Appendix F

Geotechnical Investigation

GEOTECHNICAL INVESTIGATION

PROPOSED MULTI-USE DEVELOPMENT
12805-12825 WEST VENTURA BOULEVARD
STUDIO CITY, CALIFORNIA
TRACT: P M 353; LOT: B; ARB: 2

GEOCON
WEST, INC.

GEOTECHNICAL
ENVIRONMENTAL
MATERIALS

PREPARED FOR

SPORTMAN'S LODGE OWNER, LLC
NEW YORK, NEW YORK

PROJECT NO. W1208-06-01

JUNE 30, 2021

Project No. W1208-06-01
June 30, 2021

Sportsman's Lodge Owner, LLC
c/o Midwood Management Corporation
430 Park Avenue, Suite 201
New York, New York 10022

Subject: GEOTECHNICAL INVESTIGATION
PROPOSED MULTI-USE DEVELOPMENT
12805-12825 VENTURA BOULEVARD
STUDIO CITY, CALIFORNIA
TRACT: P M 353; LOT: B; ARB: 2

Ladies and Gentlemen:

In accordance with your authorization of our proposal dated February 11, 2019, we have performed a geotechnical investigation for the proposed multi-use development located at 12805-12825 Ventura Boulevard in the Studio City district of the City of Los Angeles, California. The accompanying report presents the findings of our study and our conclusions and recommendations pertaining to the geotechnical aspects of proposed design and construction. Based on the results of our investigation, it is our opinion that the site can be developed as proposed, provided the recommendations of this report are followed and implemented during design and construction.

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

Very truly yours,

GEOCON WEST, INC.

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GEOTECHNICAL INVESTIGATION

1. PURPOSE AND SCOPE

This report presents the results of a geotechnical investigation for the proposed multi-use development located at 12805-12825 Ventura Boulevard in the City of Los Angeles, California (see Vicinity Map, Figure 1). The purpose of the investigation was to evaluate subsurface soil and geologic conditions underlying the site and, based on conditions encountered, to provide conclusions and recommendations pertaining to the geotechnical aspects of proposed design and construction.

The scope of this investigation included a review of a prior report for the adjacent site, a site reconnaissance, field exploration, laboratory testing, engineering analysis, and the preparation of this report. The site was explored on August 13 and 14, 2020, by excavating four 8-inch-diameter borings using a truck-mounted hollow-stem auger drilling machine. The borings were excavated to depths between approximately 65½ and 70½feet below the existing ground surface. The approximate locations of the exploratory borings are depicted on the Site Plan (see Figure 2A). 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 site is currently occupied by The Sportsmen's Lodge Hotel and associated asphalt paved parking lots. The site is bounded by the channelized Los Angeles River to the north, by Ventura Boulevard to the south, by a gas station, a construction site, and Coldwater Canyon Avenue to the west and southwest, and a single-story commercial development to the east. The construction site is currently being developed with seven single- and two-story, on-grade structures. The site is relatively level without any highs or lows. Surface water drainage at the site appears to have no discernible pattern. Vegetation onsite consists of a trees, shrubs, and ornamental gardens surrounding a man-made water feature.

Based on the information provided by the Client, it is our understanding that the existing structures will be demolished and the proposed development will include the construction of several multi-use structures. The proposed structures will be up to 7-levels of wood-framed construction over one to three levels of subterranean parking. A Site Plan and Cross-Section showing the limits of the proposed improvements are depicted on Figures 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 structure will be up to 1,200 kips, and wall loads will be up to 12.5 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. BACKGROUND REVIEW

As a part of our scope, we reviewed a prior geotechnical investigation report prepared for the adjacent site provided to us by the Client:

> *Geotechnical Engineering Due Diligence Study, 12825-12833 Ventura Boulevard, Studio City, Los Angeles, California*, prepared by GeoSoils Consultants, Inc., dated October 14, 2014.

The prior investigation prepared by GeoSoils Consultants, Inc. (GeoSoils) was performed for the adjacent property to the west that is currently under construction. Five borings were excavated using a hollow-stem auger drilling machine to maximum depths of approximately 75 feet below the ground surface. Bedrock was reportedly encountered in all five borings at depths ranging from 51 to 70 feet below the ground surface. Groundwater was encountered at depths ranging from 21 to 25 feet below the ground surface.

Within the report by GeoSoils, and additional 2 borings performed by Van Beveren and Butelo, Inc. (VBB) are referenced. These two borings were also excavated using a hollow-stem auger drilling machine to maximum depths of approximately 71 feet below the ground surface. Bedrock was reportedly encountered at depths of 48 and 49 feet below the ground surface. Groundwater was encountered at depths ranging from 17½ and 25 feet below the ground surface. The boring logs also indicate that artificial fill is present at both boring locations to a depth of approximate 16½ feet.

Geocon West, Inc. has reviewed the boring logs from the referenced GeoSoils and VBB reports and we assume responsibility for the utilization of the boring logs only, including the indicated depths to bedrock. Copies of the boring logs are provided in Appendix C. The recommendations presented herein are based on our own subsurface investigation, data and analyses.

4. GEOLOGIC SETTING

The site is located in the southern portion of the San Fernando Valley, an alluvial-filled basin approximately 23 miles wide and 12 miles long. The alluvium within the San Fernando Valley is 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, and locally from the Los Angeles River (Hitchcock and Wills, 2000).

The site is located near the base of the northern flank of the Santa Monica Mountains and the Los Angeles River is located adjacent to the northern site boundary. The surficial sediments underlying the site were derived primarily from the local drainages in the nearby Santa Monica Mountains, in-place weathering of the underlying sedimentary bedrock, and the ancestral Los Angeles River (prior to channelization).

5. SOIL AND GEOLOGIC CONDITIONS

Based on our field investigation and published geologic maps of the area, the site is underlain by artificial fill, Holocene-age alluvium and Miocene age sedimentary bedrock of the Modelo Formation that consists primarily of siltstone (Dibblee, 1991; Hoots, 1930). Detailed stratigraphic profiles are provided on the boring logs in Appendix A.

5.1 Artificial Fill

Artificial fill was encountered in our explorations to a maximum depth of 4 feet below existing ground surface. The artificial fill generally consists of light brown to dark brown silty sand and clayey silt. The artificial fill is characterized as dry to moist and loose or soft to stiff. 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.

5.2 Alluvium

Holocene age alluvial deposits were encountered beneath the fill. The alluvium consists of brown to dark brown, and olive brown to olive gray interbedded silty clay, sandy clay, clayey silt, sandy silt, silty sand and poorly graded sand. The sand is predominately fine- to medium-grained. The alluvium is characterized as dry to wet and soft to hard or medium dense to very dense.

5.3 Modelo Formation

The alluvial soils are underlain by sedimentary bedrock of the Miocene age Modelo Formation (Dibblee, 1991; Hoots, 1930). Bedrock was encountered in Geocon borings B1 and B4 at depths of 52½ and 60 feet below the ground surface, respectively. The bedrock is predominantly olive brown and gray to dark gray, siltstone characterized as massive to poorly bedded and highly weathered.

Bedrock was also reportedly encountered in the prior borings performed by GeoSoils and VBB (see Section 3.0). We have included the indicated depths to bedrock on the Site Plan (see Figure 2A). These depths to bedrock are for reference purposes only and may represent soft, highly-weathered bedrock. The depth to hard, less weathered bedrock may be deeper than the indicated contact depths.

6. GROUNDWATER

Review of the Seismic Hazard Zone Report for the Van Nuys Quadrangle (California Division of Mines and Geology [CDMG], 1997) indicates the historically highest groundwater level in the area is approximately 5 to 10 feet beneath the ground surface. Groundwater information presented in this document is generated from data collected in the early 1900's to the late 1990s. Based on current groundwater basin management practices, it is unlikely that groundwater levels will ever exceed the historic high levels.

Groundwater was encountered in our field explorations at depths between approximately 22 and 24 feet below the existing ground surface, and in prior site exploration at depths ranging from 17½ to 25 feet. Based on the reported historic high groundwater levels in the site vicinity (CDMG, 1998), the presence of groundwater in our borings, and the subterranean nature of the proposed development, groundwater may have an impact on the proposed structure. In addition, it is common 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 8.26).

7. GEOLOGIC HAZARDS

7.1 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 located in state-designated Alquist-Priolo Earthquake Fault Zone (CGS, 2021a) or a city-designated Preliminary Fault Rupture Study Area (City of Los Angeles, 2021) 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 active fault to the site is the Hollywood Fault located approximately 3.9 miles to the southeast (CGS, 2014). Other nearby active faults are the Santa Monica Fault, the Verdugo Fault, the Newport-Inglewood Fault Zone, and the Raymond Fault located approximately 5.0 miles south, 5.8 miles northeast, 6.4 miles south, and 10.4 miles east-southeast, respectively (Ziony and Jones, 1989; USGS, 2006). The active San Andreas Fault Zone is located approximately 34 miles northeast of the site (USGS, 2006).

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 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.

7.2 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 table on the following page.

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	67	E
Long Beach	March 10, 1933	6.4	44	SE
Tehachapi	July 21, 1952	7.5	68	NW
San Fernando	February 9, 1971	6.6	18	N
Whittier Narrows	October 1, 1987	5.9	20	ESE
Sierra Madre	June 28, 1991	5.8	25	ENE
Landers	June 28, 1992	7.3	113	E
Big Bear	June 28, 1992	6.4	91	E
Northridge	January 17, 1994	6.7	9	WNW
Hector Mine	October 16, 1999	7.1	126	ENE
Ridgecrest	July 5, 2019	7.1	121	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.

7.3 Site-Specific Ground Motion Hazard Analysis

A site-specific ground motion hazard analyses was performed in accordance with ASCE 7-16 Chapter 21 and Section 1613 of the 2019 CBC using the online applications developed by USGS.

7.3.1 Probabilistic Seismic Hazard Analysis

The risk-targeted Maximum Considered Earthquake (MCE_R) probabilistic response spectrum consists of the spectral response accelerations which are expected to achieve a 1 percent probability of collapse within a 50-year period, evaluated at 5 percent damping.

The mean spectral response accelerations having a 2 percent chance of exceedance in 50 years were evaluated at 5 percent damping using the USGS Unified Hazard Tool (UHT). The Dynamic U.S. 2014 (v4.2.0) edition was used within the analysis, which is based on the UCERF-3 fault model. The soil underlying the site was modeled with a shear wave velocity (V_{S30}) of 180 meters per second. The site class definition is based on Standard Penetration Test blow count data.

The web application uses the ground motion prediction equations (GMPEs) from the NGA-West 2 project: Abrahamson-et al. (2014) NGA West 2, Boore et al. (2014) NGA West 2, Campbell-Bozorgnia (2014) NGA West 2, and Chiou-Youngs (2014) NGA West 2. Each GMPE was assigned an equal weight and the mean value of the four GMPEs was evaluated. The mean spectral accelerations were rotated to maximum direction using the period specific ratios from Shahi et al. (2013 & 2014).

The GMPE of Campbell and Borzorgnia requires that the depth to where the shear wave velocity reaches 2.5 kilometers per second ($Z_{2.5}$) be defined. Additionally, the GMPEs of Abrahamson-et al., Boore et al. and Chiou-Youngs require that the depth to where the shear wave velocity reaches 1 kilometer per second ($Z_{1.0}$) be defined. The values of $Z_{2.5}$and $Z_{1.0}$ are internally calculated by the Uniform Hazard Tool.

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

The risk-targeted Maximum Considered Earthquake (MCE_R) probabilistic response spectrum is provided on Figure 5.

7.3.2 Deterministic Seismic Hazard Analysis

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

The earthquake magnitudes of the deterministic scenario events were based on the BSSC 2014 Scenario Event which includes the parent fault identified in the deaggregation and which has the largest earthquake magnitude. Other fault source parameters were defined by the values in the BSSC2014 Scenario Catalog. The values of $Z_{2.5}$ and $Z_{1.0}$ were estimated using data from the Community Velocity Model (CVM) Version 4, Iteration 26, Basin Depth developed by Southern California Earthquake Data Center (SCEDC) accessed by the OpenSHA Site Data Application (v1.5.0).

Six deterministic scenario events were considered for this analysis and consisted of a magnitude 6.7 event occurring on the Hollywood fault; a magnitude 6.78 event occurring on the Santa Monica fault; a magnitude 6.89 event occurring on the Northridge fault; a magnitude 7.45 event occurring on the Compton fault; a magnitude 7.15 event occurring on the Newport-Inglewood fault; and a magnitude 8.18 event occurring on the Southern San Andreas fault.

The deterministic median and standard deviation (sigma) for the scenario events were evaluated using the USGS NSHMP-HAZ-WS Response Spectra online application. The deterministic analysis used the same four GMPEs, equally weighted, to generate the median and standard deviation of the ground motion which were then used to calculate the 84th percentile at 5% damping. The median spectral accelerations were rotated to maximum direction using the period specific ratios from Shahi et al. (2013 & 2014).

The deterministic scenarios were compared, and the event occurring on the Compton fault is considered the controlling deterministic event.

The 84th percentile maximum rotated component deterministic response spectra is provided on Figure 6.

7.3.3 Site-Specific Response Spectrum

The lesser of the probabilistic and deterministic MCE_R response spectrums is the Site-Specific MCE_R. Two thirds of the Site-Specific MCE_R is the Design Earthquake (DE) Response Spectrum, provided the results are not less than 80 percent of the modified General Design Response Spectrum determined by ASCE 7-16 Section 11.4.6 with F_a and F_v determined as specified in Section 21.3.

Graphical representations of the analyses are presented on Figures 5 and 6. The Site-Specific Design Earthquake response spectrum at 5 percent damping is presented on Figure 6 and in tabular form on Figure 7.

7.3.4 Site-Specific Seismic Design Criteria

Based the site-specific ground motion hazard analysis performed, and in accordance with the ASCE 7-16 Section 21.4, site-specific design acceleration parameters shall be derived using the results of the site-specific ground motion hazard analysis.

The parameter S_{DS} shall be taken as equal to 90 percent of the maximum spectral acceleration obtained from the site-specific analysis at any period within the range from 0.2 to 5 seconds, inclusive. The parameter S_{D1} shall be taken as the maximum value of the product of the spectral acceleration and period for periods from 1 to 5 seconds, inclusive. The values of S_{MS} and S_{M1} shall be taken as 1.5 times the site-specific values of S_{DS} and S_{D1}. The site-specific design acceleration parameters shall not be less than 80 percent of the general seismic design values determined by ASCE 7-16 Section 11.4.

The following table presents the site-specific seismic design parameters based on the site-specific ground motion hazard analysis.

SITE-SPECIFIC DESIGN ACCELERATION PARAMETERS

Parameter	Value
Site Class Modified MCE_R Spectral Response Acceleration (short), S_{MS}	1.991g
Site Class Modified MCE_R Spectral Response Acceleration – (1 sec), S_{M1}	2.487g
5% Damped Design Spectral Response Acceleration (short), S_{DS}	1.328g
5% Damped Design Spectral Response Acceleration (1 sec), S_{D1}	1.658g

7.3.5 Site-Specific Peak Ground Acceleration

The site-specific Maximum Considered Earthquake (MCE_G) geometric mean peak ground acceleration was evaluated in accordance with ASCE 7-16 Section 21.5. The significant difference between the MCE_G peak ground acceleration and the analysis presented above is that the MCE_G is calculated without the risk-targeted adjustment factors.

The probabilistic and deterministic 84th percentile peak ground accelerations were analyzed using the same approaches as described above. The analysis used the same Site Class and scenario earthquake. However, within the probabilistic calculation, the risk-targeted adjustment factor was not applied.

The deterministic MCE_G shall not be less than $0.5F_{PGA}$, where F_{PGA} is determined from ASCE 7-16 Table 11.8-1 with the value of PGA taken as 0.5g. The site-specific MCE_G peak ground acceleration is taken as the lesser of the probabilistic and deterministic MCE_G, provided the value is not less than 80 percent of the value of PGA_M as determined by ASCE 7-16 Equation 11.8.1.

ASCE 7-16 SITE-SPECIFIC PEAK GROUND ACCELERATION

Parameter	Value	ASCE 7-16 Reference
Site-Specific MCE_G Peak Ground Acceleration, PGA_M	0.756g	Section 21.5

7.3.6 Deaggregated Parameters

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

Deaggregation of the MCE peak ground acceleration was performed using the USGS online Unified Hazard Tool, 2014 Conterminous U.S. Dynamic edition (v4.2.0). The result of the deaggregation analysis indicates that the predominant earthquake contributing to the MCE peak ground acceleration is characterized as a 6.82 magnitude event occurring at a hypocentral distance of 11.62 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 6.68 magnitude occurring at a hypocentral distance of 14.86 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.

7.4 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 Van Nuys Quadrangle (CDMG, 1998,) indicates that the site is located within an area designated as having a potential for liquefaction. In addition, a review of the County of Los Angeles Seismic Safety Element (Leighton, 1990) indicates that the site is located within an area identified as having a potential for liquefaction.

Liquefaction analysis of the soils underlying the site was performed using an updated version of the spreadsheet template LIQ2_30.WQ1 developed by Thomas F. Blake (1996). This program utilizes the 1996 NCEER method of analysis. This semi-empirical method is based on a correlation between values of Standard Penetration Test (SPT) resistance and field performance data.

Screening criteria presented by Bray and Sancio (2006) was used to evaluate the liquefaction susceptibility of the fine-grained soils encountered in the boring. Based on these screening criteria, fine-grained soils with a plasticity index of greater than 18 and fine-grained soils with a plasticity index of greater than 12 and a saturated water content of less than 80 percent of the liquid limit are considered not susceptible to liquefaction. Laboratory test results used for the screening criteria are presented as Figures B33 and B34.

The liquefaction analysis was performed for a Design Earthquake level by using a historic high groundwater table of 10 feet below the ground surface, a magnitude 6.68 earthquake, and a peak horizontal acceleration of 0.504g ($\frac{2}{3}PGA_M$). The enclosed liquefaction analyses, included herein for boring B1, indicate that the alluvial soils below one level of subterranean construction could be susceptible to up to 2.4 inches of liquefaction settlement during Design Earthquake ground motion. Furthermore, the alluvial soils below three levels of subterranean construction would not be prone to liquefaction induced settlement during Deign Earthquake ground motion. Calculation sheets are provided herein as Figures 8 through 11.

It is our understanding that the intent of the Building Code is to maintain “Life Safety” during Maximum Considered Earthquake level events. Therefore, additional analysis was performed to evaluate the potential for liquefaction during a MCE event. The structural engineer should evaluate the proposed structure for the anticipated MCE liquefaction induced settlements and verify that anticipated deformations would not cause the foundation system to lose the ability to support the gravity loads and/or cause collapse of the structure.

The liquefaction analysis was also performed for the Maximum Considered Earthquake level by using a groundwater table of 10 feet below the ground surface, a magnitude 6.82 earthquake, and a peak horizontal acceleration of 0.756g (PGA_M). The enclosed liquefaction analyses, included herein for boring B1, indicate that the alluvial soils below one level of subterranean construction could be susceptible to up to 2.4 inches of liquefaction settlement during Maximum Considered Earthquake ground motion. Furthermore, the alluvial soils below three levels of subterranean construction would not be prone to liquefaction induced settlement during Maximum Considered Earthquake ground motion. Calculation sheets are provided herein as Figures 12 through 15.

7.5 Lateral Spread

The site is bounded to the north by the Los Angeles River, which adjacent to the site has been channelized. Based on the absence of unsupported soil slopes, the potential for lateral spreading is considered to be low.

7.6 Slope Stability

The topography at the site slopes gently to north. The site is not located within a City of Los Angeles Hillside Grading Area or a Hillside Ordinance Area (City of Los Angeles, 2021). Additionally, the site is not located within an area identified as having a potential for seismic slope instability (CDMG, 1998). 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.

7.7 Earthquake-Induced Flooding

Earthquake-induced flooding is inundation caused by failure of dams or other water-retaining structures due to earthquakes. The County of Los Angeles (Leighton, 1990) indicates that the site is located within the Sepulveda Dam, Lopez Dam, Hansen Dam, and Los Angeles Dam inundation areas. However, these reservoirs, 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

7.8 Tsunamis, Seiches, and Flooding

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

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

The site is within an area of minimal flooding (Zone X) as defined by the Federal Emergency Management Agency (FEMA, 2021; LACDPW, 2021). Therefore, flooding is not anticipated to adversely impact the site.

7.9 Oil Fields & Methane Potential

Review of the California Geologic Energy Management Division (CalGEM) Well Finder Website indicates that the site is not located within an oil field and oil or gas wells are not documented in the immediate site vicinity (CalGEM, 2021). 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.

The site is not located within the boundaries of a city-designated Methane Zone or Methane Buffer Zone (City of Los Angeles, 2021). 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.

7.10 Subsidence

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

8. CONCLUSIONS AND RECOMMENDATIONS

8.1 General

8.1.1 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. The geotechnical design parameters presented herein should be reviewed and updated as the design progresses and as the structural loads become finalized.

8.1.2 Up 4 feet of artificial fill was encountered during site exploration. 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. It is our opinion that the existing fill, in its present condition, is not suitable for direct support of proposed foundations, slabs, or additional fill. 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 8.5). Excavations for the subterranean portions of the structures are anticipated to penetrate through the existing artificial fill and expose undisturbed alluvial soils throughout the excavation bottoms.

8.1.3 The enclosed seismically induced settlement analyses indicate that the alluvial soils below one level of subterranean construction could be prone to approximately 2.4 inches of settlement as a result of the Design Earthquake peak ground acceleration ($\frac{2}{3}PGA_M$). The resulting differential settlement at the ground surface is anticipated to be approximately half of the total settlement, or 1.2 inches of settlement over a distance of 20 feet. The grading and foundation recommendations presented herein are intended to reduce the effects of settlement on proposed improvements.

8.1.4 Groundwater has been encountered at depths ranging from 17½ feet to 25 feet below the existing ground surface. Excavation for the proposed three-level subterranean parking is anticipated to extend to depths up to approximately 38 feet below the ground surface, including foundation construction and dewatering elements. Due to the depth of the proposed excavation and the groundwater level, temporary dewatering measures will be required to mitigate groundwater during excavation and construction. Furthermore, groundwater will likely be encountered during deep drilled excavations, such as shoring piles and/or an elevator piston. Recommendations for temporary dewatering are discussed in Section 8.4 of this report.

8.1.5 The historically high groundwater level beneath the site is approximately 10 feet below the existing ground surface, and the proposed structure must be designed for hydrostatic pressure based on this groundwater level. The hydrostatic design will result in uplift forces on the structure that must be resisted by counterweight or structural design measures. The recommended floor slab uplift pressure to be used in design would be 62.4(H) in units of pounds per square foot (psf), where “H” is the height of the water above the bottom of the foundation in feet.

8.1.6 The upper 40 feet of alluvial soils consist of very soft to soft silts and clays with varying amounts of sand. Based on laboratory testing (see Figures B18 to B32), the upper alluvium is moderately to highly compressible. Based on the presence of compressible soils, as well as the potential for soils susceptible to liquefaction immediately underlying the area with one level of subterranean construction, the use of a conventional foundation system or a mat foundation system is not considered feasible for a single subterranean level. Mitigation of these soils could be achieved through excavation and placement of engineered fill; however, this would require deep shoring and dewatering, which is likely not the most economical option.

8.1.7 Based on these considerations, it is recommended that the portion of the proposed structure with one subterranean level be supported on Auger-Cast Pressure Grouted Displacement (APGD) piles which penetrate through the existing artificial fill and soft, compressible and liquefiable soils to derive support in competent bedrock found at or below a depth of 65 feet. The client should be aware that the City of Los Angeles will require a comprehensive load testing program Recommendations for the design of APGD piles are provided in Section 8.8 of this report.

8.1.8 The concrete slab for a pile-supported structure should be designed as a structural slab that derives all support from the pile, eliminating permanent reliance on the underlying soils. As a minimum, it is recommended that the upper 12 inches of slab subgrade be compacted to provide a suitable temporary surface upon which concrete can be poured and placed. Any disturbed soils should be properly compacted prior to slab construction.

8.1.9 Based on these considerations, it is recommended that the portion of the proposed structure with three subterranean levels be supported on a reinforced concrete mat foundation system deriving support in the undisturbed alluvial soils found at or below a depth of 38 feet. In order to minimize differential settlement, it is recommended that the ramp and ramp walls for the subterranean parking garage be structurally supported on the mat foundation. A mat foundation is more accommodating to subgrade stabilization, waterproofing, and hydrostatic design. Any soils unintentionally disturbed should be properly compacted. All foundation excavations must be observed and approved in writing by the Geotechnical Engineer prior to placement of steel or concrete. Recommendations for the design of a mat foundation system are provided in Section 8.7 of this report.

8.1.10 If small areas of the proposed structure will extend beyond the limits of the subterranean levels to be constructed on-grade, it is recommended that these areas be designed as structurally cantilevered from the main structure (thus eliminating permanent reliance on the underlying soils) or supported on deep foundations in order to minimize differential settlements. These areas, if any, can be further addressed as the design progresses.

8.1.11 The alluvial soils anticipated to be exposed at the excavation bottom will likely be very moist and could be subject to excessive pumping. Operation of rubber tire equipment on the subgrade soils may cause excessive disturbance of the soils. Excavation activities to establish the finished subgrade elevation must be conducted carefully and methodically to avoid excessive disturbance to the subgrade. Stabilization of the bottom of the excavation will likely be required in order to provide a firm working surface upon which heavy equipment can operate. Recommendations for bottom stabilization and earthwork are provided in the *Grading* section of this report (see Section 8.5).

8.1.12 Excavations up to 40 feet in vertical height are anticipated for construction of the subterranean parking levels, including foundation depths and temporary dewatering system. Due to the depth of the excavation and the proximity to the property lines, city streets and adjacent offsite structures, excavation of the subterranean levels will likely require sloping and/or 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 a structure, the proposed shoring should be designed to resist the surcharge imposed by the adjacent structure. Recommendations for *Temporary Excavations* are provided in Section 8.21 of this report.

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

8.1.14 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 at and below a depth of 24 inches, and should be deepened as necessary to maintain a minimum 12-inch embedment into the recommended bearing materials. If the soils exposed in the excavation bottom are soft or loose, compaction of the soils will be required prior to placing steel or concrete. Compaction of the foundation excavation bottom is typically accomplished with a compaction wheel or mechanical whacker and must be observed and approved by a Geocon representative.

8.1.15 Where new paving is to be placed, it is recommended that all existing fill and soft soils be excavated and properly compacted for paving support. The client should be aware that excavation and compaction of all existing fill and soft soils in the area of new paving is not required; however, paving constructed over existing uncertified fill may experience increased settlement and/or cracking, and may therefore have a shorter design life and increased maintenance costs. As a minimum, the upper 12 inches of subgrade soil should be scarified and properly compacted for paving support. Paving recommendations are provided in *Preliminary Pavement Recommendations* section of this report (see Section 8.13).

8.1.16 In accordance with City of Los Angeles Information Bulletin P/BC 2020-118, stormwater infiltration is not allowed for this project. It is recommended that stormwater be retained, filtered, and discharged in accordance with the requirements of the local governing agency.

8.1.17 Once the design and foundation loading configuration for the proposed structure 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.

8.1.18 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.

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

8.2 Soil and Excavation Characteristics

8.2.1 The in-situ soils can be excavated with moderate effort using conventional excavation equipment. Some caving should be anticipated in vertical excavations, especially where granular or saturated soils are encountered. Excavations for drilled piles which extend into bedrock may encounter concretions and well cemented layers which could make drilling conditions difficult. The contractor should be prepared for these conditions and should have coring equipment readily available.

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

8.2.3 All onsite excavations must be conducted in such a manner that potential surcharges from existing structures, construction equipment, and vehicle loads are resisted. The surcharge area may be defined by a 1:1 projection down and away from the bottom of an existing foundation or vehicle load. Penetrations below this 1:1 projection will require special excavation measures such as sloping and possibly shoring. Excavation recommendations are provided in the *Temporary Excavations* section of this report (see Section 8.21).

8.2.4 The existing site soils encountered between depths of 10 feet to 15 feet during this investigation are considered to have a “high” expansive potential (EI = 102); and the soils are classified as “expansive” based on the 2019 California Building Code (CBC) Section 1803.5.3. Recommendations presented herein assume that on-grade foundations and slabs will derive support in materials with a “high” expansion potential.

8.2.5 Based on depth of the proposed subterranean levels, the foundations and slabs at the subterranean levels would not be prone to the effects of expansive soils.

8.3 Minimum Resistivity, pH and Water-Soluble Sulfate

8.3.1 Potential of Hydrogen (pH) and resistivity testing as well as chloride content testing were previously 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 "severely corrosive" with respect to corrosion of buried ferrous metals on site. The results are presented in Appendix B (Figure B39) and should be considered for design of underground structures. Due to the corrosive potential of the soils, it is recommended that PVC, ABS or equivalent plastic piping be considered in lieu of cast-iron for subdrains and retaining wall drains in direct contact with the site soils.

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

8.3.3 Geocon West, Inc. does not practice in the field of corrosion engineering. 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 on buried metal pipes and concrete structures in direct contact with the soils.

8.4 Temporary Dewatering

8.4.1 Groundwater has been encountered as shallow as 17½ feet during prior explorations at the site. Based on the conditions encountered at the time of exploration, groundwater is anticipated to be encountered during construction activities. The depth to groundwater at the time of construction can be further be verified during the installation of the initial dewatering well or shoring pile. If groundwater is present above the depth of the proposed foundation excavation, temporary dewatering will be necessary to maintain a safe working environment during excavation and construction activities.

8.4.2 It is recommended that a qualified dewatering consultant be retained to design the dewatering system and to determine the design flow rates for dewatering. Temporary dewatering may consist of perimeter wells with interior well points as well as gravel filled trenches (French drains) placed adjacent to the shoring system and interior of the site. The number and locations of the wells or French drains can be adjusted during excavation activities as necessary to collect and control any encountered seepage. The French drains will then direct the collected seepage to a sump where it will be pumped out of the excavation.

8.4.3 The embedment of perimeter shoring piles should be deepened as necessary to take into account any required excavations necessary to place an adjacent French drain system, or sub-slab drainage system, should it be deemed necessary. It is not anticipated that a perimeter French drain will be more than 24 inches in depth below the proposed excavation bottom. If a French drain is to remain on a permanent basis, it must be lined with filter fabric to prevent soil migration into the gravel.

8.5 Grading

8.5.1 Grading is anticipated to include excavation of site soils for the subterranean levels, foundations, and utility trenches, as well as placement of backfill for walls and trenches.

8.5.2 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.

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

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

8.5.5 The portion of the proposed structure with three subterranean levels may be supported on a reinforced concrete mat foundation system deriving support in undisturbed alluvial soils found at and below a depth of 38 feet. Foundations should be deepened as necessary to extend into satisfactory soils and must be observed and approved in writing by the Geotechnical Engineer (a representative of Geocon West, Inc.).

8.5.6 The portion of the proposed structure with one subterranean level may be supported on deepened foundations consisting of Auger-Cast Pressure Grouted Displacement (APGD) piles which penetrate through the existing artificial fill and derive support in the competent bedrock found below at or below a depth of 65 feet. Recommendations for the design of APGD piles are provided in Section 8.8 of this report.

8.5.7 The concrete slab for a pile-supported structure penetrating through uncertified artificial fill and compressible alluvium should be designed as a structural slab that derives all support from the piles, eliminating permanent reliance on the underlying soils. As a minimum, it is recommended that the upper 12 inches of slab subgrade be compacted to provide a suitable temporary surface upon which concrete can be poured and placed. Any disturbed soils should be properly compacted prior to slab construction

8.5.8 Due to the potential for high-moisture content soils at the excavation bottom, or if construction is performed during the rainy season and the excavation bottom becomes saturated, stabilization measures may have to be implemented to prevent excessive disturbance the excavation bottom. Should this condition exist, rubber tire equipment should not be allowed in the excavation bottom until it is stabilized or extensive soil disturbance could result.

8.5.9 One method of subgrade stabilization would consist of introducing a thin lift of 3 to 6-inch diameter crushed angular rock into the soft excavation bottom. The use of crushed concrete will also be acceptable. The crushed rock should be spread thinly across the excavation bottom and pressed into the soils by track rolling or wheel rolling with heavy equipment. It is very important that voids between the rock fragments are not created so the rock must be thoroughly pressed or blended into the soils. All subgrade soils must be properly compacted and proof-rolled in the presence of the Geotechnical Engineer (a representative of Geocon West, Inc.).

