GEOTECHNICAL INVESTIGATION

PROPOSED MULTI-FAMILY RESIDENTIAL DEVELOPMENT
2325 CRENSHAW BOULEVARD
TORRANCE, CALIFORNIA

GEOCON
WEST, INC.

GEOTECHNICAL
ENVIRONMENTAL
MATERIALS

PREPARED FOR

ROSE EQUITIES, LLC
BEVERLY HILLS, CALIFORNIA

PROJECT NO. W1572-06-01A

REVISED OCTOBER 25, 2023

GEOCON
WEST, INC.
GEOTECHNICAL ■ ENVIRONMENTAL ■ MATERIALS

Project No. W1572-06-01A
Revised October 25, 2023

Mr. Brent Stoll
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8383 Wilshire Boulevard, Suite 632
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Subject: GEOTECHNICAL INVESTIGATION
PROPOSED MULTI-FAMILY RESIDENTIAL DEVELOPMENT
2325 CRENSHAW BOULEVARD
TORRANCE, CALIFORNIA

Dear Mr. Stoll:

In accordance with your authorization of our proposal dated May 4, 2022, we have performed a geotechnical investigation for the proposed multi-family residential development located at 2325 Crenshaw Boulevard in the City of Torrance, California. The accompanying report presents the findings of our study and our conclusions and recommendations pertaining to the geotechnical aspects of proposed design and construction. Based on the results of our investigation, it is our opinion that the site can be developed as proposed, provided the recommendations of this report are followed and implemented during design and construction.

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

Very truly yours,

GEOCON WEST, INC.

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GEOTECHNICAL INVESTIGATION

1. PURPOSE AND SCOPE

This report presents the results of a geotechnical investigation for the proposed multi-family residential development located at 2325 Crenshaw Boulevard in the City of Torrance, California (see Vicinity Map, Figure 1). The purpose of the investigation was to evaluate subsurface soil and geologic conditions underlying the site and, based on conditions encountered, to provide conclusions and recommendations pertaining to the geotechnical aspects of design and construction.

The scope of this investigation included a site reconnaissance, field exploration, laboratory testing, engineering analysis, and the preparation of this report. The site was explored on June 2, 2022, by excavating six 8-inch diameter borings to depths ranging from approximately 25½ to 50½ feet below the existing ground surface using a truck-mounted hollow-stem auger drilling machine. In addition, one boring was excavated with hand tools to a maximum depth of approximately 5 feet beneath the ground surface for the purpose of shallow percolation testing. Additional site exploration was performed on September 23, 2023, by excavating three 8-inch diameter borings using a truck-mounted hollow-stem auger drilling machine to depths of approximately 20½ and 30½ feet below the existing ground surface. The approximate locations of the exploratory borings are depicted on the Site Plan (see Figure 2). A detailed discussion of the field investigation, including the boring logs, is presented in Appendix A.

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

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

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

2. SITE AND PROJECT DESCRIPTION

The subject site is located at 2325 Crenshaw Boulevard in the City of Torrance, California. The site is currently occupied by single-story commercial structures and an asphalt paved parking lot. The site is bounded by two-story single-family residential structures to the north, by three-story multi-family residential structures to the west, by three-story commercial buildings with associated asphalt paved parking lots the east, and by a three-story apartment building with a partial subterranean parking level and an associated asphalt parking lot to the south. The site is relatively level with no pronounced highs or lows. Surface water drainage at the site appears to be by sheet flow along the existing ground contours towards drains in paving. Vegetation onsite consists of shrubs and trees located in isolated planter areas.

Based on the information provided by the Client, it is our understanding that proposed multi-family residential development will consist of a three- to five-story multi-family residential structure that will be constructed at or near present site grade on the north portion of the property and underlain by two levels of subterranean parking elsewhere (see Site Plan, Figure 2). The proposed subterranean levels will extend approximately up to 22 feet below the existing ground surface, including foundation depths.

Based on the preliminary nature of the design at this time, wall and column loads were not available. It is anticipated that column loads for the proposed structure will be up to 700 kips, and wall loads will be up to 8 kips per linear foot.

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

3. GEOLOGIC SETTING

The site is located on the Torrance Plain, a broad, nearly flat alluvial surface situated in the southern extreme of the Los Angeles Coastal Plain, northeast of the Palos Verdes Hills. The Torrance Plain is a Pleistocene age marine surface only slightly dissected by local streams. Both fresh water and marine fossils are commonly found at shallow depths below the surface of the plain, suggesting that there were periods of complete or nearly complete emergence of the land between periods of subsidence.

Regionally, the Torrance Plain is located within the Peninsular Ranges geomorphic province, characterized by elongate northwest-trending mountain ridges separated by straight-sided sediment-filled valleys. The northwest trend is further reflected in the direction of the dominant geologic structural features of the province that are northwest to west-northwest trending folds and faults, such as the nearby Palos Verdes Fault Zone, located approximately 2.2 miles southwest of the site.

4. SOIL AND GEOLOGIC CONDITIONS

Based on our field investigation and published geologic maps of the area, the site is underlain by artificial fill and Pleistocene age older sand dune deposits consisting of primarily fine-grained sand with varying amounts of silt and clay (California Geological Survey [CGS], 2010). Detailed stratigraphic profiles of the materials encountered at the site are provided on the boring logs in Appendix A.

4.1 Artificial Fill

Artificial fill was encountered in our borings to a maximum depth of 4 feet below existing ground surface. The artificial fill generally consists of yellowish brown to dark yellowish brown and dark grayish brown silty sand that can be characterized as dry to moist and medium dense. The fill is likely the result of past grading or construction activities at the site. Deeper fill may exist between excavations and in other portions of the site that were not directly explored.

4.2 Older Dune Sand

The fill soils are underlain by Pleistocene age (older) dune sand deposits consisting of light to dark yellowish brown or light to dark olive brown, interbedded silty sand, clayey sand, and poorly graded sand with lesser interbeds of clay, sandy clay or silty clay. The alluvial soils are primarily dry to moist and medium dense to dense or firm to hard.

5. GROUNDWATER

Review of the Seismic Hazard Zone Report for the Torrance Quadrangle (California Division of Mines and Geology [CDMG], 1998) indicates the historically highest groundwater level in the area is greater than 40 feet beneath the ground surface. Groundwater information presented in this document is generated from data collected in the early 1900's to the late 1990s. Based on current groundwater basin management practices, it is unlikely that groundwater levels will ever exceed the historic high levels.

Review of groundwater monitoring well data provided by the California Department of Water Resources (CDWR, 2022) indicates closest monitoring well to the site is State Well No. 04S14W15N001S, located approximately ¾ mile southwest of the site. Monitoring data from this well is available for the period from December 1995 through March 1998. During this time, the depth to groundwater has fluctuated from approximately 86 to 89 feet beneath the ground surface (CDWR, 2022). The most recent groundwater level measurement was recorded on March 23, 1998, and the depth to groundwater was greater than 85 feet beneath the ground surface (CDWR, 2002).

Groundwater was not encountered in the borings excavated to a maximum depth of approximately 50½ feet beneath the existing ground surface. Based on the lack of groundwater in our borings, the reported historic high groundwater levels in the site vicinity (CDMG, 1998; CDWR, 2022), and the depth of the proposed construction, static groundwater is not expected to be encountered during construction or have a detrimental effect on the project. However, it is not uncommon for groundwater levels to vary seasonally or for groundwater seepage conditions to develop where none previously existed, especially in impermeable fine-grained soils which are heavily irrigated or after seasonal rainfall. In addition, recent requirements for stormwater infiltration could result in shallower seepage conditions in the immediate site vicinity. Proper surface drainage of irrigation and precipitation will be critical for future performance of the project. Recommendations for drainage are provided in the S*urface Drainage* section of this report (see Section 7.24).

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, 2022a; CGS, 2022b; CDMG, 1986) or a city-designated Fault Hazard Management Zone (City of Torrance, 2010) 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 in Figure 3, Regional Fault Map.

The closest surface trace of an active fault to the site is the Palos Verdes Fault Zone located approximately 2.2 miles to the southwest (CDMG, 1986; USGS, 2006). Other nearby active faults include the Redondo Canyon Fault, the Cabrillo Fault, and the Newport-Inglewood Fault Zone located approximately 4.0 miles to the southwest, 4.2 miles to the south, and 5.6 miles to the northeast-northeast of the site, respectively. The active San Andreas Fault Zone is located approximately 51 miles northeast of the site (Ziony and Jones, 1989).

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

6.2 Seismicity

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

LIST OF HISTORIC EARTHQUAKES

Earthquake (Oldest to Youngest)	Date of Earthquake	Magnitude	Distance to Epicenter (Miles)	Direction to Epicenter
Near Redlands	July 23, 1923	6.3	63	E
Long Beach	March 10, 1933	6.4	25	ESE
Tehachapi	July 21, 1952	7.5	90	NW
San Fernando	February 9, 1971	6.6	41	N
Whittier Narrows	October 1, 1987	5.9	22	NE
Sierra Madre	June 28, 1991	5.8	36	NE
Landers	June 28, 1992	7.3	111	ENE
Big Bear	June 28, 1992	6.4	90	ENE
Northridge	January 17, 1994	6.7	29	NNW
Hector Mine	October 16, 1999	7.1	129	ENE
Ridgecrest	July 5, 2019	7.1	140	NNE

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

6.3 Seismic Design Criteria

The following table summarizes the site-specific design criteria obtained from the 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 1613.3.2 of the 2022 CBC and Table 20.3-1 of ASCE 7-16. The values presented on the following gpage 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	1.8g	Figure 1613.2.1(1)
MCE_R Ground Motion Spectral Response Acceleration – Class B (1 sec), S_1	0.651g	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}	1.8g	Section 1613.2.3 (Eqn 16-20)
Site Class Modified MCE_R Spectral Response Acceleration – (1 sec), S_{M1}	1.106g*	Section 1613.2.3 (Eqn 16-21)
5% Damped Design Spectral Response Acceleration (short), S_{DS}	1.2g	Section 1613.2.4 (Eqn 16-22)
5% Damped Design Spectral Response Acceleration (1 sec), S_{D1}	0.737g*	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 table on the following page 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	0.793g	Figure 22-9
Site Coefficient, F_{PGA}	1.1	Table 11.8-1
Site Class Modified MCE_G Peak Ground Acceleration, PGA_M	0.872g	Section 11.8.3 (Eqn 11.8-1)

Deaggregation of the MCE peak ground acceleration was performed using the USGS online Unified Hazard Tool, 2014 Conterminous U.S. Dynamic edition (v4.2.0). The result of the deaggregation analysis indicates that the mean earthquake contributing to the MCE peak ground acceleration is characterized as a 6.83 magnitude event occurring at a hypocentral distance of 6.93 kilometers from the site.

Deaggregation was also performed for the Design Earthquake (DE) peak ground acceleration, corresponding to two-thirds of the MCE peak ground acceleration. The result of the analysis indicates that the mean earthquake contributing to the DE peak ground acceleration is characterized as a 6.64 magnitude occurring at a hypocentral distance of 11.56 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 State of California Seismic Hazard Zone Map for the Torrance Quadrangle (CDMG, 1999) indicates that the site is not located in an area designated as having a potential for liquefaction. In addition, a review of the City of Torrance Safety Element (City of Torrance, 2010) indicates that the site is not located within an area identified as having a potential for liquefaction. The reported historic high groundwater level is greater than 40 feet beneath the ground surface (CDMG, 1998). Also, the site is underlain by moderately consolidated Pleistocene age older dune sand that is medium dense to dense or stiff to hard and not prone to liquefaction. Based on these considerations, it is our opinion that the potential for liquefaction and associated ground deformations beneath the site is low.

6.5 Slope Stability

The topography at the site and the immediate vicinity is relatively level to gently sloping downward to the south-southeast. The site is not located within a City of Torrance Landslide Hazard Zone (City of Torrance, 2010). According to the County of Los Angeles Safety Element (Leighton, 1990), the site is not located within an area identified as a “Hillside Area” or an area identified as having a potential for slope instability. Additionally, the site is not located within an area identified as having a potential for seismic slope instability (CDMG, 1999). There are no known landslides near the site, nor is the site in the path of any known or potential landslides. Therefore, the potential for slope stability hazards to adversely affect the proposed development is considered low.

6.6 Earthquake-Induced Flooding

Earthquake-induced flooding is inundation caused by failure of dams or other water-retaining structures due to earthquakes. The Los Angeles County Safety Element (Leighton, 1990) indicates that the site is not located within a dam inundation hazard area. Therefore, the potential for inundation at the site as a result of an earthquake-induced dam failure is considered low.

6.7 Tsunamis, Seiches, and Flooding

The site is located approximately 3½ miles east of the Pacific Ocean at elevations greater than approximately 80 feet above mean sea level (MSL). Also, the site is not located within a designated tsunami inundation area (City of Torrance, 2010; CGS, 2009). 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 within an area of minimal flooding (Zone X) as defined by the Federal Emergency Management Agency (FEMA, 2022; LACDPW, 2022).

6.8 Oil Fields & Methane Potential

Based on a review of the California Geologic Energy Management Division (CalGEM) Well Finder Website (CalGEM, 2022), the site is located within the Torrance Oil Field. The nearest active oil/gas well is the S & C Oil Company Well No. 3 (API: 0403717913), located approximately 0.4-mile southeast of the site. Also, there may be several abandoned or inactive oil/gas wells documented on the property (CalGEM, 2022). Additional information regarding these wells is further addressed in the Phase 1 ESA currently being prepared for the site (submitted under separate cover). Due to the voluntary nature of record reporting by the oil well drilling companies, wells may be improperly located or not documented on the location map. Undocumented or abandoned wells could be encountered during construction. Any wells encountered will need to be properly abandoned in accordance with the current requirements of the CalGEM.

Since the site is within the Torrance Oil Field, there may be a potential for methane and other volatile gases to occur at the site which may require a permanent methane gas control system beneath the proposed buildings. Should it be determined that a methane study is required for the proposed development, it is recommended that a qualified methane consultant be retained to perform the study and provide mitigation measures as necessary.

6.9 Subsidence

Subsidence occurs when a large portion of land is displaced vertically, usually due to the withdrawal of groundwater, oil, or natural gas. Soils that are particularly subject to subsidence include those with high silt or clay content. The City of Torrance Local Hazard Mitigation Plan indicates that there are no recognized past subsidence events in the Torrance area (City of Torrance, 2016). There appears to be little or no potential for ground subsidence due to withdrawal of fluids or gases at the site.

7. CONCLUSIONS AND RECOMMENDATIONS

7.1 General

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

7.1.2 Up to 4 feet of existing artificial fill was encountered during the site investigation. The existing fill encountered is believed to be the result of past grading and construction activities at the site. Deeper fill may exist in other areas of the site that were not directly explored. Future demolition of the existing structures and improvements which occupy the site will likely disturb the upper few feet of existing site soils. The existing fill and site soils are suitable for re-use as engineered fill, if needed, provided the recommendations in the *Grading* section of this report are followed (see Section 7.4).

7.1.3 Excavations for the subterranean levels are anticipated to penetrate through the existing artificial fill and expose undisturbed older dune sand throughout the excavation bottom.

7.1.4 Based on these considerations, the proposed structure may be supported on a conventional foundation system. At the subterranean levels, the conventional foundation system may derive support in the undisturbed older dune sand at or below a depth of 6 feet below the existing ground surface. For on-grade portions of the structure, the conventional foundation system may derive support in newly placed engineered fill. The transition area for the on-grade portion to the subterranean portion of the structure should be more heavily reinforced to resist differential settlement stresses which could cause cracking. All foundation excavations must be observed and approved in writing by the Geotechnical Engineer prior to placement of steel or concrete. Recommendations for the design of a conventional foundation system are provided in Section 7.5.

7.1.5 As a minimum, the upper 6 feet of existing site soils within the proposed on-grade footprint areas should be excavated and properly compacted for foundation and slab support. Excavation should be conducted as necessary to completely remove all artificial fill and any soft, unsuitable paralic deposits at the direction of the Geotechnical Engineer (a representative of Geocon). The excavation should extend laterally a minimum distance of 3 feet beyond the building footprint area or a distance equal to the depth of fill below the foundation, whichever is greater. Recommendations for earthwork are provided in the *Grading* section of this report (see Section 7.4).

7.1.6 Where miscellaneous subterranean improvements are planned (elevator pits and swimming pool), the structures may be supported on a conventional foundation system deriving support in the undisturbed older dune sand at or below a depth of 6 feet below the existing ground surface. If necessary, these miscellaneous improvements may derive support in a combination of newly placed engineered fill and undisturbed older dune sand at or below a depth of 6 feet below the existing ground surface. Recommendations for swimming pool and elevator pit design are provided in Sections 7.14 and 7.15 of this report, respectively.

7.1.7 It is the intent of the Geotechnical Engineer to allow foundations to derive support in both undisturbed older dune sand and newly placed engineered fill for this project, if project conditions warrant such an occurrence. All foundation excavations must be observed and approved in writing by the Geotechnical Engineer (a representative of Geocon), prior to placing steel or concrete.

7.1.8 Excavations up to 22 feet in vertical height are anticipated for construction of the subterranean levels, including foundation depths. Due to the depth of the excavation and the proximity to the property lines, city streets and adjacent offsite structures, excavation of the proposed subterranean level will likely require sloping and shoring measures in order to provide a stable excavation. Where shoring is required it is recommended that a soldier pile shoring system be utilized. In addition, where the proposed excavation will be deeper than and adjacent to an offsite structure, the proposed shoring should be designed to resist the surcharge imposed by the adjacent offsite structure. Recommendations for shoring are provided in Section 7.18 of this report.

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

7.1.10 Foundations for small outlying structures, such as block walls up to 6 feet in height, planter walls or trash enclosures, which will not be tied to the proposed structure, may be supported on conventional foundations deriving support on a minimum of 12 inches of newly placed engineered fill which extends laterally at least 12 inches beyond the foundation area. Where excavation and compaction cannot be performed, foundations may derive support directly in the undisturbed older dune sand at a depth of 24 inches, and should be deepened as necessary to maintain a minimum 12-inch embedment into the recommended bearing materials. If the soils exposed in the excavation bottom are soft or loose, compaction of the soils will be required prior to placing steel or concrete. Compaction of the foundation excavation bottom is typically accomplished with a compaction wheel or mechanical whacker and must be observed and approved by a Geocon representative.

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

7.1.12 Based on the results of percolation testing performed at the site, a stormwater infiltration system is considered feasible for this project. Recommendations for infiltration are provided in the *Stormwater Infiltration* section of this report (see Section 7.23).

7.1.13 Once the design and foundation loading configuration proceeds to a more finalized plan, the recommendations within this report should be reviewed and revised, if necessary. If the proposed building loads will exceed those presented herein, the potential for settlement should be reevaluated by this office.

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

7.2 Soil and Excavation Characteristics

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

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

7.2.3 All onsite excavations must be conducted in such a manner that potential surcharges from existing structures, construction equipment, and vehicle loads are resisted. The surcharge area may be defined by a 1:1 projection down and away from the bottom of an existing foundation or vehicle load. Penetrations below this 1:1 projection will require special excavation measures such as sloping and shoring. Excavation recommendations are provided in the *Temporary Excavations* section of this report (see Section 7.17).

7.2.4 The soils encountered during the investigation are considered to have a "very low" expansive potential (EI = 0) and are classified as "non-expansive" in accordance with the 2022 California Building Code (CBC) Section 1803.5.3. The recommendations presented herein assume that the proposed foundations and slabs will derive support in materials with a "very low" expansion potential.

7.3 Minimum Resistivity, pH, and Water-Soluble Sulfate

7.3.1 Potential of Hydrogen (pH) and resistivity testing as well as chloride content testing were performed on representative samples of soil to generally evaluate the corrosion potential to surface utilities. The tests were performed in accordance with California Test Method Nos. 643 and 422 and indicate that the upper site soils are considered "moderately corrosive" with respect to corrosion of buried ferrous metals on site. The results are presented in Appendix B (Figure B35) and should be considered for design of underground structures.

7.3.2 Laboratory tests were performed on representative samples of the site materials to measure the percentage of water-soluble sulfate content. Results from the laboratory water-soluble sulfate tests are presented in Appendix B (Figure B35) and indicate that the on-site materials possess a sulfate exposure class of "S0" to concrete structures as defined by 2022 CBC Section 1904 and ACI 318-19 Chapter 19.

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

7.4 Grading

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

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

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

7.4.4 The proposed structure may be supported on a conventional foundation system. At the subterranean levels, the conventional foundation system may derive support in the undisturbed older dune sand at or below a depth of 6 feet below the existing ground surface. For on-grade portions of the structure, the conventional foundation system may derive support in newly placed engineered fill. All foundation excavations must be observed and approved in writing by the Geotechnical Engineer prior to placement of steel or concrete.

7.4.5 As a minimum, the upper 6 feet of existing site soils within the proposed on-grade footprint areas should be excavated and properly compacted for foundation and slab support. Excavation should be conducted as necessary to completely remove all artificial fill and any soft, unsuitable paralic deposits at the direction of the Geotechnical Engineer (a representative of Geocon). The excavation should extend laterally a minimum distance of 3 feet beyond the building footprint area or a distance equal to the depth of fill below the foundation, whichever is greater.

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

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

7.4.8 Foundations for small outlying structures, such as block walls up to 6 feet in height, planter walls or trash enclosures, which will not be tied to the proposed structure, may be supported on conventional foundations deriving support on a minimum of 12 inches of newly placed engineered fill which extends laterally at least 12 inches beyond the foundation area. Where excavation and compaction cannot be performed, foundations may derive support directly in the undisturbed older dune sand at a depth of 24 inches, and should be deepened as necessary to maintain a minimum 12-inch embedment into the recommended bearing materials. If the soils exposed in the excavation bottom are soft or loose, compaction of the soils will be required prior to placing steel or concrete. Compaction of the foundation excavation bottom is typically accomplished with a compaction wheel or mechanical whacker and must be observed and approved by a Geocon representative.

7.4.9. Where new paving is to be placed, it is recommended that all existing fill and soft soils be excavated and properly compacted for paving support. As a minimum, the upper 12 inches of soil should be scarified, moisture conditioned to near optimum moisture content, and compacted to at least 95 percent relative compaction, as determined by ASTM Test Method D 1557 (latest edition). Paving recommendations are provided in *Preliminary Pavement Recommendations* section of this report (see Section 7.10).

7.4.10 All imported fill shall be observed, tested, and approved by Geocon West, Inc. prior to bringing soil to the site. Rocks larger than 6 inches in diameter shall not be used in the fill. If necessary, import soils used as structural fill should have an expansion index less than 20 and corrosivity properties that are equally or less detrimental to that of the existing onsite soils (see Figure B35). Import soils placed in the building area should be placed uniformly across the building pad or in a manner that is approved by the Geotechnical Engineer (a representative of Geocon).

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

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

7.5 Foundation Design

7.5.1 At the subterranean levels, the conventional foundation system may derive support in the undisturbed older dune sand at or below a depth of 6 feet below the existing ground surface. For on-grade portions of the structure, the conventional foundation system may derive support in newly placed engineered fill. The transition area for the on-grade portion to the subterranean portion of the structure should be more heavily reinforced to resist differential settlement stresses which could cause cracking. All foundation excavations must be observed and approved in writing by the Geotechnical Engineer prior to placement of steel or concrete.

7.5.2 It is the intent of the Geotechnical Engineer to allow foundations to derive support in both undisturbed older dune sand and newly placed engineered fill for this project, if project conditions warrant such an occurrence. All foundation excavations must be observed and approved in writing by the Geotechnical Engineer (a representative of Geocon), prior to placing steel or concrete.

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

7.5.4 Isolated spread foundations may be designed for an allowable bearing capacity of 2,500 psf, and should be a minimum of 24 inches in width, 18 inches in depth below the lowest adjacent grade, and 12 inches into the recommended bearing material.

7.5.5 The allowable soil bearing pressure above may be increased by 250 psf and 500 psf for each additional foot of foundation width and depth, respectively, up to a maximum allowable soil bearing pressure of 6,000 psf.

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

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

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

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

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

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

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

7.6 Foundation Settlement

7.6.1 The maximum expected static settlement for a subterranean structure supported on a conventional foundation system deriving support in undisturbed older dune sand at and below a depth of 6 feet below the existing ground surface and designed with a maximum bearing pressure of 6,000 psf is estimated to be less than 1 inch and occur below the heaviest loaded structural element. Settlement of the foundation system is expected to occur on initial application of loading. Differential settlement is not expected to exceed ½ inch over a distance of 20 feet.

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

7.6.3 In order to minimize differential settlement across the stepped transitions between the ground floor and subterranean levels, it is recommended that the transition areas be more heavily reinforced. The configuration and reinforcement of the structural connection should be designed by the project structural engineer.

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

7.7 Miscellaneous Foundations

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

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

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

7.8 Lateral Design

7.8.1 Resistance to lateral loading may be provided by friction acting at the base of foundations, slabs and by passive earth pressure. An allowable coefficient of friction of 0.35 may be used with the dead load forces in the undisturbed older dune sand or newly placed engineered fill.

7.8.2 Passive earth pressure for the sides of foundations and slabs poured against undisturbed older dune sand or newly placed engineered fill may be computed as an equivalent fluid having a density of 250 pounds per cubic foot (pcf) with a maximum earth pressure of 2,500 psf. When combining passive and friction for lateral resistance, the passive component should be reduced by one-third.

7.9 Concrete Slabs-on-Grade

7.9.1 Exterior concrete slabs-on-grade subject to vehicle loading should be designed in accordance with the recommendations in the *Preliminary Pavement Recommendations* section of this report (Section 7.10).

7.9.2 Unless specifically evaluated and designed by a qualified structural engineer, concrete slabs-on-grade for structures, not subject to vehicle loading, should be a minimum of 4 inches thick and minimum slab reinforcement should consist of No. 3 steel reinforcing bars placed 18 inches on center in both horizontal directions. The slab-on-grade subject to vehicle loading should be a minimum of 5 inches thick and minimum slab reinforcement should consist of No. 3 steel reinforcing bars placed 18 inches on center in both horizontal directions. Steel reinforcing should be positioned vertically near the slab midpoint. The concrete slab-on-grade and ramp may derive support directly on the undisturbed older dune sand at the excavation bottom as well as compacted soils, if necessary. Any disturbed soils should be properly compacted for slab support.

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

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

7.9.5 For seismic design purposes, a coefficient of friction of 0.35 may be utilized between concrete slabs and subgrade soils without a moisture barrier, and 0.15 for slabs underlain by a moisture barrier.

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

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

7.10 Preliminary Pavement Recommendations

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

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

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

PRELIMINARY PAVEMENT DESIGN SECTIONS

Location	Estimated Traffic Index (TI)	Asphalt Concrete (inches)	Class 2 Aggregate Base (inches)
Automobile Parking And Driveways	4.0	3.0	4.0
Trash Truck & Fire Lanes	7.0	4.0	9.0

7.10.4 Asphalt concrete should conform to Section 203-6 of the "*Standard Specifications for Public Works Construction"* (Green Book). Class 2 aggregate base materials should conform to Section 26-1.02A of the "*Standard Specifications of the State of California, Department of Transportation"* (Caltrans). The use of Crushed Miscellaneous Base in lieu of Class 2 aggregate base is acceptable. Crushed Miscellaneous Base should conform to Section 200-2.4 of the "*Standard Specifications for Public Works Construction"* (Green Book).

