Appendix E
Park Lane Homes Preliminary Geotechnical Investigation
(Available on the city website)

GEOCON

PRELIMINARY GEOTECHNICAL INVESTIGATION

PROPOSED MULTI-FAMILY RESIDENTIAL DEVELOPMENT
14320 PALM DRIVE
DESERT HOT SPRINGS, CALIFORNIA

SEPTEMBER 12, 2024

PROJECT NO. T3082-22-01

PREPARED FOR:

ABODE COMMUNITIES

LOS ANGELES, CALIFORNIA

GEOCON
WEST, INC.
GEOTECHNICAL ■ ENVIRONMENTAL ■ MATERIALS

Project No. T3082-22-01
September 12, 2024

Abode Communities
1149 S. Hill Street, Suite 700
Los Angeles, CA 90015

Attention: Sergio Rosas, Senior Project Manager

Subject: PRELIMINARY GEOTECHNICAL INVESTIGATION
PROPOSED MULTI-FAMILY RESIDENTIAL DEVELOPMENT
14320 PALM DRIVE
DESERT HOT SPRINGS, CALIFORNIA

Mr. Rosas:

In accordance with your authorization of our Proposal CV-24-1225-P-GT, dated July 8, 2024, Geocon West, Inc. (Geocon) performed a preliminary geotechnical investigation for the proposed multi-family residential development, planned east of the Desert Hot Springs Library, at 14320 Palm Drive in the City of Desert Hot Springs, California. The accompanying report presents our findings, conclusions, and preliminary recommendations pertaining to the geotechnical aspects of the proposed development. Based on the results of this study, it is our opinion the site is considered suitable for the proposed development provided the recommendations of this report are followed.

The primary intent of this study was to address potential geologic hazards and geotechnical conditions that could impact the project. An updated geotechnical study will be required when more finalized project plans become available for review, to provide updated geotechnical recommendations for design and construction.

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

Very truly yours,

GEOCON WEST, INC.

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PRELIMINARY GEOTECHNICAL INVESTIGATION

1. PURPOSE AND SCOPE

This report presents the results of our preliminary geotechnical investigation for the proposed multi-family residential development, planned within a square parcel located immediately east of the Desert Hot Springs Library, at 14320 Palm Drive in the City of Desert Hot Springs, California, as depicted on the *Vicinity Map*, Figure 1.

The purpose of this investigation was to perform a subsurface exploration and percolation testing, laboratory testing, and provide geotechnical analyses and, based on the conditions encountered, provide preliminary recommendations pertaining to the geotechnical aspects of developing the property. An updated geotechnical study will be required when more finalized plans become available, to provide updated geotechnical recommendations for design and construction.

The scope of this investigation included reviewing aerial photographs and published geologic information; conducting a subsurface exploration and performing sample collection, percolation testing, laboratory testing on the samples collected; engineering analyses; and preparing this preliminary geotechnical report. A summary of the information and documentation reviewed for this study is presented in the *List of References*.

Our field investigation was conducted on August 9 and 12, 2024, and included:

- Drilling of nine (9) exploratory borings (Borings B-1 through B-9) to depths ranging between approximately 16½ feet and 50½ feet, to observe the subsurface geological conditions at the site, collect relatively undisturbed in-situ and disturbed bulk samples for laboratory testing, and evaluate the depth to static groundwater, if encountered.
- Backfilling and performing percolation testing in one (1) geotechnical boring (Boring B-3), at a depth of approximately 10 feet, to provide a preliminary evaluation of the subsurface infiltration rate in areas where stormwater infiltration systems are expected. The percolation test is identified as Test P-1. A bentonite plug was installed at 10 feet of depth, after backfilling and prior to performing percolation testing. Additional percolation testing should be performed when the exact location and depth of the proposed stormwater infiltration system is known.

Appendix A presents a discussion of the field investigation, and detailed logs of the borings and percolation test data. The approximate locations of the exploratory borings and the percolation test are presented on Figure 2, *Geologic Map and Site Plan*. We performed laboratory testing on select soil samples obtained from our field investigation to evaluate physical and chemical properties for engineering analysis. Appendix B presents the results of our laboratory testing.

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 site is an approximately 8-acre square parcel that is vacant and undeveloped. Based on Google Earth aerial imagery, the site appears to have been natural since at least 1996. The site consists of a loose sand surface with moderate to sparce growth of shrubs. Access is via a gate along Park Lane. The site is bounded on the north by a retail shopping center, the west by Desert Hot Springs Library and Riverside County Behavioral Health and Nutrition Services Center, on the south by Park Lane, and on the east by the play fields of Desert Springs Middle School.

The site is relatively flat to gently sloping down toward the southeast. Existing elevations range from approximately 917 feet above mean sea level (MSL) in the northwest portion of the site, to approximately 906 feet MSL in the southeast portion of the site. Drainage appears to be by sheet flow toward the southeast. The site coordinates are at latitude 33.9441 degrees and longitude -116.4989 degrees.

The *Site Plan*, prepared by Abode Communities Architecture Studio and dated October 17, 2023, indicates the proposed development will include eight multi-family residential buildings up to three stories high, a community center, and an early childcare center. Additionally, associated utility, parking, drive aisle, flatwork, and landscape improvements are proposed for the site. The stormwater mitigation plan has not been developed for the site at this time; however, we expect infiltration systems will be constructed in the southeastern corner of the site where the lowest elevation exists.

We expect that rough grading will result in cuts and fills of less than 5 feet (exclusive of remedial grading). Graded slopes are not proposed on the site at this time.

Structural plans and loading information were not provided to us at this time; however, we expect the proposed structures will be one- to three-story buildings constructed of wood or light gauge steel framing, with shallow concrete foundations and concrete slab-on-grade floors. For preliminary evaluation purposes, we assume that column loads for the proposed structures will be up to 300 kips, and wall loads will be up to 3 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. If project details differ significantly from those described, Geocon should be contacted for review and possible revision to this report.

3. GEOLOGIC SETTING

The site is located within the northern end of the Coachella Valley approximately 35 miles northwest of the Salton Sea. The Coachella Valley is a pull apart geologic basin formed by extensional faulting and step-overs along the San Andreas fault. A thickness of more than 3,000 feet of sediment has accumulated within the Coachella Valley in the last 0.5 million years since the extension began. The site is located east of the San Jacinto Mountains and is subject to alluvial deposits carried from the nearby foothills to the west. The sediments consist primarily of sands and gravels with varying amounts of silt.

The Coachella Valley is part of the Colorado Desert geomorphic province, which is bounded on the west by the Santa Rosa Mountains and the north by the Transverse Ranges. The Colorado Desert extends beyond California to the east and south. The San Andreas fault is geologically mapped approximately ½ mile northeast of the site. Geothermal resources associated with the pull-apart basin are present near the southern area of the Salton Sea.

Regional subsidence has occurred in recent history within the Coachella Valley. Initial subsidence occurred between the 1920's and 1940's when groundwater was over pumped and ground water levels declined on the order of 50 feet. The introduction of Colorado River water in 1949 reduced groundwater pumping and the related subsidence temporarily stopped. In the 1970's overdraft of the groundwater occurred resulting in groundwater level declines of 50 to 100 feet. Subsidence resumed. In 1996 the United States Geologic Survey (USGS) in cooperation with Coachella Valley Water District (CVWD) implemented a geodetic measurement of ground levels from Palm Desert, southwestward to the Salton Sea. Subsidence was not studied in the Desert Hot Springs area. CVWD has embarked on a groundwater replenishment program which has slowed the rate of subsidence in the region. Ongoing studies from the USGS have discovered that the dominant factor in ground subsidence is the presence of silt layers which compress upon groundwater withdrawal (Sneed, APWA Presentation March 2013). Ground subsidence could occur in the future and the site could be affected especially if groundwater withdrawal were to re-initiate. We expect the subsidence to be on a regional scale that could cause settlement across the project site. However, the settlement occurs over a relatively large geographic area and typically does not cause differential settlement over a relatively short horizontal distance that should be addressed as a design concern as part of the site development.

4. GEOLOGIC MATERIALS

4.1 General

Based on the field investigation and published geologic maps of the area, the soil exposed at the surface and underlying the site to depths of several hundred feet is generally referred to as alluvium. The alluvium at the site includes cohesionless, undissected alluvial sand and gravel of the valley areas (Dibblee, 2008). Although undocumented artificial fill was not encountered in our borings, it may be present on the site. The soil and geologic units encountered at the site are discussed in general terms below. The site soil is described in detail on the boring logs in Appendix A.

4.2 Alluvial Sand and Gravel of the Valley Areas (Qa)

The alluvial soils encountered consist predominantly of poorly graded sand, poorly graded sand with silt, and silty sand. Cobbles were encountered, along with several "no recoveries" with locally high blow counts, within our borings at depth. Where explored, the alluvial soils are generally loose to very dense, dry to slightly moist, and are pale brown. This soil is highly susceptible to caving. Cobbles and boulders were observed scattered across the surface of the site. Based on what we encountered within our borings and what we observed across the surface of the site, cobbles and boulders should be expected to be encountered during grading operations. Furthermore, laboratory testing indicates site soils are dry, with average in-situ moisture contents within borings ranging between 0.7 and 6.6 percent.

5. GROUNDWATER

Static groundwater was not encountered during this investigation to the maximum depth explored of approximately 50½ feet. Based on a well record located approximately 0.8 mile west of the site (Well 03S04E01J001S), static groundwater may be as shallow as 176 feet beneath the ground surface at the site. We do not expect static groundwater to impact grading operations or the construction of improvements at the subject site. Static groundwater elevations are dependent on seasonal precipitation, irrigation, and land use, among other factors, and vary as a result.

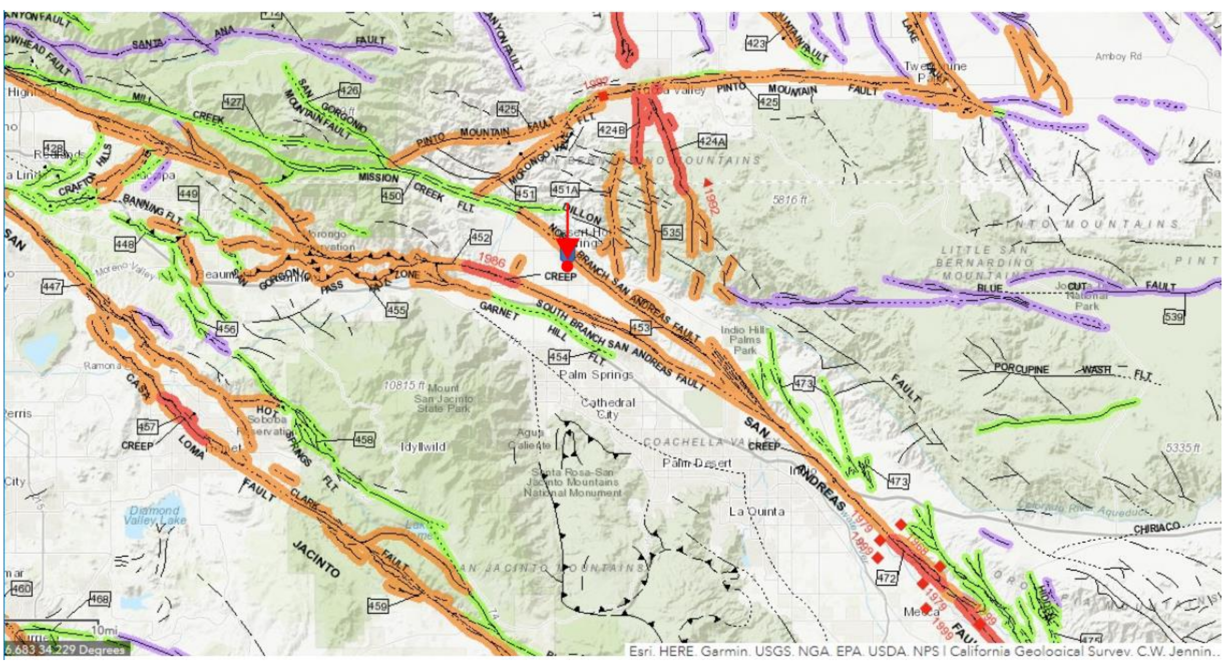
6. GEOLOGIC HAZARDS

6.1 Surface Fault Rupture

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

The site is not within a state-designated Alquist-Priolo Earthquake Fault Zone (CGS, 2023a; 2023b; 2017; Riverside County Map My County 2024) 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 active Southern California faults. The faults in the vicinity of the site are shown on the following Regional Fault Map.

REGIONAL FAULT MAP

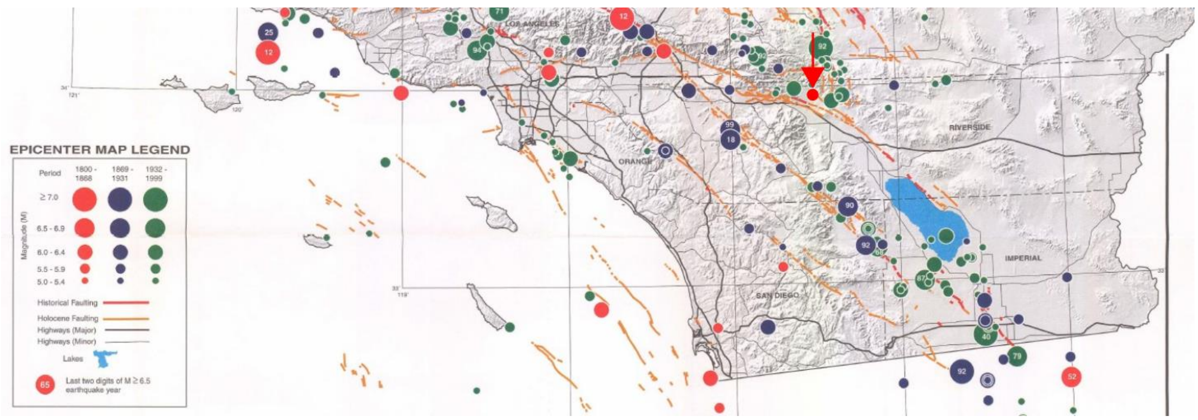


The closest surface trace of an active fault to the site is the North Branch of the San Andreas Fault located approximately ½ mile to the northeast. Other nearby active faults are the South Branch of the San Andreas Fault, San Gorgonio Pass Fault, and Morongo Fault located approximately 2½ miles southwest, 14 miles west, and 9 miles northwest, respectively (Bryant, 2010).

6.2 Seismicity

As with all 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 the following Regional Seismicity Map.

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.

HISTORIC EARTHQUAKE EVENTS WITH RESPECT TO THE SITE

Earthquake (Oldest to Youngest)	Date of Earthquake	Magnitude	Distance to Epicenter (Miles)	Direction to Epicenter
Near Redlands	March 10, 1933	6.3	43	W
Long Beach	March 10, 1933	6.4	87	WSW
Tehachapi	July 21, 1952	7.5	161	WNW
San Fernando	February 9, 1971	6.6	113	WNW
Whittier Narrows	October 1, 1987	5.9	91	W
Sierra Madre	June 28, 1991	5.8	89	WNW
Landers	June 28, 1992	7.3	18	NNE
Big Bear	June 28, 1992	6.4	26	NW
Northridge	January 17, 1994	6.7	118	W
Hector Mine	October 16, 1999	7.1	47	NNE
Ridgecrest China Lake Fault	July 5, 2019	7.1	140	NW

6.3 Seismic Design Criteria

The following table summarizes the site-specific design criteria obtained from the 2022 California Building Code (CBC; Based on the 2021 International Building Code [IBC] and ASCE 7-16), Chapter 16 Structural Design, Section 1613 Earthquake Loads. The data was calculated using the online application U.S. Seismic Design Maps, provided by the Structural Engineers Association of California (SEAOC). The short spectral response uses a period of 0.2 second. We evaluated the Site Class based on the discussion in Section 1613.2.2 of the 2022 CBC and Table 20.3-1 of ASCE 7-16. The values presented in the following table are for the risk-targeted maximum considered earthquake (MCE_R).

2022 CBC SEISMIC DESIGN PARAMETERS

Parameter	Value	2022 CBC Reference
Site Class	D	Section 1613.2.2
MCE_R Ground Motion Spectral Response Acceleration – Class B (short), S_S	2.372g	Figure 1613.2.1(1)
MCE_R Ground Motion Spectral Response Acceleration – Class B (1 sec), S_1	0.884g	Figure 1613.2.1(3)
Site Coefficient, F_A	1	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}	2.372g	Section 1613.2.3 (Eqn 16-20)
Site Class Modified MCE_R Spectral Response Acceleration – (1 sec), S_{M1}	1.503g*	Section 1613.2.3 (Eqn 16-21)
5% Damped Design Spectral Response Acceleration (short), S_{DS}	1.581g	Section 1613.2.4 (Eqn 16-22)
5% Damped Design Spectral Response Acceleration (1 sec), S_{D1}	1.002g*	Section 1613.2.4 (Eqn 16-23)

*Per Supplement 3 of ASCE 7-16, a ground motion hazard analysis (GMHA) shall be performed for projects on Site Class “D” sites with 1-second spectral acceleration (S_1) greater than or equal to 0.2g, which is true for this site. However, Supplement 3 of ASCE 7-16 provides an exception stating that that the GMHA may be waived provided that the parameter S_{M1} is increased by 50% for all applications of S_{M1}. The values for parameters S_{M1} and S_{D1} presented above have not been increased in accordance with Supplement 3 of ASCE 7-16.

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

ASCE 7-16 PEAK GROUND ACCELERATION

Parameter	Value	ASCE 7-16 Reference
Mapped MCE_G Peak Ground Acceleration, PGA	0.982g	Figure 22-9
Site Coefficient, F_{PGA}	1.1	Table 11.8-1
Site Class Modified MCE_G Peak Ground Acceleration, PGA_M	1.08g	Section 11.8.3 (Eqn 11.8-1)

Deaggregation of the Maximum Considered Earthquake (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 modal 7.5 magnitude event occurring at a hypocentral distance of 3.41 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.

The Riverside County Map My County website indicates that the site is in an area designated as having a moderate potential for liquefaction.

We performed a liquefaction analysis of the soils underlying the site using the 1996 NCEER method of analysis with the updates by Youd et al. (2001). The liquefaction potential evaluation was performed by utilizing a static groundwater depth of greater than 50 feet, a magnitude 7.5 earthquake, and the site class modified MCE_G peak ground acceleration (PGA_M) of 1.08g. This semi-empirical method is based on a correlation between values of Standard Penetration Test (SPT) resistance. An average conversion factor of 0.63 was used to derive SPT blow-count values from California Modified Sampler blow-count values.

Due to the lack of shallow static groundwater at the project site, liquefaction is not a design consideration. Our *Empirical Estimation of Liquefaction Potential* is included as Figure 3.

Additionally, an evaluation of seismically induced “dry-sand” settlement was performed, with the resulting seismic “dry-sand” settlement estimated to be up to ¾ inch, with differential settlement on the order ½ inch across 40 feet. An analysis of seismically induced “dry-sand” settlement is included as Figure 4.

6.5 Expansive Soil

The geologic units near the ground surface at the site consist of sandy soils. Laboratory testing indicates site soils have a “very low” expansion potential (Expansion Index [EI] 0 to 20).

6.6 Hydrocompression

Hydrocompression is the tendency of unsaturated soil structure to collapse upon wetting resulting in the overall settlement of the affected soil and overlying foundations or improvements supported thereon. Potentially compressible soils underlying the site are typically removed and compacted during remedial site grading. However, if compressible soil is left in-place, a potential for settlement due to hydrocompression of the soil exists.

Based on the laboratory test results, the potential for hydrocompression ranges from approximately 0.4 to 2.6 percent within the alluvial soils. We expect that the hydrocompressive characteristics of site soils will be effectively reduced as a result of remedial grading operations and adequate drainage measures; therefore, it is our opinion that hydrocompression is not a design consideration for this project.

6.7 Slope Stability

The topography at the site and surrounding areas is relatively level with a gentle slope to the south-southeast. There are no known landslides near the site, nor is the site in the path of any known or potential landslides (Dibblee, 2008). Therefore, the potential for slope stability hazards to adversely affect the proposed development is considered low.

6.8 Earthquake-Induced Flooding

Earthquake-induced flooding is inundation caused by failure of dams or other water-retaining structures due to earthquakes. Based on a review of the USGS dam inundation database, 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.

6.9 Tsunamis, Seiches, and Flooding

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

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

The site is not located in an area of flooding per Riverside County Map My County website (RCIT 2024).

6.10 Oil Fields & Methane Potential

Based on a review of the California Geologic Energy Management Division (CalGEM) Well Finder Website, the site is not located within an oil field and oil or gas wells are not documented within ½-mile of the site (CalGEM, 2023). However, due to the voluntary nature of record reporting by the oil well drilling companies, wells may be improperly located or not shown on the location map and undocumented wells could be encountered during construction. Any wells encountered during construction will need to be properly abandoned in accordance with the current requirements of the CalGEM.

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

6.11 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 (USGS, 2024). No large-scale extraction of groundwater, gas, oil, or geothermal energy is occurring or planned at the site or in the general site vicinity. There appears to be little or no potential for ground subsidence due to withdrawal of fluids or gases at the site.

Regional subsidence has occurred in recent history within the Coachella Valley. Initial subsidence occurred between the 1920's and 1940's when groundwater was over-pumped and groundwater levels declined to the order of 50 feet. The introduction of Colorado River water in 1949 reduced groundwater pumping and the related subsidence temporarily stopped. In the 1970's overdraft of the groundwater occurred resulting in groundwater level declines of 50 to 100 feet and subsidence resumed. In 1996, the United States Geologic Survey (USGS) in cooperation with CVWD implemented a geodetic measurement of ground levels from Palm Desert, southwestward to the Salton Sea. Subsidence of 0.39 to 0.57 ft. has occurred within the La Quinta Subsidence Zone, located southwest of the site, between 1996 and 2005. Subsidence at a point located near the intersection of Avenue 54 and Jackson was recorded at 44 mm in 1998. Since that time, no subsidence has been recorded at that location. CVWD has embarked on a groundwater replenishment program which has slowed the rate of subsidence in the region. Ongoing studies from the USGS have discovered that the dominant factor in ground subsidence is the presence of silt layers which compress upon groundwater withdraw (Sneed, APWA Presentation March 2013). Ground subsidence could occur in the future and the site could be affected especially if groundwater withdrawal were to re-initiate. We anticipate the subsidence to be on a regional scale that could cause settlement across the project site. However, the settlement occurs over a relatively large geographic area and typically does not cause differential settlement over a relatively short horizontal distance that should be addressed as a design concern as part of the site development.

7. SITE INFILTRATION

Preliminary percolation testing was performed in accordance with the procedures outlined in *Riverside County Flood Control and Water Conservation District LID BMP, Appendix A* (Handbook) for infiltration basins. The percolation test locations are depicted on the *Geologic Map and Site Plan*, Figure 2.

Percolation Test P-1 was performed within geotechnical Boring B-3, at a depth of 10 feet below existing grade. Initially, Boring B-3 was excavated using a CME-75 hollow-stem auger drilling machine with 8-inch-diameter augers for geotechnical logging and sampling. At the completion of the geotechnical portion of Boring B-3, the boring was backfilled with cuttings to approximately 10 feet of depth, and a bentonite plug was installed. Approximately two inches of gravel was placed at the bottom of the test hole, and a perforated pipe was placed atop the gravel to keep the test hole open. Gravel was placed around the bottom of the test hole to support the test pipe. The test location was pre-saturated prior to testing. The Boring B-3 log and the Test P-1 percolation data are presented in Appendix A. A summary of Test P-1 percolation data and infiltration rate results are provided in the following table.

CALCULATED INFILTRATION RATES FROM PERCOLATION TEST RESULTS

Parameter	P-1
Depth (inches)	120
Test Type	Sandy
Change in Head Over Time: ΔH (inches)	42.8
Average Head: H_{avg} (inches)	21.7
Time Interval: Δt (minutes)	10
Radius of Test Hole: r (inches)	4.0
Calculated Infiltration Rate: I_t (inches/hour)	21.7

The results of the preliminary percolation testing indicate that the calculated infiltration rates at the location tested is 21.7 inches per hour. The *Handbook* requires a factor of safety of 3 be applied to the values above based on the test method used.

The in-situ field percolation tests performed provide short-term infiltration rates. Where appropriate, the short-term infiltration rates shall be converted to long-term infiltration rates using reduction factors depending on the degree of infiltration quality, maintenance access and frequency, site variability, subsurface stratigraphy variation, and other factors. The small-scale percolation testing cannot model the complexity of the effect of interbedded layers of different soil composition, and our test results should be considered only as index values of infiltration rates.

Due to the presence of potentially hydrocompressive soils, the proposed infiltration system should be located a minimum distance of 20 feet from proposed settlement-sensitive structures and a minimum distance of 15 feet from site improvements to reduce the potential for induced settlements to adversely impact the proposed structures and improvements. Provided these offsets are maintained, there is a low potential for infiltration-related soil settlement to adversely affect the proposed structures; some settlement may occur locally within the area of the infiltration system.

The civil engineer should also 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 drainpipes and infiltration chambers. The project owner should understand that it is not our intent to completely prevent any soil settlement and/or associated distress of overlying pavement as a result of stormwater infiltration, as doing so would be cost-prohibitive to the proposed project.

8. CONCLUSIONS AND RECOMMENDATIONS

8.1 General

8.1.1 Soil or geologic conditions were not encountered during the investigation that would preclude the proposed development of the project, provided the recommendations presented herein are followed and implemented during design and construction. This report should be considered as preliminary, and the geotechnical design parameters presented herein should be verified once the project progresses to a more finalized state.

8.1.2 Potential geologic hazards at the site include seismic shaking, seismically induced settlement, and compressible near surface soils.

8.1.3 Based on our investigation and available geologic information, active, potentially active, or inactive faults are not present underlying or trending toward the site.

8.1.4 An evaluation of seismically induced settlement was performed, with the resulting seismic “dry-sand” settlement estimated to be up to ¾ inch, with differential settlement on the order ½ inch across 40 feet.

8.1.5 The upper portion of alluvial soils present at the site, in their current state, are not considered suitable for the support of additional compacted fill or settlement-sensitive improvements. Remedial grading of the surficial soil will be required as discussed herein. The site soils are suitable for re-use as engineered fill provided the recommendations in the *Grading* section of this report are followed.

8.1.6 Based on laboratory testing and our observations during our investigation, we expect onsite soils can be processed to meet gradation and sand equivalent requirements for trench bedding and shading.

8.1.7 Although static groundwater was not encountered during our subsurface investigation, it is possible that seepage may be encountered during the wet-weather season.

8.1.8 Cobbles and boulders were observed across the site surface, and cobbles were encountered within our borings at depth. We expect cobbles and boulders to be encountered during grading operations. The contractor should be prepared to screen cobbles and boulders from the soils during earthwork operations. Grading recommendations addressing oversize rock are discussed herein.

8.1.9 Based on the laboratory test results, the potential for hydrocompression ranges from approximately 0.4 to 2.6 percent within the alluvial soils. We expect that the hydrocompressive characteristics of site soils will be effectively reduced as a result of remedial grading operations and adequate drainage measures.

8.1.10 Site soils are generally comprised of sand with little or no cohesion that are highly susceptible to caving in un-shored excavations. It is the responsibility of the contractor to ensure that excavations and trenches are properly shored and maintained in accordance with Cal-OSHA rules and regulations to maintain the stability of adjacent existing improvements. The contractor should be aware that formwork may be required to prevent caving of shallow spread foundation excavations. Shoring recommendations are provided in the *Temporary Excavations* section of this report. In addition, the soil is susceptible to rapid erosion during a wet-weather event.

8.1.11 In-situ moisture and density laboratory testing indicate that site soils are significantly dry when compared to the optimum moisture content, determined by ASTM D1557. Significant moisture conditioning of material to be used as engineered fill should be expected during grading operations. Wet-weather events may affect the in-situ moisture content of site soils.

8.1.12 Proper drainage should be maintained to preserve the design properties of the engineered fill in the sheet-graded pads. Recommendations for site drainage are provided herein.

8.1.13 Once design or civil grading plans are made available, the recommendations within this report should be reviewed and revised, as necessary. Additionally, as the project design progresses toward a final design, changes in the design, location, or elevation of the proposed improvement should be reviewed by this office. Geocon should be contacted to evaluate the necessity for review and possible revision of this report.

8.2 Soil and Excavation Characteristics

8.2.1 The in-situ soils and oversize rock material at the site should generally be excavatable with moderate to heavy effort using conventional earth moving equipment in proper functioning order. The contractor should expect the presence of cobbles and boulders in the alluvial soils will present difficulties during the excavation process, and that formwork may be required to prevent caving of shallow spread foundation excavations. Special handling of these oversize materials should be performed in accordance with the *Recommended Grading Specifications* of Appendix C.

8.2.2 It is the responsibility of the contractor to ensure that all excavations and trenches are properly shored and maintained in accordance with applicable Cal-OSHA rules and regulations to maintain safety and the stability of existing improvements. All onsite excavations must be conducted in such a manner that potential surcharges from existing structures, construction equipment, and vehicle loads are resisted. The surcharge area may be defined by a 1:1 projection down and away from the bottom of an existing foundation or vehicle load. Penetrations below this 1:1 projection will require special excavation measures such as sloping and shoring. Excavation recommendations are provided in the *Temporary Excavations* section of this report.