8.5.10 Subgrade stabilization may also be accomplished by placing a 1-foot-thick layer of washed, angular 3/4-inch gravel atop a stabilization fabric (Mirafi 500X or equivalent), subsequent to subgrade approval. This gravel placement procedure should be conducted in sections until the entire excavation bottom has been blanketed by fabric and gravel. Heavy equipment may operate upon the gravel once it has been placed. The gravel should be compacted to a dense state utilizing a vibratory drum roller. The placement of gravel at the subgrade level should be coordinated with the temporary or permanent dewatering of the site. The gravel and fabric system will function as both a permeable material for any necessary dewatering procedures as well as a stable material upon which heavy equipment may operate. It is recommended that the contractor meet with the Geotechnical Engineer to discuss this procedure in more detail.

8.5.11 All fill and backfill soils should be placed in horizontal loose layers approximately 6 to 8 inches thick, moisture conditioned to approximately 2 percent above optimum moisture content, and properly compacted to a minimum of 90 percent of their maximum dry density in accordance with ASTM D 1557 (latest edition).

8.5.12 Where new exterior concrete slabs-on-grade are to be constructed, the upper twelve inches of soil should be scarified, moisture conditioned to 2 percent above optimum moisture content, and compacted to at least 95 percent relative compaction, as determined by ASTM Test Method D 1557 (latest edition).

8.5.13 Where new paving is to be placed, it is recommended that all existing fill and disturbed alluvium be excavated and properly compacted for paving support. The client should be aware that excavation and compaction of all existing fill and soft soils in the area of new paving is not required; however, paving constructed over existing fill or unsuitable alluvium 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 soil should be scarified, moisture conditioned to near optimum moisture content, and compacted to at least 95 percent relative compaction for paving support. Paving recommendations are provided in *Preliminary Pavement Recommendations* section of this report (see Section 8.13).

8.5.14 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. If gravel is used for trench bedding and shading (typical when seepage is present), it must be 3/16-inch rounded birds-eye rock in accordance with the City of LA plumbing department requirements. The remainder of the trench backfill may be derived from onsite soil or approved import soil, compacted as necessary, until the required compaction is obtained. The use of minimum 2-sack slurry is also acceptable (Section 8.6). 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).

8.5.15 Although not anticipated for this project, all imported fill shall be observed, tested, and approved by Geocon West, Inc. prior to bringing soil to the site. Rocks larger than 6 inches in diameter shall not be used in the fill. If necessary, import soils used as structural fill should have an expansion index less than 40 and corrosivity properties that are equally or less detrimental to that of the existing onsite soils (see Figure B39).

8.5.16 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.

8.6 Controlled Low Strength Material (CLSM)

8.6.1 Controlled Low Strength Material (CLSM) may be utilized in lieu of compacted soil as engineered fill where approved in writing by the Geotechnical Engineer. Where utilized within the City of Los Angeles use of CLSM is subject to the following requirements:

Standard Requirements

1. CLSM shall be ready-mixed by a City of Los Angeles approved batch plant;
2. CLSM shall not be placed on uncertified fill, on incompetent natural soil, nor below water;
3. CLSM shall not be placed on a sloping surface with a gradient steeper than 5:1 (horizontal to vertical);
4. Placement of the CLSM shall be under the continuous inspection of a concrete deputy inspector;
5. The excavation bottom shall be accepted by the soil engineer and the City Inspector prior to placing CLSM.

Requirements for CLSM that will be used for support of footings

1. The cement content of the CLSM shall not be less than 188 pounds per cubic yard (min. 2 sacks);
2. The excavation bottom must be level, cleaned of loose soils and approved in writing by Geocon prior to placement of the CLSM;
3. The ultimate compressive strength of the CLSM shall be no less than 100 pounds per square inch (psi) when tested on the 28th-day per ASTM D4832 (latest edition), Standard Test Method for Preparation and Testing of Controlled Low Strength Material Test Cylinders. Compression testing will be performed in accordance with ASTM C39 and City of Los Angeles requirements;
4. Samples of the CLSM will be collected during placement, a minimum of one test (two cylinders) for each 50 cubic yards or fraction thereof;
5. Overexcavation for CLSM placement shall extend laterally beyond the footprint of any proposed footings as required for placement of compacted fill, unless justified otherwise by the soil engineer that footings will have adequate vertical and horizontal bearing capacity.

8.7 Mat Foundation Design

8.7.1 It is recommended that a reinforced concrete mat foundation be utilized for support of the portion of the proposed structure with three subterranean levels. The reinforced concrete mat foundation may derive support in the undisturbed alluvial soils found at or below a depth of 38 feet below ground surface. Foundations should be deepened as necessary to extend into satisfactory soils and must be observed and approved in writing by the Geotechnical Engineer (a representative of Geocon West, Inc.).

8.7.2 The recommended maximum allowable bearing value is 4,000 pounds per square foot (psf). The allowable bearing pressure may be increased by up to one-third for transient loads due to wind or seismic forces.

8.7.3 It is recommended that a modulus of subgrade reaction of 75 pounds per cubic inch (pci) be utilized for the design of the mat foundation bearing in the undisturbed alluvial soils found at or below a depth of 38 feet. These values are unit values for use with a 1-foot square footing. The modulus should be reduced in accordance with the following equation when used with larger foundations:

$$K_R = K\left[\frac{B+1}{2B}\right]^2$$

where: K_R = reduced subgrade modulus
K = unit subgrade modulus
B = foundation width (in feet)

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

8.7.5 The thickness of and reinforcement for the mat foundation should be designed by the project structural engineer.

8.7.6 The proposed structure must be designed for hydrostatic pressure based on the groundwater level. The hydrostatic design will result in uplift forces on the structure that must be resisted by counterweight or structural design measures. The recommended floor slab uplift pressure to be used in design would be 62.4(H) in units of pounds per square foot (psf), where “H” is the height of the water above the bottom of the foundation in feet. For design purposes the groundwater table should be assumed to be at a depth of 10 feet below the ground surface. Considerations for uplift resistance are provided in Section 8.9 of this report.

8.7.7 For seismic design purposes, a coefficient of friction of 0.30 may be utilized between the concrete mat and subgrade soils without a moisture barrier, and 0.15 for slabs underlain by a moisture barrier.

8.7.8 The maximum settlement for a reinforced concrete mat foundation with a maximum allowable bearing pressure of 4,000 psf deriving support in the recommended bearing materials is expected to be less than ½ inch and occur below the heaviest loaded structural element. Most of the settlement of the foundation system is expected to occur on initial application of loading; however, some additional settlements should be expected within the first 12 months. Differential settlement is expected to be less than ½ inch between the center and corner of the mat foundation. The anticipated static settlements, including the effects of temporary dewatering, should be further verified once the design phase proceeds to a more finalized plan.

8.7.9 Differential settlement across the stepped transitions between the various building levels could be on the order of ½ inch and will likely require a heavily reinforced structural connection, or a structural separation to account for the anticipated differential movements.

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

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

8.7.12 Waterproofing of subterranean walls and slabs is recommended for this project. 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.

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

8.8 Auger-Cast Displacement Piles

8.8.1 Auger-cast pressure grouted displacement (APGD) piles are installed by advancing a hollow-stem auger with a diameter equivalent to that of the pile to the desired pile tip elevation. The specialized hollow-stem auger bit displaces the penetrated soils laterally away from the auger as it is advanced, creating increased pile capacity and minimizing the amount of soil spoils. Once the desired pile tip elevation is achieved, grout is pumped under pressure from the tip of the auger as it is withdrawn and then the pile reinforcing steel is placed in the grout.

8.8.2 The Client should be aware that APGD piles are typically designed and installed by a specialty geotechnical contractor. The recommendations presented herein for the design of APGD piles may be used for preliminary design purposes.

8.8.3 For preliminary design purposes 16 and 18 inch diameter APGD piles have been assumed, and preliminary ultimate pile capacities are provided in the following table. The pile design must also include consideration of downdrag loads of 43 and 49 kips for 16 and 18 inch diameter piles, respectively, due to downdrag from liquefiable soils. These loads are not incorporated into the capacities provided below and should be applied by the Structural Engineer. It is recommended that a factor of safety of at least 2.5 be applied to the ultimate pile capacity.

AUGER-CAST GROUTED DISPLACEMENT PILE CAPACITIES

Embedment below Ground Surface (feet)	16-Inch Diameter Pile Ultimate Capacity (kips)	18-Inch Diameter Pile Ultimate Capacity (kips)
10 feet into the competent bedrock found at and below a depth of 65 feet	430	550

8.8.4 Single pile uplift capacity can be taken as 50 percent of the allowable downward capacity.

8.8.5 The axial capacity of the APGD piles should be verified by the design-build contractor and confirmed based upon pile load testing. Geocon should review, and if necessary, can assist the design-build contractor in developing a suitable testing program. During pile load testing, a representative of Geocon should be present to observe pile installation and testing. The information obtained from the pile load testing should be used to evaluate the need to modify pile lengths to achieve design capacities, as well as develop installation criteria that can be used during construction of production piles.

8.8.6 It is recommended that at least two pre-production piles or one percent of the production pile quantity be constructed, and load tested to at least 200 percent of the design load. Additional information on the indicator pile test program are provided in Appendix D.

8.8.7 Proof testing of production piles should also be performed by the design-build contractor and verified by the Geotechnical Engineer. It is recommended that at least 5 percent of production piles be constructed, and load tested to at least 160 percent of the design load. In addition, Thermal Integrity Profiling will be required for all preproduction piles and for 10 percent of the production piles. The testing program and acceptance criteria should be configured to satisfy the requirements of the building official.

8.8.8 APGD pile construction should be performed under continuous observation of the Geotechnical Engineer (a representative of Geocon) to observe that soil conditions do not differ from those anticipated and to observe that construction of the APGD piles is performed in accordance with the project plans and specifications. Additional specifications for APGD installation are provided in Appendix D.

8.8.9 If piles are spaced at least at least 3 diameters on center, no reduction in axial capacity is considered necessary for group effects. If pile spacing is closer than three pile diameters, an evaluation for group effects including appropriate reductions should be incorporated into the pile design based on pile dimension, spacing, and the direction of loading.

8.8.10 For increased resistance to differential foundation movement and lateral drift, the pile tops should be interconnected in two horizontal directions with grade beams or tied with a structural slab / mat slab. The project structural engineer should provide slab and grade beam design, reinforcement and spacing dependent on anticipated loading. However, for grade beams we recommend a minimum embedment depth below lowest adjacent pad grade of 24 inches and a minimum width of 12 inches. In addition, minimum reinforcement should consist of four No. 5 steel reinforcing bars; two placed near the top of the grade beam and two near the bottom.

8.8.11 APGD piles should be designed based on settlement criteria of a maximum combined static and seismic differential settlement of ½ inch between adjacent columns.

8.8.12 The design of the structural connection between adjacent structures is at the discretion of the project structural engineer and should take into account potential differential settlements between structures.

8.8.13 It is recommended that flexible utility connections be utilized for all rigid utilities to minimize or prevent damage to utilities from minor differential movements.

8.9 Uplift Resistance

8.9.1 Foundation uplift may be resisted by the weight of structure, as well as by friction along the sides of foundations. If additional uplift resistance is required, the perimeter shoring piles may be utilized provided the toes of the piles are poured with structural concrete and are designed as permanent piles. Uplift resistance may also be generated by APGD piles constructed within the interior of the structure. As the project progress, an evaluation of the need for uplift resistance should be performed by the project Structural Engineer and additional recommendations can be provided under separate cover.

8.10 Lateral Design

8.10.1 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 0.30 may be used with the dead load forces in newly placed engineered fill, competent alluvial soils, or stabilized subgrade, and 0.15 for slabs underlain by a moisture barrier.

8.10.2 Passive earth pressure for the sides of foundations and slabs poured against newly placed engineered fill and alluvial soils above the groundwater table may be computed as an equivalent fluid having a density of 185 pcf with a maximum earth pressure of 1,850 psf. Passive earth pressure for the sides of foundations and slabs poured against the alluvial soils below the groundwater table may be computed as an equivalent fluid having a density of 105 pcf with a maximum earth pressure of 1,050 psf (these values have been adjusted for buoyant forces). When combining passive and friction for lateral resistance, the passive component should be reduced by one-third.

8.11 Miscellaneous Foundations

8.11.1 Foundations for small outlying structures, such as block walls up to 6 feet in height, planter walls or trash enclosures, which will not be structurally supported by 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 compaction cannot be performed, such as adjacent to property lines, foundations may bear in the undisturbed alluvial soils at and below a depth of 24 inches, and should be deepened as necessary to maintain a 12-inch embedment in to the recommended bearing materials.

8.11.2 Miscellaneous foundations may be designed for a bearing value of 1,500 psf and should be a minimum of 12 inches in width, 36 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. 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.

8.11.3 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.

8.12 Concrete Slabs-on-Grade

8.12.1 It is recommended that the concrete slab-on-grade for the pile supported structure be designed as a structural slab deriving support from the deepened foundation system. The thickness and reinforcing of the structural slab should be designed by the project structural engineer. It is recommended that the upper 12 inches of slab subgrade be compacted to provide a suitable surface upon which concrete can be placed. Any soils unintentionally disturbed should be properly compacted prior to slab construction.

8.12.2 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. 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.

8.12.3 For seismic design purposes, a coefficient of friction of 0.30 may be utilized between concrete slabs and subgrade soils without a moisture barrier.

8.12.4 Exterior slabs, 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 2 percent above optimum moisture content and properly compacted to at least 95 percent relative compaction, as determined by ASTM Test Method D 1557 (latest edition). Crack control joints should be spaced at intervals not greater than 10 feet and should be constructed using saw-cuts or other methods as soon as practical following concrete placement. Crack control joints should extend a minimum depth of ¼ the slab thickness. The project structural engineer should design construction joints as necessary.

8.12.5 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.

8.13 Preliminary Pavement Recommendations

8.13.1 Where new paving is to be placed, it is recommended that all existing fill and soft, unsuitable soils be excavated and properly compacted for paving support. The client should be aware that excavation and compaction of all soft, unsuitable soils in the area of new paving is not required, however, paving constructed over existing unsuitable soils may experience increased settlement and/or cracking, and may therefore have a shorter design life and increased maintenance costs. As a minimum, the upper 12 inches of paving subgrade should be scarified, moisture conditioned to 2 percent above optimum moisture content, and properly compacted to at least 95 percent relative compaction, as determined by ASTM Test Method D 1557 (latest edition).

8.13.2 The following pavement sections are based on an assumed R-Value of 20. Once site grading activities are complete, it is recommended that laboratory testing confirm the properties of the soils serving as paving subgrade prior to placing pavement.

8.13.3 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	12.0

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

8.13.5 Unless specifically designed and evaluated by the project structural engineer, where exterior concrete paving will be utilized for support of vehicles, it is recommended that the concrete be a minimum of 6 inches thick and be reinforced with No. 3 steel reinforcing bars placed 18 inches on center in both horizontal directions. Concrete paving supporting vehicular traffic should be underlain by a minimum of 12 inches properly compacted subgrade soil that is compacted to 95 percent relative compaction as determined by ASTM Test Method D 1557 (latest edition).

8.13.6 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.

8.14 Retaining Wall Design

8.14.1 The recommendations presented below are generally applicable to the design of rigid concrete or masonry retaining walls having a maximum height of 45 feet. In the event that walls significantly higher than 45 feet are planned, Geocon should be contacted for additional recommendations.

8.14.2 Retaining walls with a level backfill surface that are not restrained at the top should be designed utilizing a triangular distribution of pressure (active pressure). Restrained walls are those that are not allowed to rotate more than 0.001H (where H equals the height of the retaining portion of the wall in feet) at the top of the wall. Where walls are restrained from movement at the top, walls may be designed utilizing a triangular distribution of pressure (at-rest pressure). The table below presents recommended pressures to be used in retaining wall design, assuming that proper drainage will be maintained. Retaining wall pressure calculations are presented on Figures 16a through 16c.

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 20	36	71
21 to 35	48	71
36 to 45	52	71

8.14.3 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, at-rest walls is 100 pounds per cubic foot (pcf). The value includes hydrostatic pressures plus buoyant lateral earth pressures.

8.14.4 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 8.25 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.

8.14.5 The wall pressures provided above assume that the proposed retaining walls will support relatively undisturbed alluvial soils. If sloping techniques are to be utilized for construction of proposed walls, which would result in a wedge of engineered fill behind the retaining walls, revised earth pressures may be required. This should be evaluated once the use of sloping measures is established and once the geotechnical characteristics of the engineered backfill soils can be evaluated.

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

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

8.15 Dynamic (Seismic) Lateral Forces

8.15.1 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).

8.15.2 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; it is not necessary to add the seismic load to the at-rest pressure. We used the peak site acceleration, PGA_M, of 0.756g calculated from ASCE 7-16 Section 11.8.3 and applied a pseudo-static coefficient of 0.33.

8.16 Retaining Wall Drainage

8.16.1 Where retaining walls are 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 17). 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.

8.16.2 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 18). 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.

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

8.16.4 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.

8.17 Elevator Pit Design

8.17.1 The elevator pit slab and retaining wall should be designed by the project structural engineer. Elevator pit walls may be designed in accordance with the recommendations in the *Retaining Wall Design* section of this report (see Section 8.14).

8.17.2 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.

8.17.3 It is suggested that the exterior walls and slab be waterproofed to prevent excessive moisture inside of the elevator pit. The elevator pit should be waterproofed in accordance with the *Mat Foundation Design* section of this report (see Section 8.7). Waterproofing design and installation is not the responsibility of the geotechnical engineer.

8.18 Elevator Piston

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

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

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

8.19 Temporary Excavations

8.19.1 Excavations on the order of 50 feet in height are anticipated for construction of the proposed subterranean levels, including dewatering system and foundation system. The excavations are expected to expose artificial fill and alluvium, which are suitable for vertical excavations up to 5 feet in height where loose soils or caving sands are not present, and where not surcharged by adjacent traffic or structures.

8.19.2 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 15 feet. A uniform slope does not have a vertical portion. Excavations greater than 15 feet in height will require special excavations measures such as shoring. Recommendations for Shoring are provided in Section 8.20.

8.19.3 Where temporary construction slopes are utilized, the top of the slope should be barricaded to prevent vehicles and storage loads at the top of the slope within a horizontal distance equal to the height of the slope. If the temporary 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.

8.20 Shoring – Soldier Pile Design and Installation

8.20.1 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.

8.20.2 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.

8.20.3 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 subgrade stabilization activities, foundations and/or adjacent drainage systems.

8.20.4 The proposed soldier piles may be utilized to provide a component of uplift resistance. If required to provide uplift resistance, the shoring piles must be designed as permanent piles. The uplift capacity may be taken as ⅔ of the downward frictional capacity.

8.20.5 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 8.14).

8.20.6 Drilled cast-in-place soldier piles should be placed no closer than two 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 110 pounds per square foot per (value has been reduced for buoyancy). Where piles are installed by vibration techniques, the passive pressure may be assumed to mobilize across a width equal to two times the dimension of the beam flange. The allowable passive value may be doubled for isolated piles spaced a minimum of three the pile diameter. To develop the full lateral value, provisions should be implemented to assure firm contact between the soldier piles and the undisturbed soils.

8.20.7 Groundwater was encountered at depths of 22 to 24 feet during our site exploration, and the contractor should be prepared for groundwater during pile installation. Piles placed below the water level require the use of a tremie to place the concrete into the bottom of the hole. If more than 6 inches of water is present in the bottom of the excavation, a tremie is required to place the concrete into the bottom of the hole. A tremie should consist of a rigid, water-tight tube having a diameter of not less than 6 inches with a hopper at the top. The tube should be equipped with a device that will close the discharge end and prevent water from entering the tube while it is being charged with concrete. The tremie should be supported so as to permit free movement of the discharge end over the entire top surface of the work and to permit rapid lowering when necessary to retard or stop the flow of concrete. The discharge end should be closed at the start of the work to prevent water entering the tube and should be entirely sealed at all times, except when the concrete is being placed. The tremie tube should be kept full of concrete. The flow should be continuous until the work is completed and the resulting concrete seal should be monolithic and homogeneous. The tip of the tremie tube should always be kept about 5 feet below the surface of the concrete and definite steps and safeguards should be taken to insure that the tip of the tremie tube is never raised above the surface of the concrete.

8.20.8 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.

8.20.9 Caving is anticipated to occur where granular soils are encountered and the contractor should have casing available prior to commencement of pile excavation. When casing is used, extreme care should be employed so that the pile is not pulled apart as the casing is withdrawn. At no time should the distance between the surface of the concrete and the bottom of the casing be less than 5 feet. Continuous observation of the drilling and pouring of the piles by the Geotechnical Engineer (a representative of Geocon West, Inc.), is required.

8.20.10 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.

8.20.11 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, the bore diameter should be no greater than 75 percent of the largest dimension of the pile to prevent excessive loss in the frictional component of the pile capacity. The depth of the predrilled holes shall not exceed the planned excavation depth and the auger shall be backspun out of the pilot holes, leaving the soils in place.

8.20.12 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.

8.20.13 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.

8.20.14 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.

8.20.15 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.

8.20.16 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.

8.20.17 The frictional resistance between the soldier piles and retained soil may be used to resist the vertical component of the anchor load. The coefficient of friction may be taken as 0.3 based on uniform contact between the steel beam and lean-mix concrete and retained earth. The portion of soldier piles below the plane of excavation may also be employed to resist the downward loads. The downward capacity may be determined using a frictional resistance of 200 psf per foot (value has been reduced for buoyant forces).

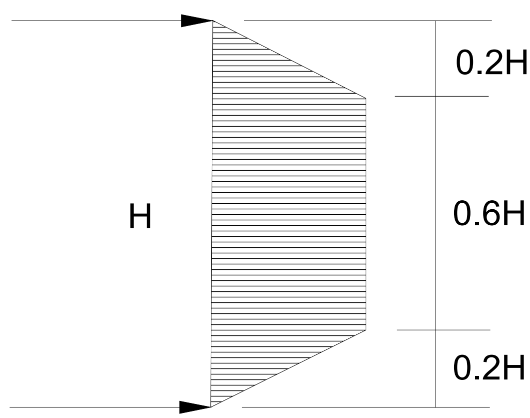
8.20.18 Due to the nature of the site soils, it is expected that continuous lagging between soldier piles will be required. However, it is recommended that the exposed soils be observed by the Geotechnical Engineer (a representative of Geocon West, Inc.), to verify the presence of any competent, cohesive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

8.20.19 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 pounds per square foot.

8.20.20 For the design of shoring, it is recommended that an equivalent fluid pressure based on the following table, be utilized for design. A trapezoidal distribution of lateral earth pressure may be used where shoring will be restrained by bracing or tie backs. The recommended active and trapezoidal pressure are provided in the following table. A diagram depicting the trapezoidal pressure distribution of lateral earth pressure is provided below the table. Shoring pressure calculations are presented on Figures 19a though 19c.

HEIGHT OF SHORING (FEET)	EQUIVALENT FLUID PRESSURE (Pounds Per Cubic Foot) (ACTIVE PRESSURE)	EQUIVALENT FLUID PRESSURE (Pounds Per Square Foot per Foot) (Where H is the height of the shoring in feet)
Up to 15 feet	25	16H
16 to 35 feet	36	23H
36 to 50 feet	41	26H

Trapezoidal Distribution of Pressure



8.20.21 Where a combination of sloped embankment and shoring is utilized, the pressure will be greater and must be determined for each combination. Additional active pressure should be added for a surcharge condition due to slopes, vehicular traffic or adjacent structures and should be designed for each condition. The surcharge pressure should be evaluated in accordance with the recommendations in Section 8.25 of this report.

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

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

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

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

8.21 Temporary Tieback Anchors

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

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

- 7 feet below the top of the excavation –1,050 pounds per square foot
- 16 feet below the top of the excavation – 1,600 pounds per square foot
- 24 feet below the top of the excavation – 1,100 pounds per square foot (reduced for buoyancy)
- 32 feet below the top of the excavation – 1,100 pounds per square foot (reduced for buoyancy)

8.21.3 Depending on the techniques utilized, and the experience of the contractor performing the installation, a maximum allowable friction capacity of 1.7 kips per linear foot for post-grouted anchors (for a minimum 20-foot length beyond the active wedge) may be assumed for design purposes. Only the frictional resistance developed beyond the active wedge should be utilized in resisting lateral loads.

8.22 Anchor Installation

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

8.23 Anchor Testing

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

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

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

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

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

8.24 Internal Bracing

8.24.1 Rakers may be utilized to brace the soldier piles in lieu of tieback anchors. The raker bracing could be supported laterally by temporary concrete footings (deadmen) or by the permanent, interior footings. For design of such temporary footings or deadmen, poured with the bearing surface normal to rakers inclined at 45 degrees, a bearing value of 2,000 psf in competent alluvial deposits may be used, provided the shallowest point of the footing is at least 1 foot below the lowest adjacent grade. The client should be aware that the utilization of rakers could significantly impact the construction schedule due to their intrusion into the construction site and potential interference with equipment. In addition, the raker footing plan should be checked by the project structural engineer to verify if there are any conflicts with the proposed structural foundations, and resolve any issues prior to commencement of construction activities.

8.25 Surcharge from Adjacent Structures and Improvements

8.25.1 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.

8.25.2 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.

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

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

8.26 Surface Drainage

8.26.1 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.

8.26.2 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.

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

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

8.27 Plan Review

8.27.1 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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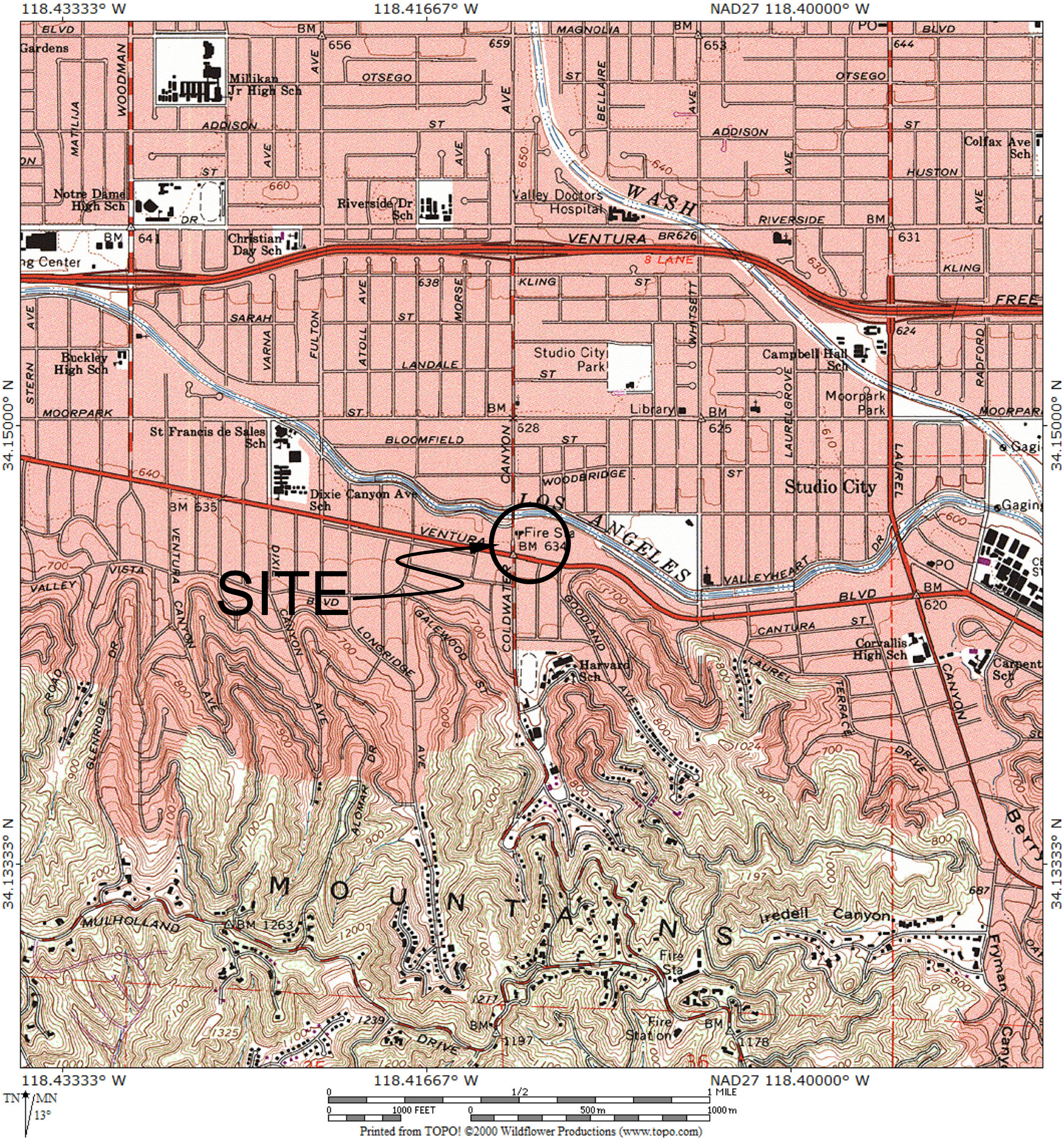

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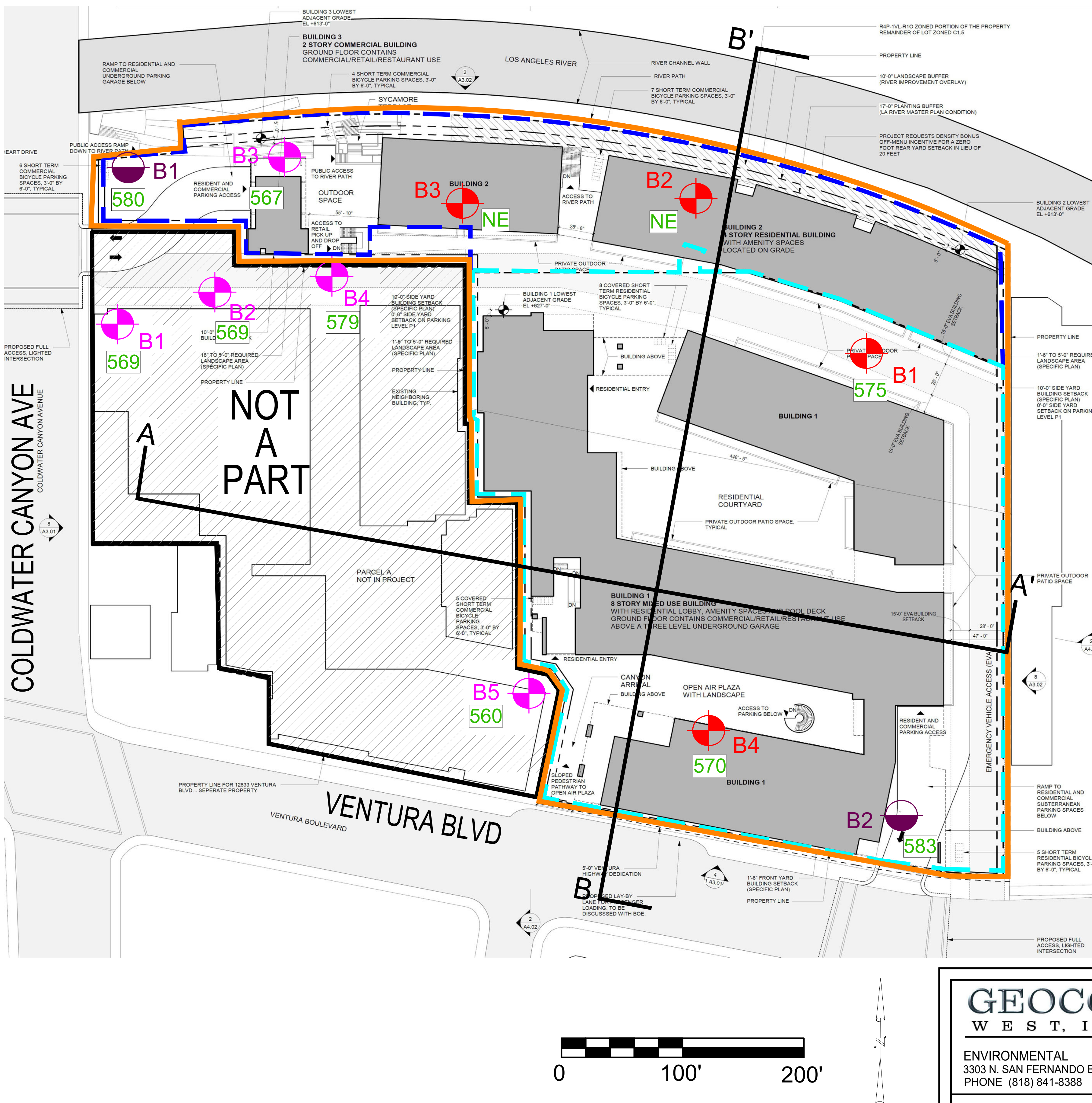
DRAFTED BY: MDS | CHECKED BY: SFK