7.10.5 Unless specifically designed and evaluated by the project structural engineer, where exterior concrete paving will be utilized for support of vehicles, it is recommended that the concrete be a minimum of 5 inches of concrete reinforced with No. 3 steel reinforcing bars placed 18 inches on center in both horizontal directions. Concrete paving supporting vehicular traffic should be underlain by a minimum of 4 inches of aggregate base and a properly compacted subgrade. The subgrade and base material should be compacted to 95 percent relative compaction as determined by ASTM Test Method D 1557 (latest edition).

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

7.11 Retaining Wall Design

7.11.1 The recommendations presented below are generally applicable to the design of rigid concrete or masonry retaining walls having a maximum height of 20 feet. In the event that walls higher than 20 feet are planned, Geocon should be contacted for additional recommendations.

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

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

RETAINING WALL WITH LEVEL BACKFILL SURFACE

HEIGHT OF RETAINING WALL (Feet)	ACTIVE PRESSURE EQUIVALENT FLUID PRESSURE (Pounds Per Cubic Foot)	AT-REST PRESSURE EQUIVALENT FLUID PRESSURE (Pounds Per Cubic Foot)
Up to 10	41	61
10 – 15	46	61
15 – 20	49	61

7.11.4 The wall pressures provided above assume that the retaining wall will be properly drained preventing the buildup of hydrostatic pressure. If retaining wall drainage is not implemented, an at-rest equivalent fluid pressure of 93 pcf should be used in design of undrained, restrained walls for the full height of the wall. The value includes hydrostatic pressures plus buoyant lateral earth pressures. If a partially drained wall is proposed, Geocon should be contacted to provide additional recommendations.

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

7.11.6 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.

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

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

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

and

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

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

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

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

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

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

and

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

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

then

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

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

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

7.11.10 Seismic lateral forces should be incorporated into the design as necessary, and recommendations for seismic lateral forces are presented below.

7.12 Dynamic (Seismic) Lateral Forces

7.12.1 The structural engineer should determine the seismic design category for the project in accordance with Section 1613 of the CBC. If the project possesses a seismic design category of D, E, or F, proposed retaining walls in excess of 6 feet in height should be designed with seismic lateral pressure (Section 1803.5.12 of the 2022 CBC).

7.12.2 A seismic load of 10 pcf should be used for design of walls that support more than 6 feet of backfill in accordance with Section 1803.5.12 of the 2022 CBC. The seismic load is applied as an equivalent fluid pressure along the height of the wall and the calculated loads result in a maximum load exerted at the base of the wall and zero at the top of the wall. This seismic load should be applied in addition to the active earth pressure. The earth pressure is based on half of two thirds of PGA_M calculated from ASCE 7-16 Section 11.8.3.

7.13 Retaining Wall Drainage

7.13.1 Retaining walls not designed for hydrostatic pressure should be provided with a drainage system. At the base of the drain system, a subdrain covered with a minimum of 12 inches of gravel should be installed, and a compacted fill blanket or other seal placed at the surface (see Figure 5). The clean bottom and subdrain pipe, behind a retaining wall, should be observed by the Geotechnical Engineer (a representative of Geocon), prior to placement of gravel or compacting backfill.

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

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

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

7.14 Swimming Pool

7.14.1 The proposed swimming pool should be designed as a free-standing structure deriving support in newly placed engineered fill and/or the undisturbed older dune sand at or below a depth of 6 feet below the existing ground surface. Swimming pool walls may be designed in accordance with the *Retaining Wall Design* section of this report (see Section 7.11). The proposed pools should be constructed utilizing an expansive soils design and 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.

7.14.2 If a spa is proposed it should be constructed independent of the swimming pool and must not be cantilevered from the swimming pool shell.

7.15 Elevator Pit Design

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

7.15.2 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.

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

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

7.16 Elevator Piston

7.16.1 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 shoring pile, or the drilled excavation could compromise the existing foundation or pile support, especially if the drilling is performed subsequent to the foundation or pile construction.

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

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

7.17 Temporary Excavations

7.17.1 Excavations up to 22 feet in height may be required for excavation and construction of the proposed subterranean levels and foundation system. The excavations are expected to expose artificial fill and older dune sand, which are subject to excessive caving where granular soils are encountered. Vertical excavations up to 5 feet in height may be attempted where loose soils or caving sands are not present, and where not surcharged by adjacent traffic or structures.

7.17.2 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. Where sufficient space is available, temporary unsurcharged embankments could be sloped back at a uniform 1:1 slope gradient or flatter up to maximum height of 8 feet. Alternatively, temporary embankments could be sloped back at a uniform 1½:1 slope gradient or flatter up to a maximum height of 16 feet. A uniform slope does not have a vertical portion. Where space is limited, shoring measures will be required. *Shoring* data is provided in Section 7.18 of this report.

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

7.18 Shoring – Soldier Pile Design and Installation

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

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

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

7.18.4 The proposed soldier piles may also be designed as permanent piles. The required pile depths, dimensions, and spacing should be determined and designed by the project structural and shoring engineers. All piles utilized for shoring can also be incorporated into a permanent retaining wall system (shotcrete wall) and should be designed in accordance with the earth pressure provided in the *Retaining Wall Design* section of this report (see Section 7.11).

7.18.5 Drilled cast-in-place soldier piles should be placed no closer than 3 diameters on center. The minimum diameter of the piles is 18 inches. Structural concrete should be used for the soldier piles below the excavation; lean-mix concrete may be employed above that level. As an alternative, lean-mix concrete may be used throughout the pile where the reinforcing consists of a wideflange section. The slurry must be of sufficient strength to impart the lateral bearing pressure developed by the wideflange section to the soil. For design purposes, an allowable passive value for the soils below the bottom plane of excavation may be assumed to be 250 psf per foot. Where piles are installed by vibration techniques, the passive pressure may be assumed to mobilize across a width equal to the two times the dimension of the beam flange. The allowable passive value may be doubled for isolated piles spaced a minimum of three times the pile diameter. To develop the full lateral value, provisions should be implemented to assure firm contact between the soldier piles and the undisturbed older dune sand.

7.18.6 Groundwater was not encountered during site exploration. However, groundwater levels can fluctuate and may be different at the time of construction. It is not uncommon for groundwater or seepage conditions to develop where none previously existed. Therefore, the contractor should be prepared for groundwater during pile installation should the need arise. If more than 6 inches of water is present in the bottom of the excavation, a tremie is required to place the concrete into the bottom of the hole. A tremie should consist of a rigid, water-tight tube having a diameter of not less than 6 inches with a hopper at the top. The tube should be equipped with a device that will close the discharge end and prevent water from entering the tube while it is being charged with concrete. The tremie should be supported so as to permit free movement of the discharge end over the entire top surface of the work and to permit rapid lowering when necessary to retard or stop the flow of concrete. The discharge end should be closed at the start of the work to prevent water entering the tube and should be entirely sealed at all times, except when the concrete is being placed. The tremie tube should be kept full of concrete. The flow should be continuous until the work is completed, and the resulting concrete seal should be monolithic and homogeneous. The tip of the tremie tube should always be kept about 5 feet below the surface of the concrete and definite steps and safeguards should be taken to ensure that the tip of the tremie tube is never raised above the surface of the concrete.

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

7.18.8 Casing will likely be required since caving is expected in the granular soils, and the contractor should have casing available prior to commencement of pile excavation. When casing is used, extreme care should be employed so that the pile is not pulled apart as the casing is withdrawn. At no time should the distance between the surface of the concrete and the bottom of the casing be less than 5 feet. As an alternative, piles may be vibrated into place; however, there is always a risk that excessive vibrations in sandy soils could induce settlements and distress to adjacent offsite improvements. Continuous observation of the drilling and pouring of the piles by the Geotechnical Engineer (a representative of Geocon West, Inc.), is required.

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

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

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

7.18.12 Based on Table 19 of the *Transportation and Construction Vibration Guidance Manual* (Caltrans 2020), a continuous source of vibrations (ex. vibratory pile driving) which generates a maximum peak particle velocity of 0.5 in/sec is considered tolerable for modern industrial/commercial buildings and new residential structures. The Client should be aware that a lower value may be necessary if older or fragile structures are in the immediate vicinity of the site.

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

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

7.18.15 The frictional resistance between the soldier piles and retained soil may be used to resist the vertical component of the anchor load. The coefficient of friction may be taken as 0.35 based on uniform contact between the steel beam and lean-mix concrete and retained earth. The portion of soldier piles below the plane of excavation may also be employed to resist the downward loads. The downward capacity may be determined using a frictional resistance of 380 psf.

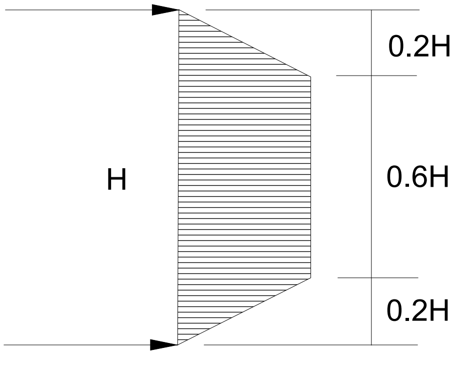
7.18.16 Due to the nature of the site soils, it is expected that continuous lagging between soldier piles will be required. However, it is recommended that the exposed soils be observed by the Geotechnical Engineer (a representative of Geocon West, Inc.), to verify the presence of any competent, cohesive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

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

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

HEIGHT OF SHORING (FEET)	**EQUIVALENT FLUID PRESSURE (Pounds Per Cubic Foot) (ACTIVE PRESSURE)**	**EQUIVALENT FLUID PRESSURE Trapezoidal (Where H is the height of the shoring in feet)**
Up to 10	25	16H
10 – 15	30	19H
15 – 22	35	22H

Trapezoidal Distribution of Pressure



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

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

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

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

and

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

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

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

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

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

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

and

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

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

then

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

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

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

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

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

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

7.19 Temporary Tie-Back Anchors

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

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

- 5 feet below the top of the excavation – 700 pounds per square foot
- 10 feet below the top of the excavation – 1,000 pounds per square foot

7.19.3 Depending on the techniques utilized, and the experience of the contractor performing the installation, a maximum allowable friction capacity of 2.2 kips per linear foot for post-grouted anchors (for a minimum 20 foot length beyond the active wedge) may be assumed for design purposes. Only the frictional resistance developed beyond the active wedge should be utilized in resisting lateral loads.

7.20 Anchor Installation

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

7.21 Anchor Testing

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

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

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

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

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

7.22 Internal Bracing

7.22.1 Rakers may be utilized to brace the soldier piles in lieu of tieback anchors. The raker bracing could be supported laterally by temporary concrete footings (deadmen) or by the permanent, interior footings. For design of such temporary footings or deadmen, poured with the bearing surface normal to rakers inclined at 45 degrees, a bearing value of 1,500 psf may be used, provided the shallowest point of the footing is at least one foot below the lowest adjacent grade. The structural engineer should review the shoring plans to determine if raker footings conflict with the structural foundation system. The client should be aware that the utilization of rakers could significantly impact the construction schedule due to their intrusion into the construction site and potential interference with equipment.

7.23 Stormwater Infiltration

7.23.1 During the site exploration on June 2, 2022, borings B1 and B7 were utilized to perform percolation testing. Boring B1 was drilled and subsequently backfilled to a depth of 40 feet. Slotted casing was placed in the borings, and the annular space between the casing and excavations were filled with filter pack. The borings were then filled with water to pre-saturate the soils. After pre-saturating the soils, the casings were refilled with water and percolation test readings were performed after repeated flooding of the cased excavation. Based on the test results, the measured percolation rate and design infiltration rate, for the earth materials encountered, are provided in the following table. These values have been calculated in accordance with the Small Diameter Boring Infiltration Test Procedure in the County of Los Angeles Department of Public Works GMED *Guidelines for Geotechnical Investigation and Reporting, Low Impact Development Stormwater Infiltration* (June 2021). Percolation test field data and calculation of the measured percolation rate and design infiltration rate are provided on Figures 7 and 8.

Boring	Soil Type	Infiltration Depth (ft)	Measured Percolation Rate (in / hour)	Design Infiltration Rate (in / hour)
B1	CL-ML / SP	30 – 40	3.36	1.12
B7	SM	3 – 5	3.96	1.32

7.23.2 Based on the test method utilized (Small Diameter Boring), the reduction factor RF_t may be taken as 1.0 in the infiltration system design. Based on the number of tests performed and consistency of the soils throughout the site, it is suggested that the reduction factor RF_v be taken as 1.0. In addition, provided proper maintenance is performed to minimize long-term siltation and plugging, the reduction factor RF_s may be taken as 1.0. Additional reduction factors may be required and should be applied by the engineer in responsible charge of the design of the stormwater infiltration system and based on applicable guidelines.

7.23.3 The results of the percolation testing indicate that the soils at depths in the above table are conducive to infiltration. It is our opinion that the soil zone encountered at the depth and location as listed in the table above are suitable for infiltration of stormwater.

7.23.4 It is our further opinion that infiltration of stormwater and will not induce excessive hydro-consolidation (see Figures B15 through B28), will not create a perched groundwater condition, will not affect soil structure interaction of existing or proposed foundations due to expansive soils, will not saturate soils supported by existing retaining walls, and will not increase the potential for liquefaction. Resulting settlements are anticipated to be less than ¼ inch, if any.

7.23.5 The infiltration system must be located such that the closest distance between an adjacent foundation is at least 15 feet in all directions from the zone of saturation. The zone of saturation may be assumed to project downward from the discharge of the infiltration facility at a gradient of 1:1. Additional property line or foundation setbacks may be required by the governing jurisdiction and should be incorporated into the stormwater infiltration system design as necessary.

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

7.23.7 Subsequent to the placement of the infiltration system, it is acceptable to backfill the resulting void space between the excavation sidewalls and the infiltration system with minimum two-sack slurry provided the slurry is not placed in the infiltration zone. It is recommended that pea gravel be utilized adjacent to the infiltration zone so communication of water to the soil is not hindered.

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

7.24 Surface Drainage

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

7.24.2 All site drainage should be collected and controlled in non-erosive drainage devices. Drainage should not be allowed to pond anywhere on the site, and especially not against any foundation or retaining wall. The site should be graded and maintained such that surface drainage is directed away from structures in accordance with 2022 CBC 1804.4 or other applicable standards. In addition, drainage should not be allowed to flow uncontrolled over any descending slope. Discharge from downspouts, roof drains and scuppers are not recommended onto unprotected soils within 5 feet of the building perimeter. Planters which are located adjacent to foundations should be sealed to prevent moisture intrusion into the soils providing foundation support. Landscape irrigation is not recommended within 5 feet of the building perimeter footings except when enclosed in protected planters.

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

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

7.25 Plan Review

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

LIMITATIONS AND UNIFORMITY OF CONDITIONS

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

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

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

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

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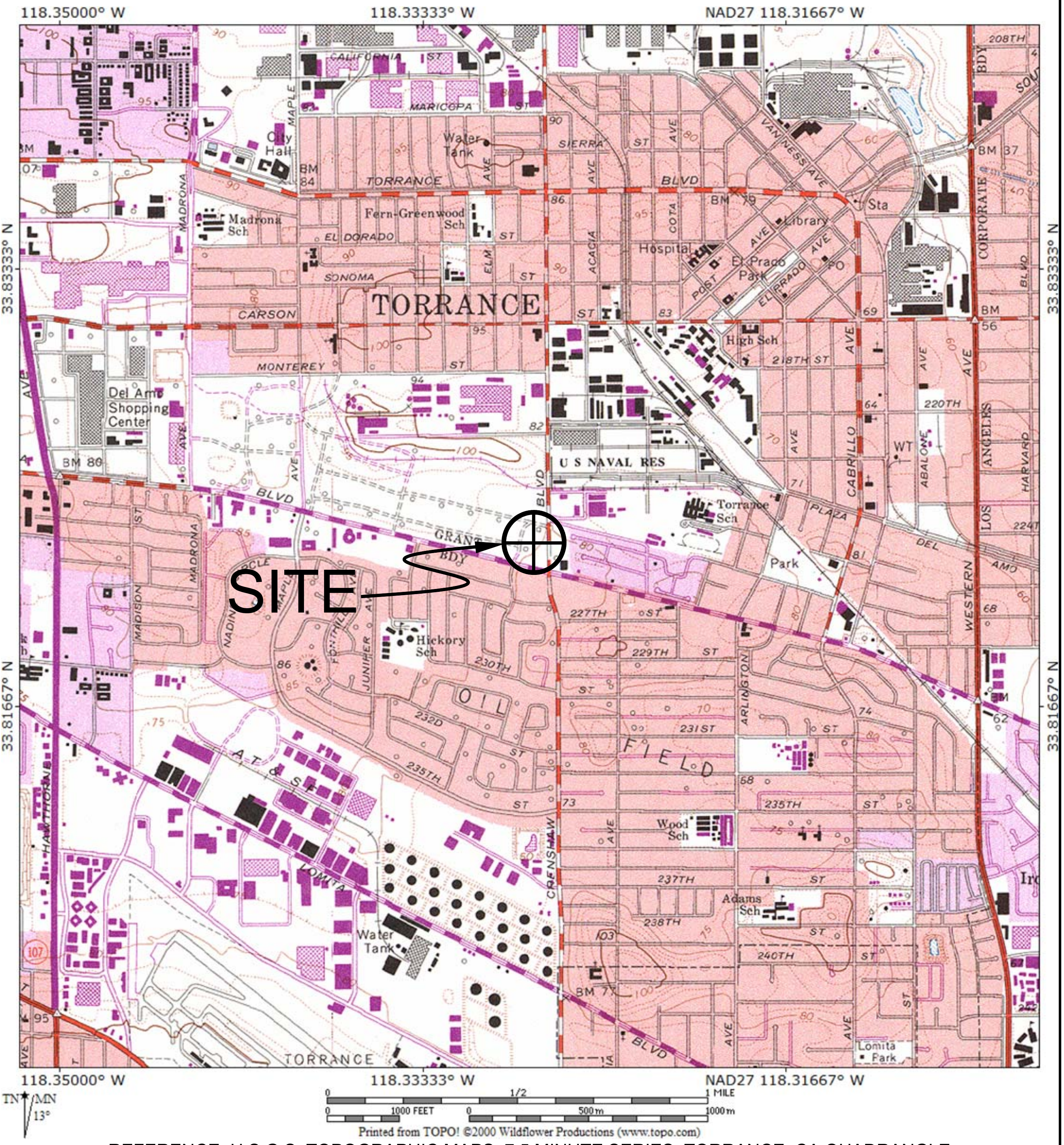

REFERENCE: U.S.G.S. TOPOGRAPHIC MAPS, 7.5 MINUTE SERIES, TORRANCE, CA QUADRANGLE

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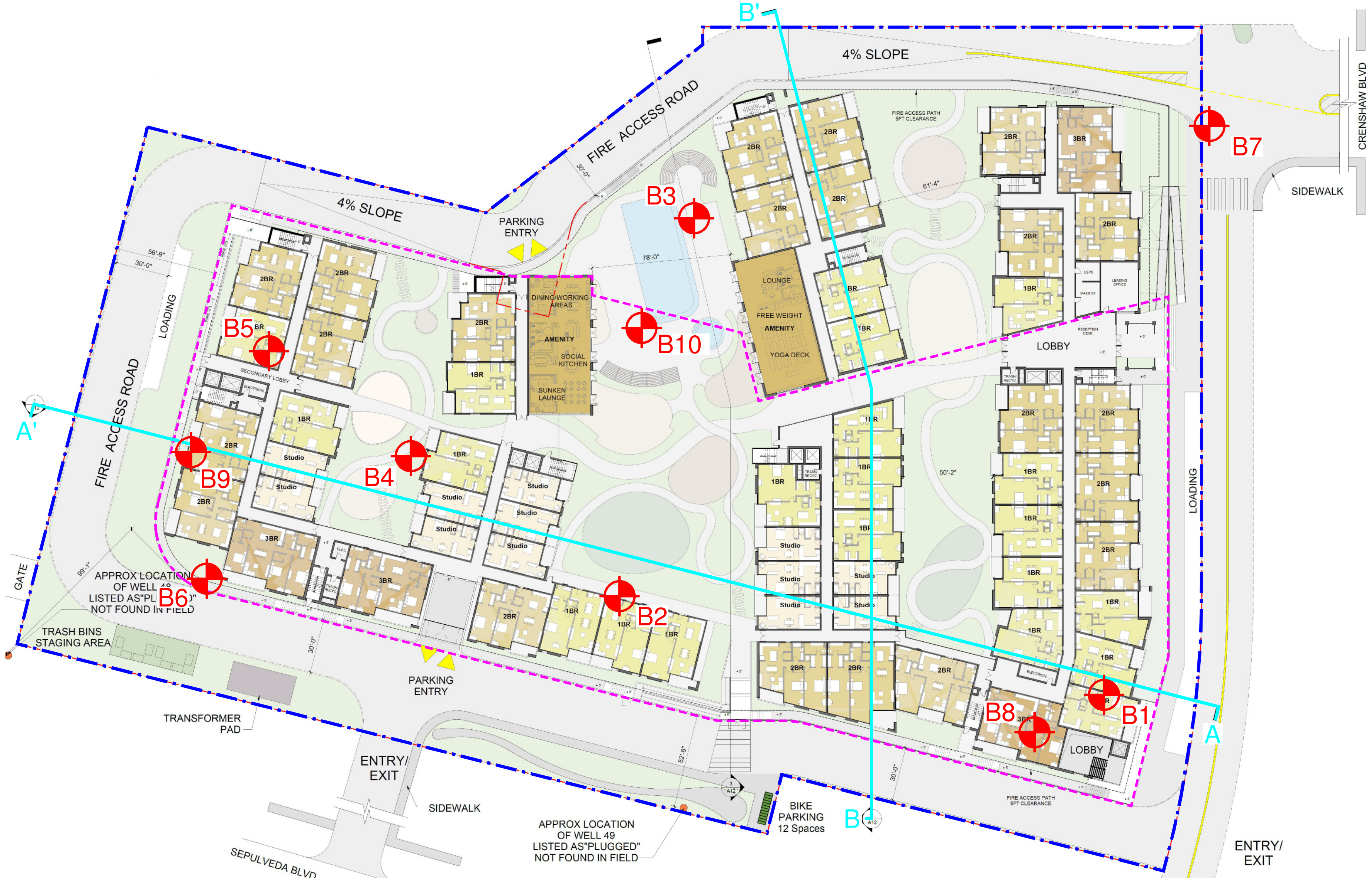
VICINITY MAP

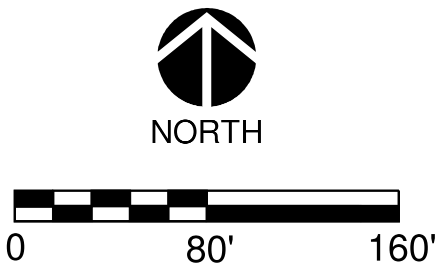
2325 CRENSHAW BOULEVARD
TORRANCE, CALIFORNIA

OCT. 2023	PROJECT NO. W1572-06-01A	FIG. 1



LEGEND

⊕ B10	Approximate Location of Boring
▭	Approximate Location of Property Line
▭	Limits of Subterranean Level Below



SECTION B - B'

SECTION A - A'

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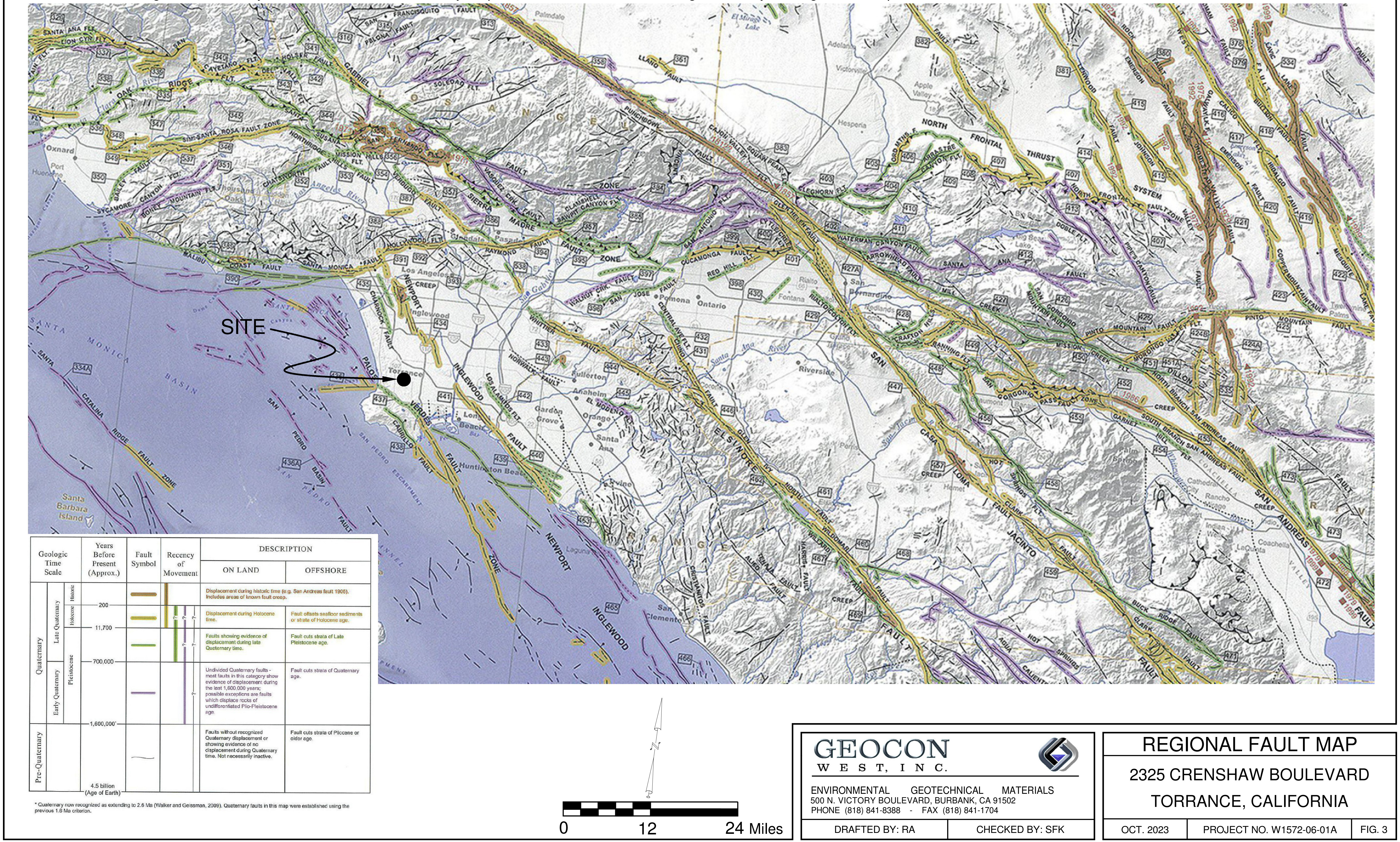
SITE PLAN

