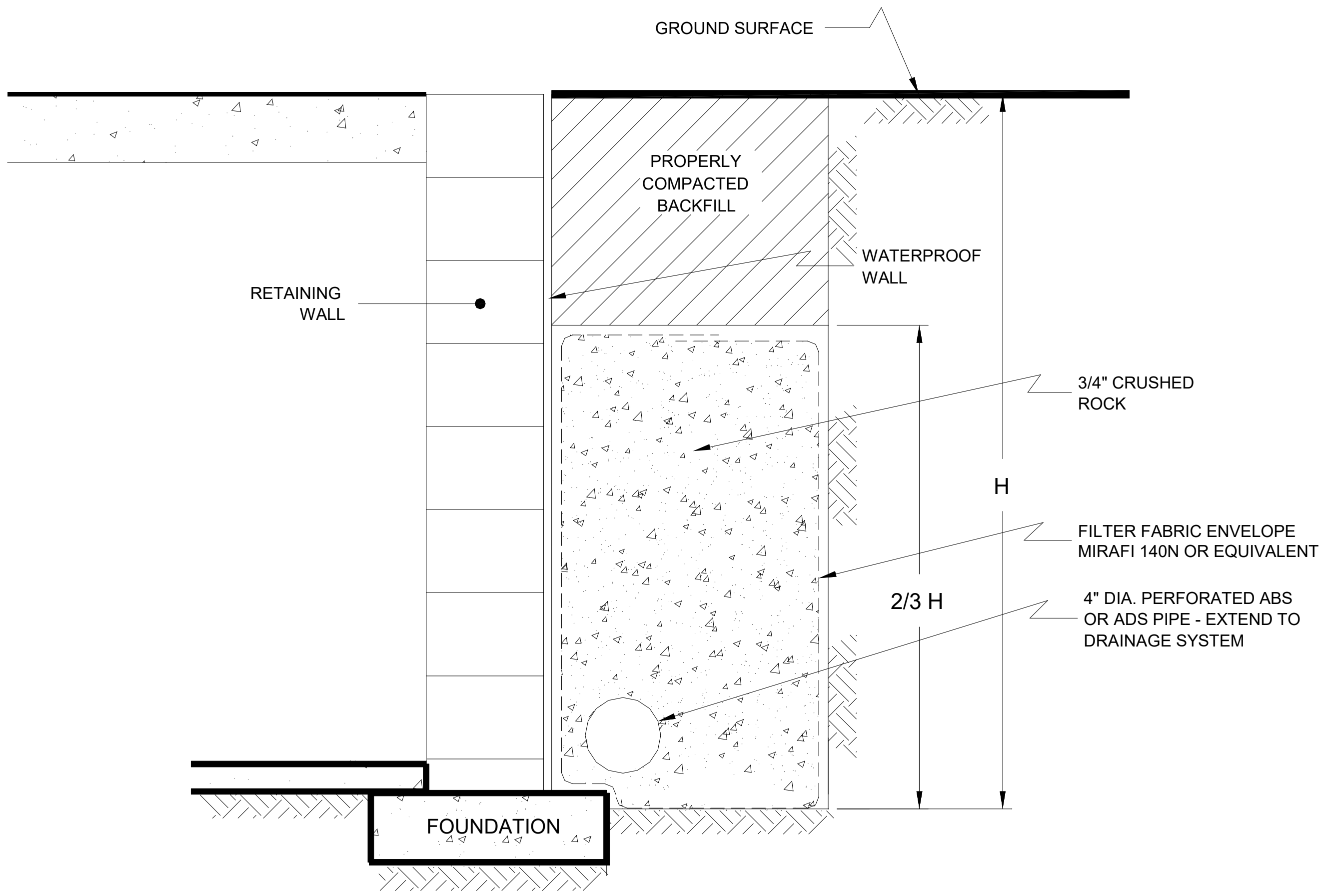
# REGIONAL SEISMICITY MAP

61650 ALTA LOMA DRIVE
JOSHUA TREE, CALIFORNIA

| DRAFTED BY: RA | CHECKED BY: SFK | JULY 2022 | PROJECT NO. W1558-99-01 | FIG.4 |
|---|---|---|---|---|




NO SCALE

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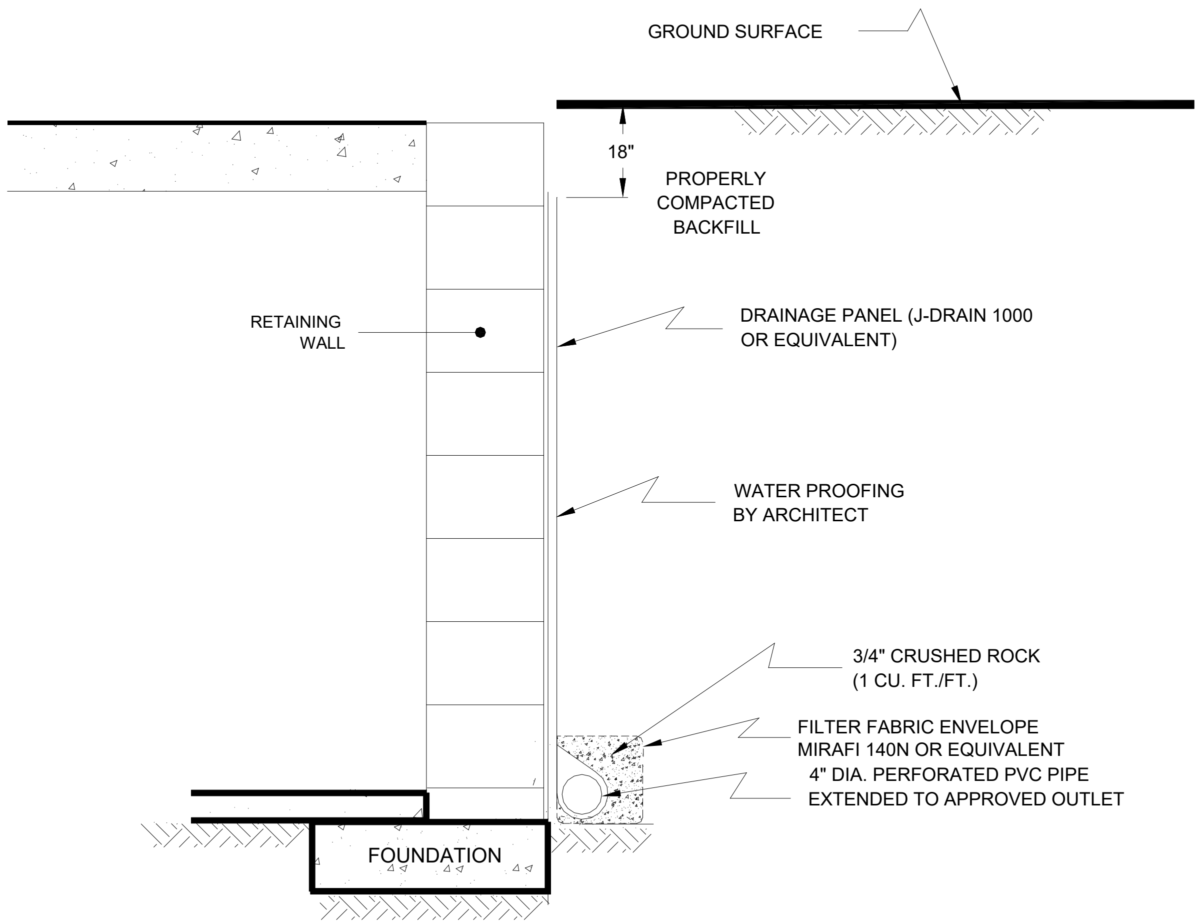
DRAFTED BY: JS | CHECKED BY: NDB

# RETAINING WALL DRAIN DETAIL

## 61650 ALTA LOMA DRIVE
## JOSHUA TREE, CALIFORNIA

JULY 2022 | PROJECT NO. W1558-99-01 | FIG. 5




NO SCALE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# RETAINING WALL DRAIN DETAIL

61650 ALTA LOMA DRIVE

JOSHUA TREE, CALIFORNIA

JULY 2022 | PROJECT NO. W1558-99-01 | FIG. 6

## PERCOLATION TEST DATA SHEET

| | | | | | | | |
|---|---|---|---|---|---|---|---|
| Project: | 61650 Alta Loma Drive | | Project No: | W1558-99-01 | | Date: | 5/27/2022 |
| Test Hole No: | | B1 | Tested By: | RA | | | |
| Depth of Test Hole, $D_T$: | | 15 | USCS Soil Classification: | | SM/SP | | |
| Test Hole Dimensions (inches) | | | | | Length | Width | |
| Diameter (if round) = | | 8 | Sides (if rectangular) = | | --- | --- | |

Sandy Soil Criteria Test*

| Trial No. | Start Time | Stop Time | Δt Time Interval (min) | $D_0$ Initial Depth to Water (in) | $D_f$ Final Depth to Water (in) | ΔD Change in Water Level (in) | Greater than or Equal to 6"? (y/n) |
|---|---|---|---|---|---|---|---|
| 1 | 7:17 | 7:47 | 30 | 120.0 | 137.5 | 17.5 | y |
| 2 | 7:48 | 8:18 | 30 | 120.0 | 135.7 | 15.7 | y |

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

| Trial No. | Start Time | Stop Time | Δt Time Interval (min) | $D_0$ Initial Depth to Water (in) | $D_f$ Final Depth to Water (in) | ΔD Change in Water Level (in) | Percolation Rate (min/in) |
|---|---|---|---|---|---|---|---|
| 1 | 8:21 | 8:51 | 30 | 120.0 | 134.4 | 14.4 | 2.08 |
| 2 | 8:53 | 9:23 | 30 | 120.0 | 133.9 | 13.9 | 2.16 |
| 3 | 9:25 | 9:55 | 30 | 120.0 | 133.6 | 13.6 | 2.21 |
| 4 | 9:57 | 10:27 | 30 | 120.0 | 133.6 | 13.6 | 2.21 |
| 5 | 10:31 | 11:01 | 30 | 120.0 | 132.7 | 12.7 | 2.36 |
| 6 | 11:02 | 11:32 | 30 | 120.0 | 133.2 | 13.2 | 2.27 |
| 7 | | | | | | | |
| 8 | | | | | | | |

Infiltration Rate Calculation:

| | | | | | |
|---|---|---|---|---|---|
| Time Interval, Δt = | 30 | minutes | Ho = | 60.0 | inches |
| Final Depth to Water, Df = | 133.2 | inches | Hf = | 46.8 | inches |
| Test Hole Radius, r = | 4 | inches | ΔH = | 13.2 | inches |
| Initial Depth to Water, Do = | 120.0 | inches | Havg = | 53.4 | inches |
| Total Depth of Test Hole, DT = | 180.0 | inches | | | |

$$I_t = \frac{\Delta H(60r)}{\Delta t(r + 2H_{avg})}$$

Infiltration Rate, It = **0.95** inches/hour

## PERCOLATION TEST DATA SHEET

| | | | | | | | |
|---|---|---|---|---|---|---|---|
| Project: | 61650 Alta Loma Drive | | Project No: | W1558-99-01 | | Date: | 5/27/2022 |
| Test Hole No: | | B3 | Tested By: | RA | | | |
| Depth of Test Hole, $D_T$: | | 15 | USCS Soil Classification: | | SP | | |
| Test Hole Dimensions (inches) | | | | | Length | Width | |
| Diameter (if round) = | | 8 | Sides (if rectangular) = | | --- | --- | |

Sandy Soil Criteria Test*

| Trial No. | Start Time | Stop Time | Δt Time Interval (min) | $D_0$ Initial Depth to Water (in) | $D_f$ Final Depth to Water (in) | ΔD Change in Water Level (in) | Greater than or Equal to 6"? (y/n) |
|---|---|---|---|---|---|---|---|
| 1 | 8:27 | 8:57 | 30 | 120.0 | 158.8 | 38.8 | y |
| 2 | 9:00 | 9:30 | 30 | 120.0 | 158.6 | 38.6 | y |

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

| Trial No. | Start Time | Stop Time | Δt Time Interval (min) | $D_0$ Initial Depth to Water (in) | $D_f$ Final Depth to Water (in) | ΔD Change in Water Level (in) | Percolation Rate (min/in) |
|---|---|---|---|---|---|---|---|
| 1 | 9:35 | 10:05 | 30 | 120.0 | 155.8 | 35.8 | 0.84 |
| 2 | 10:08 | 10:38 | 30 | 120.0 | 157.9 | 37.9 | 0.79 |
| 3 | 10:39 | 11:09 | 30 | 120.0 | 157.8 | 37.8 | 0.79 |
| 4 | 11:10 | 11:40 | 30 | 120.0 | 158.4 | 38.4 | 0.78 |
| 5 | 11:43 | 12:13 | 30 | 120.0 | 158.8 | 38.8 | 0.77 |
| 6 | 12:15 | 12:45 | 30 | 120.0 | 158.4 | 38.4 | 0.78 |
| 7 | | | | | | | |
| 8 | | | | | | | |

Infiltration Rate Calculation:

| | | | | | |
|---|---|---|---|---|---|
| Time Interval, Δt = | 30 | minutes | Ho = | 60.0 | inches |
| Final Depth to Water, Df = | 158.4 | inches | Hf = | 21.6 | inches |
| Test Hole Radius, r = | 4 | inches | ΔH = | 38.4 | inches |
| Initial Depth to Water, Do = | 120.0 | inches | Havg = | 40.8 | inches |
| Total Depth of Test Hole, DT = | 180.0 | inches | | | |

$$I_t = \frac{\Delta H(60r)}{\Delta t(r + 2H_{avg})}$$

Infiltration Rate, It = **3.59** inches/hour

# APPENDIX A

## APPENDIX A

## FIELD INVESTIGATION

The site was explored on May 26 2022 and May 31, 2022, by excavating 12 8-inch diameter borings to a maximum depth of 15½ feet below the existing ground surface utilizing a truck-mounted hollow-stem auger drilling machine and a limited access track mounted drilling machine. Representative and relatively undisturbed samples were obtained by driving a 3-inch O. D., California Modified Sampler into the "undisturbed" soil mass with blows from a 140-pound auto-hammer falling 30 inches. The California Modified Sampler was equipped with 1-inch high by $2^{3}/_{8}$-inch diameter brass rings to facilitate soil removal and testing. Bulk samples were 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 boring logs are presented on Figures A1 and A12. 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 log 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. W1558-99-01

## BORING 1

ELEV. (MSL.) - - DATE COMPLETED 05/26/2022

EQUIPMENT HOLLOW STEM AUGER BY: RA

| DEPTH IN FEET | SAMPLE NO. | LITHOLOGY | GROUNDWATER | SOIL CLASS (USCS) | MATERIAL DESCRIPTION | PENETRATION RESISTANCE (BLOWS/FT)* | DRY DENSITY (P.C.F.) | MOISTURE CONTENT (%) |
|---|---|---|---|---|---|---|---|---|
| 0 | | | | | **ALLUVIUM**<br>Sand, poorly graded, loose to medium dense, dry to slightly moist, reddish brown, trace coarse-grained and silt. | | | |
| 2 | | | | SP | | | | |
| 3 | B1@3' | | | | - light brown to reddish brown | 39 | 107.6 | 2.2 |
| 4 | | | | | | | | |
| 6 | B1@6' | | | SP-SM | Sand with Silt, poorly graded, medium dense, dry, light brown, fine-grained, trace medium-grained and fine gravel. | 33 | 117.6 | 1.4 |
| 8 | | | | | | | | |
| 9 | B1@9' | | | SM | Silty Sand, medium dense, dry, light brown, fine-grained, trace fine gravel. | 43 | 102.2 | 2.1 |
| 10 | | | | | | | | |
| 11 | | | | SP | Sand, poorly graded, medium dense, dry, brown, fine- to medium-grained, some coarse-grained. | | | |
| 12 | B1@12' | | | | | 28 | 112.4 | 0.9 |
| 14 | | | | | | | | |
| 15 | B1@15' | | | | - no recovery | 24 | | |

Total depth of boring: 15.5 feet
No fill.
No groundwater encountered.
Percolation testing performed.

*Penetration resistance for 140-pound hammer falling 30 inches by auto-hammer.
NOTE: The stratification lines presented herein represent the approximate boundary between earth types; the transitions may be gradual.

W1558-99-01 BORING LOGS.GPJ

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

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

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

GEOCON

## BORING 2

ELEV. (MSL.) - - DATE COMPLETED 05/26/2022

EQUIPMENT HOLLOW STEM AUGER BY: RA

| 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' | | | SP-SM | **ALLUVIUM** Sand with Silt, medium dense, dry, reddish brown to brown, fine-grained, trace medium- to coarse-grained. | | | |
| 2 | B2@2.5' | | | | | 25 | 115.4 | 3.8 |
| 4 | | | | | | | | |
| | B2@5' | | | SP | Sand, poorly graded, medium dense, dry to slightly moist, reddish brown, fine- to medium-grained, some coarse-grained. | 42 | 110.5 | 1.9 |
| 6 | | | | | | | | |
| 8 | B2@7.5' | | | | - light reddish brown to reddish brown, fine-grained, trace medium- to coarse-grained | 53 | 118.1 | 2.5 |
| 10 | B2@10' | | | | - yellowish brown, fine-grained, trace fine to medium gravel | 31 | 112.9 | 0.9 |
| 12 | B2@12.5' | | | | - some medium-grained, trace coarse-grained | 31 | 117.7 | 1.4 |
| 14 | B2@15' | | | | - dense, trace fine to medium gravel | 56 | 111.4 | 1.1 |

Total depth of boring: 15.5 feet
No fill.
No groundwater encountered.
Backfilled with soil cuttings and tamped.

*Penetration resistance for 140-pound hammer falling 30 inches by auto-hammer.
NOTE: The stratification lines presented herein represent the approximate boundary between earth types; the transitions may be gradual.

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

W1558-99-01 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.

PROJECT NO. W1558-99-01

## BORING 3

ELEV. (MSL.) - - DATE COMPLETED 05/26/2022

EQUIPMENT HOLLOW STEM AUGER BY: RA

| DEPTH IN FEET | SAMPLE NO. | LITHOLOGY | GROUNDWATER | SOIL CLASS (USCS) | MATERIAL DESCRIPTION | PENETRATION RESISTANCE (BLOWS/FT)* | DRY DENSITY (P.C.F.) | MOISTURE CONTENT (%) |
|---|---|---|---|---|---|---|---|---|
| 0 | | | | | **ALLUVIUM**<br>Sand, poorly graded, medium dense, dry to slightly moist, light brown to reddish brown, fine-grained, some medium- to coarse-grained. | | | |
| 2 | | | | | | | | |
| | B3@3' | | | | | 44 | | |
| 4 | | | | | | | | |
| 6 | B3@6' | | | SP | - reddish brown, trace fine gravel | 53 | 109.3 | 3.8 |
| 8 | B3@8.5' | | | | - very dense | 50 (6") | 114.4 | 4.8 |
| 10 | | | | | | | | |
| 12 | B3@12' | | | | - medium dense, fine-grained, trace medium- to coarse-grained | 50 | 116.1 | 3.4 |
| 14 | | | | | | | | |
| | B3@15' | | | | - light reddish brown to light brown | 53 | 118.3 | 2.8 |

Total depth of boring: 15.5 feet
No fill.
No groundwater encountered.
Backfilled with soil cuttings and tamped.

*Penetration resistance for 140-pound hammer falling 30 inches by auto-hammer.
NOTE: The stratification lines presented herein represent the approximate boundary between earth types; the transitions may be gradual.

W1558-99-01 BORING LOGS.GPJ

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

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

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

GEOCON

## BORING 4

ELEV. (MSL.) - - DATE COMPLETED 05/26/2022

EQUIPMENT HOLLOW STEM AUGER BY: RA

| DEPTH IN FEET | SAMPLE NO. | LITHOLOGY | GROUNDWATER | SOIL CLASS (USCS) | MATERIAL DESCRIPTION | PENETRATION RESISTANCE (BLOWS/FT)* | DRY DENSITY (P.C.F.) | MOISTURE CONTENT (%) |
|---|---|---|---|---|---|---|---|---|
| 0 | | | | | **ALLUVIUM**<br>Sand with Silt, medium dense, dry, reddish brown, fine-grained, trace fine gravel. | | | |
| 2 | B4@2.5' | | | | | 28 | 112.8 | 3.9 |
| 4 | B4@5'<br>BULK<br>5-10' | | | SP-SM | | 22 | 96.2 | 0.9 |
| 8 | B4@7.5' | | | | - trace coarse-grained | 45 | 116.1 | 0.8 |
| 10 | B4@9.5' | | | | - very dense, fine-grained | 50 (6") | 86.9 | 1.0 |
| 12 | B4@12.5' | | | SW | Sand, well-graded, very dense, dry, light reddish brown, fine- to coarse-grained, trace fine gravel. | 78 | 119.7 | 1.1 |
| 14 | B4@14.5' | | | SP-SW | Sand with Silt, well-graded, very dense, dry, light reddish brown, interbeds of poorly-graded sand with silt. | 50 (6") | 118.2 | 1.1 |

Total depth of boring: 15 feet
No fill.
No groundwater encountered.
Backfilled with soil cuttings and tamped.

*Penetration resistance for 140-pound hammer falling 30 inches by auto-hammer.
NOTE: The stratification lines presented herein represent the approximate boundary between earth types; the transitions may be gradual.

W1558-99-01 BORING LOGS.GPJ

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

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

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

PROJECT NO. W1558-99-01

# BORING 5

ELEV. (MSL.) - - DATE COMPLETED 05/26/2022

EQUIPMENT HOLLOW STEM AUGER BY: RA

| DEPTH IN FEET | SAMPLE NO. | LITHOLOGY | GROUNDWATER | SOIL CLASS (USCS) | MATERIAL DESCRIPTION | PENETRATION RESISTANCE (BLOWS/FT)* | DRY DENSITY (P.C.F.) | MOISTURE CONTENT (%) |
|---|---|---|---|---|---|---|---|---|
| 0 | | | | SP | **ALLUVIUM** Sand, poorly graded, dense, dry, brown to light brown, fine-grained to very fine-grained, trace silt. | | | |
| 3 | B5@3' | | | | | 59 | 110.2 | 4.7 |
| 6 | B5@6' | | | | - no recovery | 55 | | |
| 10 | B5@10' | | | | - medium dense, light brown to light reddish brown, fine-grained, some medium- to coarse-grained, trace fine gravel | 43 | 112.0 | 1.4 |
| 12 | B5@12' | | | SW | Sand, well-graded, medium dense, dry, light reddish brown, fine- to coarse-grained, trace fine gravel. | 45 | 124.8 | 0.8 |
| 14.5 | B5@14.5' | | | SP | Sand, poorly graded, very dense, dry, light reddish brown, fine- to medium-grained, trace fine to medium gravel. | 50 (6") | 115.7 | 1.6 |

Total depth of boring: 15 feet
No fill.
No groundwater encountered.
Backfilled with soil cuttings and tamped.

*Penetration resistance for 140-pound hammer falling 30 inches by auto-hammer.
NOTE: The stratification lines presented herein represent the approximate boundary between earth types; the transitions may be gradual.

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

W1558-99-01 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.