8.2.3 Based on laboratory expansion index (EI) testing, site soils generally possess a "very low" expansion potential, EI of 0 to 20, and are considered "non-expansive" as defined by 2022 CBC Section 1803.5.3. The following table presents soil classifications based on the EI.

SOIL CLASSIFICATION BASED ON EXPANSION INDEX

Expansion Index (EI)	Expansion Classification	2022 CBC Expansion Classification
0 – 20	**Very Low**	**Non-Expansive**
21 – 50	Low	Expansive
51 – 90	Medium	
91 – 130	High	
Greater Than 130	Very High	

8.2.4 The recommendations presented herein assume that foundations and slabs will derive support in these materials.

8.2.5 Testing for expansion potential should be performed during finish grading to confirm the expansion potential of building pad fill material. Plasticity index testing should be performed on soils with expansion indices greater than 20.

8.3 Minimum Resistivity, pH, and Water-Soluble Chloride and Sulfate

8.3.1 We performed laboratory tests on samples of the site materials to evaluate the percentage of water-soluble sulfate content. Appendix B presents results of the laboratory water-soluble sulfate content tests. Laboratory tests performed on samples of the site materials indicate that the on-site materials possess an "S0" sulfate exposure to concrete structures as defined by 2022 CBC Section 1904 and ACI 318-19, Chapter 19. The following table presents a summary of concrete requirements set forth by 2022 CBC Section 1904.3 and ACI 318. The presence of water-soluble sulfates is not a visually discernible characteristic;

therefore, other soil samples from the site could yield different concentrations. Additionally, over time landscaping activities (i.e., addition of fertilizers and other soil nutrients) may affect the concentration.

REQUIREMENTS FOR CONCRETE EXPOSED TO SULFATE-CONTAINING SOLUTIONS

<table>
<tr><th colspan="2">Exposure Class</th><th>Water-Soluble Sulfate (SO_4) Percent by Weight</th><th>Cement Type (ASTM C 150)</th><th>Maximum Water to Cement Ratio by Weight[1]</th><th>Minimum Compressive Strength (psi)</th></tr>
<tr><td colspan="2">S0</td><td>SO_4<0.10</td><td>No Type Restriction</td><td>n/a</td><td>2,500</td></tr>
<tr><td colspan="2">S1</td><td>0.10≤SO_4<0.20</td><td>II</td><td>0.50</td><td>4,000</td></tr>
<tr><td colspan="2">S2</td><td>0.20≤SO_4≤2.00</td><td>V</td><td>0.45</td><td>4,500</td></tr>
<tr><td rowspan="2">S3</td><td>Option 1</td><td rowspan="2">SO_4>2.00</td><td>V+Pozzolan or Slag</td><td>0.45</td><td>4,500</td></tr>
<tr><td>Option 2</td><td>V</td><td>0.40</td><td>5,000</td></tr>
</table>

[1] Maximum water to cement ratio limits do not apply to lightweight concrete.

8.3.2 Laboratory test results indicate a resistivity of 13,000 ohm-cm, pH of 8.8, chloride content of 150 ppm, and sulfate content of 10 ppm. Based on the laboratory test results, the site soils would not be considered corrosive to metal improvements based on resistivity in accordance with Caltrans *Corrosion Guidelines* (Caltrans, 2021) as shown in the following table.

CALTRANS CORROSION GUIDELINES

Corrosion Exposure	Resistivity (ohm-cm)	Chloride (ppm)	Sulfate (ppm)	pH
Corrosive	<1,500	500 or greater	1,500 or greater	5.5 or less

8.3.3 Geocon does not practice in the field of corrosion engineering. Therefore, further evaluation by a corrosion engineer may be performed if improvements that could be susceptible to corrosion are planned.

8.4 Grading

8.4.1 Grading should be performed in accordance with the *Recommended Grading Specifications* contained in Appendix C and the grading ordinances of the City of Desert Hot Springs.

8.4.2 Prior to commencing grading, a preconstruction conference should be held at the site with the City inspector, owner or developer, grading contractor, civil engineer, and geotechnical engineer in attendance. Special soil handling and/or the grading plans can be discussed at that time.

8.4.3 Site preparation should begin with the removal of deleterious material, debris, buried and surficial trash, and vegetation. The depth of removal should be such that material exposed in cut areas or soil to be used as fill is relatively free of organic matter. Material generated during stripping and/or site demolition should be exported from the site. Rock greater than 6 inches in dimension should not be used in the engineered fill, and rock greater than 3 inches in dimension should not be used in backfill within utility trench corridors.

8.4.4 Dry, loose, soft, or compressible alluvial soils within a 1:1 (h:v) projection of the limits of grading should be removed to expose competent alluvial soils with a relative compaction of at least 85 percent, based on ASTM D1557. Based on our findings, we expect surficial alluvial soils will require remedial excavation and proper compaction. Removals should extend at least 5 feet below the existing ground surface, or at least 2 feet below the bottom of the planned foundations, whichever is deeper. Removals in pavement and walkway areas should extend at least 2 feet below subgrade and into competent alluvial soils. The engineering geologist should evaluate the actual depth of removal during grading operations to ensure the excavation bottoms do not contain dry, loose, soft, or compressible soils. Where over-excavation and compaction is to be conducted, the excavations should be extended laterally a minimum distance of 5 feet beyond the building footprint or for a distance equal to the depth of removal, whichever is greater. Patios and building appurtenances should be considered a part of the building footprint when determining the limits of lateral excavation. The bottom of the excavations should be competent alluvial soils, as defined above, and should be scarified to a depth of at least 1 foot, moisture conditioned at or slightly above optimum moisture content, and properly compacted to 90 percent of the laboratory maximum dry density, as determined by ASTM D1557.

8.4.5 Additional grading should be conducted as necessary to maintain the required 2 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 2 feet of engineered fill below foundations.

8.4.6 Geocon should observe the removal bottoms to check the competence of the exposed soil. Deeper excavations may be required if dry, loose, soft, or compressible soils are present at the base of the removals.

8.4.7 The fill placed within 3 feet of proposed foundations should possess a “very low” expansion potential (EI of 20 or less).

8.4.8 The site should be brought to finish grade elevations with fill compacted in layers. Layers of fill should be no thicker than will allow for adequate bonding and compaction. Fill, including backfill and scarified ground surfaces, should be compacted to a dry density of at least 90 percent of the laboratory maximum dry density, at or slightly above optimum moisture content as determined by ASTM D1557. Fill materials placed below optimum moisture content may require additional moisture conditioning prior to placing additional fill. Earthwork should be observed, and compacted fill tested by representatives of Geocon.

8.4.9 Oversized rock should be expected to be encountered during grading operations. The oversize rock will require special handling and placement. Rocks greater than 3 inches in maximum dimensions should not be placed within utility trench backfill. Rocks greater than 6 inches in maximum dimension should not be placed in soil fill within the upper 3 feet of finish grade. Rocks 6 to 12 inches in maximum dimension should be placed deeper than 3 feet below finished grade elevations. Rocks 12 inches or larger in maximum dimension should be exported from the site or placed at least 10 feet below finished grade elevations, in accordance with the *Recommended Grading Specifications* of Appendix C.

8.4.10 If needed, import fill should consist of granular materials with a “very low” expansion potential (EI of 20 or less), non-corrosive, generally free of deleterious material, and contain rock no larger than 6 inches. Geocon should be notified of the import soil source and should be afforded the opportunity to perform laboratory testing of the import soil prior to its arrival at the site to evaluate its suitability as fill material.

8.4.11 We do not expect perched groundwater or saturated materials to be encountered during remedial grading; however, should they be encountered (such as a result of seepage during the wet-weather season) extensive drying and mixing with dryer soil may be required if the saturated material is to be utilized as fill material in achieving finished grades. The materials should then be moisture conditioned at or slightly above optimum moisture content, prior to placement as compacted fill.

8.4.12 Foundation excavation bottoms must be observed and approved in writing by the Geotechnical Engineer, prior to placing fill, steel, gravel, or concrete.

8.5 Earthwork Grading Factors

8.5.1 Estimates of shrinkage factors are based on empirical judgments comparing the material in its existing or natural state as encountered in the exploratory excavations to a compacted state. Variations in natural soil density and in compacted fill density render shrinkage value estimates as rough approximations. As an example, the contractor can compact the fill to a dry density of 90 percent or higher of the laboratory maximum dry density. Thus, the contractor has an approximately 10 percent range of control over the fill volume. Due to the variations in the actual shrinkage/bulking factors, a balance area should be provided to accommodate variations.

8.6 Utility Trench Backfill

8.6.1 Utility trenches should be properly backfilled in accordance with the requirements of the City of Desert Hot Springs and the following recommendations. Pipes should be bedded with well-graded crushed rock or clean sands (sand equivalent greater than 30) to a depth of at least one foot over the pipe; based on our experience with site soils, we expect site soils will have a sand equivalent of greater than 30. The bedding material must be inspected and approved in writing by a qualified representative of Geocon. <u>The use of well-graded crushed rock is only acceptable if used in conjunction with filter fabric to prevent the gravel from having direct contact with soil.</u> The remainder of the trench backfill may be derived from onsite soil or approved import soil. Backfill of utility trenches should not contain rocks greater than 3 inches in diameter. The use of 2-sack slurry and controlled low strength material (CLSM) are also acceptable as backfill. However, consideration should be given to the possibility of differential settlement where the slurry ends and earthen backfill begins. These transitions should be minimized and additional stabilization should be considered at these transitions.

8.6.2 Trench excavation bottoms must be observed and approved in writing by a representative of Geocon, prior to placing bedding materials, fill, gravel, or concrete.

8.6.3 Utility trench backfill should be placed in layers no thicker than will allow for adequate bonding and compaction. Utility backfill should be compacted to a dry density of at least 90 percent of the laboratory maximum dry density and moisture conditioned at or slightly above optimum moisture content as determined by ASTM D1557. Backfill at the finish subgrade elevation of new pavements should be compacted to at least 95 percent of the maximum dry density. Backfill materials placed below the recommended moisture content may require additional moisture conditioning prior to placing additional fill.

8.7 Conventional Foundation Design

8.7.1 Proposed structures can be supported on shallow foundation systems supported on newly placed engineered fill, following the completion of grading, per the recommendations provided in the *Grading* section of this report. Due to the presence of abundant gravel, cobbles, and boulders, foundation excavations may result in irregular surfaces where not appropriately screened from the engineered fill; here cobbles and boulders are removed from the bottom of the foundation excavations, the resulting depression should be backfilled with site soils and compacted as necessary. In addition, due to the granular nature of soils and potential for caving, the contractor should be prepared to form foundation excavations, if necessary.

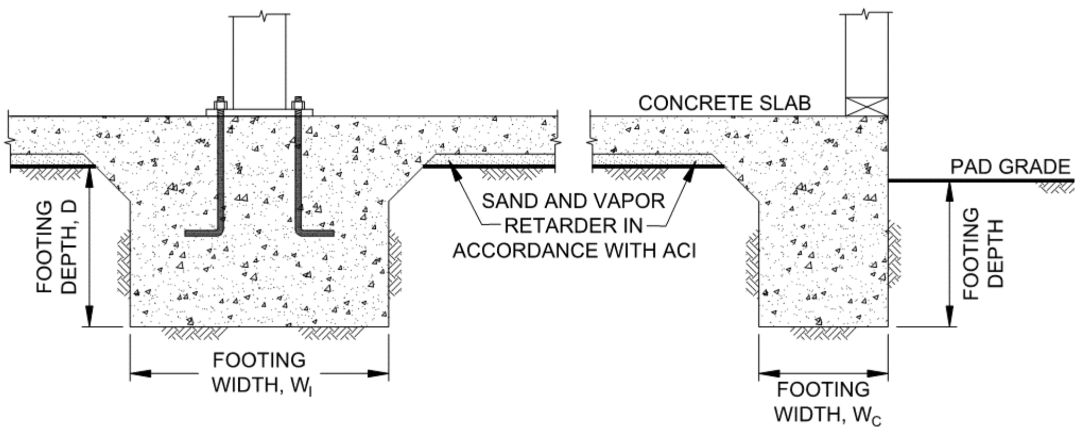
8.7.2 Foundations deriving support in newly placed engineered fill should be underlain by a minimum of 2 feet of engineered fill. Foundations for the structure should consist of continuous strip footings and/or isolated spread footings. The following table provides a summary of the foundation design recommendations.

SUMMARY OF FOUNDATION RECOMMENDATIONS

Parameter	Value
Minimum Continuous Foundation Width, W_C	12 inches
Minimum Isolated Foundation Width, W_I	24 inches
Minimum Foundation Depth, D	18 inches Below Lowest Adjacent Grade
Minimum Steel Reinforcement	Four No. 4 Bars, Two at the Top and Two at the Bottom
Allowable Bearing Pressure	3,000 psf
Bearing Pressure Increase	500 psf per Foot of Depth
	250 psf per Foot of Width
Maximum Allowable Bearing Pressure	4,000 psf
*Estimated Total Static Settlement	1¼ inches
*Estimated Static Differential Settlement	⅝ inch in 20 Feet
Design Expansion Index	20 or less

*The calculated seismic settlements provided in the *Liquefaction Potential* section of this report should be added to the static settlements for design purposes.

8.7.3 The foundations should be embedded in accordance with the recommendations herein and the *Wall/Column Footing Dimension Detail* below. The embedment depths should be measured from the lowest adjacent pad grade for both interior and exterior footings. Footings should be deepened such that the bottom outside edge of the footing is at least 7 feet horizontally from the face of the slope.



Wall/Column Footing Dimension Detail

8.7.4 The bearing capacity values presented herein are for dead plus live loads and may be increased by one-third when considering transient loads due to wind or seismic forces.

8.7.5 We should observe the foundation excavations prior to the placement of reinforcing steel and concrete to check that the exposed soil conditions are similar to those expected and that they have been extended to the appropriate bearing strata. Foundation modifications may be required if unexpected soil conditions are encountered.

8.7.6 Geocon should be consulted to provide additional design parameters as required by the structural engineer.

8.8 Concrete Slabs-On-Grade

8.8.1 Concrete slabs-on-grade for the structures should be constructed in accordance with the following table.

MINIMUM CONCRETE SLAB-ON-GRADE RECOMMENDATIONS

Parameter	Value
Minimum Concrete Slab Thickness	4 inches
Minimum Steel Reinforcement	No. 3 Bars 18 Inches on Center, Both Directions
Typical Slab Underlayment	3 to 4 Inches of Sand/Gravel/Base
Design Expansion Index	20 or less

8.8.2 Slabs that may receive moisture-sensitive floor coverings or may be used to store moisture-sensitive materials should be underlain by a vapor retarder. The vapor retarder design should be consistent with the guidelines presented in 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. In addition, the membrane should be installed in accordance with manufacturer's recommendations and ASTM requirements and installed in a manner that prevents puncture. The vapor retarder used should be specified by the project architect or developer based on the type of floor covering that will be installed and if the structure will possess a humidity controlled environment.

8.8.3 The bedding sand thickness should be determined by the project foundation engineer, architect, and/or developer. It is common to have 3 to 4 inches of sand for 5-inch and 4-inch thick slabs, respectively, in the Southern California region. However, we should be contacted to provide recommendations if the bedding sand is thicker than 6 inches. The foundation design engineer should provide appropriate concrete mix design criteria and curing measures to assure proper curing of the slab by reducing the potential for rapid moisture loss and subsequent cracking and/or slab curl. We suggest that the foundation design engineer present the concrete mix design and proper curing methods on the foundation plans. It is critical that the foundation contractor understands and follows the recommendations presented on the foundation plans.

8.8.4 Some projects remove the sand layer below the slab in parking structure areas. This is acceptable from a geotechnical engineering standpoint; however, relatively minor cracks could form due to differential curing. Therefore, the structural engineer and/or the concrete contractor should provide recommendations for proper curing techniques to help prevent cracking.

8.8.5 Concrete slabs should be provided with adequate crack-control joints, construction joints and/or expansion joints to reduce unsightly shrinkage cracking. The design of joints should consider criteria of the American Concrete Institute (ACI) when establishing crack-control spacing. Crack-control joints should be spaced at intervals no greater than 12 feet. Additional steel reinforcing, concrete admixtures and/or closer crack control joint spacing should be considered where concrete-exposed finished floors are planned.

8.8.6 Special subgrade presaturation is not deemed necessary prior to placing concrete; however, the exposed foundation and slab subgrade soil should be moisturized to maintain a moist condition as would be expected in any such concrete placement.

8.8.7 The concrete slab-on-grade recommendations are based on soil support characteristics only. The project structural engineer should evaluate the structural requirements of the concrete slabs for supporting expected loads.

8.8.8 Where exterior flatwork abuts the structure at entrant or exit areas, the exterior slab should be dowelled into the structure's foundation stemwall. This recommendation is intended to reduce the potential for differential elevations that could result from differential settlement or minor heave of the flatwork. Dowelling details should be designed by the project structural engineer.

8.8.9 The recommendations of this report are intended to reduce the potential for cracking of slabs due to expansive soil (if present), differential settlement of existing soil or 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 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.

8.9 Miscellaneous Foundations

8.9.1 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, can be supported on shallow foundation systems supported by a minimum 2 feet of engineered fill.. 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.

8.9.2 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.

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

8.10 Conventional Retaining Walls

8.10.1 The recommendations presented herein are generally applicable to the design of rigid concrete or masonry retaining walls having a maximum height of 5 feet. In the event that walls higher than 5 feet or other types of walls are planned, Geocon should be consulted for additional recommendations.

8.10.2 Retaining walls not restrained at the top and having a level backfill surface should be designed for an active soil pressure equivalent to the pressure exerted by a fluid density of 40 pounds per cubic foot (pcf). These soil pressures assume that the backfill materials within an area bounded by the wall and a 1:1 plane extending upward from the base of the wall possess an EI of 20 or less. For walls where backfill materials do not conform to the criteria herein, Geocon should be consulted for additional recommendations.

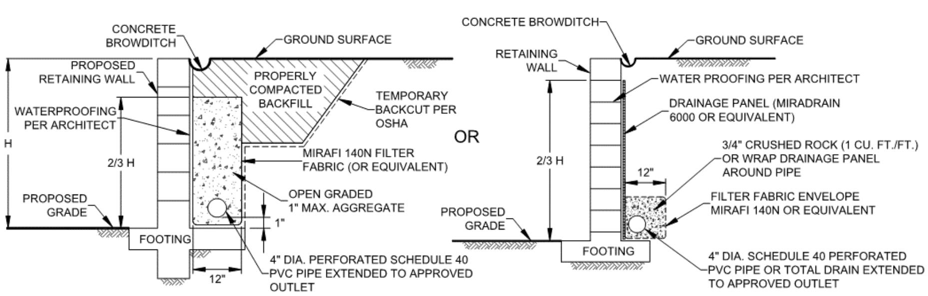
8.10.3 Unrestrained walls are those that are allowed to rotate more than 0.001H (where H equals the height of the retaining portion of the wall in feet) at the top of the wall. Where level walls are restrained from movement at the top, the walls should be designed for a soil pressure equivalent to the pressure exerted by a fluid density of 58 pcf.

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

8.10.5 It is common to see retaining walls constructed in the areas of the elevator pits. The retaining walls should be properly drained and designed in accordance with the recommendations presented herein. If the elevator pit walls are not drained, the walls should be designed with an at-rest pressure with an equivalent fluid density of 90 pcf. It is also common to see seepage and water collection within the elevator pit. The pit should be designed and properly waterproofed to prevent seepage and water migration into the elevator pit.

8.10.6 Unrestrained walls will move laterally when backfilled and loading is applied. The amount of lateral deflection is dependent on the wall height, the type of soil used for backfill, and loads acting on the wall. The retaining walls and improvements above the retaining walls should be designed to incorporate an appropriate amount of lateral deflection as determined by the structural engineer.

8.10.7 Retaining walls should be provided with a drainage system adequate to prevent the buildup of hydrostatic forces and waterproofed as required by the project architect. The soil immediately adjacent to the backfilled retaining wall should be composed of free draining material completely wrapped in Mirafi 140N (or equivalent) filter fabric for a lateral distance of 1 foot for the bottom two-thirds of the height of the retaining wall. The upper one-third should be backfilled with less permeable compacted fill to reduce water infiltration. Alternatively, a drainage panel, such as a Miradrain 6000 or equivalent, can be placed along the back of the wall. The use of drainage openings through the base of the wall (weep holes) is not recommended where the seepage could be a nuisance or otherwise adversely affect the property adjacent to the base of the wall. The recommendations herein assume a properly compacted backfill (EI of 20 or less) with no hydrostatic forces or imposed surcharge load. If conditions different than those described are expected or if specific drainage details are desired, Geocon should be contacted for additional recommendations. A graphic depicting typical retaining wall drainage is provided below.



Typical Retaining Wall Drainage Detail

8.10.8 Wall foundations should be designed in accordance with the foundation recommendations in the *Conventional Foundation Design* section of this report.

8.10.9 Additional 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.

8.10.10 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*.

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

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

8.11 Elevator Pit Design

8.11.1 If used, the elevator pit slab and retaining wall should be designed by the project structural engineer. Elevator pit foundation and walls may be designed in accordance with the recommendations in the *Conventional Foundation Design* and *Conventional Retaining Walls* sections of this report.

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

8.11.3 If retaining wall drainage is to be provided, the drainage system should be designed in accordance with the *Conventional Retaining Walls* section of this report.

8.11.4 We recommend that the exterior walls and slab be waterproofed to prevent excessive moisture inside of the elevator pit. Waterproofing design and installation are not the responsibility of the Geotechnical Engineer.

8.12 Elevator Piston

8.12.1 If a plunger-type elevator piston is installed for this project, a deep drilled excavation will be required. It is important to verify that the drilled excavation is not situated immediately adjacent to a foundation, or the drilled excavation could compromise the existing foundation support, especially if the drilling is performed subsequent to the foundation construction. In addition, boulders and cobbles may be encountered in the existing fill or alluvial soils, and some of the site soils have little to no cohesion and are prone to excessive caving. The contractor should be prepared for difficult drilling conditions.

8.12.2 Caving is expected, and the contractor should be prepared to use casing and should have it readily available at the commencement of drilling activities. Continuous observation of the drilling and installation of the elevator piston by the Geotechnical Engineer should be performed.

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

8.13 Swimming Pools

8.13.1 For the proposed pools, the shell bottoms should be designed as a free-standing structure and may derive support on a minimum of 2 feet of engineered fill compacted to a dry density of at least 90 percent of the laboratory maximum dry density at or slightly above optimum moisture content as determined by ASTM D1557.

8.13.2 Swimming pool foundations and walls may be designed in accordance with the recommendations in the *Conventional Foundation Design* and *Conventional Retaining Walls* sections of this report. A hydrostatic relief valve should be considered as part of the swimming pool design unless a gravity drain system can be placed beneath the pool shell.

8.13.3 Based on the soil overburden load that will be removed during excavation of the swimming pool, anticipated settlements are expected to be small. Static differential settlement of the pool is not expected to exceed ¼ inch over a horizontal distance of 40 feet.

8.13.4 Surface drainage around the pool/spa should be designed to prevent water from ponding and seeping into the ground. Surface water should be collected and conducted through non-erosive devices to the street, storm drain or other approved water course or disposal area. Leakage from the proposed pool/spa could create an artificial groundwater condition that will likely create instability problems. Therefore, all plumbing and the pool/spa should be leak free.

8.13.5 The deck for the swimming pool/spa should be cast separately from the swimming pool/spa, and water stops should be provided between the bond beam and the deck. Jointing for concrete flatwork should be provided in accordance with the recommendations of the American Concrete Institute. The joints should be sealed with an approved flexible sealant to reduce the potential for introduction of surface water into the underlying soil.

8.13.6 To mitigate the potential for moisture infiltration into the subgrade soils beneath the pool deck, we recommend the construction of a deepened footing along the outside edge of the pool deck flatwork. A subdrain consisting of 4-inch diameter perforated PVC pipe should be installed inside the deepened footing and sloped to drain into an approved outlet. The pipe should be surrounded by ¾ inch open-graded gravel and wrapped with filter fabric.

8.13.7 If the proposed pools are in proximity to a proposed or existing structure, consideration should be given to the construction sequence. If the proposed pool is to be constructed near an existing structure, or a proposed structure that is constructed before the pool construction, the excavation required for the pool could remove a critical component of lateral support from the foundations of the structure and would therefore require shoring to safeguard the foundations. Once information regarding the pool locations and depth becomes available, this information should be provided to Geocon for review and possible revision of these recommendations.

8.14 Lateral Design

8.14.1 To resist lateral loads, a passive pressure exerted by an equivalent fluid weight of 270 pounds per cubic foot (pcf) with a maximum earth pressure of 2,700 pcf should be used for the design of footings or shear keys poured neat against properly compacted fill. The allowable passive pressure assumes a horizontal surface extending at least 5 feet, or three times the surface generating the passive pressure, whichever is greater. The upper 12 inches of material in areas not protected by floor slabs or pavement should not be included in design for passive resistance.

8.14.2 If friction is to be used to resist lateral loads, an allowable coefficient of friction between soil and concrete of 0.4 should be used for design.

8.14.3 The passive and frictional resistant loads can be combined for design purposes. When combining passive and friction for lateral resistance, the passive component should be reduced by one-third. The lateral passive pressures may be increased by one-third when considering transient loads due to wind or seismic forces.

8.15 Exterior Concrete Flatwork

8.15.1 Exterior concrete flatwork not subject to vehicular traffic should be constructed in accordance with the recommendations presented in the following table. The recommended steel reinforcement would help reduce the potential for cracking.

MINIMUM CONCRETE FLATWORK RECOMMENDATIONS

Expansion Index, EI	Minimum Reinforcing Steel* Options	Minimum Thickness
EI $\leq$ 20	6x6-W2.9/W2.9 (6x6-6/6) welded wire mesh	4 Inches
	No. 3 Bars 18 inches on center, Both Directions	

*In excess of 8 feet square.

8.15.2 The subgrade soil should be properly moisturized and compacted prior to the placement of steel and concrete. The subgrade soil should be compacted to a dry density of at least 90 percent of the laboratory maximum dry density at or slightly above optimum moisture content in accordance with ASTM D1557.

8.15.3 Even with the incorporation of the recommendations of this report. The reinforcing steel should overlap continuously in flatwork to reduce the potential for vertical offsets within flatwork. Additionally, flatwork should be structurally connected to the curbs, where possible, to reduce the potential for offsets between the curbs and the flatwork.

8.15.4 Concrete flatwork should be provided with crack control joints to reduce and/or control shrinkage cracking. Crack control spacing should be determined by the project Structural Engineer based upon the slab thickness and intended usage. Criteria of the American Concrete Institute (ACI) should be taken into consideration when establishing crack control spacing. Subgrade soil for exterior slabs not subjected to vehicle loads should be compacted in accordance with criteria presented in the grading section prior to concrete placement. Subgrade soil should be properly compacted and the moisture content of subgrade soil should be verified prior to placing concrete. Base materials will not be required below concrete improvements.

8.15.5 Where exterior flatwork abuts the structure at entrant or exit points, the exterior slab should be dowelled into the structure's foundation stemwall. This recommendation is intended to reduce the potential for differential elevations that could result from differential settlement or minor heave of the flatwork. Dowelling details should be designed by the project structural engineer.

8.15.6 The recommendations presented herein are intended to reduce the potential for cracking of exterior slabs as a result of differential movement. However, even with the incorporation of the recommendations presented herein, slabs-on-grade will still crack. The occurrence of concrete shrinkage cracks is independent of the soil supporting characteristics. Their occurrence may be reduced and/or controlled by limiting the slump of the concrete, the use of crack control joints and proper concrete placement and curing. Crack control joints should be spaced at intervals no greater than 12 feet. Literature provided by the Portland Concrete Association (PCA) and American Concrete Institute (ACI) present recommendations for proper concrete mix, construction, and curing practices, and should be incorporated into project construction.

8.16 Preliminary Pavement Design

8.16.1 Where new paving is to be placed, we recommend that undocumented fill or soft/loose soils be excavated and properly compacted for paving support in accordance with the recommendations provided in the *Grading* section of this report. The client should be aware that excavation and compaction of undocumented fill or soft/loose soils in the area of new paving is not required; however, paving constructed over existing uncertified fill or unsuitable soils may experience increased settlement and/or cracking, and may therefore have a shorter design life and increased maintenance costs. As a minimum, the upper 12 inches of paving subgrade should be scarified, moisture conditioned at or slightly above optimum moisture content, and properly compacted to at least 95 percent relative compaction, as determined by ASTM D1557.