2325 CRENSHAW BOULEVARD
TORRANCE, CALIFORNIA

OCT. 2023	PROJECT NO. W1572-06-01A	FIG. 2

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



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

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

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DRAFTED BY: RA
CHECKED BY: SFK

REGIONAL FAULT MAP

2325 CRENSHAW BOULEVARD
TORRANCE, CALIFORNIA

OCT. 2023
PROJECT NO. W1572-06-01A
FIG. 3

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

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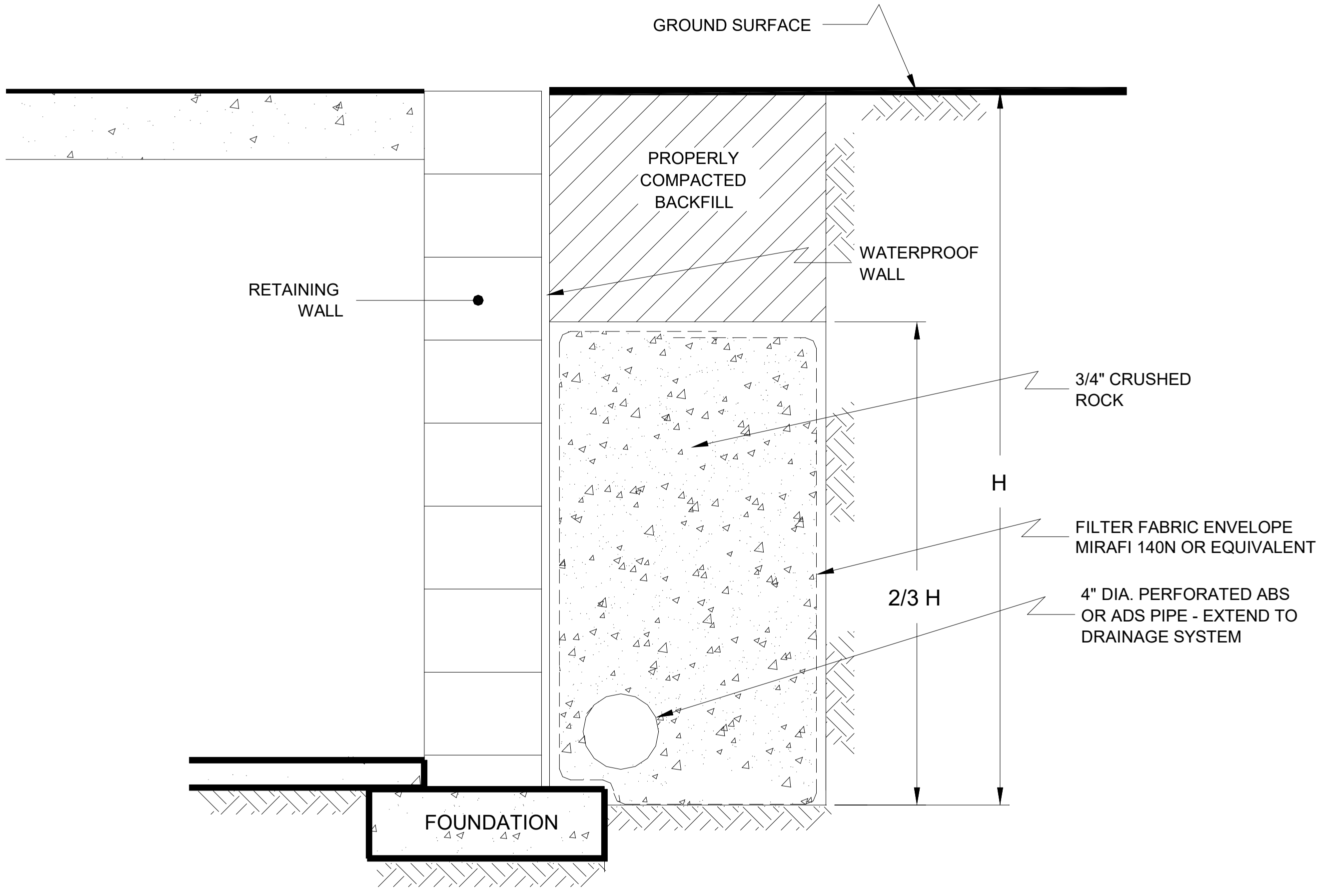
REGIONAL SEISMICITY MAP

2325 CRENSHAW BOULEVARD
TORRANCE, CALIFORNIA

OCT. 2023	PROJECT NO. W1572-06-01A	FIG. 4



NO SCALE

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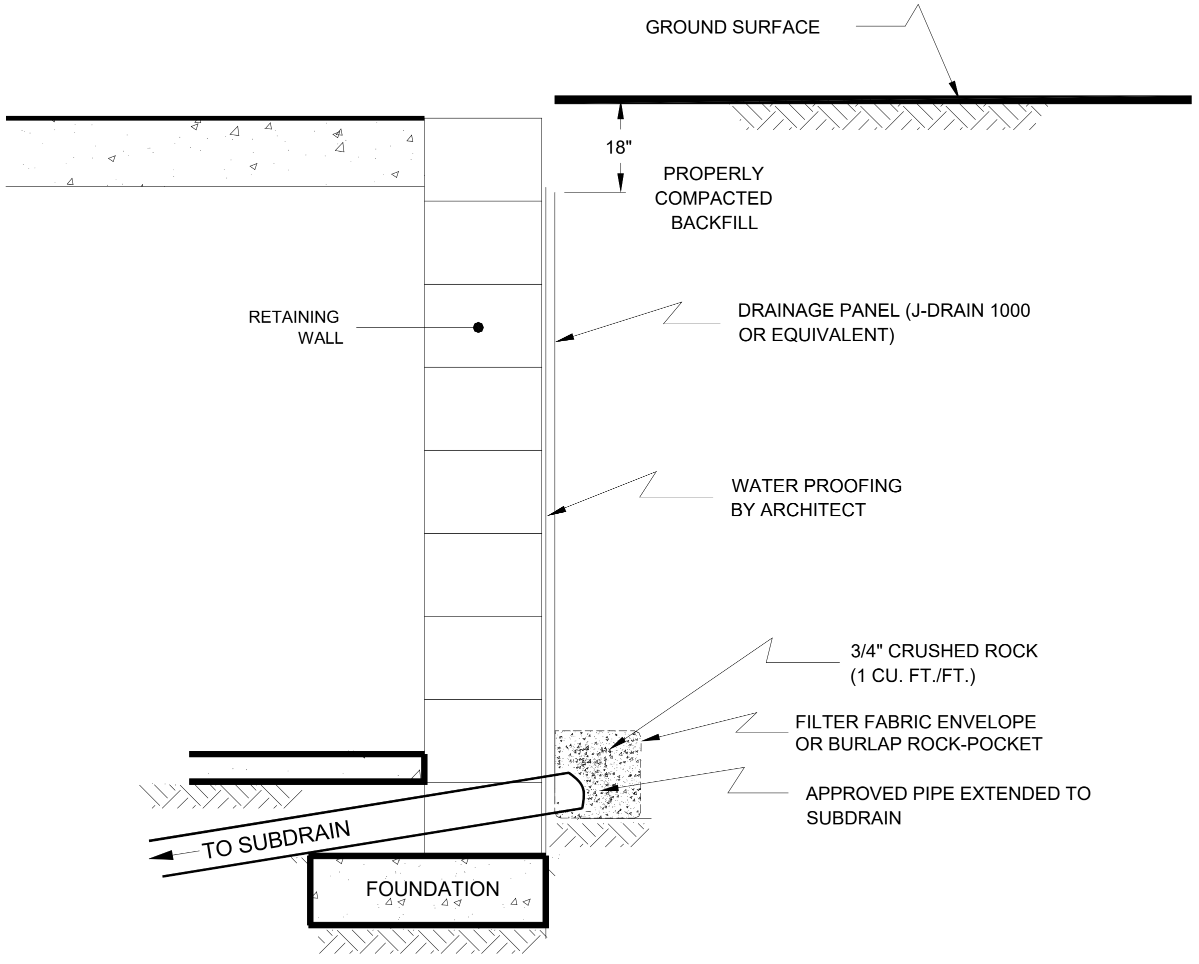
RETAINING WALL DRAIN DETAIL

2325 CRENSHAW BOULEVARD
TORRANCE, CALIFORNIA

DRAFTED BY: PZ	CHECKED BY: HHD	OCT. 2023	PROJECT NO. W1572-06-01A	FIG. 5



NO SCALE

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RETAINING WALL DRAIN DETAIL

2325 CRENSHAW BOULEVARD
TORRANCE, CALIFORNIA

DRAFTED BY: PZ	CHECKED BY: HHD	OCT. 2023	PROJECT NO. W1572-06-01A	FIG. 6

BORING PERCOLATION TEST FIELD LOG

Date: 6/3/2022
Project Number: W1572-06-01A
Project Location: 2325 Crenshaw Blvd, Torrance
Earth Description: CL-ML / SP
Tested By: JS
Liquid Description: Clear Clean Tap Water
Measurement Method: Sounder

Boring/Test Number: B1
Diameter of Boring: 8 inches
Diameter of Casing: 2 inches
Depth of Boring: 40 feet
Depth to Invert of BMP: 30 feet
Depth to Water Table: > 50 feet
Depth to Initial Water Depth (d_1): 360 inches

Start Time for Pre-Soak: 7:13 AM
Start Time for Standard: 7:40 AM

Water Remaining in Boring (Y/N): No
Standard Time Interval Between Readings: 10 min

Reading Number	**Time Start (hh:mm)**	**Time End (hh:mm)**	**Elapsed Time Δtime (min)**	**Water Drop During Standard Time Interval, Δd (in)**	**Soil Description Notes Comments**
1	7:40 AM	7:50 AM	10	40.3	
2	7:52 AM	8:02 AM	10	37.1	
3	8:05 AM	8:15 AM	10	36.4	
4	8:18 AM	8:28 AM	10	37.1	
5	8:31 AM	8:41 AM	10	36.4	
6	9:43 AM	9:53 AM	10	35.3	
7	8:56 AM	9:06 AM	10	35.5	
8	9:09 AM	9:19 AM	10	37.1	
9	9:22 AM	9:32 AM	10	35.5	
10	9:35 AM	9:45 AM	10	35.3	
11	9:48 AM	9:58 AM	10	34.9	
12	10:01 AM	10:11 AM	10	36.4	
13	10:16 AM	10:26 AM	10	34.6	
14	10:30 AM	10:40 AM	10	34.3	
15	10:42 AM	10:52 AM	10	34.1	
16	4:56 PM	5:06 PM	10	34.6	
17	11:09 AM	11:19 AM	10	34.1	
18	11:21 AM	11:31 AM	10	33.8	Stabilized Readings Achieved

MEASURED PERCOLATION RATE & DESIGN INFILTRATION RATE CALCULATIONS*

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

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

A = **3066** in^2

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

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

Reading 16 V = **1737** in^3
Reading 17 V = **1713** in^3
Reading 18 V = **1701** in^3

Percolation Rate = **3.40** inches/hour
Percolation Rate = **3.35** inches/hour
Percolation Rate = **3.33** inches/hour

Measured Percolation Rate = **3.36** inches/hour

Reduction Factors

Small Diameter Boring, RF_t = 1
Site Variability, RF_v = 1
Long Term Siltation, RF_s = 1

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

Total Reduction Factor = 3

Design Infiltration Rate

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

Design Infiltration Rate = **1.12** inches/hour

FIGURE 7

BORING PERCOLATION TEST FIELD LOG

Date: 6/3/2022
Project Number: W1572-06-01A
Project Location: 2325 Crenshaw Blvd, Torrance
Earth Description: SM
Tested By: JS
Liquid Description: Clear Clean Tap Water
Measurement Method: Sounder

Boring/Test Number: B7
Diameter of Boring: 4 inches
Diameter of Casing: 2 inches
Depth of Boring: 5 feet
Depth to Invert of BMP: 3 feet
Depth to Water Table: > 50 feet
Depth to Initial Water Depth (d_1): 36 inches

Start Time for Pre-Soak: 7:19 AM
Start Time for Standard: 7:46 AM

Water Remaining in Boring (Y/N): No
Standard Time Interval Between Readings: 10 min

Reading Number	Time Start (hh:mm)	Time End (hh:mm)	Elapsed Time Δtime (min)	Water Drop During Standard Time Interval, Δd (in)	Soil Description Notes Comments
1	7:46 AM	7:56 AM	10	18.1	
2	7:59 AM	8:09 AM	10	17.9	
3	8:12 AM	8:22 AM	10	17.9	
4	8:25 AM	8:35 AM	10	17.5	
5	8:38 AM	8:48 AM	10	24.0	
6	9:01 AM	9:11 AM	10	16.7	
7	9:14 AM	9:24 AM	10	16.7	
8	9:28 AM	9:38 AM	10	16.6	
9	9:41 AM	9:51 AM	10	16.3	
10	9:55 AM	10:05 AM	10	16.3	
11	10:08 AM	10:18 AM	10	16.3	
12	10:21 AM	10:31 AM	10	16.7	
13	10:35 AM	10:45 AM	10	16.6	
14	10:49 AM	10:59 AM	10	16.7	
15	11:03 AM	11:13 AM	10	16.3	
16	11:17 AM	11:27 AM	10	16.4	
17	11:30 AM	11:40 AM	10	16.4	
18	11:43 AM	11:53 AM	10	16.6	Stabilized Readings Achieved

MEASURED PERCOLATION RATE & DESIGN INFILTRATION RATE CALCULATIONS*

Boring Radius, r: 2 inches
Test Section Height, h: 24.0 inches

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

A = **314** in^2

$$\mathit{Discharged\ Water\ Volume}, V = \pi r^2 \Delta \text{d}$$

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

Reading 16	V =	**207**	in^3	Percolation Rate =	**3.95**	inches/hour
Reading 17	V =	**207**	in^3	Percolation Rate =	**3.95**	inches/hour
Reading 18	V =	**208**	in^3	Percolation Rate =	**3.97**	inches/hour

Measured Percolation Rate = **3.96** inches/hour

Reduction Factors

Small Diameter Boring, RF_t = 1
Site Variability, RF_v = 1
Long Term Siltation, RF_s = 1

$$\mathit{Total\ Reduction\ Factor}, RF = RF_t + RF_v + RF_s$$

Total Reduction Factor = 3

Design Infiltration Rate

$$\mathit{Design\ Infiltration\ Rate} = \mathit{Measured\ Percolation\ Rate}\ /RF$$

Design Infiltration Rate = **1.32** inches/hour

FIGURE 8

APPENDIX A

APPENDIX A

FIELD INVESTIGATION

The site was explored on June 2, 2022, by excavating six 8-inch diameter borings to depths ranging from approximately 25½ to 50½ feet below the existing ground surface using a truck-mounted hollow-stem auger drilling machine. In addition, one boring was excavated with hand tools to a depth of approximately 5 feet beneath the ground surface for the purpose of percolation testing. Additional site exploration was performed on September 23, 2023, by excavating three 8-inch diameter borings using a truck-mounted hollow-stem auger drilling machine to depths of approximately 20½ and 30½ feet below the existing ground surface. Representative and relatively undisturbed samples were obtained by driving a 3-inch, O. D., California Modified Sampler into the "undisturbed" soil mass with blows from a 140-pound auto-hammer falling 30 inches. The California Modified Sampler was equipped with 1-inch by 2³/₈-inch diameter brass sampler rings to facilitate soil removal and testing. Bulk samples were also obtained.

The soil conditions encountered in the borings were visually examined, classified and logged in general accordance with the Unified Soil Classification System (USCS). The logs of the borings are presented on Figures A1 through A10. The logs depict the soil and geologic conditions encountered and the depth at which samples were obtained. The logs also include our interpretation of the conditions between sampling intervals. Therefore, the logs contain both observed and interpreted data. We determined the lines designating the interface between soil materials on the logs using visual observations, penetration rates, excavation characteristics and other factors. The transition between materials may be abrupt or gradual. Where applicable, the logs were revised based on subsequent laboratory testing. The location of the borings are shown on Figure 2.

PROJECT NO. W1572-06-01

BORING 1

ELEV. (MSL.) **- -** DATE COMPLETED **06/02/2022**

EQUIPMENT **HOLLOW STEM AUGER** BY: **JC**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0	BULK 0-5' ☒				**CONCRETE: 5" BASE: NONE** **ARTIFICIAL FILL** Silty Sand, medium dense, dry to slightly moist, light yellowish brown, fine- to medium-grained.			
2								
4	B1@5' ■				**ALLUVIUM** Silty Sand, medium dense, dry to slightly moist, dark yellowish brown, fine- to medium-grained.	41	109.2	5.3
6								
					- light yellowish brown			
8								
10	B1@10' ■					27	105.6	3.9
12								
				SM				
14								
	B1@15' ■				- mottled, oxidation staining	37	116.3	11.5
16								
18								
20	B1@20' ◧				- trace clay	39		9.1
22								
24								
					Silty Clay, firm, moist, dark yellowish brown, fine- to medium-grained.			
26								
				CL-ML				
28								

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

W1572-06-01A 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.

BORING 1

ELEV. (MSL.) **- -** DATE COMPLETED **06/02/2022**

EQUIPMENT **HOLLOW STEM AUGER** BY: **JC**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
30	B1@30' ■					10		36.2
32								
34				CL-ML				
36								
38								
40	B1@40' ■				Sand, poorly graded, dense, slightly moist, light olive brown, oxidation staining, trace silt.	50 (5")	99.3	7.3
42								
44				SP				
46								
48								
50	B1@50' ■					50 (4")	102.5	7.1

Total depth of boring: 50.5 feet
Fill to 4 feet.
No groundwater encountered.
Percolation testing performed.
Backfilled with soil cuttings and tamped.
Concrete patched.

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

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

SAMPLE SYMBOLS

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

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

PROJECT NO. W1572-06-01

BORING 2

ELEV. (MSL.) - - DATE COMPLETED **06/02/2022**

EQUIPMENT **HOLLOW STEM AUGER** BY: **JC**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0					**AC: 5" BASE: 5"** **ARTIFICIAL FILL** Silty Sand, medium dense, slightly moist, very dark grayish brown.			
2	B2@2.5'				- mottled, oxidation staining	51	113.7	7.6
4					**ALLUVIUM** Silty Sand, medium dense, slightly moist, dark yellowish brown.			
6	B2@5'					51	107.7	4.7
8	B2@7.5'				- dense, mottled, oxidation stains	72	121.0	6.5
10	B2@10'			SM	- light brown	50 (6")	119.7	7.6
12								
14								
16	B2@15'				- increase in mottling	32	125.5	10.2
18								
20	B2@20'			CL	Clay, stiff, slightly moist to moist, olive brown.	41	115.9	20.3
22					Sand, poorly graded, dense, slightly moist to moist, dark yellowish brown, fine- to medium-grained, trace silt.			
24								
26	B2@25'			SP		50 (5")	116.0	15.4
28								

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

W1572-06-01A BORING LOGS.GPJ

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

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

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BORING 2

ELEV. (MSL.) **- -** DATE COMPLETED **06/02/2022**

EQUIPMENT **HOLLOW STEM AUGER** BY: **JC**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
30	B2@30' ■				- dark brown, increase in medium-grained	50 (5.5")	104.0	2.0
32				SP				
34								
	B2@35' ■				- light olive brown, increase in fine-grained	50 (6")	99.4	9.8

Total depth of boring: 35.5 feet
Fill to 3 feet.
No groundwater encountered.
Backfilled with soil cuttings and tamped.
AC patched.

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

W1572-06-01A BORING LOGS.GPJ

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

SAMPLE SYMBOLS

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

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

PROJECT NO. W1572-06-01

BORING 3

ELEV. (MSL.) **- -** DATE COMPLETED **06/02/2022**

EQUIPMENT **HOLLOW STEM AUGER** BY: **JC**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0	BULK 0-5'				**AC: 3" BASE: 5"** **ARTIFICIAL FILL** Silty Sand, medium dense, slightly moist, dark yellowish brown, fine- to medium-grained. - dark grayish brown			
4				SM	**ALLUVIUM** Silty Sand, dense, slightly moist to moist, brown, fine- to medium-grained.			
	B3@5'					60	105.2	16.5
8	B3@7.5'				- mottled, dark yellowish brown and brown	62	121.3	11.7
10	B3@10'					67	115.9	13.6
				CL	Sandy Clay, stiff, slightly moist to moist, dark brown, fine- to medium-grained.			
	B3@15'					47	129.6	9.9
20	B3@20'				- hard, slightly moist, dark olive brown	62	124.4	13.2
				SP	Sand, poorly graded, dense, slightly moist, dark olive brown, fine- to medium-grained.			
	B3@25'					50 (5")	109.3	7.8

W1572-06-01A BORING LOGS.GPJ

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

SAMPLE SYMBOLS

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

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. W1572-06-01

BORING 3

ELEV. (MSL.) - - DATE COMPLETED 06/02/2022

EQUIPMENT **HOLLOW STEM AUGER** BY: **JC**

DEPTH IN FEET	SAMPLE NO.		LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
30	B3@30'	■	░		SP	- light olive brown	50 (6")	97.7	3.5

Total depth of boring: 30.5 feet
Fill to 4 feet.
No groundwater encountered.
Backfilled with soil cuttings and tamped.
AC patched.

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

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

W1572-06-01A BORING LOGS.GPJ

SAMPLE SYMBOLS

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

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

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BORING 4

ELEV. (MSL.) - - DATE COMPLETED 06/02/2022

EQUIPMENT **HOLLOW STEM AUGER** BY: **JC**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0					**AC: 3" BASE: 3"**			
					ARTIFICIAL FILL Silty Sand, medium dense, slightly moist, dark grayish brown.			
2					**ALLUVIUM** Silty Sand, dense, slightly moist, brown, fine- to medium-grained.			
4								
	B4@5' ■					74	110.5	6.1
6								
8				SM				
10	B4@10' ■				- medium dense, dark yellowish brown, mottled	42	106.8	10.8
12								
14								
	B4@15' ■				Sandy Clay, stiff, slightly moist to moist, dark yellowish brown, fine- to medium-grained, mottled.	41	125.9	10.4
16								
18								
				CL				
20	B4@20' ■					68	118.9	14.1
22								
24								
	B4@25' ■				Silty Clay, stiff, moist, dark olive brown, mottled with dark yellowish brown.	30	90.2	33.1
26								
				CL-ML				
28								

Figure A4,
Log of Boring 4, Page 1 of 2

SAMPLE SYMBOLS

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

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

PROJECT NO. W1572-06-01

BORING 4

ELEV. (MSL.) **- -** DATE COMPLETED **06/02/2022**

EQUIPMENT **HOLLOW STEM AUGER** BY: **JC**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
30	B4@30' ■	⁙		SP	Sand, poorly graded, dense, slightly moist, mottled with dark yellowish brown and dark olive brown, fine- to medium-grained.	50 (5")	113.7	15.6

Total depth of boring: 30.5 feet
Fill to 1 foot.
No groundwater encountered.
Backfilled with soil cuttings and tamped.
AC patched.

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

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

W1572-06-01A 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. W1572-06-01

BORING 5

ELEV. (MSL.) **- -** DATE COMPLETED **06/02/2022**

EQUIPMENT **HOLLOW STEM AUGER** BY: **JC**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0					**AC: 3" BASE: 3"** **ARTIFICIAL FILL** Silty Sand, medium dense, slightly moist, dark grayish brown, fine- to medium-grained.			
2 4 6	B5@5'			SM	**ALLUVIUM** Silty Sand, medium dense, slightly moist, dark yellowish brown, fine- to medium-grained.	35	88.6	15.6
8 10	B5@10'			SC	Clayey Sand, dense, slightly moist, dark yellowish brown, fine- to medium-grained.	62	111.7	13.5
12 14 16 18	B5@15'			CL	Sandy Clay, stiff, slightly moist to moist, dark yellowish brown, fine- to medium-grained.	60	125.9	18.8
20 22 24 26	B5@20'			SC	Clayey Sand, dense, dry to slightly moist, mottled with dark yellowish brown and dark olive brown, fine- to medium-grained.	50 (6")	118.2	10.6
28				SP	Sand, poorly graded, dense, dry to slightly moist, light olive brown, fine- to medium-grained.			

W1572-06-01A BORING LOGS.GPJ

Figure A5,
Log of Boring 5, Page 1 of 2

SAMPLE SYMBOLS

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

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

GEOCON

BORING 5

ELEV. (MSL.) **- -** DATE COMPLETED **06/02/2022**

EQUIPMENT **HOLLOW STEM AUGER** BY: **JC**

DEPTH IN FEET	SAMPLE NO.		LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
30	B5@30'	■	░		SP		50 (6")	98.5	15.0

Total depth of boring: 30.5 feet
Fill to 1.5 feet.
No groundwater encountered.
Backfilled with soil cuttings and tamped.
AC patched.

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

Figure A5,
Log of Boring 5, Page 2 of 2

W1572-06-01A 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. W1572-06-01

BORING 6

ELEV. (MSL.) **- -** DATE COMPLETED **06/02/2022**

EQUIPMENT **HOLLOW STEM AUGER** BY: **JC**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0	BULK 0-5'				**AC: 4" BASE: 4"** **ARTIFICIAL FILL** Silty Sand, medium dense, slightly moist, dark yellowish brown, fine- to medium-grained.			
2					**ALLUVIUM** Silty Sand, medium dense, dry to slightly moist, brown, fine- to medium-grained.			
4								
6	B6@5' BULK 5-10'			SM		43	93.3	14.1
8								
10	B6@10'				- dense, mottled	50 (6")	104.9	5.0
12								
					Sandy Clay, stiff, dry to slightly moist, dark yellowish brown.			
14								
16	B6@15'			CL		50 (3.5")	121.9	7.5
18					Clayey Sand, dry, slightly moist, mottled with dark yellowish brown and dark olive brown, fine- to medium-grained.			
20	B6@20'					56	103.5	17.5
22				SC				
24								
	B6@25'					49	100.6	26.7

Total depth of boring: 25.5 feet
Fill to 1 foot.
No groundwater encountered.
Backfilled with soil cuttings and tamped.
AC patched.

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

W1572-06-01A BORING LOGS.GPJ

Figure A6,
Log of Boring 6, Page 1 of 2

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

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

GEOCON

BORING 6

ELEV. (MSL.) **- -** DATE COMPLETED **06/02/2022**

EQUIPMENT **HOLLOW STEM AUGER** BY: **JC**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
					auto-hammer. NOTE: The stratification lines presented herein represent the approximate boundary between earth types; the transitions may be gradual.			

Figure A6,
Log of Boring 6, Page 2 of 2

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

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

PROJECT NO. W1572-06-01

BORING 7

ELEV. (MSL.) **- -** DATE COMPLETED **06/02/2022**

EQUIPMENT **HAND AUGER** BY: **JC**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0					**ARTIFICIAL FILL** Silty Sand, medium dense, slightly moist to moist, dark grayish brown, fine- to medium-grained. - thin layer of base			
2				SM	**ALLUVIUM** Silty Sand, medium dense, moist, dark yellowish brown, fine- to medium-grained.			
4								

Total depth of boring: 5 feet
Fill to 1.5 feet.
No groundwater encountered.
Percolation testing performed.
Backfilled with soil cuttings and tamped.