GEOCON

# BORING 6

ELEV. (MSL.) - - DATE COMPLETED 05/31/2022

EQUIPMENT HOLLOW STEM AUGER BY: RA

| DEPTH IN FEET | SAMPLE NO. | SOIL CLASS (USCS) | MATERIAL DESCRIPTION | PENETRATION RESISTANCE (BLOWS/FT)* | DRY DENSITY (P.C.F.) | MOISTURE CONTENT (%) |
|---|---|---|---|---|---|---|
| 0 | BULK 0-5' | SP | **ALLUVIUM**<br>Sand, poorly graded, medium dense, dry to slightly moist, reddish brown, fine- to medium-grained, trace coarse-grained. | | | |
| 2 | B6@2.5' | | | 46 | 116.5 | 6.0 |
| 4 | | | | | | |
| 6 | B6@5'<br>BULK 5-10' | | - loose, trace fine gravel and silt | 13 | 98.6 | 4.9 |
| 8 | B6@7.5' | | - increase in coarse-grained | 17 | 115.4 | 3.7 |
| 10 | B6@10' | SW | Sand, well-graded, medium dense, dry to slightly moist, reddish brown, fine- to coarse-grained, trace fine gravel. | 40 | 124.4 | 3.6 |
| 12 | B6@12.5' | SP | Sand, poorly graded, medium dense, dry, reddish brown to orangeish brown, fine- to medium-grained, trace coarse-grained. | 44 | 113.7 | 1.8 |
| 14 | B6@15' | | - increase in coarse-grained | 39 | 114.1 | 1.5 |

Total depth of boring: 15.5 feet
No fill.
No groundwater encountered.
Backfilled with soil cuttings and tamped.

*Penetration resistance for 140-pound hammer falling 30 inches by auto-hammer.
NOTE: The stratification lines presented herein represent the approximate boundary between earth types; the transitions may be gradual.

W1558-99-01 BORING LOGS.GPJ

**Figure A6,**
**Log of Boring 6, Page 1 of 1**

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

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

GEOCON

PROJECT NO. W1558-99-01

## BORING 7

ELEV. (MSL.) - - DATE COMPLETED 05/31/2022

EQUIPMENT HOLLOW STEM AUGER BY: RA

| DEPTH IN FEET | SAMPLE NO. | LITHOLOGY | GROUNDWATER | SOIL CLASS (USCS) | MATERIAL DESCRIPTION | PENETRATION RESISTANCE (BLOWS/FT)* | DRY DENSITY (P.C.F.) | MOISTURE CONTENT (%) |
|---|---|---|---|---|---|---|---|---|
| 0 | | | | | **ALLUVIUM** Sand, poorly graded, loose to medium dense, dry, light brown to light reddish brown, fine- to medium-grained, trace coarse-grained and gravel. | | | |
| 2 | | | | | | | | |
| 4 | B7@3' | | | SP | - dense, reddish brown to light reddish brown, fine-grained, trace medium- to coarse-grained | 75 | 118.0 | 3.2 |
| 6 | B7@6' | | | | - medium dense, increase in medium- to coarse-grained | 24 | 116.0 | 2.1 |
| 8 | | | | | | | | |
| 10 | B7@9' | | | SW | Sand, well-graded, dense, dry, light brown to light reddish brown, fine- to coarse-grained, trace fine to medium gravel. | 63 | 116.7 | 1.7 |
| 12 | B7@12' | | | SP | Sand, poorly graded, medium dense, dry, brown, very fine to fine-grained, trace fine gravel and coarse-grained. | 51 | 125.4 | 1.2 |
| 14 | B7@15' | | | | | 69 | 124.7 | 1.6 |

Total depth of boring: 15.5 feet
No fill.
No groundwater encountered.
Backfilled with soil cuttings and tamped.

*Penetration resistance for 140-pound hammer falling 30 inches by auto-hammer.
NOTE: The stratification lines presented herein represent the approximate boundary between earth types; the transitions may be gradual.

W1558-99-01 BORING LOGS.GPJ

**Figure A7,**
**Log of Boring 7, Page 1 of 1**

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

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

GEOCON

# BORING 8

ELEV. (MSL.) - - DATE COMPLETED 05/31/2022

EQUIPMENT HOLLOW STEM AUGER BY: RA

| DEPTH IN FEET | SAMPLE NO. | LITHOLOGY | GROUNDWATER | SOIL CLASS (USCS) | MATERIAL DESCRIPTION | PENETRATION RESISTANCE (BLOWS/FT)* | DRY DENSITY (P.C.F.) | MOISTURE CONTENT (%) |
|---|---|---|---|---|---|---|---|---|
| 0 | | | | | **ALLUVIUM** | | | |
| 1 | B8@1' | | | SP | Sand, poorly graded, medium dense, dry, light reddish brown, fine-grained, trace medium-grained, trace fine gravel. | 30 | 111.1 | 3.4 |
| 3 | B8@3' | | | SW | Sand, well-graded, medium dense, dry, reddish brown, fine- to coarse-grained. | 23 | 114.9 | 2.0 |
| 5-10 | BULK 5-10' | | | | | | | |
| 6 | B8@6' | | | | | 42 | 115.4 | 1.5 |
| 9 | B8@9' | | | | - light reddish brown, trace fine to medium gravel | 41 | 121.4 | 1.0 |
| 12 | B8@12' | | | | - increase in fine to medium gravel | 40 | 107.9 | 5.7 |
| 15 | B8@15' | | | | - decrease in medium-grained and medium gravel | 49 | 116.9 | 1.0 |

Total depth of boring: 15.5 feet
No fill.
No groundwater encountered.
Backfilled with soil cuttings and tamped.

*Penetration resistance for 140-pound hammer falling 30 inches by auto-hammer.
NOTE: The stratification lines presented herein represent the approximate boundary between earth types; the transitions may be gradual.

W1558-99-01 BORING LOGS.GPJ

**Figure A8,**
**Log of Boring 8, Page 1 of 1**

SAMPLE SYMBOLS

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

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

GEOCON

## BORING 9

ELEV. (MSL.) - - DATE COMPLETED 05/31/2022

EQUIPMENT HOLLOW STEM AUGER BY: RA

| 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' | | | | **ALLUVIUM** Sand, poorly graded, loose, dry, light brown to light reddish brown, fine-grained, trace medium- to coarse-grained. | | | |
| 2 | B9@3' | | | SP | - no recovery | 11 | | |
| 4 | | | | | | | | |
| 6 | B9@6' | | | | - medium dense, brown | 40 | 120.7 | 1.6 |
| 8 | | | | | Sand, well-graded, medium dense, dry, reddish brown to brown, fine- to coarse-grained, trace fine gravel. | | | |
| | B9@9' | | | | | 31 | 115.5 | 0.8 |
| 10 | | | | SW | | | | |
| 12 | B9@12' | | | | - increase in medium- to coarse-grained, no gravel | 38 | 112.2 | 0.8 |
| 14 | | | | | | | | |
| | B9@15' | | | | - dense | 58 | 120.2 | 1.1 |

Total depth of boring: 15.5 feet
No fill.
No groundwater encountered.
Backfilled with soil cuttings and tamped.

*Penetration resistance for 140-pound hammer falling 30 inches by auto-hammer.
NOTE: The stratification lines presented herein represent the approximate boundary between earth types; the transitions may be gradual.

**Figure A9,**
**Log of Boring 9, Page 1 of 1**

W1558-99-01 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.

GEOCON

## BORING 10

ELEV. (MSL.) - - DATE COMPLETED 05/31/2022

EQUIPMENT HOLLOW STEM AUGER BY: RA

| 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' | | | | **ALLUVIUM** Sand, poorly graded, medium dense, dry, reddish brown, fine-grained, trace fine gravel. | | | |
| 2 | | | | | | | | |
| | B10@3' | | | | | 27 | 110.6 | 3.2 |
| 4 | | | | SP | | | | |
| 6 | B10@6' | | | | - trace medium- to coarse-grained | 37 | 122.9 | 1.3 |
| 8 | | | | | | | | |
| | B10@9' | | | | Sand, well-graded, medium dense, dry, reddish brown to brown, fine- to coarse-grained, trace fine gravel. | 46 | 114.4 | 1.0 |
| 10 | BULK 10-15' | | | | | | | |
| 12 | B10@12' | | | SW | - decrease in medium- to coarse-grained and gravel | 38 | 117.8 | 4.5 |
| 14 | | | | | - interbeds of well-graded sand with silt | | | |
| | B10@15' | | | | | 50 | 116.7 | 2.0 |

Total depth of boring: 15.5 feet
No fill.
No groundwater encountered.
Backfilled with soil cuttings and tamped.

*Penetration resistance for 140-pound hammer falling 30 inches by auto-hammer.
NOTE: The stratification lines presented herein represent the approximate boundary between earth types; the transitions may be gradual.

W1558-99-01 BORING LOGS.GPJ

**Figure A10,**
**Log of Boring 10, Page 1 of 1**

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

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

GEOCON

## BORING 11

ELEV. (MSL.) - - DATE COMPLETED 05/31/2022

EQUIPMENT HOLLOW STEM AUGER BY: RA

| DEPTH IN FEET | SAMPLE NO. | LITHOLOGY | GROUNDWATER | SOIL CLASS (USCS) | MATERIAL DESCRIPTION | PENETRATION RESISTANCE (BLOWS/FT)* | DRY DENSITY (P.C.F.) | MOISTURE CONTENT (%) |
|---|---|---|---|---|---|---|---|---|
| 0 | | | | | **ALLUVIUM** | | | |
| 1 | B11@1' | | | SP | Sand, poorly graded, loose, dry, reddish brown, fine-grained, trace fine gravel and coarse-grained. | 17 | 114.1 | 2.0 |
| 3 | B11@3' | | | | - trace medium- to coarse-grained, decrease in gravel | 19 | 118.3 | 2.4 |
| 6 | B11@6' | | | | - decrease in medium- to coarse-grained, increase in fine gravel | 13 | 104.2 | 1.2 |
| 8 | | | | | - interbeds of well-graded sand with silt | | | |
| 9 | B11@9' | | | | | 14 | 109.4 | 1.3 |
| 10 | | | | SW | Sand, well-graded, loose, dry, olive brown to reddish brown, fine- to coarse-grained, fine gravel. | | | |
| | | | | | - increase in fine-grained, decrease in medium- to coarse-grained | | | |
| 12 | B11@12' | | | SP | Sand, poorly graded, medium dense, dry, brown to light reddish brown, medium- to coarse-grained, trace fine-grained. | 35 | 115.9 | 0.8 |
| 14 | | | | SW | Sand, well-graded, medium dense, dry, brown, fine- to coarse-grained, trace fine to medium gravel. | | | |
| 15 | B11@15' | | | | | 34 | 116.1 | 0.6 |

Total depth of boring: 15.5 feet
No fill.
No groundwater encountered.
Backfilled with soil cuttings and tamped.

*Penetration resistance for 140-pound hammer falling 30 inches by auto-hammer.
NOTE: The stratification lines presented herein represent the approximate boundary between earth types; the transitions may be gradual.

**Figure A11,**
**Log of Boring 11, Page 1 of 1**

W1558-99-01 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.

GEOCON

PROJECT NO. W1558-99-01

## BORING 12

ELEV. (MSL.) - - DATE COMPLETED 05/31/2022

EQUIPMENT HOLLOW STEM AUGER BY: RA

| 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' | | | | **ALLUVIUM**<br>Sand, poorly graded, loose, dry, light reddish brown, fine-grained, trace fine gravel. | | | |
| 2 | | | | SP | | | | |
| | B12@3' | | | | | 15 | 112.4 | 1.7 |
| 4 | | | | | | | | |
| 6 | BULK 5-10'<br>B12@6' | | | | Sand, well-graded, medium dense, brown, fine- to coarse-grained, trace fine gravel. | 20 | 116.6 | 0.8 |
| 8 | | | | SW | | | | |
| | B12@9' | | | | - reddish brown | 52 | 112.8 | 3.6 |
| 10 | | | | | | | | |
| 12 | B12@12' | | | | Sand, poorly graded, medium dense, brown to light brown, fine-grained, trace medium- to coarse-grained, trace fine gravel. | 42 | 119.6 | 1.2 |
| 14 | | | | SP | | | | |
| | B12@15' | | | | | 50 | 116.6 | 1.6 |

Total depth of boring: 15.5 feet
No fill.
No groundwater encountered.
Backfilled with soil cuttings and tamped.

*Penetration resistance for 140-pound hammer falling 30 inches by auto-hammer.
NOTE: The stratification lines presented herein represent the approximate boundary between earth types; the transitions may be gradual.

**Figure A12,**
**Log of Boring 12, Page 1 of 1**

W1558-99-01 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.

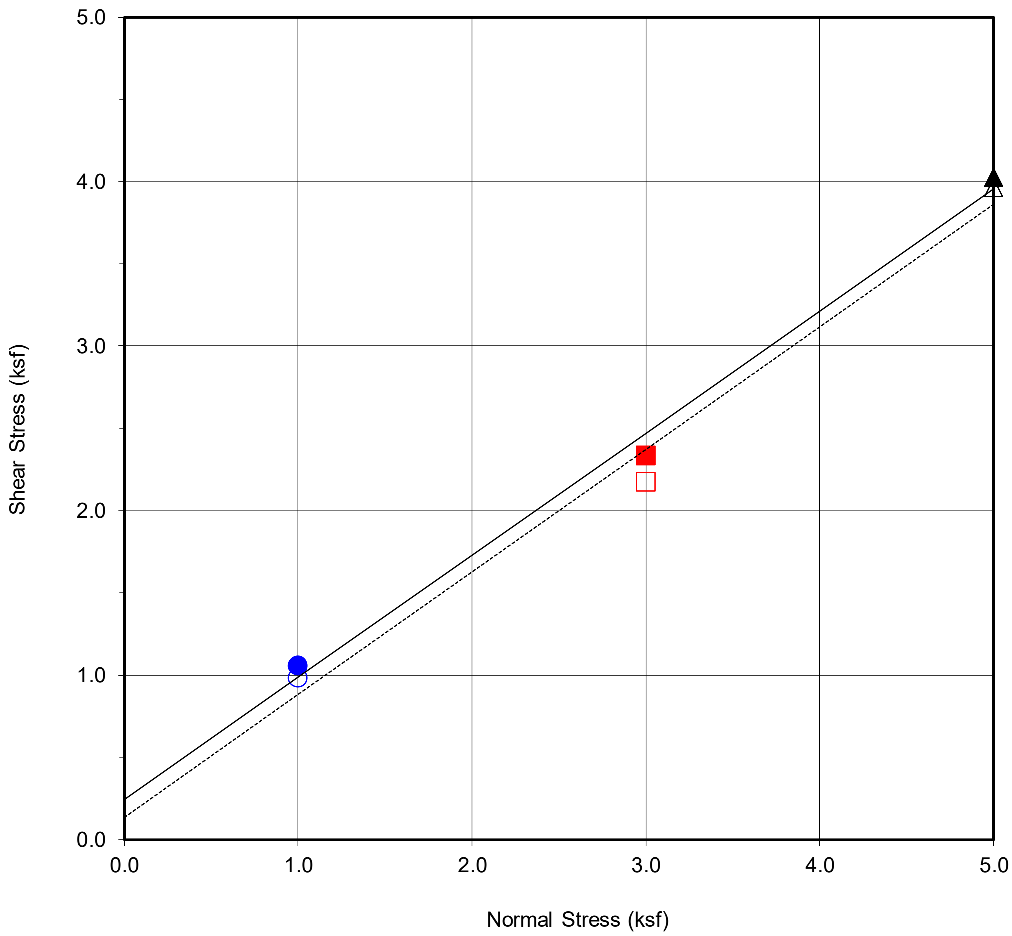
GEOCON

# APPENDIX B

# APPENDIX B

## LABORATORY TESTING

Laboratory tests were performed in accordance with generally accepted test methods of the "American Society for Testing and Materials (ASTM)", or other suggested procedures. Selected samples were tested for direct shear strength, moisture density relationship, expansion and consolidation characteristics, corrosivity and 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 log, Appendix A.




| Boring No. | B2 |
|---|---|
| Sample No. | B2@2.5' |
| Depth (ft) | 2.5 |
| Sample Type: | Ring |

| Soil Identification: | | |
|---|---|---|
| Silty Sand (SM) | | |
| **Strength Parameters** | | |
| | C (psf) | $\phi$ (°) |
| Peak | 245 | 36.6 |
| Ultimate | 137 | 36.7 |

| Normal Stress (kip/ft2) | 1 | 3 | 5 |
|---|---|---|---|
| Peak Shear Stress (kip/ft²) | ● 1.06 | ■ 2.33 | ▲ 4.02 |
| Shear Stress @ End of Test (ksf) | ○ 0.98 | □ 2.17 | △ 3.96 |
| 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 (%) | 3.8 | 3.8 | 4.7 |
| Initial Dry Density (pcf) | 113.4 | 116.5 | 113.2 |
| Initial Degree of Saturation (%) | 21.4 | 22.9 | 25.8 |
| Soil Height Before Shearing (in.) | 1.2 | 1.2 | 1.2 |
| Final Moisture Content (%) | 14.4 | 13.0 | 13.2 |

GEOCON

**DIRECT SHEAR TEST RESULTS**

Consolidated Drained ASTM D-3080

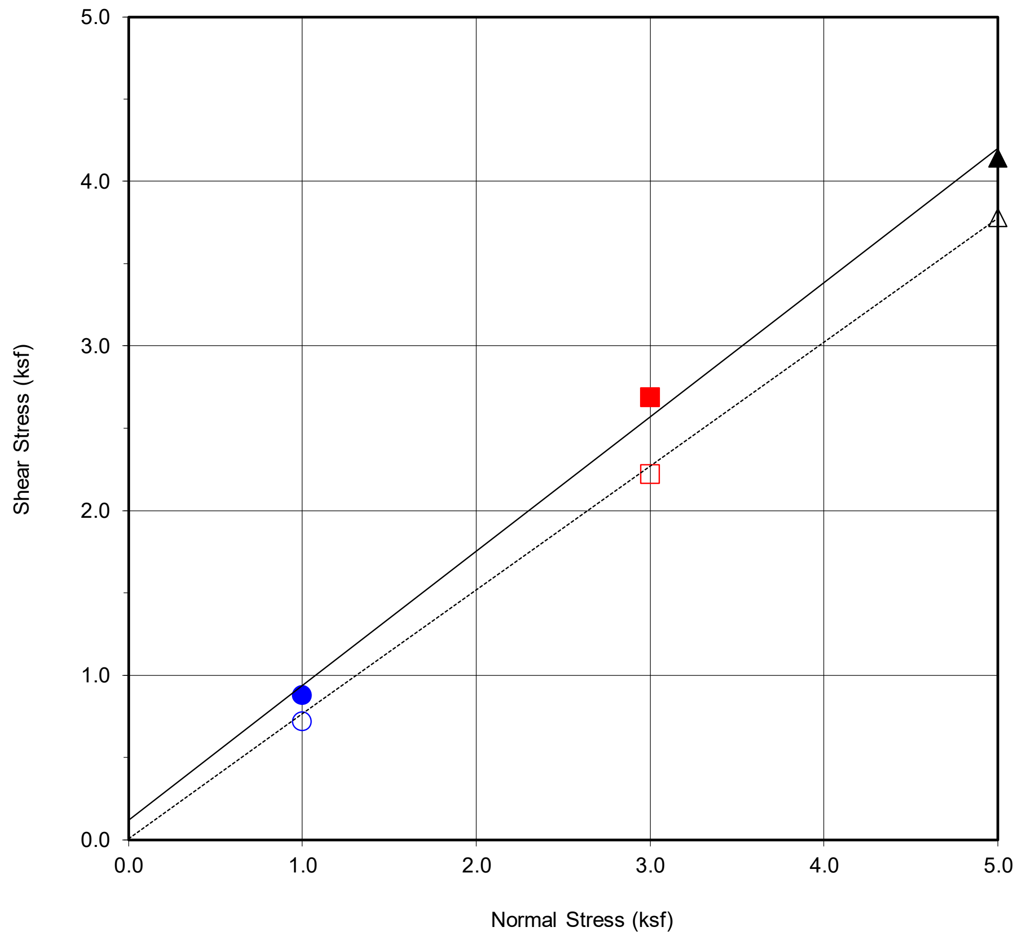
Checked by: JS

Project No.: W1558-99-01

61650 Alta Loma Drive
Joshua Tree, California

July 2022

Figure B1




| | |
|---|---|
| **Boring No.** | **B4** |
| **Sample No.** | **B4@2.5'** |
| **Depth (ft)** | **2.5'** |
| Sample Type: | Ring |

| Soil Identification: | | |
|---|---|---|
| Sand with Silt (SP-SM), with gravel | | |
| **Strength Parameters** | | |
| | C (psf) | ϕ (°) |
| Peak | 120 | 39.2 |
| Ultimate | 10 | 37.0 |

| Normal Stress (kip/ft2) | 1 | 3 | 5 |
|---|---|---|---|
| Peak Shear Stress (kip/ft²) | ● 0.88 | ■ 2.69 | ▲ 4.14 |
| Shear Stress @ End of Test (ksf) | ○ 0.72 | □ 2.22 | △ 3.78 |
| 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 (%) | 3.9 | 3.0 | 4.7 |
| Initial Dry Density (pcf) | 113.2 | 112.3 | 107.7 |
| Initial Degree of Saturation (%) | 21.7 | 16.3 | 22.5 |
| Soil Height Before Shearing (in.) | 1.2 | 1.2 | 1.2 |
| Final Moisture Content (%) | 14.4 | 14.2 | 14.5 |