8.16.2 The final pavement design should be based on R-value testing of soils at roadway subgrade elevation. Roadways should be designed in accordance with the City of Desert Hot Springs *Standard Plans & Specifications* when final Traffic Indices (TI) and R-Value test results of subgrade soils are completed. The roadway classifications and TI's selected for our preliminary evaluation are in accordance with those specified in *Section III.C., Street Standards* of the City of Desert Hot Springs *Standard Plans & Specifications*. Based on our observation and experience with site soils, we used an assumed R-value of 50 for our preliminary evaluation of pavements. Preliminary flexible pavement sections are presented in the following table. Geocon should be contacted if other roadway classifications and traffic indices are appropriate for the project.

PRELIMINARY FLEXIBLE PAVEMENT SECTIONS

Road Classification	Assumed Traffic Index	Assumed Subgrade R-Value	Asphalt Concrete (inches)	Crushed Aggregate Base (inches)
Alley/Cul-de-Sac	3.5	50	3	4
Local Collector	4.0		3	6
Collector	5.5		3	8

8.16.3 The crushed aggregated base and asphalt concrete materials should conform to Section 200-2.2 and Section 203-6, respectively, of the *Greenbook*. Base materials should be moisture conditioned at or slightly above optimum moisture content and properly compacted to at least 95 percent relative compaction, as determined by ASTM D1557. Asphalt concrete should be compacted to a density of 95 percent of the laboratory Hveem density in accordance with ASTM D1561.

8.16.4 A rigid Portland cement concrete (PCC) pavement section should be placed in roadway aprons and cross gutters. We calculated the rigid pavement section in general conformance with the procedure recommended by the American Concrete Institute report ACI 330-21 Commercial Concrete Parking Lots and Site Paving Design and Construction – Guide. The following table provides the traffic categories and design parameters used for the calculations for 20-year design life.

TRAFFIC CATEGORIES

Traffic Category	Description	Reliability (%)	Slabs Cracked at End of Design Life (%)
A	Car Parking Areas and Access Lanes	60	15
B	Entrance and Truck Service Lanes	60	15
C	School or City Buses (Excluding Large Articulated Buses)	75	15
D	Heavy Duty Trucks (Gross Weight of 80 Kips)	75	15
E	Garbage or Fire Truck Lane	75	15

8.16.5 We used the parameters presented in the following table to calculate the pavement design sections. We should be contacted to provide updated design sections, if necessary.

RIGID PAVEMENT DESIGN PARAMETERS

Design Parameter	Design Value
Modulus of subgrade reaction, k	100 pci
Modulus of rupture for concrete, M_R	500 psi
Concrete Compressive Strength	3,000 psi
Concrete Modulus of Elasticity, E	3,150,000

8.16.6 Based on the criteria presented herein, the PCC pavement sections should have a minimum thickness as presented in the following table.

RIGID VEHICULAR PAVEMENT RECOMMENDATIONS

Traffic Category	Trucks Per Day	Portland Cement Concrete, T (Inches)
A = Car Parking Areas and Access Lanes	10	5½
B = Entrance and Truck Service Lanes	10	6
	50	6½
	100	6½
C = School or City Buses	50	9½
	100	9½
D = Heavy Duty Trucks	50	6½
	100	7
E = Garbage or Fire Truck Lanes	5	6½
	10	7

8.16.7 The PCC vehicular pavement should be placed over subgrade soil that is compacted to a dry density of at least 95 percent of the laboratory maximum dry density, at or slightly above optimum moisture content, as determined by ASTM D1557.

8.16.8 Adequate joint spacing should be incorporated into the design and construction of the rigid pavement in accordance with the following table.

MAXIMUM JOINT SPACING

Pavement Thickness, T (Inches)	Maximum Joint Spacing (Feet)
4<T<5	10
5≤T<6	12.5
6≤T	15

8.16.9 The rigid pavement should also be designed and constructed incorporating the parameters presented in the following table.

ADDITIONAL RIGID PAVEMENT RECOMMENDATIONS

Subject	Value
Thickened Edge	1.2 Times Slab Thickness Adjacent to Structures
	1.5 Times Slab Thickness Adjacent to Soil
	Minimum Increase of 2 Inches
	4 Feet Wide
Crack Control Joint Depth	Early Entry Sawn = T/6 to T/5, 1.25 Inch Minimum
	Conventional (Tooled or Conventional Sawing) = T/4 to T/3
Crack Control Joint Width	¼-Inch for Sealed Joints and Per Sealer Manufacturer's Recommendations
	$^1/_{16}$- to $^1/_4$-Inch is Common for Unsealed Joints

8.16.10 Reinforcing steel will not be necessary within the concrete for geotechnical purposes with the possible exception of dowels at construction joints as discussed herein.

8.16.11 To control the location and spread of concrete shrinkage cracks, crack-control joints (weakened plane joints) should be included in the design of the concrete pavement slab. Crack-control joints should be sealed with an appropriate sealant to prevent the migration of water through the control joint to the subgrade materials. The depth of the crack-control joints should be in accordance with the referenced ACI guide.

8.16.12 To provide load transfer between adjacent pavement slab sections, a butt-type construction joint should be constructed. The butt-type joint should be thickened by at least 20 percent at the edge and taper back at least 4 feet from the face of the slab.

8.16.13 Concrete curb and gutter should be placed on soil subgrade compacted to a dry density of at least 95 percent of the laboratory maximum dry density at or slightly above optimum moisture content. Cross-gutters that receive vehicular traffic should be placed on subgrade soil compacted to a dry density of at least 95 percent of the laboratory maximum dry density at or slightly above optimum moisture content. Base materials should not be placed below the curb and gutter, or cross-gutters so water is not able to migrate from the adjacent parkways to the pavement sections. Where flatwork is located directly adjacent to the curb and gutter, the concrete flatwork should be structurally connected to the curbs to help reduce the potential for offsets between the curbs and the flatwork.

8.17 Temporary Excavations

8.17.1 Excavations of up to 10 feet in height may be required during earthwork and utility installation operations. The excavations are expected to expose engineered fill or alluvial soils that are highly susceptible to caving. Vertical excavations up to 5 feet in height may be attempted where not surcharged by adjacent foundations or traffic; however, the contractor should be prepared for caving sands to be present in open excavations and formwork may be required in foundation excavations. Sloping measures will likely be required to provide a stable excavation. Excavations should be observed for the presence of cobbles and boulders to determine if further safety measures are required.

8.17.2 Vertical excavations greater than 5 feet or where surcharged by existing structures will require sloping or shoring measures in order to provide a stable excavation. The contractor’s competent person should evaluate the appropriate slope based on soil type, per Cal-OSHA regulations. We anticipate that sufficient space is available to complete the required earthwork for this project using sloping measures.

8.17.3 Where there is insufficient space for sloped excavations, shoring or trench shields should be used to support excavations. Shoring may also be necessary where sloped excavation could remove vertical or lateral support of existing improvements, including existing utilities and adjacent structures. The contractor’s competent person should evaluate the appropriate shoring system to provide per Cal-OSHA regulations.

8.17.4 Where temporary construction slopes are utilized, the top of the slope should be barricaded to prevent vehicles and storage loads at the top of the slope within a horizontal distance equal to the height of the slope. If the temporary construction slopes are to be maintained during the rainy season, berms are suggested along the tops of the slopes where necessary to prevent runoff water from entering the excavation and eroding the slope faces. The contractor's competent person should inspect the soils exposed in the cut slopes during excavation in accordance with Cal-OSHA regulations so that modifications of the slopes can be made if variations in the soil conditions occur.

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

8.18 Site Drainage and Moisture Protection

8.18.1 Adequate site drainage is critical to reduce the potential for differential soil movement, erosion and subsurface seepage. Under no circumstances should water be allowed to pond adjacent to footings. The site should be graded and maintained such that surface drainage is directed away from structures in accordance with 2022 CBC 1804.4 or other applicable standards. In addition, surface drainage should be directed away from the top of slopes into swales or other controlled drainage devices. Roof and pavement drainage should be directed into conduits that carry runoff away from the proposed structure.

8.18.2 Underground utilities should be leak free. Utility and irrigation lines should be checked periodically for leaks and detected leaks should be repaired promptly. Detrimental soil movement could occur if water can infiltrate the soil for prolonged periods of time.

8.18.3 Landscaping planters 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. We recommend area drains to collect excess irrigation water and transmit it to drainage structures or impervious above-grade planter boxes be used. In addition, where landscaping is planned adjacent to the pavement, we recommend construction of a cutoff wall or the use of an impermeable geosynthetic along the edge of the pavement that extends at least 6 inches below the bottom of the base material.

8.18.4 Proposed infiltration systems should be offset from the outside edge of planned foundations a minimum lateral distance of 20 feet to reduce the occurrence of water migrating below the load projection of planned structures, and a minimum lateral distance of 15 feet from site improvements. These minimum offsets will reduce the potential for settlements induced by migrating water that could adversely impact the proposed structures and improvements.

8.18.5 If not properly constructed, there is a potential for distress to improvements and properties located hydrologically down gradient or adjacent to infiltration areas. Factors such as the amount of water to be detained, its residence time, and soil permeability have an important effect on seepage transmission and the potential adverse impacts that may occur if the storm water management features are not properly designed and constructed. We have not performed a hydrogeology study at the site. Downgradient and adjacent structures may be subjected to seeps, movement of foundations and slabs, or other impacts as a result of water infiltration.

8.19 Plan Review

8.19.1 Grading and structural/foundation plans should be reviewed by Geocon prior to finalization of design to check that the plans have been prepared in substantial conformance with the recommendations of this report, and to provide additional analyses or recommendations, if necessary.

LIMITATIONS AND UNIFORMITY OF CONDITIONS

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 this investigation. If any variations or undesirable conditions are encountered during construction, or if the proposed construction will differ from that expected herein, Geocon West, Inc., should be notified so that supplemental recommendations can be given. The evaluation or identification of the potential presence of hazardous materials was not part of the scope of services provided by Geocon West, Inc.

This report is issued with the understanding that it is the responsibility of the owner, or of their 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.

The requirements for concrete and reinforcing steel presented in this report are preliminary recommendations from a geotechnical perspective. The Structural Engineer should provide the final recommendations for structural design of concrete and reinforcing steel for foundation systems, floor slabs, exterior concrete, or other systems where concrete and reinforcing steel are utilized, in accordance with the latest version of applicable codes.

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.

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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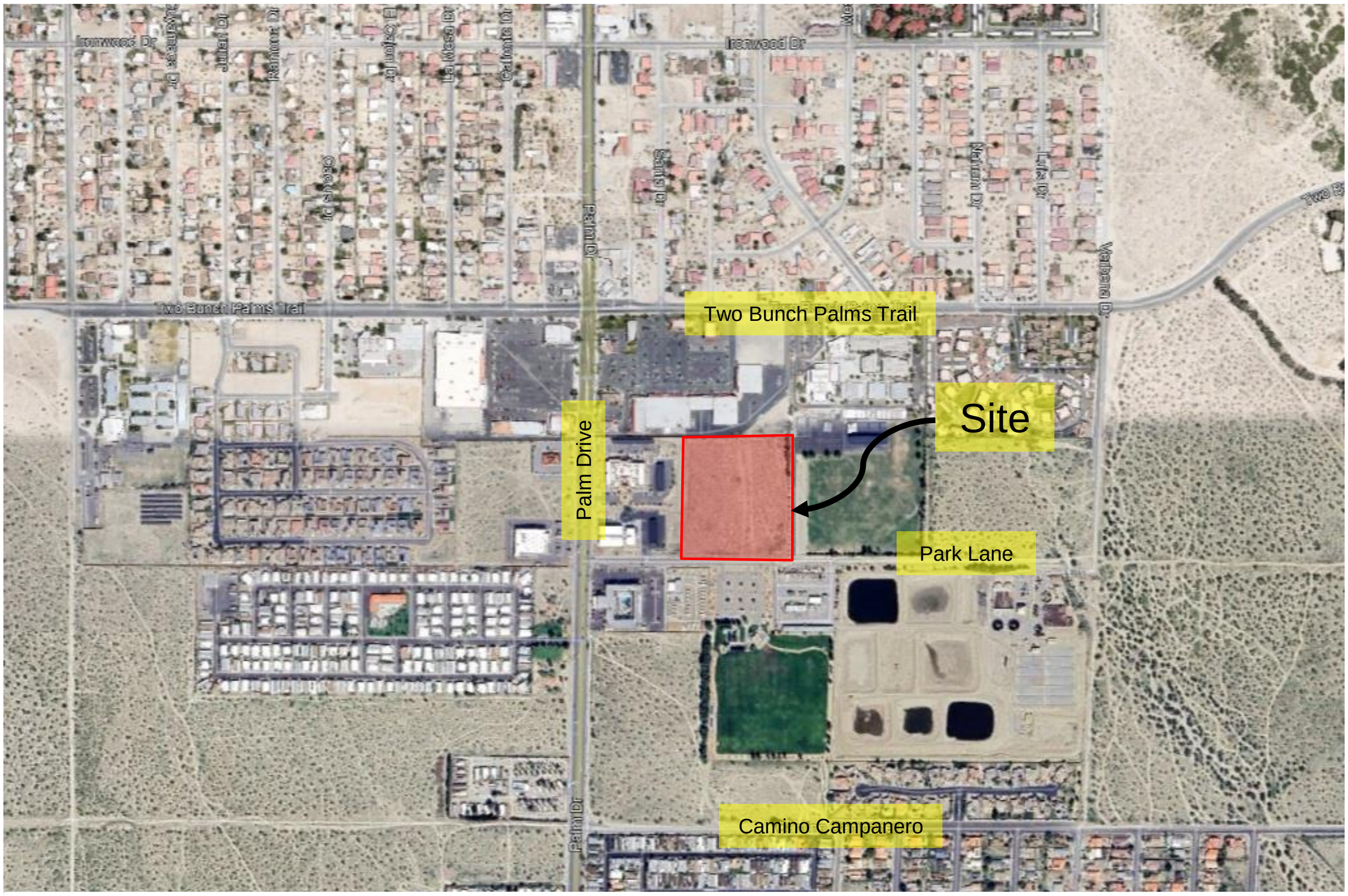

0’ 1,000’ 2,000’

SCALE: 1” = 1,000’

N

SOURCE: Google Earth, 2024

VICINITY MAP

GEOCON WEST, INC.

GEOTECHNICAL, ENVIRONMENTAL, MATERIALS
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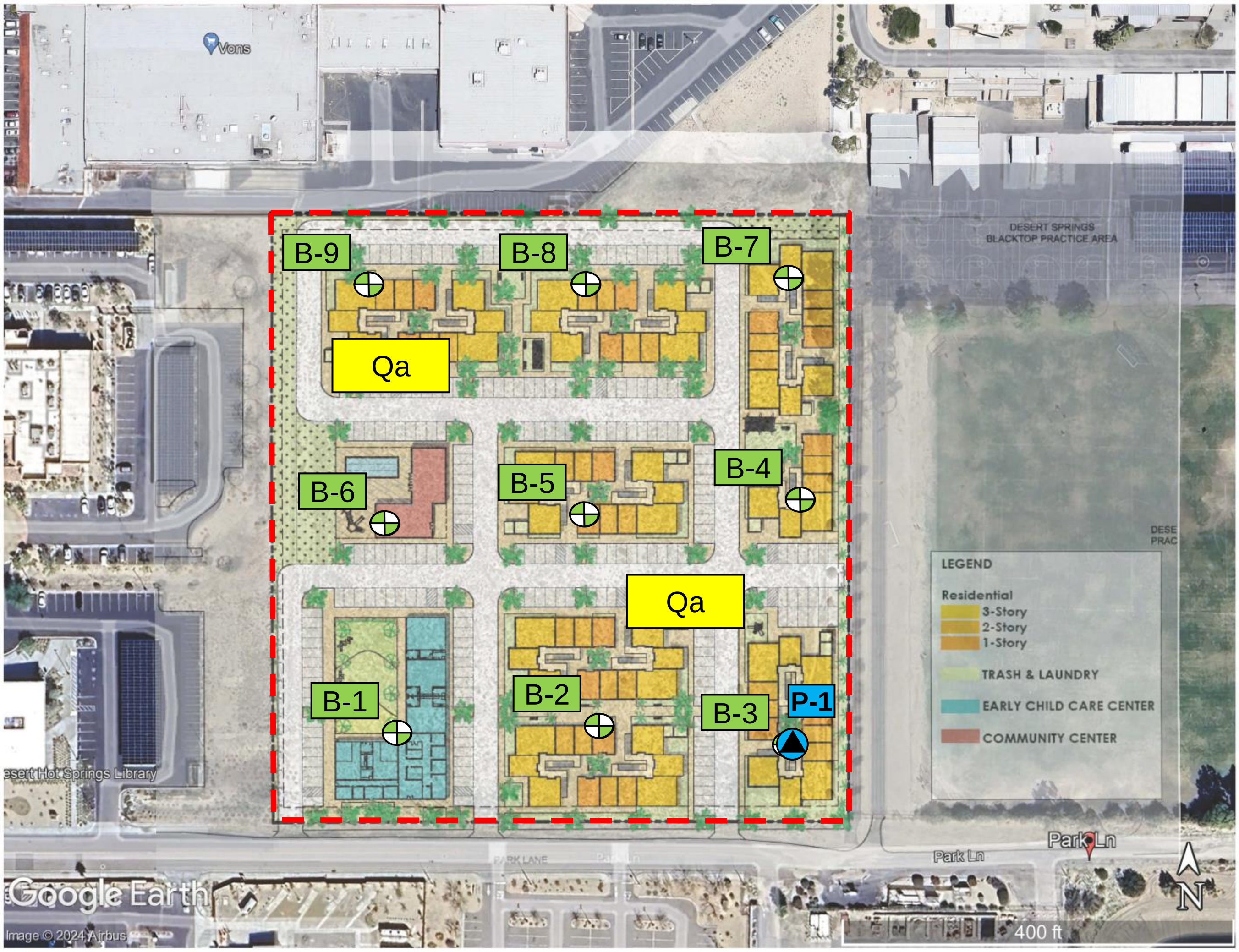
PHONE 951-304-2300 www.geoconinc.com

DRAFT BY: KD | CHECKED BY: ATS

MULTI-FAMILY RESIDENTIAL DEVELOPMENT
14320 PALM DRIVE
DESERT HOT SPRINGS, CALIFORNIA

SEPTEMBER 2024 | PROJECT NO. T3082-22-01 | FIG. 1



GEOCON LEGEND

Locations are approximate

B-9 ……. BORING LOCATION

P-1 ……. PERCOLATION TEST LOCATION

……. PROJECT LIMITS

Qa ……. ALLUVIAL SAND AND GRAVEL OF THE VALLEY AREAS

0' 165' 330'

SCALE: 1" = 165'

N

SOURCE: Google Earth, 2024

GEOLOGIC MAP AND SITE PLAN

GEOCON WEST, INC.

GEOTECHNICAL, ENVIRONMENTAL, MATERIALS
78-075 Main Street #G-203, La Quinta, California 92253
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MULTI-FAMILY RESIDENTIAL DEVELOPMENT
14320 PALM DRIVE
DESERT HOT SPRINGS, CALIFORNIA

DRAFT BY: KD	CHECKED BY: ATS	SEPTEMBER 2024	PROJECT NO. T3082-22-01	FIG. 2

Project Name : Multi-Family Residential Deve
Project No : T3033-22-01
Boring : B-5

GEOCON

EMPIRICAL ESTIMATION OF LIQUEFACTION POTENTIAL
MAXIMUM CONSIDERED EARTHQUAKE

NCEER (1996) METHOD W 2001 UPDATES

EARTHQUAKE INFORMATION:

Earthquake Magnitude:	7.50
Peak Horiz. Acceleration PGA_M (g):	1.080
Magnitude Scaling Factor:	1.000
Historic High Groundwater:	176.0
Groundwater Depth During Exploration:	51.5

ENERGY & ROD CORRECTIONS:

Energy Correction (CE) for N60:	1.25
Rod Len.Corr.(CR) (0-no or 1-yes):	1
Bore Dia. Corr. (CB):	1.00
Sampler Corr. (CS):	1.20
Use Ksigma (0-no or 1-yes):	1

LIQUEFACTION CALCULATIONS:

Unit Wt. Water (pcf):	62.4

Depth to Base (ft)	Total Unit Wt. (pcf)	Water (0 or 1)	Field SPT (N)	Depth of SPT (ft)	Liq.Sus. (0 or 1)	-200 (%)		Est. Dr (%)	CN Factor	Corrected (N1)60cs	Eff. Unit Wt. (psf)	Resist. CRR 7.5	rd Factor	Induced CSR	Liquefac. Safe.Fact.
1.0	100.7	0	10.0	2.5	1	5		73	1.700	19.1	100.7	0.205	1.000	0.702	--
2.0	100.7	0	10.0	2.5	1	5		73	1.700	19.1	100.7	0.205	0.998	0.701	--
3.0	100.7	0	10.0	2.5	1	5		73	1.700	19.1	100.7	0.205	0.996	0.699	--
4.0	100.7	0	10.0	2.5	1	5		73	1.700	19.1	100.7	0.205	0.994	0.698	--
5.0	113.4	0	9.0	5.0	1	5		66	1.700	17.2	113.4	0.183	0.991	0.696	--
6.0	113.4	0	9.0	5.0	1	5		66	1.700	17.2	113.4	0.183	0.989	0.694	--
7.0	113.4	0	9.0	5.0	1	5		66	1.700	17.2	113.4	0.183	0.987	0.693	--
8.0	109.7	0	11.0	7.5	1	4		69	1.618	20.0	109.7	0.216	0.985	0.691	--
9.0	109.7	0	11.0	7.5	1	4		69	1.517	18.8	109.7	0.201	0.982	0.690	--
10.0	121.5	0	18.0	10.0	1	4		85	1.429	28.9	121.5	0.407	0.980	0.688	--
11.0	121.5	0	18.0	10.0	1	4		85	1.351	27.3	121.5	0.348	0.978	0.687	--
12.5	121.5	0	18.0	10.0	1	4		85	1.269	25.7	121.5	0.306	0.975	0.685	--
13.0	125.9	0	19.0	12.5	1	9		83	1.240	27.3	125.9	0.348	0.973	0.683	--
14.0	125.9	0	19.0	12.5	1	9		83	1.174	25.9	125.9	0.311	0.972	0.682	--
15.0	116.6	0	29.0	15.0	1	9		98	1.129	40.7	116.6	Infin.	0.970	0.681	--
16.0	116.6	0	29.0	15.0	1	9		98	1.091	39.3	116.6	Infin.	0.967	0.679	--
17.0	116.6	0	29.0	15.0	1	9		98	1.057	38.1	116.6	Infin.	0.965	0.678	--
18.0	125.9	0	23.0	17.5	1	6		83	1.024	30.4	125.9	Infin.	0.963	0.676	--
19.0	125.9	0	23.0	17.5	1	6		83	0.993	29.5	125.9	0.434	0.961	0.674	--
20.0	125.9	0	65.0	20.0	1	6		136	0.965	84.6	125.9	Infin.	0.958	0.673	--
21.0	125.9	0	65.0	20.0	1	6		136	0.939	82.3	125.9	Infin.	0.956	0.671	--
22.0	125.9	0	65.0	20.0	1	6		136	0.915	80.2	125.9	Infin.	0.953	0.669	--
23.0	125.9	0	65.0	20.0	1	6		136	0.893	78.2	125.9	Infin.	0.950	0.667	--
24.0	125.9	0	65.0	20.0	1	6		136	0.872	76.4	125.9	Infin.	0.947	0.665	--
25.0	125.9	0	41.0	25.0	1	6		101	0.853	50.3	125.9	Infin.	0.944	0.662	--
26.0	125.9	0	41.0	25.0	1	6		101	0.834	49.3	125.9	Infin.	0.940	0.660	--
27.0	125.9	0	41.0	25.0	1	6		101	0.817	48.3	125.9	Infin.	0.936	0.657	--
28.0	125.9	0	41.0	25.0	1	6		101	0.801	47.3	125.9	Infin.	0.932	0.654	--
29.0	125.9	0	41.0	25.0	1	6		101	0.786	46.4	125.9	Infin.	0.928	0.651	--
30.0	125.9	0	63.0	30.0	1	6		118	0.772	73.3	125.9	Infin.	0.923	0.648	--
31.0	125.9	0	63.0	30.0	1	6		118	0.759	72.0	125.9	Infin.	0.918	0.644	--
32.0	125.9	0	63.0	30.0	1	15	*	118	0.746	76.4	125.9	Infin.	0.912	0.641	--
33.0	125.9	0	63.0	30.0	1	15	*	118	0.734	75.2	125.9	Infin.	0.907	0.636	--
34.0	125.9	0	63.0	30.0	1	15	*	118	0.722	74.0	125.9	Infin.	0.900	0.632	--
35.0	125.9	0	63.0	30.0	1	15	*	118	0.711	72.9	125.9	Infin.	0.894	0.628	--
36.0	125.9	0	37.0	35.0	1	6	*	85	0.700	39.1	125.9	Infin.	0.887	0.623	--
37.0	125.9	0	37.0	35.0	1	6	*	85	0.690	38.5	125.9	Infin.	0.880	0.617	--
38.0	125.9	0	37.0	35.0	1	6	*	85	0.680	38.0	125.9	Infin.	0.872	0.612	--
39.0	125.9	0	37.0	35.0	1	6	*	85	0.671	37.5	125.9	Infin.	0.864	0.606	--
40.0	125.9	0	37.0	35.0	1	6	*	85	0.662	37.0	125.9	Infin.	0.855	0.600	--
41.0	108.2	0	27.0	40.0	1	15	*	69	0.654	30.3	108.2	Infin.	0.846	0.594	--
42.0	108.2	0	27.0	40.0	1	4	*	69	0.647	26.2	108.2	0.318	0.837	0.588	--
43.0	108.2	0	27.0	40.0	1	4	*	69	0.640	25.9	108.2	0.311	0.828	0.581	--
44.0	108.2	0	27.0	40.0	1	4	*	69	0.633	25.7	108.2	0.305	0.818	0.574	--
45.0	108.2	0	27.0	40.0	1	4	*	69	0.627	25.4	108.2	0.300	0.808	0.567	--
46.0	108.2	0	27.0	45.0	1	15	*	67	0.621	28.8	108.2	0.403	0.798	0.560	--
47.0	108.2	0	27.0	45.0	1	15	*	67	0.615	28.6	108.2	0.392	0.788	0.553	--
48.0	108.2	0	27.0	45.0	1	15	*	67	0.609	28.3	108.2	0.382	0.778	0.546	--
49.0	108.2	0	27.0	45.0	1	15	*	67	0.603	28.1	108.2	0.373	0.768	0.539	--
50.5	108.2	0	63.0	50.0	1	15	*	195	0.596	61.5	108.2	Infin.	0.755	0.530	--

* Indicates Assumed Value

GEOCON

Project Name : Multi-Family Residential Development, Desert Hot Spr
Project No : T3033-22-01
Boring : B-5

TECHNICAL ENGINEERING AND DESIGN GUIDES AS ADAPTED FROM THE US ARMY CORPS OF ENGINEERS, NO. 9
EVALUATION OF EARTHQUAKE-INDUCED SETTLEMENTS IN DRY SANDY SOILS
MAXIMUM CONSIDERED EARTHQUAKE

MCE EARTHQUAKE INFORMATION:

Earthquake Magnitude:	7.50
Peak Horiz. Acceleration (g):	1.080

Fig 4.1 Fig 4.2 Fig 4.4

Depth of Base of Strata (ft)	Thickness of Layer (ft)	Depth of Mid-point of Layer (ft)	Soil Unit Weight (pcf)	Overburden Pressure at Mid-point (tsf	Mean Effective Pressure at Mid-point (tsf)	Average Cyclic Shear Stress [Tav]	Field SPT [N]	Correction Factor [Cer]	Relative Density [Dr] (%)	Correction Factor [Cn]	Corrected [N1]60	rd Factor	Maximum Shear Mod. [Gmax] (tsf)	[yeff]*[Geff] [Gmax]	yeff Shear Strain	[yeff]*100%	Volumetric Strain M7.5 [E15} (%)	Number of Strain Cycles [Nc]	Corrected Vol. Strains [Ec]	Estimated Settlement [S] (inches)
1.0	1.0	0.5	100.7	0.03	0.02	0.018	10	1.25	73.3	1.7	19.1	1.0	155.2	1.13E-04	2.30E-04	0.023	2.43E-02	15.0	2.43E-02	Grading
2.0	1.0	1.5	100.7	0.08	0.05	0.053	10	1.25	73.3	1.7	19.1	1.0	268.9	1.91E-04	2.30E-04	0.023	2.43E-02	15.0	2.43E-02	Grading
3.0	1.0	2.5	100.7	0.13	0.08	0.088	10	1.25	73.3	1.7	19.1	1.0	347.1	2.42E-04	3.00E-03	0.300	3.17E-01	15.0	3.17E-01	Grading
4.0	1.0	3.5	100.7	0.18	0.12	0.124	10	1.25	73.3	1.7	19.1	1.0	410.8	2.81E-04	8.10E-04	0.081	8.55E-02	15.0	8.55E-02	Grading
5.0	1.0	4.5	113.4	0.23	0.15	0.161	9	1.25	66.5	1.7	17.2	1.0	452.8	3.26E-04	5.00E-03	0.500	5.99E-01	15.0	5.99E-01	Grading
6.0	1.0	5.5	113.4	0.29	0.19	0.200	9	1.25	66.5	1.7	17.2	1.0	505.6	3.57E-04	5.00E-03	0.500	5.99E-01	15.0	5.99E-01	0.14
7.0	1.0	6.5	113.4	0.34	0.23	0.240	9	1.25	66.5	1.7	17.2	1.0	553.4	3.84E-04	1.00E-03	0.100	1.20E-01	15.0	1.20E-01	0.03
8.0	1.0	7.5	109.7	0.40	0.27	0.278	11	1.25	68.9	1.6	20.0	1.0	627.5	3.86E-04	1.00E-03	0.100	9.99E-02	15.0	9.99E-02	0.02
9.0	1.0	8.5	109.7	0.45	0.30	0.316	11	1.25	68.9	1.5	18.8	1.0	655.0	4.13E-04	2.70E-03	0.270	2.91E-01	15.0	2.91E-01	0.07
10.0	1.0	9.5	121.5	0.51	0.34	0.356	18	1.25	84.8	1.4	28.9	1.0	803.4	3.73E-04	1.00E-03	0.100	6.42E-02	15.0	6.42E-02	0.02
11.0	1.0	10.5	121.5	0.57	0.38	0.397	18	1.25	84.8	1.4	27.3	1.0	834.0	3.95E-04	1.00E-03	0.100	6.87E-02	15.0	6.87E-02	0.02
12.5	1.5	11.8	121.5	0.65	0.43	0.449	18	1.25	84.8	1.3	25.7	1.0	869.3	4.19E-04	2.70E-03	0.270	2.00E-01	15.0	2.00E-01	0.07
13.0	0.5	12.8	125.9	0.71	0.48	0.490	19	1.25	83.5	1.2	27.3	1.0	928.6	4.22E-04	2.70E-03	0.270	1.85E-01	15.0	1.85E-01	0.02
14.0	1.0	13.5	125.9	0.76	0.51	0.522	19	1.25	83.5	1.2	25.9	1.0	941.9	4.38E-04	1.20E-03	0.120	8.79E-02	15.0	8.79E-02	0.02
15.0	1.0	14.5	116.6	0.82	0.55	0.562	29	1.25	98.2	1.1	40.7	1.0	1137.6	3.85E-04	7.10E-04	0.071	3.03E-02	15.0	3.03E-02	0.01
16.0	1.0	15.5	116.6	0.88	0.59	0.600	29	1.25	98.2	1.1	39.3	1.0	1164.2	3.96E-04	7.10E-04	0.071	3.16E-02	15.0	3.16E-02	0.01
17.0	1.0	16.5	116.6	0.93	0.63	0.638	29	1.25	98.2	1.1	38.1	1.0	1189.7	4.06E-04	1.20E-03	0.120	5.54E-02	15.0	5.54E-02	0.01
18.0	1.0	17.5	125.9	0.99	0.67	0.677	23	1.25	83.4	1.0	30.4	1.0	1138.6	4.44E-04	1.20E-03	0.120	7.27E-02	15.0	7.27E-02	0.02
19.0	1.0	18.5	125.9	1.06	0.71	0.718	23	1.25	83.4	1.0	29.5	1.0	1162.1	4.55E-04	1.20E-03	0.120	7.54E-02	15.0	7.54E-02	0.02
20.0	1.0	19.5	125.9	1.12	0.75	0.758	65	1.25	136.0	1.0	84.6	1.0	1700.1	3.24E-04	7.10E-04	0.071	1.26E-02	15.0	1.26E-02	0.00
21.0	1.0	20.5	125.9	1.18	0.79	0.797	65	1.25	136.0	0.9	82.3	1.0	1731.4	3.30E-04	7.10E-04	0.071	1.30E-02	15.0	1.30E-02	0.00
22.0	1.0	21.5	125.9	1.25	0.84	0.836	65	1.25	136.0	0.9	80.2	1.0	1761.5	3.36E-04	7.10E-04	0.071	1.34E-02	15.0	1.34E-02	0.00
23.0	1.0	22.5	125.9	1.31	0.88	0.875	65	1.25	136.0	0.9	78.2	0.9	1790.7	3.42E-04	7.10E-04	0.071	1.38E-02	15.0	1.38E-02	0.00
24.0	1.0	23.5	125.9	1.37	0.92	0.913	65	1.25	136.0	0.9	76.4	0.9	1819.0	3.47E-04	7.10E-04	0.071	1.42E-02	15.0	1.42E-02	0.00
25.0	1.0	24.5	125.9	1.44	0.96	0.951	41	1.25	100.8	0.9	50.3	0.9	1618.4	4.02E-04	1.20E-03	0.120	3.96E-02	15.0	3.96E-02	0.01
26.0	1.0	25.5	125.9	1.50	1.00	0.988	41	1.25	100.8	0.8	49.3	0.9	1641.7	4.07E-04	8.10E-04	0.081	2.75E-02	15.0	2.75E-02	0.01
27.0	1.0	26.5	125.9	1.56	1.05	1.024	41	1.25	100.8	0.8	48.3	0.9	1664.4	4.11E-04	8.10E-04	0.081	2.81E-02	15.0	2.81E-02	0.01
28.0	1.0	27.5	125.9	1.62	1.09	1.060	41	1.25	100.8	0.8	47.3	0.9	1686.5	4.16E-04	8.10E-04	0.081	2.88E-02	15.0	2.88E-02	0.01
29.0	1.0	28.5	125.9	1.69	1.13	1.095	41	1.25	100.8	0.8	46.4	0.9	1708.1	4.20E-04	8.10E-04	0.081	2.95E-02	15.0	2.95E-02	0.01
30.0	1.0	29.5	125.9	1.75	1.17	1.130	63	1.25	117.6	0.8	73.3	0.9	2025.7	3.62E-04	5.20E-04	0.052	1.09E-02	15.0	1.09E-02	0.00
31.0	1.0	30.5	125.9	1.81	1.21	1.164	63	1.25	117.6	0.8	72.0	0.9	2049.8	3.65E-04	5.20E-04	0.052	1.12E-02	15.0	1.12E-02	0.00
32.0	1.0	31.5	125.9	1.88	1.26	1.198	63	1.25	117.6	0.7	76.4	0.9	2126.1	3.58E-04	5.20E-04	0.052	1.04E-02	15.0	1.04E-02	0.00
33.0	1.0	32.5	125.9	1.94	1.30	1.231	63	1.25	117.6	0.7	75.2	0.9	2150.0	3.61E-04	5.20E-04	0.052	1.06E-02	15.0	1.06E-02	0.00
34.0	1.0	33.5	125.9	2.00	1.34	1.263	63	1.25	117.6	0.7	74.0	0.9	2173.4	3.63E-04	5.20E-04	0.052	1.08E-02	15.0	1.08E-02	0.00
35.0	1.0	34.5	125.9	2.06	1.38	1.295	63	1.25	117.6	0.7	72.9	0.9	2196.4	3.65E-04	5.20E-04	0.052	1.10E-02	15.0	1.10E-02	0.00
36.0	1.0	35.5	125.9	2.13	1.43	1.326	37	1.25	85.4	0.7	39.1	0.9	1811.1	4.49E-04	8.10E-04	0.081	3.63E-02	15.0	3.63E-02	0.01
37.0	1.0	36.5	125.9	2.19	1.47	1.357	37	1.25	85.4	0.7	38.5	0.9	1828.8	4.51E-04	8.10E-04	0.081	3.69E-02	15.0	3.69E-02	0.01
38.0	1.0	37.5	125.9	2.25	1.51	1.387	37	1.25	85.4	0.7	38.0	0.9	1846.2	4.53E-04	8.10E-04	0.081	3.75E-02	15.0	3.75E-02	0.01
39.0	1.0	38.5	125.9	2.32	1.55	1.416	37	1.25	85.4	0.7	37.5	0.9	1863.2	4.55E-04	8.10E-04	0.081	3.82E-02	15.0	3.82E-02	0.01
40.0	1.0	39.5	125.9	2.38	1.59	1.445	37	1.25	85.4	0.7	37.0	0.9	1880.0	4.57E-04	8.10E-04	0.081	3.88E-02	15.0	3.88E-02	0.01
41.0	1.0	40.5	108.2	2.44	1.63	1.470	27	1.25	69.5	0.7	30.3	0.8	1780.5	4.87E-04	8.10E-04	0.081	4.93E-02	15.0	4.93E-02	0.01
42.0	1.0	41.5	108.2	2.49	1.67	1.492	27	1.25	69.5	0.6	26.2	0.8	1715.7	5.09E-04	1.30E-03	0.130	9.40E-02	15.0	9.40E-02	0.02
43.0	1.0	42.5	108.2	2.55	1.71	1.514	27	1.25	69.5	0.6	25.9	0.8	1728.0	5.09E-04	1.30E-03	0.130	9.52E-02	15.0	9.52E-02	0.02
44.0	1.0	43.5	108.2	2.60	1.74	1.535	27	1.25	69.5	0.6	25.7	0.8	1740.2	5.09E-04	1.30E-03	0.130	9.64E-02	15.0	9.64E-02	0.02
45.0	1.0	44.5	108.2	2.65	1.78	1.555	27	1.25	69.5	0.6	25.4	0.8	1752.2	5.09E-04	1.30E-03	0.130	9.76E-02	15.0	9.76E-02	0.02
46.0	1.0	45.5	108.2	2.71	1.81	1.575	27	1.25	66.8	0.6	28.8	0.8	1846.8	4.86E-04	8.10E-04	0.081	5.22E-02	15.0	5.22E-02	0.01
47.0	1.0	46.5	108.2	2.76	1.85	1.595	27	1.25	66.8	0.6	28.6	0.8	1859.5	4.86E-04	8.10E-04	0.081	5.28E-02	15.0	5.28E-02	0.01
48.0	1.0	47.5	108.2	2.82	1.89	1.613	27	1.25	66.8	0.6	28.3	0.8	1872.1	4.85E-04	8.10E-04	0.081	5.33E-02	15.0	5.33E-02	0.01
49.0	1.0	48.5	108.2	2.87	1.92	1.632	27	1.25	66.8	0.6	28.1	0.8	1884.5	4.85E-04	8.10E-04	0.081	5.39E-02	15.0	5.39E-02	0.01
50.5	1.5	49.8	108.2	2.94	1.97	1.654	63	1.25	195.1	0.6	61.5	0.8	2476.0	3.72E-04	5.20E-04	0.052	1.35E-02	15.0	1.35E-02	0.00

TOTAL SETTLEMENT = **0.75**

Figure 4

APPENDIX A

APPENDIX A
FIELD EXPLORATION

Our field investigation was conducted on August 9 and 12, 2024, and included:

- Drilling of nine (9) exploratory borings (Borings B-1 through B-9) to depths ranging between approximately 16½ feet and 50½ feet, to observe the subsurface geological conditions at the site, collect relatively undisturbed in-situ and disturbed bulk samples for laboratory testing, and evaluate the depth to static groundwater, if encountered.
- Backfilling and performing percolation testing in one (1) geotechnical boring (Boring B-3), at a depth of approximately 10 feet, to provide a preliminary evaluation of the subsurface infiltration rate in areas where storm water infiltration systems are expected. The percolation test is identified as Test P-1. A bentonite plug was installed at 10 feet of depth, after backfilling and prior to performing percolation testing. Additional percolation testing should be performed when the exact location and depth of the proposed storm water infiltration system is known.

We collected bulk and relatively undisturbed samples from the borings by driving a 3-inch O. D., California Modified Sampler into the "undisturbed" soil mass with blows from a 140-pound hammer falling 30 inches. The California Modified Sampler was equipped with 1-inch high by $2^3/_8$-inch inside diameter brass sampler rings to facilitate removal and testing. Relatively undisturbed samples and bulk samples of disturbed soils were transported to our laboratory for testing.

The soil conditions encountered in the borings were visually examined, classified and logged in general accordance with the Unified Soil Classification System (USCS). Logs of the borings are presented on Figures A-1 through A-9. The logs depict the soil and geologic conditions encountered and the depth at which samples were obtained. The approximate locations of the borings are depicted on the *Geologic Map and Site Plan*, Figure 2.

Preliminary percolation testing was performed in accordance with *Riverside County Flood Control and Water Conservation District, LID BMP Manual, Appendix A.* The percolation test data is presented on Figure A-10.

PROJECT NO. T3082-22-01

BORING B-1

ELEV. (MSL.) 910 DATE COMPLETED 8/9/2024

EQUIPMENT CME-75 BY: KD

DEPTH IN FT	SAMPLE NO.	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0	B-1@0-5'	SP-SM	**ALLUVIAL SAND AND GRAVEL OF VALLEY AREAS (Qa)** Poorly-graded SAND with silt, medium dense, dry, pale brown, fine to coarse			
	B-1@2.5'			22	106.2	1.1
	B-1@5'			26	116.4	0.5
	B-1@7.5'			31	107.8	0.8
10	B-1@10'	SM	Silty SAND, medium dense, slightly moist, light brown, fine to medium	28	117.6	0.3
	B-1@15'			40	106.4	0.6
16		SP	Poorly-graded SAND, medium dense, slightly moist, pale brown, fine to coarse			
	B-1@20'			42	116.1	1.3
	B-1@25'		- Becomes dense	58	118.8	0.9

Total Depth = 26 1/2 feet
Groundwater not encountered
Penetration resistance for 140-lb hammer falling 30 inches by auto hammer
Backfilled with cuttings 8/9/2024

Figure A-1,
Log of Boring B-1, Page 1 of 1

T3082-22-01 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 B-2

ELEV. (MSL.) 908 DATE COMPLETED 8/9/2024

EQUIPMENT CME-75 BY: KD

DEPTH IN FT	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0				SP-SM	**ALLUVIAL SAND AND GRAVEL OF VALLEY AREAS (Qa)** Poorly-graded SAND with silt, dense, dry, pale brown, fine to coarse			
3					- Cobbles encountered			
5	B-2@5'					53	118.1	0.5
9				SM	Silty SAND, medium dense, slightly moist, light brown, fine to medium			
10	B-2@10'					31	104.0	0.7
15	B-2@15'				- Becomes dense	49	123.3	1.1

Total Depth = 16 1/2 feet
Groundwater not encountered
Penetration resistance for 140-lb hammer falling 30 inches by auto hammer
Backfilled with cuttings 8/9/2024

Figure A-2,
Log of Boring B-2, Page 1 of 1

T3082-22-01 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. T3082-22-01

BORING B-3

ELEV. (MSL.) 906 DATE COMPLETED 8/9/2024

EQUIPMENT CME-75 BY: KD

DEPTH IN FT	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0	B-3@0-5'			SP	**ALLUVIAL SAND AND GRAVEL OF VALLEY AREAS (Qa)** Poorly-graded SAND, loose, dry, pale gray, fine to medium with few coarse; coarsening downward			
2	B-3@2.5'					12	98.2	4.4
4								
6	B-3@5'				- Becomes slightly moist	16	110.6	1.3
8	B-3@7.5'				- Becomes light brown	15	107.0	0.9
					- Thin silt layer			
10	B-3@10'			SP-SM	Poorly-graded SAND with silt, medium dense, slightly moist, light brown; fine to coarse	22	118.5	0.7
12								
14								
16	B-3@15'				- Becomes pale brown	40	113.8	2.2
18								
20	B-3@20'				- Becomes dense; increase in fine sand	53	96.6	0.5
22								
24								
26	B-3@25'				- Becomes very dense; NO RECOVERY	50/6"		

Total Depth = 26 1/2 feet
Groundwater not encountered
Penetration resistance for 140-lb hammer falling 30 inches by auto hammer
Backfilled with cuttings to 10'
Percolation Test Equipment Set on 8/9/2024
Presaturated with 5 gallons of water
Backfilled with cuttings 8/12/2024

Figure A-3,
Log of Boring B-3, Page 1 of 1

T3082-22-01 BORING LOGS.GPJ

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 B-4

ELEV. (MSL.) **910** DATE COMPLETED **8/9/2024**

EQUIPMENT **CME-75** BY: **KD**

DEPTH IN FT	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0				SP	**ALLUVIAL SAND AND GRAVEL OF VALLEY AREAS (Qa)** Poorly-graded SAND, medium dense, dry, pale brown; fine to medium with few coarse sand; coarsening downward			
5	B-4@5'					22	103.0	3.1
10	B-4@10'				-Becomes dense	53	112.3	1.0
15	B-4@15'				NO RECOVERY	50/2"		

Total Depth = 16 1/2 feet
Groundwater not encountered
Penetration resistance for 140-lb hammer falling 30 inches by auto hammer
Backfilled with cuttings 8/9/2024

Figure A-4,
Log of Boring B-4, Page 1 of 1

T3082-22-01 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. T3082-22-01

BORING B-5

ELEV. (MSL.) 914   DATE COMPLETED 8/9/2024

EQUIPMENT CME-75   BY: KD

DEPTH IN FT	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0	B-5@0-5'			SP	**ALLUVIAL SAND AND GRAVEL OF VALLEY AREAS (Qa)** Poorly-graded SAND, loose, dry, pale brown; fine to coarse sand			
2	B-5@2.5'					10		
4	B-5@5'				- Fining downward; NO RECOVERY	14		
8	B-5@7.5'				- Thin silt lens	11		
10					- Increase in medium to coarse sand			
10	B-5@10'				- Becomes medium dense; NO RECOVERY	28		
12	B-5@12.5'			SP-SM	Poorly-graded SAND with Silt, medium dense, dry, pale brown; fine to medium sand with few coarse sand; few large gravel	19		
14	B-5@15'				- Increase in coarse sand	46	122.8	0.5
18	B-5@17.5'					23		
20	B-5@20'				- Becomes very dense; NO RECOVERY	77/9"		
22					- Large cobbes encountered			
24	B-5@25'				- Becomes dense	41		
30	B-5@30'				- Becomes very dense; large gravels (6-8') encountered	50/6"	154.4	0.4
32				SM	Silty SAND, very dense, slightly moist, light brown; fine to medium with few coarse sand			
34								

Figure A-5,
Log of Boring B-5, Page 1 of 2

T3082-22-01 BORING LOGS.GPJ

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

GEOCON

BORING B-5

ELEV. (MSL.) 914 DATE COMPLETED 8/9/2024

EQUIPMENT CME-75 BY: KD

DEPTH IN FT	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
36	B-5@35'			SP-SM	Poorly-graded SAND with silt, dense, slightly moist, pale brown; medium to coarse sand	37		
38								
40	B-5@40'			SM	Silty SAND, medium dense, slightly moist, light brown; fine sand	43	105.6	1.9
42				SP	Poorly-graded SAND, medium dense, slightly moist, pale brown; fine to coarse sand			
44								
46	B-5@45'			SM	Silty SAND, medium dense, slightly moist, pale brown; fine sand	27		
48								
50	B-5@50'					50/6"		

Total Depth = 50 1/2 feet
Groundwater not encountered
Penetration resistance for 140-lb hammer falling 30 inches by auto hammer
Backfilled with cuttings 8/9/2024

Figure A-5,
Log of Boring B-5, Page 2 of 2

T3082-22-01 BORING LOGS.GPJ

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

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

GEOCON

PROJECT NO. T3082-22-01

BORING B-6

ELEV. (MSL.) 910 DATE COMPLETED 8/9/2024

EQUIPMENT CME-75 BY: KD

DEPTH IN FT	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0				SP	**ALLUVIAL SAND AND GRAVEL OF VALLEY AREAS (Qa)** Poorly-graded SAND, medium dense, slightly moist, pale gray; fine to medium with few coarse sand			
5	B-6@5'				- NO RECOVERY	20		
10	B-6@10'					37	104.1	6.1
15	B-6@15'				- Becomes dense	72	124.1	0.5

Total Depth = 16 1/2 feet
Groundwater not encountered
Penetration resistance for 140-lb hammer falling 30 inches by auto hammer
Backfilled with cuttings 8/9/2024

Figure A-6,
Log of Boring B-6, Page 1 of 1

T3082-22-01 BORING LOGS.GPJ

SAMPLE SYMBOLS

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

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

GEOCON

PROJECT NO. T3082-22-01

BORING B-7

ELEV. (MSL.) 913 DATE COMPLETED 8/9/2024

EQUIPMENT CME-75 BY: KD

DEPTH IN FT	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0	B-7@0-5'			SP-SM	**ALLUVIAL SAND AND GRAVEL OF VALLEY AREAS (Qa)** Poorly-graded SAND with silt, medium dense, dry, pale brown; fine to coarse sand			
2	B-7@2.5'					38	115.0	1.8
4					- Rootlets			
6	B-7@5'					29	107.2	1.1
8	B-7@7.5'					38	100.3	0.7
					- Increase in coarse sand			
10	B-7@10'					43	109.2	0.4
12								
14								
16	B-7@15'				NO RECOVERY	50/2"		

REFUSAL, LARGE COBBLES ENCOUNTERED
Total Depth = 17 feet
Groundwater not encountered
Penetration resistance for 140-lb hammer falling 30 inches by auto hammer
Backfilled with cuttings 8/9/2024

Figure A-7,
Log of Boring B-7, Page 1 of 1

T3082-22-01 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 B-8

ELEV. (MSL.) **915** DATE COMPLETED **8/9/2024**

EQUIPMENT **CME-75** BY: **KD**

DEPTH IN FT	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0				SP	**ALLUVIAL SAND AND GRAVEL OF VALLEY AREAS (Qa)** Poorly-graded SAND, medium dense, dry, pale brown, fine to coarse sand			
2								
4								
	B-8@5'					33	114.1	0.6
6								
8								
10	B-8@10'				- Becomes very dense; NO RECOVERY	50/3"		
12								
14					- Large cobbles encountered			
	B-8@15'					43	111.8	0.7
16								

Total Depth = 16 1/2 feet
Groundwater not encountered
Penetration resistance for 140-lb hammer falling 30 inches by auto hammer
Backfilled with cuttings 8/9/2024

Figure A-8,
Log of Boring B-8, Page 1 of 1

T3082-22-01 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. T3082-22-01

BORING B-9

ELEV. (MSL.) 910 DATE COMPLETED 8/9/2024

EQUIPMENT CME-75 BY: KD

DEPTH IN FT	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0	B-9@0-5'			SP	**ALLUVIAL SAND AND GRAVEL OF VALLEY AREAS (Qa)** Poorly-graded SAND, medium dense, dry, pale brown; medium to coarse sand			
2	B-9@2.5'					19	113.3	0.9
4								
6	B-9@5'				- Increase in fine and medium sand	24	107.0	0.8
8	B-9@7.5'					44	107.0	0.4
10	B-9@10'					27	117.2	1.4
12								
14								
16	B-9@15'					41	137.7	0.9
18								
20	B-9@20'				- Becomes very dense; NO RECOVERY	50/3"		
22								
24								
26	B-9@25'				NO RECOVERY	50/3"		

Total Depth = 26 1/2 feet
Groundwater not encountered
Penetration resistance for 140-lb hammer falling 30 inches by auto hammer
Backfilled with cuttings 8/9/2024

Figure A-9,
Log of Boring B-9, Page 1 of 1

T3082-22-01 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

PERCOLATION TEST REPORT

Project Name:		DHS			**Project No.:**		T3082-22-01
Test Hole No.:		B-3			**Date Excavated:**		8/9/2024
Length of Test Pipe:			126.0	inches	**Soil Classification:**		SP
Height of Pipe above Ground:			6.0	inches	**Presoak Date:**		8/9/2024
Depth of Test Hole:			120.0	inches	**Perc Test Date:**		8/13/2024
Check for Sandy Soil Criteria Tested by:					**Percolation Tested by:**		KD

Water level measured from BOTTOM of hole

Sandy Soil Criteria Test

Trial No.	Time	Time Interval (min)	Total Elapsed Time (min)	Initial Water Head (in)	Final Water Head (in)	Δ in Water Level (in)	Percolation Rate (min/inch)
1							
2							

Soil Criteria: Sandy

Percolation Test

Reading No.	Time	Time Interval (min)	Total Elapsed Time (min)	Initial Water Head (in)	Final Water Head (in)	Δ in Water Level (in)	Percolation Rate (min/inch)
1	8:16 AM 8:26 AM	10	10	30.0	0.0	30.0	0.3
2	8:26 AM 8:36 AM	10	20	46.2	0.7	45.5	0.2
3	8:36 AM 8:46 AM	10	30	43.8	1.8	42.0	0.2
4	8:46 AM 8:56 AM	10	40	52.2	1.3	50.9	0.2
5	8:56 AM 9:06 AM	10	50	49.2	0.6	48.6	0.2
6	9:06 AM 9:16 AM	10	60	43.1	0.2	42.8	0.2

Infiltration Rate (in/hr):	21.7
Radius of test hole (in):	4
Average Head (in):	21.7

Figure A-10

APPENDIX B

APPENDIX B
LABORATORY TESTING

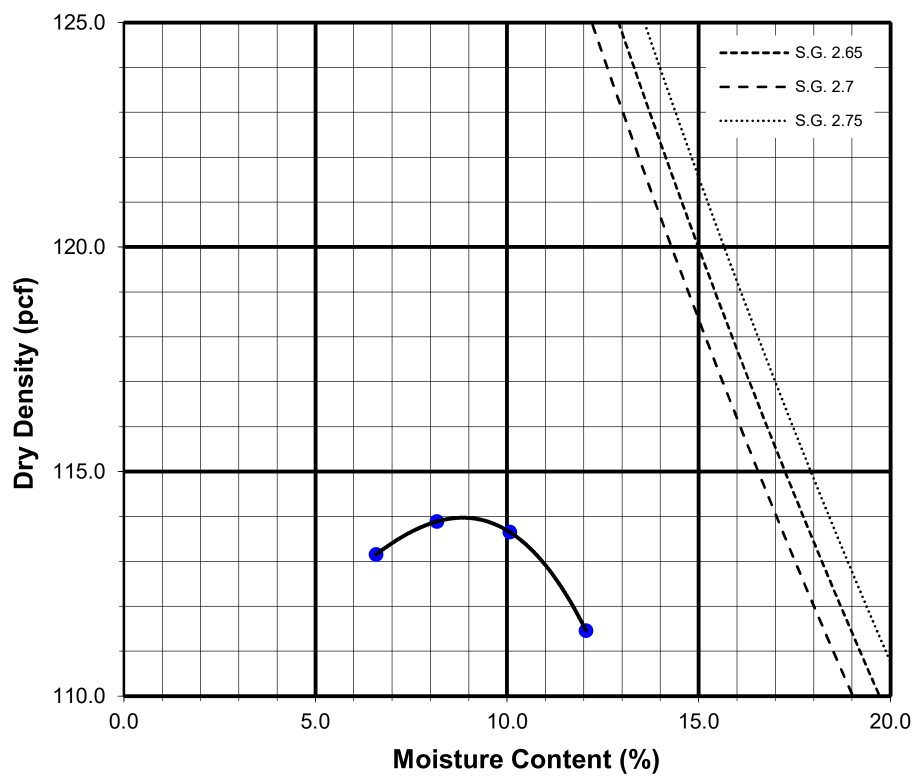
We performed laboratory tests in accordance with current, generally accepted test methods of ASTM International (ASTM) or other suggested procedures. We analyzed selected soil samples for in-situ density and moisture content, maximum dry density and optimum moisture content, corrosivity, expansion, grain size distribution, consolidation characteristics, and direct shear strength. The results of the laboratory tests are presented in Figures B-1 through B-23. The in-place dry density and moisture content of the samples tested are presented on the boring logs in Appendix A.