NOTE: The stratification lines presented herein represent the approximate boundary between earth types; the transitions may be gradual.

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

W1572-06-01A 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. W1572-06-01

BORING 8

ELEV. (MSL.) - - DATE COMPLETED 09/23/2023

EQUIPMENT HOLLOW STEM AUGER BY: CT

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0					**ARTIFICIAL FILL** Silty Sand, dense, slightly moist, brown, fine-grained, trace rootlets.			
2					**ALLUVIUM** Silty Sand, very dense, dry to slightly moist, brown, fine-grained.			
4								
	B8@5'					50 (6")	112.5	6.5
6								
8				SM				
10	B8@10'				- loose	15	105.6	13.8
12								
	B8@12.5'				- increase in silt - medium dense, moist, reddish brown	23	113.5	16.3
14								
	B8@15' BULK 15-20'				- increase in moisture, manganese oxide specs	43	109.2	17.2
16								
					Clayey Sand, medium dense, moist, brown, fine- to medium-grained.			
18	B8@17.5'			SC		35	127.7	12.8
20	B8@20'				Sand with Silt, dense, moist, brown, fine- to medium-grained, trace clay.	76	131.0	10.2
22				SP-SM				
24					Silty Sand, dense, moist, reddish brown, fine- to medium-grained, oxidation staining.			
	B8@25'					70	123.0	16.1
26				SM				
28								
				SP	Sand, poorly graded, dense, slightly moist, light reddish brown, fine-grained.			

W1572-06-01A BORING LOGS.GPJ

Figure A8,
Log of Boring 8, Page 1 of 2

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

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

GEOCON

PROJECT NO. W1572-06-01

BORING 8

ELEV. (MSL.) **- -** DATE COMPLETED **09/23/2023**

EQUIPMENT **HOLLOW STEM AUGER** BY: **CT**

DEPTH IN FEET	SAMPLE NO.		LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
30	B8@30'	■	⁚⁚⁚		SP		75	105.2	6.1
						Total depth of boring: 30.5 feet Fill to 2 feet. No groundwater encountered. Backfilled with soil cuttings and tamped. Surface restored. *Penetration resistance for 140-pound hammer falling 30 inches by auto-hammer. NOTE: The stratification lines presented herein represent the approximate boundary between earth types; the transitions may be gradual.			

Figure A8,
Log of Boring 8, Page 2 of 2

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

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PROJECT NO. W1572-06-01

BORING 9

ELEV. (MSL.) - - DATE COMPLETED 09/23/2023

EQUIPMENT HOLLOW STEM AUGER BY: CT

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0					**AC: 3" BASE: 3"**			
					ARTIFICIAL FILL Silty Sand, medium dense, dry, brown, fine-grained.			
2					**ALLUVIUM** Silty Sand, dense, dry, brown, fine-grained.			
4								
	B9@5'				- very dense	50 (4")	114.1	6.5
6				SM				
8								
10	B9@10'					50 (5")	106.0	4.9
	BULK 10-15'							
12					Clayey Sand, very dense, slightly moist, reddish brown, fine- to medium-grained.			
	B9@12.5'					50 (6")	118.7	10.0
14				SC				
	B9@15'					50 (3")	108.2	9.9
16								
					Clay, hard, slightly moist, reddish brown, mottled, trace fine-grained sand.			
18	B9@17.5'					50 (6")	109.5	19.1
20	B9@20'			CL		50 (6")	114.6	10.3
22								
					- increase in sand			
24					Silty Sand, very dense, dry, reddish brown, oxidation staining, fine-grained.			
	B9@25'					50 (2")	102.5	7.3
26				SM				
28								
				SP	Sand, poorly graded, dense, dry, whitish gray, fine-grained.			

W1572-06-01A BORING LOGS.GPJ

Figure A9,
Log of Boring 9, Page 1 of 2

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

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

PROJECT NO. W1572-06-01

BORING 9

ELEV. (MSL.) **- -** DATE COMPLETED **09/23/2023**

EQUIPMENT **HOLLOW STEM AUGER** BY: **CT**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
30	B9@30' ■	⁚⁚⁚		SP		79	98.3	1.9

Total depth of boring: 30.5 feet
Fill to 1 feet.
No groundwater encountered.
Backfilled with soil cuttings and tamped.
AC patched.

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

Figure A9,
Log of Boring 9, Page 2 of 2

SAMPLE SYMBOLS

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

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

PROJECT NO. W1572-06-01

BORING 10

ELEV. (MSL.) - - DATE COMPLETED 09/23/2023

EQUIPMENT **HOLLOW STEM AUGER** BY: **CT**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0					**AC: 3" BASE: 5"** **ARTIFICIAL FILL** Silty Sand, dense, slightly moist, brown, fine-grained.			
2 4				SM	**ALLUVIUM** Silty Sand, dense, slightly moist, brown, fine-grained.			
6 8	B10@5' BULK 5-10'				- very dense	50 (4")	114.7	5.9
10	B10@10'			SC	Clayey Sand, dense, slightly moist, reddish brown, oxidation staining, manganese oxide specs, fine-grained.	57	114.9	16.4
12	B10@12.5				- not mottled, no oxidation staining	48	120.1	12.1
14 16	B10@15'				- fine- to medium-grained	46	130.1	10.8
18	B10@17.5			CL	Sandy Clay, hard, slightly moist, brown, fine- to medium-grained.	47	131.4	10.9
20	B10@20'			SC	Clayey Sand, medium dense, slightly moist, reddish brown, fine- to medium-grained.	49	121.8	15.0

Total depth of boring: 20.5 feet
Fill to 1.5 feet.
No groundwater encountered.
Backfilled with soil cuttings and tamped.
AC patched.

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

W1572-06-01A BORING LOGS.GPJ

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

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

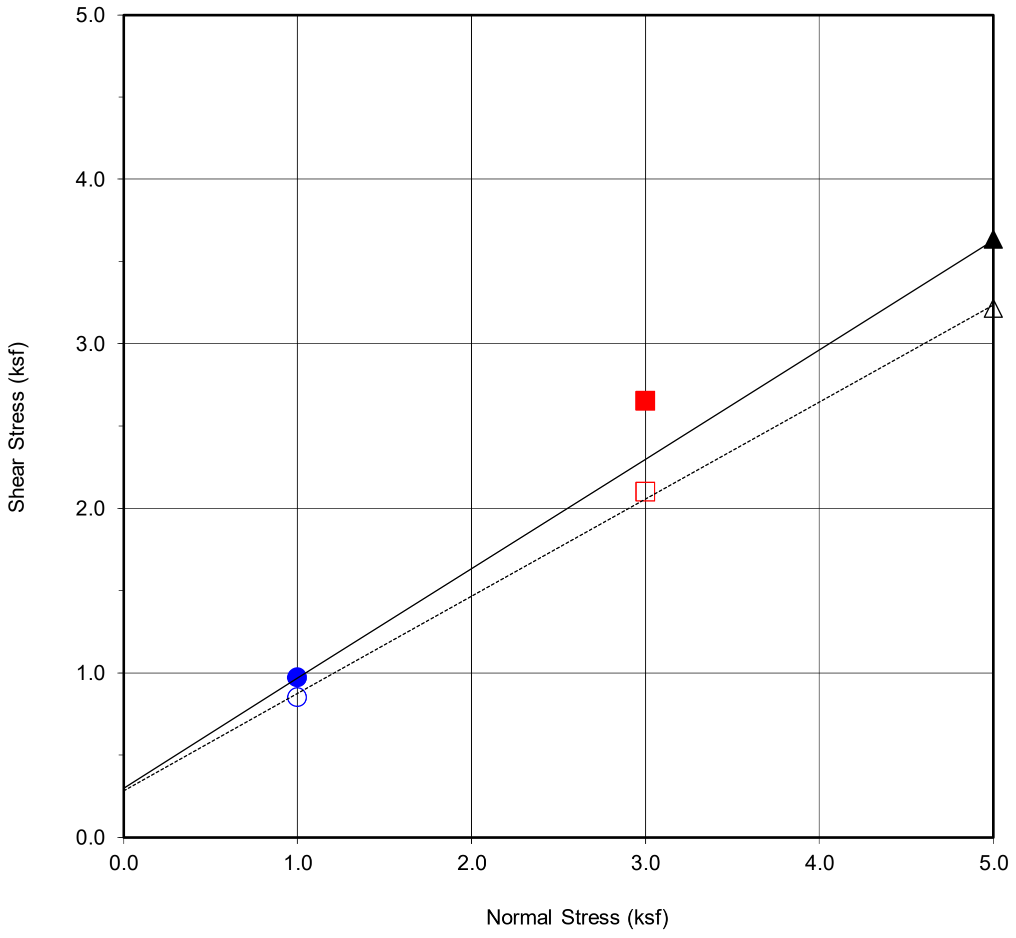
NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

APPENDIX B

APPENDIX B

LABORATORY TESTING

Laboratory tests were performed in accordance with generally accepted test methods of the International ASTM, or other suggested procedures. Selected samples were tested for direct shear strength, compaction, consolidation and expansion characteristics, corrosivity, in-place dry density and moisture content. The results of the laboratory tests are summarized in Figures B1 through B35. The in-place dry density and moisture content of the samples tested are presented on the boring logs, Appendix A.



Boring No.	**B-2**
Sample No.	**B2@2.5'**
Depth (ft)	**2.5'**
Sample Type:	Ring

Soil Identification:		
Silty Sand (SM)		
Strength Parameters		
	C (psf)	ϕ (°)
Peak	300	34
Ultimate	283	31

Normal Stress (kip/ft²)	1	3	5
Peak Shear Stress (kip/ft²)	● 0.97	■ 2.65	▲ 3.64
Shear Stress @ End of Test (ksf)	○ 0.85	□ 2.10	△ 3.22
Deformation Rate (in./min.)	0.01	0.01	0.01
Initial Sample Height (in.)	1.0	1.0	1.0
Ring Inside Diameter (in.)	2.375	2.375	2.375
Initial Moisture Content (%)	12.4	13.0	14.5
Initial Dry Density (pcf)	108.8	108.5	106.2
Initial Degree of Saturation (%)	61.1	63.6	66.7
Soil Height Before Shearing (in.)	1.2	1.2	1.2
Final Moisture Content (%)	15.8	15.4	16.1

GEOCON

DIRECT SHEAR TEST RESULTS

Consolidated Drained ASTM D-3080

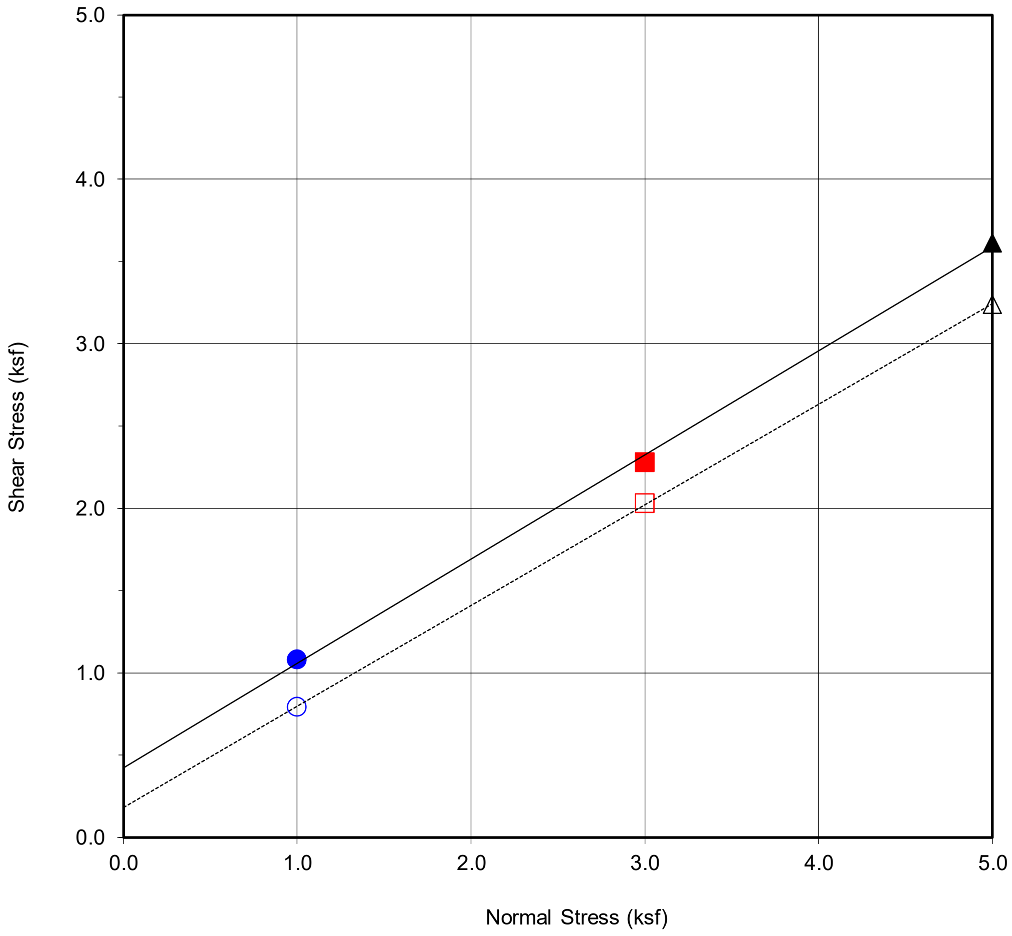
Checked by: PZ

Project No.: W1572-06-01A

2325 Crenshaw Boulevard
Torrance, California

Oct. 2023

Figure B1



Boring No.	**B-1**
Sample No.	**B1@5'**
Depth (ft)	**5'**
Sample Type:	Ring

Soil Identification:		
Silty Sand (SM)		
Strength Parameters		
	C (psf)	ϕ (°)
Peak	425	32
Ultimate	184	31

Normal Stress (kip/ft²)	1	3	5
Peak Shear Stress (kip/ft²)	● 1.08	■ 2.28	▲ 3.61
Shear Stress @ End of Test (ksf)	○ 0.79	□ 2.03	△ 3.24
Deformation Rate (in./min.)	0.01	0.01	0.01
Initial Sample Height (in.)	1.0	1.0	1.0
Ring Inside Diameter (in.)	2.375	2.375	2.375
Initial Moisture Content (%)	11.9	10.7	9.1
Initial Dry Density (pcf)	101.3	103.7	105.7
Initial Degree of Saturation (%)	48.5	46.1	41.4
Soil Height Before Shearing (in.)	1.2	1.2	1.2
Final Moisture Content (%)	19.1	18.9	18.0

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DIRECT SHEAR TEST RESULTS

Consolidated Drained ASTM D-3080

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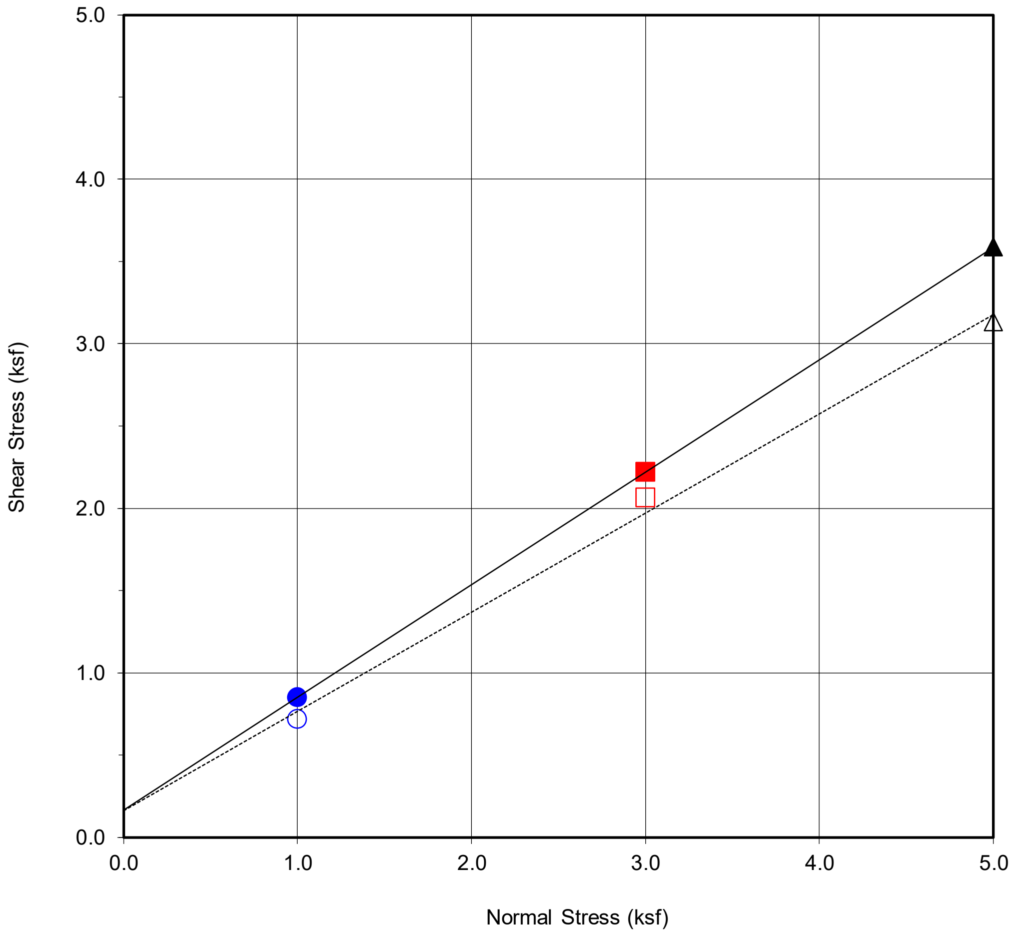
Figure B2



Boring No.	**B-3**
Sample No.	**B3@5'**
Depth (ft)	**5'**
Sample Type:	Ring

Soil Identification:

Silty Sand (SM)

Strength Parameters

	C (psf)	ϕ (°)
Peak	168	34
Ultimate	163	31

Normal Stress (kip/ft²)	1	3	5
Peak Shear Stress (kip/ft²)	● 0.85	■ 2.22	▲ 3.59
Shear Stress @ End of Test (ksf)	○ 0.72	□ 2.06	△ 3.13
Deformation Rate (in./min.)	0.01	0.01	0.01
Initial Sample Height (in.)	1.0	1.0	1.0
Ring Inside Diameter (in.)	2.375	2.375	2.375
Initial Moisture Content (%)	14.1	17.8	16.5
Initial Dry Density (pcf)	106.2	101.9	104.7
Initial Degree of Saturation (%)	64.8	73.4	73.1
Soil Height Before Shearing (in.)	1.2	1.2	1.2
Final Moisture Content (%)	17.5	17.7	16.8

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DIRECT SHEAR TEST RESULTS

Consolidated Drained ASTM D-3080

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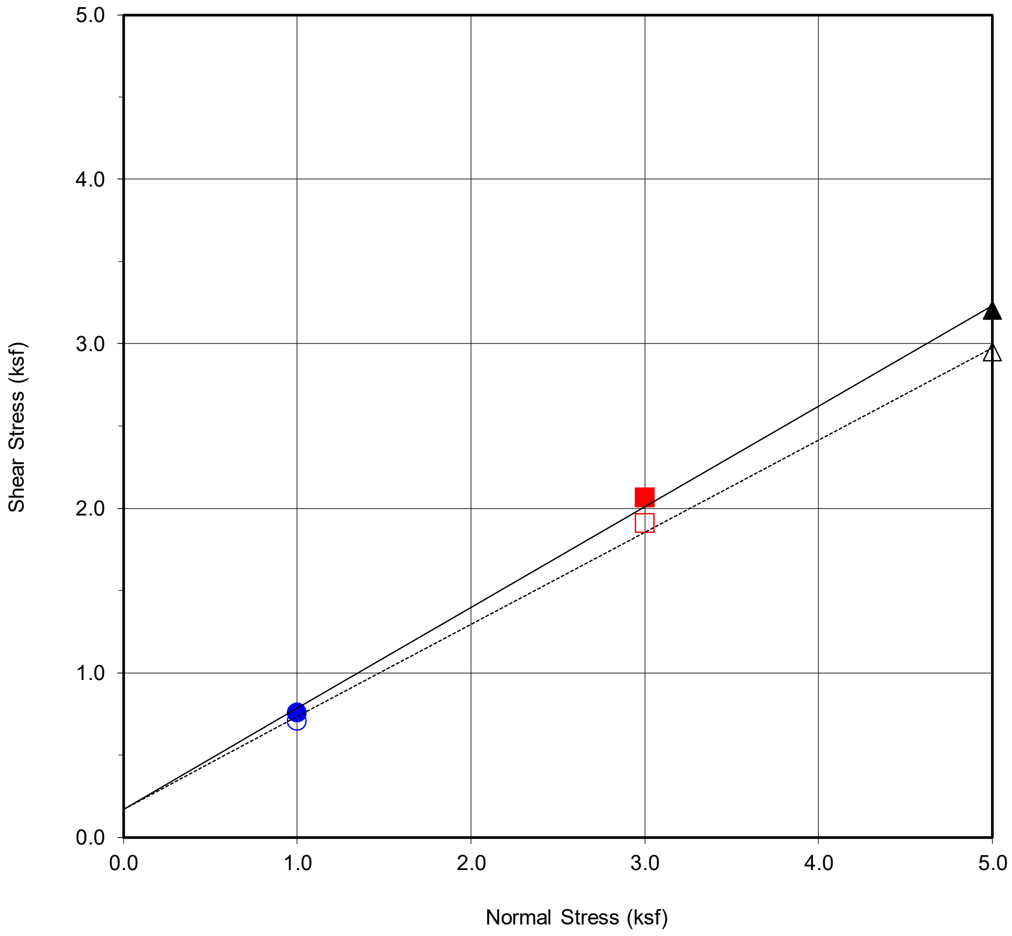
Figure B3



Boring No.	**B-5**
Sample No.	**B5@5'**
Depth (ft)	**5'**
Sample Type:	Ring

Soil Identification:	
Silty Sand (SM)	

Strength Parameters

	C (psf)	ϕ (°)
Peak	172	31
Ultimate	173	29

Normal Stress (kip/ft²)	1	3	5
Peak Shear Stress (kip/ft²)	● 0.76	■ 2.06	▲ 3.20
Shear Stress @ End of Test (ksf)	○ 0.71	□ 1.91	△ 2.95
Deformation Rate (in./min.)	0.01	0.01	0.01
Initial Sample Height (in.)	1.0	1.0	1.0
Ring Inside Diameter (in.)	2.375	2.375	2.375
Initial Moisture Content (%)	14.3	15.4	13.5
Initial Dry Density (pcf)	88.9	84.7	87.5
Initial Degree of Saturation (%)	43.0	42.0	39.5
Soil Height Before Shearing (in.)	1.2	1.2	1.2
Final Moisture Content (%)	23.7	23.4	23.0

GEOCON

DIRECT SHEAR TEST RESULTS

Consolidated Drained ASTM D-3080

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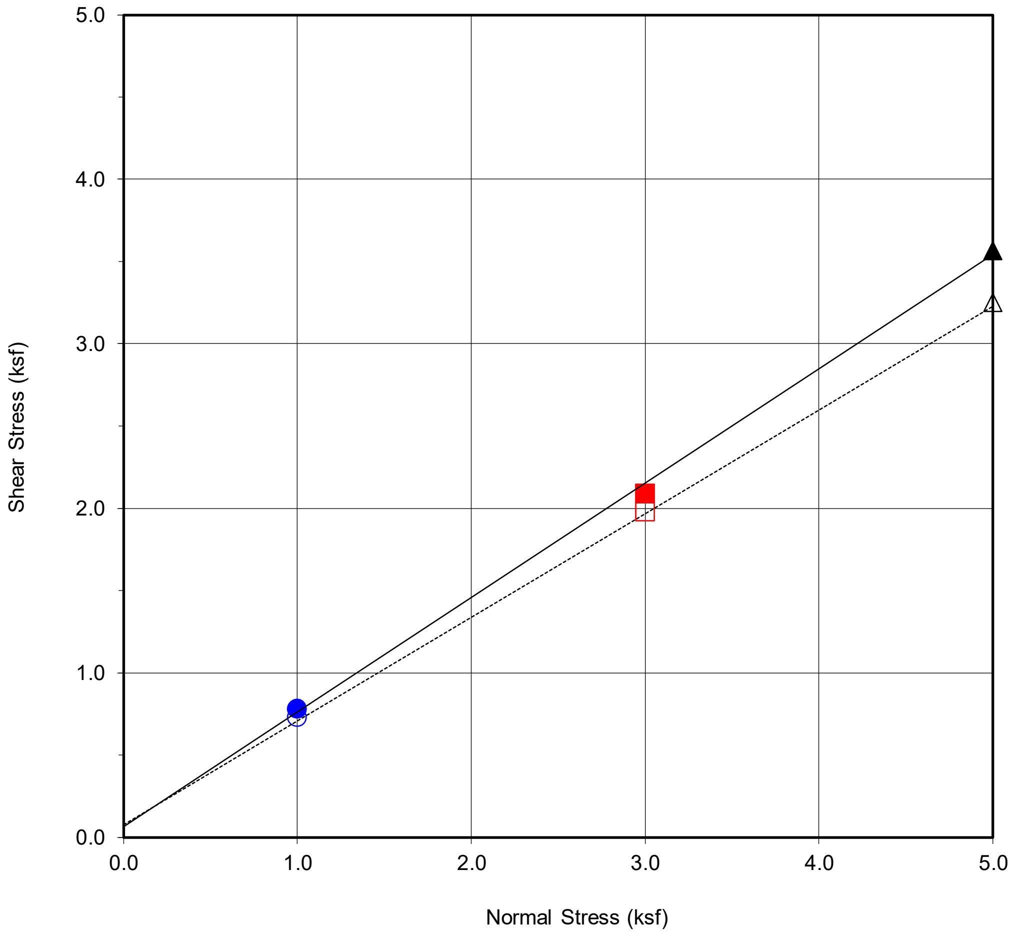
Figure B4



Boring No.	**B-6**
Sample No.	**B6@5'**
Depth (ft)	**5'**
Sample Type:	Ring

Soil Identification:		
Silty Sand (SM)		
Strength Parameters		
	C (psf)	ϕ (°)
Peak	66	35
Ultimate	78	32

Normal Stress (kip/ft²)	1	3	5
Peak Shear Stress (kip/ft²)	● 0.78	■ 2.09	▲ 3.56
Shear Stress @ End of Test (ksf)	○ 0.73	□ 1.98	△ 3.25
Deformation Rate (in./min.)	0.01	0.01	0.01
Initial Sample Height (in.)	1.0	1.0	1.0
Ring Inside Diameter (in.)	2.375	2.375	2.375
Initial Moisture Content (%)	11.0	15.5	14.1
Initial Dry Density (pcf)	95.7	91.4	93.2
Initial Degree of Saturation (%)	39.1	49.7	47.1
Soil Height Before Shearing (in.)	1.2	1.2	1.2
Final Moisture Content (%)	20.8	21.2	21.2

