Appendix E

Updated Geotechnical Investigation

UPDATED GEOTECHNICAL INVESTIGATION

PROPOSED MIXED-USE DEVELOPMENT
2311 NORTH HOLLYWOOD WAY
BURBANK, CALIFORNIA
PM 269-99-100 LOT 1

GEOCON
WEST, INC.

GEOTECHNICAL
ENVIRONMENTAL
MATERIALS

PREPARED FOR

NHW INVESTORS, LLC

PROJECT NO. W1233-06-01

MAY 7, 2021

GEOCON
WEST, INC.
GEOTECHNICAL ■ ENVIRONMENTAL ■ MATERIALS

Project No. W1233-06-01
May 7, 2021

NHW Investors, LLC
1880 Century Park East, Suite 1017
Los Angeles, CA 90067

Subject: UPDATED GEOTECHNICAL INVESTIGATION
PROPOSED MIXED-USE DEVELOPMENT
2311 NORTH HOLLYWOOD WAY
BURBANK, CALIFORNIA
PM 269-99-100 LOT 1

Ladies and Gentlemen:

In accordance with your authorization of our proposal dated April 19, 2021, we have updated our geotechnical investigation report for the proposed mixed-use development located at 2311 North Hollywood Way in the City of Burbank, 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 mixed-use development located at 2311 North Hollywood Way in the City of Burbank, California (see Vicinity Map, Figure1). The purpose of the investigation was to evaluate subsurface soil and geologic conditions underlying the site and, based on conditions encountered, to provide conclusions and recommendations pertaining to the geotechnical aspects of proposed design and construction.

The scope of this investigation included a site reconnaissance, field exploration, laboratory testing, engineering analysis, and the preparation of this report. The site was explored on October 14, 2020 by excavating six 8-inch-diameter borings using a truck-mounted hollow-stem auger drilling machine. All borings were excavated to a depth of 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 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 2311 North Hollywood Way in the City of Burbank, California. The site is currently occupied by Fry's Electronics with associated paved parking lots, miscellaneous landscaping and flatwork improvements. The site is bounded by Vanowen Street to north, by North Hollywood Way to the east, by Valhalla Drive to the south, and by commercial developments to the west. The site is relatively level and surface water drainage at the site appears to flow to the city streets. There are grass lawns, miscellaneous landscaping, and trees localized in planter areas throughout the site.

Based on the information provided by the Client and a review of the updated plan set dated March 15, 2021, it is our understanding that the proposed development has changed to include a subterranean level beneath the proposed mixed-use structures, and the proposed office building will now be up to 5-stories in height.

The development, as presently proposed, will consist of two five- to seven-story mixed-use structures wrapped around two five-story parking structures on the eastern portion of the property. The mixed-use structures will include townhomes, multi-family residential units and retail space. It is anticipated that the mixed use structures will be constructed over one subterranean level extending up to 9 feet below the existing grade. The development will also include a 5-story office building tied to a 5 level parking structure on the western portion of the property which will be constructed at or near the existing grade. The limits of the proposed structures and adjacent structures are depicted on the Site Plans (see Figure 2A and 2B).

Based on the preliminary nature of the design at this time, wall and column loads were not available. It is anticipated that column loads for the proposed structures will be up to 900 kips, and wall loads will be up to 9.5 kips per linear foot.

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

3. GEOLOGIC SETTING

The site is located within the eastern portion of the San Fernando Valley, an alluvial-filled basin approximately 23 miles wide and 12 miles long (Hitchcock & Wills, 2000). The alluvium within the San Fernando Valley is mainly derived from the Santa Monica Mountains to the south, the Santa Susana Mountains to the north, the Simi Hills to the west, the San Gabriel Mountains to the northeast, and the Verdugo Mountains to the east. Regionally, the site is located in the southern portion Transverse Ranges geomorphic province which is characterized by east-west trending geologic structures such as the nearby Santa Monica Mountains and the east-west trending active San Fernando Fault Zone.

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 unconsolidated Holocene age alluvial fan deposits consisting of silt, sand, and gravel (California Geological Survey, 2012; Hitchcock and Wills, 2000). Detailed stratigraphic profiles are provided on the boring logs in Appendix A.

4.1 Artificial Fill

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

4.2 Alluvial Fan Deposits

Holocene age alluvial fan deposits were encountered beneath the fill. The alluvium generally consists of light brown to brown, grayish brown, and light gray silty sand and poorly graded sand with varying amounts of fine to coarse gravel and cobbles. The alluvial soils are characterized as dry to moist and loose to very dense.

5. GROUNDWATER

Review of the Seismic Hazard Zone Report for the Burbank Quadrangle (California Division of Mines and Geology [CDMG], 1998) indicates that the historically highest groundwater level in the area is approximately 50 to 60 feet beneath the existing ground surface. Groundwater information presented in this document is generated from data collected in the early 1900's to the late 1990s.

The site is located within the San Fernando Groundwater Basin. Based on information provided by EFI Global, Inc. (2020), the depth to groundwater in the vicinity of the site is approximately 205 to 210 feet beneath the existing ground surface. Groundwater flow is toward the east (EFI Global, Inc., 2020)

Groundwater was not encountered in our field explorations, drilled to a maximum depth of 30½ feet below the existing ground surface. Based on the reported historic high groundwater level in the site vicinity, the lack of groundwater in our borings, and the depth of proposed construction, groundwater is neither expected to be encountered during construction nor have a detrimental effect on the project. However, it is not uncommon for groundwater levels to vary seasonally or for groundwater seepage conditions to develop where none previously existed, especially in impermeable fine-grained soils which are heavily irrigated or after seasonal rainfall. In addition, recent requirements for stormwater infiltration could result in shallower seepage conditions in the immediate site vicinity. Proper surface drainage of irrigation and precipitation will be critical for future performance of the project. Recommendations for drainage are provided in the Surface Drainage section of this report (see Section 7.21).

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 (CDMG, 1979; CGS, 2020a; 2020b) 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 Verdugo Fault located approximately 1.2 miles to the northeast (USGS, 2006; CDMG, 1979). Other nearby active faults are an Unnamed Fault, the San Fernando Fault Zone, the Hollywood Fault, the Sierra Madre Fault Zone, and the Raymond Fault located approximately 1.2 miles southwest, 5.9 miles north, 5.9 miles south, 6.1 miles northeast, and 8.2 miles southeast of the site, respectively (USGS, 2006; Ziony and Jones, 1989). The active San Andreas Fault Zone is located approximately 28 miles northeast of the site (Ziony and Jones, 1989).

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

6.2 Seismicity

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

LIST OF HISTORIC EARTHQUAKES

Earthquake (Oldest to Youngest)	Date of Earthquake	Magnitude	Distance to Epicenter (Miles)	Direction to Epicenter
Near Redlands	July 23, 1923	6.3	64	E
Long Beach	March 10, 1933	6.4	45	SE
Tehachapi	July 21, 1952	7.5	67	NW
San Fernando	February 9, 1971	6.6	15	NNW
Whittier Narrows	October 1, 1987	5.9	18	ENE
Sierra Madre	June 28, 1991	5.8	20	ENE
Landers	June 28, 1992	7.3	109	E
Big Bear	June 28, 1992	6.4	87	E
Northridge	January 17, 1994	6.7	11	WNW
Hector Mine	October 16, 1999	7.1	122	ENE
Ridgecrest	July 5, 2019	7.1	117	NNE

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

6.3 Seismic Design Criteria

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

2019 CBC SEISMIC DESIGN PARAMETERS

Parameter	Value	2019 CBC Reference
Site Class	D	Section 1613.2.2
MCE_R Ground Motion Spectral Response Acceleration – Class B (short), S_S	1.995g	Figure 1613.2.1(1)
MCE_R Ground Motion Spectral Response Acceleration – Class B (1 sec), S_1	0.643g	Figure 1613.2.1(2)
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}	1.995g	Section 1613.2.3 (Eqn 16-36)
Site Class Modified MCE_R Spectral Response Acceleration – (1 sec), S_{M1}	1.093g*	Section 1613.2.3 (Eqn 16-37)
5% Damped Design Spectral Response Acceleration (short), S_{DS}	1.33g	Section 1613.2.4 (Eqn 16-38)
5% Damped Design Spectral Response Acceleration (1 sec), S_{D1}	0.729g*	Section 1613.2.4 (Eqn 16-39)

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

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

ASCE 7-16 PEAK GROUND ACCELERATION

Parameter	Value	ASCE 7-16 Reference
Mapped MCE_G Peak Ground Acceleration, PGA	0.803g	Figure 22-7
Site Coefficient, F_{PGA}	1.1	Table 11.8-1
Site Class Modified MCE_G Peak Ground Acceleration, PGA_M	0.883g	Section 11.8.3 (Eqn 11.8-1)

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

Deaggregation of the MCE peak ground acceleration was performed using the USGS online Unified Hazard Tool, 2014 Conterminous U.S. Dynamic edition (v4.2.0). The result of the deaggregation analysis indicates that the predominant earthquake contributing to the MCE peak ground acceleration is characterized as a 6.91 magnitude event occurring at a hypocentral distance of 11.11 kilometers from the site.

Deaggregation was also performed for the Design Earthquake (DE) peak ground acceleration, and the result of the analysis indicates that the predominant earthquake contributing to the DE peak ground acceleration is characterized as a 6.77 magnitude occurring at a hypocentral distance of 14.03 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 Burbank Quadrangle (CDMG, 1999) indicates that the site is not located in an area designated as having a potential for liquefaction. The historic high groundwater level in the vicinity of the site is at a depth of 50 to 60 feet (CDMG, 1998). Based on these considerations, the potential for liquefaction and associated ground deformations beneath the site is considered very low.

6.5 Slope Stability

The topography at the site is relatively level and the topography in the immediate site vicinity slopes gently to the southeast. The City of Burbank Safety Element (2013) and the County of Los Angeles Safety Element (Leighton, 1990) indicate the site is not located within a "hillside area" or within an area identified as having a potential for slope instability. Additionally, the site is not within an area identified as having a potential for seismic slope instability (CGS, 2020b; 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 (County of Los Angeles, 1990; Leighton, 1990) indicates that the site is located within the Hansen Dam inundation area. However, this reservoir, as well as others in California, are continually monitored by various governmental agencies (such as the State of California Division of Safety of Dams and the U.S. Army Corps of Engineers) to guard against the threat of dam failure. Current design, construction practices, and ongoing programs of review, modification, or total reconstruction of existing dams are intended to ensure that all dams are capable of withstanding the maximum considered earthquake (MCE) for the site. 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 not located within a coastal area. Therefore, tsunamis are not considered a significant hazard at the site.

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

The site is located within an area of minimal flooding (Zone X) as defined by the Federal Emergency Management Agency (FEMA, 2020; LACDPW, 2020). Therefore, flooding is not anticipated to adversely impact the site.

6.8 Oil Fields & Methane Potential

Based on a review of the California Geologic Energy Management Division (CalGEM) Well Finder Website, the site is not located within an oil field and active oil or gas wells are not documented in the immediate site vicinity (CalGEM, 2020). However, due to the voluntary nature of record reporting by the oil well drilling companies, wells may be improperly located or not shown on the location map and undocumented wells could be encountered during construction. Any wells encountered during construction will need to be properly abandoned in accordance with the current requirements of the CalGEM.

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

6.9 Subsidence

Subsidence occurs when a large portion of land is displaced vertically, usually due to the withdrawal of groundwater, oil, or natural gas. Soils that are particularly subject to subsidence include those with high silt or clay content. The site is not located within an area of known ground subsidence. No known large-scale extraction of groundwater, gas, oil, or geothermal energy is occurring or planned at the site or in the general site vicinity. There appears to be little or no potential for ground subsidence due to withdrawal of fluids or gases at the site.

7. CONCLUSIONS AND RECOMMENDATIONS

7.1 General

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

7.1.2 Up to 2 feet of existing artificial fill was encountered during the site investigation. The existing fill encountered is believed to be the result of past grading and construction activities at the site. Deeper fill may exist in other areas of the site that were not directly explored. Future demolition of the existing structures and improvements which occupy the site will likely disturb the upper few feet of soil. It is our opinion that the existing fill in its present condition, is not suitable for direct support of proposed foundations or slabs. The existing fill and site soils are suitable for re-use as engineered fill provided the recommendations in the Grading section of this report are followed (see Section 7.4).

7.1.3 Where proposed structures will be constructed with a subterranean level the structure may be supported on a conventional foundation system deriving support in the competent alluvium found below a depth of 9 feet below the existing ground surface. Foundations should be deepened as necessary to penetrate through any encountered soft or unsuitable alluvium at the direction of the Geotechnical Engineer.

7.1.4 Where structures will be constructed at or near present site grade the structure may be supported on a conventional foundation system deriving support in newly placed engineered fill. All foundations less than 9 feet deep should be underlain by a minimum of 3 feet of newly placed engineered fill. As a minimum it is recommended that the upper 5 feet of existing earth materials within the building footprint areas be excavated and properly compacted for foundation and slab support. Deeper excavations should be conducted as needed to remove any encountered artificial fill or soft soils as necessary at the direction of the Geotechnical Engineer (a representative of Geocon) and or to maintain the required 3-foot-thick fill blanket beneath foundations. The excavation should extend laterally a minimum distance of 3 feet beyond the building footprint areas, including building appurtenances, or a distance equal to the depth of fill below the foundation, whichever is greater. The limits of existing fill and/or soft soil removal will be verified by the Geocon representative during site grading activities. Recommendations for earthwork are provided in the *Grading* section of this report (see Section 7.4). Recommendations for the design of a conventional foundation system are provided in Section 7.6.

7.1.5 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 scarified, moistened, and proof-rolled with heavy equipment in the presence of the Geotechnical Engineer (a representative of Geocon West, Inc.).

7.1.6 Where sufficient space is available it is anticipated that stable excavations for the recommended excavations and grading can be achieved with sloping measures. Recommendations for temporary excavations are provided in Section 7.17 of this report.

7.1.7 It is anticipated that excavations on the order of 12 feet in vertical height may be required for construction of the subterranean level, including foundation depths. Due to the depth of the excavation and the proximity to the property lines, city streets and adjacent offsite improvements, excavations may require 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.

7.1.8 Foundations for small outlying structures, such as block walls less than 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 or is undesirable, foundations may derive support directly in the competent undisturbed alluvial soils and should be deepened as necessary to maintain a minimum 12 inch embedment into the recommended bearing materials. 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.

7.1.9 Where new paving is to be placed, it is recommended that all existing fill and soft alluvial soils be excavated and properly compacted for paving support. The client should be aware that excavation and compaction of all existing fill and soft alluvial soils in the area of new paving is not required; however, paving constructed over existing uncertified fill or unsuitable alluvial soil may experience increased settlement and/or cracking, and may therefore have a shorter design life and increased maintenance costs. As a minimum, the upper 12 inches of subgrade soil should be scarified and properly compacted for paving support. Paving recommendations are provided in *Preliminary Paving Design* section of this report (see Section 7.11).

7.1.10 Based on the results of the percolation testing performed at the site, a stormwater infiltration system is considered feasible for this project. Preliminary recommendations are provided in the *Stormwater Infiltration* section of this report (Section 7.20)

7.1.11 Once the design and foundation loading configuration for the proposed structures proceeds to a more finalized plan, the recommendations within this report should be reviewed and revised, if necessary. Based on the final foundation loading configurations, the potential for settlement should be reevaluated by this office.

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

7.2 Soil and Excavation Characteristics

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

7.2.2 It is the responsibility of the contractor to ensure that all excavations and trenches are properly shored and maintained in accordance with applicable OSHA rules and regulations to maintain safety and maintain the stability of 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 upper 5 feet of existing site soils encountered during the investigation are considered to have a “very low” expansive potential (EI = 0) and are classified as “non-expansive” in accordance with the 2019 California Building Code (CBC) Section 1803.5.3. The recommendations presented herein assume that the building foundations and slabs will derive support in these materials.

7.3 Minimum Resistivity, pH, and Water-Soluble Sulfate

7.3.1 Potential of Hydrogen (pH) and resistivity testing as well as chloride content testing were performed on representative samples of soil to generally evaluate the corrosion potential to surface utilities. The tests were performed in accordance with California Test Method Nos. 643 and 422 and indicate that the soils are considered "mildly corrosive" to "moderately corrosive" with respect to corrosion of buried ferrous metals on site. The results are presented in Appendix B (Figure B33) and should be considered for design of underground structures. Due to the corrosive potential of the soils, it is recommended that PVC, ABS or other approved plastic piping be utilized in lieu of cast-iron when in direct contact with the site soils.

7.3.2 Laboratory tests were performed on representative samples of the site materials to measure the percentage of water-soluble sulfate content. Results from the laboratory water-soluble sulfate tests are presented in Appendix B (Figure B33) and indicate that the on-site materials possess a sulfate exposure class of "S0" to concrete structures as defined by 2019 CBC Section 1904 and ACI 318-14 Table 19.3.1.1.

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

7.4 Grading

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

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

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

7.4.4 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.) and the City Inspector.

7.4.5 Where proposed structures will be constructed with a subterranean level the structure may be supported on a conventional foundation system deriving support in the competent alluvium found below a depth of 9 feet below the existing ground surface. Foundations should be deepened as necessary to penetrate through any encountered soft or unsuitable alluvium at the direction of the Geotechnical Engineer.

7.4.6 Where structures will be constructed at or near present site grade the structure may be supported on a conventional foundation system deriving support in newly placed engineered fill. All foundations less than 9 feet deep should be underlain by a minimum of 3 feet of newly placed engineered fill. As a minimum it is recommended that the upper 5 feet of existing earth materials within the building footprint areas be excavated and properly compacted for foundation and slab support. Deeper excavations should be conducted as needed to remove any encountered artificial fill or soft soils as necessary at the direction of the Geotechnical Engineer (a representative of Geocon) and or to maintain the required 3-foot-thick fill blanket beneath foundations. The excavation should extend laterally a minimum distance of 3 feet beyond the building footprint areas, including building appurtenances, or a distance equal to the depth of fill below the foundation, whichever is greater. The limits of existing fill and/or soft soil removal will be verified by the Geocon representative during site grading activities.

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

7.4.8 All fill and backfill soils within the building areas should be placed in horizontal loose layers approximately 6 to 8 inches thick, moisture conditioned to near optimum moisture content, and properly compacted to at least 95 percent relative compaction in accordance with ASTM D 1557 (latest edition).

7.4.9. Where new paving is to be placed, it is recommended that all existing fill and soft alluvium be excavated and properly compacted for paving support. As a minimum, the upper 12 inches of soil should be scarified, moisture conditioned to 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 7.11).

7.4.10 It is anticipated that stable excavations for the recommended grading associated with the proposed structures can be achieved with sloping measures. Due to the granular nature of the soils there is a potential for excessive caving in vertical excavations. If excavations in close proximity to an adjacent property line and/or existing improvement are required, special excavation measures may be necessary in order to maintain lateral support of existing improvements. Excavation recommendations are provided in the *Temporary Excavations* section of this report (Section 7.17)

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

7.4.12 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 B33). 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).

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

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

7.5 Shrinkage

7.5.1 Shrinkage results when a volume of material removed at one density is compacted to a higher density. A shrinkage factor between 5 and 15 percent should be anticipated when excavating and compacting the upper site soils to an average relative compaction of 95 percent.

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

7.6 Foundation Design – Spread Foundation

7.6.1 Subsequent to the recommended grading, a conventional shallow spread foundation system may be utilized for support of the proposed structure. Where not supported directly in the competent alluvial soils found at or below a depth of 9 feet, all building foundations should be underlain by a minimum of 3 feet of newly placed engineered fill.

7.6.2 Continuous footings may be designed for an allowable bearing capacity of 2,300 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 materials.

7.6.3 Isolated spread foundations may be designed for an allowable bearing capacity of 2,700 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 materials.

7.6.4 The soil bearing pressures above may be increased by 500 psf and 1,000 psf for each additional foot of foundation width and depth, respectively, up to a maximum allowable soil bearing pressure of 5,000 psf.

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

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

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

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

7.6.9 No special subgrade presaturation is required prior to placement of concrete. However, the moisture in the foundation subgrade should be sprinkled as necessary to maintain a moist condition at the time of concrete placement.

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

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

7.7 Foundation Settlement

7.7.1 The maximum expected static settlement for a proposed structure supported on a conventional foundation system deriving support in the engineered fill and or competent alluvium at or below a depth of 9 feet, and designed with a maximum bearing pressure of 5,000 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.

7.7.2 Once the design and foundation loading configurations for the proposed structures proceeds to a more finalized plan, the estimated total and differential 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.

7.8 Miscellaneous Foundations

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

7.8.2 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.

7.8.3 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.

7.9 Lateral Design

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

7.9.2 Passive earth pressure for the sides of foundations and slabs poured against the alluvial soils or properly compacted engineered fill may be computed as an equivalent fluid having a density of 270 pcf with a maximum earth pressure of 2,700 pcf. When combining passive and friction for lateral resistance, the passive component should be reduced by one-third.

7.10 Concrete Slabs-on-Grade

7.10.1 Concrete slabs-on-grade subject to vehicle loading should be designed in accordance with the recommendations in the *Preliminary Paving Design* section of this report (Section 7.11).

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

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

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

7.10.5 Exterior slabs for walkways or flatwork, not subject to traffic loads, should be at least 4 inches thick and reinforced with No. 3 steel reinforcing bars placed 18 inches on center in both horizontal directions, positioned near the slab midpoint. Prior to construction of slabs, the upper 12 inches of subgrade should be moistened to 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 12 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.

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

7.11 Preliminary Paving Design

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

7.11.2 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.

7.11.3 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

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

7.11.5 Unless specifically designed and evaluated by the project structural engineer, where 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 properly compacted subgrade. 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). The base material should be compacted to 95 percent relative compaction, as determined by ASTM Test Method D 1557 (latest edition).

7.11.6 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.

7.12 Retaining Wall Design

7.12.1 The recommendations presented below are generally applicable to the design of rigid concrete or masonry retaining walls having a maximum height of 10 feet. In the event that walls significantly higher than 5 feet are planned, Geocon should be contacted for additional recommendations.

7.12.2 Retaining wall foundations should be designed in accordance with the recommendations provided in the foundation section of this report.

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

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	30	51

7.12.4 The wall pressures provided above assume that the proposed retaining walls will support relatively undisturbed alluvium 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.

7.12.5 The wall pressures provided above assume that the retaining wall will be properly drained preventing the buildup of hydrostatic pressure. If retaining wall drainage is not implemented, the equivalent fluid pressure to be used in design of undrained walls is 90 pcf. The value includes hydrostatic pressures plus buoyant lateral earth pressures.

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

7.12.7 Additional active pressure should be added for a surcharge condition due to sloping ground, vehicular traffic or adjacent structures and should be designed for each condition as the project progresses. Surcharges may be evaluated using Section 7.19 of this report. Once the design becomes more finalized, an addendum letter can be prepared revising recommendations and addressing specific surcharge conditions throughout the project, if necessary.

7.13 Dynamic (Seismic) Lateral Forces

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

7.13.2 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 2019 CBC. The seismic load is applied as an equivalent fluid pressure along the height of the wall and the calculated loads result in a maximum load exerted at the base of the wall and zero at the top of the wall. This seismic load should be applied in addition to the active earth pressure. The earth pressure is based on half of two-thirds of PGA_M calculated from ASCE 7-16 Section 11.8.3.

7.14 Retaining Wall Drainage

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

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

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

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

7.15 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 7.6 and 7.12).

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

7.15.3 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 7.14).

7.15.4 Subdrainage pipes at the base of the retaining wall drainage system should outlet to a location acceptable to the building official.

7.15.5 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 the drilled excavation could compromise the existing foundation support, especially if the drilling is performed subsequent to the foundation construction.

7.16.2 Excessive caving will occur in the granular soils and casing will be required for this project. Cobbles and boulders may also be encountered. The contractor should be prepared for difficult drilling conditions and the requirement 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 on the order of 12 feet in height may be required for excavation and construction of the subterranean level, including foundation depths. The excavations are expected to expose artificial fill and alluvial soils, which are subject to excessive caving. Vertical excavations up to 3 feet in height may be attempted where not surcharged by adjacent foundations or traffic; however, the contractor should be prepared for caving sands in open excavations and formwork may be required in foundation excavations.

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 7 feet. Temporary unsurcharged embankments greater than 7 feet and less than 15 feet may be sloped back at a uniform 1.5:1 (H:V) slope gradient or flatter. 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 slopes are utilized, the top of the slope should be barricaded to prevent vehicles and storage loads at the top of the slope within a horizontal distance equal to the height of the slope. If the temporary construction slopes are to be maintained during the rainy season, berms are suggested along the tops of the slopes where necessary to prevent runoff water from entering the excavation and eroding the slope faces. Geocon personnel should inspect the soils exposed in the cut slopes during excavation so that modifications of the slopes can be made if variations in the soil conditions occur. All excavations should be stabilized within 30 days of initial excavation.

7.18 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 foundations 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 7.12).

7.18.5 Drilled cast-in-place soldier piles should be placed no closer than three 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 270 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 alluvium.

7.18.6 If caving is experienced the contractor may require casing and should have casing available prior to commencement of drilling activities. 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.