Sample No:

B1,B3@0-5	Poorly Graded SAND with Silt (SP-SM), light gray

TEST NO.		1	2	3	4	5	6
Wt. Compacted Soil + Mold	(g)	6113	6142	6139	6074		
Weight of Mold	(g)	4252	4252	4252	4252		
Net Weight of Soil	(g)	1861	1890	1887	1822		
Wet Weight of Soil + Cont.	(g)	895.9	764.4	824.6	937.5		
Dry Weight of Soil + Cont.	(g)	847.7	718.2	763.8	895.6		
Weight of Container	(g)	257.7	259.5	259.5	258.7		
Moisture Content	(%)	8.2	10.1	12.1	6.6		
Wet Density	(pcf)	123.2	125.1	124.9	120.6		
Dry Density	(pcf)	113.9	113.7	111.5	113.2		

Maximum Dry Density (pcf)	**114.0**	**Optimum Moisture Content (%)**	**9.0**



Preparation Method: A

COMPACTION CHARACTERISTICS USING MODIFIED EFFORT TEST RESULTS

ASTM D-1557

GEOCON

Checked by: ATS

Project No.: T3082-22-01

Multi-Family Residential Development
14320 Palm Drive
Desert Hot Springs

September 2024

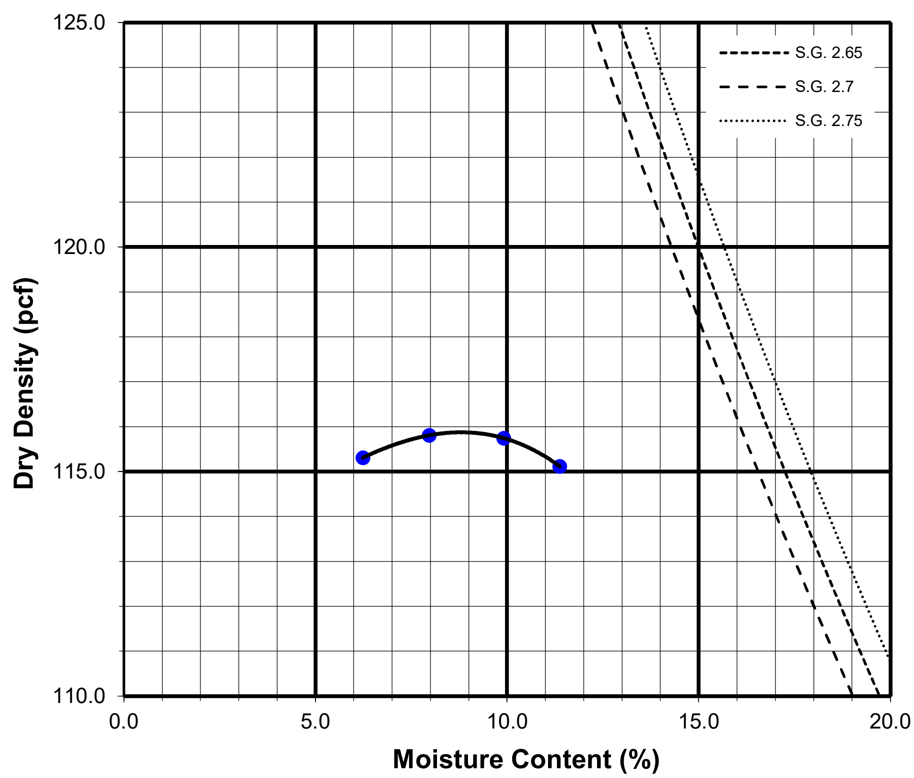
Figure B-1

Sample No:

B7,B9@0-5	Poorly Graded SAND with Silt (SP-SM), pale brown

TEST NO.		1	2	3	4	5	6
Wt. Compacted Soil + Mold	(g)	6103	6141	6174	6189		
Weight of Mold	(g)	4252	4252	4252	4252		
Net Weight of Soil	(g)	1850	1889	1922	1937		
Wet Weight of Soil + Cont.	(g)	835.7	868.0	900.2	886.0		
Dry Weight of Soil + Cont.	(g)	801.7	822.8	842.2	821.5		
Weight of Container	(g)	256.7	255.4	257.1	254.4		
Moisture Content	(%)	6.2	8.0	9.9	11.4		
Wet Density	(pcf)	122.5	125.0	127.2	128.2		
Dry Density	(pcf)	115.3	115.8	115.7	115.1		

Maximum Dry Density (pcf)	**116.0**	**Optimum Moisture Content (%)**	**9.0**



Preparation Method: A

GEOCON	**COMPACTION CHARACTERISTICS USING MODIFIED EFFORT TEST RESULTS** ASTM D-1557	Project No.: T3082-22-01
		Multi-Family Residential Development 14320 Palm Drive Desert Hot Springs
	Checked by: ATS	September 2024 Figure B-2

B1,B3@0-5

MOLDED SPECIMEN		BEFORE TEST	AFTER TEST
Specimen Diameter	(in.)	4.0	4.0
Specimen Height	(in.)	1.0	1.0
Wt. Comp. Soil + Mold	(gm)	599.6	615.1
Wt. of Mold	(gm)	196.8	196.8
Specific Gravity	(Assumed)	2.7	2.7
Wet Wt. of Soil + Cont.	(gm)	473.2	615.1
Dry Wt. of Soil + Cont.	(gm)	445.7	365.8
Wt. of Container	(gm)	173.2	196.8
Moisture Content	(%)	10.1	14.3
Wet Density	(pcf)	121.5	126.0
Dry Density	(pcf)	110.4	110.2
Void Ratio		0.5	0.5
Total Porosity		0.3	0.3
Pore Volume	(cc)	71.5	71.2
Degree of Saturation	(%) [S_{meas}]	52.1	73.6

Date	Time	Pressure (psi)	Elapsed Time (min)	Dial Readings (in.)
8/20/2024	10:00	1.0	0	0.3573
8/20/2024	10:10	1.0	10	0.3573
Add Distilled Water to the Specimen				
8/21/2024	10:00	1.0	1430	0.3561
8/21/2024	11:00	1.0	1490	0.3561

Expansion Index (EI meas) =	-1.2
Expansion Index (Report) =	**0**

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

* Reference: 2022 California Building Code, Section 1803.5.3
** Reference: 1997 Uniform Building Code, Table 18-I-B.

GEOCON

EXPANSION INDEX TEST RESULTS

ASTM D-4829

Checked by: ATS

Project No.: T3082-22-01

Multi-Family Residential Development
14320 Palm Drive
Desert Hot Springs

September 2024 Figure B-3

B7,B9@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)	605.0	626.3
Wt. of Mold	(gm)	201.6	201.6
Specific Gravity	(Assumed)	2.7	2.7
Wet Wt. of Soil + Cont.	(gm)	473.2	626.3
Dry Wt. of Soil + Cont.	(gm)	447.2	368.4
Wt. of Container	(gm)	173.2	201.6
Moisture Content	(%)	9.5	15.3
Wet Density	(pcf)	121.7	127.9
Dry Density	(pcf)	111.1	111.0
Void Ratio		0.5	0.5
Total Porosity		0.3	0.3
Pore Volume	(cc)	70.5	69.7
Degree of Saturation	(%) [S_{meas}]	50.0	80.7

Date	Time	Pressure (psi)	Elapsed Time (min)	Dial Readings (in.)
8/20/2024	10:00	1.0	0	0.3395
8/20/2024	10:10	1.0	10	0.3396
Add Distilled Water to the Specimen				
8/21/2024	10:00	1.0	1430	0.3356
8/21/2024	11:00	1.0	1490	0.3356

Expansion Index (EI meas) =	-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: 2022 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: ATS

Project No.: T3082-22-01

Multi-Family Residential Development
14320 Palm Drive
Desert Hot Springs

September 2024 Figure B-4

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

Sample No.	pH	Resistivity (ohm centimeters)
B1,B3@0-5	8.8	13000
B7,B9@0-5	9.2	15000

SUMMARY OF LABORATORY CHLORIDE CONTENT TEST RESULTS
AASHTO T291 ASTM C1218

Sample No.	Chloride Ion Content (%)
B1,B3@0-5	0.009
B7,B9@0-5	0.015

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

Sample No.	Water Soluble Sulfate (% SO_4)	Sulfate Exposure
B1,B3@0-5	0.001	S0
B7,B9@0-5	0.000	S0

GEOCON

CORROSIVITY TEST RESULTS

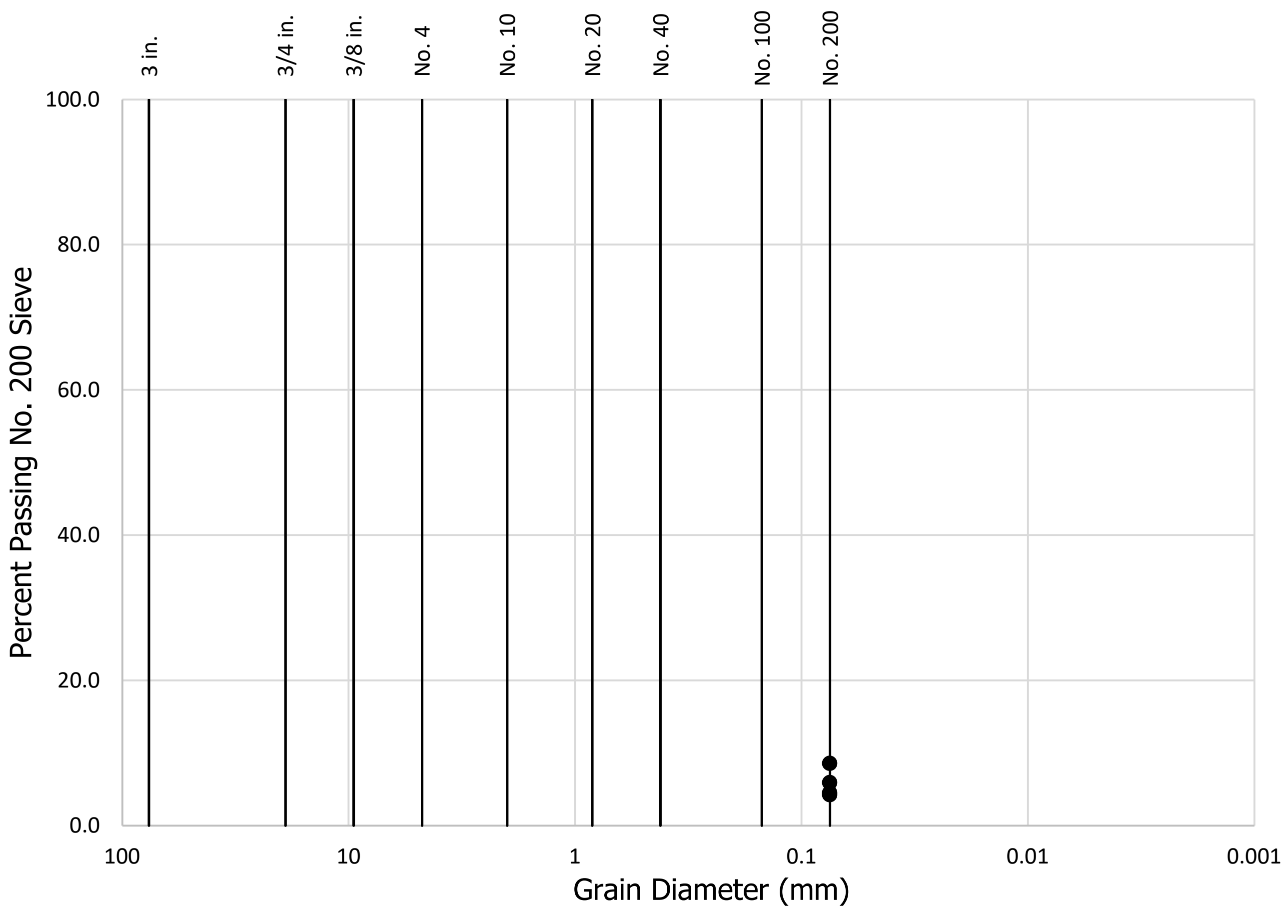
Checked by: ATS

Project No.: T3082-22-01

Multi-Family Residential Development
14320 Palm Drive
Desert Hot Springs

September 2024 Figure B-5

GRAVEL		SAND			SILT AND CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	



Sample No.	Percent Passing No. 200 Sieve
B5 @ 2.5'	4.5
B5 @ 7.5'	4.3
B5 @ 12.5'	8.6
B5 @ 17.5'	5.9

GEOCON

GRAIN SIZE ANALYSIS

ASTM D-1140

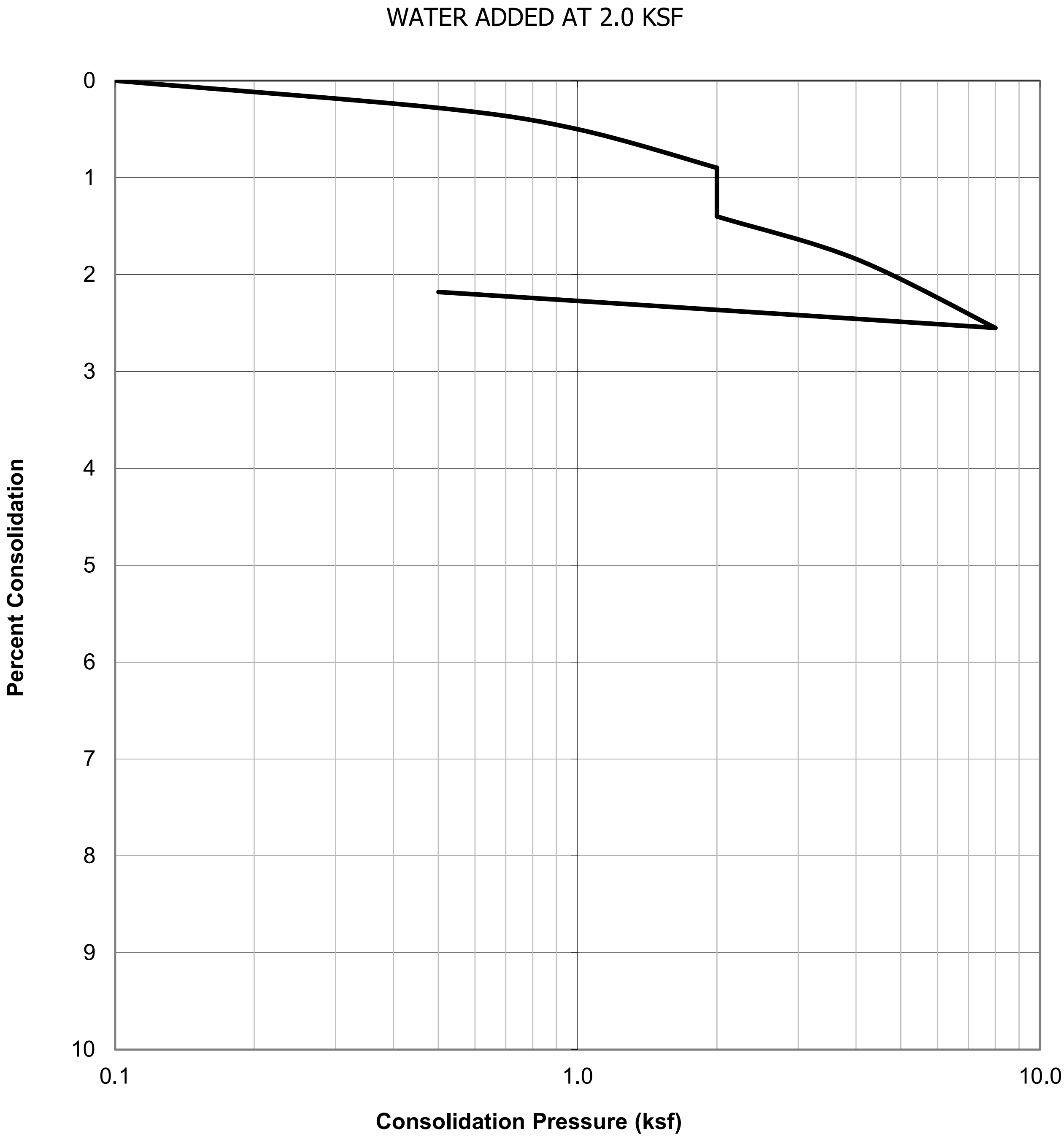
Checked by: ATS

Project No.: T3082-22-01

Multi-Family Residential Development
14320 Palm Drive
Desert Hot Springs

September 2024

Figure B-6



SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B1@2.5	Poorly Graded SAND with Silt (SP-SM), pale brown	110.2	1.1	14.5

GEOCON

CONSOLIDATION TEST RESULTS

ASTM D-2435

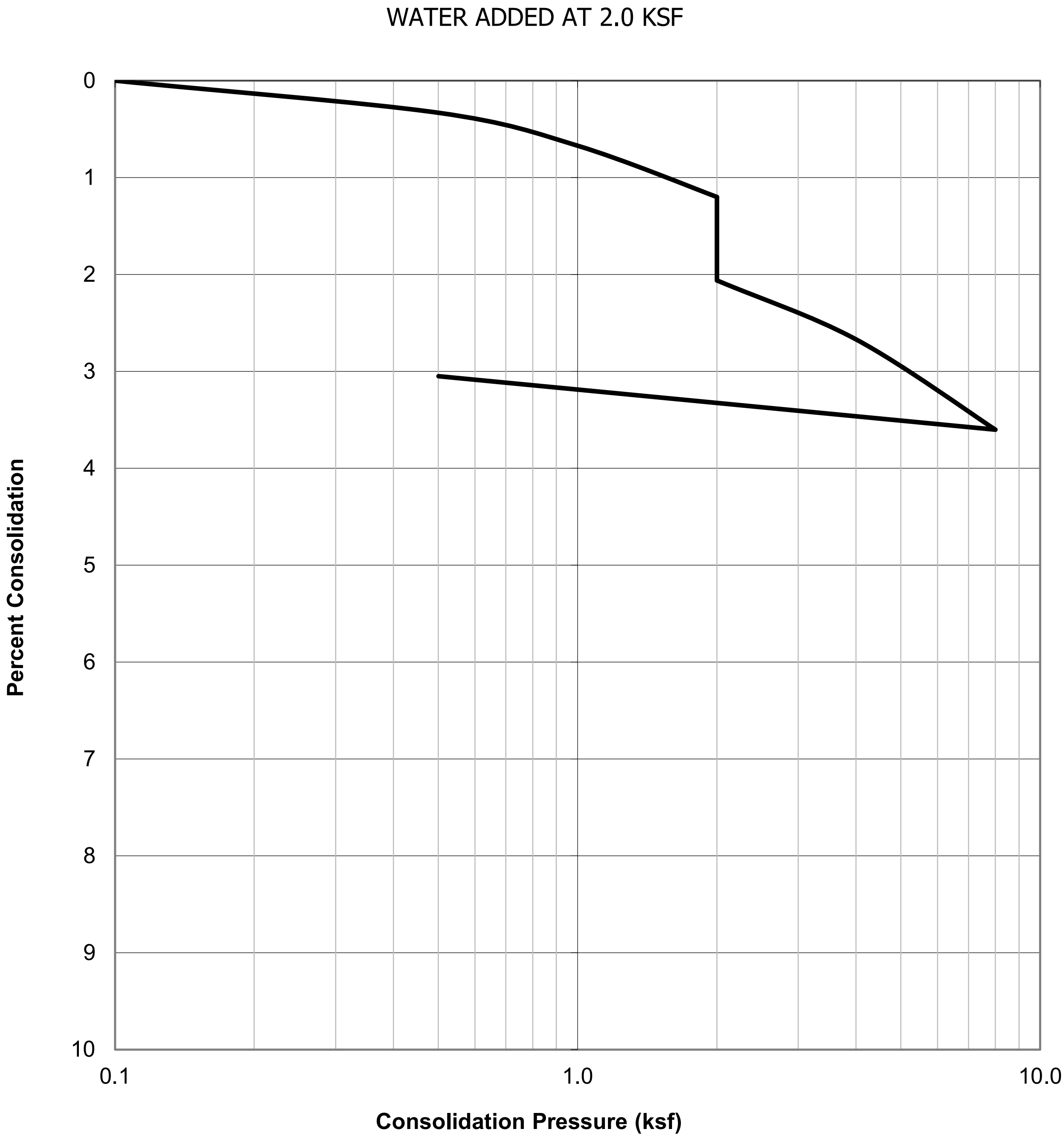
Checked by: ATS

Project No.: T3082-22-01

Multi-Family Residential Development
14320 Palm Drive
Desert Hot Springs

September 2024

Figure B-7



SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B1@5	Poorly Graded SAND with Silt (SP-SM), pale brown	112.8	0.5	14.2

GEOCON

CONSOLIDATION TEST RESULTS

ASTM D-2435

Checked by: ATS

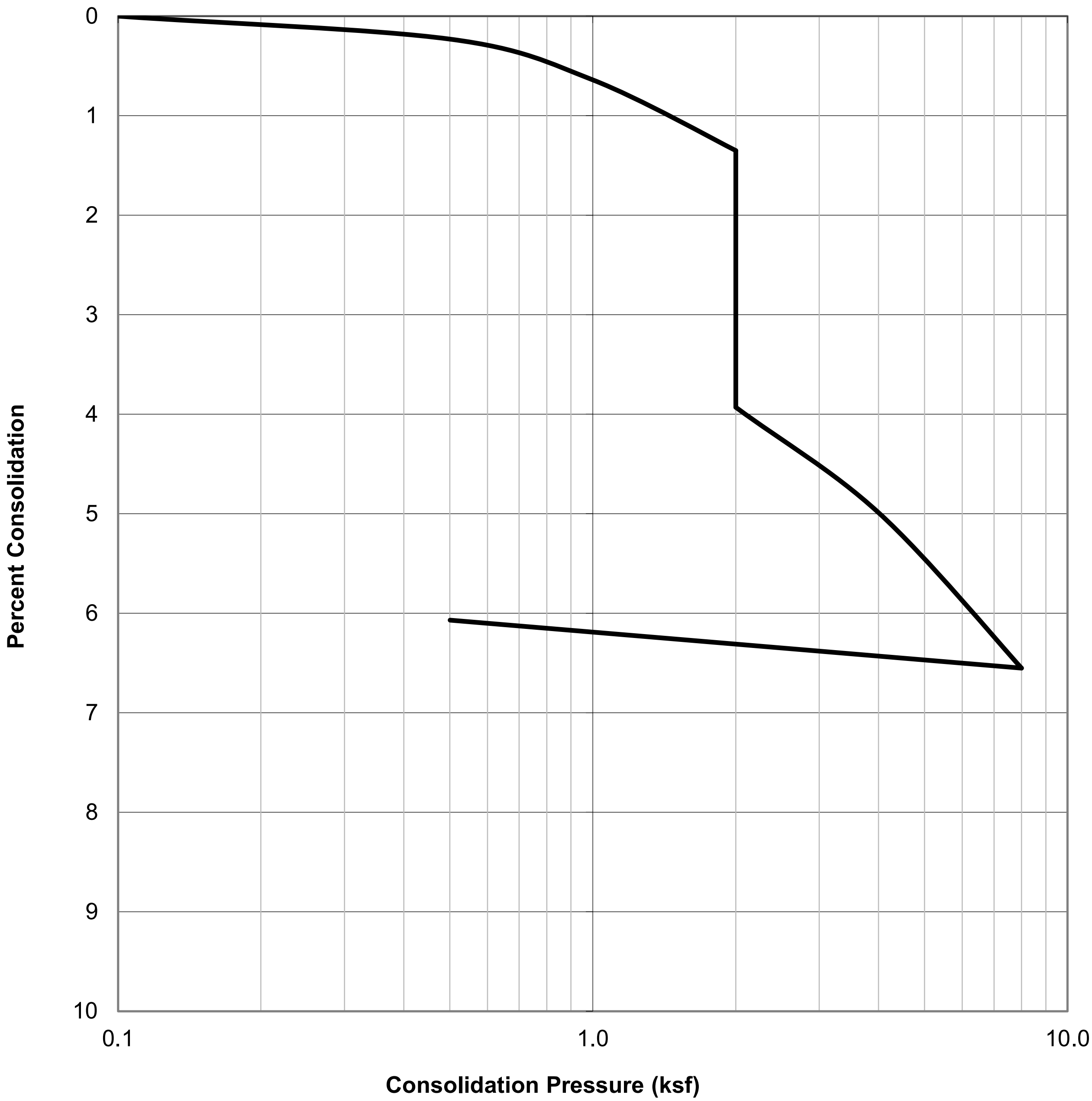
Project No.: T3082-22-01

Multi-Family Residential Development
14320 Palm Drive
Desert Hot Springs

September 2024

Figure B-8

WATER ADDED AT 2.0 KSF



SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B1@10	Silty SAND (SM), light brown	113.9	0.2	7.0

GEOCON

CONSOLIDATION TEST RESULTS

ASTM D-2435

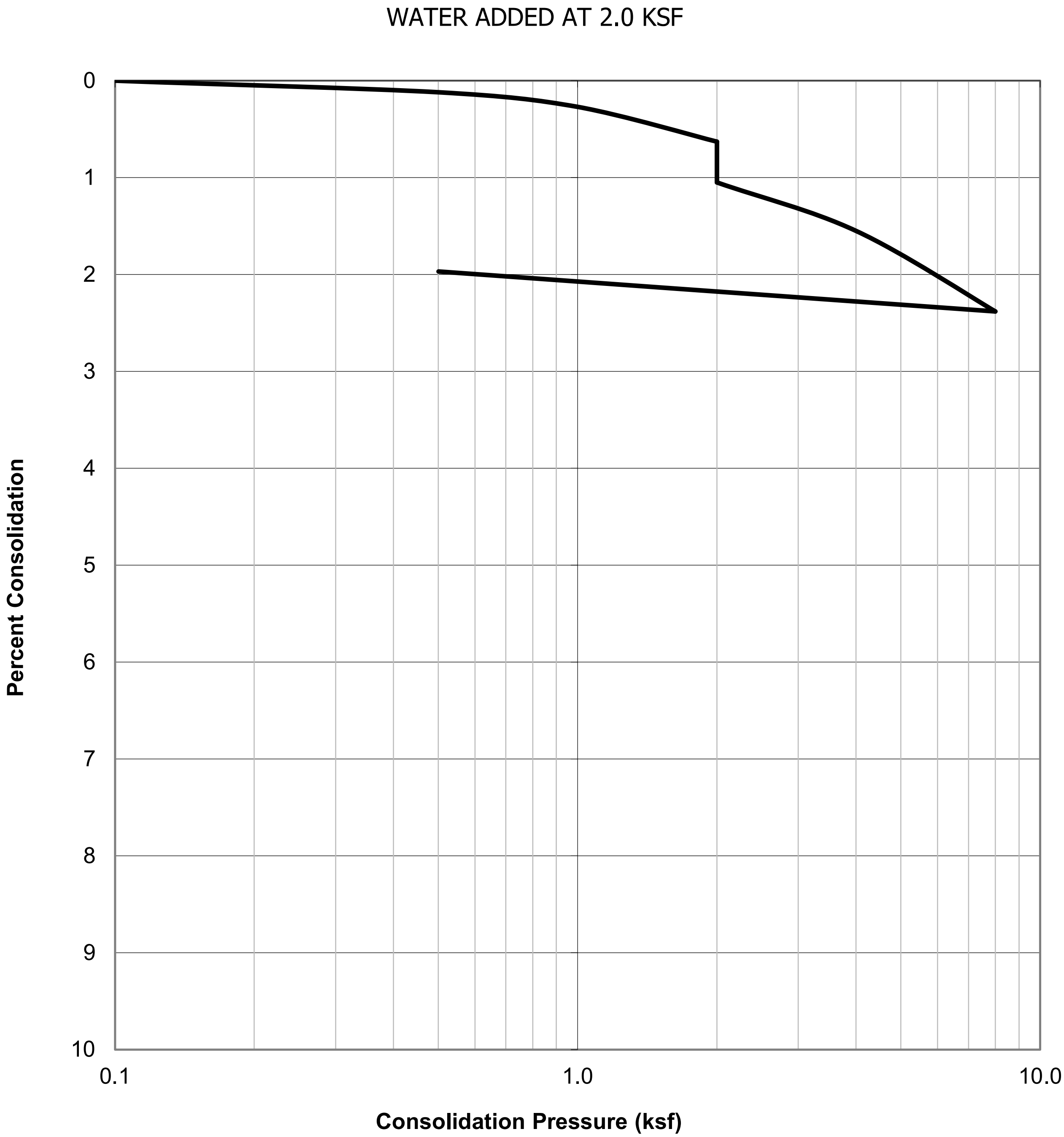
Checked by: ATS

Project No.: T3082-22-01

Multi-Family Residential Development
14320 Palm Drive
Desert Hot Springs

September 2024

Figure B-9



SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B1@20	Poorly Graded SAND (SP), pale brown	109.3	1.3	15.3

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CONSOLIDATION TEST RESULTS

ASTM D-2435

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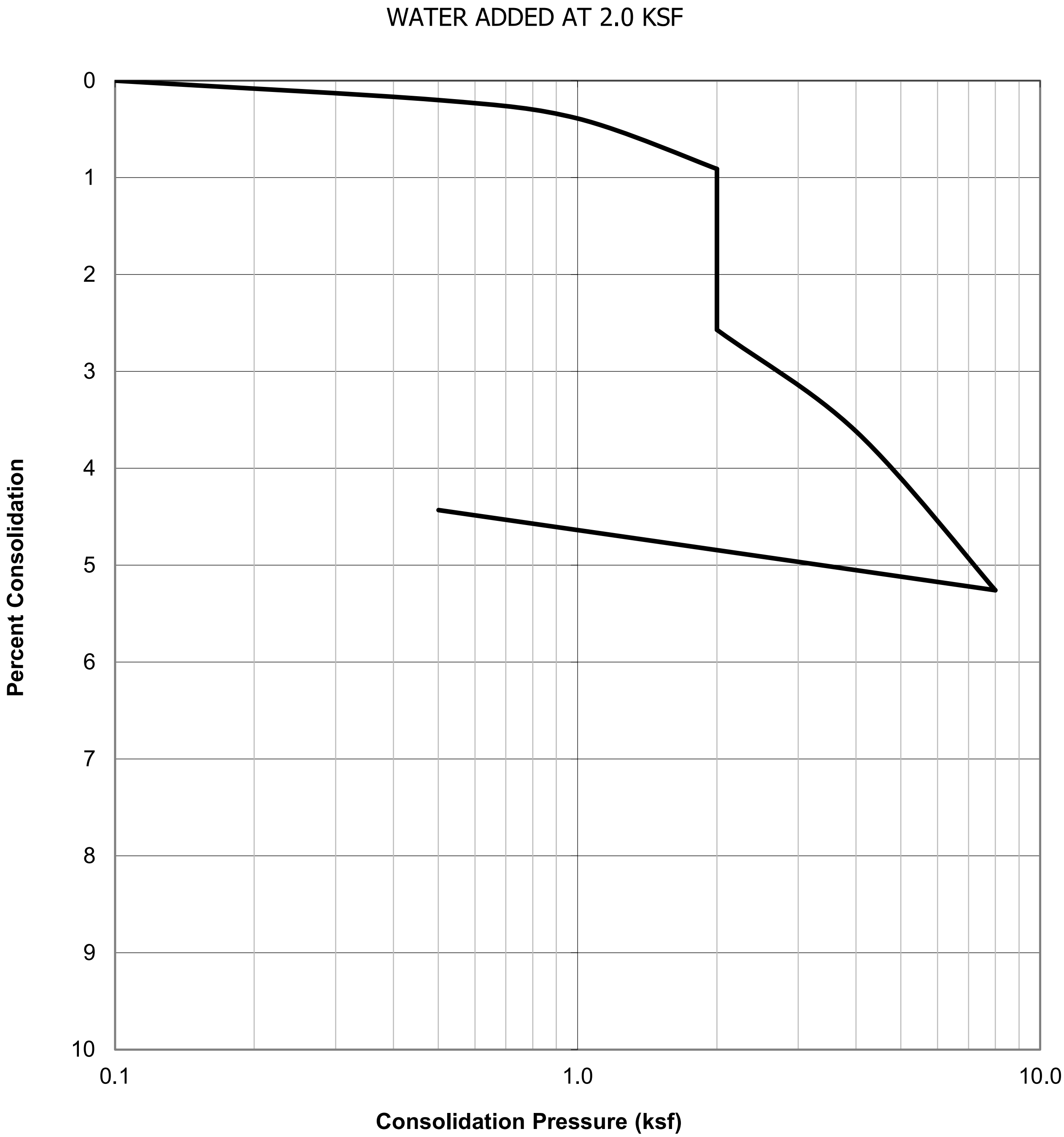
Figure B-10



SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B4@5	Poorly graded SAND (SP), pale brown	101.5	3.1	19.4

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CONSOLIDATION TEST RESULTS

ASTM D-2435

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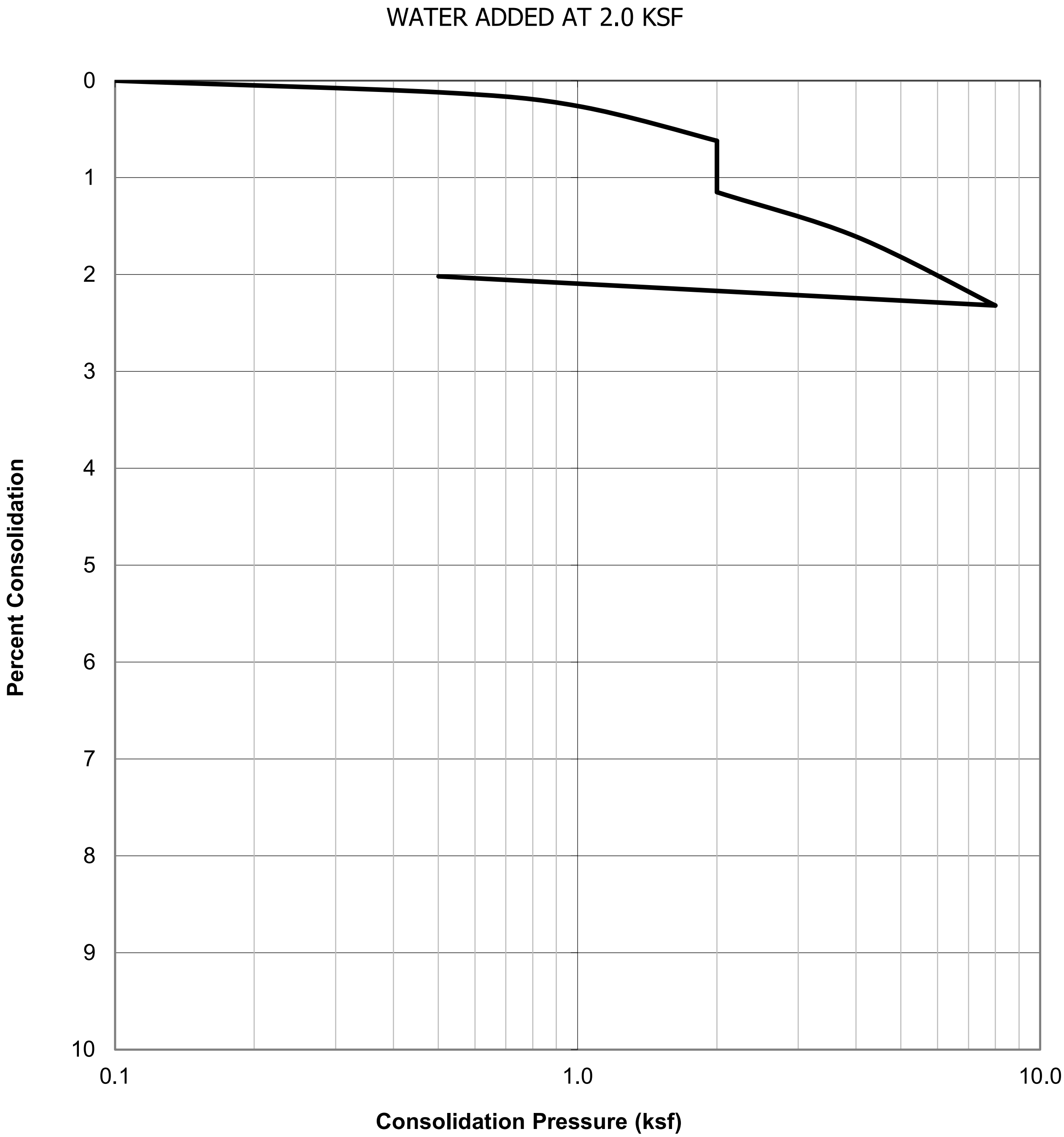
Figure B-11



SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B4@10	Poorly graded SAND (SP), pale brown	113.1	1.0	13.1

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CONSOLIDATION TEST RESULTS

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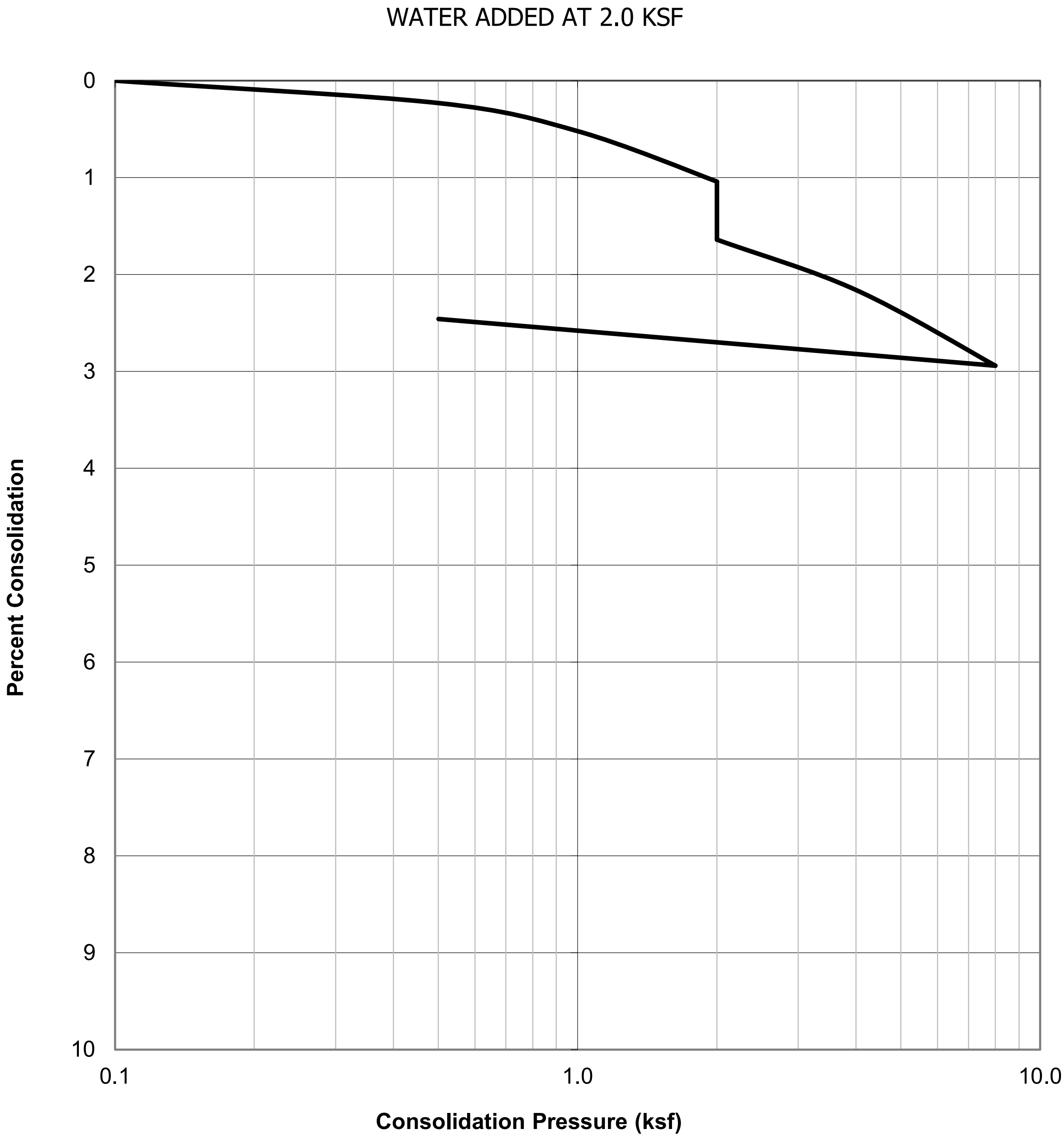
Figure B-12



SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B7@2.5	Poorly graded SAND with Silt (SP-SM), pale brown	111.7	1.8	14.8

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CONSOLIDATION TEST RESULTS

ASTM D-2435

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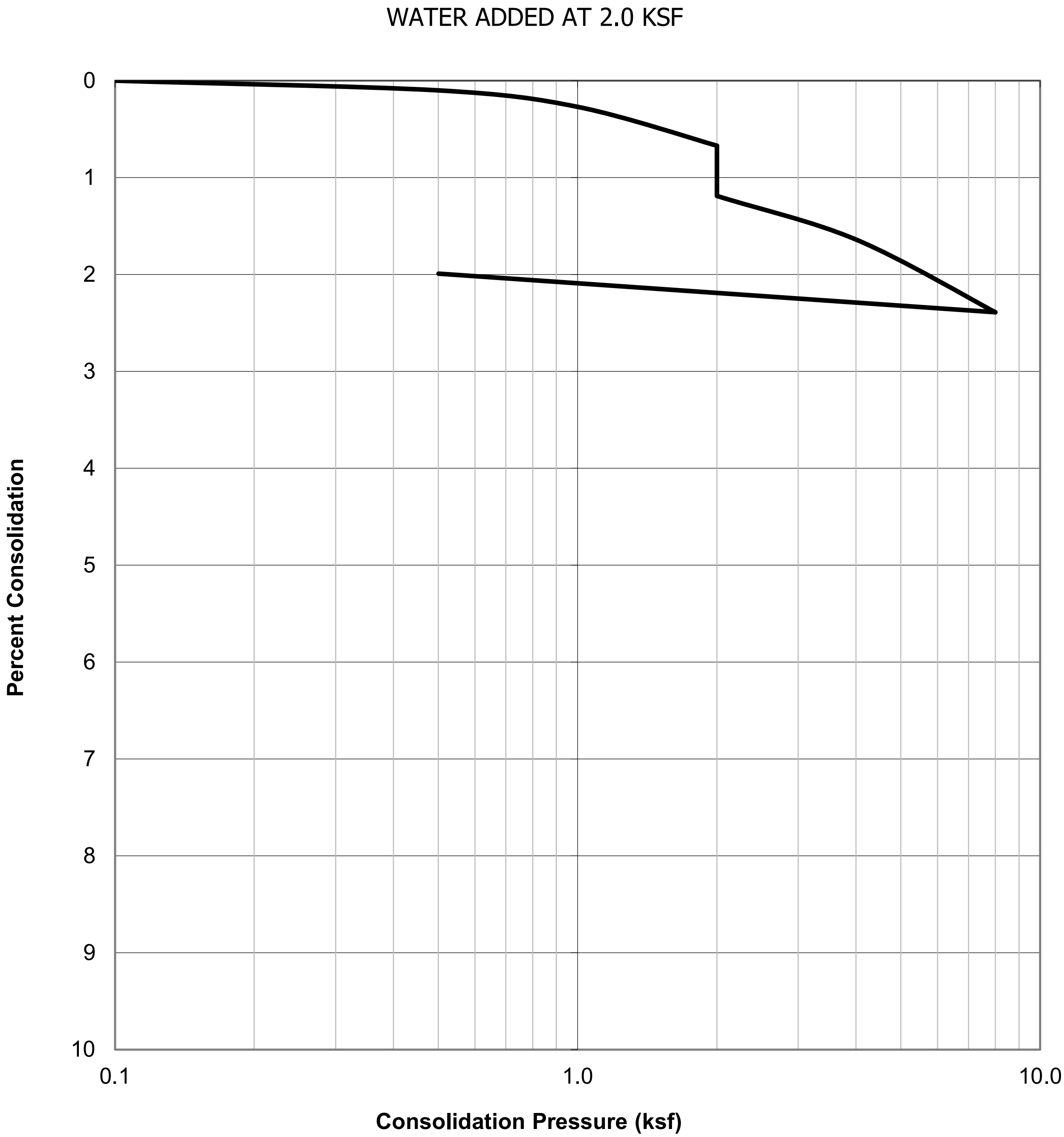
Figure B-13



SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B7@5	Poorly graded SAND with Silt (SP-SM), pale brown	110.0	1.1	15.9

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CONSOLIDATION TEST RESULTS

ASTM D-2435

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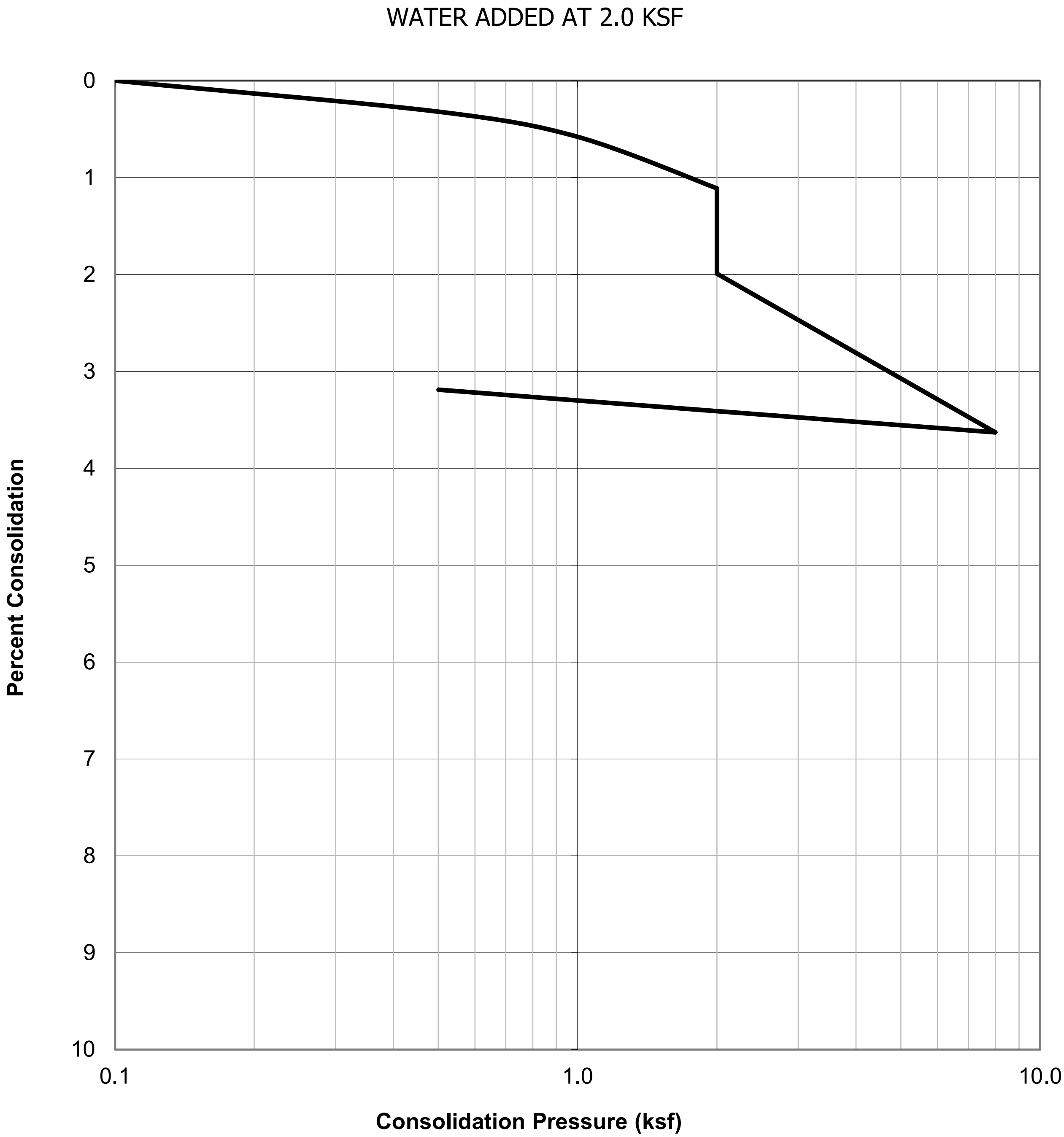
Figure B-14



SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B9@2.5	Poorly graded SAND (SP), pale brown	112.7	0.9	14.3

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CONSOLIDATION TEST RESULTS

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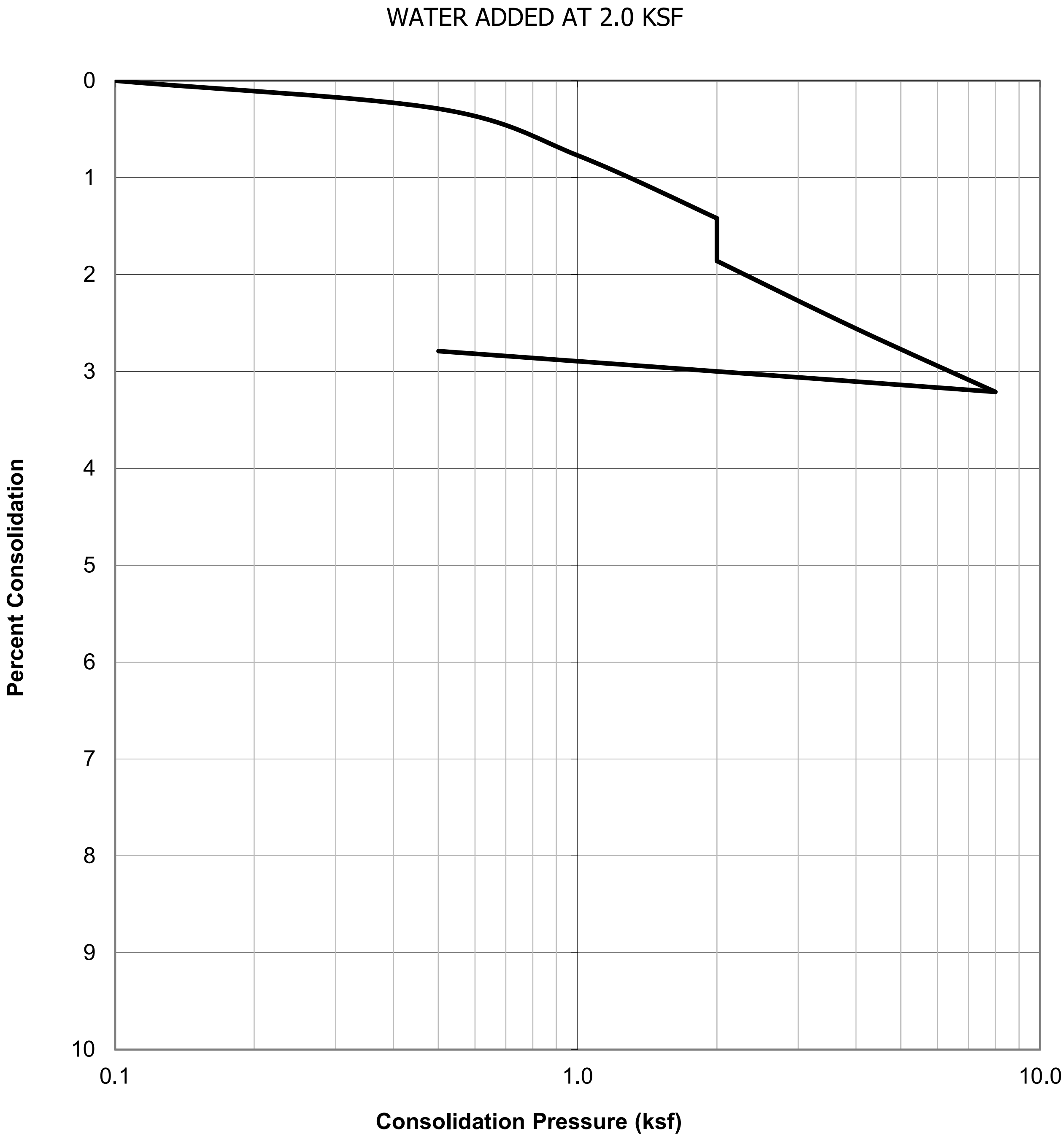
Figure B-15



SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B9@5	Poorly graded SAND (SP), pale brown	114.3	0.8	14.5

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CONSOLIDATION TEST RESULTS

ASTM D-2435

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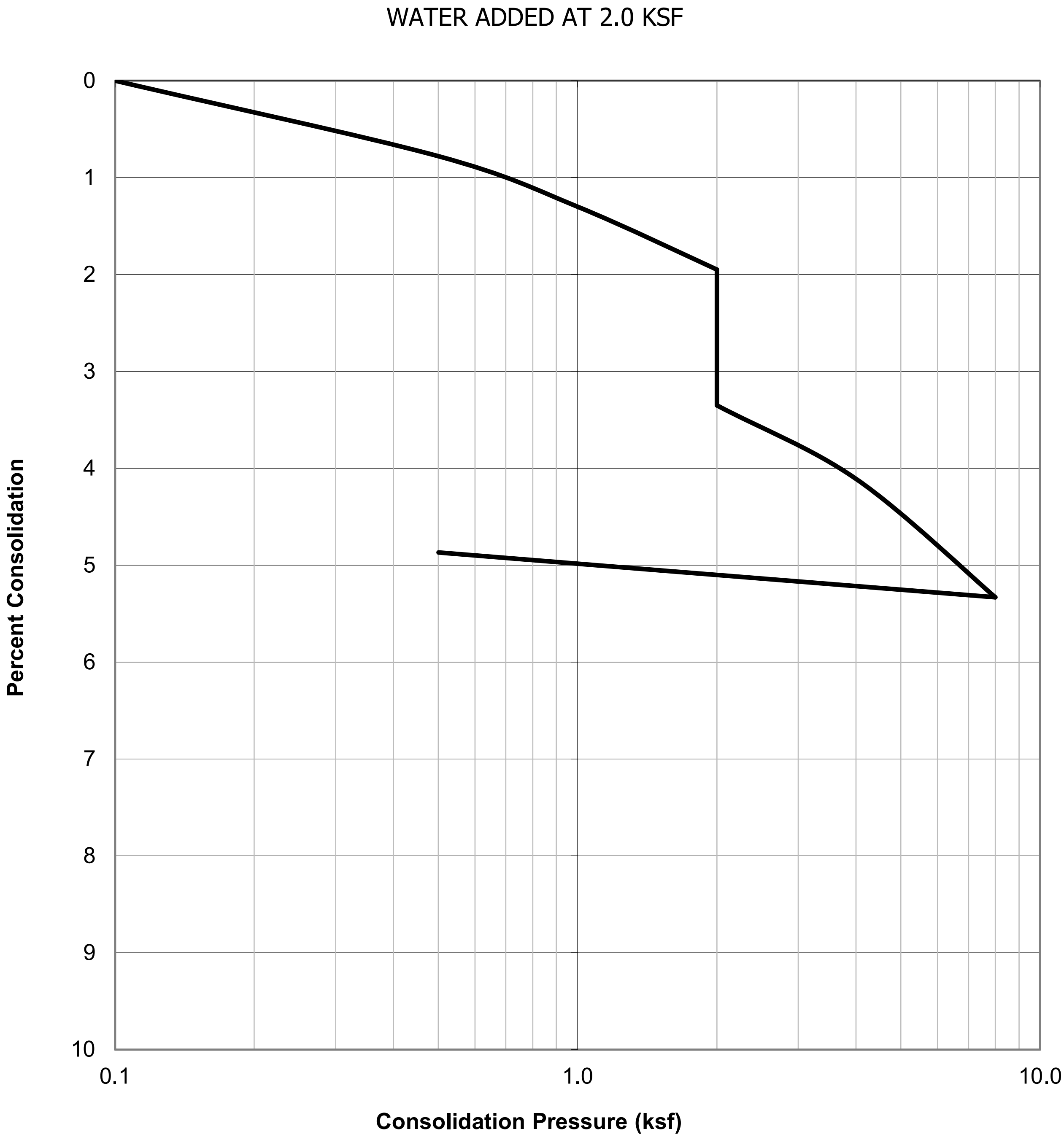
Figure B-16



SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B9@10	Poorly graded SAND (SP), pale brown	114.7	1.4	12.9

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CONSOLIDATION TEST RESULTS

ASTM D-2435

Checked by: ATS

Project No.: T3082-22-01

Multi-Family Residential Development
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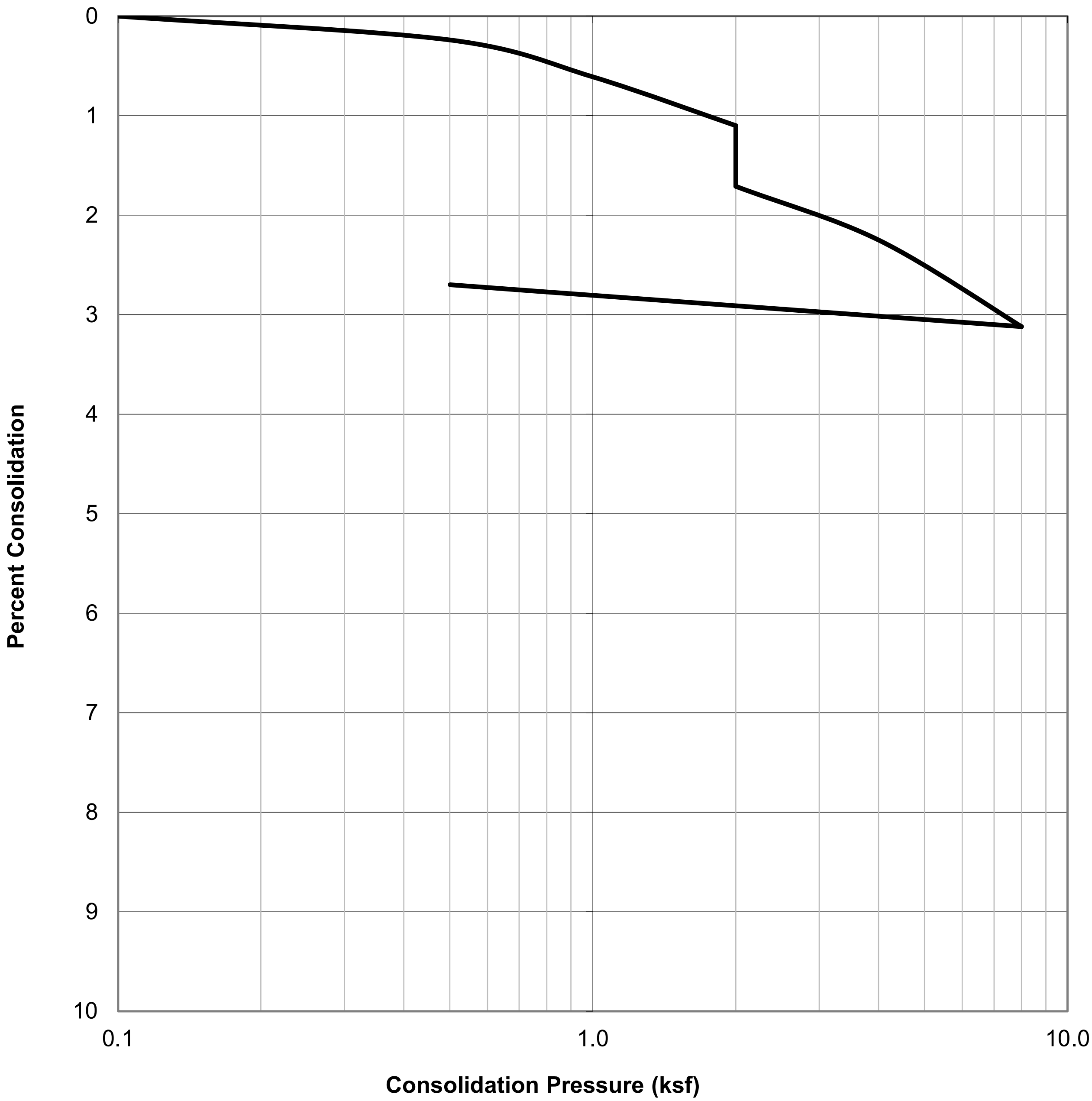
Figure B-17