GEOCON

DIRECT SHEAR TEST RESULTS

Consolidated Drained ASTM D-3080

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Project No.: W1572-06-01A

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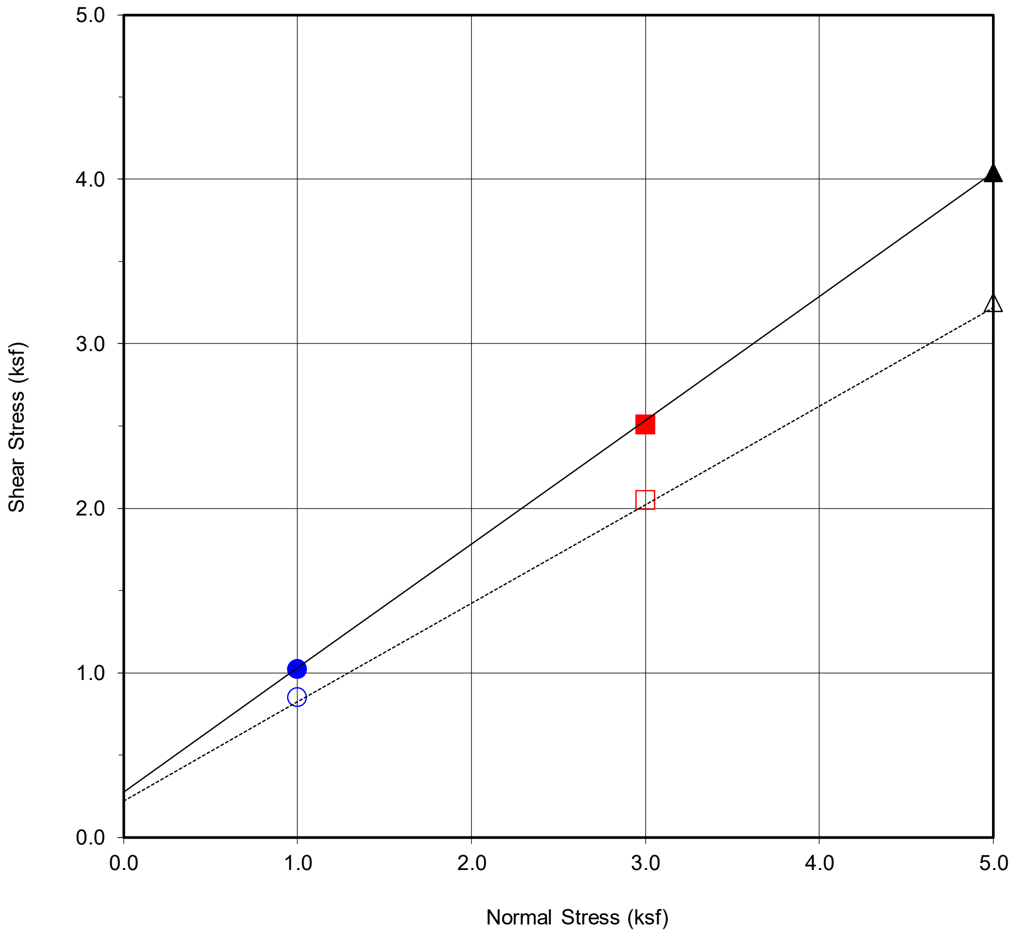
Oct. 2023

Figure B5



Boring No.	**B10**
Sample No.	**B10@5'**
Depth (ft)	**5'**
Sample Type:	RING

Soil Identification:

Silty Sand (SM)

Strength Parameters		
	C (psf)	ϕ (°)
Peak	276	37
Ultimate	222	31

Normal Stress (kip/ft²)	1	3	5
Peak Shear Stress (kip/ft²)	● 1.02	■ 2.51	▲ 4.04
Shear Stress @ End of Test (ksf)	○ 0.85	□ 2.05	△ 3.25
Deformation Rate (in./min.)	0.05	0.05	0.05
Initial Sample Height (in.)	1.0	1.0	1.0
Ring Inside Diameter (in.)	2.375	2.375	2.375
Initial Moisture Content (%)	8.4	9.0	8.5
Initial Dry Density (pcf)	112.8	110.9	113.7
Initial Degree of Saturation (%)	45.7	47.0	47.5
Soil Height Before Shearing (in.)	1.2	1.2	1.2
Final Moisture Content (%)	13.7	13.4	12.9

GEOCON

DIRECT SHEAR TEST RESULTS

Consolidated Drained ASTM D-3080

Checked by: PZ

Project No.: W1572-06-01A

2325 Crenshaw Boulevard
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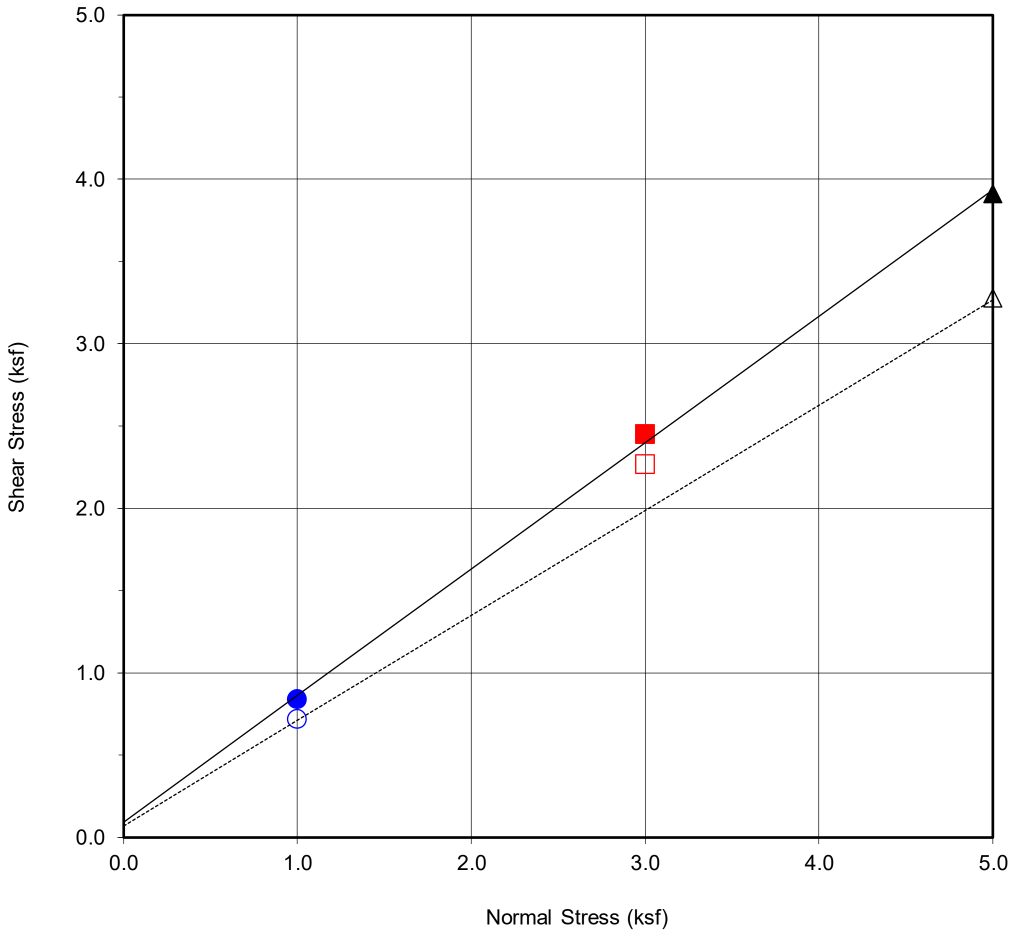
Figure B6



Boring No.	**B-2**
Sample No.	**B2@7.5'**
Depth (ft)	**7.5'**
Sample Type:	Ring

Soil Identification:
Silty Sand (SM)

Strength Parameters

	C (psf)	ϕ (°)
Peak	96	38
Ultimate	71	33

Normal Stress (kip/ft^2)	1	3	5
Peak Shear Stress (kip/ft^2)	● 0.84	■ 2.45	▲ 3.91
Shear Stress @ End of Test (ksf)	○ 0.72	□ 2.27	△ 3.28
Deformation Rate (in./min.)	0.01	0.01	0.01
Initial Sample Height (in.)	1.0	1.0	1.0
Ring Inside Diameter (in.)	2.375	2.375	2.375
Initial Moisture Content (%)	10.8	11.1	10.6
Initial Dry Density (pcf)	112.0	118.1	118.9
Initial Degree of Saturation (%)	57.7	69.9	68.3
Soil Height Before Shearing (in.)	1.2	1.2	1.2
Final Moisture Content (%)	15.4	14.9	14.4

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DIRECT SHEAR TEST RESULTS

Consolidated Drained ASTM D-3080

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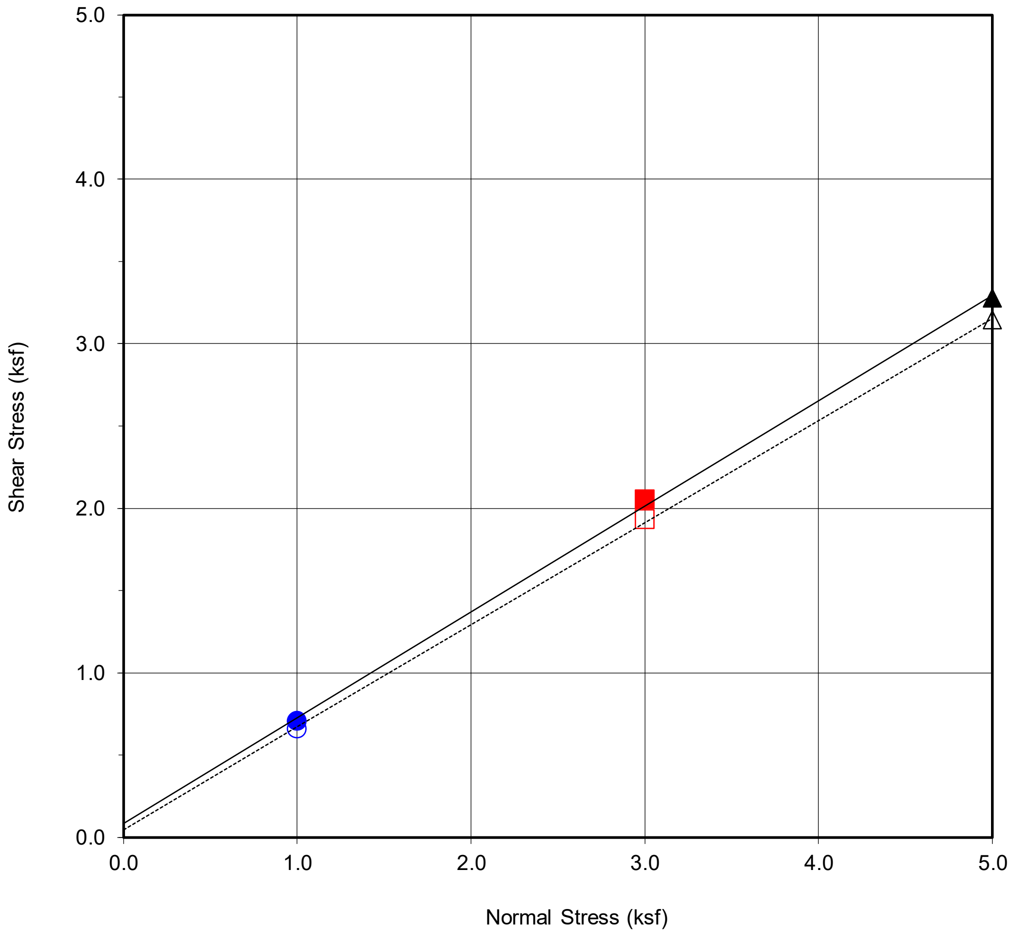
Figure B7



Boring No.	**B-1**
Sample No.	**B1@10'**
Depth (ft)	**10'**
Sample Type:	Ring

Soil Identification:		
Silty Sand (SM)		
Strength Parameters		
	C (psf)	ϕ (°)
Peak	86	33
Ultimate	49	32

Normal Stress (kip/ft²)	1	3	5
Peak Shear Stress (kip/ft²)	● 0.71	■ 2.05	▲ 3.28
Shear Stress @ End of Test (ksf)	○ 0.66	□ 1.93	△ 3.14
Deformation Rate (in./min.)	0.01	0.01	0.01
Initial Sample Height (in.)	1.0	1.0	1.0
Ring Inside Diameter (in.)	2.375	2.375	2.375
Initial Moisture Content (%)	12.5	14.4	13.0
Initial Dry Density (pcf)	94.7	98.1	102.2
Initial Degree of Saturation (%)	43.4	54.3	54.1
Soil Height Before Shearing (in.)	1.2	1.2	1.2
Final Moisture Content (%)	21.9	20.0	18.8

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DIRECT SHEAR TEST RESULTS

Consolidated Drained ASTM D-3080

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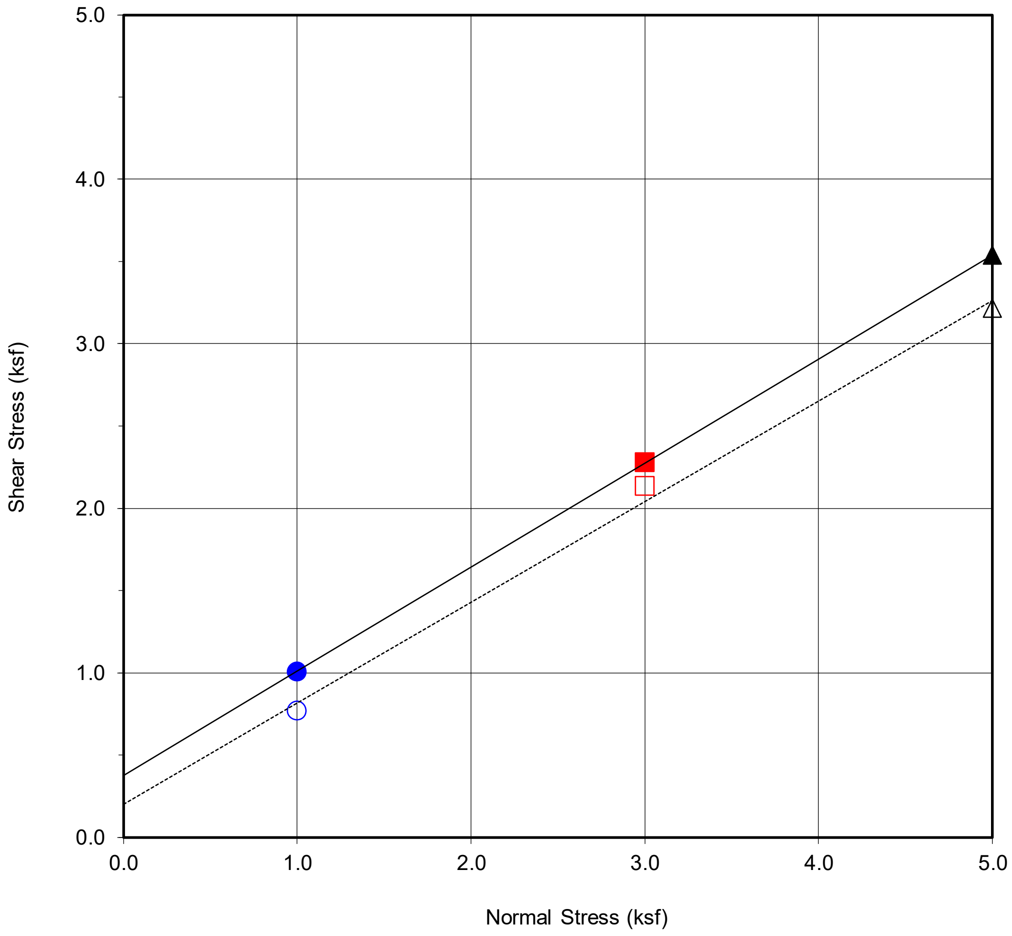
Figure B8



Boring No.	**B-4**
Sample No.	**B4@10'**
Depth (ft)	**10'**
Sample Type:	Ring

Soil Identification:
Silty Sand (SM)

Strength Parameters

	C (psf)	ϕ (°)
Peak	377	32
Ultimate	204	31

Normal Stress (kip/ft²)	1	3	5
Peak Shear Stress (kip/ft²)	● 1.01	■ 2.28	▲ 3.54
Shear Stress @ End of Test (ksf)	○ 0.77	□ 2.14	△ 3.22
Deformation Rate (in./min.)	0.01	0.01	0.01
Initial Sample Height (in.)	1.0	1.0	1.0
Ring Inside Diameter (in.)	2.375	2.375	2.375
Initial Moisture Content (%)	10.8	14.5	12.5
Initial Dry Density (pcf)	108.0	104.2	105.5
Initial Degree of Saturation (%)	51.8	63.6	56.5
Soil Height Before Shearing (in.)	1.2	1.2	1.2
Final Moisture Content (%)	18.1	18.2	18.0

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DIRECT SHEAR TEST RESULTS

Consolidated Drained ASTM D-3080

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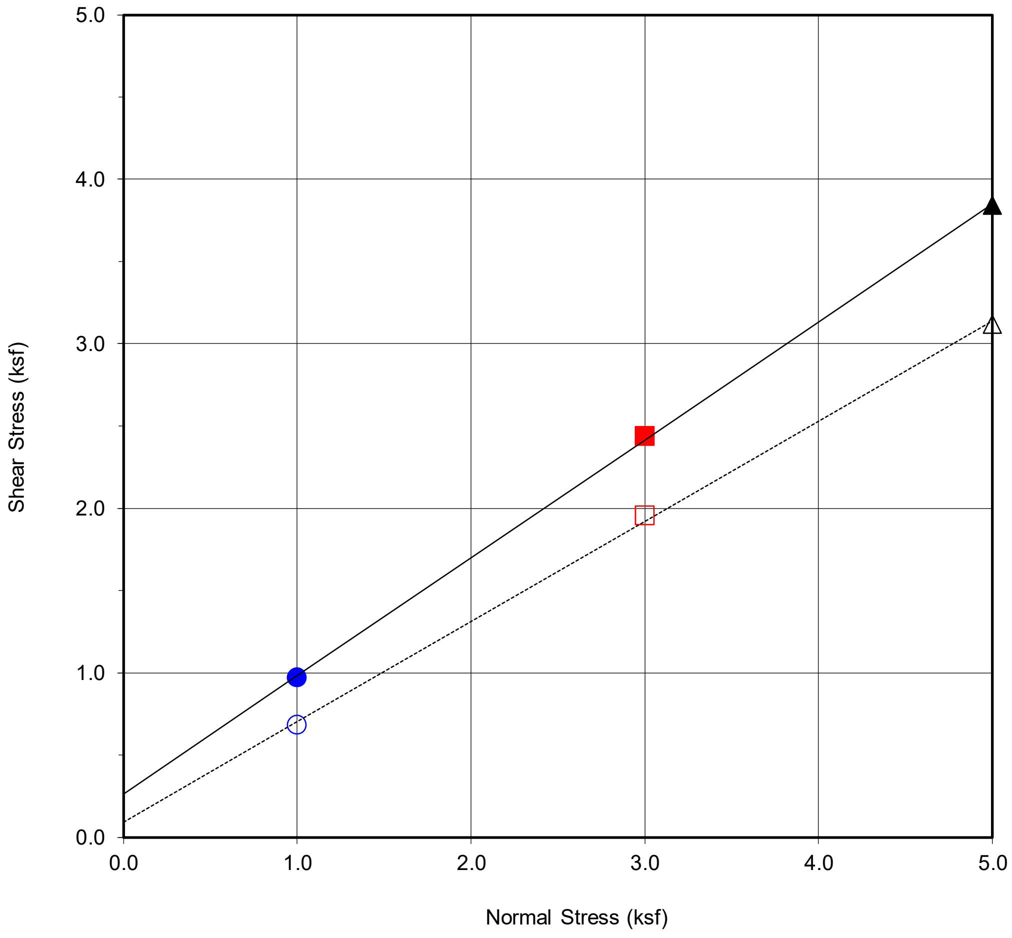
Figure B9



Boring No.	**B-5**
Sample No.	**B5@10'**
Depth (ft)	**10'**
Sample Type:	Ring

Soil Identification:		
Clayey Sand (SC)		
Strength Parameters		
	C (psf)	ϕ (°)
Peak	265	36
Ultimate	93	31

Normal Stress (kip/ft^2)	1	3	5
Peak Shear Stress (kip/ft^2)	● 0.97	■ 2.44	▲ 3.84
Shear Stress @ End of Test (ksf)	○ 0.68	□ 1.96	△ 3.12
Deformation Rate (in./min.)	0.01	0.01	0.01
Initial Sample Height (in.)	1.0	1.0	1.0
Ring Inside Diameter (in.)	2.375	2.375	2.375
Initial Moisture Content (%)	14.5	15.1	18.8
Initial Dry Density (pcf)	112.2	111.8	114.0
Initial Degree of Saturation (%)	77.7	80.5	105.9
Soil Height Before Shearing (in.)	1.2	1.2	1.2
Final Moisture Content (%)	19.2	18.5	18.1

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DIRECT SHEAR TEST RESULTS

Consolidated Drained ASTM D-3080

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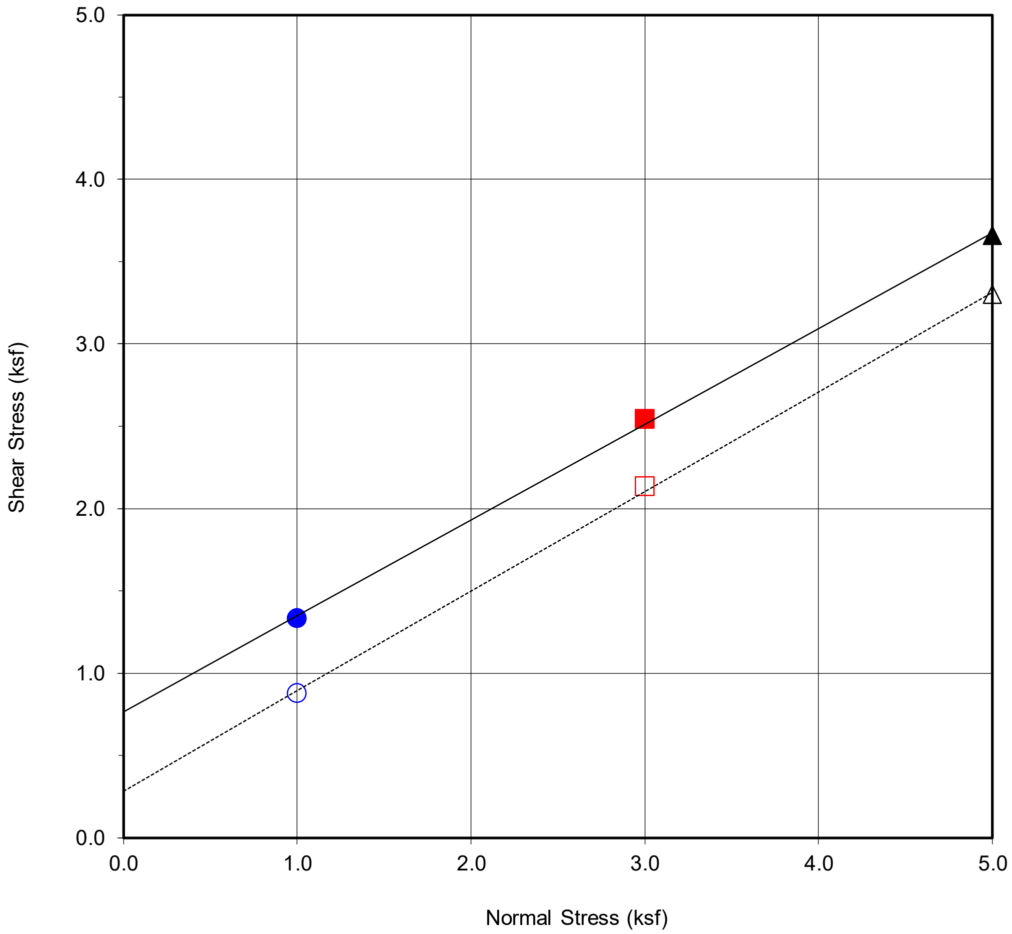
Figure B10



Boring No.	**B10**
Sample No.	**B10@15'**
Depth (ft)	**15'**
Sample Type:	RING

Soil Identification:	
Clayey Sand (SC)	

Strength Parameters		
	C (psf)	ϕ (°)
Peak	766	30
Ultimate	286	31

Normal Stress (kip/ft²)	1	3	5
Peak Shear Stress (kip/ft²)	● 1.33	■ 2.54	▲ 3.66
Shear Stress @ End of Test (ksf)	○ 0.88	□ 2.14	△ 3.30
Deformation Rate (in./min.)	0.05	0.05	0.05
Initial Sample Height (in.)	1.0	1.0	1.0
Ring Inside Diameter (in.)	2.375	2.375	2.375
Initial Moisture Content (%)	11.4	10.8	11.3
Initial Dry Density (pcf)	127.3	127.1	129.7
Initial Degree of Saturation (%)	95.0	89.2	101.6
Soil Height Before Shearing (in.)	1.2	1.2	1.2
Final Moisture Content (%)	11.9	11.5	11.3

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DIRECT SHEAR TEST RESULTS

Consolidated Drained ASTM D-3080

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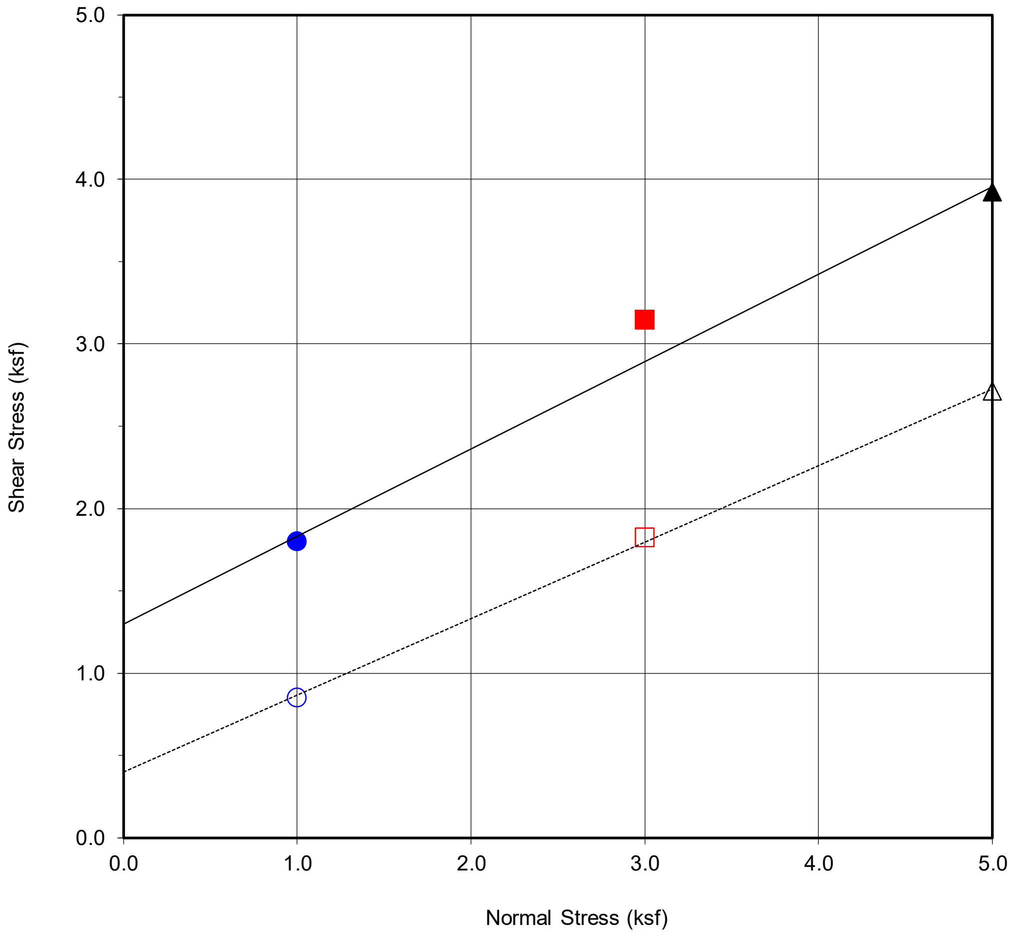
Figure B11