VICINITY MAP

12805-12825 WEST VENTURA BOULEVARD
STUDIO CITY, CALIFORNIA

JUNE 2021 | PROJECT NO. A1208-06-01 | FIG. 1



Legend

- B1 Approximate Boring Location
- Limit of Proposed 3-Level Subterranean Parking
- Limit of Proposed 1-Level Subterranean Parking (Sub-Level to Match Grade of Existing River Path)
- Limit of Existing 1- to 2-Story On-Grade Commercial Structures
- Property Limits
- B5 Approximate Boring Location (GeoSoils, 2014)
- B2 Approximate Boring Location (VBB, 2014)
- 123 Approximate Elev of Bedrock +/- (ft, MSL)

Notes: NE indicates Not Encountered
Elevations based on assumed GSE take from 2019 Survey

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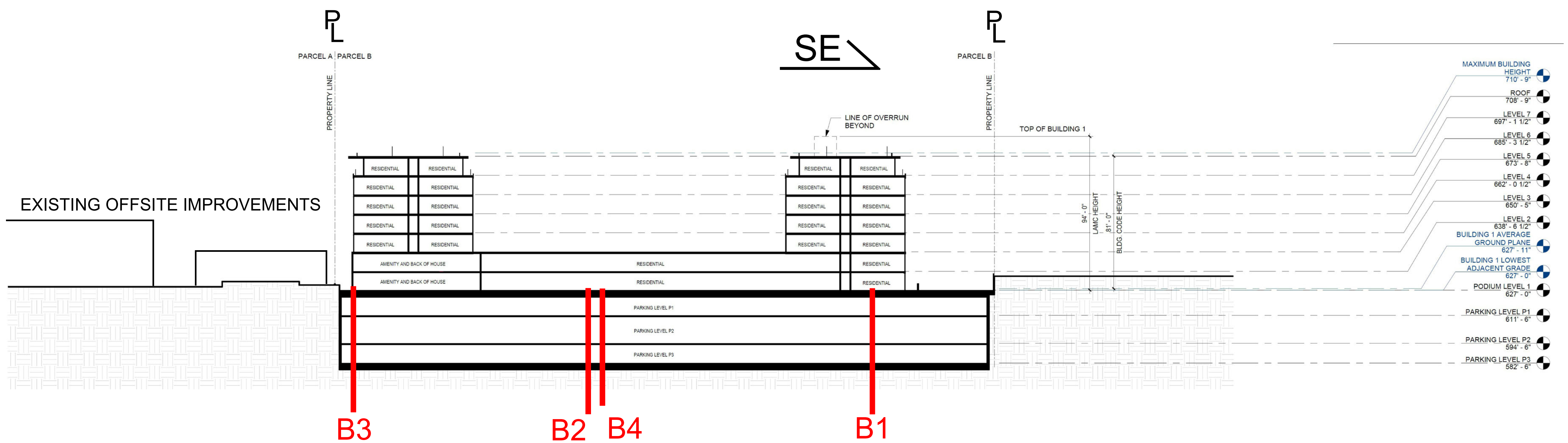
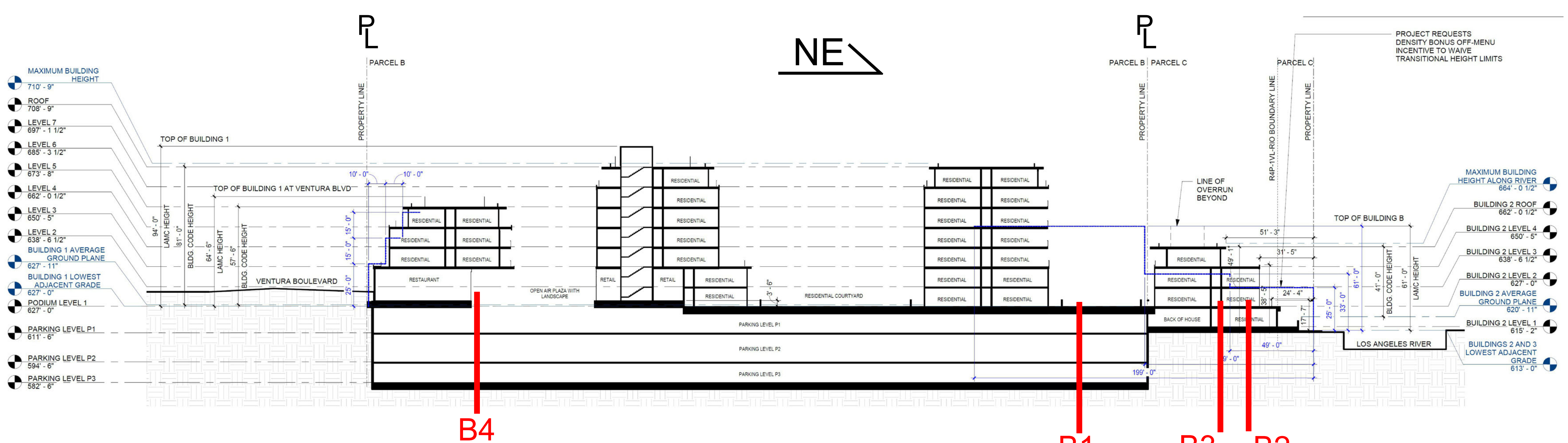
SITE PLAN

12805-12825 WEST VENTURA BOULEVARD
STUDIO CITY, CALIFORNIA

JUNE 2021 | PROJECT NO: W1208-06-01 | FIG. 2A



SECTION A - A'



SECTION B - B'

Legend

B1 Approximate Boring Locations

0 100' 200'

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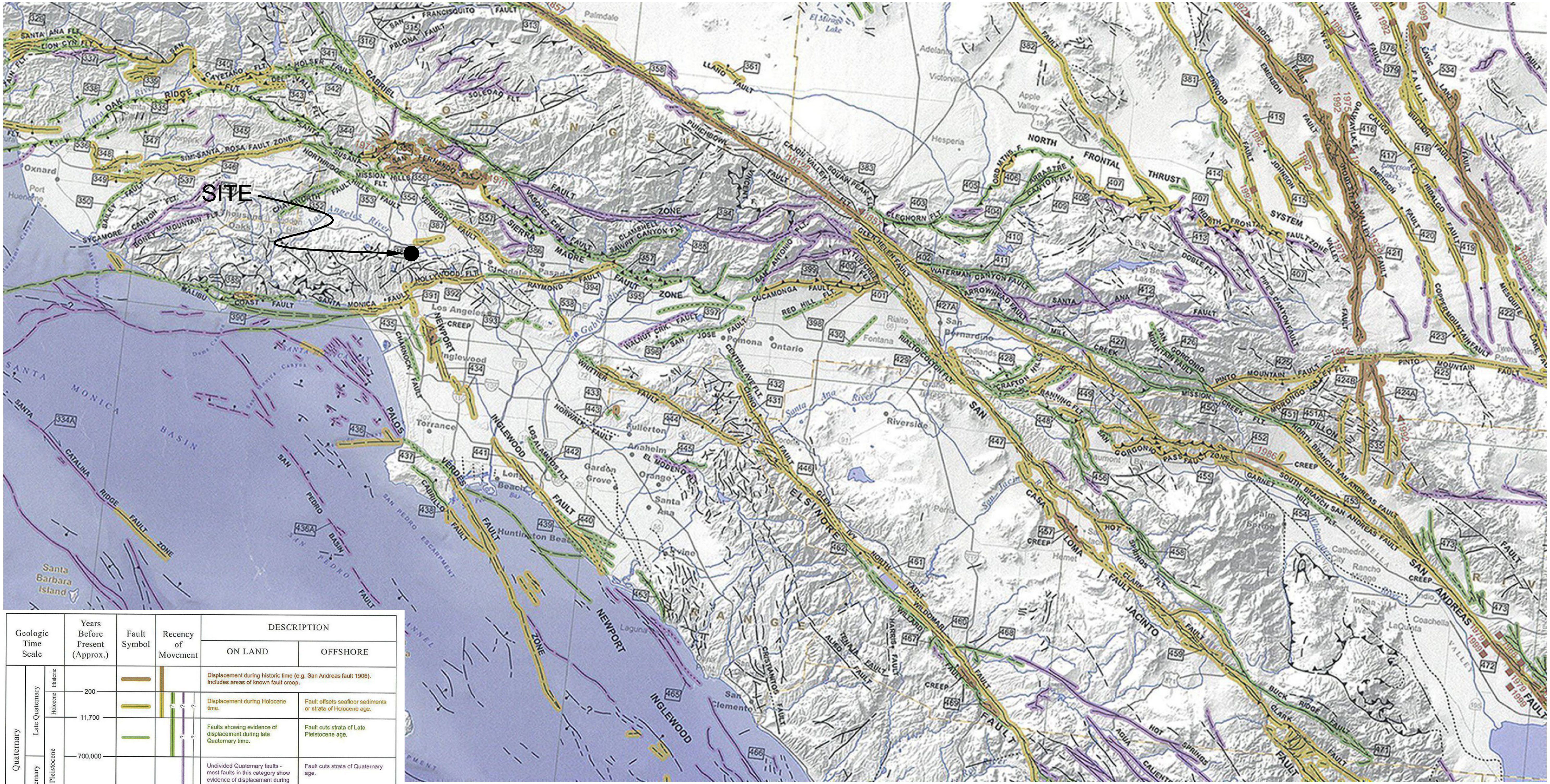
CROSS SECTION

12805-12825 WEST VENTURA BOULEVARD
STUDIO CITY, CALIFORNIA

JUNE 2021	PROJECT NO: W1208-06-01	FIG. 2B

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.

0 12 24 Miles

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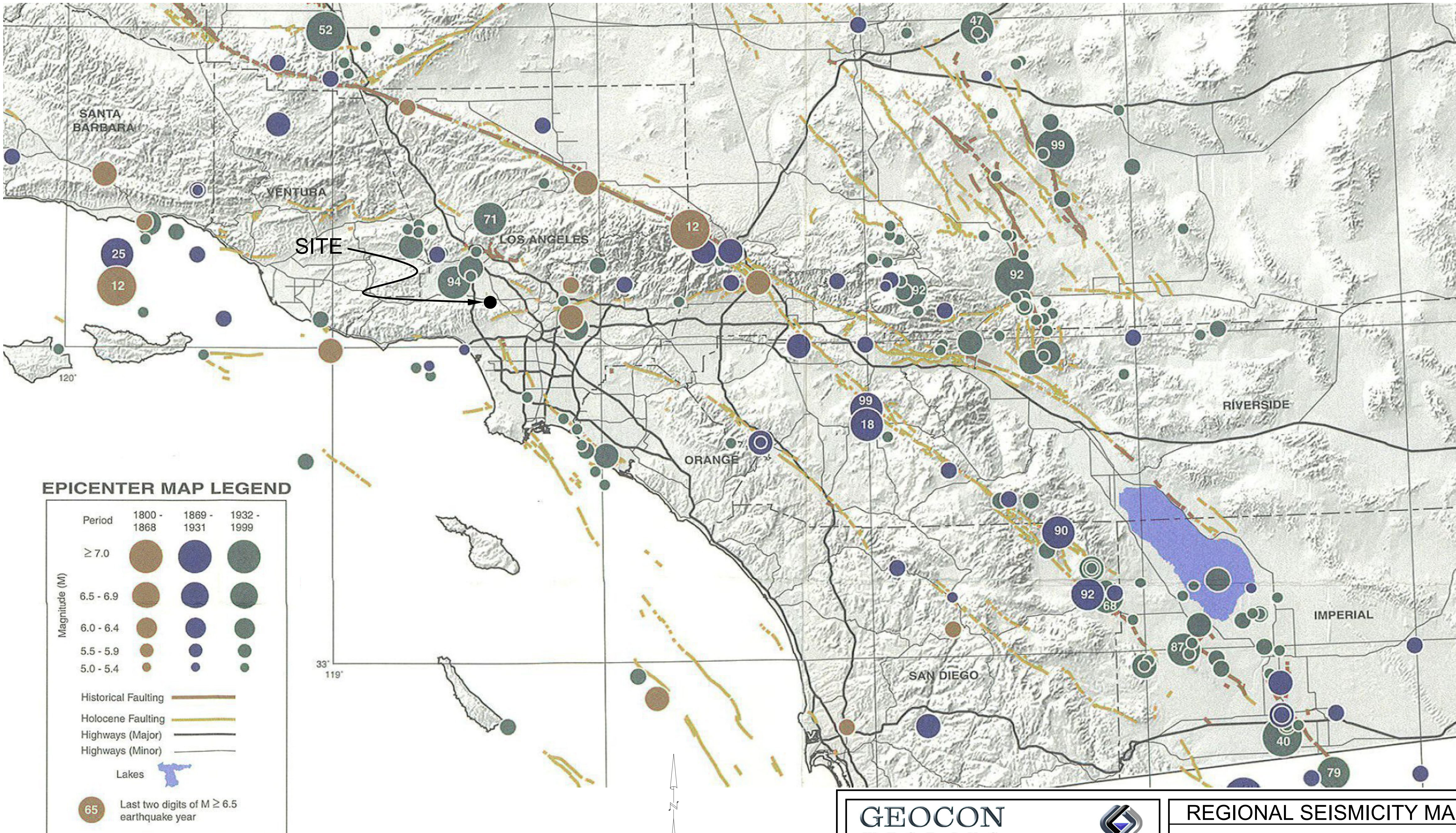
REGIONAL FAULT MAP

12805-12825 WEST VENTURA BOUELVARD
STUDIO CITY, CALIFORNIA

JUNE 2021 | PROJECT NO. W1208-06-01 | FIG. 3



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N

0 20 40 Miles

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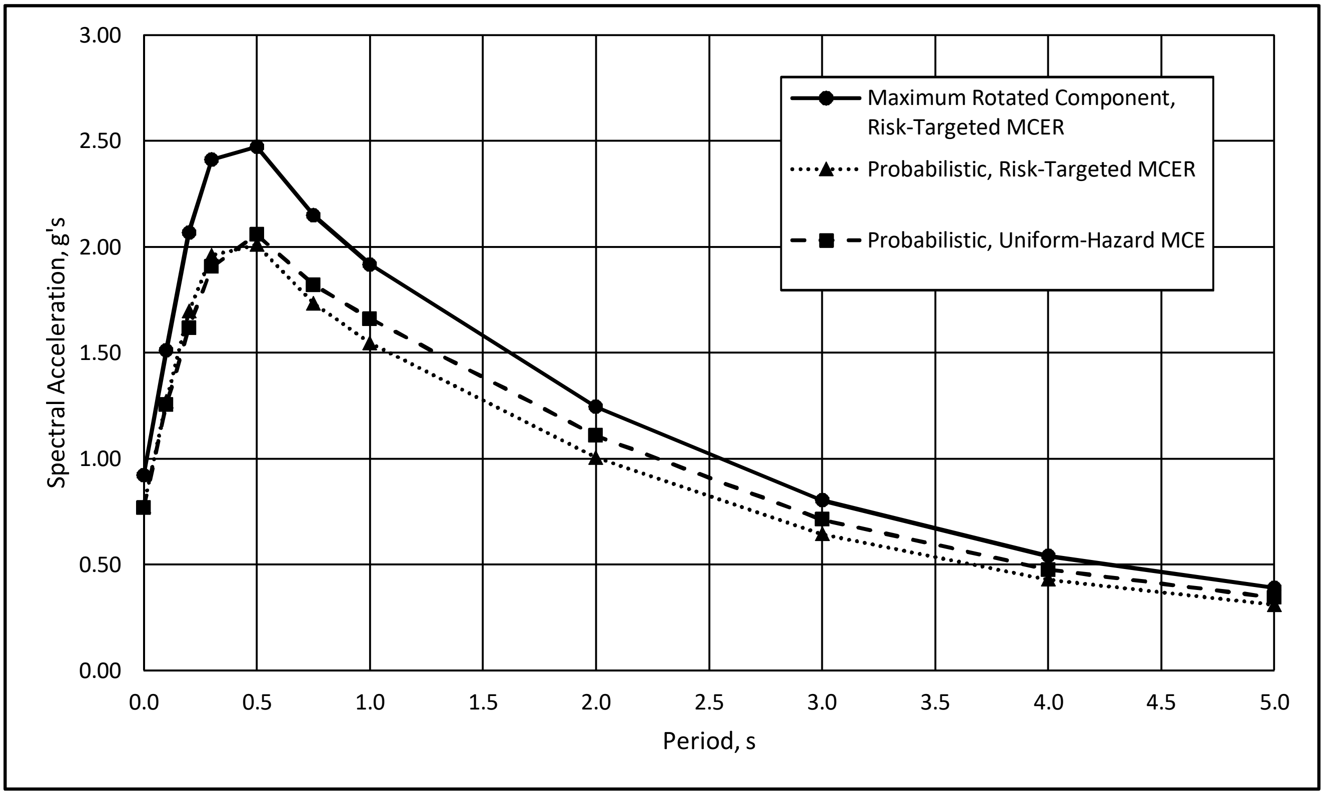
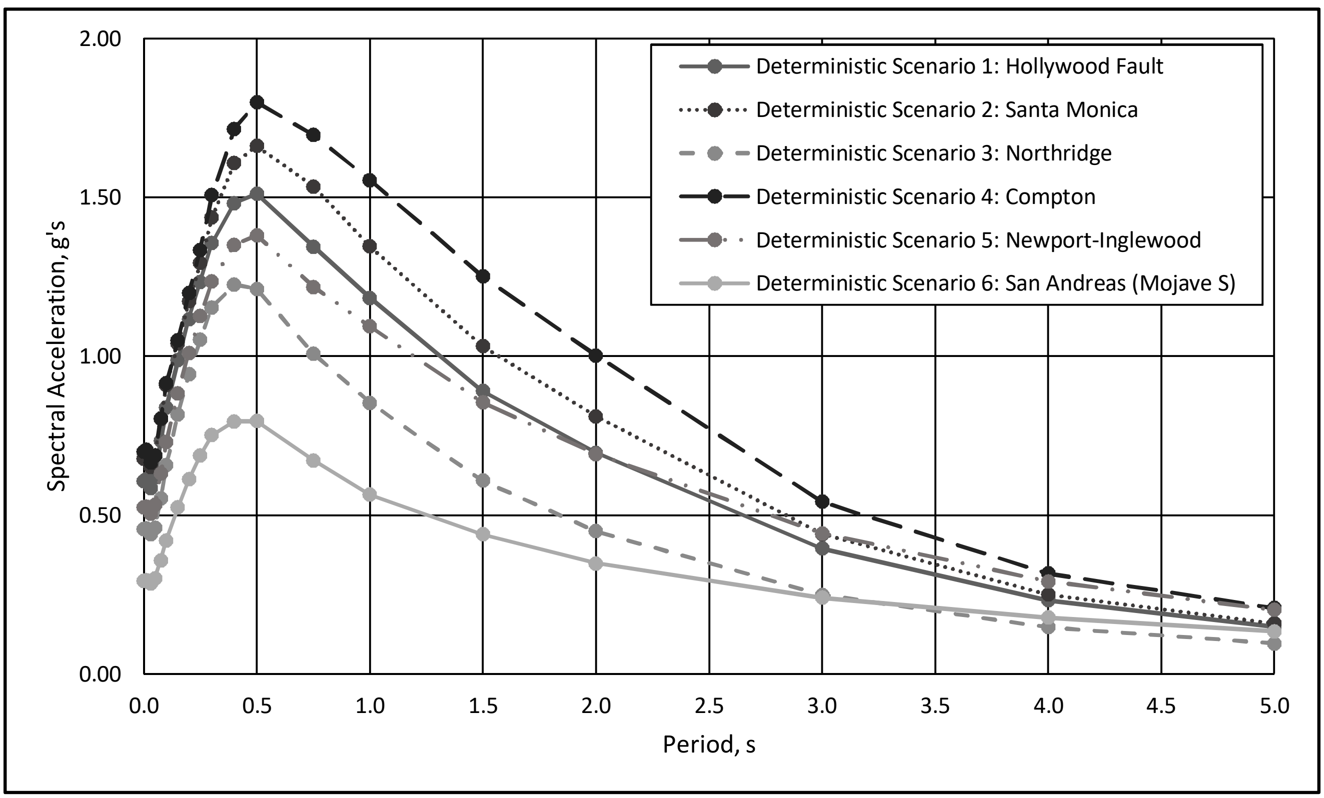
REGIONAL SEISMICITY MAP

12805-12825 WEST VENTURA BOULEVARD
STUDIO CITY, CALIFORNIA

JUNE 2021	PROJECT NO. W1208-06-01	FIG.4





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DESIGN RESPONSE SPECTRUM

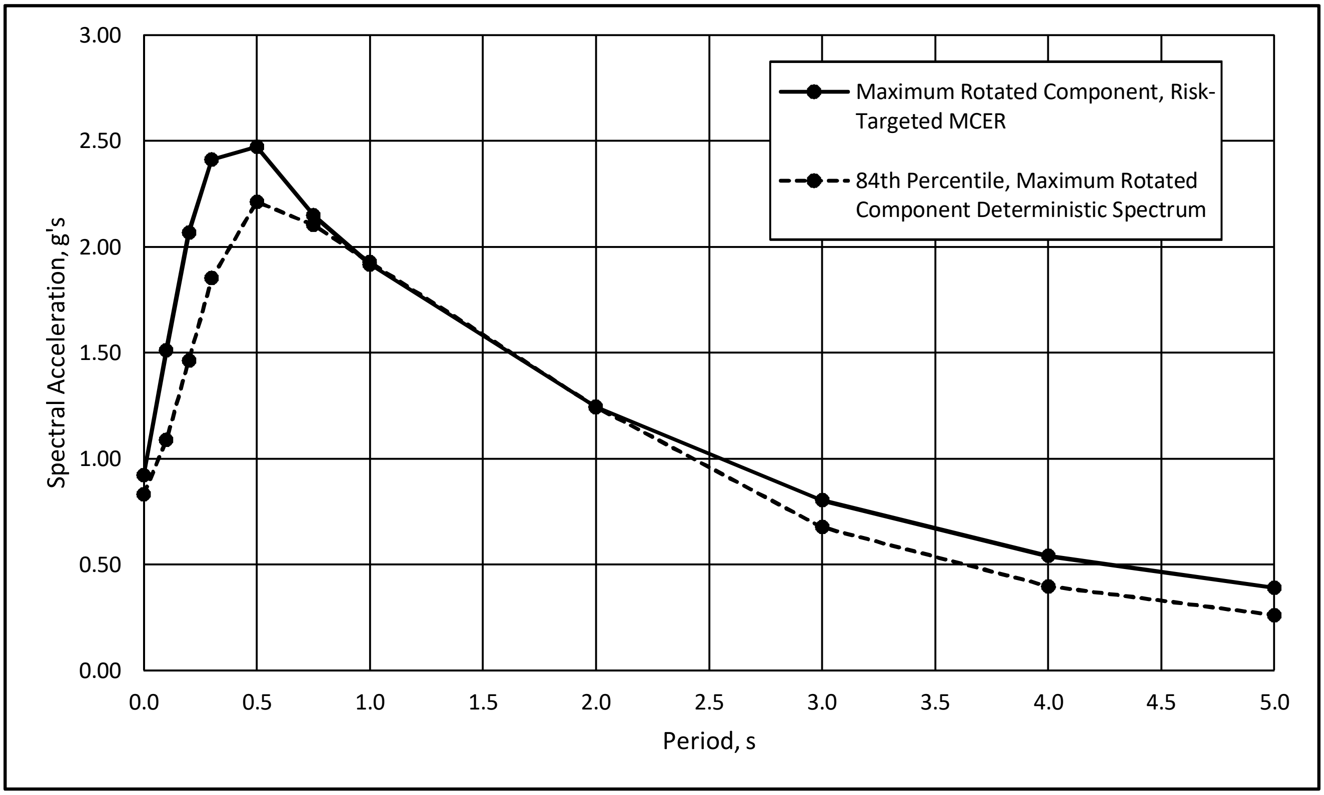
Checked by: JJK

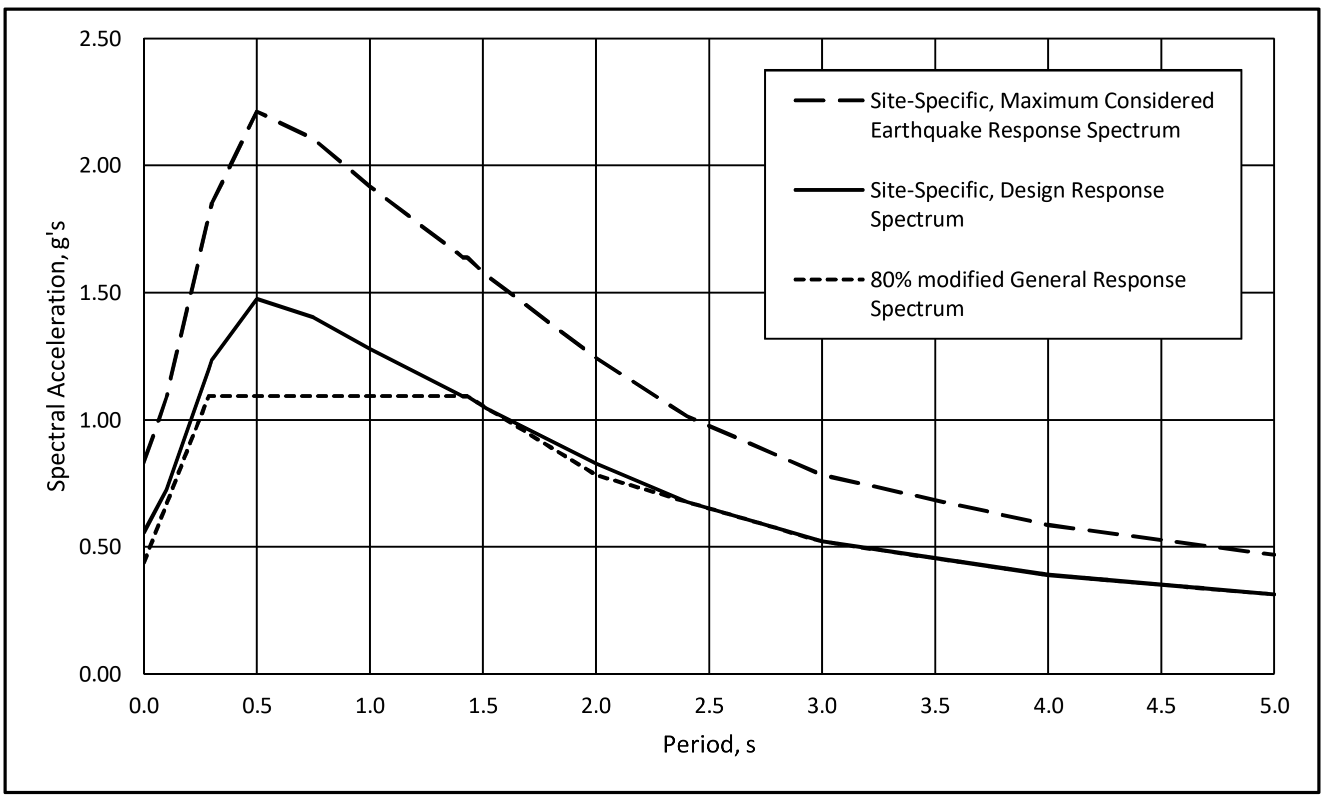
Project No.: W1208-06-01

12805-12825 WEST VENTURA BOULEVARD
STUDIO CITY, CALIFORNIA

JUNE 2021

Figure 5





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DESIGN RESPONSE SPECTRUM

Checked by: JJK

Project No.: W1208-06-01

12805-12825 WEST VENTURA BOULEVARD
STUDIO CITY, CALIFORNIA

JUNE 2021

Figure 6

Spectral Period (seconds)	Probabilistic Uniform-Hazard	Risk-Targeted, Probabilistic	Risk Factor, Cr	Maximum-Rotated Componet Scale Factor	MRC, Risk-Targeted Probablistic	84th Percentile, Deterministic	Site-Specific Design Earthquake	80% Modifed General Response Spectrum	Site-Specific Maximum Considered Earthquake
0.00	0.769	0.775	1.008	1.190	0.923	0.834	0.556	0.437	0.834
0.10	1.256	1.271	1.012	1.190	1.513	1.088	0.726	0.666	1.088
0.20	1.619	1.695	1.047	1.220	2.068	1.464	0.976	0.895	1.464
0.29	--	--	--	--	--	--	1.199	1.093	1.799
0.30	1.906	1.960	1.028	1.230	2.411	1.853	1.235	1.093	1.853
0.50	2.058	2.009	0.976	1.230	2.472	2.213	1.475	1.093	2.213
0.75	1.820	1.733	0.952	1.240	2.149	2.103	1.402	1.093	2.103
1.00	1.660	1.546	0.931	1.240	1.917	1.927	1.278	1.093	1.917
1.41	--	--	--	--	--	--	1.093	1.093	1.639
1.43	--	--	--	--	--	--	1.093	1.093	1.639
1.52	--	--	--	--	--	--	1.046	1.046	1.569
2.00	1.109	1.005	0.906	1.240	1.246	1.243	0.829	0.782	1.243
2.40	--	--	--	--	--	--	0.677	0.677	1.015
3.00	0.714	0.644	0.902	1.250	0.804	0.677	0.521	0.521	0.782
4.00	0.476	0.430	0.902	1.260	0.541	0.398	0.391	0.391	0.586
5.000	0.345	0.311	0.902	1.260	0.392	0.262	0.313	0.313	0.469

SM_S = 1.991 g
SM_1 = 2.487 g

SD_S = 1.328 g
SD_1 = 1.658 g

Reference: ASCE 7-16 21.4 DESIGN ACCELERATION PARAMETERS

Where the site-specific procedure is used to determine the design ground motion in accordance with Section 21.3, the parameter S_{DS} shall be taken as 90% of the maximum spectral acceleration, S_a, obtained from the site-specific spectrum, at any period within the range from 0.2 to 5 s, inclusive. The parameter S_{D1} shall be taken as the maximum value of the product, TS_a, for periods from 1 to 2 s for sites with $v_{s,30}$ > 1,200 ft/s ($v_{s,30}$ > 365.76 m/s) and for periods from 1 to 5 s for sites with $v_{s,30} \leq$ 1,200 ft/=s ($v_{s,30} \leq$ 365.76 m/s). The parameters S_{MS} and S_{M1} shall be taken as 1.5 times S_{DS} and S_{D1}, respectively. The values so obtained shall not be less than 80% of the values determined in accordance with Section 11.4.3 for S_{MS} and S_{M1} and Section 11.4.5 for S_{DS} and S_{D1}.