7.18.7 Groundwater was not encountered at the time of exploration; however, groundwater seepage may be encountered due to heavy seasonal rainfall at the time of construction. The contractor should be aware of the requirements for pile installation should groundwater be encountered. Piles placed below the water level require the use of a tremie to place the concrete into the bottom of the hole. A tremie should consist of a rigid, water-tight tube having a diameter of not less than 6 inches with a hopper at the top. The tube should be equipped with a device that will close the discharge end and prevent water from entering the tube while it is being charged with concrete. The tremie should be supported so as to permit free movement of the discharge end over the entire top surface of the work and to permit rapid lowering when necessary to retard or stop the flow of concrete. The discharge end should be closed at the start of the work to prevent water entering the tube and should be entirely sealed at all times, except when the concrete is being placed. The tremie tube should be kept full of concrete. The flow should be continuous until the work is completed and the resulting concrete seal should be monolithic and homogeneous. The tip of the tremie tube should always be kept about 5 feet below the surface of the concrete and definite steps and safeguards should be taken to insure that the tip of the tremie tube is never raised above the surface of the concrete.

7.18.8 A special concrete mix should be used for concrete to be placed below water. The design should provide for concrete with an unconfined compressive strength psi of 1,000 pounds per square inch (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.

7.18.9 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.

7.18.10 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.

7.18.11 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.

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

7.18.13 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.

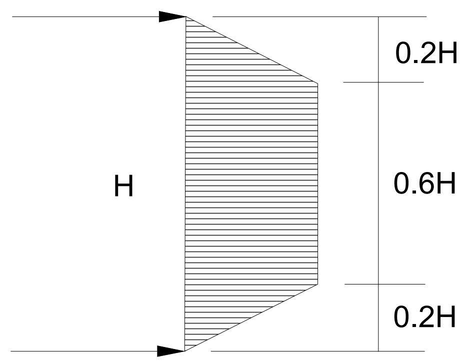
7.18.15 The frictional resistance between the soldier piles and retained soil may be used to resist the vertical component of the load. The coefficient of friction may be taken as 0.40 based on uniform contact between the steel beam and lean-mix concrete and alluvial soils. The portion of soldier piles 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 350 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 cohesive soils and the areas where lagging may be omitted.

7.18.17 For the design of unbraced shoring, it is recommended that an equivalent fluid pressure be utilized for design. A trapezoidal distribution of lateral earth pressure may be used where shoring will be restrained at the top by bracing or tie backs. The recommended active and trapezoidal pressures 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 Active Trapezoidal (Where H is the height of the shoring in feet)
Up to 12	25	16H

Trapezoidal Distribution of Pressure



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

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

7.18.20 Additional active pressure should be added for a surcharge condition due to sloping ground, vehicular traffic or adjacent structures and should be designed for each condition as the project progresses. Surcharges may be evaluated using Section 7.19 of this report. Once the design becomes more finalized, an addendum letter can be prepared revising recommendations and addressing specific surcharge conditions throughout the project, if necessary.

7.19 Surcharge from Adjacent Structures and Improvements

7.19.1 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.

7.19.2 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.

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

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

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

and

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

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

then

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

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

7.20 Stormwater Infiltration

7.20.1 During the October 14, 2020 site exploration, boring B1 was utilized to perform percolation testing. The boring was over excavated below the anticipated infiltration invert to collect samples, and then backfilled with bentonite to the anticipated invert elevation for percolation testing. Slotted casing was placed in the boring, and the annular space between the casing and excavation was filled with gravel. The casing was filled with water and percolation test readings were performed after repeated flooding of the cased excavations. Based on the test results, the measured percolation rates and design infiltration rates, for the earth materials encountered, are provided in the following table. These values have been calculated in accordance with the Boring Percolation Test Procedure in the County of Los Angeles Department of Public Works GMED *Guidelines for Geotechnical Investigation and Reporting, Low Impact Development Stormwater Infiltration* (June 2017). Percolation test field data and calculation of the measured percolation rate and design infiltration rate are provided on Figure 7.

Boring	Soil Type	Infiltration Depth (ft)	Measured Percolation Rate (in / hour)	Design Infiltration Rate (in / hour)
B1	Poorly Graded Sand	5-7	3.86	1.93

7.20.2 Based on the test method utilized (Boring Percolation Test), the reduction factor RF_t may be taken as 2.0 in the infiltration system design. Based on the number of tests performed and consistency of the soils throughout the site, it is suggested that the reduction factor RF_v be taken as 1.0. In addition, provided proper maintenance is performed to minimize long-term siltation and plugging, the reduction factor RF_s may be taken as 1.0. Additional reduction factors may be required and should be applied by the engineer in responsible charge of the design of the stormwater infiltration system and based on applicable guidelines.

7.20.3 The results of the percolation testing indicate that the alluvial soils at depths in the above table are conductive to infiltration. It is our opinion that the alluvial soils encountered are suitable for infiltration of stormwater.

7.20.4 It is our further opinion that infiltration of stormwater and will not induce excessive hydro-consolidation (see Figures B11 through B26), will not create a perched groundwater condition, will not affect soil structure interaction of existing or proposed foundations due to expansive soils, will not saturate soils supported by existing or proposed retaining walls, and will not increase the potential for liquefaction. Resulting settlements are anticipated to be less than ¼ inch, if any.

7.20.5 The infiltration system must be located such that the closest distance between an adjacent foundation is at least 10 feet in all directions from the zone of saturation. The zone of saturation may be assumed to project downward from the discharge of the infiltration facility at a gradient of 1:1. Additional property line or foundation setbacks may be required by the governing jurisdiction and should be incorporated into the stormwater infiltration system design as necessary.

7.20.6 Where the 10-foot horizontal setback cannot be maintained between the infiltration system and an adjacent footing, and the infiltration system penetrates below the foundation influence line, the proposed stormwater infiltration system must be designed to resist the surcharge from the adjacent foundation. The foundation surcharge line may be assumed to project down away from the bottom of the foundation at a 1:1 gradient. The stormwater infiltration system must still be sufficiently deep to maintain the 10-foot vertical offset between the bottom of the footing and the zone of saturation.

7.20.7 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 or impede 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.

7.20.8 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).

7.21 Surface Drainage

7.21.1 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.

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

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

7.21.4 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.

7.22 Plan Review

7.22.1 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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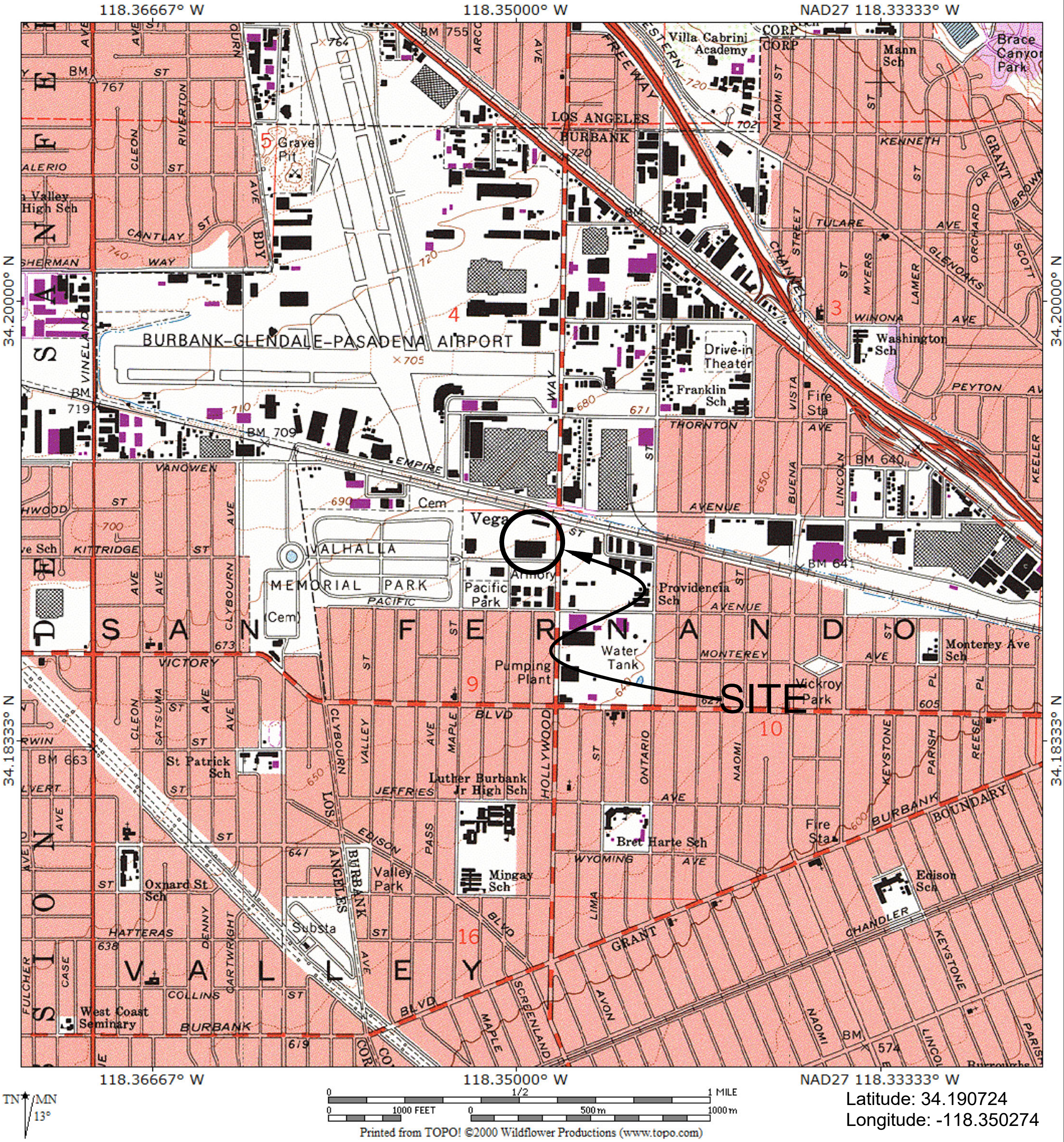

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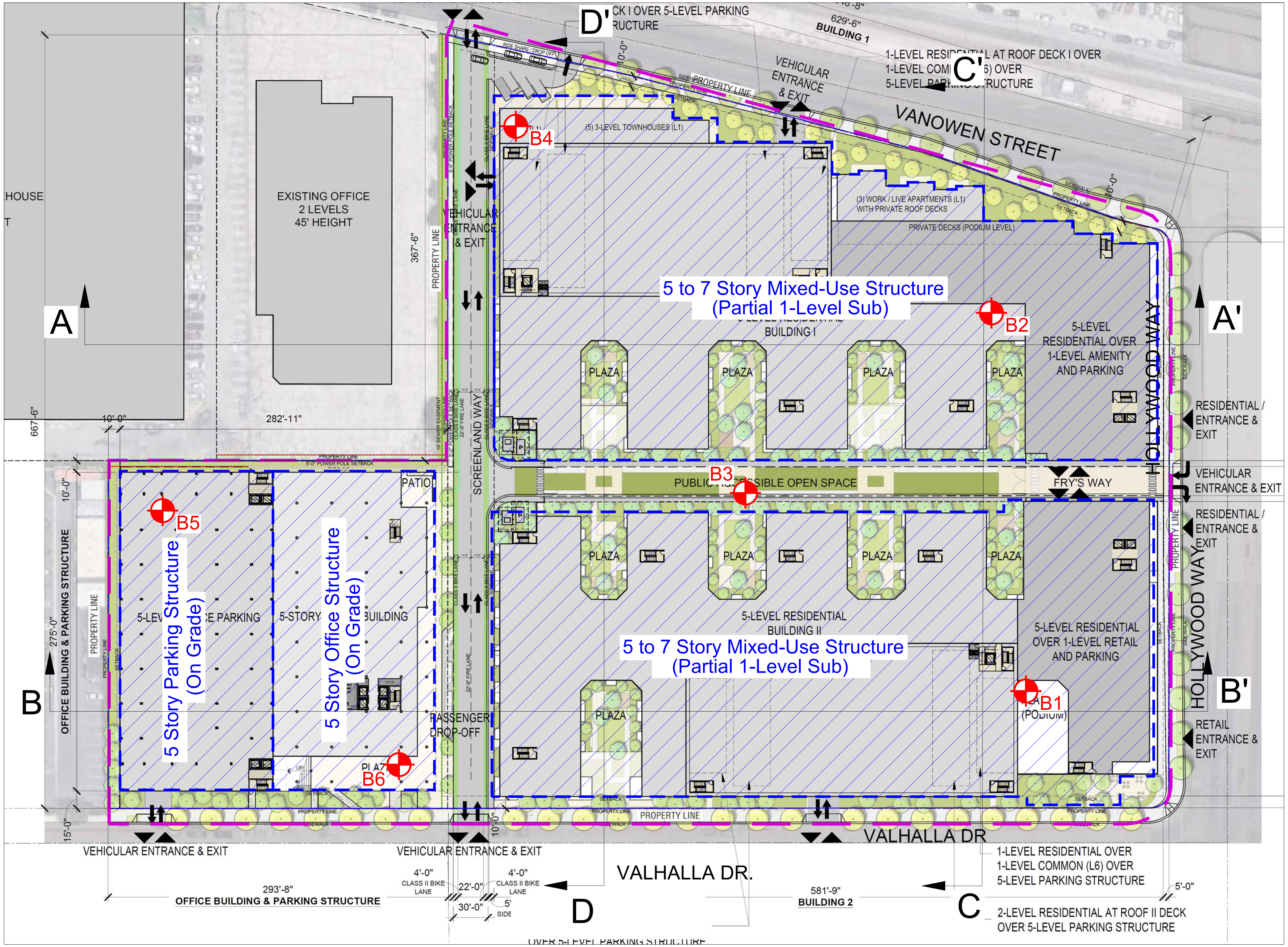
VICINITY MAP

2311 NORTH HOLLYWOOD WAY
BURBANK, CALIFORNIA

DRAFTED BY: RA	CHECKED BY: SFK	MAY 2021	PROJECT NO. W1233-06-01	FIG. 1



LEGEND

Approximate Property Line

Approximate Location of Boring

Proposed Structure

0 100 200

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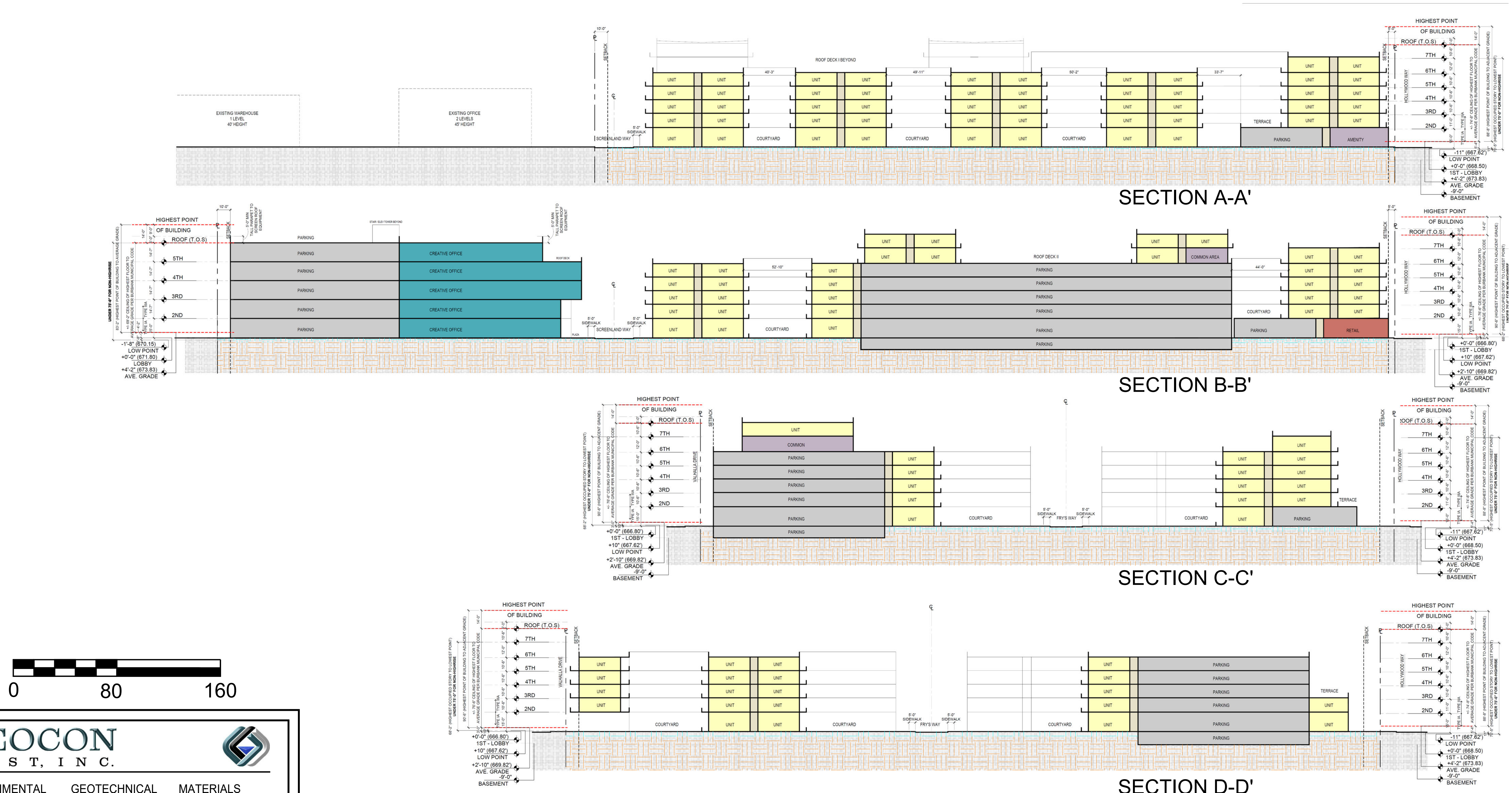
DRAFTED BY: JMH	CHECKED BY: NDB

SITE PLAN

2311 N HOLLYWOOD WAY
BURBANK, CALIFORNIA

MAY 2021	PROJECT NO. W1233-06-01	FIG. 2A

PLANS BY: URBAN ARCHITECTURE LAB (03/15/21)



NEWLY PLACED ENGINEERED FILL (PER REPORT RECOMMENDATIONS)

COMPETENT UNDISTURBED ALLUVIUM

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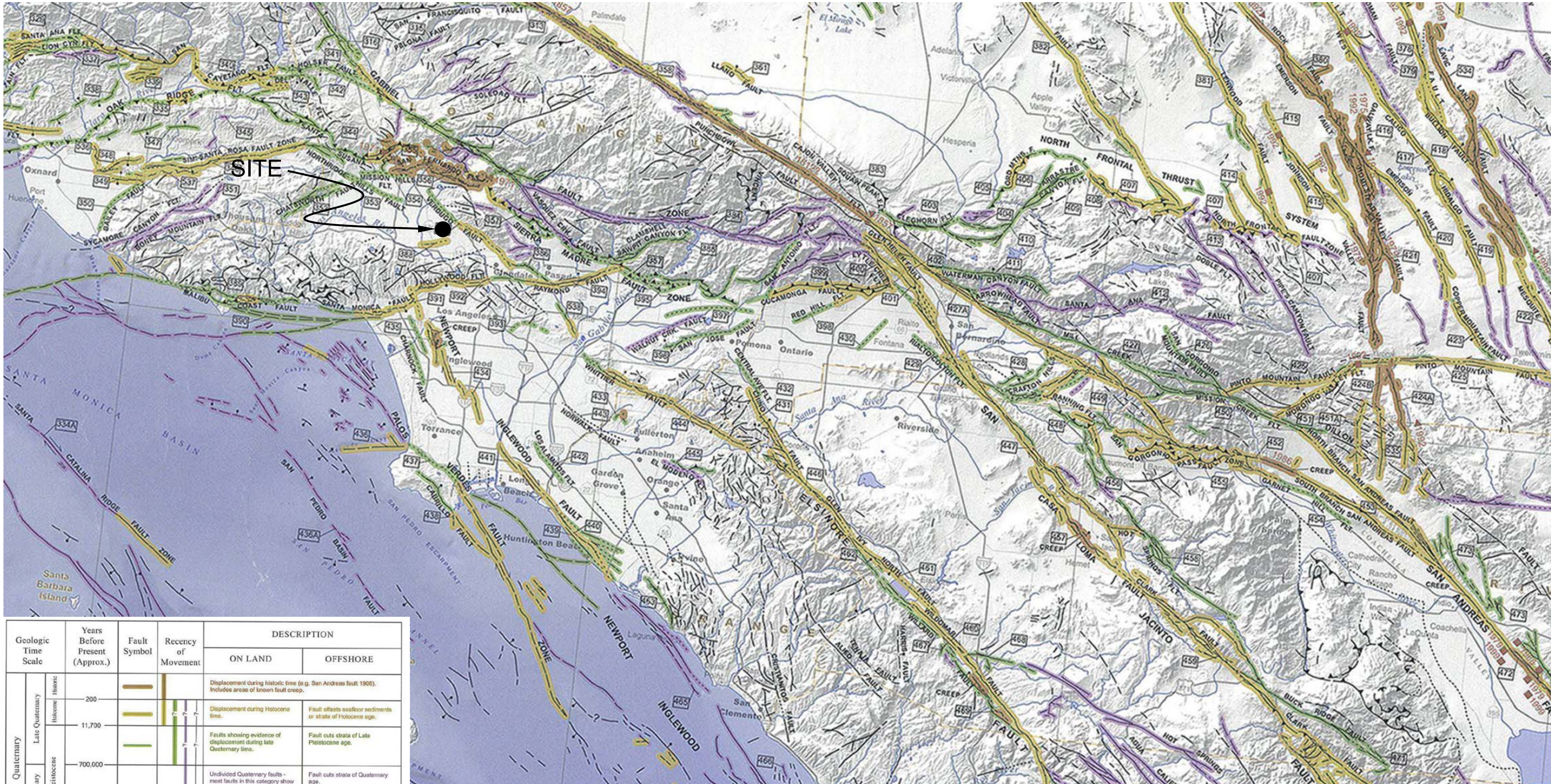
SITE PLAN SECTIONS

2311 N HOLLYWOOD WAY
BURBANK, CALIFORNIA

MAY 2021	PROJECT NO. W1233-06-01	FIG. 2B

PLANS BY: URBAN ARCHITECTURE LAB (03/15/21)

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

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

Latitude: 34.190724
Longitude: -118.350274

0 12 24 Miles

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REGIONAL FAULT MAP

2311 NORTH HOLLYWOOD WAY
BURBANK, CALIFORNIA

MAY 2021 | PROJECT NO. W1233-06-01 | FIG. 3

Latitude: 34.190724
Longitude: -118.350274

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

2311 NORTH HOLLYWOOD WAY
BURBANK, CALIFORNIA

MAY 2021 | PROJECT NO. W1233-06-01 | FIG.4

BORING PERCOLATION TEST FIELD LOG

Date:	Wednesday, October 14, 2020	**Boring/Test Number:**	Boring 1 / Test 1
Project Number:	W1233-06-01	**Diameter of Boring:**	8 inches
Project Location:	2311 N Hollywod Way	**Diameter of Casing:**	2 inches
Earth Description:	SP	**Depth of Boring:**	7 feet
Tested By:	JH	**Depth to Invert of BMP:**	5 feet
Liquid Description:	Clear Clean Tap Water	**Depth to Water Table:**	100 feet
Measurement Method:	Sounder	**Depth to Initial Water Depth (d_1):**	60 inches
Start Time for Pre-Soak:	9:30 AM	**Water Remaining in Boring (Y/N):**	Yes
Start Time for Standard:	10:30 AM	**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	10:30 AM	10:40 AM	10	24.0	
2	11:00 AM	11:10 AM	10	21.6	
3	11:30 AM	11:40 AM	10	11.6	
4	12:00 PM	12:10 PM	10	10.3	
5	12:30 PM	12:40 PM	10	9.0	
6	1:00 PM	1:10 PM	10	8.4	Stabilized Readings
7	1:30 PM	1:40 PM	10	8.3	Achieved with Readings
8	2:00 PM	2:10 PM	10	8.4	6, 7, and 8

MEASURED PERCOLATION RATE & DESIGN INFILTRATION RATE CALCULATIONS*

* Calculations Below Based on Stabilized Readings Only

Boring Radius, r: 4 inches

Test Section Height, h: 24.0 inches

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

A = **653** in^2

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

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

Reading 6	V = **422** in^3	Percolation Rate =	**3.88**	inches/hour
Reading 7	V = **416** in^3	Percolation Rate =	**3.82**	inches/hour
Reading 8	V = **422** in^3	Percolation Rate =	**3.88**	inches/hour

Measured Percolation Rate = **3.86** inches/hour

Reduction Factors

Boring Percolation Test, RF_t = 2

Site Variability, RF_v = 1

Long Term Siltation, RF_s = 1

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

Total Reduction Factor = 2

Design Infiltration Rate

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

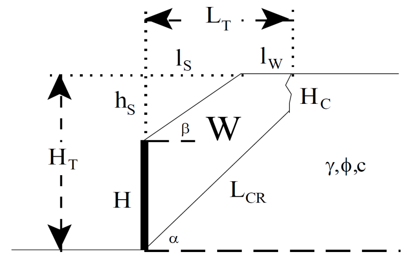
Design Infiltration Rate = **1.93** inches/hour