Boring No.	**B-2**
Sample No.	**B2@20'**
Depth (ft)	**20'**
Sample Type:	Ring

Soil Identification:	
Clay (CL)	

Strength Parameters		
	C (psf)	ϕ (°)
Peak	1300	28
Ultimate	401	25

Normal Stress (kip/ft²)	1	3	5
Peak Shear Stress (kip/ft²)	● 1.80	■ 3.14	▲ 3.92
Shear Stress @ End of Test (ksf)	○ 0.85	□ 1.82	△ 2.71
Deformation Rate (in./min.)	0.01	0.01	0.01
Initial Sample Height (in.)	1.0	1.0	1.0
Ring Inside Diameter (in.)	2.375	2.375	2.375
Initial Moisture Content (%)	22.9	21.8	23.8
Initial Dry Density (pcf)	105.3	106.2	106.2
Initial Degree of Saturation (%)	102.9	100.5	109.6
Soil Height Before Shearing (in.)	1.2	1.2	1.2
Final Moisture Content (%)	26.5	25.3	27.0

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DIRECT SHEAR TEST RESULTS

Consolidated Drained ASTM D-3080

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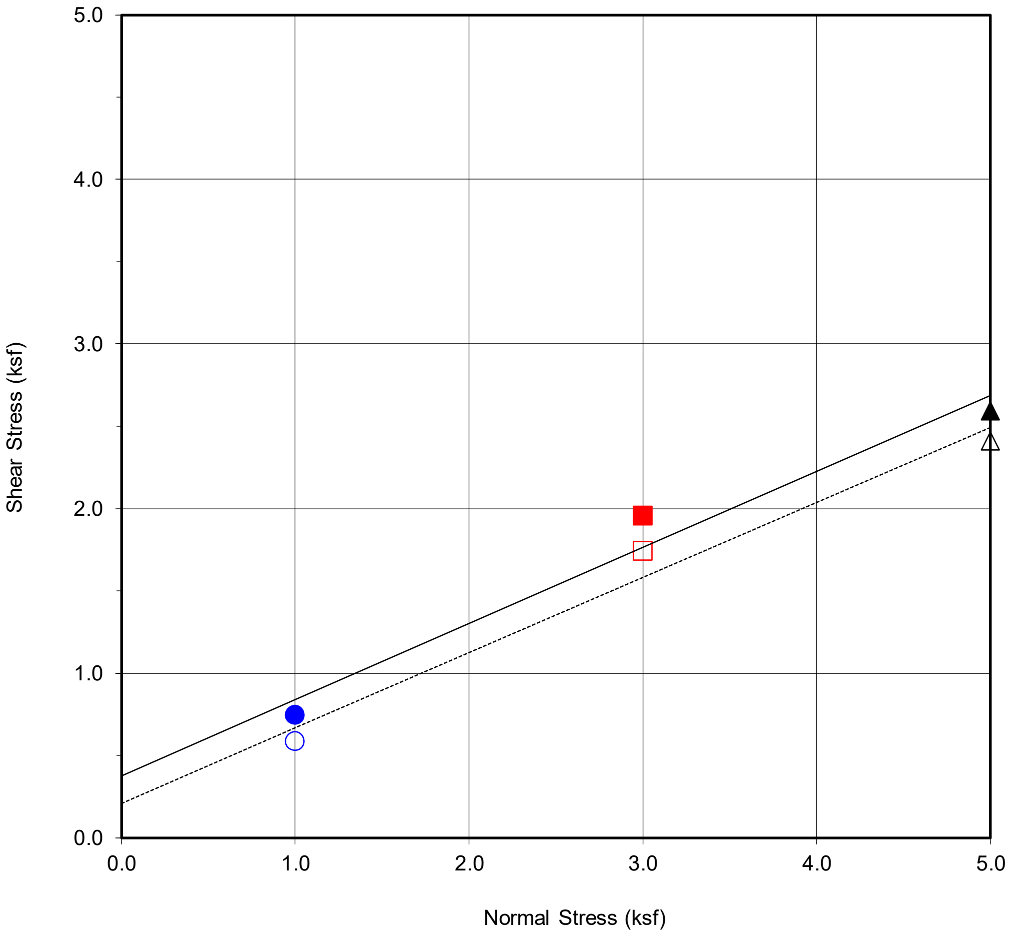
Figure B12



Boring No.	**B-5**
Sample No.	**B5@20'**
Depth (ft)	**20'**
Sample Type:	Ring

Normal Stress (kip/ft²)	1	3	5
Peak Shear Stress (kip/ft²)	● 0.74	■ 1.96	▲ 2.59
Shear Stress @ End of Test (ksf)	○ 0.59	□ 1.74	△ 2.41
Deformation Rate (in./min.)	0.01	0.01	0.01
Initial Sample Height (in.)	1.0	1.0	1.0
Ring Inside Diameter (in.)	2.375	2.375	2.375
Initial Moisture Content (%)	23.8	23.4	28.1
Initial Dry Density (pcf)	103.6	105.6	102.2
Initial Degree of Saturation (%)	102.8	105.8	116.8
Soil Height Before Shearing (in.)	1.2	1.2	1.2
Final Moisture Content (%)	28.8	25.7	27.1

Soil Identification:

Clayey Sand (SC)

Strength Parameters

	C (psf)	ϕ (°)
Peak	378	25
Ultimate	212	25

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DIRECT SHEAR TEST RESULTS

Consolidated Drained ASTM D-3080

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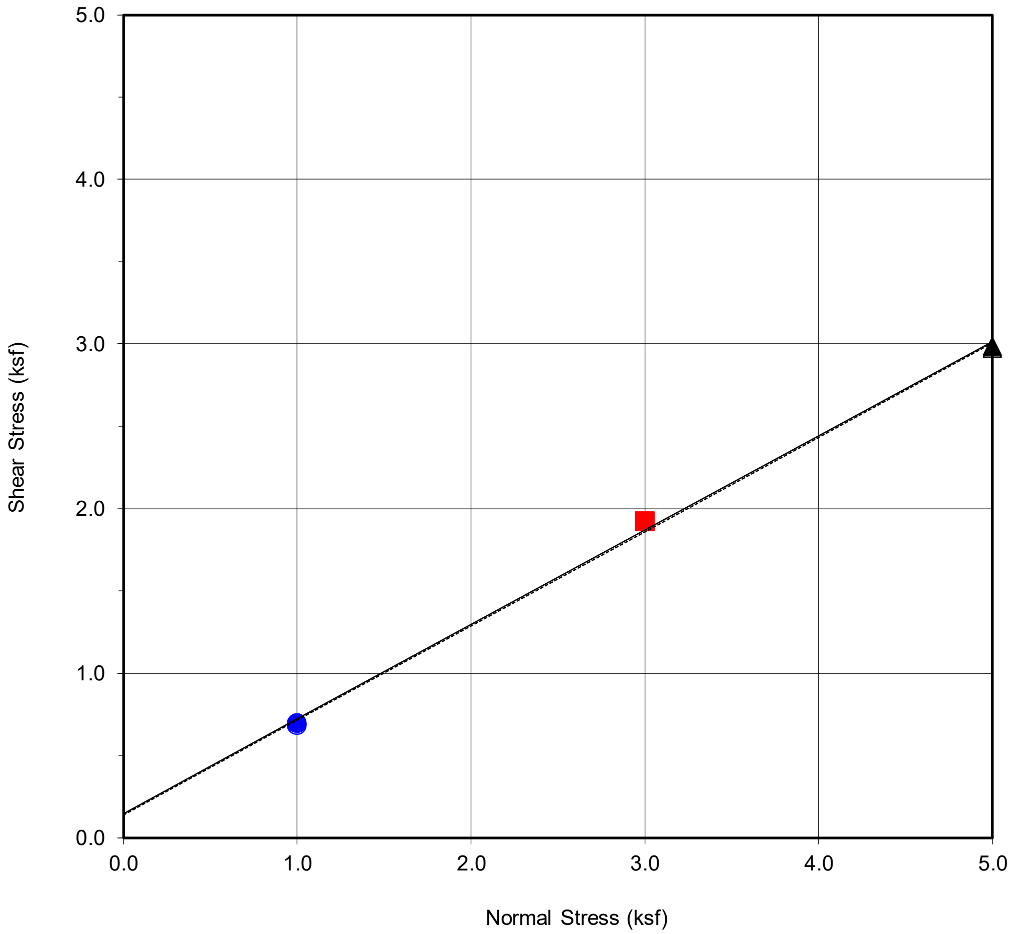
Figure B13



Boring No.	**B10**
Sample No.	**B10@5-10'**
Depth (ft)	**5-10'**
Sample Type:	REMOLD

Normal Stress (kip/ft²)	1	3	5
Peak Shear Stress (kip/ft²)	● 0.70	🟥 1.92	▲ 2.99
Shear Stress @ End of Test (ksf)	○ 0.68	□ 1.92	△ 2.98
Deformation Rate (in./min.)	0.05	0.05	0.05
Initial Sample Height (in.)	1.0	1.0	1.0
Ring Inside Diameter (in.)	2.375	2.375	2.375
Initial Moisture Content (%)	12.7	13.1	14.0
Initial Dry Density (pcf)	100.5	100.2	99.3
Initial Degree of Saturation (%)	50.7	51.7	54.1
Soil Height Before Shearing (in.)	1.2	1.2	1.2
Final Moisture Content (%)	19.6	19.2	19.4

Soil Identification:

Silty Sand (SM)

Strength Parameters

	C (psf)	ϕ (^{o})
Peak	149	30
Ultimate	141	30

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DIRECT SHEAR TEST RESULTS

Consolidated Drained ASTM D-3080

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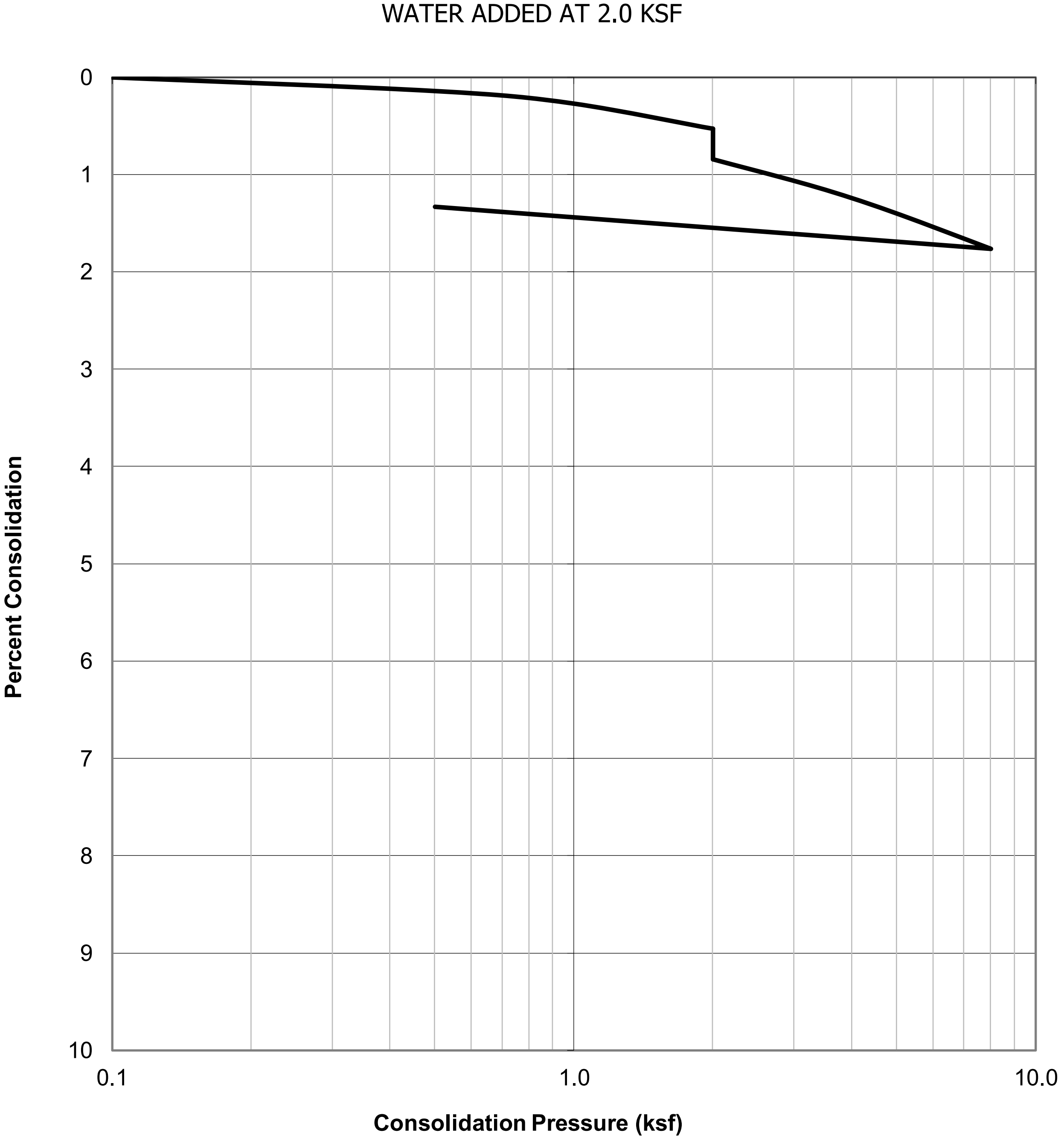
Figure B14



SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B-2@5'	Silty Sand (SM)	105.0	6.7	17.7

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CONSOLIDATION TEST RESULTS

ASTM D-2435

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Project No.: W1572-06-01A

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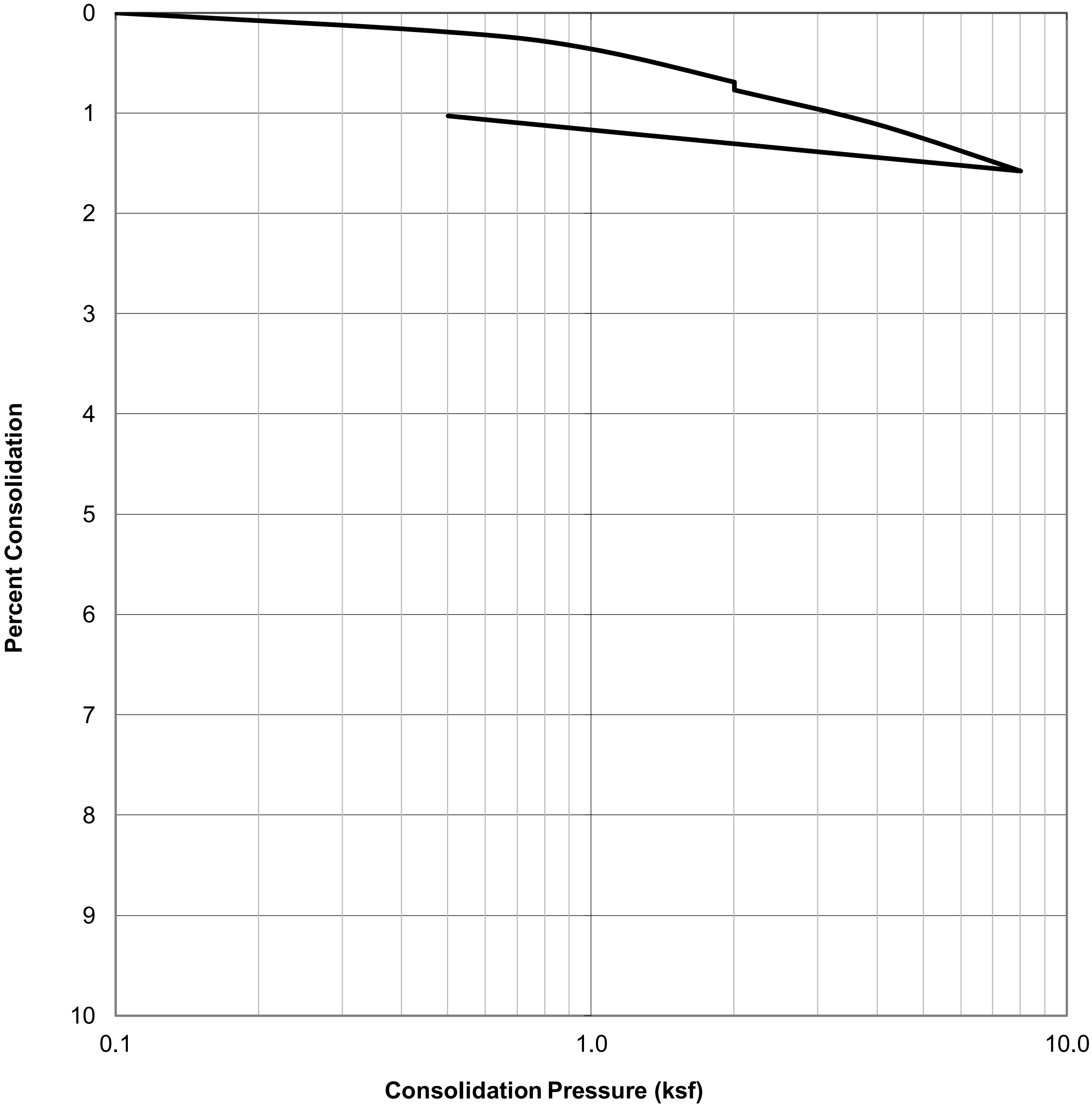
Figure B15

WATER ADDED AT 2.0 KSF



SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B-4@5'	Silty Sand (SM)	109.4	10.4	14.9

GEOCON

CONSOLIDATION TEST RESULTS

ASTM D-2435

Checked by: PZ

Project No.: W1572-06-01A

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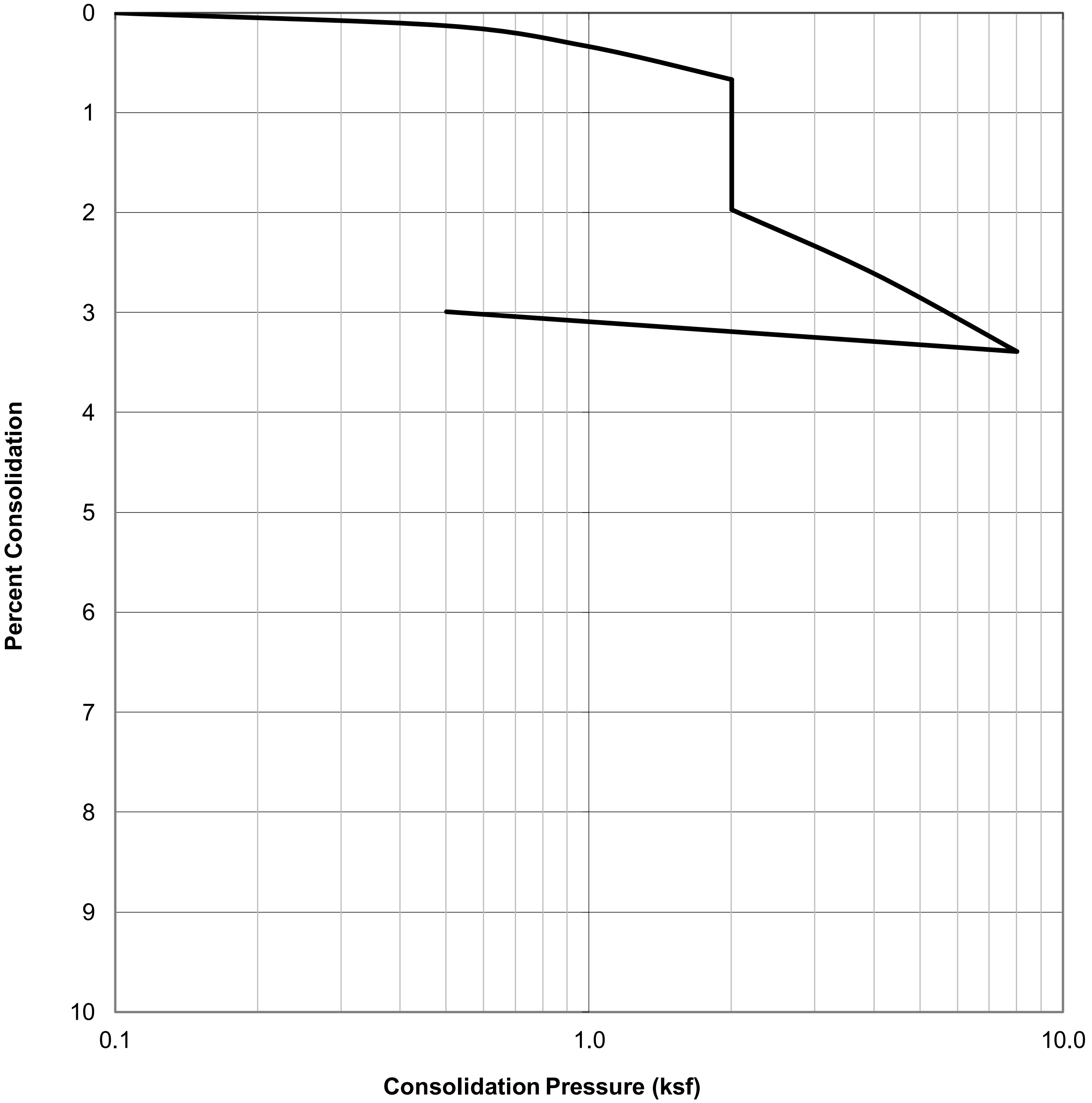
Figure B16

WATER ADDED AT 2.0 KSF



SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B-6@5'	Silty Sand (SM)	100.7	5.8	19.8

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CONSOLIDATION TEST RESULTS

ASTM D-2435

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Project No.: W1572-06-01A

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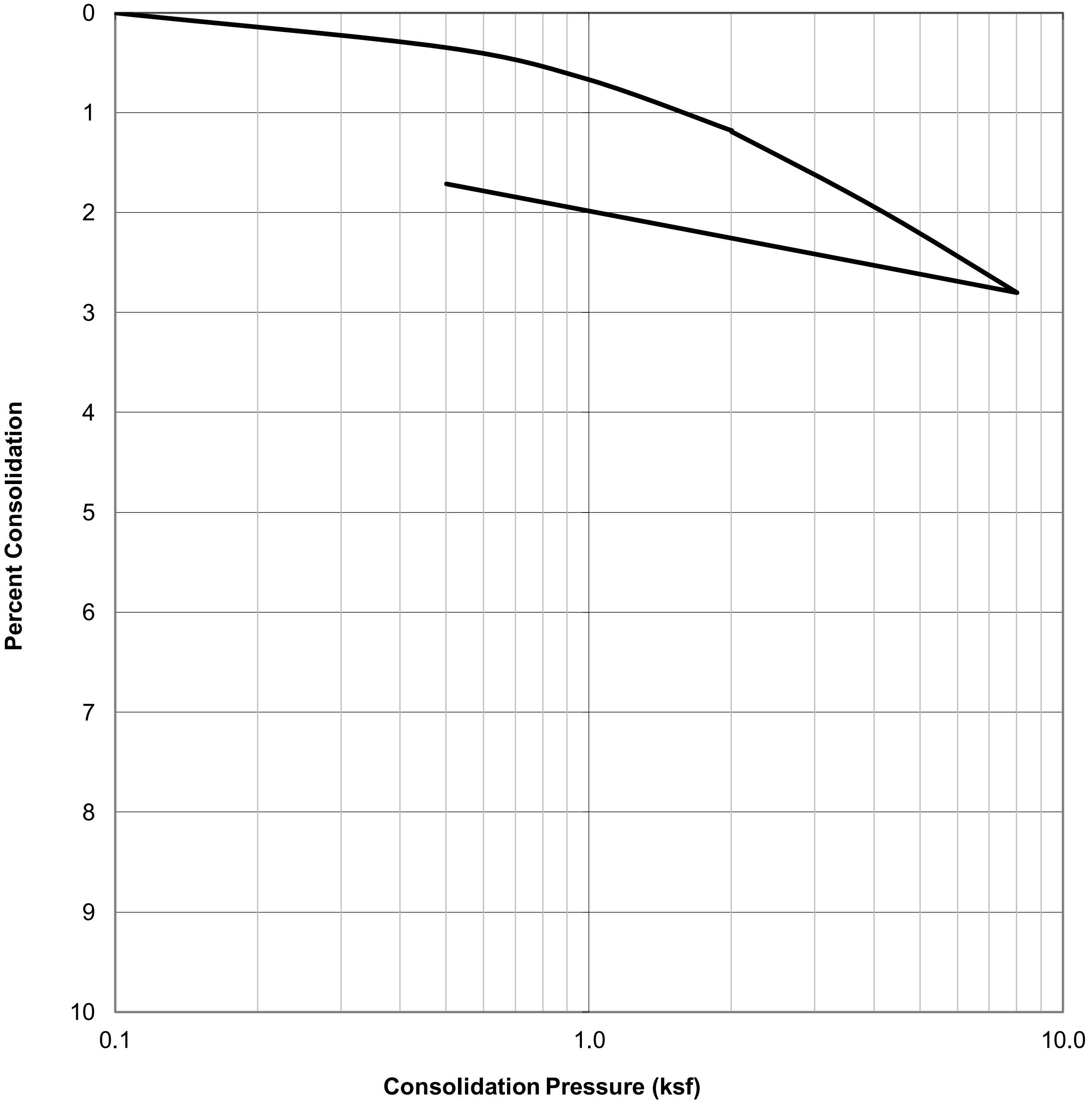
Figure B17

WATER ADDED AT 2.0 KSF



SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B-10@5'	Silty Sand (SM)	113.8	12.1	20.1