"--" Indicates that spectral period was not used at that calculation step

GEOCON

File No. : W1208-06-01
Boring : 1

EMPIRICAL ESTIMATION OF LIQUEFACTION POTENTIAL
DESIGN EARTHQUAKE

NCEER (1996) METHOD

By Thomas F. Blake (1994-1996)

EARTHQUAKE INFORMATION:

Earthquake Magnitude:	6.68
Peak Horiz. Acceleration PGA_M (g):	0.756
2/3 PGA_M (g):	0.504
Calculated Mag.Wtg.Factor:	0.747
Historic High Groundwater:	10.0
Groundwater Depth During Exploration:	22.0

ENERGY & ROD CORRECTIONS:

Energy Correction (CE) for N60:	1.25
Rod Len.Corr.(CR)(0-no or 1-yes):	1.0
Bore Dia. Corr. (CB):	1.00
Sampler Corr. (CS):	1.20
Use Ksigma (0 or 1):	1.0

LIQUEFACTION CALCULATIONS:

Unit Wt. Water (pcf): 62.4

Depth to Base (ft)	Total Unit Wt. (pcf)	Water (0 or 1)	FIELD SPT (N)	Depth of SPT (ft)	Liq.Sus. (0 or 1)	-200 (%)	Est. Dr (%)	CN Factor	Corrected (N1)60	Eff. Unit Wt. (psf)	Resist. CRR	rd Factor	Induced CSR	Liquefac. Safe.Fact.
1.0	120.0	0	10.0	1.0	0			1.700	19.1	120.0	~	0.998	0.244	~
2.0	120.0	0	10.0	2.0	0			1.700	19.1	120.0	~	0.993	0.243	~
3.0	120.0	0	10.0	3.0	0			1.700	19.1	120.0	~	0.989	0.242	~
4.0	120.0	0	14.0	4.0	0			1.700	26.8	120.0	~	0.984	0.241	~
5.0	120.0	0	14.0	5.0	0			1.700	26.8	120.0	~	0.979	0.240	~
6.0	120.0	0	14.0	6.0	0			1.700	26.8	120.0	~	0.975	0.239	~
7.0	120.0	0	14.0	7.0	0			1.636	25.8	120.0	~	0.970	0.238	~
8.0	120.0	0	14.0	8.0	0			1.523	24.0	120.0	~	0.966	0.236	~
9.0	120.0	0	14.0	9.0	0			1.431	22.5	120.0	~	0.961	0.235	~
10.0	120.0	0	14.0	10.0	1	54	73	1.353	28.3	120.0	0.359	0.957	0.234	--
11.0	120.0	1	14.0	10.0	1	54	73	1.287	27.3	57.6	0.330	0.952	0.239	1.38
12.0	120.0	1	14.0	10.0	1	54	73	1.230	26.4	57.6	0.311	0.947	0.249	1.25
13.0	120.0	1	14.0	10.0	1	54	73	1.180	25.6	57.6	0.296	0.943	0.258	1.15
14.0	120.0	1	14.0	10.0	1	54	73	1.135	24.9	57.6	0.284	0.938	0.266	1.07
15.0	120.0	1	8.0	15.0	1	60	53	1.095	17.6	57.6	0.191	0.934	0.273	0.70
16.0	120.0	1	8.0	15.0	1	60	53	1.060	17.3	57.6	0.188	0.929	0.279	0.67
17.0	120.0	1	8.0	15.0	1	60	53	1.027	16.9	57.6	0.184	0.925	0.285	0.65
18.0	120.0	1	8.0	15.0	1	60	53	0.997	16.7	57.6	0.181	0.920	0.290	0.62
19.0	120.0	1	8.0	15.0	1	60	53	0.970	16.4	57.6	0.178	0.915	0.295	0.61
20.0	120.0	1	1.0	20.0	1	54	18	0.945	8.3	57.6	0.092	0.911	0.299	0.31
21.0	120.0	1	1.0	20.0	1	54	18	0.921	8.2	57.6	0.092	0.906	0.302	0.30
22.0	120.0	1	1.0	20.0	1	54	18	0.905	8.2	57.6	0.092	0.902	0.306	0.30
23.0	120.0	1	1.0	20.0	1	54	18	0.895	8.2	57.6	0.092	0.897	0.309	0.30
24.0	120.0	1	1.0	20.0	1	54	18	0.885	8.2	57.6	0.091	0.893	0.312	0.29
25.0	120.0	1	9.0	25.0	0	63		0.876	18.3	57.6	~	0.888	0.314	~
26.0	120.0	1	9.0	25.0	0	63		0.867	18.2	57.6	~	0.883	0.316	~
27.0	120.0	1	9.0	25.0	0	63		0.858	18.1	57.6	~	0.879	0.318	~
28.0	120.0	1	9.0	25.0	0	63		0.849	18.0	57.6	~	0.874	0.320	~
29.0	120.0	1	9.0	25.0	0	63		0.841	17.8	57.6	~	0.870	0.321	~
30.0	120.0	1	4.0	30.0	0	62		0.833	12.0	57.6	~	0.865	0.323	~
31.0	120.0	1	4.0	30.0	0	62		0.825	12.0	57.6	~	0.861	0.324	~
32.0	120.0	1	4.0	30.0	0	62		0.817	11.9	57.6	~	0.856	0.325	~
33.0	120.0	1	4.0	30.0	0	62		0.810	11.9	57.6	~	0.851	0.326	~
34.0	120.0	1	4.0	30.0	0	62		0.803	11.8	57.6	~	0.847	0.326	~
35.0	120.0	1	2.0	35.0	0	60		0.796	9.4	57.6	~	0.842	0.327	~
36.0	120.0	1	2.0	35.0	0	60		0.789	9.4	57.6	~	0.838	0.327	~
37.0	120.0	1	2.0	35.0	0	60		0.782	9.3	57.6	~	0.833	0.328	~
38.0	120.0	1	2.0	35.0	0	60		0.776	9.3	57.6	~	0.829	0.328	~
39.0	120.0	1	2.0	35.0	0	60		0.769	9.3	57.6	~	0.824	0.328	~
40.0	120.0	1	2.0	40.0	0	80		0.763	9.3	57.6	~	0.819	0.328	~
41.0	120.0	1	2.0	40.0	0	80		0.757	9.3	57.6	~	0.815	0.328	~
42.0	120.0	1	2.0	40.0	0	80		0.751	9.3	57.6	~	0.810	0.328	~
43.0	120.0	1	2.0	40.0	0	80		0.745	9.2	57.6	~	0.806	0.328	~
44.0	120.0	1	2.0	40.0	0	80		0.740	9.2	57.6	~	0.801	0.327	~
45.0	120.0	1	48.0	45.0	1	3	106	0.734	52.9	57.6	Infin.	0.797	0.327	Non-Liq.
46.0	120.0	1	48.0	45.0	1	3	106	0.729	52.5	57.6	Infin.	0.792	0.326	Non-Liq.
47.0	120.0	1	48.0	45.0	1	3	106	0.724	52.1	57.6	Infin.	0.787	0.326	Non-Liq.
48.0	120.0	1	48.0	45.0	1	3	106	0.718	51.7	57.6	Infin.	0.783	0.325	Non-Liq.
49.0	120.0	1	48.0	45.0	1	3	106	0.713	51.4	57.6	Infin.	0.778	0.325	Non-Liq.
50.0	120.0	1	10.0	50.0	0	63		0.708	17.6	57.6	~	0.774	0.324	~
51.0	120.0	1	10.0	50.0	0	63		0.703	17.6	57.6	~	0.769	0.323	~
52.0	120.0	1	10.0	50.0	0	63		0.699	17.5	57.6	~	0.765	0.322	~
53.0	120.0	1	18.0	50.0	0			0.694	18.7	57.6	~	0.760	0.321	~
54.0	120.0	1	18.0	50.0	0			0.689	18.6	57.6	~	0.755	0.320	~
55.0	120.0	1	18.0	55.0	0			0.685	18.5	57.6	~	0.751	0.320	~
56.0	120.0	1	18.0	55.0	0			0.681	18.4	57.6	~	0.746	0.319	~
57.0	120.0	1	18.0	55.0	0			0.676	18.3	57.6	~	0.742	0.318	~
58.0	120.0	1	18.0	55.0	0			0.672	18.1	57.6	~	0.737	0.316	~
59.0	120.0	1	18.0	55.0	0			0.668	18.0	57.6	~	0.733	0.315	~
60.0	120.0	1	55.0	60.0	0			0.664	54.8	57.6	~	0.728	0.314	~
61.0	120.0	1	55.0	60.0	0			0.660	54.4	57.6	~	0.723	0.313	~
62.0	120.0	1	55.0	60.0	0			0.656	54.1	57.6	~	0.719	0.312	~
63.0	120.0	1	55.0	60.0	0			0.652	53.8	57.6	~	0.714	0.311	~
64.0	120.0	1	55.0	60.0	0			0.648	53.5	57.6	~	0.710	0.309	~
65.0	120.0	1	11.0	65.0	0			0.645	10.6	57.6	~	0.705	0.308	~
66.0	120.0	1	11.0	65.0	0			0.641	10.6	57.6	~	0.701	0.307	~
67.0	120.0	1	11.0	65.0	0			0.637	10.5	57.6	~	0.696	0.305	~
68.0	120.0	1	11.0	65.0	0			0.634	10.5	57.6	~	0.691	0.304	~
69.0	120.0	1	11.0	65.0	0			0.630	10.4	57.6	~	0.687	0.303	~
70.0	120.0	1	100.0	70.0	0			0.627	94.0	57.6	~	0.682	0.301	~

GEOCON

File No. : W1208-06-01
Boring : 1

LIQUEFACTION SETTLEMENT ANALYSIS
DESIGN EARTHQUAKE

(SATURATED SAND AT INITIAL LIQUEFACTION CONDITION)

NCEER (1996) METHOD
EARTHQUAKE INFORMATION:

Earthquake Magnitude:	6.68
PGAM (g):	0.756
2/3 PGAM (g):	0.50
Calculated Mag.Wtg.Factor:	0.747
Historic High Groundwater:	10.0
Groundwater @ Exploration:	22.0

DEPTH TO BASE	BLOW COUNT N	WET DENSITY (PCF)	TOTAL STRESS O (TSF)	EFFECT STRESS O' (TSF)	REL. DEN. Dr (%)	ADJUST BLOWS (N1)60	Tav/σ'o	LIQUEFACTION SAFETY FACTOR	Volumetric Strain [e15} (%)	EQ. SETTLE. Pe (in.)
1	10	120	0.030	0.030		19	0.328	~	0.00	SUBTERRANEAN LEVEL
2	10	120	0.090	0.090		19	0.328	~	0.00	
3	10	120	0.150	0.150		19	0.328	~	0.00	
4	14	120	0.210	0.210		27	0.328	~	0.00	
5	14	120	0.270	0.270		27	0.328	~	0.00	
6	14	120	0.330	0.330		27	0.328	~	0.00	
7	14	120	0.390	0.390		26	0.328	~	0.00	
8	14	120	0.450	0.450		24	0.328	~	0.00	
9	14	120	0.510	0.510		23	0.328	~	0.00	
10	14	120	0.570	0.570	73	28	0.328	--	0.00	
11	14	120	0.630	0.614	73	27	0.336	1.38	0.00	
12	14	120	0.690	0.643	73	26	0.352	1.25	0.00	
13	14	120	0.750	0.672	73	26	0.366	1.15	0.00	
14	14	120	0.810	0.701	73	25	0.379	1.07	1.30	
15	8	120	0.870	0.730	53	18	0.391	0.70	1.70	
16	8	120	0.930	0.758	53	17	0.402	0.67	1.70	0.20
17	8	120	0.990	0.787	53	17	0.412	0.65	1.70	0.20
18	8	120	1.050	0.816	53	17	0.422	0.62	1.70	0.20
19	8	120	1.110	0.845	53	16	0.431	0.61	1.70	0.20
20	1	120	1.170	0.874	18	8	0.439	0.31	2.70	0.32
21	1	120	1.230	0.902	18	8	0.447	0.30	2.70	0.32
22	1	120	1.290	0.931	18	8	0.454	0.30	2.70	0.32
23	1	120	1.350	0.960	18	8	0.461	0.30	2.70	0.32
24	1	120	1.410	0.989	18	8	0.467	0.29	2.70	0.32
25	9	120	1.470	1.018		18	0.473	~	0.00	0.00
26	9	120	1.530	1.046		18	0.479	~	0.00	0.00
27	9	120	1.590	1.075		18	0.485	~	0.00	0.00
28	9	120	1.650	1.104		18	0.490	~	0.00	0.00
29	9	120	1.710	1.133		18	0.495	~	0.00	0.00
30	4	120	1.770	1.162		12	0.499	~	0.00	0.00
31	4	120	1.830	1.190		12	0.504	~	0.00	0.00
32	4	120	1.890	1.219		12	0.508	~	0.00	0.00
33	4	120	1.950	1.248		12	0.512	~	0.00	0.00
34	4	120	2.010	1.277		12	0.516	~	0.00	0.00
35	2	120	2.070	1.306		9	0.520	~	0.00	0.00
36	2	120	2.130	1.334		9	0.523	~	0.00	0.00
37	2	120	2.190	1.363		9	0.527	~	0.00	0.00
38	2	120	2.250	1.392		9	0.530	~	0.00	0.00
39	2	120	2.310	1.421		9	0.533	~	0.00	0.00
40	2	120	2.370	1.450		9	0.536	~	0.00	0.00
41	2	120	2.430	1.478		9	0.539	~	0.00	0.00
42	2	120	2.490	1.507		9	0.541	~	0.00	0.00
43	2	120	2.550	1.536		9	0.544	~	0.00	0.00
44	2	120	2.610	1.565		9	0.547	~	0.00	0.00
45	48	120	2.670	1.594	106	53	0.549	Non-Liq.	0.00	0.00
46	48	120	2.730	1.622	106	52	0.552	Non-Liq.	0.00	0.00
47	48	120	2.790	1.651	106	52	0.554	Non-Liq.	0.00	0.00
48	48	120	2.850	1.680	106	52	0.556	Non-Liq.	0.00	0.00
49	48	120	2.910	1.709	106	51	0.558	Non-Liq.	0.00	0.00
50	10	120	2.970	1.738		18	0.560	~	0.00	0.00
51	10	120	3.030	1.766		18	0.562	~	0.00	0.00
52	10	120	3.090	1.795		17	0.564	~	0.00	0.00
53	18	120	3.150	1.824		19	0.566	~	0.00	0.00
54	18	120	3.210	1.853		19	0.568	~	0.00	0.00
55	18	120	3.270	1.882		18	0.570	~	0.00	0.00
56	18	120	3.330	1.910		18	0.571	~	0.00	0.00
57	18	120	3.390	1.939		18	0.573	~	0.00	0.00
58	18	120	3.450	1.968		18	0.575	~	0.00	0.00
59	18	120	3.510	1.997		18	0.576	~	0.00	0.00
60	55	120	3.570	2.026		55	0.578	~	0.00	0.00
61	55	120	3.630	2.054		54	0.579	~	0.00	0.00
62	55	120	3.690	2.083		54	0.581	~	0.00	0.00
63	55	120	3.750	2.112		54	0.582	~	0.00	0.00
64	55	120	3.810	2.141		53	0.583	~	0.00	0.00
65	11	120	3.870	2.170		11	0.585	~	0.00	0.00
66	11	120	3.930	2.198		11	0.586	~	0.00	0.00
67	11	120	3.990	2.227		11	0.587	~	0.00	0.00
68	11	120	4.050	2.256		10	0.588	~	0.00	0.00
69	11	120	4.110	2.285		10	0.590	~	0.00	0.00
70	100	120	4.170	2.314		94	0.591	~	0.00	0.00

TOTAL SETTLEMENT = 2.4 INCHES

File No. : W1208-06-01
Boring : 1

GEOCON

EMPIRICAL ESTIMATION OF LIQUEFACTION POTENTIAL
DESIGN EARTHQUAKE

NCEER (1996) METHOD

By Thomas F. Blake (1994-1996)

EARTHQUAKE INFORMATION:

Earthquake Magnitude:	6.68
Peak Horiz. Acceleration PGA_M (g):	0.756
2/3 PGA_M (g):	0.504
Calculated Mag.Wtg.Factor:	0.747
Historic High Groundwater:	10.0
Groundwater Depth During Exploration:	22.0

ENERGY & ROD CORRECTIONS:

Energy Correction (CE) for N60:	1.25
Rod Len.Corr.(CR)(0-no or 1-yes):	1.0
Bore Dia. Corr. (CB):	1.00
Sampler Corr. (CS):	1.20
Use Ksigma (0 or 1):	1.0

LIQUEFACTION CALCULATIONS:

Unit Wt. Water (pcf):	62.4

Depth to Base (ft)	Total Unit Wt. (pcf)	Water (0 or 1)	FIELD SPT (N)	Depth of SPT (ft)	Liq.Sus. (0 or 1)	-200 (%)	Est. Dr (%)	CN Factor	Corrected (N1)60	Eff. Unit Wt. (psf)	Resist. CRR	rd Factor	Induced CSR	Liquefac. Safe.Fact.
1.0	120.0	0	10.0	1.0	0			1.700	19.1	120.0	~	0.998	0.244	~
2.0	120.0	0	10.0	2.0	0			1.700	19.1	120.0	~	0.993	0.243	~
3.0	120.0	0	10.0	3.0	0			1.700	19.1	120.0	~	0.989	0.242	~
4.0	120.0	0	14.0	4.0	0			1.700	26.8	120.0	~	0.984	0.241	~
5.0	120.0	0	14.0	5.0	0			1.700	26.8	120.0	~	0.979	0.240	~
6.0	120.0	0	14.0	6.0	0			1.700	26.8	120.0	~	0.975	0.239	~
7.0	120.0	0	14.0	7.0	0			1.636	25.8	120.0	~	0.970	0.238	~
8.0	120.0	0	14.0	8.0	0			1.523	24.0	120.0	~	0.966	0.236	~
9.0	120.0	0	14.0	9.0	0			1.431	22.5	120.0	~	0.961	0.235	~
10.0	120.0	0	14.0	10.0	1	54	73	1.353	28.3	120.0	0.359	0.957	0.234	--
11.0	120.0	1	14.0	10.0	1	54	73	1.287	27.3	57.6	0.330	0.952	0.239	1.38
12.0	120.0	1	14.0	10.0	1	54	73	1.230	26.4	57.6	0.311	0.947	0.249	1.25
13.0	120.0	1	14.0	10.0	1	54	73	1.180	25.6	57.6	0.296	0.943	0.258	1.15
14.0	120.0	1	14.0	10.0	1	54	73	1.135	24.9	57.6	0.284	0.938	0.266	1.07
15.0	120.0	1	8.0	15.0	1	60	53	1.095	17.6	57.6	0.191	0.934	0.273	0.70
16.0	120.0	1	8.0	15.0	1	60	53	1.060	17.3	57.6	0.188	0.929	0.279	0.67
17.0	120.0	1	8.0	15.0	1	60	53	1.027	16.9	57.6	0.184	0.925	0.285	0.65
18.0	120.0	1	8.0	15.0	1	60	53	0.997	16.7	57.6	0.181	0.920	0.290	0.62
19.0	120.0	1	8.0	15.0	1	60	53	0.970	16.4	57.6	0.178	0.915	0.295	0.61
20.0	120.0	1	1.0	20.0	1	54	18	0.945	8.3	57.6	0.092	0.911	0.299	0.31
21.0	120.0	1	1.0	20.0	1	54	18	0.921	8.2	57.6	0.092	0.906	0.302	0.30
22.0	120.0	1	1.0	20.0	1	54	18	0.905	8.2	57.6	0.092	0.902	0.306	0.30
23.0	120.0	1	1.0	20.0	1	54	18	0.895	8.2	57.6	0.092	0.897	0.309	0.30
24.0	120.0	1	1.0	20.0	1	54	18	0.885	8.2	57.6	0.091	0.893	0.312	0.29
25.0	120.0	1	9.0	25.0	0	63		0.876	18.3	57.6	~	0.888	0.314	~
26.0	120.0	1	9.0	25.0	0	63		0.867	18.2	57.6	~	0.883	0.316	~
27.0	120.0	1	9.0	25.0	0	63		0.858	18.1	57.6	~	0.879	0.318	~
28.0	120.0	1	9.0	25.0	0	63		0.849	18.0	57.6	~	0.874	0.320	~
29.0	120.0	1	9.0	25.0	0	63		0.841	17.8	57.6	~	0.870	0.321	~
30.0	120.0	1	4.0	30.0	0	62		0.833	12.0	57.6	~	0.865	0.323	~
31.0	120.0	1	4.0	30.0	0	62		0.825	12.0	57.6	~	0.861	0.324	~
32.0	120.0	1	4.0	30.0	0	62		0.817	11.9	57.6	~	0.856	0.325	~
33.0	120.0	1	4.0	30.0	0	62		0.810	11.9	57.6	~	0.851	0.326	~
34.0	120.0	1	4.0	30.0	0	62		0.803	11.8	57.6	~	0.847	0.326	~
35.0	120.0	1	2.0	35.0	0	60		0.796	9.4	57.6	~	0.842	0.327	~
36.0	120.0	1	2.0	35.0	0	60		0.789	9.4	57.6	~	0.838	0.327	~
37.0	120.0	1	2.0	35.0	0	60		0.782	9.3	57.6	~	0.833	0.328	~
38.0	120.0	1	2.0	35.0	0	60		0.776	9.3	57.6	~	0.829	0.328	~
39.0	120.0	1	2.0	35.0	0	60		0.769	9.3	57.6	~	0.824	0.328	~
40.0	120.0	1	2.0	40.0	0	80		0.763	9.3	57.6	~	0.819	0.328	~
41.0	120.0	1	2.0	40.0	0	80		0.757	9.3	57.6	~	0.815	0.328	~
42.0	120.0	1	2.0	40.0	0	80		0.751	9.3	57.6	~	0.810	0.328	~
43.0	120.0	1	2.0	40.0	0	80		0.745	9.2	57.6	~	0.806	0.328	~
44.0	120.0	1	2.0	40.0	0	80		0.740	9.2	57.6	~	0.801	0.327	~
45.0	120.0	1	48.0	45.0	1	3	106	0.734	52.9	57.6	Infin.	0.797	0.327	Non-Liq.
46.0	120.0	1	48.0	45.0	1	3	106	0.729	52.5	57.6	Infin.	0.792	0.326	Non-Liq.
47.0	120.0	1	48.0	45.0	1	3	106	0.724	52.1	57.6	Infin.	0.787	0.326	Non-Liq.
48.0	120.0	1	48.0	45.0	1	3	106	0.718	51.7	57.6	Infin.	0.783	0.325	Non-Liq.
49.0	120.0	1	48.0	45.0	1	3	106	0.713	51.4	57.6	Infin.	0.778	0.325	Non-Liq.
50.0	120.0	1	10.0	50.0	0	63		0.708	17.6	57.6	~	0.774	0.324	~
51.0	120.0	1	10.0	50.0	0	63		0.703	17.6	57.6	~	0.769	0.323	~
52.0	120.0	1	10.0	50.0	0	63		0.699	17.5	57.6	~	0.765	0.322	~
53.0	120.0	1	18.0	50.0	0			0.694	18.7	57.6	~	0.760	0.321	~
54.0	120.0	1	18.0	50.0	0			0.689	18.6	57.6	~	0.755	0.320	~
55.0	120.0	1	18.0	55.0	0			0.685	18.5	57.6	~	0.751	0.320	~
56.0	120.0	1	18.0	55.0	0			0.681	18.4	57.6	~	0.746	0.319	~
57.0	120.0	1	18.0	55.0	0			0.676	18.3	57.6	~	0.742	0.318	~
58.0	120.0	1	18.0	55.0	0			0.672	18.1	57.6	~	0.737	0.316	~
59.0	120.0	1	18.0	55.0	0			0.668	18.0	57.6	~	0.733	0.315	~
60.0	120.0	1	55.0	60.0	0			0.664	54.8	57.6	~	0.728	0.314	~
61.0	120.0	1	55.0	60.0	0			0.660	54.4	57.6	~	0.723	0.313	~
62.0	120.0	1	55.0	60.0	0			0.656	54.1	57.6	~	0.719	0.312	~
63.0	120.0	1	55.0	60.0	0			0.652	53.8	57.6	~	0.714	0.311	~
64.0	120.0	1	55.0	60.0	0			0.648	53.5	57.6	~	0.710	0.309	~
65.0	120.0	1	11.0	65.0	0			0.645	10.6	57.6	~	0.705	0.308	~
66.0	120.0	1	11.0	65.0	0			0.641	10.6	57.6	~	0.701	0.307	~
67.0	120.0	1	11.0	65.0	0			0.637	10.5	57.6	~	0.696	0.305	~
68.0	120.0	1	11.0	65.0	0			0.634	10.5	57.6	~	0.691	0.304	~
69.0	120.0	1	11.0	65.0	0			0.630	10.4	57.6	~	0.687	0.303	~
70.0	120.0	1	100.0	70.0	0			0.627	94.0	57.6	~	0.682	0.301	~

GEOCON

File No. : W1208-06-01
Boring : 1

LIQUEFACTION SETTLEMENT ANALYSIS
DESIGN EARTHQUAKE

(SATURATED SAND AT INITIAL LIQUEFACTION CONDITION)

NCEER (1996) METHOD
EARTHQUAKE INFORMATION:

Earthquake Magnitude:	6.68
PGAM (g):	0.756
2/3 PGAM (g):	0.50
Calculated Mag.Wtg.Factor:	0.747
Historic High Groundwater:	10.0
Groundwater @ Exploration:	22.0

DEPTH TO BASE	BLOW COUNT N	WET DENSITY (PCF)	TOTAL STRESS O (TSF)	EFFECT STRESS O' (TSF)	REL. DEN. Dr (%)	ADJUST BLOWS (N1)60	T_{av}/σ'_o	LIQUEFACTION SAFETY FACTOR	Volumetric Strain $[e_{15}]$ (%)	EQ. SETTLE. Pe (in.)
1	10	120	0.030	0.030		19	0.328	~	0.00	SUBTERRANEAN LEVEL
2	10	120	0.090	0.090		19	0.328	~	0.00	
3	10	120	0.150	0.150		19	0.328	~	0.00	
4	14	120	0.210	0.210		27	0.328	~	0.00	
5	14	120	0.270	0.270		27	0.328	~	0.00	
6	14	120	0.330	0.330		27	0.328	~	0.00	
7	14	120	0.390	0.390		26	0.328	~	0.00	
8	14	120	0.450	0.450		24	0.328	~	0.00	
9	14	120	0.510	0.510		23	0.328	~	0.00	
10	14	120	0.570	0.570	73	28	0.328	--	0.00	
11	14	120	0.630	0.614	73	27	0.336	1.38	0.00	
12	14	120	0.690	0.643	73	26	0.352	1.25	0.00	
13	14	120	0.750	0.672	73	26	0.366	1.15	0.00	
14	14	120	0.810	0.701	73	25	0.379	1.07	1.30	
15	8	120	0.870	0.730	53	18	0.391	0.70	1.70	
16	8	120	0.930	0.758	53	17	0.402	0.67	1.70	
17	8	120	0.990	0.787	53	17	0.412	0.65	1.70	
18	8	120	1.050	0.816	53	17	0.422	0.62	1.70	
19	8	120	1.110	0.845	53	16	0.431	0.61	1.70	
20	1	120	1.170	0.874	18	8	0.439	0.31	2.70	
21	1	120	1.230	0.902	18	8	0.447	0.30	2.70	
22	1	120	1.290	0.931	18	8	0.454	0.30	2.70	
23	1	120	1.350	0.960	18	8	0.461	0.30	2.70	
24	1	120	1.410	0.989	18	8	0.467	0.29	2.70	
25	9	120	1.470	1.018		18	0.473	~	0.00	
26	9	120	1.530	1.046		18	0.479	~	0.00	
27	9	120	1.590	1.075		18	0.485	~	0.00	
28	9	120	1.650	1.104		18	0.490	~	0.00	
29	9	120	1.710	1.133		18	0.495	~	0.00	
30	4	120	1.770	1.162		12	0.499	~	0.00	
31	4	120	1.830	1.190		12	0.504	~	0.00	
32	4	120	1.890	1.219		12	0.508	~	0.00	
33	4	120	1.950	1.248		12	0.512	~	0.00	
34	4	120	2.010	1.277		12	0.516	~	0.00	
35	2	120	2.070	1.306		9	0.520	~	0.00	
36	2	120	2.130	1.334		9	0.523	~	0.00	
37	2	120	2.190	1.363		9	0.527	~	0.00	
38	2	120	2.250	1.392		9	0.530	~	0.00	
39	2	120	2.310	1.421		9	0.533	~	0.00	0.00
40	2	120	2.370	1.450		9	0.536	~	0.00	0.00
41	2	120	2.430	1.478		9	0.539	~	0.00	0.00
42	2	120	2.490	1.507		9	0.541	~	0.00	0.00
43	2	120	2.550	1.536		9	0.544	~	0.00	0.00
44	2	120	2.610	1.565		9	0.547	~	0.00	0.00
45	48	120	2.670	1.594	106	53	0.549	Non-Liq.	0.00	0.00
46	48	120	2.730	1.622	106	52	0.552	Non-Liq.	0.00	0.00
47	48	120	2.790	1.651	106	52	0.554	Non-Liq.	0.00	0.00
48	48	120	2.850	1.680	106	52	0.556	Non-Liq.	0.00	0.00
49	48	120	2.910	1.709	106	51	0.558	Non-Liq.	0.00	0.00
50	10	120	2.970	1.738		18	0.560	~	0.00	0.00
51	10	120	3.030	1.766		18	0.562	~	0.00	0.00
52	10	120	3.090	1.795		17	0.564	~	0.00	0.00
53	18	120	3.150	1.824		19	0.566	~	0.00	0.00
54	18	120	3.210	1.853		19	0.568	~	0.00	0.00
55	18	120	3.270	1.882		18	0.570	~	0.00	0.00
56	18	120	3.330	1.910		18	0.571	~	0.00	0.00
57	18	120	3.390	1.939		18	0.573	~	0.00	0.00
58	18	120	3.450	1.968		18	0.575	~	0.00	0.00
59	18	120	3.510	1.997		18	0.576	~	0.00	0.00
60	55	120	3.570	2.026		55	0.578	~	0.00	0.00
61	55	120	3.630	2.054		54	0.579	~	0.00	0.00
62	55	120	3.690	2.083		54	0.581	~	0.00	0.00
63	55	120	3.750	2.112		54	0.582	~	0.00	0.00
64	55	120	3.810	2.141		53	0.583	~	0.00	0.00
65	11	120	3.870	2.170		11	0.585	~	0.00	0.00
66	11	120	3.930	2.198		11	0.586	~	0.00	0.00
67	11	120	3.990	2.227		11	0.587	~	0.00	0.00
68	11	120	4.050	2.256		10	0.588	~	0.00	0.00
69	11	120	4.110	2.285		10	0.590	~	0.00	0.00
70	100	120	4.170	2.314		94	0.591	~	0.00	0.00

TOTAL SETTLEMENT = 0.0 INCHES

GEOCON

File No. : W1208-06-01
Boring : 1

EMPIRICAL ESTIMATION OF LIQUEFACTION POTENTIAL
MAXIMUM CONSIDERED EARTHQUAKE

NCEER (1996) METHOD
EARTHQUAKE INFORMATION:

Earthquake Magnitude:	6.82
Peak Horiz. Acceleration PGA_M (g):	0.756
Calculated Mag.Wtg.Factor:	0.788
Historic High Groundwater:	10.0
Groundwater Depth During Exploration:	22.0

By Thomas F. Blake (1994-1996)
ENERGY & ROD CORRECTIONS:

Energy Correction (CE) for N60:	1.25
Rod Len.Corr.(CR)(0-no or 1-yes):	1.0
Bore Dia. Corr. (CB):	1.00
Sampler Corr. (CS):	1.20
Use Ksigma (0 or 1):	1.0

LIQUEFACTION CALCULATIONS:

Unit Wt. Water (pcf):	62.4

Depth to Base (ft)	Total Unit Wt. (pcf)	Water (0 or 1)	FIELD SPT (N)	Depth of SPT (ft)	Liq.Sus. (0 or 1)	-200 (%)	Est. Dr (%)	CN Factor	Corrected (N1)60	Eff. Unit Wt. (psf)	Resist. CRR	rd Factor	Induced CSR	Liquefac. Safe.Fact.
1.0	120.0	0	10.0	1.0	0			1.700	19.1	120.0	~	0.998	0.386	~
2.0	120.0	0	10.0	2.0	0			1.700	19.1	120.0	~	0.993	0.384	~
3.0	120.0	0	10.0	3.0	0			1.700	19.1	120.0	~	0.989	0.383	~
4.0	120.0	0	14.0	4.0	0			1.700	26.8	120.0	~	0.984	0.381	~
5.0	120.0	0	14.0	5.0	0			1.700	26.8	120.0	~	0.979	0.379	~
6.0	120.0	0	14.0	6.0	0			1.700	26.8	120.0	~	0.975	0.377	~
7.0	120.0	0	14.0	7.0	0			1.636	25.8	120.0	~	0.970	0.376	~
8.0	120.0	0	14.0	8.0	0			1.523	24.0	120.0	~	0.966	0.374	~
9.0	120.0	0	14.0	9.0	0			1.431	22.5	120.0	~	0.961	0.372	~
10.0	120.0	0	14.0	10.0	1	54	73	1.353	28.3	120.0	0.359	0.957	0.370	--
11.0	120.0	1	14.0	10.0	1	54	73	1.287	27.3	57.6	0.330	0.952	0.378	0.87
12.0	120.0	1	14.0	10.0	1	54	73	1.230	26.4	57.6	0.311	0.947	0.393	0.79
13.0	120.0	1	14.0	10.0	1	54	73	1.180	25.6	57.6	0.296	0.943	0.407	0.73
14.0	120.0	1	14.0	10.0	1	54	73	1.135	24.9	57.6	0.284	0.938	0.420	0.68
15.0	120.0	1	8.0	15.0	1	60	53	1.095	17.6	57.6	0.191	0.934	0.431	0.44
16.0	120.0	1	8.0	15.0	1	60	53	1.060	17.3	57.6	0.188	0.929	0.441	0.43
17.0	120.0	1	8.0	15.0	1	60	53	1.027	16.9	57.6	0.184	0.925	0.450	0.41
18.0	120.0	1	8.0	15.0	1	60	53	0.997	16.7	57.6	0.181	0.920	0.458	0.40
19.0	120.0	1	8.0	15.0	1	60	53	0.970	16.4	57.6	0.178	0.915	0.466	0.38
20.0	120.0	1	1.0	20.0	1	54	18	0.945	8.3	57.6	0.092	0.911	0.472	0.20
21.0	120.0	1	1.0	20.0	1	54	18	0.921	8.2	57.6	0.092	0.906	0.478	0.19
22.0	120.0	1	1.0	20.0	1	54	18	0.905	8.2	57.6	0.092	0.902	0.484	0.19
23.0	120.0	1	1.0	20.0	1	54	18	0.895	8.2	57.6	0.092	0.897	0.488	0.19
24.0	120.0	1	1.0	20.0	1	54	18	0.885	8.2	57.6	0.091	0.893	0.493	0.19
25.0	120.0	1	9.0	25.0	0	63		0.876	18.3	57.6	~	0.888	0.497	~
26.0	120.0	1	9.0	25.0	0	63		0.867	18.2	57.6	~	0.883	0.500	~
27.0	120.0	1	9.0	25.0	0	63		0.858	18.1	57.6	~	0.879	0.503	~
28.0	120.0	1	9.0	25.0	0	63		0.849	18.0	57.6	~	0.874	0.506	~
29.0	120.0	1	9.0	25.0	0	63		0.841	17.8	57.6	~	0.870	0.508	~
30.0	120.0	1	4.0	30.0	0	62		0.833	12.0	57.6	~	0.865	0.510	~
31.0	120.0	1	4.0	30.0	0	62		0.825	12.0	57.6	~	0.861	0.512	~
32.0	120.0	1	4.0	30.0	0	62		0.817	11.9	57.6	~	0.856	0.514	~
33.0	120.0	1	4.0	30.0	0	62		0.810	11.9	57.6	~	0.851	0.515	~
34.0	120.0	1	4.0	30.0	0	62		0.803	11.8	57.6	~	0.847	0.516	~
35.0	120.0	1	2.0	35.0	0	60		0.796	9.4	57.6	~	0.842	0.517	~
36.0	120.0	1	2.0	35.0	0	60		0.789	9.4	57.6	~	0.838	0.518	~
37.0	120.0	1	2.0	35.0	0	60		0.782	9.3	57.6	~	0.833	0.518	~
38.0	120.0	1	2.0	35.0	0	60		0.776	9.3	57.6	~	0.829	0.518	~
39.0	120.0	1	2.0	35.0	0	60		0.769	9.3	57.6	~	0.824	0.519	~
40.0	120.0	1	2.0	40.0	0	80		0.763	9.3	57.6	~	0.819	0.519	~
41.0	120.0	1	2.0	40.0	0	80		0.757	9.3	57.6	~	0.815	0.519	~
42.0	120.0	1	2.0	40.0	0	80		0.751	9.3	57.6	~	0.810	0.518	~
43.0	120.0	1	2.0	40.0	0	80		0.745	9.2	57.6	~	0.806	0.518	~
44.0	120.0	1	2.0	40.0	0	80		0.740	9.2	57.6	~	0.801	0.517	~
45.0	120.0	1	48.0	45.0	1	3	106	0.734	52.9	57.6	Infin.	0.797	0.517	Non-Liq.
46.0	120.0	1	48.0	45.0	1	3	106	0.729	52.5	57.6	Infin.	0.792	0.516	Non-Liq.
47.0	120.0	1	48.0	45.0	1	3	106	0.724	52.1	57.6	Infin.	0.787	0.515	Non-Liq.
48.0	120.0	1	48.0	45.0	1	3	106	0.718	51.7	57.6	Infin.	0.783	0.514	Non-Liq.
49.0	120.0	1	48.0	45.0	1	3	106	0.713	51.4	57.6	Infin.	0.778	0.513	Non-Liq.
50.0	120.0	1	10.0	50.0	0	63		0.708	17.6	57.6	~	0.774	0.512	~
51.0	120.0	1	10.0	50.0	0	63		0.703	17.6	57.6	~	0.769	0.511	~
52.0	120.0	1	10.0	50.0	0	63		0.699	17.5	57.6	~	0.765	0.509	~
53.0	120.0	1	18.0	50.0	0			0.694	18.7	57.6	~	0.760	0.508	~
54.0	120.0	1	18.0	50.0	0			0.689	18.6	57.6	~	0.755	0.507	~
55.0	120.0	1	18.0	55.0	0			0.685	18.5	57.6	~	0.751	0.505	~
56.0	120.0	1	18.0	55.0	0			0.681	18.4	57.6	~	0.746	0.504	~
57.0	120.0	1	18.0	55.0	0			0.676	18.3	57.6	~	0.742	0.502	~
58.0	120.0	1	18.0	55.0	0			0.672	18.1	57.6	~	0.737	0.500	~
59.0	120.0	1	18.0	55.0	0			0.668	18.0	57.6	~	0.733	0.499	~
60.0	120.0	1	55.0	60.0	0			0.664	54.8	57.6	~	0.728	0.497	~
61.0	120.0	1	55.0	60.0	0			0.660	54.4	57.6	~	0.723	0.495	~
62.0	120.0	1	55.0	60.0	0			0.656	54.1	57.6	~	0.719	0.493	~
63.0	120.0	1	55.0	60.0	0			0.652	53.8	57.6	~	0.714	0.491	~
64.0	120.0	1	55.0	60.0	0			0.648	53.5	57.6	~	0.710	0.489	~
65.0	120.0	1	11.0	65.0	0			0.645	10.6	57.6	~	0.705	0.487	~
66.0	120.0	1	11.0	65.0	0			0.641	10.6	57.6	~	0.701	0.485	~
67.0	120.0	1	11.0	65.0	0			0.637	10.5	57.6	~	0.696	0.483	~
68.0	120.0	1	11.0	65.0	0			0.634	10.5	57.6	~	0.691	0.481	~
69.0	120.0	1	11.0	65.0	0			0.630	10.4	57.6	~	0.687	0.478	~
70.0	120.0	1	100.0	70.0	0			0.627	94.0	57.6	~	0.682	0.476	~

GEOCON

File No. : W1208-06-01
Boring : 1

LIQUEFACTION SETTLEMENT ANALYSIS
MAXIMUM CONSIDERED EARTHQUAKE

(SATURATED SAND AT INITIAL LIQUEFACTION CONDITION)

NCEER (1996) METHOD

EARTHQUAKE INFORMATION:

Earthquake Magnitude:	6.82
PGA_M (g):	0.756
Calculated Mag.Wtg.Factor:	0.788
Historic High Groundwater:	10.0
Groundwater @ Exploration:	22.0

DEPTH TO BASE	BLOW COUNT N	WET DENSITY (PCF)	TOTAL STRESS σ (TSF)	EFFECT STRESS σ' (TSF)	REL. DEN. Dr (%)	ADJUST BLOWS (N1)60	Tav/σ'o	LIQUEFACTION SAFETY FACTOR	Volumetric Strain [e15] (%)	EQ. SETTLE. Pe (in.)
1	10	120	0.030	0.030		19	0.491	~	0.00	SUBTERRANEAN LEVEL
2	10	120	0.090	0.090		19	0.491	~	0.00	
3	10	120	0.150	0.150		19	0.491	~	0.00	
4	14	120	0.210	0.210		27	0.491	~	0.00	
5	14	120	0.270	0.270		27	0.491	~	0.00	
6	14	120	0.330	0.330		27	0.491	~	0.00	
7	14	120	0.390	0.390		26	0.491	~	0.00	
8	14	120	0.450	0.450		24	0.491	~	0.00	
9	14	120	0.510	0.510		23	0.491	~	0.00	
10	14	120	0.570	0.570	73	28	0.491	--	0.00	
11	14	120	0.630	0.614	73	27	0.504	0.87	1.10	
12	14	120	0.690	0.643	73	26	0.527	0.79	1.10	
13	14	120	0.750	0.672	73	26	0.548	0.73	1.10	
14	14	120	0.810	0.701	73	25	0.568	0.68	1.30	
15	8	120	0.870	0.730	53	18	0.586	0.44	1.70	
16	8	120	0.930	0.758	53	17	0.603	0.43	1.70	0.20
17	8	120	0.990	0.787	53	17	0.618	0.41	1.70	0.20
18	8	120	1.050	0.816	53	17	0.632	0.40	1.70	0.20
19	8	120	1.110	0.845	53	16	0.646	0.38	1.70	0.20
20	1	120	1.170	0.874	18	8	0.658	0.20	2.70	0.32
21	1	120	1.230	0.902	18	8	0.670	0.19	2.70	0.32
22	1	120	1.290	0.931	18	8	0.681	0.19	2.70	0.32
23	1	120	1.350	0.960	18	8	0.691	0.19	2.70	0.32
24	1	120	1.410	0.989	18	8	0.701	0.19	2.70	0.32
25	9	120	1.470	1.018		18	0.710	~	0.00	0.00
26	9	120	1.530	1.046		18	0.719	~	0.00	0.00
27	9	120	1.590	1.075		18	0.727	~	0.00	0.00
28	9	120	1.650	1.104		18	0.734	~	0.00	0.00
29	9	120	1.710	1.133		18	0.742	~	0.00	0.00
30	4	120	1.770	1.162		12	0.749	~	0.00	0.00
31	4	120	1.830	1.190		12	0.755	~	0.00	0.00
32	4	120	1.890	1.219		12	0.762	~	0.00	0.00
33	4	120	1.950	1.248		12	0.768	~	0.00	0.00
34	4	120	2.010	1.277		12	0.774	~	0.00	0.00
35	2	120	2.070	1.306		9	0.779	~	0.00	0.00
36	2	120	2.130	1.334		9	0.784	~	0.00	0.00
37	2	120	2.190	1.363		9	0.789	~	0.00	0.00
38	2	120	2.250	1.392		9	0.794	~	0.00	0.00
39	2	120	2.310	1.421		9	0.799	~	0.00	0.00
40	2	120	2.370	1.450		9	0.803	~	0.00	0.00
41	2	120	2.430	1.478		9	0.808	~	0.00	0.00
42	2	120	2.490	1.507		9	0.812	~	0.00	0.00
43	2	120	2.550	1.536		9	0.816	~	0.00	0.00
44	2	120	2.610	1.565		9	0.820	~	0.00	0.00
45	48	120	2.670	1.594	106	53	0.823	Non-Liq.	0.00	0.00
46	48	120	2.730	1.622	106	52	0.827	Non-Liq.	0.00	0.00
47	48	120	2.790	1.651	106	52	0.830	Non-Liq.	0.00	0.00
48	48	120	2.850	1.680	106	52	0.834	Non-Liq.	0.00	0.00
49	48	120	2.910	1.709	106	51	0.837	Non-Liq.	0.00	0.00
50	10	120	2.970	1.738		18	0.840	~	0.00	0.00
51	10	120	3.030	1.766		18	0.843	~	0.00	0.00
52	10	120	3.090	1.795		17	0.846	~	0.00	0.00
53	18	120	3.150	1.824		19	0.849	~	0.00	0.00
54	18	120	3.210	1.853		19	0.851	~	0.00	0.00
55	18	120	3.270	1.882		18	0.854	~	0.00	0.00
56	18	120	3.330	1.910		18	0.857	~	0.00	0.00
57	18	120	3.390	1.939		18	0.859	~	0.00	0.00
58	18	120	3.450	1.968		18	0.861	~	0.00	0.00
59	18	120	3.510	1.997		18	0.864	~	0.00	0.00
60	55	120	3.570	2.026		55	0.866	~	0.00	0.00
61	55	120	3.630	2.054		54	0.868	~	0.00	0.00
62	55	120	3.690	2.083		54	0.870	~	0.00	0.00
63	55	120	3.750	2.112		54	0.873	~	0.00	0.00
64	55	120	3.810	2.141		53	0.875	~	0.00	0.00
65	11	120	3.870	2.170		11	0.877	~	0.00	0.00
66	11	120	3.930	2.198		11	0.878	~	0.00	0.00
67	11	120	3.990	2.227		11	0.880	~	0.00	0.00
68	11	120	4.050	2.256		10	0.882	~	0.00	0.00
69	11	120	4.110	2.285		10	0.884	~	0.00	0.00
70	100	120	4.170	2.314		94	0.886	~	0.00	0.00

TOTAL SETTLEMENT = 2.4 INCHES

File No. : W1208-06-01
Boring : 1

EMPIRICAL ESTIMATION OF LIQUEFACTION POTENTIAL
MAXIMUM CONSIDERED EARTHQUAKE

NCEER (1996) METHOD

By Thomas F. Blake (1994-1996)

EARTHQUAKE INFORMATION:

Earthquake Magnitude:	6.82
Peak Horiz. Acceleration PGA_M (g):	0.756
Calculated Mag.Wtg.Factor:	0.788
Historic High Groundwater:	10.0
Groundwater Depth During Exploration:	22.0

ENERGY & ROD CORRECTIONS:

Energy Correction (CE) for N60:	1.25
Rod Len.Corr.(CR)(0-no or 1-yes):	1.0
Bore Dia. Corr. (CB):	1.00
Sampler Corr. (CS):	1.20
Use Ksigma (0 or 1):	1.0

LIQUEFACTION CALCULATIONS:

Unit Wt. Water (pcf):	62.4

Depth to Base (ft)	Total Unit Wt. (pcf)	Water (0 or 1)	FIELD SPT (N)	Depth of SPT (ft)	Liq.Sus. (0 or 1)	-200 (%)	Est. Dr (%)	CN Factor	Corrected (N1)60	Eff. Unit Wt. (psf)	Resist. CRR	rd Factor	Induced CSR	Liquefac. Safe.Fact.
1.0	120.0	0	10.0	1.0	0			1.700	19.1	120.0	~	0.998	0.386	~
2.0	120.0	0	10.0	2.0	0			1.700	19.1	120.0	~	0.993	0.384	~
3.0	120.0	0	10.0	3.0	0			1.700	19.1	120.0	~	0.989	0.383	~
4.0	120.0	0	14.0	4.0	0			1.700	26.8	120.0	~	0.984	0.381	~
5.0	120.0	0	14.0	5.0	0			1.700	26.8	120.0	~	0.979	0.379	~
6.0	120.0	0	14.0	6.0	0			1.700	26.8	120.0	~	0.975	0.377	~
7.0	120.0	0	14.0	7.0	0			1.636	25.8	120.0	~	0.970	0.376	~
8.0	120.0	0	14.0	8.0	0			1.523	24.0	120.0	~	0.966	0.374	~
9.0	120.0	0	14.0	9.0	0			1.431	22.5	120.0	~	0.961	0.372	~
10.0	120.0	0	14.0	10.0	1	54	73	1.353	28.3	120.0	0.359	0.957	0.370	--
11.0	120.0	1	14.0	10.0	1	54	73	1.287	27.3	57.6	0.330	0.952	0.378	0.87
12.0	120.0	1	14.0	10.0	1	54	73	1.230	26.4	57.6	0.311	0.947	0.393	0.79
13.0	120.0	1	14.0	10.0	1	54	73	1.180	25.6	57.6	0.296	0.943	0.407	0.73
14.0	120.0	1	14.0	10.0	1	54	73	1.135	24.9	57.6	0.284	0.938	0.420	0.68
15.0	120.0	1	8.0	15.0	1	60	53	1.095	17.6	57.6	0.191	0.934	0.431	0.44
16.0	120.0	1	8.0	15.0	1	60	53	1.060	17.3	57.6	0.188	0.929	0.441	0.43
17.0	120.0	1	8.0	15.0	1	60	53	1.027	16.9	57.6	0.184	0.925	0.450	0.41
18.0	120.0	1	8.0	15.0	1	60	53	0.997	16.7	57.6	0.181	0.920	0.458	0.40
19.0	120.0	1	8.0	15.0	1	60	53	0.970	16.4	57.6	0.178	0.915	0.466	0.38
20.0	120.0	1	1.0	20.0	1	54	18	0.945	8.3	57.6	0.092	0.911	0.472	0.20
21.0	120.0	1	1.0	20.0	1	54	18	0.921	8.2	57.6	0.092	0.906	0.478	0.19
22.0	120.0	1	1.0	20.0	1	54	18	0.905	8.2	57.6	0.092	0.902	0.484	0.19
23.0	120.0	1	1.0	20.0	1	54	18	0.895	8.2	57.6	0.092	0.897	0.488	0.19
24.0	120.0	1	1.0	20.0	1	54	18	0.885	8.2	57.6	0.091	0.893	0.493	0.19
25.0	120.0	1	9.0	25.0	0	63		0.876	18.3	57.6	~	0.888	0.497	~
26.0	120.0	1	9.0	25.0	0	63		0.867	18.2	57.6	~	0.883	0.500	~
27.0	120.0	1	9.0	25.0	0	63		0.858	18.1	57.6	~	0.879	0.503	~
28.0	120.0	1	9.0	25.0	0	63		0.849	18.0	57.6	~	0.874	0.506	~
29.0	120.0	1	9.0	25.0	0	63		0.841	17.8	57.6	~	0.870	0.508	~
30.0	120.0	1	4.0	30.0	0	62		0.833	12.0	57.6	~	0.865	0.510	~
31.0	120.0	1	4.0	30.0	0	62		0.825	12.0	57.6	~	0.861	0.512	~
32.0	120.0	1	4.0	30.0	0	62		0.817	11.9	57.6	~	0.856	0.514	~
33.0	120.0	1	4.0	30.0	0	62		0.810	11.9	57.6	~	0.851	0.515	~
34.0	120.0	1	4.0	30.0	0	62		0.803	11.8	57.6	~	0.847	0.516	~
35.0	120.0	1	2.0	35.0	0	60		0.796	9.4	57.6	~	0.842	0.517	~
36.0	120.0	1	2.0	35.0	0	60		0.789	9.4	57.6	~	0.838	0.518	~
37.0	120.0	1	2.0	35.0	0	60		0.782	9.3	57.6	~	0.833	0.518	~
38.0	120.0	1	2.0	35.0	0	60		0.776	9.3	57.6	~	0.829	0.518	~
39.0	120.0	1	2.0	35.0	0	60		0.769	9.3	57.6	~	0.824	0.519	~
40.0	120.0	1	2.0	40.0	0	80		0.763	9.3	57.6	~	0.819	0.519	~
41.0	120.0	1	2.0	40.0	0	80		0.757	9.3	57.6	~	0.815	0.519	~
42.0	120.0	1	2.0	40.0	0	80		0.751	9.3	57.6	~	0.810	0.518	~
43.0	120.0	1	2.0	40.0	0	80		0.745	9.2	57.6	~	0.806	0.518	~
44.0	120.0	1	2.0	40.0	0	80		0.740	9.2	57.6	~	0.801	0.517	~
45.0	120.0	1	48.0	45.0	1	3	106	0.734	52.9	57.6	Infin.	0.797	0.517	Non-Liq.
46.0	120.0	1	48.0	45.0	1	3	106	0.729	52.5	57.6	Infin.	0.792	0.516	Non-Liq.
47.0	120.0	1	48.0	45.0	1	3	106	0.724	52.1	57.6	Infin.	0.787	0.515	Non-Liq.
48.0	120.0	1	48.0	45.0	1	3	106	0.718	51.7	57.6	Infin.	0.783	0.514	Non-Liq.
49.0	120.0	1	48.0	45.0	1	3	106	0.713	51.4	57.6	Infin.	0.778	0.513	Non-Liq.
50.0	120.0	1	10.0	50.0	0	63		0.708	17.6	57.6	~	0.774	0.512	~
51.0	120.0	1	10.0	50.0	0	63		0.703	17.6	57.6	~	0.769	0.511	~
52.0	120.0	1	10.0	50.0	0	63		0.699	17.5	57.6	~	0.765	0.509	~
53.0	120.0	1	18.0	50.0	0			0.694	18.7	57.6	~	0.760	0.508	~
54.0	120.0	1	18.0	50.0	0			0.689	18.6	57.6	~	0.755	0.507	~
55.0	120.0	1	18.0	55.0	0			0.685	18.5	57.6	~	0.751	0.505	~
56.0	120.0	1	18.0	55.0	0			0.681	18.4	57.6	~	0.746	0.504	~
57.0	120.0	1	18.0	55.0	0			0.676	18.3	57.6	~	0.742	0.502	~
58.0	120.0	1	18.0	55.0	0			0.672	18.1	57.6	~	0.737	0.500	~
59.0	120.0	1	18.0	55.0	0			0.668	18.0	57.6	~	0.733	0.499	~
60.0	120.0	1	55.0	60.0	0			0.664	54.8	57.6	~	0.728	0.497	~
61.0	120.0	1	55.0	60.0	0			0.660	54.4	57.6	~	0.723	0.495	~
62.0	120.0	1	55.0	60.0	0			0.656	54.1	57.6	~	0.719	0.493	~
63.0	120.0	1	55.0	60.0	0			0.652	53.8	57.6	~	0.714	0.491	~
64.0	120.0	1	55.0	60.0	0			0.648	53.5	57.6	~	0.710	0.489	~
65.0	120.0	1	11.0	65.0	0			0.645	10.6	57.6	~	0.705	0.487	~
66.0	120.0	1	11.0	65.0	0			0.641	10.6	57.6	~	0.701	0.485	~
67.0	120.0	1	11.0	65.0	0			0.637	10.5	57.6	~	0.696	0.483	~
68.0	120.0	1	11.0	65.0	0			0.634	10.5	57.6	~	0.691	0.481	~
69.0	120.0	1	11.0	65.0	0			0.630	10.4	57.6	~	0.687	0.478	~
70.0	120.0	1	100.0	70.0	0			0.627	94.0	57.6	~	0.682	0.476	~

GEOCON

File No. : W1208-06-01
Boring : 1

LIQUEFACTION SETTLEMENT ANALYSIS
MAXIMUM CONSIDERED EARTHQUAKE

(SATURATED SAND AT INITIAL LIQUEFACTION CONDITION)

NCEER (1996) METHOD
EARTHQUAKE INFORMATION:

Earthquake Magnitude:	6.82
PGA_M (g):	0.756
Calculated Mag.Wtg.Factor:	0.788
Historic High Groundwater:	10.0
Groundwater @ Exploration:	22.0

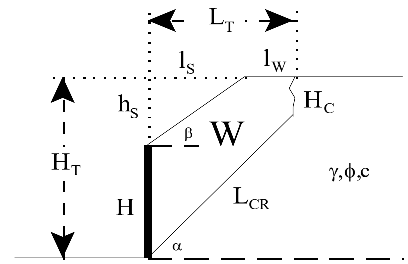
DEPTH TO BASE	BLOW COUNT N	WET DENSITY (PCF)	TOTAL STRESS O (TSF)	EFFECT STRESS O' (TSF)	REL. DEN. Dr (%)	ADJUST BLOWS (N1)60	Tav/σ'o	LIQUEFACTION SAFETY FACTOR	Volumetric Strain [e_{15}} (%)	EQ. SETTLE. Pe (in.)
1	10	120	0.030	0.030		19	0.491	~	0.00	SUBTERRANEAN LEVEL
2	10	120	0.090	0.090		19	0.491	~	0.00	
3	10	120	0.150	0.150		19	0.491	~	0.00	
4	14	120	0.210	0.210		27	0.491	~	0.00	
5	14	120	0.270	0.270		27	0.491	~	0.00	
6	14	120	0.330	0.330		27	0.491	~	0.00	
7	14	120	0.390	0.390		26	0.491	~	0.00	
8	14	120	0.450	0.450		24	0.491	~	0.00	
9	14	120	0.510	0.510		23	0.491	~	0.00	
10	14	120	0.570	0.570	73	28	0.491	--	0.00	
11	14	120	0.630	0.614	73	27	0.504	0.87	1.10	
12	14	120	0.690	0.643	73	26	0.527	0.79	1.10	
13	14	120	0.750	0.672	73	26	0.548	0.73	1.10	
14	14	120	0.810	0.701	73	25	0.568	0.68	1.30	
15	8	120	0.870	0.730	53	18	0.586	0.44	1.70	
16	8	120	0.930	0.758	53	17	0.603	0.43	1.70	
17	8	120	0.990	0.787	53	17	0.618	0.41	1.70	
18	8	120	1.050	0.816	53	17	0.632	0.40	1.70	
19	8	120	1.110	0.845	53	16	0.646	0.38	1.70	
20	1	120	1.170	0.874	18	8	0.658	0.20	2.70	
21	1	120	1.230	0.902	18	8	0.670	0.19	2.70	
22	1	120	1.290	0.931	18	8	0.681	0.19	2.70	
23	1	120	1.350	0.960	18	8	0.691	0.19	2.70	
24	1	120	1.410	0.989	18	8	0.701	0.19	2.70	
25	9	120	1.470	1.018		18	0.710	~	0.00	
26	9	120	1.530	1.046		18	0.719	~	0.00	
27	9	120	1.590	1.075		18	0.727	~	0.00	
28	9	120	1.650	1.104		18	0.734	~	0.00	
29	9	120	1.710	1.133		18	0.742	~	0.00	
30	4	120	1.770	1.162		12	0.749	~	0.00	
31	4	120	1.830	1.190		12	0.755	~	0.00	
32	4	120	1.890	1.219		12	0.762	~	0.00	
33	4	120	1.950	1.248		12	0.768	~	0.00	
34	4	120	2.010	1.277		12	0.774	~	0.00	
35	2	120	2.070	1.306		9	0.779	~	0.00	
36	2	120	2.130	1.334		9	0.784	~	0.00	
37	2	120	2.190	1.363		9	0.789	~	0.00	
38	2	120	2.250	1.392		9	0.794	~	0.00	
39	2	120	2.310	1.421		9	0.799	~	0.00	0.00
40	2	120	2.370	1.450		9	0.803	~	0.00	0.00
41	2	120	2.430	1.478		9	0.808	~	0.00	0.00
42	2	120	2.490	1.507		9	0.812	~	0.00	0.00
43	2	120	2.550	1.536		9	0.816	~	0.00	0.00
44	2	120	2.610	1.565		9	0.820	~	0.00	0.00
45	48	120	2.670	1.594	106	53	0.823	Non-Liq.	0.00	0.00
46	48	120	2.730	1.622	106	52	0.827	Non-Liq.	0.00	0.00
47	48	120	2.790	1.651	106	52	0.830	Non-Liq.	0.00	0.00
48	48	120	2.850	1.680	106	52	0.834	Non-Liq.	0.00	0.00
49	48	120	2.910	1.709	106	51	0.837	Non-Liq.	0.00	0.00
50	10	120	2.970	1.738		18	0.840	~	0.00	0.00
51	10	120	3.030	1.766		18	0.843	~	0.00	0.00
52	10	120	3.090	1.795		17	0.846	~	0.00	0.00
53	18	120	3.150	1.824		19	0.849	~	0.00	0.00
54	18	120	3.210	1.853		19	0.851	~	0.00	0.00
55	18	120	3.270	1.882		18	0.854	~	0.00	0.00
56	18	120	3.330	1.910		18	0.857	~	0.00	0.00
57	18	120	3.390	1.939		18	0.859	~	0.00	0.00
58	18	120	3.450	1.968		18	0.861	~	0.00	0.00
59	18	120	3.510	1.997		18	0.864	~	0.00	0.00
60	55	120	3.570	2.026		55	0.866	~	0.00	0.00
61	55	120	3.630	2.054		54	0.868	~	0.00	0.00
62	55	120	3.690	2.083		54	0.870	~	0.00	0.00
63	55	120	3.750	2.112		54	0.873	~	0.00	0.00
64	55	120	3.810	2.141		53	0.875	~	0.00	0.00
65	11	120	3.870	2.170		11	0.877	~	0.00	0.00
66	11	120	3.930	2.198		11	0.878	~	0.00	0.00
67	11	120	3.990	2.227		11	0.880	~	0.00	0.00
68	11	120	4.050	2.256		10	0.882	~	0.00	0.00
69	11	120	4.110	2.285		10	0.884	~	0.00	0.00
70	100	120	4.170	2.314		94	0.886	~	0.00	0.00

TOTAL SETTLEMENT = 0.0 INCHES

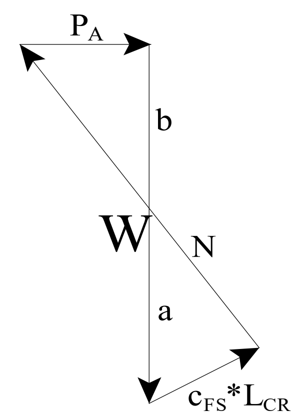
Retaining Wall Design with Transitioned Backfill
(Vector Analysis)

Input:

Retaining Wall Height	(H)	20.00	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)	20.0	feet
Unit Weight of Retained Soils	(g)	125.0	pcf
Friction Angle of Retained Soils	(f)	25.4	degrees
Cohesion of Retained Soils	(c)	367.0	psf
Factor of Safety	(FS)	1.50	
Factored Parameters:	(f_{FS})	17.6	degrees
	(c_{FS})	244.7	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	5.7	184	22949.5	20.2	10218.6	12730.9	6608.8
46	5.6	178	22221.2	20.0	9778.2	12443.0	6737.6
47	5.6	172	21506.1	19.7	9366.6	12139.5	6849.9
48	5.5	166	20804.4	19.5	8981.4	11823.0	6946.1
49	5.5	161	20116.1	19.3	8620.4	11495.6	7026.5
50	5.4	156	19441.0	19.0	8281.7	11159.3	7091.3
51	5.4	150	18778.8	18.8	7963.3	10815.4	7140.8
52	5.4	145	18129.2	18.6	7663.7	10465.5	7175.1
53	5.3	140	17491.7	18.3	7381.1	10110.6	7194.4
54	5.3	135	16866.0	18.1	7114.3	9751.7	7198.6
55	5.4	130	16251.5	17.9	6861.9	9389.6	7187.9
56	5.4	125	15647.8	17.6	6622.7	9025.1	7162.1
57	5.4	120	15054.3	17.4	6395.4	8658.8	7121.2
58	5.4	116	14470.5	17.2	6179.2	8291.3	7065.0
59	5.5	111	13895.8	16.9	5972.9	7922.9	6993.5
60	5.5	107	13329.8	16.7	5775.6	7554.2	6906.2
61	5.6	102	12771.9	16.5	5586.5	7185.4	6803.1
62	5.7	98	12221.5	16.2	5404.6	6816.9	6683.7
63	5.8	93	11678.1	16.0	5229.1	6449.0	6547.6
64	5.9	89	11141.2	15.7	5059.2	6082.0	6394.5
65	6.0	85	10610.2	15.5	4894.0	5716.2	6223.8
66	6.1	81	10084.4	15.2	4732.7	5351.7	6035.1
67	6.3	77	9563.3	14.9	4574.4	4989.0	5827.8
68	6.5	72	9046.3	14.6	4418.1	4628.2	5601.4
69	6.7	68	8532.6	14.3	4262.9	4269.7	5355.1
70	6.9	64	8021.5	14.0	4107.7	3913.8	5088.5



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

$P_{A,\ max}$ 7198.6 lbs/lineal foot

Equivalent Fluid Pressure (per lineal foot of wall)

$EFP = 2*P_A/H^2$

EFP 36.0 pcf 71.4 pcf

Design Wall for an Equivalent Fluid Pressure **36 pcf** 71 pcf

Active At-Rest

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RETAINING WALL PRESSURE

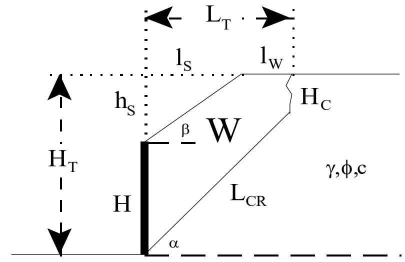
12805-12825 WEST VENTURA BOULEVARD
STUDIO CITY, CALIFORNIA

DRAFTED BY: JJK	CHECKED BY: NDB	JUNE 2021	PROJECT NO. W1208-06-01	FIG. 16a

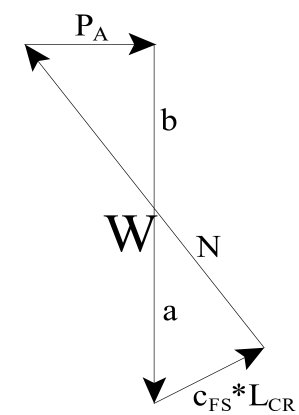
Retaining Wall Design with Transitioned Backfill
(Vector Analysis)