FIGURE 5

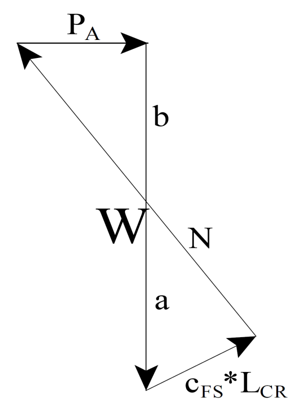
Retaining Wall Design with Transitioned Backfill
(Vector Analysis)

Input:

Retaining Wall Height	(H)	10.00	feet
Slope Angle of Backfill	(b)	0.0	degrees
Height of Slope above Wall	(h_s)	0.0	feet
Horizontal Length of Slope	(l_s)	0.0	feet
Total Height (Wall + Slope)	(H_T)	10.0	feet
Unit Weight of Retained Soils	(g)	125.0	pcf
Friction Angle of Retained Soils	(f)	36.2	degrees
Cohesion of Retained Soils	(c)	133.0	psf
Factor of Safety	(FS)	1.50	
Factored Parameters:	(f_{FS})	26.0	degrees
	(c_{FS})	88.7	psf



Failure Angle (a) degrees	Height of Tension Crack (H_C) feet	Area of Wedge (A) $feet^2$	Weight of Wedge (W) lbs/lineal foot	Length of Failure Plane (L_{CR}) feet	a lbs/lineal foot	b lbs/lineal foot	Active Pressure (P_A) lbs/lineal foot
45	2.8	46	5770.3	10.2	2503.6	3266.7	1124.2
46	2.7	45	5600.6	10.2	2370.5	3230.1	1175.1
47	2.6	43	5431.4	10.1	2248.0	3183.4	1221.4
48	2.5	42	5263.2	10.0	2135.0	3128.2	1263.3
49	2.5	41	5096.8	10.0	2030.7	3066.0	1300.9
50	2.4	39	4932.3	9.9	1934.4	2998.0	1334.2
51	2.4	38	4770.2	9.8	1845.1	2925.1	1363.4
52	2.4	37	4610.4	9.7	1762.3	2848.1	1388.6
53	2.3	36	4453.1	9.6	1685.4	2767.8	1409.7
54	2.3	34	4298.4	9.5	1613.7	2684.7	1426.9
55	2.3	33	4146.2	9.4	1546.9	2599.3	1440.3
56	2.3	32	3996.4	9.3	1484.4	2512.0	1449.8
57	2.3	31	3849.1	9.2	1425.8	2423.2	1455.5
58	2.3	30	3704.1	9.1	1370.9	2333.2	1457.4
59	2.3	28	3561.3	9.0	1319.2	2242.1	1455.5
60	2.3	27	3420.8	8.9	1270.5	2150.2	1449.9
61	2.3	26	3282.3	8.8	1224.5	2057.8	1440.4
62	2.3	25	3145.7	8.7	1180.9	1964.8	1427.1
63	2.3	24	3011.1	8.6	1139.5	1871.6	1409.9
64	2.4	23	2878.2	8.5	1100.1	1778.1	1388.8
65	2.4	22	2746.9	8.4	1062.4	1684.5	1363.7
66	2.4	21	2617.2	8.3	1026.3	1590.9	1334.5
67	2.5	20	2488.8	8.2	991.5	1497.3	1301.2
68	2.5	19	2361.8	8.0	957.9	1403.9	1263.7
69	2.6	18	2235.9	7.9	925.2	1310.7	1221.8
70	2.7	17	2111.0	7.8	893.3	1217.7	1175.5



Design Equations (Vector Analysis):

$a = c_{FS}*L_{CR}*\sin(90+f_{FS})/\sin(a-f_{FS})$

$b = W-a$

$P_A = b*\tan(a-f_{FS})$

$EFP = 2*P_A/H^2$

Maximum Active Pressure Resultant

$P_{A,\ max}$ 1457.4 lbs/lineal foot

Equivalent Fluid Pressure (per lineal foot of wall) At-Rest= $\gamma*(1-\sin(\phi))$

$EFP = 2*P_A/H^2$

EFP 29.1 pcf 51.2 pcf

Design Wall for an Equivalent Fluid Pressure: **30** pcf **51** pcf

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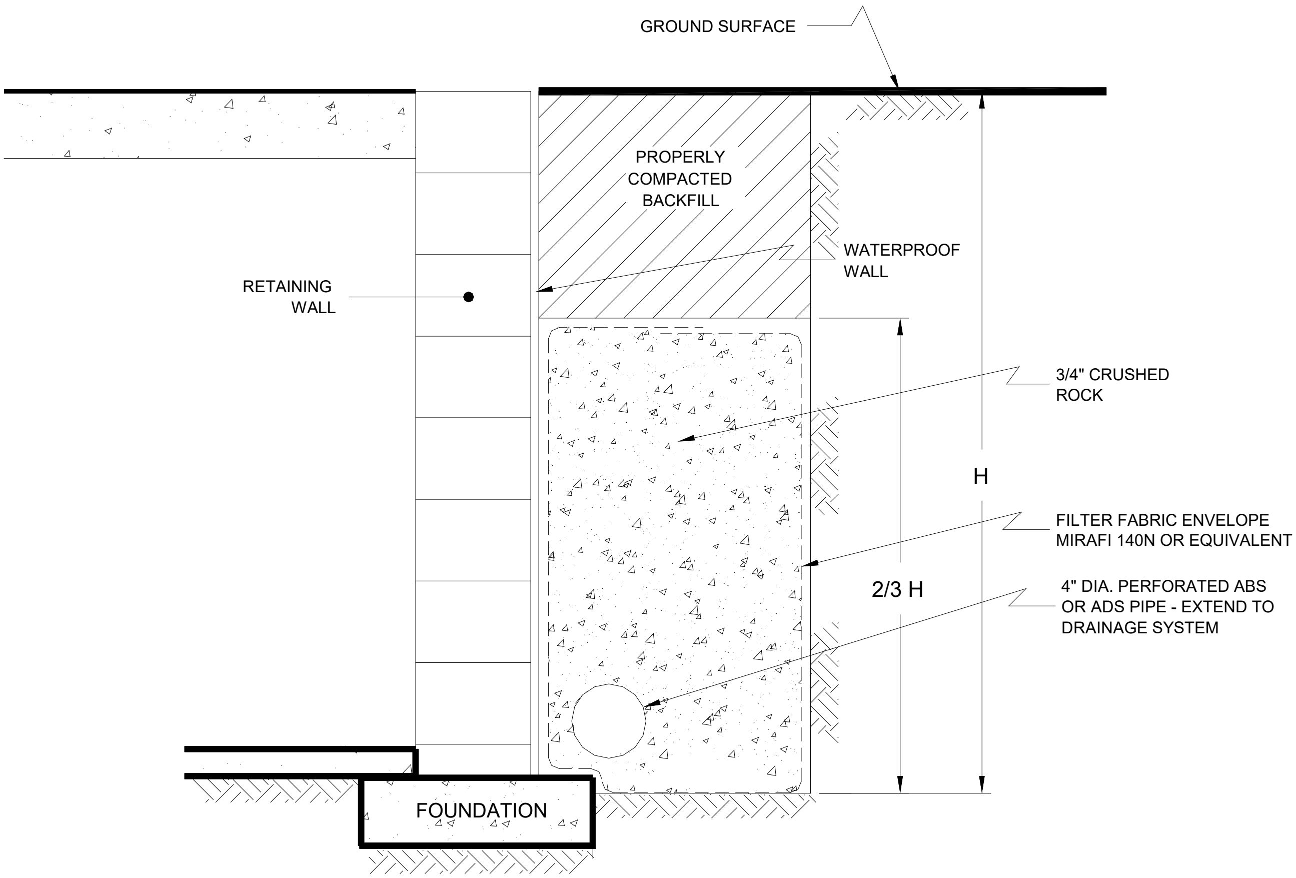
DRAFTED BY: JMH CHECKED BY: NDB

RETAINING WALL CALCULATION

2311 N HOLLYWOOD WAY
BURBANK, CALIFORNIA

MAY 2021 | PROJECT NO. W1233-06-01 | FIG. 6



NO SCALE

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RETAINING WALL DRAIN DETAIL

2311 N HOLLYWOOD WAY
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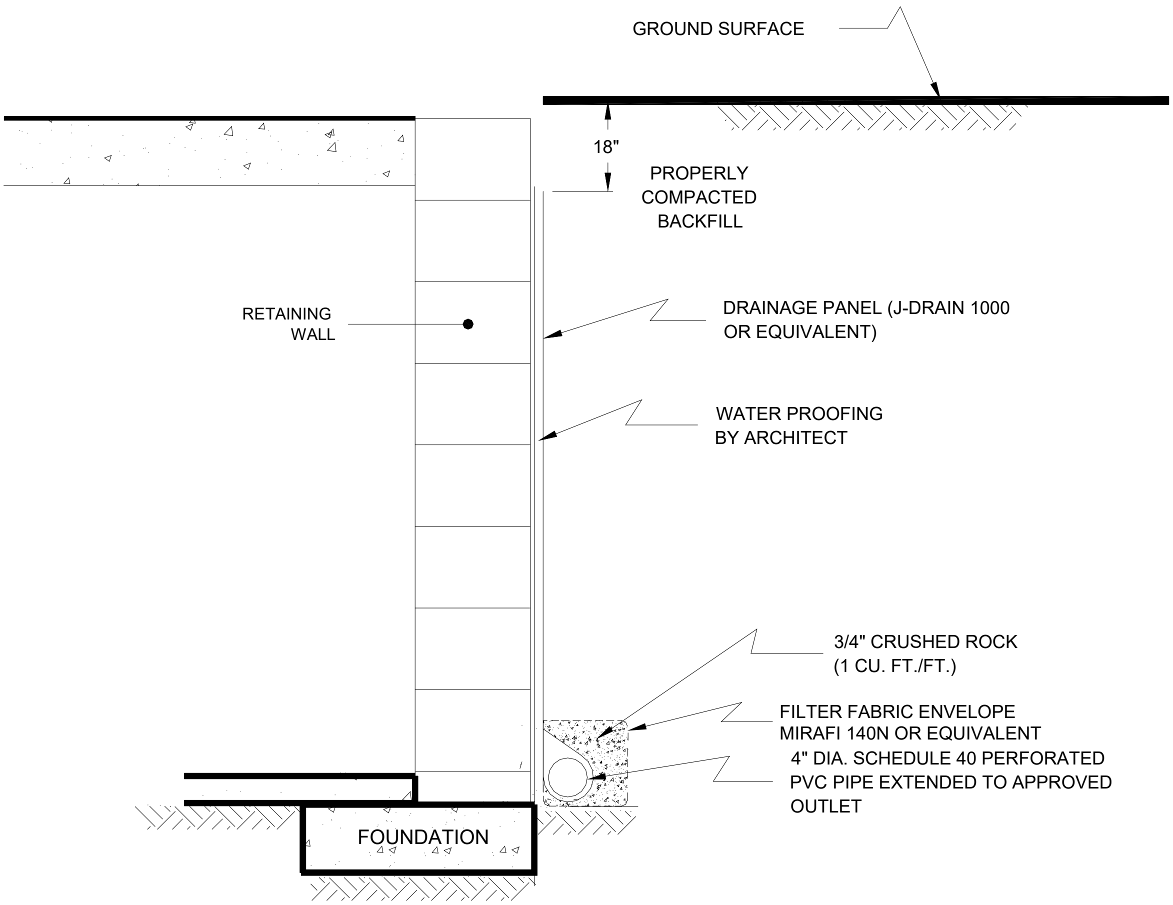
MAY 2021 | PROJECT NO. W1233-06-01 | FIG. 7



NO SCALE

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DRAFTED BY: JMH	CHECKED BY: NDB

RETAINING WALL DRAIN DETAIL

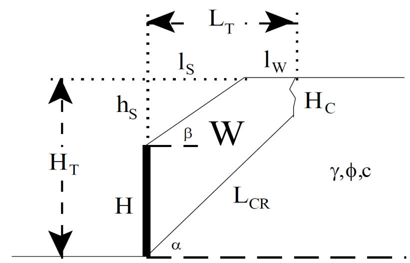
2311 N HOLLYWOOD WAY
BURBANK, CALIFORNIA

MAY 2021	PROJECT NO. W1233-06-01	FIG. 8

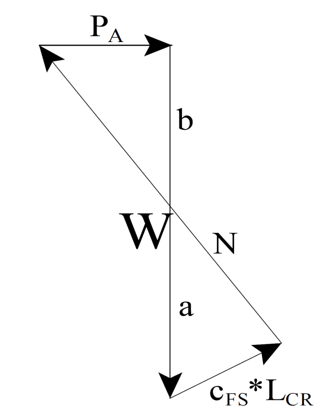
Shoring Design with Transitioned Backfill
(Vector Analysis)

Input:

Shoring Height	(H)	12.00	feet
Slope Angle of Backfill	(b)	0.0	degrees
Height of Slope above Shoring	(h_s)	0.0	feet
Horizontal Length of Slope	(l_s)	0.0	feet
Total Height (Shoring + Slope)	(H_T)	12.0	feet
Unit Weight of Retained Soils	(g)	125.0	pcf
Friction Angle of Retained Soils	(f)	36.2	degrees
Cohesion of Retained Soils	(c)	133.0	psf
Factor of Safety	(FS)	1.25	
Factored Parameters:	(f_{FS})	30.3	degrees
	(c_{FS})	106.4	psf



Failure Angle (a) degrees	Height of Tension Crack (H_C) feet	Area of Wedge (A) $feet^2$	Weight of Wedge (W) lbs/lineal foot	Length of Failure Plane (L_{CR}) feet	a lbs/lineal foot	b lbs/lineal foot	Active Pressure (P_A) lbs/lineal foot
45	4.1	64	7945.7	11.2	4052.2	3893.5	1017.8
46	3.9	62	7763.9	11.2	3823.2	3940.6	1104.0
47	3.8	61	7569.1	11.3	3610.9	3958.3	1183.8
48	3.6	59	7366.0	11.3	3414.5	3951.4	1257.3
49	3.5	57	7157.6	11.3	3233.3	3924.3	1324.5
50	3.4	56	6946.3	11.2	3065.9	3880.4	1385.6
51	3.3	54	6733.7	11.2	2911.4	3822.3	1440.5
52	3.2	52	6520.9	11.1	2768.5	3752.4	1489.5
53	3.2	50	6308.9	11.1	2636.3	3672.6	1532.6
54	3.1	49	6098.2	11.0	2513.7	3584.5	1569.8
55	3.1	47	5889.3	10.9	2399.8	3489.5	1601.3
56	3.0	45	5682.4	10.8	2293.8	3388.5	1627.2
57	3.0	44	5477.7	10.7	2195.1	3282.7	1647.5
58	3.0	42	5275.4	10.6	2102.8	3172.6	1662.2
59	3.0	41	5075.5	10.5	2016.4	3059.1	1671.4
60	3.0	39	4877.9	10.4	1935.3	2942.6	1675.1
61	3.0	37	4682.8	10.3	1859.1	2823.7	1673.3
62	3.0	36	4489.9	10.2	1787.2	2702.7	1666.0
63	3.0	34	4299.3	10.1	1719.3	2580.1	1653.2
64	3.0	33	4110.8	10.0	1654.8	2456.0	1634.9
65	3.1	31	3924.4	9.9	1593.5	2330.9	1611.0
66	3.1	30	3739.9	9.7	1535.0	2204.9	1581.5
67	3.1	28	3557.1	9.6	1479.0	2078.2	1546.2
68	3.2	27	3376.0	9.5	1425.0	1951.0	1505.2
69	3.3	26	3196.4	9.3	1372.9	1823.5	1458.3
70	3.4	24	3018.0	9.2	1322.2	1695.9	1405.5



Design Equations (Vector Analysis):

$a = c_{FS}*L_{CR}*\sin(90+f_{FS})/\sin(a-f_{FS})$

$b = W-a$

$P_A = b*\tan(a-f_{FS})$

$EFP = 2*P_A/H^2$

Maximum Active Pressure Resultant

$P_{A,\ max}$ 1675.1 lbs/lineal foot

Equivalent Fluid Pressure (per lineal foot of shoring)

$EFP = 2*P_A/H^2$

EFP 23.3 pcf

Design Shoring for an Equivalent Fluid Pressure: 25 pcf

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SHORING CALCULATION

2311 N HOLLYWOOD WAY
BURBANK, CALIFORNIA

DRAFTED BY: JMH	CHECKED BY: NDB	MAY 2021	PROJECT NO. W1233-06-01	FIG. 9

APPENDIX A

APPENDIX A

FIELD INVESTIGATION

The site was explored on October 14, 2020 by excavating six 8-inch-diameter borings using a truck-mounted hollow-stem auger drilling machine. The hollow-stem auger borings were excavated to depths of approximately 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 2⅜-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 A6. The logs depict the soil and geologic conditions encountered and the depth at which samples were obtained. The logs also include our interpretation of the conditions between sampling intervals. Therefore, the logs contain both observed and interpreted data. We determined the lines designating the interface between soil materials on the logs using visual observations, penetration rates, excavation characteristics and other factors. The transition between materials may be abrupt or gradual. Where applicable, the logs were revised based on subsequent laboratory testing. The approximate locations of the borings are shown on Figure 2.

PROJECT NO. W1233-06-01

BORING 1

ELEV. (MSL.) -- DATE COMPLETED 10/14/2020

EQUIPMENT HOLLOW STEM AUGER BY: JMH

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: 6" BASE: NONE**			
					ARTIFICIAL FILL Silty Sand, poorly graded, medium dense, slightly moist, grayish brown, fine-grained.			
2								
	B1@3'				**ALLUVIUM** Sand, poorly graded, medium dense, light brown, slightly moist, fine-grained, some medium-grained.	29	99.8	8.5
4								
6	B1@6'				- brown, fine- to medium-grained, trace fine gravel	28	114.5	2.6
8								
	B1@9'				- dense, moist, trace coarse-grained, coarse gravel	62	117.2	5.3
10								
12	B1@12'				- medium dense, fine- to medium-grained, trace fine gravel	50	117.1	4.0
14				SP				
	B1@15'				- no gravel	34	105.6	9.9
16								
18								
20	B1@20'				- trace coarse-grained, fine gravel	39	119.9	4.2
22								
24								
	B1@24.5'				- very dense, trace cobbles	50 (6")	117.8	2.5
26								
28								

W1233-06-01 BORING LOGS.GPJ

Figure A1,
Log of Boring 1, 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

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	BORING 1 ELEV. (MSL.) -- DATE COMPLETED 10/14/2020 EQUIPMENT HOLLOW STEM AUGER BY: JMH	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
					MATERIAL DESCRIPTION			
30	B1@30'			SP	- medium dense, no cobbles	46	109.2	5.3
					Total depth of boring: 30.5 feet Fill to 2 feet. No groundwater encountered. Percolation testing 5-7 feet. Backfilled with soil cuttings and tamped. *Penetration resistance for 140-pound hammer falling 30 inches by auto-hammer.			

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

W1233-06-01 BORING LOGS.GPJ

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

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

PROJECT NO. W1233-06-01

BORING 2

ELEV. (MSL.) -- DATE COMPLETED 10/14/2020

EQUIPMENT HOLLOW STEM AUGER BY: JMH

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: 6" BASE: NONE**			
					ARTIFICIAL FILL Silty Sand, poorly graded, medium dense, slightly moist, brown, fine-grained.			
2	B2@2'			SM	**ALLUVIUM** Silty Sand, poorly graded, dense, moist, brown, fine-grained, trace fine gravel.	64	114.4	10.7
4					Sand, poorly graded, medium dense, moist, light brown, fine-grained, some fine gravel.			
6	B2@6'					31	114.3	2.2
8	B2@8'				- dense, fine- to medium-grained, cobble	76	114.3	1.9
10	B2@10'				- very dense	50 (5")	108.4	3.6
12								
14				SP				
16	B2@15'				- medium dense, trace fine gravel	42	113.7	2.8
18								
20	B2@20'				- dense, trace coarse-grained	60	132.3	2.6
22								
24								
26	B2@25'					63	117.4	3.0
28								

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

W1233-06-01 BORING LOGS.GPJ

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

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

GEOCON

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	**BORING 2** ELEV. (MSL.) **--** DATE COMPLETED **10/14/2020** EQUIPMENT **HOLLOW STEM AUGER** BY: **JMH**	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
					MATERIAL DESCRIPTION			
30	B2@30'			SP	- very dense, cobble	89	121.6	2.8

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

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

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

W1233-06-01 BORING LOGS.GPJ

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

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

BORING 3

ELEV. (MSL.) -- DATE COMPLETED 10/14/2020

EQUIPMENT HOLLOW STEM AUGER BY: JMH

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: 6" BASE: NONE**			
					ARTIFICIAL FILL Silty Sand, poorly graded, medium dense, slightly moist, brown, fine-grained.			
2					**ALLUVIUM** Sand, poorly graded, medium dense, slightly moist, light gray, fine- to medium-grained, some coarse-grained, fine gravel.			
	B3@3'					35	128.0	1.6
4								
6	B3@6'				- light brown, trace fine gravel	45	107.5	4.8
8								
	B3@9'				- moist, no gravel	38	110.3	2.5
10								
12	B3@12'			SP	- dense, trace coarse-grained, fine gravel, and cobble	73	131.1	1.2
14								
	B3@15'				- medium dense, fine-grained, trace medium-grained and fine gravel	44	122.8	2.5
16								
18								
20	B3@20'				- brown, dense, fine- to medium-grained	58	100.9	6.9
22								
24								
	B3@25'				- trace cobbles	67	123.8	2.9
26								
28								
				SM	Silty Sand, poorly graded, medium dense, moist, brown, fine-grained.			

W1233-06-01 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.

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	BORING 3 ELEV. (MSL.) -- DATE COMPLETED 10/14/2020 EQUIPMENT HOLLOW STEM AUGER BY: JMH	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
					MATERIAL DESCRIPTION			
30	B3@30'			SM		25	102.3	13.5
					Total depth of boring: 30.5 feet Fill to 1 foot. No groundwater encountered. Backfilled with soil cuttings and tamped. *Penetration resistance for 140-pound hammer falling 30 inches by auto-hammer.			

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

W1233-06-01 BORING LOGS.GPJ

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

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

GEOCON

BORING 4

ELEV. (MSL.) **--** DATE COMPLETED **10/14/2020**

EQUIPMENT **HOLLOW STEM AUGER** BY: **JMH**

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: NONE**			
					ARTIFICIAL FILL Silty Sand, poorly graded, medium dense, slightly moist, brown, fine-grained.			
2				SM	**ALLUVIUM** Silty Sand, poorly graded, medium dense, moist, brown, fine-grained.			
	B4@3'					18		
4					Sand, poorly graded, medium dense, slightly moist, light brown, fine-graind, some medium-grained.			
6	B4@6'					15	109.5	2.7
8								
	B4@9'				- moist, fine- to medium-grained, trace coarse-grained	36	115.6	2.9
10								
12	B4@12'				- trace fine gravel	44	114.4	3.0
14				SP				
	B4@15'					43	130.7	3.1
16								
18								
20	B4@20'				- dense, fine gravel	64	122.2	2.7
22								
24								
	B4@25'				- cobble	86	114.9	3.1
26								
28								

W1233-06-01 BORING LOGS.GPJ

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.

BORING 4

ELEV. (MSL.) -- DATE COMPLETED 10/14/2020

EQUIPMENT HOLLOW STEM AUGER BY: JMH

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		58	120.4	1.4
					Total depth of boring: 30.5 feet Fill to 1 foot. No groundwater encountered. Backfilled with soil cuttings and tamped. *Penetration resistance for 140-pound hammer falling 30 inches by auto-hammer.			

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

W1233-06-01 BORING LOGS.GPJ

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

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

PROJECT NO. W1233-06-01

BORING 5

ELEV. (MSL.) -- DATE COMPLETED 10/14/2020

EQUIPMENT HOLLOW STEM AUGER BY: JMH

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: NONE**			
					ARTIFICIAL FILL Silty Sand, poorly graded, loose, slightly moist, brown, fine-grained.			
2	B5@2'			SM	**ALLUVIUM** Silty Sand, poorly graded, loose, slightly moist, grayish brown, fine-grained.	12	100.9	7.2
4								
6	B5@6'			SP	Sand, poorly graded, medium dense, dry, light brown, fine-grained.	22	103.2	2.0
8	B5@8'					45	136.3	1.2
10	B5@10'				- dense, light gray	59	121.9	2.4
12								
14								
	B5@15'				- brown, medium dense, trace coarse-grained	47	117.4	4.4
16								
18								
20	B5@20'				- dense, cobble	77	136.3	1.2
22								
24								
	B5@25'				- cobble	72	122.7	2.6
26								
28				SM	Silty Sand, poorly graded, medium dense, moist, brown, fine-grained.			