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CONSOLIDATION TEST RESULTS

ASTM D-2435

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Project No.: W1572-06-01A

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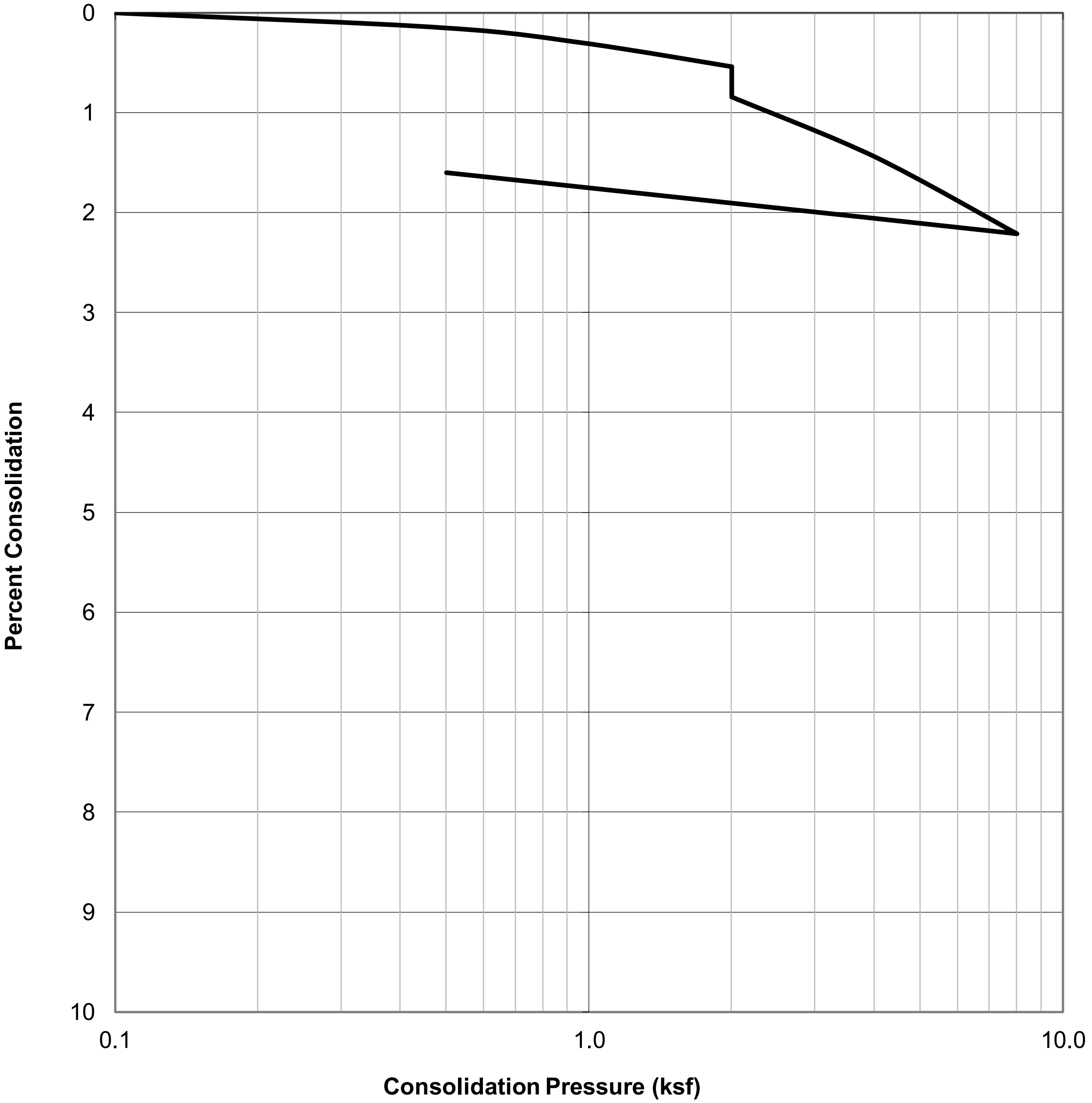
Figure B18

WATER ADDED AT 2.0 KSF



SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B-2@7.5'	Silty Sand (SM)	121.7	9.2	13.5

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CONSOLIDATION TEST RESULTS

ASTM D-2435

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Project No.: W1572-06-01A

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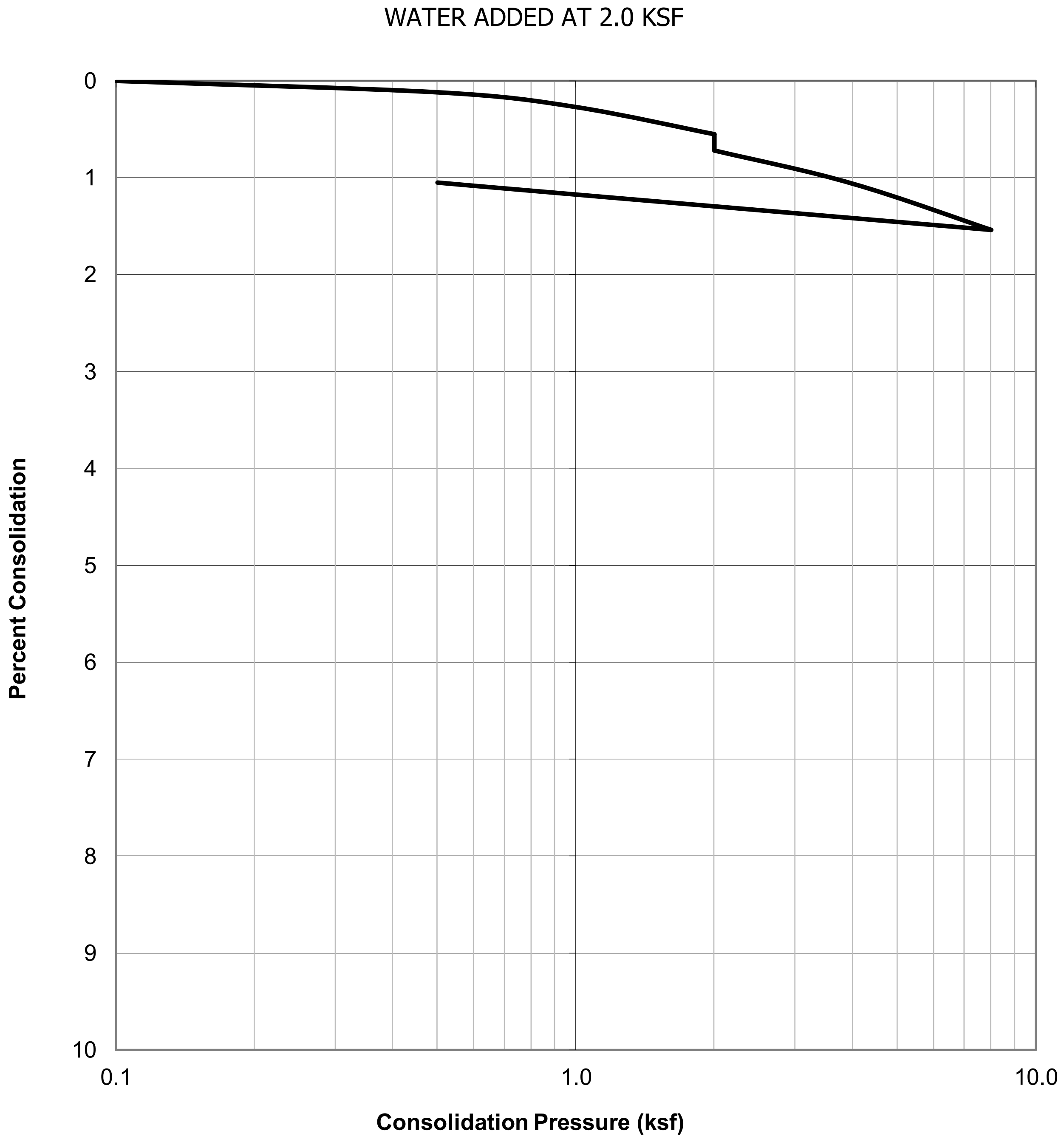
Figure B19



SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B-4@10'	Silty Sand (SM)	113.4	8.4	16.0

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CONSOLIDATION TEST RESULTS

ASTM D-2435

Checked by: PZ

Project No.: W1572-06-01A

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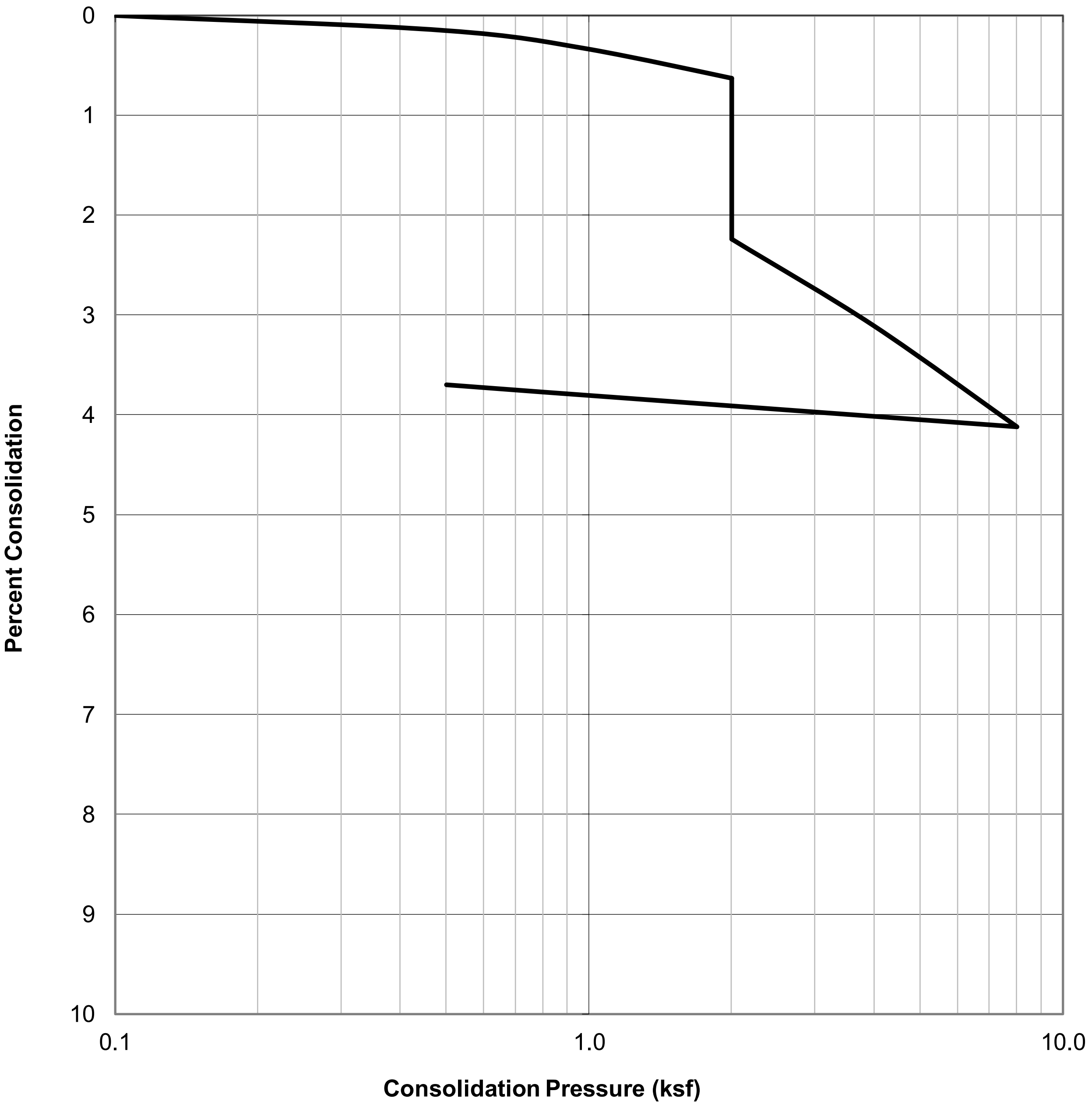
WATER ADDED AT 2.0 KSF



SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B-6@10'	Silty Sand (SM)	109.6	4.8	15.7

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CONSOLIDATION TEST RESULTS

ASTM D-2435

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Project No.: W1572-06-01A

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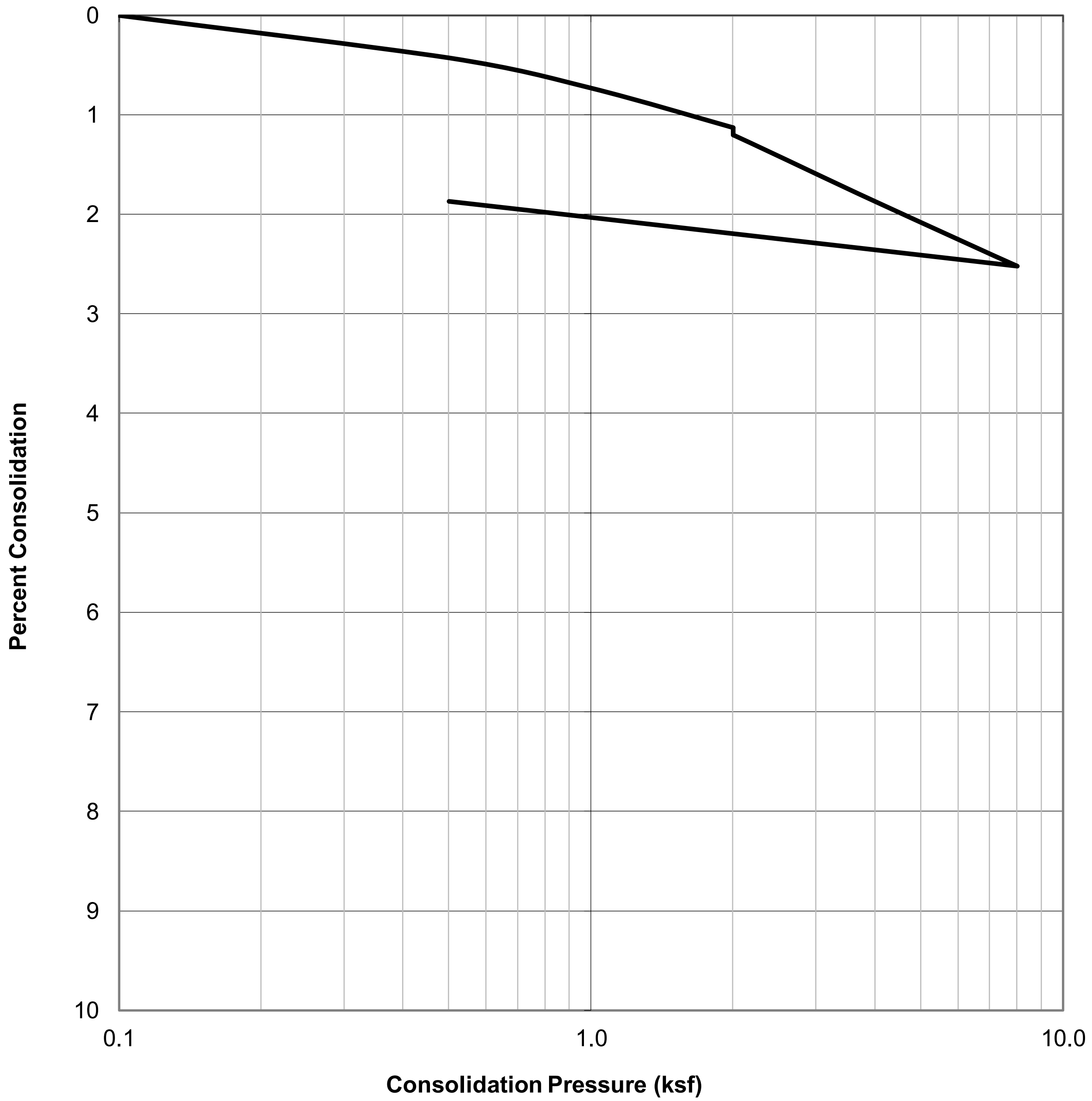
Figure B21

WATER ADDED AT 2.0 KSF



SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B-10@10'	Clayey Sand (SC)	105.0	30.8	32.5

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CONSOLIDATION TEST RESULTS

ASTM D-2435

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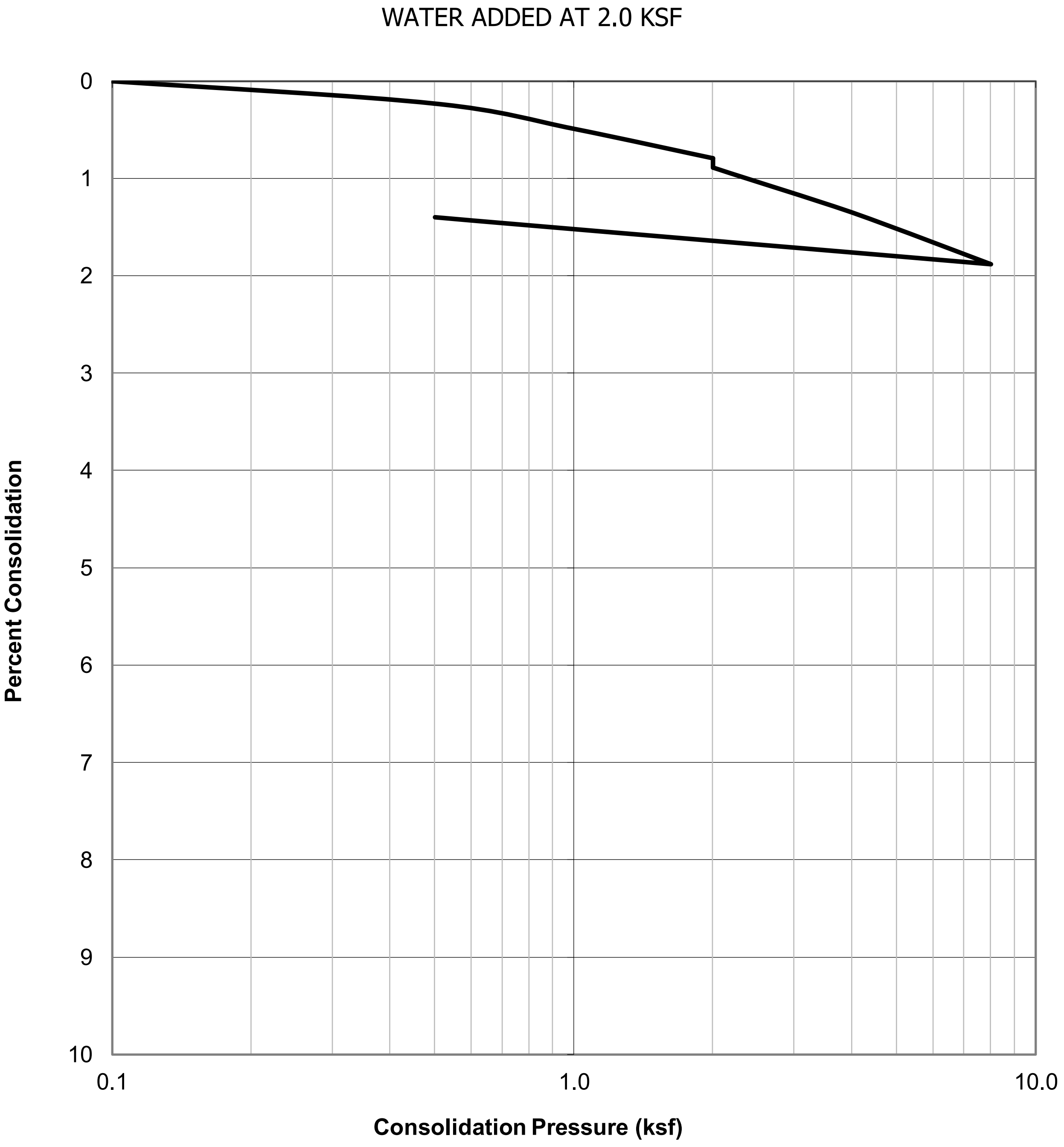
Figure B22



SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B-2@15'	Silty Sand (SM)	124.1	11.3	12.9

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CONSOLIDATION TEST RESULTS

ASTM D-2435

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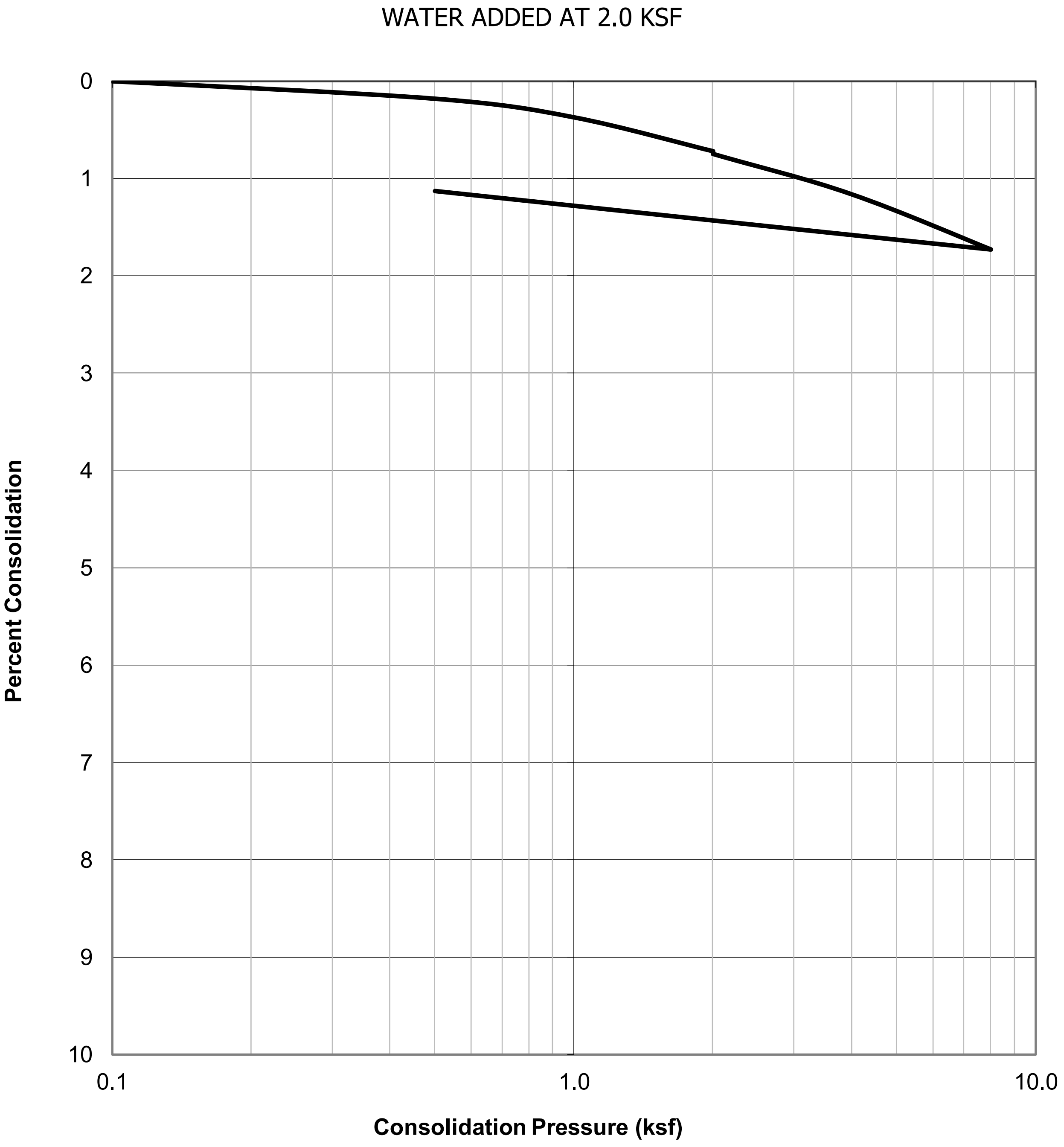
Figure B23



SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B-4@20'	Sandy Clay (CL)	121.8	14.2	15.0

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CONSOLIDATION TEST RESULTS

ASTM D-2435

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Project No.: W1572-06-01A

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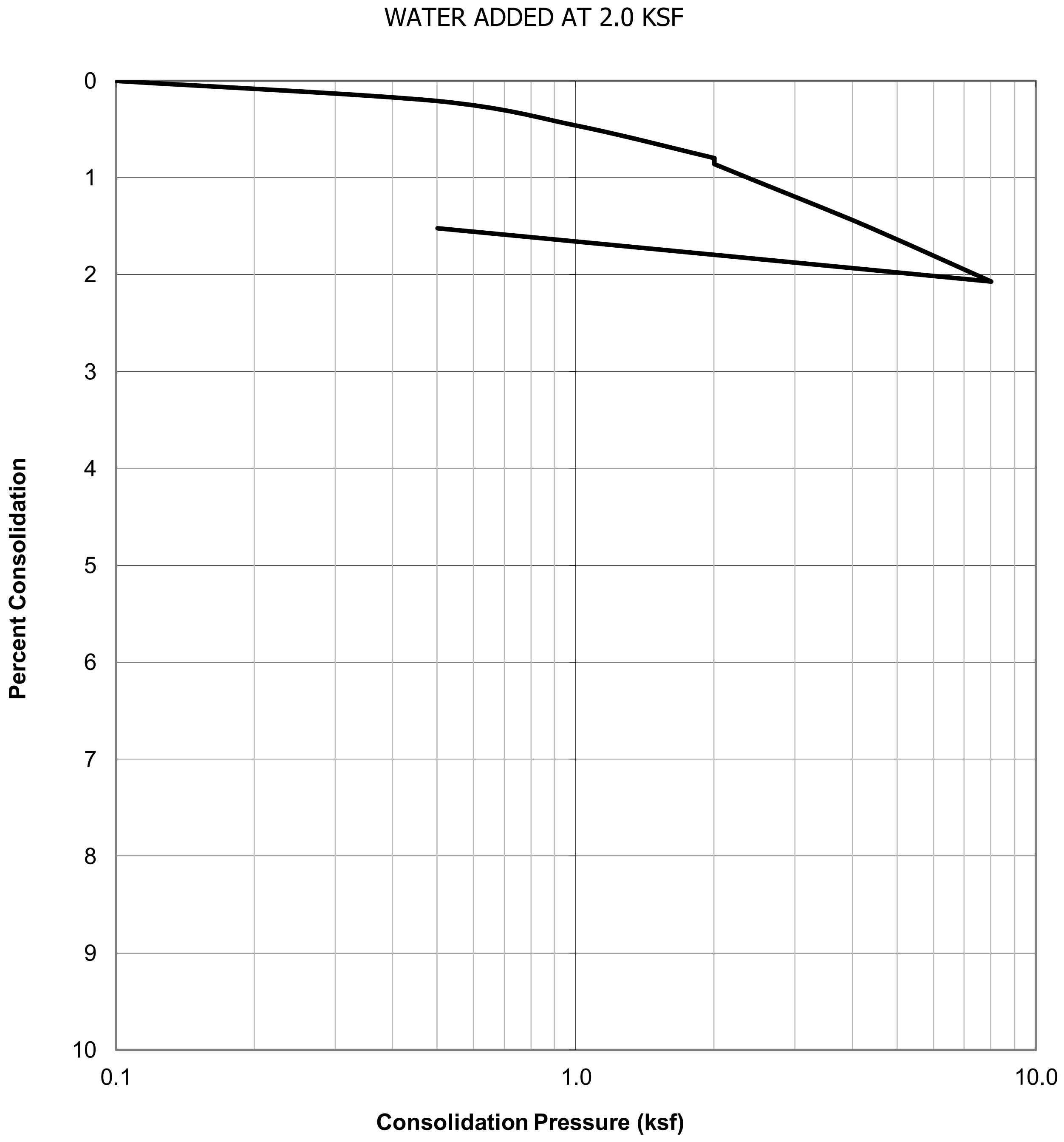
Figure B24



SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B-2@25'	Poorly Graded Sand (SP)	112.4	15.5	17.8

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CONSOLIDATION TEST RESULTS

ASTM D-2435

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Project No.: W1572-06-01A

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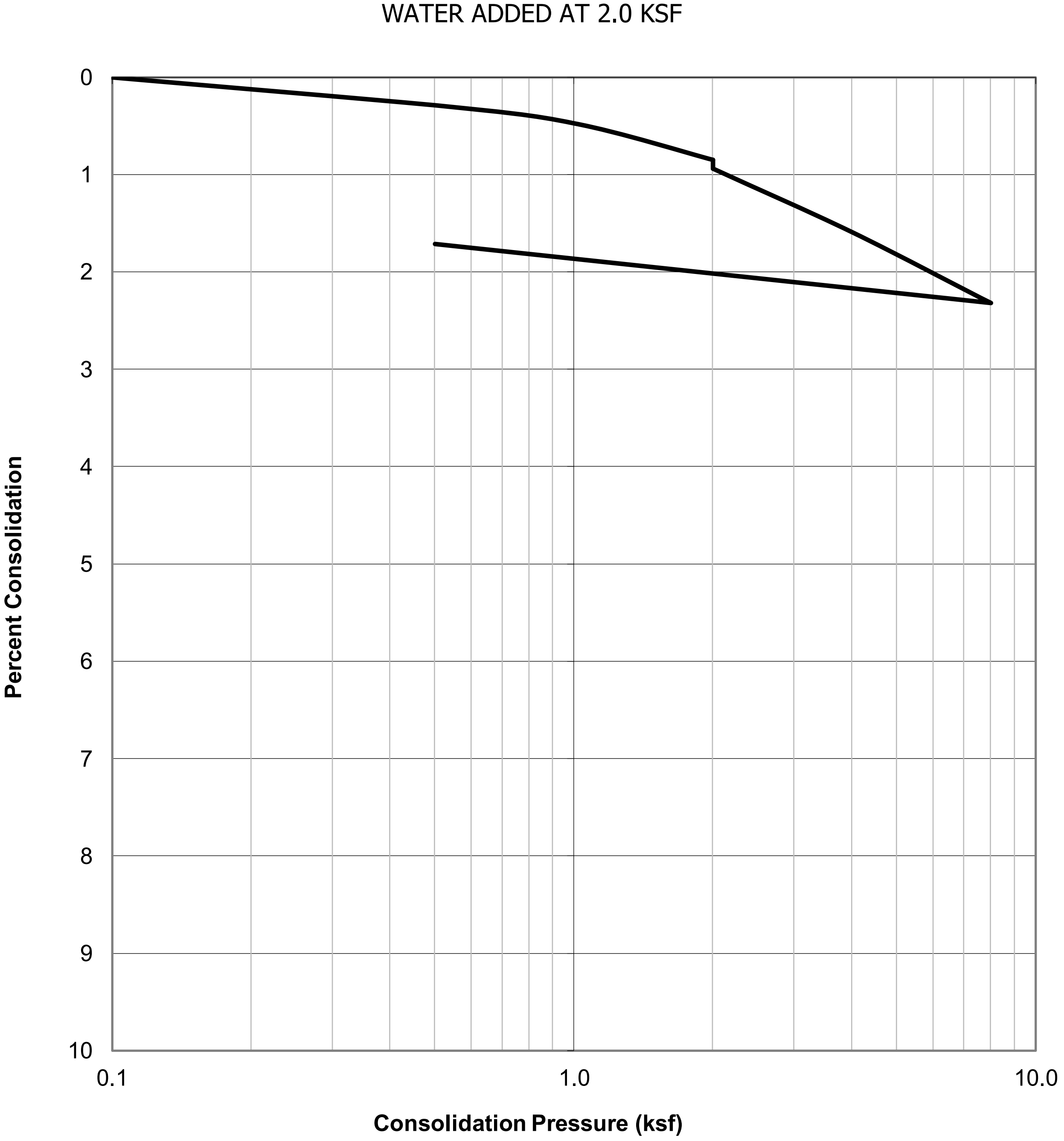
Figure B25



SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B-4@30'	Poorly Graded Sand (SP)	111.1	16.0	16.7

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CONSOLIDATION TEST RESULTS

ASTM D-2435

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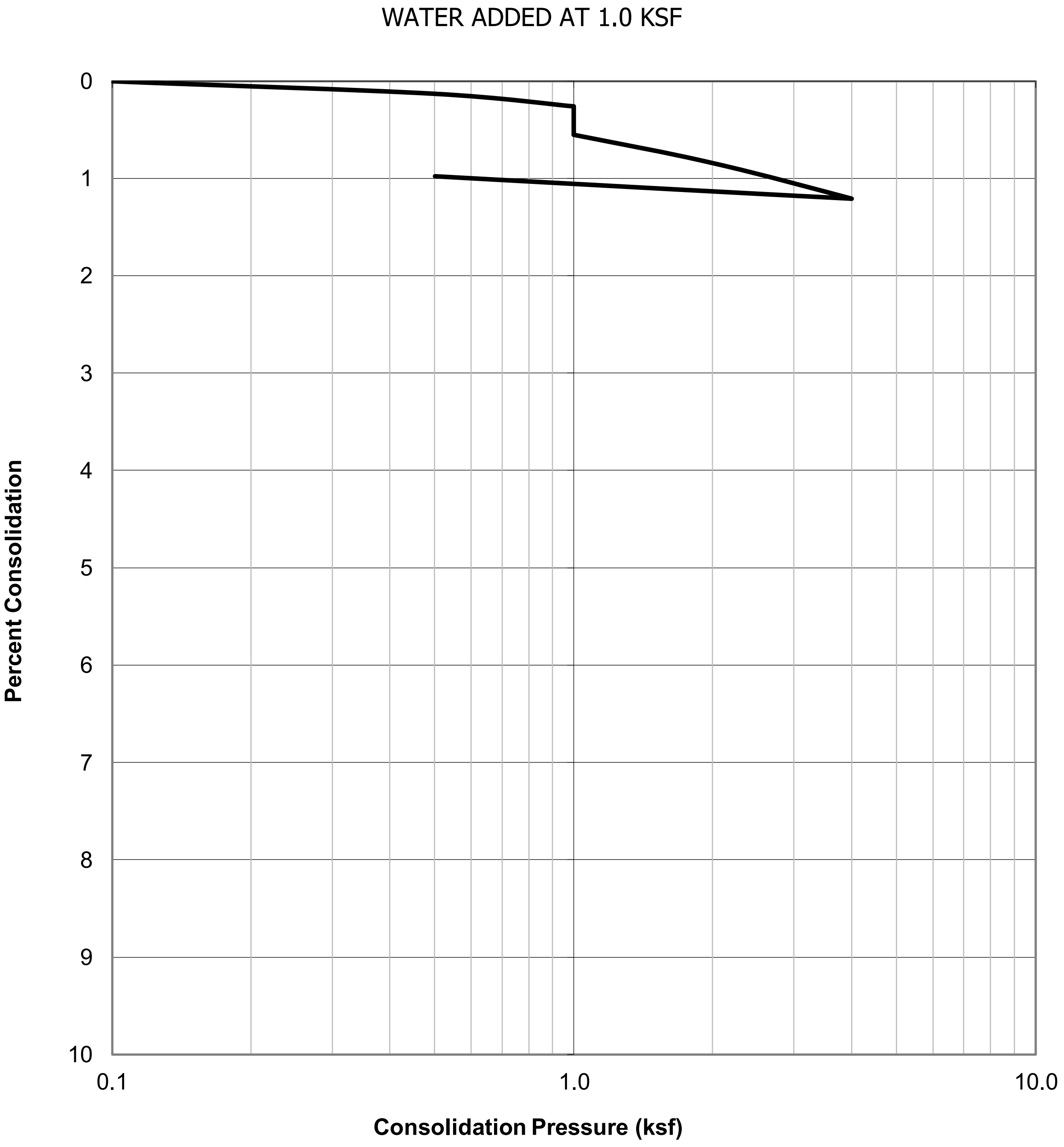
Figure B26



SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B-1@40'	Poorly Graded Sand (SP)	95.5	11.6	25.7

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CONSOLIDATION TEST RESULTS

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Project No.: W1572-06-01A

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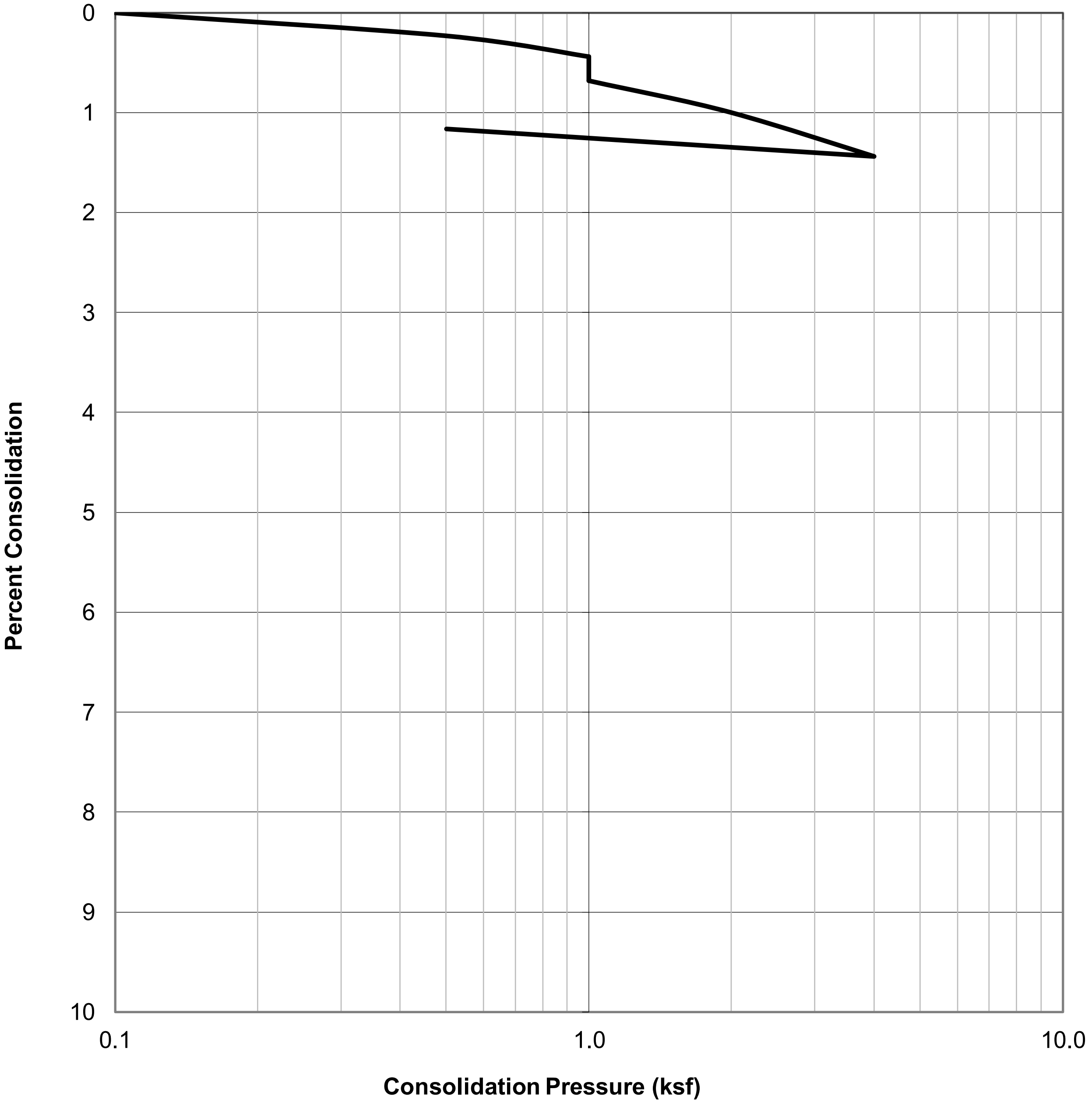
Figure B27

WATER ADDED AT 1.0 KSF



SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B-1@50'	Poorly Graded Sand (SP)	95.8	10.4	25.8

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CONSOLIDATION TEST RESULTS

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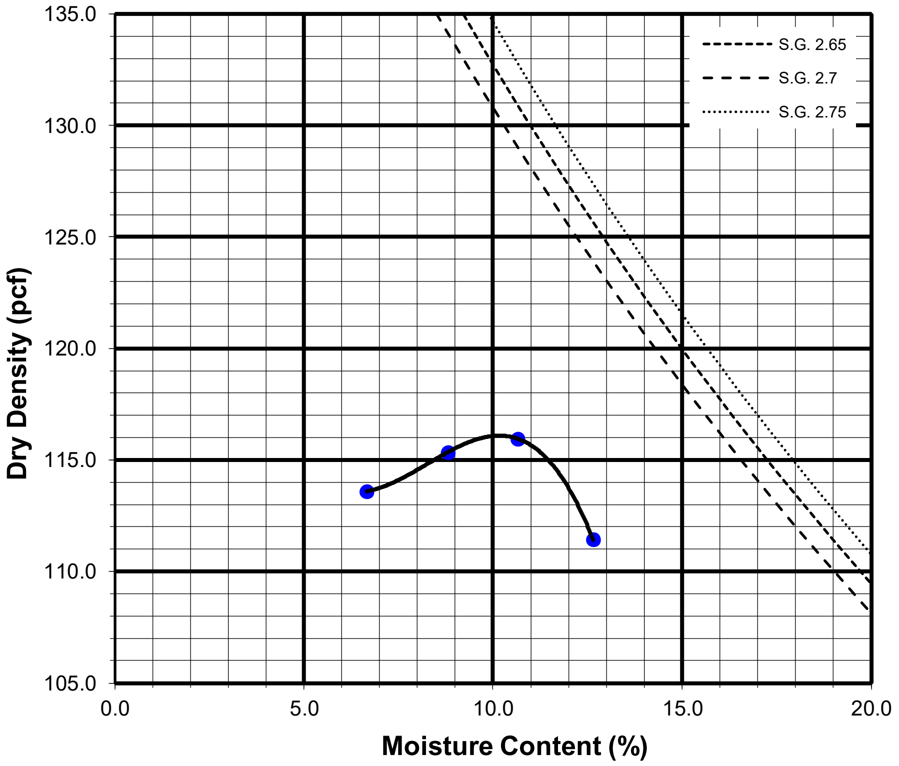
Sample No:

B3@0-5'	Silty Sand (SM)

TEST NO.	1	2	3	4	5	6
Wt. Compacted Soil + Mold (g)	6112	6178	6220	6178		
Weight of Mold (g)	4282	4282	4282	4282		
Net Weight of Soil (g)	1830	1896	1938	1896		
Wet Weight of Soil + Cont. (g)	2201.1	2271.3	2346.7	2260.7		
Dry Weight of Soil + Cont. (g)	2087.3	2118.0	2160.4	2049.4		
Weight of Container (g)	377.1	378.7	411.1	378.5		
Moisture Content (%)	6.7	8.8	10.6	12.6		
Wet Density (pcf)	121.2	125.5	128.3	125.5		
Dry Density (pcf)	113.6	115.4	116.0	111.4		

Maximum Dry Density (pcf)	**116.5**	**Optimum Moisture Content (%)**	**9.7**



Preparation Method: A

COMPACTION CHARACTERISTICS USING MODIFIED EFFORT TEST RESULTS

ASTM D-1557

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Figure B29

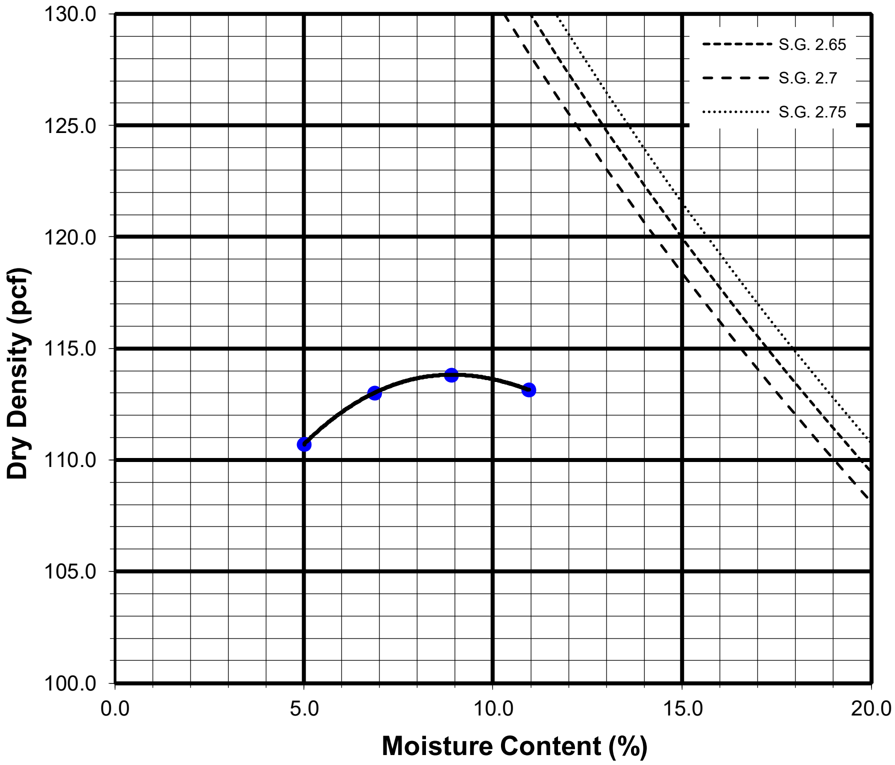
Sample No:

B6@5-10'	Silty Sand (SM)

TEST NO.		1	2	3	4	5	6
Wt. Compacted Soil + Mold	(g)	6038	6106	6154	6178		
Weight of Mold	(g)	4282	4282	4282	4282		
Net Weight of Soil	(g)	1756	1824	1872	1896		
Wet Weight of Soil + Cont.	(g)	2152.8	2199.0	2186.7	2265.4		
Dry Weight of Soil + Cont.	(g)	2069.7	2082.2	2039.1	2079.3		
Weight of Container	(g)	409.6	379.5	377.9	378.8		
Moisture Content	(%)	5.0	6.9	8.9	10.9		
Wet Density	(pcf)	116.3	120.8	123.9	125.5		
Dry Density	(pcf)	110.7	113.0	113.8	113.1		

Maximum Dry Density (pcf) **114.2**

Optimum Moisture Content (%) **7.9**



Preparation Method: A

COMPACTION CHARACTERISTICS USING MODIFIED EFFORT TEST RESULTS

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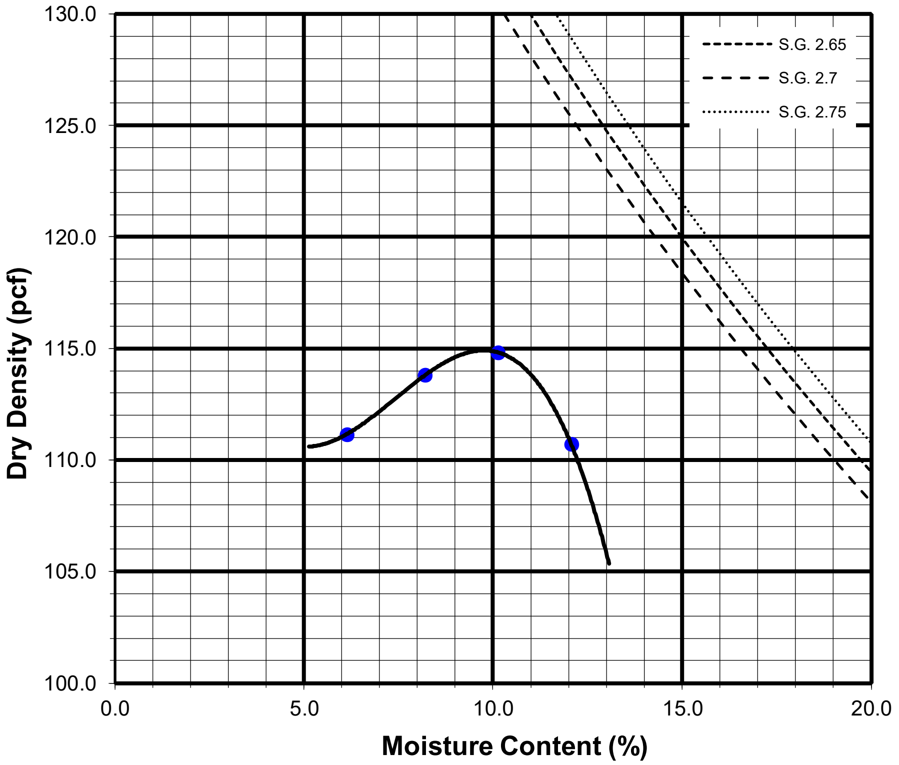
Figure B30

Sample No:

B10@5-10'	Silty Sand (SM)

TEST NO.		1	2	3	4	5	6
Wt. Compacted Soil + Mold	(g)	6064	6142	6192	6156		
Weight of Mold	(g)	4282	4282	4282	4282		
Net Weight of Soil	(g)	1782	1860	1910	1874		
Wet Weight of Soil + Cont.	(g)	2171.1	2249.1	2295.5	2258.9		
Dry Weight of Soil + Cont.	(g)	2067.5	2107.2	2119.2	2056.2		
Weight of Container	(g)	378.8	377.9	377.3	376.4		
Moisture Content	(%)	6.1	8.2	10.1	12.1		
Wet Density	(pcf)	118.0	123.1	126.4	124.1		
Dry Density	(pcf)	111.2	113.8	114.8	110.7		

Maximum Dry Density (pcf)	**115.1**	**Optimum Moisture Content (%)**	**9.2**



Preparation Method: A

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COMPACTION CHARACTERISTICS USING MODIFIED EFFORT TEST RESULTS

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Figure B31

B6@5-10'

MOLDED SPECIMEN		BEFORE TEST	AFTER TEST
Specimen Diameter	(in.)	4.0	4.0
Specimen Height	(in.)	1.0	1.0
Wt. Comp. Soil + Mold	(gm)	598.2	602.6
Wt. of Mold	(gm)	198.6	198.6
Specific Gravity	(Assumed)	2.7	2.7
Wet Wt. of Soil + Cont.	(gm)	676.7	602.6
Dry Wt. of Soil + Cont.	(gm)	649.5	363.3
Wt. of Container	(gm)	376.7	198.6
Moisture Content	(%)	10.0	11.2
Wet Density	(pcf)	120.5	121.7
Dry Density	(pcf)	109.6	109.4
Void Ratio		0.5	0.5
Total Porosity		0.4	0.3
Pore Volume	(cc)	72.4	72.4
Degree of Saturation	(%) [S_{meas}]	50.5	56.3

Date	Time	Pressure (psi)	Elapsed Time (min)	Dial Readings (in.)
6/15/2022	10:00	1.0	0	0.4007
6/15/2022	10:10	1.0	10	0.4007
Add Distilled Water to the Specimen				
6/16/2022	10:00	1.0	1430	0.4004
6/16/2022	11:00	1.0	1490	0.4004

Expansion Index (EI meas) =	-0.3
Expansion Index (Report) =	**0**

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

* Reference: 2019 California Building Code, Section 1803.5.3

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

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EXPANSION INDEX TEST RESULTS

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Figure B32

B9@10-15'

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)	584.2	595.0
Wt. of Mold	(gm)	176.4	176.4
Specific Gravity	(Assumed)	2.7	2.7
Wet Wt. of Soil + Cont.	(gm)	308.7	595.0
Dry Wt. of Soil + Cont.	(gm)	283.9	374.1
Wt. of Container	(gm)	8.7	176.4
Moisture Content	(%)	9.0	11.9
Wet Density	(pcf)	123.0	126.1
Dry Density	(pcf)	112.9	112.7
Void Ratio		0.5	0.5
Total Porosity		0.3	0.3
Pore Volume	(cc)	68.4	68.2
Degree of Saturation	(%) [S_{meas}]	49.6	65.2

Date	Time	Pressure (psi)	Elapsed Time (min)	Dial Readings (in.)
10/11/2023	10:00	1.0	0	0.4169
10/11/2023	10:10	1.0	10	0.4168
Add Distilled Water to the Specimen				
10/12/2023	10:00	1.0	1430	0.4159
10/12/2023	11:00	1.0	1490	0.4159

Expansion Index (EI meas) =	-0.9
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.

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EXPANSION INDEX TEST RESULTS

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Figure B33

B8@15-20'

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)	615.8	641.2
Wt. of Mold	(gm)	209.8	209.8
Specific Gravity	(Assumed)	2.7	2.7
Wet Wt. of Soil + Cont.	(gm)	308.6	641.2
Dry Wt. of Soil + Cont.	(gm)	283.8	372.5
Wt. of Container	(gm)	8.6	209.8
Moisture Content	(%)	9.0	15.8
Wet Density	(pcf)	122.5	130.0
Dry Density	(pcf)	112.4	112.2
Void Ratio		0.5	0.5
Total Porosity		0.3	0.3
Pore Volume	(cc)	69.0	68.4
Degree of Saturation	(%) [S_{meas}]	49.0	86.1

Date	Time	Pressure (psi)	Elapsed Time (min)	Dial Readings (in.)
10/11/2023	10:00	1.0	0	0.3851
10/11/2023	10:10	1.0	10	0.3849
Add Distilled Water to the Specimen				
10/12/2023	10:00	1.0	1430	0.382
10/12/2023	11:00	1.0	1490	0.382

Expansion Index (EI meas) =	-2.9
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.

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EXPANSION INDEX TEST RESULTS

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Figure B34

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

Sample No.	**pH**	**Resistivity (ohm centimeters)**
B6@5-10'	8.1	3200 (Moderately Corrosive)
B9@10-15'	8.2	3200 (Moderately Corrosive)
B8@15-20'	7.7	2900 (Moderately Corrosive)

SUMMARY OF LABORATORY CHLORIDE CONTENT TEST RESULTS
EPA NO. 325.3

Sample No.	Chloride Ion Content (%)
B6@5-10'	0.015
B9@10-15'	0.007
B8@15-20'	0.006

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

Sample No.	**Water Soluble Sulfate (% SQ_4)**	**Sulfate Exposure***
B6@5-10'	0.000	S0
B9@10-15'	0.003	S0
B8@15-20'	0.003	S0

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CORROSIVITY TEST RESULTS

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Figure B35