Input:

Retaining Wall Height	(H)	35.00	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)	35.0	feet
Unit Weight of Retained Soils	(g)	125.0	pcf
Friction Angle of Retained Soils	(f)	25.4	degrees
Cohesion of Retained Soils	(c)	367.0	psf
Factor of Safety	(FS)	1.50	
Factored Parameters:	(f_{FS})	17.6	degrees
	(c_{FS})	244.7	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	5.7	596	74512.0	41.4	20958.3	53553.6	27800.5
46	5.6	576	72014.5	40.8	19993.4	52021.1	28168.2
47	5.6	557	69588.9	40.2	19101.7	50487.2	28488.1
48	5.5	538	67231.5	39.7	18276.0	48955.5	28761.6
49	5.5	520	64938.7	39.1	17509.9	47428.8	28989.8
50	5.4	502	62707.0	38.6	16797.7	45909.3	29173.7
51	5.4	484	60533.3	38.1	16134.6	44398.7	29313.9
52	5.4	467	58414.2	37.6	15515.9	42898.3	29411.0
53	5.3	451	56346.9	37.1	14937.7	41409.2	29465.5
54	5.3	435	54328.4	36.7	14396.4	39932.0	29477.5
55	5.4	419	52356.0	36.2	13888.8	38467.2	29447.0
56	5.4	403	50427.1	35.7	13412.0	37015.1	29374.1
57	5.4	388	48539.4	35.3	12963.4	35576.0	29258.3
58	5.4	374	46690.3	34.9	12540.5	34149.8	29099.2
59	5.5	359	44877.7	34.4	12141.1	32736.6	28896.2
60	5.5	345	43099.4	34.0	11763.3	31336.1	28648.4
61	5.6	331	41353.4	33.6	11405.1	29948.3	28354.9
62	5.7	317	39637.8	33.2	11064.9	28572.9	28014.4
63	5.8	304	37950.6	32.8	10740.9	27209.6	27625.5
64	5.9	290	36289.9	32.4	10431.7	25858.2	27186.6
65	6.0	277	34654.2	32.0	10135.8	24518.4	26695.8
66	6.1	264	33041.5	31.6	9851.6	23189.9	26151.1
67	6.3	252	31450.3	31.2	9577.9	21872.4	25550.0
68	6.5	239	29878.9	30.8	9313.3	20565.6	24890.0
69	6.7	227	28325.6	30.4	9056.1	19269.4	24168.2
70	6.9	214	26788.7	29.9	8805.1	17983.6	23381.3



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

$P_{A,\ max}$ 29477.5 lbs/lineal foot

Equivalent Fluid Pressure (per lineal foot of wall)

$EFP = 2*P_A/H^2$

EFP 48.1 pcf 71.4 pcf

Design Wall for an Equivalent Fluid Pressure **48 pcf** 71 pcf

Active At-Rest

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RETAINING WALL PRESSURE

12805-12825 WEST VENTURA BOULEVARD

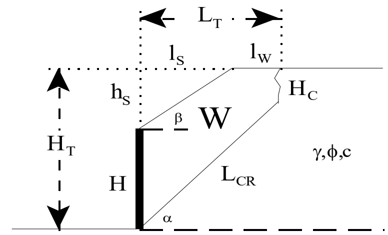
STUDIO CITY, CALIFORNIA

JUNE 2021 | PROJECT NO. W1208-06-01 | FIG. 16b

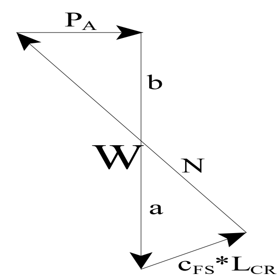
Retaining Wall Design with Transitioned Backfill
(Vector Analysis)

Input:

Retaining Wall Height	(H)	45.00 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)	45.0 feet
Unit Weight of Retained Soils	(g)	125.0 pcf
Friction Angle of Retained Soils	(f)	25.4 degrees
Cohesion of Retained Soils	(c)	367.0 psf
Factor of Safety	(FS)	1.50
Factored Parameters:	(f_{FS})	17.6 degrees
	(c_{FS})	244.7 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	5.7	996	124512.0	55.5	28118.2	96393.8	50039.4
46	5.6	962	120298.9	54.7	26803.6	93495.4	50625.5
47	5.6	930	116214.7	53.9	25591.8	90622.8	51135.2
48	5.5	898	112251.7	53.1	24472.4	87779.3	51570.8
49	5.5	867	108403.0	52.4	23436.2	84966.8	51934.2
50	5.4	837	104662.0	51.7	22475.1	82186.9	52226.8
51	5.4	808	101022.5	51.0	21582.1	79440.4	52449.9
52	5.4	780	97478.5	50.3	20750.7	76727.8	52604.5
53	5.3	752	94024.6	49.6	19975.4	74049.2	52691.1
54	5.3	725	90655.5	49.0	19251.1	71404.4	52710.2
55	5.4	699	87366.4	48.4	18573.4	68792.9	52661.8
56	5.4	673	84152.6	47.8	17938.3	66214.3	52545.7
57	5.4	648	81009.7	47.2	17342.0	63667.7	52361.4
58	5.4	623	77933.7	46.7	16781.3	61152.4	52108.2
59	5.5	599	74920.7	46.1	16253.3	58667.4	51785.1
60	5.5	576	71966.9	45.6	15755.1	56211.9	51390.5
61	5.6	553	69068.9	45.0	15284.2	53784.7	50923.0
62	5.7	530	66223.3	44.5	14838.4	51384.9	50380.4
63	5.8	507	63426.8	44.0	14415.5	49011.4	49760.4
64	5.9	485	60676.6	43.5	14013.4	46663.2	49060.2
65	6.0	464	57969.5	43.0	13630.3	44339.3	48276.9
66	6.1	442	55302.9	42.5	13264.3	42038.7	47406.7
67	6.3	421	52674.0	42.1	12913.7	39760.4	46445.7
68	6.5	401	50080.2	41.6	12576.7	37503.5	45389.5
69	6.7	380	47518.8	41.1	12251.6	35267.1	44232.9
70	6.9	360	44987.2	40.6	11936.7	33050.5	42970.5



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

$P_{A,\ max}$ 52710.2 lbs/lineal foot

Equivalent Fluid Pressure (per lineal foot of wall)

$EFP = 2*P_A/H^2$

EFP 52.1 pcf 71.4 pcf

Design Wall for an Equivalent Fluid Pressure: **52 pcf** Active 71 pcf At-Rest

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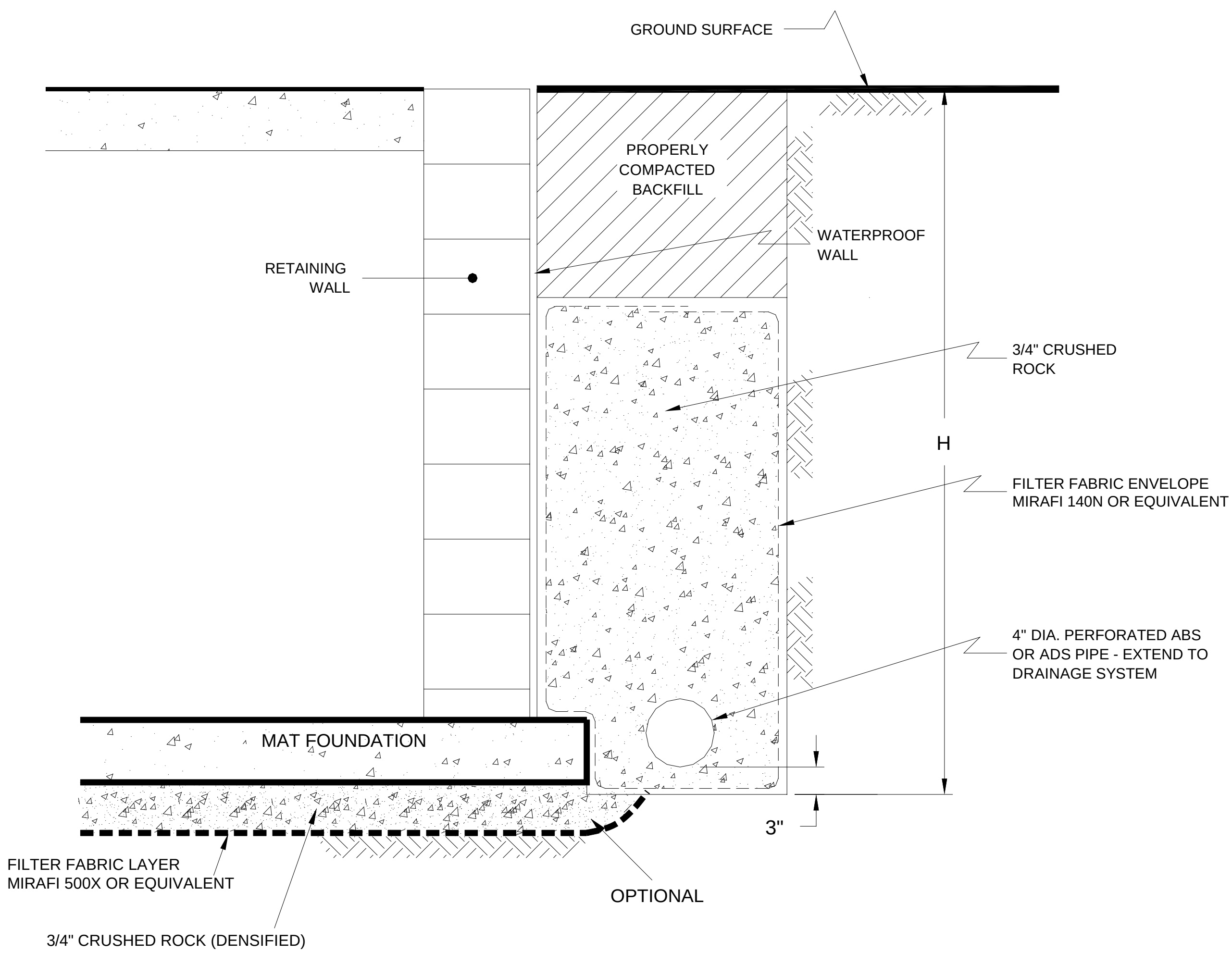
RETAINING WALL PRESSURE

12805-12825 WEST VENTURA BOULEVARD

STUDIO CITY, CALIFORNIA

DRAFTED BY: JJK	CHECKED BY: NDB	JUNE 2021	PROJECT NO. W1208-06-01	FIG. 16c



NO SCALE

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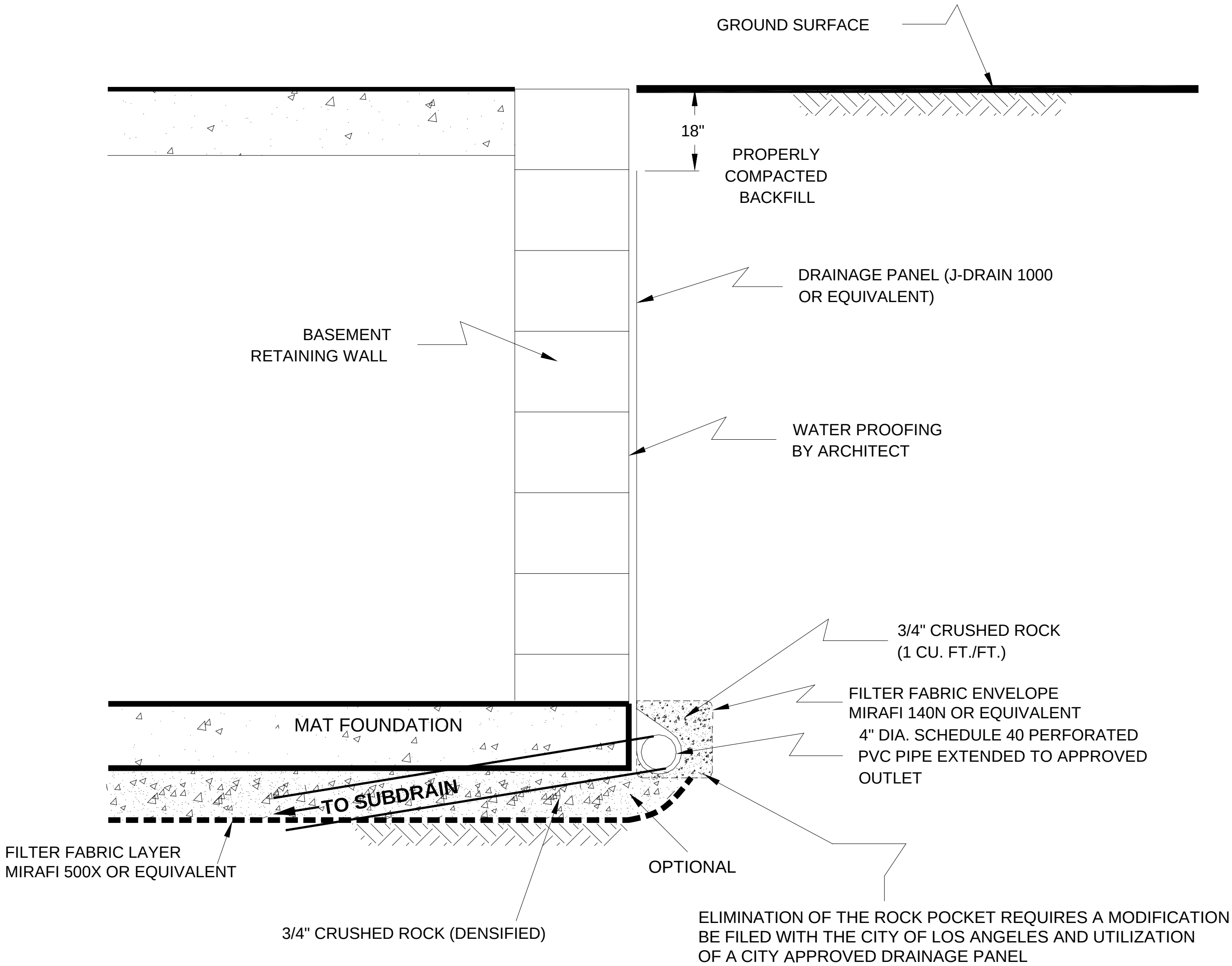
RETAINING WALL DRAIN DETAIL

12805-12825 WEST VENTURA BOULEVARD
STUDIO CITY, CALIFORNIA

JUNE 2021 | PROJECT NO. W1208-06-01 | FIG. 17



NOTE: TOP OF DRAINAGE PANEL NOT MORE THAN 18 INCHES FROM GROUND SURFACE

NO SCALE

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RETAINING WALL DRAIN DETAIL

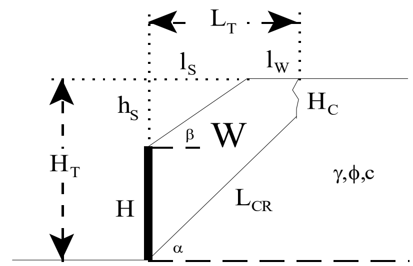
12805-12825 WEST VENTURA BOULEVARD
STUDIO CITY, CALIFORNIA

JUNE 2021 | PROJECT NO. W1208-06-01 | FIG. 18

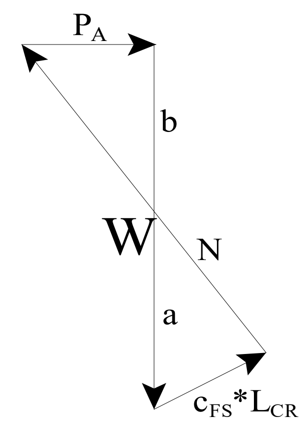
Shoring Design with Transitioned Backfill
(Vector Analysis)

Input:

Shoring Height	(H)	15.00	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)	15.0	feet
Unit Weight of Retained Soils	(g)	125.0	pcf
Friction Angle of Retained Soils	(f)	25.4	degrees
Cohesion of Retained Soils	(c)	367.0	psf
Factor of Safety	(FS)	1.25	
Factored Parameters:	(f_{FS})	20.8	degrees
	(c_{FS})	293.6	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	7.6	84	10476.1	10.5	7030.5	3445.5	1548.5
46	7.4	82	10253.7	10.5	6789.3	3464.4	1630.2
47	7.3	80	10014.3	10.5	6551.7	3462.5	1703.8
48	7.2	78	9761.7	10.5	6319.4	3442.3	1769.1
49	7.1	76	9499.0	10.5	6093.2	3405.8	1826.2
50	7.0	74	9228.7	10.4	5873.9	3354.8	1874.9
51	6.9	72	8952.6	10.4	5661.6	3291.0	1915.4
52	6.9	69	8672.3	10.3	5456.4	3215.9	1947.6
53	6.8	67	8389.0	10.2	5258.2	3130.8	1971.5
54	6.8	65	8103.6	10.1	5066.8	3036.8	1987.2
55	6.8	63	7816.7	10.0	4881.7	2935.0	1994.6
56	6.8	60	7529.1	9.9	4702.7	2826.4	1993.8
57	6.8	58	7241.1	9.7	4529.3	2711.8	1984.7
58	6.9	56	6952.9	9.6	4360.9	2592.0	1967.4
59	6.9	53	6664.8	9.5	4197.2	2467.6	1941.8
60	6.9	51	6376.9	9.3	4037.5	2339.4	1908.0
61	7.0	49	6089.2	9.1	3881.3	2207.9	1865.8
62	7.1	46	5801.7	8.9	3728.0	2073.7	1815.4
63	7.2	44	5514.3	8.8	3576.9	1937.4	1756.7
64	7.3	42	5226.7	8.5	3427.3	1799.4	1689.7
65	7.5	40	4938.9	8.3	3278.6	1660.3	1614.5
66	7.6	37	4650.4	8.1	3129.9	1520.5	1531.2
67	7.8	35	4361.0	7.8	2980.3	1380.7	1439.8
68	8.0	33	4070.2	7.6	2828.8	1241.4	1340.6
69	8.2	30	3777.4	7.3	2674.2	1103.2	1233.9
70	8.5	28	3482.2	6.9	2515.4	966.8	1120.1



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

$P_{A,\ max}$ 1994.6 lbs/lineal foot

Equivalent Fluid Pressure (per lineal foot of shoring)

$EFP = 2*P_A/H^2$

EFP 17.7 pcf

Design Shoring for an Equivalent Fluid Pressure: **25 pcf**
Active

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SHORING PRESSURE CALCULATION

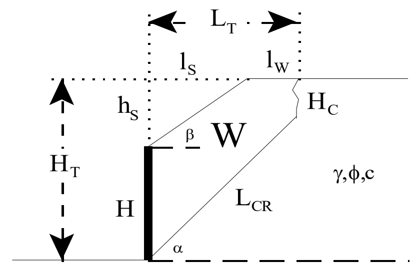
12805-12825 WEST VENTURA BOULEVARD
STUDIO CITY, CALIFORNIA

JUNE 2021 | PROJECT NO. W1208-06-01 | FIG. 19a

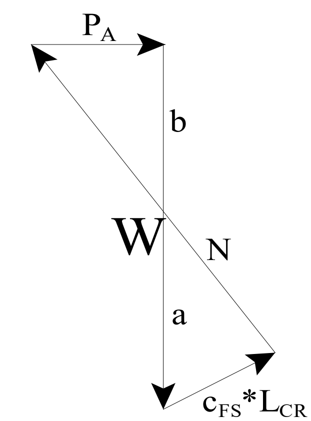
Shoring Design with Transitioned Backfill
(Vector Analysis)

Input:

Shoring Height	(H)	35.00 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)	35.0 feet
Unit Weight of Retained Soils	(g)	125.0 pcf
Friction Angle of Retained Soils	(f)	25.4 degrees
Cohesion of Retained Soils	(c)	367.0 psf
Factor of Safety	(FS)	1.25
Factored Parameters:	(f_{FS})	20.8 degrees
	(c_{FS})	293.6 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	7.6	584	72976.1	38.8	25968.4	47007.6	21125.9
46	7.4	565	70609.2	38.3	24711.9	45897.4	21597.5
47	7.3	546	68296.5	37.9	23552.0	44744.5	22016.9
48	7.2	528	66036.9	37.4	22479.2	43557.8	22385.5
49	7.1	511	63829.4	37.0	21485.1	42344.4	22704.7
50	7.0	493	61672.4	36.5	20562.1	41110.3	22975.6
51	6.9	477	59564.1	36.1	19703.7	39860.5	23199.2
52	6.9	460	57502.7	35.7	18903.7	38599.0	23376.2
53	6.8	444	55486.1	35.3	18156.9	37329.3	23507.3
54	6.8	428	53512.5	34.8	17458.4	36054.1	23593.0
55	6.8	413	51579.7	34.4	16803.8	34775.9	23633.5
56	6.8	397	49685.9	34.0	16189.4	33496.5	23629.0
57	6.8	383	47829.1	33.6	15611.6	32217.5	23579.5
58	6.9	368	46007.3	33.2	15067.0	30940.2	23484.8
59	6.9	354	44218.6	32.8	14552.8	29665.8	23344.5
60	6.9	340	42461.3	32.4	14066.3	28395.0	23158.2
61	7.0	326	40733.6	32.0	13605.0	27128.6	22925.3
62	7.1	312	39033.6	31.6	13166.5	25867.1	22644.8
63	7.2	299	37359.6	31.2	12748.5	24611.1	22315.8
64	7.3	286	35710.0	30.8	12349.2	23360.8	21937.2
65	7.5	273	34083.1	30.4	11966.3	22116.8	21507.5
66	7.6	260	32477.2	30.0	11598.1	20879.1	21025.2
67	7.8	247	30890.6	29.6	11242.5	19648.1	20488.8
68	8.0	235	29321.8	29.1	10897.7	18424.1	19896.1
69	8.2	222	27769.0	28.7	10561.6	17207.3	19245.2
70	8.5	210	26230.4	28.2	10232.2	15998.1	18533.9



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

$P_{A,\ max}$ 23633.5 lbs/lineal foot

Equivalent Fluid Pressure (per lineal foot of shoring)

$EFP = 2*P_A/H^2$

EFP 38.6 pcf

Design Shoring for an Equivalent Fluid Pressure: **39 pcf**
Active

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SHORING PRESSURE CALCULATION

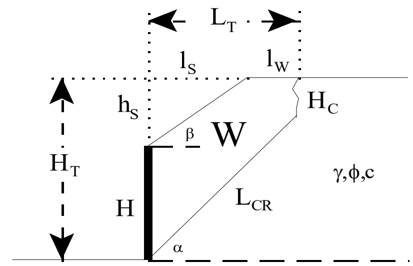
12805-12825 WEST VENTURA BOULEVARD
STUDIO CITY, CALIFORNIA

DRAFTED BY: JJK	CHECKED BY: NDB	JUNE 2021	PROJECT NO. W1208-06-01	FIG. 19b

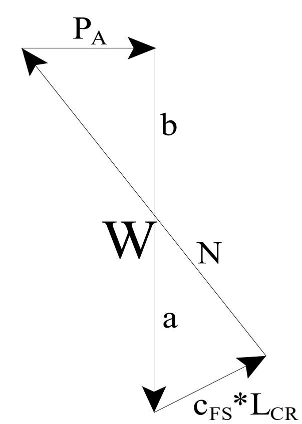
Shoring Design with Transitioned Backfill
(Vector Analysis)

Input:

Shoring Height	(H)	50.00	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)	50.0	feet
Unit Weight of Retained Soils	(g)	125.0	pcf
Friction Angle of Retained Soils	(f)	25.4	degrees
Cohesion of Retained Soils	(c)	367.0	psf
Factor of Safety	(FS)	1.25	
Factored Parameters:	(f_{FS})	20.8	degrees
	(c_{FS})	293.6	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	7.6	1221	152663.6	60.0	40171.9	112491.7	50555.3
46	7.4	1181	147562.6	59.2	38153.8	109408.7	51483.4
47	7.3	1141	142606.3	58.4	36302.1	106304.1	52307.7
48	7.2	1102	137787.9	57.6	34599.0	103188.9	53031.5
49	7.1	1065	133100.7	56.9	33028.9	100071.8	53657.6
50	7.0	1028	128538.2	56.1	31578.3	96959.9	54188.6
51	6.9	993	124093.8	55.4	30235.2	93858.6	54626.6
52	6.9	958	119761.4	54.7	28989.1	90772.2	54973.2
53	6.8	924	115535.0	54.0	27830.8	87704.1	55229.9
54	6.8	891	111408.8	53.4	26752.0	84656.8	55397.5
55	6.8	859	107377.5	52.7	25745.4	81632.1	55476.7
56	6.8	827	103435.8	52.1	24804.5	78631.3	55467.9
57	6.8	797	99578.7	51.5	23923.3	75655.5	55371.1
58	6.9	766	95801.5	50.9	23096.6	72705.0	55185.7
59	6.9	737	92099.7	50.3	22319.6	69780.1	54911.2
60	6.9	708	88468.9	49.7	21588.0	66881.0	54546.4
61	7.0	679	84905.1	49.1	20897.8	64007.3	54090.0
62	7.1	651	81404.2	48.6	20245.3	61158.8	53540.2
63	7.2	624	77962.4	48.0	19627.3	58335.1	52894.7
64	7.3	597	74576.2	47.5	19040.5	55535.7	52151.1
65	7.5	570	71242.0	46.9	18482.1	52759.9	51306.4
66	7.6	544	67956.3	46.4	17949.2	50007.1	50357.1
67	7.8	518	64716.0	45.9	17439.2	47276.8	49299.5
68	8.0	492	61517.6	45.3	16949.4	44568.3	48129.0
69	8.2	467	58358.1	44.8	16477.1	41881.0	46841.0
70	8.5	442	55234.2	44.2	16019.8	39214.4	45430.0



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

$P_{A,\ max}$ 55476.7 lbs/lineal foot

Equivalent Fluid Pressure (per lineal foot of shoring)

$EFP = 2*P_A/H^2$

EFP 44.4 pcf

Design Shoring for an Equivalent Fluid Pressure: **44 pcf**

Active

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SHORING PRESSURE CALCULATION

12805-12825 WEST VENTURA BOULEVARD
STUDIO CITY, CALIFORNIA

DRAFTED BY: JJK	CHECKED BY: NDB	JUNE 2021	PROJECT NO. W1208-06-01	FIG. 19c

APPENDIX A

APPENDIX A

FIELD INVESTIGATION

The site was explored on August 13 and 14, 2020, by excavating four 8-inch-diameter borings using a truck-mounted hollow-stem auger drilling machine. The borings were excavated to depths between approximately 65½ and 70½ feet below the existing ground surface. Representative and relatively undisturbed samples were obtained by driving a 3-inch, O. D., California Modified Sampler into the “undisturbed” soil mass with blows from a 140-pound auto-hammer falling 30 inches. The California Modified Sampler was equipped with 1-inch by 2³/₈-inch diameter brass sampler rings to facilitate soil removal and testing. Bulk samples was also obtained. Standard penetration tests (SPTs) were performed in boring B1.

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 A4. 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 2A.

BORING 1

ELEV. (MSL.) 627 DATE COMPLETED 8/13/2020

EQUIPMENT HOLLOW STEM AUGER BY: JK

DEPTH IN FEET	SAMPLE NO.	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0			**ASPHALT: 5"** **ARTIFICIAL FILL** Clayey Silt, stiff, dry, dark brown, medium-grained sand, rootlets.			
2	B1@2.5'			34	87.1	16.4
4		ML	**ALLUVIUM** Clayey Silt, firm, slightly moist, dark brown.			
6	B1@5'			14		
8	B1@7.5'		- hard	49	101.7	20.7
10	B1@10'	ML	Sandy Silt, firm, slightly moist, brown.	14		
12	B1@12.5'	CL	Sandy Clay, firm, moist, brown, trace medium-grained sand.	18	94.2	24.0
14	B1@15'		- soft, trace coarse-grained sand	8		
16	B1@17.5'			7		
18						
20	B1@20'		- very soft, very moist	1		
22	B1@22.5'			4	93.0	30.0
24			- dark brown, wet			
26	B1@25'			9		
28	B1@27.5'		- moist	14	106.3	22.4

W1208-06-01 BORING LOGS.GPJ

Figure A1,
Log of Boring 1, Page 1 of 3

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

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

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PROJECT NO. W1208-06-01

BORING 1

ELEV. (MSL.) 627 DATE COMPLETED 8/13/2020

EQUIPMENT HOLLOW STEM AUGER BY: JK

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
30	B1@30' BULK 30-35'			CL	- soft, very moist	4		
32	B1@32.5'			CL	Clay with Silt, soft, very moist, brown.	6	89.2	32.5
34	B1@35'			CL	Sandy Clay, soft, wet, brown, trace fine gravel.	2		
36	B1@37.5'			CL	Silty Clay, soft, very moist, brown.	7	86.0	39.3
38	B1@40'			CH	Clay with Sand, soft, very moist to wet, brown, some fine- to medium-grained sand.	2		
40	B1@42.5'			SM	Silty Sand, dense, wet, light brown, fine-grained, trace fine gravel.	70	122.8	15.4
44	B1@45'			SP	Sand, poorly graded, dense, wet, grayish brown, fine- to medium-grained, some coarse-grained, trace fine gravel.	48		
48	B1@47.5'				- very dense	50 (6")	124.0	12.8
50	B1@50'			CH	Sandy Clay, firm, moist, olive brown, with fine-grained sand.	10		
52	B1@52.5'				**MODELO FORMATION** Siltstone, hard, olive brown and gray, highly weathered, massive.	50 (5")	80.5	40.6
54	B1@55'				- stiff, slightly moist, gray, thinly bedded, dark gray	18		
58	B1@57.5'				- hard, thin (1/4") sandstone beds	50 (5")	79.4	43.9

Figure A1,
Log of Boring 1, Page 2 of 3

W1208-06-01 BORING LOGS.GPJ

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

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

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BORING 1

ELEV. (MSL.) 627 DATE COMPLETED 8/13/2020

EQUIPMENT HOLLOW STEM AUGER BY: JK

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
60	B1@60'				- medium hard, poorly bedded to massive	55		
62	B1@62.5'				- hard	50 (3")	74.9	45.4
64	B1@65'				- firm, poorly bedded	11		
66								
68	B1@67.5'				- hard, dry to slightly moist	50 (6")	75.1	42.9
70	B1@70'				- refusal at 70.5'	50 (3")		

Total depth of boring: 71 feet
Fill to 3 feet.
Groundwater encountered at 22 feet.
Backfilled with soil cuttings and tamped.
Concrete patched.

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

Figure A1,
Log of Boring 1, Page 3 of 3

W1208-06-01 BORING LOGS.GPJ

SAMPLE SYMBOLS

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

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

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BORING 2

ELEV. (MSL.) 626 DATE COMPLETED 8/13/2020

EQUIPMENT HOLLOW STEM AUGER BY: JK

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0					**ARTIFICIAL FILL** Silty Sand, loose, dry, light brown.			
2					**ALLUVIUM** Clayey Silt, stiff, dry to slightly moist, dark brown, abundant mica, trace medium-grained sand.			
	B2@2.5'					24	110.4	10.4
4								
	B2@5'				- slightly moist, brown	23	91.7	25.7
6								
	B2@7'				- dark brown, trace medium-grained sand	39	101.4	24.8
8				ML				
10	B2@10'				- slightly moist to moist, brown, no sand	38	100.3	19.8
	BULK 10-15'							
12								
	B2@12.5'					27	95.7	25.1
14								
	B2@15'				- firm, trace coarse-grained sand, fine gravel	16	95.8	25.6
16								
	B2@17.5'				- soft, very moist, no sand or gravel	10	92.7	29.7
18					Sandy Silt, soft, very moist, brown with medium- to coarse-grained sand, some fine gravel.			
20	B2@20'			ML		10	90.0	31.7
22			▼					
					Silt with Sand, soft, very moist, brown, some medium- to coarse-grained sand.			
24								
	B2@25'			ML		8	91.4	32.0
26								
28				CL	Silty Clay, soft, very moist, dark brown.			