W1233-06-01 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

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	BORING 5 ELEV. (MSL.) -- DATE COMPLETED 10/14/2020 EQUIPMENT HOLLOW STEM AUGER BY: JMH	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
					MATERIAL DESCRIPTION			
30	B5@30'			SM		40	108.8	6.4
					Total depth of boring: 30.5 feet Fill to 1 foot. No groundwater encountered. Backfilled with soil cuttings and tamped. *Penetration resistance for 140-pound hammer falling 30 inches by auto-hammer.			

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

W1233-06-01 BORING LOGS.GPJ

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

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

PROJECT NO. W1233-06-01

BORING 6

ELEV. (MSL.) -- DATE COMPLETED 10/14/2020

EQUIPMENT HOLLOW STEM AUGER BY: JMH

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: 4.5" BASE: NONE**			
					ARTIFICIAL FILL Silty Sand, poorly graded, loose, slightly moist, brown, fine-grained.			
2				SM	**ALLUVIUM** Silty Sand, poorly graded, loose, moist, brown, fine-grained.			
	B6@3'				Sand, poorly graded, loose, slightly moist, light brown, fine-grained, some medium-grained.	8	98.3	13.0
4								
6	B6@6'				- medium dense	26	115.8	2.5
8								
	B6@9'				- light gray, fine- to medium-grained	36	113.6	3.6
10								
12	B6@12'				- brown, trace fine gravel	41	117.4	3.1
14								
	B6@15'			SP	- trace cobble	42	114.2	4.2
16								
18								
20	B6@20'				- trace coarse-grained sand, cobble	38	109.5	2.9
22								
24								
	B6@25'					62	120.8	4.2
26								
28								

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

W1233-06-01 BORING LOGS.GPJ

SAMPLE SYMBOLS

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

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

GEOCON

BORING 6

ELEV. (MSL.) -- DATE COMPLETED 10/14/2020

EQUIPMENT HOLLOW STEM AUGER BY: JMH

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
30	B6@30'			SP	- cobble	67	126.1	2.5

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

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

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

W1233-06-01 BORING LOGS.GPJ

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

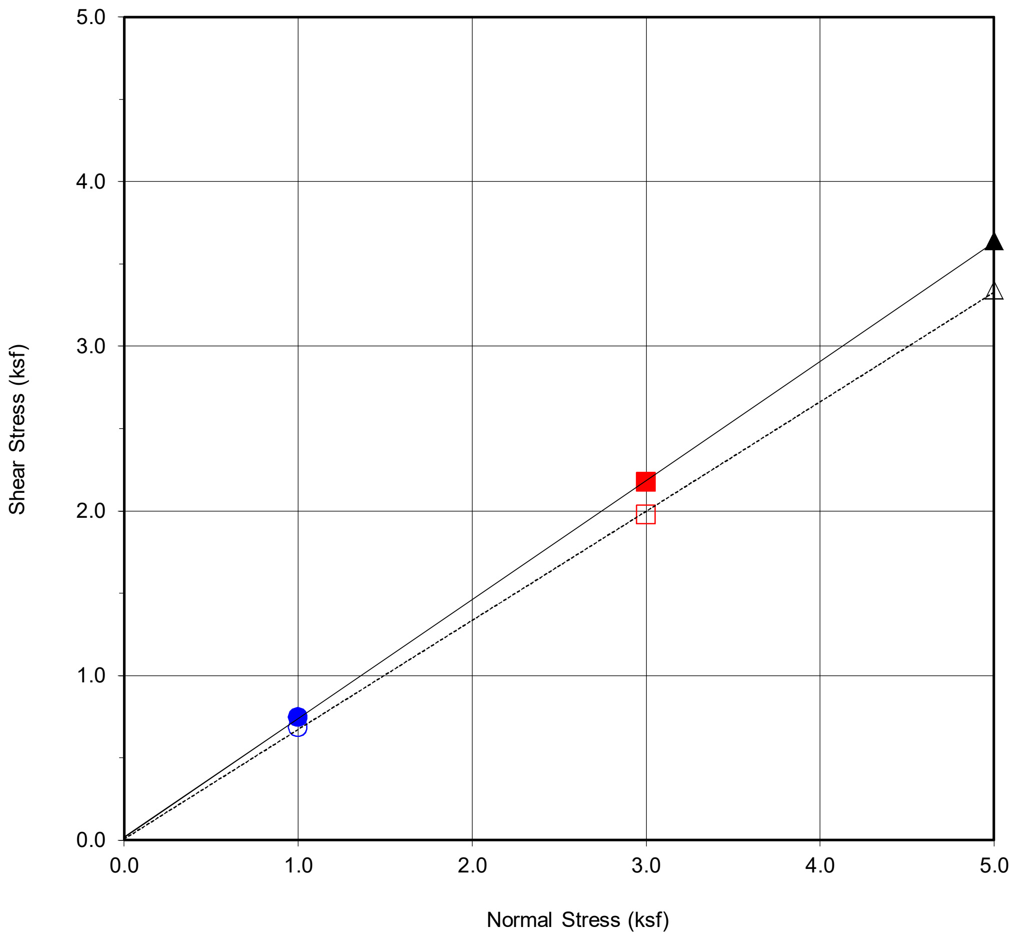
NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

APPENDIX B

APPENDIX B

LABORATORY TESTING

Laboratory tests were performed in accordance with “American Society for Testing and Materials (ASTM)”, or other suggested procedures Selected samples were tested for direct shear strength, expansion and consolidation characteristics, maximum dry density, corrosivity, in-place dry density and moisture content. The results of the laboratory tests are summarized in Figures B1 through B33. The in-place dry density and moisture content of the samples tested are presented on the boring logs, Appendix A.



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

Soil Identification:		
Light Brown Poorly Graded Sand (SP)		
Strength Parameters		
	C (psf)	ϕ (°)
Peak	16	35.9
Ultimate	6	33.6

Normal Stress (kip/ft2)	1	3	5
Peak Shear Stress (kip/ft²)	● 0.75	■ 2.17	▲ 3.64
Shear Stress @ End of Test (ksf)	○ 0.68	□ 1.97	△ 3.34
Deformation Rate (in./min.)	0.05	0.05	0.05
Initial Sample Height (in.)	1.0	1.0	1.0
Ring Inside Diameter (in.)	2.375	2.375	2.375
Initial Moisture Content (%)	10.5	10.6	10.6
Initial Dry Density (pcf)	104.0	104.0	104.0
Initial Degree of Saturation (%)	45.7	45.9	46.2
Soil Height Before Shearing (in.)	1.2	1.2	1.2
Final Moisture Content (%)	14.3	13.4	14.4

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DIRECT SHEAR TEST RESULTS

Consolidated Drained ASTM D-3080

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Project No.: W1233-06-01

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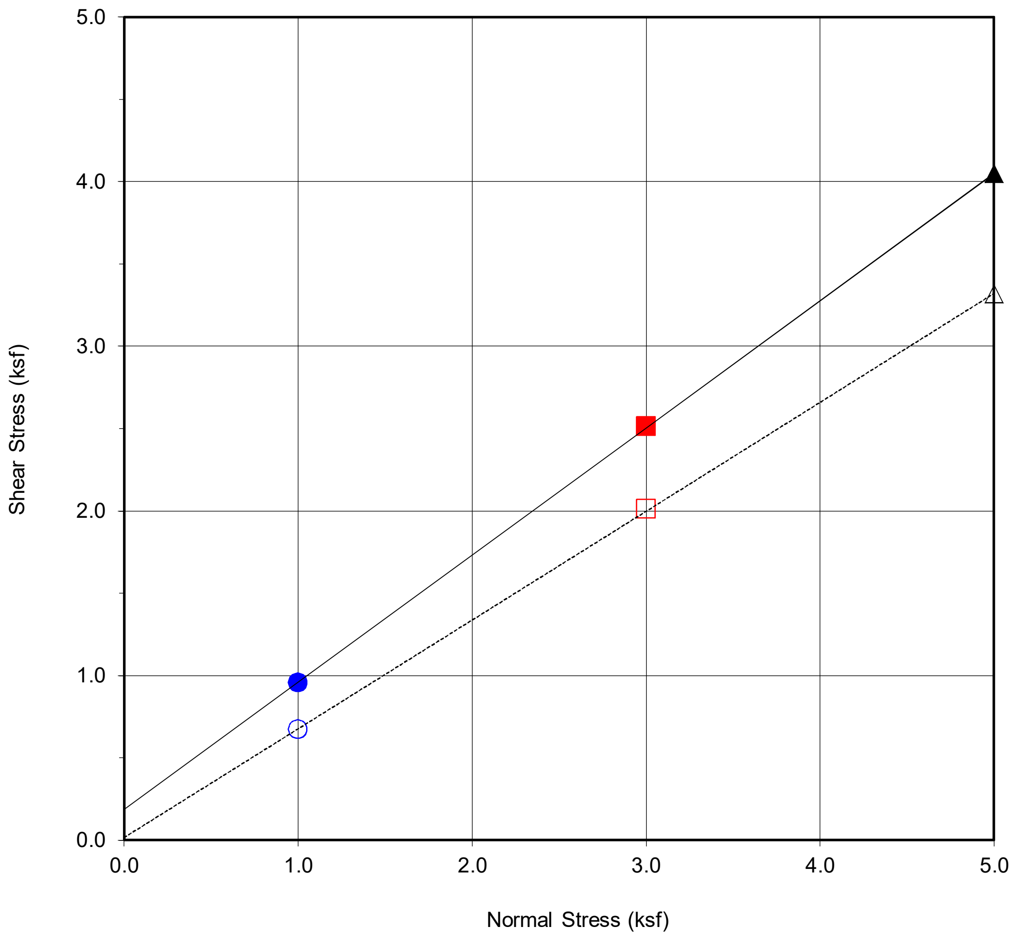
Figure B1



Boring No.	**B1**
Sample No.	**B1@3'**
Depth (ft)	**3**
Sample Type:	Ring

Soil Identification:		
Light Brown Poorly Graded Sand (SP)		
Strength Parameters		
	C (psf)	ϕ (°)
Peak	186	37.7
Ultimate	16	33.5

Normal Stress (kip/ft2)	1	3	5
Peak Shear Stress (kip/ft²)	● 0.96	■ 2.51	▲ 4.05
Shear Stress @ End of Test (ksf)	○ 0.67	□ 2.01	△ 3.32
Deformation Rate (in./min.)	0.05	0.05	0.05
Initial Sample Height (in.)	1.0	1.0	1.0
Ring Inside Diameter (in.)	2.375	2.375	2.375
Initial Moisture Content (%)	5.9	5.6	8.5
Initial Dry Density (pcf)	98.3	102.1	103.0
Initial Degree of Saturation (%)	22.2	23.2	36.0
Soil Height Before Shearing (in.)	1.2	1.2	1.2
Final Moisture Content (%)	20.2	18.5	18.9

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DIRECT SHEAR TEST RESULTS

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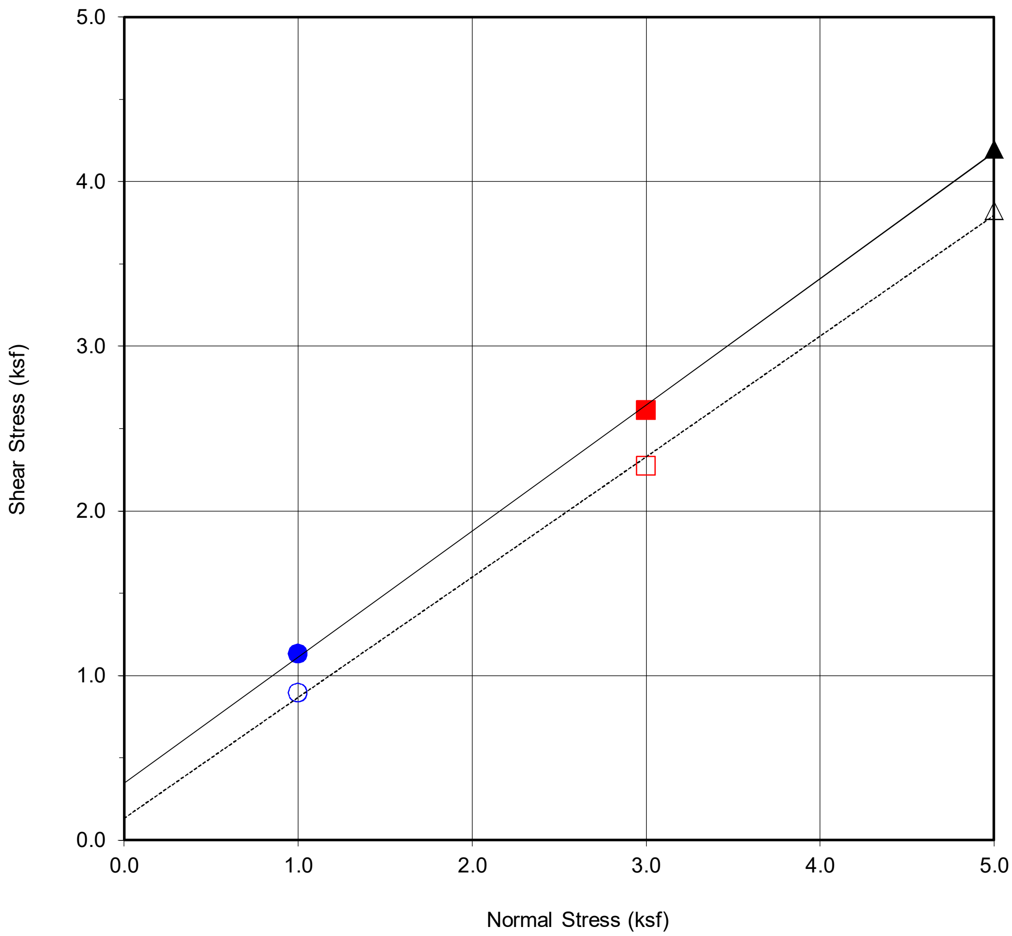
Figure B2



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

Soil Identification:		
Brown Silty Sand with Gravel (SM)		
Strength Parameters		
	C (psf)	ϕ (^{o})
Peak	345	37.5
Ultimate	133	36.2

Normal Stress (kip/ft2)	1	3	5
Peak Shear Stress (kip/ft²)	● 1.13	■ 2.61	▲ 4.20
Shear Stress @ End of Test (ksf)	○ 0.89	□ 2.27	△ 3.82
Deformation Rate (in./min.)	0.05	0.05	0.05
Initial Sample Height (in.)	1.0	1.0	1.0
Ring Inside Diameter (in.)	2.375	2.375	2.375
Initial Moisture Content (%)	11.5	11.1	10.7
Initial Dry Density (pcf)	111.4	114.4	114.8
Initial Degree of Saturation (%)	60.4	63.5	61.6
Soil Height Before Shearing (in.)	1.2	1.2	1.2
Final Moisture Content (%)	16.2	15.2	15.5

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DIRECT SHEAR TEST RESULTS

Consolidated Drained ASTM D-3080

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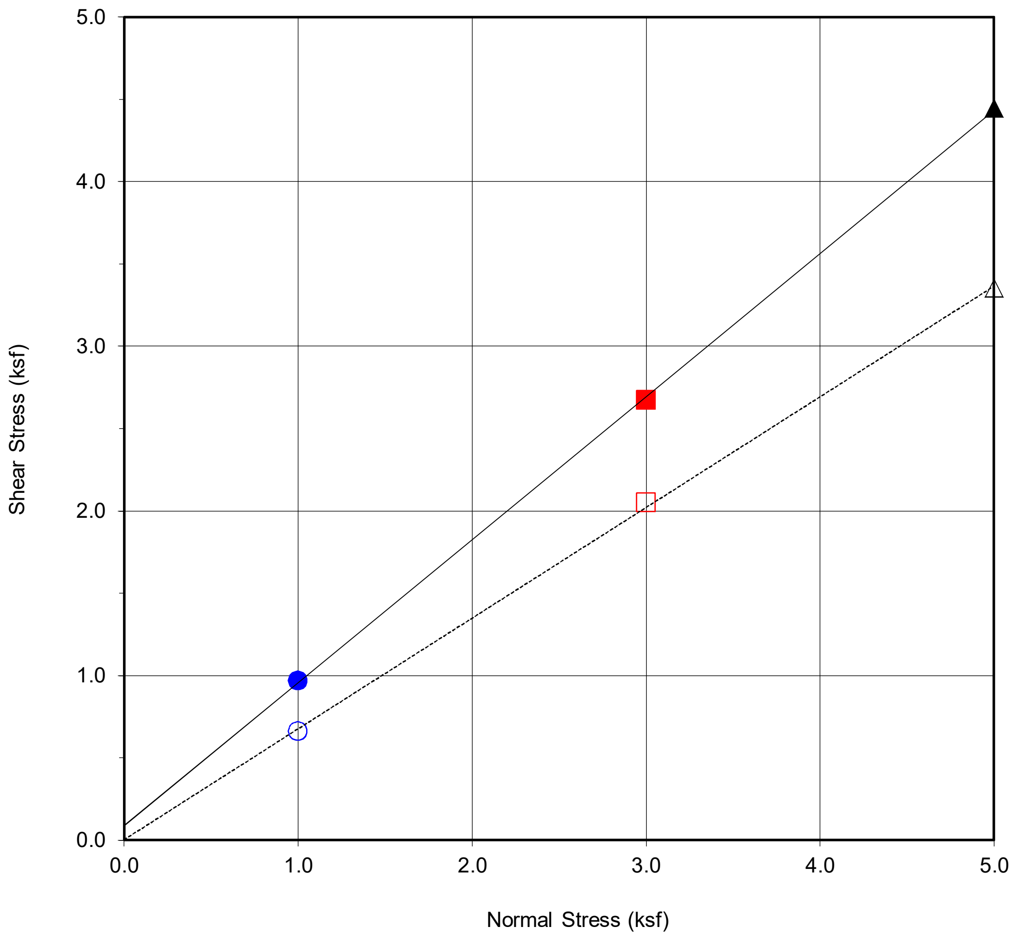
Figure B3



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

Soil Identification:		
Light Brown Poorly Graded Sand (SP)		
Strength Parameters		
	C (psf)	ϕ (°)
Peak	88	41.0
Ultimate	2	33.9

Normal Stress (kip/ft2)	1	3	5
Peak Shear Stress (kip/ft²)	● 0.97	■ 2.67	▲ 4.45
Shear Stress @ End of Test (ksf)	○ 0.66	□ 2.05	△ 3.35
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 (%)	12.2	6.3	6.9
Initial Dry Density (pcf)	93.0	98.9	102.3
Initial Degree of Saturation (%)	40.6	24.1	29.0
Soil Height Before Shearing (in.)	1.2	1.2	1.2
Final Moisture Content (%)	20.4	18.8	16.7

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DIRECT SHEAR TEST RESULTS

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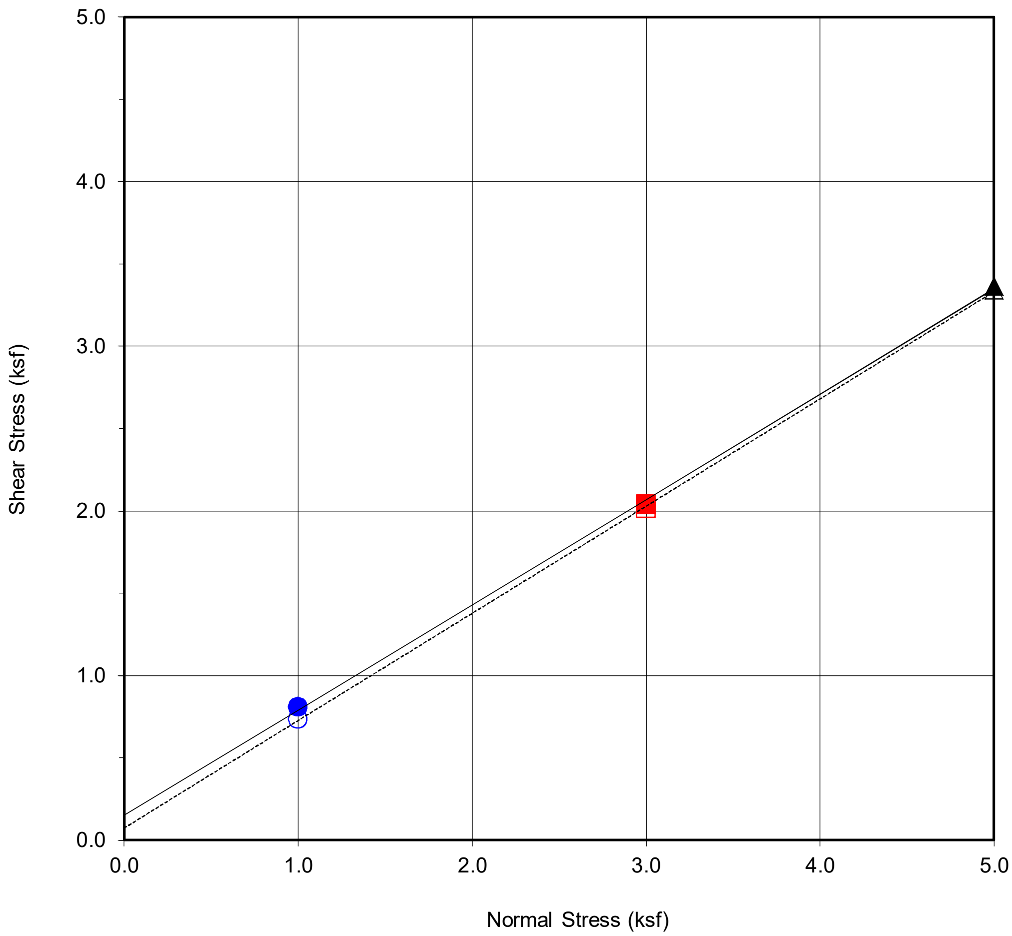
Figure B4



Boring No.	**B2 + B4**
Sample No.	**Mix B2&B4**
Depth (ft)	**0-5'**
Sample Type:	Ring

Soil Identification:		
Light Brown Silty Sand (SM)		
Strength Parameters		
	C (psf)	ϕ (^{o})
Peak	151	32.6
Ultimate	73	33.1

Normal Stress (kip/ft2)	1	3	5
Peak Shear Stress (kip/ft²)	● 0.81	■ 2.04	▲ 3.37
Shear Stress @ End of Test (ksf)	○ 0.73	□ 2.01	△ 3.34
Deformation Rate (in./min.)	0.05	0.05	0.05
Initial Sample Height (in.)	1.0	1.0	1.0
Ring Inside Diameter (in.)	2.375	2.375	2.375
Initial Moisture Content (%)	8.3	8.3	8.3
Initial Dry Density (pcf)	113.0	113.0	113.0
Initial Degree of Saturation (%)	45.6	45.7	45.7
Soil Height Before Shearing (in.)	1.2	1.2	1.2
Final Moisture Content (%)	11.3	12.4	11.9

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DIRECT SHEAR TEST RESULTS

Consolidated Drained ASTM D-3080

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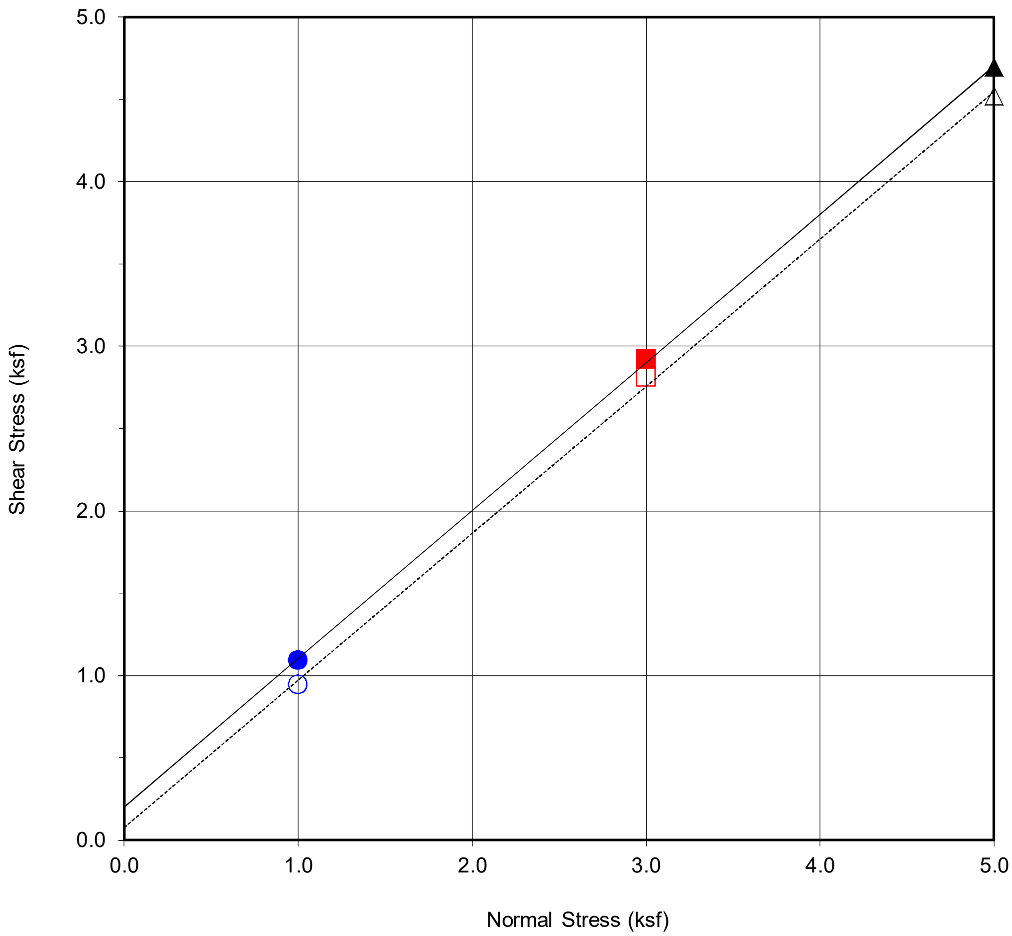
Figure B5