WATER ADDED AT 2.0 KSF



SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B9@15	Poorly graded SAND (SP), pale brown	107.9	0.9	25.1

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CONSOLIDATION TEST RESULTS

ASTM D-2435

Checked by: ATS

Project No.: T3082-22-01

Multi-Family Residential Development
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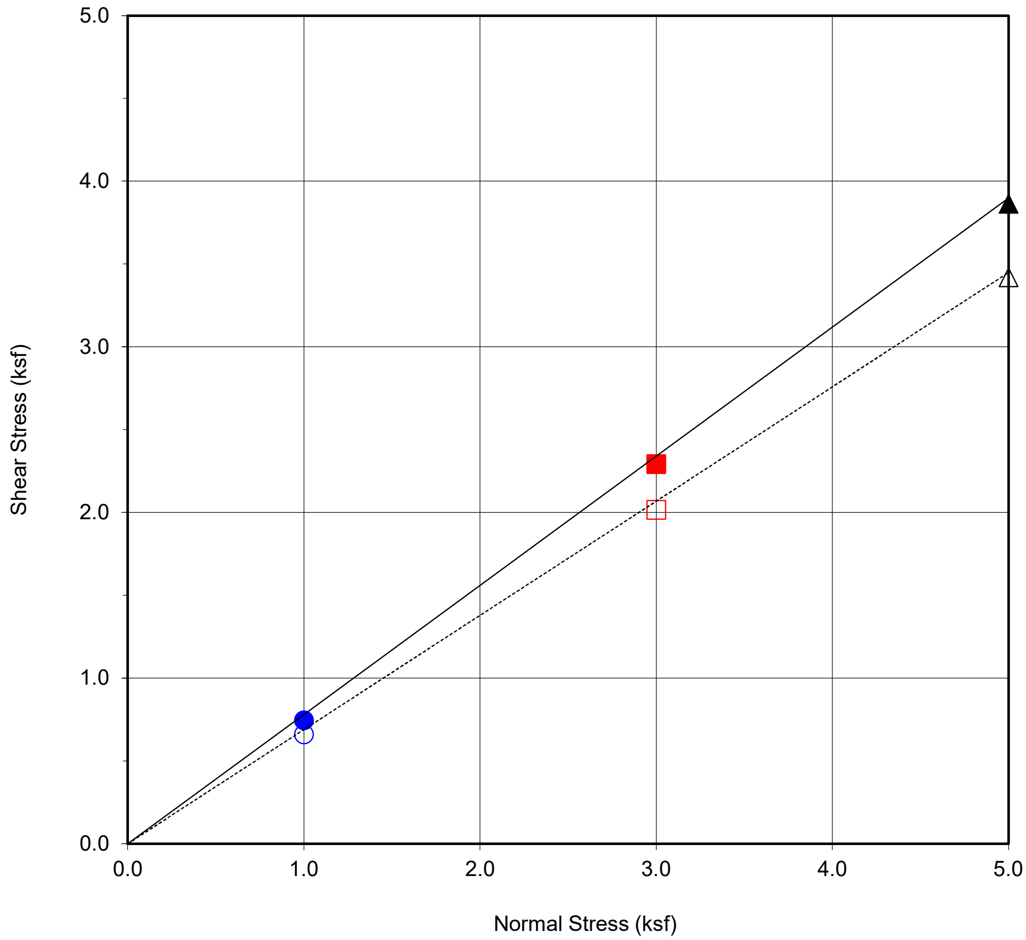
Figure B-18



Boring No.	**B1,B3**
Sample No.	**B1,B3@0-5**
Depth (ft)	**0-5**
Sample Type:	Bulk

Soil Identification:		
Poorly Graded SAND with Silt (SP-SM), light gray		
Strength Parameters		
	C (psf)	ϕ (^{o})
Peak	0	38
Ultimate	0	35

Normal Stress (kip/ft²)	1	3	5
Peak Shear Stress (kip/ft²)	● 0.74	■ 2.29	▲ 3.86
Shear Stress @ End of Test (ksf)	○ 0.66	□ 2.02	△ 3.42
Deformation Rate (in./min.)	0.05	0.05	0.05
Initial Sample Height (in.)	1.0	1.0	1.0
Ring Inside Diameter (in.)	2.375	2.375	2.375
Initial Moisture Content (%)	9.1	8.9	9.1
Initial Dry Density (pcf)	103.0	102.9	103.0
Initial Degree of Saturation (%)	38.6	37.7	38.6
Soil Height Before Shearing (in.)	1.2	1.2	1.2
Final Moisture Content (%)	12.6	7.8	13.2

GEOCON

DIRECT SHEAR TEST RESULTS

Consolidated Drained ASTM D-3080

Checked by: ATS

Project No.: T3082-22-01

Multi-Family Residential Development
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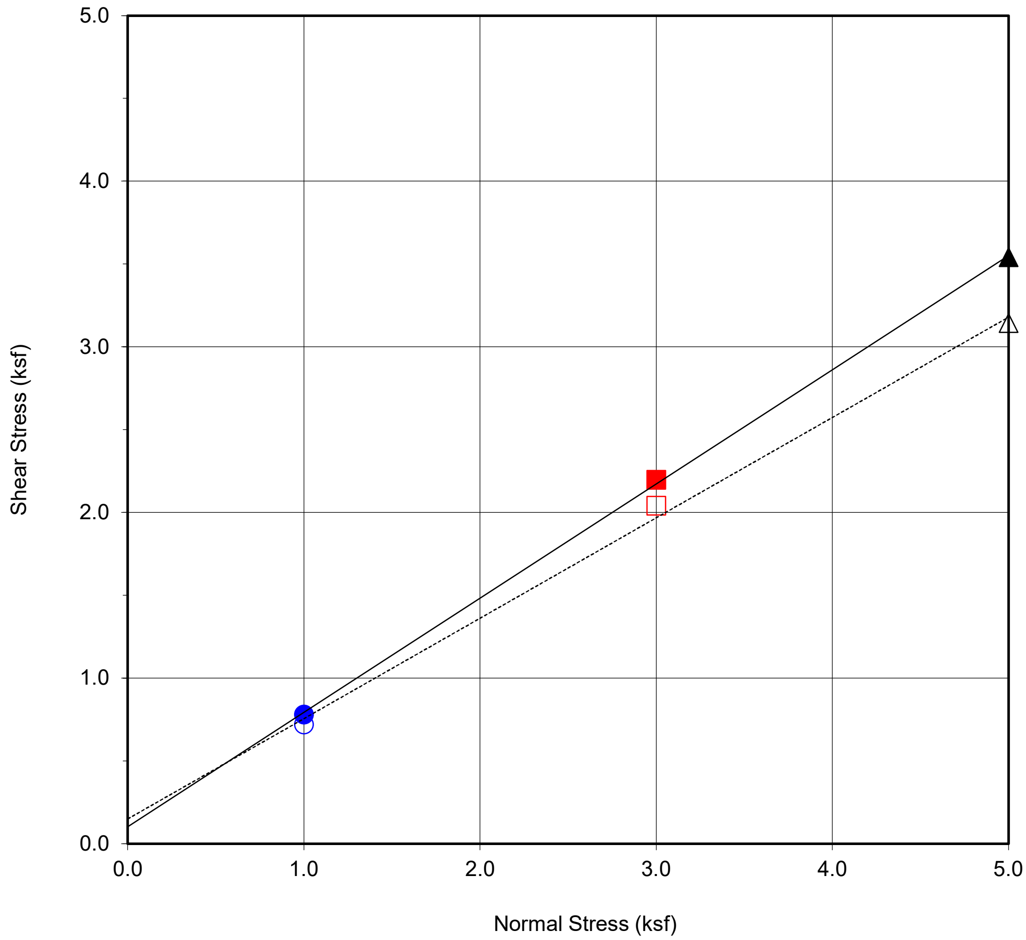
Figure B-19



Boring No.	**B3**
Sample No.	**B3@2.5**
Depth (ft)	**2.5**
Sample Type:	Ring

Soil Identification:		
Poorly Graded SAND (SP), pale gray		
Strength Parameters		
	C (psf)	ϕ (°)
Peak	102	35
Ultimate	150	31

Normal Stress (kip/ft²)	1	3	5
Peak Shear Stress (kip/ft²)	● 0.78	■ 2.20	▲ 3.54
Shear Stress @ End of Test (ksf)	○ 0.72	□ 2.04	△ 3.14
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	4.7	4.2
Initial Dry Density (pcf)	98.7	103.0	93.7
Initial Degree of Saturation (%)	17.3	20.0	14.3
Soil Height Before Shearing (in.)	1.2	1.2	1.2
Final Moisture Content (%)	18.7	18.7	15.8

GEOCON

DIRECT SHEAR TEST RESULTS

Consolidated Drained ASTM D-3080

Checked by: ATS

Project No.: T3082-22-01

Multi-Family Residential Development
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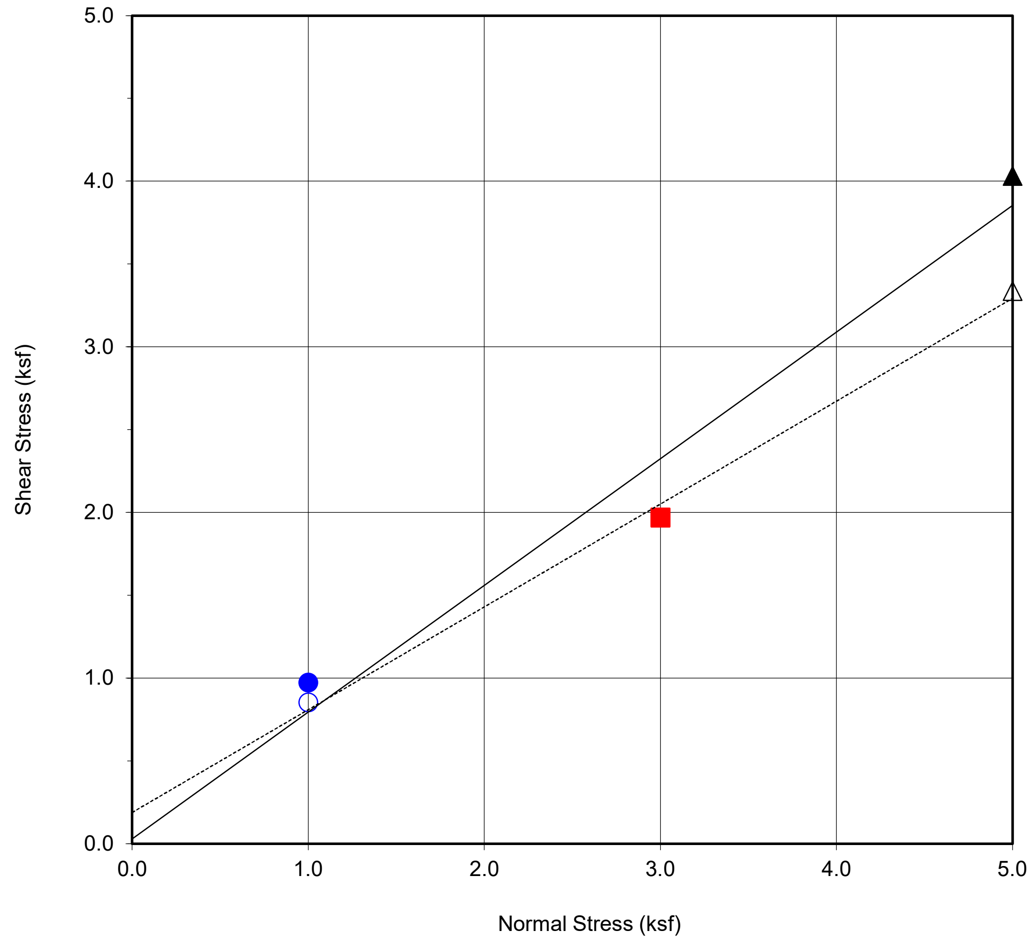
Figure B-20



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

Soil Identification:		
Poorly Graded SAND (SP), pale gray		
Strength Parameters		
	C (psf)	ϕ (°)
Peak	29	37
Ultimate	189	32

Normal Stress (kip/ft²)	1	3	5
Peak Shear Stress (kip/ft²)	● 0.97	■ 1.97	▲ 4.03
Shear Stress @ End of Test (ksf)	○ 0.85	□ 1.97	△ 3.34
Deformation Rate (in./min.)	0.05	0.05	0.05
Initial Sample Height (in.)	1.0	1.0	1.0
Ring Inside Diameter (in.)	2.375	2.375	2.375
Initial Moisture Content (%)	6.1	6.0	7.5
Initial Dry Density (pcf)	105.4	102.4	101.0
Initial Degree of Saturation (%)	27.4	25.0	30.4
Soil Height Before Shearing (in.)	1.2	1.2	1.2
Final Moisture Content (%)	19.2	19.5	19.6

GEOCON

DIRECT SHEAR TEST RESULTS

Consolidated Drained ASTM D-3080

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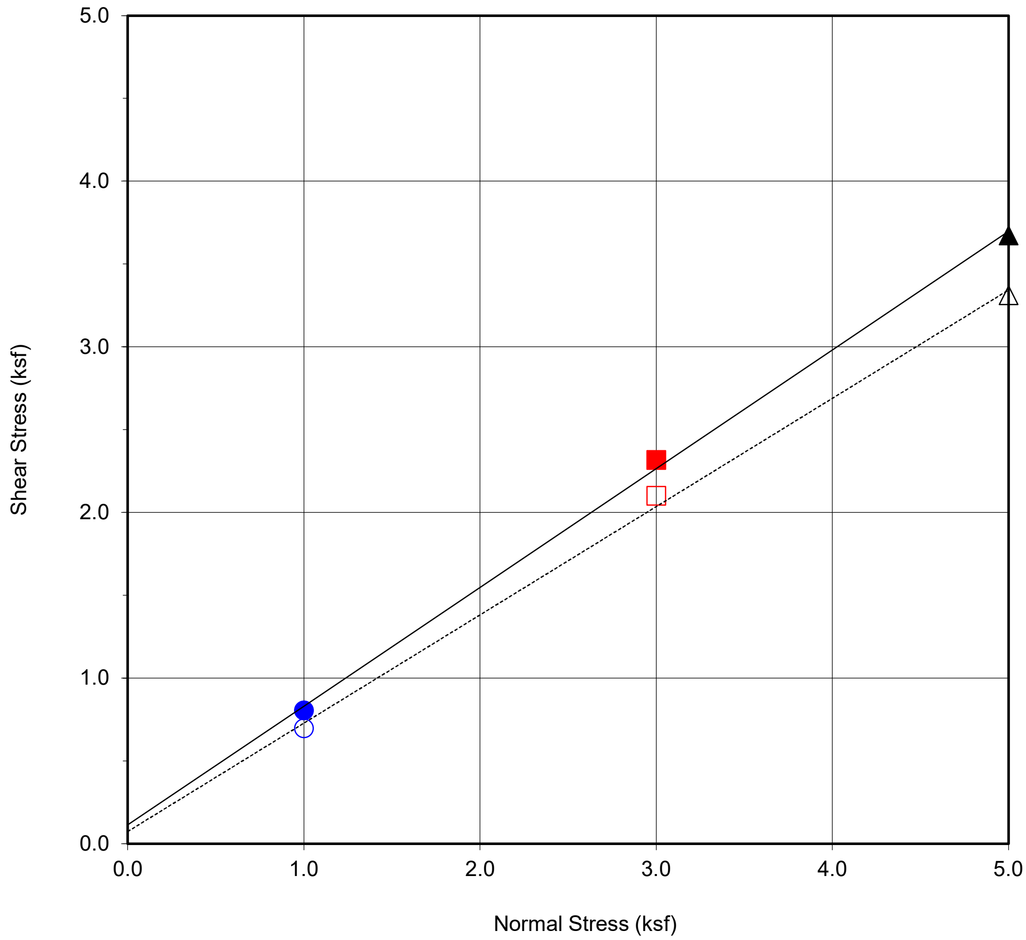
Figure B-21



Boring No.	**B7,B9**
Sample No.	**B7,B9@0-5**
Depth (ft)	**0-5**
Sample Type:	Bulk

Normal Stress (kip/ft²)	1	3	5
Peak Shear Stress (kip/ft²)	● 0.80	■ 2.32	▲ 3.67
Shear Stress @ End of Test (ksf)	○ 0.70	□ 2.10	△ 3.31
Deformation Rate (in./min.)	0.05	0.05	0.05
Initial Sample Height (in.)	1.0	1.0	1.0
Ring Inside Diameter (in.)	2.375	2.375	2.375
Initial Moisture Content (%)	8.9	9.0	9.0
Initial Dry Density (pcf)	104.0	104.0	104.0
Initial Degree of Saturation (%)	38.8	39.2	39.4
Soil Height Before Shearing (in.)	1.2	1.2	1.2
Final Moisture Content (%)	12.4	14.4	13.5

Soil Identification:

Poorly Graded SAND with Silt (SP-SM), pale brown

Strength Parameters

	C (psf)	ϕ (°)
Peak	113	36
Ultimate	74	33

GEOCON

DIRECT SHEAR TEST RESULTS

Consolidated Drained ASTM D-3080

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Multi-Family Residential Development
14320 Palm Drive
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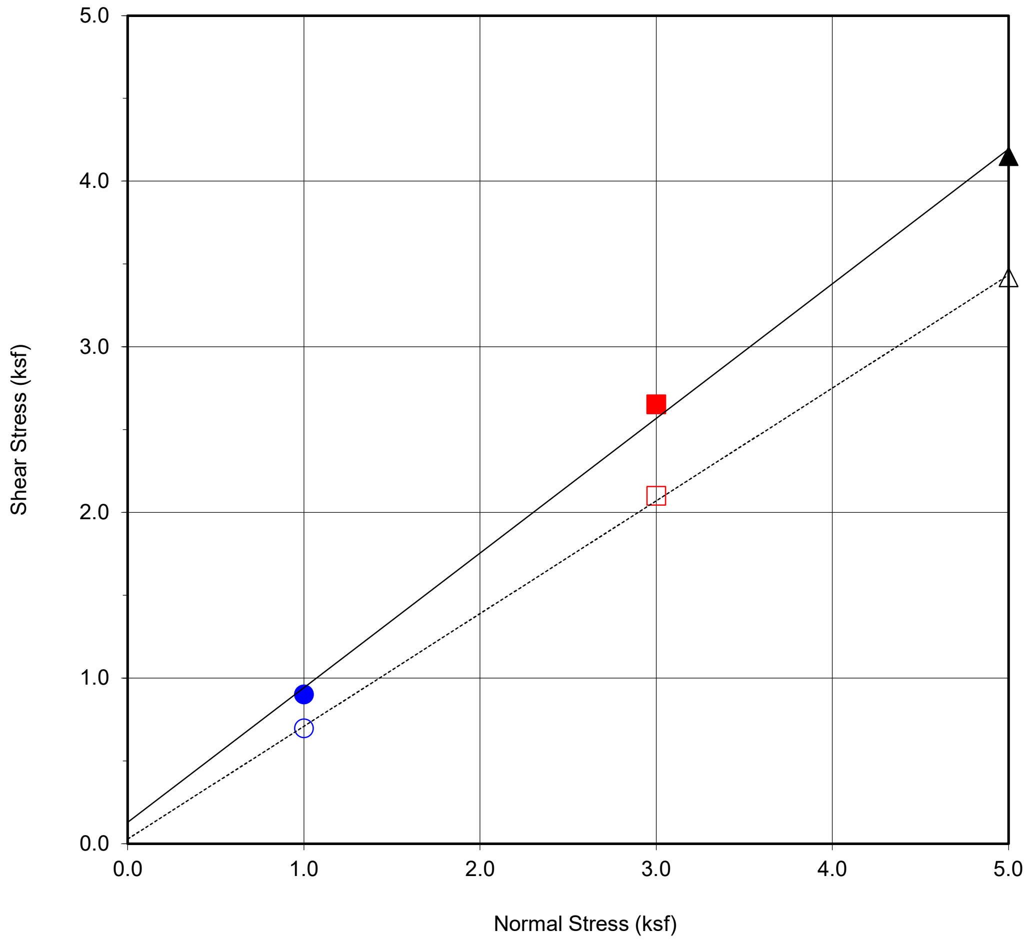
Figure B-22



Boring No.	**B7**
Sample No.	**B7@2.5**
Depth (ft)	**2.5**
Sample Type:	Ring

Soil Identification:		
Poorly Graded SAND with Silt (SP-SM), pale brown		
Strength Parameters		
	C (psf)	ϕ (°)
Peak	129	39
Ultimate	29	34

Normal Stress (kip/ft²)	1	3	5
Peak Shear Stress (kip/ft²)	● 0.90	■ 2.65	▲ 4.15
Shear Stress @ End of Test (ksf)	○ 0.70	□ 2.10	△ 3.42
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	3.7
Initial Dry Density (pcf)	105.3	106.0	108.2
Initial Degree of Saturation (%)	17.1	17.3	17.8
Soil Height Before Shearing (in.)	1.2	1.2	1.2
Final Moisture Content (%)	11.5	17.3	18.3

GEOCON

DIRECT SHEAR TEST RESULTS

Consolidated Drained ASTM D-3080

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Multi-Family Residential Development
14320 Palm Drive
Desert Hot Springs

September 2024

Figure B-23

APPENDIX C

APPENDIX C

RECOMMENDED GRADING SPECIFICATIONS

FOR

PROPOSED MULTI-FAMILY RESIDENTIAL DEVELOPMENT
14320 PALM DRIVE
DESERT HOT SPRINGS, CALIFORNIA

PROJECT NO. T3082-22-01

RECOMMENDED GRADING SPECIFICATIONS

1. GENERAL

1.1 These Recommended Grading Specifications shall be used in conjunction with the Geotechnical Report for the project prepared by Geocon. The recommendations contained in the text of the Geotechnical Report are a part of the earthwork and grading specifications and shall supersede the provisions contained hereinafter in the case of conflict.

1.2 Prior to the commencement of grading, a geotechnical consultant (Consultant) shall be employed for the purpose of observing earthwork procedures and testing the fills for substantial conformance with the recommendations of the Geotechnical Report and these specifications. The Consultant should provide adequate testing and observation services so that they may assess whether, in their opinion, the work was performed in substantial conformance with these specifications. It shall be the responsibility of the Contractor to assist the Consultant and keep them apprised of work schedules and changes so that personnel may be scheduled accordingly.

1.3 It shall be the sole responsibility of the Contractor to provide adequate equipment and methods to accomplish the work in accordance with applicable grading codes or agency ordinances, these specifications and the approved grading plans. If, in the opinion of the Consultant, unsatisfactory conditions such as questionable soil materials, poor moisture condition, inadequate compaction, and/or adverse weather result in a quality of work not in conformance with these specifications, the Consultant will be empowered to reject the work and recommend to the Owner that grading be stopped until the unacceptable conditions are corrected.

2. DEFINITIONS

2.1 **Owner** shall refer to the owner of the property or the entity on whose behalf the grading work is being performed and who has contracted with the Contractor to have grading performed.

2.2 **Contractor** shall refer to the Contractor performing the site grading work.

2.3 **Civil Engineer** or **Engineer of Work** shall refer to the California licensed Civil Engineer or consulting firm responsible for preparation of the grading plans, surveying and verifying as-graded topography.

2.4 **Consultant** shall refer to the soil engineering and engineering geology consulting firm retained to provide geotechnical services for the project.

2.5 **Soil Engineer** shall refer to a California licensed Civil Engineer retained by the Owner, who is experienced in the practice of geotechnical engineering. The Soil Engineer shall be responsible for having qualified representatives on-site to observe and test the Contractor's work for conformance with these specifications.

2.6 **Engineering Geologist** shall refer to a California licensed Engineering Geologist retained by the Owner to provide geologic observations and recommendations during the site grading.

2.7 **Geotechnical Report** shall refer to a soil report (including all addenda) which may include a geologic reconnaissance or geologic investigation that was prepared specifically for the development of the project for which these Recommended Grading Specifications are intended to apply.

3. MATERIALS

3.1 Materials for compacted fill shall consist of any soil excavated from the cut areas or imported to the site that, in the opinion of the Consultant, is suitable for use in construction of fills. In general, fill materials can be classified as *soil* fills, *soil-rock* fills or *rock* fills, as defined below.

3.1.1 **Soil fills** are defined as fills containing no rocks or hard lumps greater than 12 inches in maximum dimension and containing at least 40 percent by weight of material smaller than ¾ inch in size.

3.1.2 **Soil-rock fills** are defined as fills containing no rocks or hard lumps larger than 4 feet in maximum dimension and containing a sufficient matrix of soil fill to allow for proper compaction of soil fill around the rock fragments or hard lumps as specified in Paragraph 6.2. **Oversize rock** is defined as material greater than 12 inches.

3.1.3 **Rock fills** are defined as fills containing no rocks or hard lumps larger than 3 feet in maximum dimension and containing little or no fines. Fines are defined as material smaller than ¾ inch in maximum dimension. The quantity of fines shall be less than approximately 20 percent of the rock fill quantity.

3.2 Material of a perishable, spongy, or otherwise unsuitable nature as determined by the Consultant shall not be used in fills.

3.3 Materials used for fill, either imported or on-site, shall not contain hazardous materials as defined by the California Code of Regulations, Title 22, Division 4, Chapter 30, Articles 9 and 10; 40CFR; and any other applicable local, state or federal laws. The Consultant shall not be responsible for the identification or analysis of the potential presence of hazardous materials. However, if observations, odors or soil discoloration cause Consultant to suspect the presence of hazardous materials, the Consultant may request from the Owner the termination of grading operations within the affected area. Prior to resuming grading operations, the Owner shall provide a written report to the Consultant indicating that the suspected materials are not hazardous as defined by applicable laws and regulations.

3.4 The outer 15 feet of *soil-rock* fill slopes, measured horizontally, should be composed of properly compacted *soil* fill materials approved by the Consultant. *Rock* fill may extend to the slope face, provided that the slope is not steeper than 2:1 (horizontal:vertical) and a soil layer no thicker than 12 inches is track-walked onto the face for landscaping purposes. This procedure may be utilized provided it is acceptable to the governing agency, Owner and Consultant.

3.5 Samples of soil materials to be used for fill should be tested in the laboratory by the Consultant to determine the maximum density, optimum moisture content, and, where appropriate, shear strength, expansion, and gradation characteristics of the soil.

3.6 During grading, soil or groundwater conditions other than those identified in the Geotechnical Report may be encountered by the Contractor. The Consultant shall be notified immediately to evaluate the significance of the unanticipated condition

4. CLEARING AND PREPARING AREAS TO BE FILLED

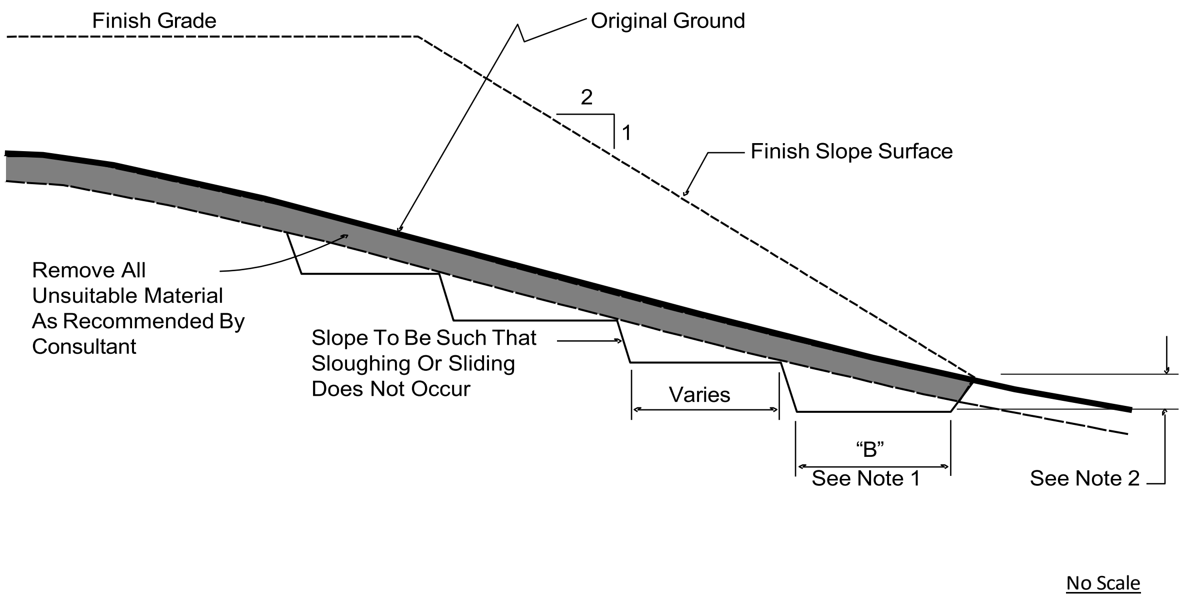
4.1 Areas to be excavated and filled shall be cleared and grubbed. Clearing shall consist of complete removal above the ground surface of trees, stumps, brush, vegetation, man-made structures, and similar debris. Grubbing shall consist of removal of stumps, roots, buried logs and other unsuitable material and shall be performed in areas to be graded. Roots and other projections exceeding 1½ inches in diameter shall be removed to a depth of 3 feet below the surface of the ground. Borrow areas shall be grubbed to the extent necessary to provide suitable fill materials.