GEOCON

**DIRECT SHEAR TEST RESULTS**

Consolidated Drained ASTM D-3080

Checked by: JS

Project No.: W1558-99-01

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Joshua Tree, California

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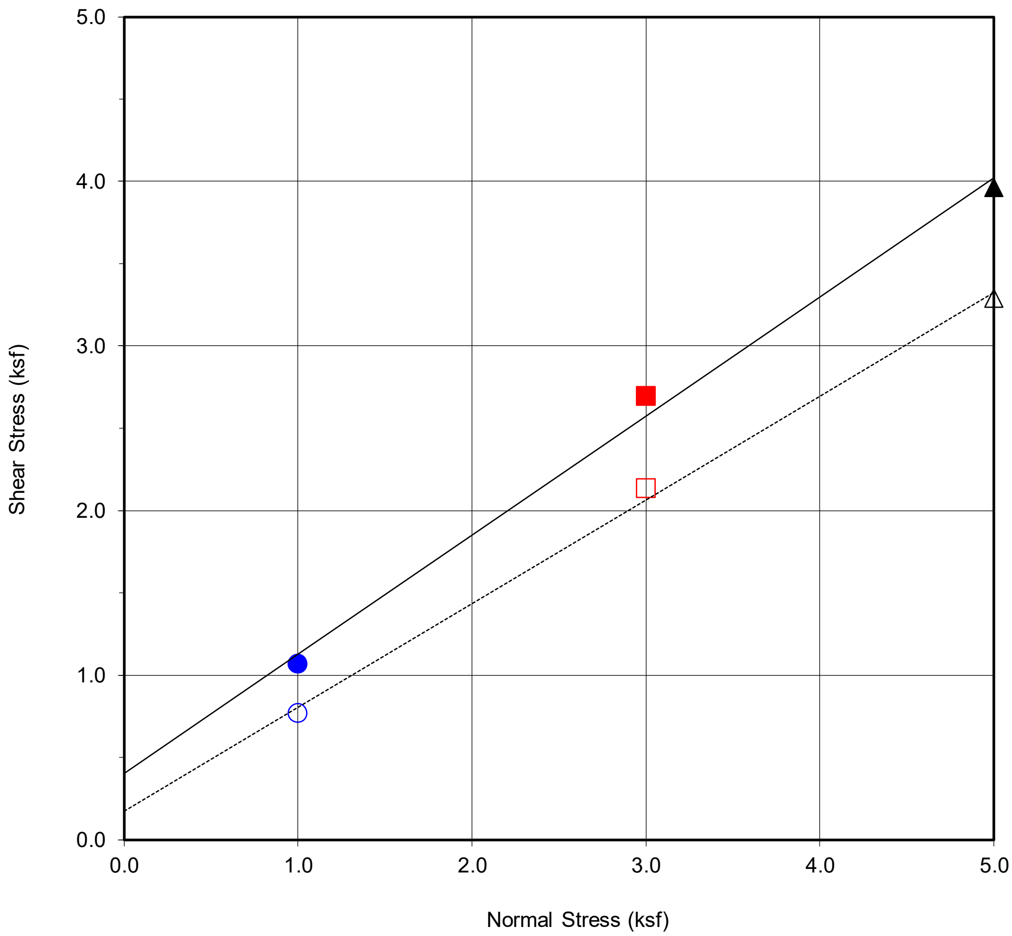
Figure B2




| Boring No. | B5 |
|---|---|
| **Sample No.** | **B5@3'** |
| **Depth (ft)** | **3** |
| Sample Type: | Ring |

| Soil Identification: | | |
|---|---|---|
| Sand (SP), trace Silt | | |
| **Strength Parameters** | | |
| | C (psf) | $\phi$ (°) |
| Peak | 405 | 35.9 |
| Ultimate | 175 | 32.2 |

| Normal Stress (kip/ft2) | 1 | 3 | 5 |
|---|---|---|---|
| Peak Shear Stress (kip/ft²) | ● 1.07 | ■ 2.70 | ▲ 3.96 |
| Shear Stress @ End of Test (ksf) | ○ 0.77 | □ 2.14 | △ 3.29 |
| Deformation Rate (in./min.) | 0.05 | 0.05 | 0.05 |
| | | | |
| Initial Sample Height (in.) | 1.0 | 1.0 | 1.0 |
| Ring Inside Diameter (in.) | 2.375 | 2.375 | 2.375 |
| Initial Moisture Content (%) | 1.1 | 4.3 | 4.7 |
| Initial Dry Density (pcf) | 111.1 | 109.5 | 112.3 |
| Initial Degree of Saturation (%) | 5.8 | 21.5 | 25.1 |
| Soil Height Before Shearing (in.) | 1.2 | 1.2 | 1.2 |
| Final Moisture Content (%) | 13.8 | 16.6 | 15.0 |

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**DIRECT SHEAR TEST RESULTS**

Consolidated Drained ASTM D-3080

Checked by: JS

Project No.: W1558-99-01

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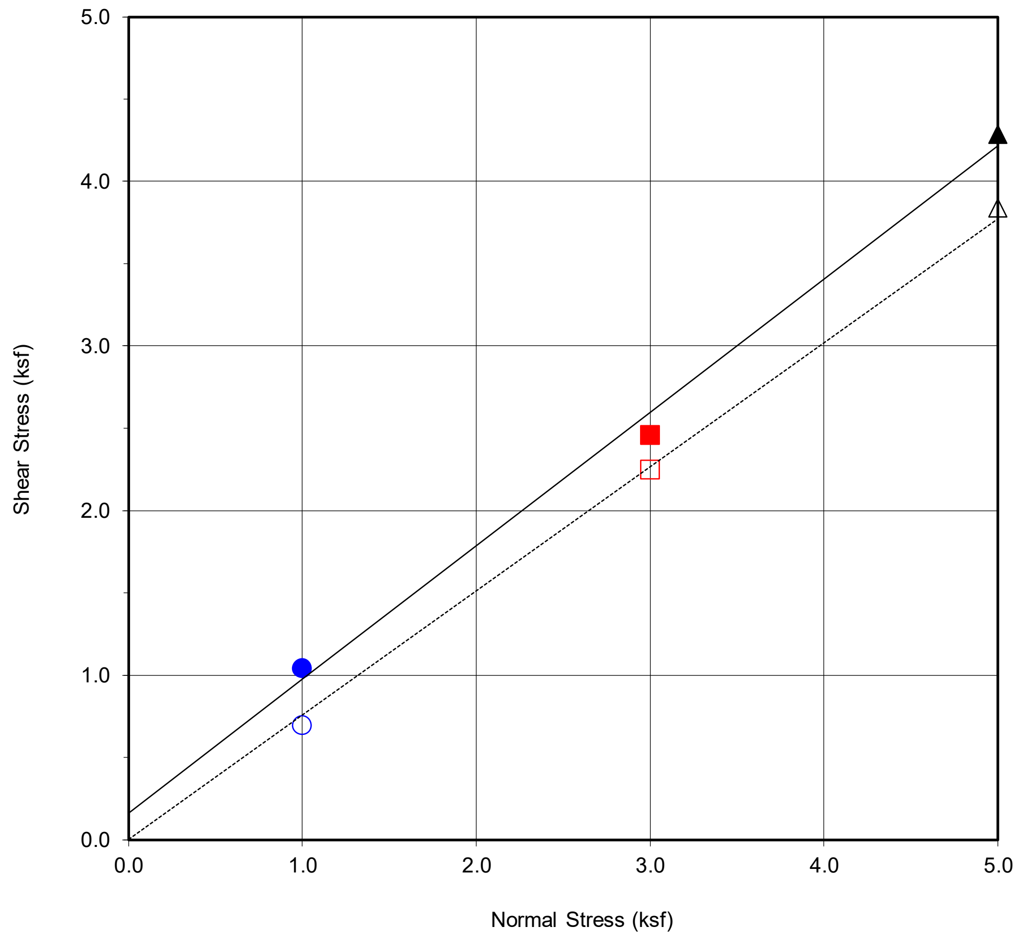
Figure B3




| | |
|---|---|
| **Boring No.** | **B8** |
| **Sample No.** | **B8@12'** |
| **Depth (ft)** | **12** |
| Sample Type: | Ring |

| Normal Stress (kip/ft2) | 1 | 3 | 5 |
|---|---|---|---|
| Peak Shear Stress (kip/ft²) | ● 1.04 | ■ 2.46 | ▲ 4.28 |
| Shear Stress @ End of Test (ksf) | ○ 0.70 | □ 2.25 | △ 3.84 |
| 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 (%) | 5.7 | 3.5 | 3.3 |
| Initial Dry Density (pcf) | 108.0 | 105.0 | 112.1 |
| Initial Degree of Saturation (%) | 27.5 | 15.5 | 17.8 |
| Soil Height Before Shearing (in.) | 1.2 | 1.2 | 1.2 |
| Final Moisture Content (%) | 16.4 | 15.9 | 15.4 |

Soil Identification:

Sand (SW), fine to medium gravel

**Strength Parameters**

| | C (psf) | ϕ (°) |
|---|---|---|
| Peak | 165 | 39.0 |
| Ultimate | 5 | 37.0 |

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**DIRECT SHEAR TEST RESULTS**

Consolidated Drained ASTM D-3080

Checked by: JS

Project No.: W1558-99-01

61650 Alta Loma Drive
Joshua Tree, California

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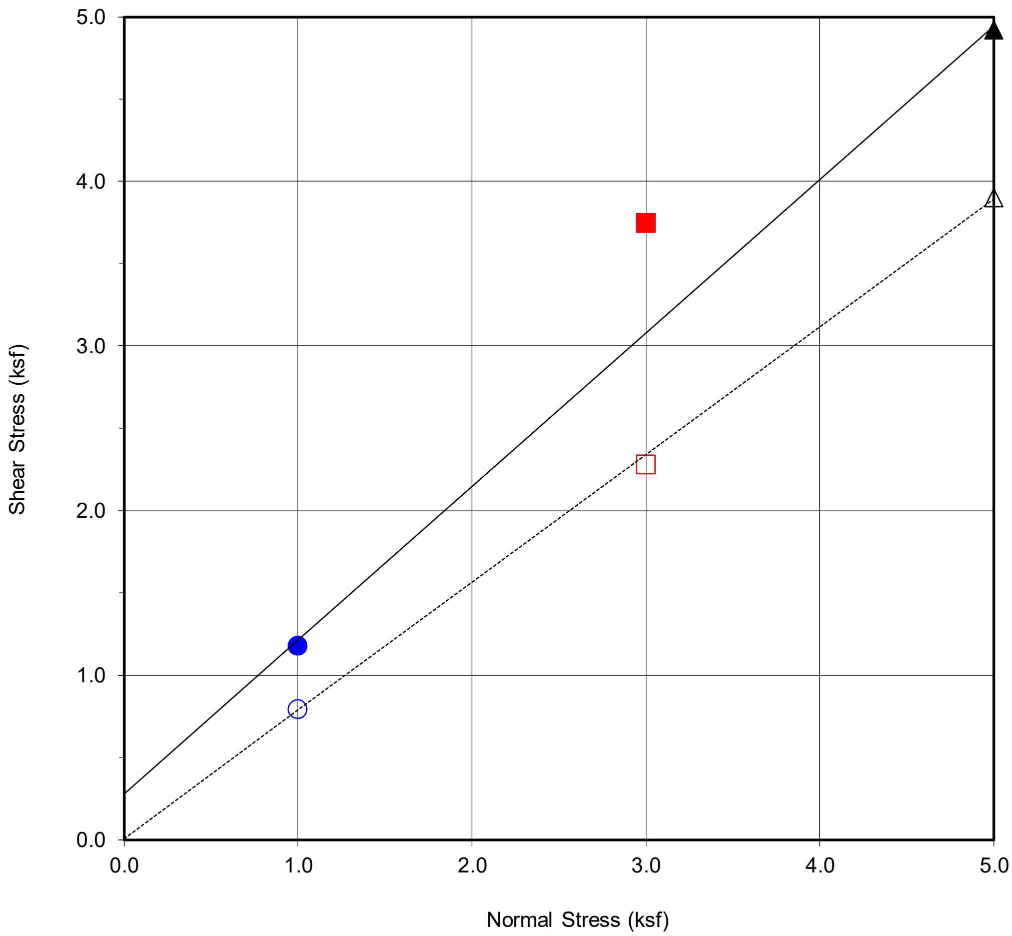
Figure B4




| **Boring No.** | **B10** |
|---|---|
| **Sample No.** | **B10@12'** |
| **Depth (ft)** | **12** |
| Sample Type: | Ring |

| Soil Identification: | | |
|---|---|---|
| Sand (SW), trace fine gravel | | |
| **Strength Parameters** | | |
| | C (psf) | ϕ (°) |
| Peak | 280 | 43.0 |
| Ultimate | 10 | 37.8 |

| Normal Stress (kip/ft2) | 1 | 3 | 5 |
|---|---|---|---|
| Peak Shear Stress (kip/ft²) | ● 1.18 | ■ 3.74 | ▲ 4.92 |
| Shear Stress @ End of Test (ksf) | ○ 0.79 | □ 2.28 | △ 3.90 |
| 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 (%) | 4.5 | 9.7 | 3.9 |
| Initial Dry Density (pcf) | 117.5 | 109.9 | 114.7 |
| Initial Degree of Saturation (%) | 28.3 | 49.2 | 22.4 |
| Soil Height Before Shearing (in.) | 1.2 | 1.2 | 1.2 |
| Final Moisture Content (%) | 14.9 | 15.9 | 14.7 |

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**DIRECT SHEAR TEST RESULTS**

Consolidated Drained ASTM D-3080

Checked by: JS

Project No.: W1558-99-01

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Joshua Tree, California

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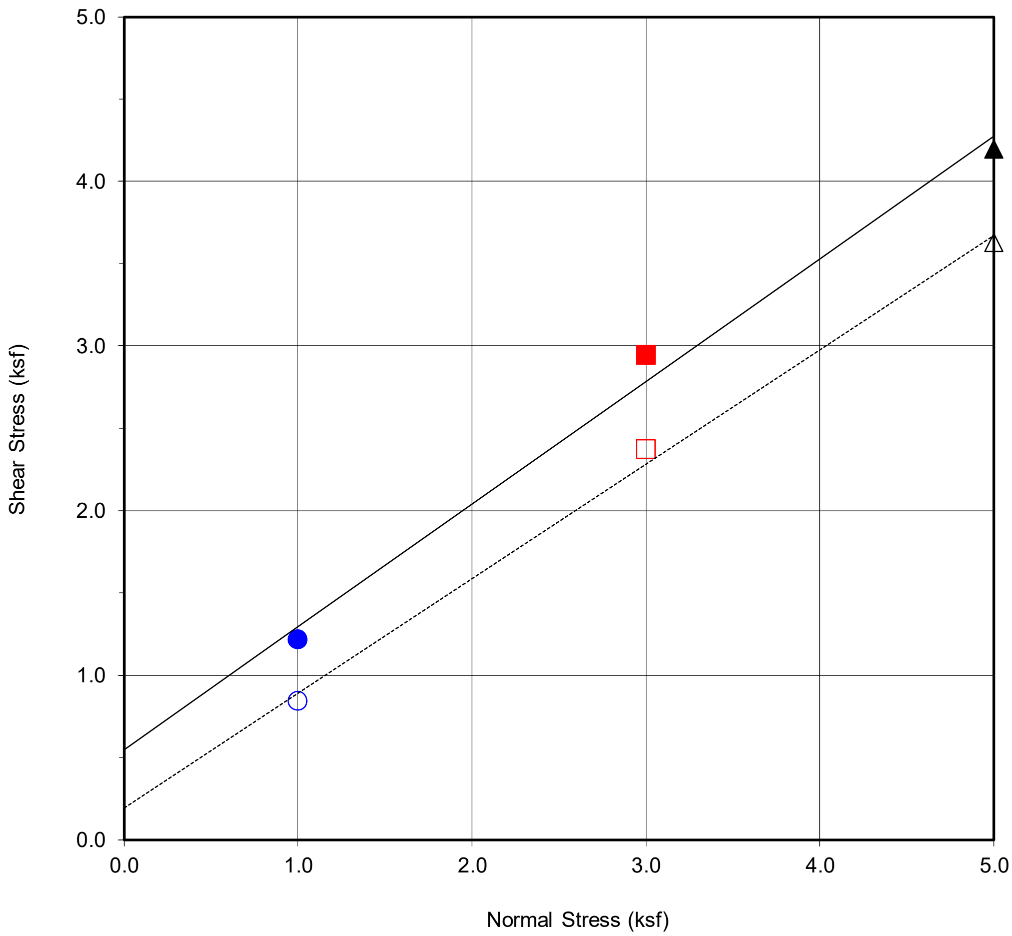
Figure B5




| | |
|---|---|
| **Boring No.** | **B12** |
| **Sample No.** | **B12@9'** |
| **Depth (ft)** | **9** |
| Sample Type: | Ring |

| Soil Identification: | | |
|---|---|---|
| Sand (SW), trace fine gravel | | |
| **Strength Parameters** | | |
| | C (psf) | $\phi$ (°) |
| Peak | 551 | 36.7 |
| Ultimate | 194 | 34.8 |

| Normal Stress (kip/ft2) | 1 | 3 | 5 |
|---|---|---|---|
| Peak Shear Stress (kip/ft²) | ● 1.22 | ■ 2.94 | ▲ 4.20 |
| Shear Stress @ End of Test (ksf) | ○ 0.84 | □ 2.37 | △ 3.63 |
| Deformation Rate (in./min.) | 0.05 | 0.05 | 0.05 |
| | | | |
| Initial Sample Height (in.) | 1.0 | 1.0 | 1.0 |
| Ring Inside Diameter (in.) | 2.375 | 2.375 | 2.375 |
| Initial Moisture Content (%) | 3.6 | 4.7 | 6.1 |
| Initial Dry Density (pcf) | 113.4 | 110.4 | 111.5 |
| Initial Degree of Saturation (%) | 20.1 | 24.1 | 32.0 |
| Soil Height Before Shearing (in.) | 1.2 | 1.2 | 1.2 |
| Final Moisture Content (%) | 15.8 | 15.7 | 15.1 |

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**DIRECT SHEAR TEST RESULTS**

Consolidated Drained ASTM D-3080

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Project No.: W1558-99-01

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Joshua Tree, California

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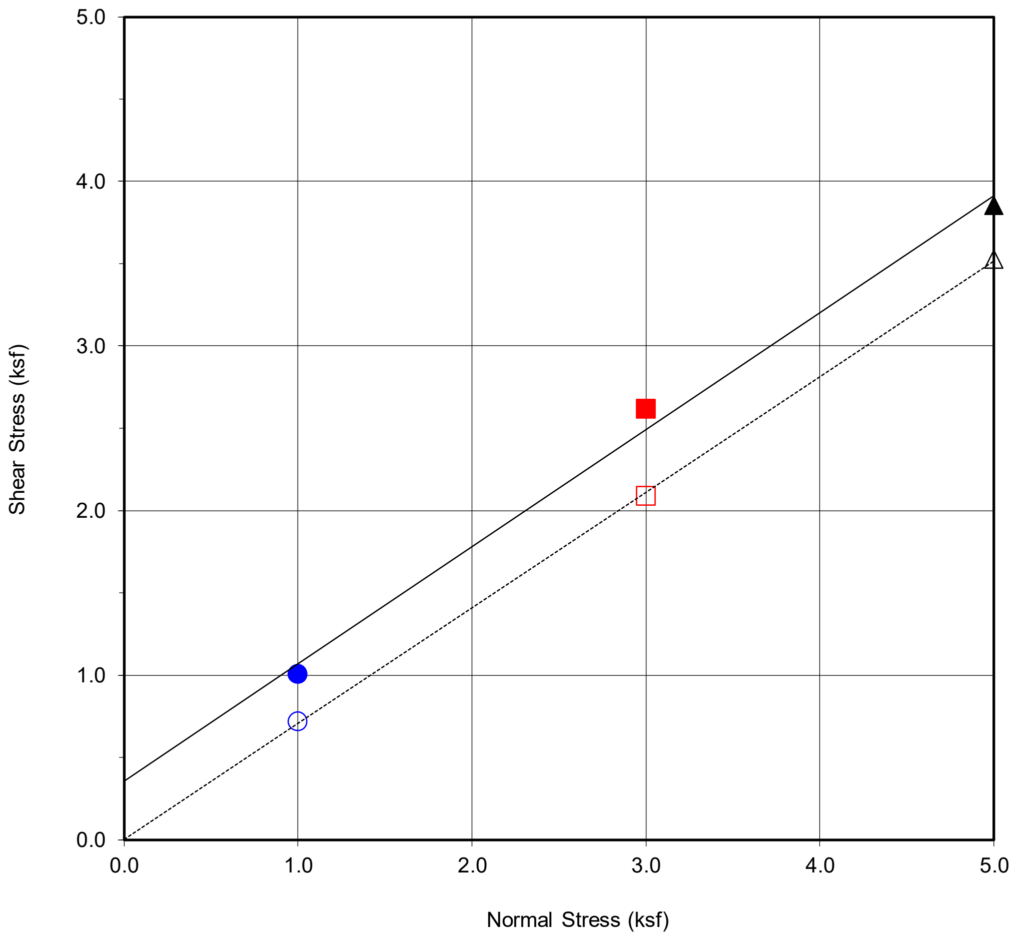
Figure B6




| | |
|---|---|
| **Boring No.** | **B6** |
| **Sample No.** | **B6@0-5'** |
| **Depth (ft)** | **0-5'** |
| Sample Type: | Bulk |

| Soil Identification: | | |
|---|---|---|
| Poorly Graded Sand (SP) | | |
| **Strength Parameters** | | |
| | C (psf) | $\phi$ (°) |
| Peak | 359 | 35.4 |
| Ultimate | 6 | 35.1 |

| Normal Stress (kip/ft2) | 1 | 3 | 5 |
|---|---|---|---|
| Peak Shear Stress (kip/ft²) | ● 1.01 | ■ 2.62 | ▲ 3.85 |
| Shear Stress @ End of Test (ksf) | ○ 0.72 | □ 2.09 | △ 3.53 |
| Deformation Rate (in./min.) | 0.05 | 0.05 | 0.05 |
| | | | |
| Initial Sample Height (in.) | 1.0 | 1.0 | 1.0 |
| Ring Inside Diameter (in.) | 2.375 | 2.375 | 2.375 |
| Initial Moisture Content (%) | 7.1 | 7.1 | 7.1 |
| Initial Dry Density (pcf) | 118.0 | 118.0 | 118.0 |
| Initial Degree of Saturation (%) | 44.8 | 44.9 | 45.1 |
| Soil Height Before Shearing (in.) | 1.2 | 1.2 | 1.2 |
| Final Moisture Content (%) | 13.0 | 12.9 | 12.4 |

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**DIRECT SHEAR TEST RESULTS**