Boring No.	B3
Sample No.	B3@6
Depth (ft)	6
Sample Type:	Ring

Normal Stress (kip/ft2)	1	3	5
Peak Shear Stress (kip/ft²)	● 1.09	■ 2.92	▲ 4.69
Shear Stress @ End of Test (ksf)	○ 0.94	□ 2.81	△ 4.52
Deformation Rate (in./min.)	0.05	0.05	0.05
Initial Sample Height (in.)	1.0	1.0	1.0
Ring Inside Diameter (in.)	2.375	2.375	2.375
Initial Moisture Content (%)	5.5	8.8	4.8
Initial Dry Density (pcf)	105.3	104.0	106.5
Initial Degree of Saturation (%)	24.8	38.3	22.4
Soil Height Before Shearing (in.)	1.2	1.2	1.2
Final Moisture Content (%)	13.8	18.7	13.6

Soil Identification:

Light Grayish Brown Poorly Graded Sand (SP)

Strength Parameters

	C (psf)	ϕ (°)
Peak	202	42.0
Ultimate	76	41.8

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DIRECT SHEAR TEST RESULTS

Consolidated Drained ASTM D-3080

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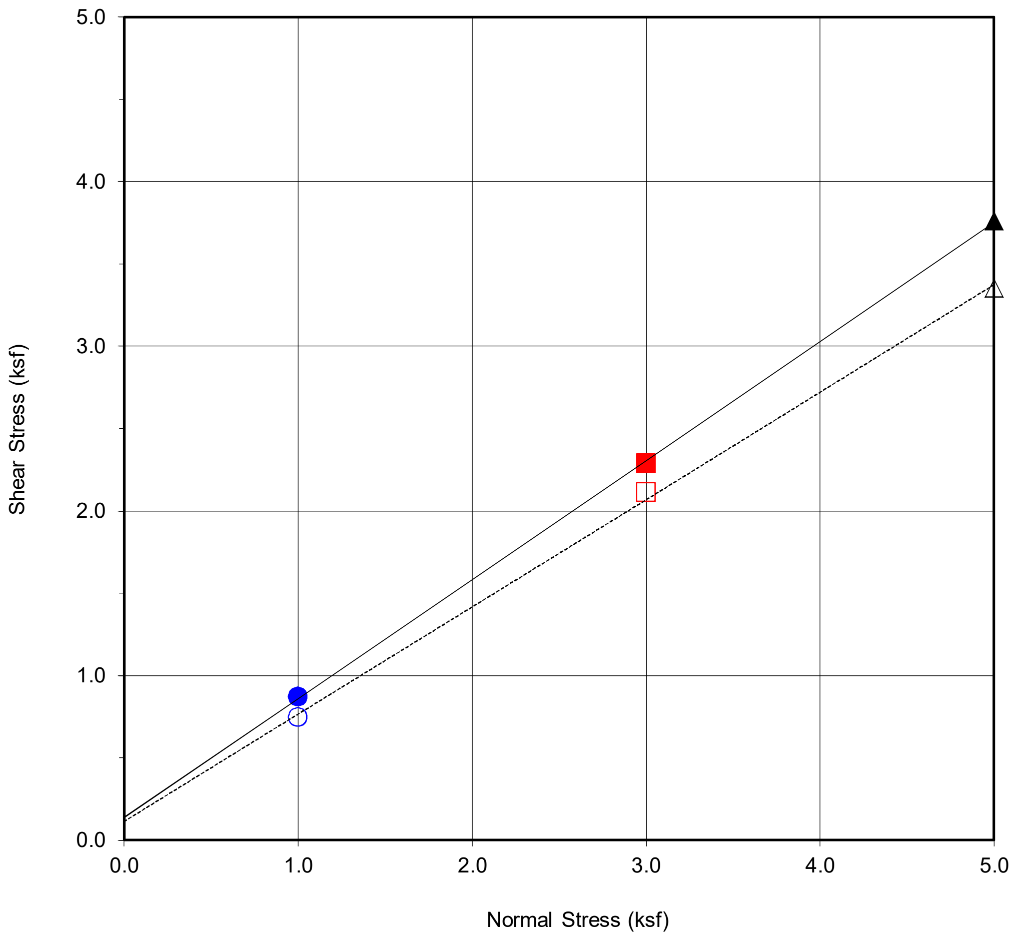
Figure B6



Boring No.	**B3**
Sample No.	**B3@30**
Depth (ft)	**30**
Sample Type:	Ring

Soil Identification:		
Brown Silty Sand (SM)		
Strength Parameters		
	C (psf)	ϕ (°)
Peak	136	35.9
Ultimate	114	33.1

Normal Stress (kip/ft2)	1	3	5
Peak Shear Stress (kip/ft²)	● 0.87	■ 2.29	▲ 3.76
Shear Stress @ End of Test (ksf)	○ 0.75	□ 2.11	△ 3.35
Deformation Rate (in./min.)	0.05	0.05	0.05
Initial Sample Height (in.)	1.0	1.0	1.0
Ring Inside Diameter (in.)	2.375	2.375	2.375
Initial Moisture Content (%)	13.2	15.2	13.5
Initial Dry Density (pcf)	98.4	101.2	104.4
Initial Degree of Saturation (%)	50.0	61.5	59.5
Soil Height Before Shearing (in.)	1.2	1.2	1.2
Final Moisture Content (%)	23.4	22.1	20.6

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DIRECT SHEAR TEST RESULTS

Consolidated Drained ASTM D-3080

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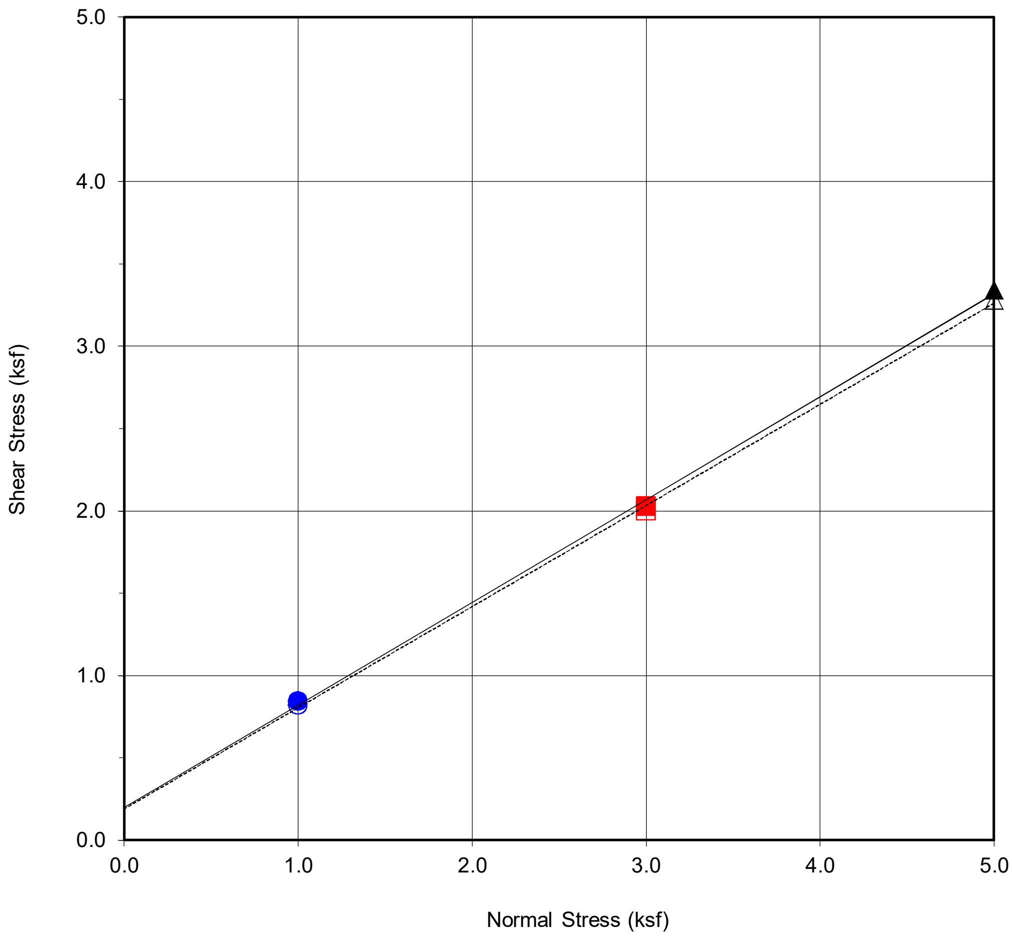
Figure B7



Boring No.	B5
Sample No.	B5@0-5'
Depth (ft)	0-5'
Sample Type:	Ring

Soil Identification:		
Grayish Brown Silty Sand (SM)		
Strength Parameters		
	C (psf)	ϕ (°)
Peak	197	32.0
Ultimate	189	31.6

Normal Stress (kip/ft2)	1	3	5
Peak Shear Stress (kip/ft²)	● 0.84	■ 2.02	▲ 3.34
Shear Stress @ End of Test (ksf)	○ 0.82	□ 2.00	△ 3.28
Deformation Rate (in./min.)	0.05	0.05	0.05
Initial Sample Height (in.)	1.0	1.0	1.0
Ring Inside Diameter (in.)	2.375	2.375	2.375
Initial Moisture Content (%)	9.7	9.7	9.7
Initial Dry Density (pcf)	108.0	108.0	108.0
Initial Degree of Saturation (%)	46.7	46.7	46.7
Soil Height Before Shearing (in.)	1.2	1.2	1.2
Final Moisture Content (%)	16.1	16.2	16.8

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DIRECT SHEAR TEST RESULTS

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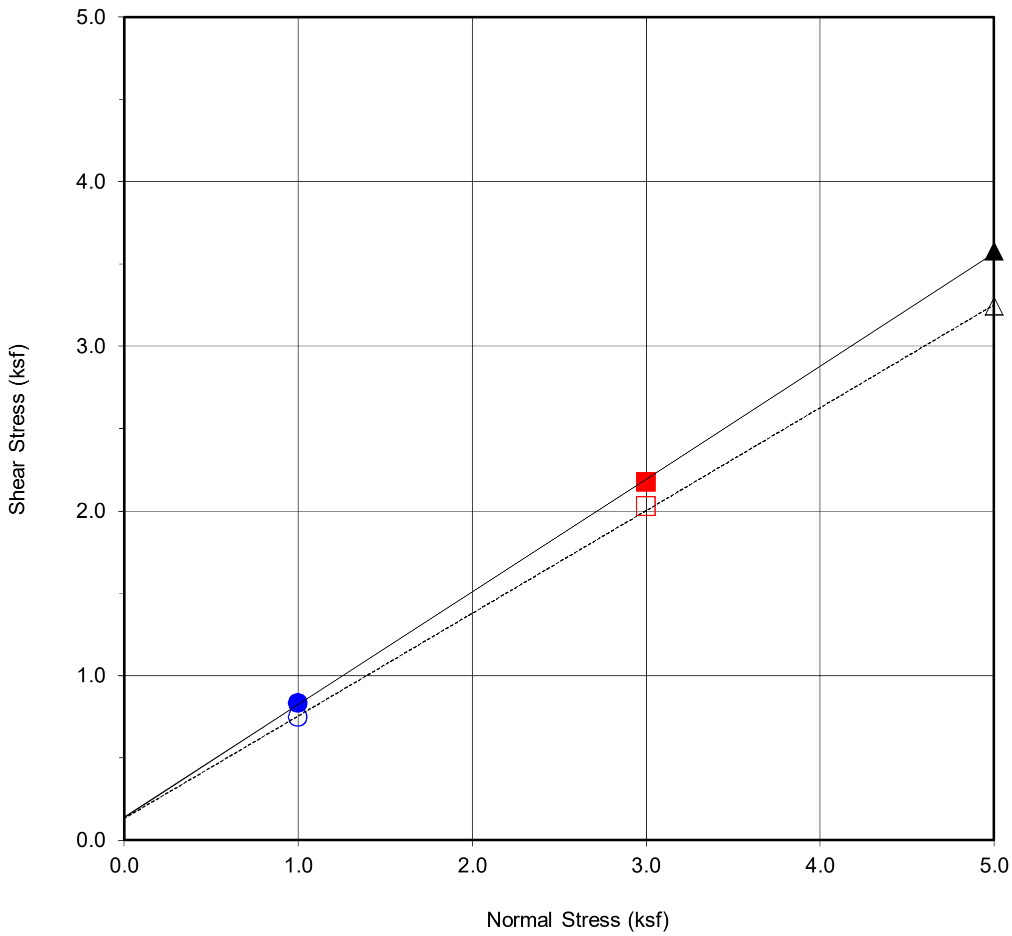
Figure B8



Boring No.	**B5**
Sample No.	**B5@2**
Depth (ft)	**2**
Sample Type:	Ring

Soil Identification:		
Grayish Brown Silty Sand (SM)		
Strength Parameters		
	C (psf)	ϕ (°)
Peak	136	34.5
Ultimate	131	32.0

Normal Stress (kip/ft2)	1	3	5
Peak Shear Stress (kip/ft²)	● 0.83	■ 2.17	▲ 3.58
Shear Stress @ End of Test (ksf)	○ 0.75	□ 2.02	△ 3.24
Deformation Rate (in./min.)	0.05	0.05	0.05
Initial Sample Height (in.)	1.0	1.0	1.0
Ring Inside Diameter (in.)	2.375	2.375	2.375
Initial Moisture Content (%)	6.1	7.2	5.9
Initial Dry Density (pcf)	100.8	101.4	100.0
Initial Degree of Saturation (%)	24.6	29.4	23.1
Soil Height Before Shearing (in.)	1.2	1.2	1.2
Final Moisture Content (%)	19.3	18.8	19.3

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DIRECT SHEAR TEST RESULTS

Consolidated Drained ASTM D-3080

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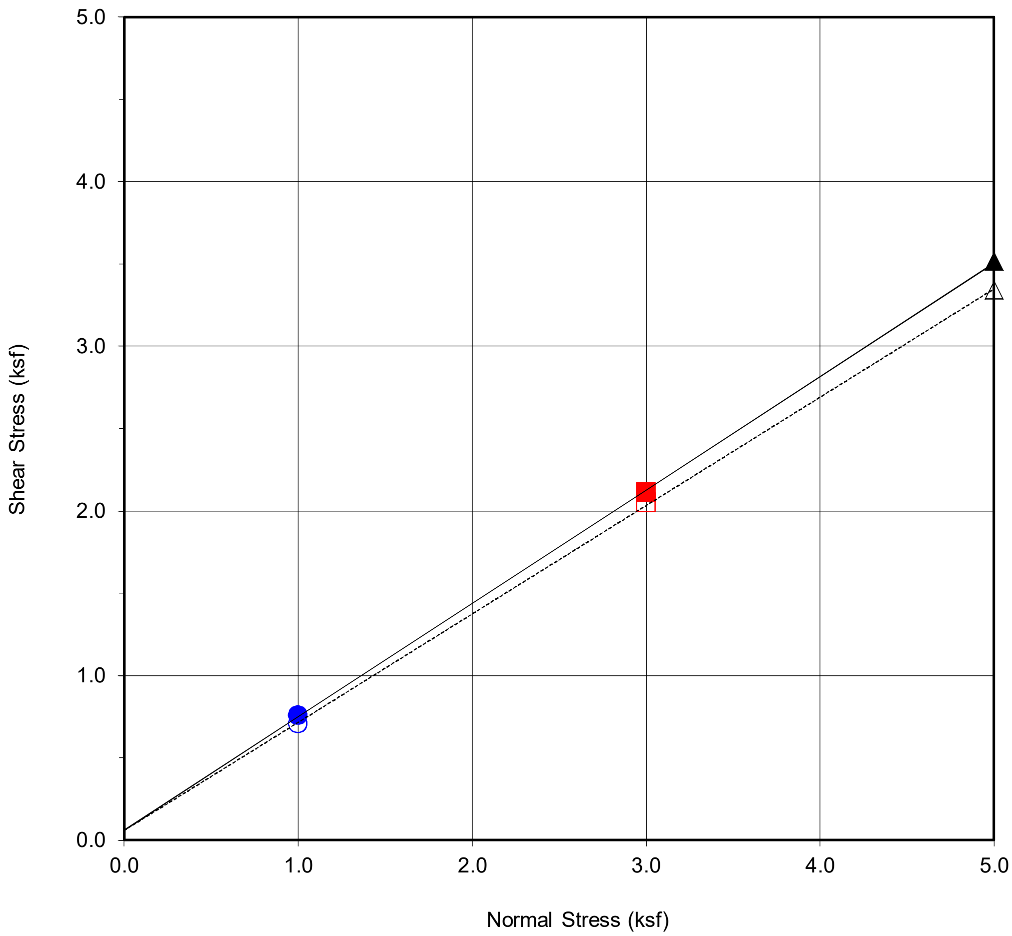
Figure B9



Boring No.	**B6**
Sample No.	**B6@3'**
Depth (ft)	**3**
Sample Type:	Ring

Normal Stress (kip/ft2)	1	3	5
Peak Shear Stress (kip/ft²)	● 0.76	■ 2.11	▲ 3.51
Shear Stress @ End of Test (ksf)	○ 0.71	□ 2.05	△ 3.34
Deformation Rate (in./min.)	0.05	0.05	0.05
Initial Sample Height (in.)	1.0	1.0	1.0
Ring Inside Diameter (in.)	2.375	2.375	2.375
Initial Moisture Content (%)	13.0	13.5	14.0
Initial Dry Density (pcf)	101.9	100.1	99.7
Initial Degree of Saturation (%)	53.8	53.2	54.9
Soil Height Before Shearing (in.)	1.2	1.2	1.2
Final Moisture Content (%)	19.0	18.8	17.9

Soil Identification:

Light Brown Poorly Graded Sand (SP)

Strength Parameters

	C (psf)	ϕ (°)
Peak	60	34.6
Ultimate	58	33.3

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DIRECT SHEAR TEST RESULTS

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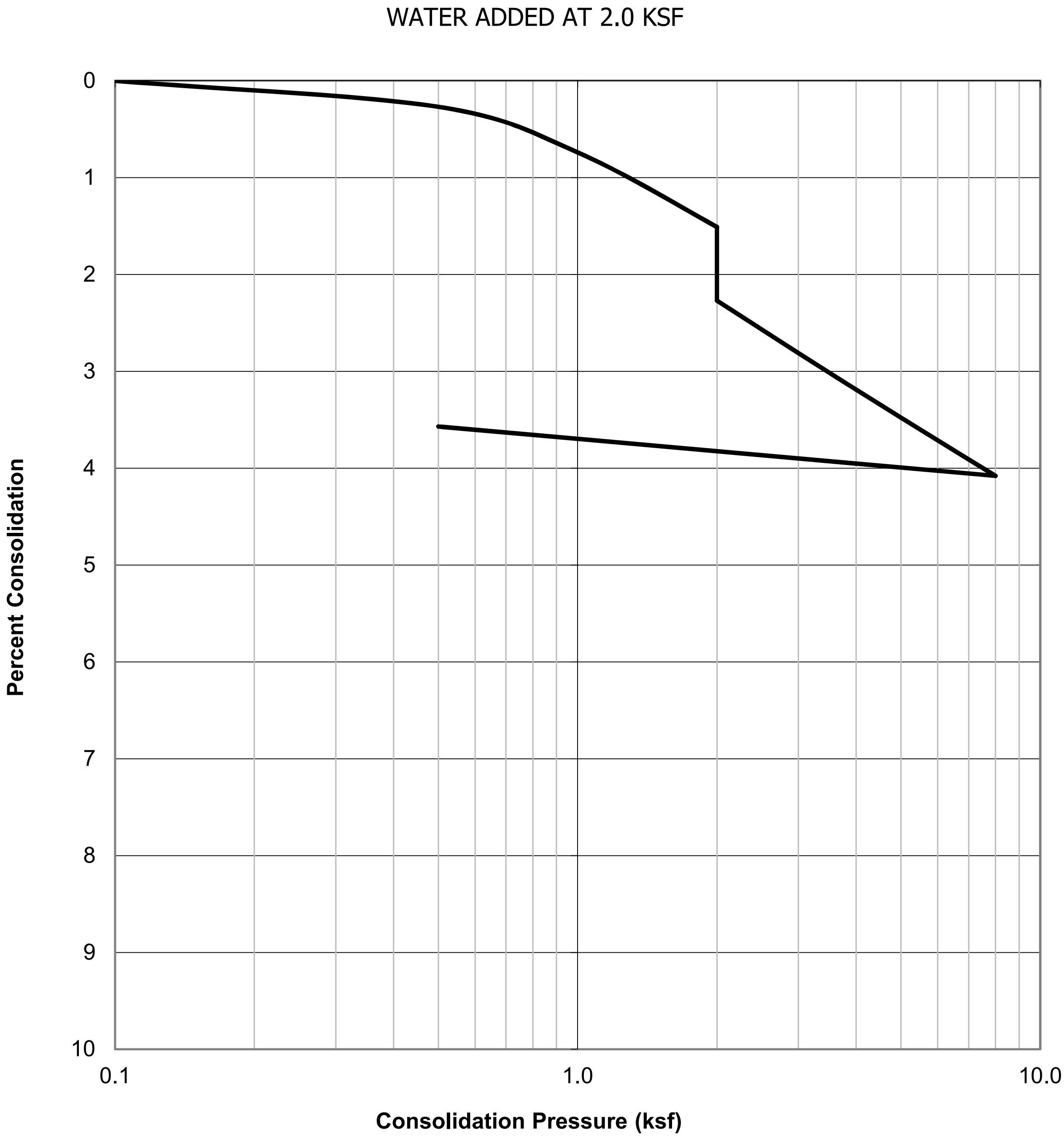
Figure B10



SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B1@3	Light Brown Poorly Graded Sand (SP)	103.3	4.9	16.6

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CONSOLIDATION TEST RESULTS

ASTM D-2435

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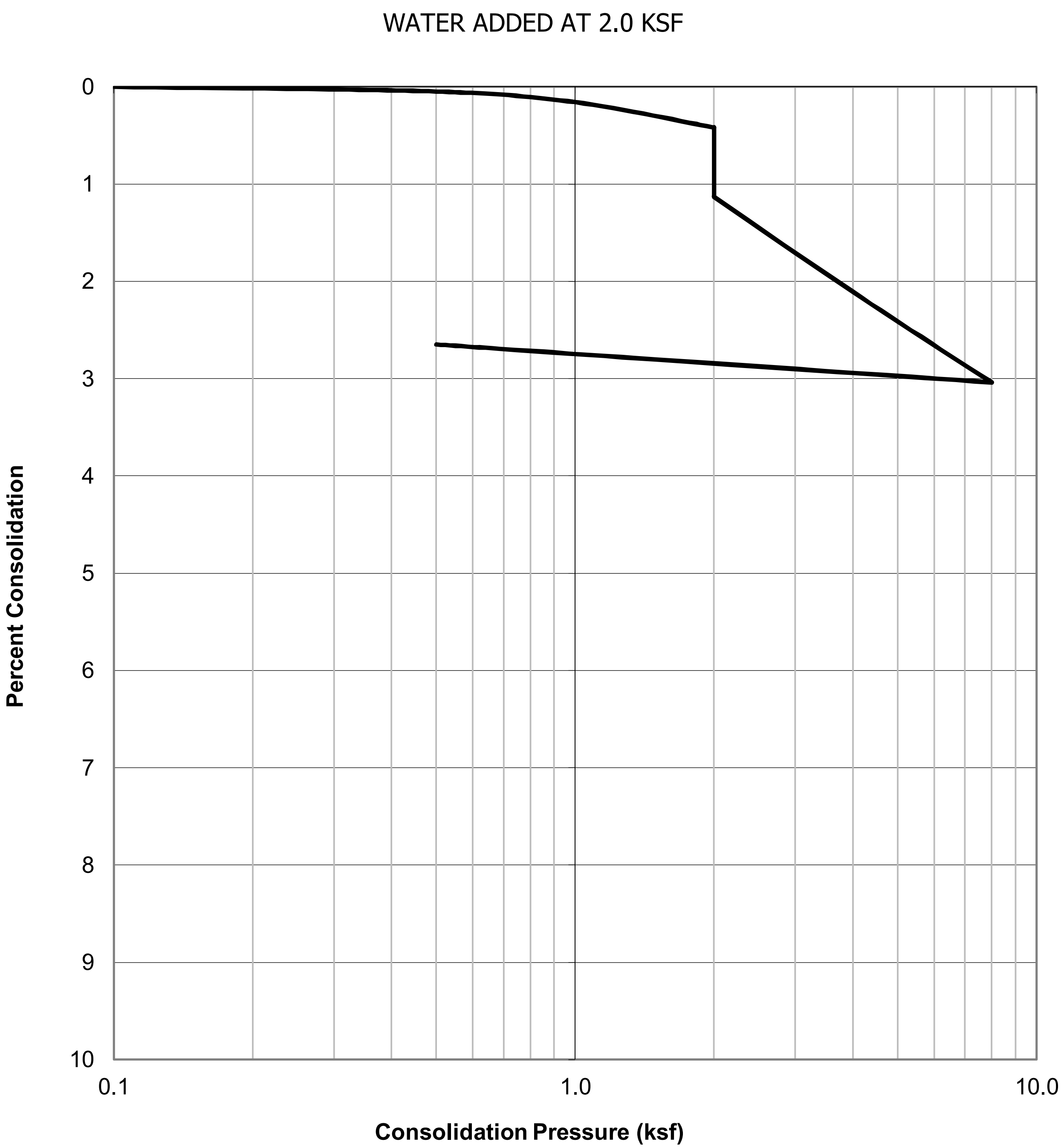
Figure B11



SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B1@6	Brown Poorly Graded Sand Trace Gravel (SP)	108.3	2.6	14.6

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CONSOLIDATION TEST RESULTS

ASTM D-2435

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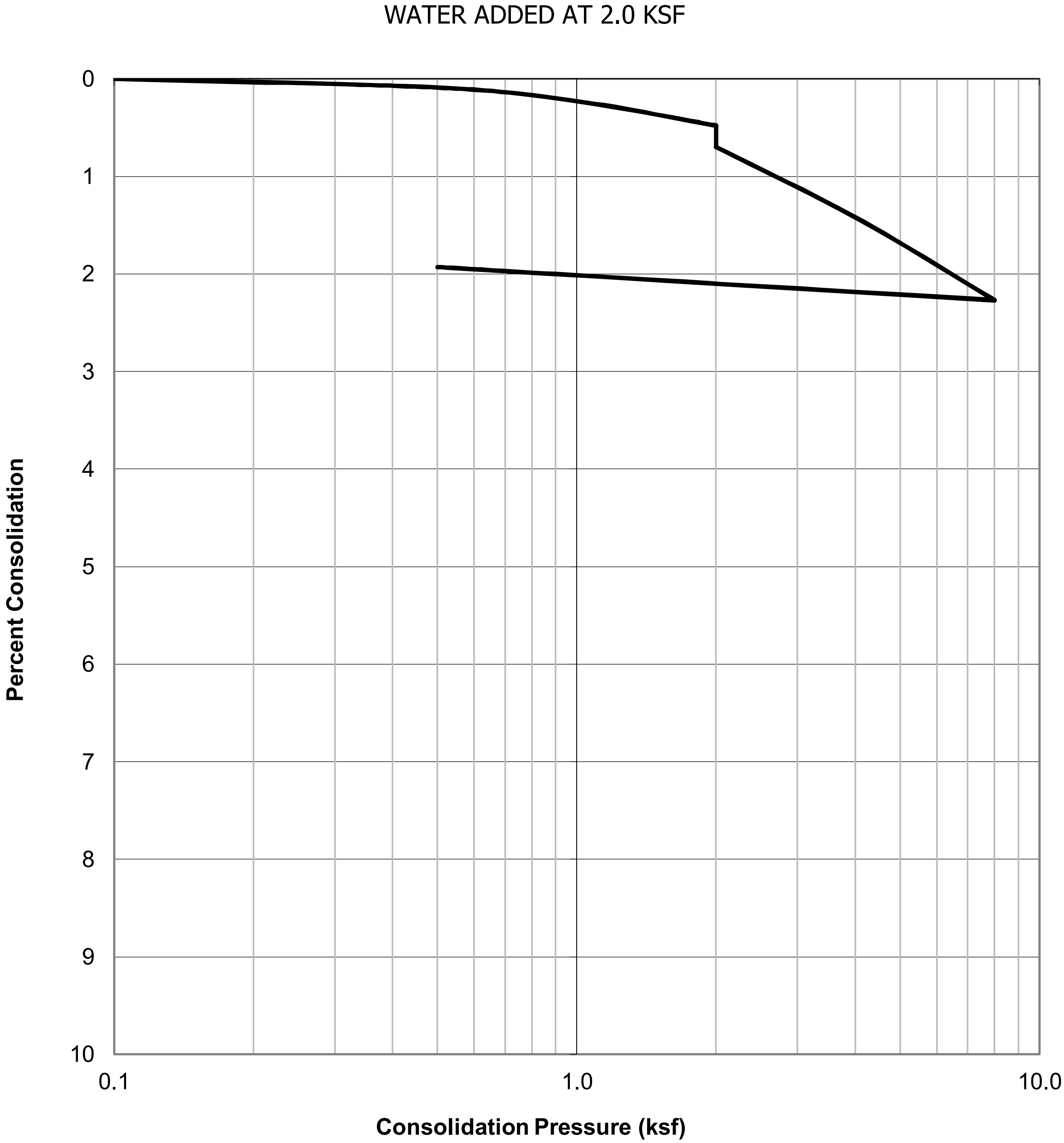
Figure B12



SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B1@9	Brown Poorly Graded Sand Trace Gravel (SP)	107.2	5.3	16.5

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CONSOLIDATION TEST RESULTS

ASTM D-2435

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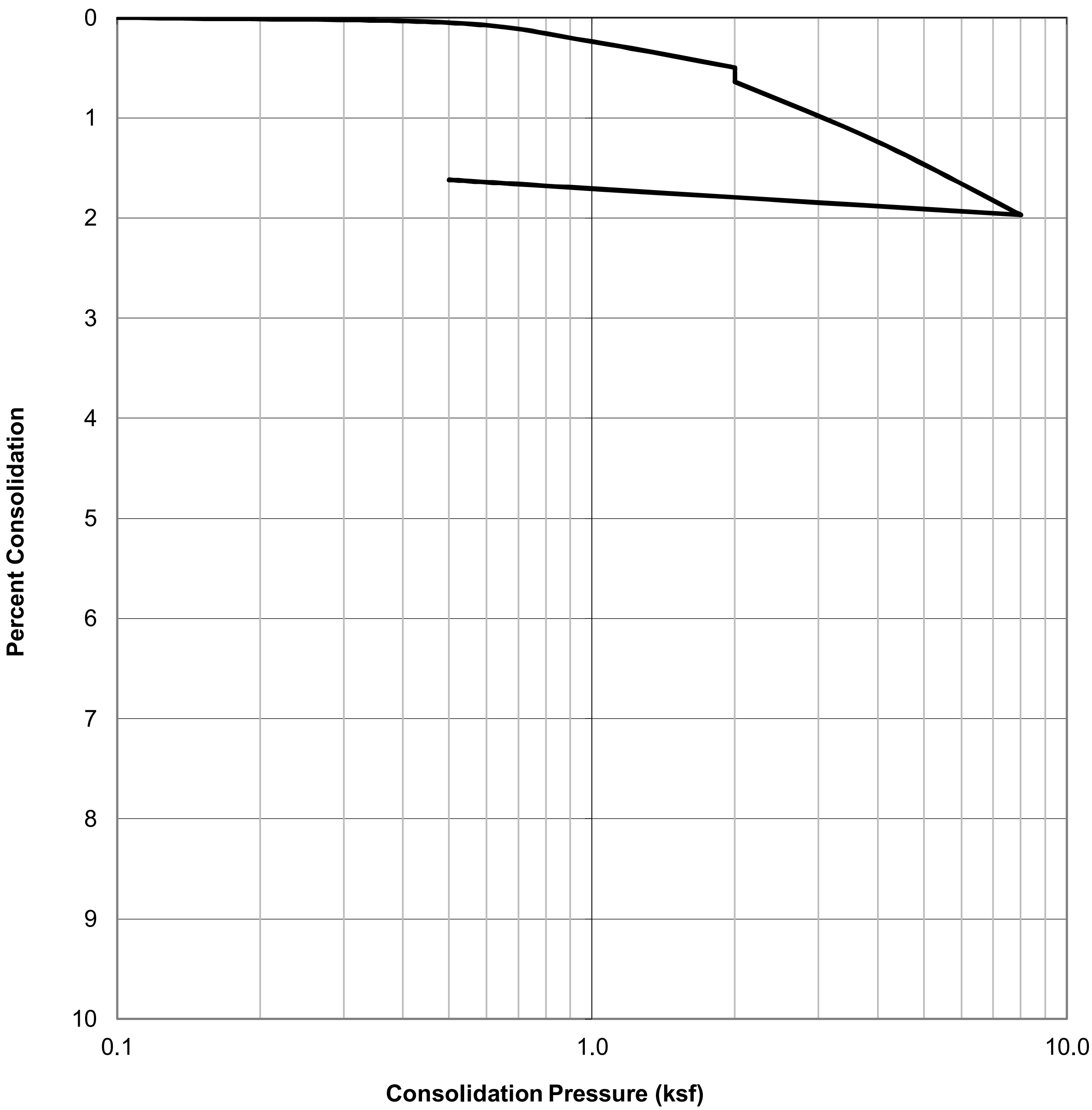
Figure B13

WATER ADDED AT 2.0 KSF



SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B1@15	Brown Poorly Graded Sand (SP)	100.1	9.9	22.3

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CONSOLIDATION TEST RESULTS

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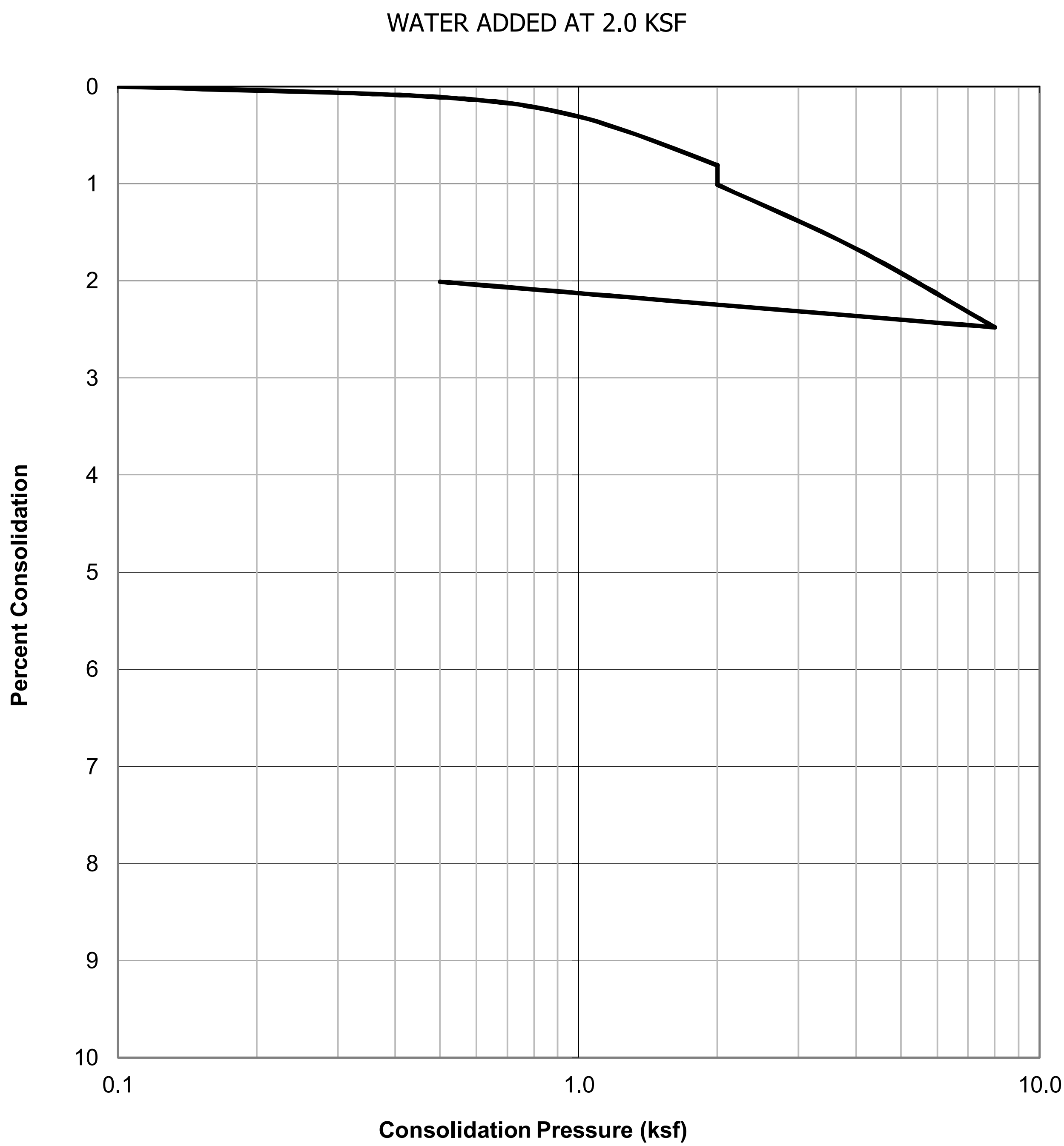
Figure B14



SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B1@20	Brown Poorly Graded Sand Trace Gravel (SP)	106.9	4.2	14.5

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CONSOLIDATION TEST RESULTS

ASTM D-2435

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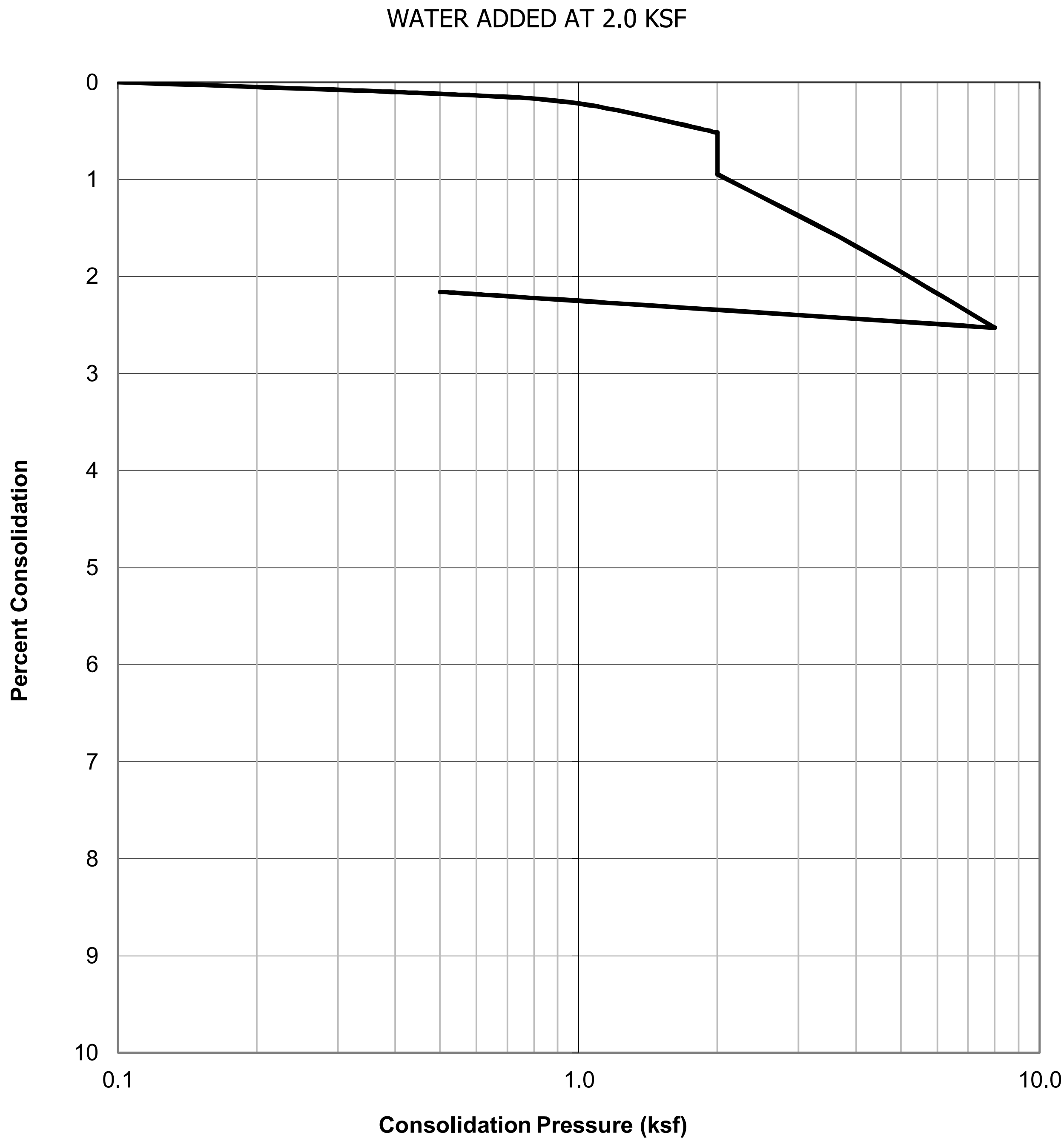
Figure B15



SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B1@30	Brown Poorly Graded Sand (SP)	106.6	4.5	15.9

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CONSOLIDATION TEST RESULTS

ASTM D-2435

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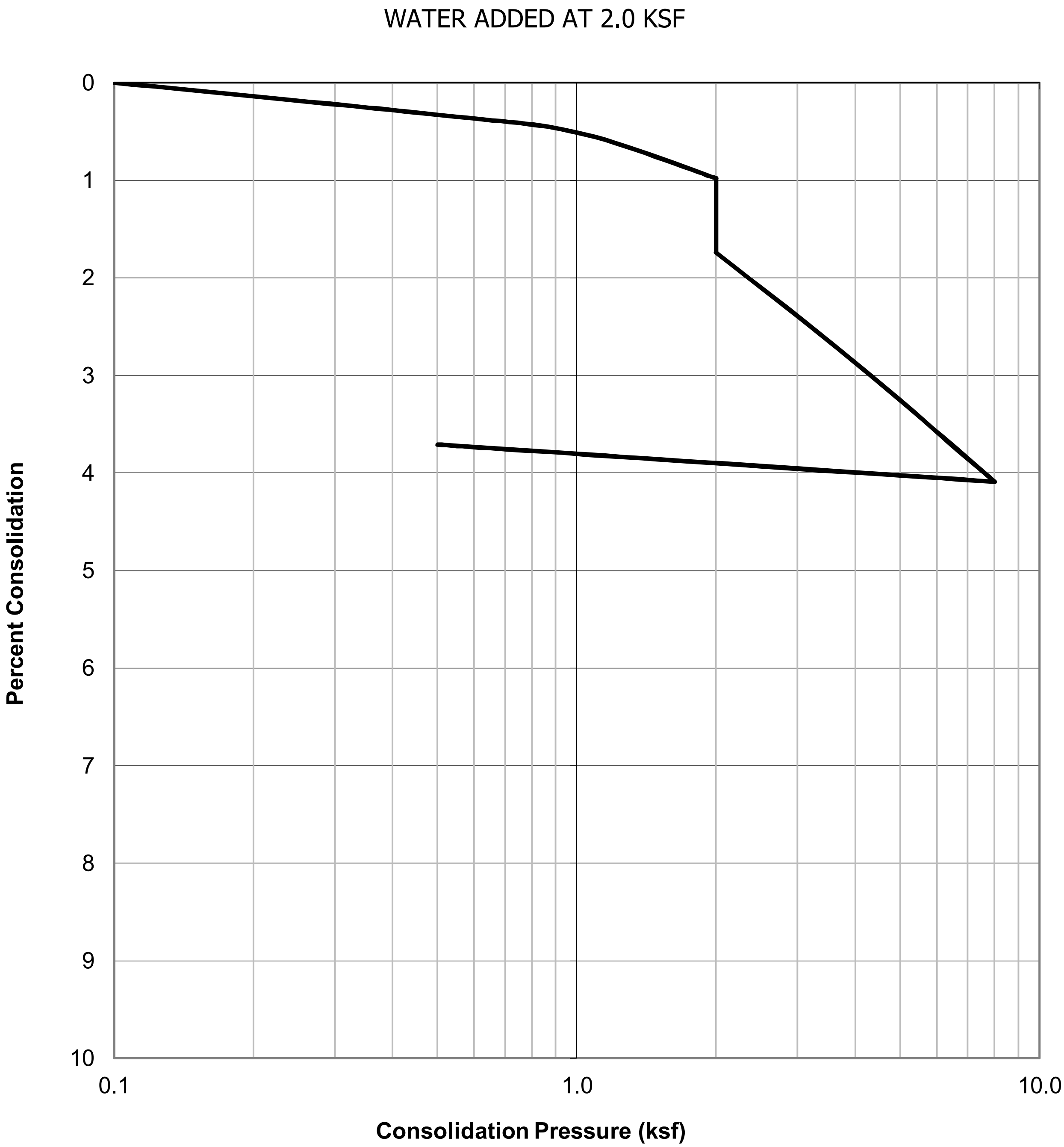
Figure B16



SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B3@6	Light Brown Poorly Graded Sand Trace Gravel (SP)	108.0	1.8	14.3

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CONSOLIDATION TEST RESULTS

ASTM D-2435

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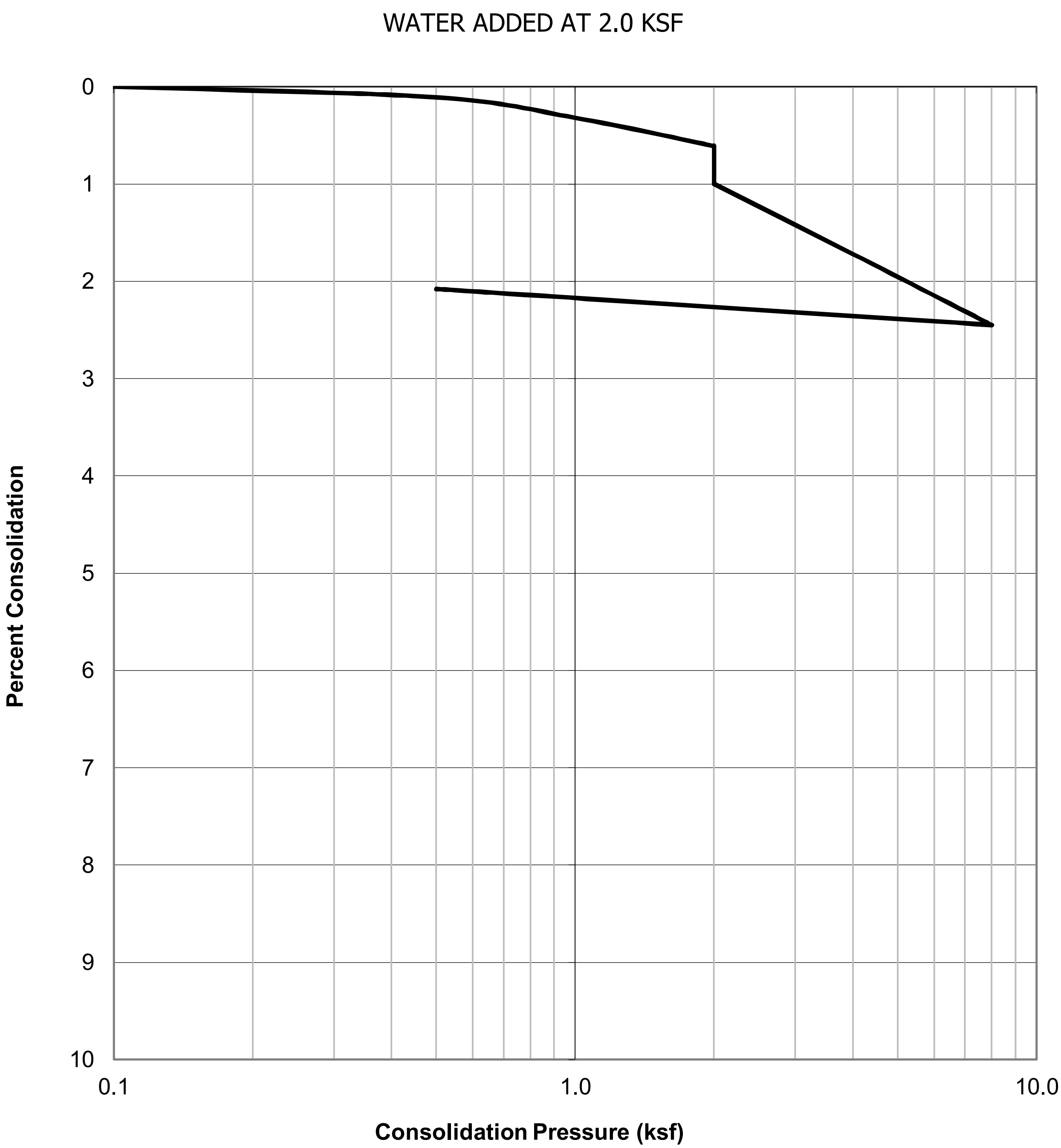
Figure B17



SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B3@9	Light Brown Poorly Graded Sand (SP)	98.7	2.5	20.2

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CONSOLIDATION TEST RESULTS

ASTM D-2435

Checked by: JMH

Project No.: W1233-06-01

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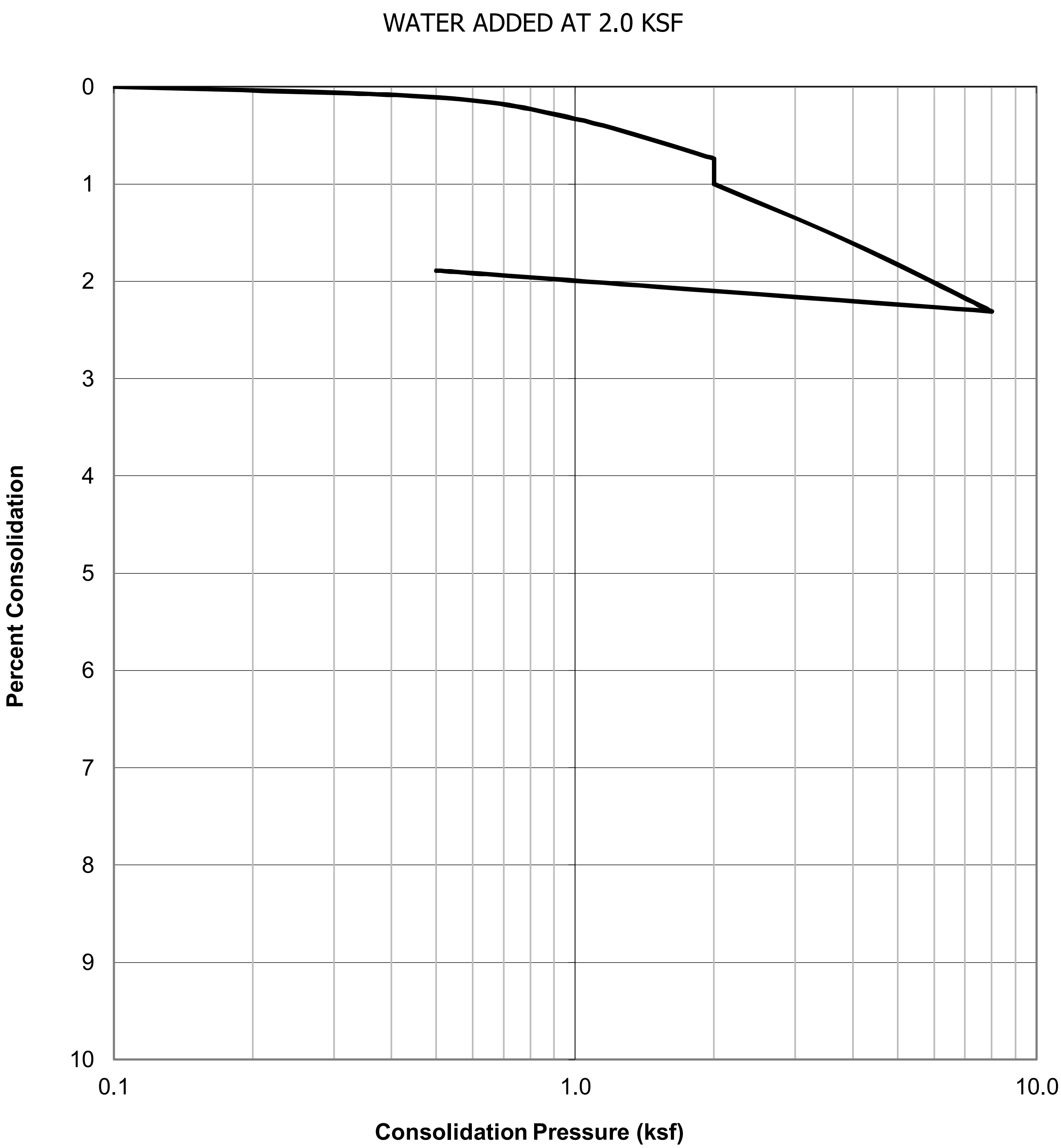
Figure B18



SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B3@15	Light Brown Poorly Graded Sand Trace Gravel (SP)	100.9	2.5	19.7

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CONSOLIDATION TEST RESULTS

ASTM D-2435

Checked by: JMH

Project No.: W1233-06-01

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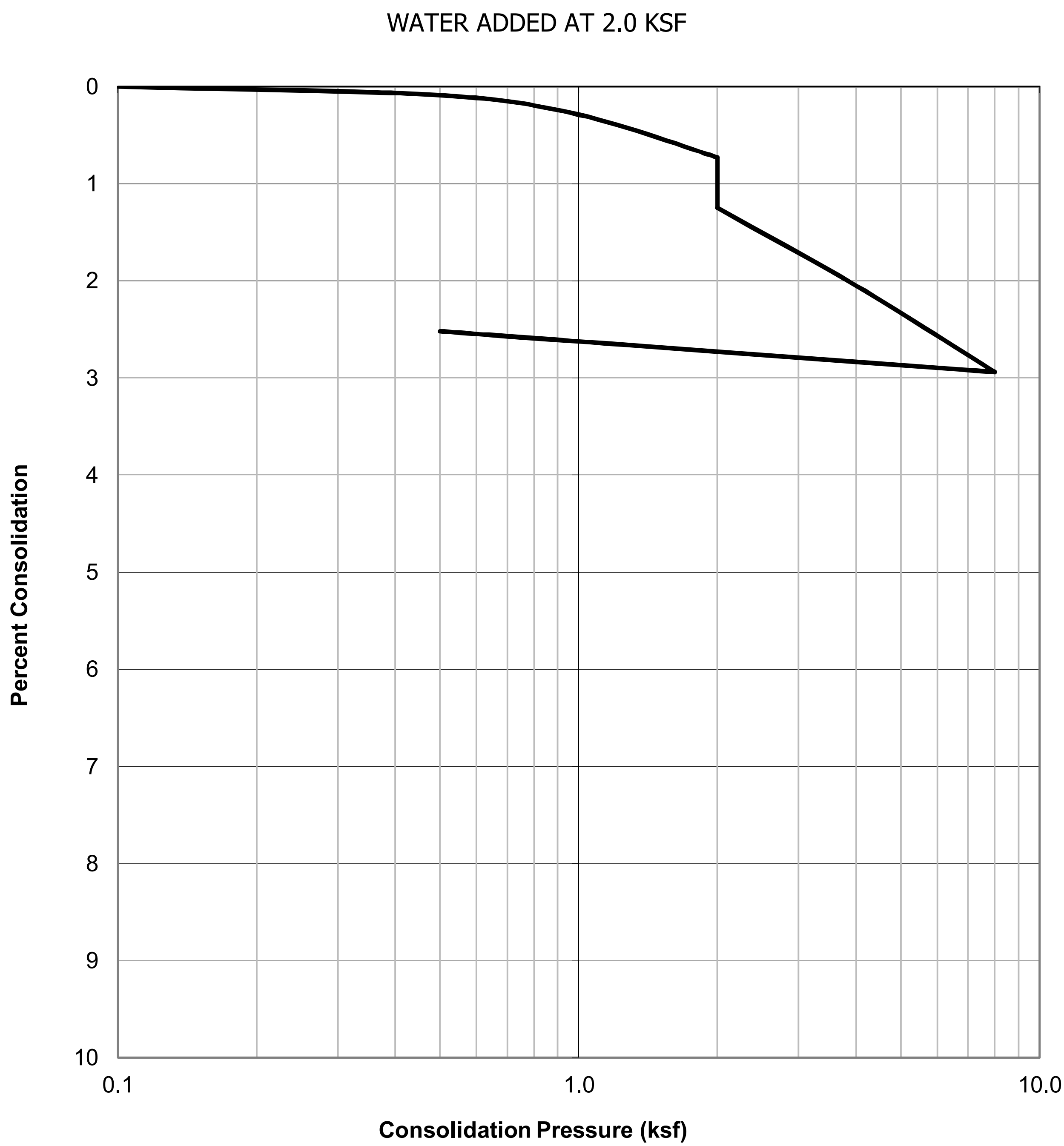
Figure B19



SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B3@25	Light Brown Poorly Graded Sand (SP)	109.5	2.9	15.9

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CONSOLIDATION TEST RESULTS

ASTM D-2435

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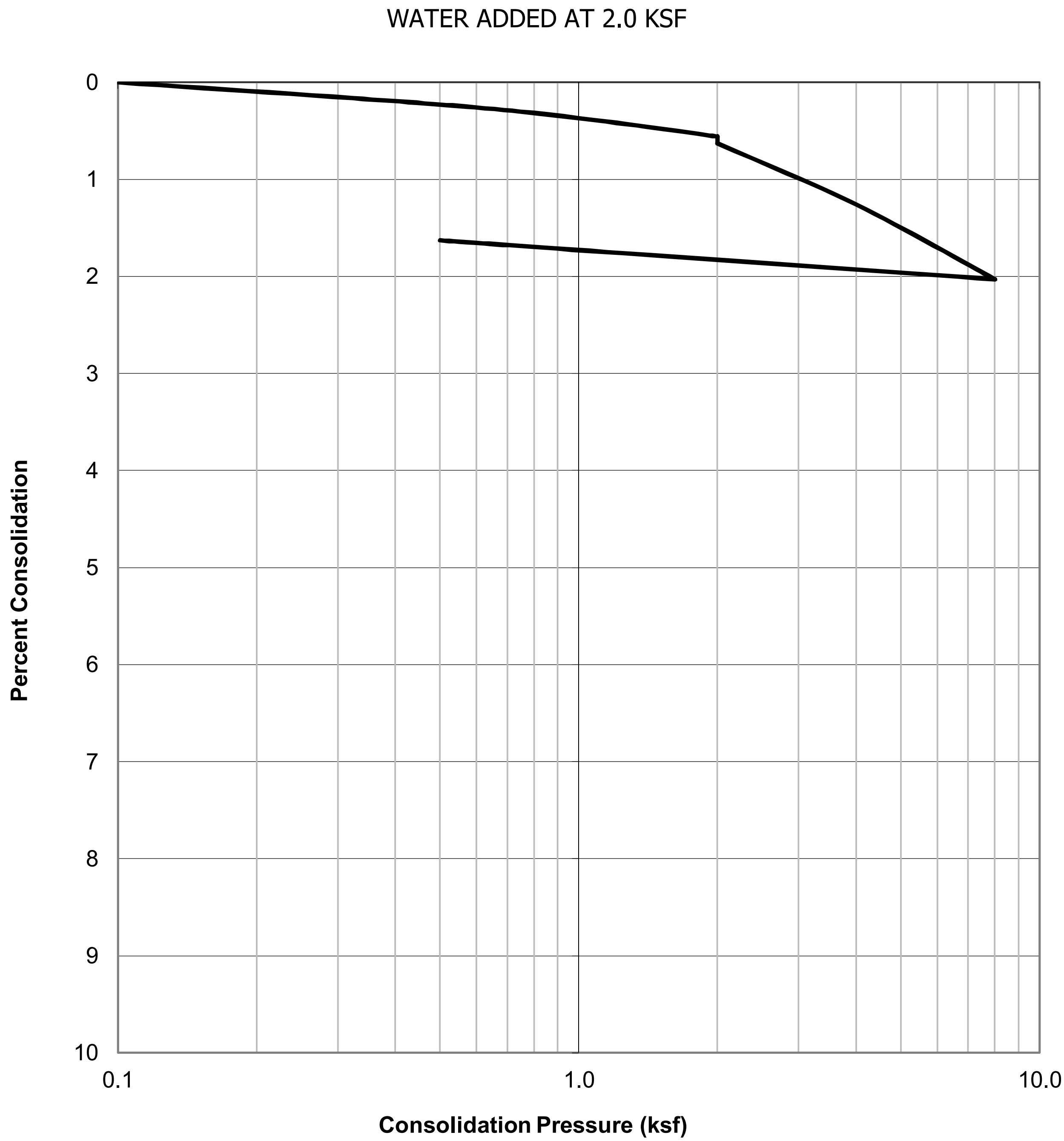
Figure B20



SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B4@6	Brown Poorly Graded Sand (SP)	100.3	2.7	20.3

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CONSOLIDATION TEST RESULTS

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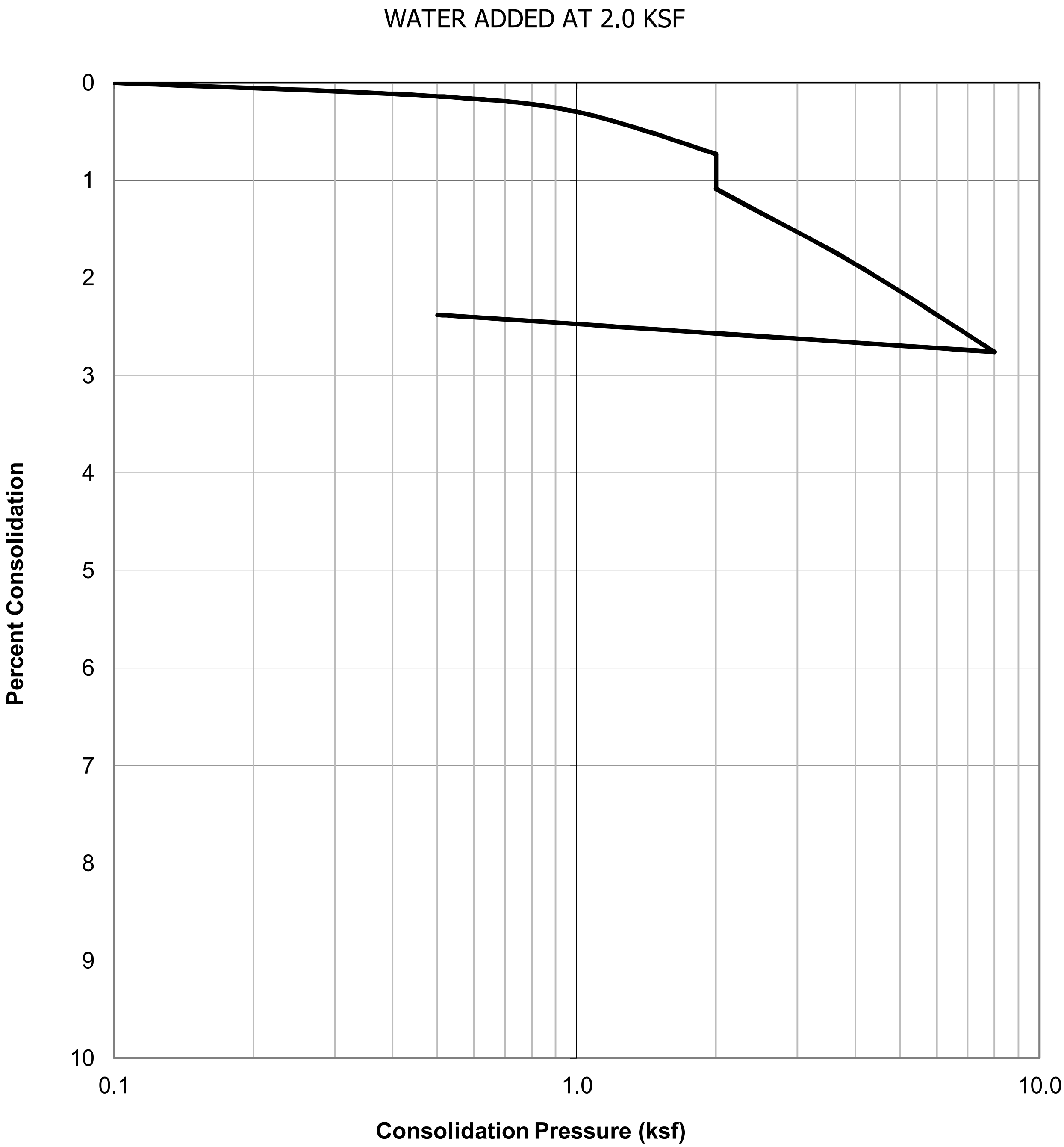
Figure B21



SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B4@9	Brown Poorly Graded Sand (SP)	108.5	2.9	15.8

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CONSOLIDATION TEST RESULTS

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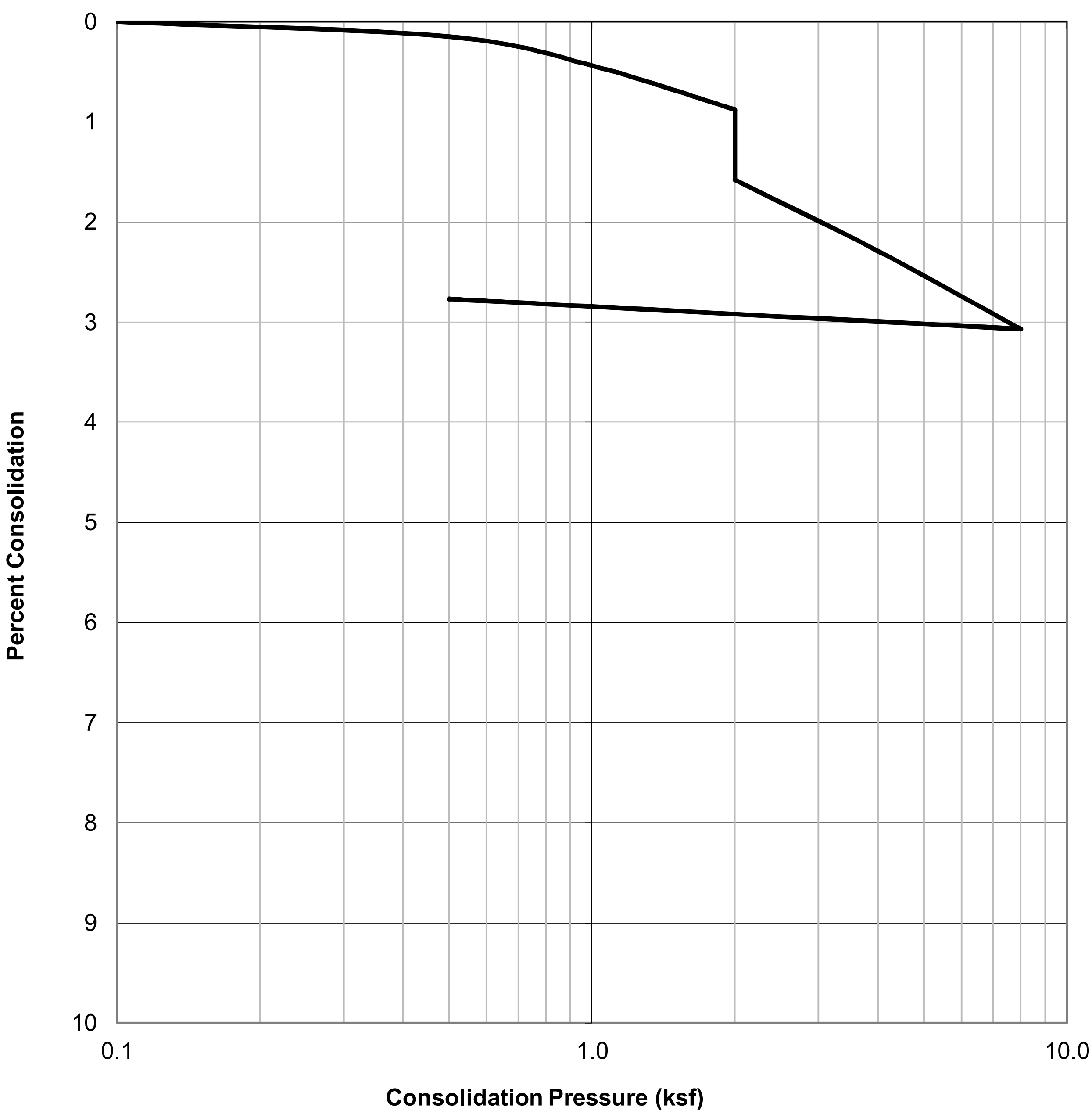
Figure B22

WATER ADDED AT 2.0 KSF



SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B4@15	Brown Poorly Graded Sand with Gravel (SP)	107.6	3.1	14.0

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CONSOLIDATION TEST RESULTS

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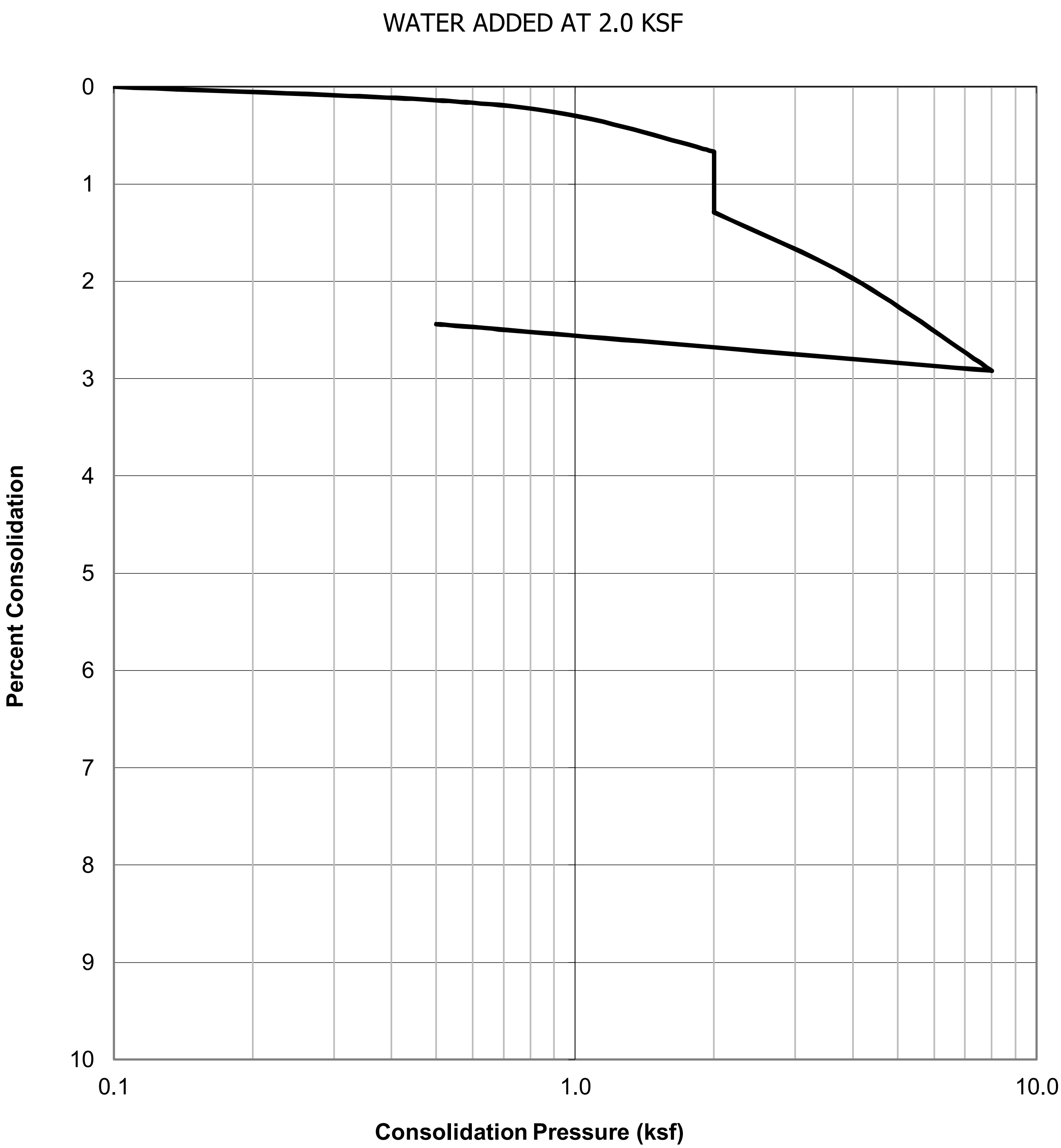
Figure B23



SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B5@6	Light Brown Brown Sand (SP)	106.7	2.0	15.8