W1208-06-01 BORING LOGS.GPJ

Figure A2,
Log of Boring 2, Page 1 of 3

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 2

ELEV. (MSL.) **626** DATE COMPLETED **8/13/2020**

EQUIPMENT **HOLLOW STEM AUGER** BY: **JK**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
30	B2@30'			CL		10	91.9	30.8
35	B2@35'			CL	- trace fine to coarse angular gravel	8	94.3	29.4
40	B2@40'			SM	Silty Sand, loose, very moist, brown, fine-grained.	13	100.6	24.9
45	B2@45'			SM		18	91.9	34.1
48				SP-SM	Sand with Silt, very dense, wet, olive gray, fine-grained.			
50	B2@50'			SP-SM		50 (6")	115.4	19.2
53				ML	Silt, firm, very moist, olive brown, some fine gravel.			
55	B2@55'			ML		16	89.5	24.6

W1208-06-01 BORING LOGS.GPJ

Figure A2,
Log of Boring 2, Page 2 of 3

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

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

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BORING 2

ELEV. (MSL.) **626** DATE COMPLETED **8/13/2020**

EQUIPMENT **HOLLOW STEM AUGER** BY: **JK**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
60	B2@60'				- stiff, moist, angular coarse-grained sand	34	96.7	27.5
62								
64								
	B2@65'			ML	- firm, gray	15	97.3	27.1
66								
68								
70	B2@70'					17	93.6	31.3

Total depth of boring: 70.5 feet
Fill to 2 feet.
Groundwater encountered at 22 feet.
Backfilled with soil cuttings and tamped.

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

W1208-06-01 BORING LOGS.GPJ

Figure A2,
Log of Boring 2, Page 3 of 3

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. W1208-06-01

BORING 3

ELEV. (MSL.) 626 DATE COMPLETED 8/14/2020

EQUIPMENT HOLLOW STEM AUGER BY: JK

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'				**ARTIFICIAL FILL** Silty Sand, loose, dry, light brown.			
2					Clayey Silt, soft, moist, dark brown.			
4					**ALLUVIUM** Clayey Silt, soft, moist, dark brown.			
	B3@5'				- trace medium-grained sand	11	89.9	30.6
6								
8								
10	B3@10'				- stiff, brown	25	96.4	24.5
12				ML				
	B3@12.5'				- firm	13	94.0	28.3
14								
	B3@15'				- trace angular coarse-grained sand	12	93.8	29.2
16								
18	B3@17.5'				- soft, very moist	7	90.3	33.5
20	B3@20'				- no coarse-grained sand	5	90.2	31.6
22			▼		Sandy Silt, soft, very moist, brown, fine-grained, some medium- to coarse-grained.			
24								
	B3@25'			ML		7	93.2	30.9
26								
28				CL	Silty Clay, soft, very moist, brown to dark brown, trace medium-grained sand.			

W1208-06-01 BORING LOGS.GPJ

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

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

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BORING 3

ELEV. (MSL.) **626** DATE COMPLETED **8/14/2020**

EQUIPMENT **HOLLOW STEM AUGER** BY: **JK**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
30	B3@30'					9	99.3	27.9
35	B3@35'			CL	- no recovery	8		
40	B3@40'				- firm, wet, no medium- to coarse-grained sand	20	84.1	31.0
42					Sand, well-graded, medium dense, wet, fine gravel.			
45	B3@45'					40	121.6	13.4
				SW				
50	B3@50'				- dense, no recovery	75		
53					Clayey Silt, hard, moist, brown.			
55	B3@55'			ML		43	100.9	24.5
58				ML	Silt with Sand, firm, moist, light brown, trace clay.			

W1208-06-01 BORING LOGS.GPJ

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

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

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

BORING 3

ELEV. (MSL.) 626 DATE COMPLETED 8/14/2020

EQUIPMENT HOLLOW STEM AUGER BY: JK

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
60	B3@60'			ML		20	97.3	27.6
62								
64								
					Silt, hard, moist, light brown, some medium- to coarse-grained angular sand.			
	B3@65'			ML		42	98.3	24.5
66								
68								
70	B3@70'				- firm	18	92.4	28.7

Total depth of boring: 70.5 feet
Fill to 4 feet.
Groundwater encountered at 22.5 feet.
Backfilled with soil cuttings and tamped.

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

W1208-06-01 BORING LOGS.GPJ

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

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.) 630 DATE COMPLETED 8/14/2020

EQUIPMENT HOLLOW STEM AUGER BY: JK

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0					**ASPHALT: 3"** **ARTIFICIAL FILL** Clayey Silt, soft, moist, dark brown.			
4					**ALLUVIUM**			
5	B4@5'				Clayey Silt, stiff, moist, dark brown, some medium-grained sand, small cobbles (to 4").	23	95.3	26.7
10	B4@10'			ML	- firm, brown	14	94.3	25.3
15	B4@15'				- soft, slightly moist to moist, some angular fine gravel	10	92.7	22.8
17.5					Sandy Silt, loose, yellowish brown, angular fine gravel pieces.			
20	B4@20'			ML		9	80.8	19.9
22.5					Silty Clay, soft, very moist to wet, brown, trace angular fine gravel.			
24			▼					
25	B4@25'			CL		5	91.6	31.3

W1208-06-01 BORING LOGS.GPJ

Figure A4,
Log of Boring 4, Page 1 of 3

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

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

BORING 4

ELEV. (MSL.) 630 DATE COMPLETED 8/14/2020

EQUIPMENT HOLLOW STEM AUGER BY: JK

DEPTH IN FEET	SAMPLE NO.	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
30	B4@30'	CL	- dark gray	10	96.6	26.4
32.5	B4@32.5'	CL	- brown	4	93.6	30.1
35	B4@35'	CL		6	89.0	31.0
37.5	B4@37.5'	CL		10	91.9	28.5
38		SM	Silty Sand, medium dense, very moist, brown, fine-grained.			
40	B4@40'	SM	- very moist to wet, coarse-grained sand, some fine gravel	36	98.4	25.9
45	B4@45'	SM	- loose, wet, fine-grained, trace fine gravel	6	92.8	27.0
48		ML	Clayey Silt, soft, very moist, gray.			
50	B4@50'	ML		11	88.6	36.7
55	B4@55'	ML	- no recovery	11		

W1208-06-01 BORING LOGS.GPJ

Figure A4,
Log of Boring 4, Page 2 of 3

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.) 630 DATE COMPLETED 8/14/2020

EQUIPMENT HOLLOW STEM AUGER BY: JK

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
60	B4@60'				**MODELO FORMATION** Siltstone, hard, slightly moist, dark gray, highly weathered, massive to poorly bedded.	50 (2")	83.4	36.6
62								
64					- dark gray with light gray mottles			
	B4@65'				- refusal	50 (4")	87.7	32.8

Total depth of boring: 65.5 feet
Fill to 4 feet.
Groundwater encountered at 24 feet.
Backfilled with soil cuttings and tamped.
Concrete patched.

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

W1208-06-01 BORING LOGS.GPJ

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

SAMPLE SYMBOLS

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

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

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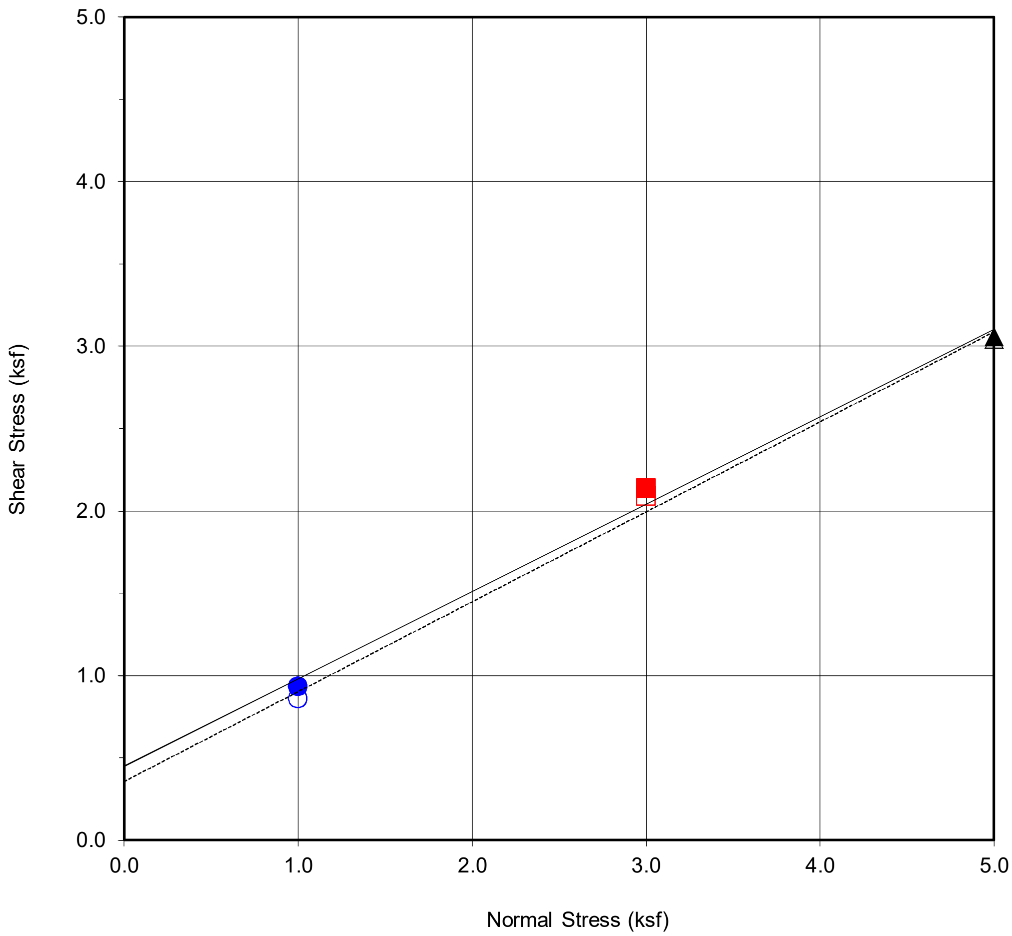
APPENDIX B

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, Atterburg Limits, grain size analysis, maximum dry density, corrosivity, in-place dry density and moisture content. The results of the laboratory tests are summarized in Figures B1 through B39. The in-place dry density and moisture content of the samples tested are presented on the boring logs, Appendix A.



Boring No.	B1
Sample No.	B1@32.5'
Depth (ft)	32.5
Sample Type:	Ring

Soil Identification:		
Brown Clay w/ Silt (CL)		
Strength Parameters		
	C (psf)	ϕ (°)
Peak	448	28.0
Ultimate	356	28.6

Normal Stress (kip/ft2)	1	3	5
Peak Shear Stress (kip/ft²)	● 0.93	■ 2.14	▲ 3.06
Shear Stress @ End of Test (ksf)	○ 0.86	□ 2.09	△ 3.04
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 (%)	33.8	32.5	33.3
Initial Dry Density (pcf)	87.7	89.7	88.1
Initial Degree of Saturation (%)	99.0	99.8	98.4
Soil Height Before Shearing (in.)	1.2	1.2	1.2
Final Moisture Content (%)	31.9	28.1	27.1

GEOCON

DIRECT SHEAR TEST RESULTS

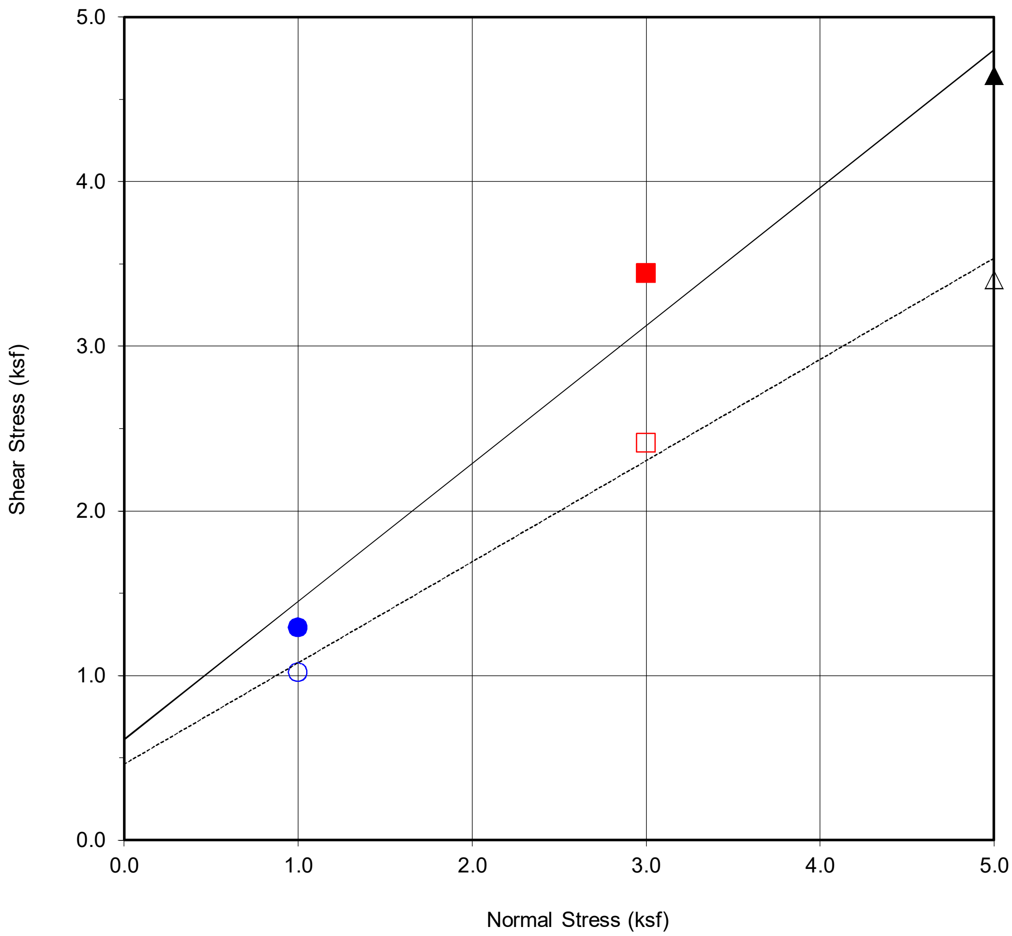
Consolidated Drained ASTM D-3080

Checked by: JJK

Project No.: W1208-06-01

12805-12825 WEST VENTURA BOULEVARD
STUDIO CITY, CALIFORNIA

JUNE 2021 Figure B1



Boring No.	**B1**
Sample No.	**B1@42.5'**
Depth (ft)	**42.5**
Sample Type:	Ring

Soil Identification:		
Light Brown Silty Sand (SM)		
Strength Parameters		
	C (psf)	ϕ (°)
Peak	610	40.0
Ultimate	461	31.6

Normal Stress (kip/ft2)	1	3	5
Peak Shear Stress (kip/ft²)	● 1.29	■ 3.44	▲ 4.64
Shear Stress @ End of Test (ksf)	○ 1.02	□ 2.41	△ 3.40
Deformation Rate (in./min.)	0.05	0.05	0.05
Initial Sample Height (in.)	1.0	1.0	1.0
Ring Inside Diameter (in.)	2.375	2.375	2.375
Initial Moisture Content (%)	16.7	15.7	18.5
Initial Dry Density (pcf)	112.9	115.1	111.6
Initial Degree of Saturation (%)	91.3	91.3	98.1
Soil Height Before Shearing (in.)	1.2	1.2	1.2
Final Moisture Content (%)	17.2	15.7	18.1

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DIRECT SHEAR TEST RESULTS

Consolidated Drained ASTM D-3080

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12805-12825 WEST VENTURA BOULEVARD
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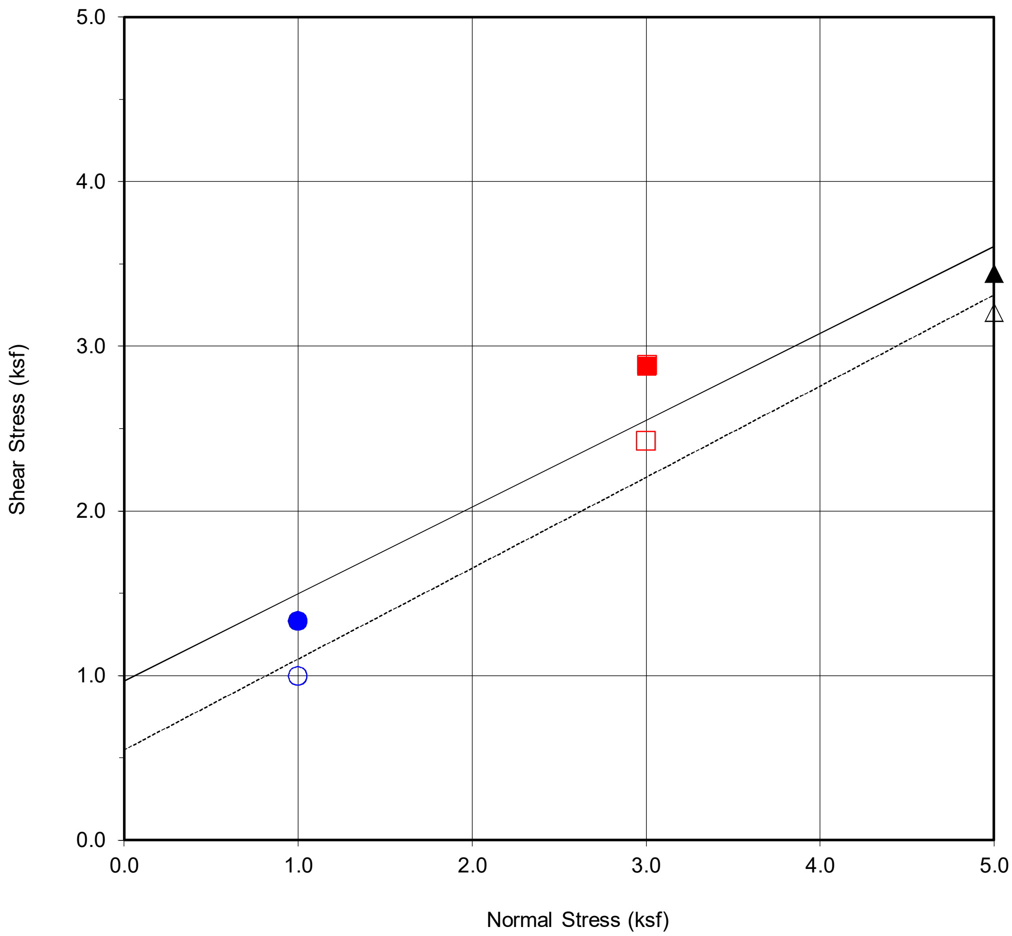
Figure B2



Boring No.	**B1**
Sample No.	**B1@52.5'**
Depth (ft)	**52.5**
Sample Type:	Ring

Soil Identification:		
Olive Brown & Gray Bedrock (BDRX)		
Strength Parameters		
	C (psf)	ϕ (^{o})
Peak	967	27.8
Ultimate	548	28.9

Normal Stress (kip/ft2)	1	3	5
Peak Shear Stress (kip/ft²)	● 1.33	■ 2.81	▲ 3.44
Shear Stress @ End of Test (ksf)	○ 0.99	□ 2.42	△ 3.20
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 (%)	56.2	48.3	40.9
Initial Dry Density (pcf)	65.8	73.4	80.2
Initial Degree of Saturation (%)	97.1	100.5	100.2
Soil Height Before Shearing (in.)	1.2	1.2	1.2
Final Moisture Content (%)	59.8	48.6	40.7

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DIRECT SHEAR TEST RESULTS

Consolidated Drained ASTM D-3080

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12805-12825 WEST VENTURA BOULEVARD
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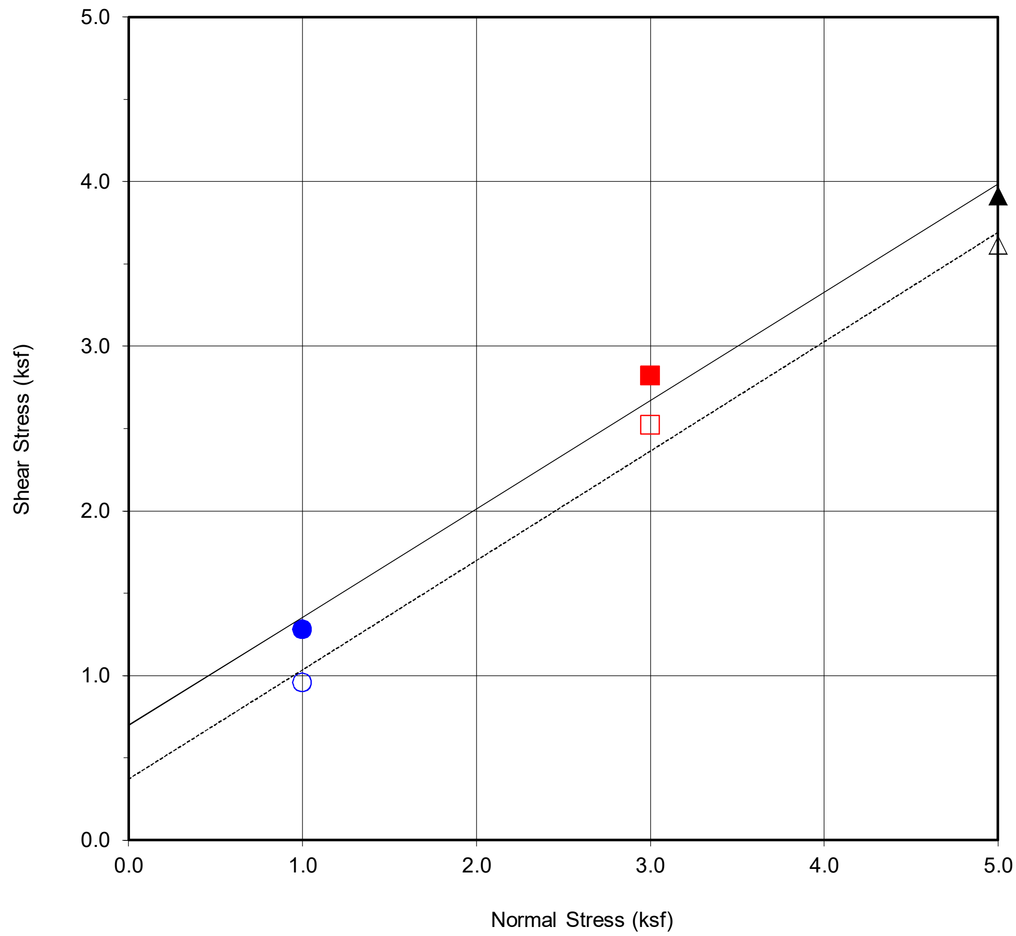
Figure B3



Boring No.	**B1**
Sample No.	**B1@62.5'**
Depth (ft)	**62.5**
Sample Type:	Ring

Soil Identification:		
Dark Gray Bedrock (BDRX)		
Strength Parameters		
	C (psf)	ϕ (^{o})
Peak	696	33.3
Ultimate	371	33.6

Normal Stress (kip/ft2)	1	3	5
Peak Shear Stress (kip/ft²)	● 1.28	■ 2.82	▲ 3.91
Shear Stress @ End of Test (ksf)	○ 0.96	□ 2.52	△ 3.61
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 (%)	43.7	45.4	44.5
Initial Dry Density (pcf)	74.3	74.2	75.0
Initial Degree of Saturation (%)	92.9	96.3	96.2
Soil Height Before Shearing (in.)	1.2	1.2	1.2
Final Moisture Content (%)	50.6	48.8	46.4

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DIRECT SHEAR TEST RESULTS

Consolidated Drained ASTM D-3080

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LOS ANGELES, CALIFORNIA

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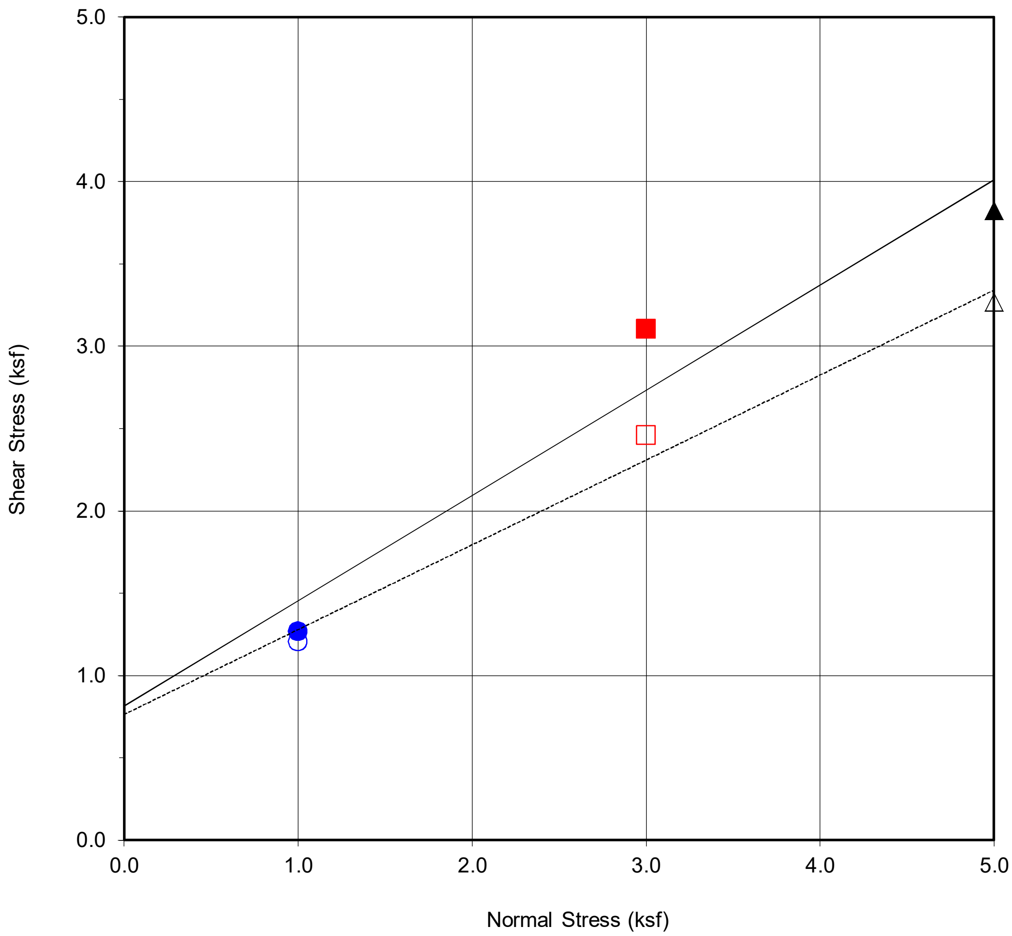
Figure B4



Boring No.	**B1**
Sample No.	**B1@67.5'**
Depth (ft)	**67.5**
Sample Type:	Ring

Soil Identification:		
Dark Gray Bedrock (BDRX)		
Strength Parameters		
	C (psf)	ϕ (°)
Peak	814	32.6
Ultimate	763	27.3

Normal Stress (kip/ft2)	1	3	5
Peak Shear Stress (kip/ft²)	● 1.27	■ 3.10	▲ 3.82
Shear Stress @ End of Test (ksf)	○ 1.20	□ 2.46	△ 3.27
Deformation Rate (in./min.)	0.05	0.05	0.05
Initial Sample Height (in.)	1.0	1.0	1.0
Ring Inside Diameter (in.)	2.375	2.375	2.375
Initial Moisture Content (%)	43.2	56.2	42.9
Initial Dry Density (pcf)	74.9	64.6	76.2
Initial Degree of Saturation (%)	93.2	94.3	95.5
Soil Height Before Shearing (in.)	1.2	1.2	1.2
Final Moisture Content (%)	48.0	58.1	45.0

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DIRECT SHEAR TEST RESULTS

Consolidated Drained ASTM D-3080

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12805-12825 WEST VENTURA BOULEVARD
LOS ANGELES, CALIFORNIA

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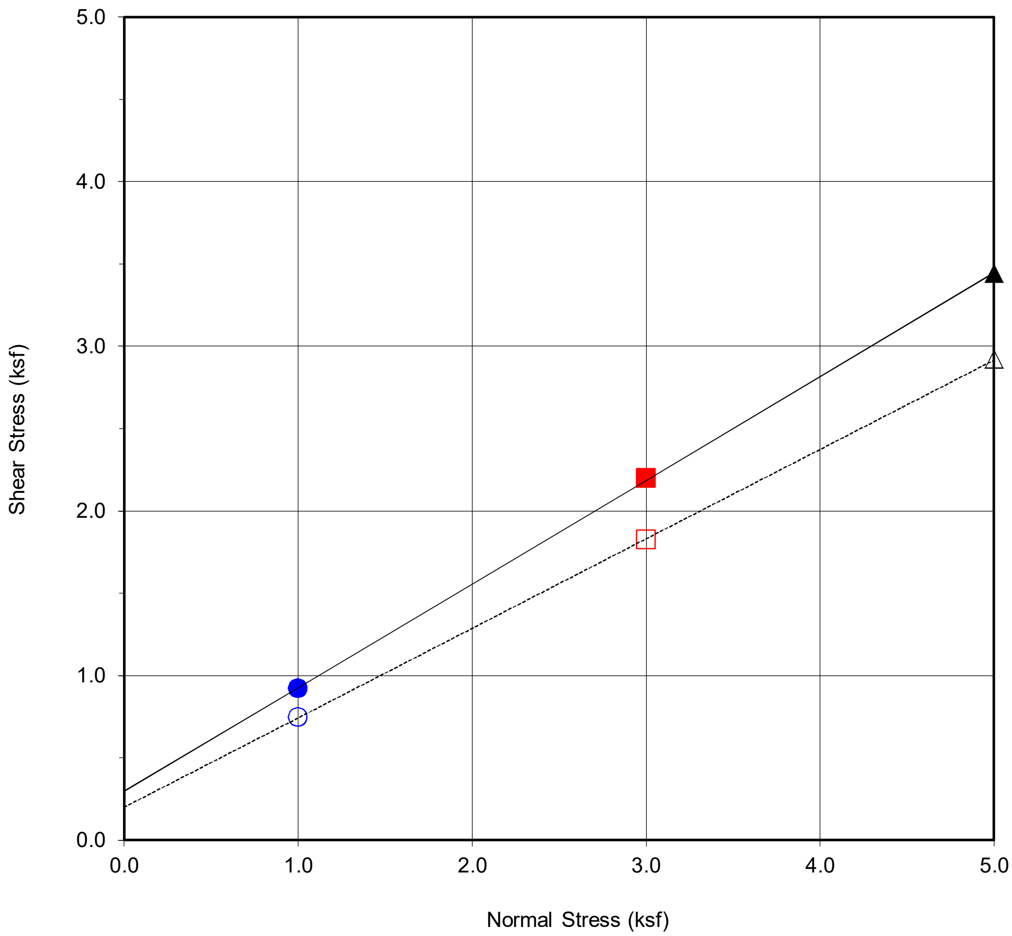
Figure B5



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

Soil Identification:		
Brown Clayey Silt (ML)		
Strength Parameters		
	C (psf)	ϕ (°)
Peak	295	32.2
Ultimate	200	28.5

Normal Stress (kip/ft2)	1	3	5
Peak Shear Stress (kip/ft²)	● 0.92	■ 2.20	▲ 3.44
Shear Stress @ End of Test (ksf)	○ 0.75	□ 1.83	△ 2.92
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 (%)	25.6	25.1	25.7
Initial Dry Density (pcf)	87.2	90.0	95.6
Initial Degree of Saturation (%)	74.1	77.7	91.2
Soil Height Before Shearing (in.)	1.2	1.2	1.2
Final Moisture Content (%)	35.6	32.1	30.6