Consolidated Drained ASTM D-3080

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Project No.: W1558-99-01

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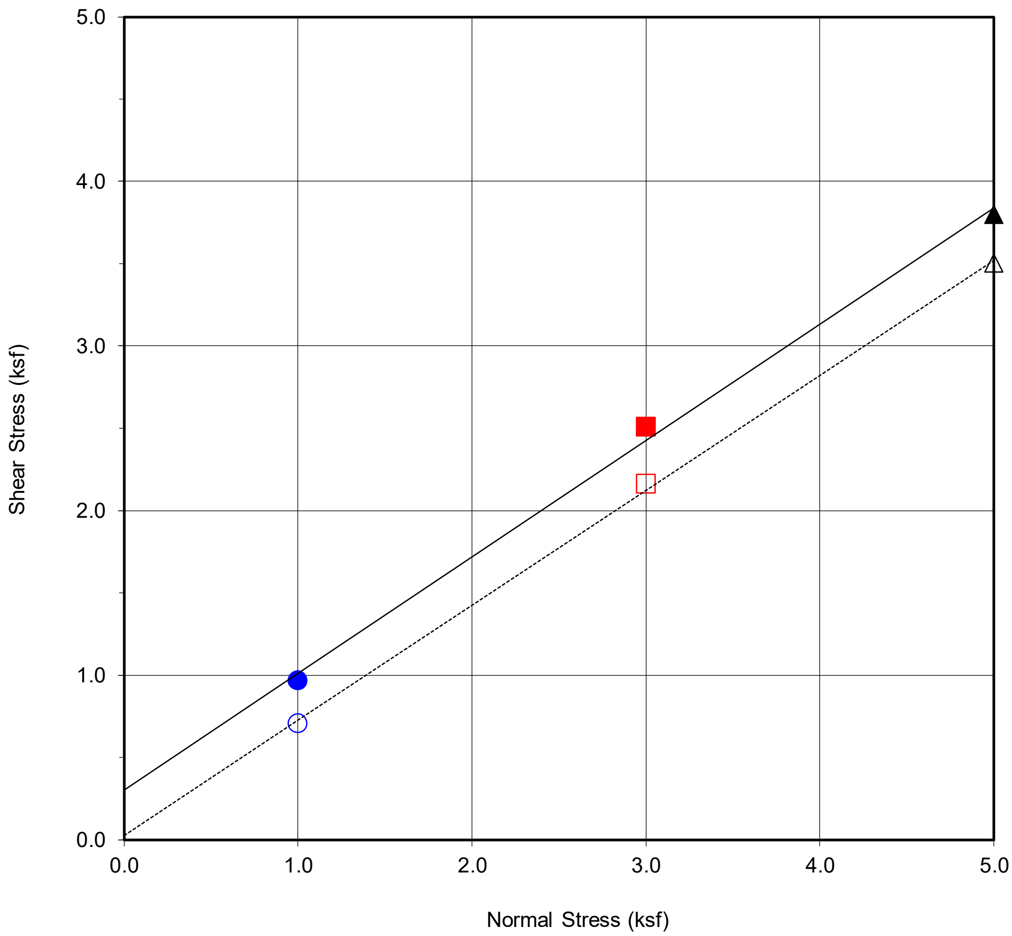
Figure B7




| | |
|---|---|
| **Boring No.** | **B8** |
| **Sample No.** | **B8@5-10'** |
| **Depth (ft)** | **5-10'** |
| Sample Type: | Bulk |

| Soil Identification: | | |
|---|---|---|
| Poorly Graded Sand (SP) | | |
| **Strength Parameters** | | |
| | C (psf) | $\phi$ (°) |
| Peak | 302 | 35.3 |
| Ultimate | 28 | 34.9 |

| Normal Stress (kip/ft2) | 1 | 3 | 5 |
|---|---|---|---|
| Peak Shear Stress (kip/ft²) | ● 0.97 | ■ 2.51 | ▲ 3.80 |
| Shear Stress @ End of Test (ksf) | ○ 0.71 | □ 2.16 | △ 3.50 |
| Deformation Rate (in./min.) | 0.05 | 0.05 | 0.05 |
| | | | |
| Initial Sample Height (in.) | 1.0 | 1.0 | 1.0 |
| Ring Inside Diameter (in.) | 2.375 | 2.375 | 2.375 |
| Initial Moisture Content (%) | 6.8 | 8.5 | 8.3 |
| Initial Dry Density (pcf) | 116.0 | 116.0 | 116.0 |
| Initial Degree of Saturation (%) | 40.3 | 50.5 | 49.4 |
| Soil Height Before Shearing (in.) | 1.2 | 1.2 | 1.2 |
| Final Moisture Content (%) | 14.2 | 14.1 | 14.5 |

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**DIRECT SHEAR TEST RESULTS**

Consolidated Drained ASTM D-3080

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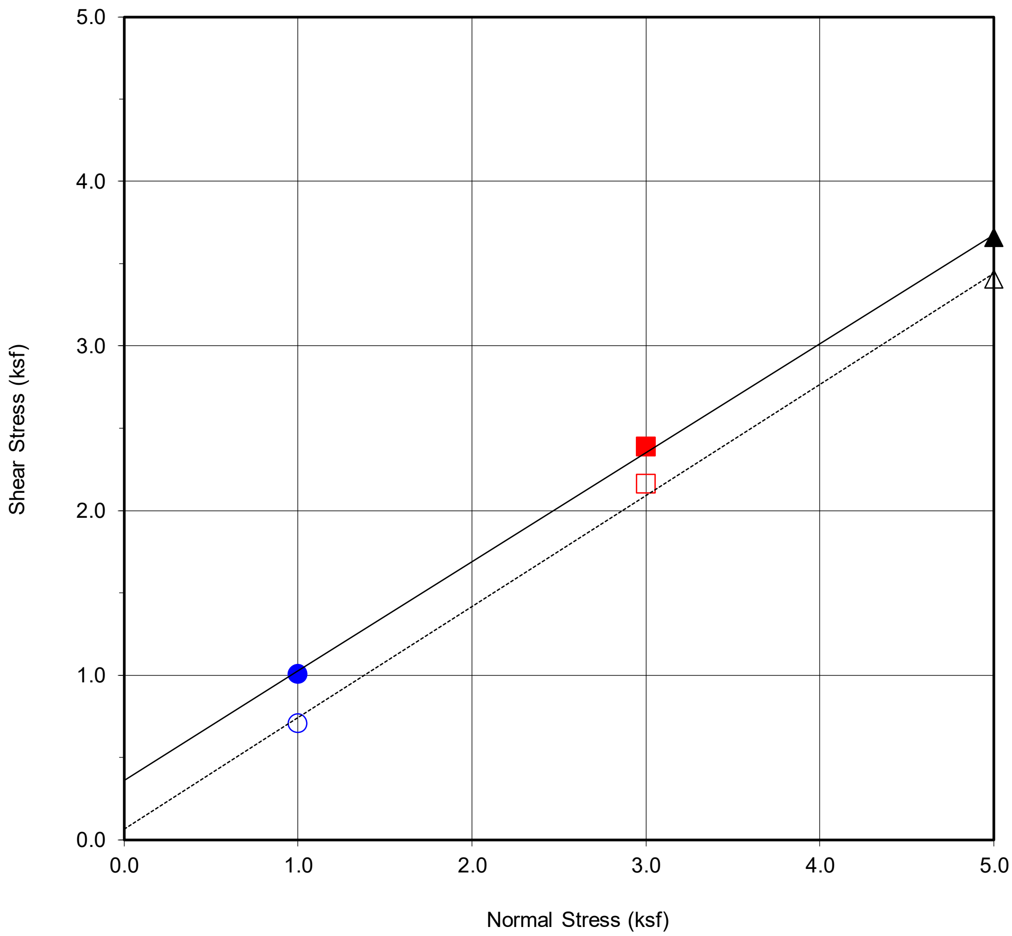
Figure B8




| Boring No. | B10 |
|---|---|
| Sample No. | B10@0-5' |
| Depth (ft) | 0-5' |
| Sample Type: | Bulk |

| Soil Identification: | | |
|---|---|---|
| Poorly Graded Sand (SP) | | |
| **Strength Parameters** | | |
| | C (psf) | $\phi$ (°) |
| Peak | 363 | 33.5 |
| Ultimate | 67 | 34.0 |

| Normal Stress (kip/ft2) | 1 | 3 | 5 |
|---|---|---|---|
| Peak Shear Stress (kip/ft²) | ● 1.01 | ■ 2.39 | ▲ 3.66 |
| Shear Stress @ End of Test (ksf) | ○ 0.71 | □ 2.16 | △ 3.41 |
| 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 | 8.2 | 8.3 |
| Initial Dry Density (pcf) | 117.0 | 117.0 | 117.0 |
| Initial Degree of Saturation (%) | 50.7 | 50.2 | 51.1 |
| Soil Height Before Shearing (in.) | 1.2 | 1.2 | 1.2 |
| Final Moisture Content (%) | 14.2 | 14.3 | 12.8 |

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## DIRECT SHEAR TEST RESULTS

Consolidated Drained ASTM D-3080

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Project No.: W1558-99-01

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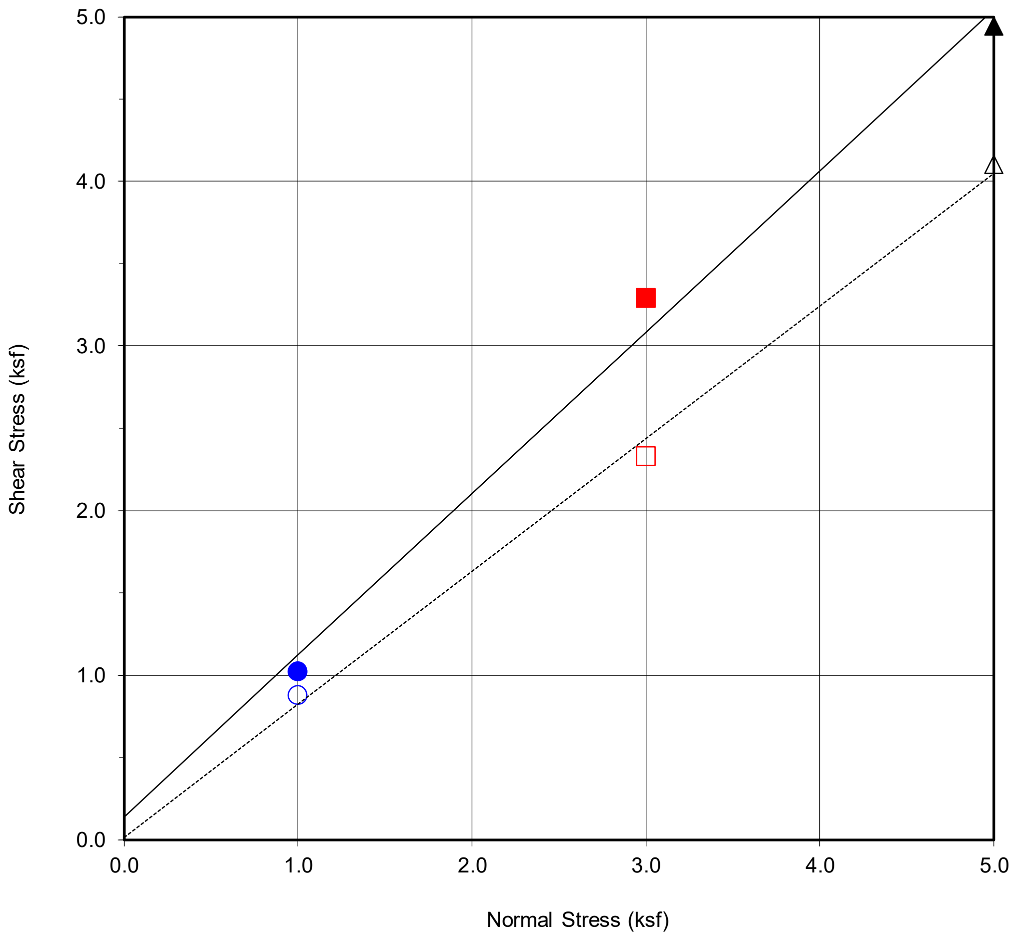
Figure B9




| Boring No. | B12 |
|---|---|
| Sample No. | B12@5-10' |
| Depth (ft) | 5-10' |
| Sample Type: | Bulk |

| Soil Identification: | | |
|---|---|---|
| Well Graded Sand (SW), trace fine gravel | | |
| **Strength Parameters** | | |
| | C (psf) | $\phi$ ($^o$) |
| Peak | 141 | 44.5 |
| Ultimate | 15 | 38.9 |

| Normal Stress (kip/ft2) | 1 | 3 | 5 |
|---|---|---|---|
| Peak Shear Stress (kip/ft²) | ● 1.02 | ■ 3.29 | ▲ 4.94 |
| Shear Stress @ End of Test (ksf) | ○ 0.88 | □ 2.33 | △ 4.10 |
| Deformation Rate (in./min.) | 0.05 | 0.05 | 0.05 |
| | | | |
| Initial Sample Height (in.) | 1.0 | 1.0 | 1.0 |
| Ring Inside Diameter (in.) | 2.375 | 2.375 | 2.375 |
| Initial Moisture Content (%) | 13.8 | 13.7 | 13.4 |
| Initial Dry Density (pcf) | 106.0 | 106.0 | 106.0 |
| Initial Degree of Saturation (%) | 63.0 | 62.8 | 61.4 |
| Soil Height Before Shearing (in.) | 1.2 | 1.2 | 1.2 |
| Final Moisture Content (%) | 17.5 | 17.9 | 18.5 |

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**DIRECT SHEAR TEST RESULTS**

Consolidated Drained ASTM D-3080

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Project No.: W1558-99-01

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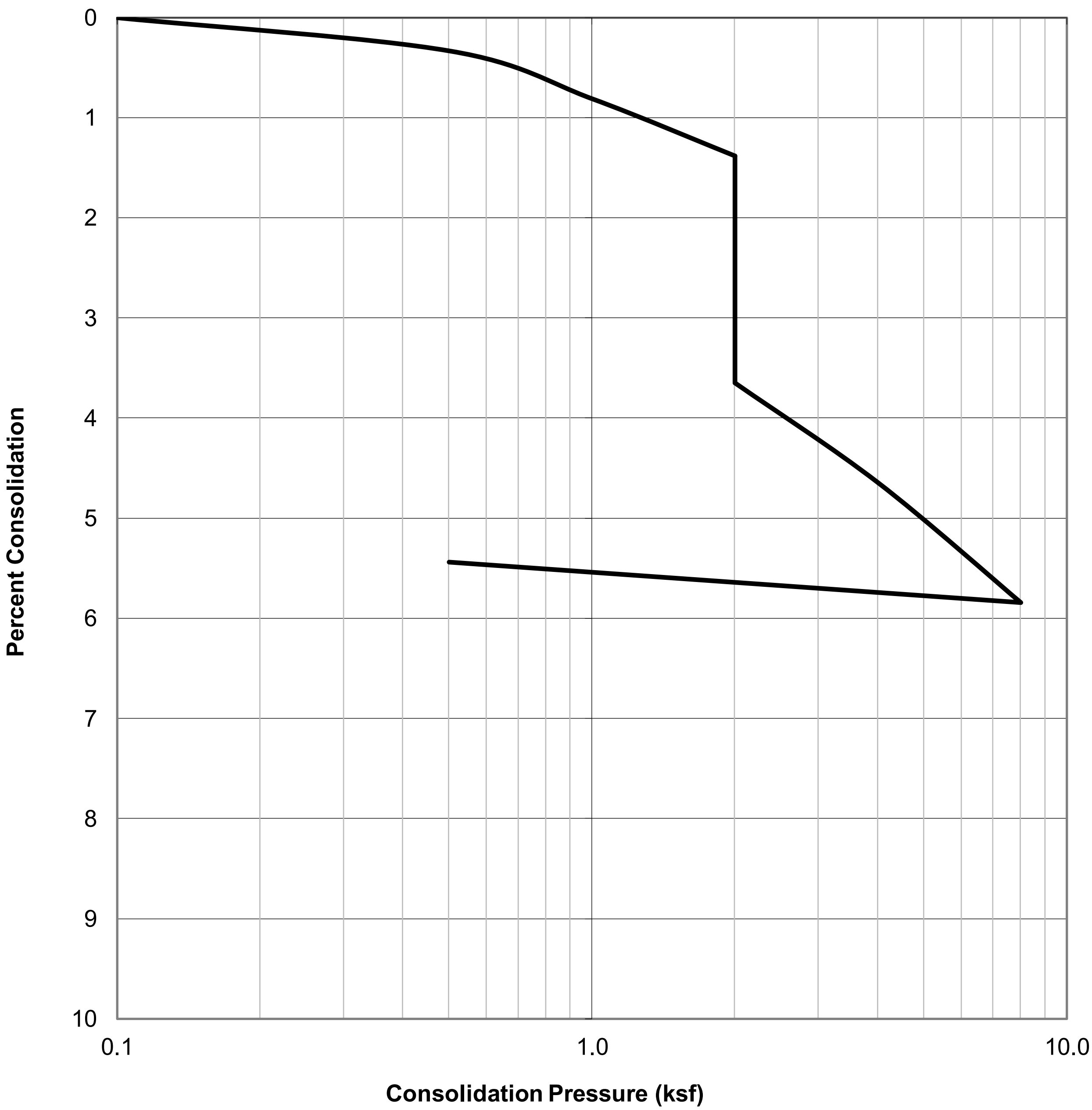
Figure B10

WATER ADDED AT 2.0 KSF




| SAMPLE ID. | SOIL TYPE | DRY DENSITY (PCF) | INITIAL MOISTURE (%) | FINAL MOISTURE (%) |
|---|---|---|---|---|
| B2@2.5 | Sand with Silt (SP-SM) | 116.5 | 2.5 | 12.7 |

GEOCON

**CONSOLIDATION TEST RESULTS**

ASTM D-2435

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Project No.: W1558-99-01

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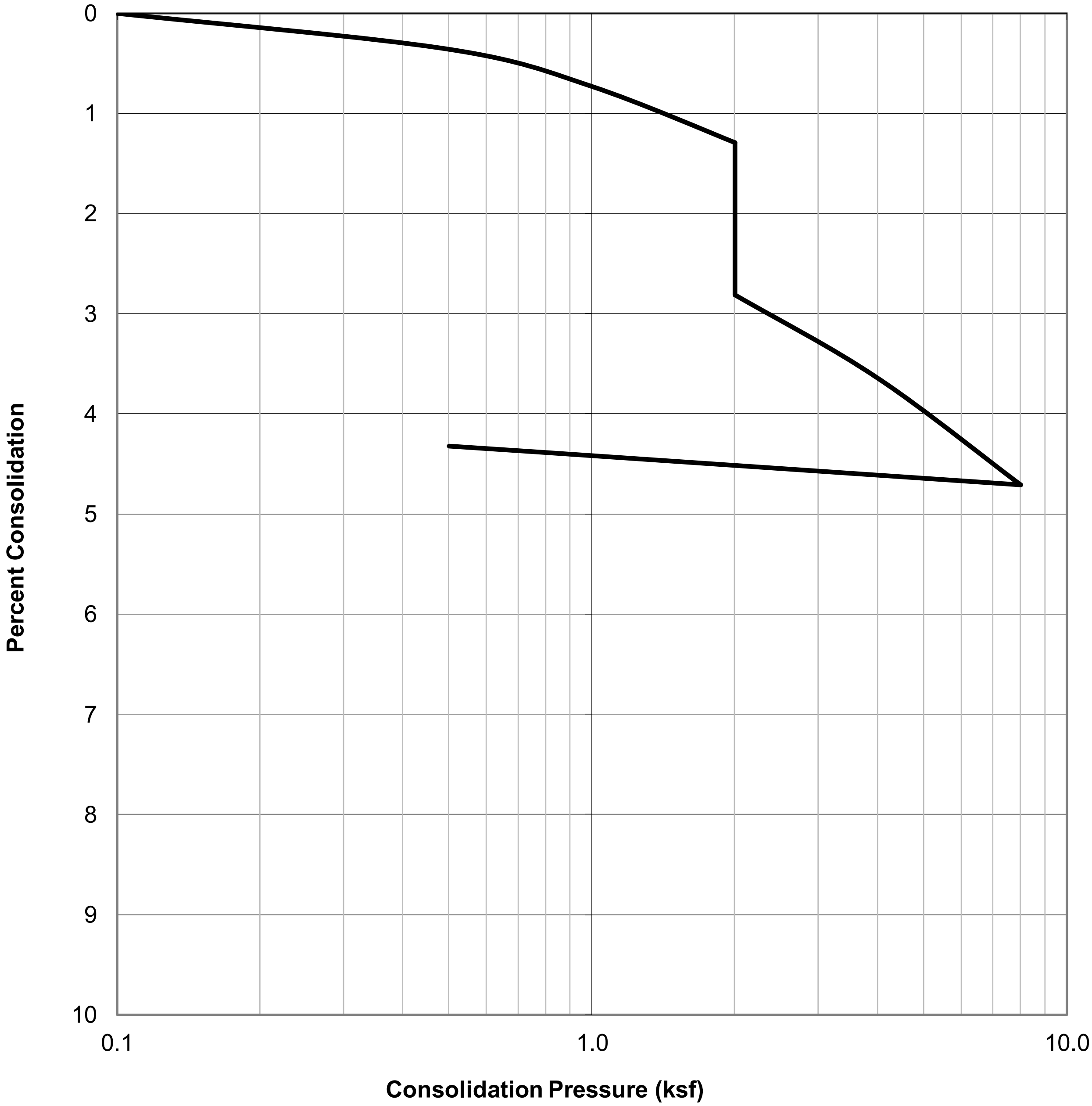
Figure B11

WATER ADDED AT 2.0 KSF




| SAMPLE ID. | SOIL TYPE | DRY DENSITY (PCF) | INITIAL MOISTURE (%) | FINAL MOISTURE (%) |
|---|---|---|---|---|
| B2@7.5 | Poorly Graded Sand (SP) | 112.1 | 2.5 | 14.7 |

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**CONSOLIDATION TEST RESULTS**

ASTM D-2435

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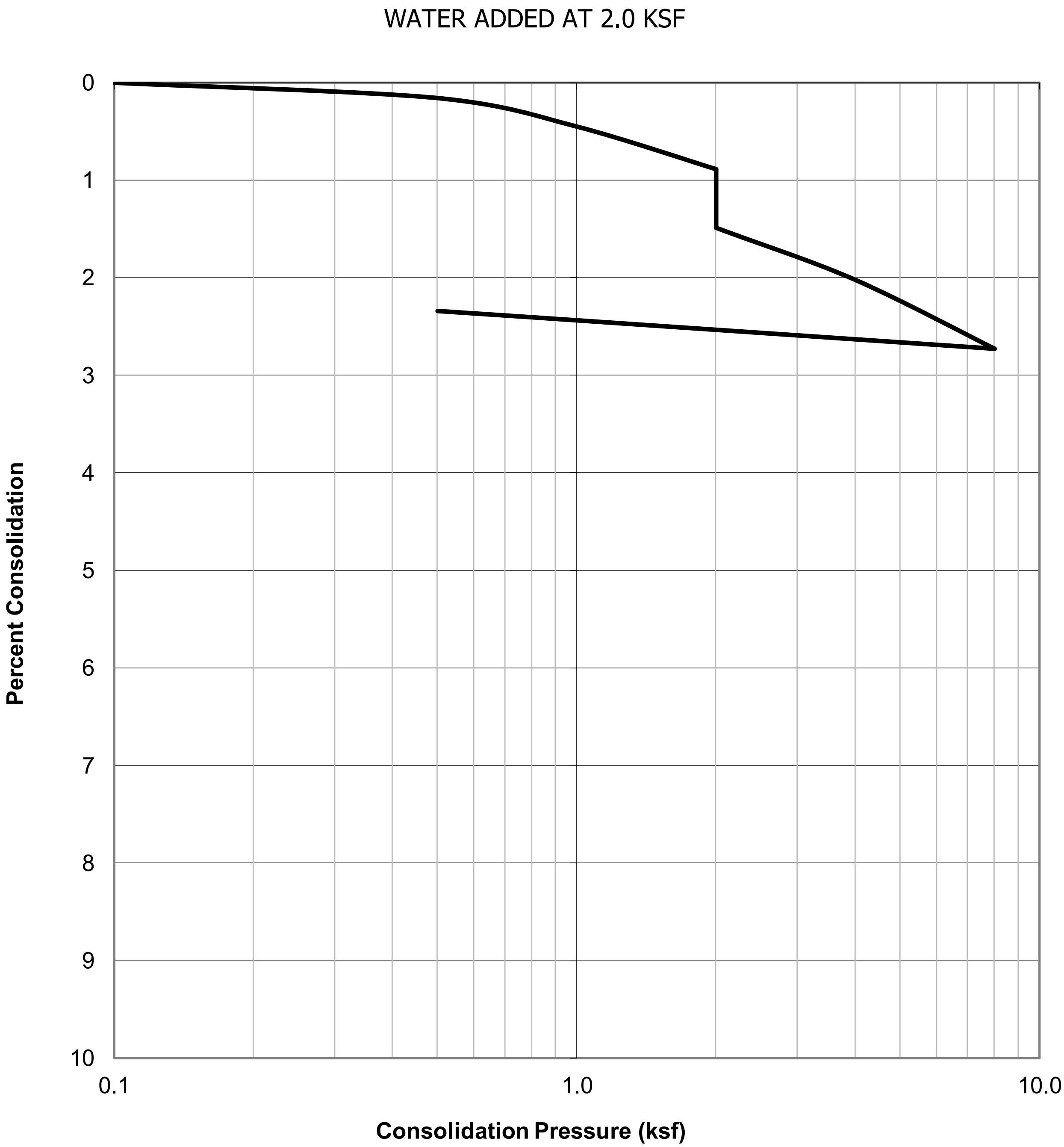
Figure B12




| SAMPLE ID. | SOIL TYPE | DRY DENSITY (PCF) | INITIAL MOISTURE (%) | FINAL MOISTURE (%) |
|---|---|---|---|---|
| B2@12.5 | Poorly Graded Sand (SP) | 111.4 | 1.4 | 16.1 |