4.2 Asphalt pavement material removed during clearing operations should be properly disposed at an approved off-site facility or in an acceptable area of the project evaluated by Geocon and the property owner. Concrete fragments that are free of reinforcing steel may be placed in fills, provided they are placed in accordance with Section 6.2 or 6.3 of this document.

4.3 After clearing and grubbing of organic matter and other unsuitable material, loose or porous soils shall be removed to the depth recommended in the Geotechnical Report. The depth of removal and compaction should be observed and approved by a representative of the Consultant. The exposed surface shall then be plowed or scarified to a minimum depth of 6 inches and until the surface is free from uneven features that would tend to prevent uniform compaction by the equipment to be used.

4.4 Where the slope ratio of the original ground is steeper than 5:1 (horizontal:vertical), or where recommended by the Consultant, the original ground should be benched in accordance with the following illustration.

TYPICAL BENCHING DETAIL



DETAIL NOTES: (1) Key width "B" should be a minimum of 10 feet, or sufficiently wide to permit complete coverage with the compaction equipment used. The base of the key should be graded horizontal, or inclined slightly into the natural slope.

(2) The outside of the key should be below the topsoil or unsuitable surficial material and at least 2 feet into dense formational material. Where hard rock is exposed in the bottom of the key, the depth and configuration of the key may be modified as approved by the Consultant.

4.5 After areas to receive fill have been cleared and scarified, the surface should be moisture conditioned to achieve the proper moisture content, and compacted as recommended in Section 6 of these specifications.

5. COMPACTION EQUIPMENT

5.1 Compaction of *soil* or *soil-rock* fill shall be accomplished by sheepsfoot or segmented-steel wheeled rollers, vibratory rollers, multiple-wheel pneumatic-tired rollers, or other types of acceptable compaction equipment. Equipment shall be of such a design that it will be capable of compacting the *soil* or *soil-rock* fill to the specified relative compaction at the specified moisture content.

5.2 Compaction of *rock* fills shall be performed in accordance with Section 6.3.

6. PLACING, SPREADING AND COMPACTION OF FILL MATERIAL

6.1 *Soil* fill, as defined in Paragraph 3.1.1, shall be placed by the Contractor in accordance with the following recommendations:

6.1.1 *Soil* fill shall be placed by the Contractor in layers that, when compacted, should generally not exceed 8 inches. Each layer shall be spread evenly and shall be thoroughly mixed during spreading to obtain uniformity of material and moisture in each layer. The entire fill shall be constructed as a unit in nearly level lifts. Rock materials greater than 12 inches in maximum dimension shall be placed in accordance with Section 6.2 or 6.3 of these specifications.

6.1.2 In general, the *soil* fill shall be compacted at a moisture content at or above the optimum moisture content as determined by ASTM D 1557.

6.1.3 When the moisture content of *soil* fill is below that specified by the Consultant, water shall be added by the Contractor until the moisture content is in the range specified.

6.1.4 When the moisture content of the *soil* fill is above the range specified by the Consultant or too wet to achieve proper compaction, the *soil* fill shall be aerated by the Contractor by blading/mixing, or other satisfactory methods until the moisture content is within the range specified.

6.1.5 After each layer has been placed, mixed, and spread evenly, it shall be thoroughly compacted by the Contractor to a relative compaction of at least 90 percent. Relative compaction is defined as the ratio (expressed in percent) of the in-place dry density of the compacted fill to the maximum laboratory dry density as determined in accordance with ASTM D 1557. Compaction shall be continuous over the entire area, and compaction equipment shall make sufficient passes so that the specified minimum relative compaction has been achieved throughout the entire fill.

6.1.6 Where practical, soils having an Expansion Index greater than 50 should be placed at least 3 feet below finish pad grade and should be compacted at a moisture content generally 2 to 4 percent greater than the optimum moisture content for the material.

6.1.7 Properly compacted *soil* fill shall extend to the design surface of fill slopes. To achieve proper compaction, it is recommended that fill slopes be over-built by at least 3 feet and then cut to the design grade. This procedure is considered preferable to track-walking of slopes, as described in the following paragraph.

6.1.8 As an alternative to over-building of slopes, slope faces may be back-rolled with a heavy-duty loaded sheepsfoot or vibratory roller at maximum 4-foot fill height intervals. Upon completion, slopes should then be track-walked with a D-8 dozer or similar equipment, such that a dozer track covers all slope surfaces at least twice.

6.2 *Soil-rock* fill, as defined in Paragraph 3.1.2, shall be placed by the Contractor in accordance with the following recommendations:

6.2.1 Rocks larger than 12 inches but less than 4 feet in maximum dimension may be incorporated into the compacted *soil* fill, but shall be limited to the area measured 15 feet minimum horizontally from the slope face and 5 feet below finish grade or 3 feet below the deepest utility, whichever is deeper.

6.2.2 Rocks or rock fragments up to 4 feet in maximum dimension may either be individually placed or placed in windrows. Under certain conditions, rocks or rock fragments up to 10 feet in maximum dimension may be placed using similar methods. The acceptability of placing rock materials greater than 4 feet in maximum dimension shall be evaluated during grading as specific cases arise and shall be approved by the Consultant prior to placement.

6.2.3 For individual placement, sufficient space shall be provided between rocks to allow for passage of compaction equipment.

6.2.4 For windrow placement, the rocks should be placed in trenches excavated in properly compacted *soil* fill. Trenches should be approximately 5 feet wide and 4 feet deep in maximum dimension. The voids around and beneath rocks should be filled with approved granular soil having a Sand Equivalent of 30 or greater and should be compacted by flooding. Windrows may also be placed utilizing an "open-face" method in lieu of the trench procedure, however, this method should first be approved by the Consultant.

6.2.5 Windrows should generally be parallel to each other and may be placed either parallel to or perpendicular to the face of the slope depending on the site geometry. The minimum horizontal spacing for windrows shall be 12 feet center-to-center with a 5-foot stagger or offset from lower courses to next overlying course. The minimum vertical spacing between windrow courses shall be 2 feet from the top of a lower windrow to the bottom of the next higher windrow.

6.2.6 Rock placement, fill placement and flooding of approved granular soil in the windrows should be continuously observed by the Consultant.

6.3 *Rock* fills, as defined in Section 3.1.3, shall be placed by the Contractor in accordance with the following recommendations:

6.3.1 The base of the *rock* fill shall be placed on a sloping surface (minimum slope of 2 percent). The surface shall slope toward suitable subdrainage outlet facilities. The *rock* fills shall be provided with subdrains during construction so that a hydrostatic pressure buildup does not develop. The subdrains shall be permanently connected to controlled drainage facilities to control post-construction infiltration of water.

6.3.2 *Rock* fills shall be placed in lifts not exceeding 3 feet. Placement shall be by rock trucks traversing previously placed lifts and dumping at the edge of the currently placed lift. Spreading of the *rock* fill shall be by dozer to facilitate *seating* of the rock. The *rock* fill shall be watered heavily during placement. Watering shall consist of water trucks traversing in front of the current rock lift face and spraying water continuously during rock placement. Compaction equipment with compactive energy comparable to or greater than that of a 20-ton steel vibratory roller or other compaction equipment providing suitable energy to achieve the required compaction or deflection as recommended in Paragraph 6.3.3 shall be utilized. The number of passes to be made should be determined as described in Paragraph 6.3.3. Once a *rock* fill lift has been covered with *soil* fill, no additional *rock* fill lifts will be permitted over the *soil* fill.

6.3.3 Plate bearing tests, in accordance with ASTM D 1196, may be performed in both the compacted *soil* fill and in the *rock* fill to aid in determining the required minimum number of passes of the compaction equipment. If performed, a minimum of three plate bearing tests should be performed in the properly compacted *soil* fill (minimum relative compaction of 90 percent). Plate bearing tests shall then be performed on areas of *rock* fill having two passes, four passes and six passes of the compaction equipment, respectively. The number of passes required for the *rock* fill shall be determined by comparing the results of the plate bearing tests for the *soil* fill and the *rock* fill and by evaluating the deflection variation with number of passes. The required number of passes of the compaction equipment will be performed as necessary until the plate bearing deflections are equal to or less than that determined for the properly compacted *soil* fill. In no case will the required number of passes be less than two.

6.3.4 A representative of the Consultant should be present during *rock* fill operations to observe that the minimum number of “passes” have been obtained, that water is being properly applied and that specified procedures are being followed. The actual number of plate bearing tests will be determined by the Consultant during grading.

6.3.5 Test pits shall be excavated by the Contractor so that the Consultant can state that, in their opinion, sufficient water is present and that voids between large rocks are properly filled with smaller rock material. In-place density testing will not be required in the *rock* fills.

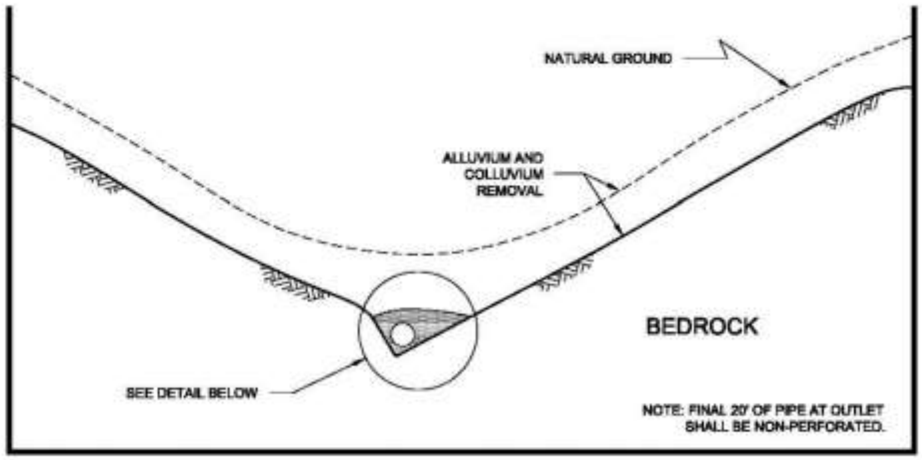
6.3.6 To reduce the potential for “piping” of fines into the *rock* fill from overlying *soil* fill material, a 2-foot layer of graded filter material shall be placed above the uppermost lift of *rock* fill. The need to place graded filter material below the *rock* should be determined by the Consultant prior to commencing grading. The gradation of the graded filter material will be determined at the time the *rock* fill is being excavated. Materials typical of the *rock* fill should be submitted to the Consultant in a timely manner, to allow design of the graded filter prior to the commencement of *rock* fill placement.

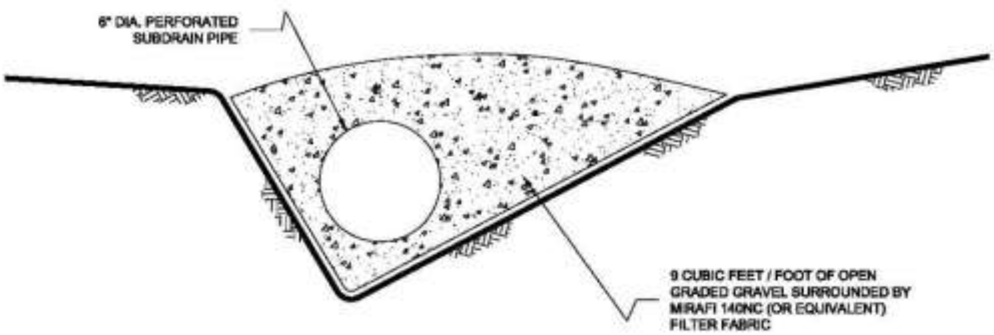
6.3.7 *Rock* fill placement should be continuously observed during placement by the Consultant.

7. SUBDRAINS

7.1 The geologic units on the site may have permeability characteristics and/or fracture systems that could be susceptible under certain conditions to seepage. The use of canyon subdrains may be necessary to mitigate the potential for adverse impacts associated with seepage conditions. Canyon subdrains with lengths in excess of 500 feet or extensions of existing offsite subdrains should use 8-inch-diameter pipes. Canyon subdrains less than 500 feet in length should use 6-inch-diameter pipes.

TYPICAL CANYON DRAIN DETAIL





NOTES:

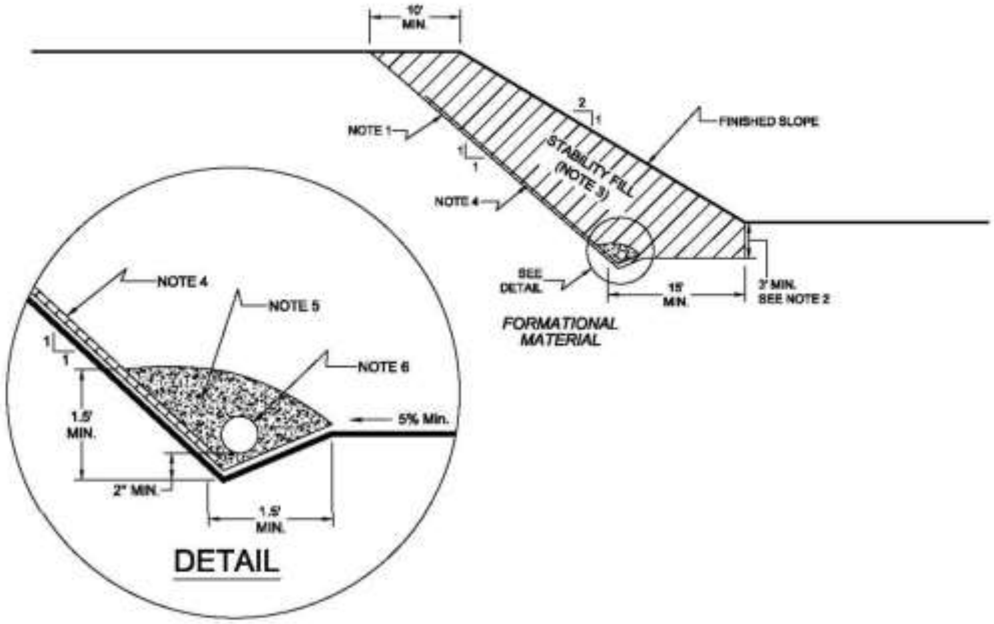
1......6-INCH DIAMETER, SCHEDULE 80 PVC PERFORATED PIPE FOR FILLS IN EXCESS OF 100-FEET IN DEPTH OR A PIPE LENGTH OF LONGER THAN 500 FEET.

2......6-INCH DIAMETER, SCHEDULE 40 PVC PERFORATED PIPE FOR FILLS LESS THAN 100-FEET IN DEPTH OR A PIPE LENGTH SHORTER THAN 500 FEET.

NO SCALE

7.2 Slope drains within stability fill keyways should use 4-inch-diameter (or lager) pipes.

TYPICAL STABILITY FILL DETAIL



NOTES:

1....EXCAVATE BACKCUT AT 1:1 INCLINATION (UNLESS OTHERWISE NOTED).

2....BASE OF STABILITY FILL TO BE 3 FEET INTO FORMATIONAL MATERIAL, SLOPING A MINIMUM 5% INTO SLOPE.

3....STABILITY FILL TO BE COMPOSED OF PROPERLY COMPACTED GRANULAR SOIL.

4....CHIMNEY DRAINS TO BE APPROVED PREFABRICATED CHIMNEY DRAIN PANELS (MIRADRAIN G200N OR EQUIVALENT) SPACED APPROXIMATELY 20 FEET CENTER TO CENTER AND 4 FEET WIDE. CLOSER SPACING MAY BE REQUIRED IF SEEPAGE IS ENCOUNTERED.

5....FILTER MATERIAL TO BE 3/4-INCH, OPEN-GRADED CRUSHED ROCK ENCLOSED IN APPROVED FILTER FABRIC (MIRAFI 140NC).

6....COLLECTOR PIPE TO BE 4-INCH MINIMUM DIAMETER, PERFORATED, THICK-WALLED PVC SCHEDULE 40 OR EQUIVALENT, AND SLOPED TO DRAIN AT 1 PERCENT MINIMUM TO APPROVED OUTLET.

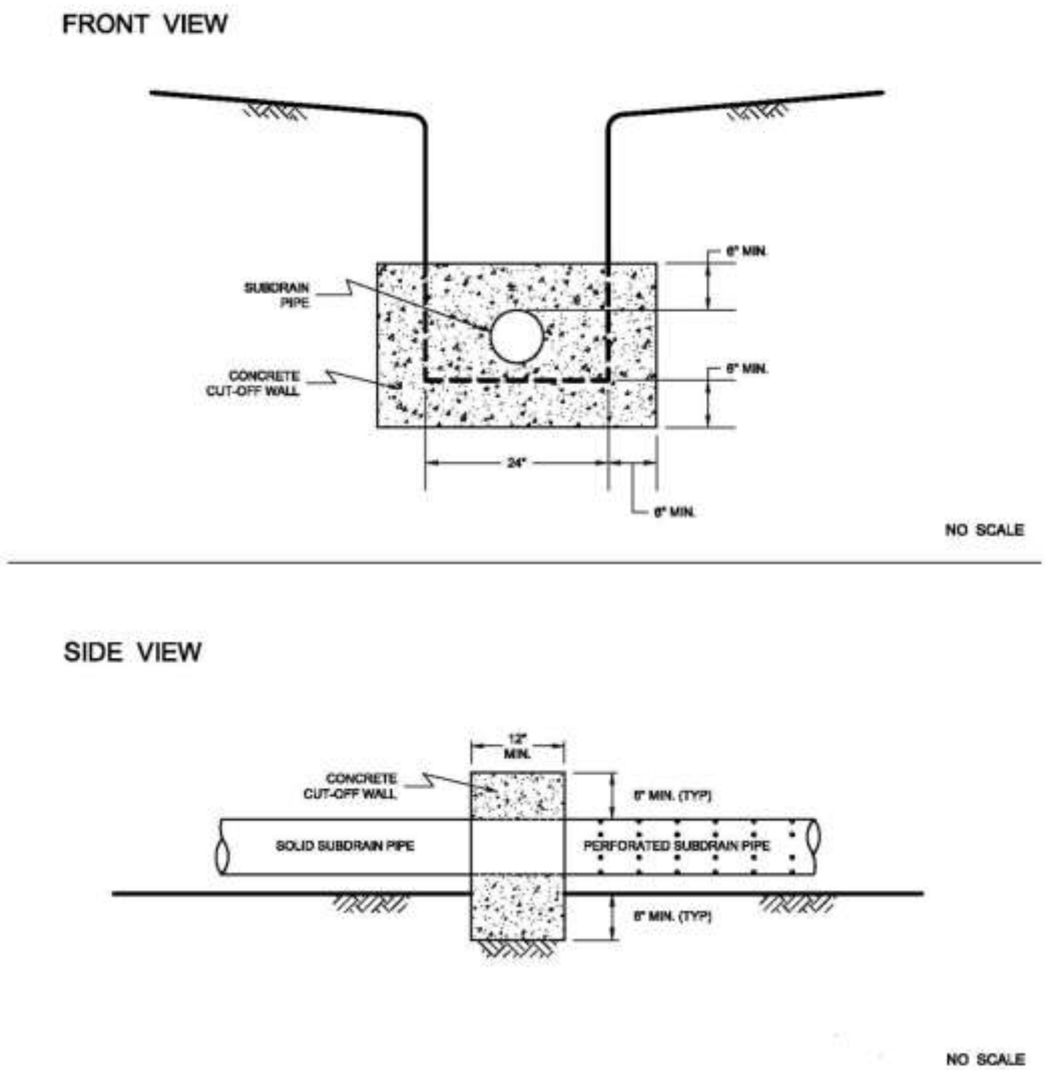
NO SCALE

7.3 The actual subdrain locations will be evaluated in the field during the remedial grading operations. Additional drains may be necessary depending on the conditions observed and the requirements of the local regulatory agencies. Appropriate subdrain outlets should be evaluated prior to finalizing 40-scale grading plans.

7.4 *Rock* fill or *soil-rock* fill areas may require subdrains along their down-slope perimeters to mitigate the potential for buildup of water from construction or landscape irrigation. The subdrains should be at least 6-inch-diameter pipes encapsulated in gravel and filter fabric. *Rock* fill drains should be constructed using the same requirements as canyon subdrains.

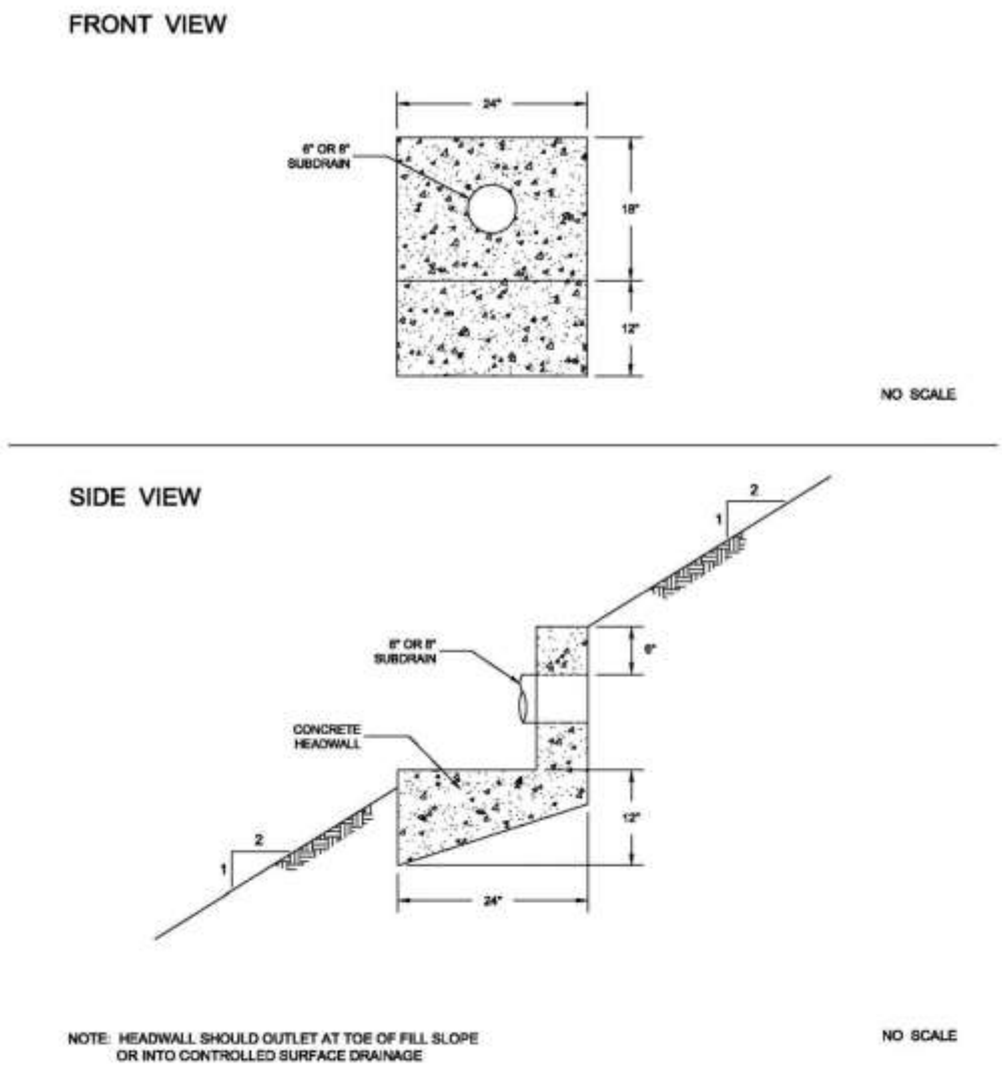
7.5 Prior to outletting, the final 20-foot segment of a subdrain that will not be extended during future development should consist of non-perforated drainpipe. At the non-perforated/ perforated interface, a seepage cutoff wall should be constructed on the downslope side of the pipe.

TYPICAL CUT OFF WALL DETAIL



7.6 Subdrains that discharge into a natural drainage course or open space area should be provided with a permanent headwall structure.

TYPICAL HEADWALL DETAIL



7.7 The final grading plans should show the location of the proposed subdrains. After completion of remedial excavations and subdrain installation, the project civil engineer should survey the drain locations and prepare an "as-built" map showing the drain locations. The final outlet and connection locations should be determined during grading operations. Subdrains that will be extended on adjacent projects after grading can be placed on formational material and a vertical riser should be placed at the end of the subdrain. The grading contractor should consider videoing the subdrains shortly after burial to check proper installation and functionality. The contractor is responsible for the performance of the drains.

8. OBSERVATION AND TESTING

8.1 The Consultant shall be the Owner's representative to observe and perform tests during clearing, grubbing, filling, and compaction operations. In general, no more than 2 feet in vertical elevation of *soil* or *soil-rock* fill should be placed without at least one field density test being performed within that interval. In addition, a minimum of one field density test should be performed for every 2,000 cubic yards of *soil* or *soil-rock* fill placed and compacted.

8.2 The Consultant should perform a sufficient distribution of field density tests of the compacted *soil* or *soil-rock* fill to provide a basis for expressing an opinion whether the fill material is compacted as specified. Density tests shall be performed in the compacted materials below any disturbed surface. When these tests indicate that the density of any layer of fill or portion thereof is below that specified, the particular layer or areas represented by the test shall be reworked until the specified density has been achieved.

8.3 During placement of *rock* fill, the Consultant should observe that the minimum number of passes have been obtained per the criteria discussed in Section 6.3.3. The Consultant should request the excavation of observation pits and may perform plate bearing tests on the placed *rock* fills. The observation pits will be excavated to provide a basis for expressing an opinion as to whether the *rock* fill is properly seated and sufficient moisture has been applied to the material. When observations indicate that a layer of *rock* fill or any portion thereof is below that specified, the affected layer or area shall be reworked until the *rock* fill has been adequately seated and sufficient moisture applied.

8.4 A settlement monitoring program designed by the Consultant may be conducted in areas of *rock* fill placement. The specific design of the monitoring program shall be as recommended in the Conclusions and Recommendations section of the project Geotechnical Report or in the final report of testing and observation services performed during grading.

8.5 We should observe the placement of subdrains, to check that the drainage devices have been placed and constructed in substantial conformance with project specifications.

8.6 Testing procedures shall conform to the following Standards as appropriate:

8.6.1 **Soil and Soil-Rock Fills:**

8.6.1.1 Field Density Test, ASTM D 1556, *Density of Soil In-Place By the Sand-Cone Method.*

8.6.1.2 Field Density Test, Nuclear Method, ASTM D 6938, *Density of Soil and Soil-Aggregate In-Place by Nuclear Methods (Shallow Depth).*

8.6.1.3 Laboratory Compaction Test, ASTM D 1557, *Moisture-Density Relations of Soils and Soil-Aggregate Mixtures Using 10-Pound Hammer and 18-Inch Drop*.

8.6.1.4 Expansion Index Test, ASTM D 4829, *Expansion Index Test*.

9. PROTECTION OF WORK

9.1 During construction, the Contractor shall properly grade all excavated surfaces to provide positive drainage and prevent ponding of water. Drainage of surface water shall be controlled to avoid damage to adjoining properties or to finished work on the site. The Contractor shall take remedial measures to prevent erosion of freshly graded areas until such time as permanent drainage and erosion control features have been installed. Areas subjected to erosion or sedimentation shall be properly prepared in accordance with the Specifications prior to placing additional fill or structures.

9.2 After completion of grading as observed and tested by the Consultant, no further excavation or filling shall be conducted except in conjunction with the services of the Consultant.

10. CERTIFICATIONS AND FINAL REPORTS

10.1 Upon completion of the work, Contractor shall furnish Owner a certification by the Civil Engineer stating that the lots and/or building pads are graded to within 0.1 foot vertically of elevations shown on the grading plan and that all tops and toes of slopes are within 0.5 foot horizontally of the positions shown on the grading plans. After installation of a section of subdrain, the project Civil Engineer should survey its location and prepare an *as-built* plan of the subdrain location. The project Civil Engineer should verify the proper outlet for the subdrains and the Contractor should ensure that the drain system is free of obstructions.

10.2 The Owner is responsible for furnishing a final as-graded soil and geologic report satisfactory to the appropriate governing or accepting agencies. The as-graded report should be prepared and signed by a California licensed Civil Engineer experienced in geotechnical engineering and by a California Certified Engineering Geologist, indicating that the geotechnical aspects of the grading were performed in substantial conformance with the Specifications or approved changes to the Specifications.