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CONSOLIDATION TEST RESULTS

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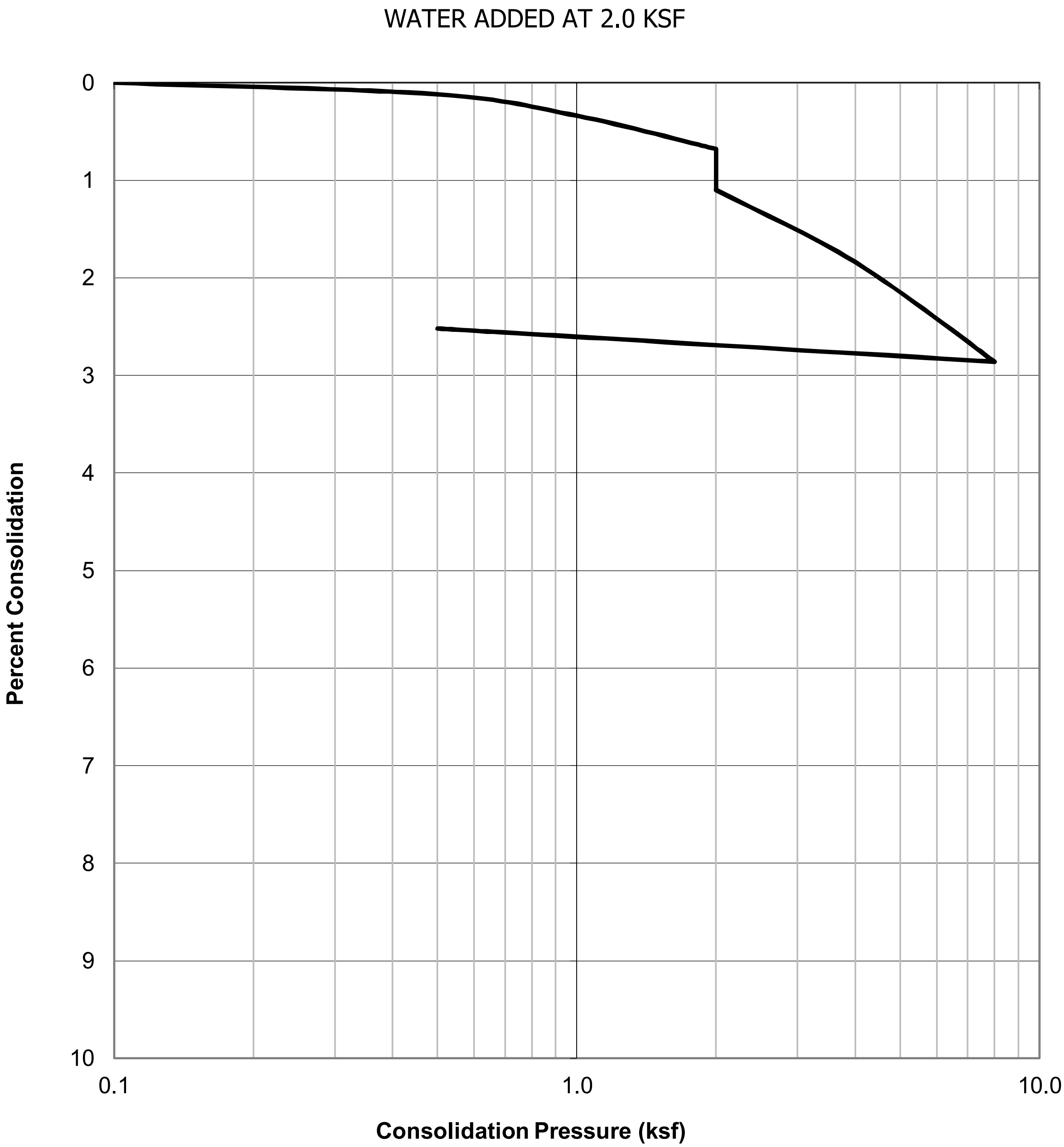
Figure B24



SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B5@10	Light Gray Poorly Graded Sand (SP)	107.5	2.4	15.9

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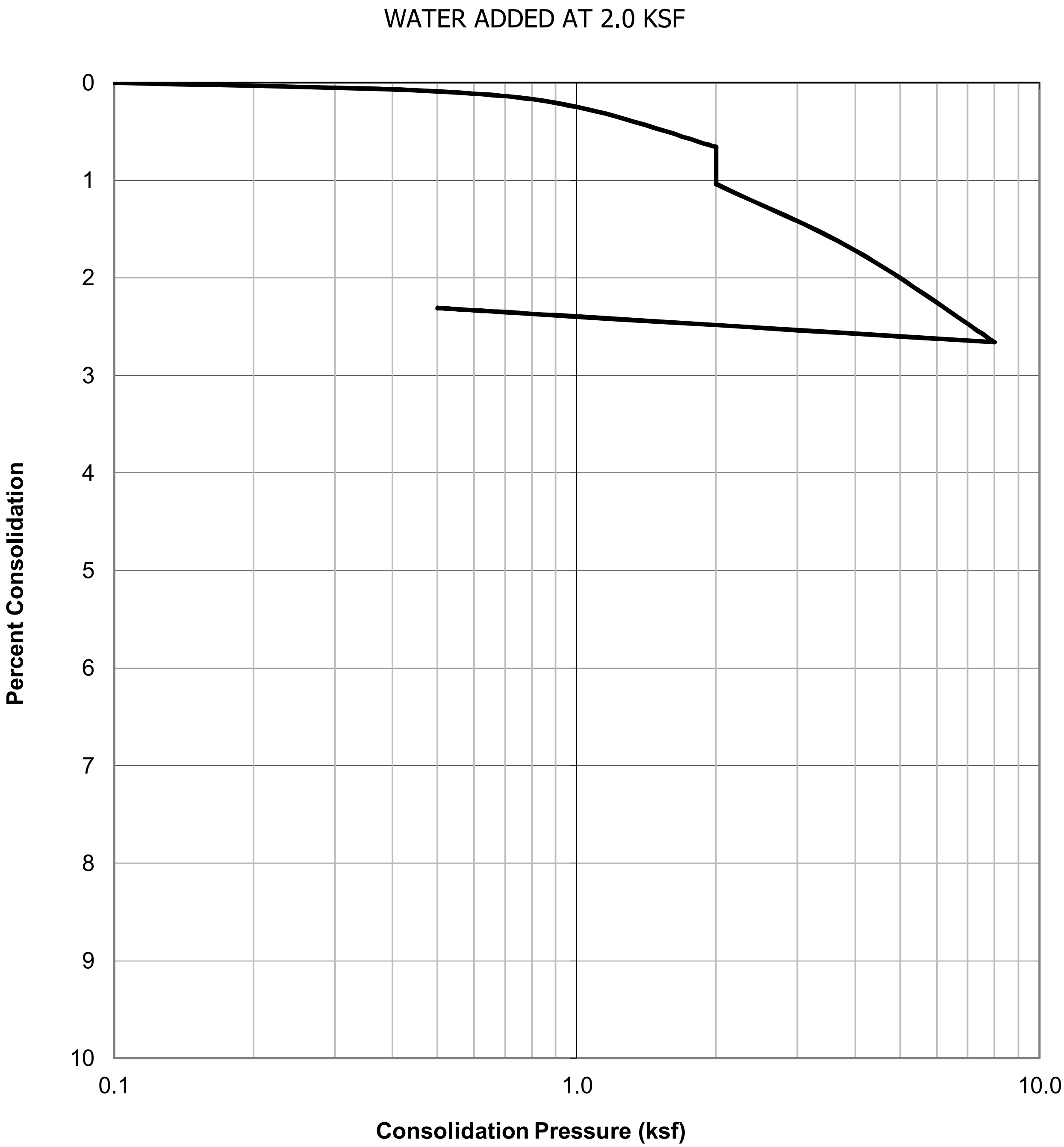
Figure B25



SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B5@15	Brown Poorly Graded Sand (SP)	107.1	4.4	16.7

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CONSOLIDATION TEST RESULTS

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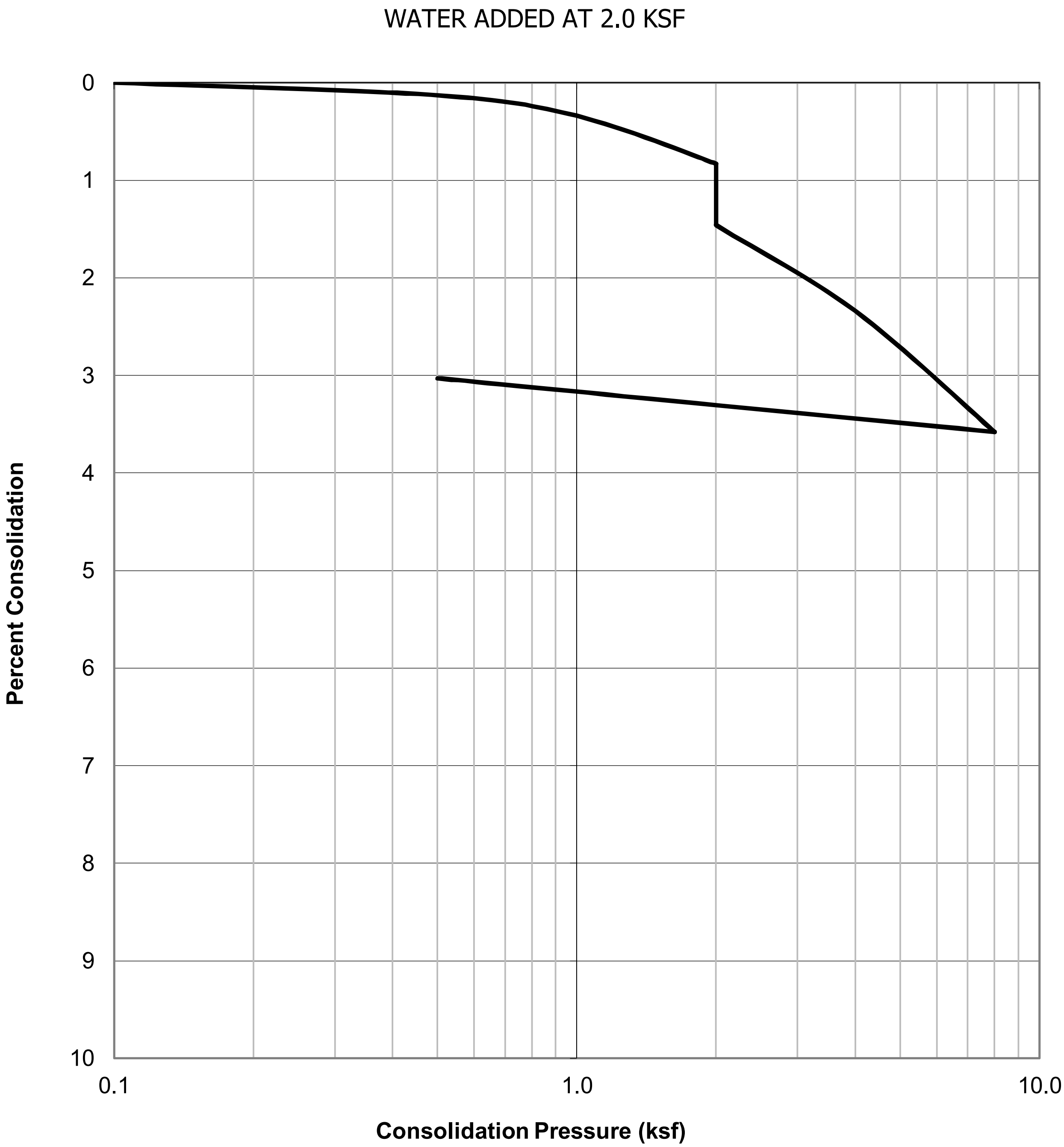
Figure B26



SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B5@30	Brown Silty Sand (SM)	98.9	6.4	21.0

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CONSOLIDATION TEST RESULTS

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Figure B27

B1@0-5

MOLDED SPECIMEN		BEFORE TEST	AFTER TEST
Specimen Diameter	(in.)	4.0	4.0
Specimen Height	(in.)	1.0	1.0
Wt. Comp. Soil + Mold	(gm)	764.8	783.0
Wt. of Mold	(gm)	367.8	367.8
Specific Gravity	(Assumed)	2.7	2.7
Wet Wt. of Soil + Cont.	(gm)	500.2	783.0
Dry Wt. of Soil + Cont.	(gm)	473.4	361.6
Wt. of Container	(gm)	200.2	367.8
Moisture Content	(%)	9.8	14.8
Wet Density	(pcf)	119.8	125.1
Dry Density	(pcf)	109.1	108.9
Void Ratio		0.5	0.5
Total Porosity		0.4	0.4
Pore Volume	(cc)	73.1	72.4
Degree of Saturation	(%) [S_{meas}]	48.9	74.1

Date	Time	Pressure (psi)	Elapsed Time (min)	Dial Readings (in.)
10/16/2020	10:00	1.0	0	0.2425
10/16/2020	10:10	1.0	10	0.2425
Add Distilled Water to the Specimen				
10/17/2020	10:00	1.0	1430	0.239
10/17/2020	11:00	1.0	1490	0.239

Expansion Index (EI meas) =	-3.5
Expansion Index (Report) =	**0**

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

* Reference: 2019 California Building Code, Section 1803.5.3

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

GEOCON

EXPANSION INDEX TEST RESULTS

ASTM D-4829

Checked by: JMH

Project No.: W1233-06-01

2311 N Hollywood Way
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Figure B28

B5@0-5'

MOLDED SPECIMEN		BEFORE TEST	AFTER TEST
Specimen Diameter	(in.)	4.0	4.0
Specimen Height	(in.)	1.0	1.0
Wt. Comp. Soil + Mold	(gm)	779.1	801.8
Wt. of Mold	(gm)	368.6	368.6
Specific Gravity	(Assumed)	2.7	2.7
Wet Wt. of Soil + Cont.	(gm)	500.2	801.8
Dry Wt. of Soil + Cont.	(gm)	476.4	378.0
Wt. of Container	(gm)	200.2	368.6
Moisture Content	(%)	8.6	14.6
Wet Density	(pcf)	123.8	130.5
Dry Density	(pcf)	114.0	113.9
Void Ratio		0.5	0.5
Total Porosity		0.3	0.3
Pore Volume	(cc)	67.0	66.8
Degree of Saturation	(%) [S_{meas}]	48.9	82.6

Date	Time	Pressure (psi)	Elapsed Time (min)	Dial Readings (in.)
10/19/2020	10:00	1.0	0	0.2795
10/19/2020	10:10	1.0	10	0.2795
Add Distilled Water to the Specimen				
10/20/2020	10:00	1.0	1430	0.2785
10/20/2020	11:00	1.0	1490	0.2785

Expansion Index (EI meas) =	-1
Expansion Index (Report) =	**-1**

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

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

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EXPANSION INDEX TEST RESULTS
ASTM D-4829

Checked by: JMH

Project No.: W1233-06-01

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Figure B29

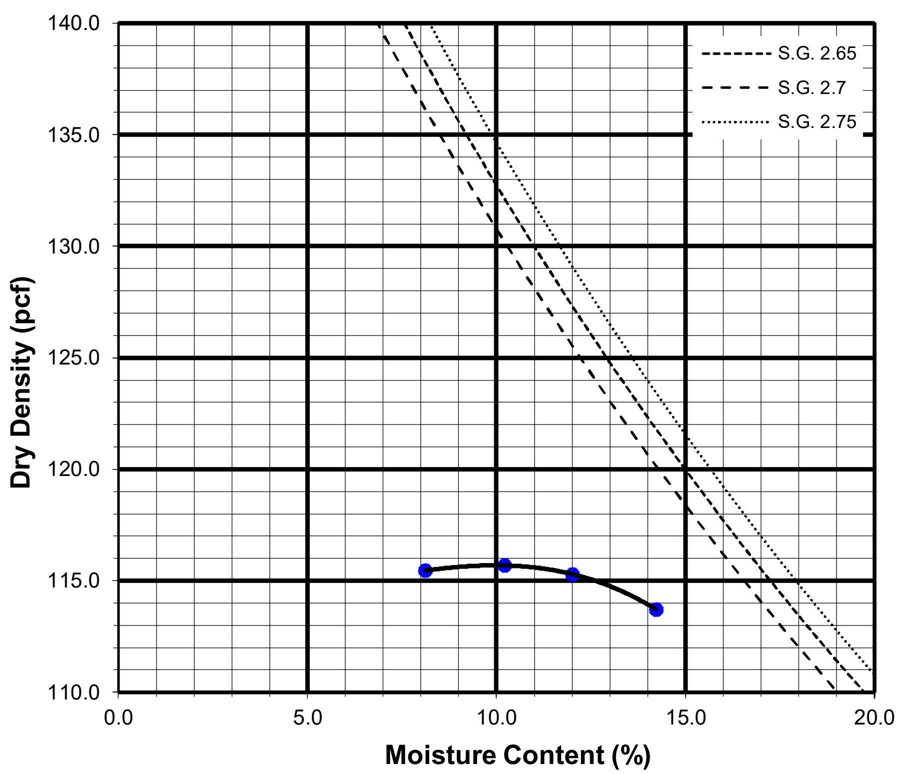
Sample No:

B1@0-5'	Light Gray Brown Poorly Graded Sand (SP)

TEST NO.		1	2	3	4	5	6
Wt. Compacted Soil + Mold	(g)	6024	6064	6089	6100		
Weight of Mold	(g)	4137	4137	4137	4137		
Net Weight of Soil	(g)	1887	1928	1952	1964		
Wet Weight of Soil + Cont.	(g)	652.5	641.5	643.0	680.0		
Dry Weight of Soil + Cont.	(g)	614.5	595.8	589.8	612.0		
Weight of Container	(g)	145.9	148.3	146.4	134.0		
Moisture Content	(%)	8.1	10.2	12.0	14.2		
Wet Density	(pcf)	124.8	127.5	129.1	129.9		
Dry Density	(pcf)	115.5	115.7	115.3	113.7		

Maximum Dry Density (pcf)	**116.0**
Bulk Specific Gravity (dry)	**2.65**
Corrected Maximum Dry Density (pcf)	**119.0**

Optimum Moisture Content (%)	**10.5**
Oversized Fraction (%)	**8.0**
Corrected Moisture Content (%)	**10.0**



Preparation Method: A

GEOCON

MODIFIED COMPACTION TEST OF SOILS

ASTM D-1557

Checked by: JMH

Project No.: W1233-06-01

2311 N Hollywood Way
Burbank, CA

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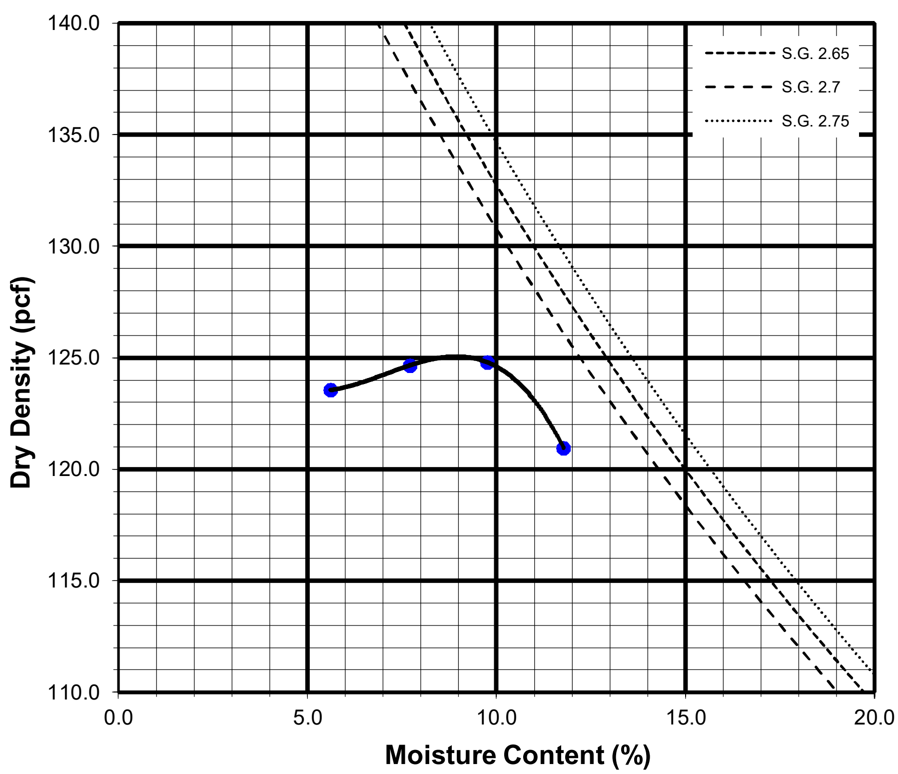
Figure B30

Sample No:

B2&B4@0-5	Light Brown Silty Sand (SM)

TEST NO.		1	2	3	4	5	6
Wt. Compacted Soil + Mold	(g)	6165	6206	6179	6108		
Weight of Mold	(g)	4137	4137	4137	4137		
Net Weight of Soil	(g)	2028	2069	2042	1971		
Wet Weight of Soil + Cont.	(g)	650.1	719.0	656.4	654.6		
Dry Weight of Soil + Cont.	(g)	613.3	667.0	602.7	627.7		
Weight of Container	(g)	135.7	134.2	146.7	148.4		
Moisture Content	(%)	7.7	9.8	11.8	5.6		
Wet Density	(pcf)	134.3	137.0	135.2	130.5		
Dry Density	(pcf)	124.7	124.8	120.9	123.6		

Maximum Dry Density (pcf)	125.0	Optimum Moisture Content (%)	9.0



Preparation Method: A

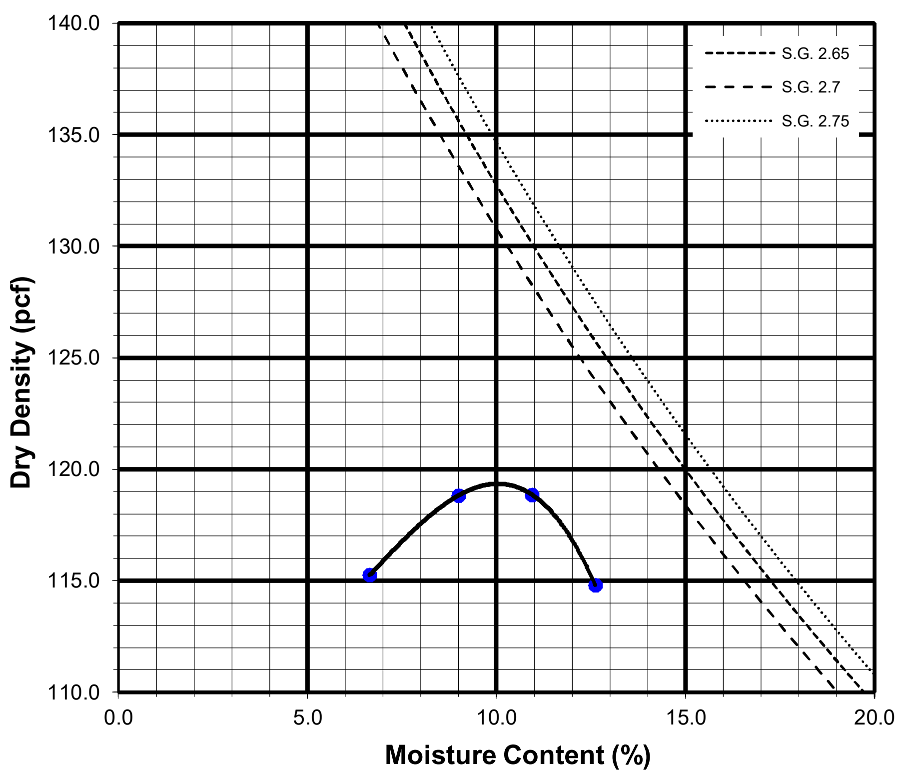
	MODIFIED COMPACTION TEST OF SOILS	Project No.: W1233-06-01
GEOCON	ASTM D-1557	2311 N Hollywood Way Burbank, CA
	Checked by: JMH	MAY 2021 Figure B31

Sample No:

B5@0-5'	Grayish Brown Silty Sand (SM)

TEST NO.		1	2	3	4	5	6
Wt. Compacted Soil + Mold	(g)	5993	6093	6129	6090		
Weight of Mold	(g)	4137	4137	4137	4137		
Net Weight of Soil	(g)	1856	1956	1992	1953		
Wet Weight of Soil + Cont.	(g)	638.5	719.9	759.2	783.5		
Dry Weight of Soil + Cont.	(g)	606.5	671.5	698.9	710.0		
Weight of Container	(g)	124.6	132.9	147.9	127.0		
Moisture Content	(%)	6.6	9.0	10.9	12.6		
Wet Density	(pcf)	122.9	129.5	131.9	129.3		
Dry Density	(pcf)	115.2	118.8	118.9	114.8		

Maximum Dry Density (pcf) **120.0** **Optimum Moisture Content (%)** **10.0**



Preparation Method: A

GEOCON

MODIFIED COMPACTION TEST OF SOILS

ASTM D-1557

Checked by: JMH

Project No.: W1233-06-01

2311 N Hollywood Way
Burbank, CA

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Figure B32

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

Sample No.	**pH**	**Resistivity (ohm centimeters)**
B1 @ 0-5'	8.1	17000 (Mildly Corrosive)
B2&B4 @ 0-5'	8.1	10000 (Moderately Corrosive)
B5 @ 0-5'	8.0	12000 (Mildly Corrosive)

SUMMARY OF LABORATORY CHLORIDE CONTENT TEST RESULTS
EPA NO. 325.3

Sample No.	Chloride Ion Content (%)
B1@0-5'	0.006
B2&B4@0-5'	0.003
B5@0-5'	0.002

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

Sample No.	**Water Soluble Sulfate (% SQ_4)**	**Sulfate Exposure***
B1@0-5'	0.000	S0
B2&B4@0-5'	0.000	S0
B5@0-5'	0.000	S0

GEOCON	**CORROSIVITY TEST RESULTS**	Project No.: W1233-06-01
		2311 N Hollywood Way Burbank, CA
	Checked by: JMH	MAY 2021 Figure B33