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**CONSOLIDATION TEST RESULTS**

ASTM D-2435

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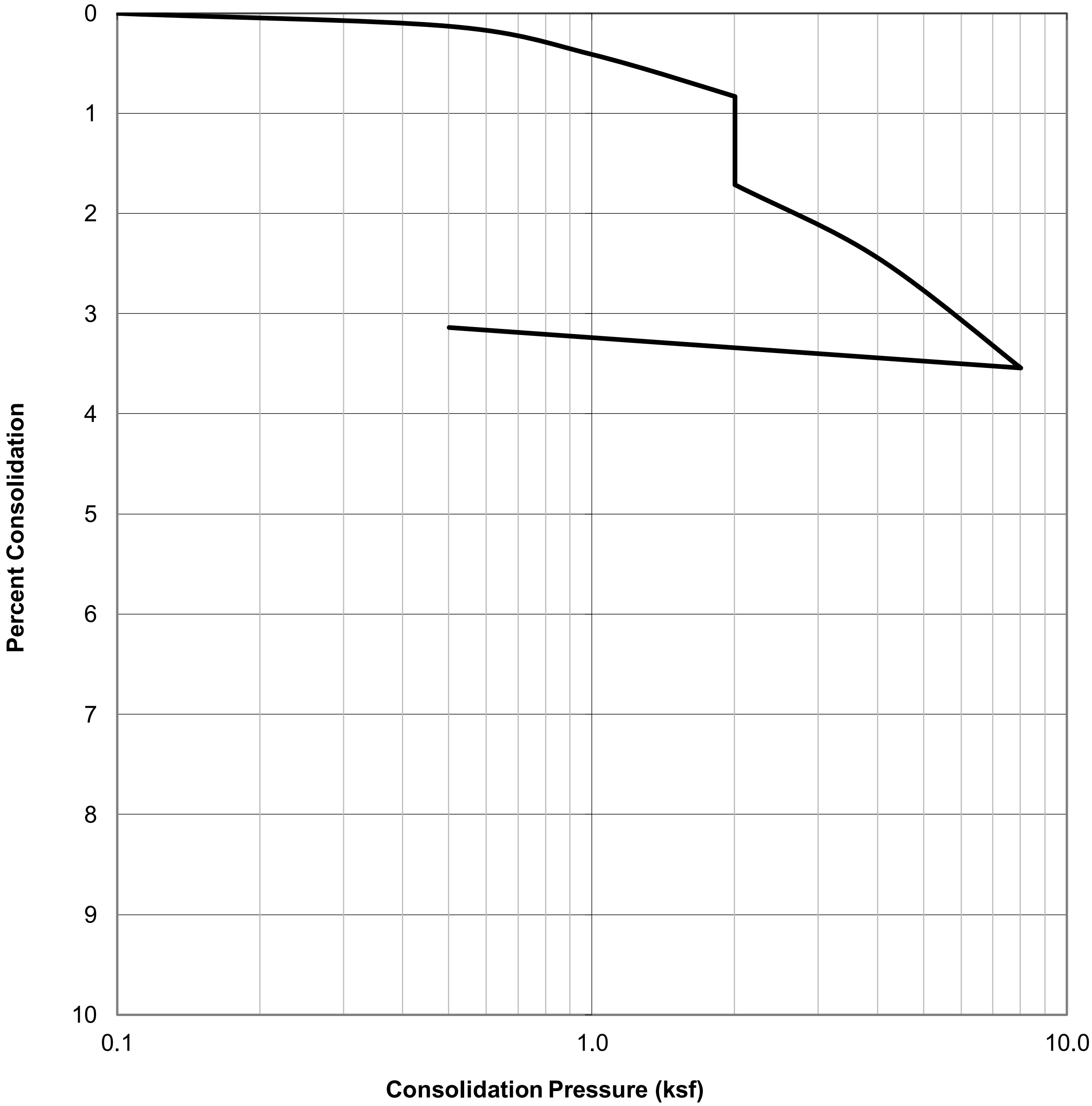
Figure B13

WATER ADDED AT 2.0 KSF




| SAMPLE ID. | SOIL TYPE | DRY DENSITY (PCF) | INITIAL MOISTURE (%) | FINAL MOISTURE (%) |
|---|---|---|---|---|
| B3@6 | Poorly Graded Sand (SP) | 104.9 | 3.8 | 16.5 |

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**CONSOLIDATION TEST RESULTS**

ASTM D-2435

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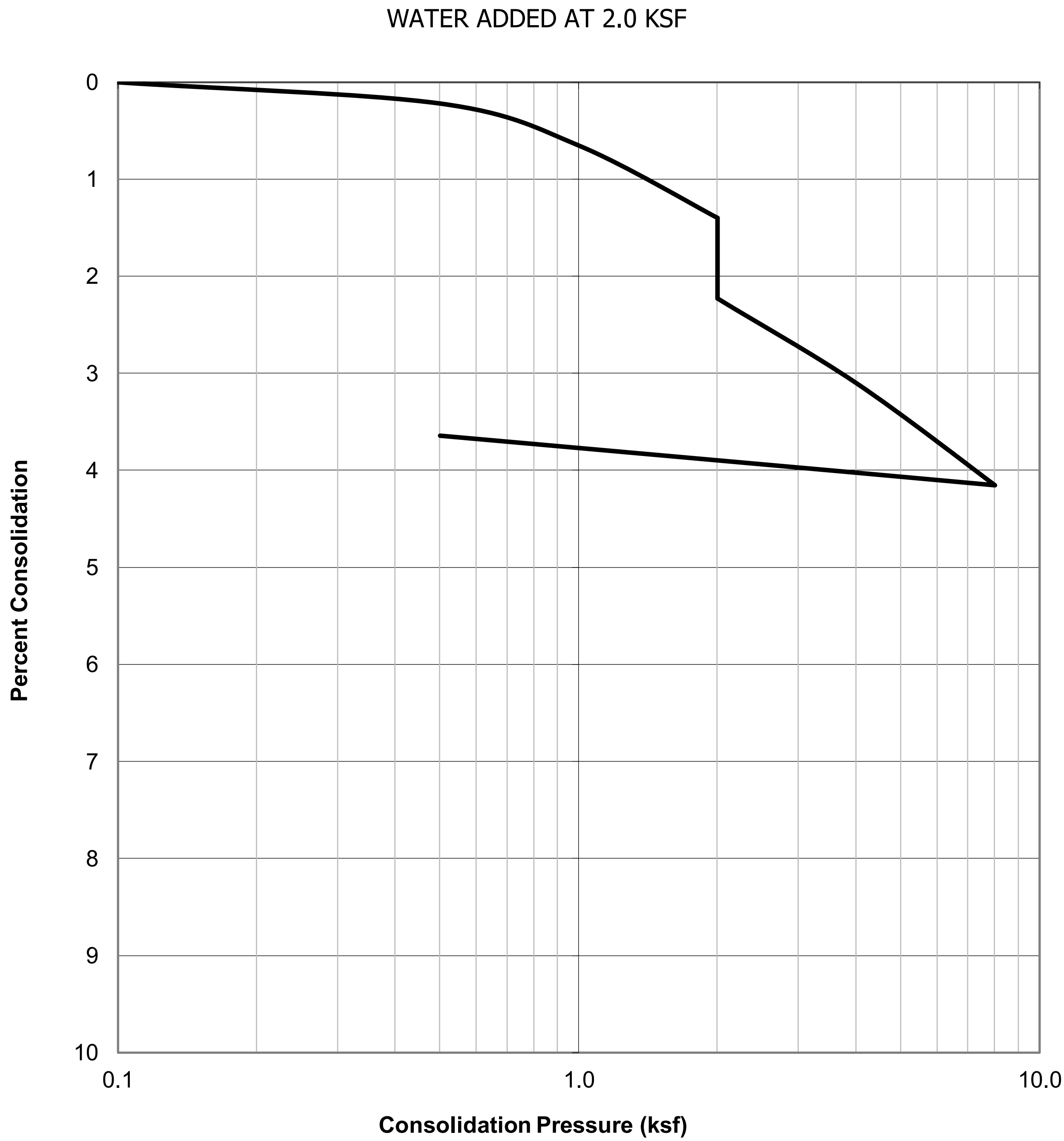
Figure B14




| SAMPLE ID. | SOIL TYPE | DRY DENSITY (PCF) | INITIAL MOISTURE (%) | FINAL MOISTURE (%) |
|---|---|---|---|---|
| B5@3 | Poorly Graded Sand (SP) trace silt | 108.6 | 3.3 | 15.7 |

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**CONSOLIDATION TEST RESULTS**

ASTM D-2435

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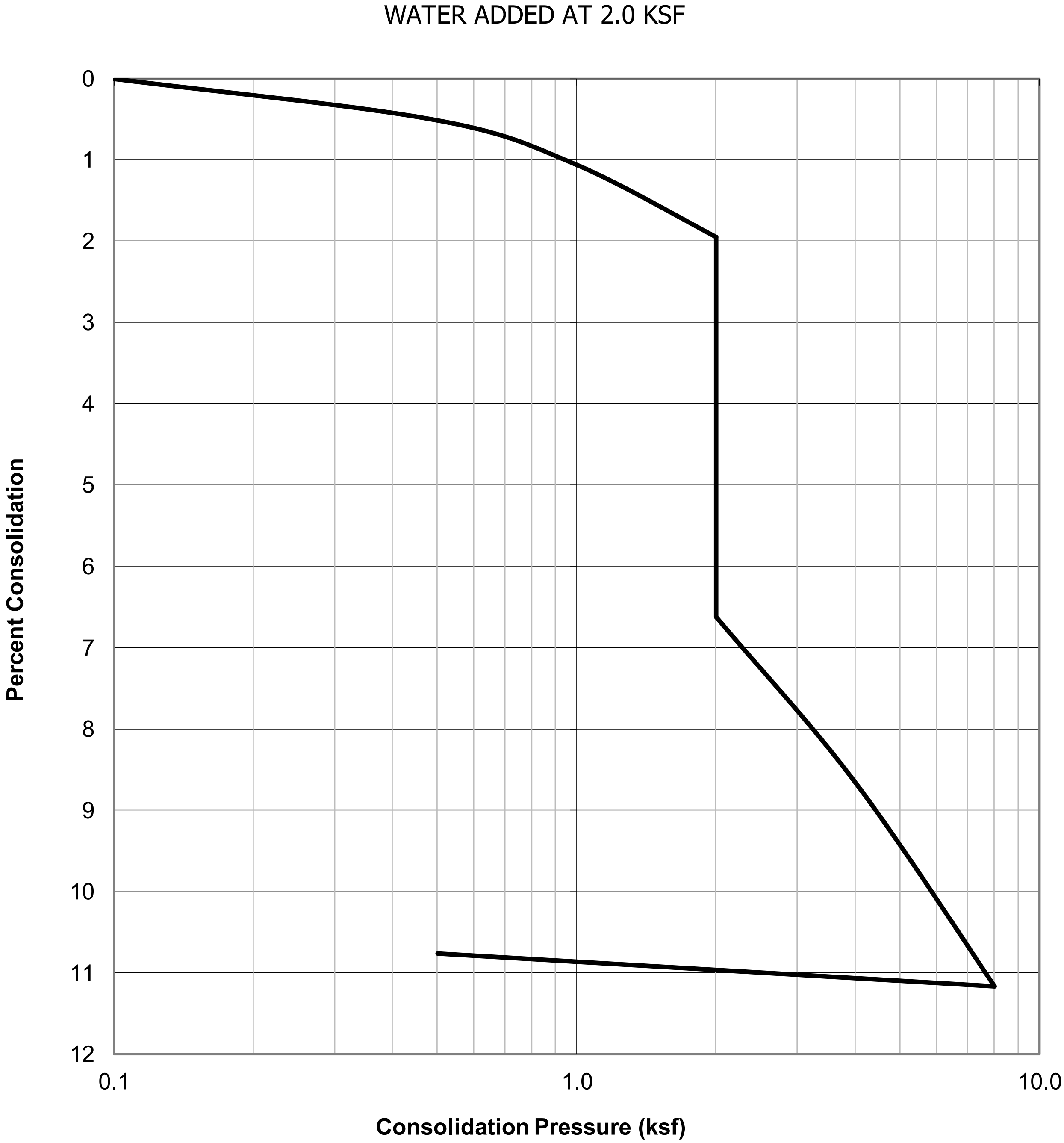
Figure B15




| SAMPLE ID. | SOIL TYPE | DRY DENSITY (PCF) | INITIAL MOISTURE (%) | FINAL MOISTURE (%) |
|---|---|---|---|---|
| B8@1 | Poorly Graded Sand (SP) | 106.5 | 3.4 | 13.5 |

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**CONSOLIDATION TEST RESULTS**

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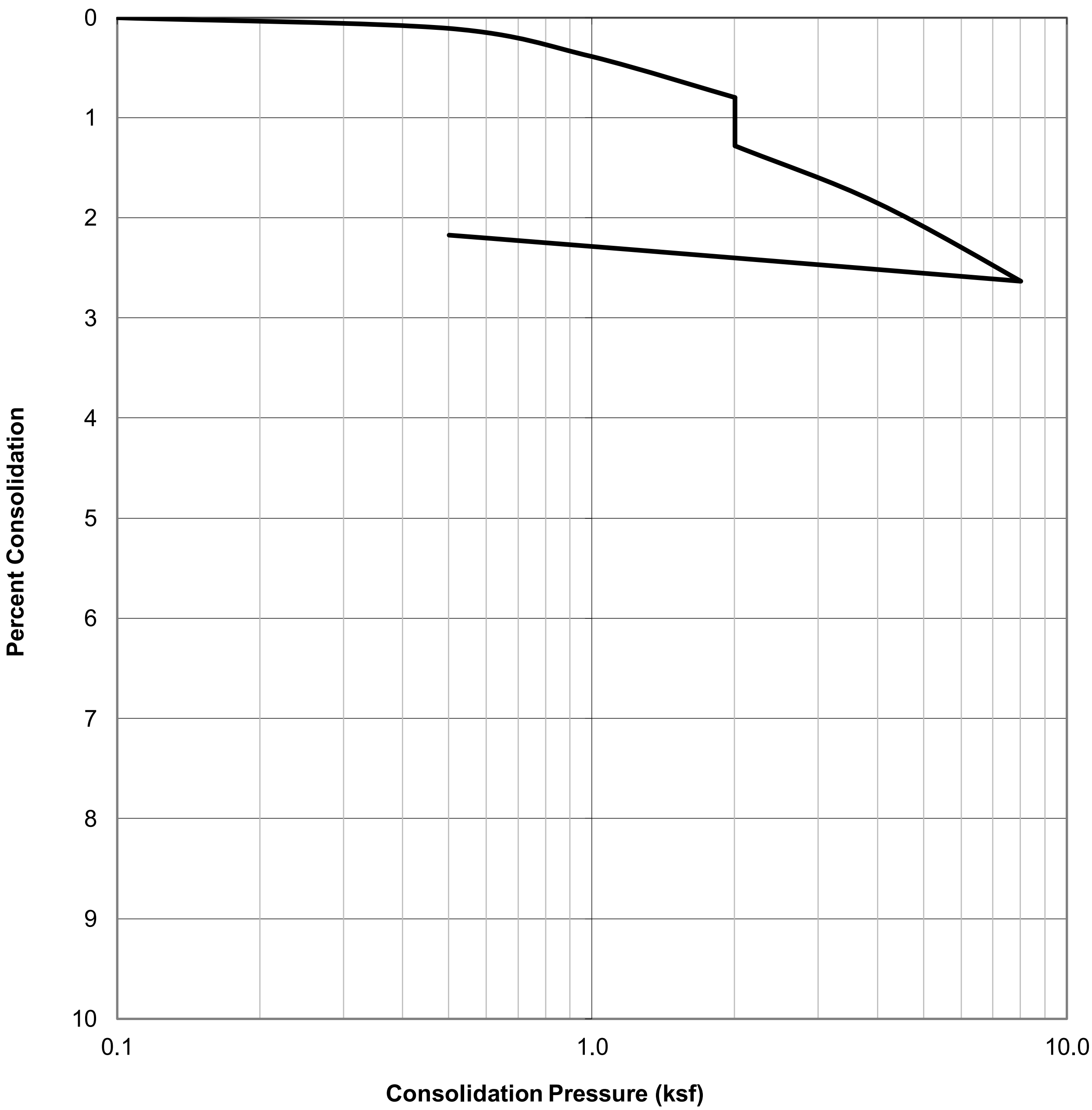
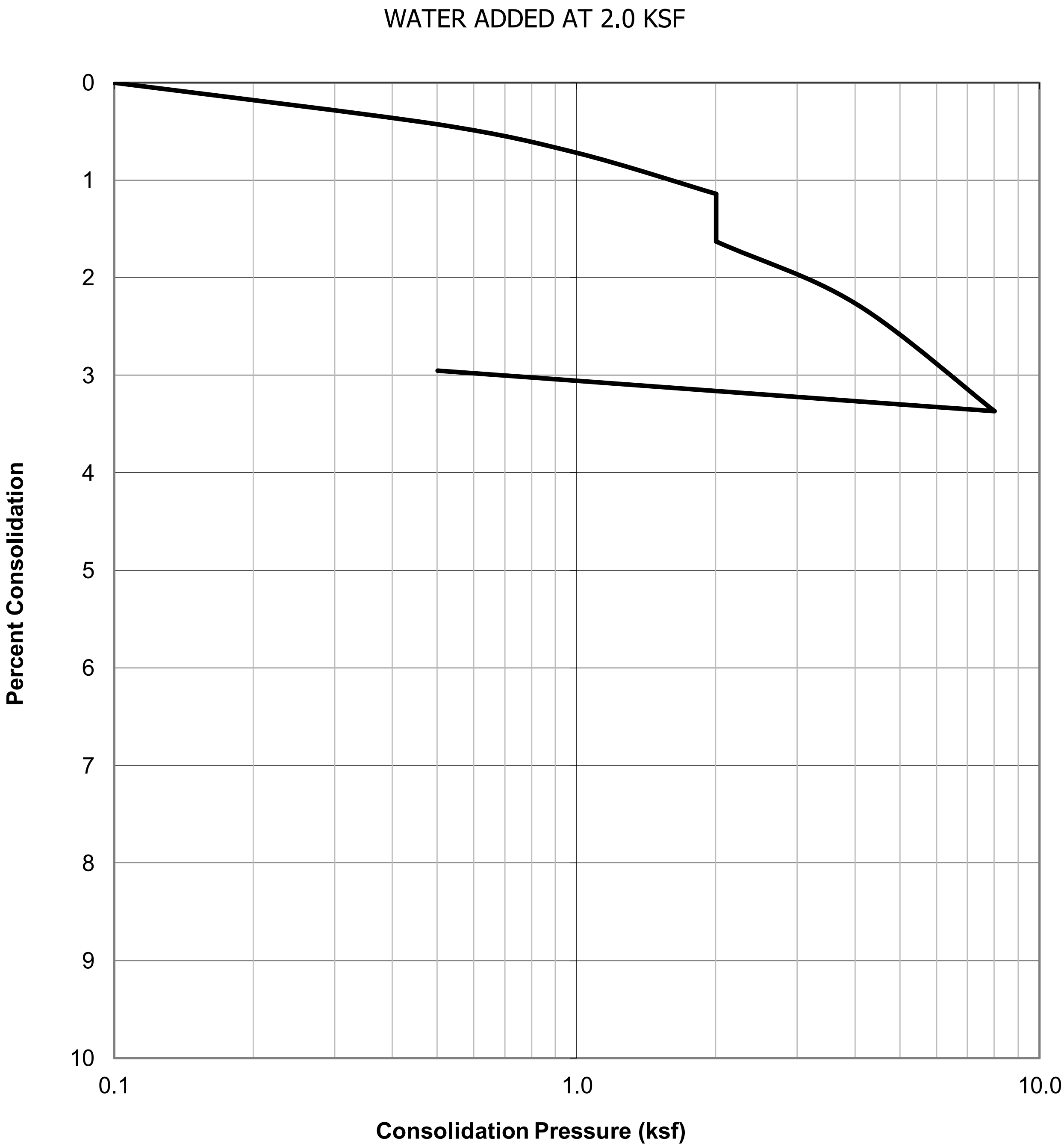
Figure B16

WATER ADDED AT 2.0 KSF




| SAMPLE ID. | SOIL TYPE | DRY DENSITY (PCF) | INITIAL MOISTURE (%) | FINAL MOISTURE (%) |
|---|---|---|---|---|
| B8@6 | Well Graded Sand (SW) | 110.8 | 1.5 | 14.2 |

| | | |
|---|---|---|
| GEOCON | **CONSOLIDATION TEST RESULTS**<br>ASTM D-2435 | Project No.: W1558-99-01 |
| | | 61650 Alta Loma Drive<br>Joshua Tree, California |
| | Checked by: JS | July 2022 Figure B17 |




| SAMPLE ID. | SOIL TYPE | DRY DENSITY (PCF) | INITIAL MOISTURE (%) | FINAL MOISTURE (%) |
|---|---|---|---|---|
| B8@12 | Well Graded Sand (SW) | 112.5 | 2.1 | 15.3 |

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**CONSOLIDATION TEST RESULTS**

ASTM D-2435

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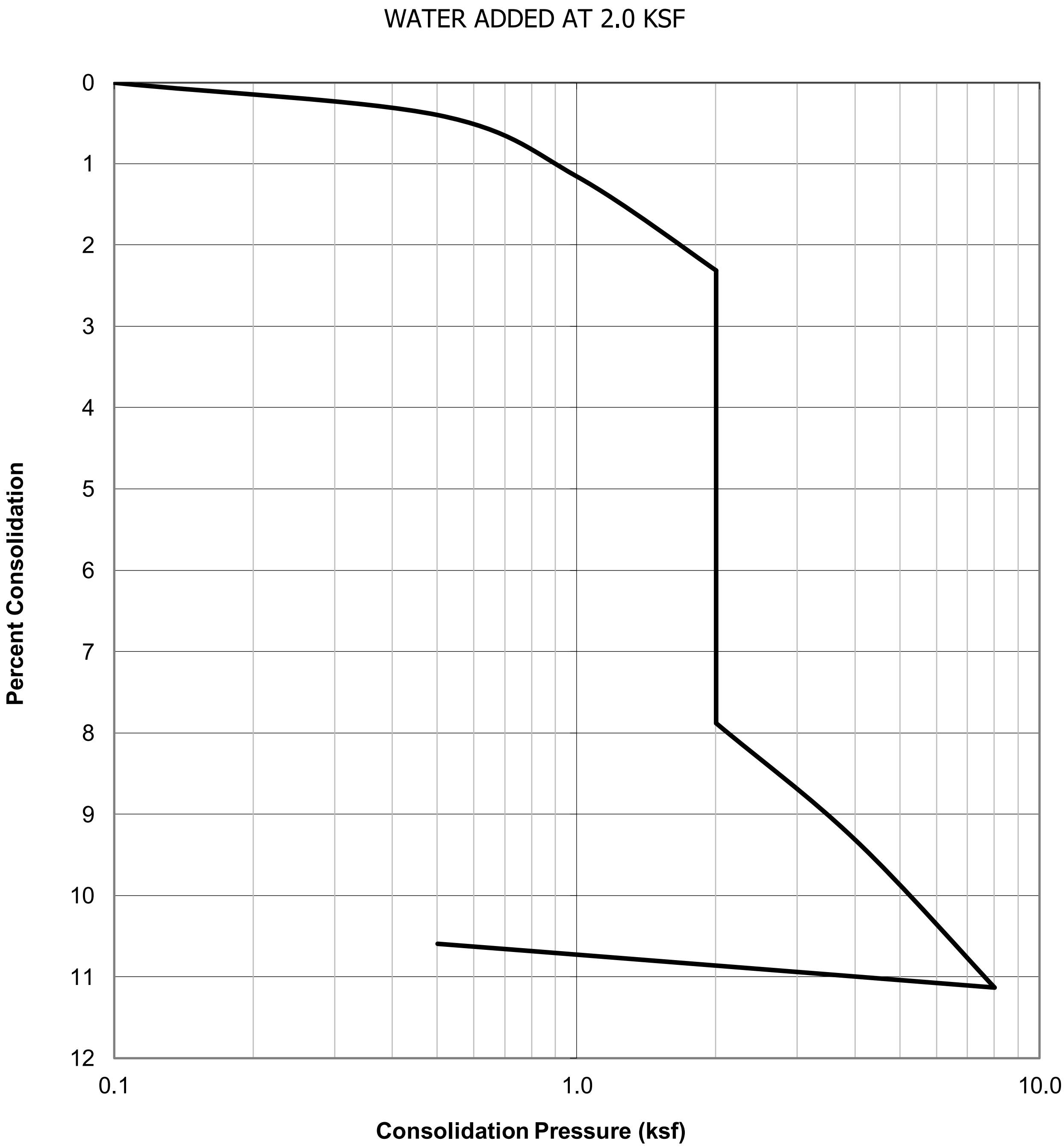
Figure B18




| SAMPLE ID. | SOIL TYPE | DRY DENSITY (PCF) | INITIAL MOISTURE (%) | FINAL MOISTURE (%) |
|---|---|---|---|---|
| B10@3 | Poorly Graded Sand (SP) | 106.8 | 3.3 | 14.0 |

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**CONSOLIDATION TEST RESULTS**

ASTM D-2435

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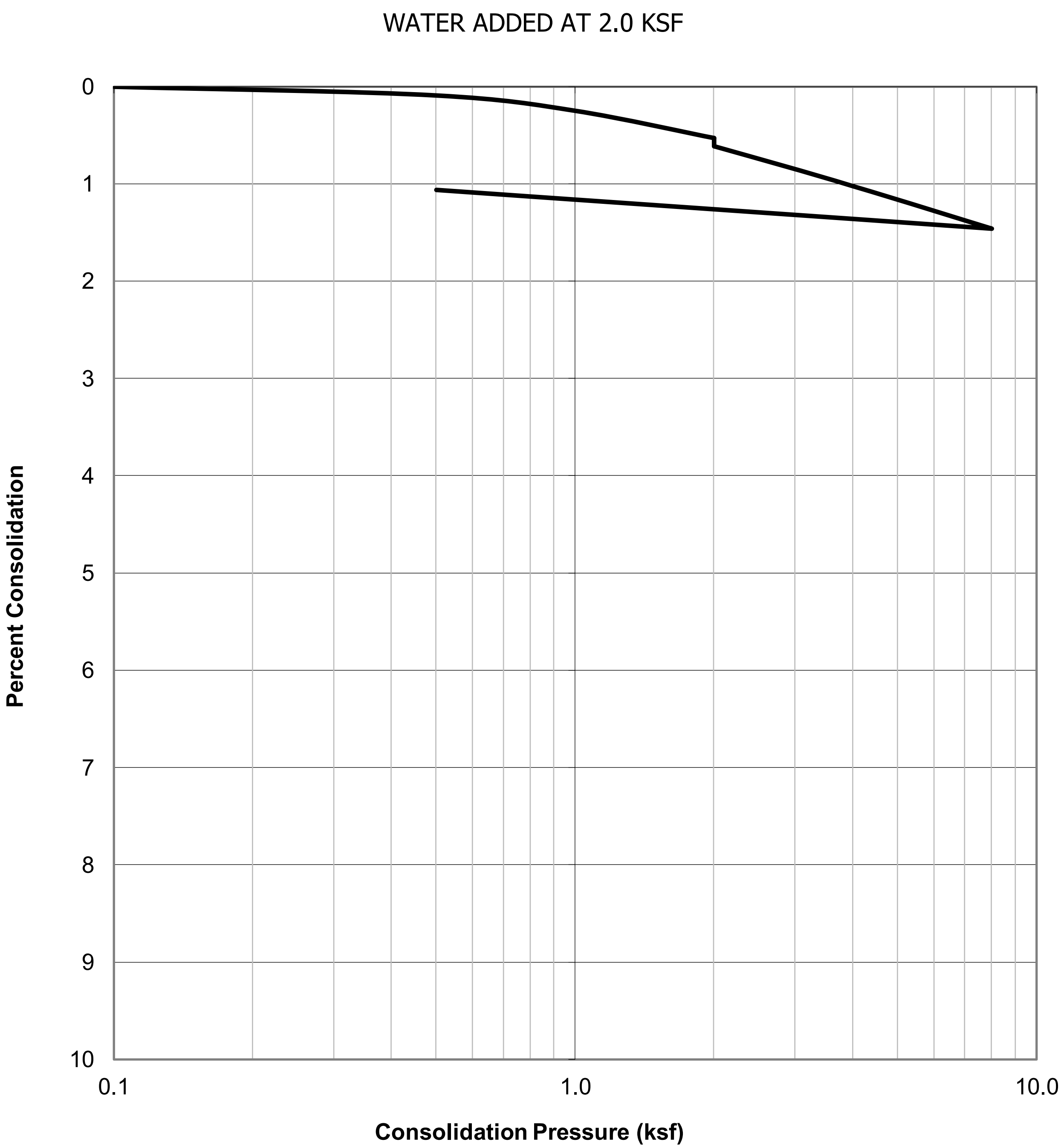
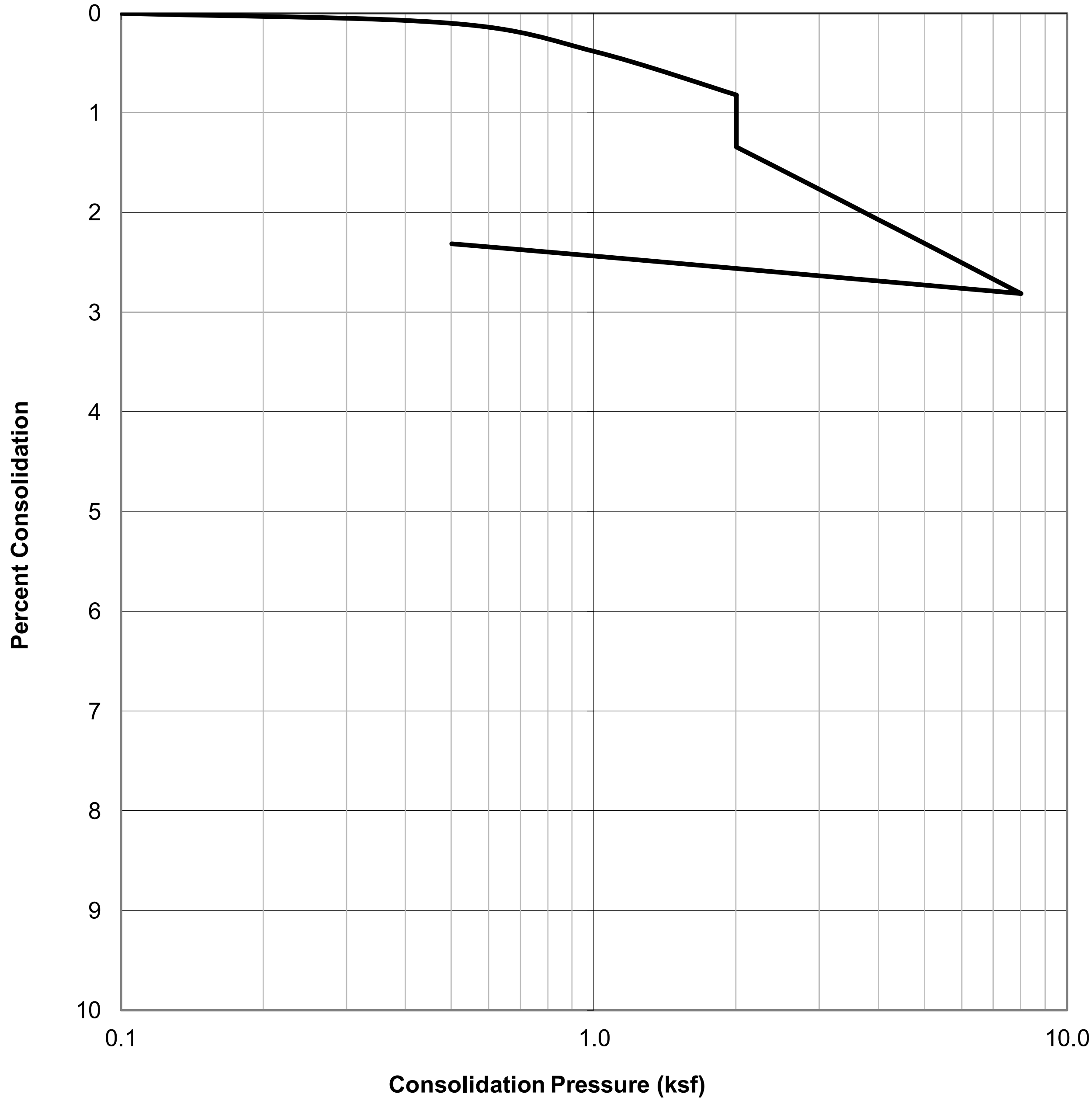
Figure B19




| SAMPLE ID. | SOIL TYPE | DRY DENSITY (PCF) | INITIAL MOISTURE (%) | FINAL MOISTURE (%) |
|---|---|---|---|---|
| B10@9 | Well Graded Sand (SW) | 110.6 | 1.0 | 17.2 |

| | | |
|---|---|---|
| GEOCON | CONSOLIDATION TEST RESULTS<br>ASTM D-2435 | Project No.: W1558-99-01 |
| | | 61650 Alta Loma Drive<br>Joshua Tree, California |
| | Checked by: JS | July 2022 Figure B20 |

WATER ADDED AT 2.0 KSF




| SAMPLE ID. | SOIL TYPE | DRY DENSITY (PCF) | INITIAL MOISTURE (%) | FINAL MOISTURE (%) |
|---|---|---|---|---|
| B10@15 | Well Graded Sand (SW) | 112.4 | 2.0 | 15.5 |

GEOCON

**CONSOLIDATION TEST RESULTS**

ASTM D-2435

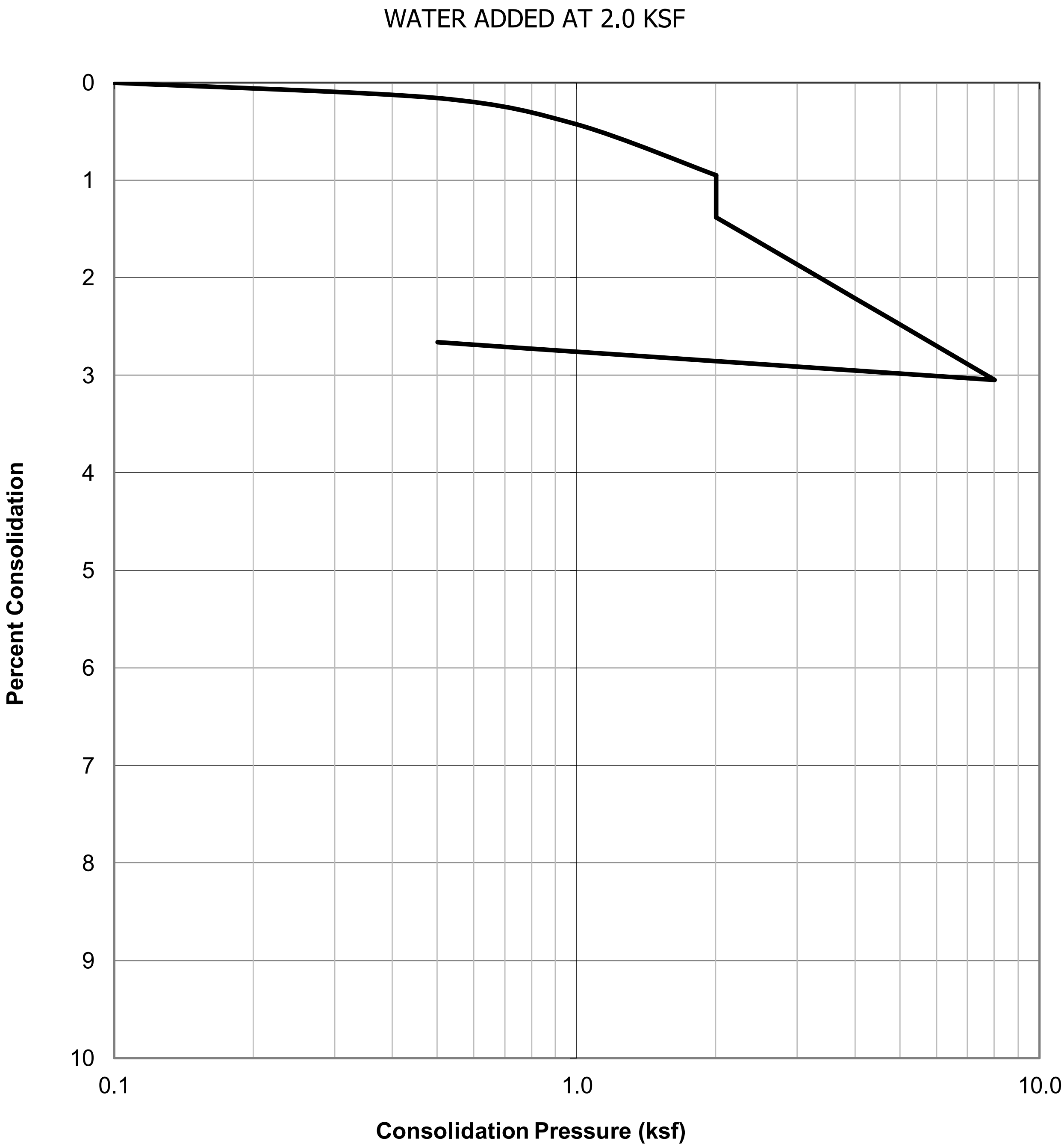
Checked by: JS

Project No.: W1558-99-01

61650 Alta Loma Drive
Joshua Tree, California

July 2022

Figure B21




| SAMPLE ID. | SOIL TYPE | DRY DENSITY (PCF) | INITIAL MOISTURE (%) | FINAL MOISTURE (%) |
|---|---|---|---|---|
| B12@3 | Poorly Graded Sand (SP) | 117.4 | 1.7 | 13.5 |

GEOCON

**CONSOLIDATION TEST RESULTS**

ASTM D-2435

Checked by: JS

Project No.: W1558-99-01

61650 Alta Loma Drive
Joshua Tree, California

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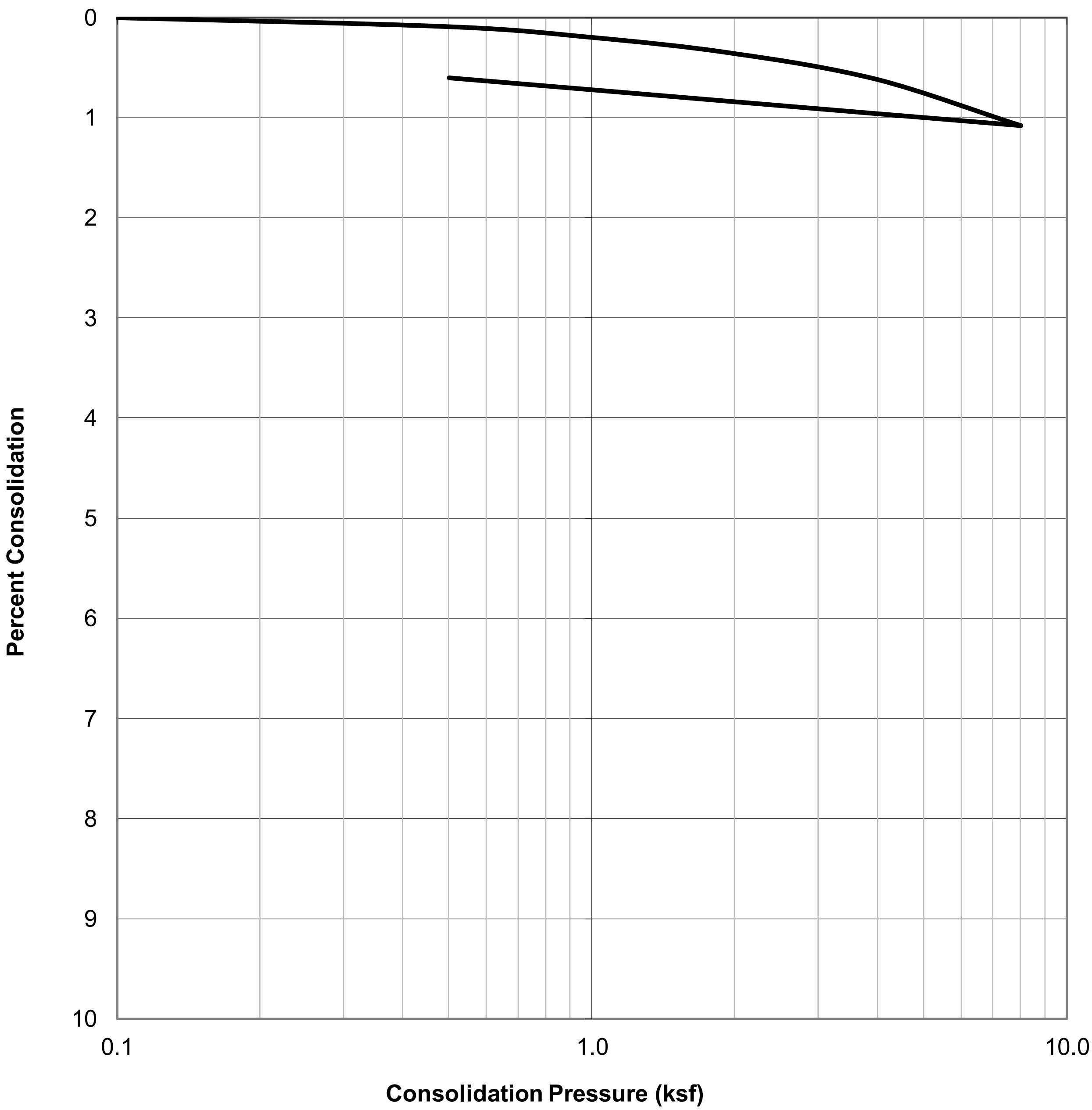
Figure B22

WATER ADDED AT 2.0 KSF




| SAMPLE ID. | SOIL TYPE | DRY DENSITY (PCF) | INITIAL MOISTURE (%) | FINAL MOISTURE (%) |
|---|---|---|---|---|
| B6@0-5' | Poorly Graded Sand (SP) | 118.0 | 7.2 | 13.0 |

GEOCON

**CONSOLIDATION TEST RESULTS**

ASTM D-2435

Checked by: JS

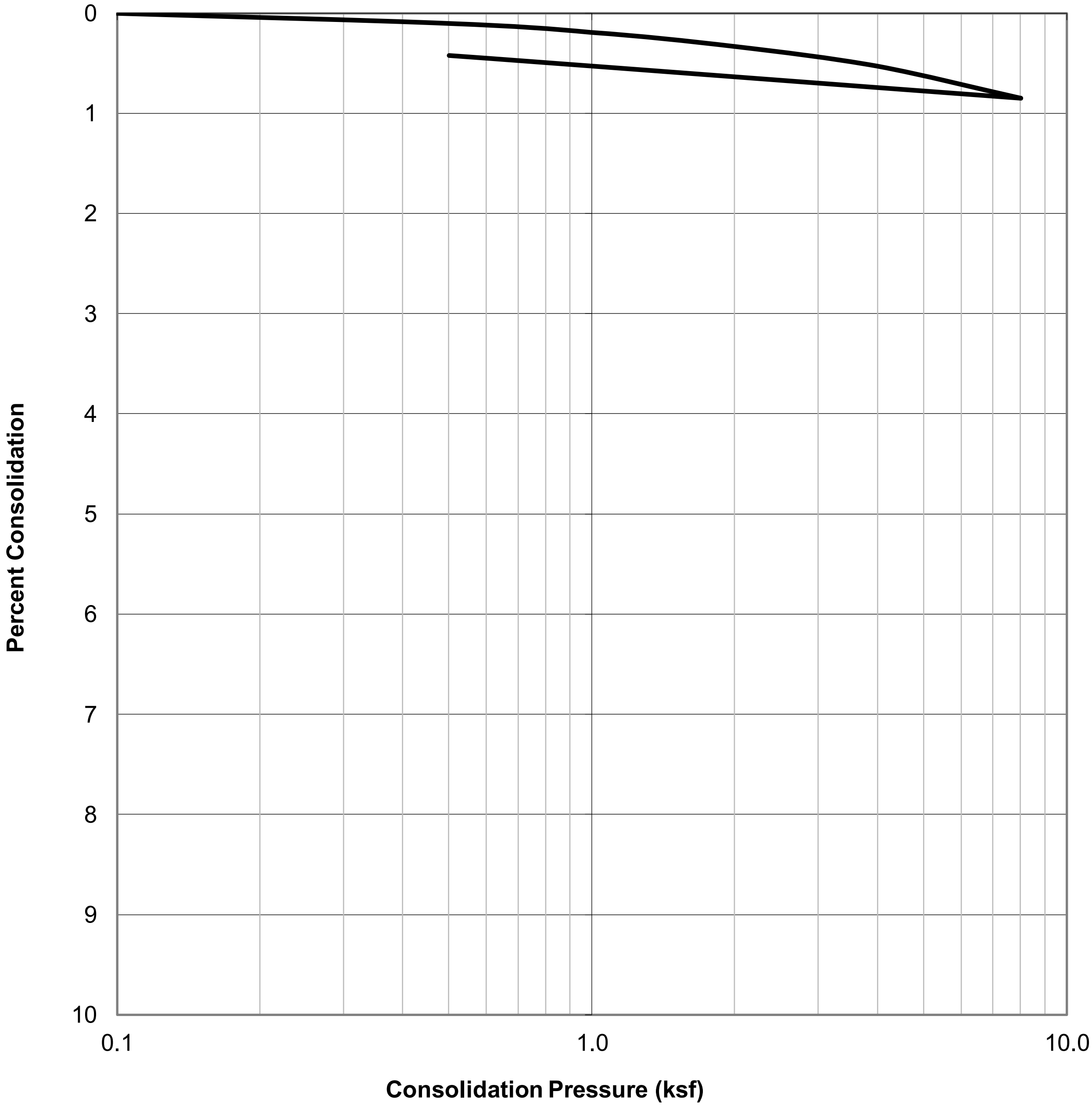
Project No.: W1558-99-01

61650 Alta Loma Drive
Joshua Tree, California

July 2022

Figure B23

WATER ADDED AT 2.0 KSF




| SAMPLE ID. | SOIL TYPE | DRY DENSITY (PCF) | INITIAL MOISTURE (%) | FINAL MOISTURE (%) |
|---|---|---|---|---|
| B8@5-10' | Well Graded Sand (SW) | 116.2 | 8.1 | 13.7 |

GEOCON

**CONSOLIDATION TEST RESULTS**

ASTM D-2435

Checked by: JS

Project No.: W1558-99-01

61650 Alta Loma Drive
Joshua Tree, California

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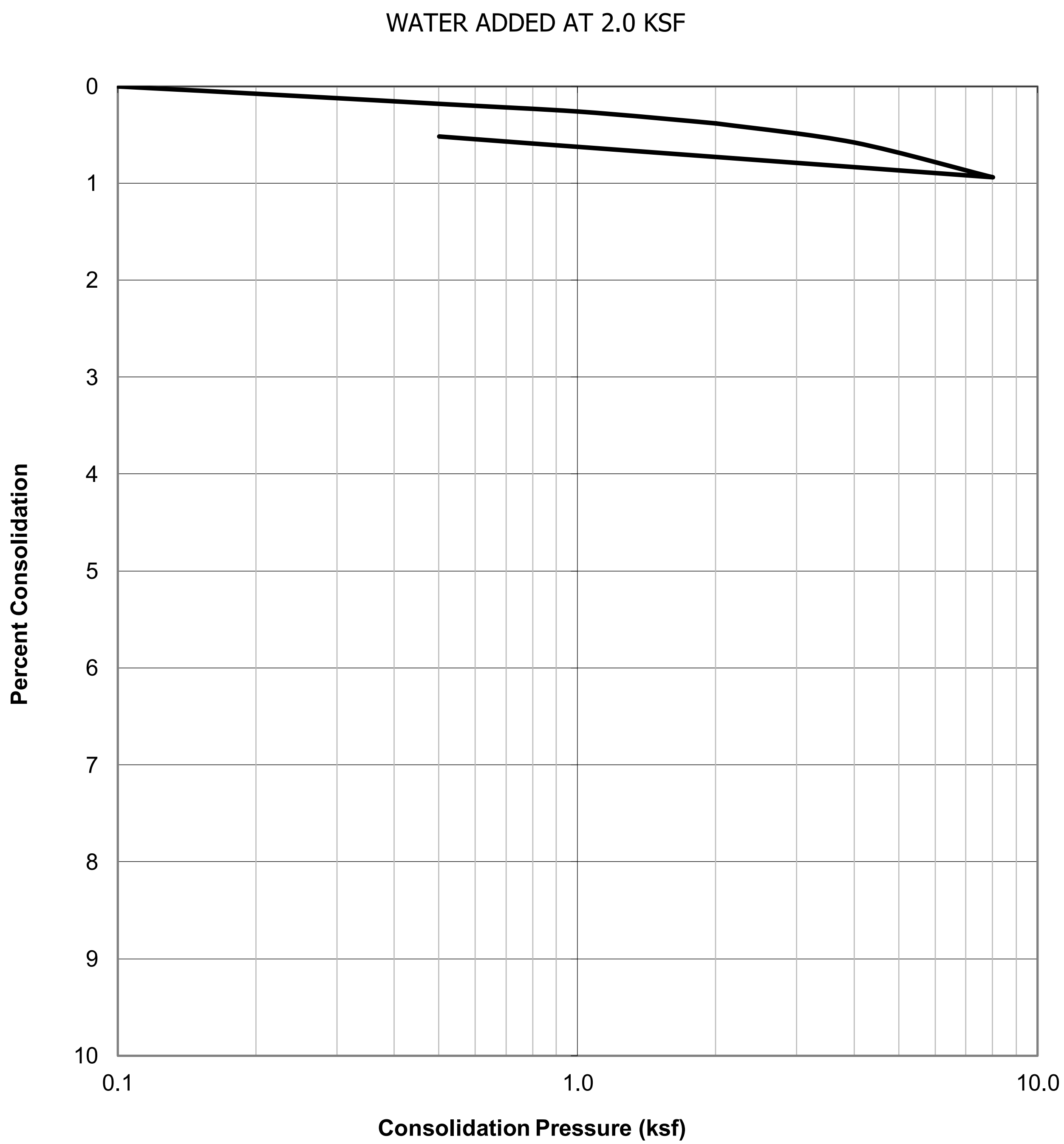



| SAMPLE ID. | SOIL TYPE | DRY DENSITY (PCF) | INITIAL MOISTURE (%) | FINAL MOISTURE (%) |
|---|---|---|---|---|
| B10@0-5' | Poorly Graded Sand (SP) | 117.3 | 8.1 | 13.6 |

GEOCON

**CONSOLIDATION TEST RESULTS**

ASTM D-2435

Checked by: JS

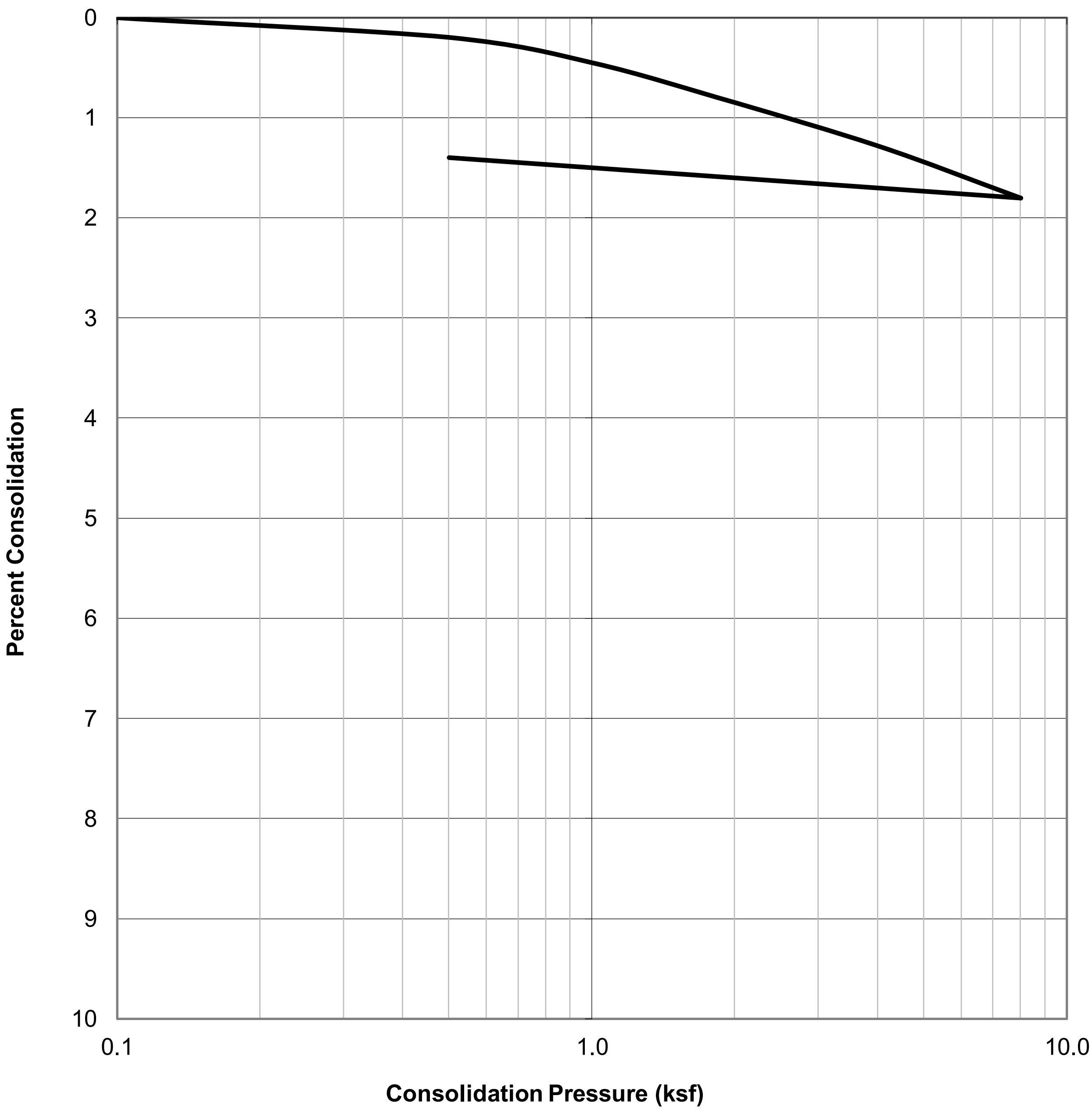
Project No.: W1558-99-01

61650 Alta Loma Drive
Joshua Tree, California

July 2022

Figure B25

WATER ADDED AT 2.0 KSF




| SAMPLE ID. | SOIL TYPE | DRY DENSITY (PCF) | INITIAL MOISTURE (%) | FINAL MOISTURE (%) |
|---|---|---|---|---|
| B12@0-5' | Poorly Graded Sand (SP) | 107.8 | 11.9 | 15.8 |

GEOCON

**CONSOLIDATION TEST RESULTS**

ASTM D-2435

Checked by: JS

Project No.: W1558-99-01

61650 Alta Loma Drive
Joshua Tree, California

July 2022

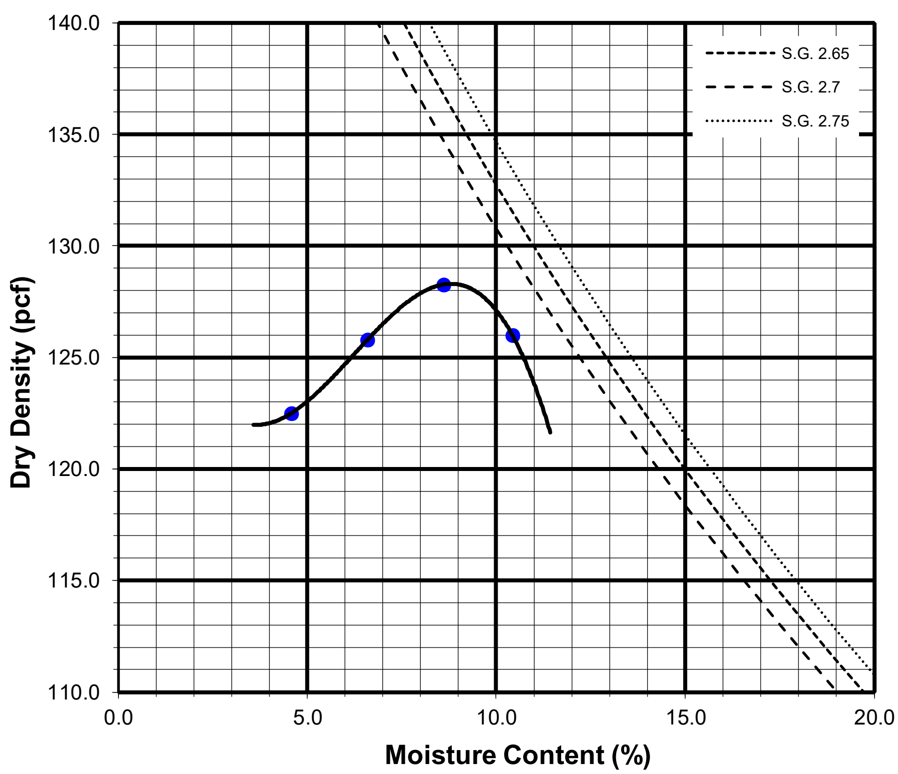
Figure B26

Sample No:

| B2@0-5' | Sand with Silt (SP-SM), brown |
|---|---|

| TEST NO. | | 1 | 2 | 3 | 4 | 5 | 6 |
|---|---|---|---|---|---|---|---|
| Wt. Compacted Soil + Mold | (g) | 6037 | 6127 | 6206 | 6204 | | |
| Weight of Mold | (g) | 4102 | 4102 | 4102 | 4102 | | |
| Net Weight of Soil | (g) | 1935 | 2025 | 2104 | 2102 | | |
| Wet Weight of Soil + Cont. | (g) | 658.8 | 690.6 | 760.2 | 685.0 | | |
| Dry Weight of Soil + Cont. | (g) | 635.7 | 656.2 | 710.1 | 634.1 | | |
| Weight of Container | (g) | 130.2 | 134.2 | 127.5 | 146.0 | | |
| Moisture Content | (%) | 4.6 | 6.6 | 8.6 | 10.4 | | |
| Wet Density | (pcf) | 128.1 | 134.1 | 139.3 | 139.1 | | |
| Dry Density | (pcf) | 122.5 | 125.8 | 128.3 | 126.0 | | |

| Maximum Dry Density (pcf) | 128.0 | Optimum Moisture Content (%) | 8.5 |
|---|---|---|---|




Preparation Method: A

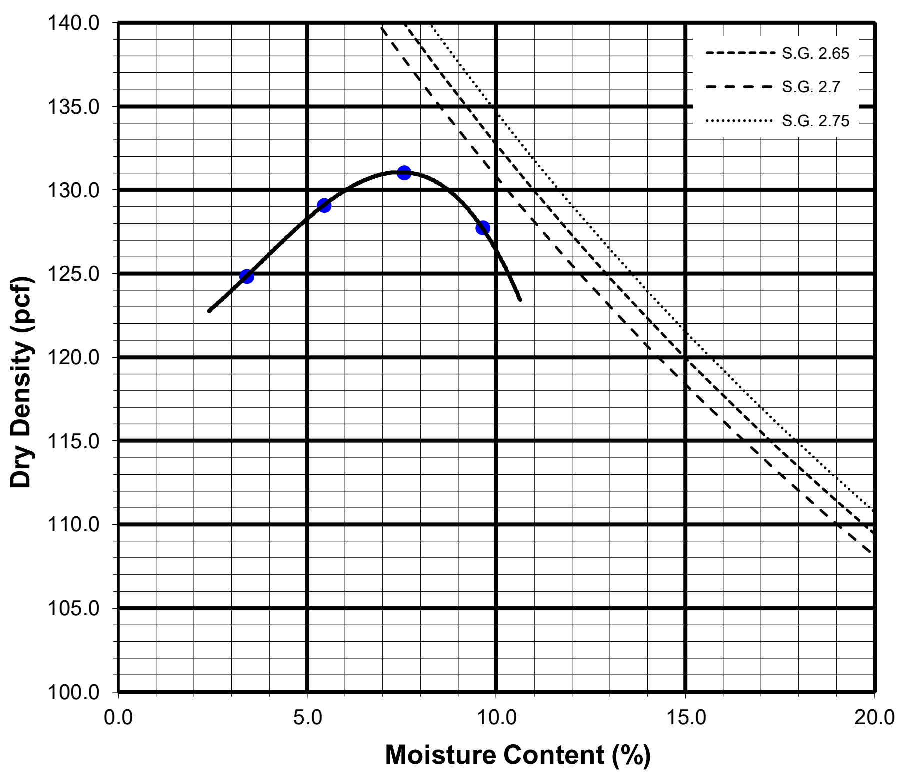
| GEOCON | **COMPACTION CHARACTERISTICS USING MODIFIED EFFORT TEST RESULTS**<br>ASTM D-1557 | Project No.: W1558-99-01 |
|---|---|---|
| | | 61650 Alta Loma Drive<br>Joshua Tree, California |
| | Checked by: JS | July 2022 — Figure B27 |

Sample No:

| B6@0-5' | Poorly Graded Sand (SP), reddish brown |
|---|---|

| TEST NO. | | 1 | 2 | 3 | 4 | 5 | 6 |
|---|---|---|---|---|---|---|---|
| Wt. Compacted Soil + Mold | (g) | 6051 | 6158 | 6231 | 6217 | | |
| Weight of Mold | (g) | 4102 | 4102 | 4102 | 4102 | | |
| Net Weight of Soil | (g) | 1950 | 2056 | 2129 | 2115 | | |
| Wet Weight of Soil + Cont. | (g) | 687.0 | 679.3 | 732.3 | 742.3 | | |
| Dry Weight of Soil + Cont. | (g) | 668.7 | 651.1 | 691.1 | 690.1 | | |
| Weight of Container | (g) | 130.1 | 133.5 | 145.4 | 148.0 | | |
| Moisture Content | (%) | 3.4 | 5.4 | 7.5 | 9.6 | | |
| Wet Density | (pcf) | 129.1 | 136.1 | 141.0 | 140.0 | | |
| Dry Density | (pcf) | 124.8 | 129.1 | 131.1 | 127.7 | | |

| Maximum Dry Density (pcf) | 131.0 | Optimum Moisture Content (%) | 7.5 |
|---|---|---|---|




Preparation Method: A

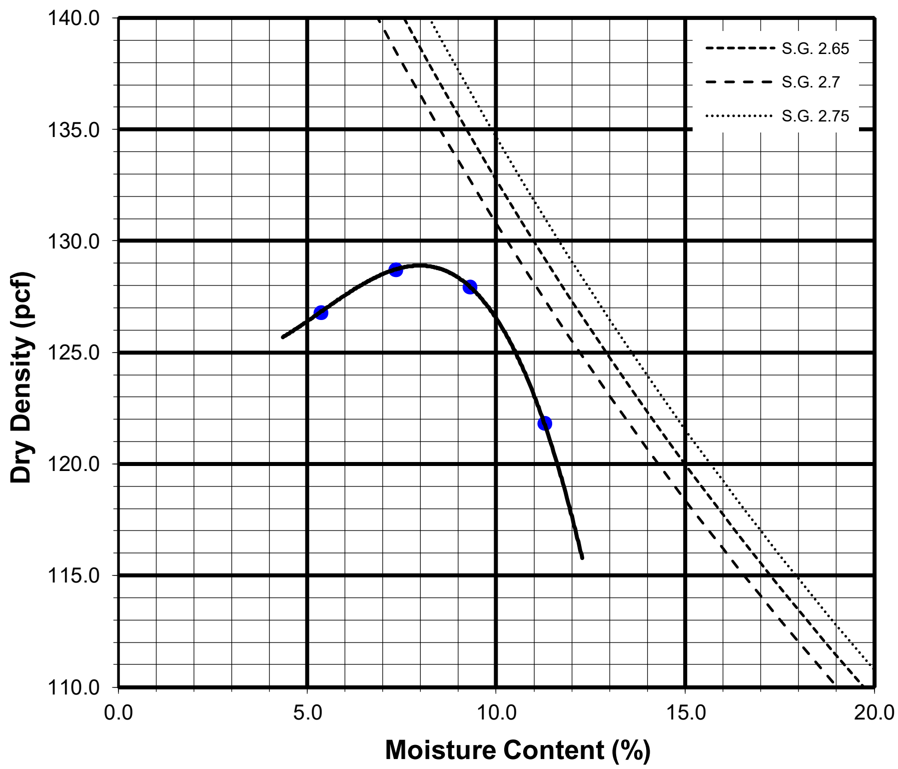
| GEOCON | **COMPACTION CHARACTERISTICS USING MODIFIED EFFORT TEST RESULTS** ASTM D-1557 | Project No.: W1558-99-01 |
|---|---|---|
| | | 61650 Alta Loma Drive Joshua Tree, California |
| | Checked by: JS | July 2022 Figure B28 |

Sample No:

| B8@5-10' | Well Graded Sand (SW), reddish brown |
|---|---|

| TEST NO. | | 1 | 2 | 3 | 4 | 5 | 6 |
|---|---|---|---|---|---|---|---|
| Wt. Compacted Soil + Mold | (g) | 6120 | 6189 | 6214 | 6149 | | |
| Weight of Mold | (g) | 4102 | 4102 | 4102 | 4102 | | |
| Net Weight of Soil | (g) | 2018 | 2087 | 2113 | 2048 | | |
| Wet Weight of Soil + Cont. | (g) | 685.3 | 767.7 | 773.5 | 655.9 | | |
| Dry Weight of Soil + Cont. | (g) | 657.0 | 725.2 | 719.3 | 604.4 | | |
| Weight of Container | (g) | 127.4 | 145.9 | 136.3 | 147.5 | | |
| Moisture Content | (%) | 5.4 | 7.3 | 9.3 | 11.3 | | |
| Wet Density | (pcf) | 133.6 | 138.2 | 139.9 | 135.6 | | |
| Dry Density | (pcf) | 126.8 | 128.7 | 128.0 | 121.8 | | |

| **Maximum Dry Density (pcf)** | **129.0** | **Optimum Moisture Content (%)** | **8.5** |
|---|---|---|---|




Preparation Method: A

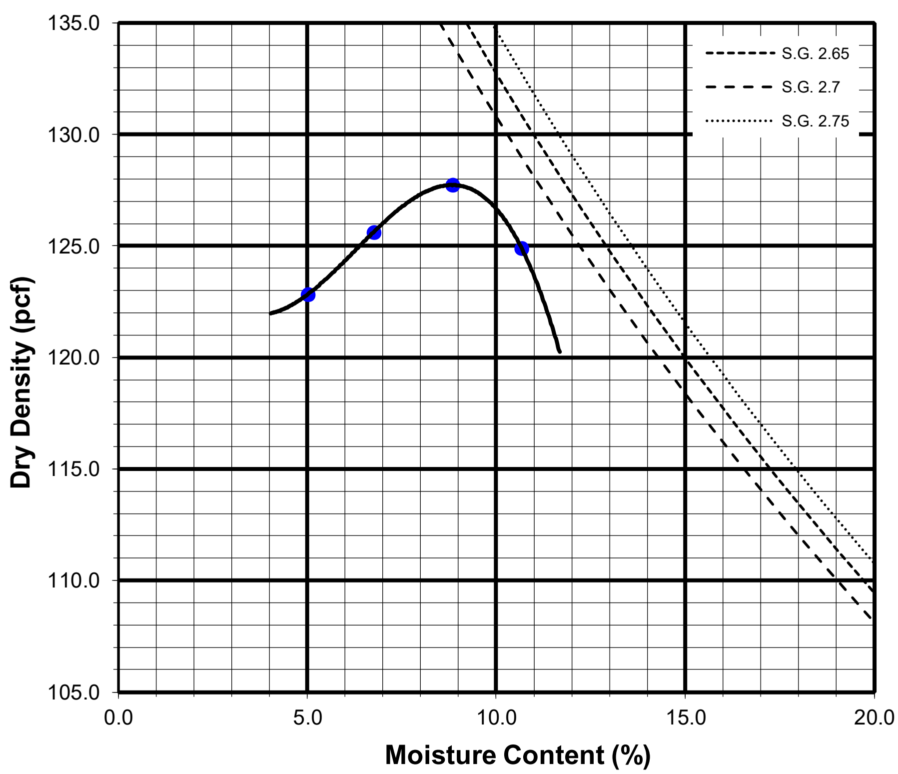
| | | |
|---|---|---|
| GEOCON | **COMPACTION CHARACTERISTICS USING MODIFIED EFFORT TEST RESULTS**<br>ASTM D-1557 | Project No.: W1558-99-01 |
| | | 61650 Alta Loma Drive<br>Joshua Tree, California |
| | Checked by: JS | July 2022 Figure B29 |

Sample No:

| | |
|---|---|
| **B9@0-5'** | Poorly Graded Sand (SP), brown |

| TEST NO. | | 1 | 2 | 3 | 4 | 5 | 6 |
|---|---|---|---|---|---|---|---|
| Wt. Compacted Soil + Mold | (g) | 6051 | 6128 | 6202 | 6190 | | |
| Weight of Mold | (g) | 4102 | 4102 | 4102 | 4102 | | |
| Net Weight of Soil | (g) | 1949 | 2026 | 2100 | 2088 | | |
| Wet Weight of Soil + Cont. | (g) | 712.2 | 644.0 | 686.3 | 687.4 | | |
| Dry Weight of Soil + Cont. | (g) | 685.2 | 611.7 | 642.6 | 635.1 | | |
| Weight of Container | (g) | 147.8 | 133.7 | 147.9 | 145.2 | | |
| Moisture Content | (%) | 5.0 | 6.8 | 8.8 | 10.7 | | |
| Wet Density | (pcf) | 129.0 | 134.1 | 139.0 | 138.2 | | |
| Dry Density | (pcf) | 122.8 | 125.6 | 127.7 | 124.9 | | |

| | | | |
|---|---|---|---|
| **Maximum Dry Density (pcf)** | **128.0** | **Optimum Moisture Content (%)** | **9.0** |




Preparation Method: A

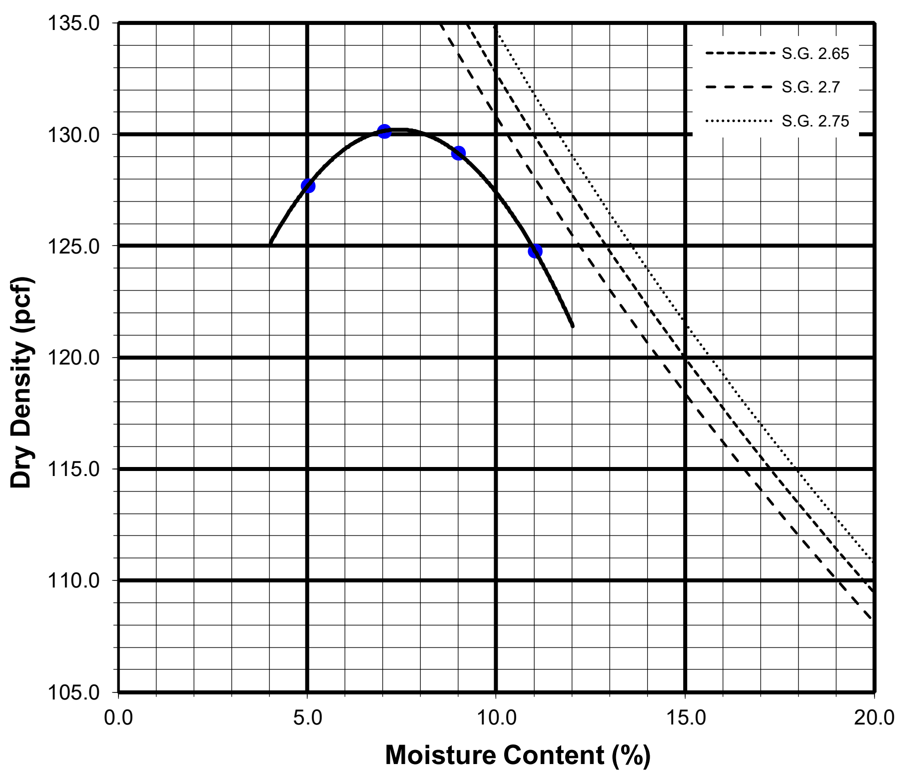
| | | |
|---|---|---|
| GEOCON | **COMPACTION CHARACTERISTICS USING MODIFIED EFFORT TEST RESULTS**<br>ASTM D-1557 | Project No.: W1558-99-01 |
| | | 61650 Alta Loma Drive<br>Joshua Tree, California |
| | Checked by: JS | July 2022 — Figure B30 |

Sample No:

| | |
|---|---|
| **B10@0-5'** | Poorly Graded Sand (SP), reddish brown |

| TEST NO. | | 1 | 2 | 3 | 4 | 5 | 6 |
|---|---|---|---|---|---|---|---|
| Wt. Compacted Soil + Mold | (g) | 6128 | 6207 | 6229 | 6195 | | |
| Weight of Mold | (g) | 4102 | 4102 | 4102 | 4102 | | |
| Net Weight of Soil | (g) | 2026 | 2104 | 2126 | 2093 | | |
| Wet Weight of Soil + Cont. | (g) | 677.9 | 749.8 | 731.1 | 703.8 | | |
| Dry Weight of Soil + Cont. | (g) | 651.5 | 709.5 | 681.2 | 648.5 | | |
| Weight of Container | (g) | 126.2 | 136.2 | 125.9 | 146.8 | | |
| Moisture Content | (%) | 5.0 | 7.0 | 9.0 | 11.0 | | |
| Wet Density | (pcf) | 134.1 | 139.3 | 140.8 | 138.5 | | |
| Dry Density | (pcf) | 127.7 | 130.2 | 129.2 | 124.8 | | |

| | | | |
|---|---|---|---|
| **Maximum Dry Density (pcf)** | **130.5** | **Optimum Moisture Content (%)** | **8.0** |




Preparation Method: A

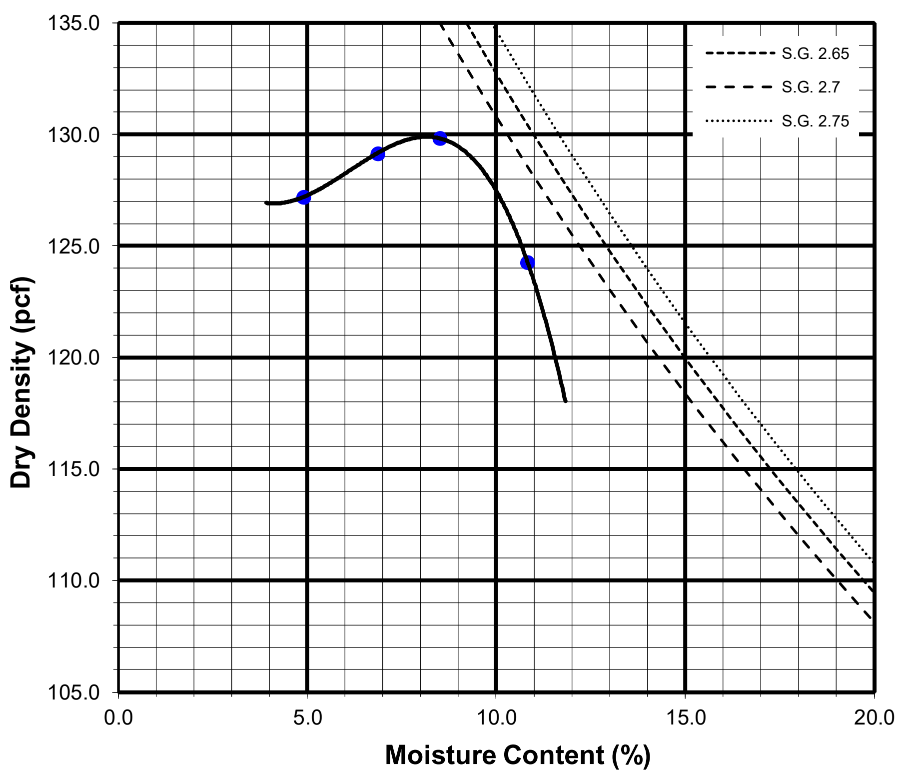
| | | |
|---|---|---|
| GEOCON | **COMPACTION CHARACTERISTICS USING MODIFIED EFFORT TEST RESULTS** | Project No.: W1558-99-01 |
| | ASTM D-1557 | 61650 Alta Loma Drive<br>Joshua Tree, California |
| | Checked by: JS | July 2022 — Figure B31 |

Sample No:

| | |
|---|---|
| **B12@0-5'** | Poorly Graded Sand (SP), light reddish brown |

| TEST NO. | | 1 | 2 | 3 | 4 | 5 | 6 |
|---|---|---|---|---|---|---|---|
| Wt. Compacted Soil + Mold | (g) | 6118 | 6187 | 6230 | 6183 | | |
| Weight of Mold | (g) | 4102 | 4102 | 4102 | 4102 | | |
| Net Weight of Soil | (g) | 2016 | 2085 | 2128 | 2080 | | |
| Wet Weight of Soil + Cont. | (g) | 648.1 | 697.3 | 643.2 | 659.5 | | |
| Dry Weight of Soil + Cont. | (g) | 624.7 | 662.1 | 603.5 | 608.3 | | |
| Weight of Container | (g) | 147.8 | 148.7 | 136.4 | 135.5 | | |
| Moisture Content | (%) | 4.9 | 6.9 | 8.5 | 10.8 | | |
| Wet Density | (pcf) | 133.4 | 138.0 | 140.9 | 137.7 | | |
| Dry Density | (pcf) | 127.2 | 129.2 | 129.8 | 124.3 | | |

| | | | |
|---|---|---|---|
| **Maximum Dry Density (pcf)** | **130.0** | **Optimum Moisture Content (%)** | **8.0** |




Preparation Method: A

**COMPACTION CHARACTERISTICS USING MODIFIED EFFORT TEST RESULTS**

ASTM D-1557

GEOCON

Checked by: JS

Project No.: W1558-99-01

61650 Alta Loma Drive
Joshua Tree, California

July 2022

Figure B32

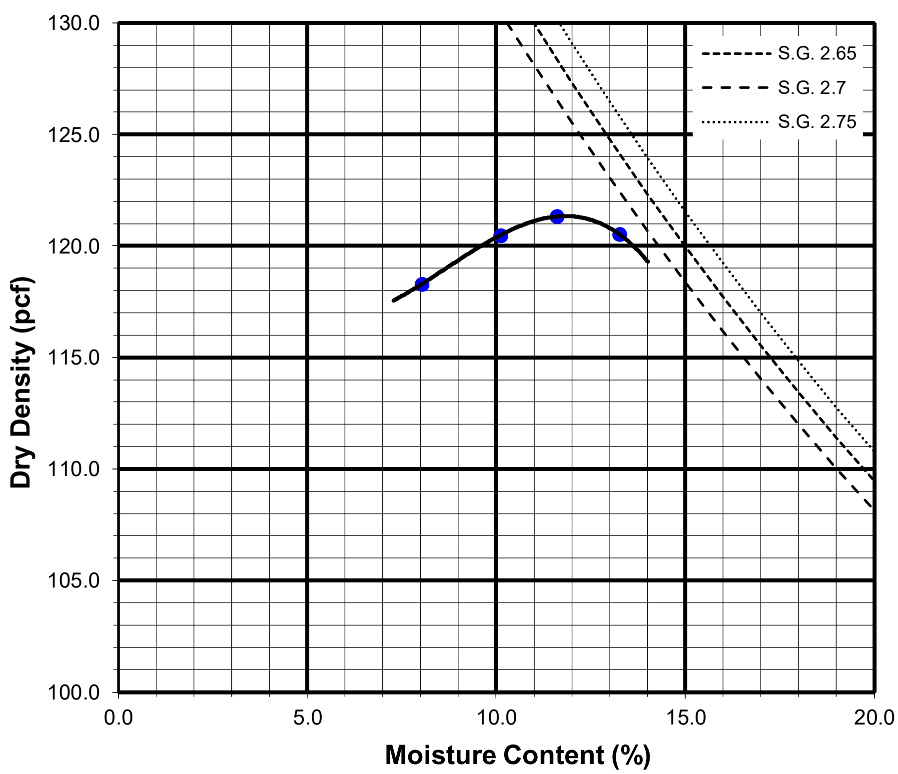
Sample No:

| | |
|---|---|
| **B12@5-10'** | Well Graded Sand (SW), trace fine gravel, brown |

| TEST NO. | | 1 | 2 | 3 | 4 | 5 | 6 |
|---|---|---|---|---|---|---|---|
| Wt. Compacted Soil + Mold | (g) | 6026 | 6099 | 6141 | 6157 | | |
| Weight of Mold | (g) | 4102 | 4102 | 4102 | 4102 | | |
| Net Weight of Soil | (g) | 1925 | 1998 | 2039 | 2056 | | |
| Wet Weight of Soil + Cont. | (g) | 625.2 | 736.4 | 739.0 | 725.3 | | |
| Dry Weight of Soil + Cont. | (g) | 588.8 | 682.4 | 677.6 | 655.1 | | |
| Weight of Container | (g) | 135.6 | 148.6 | 147.9 | 125.3 | | |
| Moisture Content | (%) | 8.0 | 10.1 | 11.6 | 13.3 | | |
| Wet Density | (pcf) | 127.8 | 132.7 | 135.4 | 136.5 | | |
| Dry Density | (pcf) | 118.3 | 120.5 | 121.3 | 120.5 | | |

| | |
|---|---|
| **Maximum Dry Density (pcf)** | **121.0** |
| **Bulk Specific Gravity (dry)** | **2.65** |
| **Corrected Maximum Dry Density (pcf)** | **123.5** |

| | |
|---|---|
| **Optimum Moisture Content (%)** | **11.5** |
| **Oversized Fraction (%)** | **7.0** |
| **Corrected Moisture Content (%)** | **10.5** |




Preparation Method: A

| | | |
|---|---|---|
| GEOCON | **COMPACTION CHARACTERISTICS USING MODIFIED EFFORT TEST RESULTS**<br>ASTM D-1557 | Project No.: W1558-99-01 |
| | | 61650 Alta Loma Drive<br>Joshua Tree, California |
| | Checked by: JS | July 2022 Figure B33 |

# B6@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) | 785.5 | 801.5 |
| Wt. of Mold | (gm) | 370.2 | 370.2 |
| Specific Gravity | (Assumed) | 2.7 | 2.7 |
| Wet Wt. of Soil + Cont. | (gm) | 515.5 | 801.5 |
| Dry Wt. of Soil + Cont. | (gm) | 493.3 | 384.5 |
| Wt. of Container | (gm) | 215.5 | 370.2 |
| Moisture Content | (%) | 8.0 | 12.2 |
| Wet Density | (pcf) | 125.3 | 129.9 |
| Dry Density | (pcf) | 116.0 | 115.8 |
| Void Ratio | | 0.5 | 0.5 |
| Total Porosity | | 0.3 | 0.3 |
| Pore Volume | (cc) | 64.6 | 64.4 |
| Degree of Saturation | (%) [$S_{meas}$] | 48.0 | 72.5 |

| Date | Time | Pressure (psi) | Elapsed Time (min) | Dial Readings (in.) |
|---|---|---|---|---|
| 6/7/2022 | 10:00 | 1.0 | 0 | 0.3167 |
| 6/7/2022 | 10:10 | 1.0 | 10 | 0.3165 |
| Add Distilled Water to the Specimen | | | | |
| 6/8/2022 | 10:00 | 1.0 | 1430 | 0.3159 |
| 6/8/2022 | 11:00 | 1.0 | 1490 | 0.3159 |

| | |
|---|---|
| Expansion Index (EI meas) = | -0.6 |
| Expansion Index ( Report ) = | **0** |

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

* Reference: 2019 California Building Code, Section 1803.5.3
** Reference: 1997 Uniform Building Code, Table 18-I-B.

GEOCON

**EXPANSION INDEX TEST RESULTS**
ASTM D-4829

Checked by: JS

Project No.: W1558-99-01

61650 Alta Loma Drive
Joshua Tree, California

July 2022

Figure B34

# B8@5-10'

| MOLDED SPECIMEN | | BEFORE TEST | AFTER TEST |
|---|---|---|---|
| Specimen Diameter | (in.) | 4.0 | 4.0 |
| Specimen Height | (in.) | 1.0 | 1.0 |
| Wt. Comp. Soil + Mold | (gm) | 785.2 | 802.1 |
| Wt. of Mold | (gm) | 367.5 | 367.5 |
| Specific Gravity | (Assumed) | 2.7 | 2.7 |
| Wet Wt. of Soil + Cont. | (gm) | 515.5 | 802.1 |
| Dry Wt. of Soil + Cont. | (gm) | 493.0 | 386.4 |
| Wt. of Container | (gm) | 215.5 | 367.5 |
| Moisture Content | (%) | 8.1 | 12.5 |
| Wet Density | (pcf) | 126.0 | 130.9 |
| Dry Density | (pcf) | 116.6 | 116.4 |
| Void Ratio | | 0.4 | 0.4 |
| Total Porosity | | 0.3 | 0.3 |
| Pore Volume | (cc) | 63.9 | 63.8 |
| Degree of Saturation | (%) [$S_{meas}$] | 49.4 | 75.5 |

| Date | Time | Pressure (psi) | Elapsed Time (min) | Dial Readings (in.) |
|---|---|---|---|---|
| 6/7/2022 | 10:00 | 1.0 | 0 | 0.3498 |
| 6/7/2022 | 10:10 | 1.0 | 10 | 0.3498 |
| Add Distilled Water to the Specimen | | | | |
| 6/8/2022 | 10:00 | 1.0 | 1430 | 0.3496 |
| 6/8/2022 | 11:00 | 1.0 | 1490 | 0.3496 |

| | |
|---|---|
| Expansion Index (EI meas) = | -0.2 |
| Expansion Index ( Report ) = | **0** |

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

* Reference: 2019 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: JS

Project No.: W1558-99-01

61650 Alta Loma Drive
Joshua Tree, California

July 2022

# B10@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) | 786.5 | 800.9 |
| Wt. of Mold | (gm) | 367.8 | 367.8 |
| Specific Gravity | (Assumed) | 2.7 | 2.7 |
| Wet Wt. of Soil + Cont. | (gm) | 515.5 | 800.9 |
| Dry Wt. of Soil + Cont. | (gm) | 493.8 | 388.4 |
| Wt. of Container | (gm) | 215.5 | 367.8 |
| Moisture Content | (%) | 7.8 | 11.5 |
| Wet Density | (pcf) | 126.3 | 130.5 |
| Dry Density | (pcf) | 117.2 | 117.0 |
| Void Ratio | | 0.4 | 0.4 |
| Total Porosity | | 0.3 | 0.3 |
| Pore Volume | (cc) | 63.1 | 63.1 |
| Degree of Saturation | (%) [$S_{meas}$] | 48.4 | 70.9 |

| Date | Time | Pressure (psi) | Elapsed Time (min) | Dial Readings (in.) |
|---|---|---|---|---|
| 6/7/2022 | 10:00 | 1.0 | 0 | 0.2774 |
| 6/7/2022 | 10:10 | 1.0 | 10 | 0.2773 |
| Add Distilled Water to the Specimen | | | | |
| 6/8/2022 | 10:00 | 1.0 | 1430 | 0.2769 |
| 6/8/2022 | 11:00 | 1.0 | 1490 | 0.2769 |

| | |
|---|---|
| Expansion Index (EI meas) = | -0.4 |
| Expansion Index ( Report ) = | **0** |

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

* Reference: 2019 California Building Code, Section 1803.5.3
** Reference: 1997 Uniform Building Code, Table 18-I-B.

GEOCON

**EXPANSION INDEX TEST RESULTS**

ASTM D-4829

Checked by: JS

Project No.: W1558-99-01

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Joshua Tree, California

July 2022

Figure B36

# B12@5-10'

| MOLDED SPECIMEN | | BEFORE TEST | AFTER TEST |
|---|---|---|---|
| Specimen Diameter | (in.) | 4.0 | 4.0 |
| Specimen Height | (in.) | 1.0 | 1.0 |
| Wt. Comp. Soil + Mold | (gm) | 787.6 | 791.6 |
| Wt. of Mold | (gm) | 370.3 | 370.3 |
| Specific Gravity | (Assumed) | 2.7 | 2.7 |
| Wet Wt. of Soil + Cont. | (gm) | 515.5 | 791.6 |
| Dry Wt. of Soil + Cont. | (gm) | 493.0 | 386.0 |
| Wt. of Container | (gm) | 215.5 | 370.3 |
| Moisture Content | (%) | 8.1 | 9.1 |
| Wet Density | (pcf) | 125.9 | 126.9 |
| Dry Density | (pcf) | 116.4 | 116.3 |
| Void Ratio | | 0.4 | 0.4 |
| Total Porosity | | 0.3 | 0.3 |
| Pore Volume | (cc) | 64.0 | 63.2 |
| Degree of Saturation | (%) [$S_{meas}$] | 49.2 | 55.8 |

| Date | Time | Pressure (psi) | Elapsed Time (min) | Dial Readings (in.) |
|---|---|---|---|---|
| 6/7/2022 | 10:00 | 1.0 | 0 | 0.3643 |
| 6/7/2022 | 10:10 | 1.0 | 10 | 0.3644 |
| Add Distilled Water to the Specimen | | | | |
| 6/8/2022 | 10:00 | 1.0 | 1430 | 0.3603 |
| 6/8/2022 | 11:00 | 1.0 | 1490 | 0.3603 |

| | |
|---|---|
| Expansion Index (EI meas) = | -4.1 |
| Expansion Index ( Report ) = | **0** |

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

* Reference: 2019 California Building Code, Section 1803.5.3
** Reference: 1997 Uniform Building Code, Table 18-I-B.

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**EXPANSION INDEX TEST RESULTS**
ASTM D-4829

Checked by: JS

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Figure B37

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

| Sample No. | pH | Resistivity (ohm centimeters) |
|---|---|---|
| B6@0-5' | 8.4 | 4700 (Moderately Corrosive) |
| B8@5-10' | 8.4 | 6500 (Moderately Corrosive) |
| B10@0-5' | 7.7 | 5100 (Moderately Corrosive) |

SUMMARY OF LABORATORY CHLORIDE CONTENT TEST RESULTS
AASHTO T291 ASTM C1218

| Sample No. | Chloride Ion Content (%) |
|---|---|
| B6@0-5' | 0.008 |
| B8@5-10' | 0.005 |
| B10@0-5' | 0.004 |

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

| Sample No. | Water Soluble Sulfate (% $SO_4$) | Sulfate Exposure |
|---|---|---|
| B6@0-5' | 0.000 | S0 |
| B8@5-10' | 0.000 | S0 |
| B10@0-5' | 0.000 | S0 |

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**CORROSIVITY TEST RESULTS**

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Figure B38

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

| Sample No. | pH | Resistivity (ohm centimeters) |
|---|---|---|
| B12@5-10' | 8.6 | 16000 (Mildly Corrosive) |
| | | |
| | | |

SUMMARY OF LABORATORY CHLORIDE CONTENT TEST RESULTS
AASHTO T291 ASTM C1218

| Sample No. | Chloride Ion Content (%) |
|---|---|
| B12@5-10' | 0.008 |
| | |
| | |

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

| Sample No. | Water Soluble Sulfate (% $SO_4$) | Sulfate Exposure |
|---|---|---|
| B12@5-10' | 0.000 | S0 |
| | | |
| | | |

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**CORROSIVITY TEST RESULTS**

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Figure B39