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DIRECT SHEAR TEST RESULTS

Consolidated Drained ASTM D-3080

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Project No.: W1208-06-01

12805-12825 WEST VENTURA BOULEVARD
STUDIO CITY, CALIFORNIA

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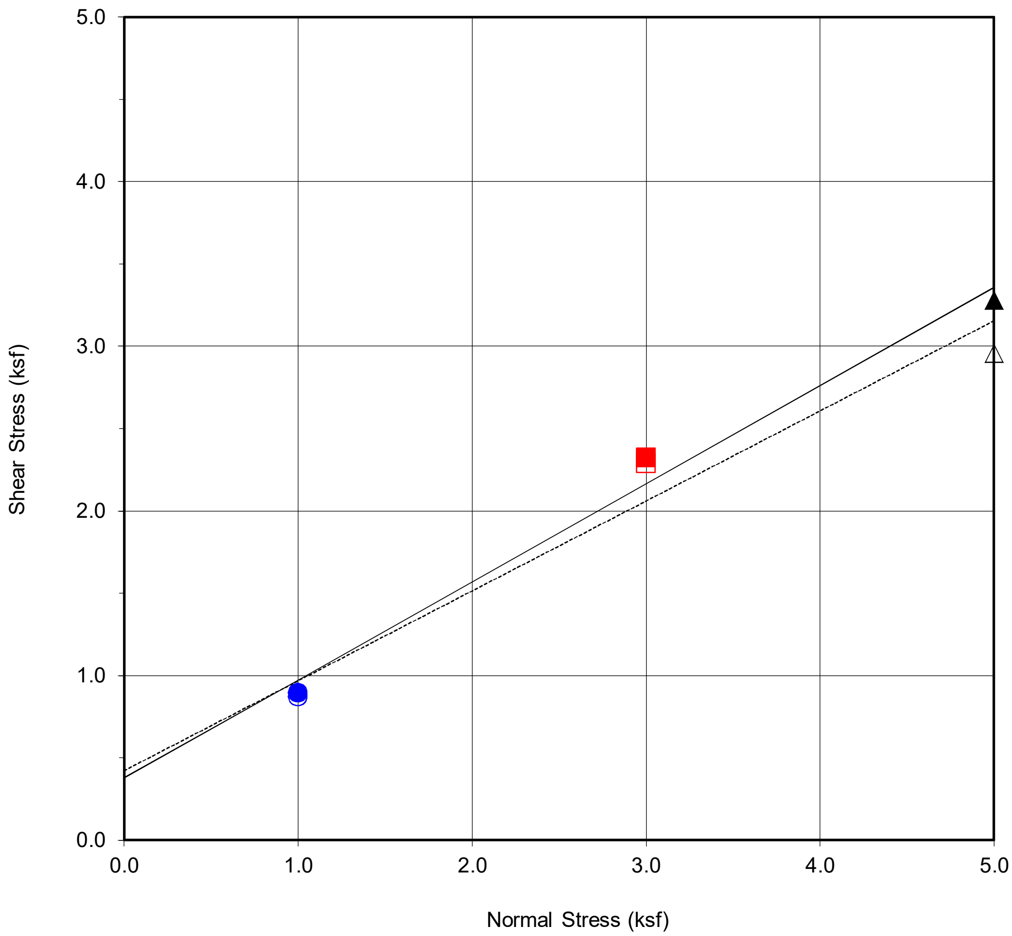
Figure B6



Boring No.	**B2**
Sample No.	**B2@10'**
Depth (ft)	**10**
Sample Type:	Ring

Soil Identification:		
Brown Clayey Silt (ML)		
Strength Parameters		
	C (psf)	ϕ (^{o})
Peak	377	30.8
Ultimate	422	28.7

Normal Stress (kip/ft2)	1	3	5
Peak Shear Stress (kip/ft²)	● 0.89	■ 2.32	▲ 3.28
Shear Stress @ End of Test (ksf)	○ 0.87	□ 2.29	△ 2.96
Deformation Rate (in./min.)	0.05	0.05	0.05
Initial Sample Height (in.)	1.0	1.0	1.0
Ring Inside Diameter (in.)	2.375	2.375	2.375
Initial Moisture Content (%)	20.8	18.3	19.8
Initial Dry Density (pcf)	94.1	100.6	101.5
Initial Degree of Saturation (%)	71.0	73.1	81.0
Soil Height Before Shearing (in.)	1.2	1.2	1.2
Final Moisture Content (%)	27.8	23.7	23.6

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DIRECT SHEAR TEST RESULTS

Consolidated Drained ASTM D-3080

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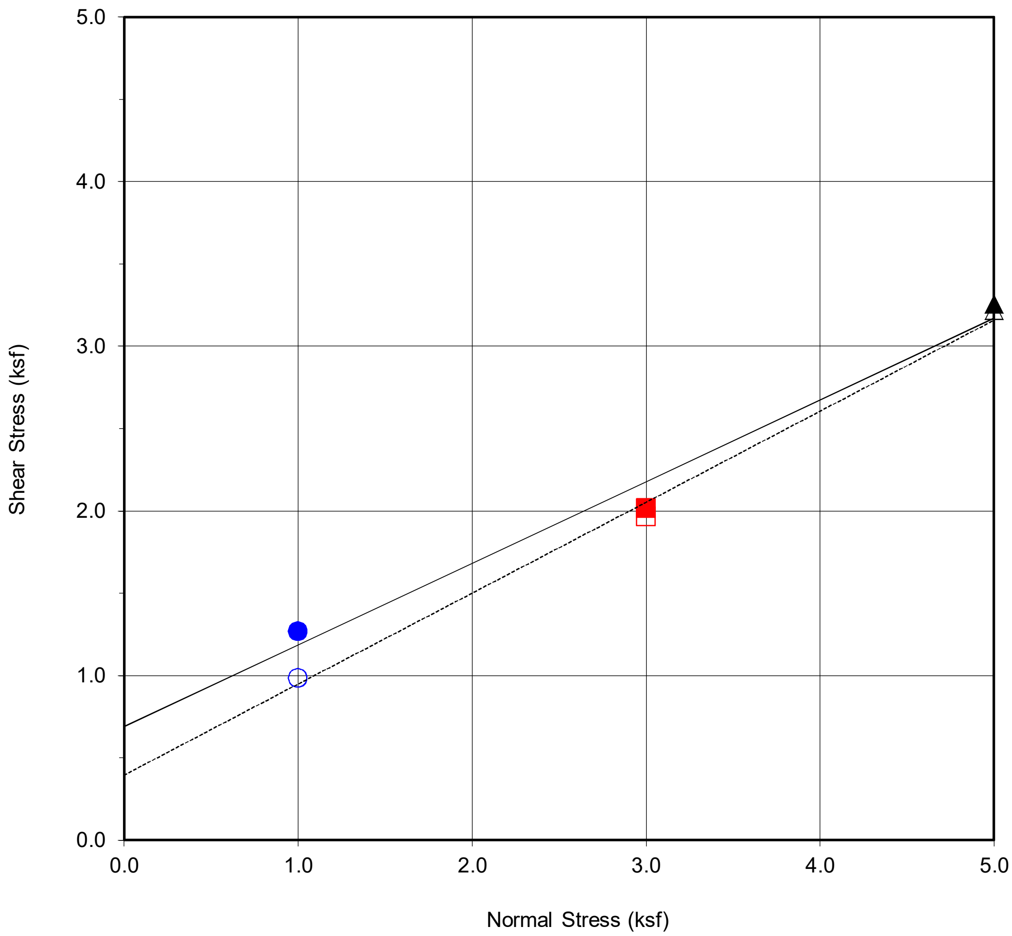
Figure B7



Boring No.	B2
Sample No.	B2@12.5'
Depth (ft)	12.5'
Sample Type:	Ring

Normal Stress (kip/ft2)	1	3	5
Peak Shear Stress (kip/ft²)	● 1.27	■ 2.01	▲ 3.25
Shear Stress @ End of Test (ksf)	○ 0.98	□ 1.96	△ 3.22
Deformation Rate (in./min.)	0.05	0.05	0.05
Initial Sample Height (in.)	1.0	1.0	1.0
Ring Inside Diameter (in.)	2.375	2.375	2.375
Initial Moisture Content (%)	25.1	26.8	24.7
Initial Dry Density (pcf)	95.5	92.4	95.6
Initial Degree of Saturation (%)	88.5	87.7	87.5
Soil Height Before Shearing (in.)	1.2	1.2	1.2
Final Moisture Content (%)	29.5	28.9	27.0

Soil Identification:

Brown Clayey Silt (ML)

Strength Parameters

	C (psf)	ϕ (°)
Peak	688	26.4
Ultimate	395	28.9

GEOCON

DIRECT SHEAR TEST RESULTS

Consolidated Drained ASTM D-3080

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Project No.: W1208-06-01

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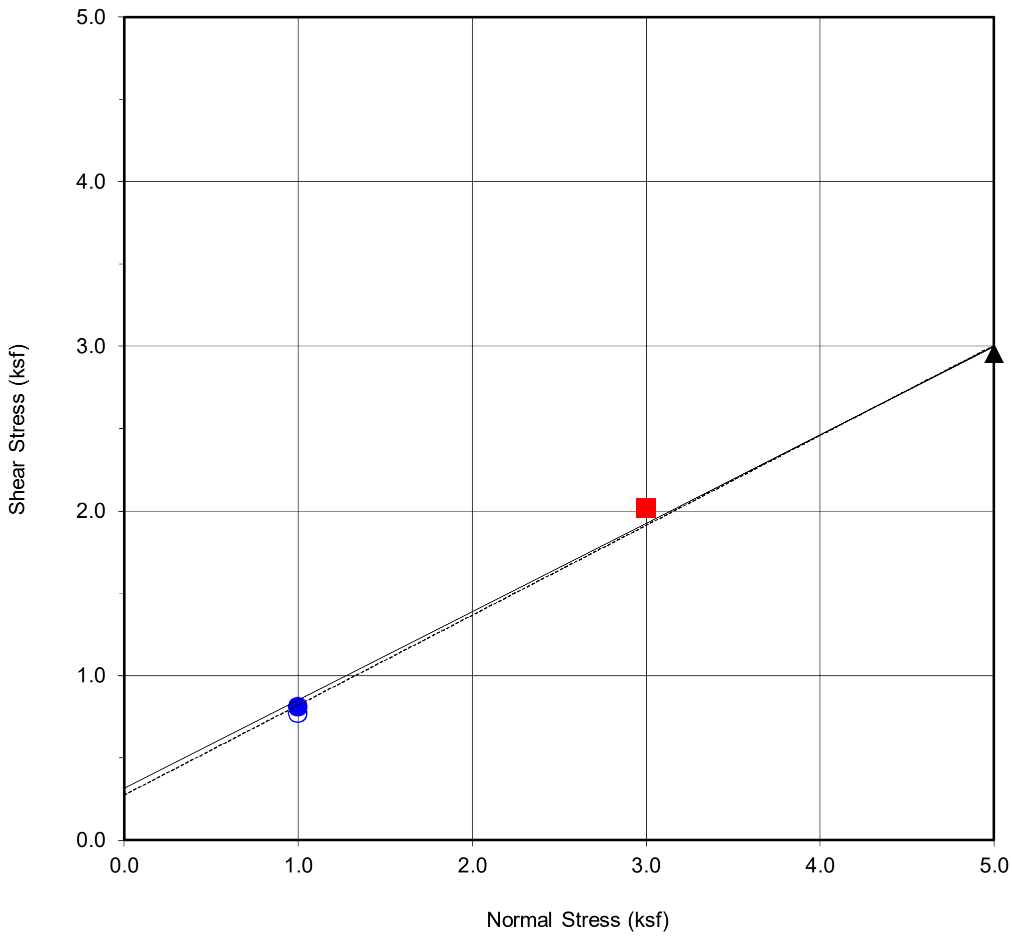
Figure B8



Boring No.	B2
Sample No.	B2@17.5'
Depth (ft)	17.5'
Sample Type:	Ring

Soil Identification:		
Brown Clayey Silt (ML)		
Strength Parameters		
	C (psf)	ϕ (°)
Peak	314	28.2
Ultimate	274	28.6

Normal Stress (kip/ft2)	1	3	5
Peak Shear Stress (kip/ft²)	● 0.81	■ 2.01	▲ 2.96
Shear Stress @ End of Test (ksf)	○ 0.77	□ 2.01	△ 2.96
Deformation Rate (in./min.)	0.05	0.05	0.05
Initial Sample Height (in.)	1.0	1.0	1.0
Ring Inside Diameter (in.)	2.375	2.375	2.375
Initial Moisture Content (%)	29.1	37.1	29.7
Initial Dry Density (pcf)	92.4	87.2	91.8
Initial Degree of Saturation (%)	95.3	107.2	96.0
Soil Height Before Shearing (in.)	1.2	1.2	1.2
Final Moisture Content (%)	29.1	28.4	25.3

GEOCON

DIRECT SHEAR TEST RESULTS

Consolidated Drained ASTM D-3080

Checked by: JJK

Project No.: W1208-06-01

12805-12825 WEST VENTURA BOULEVARD
STUDIO CITY, CALIFORNIA

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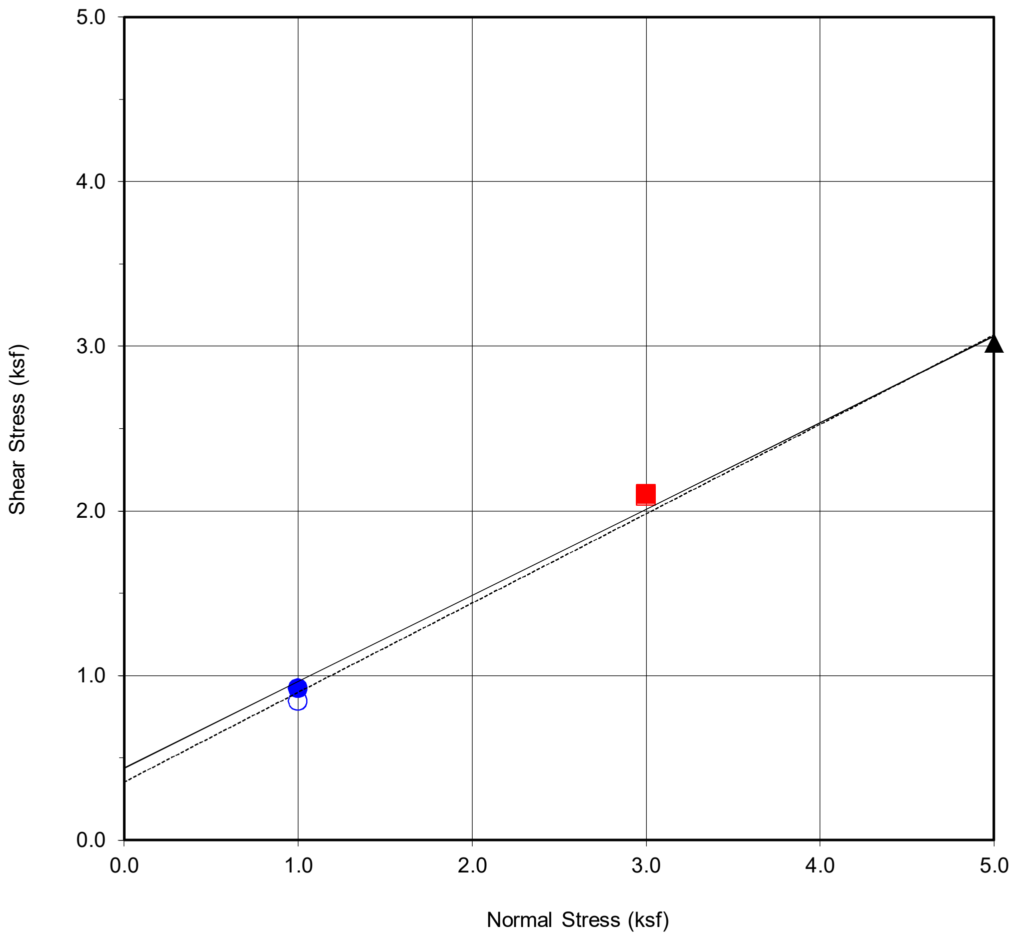
Figure B9



Boring No.	**B2**
Sample No.	**B2@25'**
Depth (ft)	**25**
Sample Type:	Ring

Soil Identification:		
Brown Silt w/ Sand (ML)		
Strength Parameters		
	C (psf)	ϕ (°)
Peak	438	27.7
Ultimate	353	28.5

Normal Stress (kip/ft2)	1	3	5
Peak Shear Stress (kip/ft²)	● 0.92	■ 2.10	▲ 3.02
Shear Stress @ End of Test (ksf)	○ 0.84	□ 2.09	△ 3.02
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 (%)	32.2	32.0	33.7
Initial Dry Density (pcf)	88.5	90.8	88.5
Initial Degree of Saturation (%)	96.1	100.9	100.5
Soil Height Before Shearing (in.)	1.2	1.2	1.2
Final Moisture Content (%)	29.7	27.5	27.0

GEOCON

DIRECT SHEAR TEST RESULTS

Consolidated Drained ASTM D-3080

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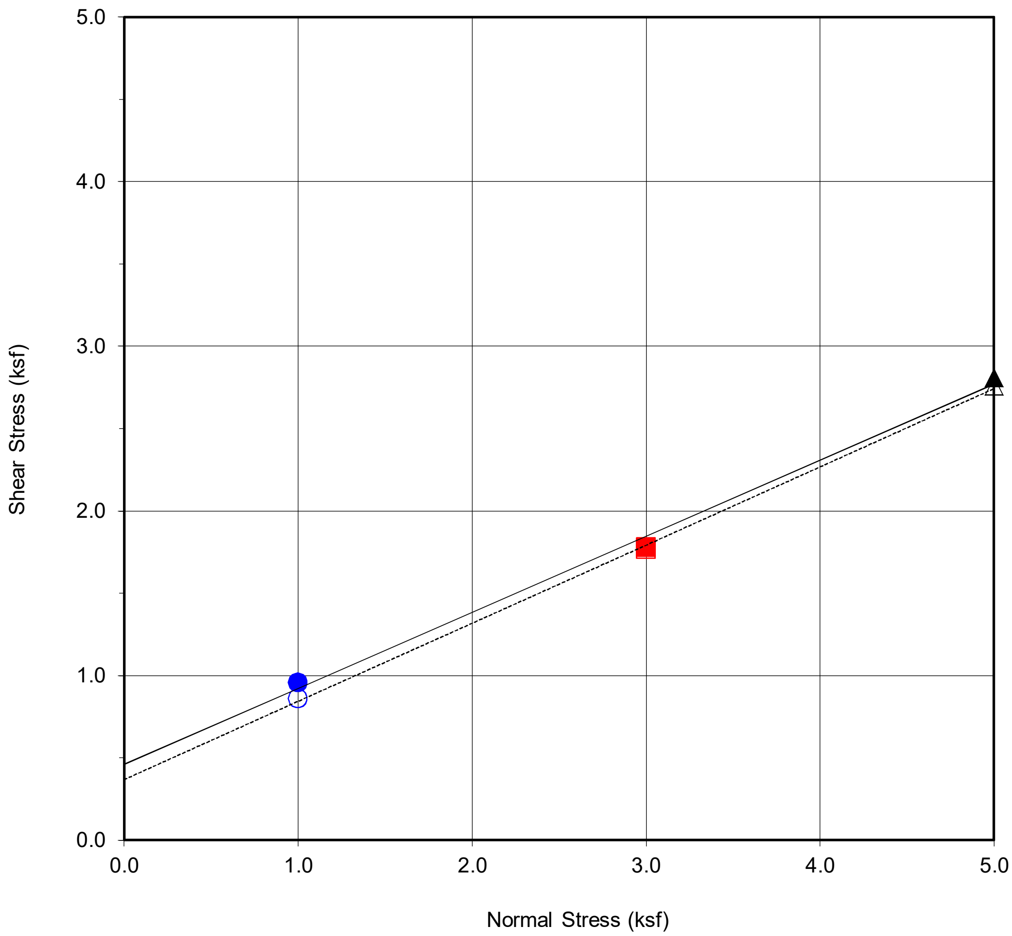
Figure B10



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

Soil Identification:		
Brown Clayey Silt (ML)		
Strength Parameters		
	C (psf)	ϕ (°)
Peak	459	24.8
Ultimate	367	25.4

Normal Stress (kip/ft2)	1	3	5
Peak Shear Stress (kip/ft²)	● 0.96	■ 1.78	▲ 2.81
Shear Stress @ End of Test (ksf)	○ 0.86	□ 1.76	△ 2.76
Deformation Rate (in./min.)	0.05	0.05	0.05
Initial Sample Height (in.)	1.0	1.0	1.0
Ring Inside Diameter (in.)	2.375	2.375	2.375
Initial Moisture Content (%)	30.6	33.5	31.7
Initial Dry Density (pcf)	88.9	85.5	88.8
Initial Degree of Saturation (%)	92.1	93.1	95.2
Soil Height Before Shearing (in.)	1.2	1.2	1.2
Final Moisture Content (%)	32.2	33.1	29.7

GEOCON

DIRECT SHEAR TEST RESULTS

Consolidated Drained ASTM D-3080

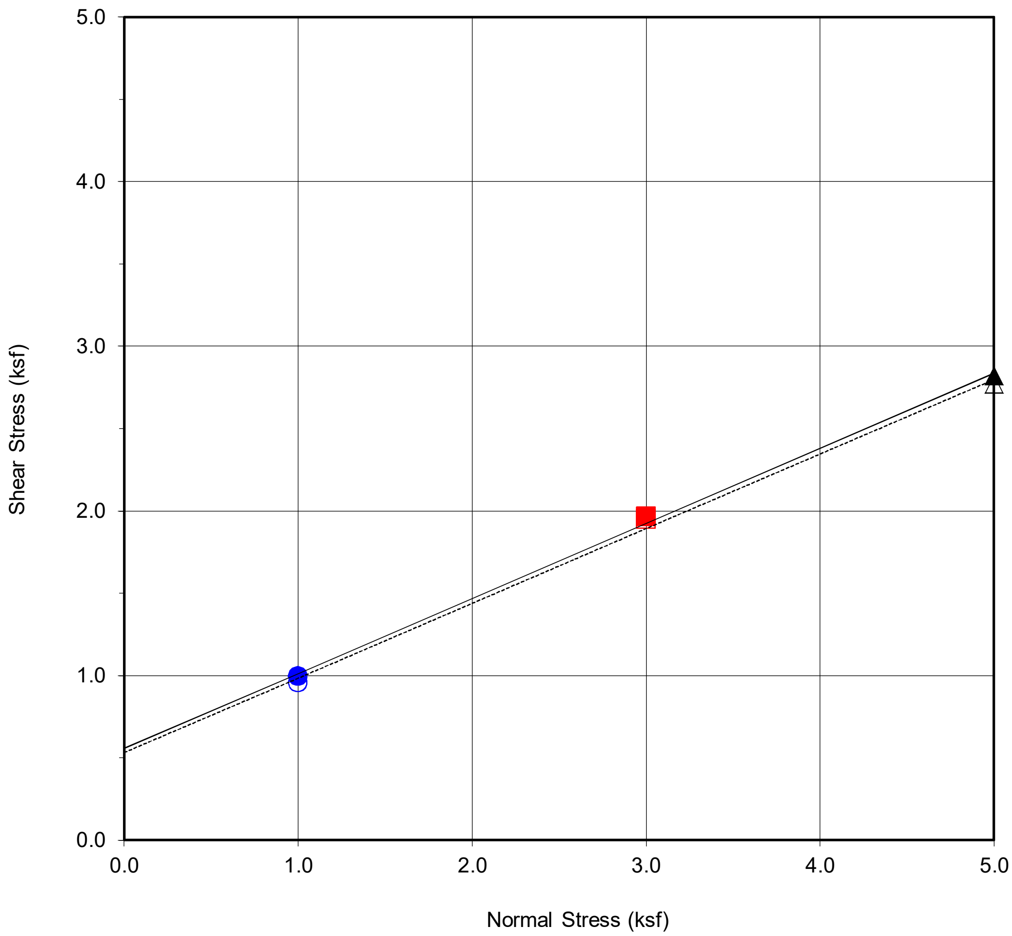
Checked by: JJK

Project No.: W1208-06-01

12805-12825 WEST VENTURA BOULEVARD
STUDIO CITY, CALIFORNIA

JUNE 2021

Figure B11



Boring No.	**B3**
Sample No.	**B3@12.5'**
Depth (ft)	**12.5**
Sample Type:	Ring

Soil Identification:		
Brown Clayey Silt (ML)		
Strength Parameters		
	C (psf)	ϕ (°)
Peak	555	24.5
Ultimate	532	24.4

Normal Stress (kip/ft2)	1	3	5
Peak Shear Stress (kip/ft²)	● 0.99	■ 1.96	▲ 2.82
Shear Stress @ End of Test (ksf)	○ 0.96	□ 1.95	△ 2.77
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 (%)	28.3	28.3	28.9
Initial Dry Density (pcf)	92.0	93.4	93.8
Initial Degree of Saturation (%)	91.8	94.9	98.0
Soil Height Before Shearing (in.)	1.2	1.2	1.2
Final Moisture Content (%)	30.3	27.9	27.4

GEOCON

DIRECT SHEAR TEST RESULTS

Consolidated Drained ASTM D-3080

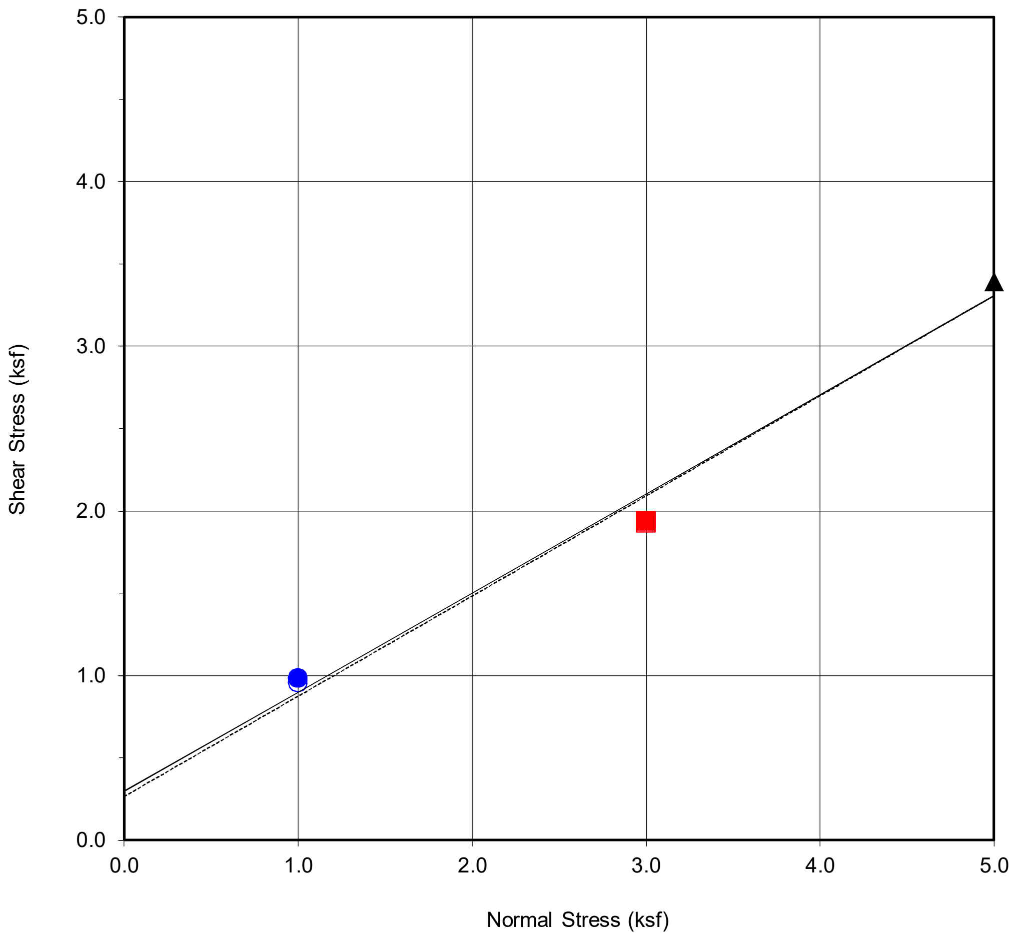
Checked by: JJK

Project No.: W1208-06-01

12805-12825 WEST VENTURA BOULEVARD
STUDIO CITY, CALIFORNIA

JUNE 2021

Figure B12



Boring No.	**B3**
Sample No.	**B3@15'**
Depth (ft)	**15**
Sample Type:	Ring

Soil Identification:		
Brown Clayey Silt (ML)		
Strength Parameters		
	C (psf)	ϕ (°)
Peak	296	31.1
Ultimate	265	31.3

Normal Stress (kip/ft2)	1	3	5
Peak Shear Stress (kip/ft²)	● 0.98	■ 1.94	▲ 3.39
Shear Stress @ End of Test (ksf)	○ 0.96	□ 1.93	△ 3.39
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 (%)	29.2	29.1	29.0
Initial Dry Density (pcf)	93.0	93.0	91.1
Initial Degree of Saturation (%)	96.9	96.8	92.2
Soil Height Before Shearing (in.)	1.2	1.2	1.2
Final Moisture Content (%)	30.6	28.6	28.0

GEOCON

DIRECT SHEAR TEST RESULTS

Consolidated Drained ASTM D-3080

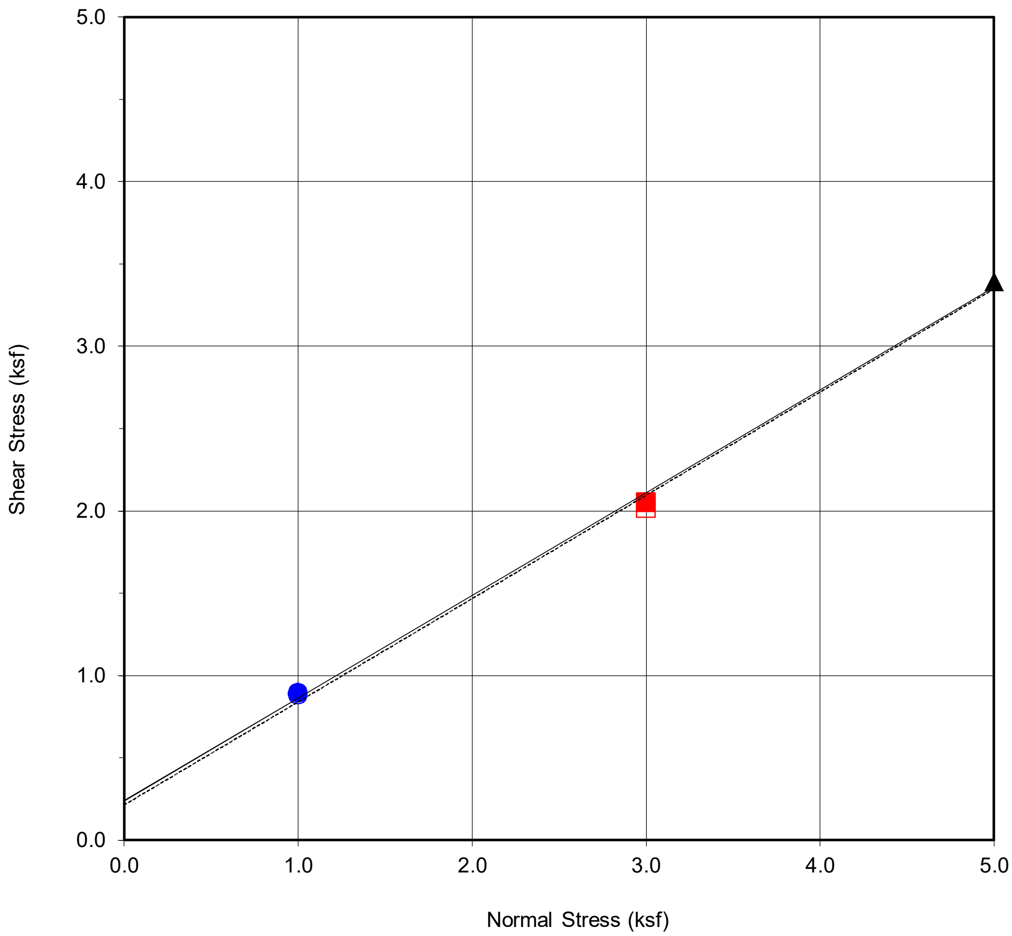
Checked by: JJK

Project No.: W1208-06-01

12805-12825 WEST VENTURA BOULEVARD
STUDIO CITY, CALIFORNIA

JUNE 2021

Figure B13



Boring No.	B3
Sample No.	B3@20'
Depth (ft)	20
Sample Type:	Ring

Normal Stress (kip/ft2)	1	3	5
Peak Shear Stress (kip/ft²)	● 0.89	■ 2.05	▲ 3.39
Shear Stress @ End of Test (ksf)	○ 0.88	□ 2.01	△ 3.39
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 (%)	32.0	31.6	34.5
Initial Dry Density (pcf)	89.8	90.8	87.3
Initial Degree of Saturation (%)	98.4	99.8	99.9
Soil Height Before Shearing (in.)	1.2	1.2	1.2
Final Moisture Content (%)	29.2	26.5	26.2

Soil Identification:

Brown Clayey Silt (ML)

Strength Parameters

	C (psf)	ϕ (°)
Peak	239	32.0
Ultimate	214	32.1

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DIRECT SHEAR TEST RESULTS

Consolidated Drained ASTM D-3080

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12805-12825 WEST VENTURA BOULEVARD
STUDIO CITY, CALIFORNIA

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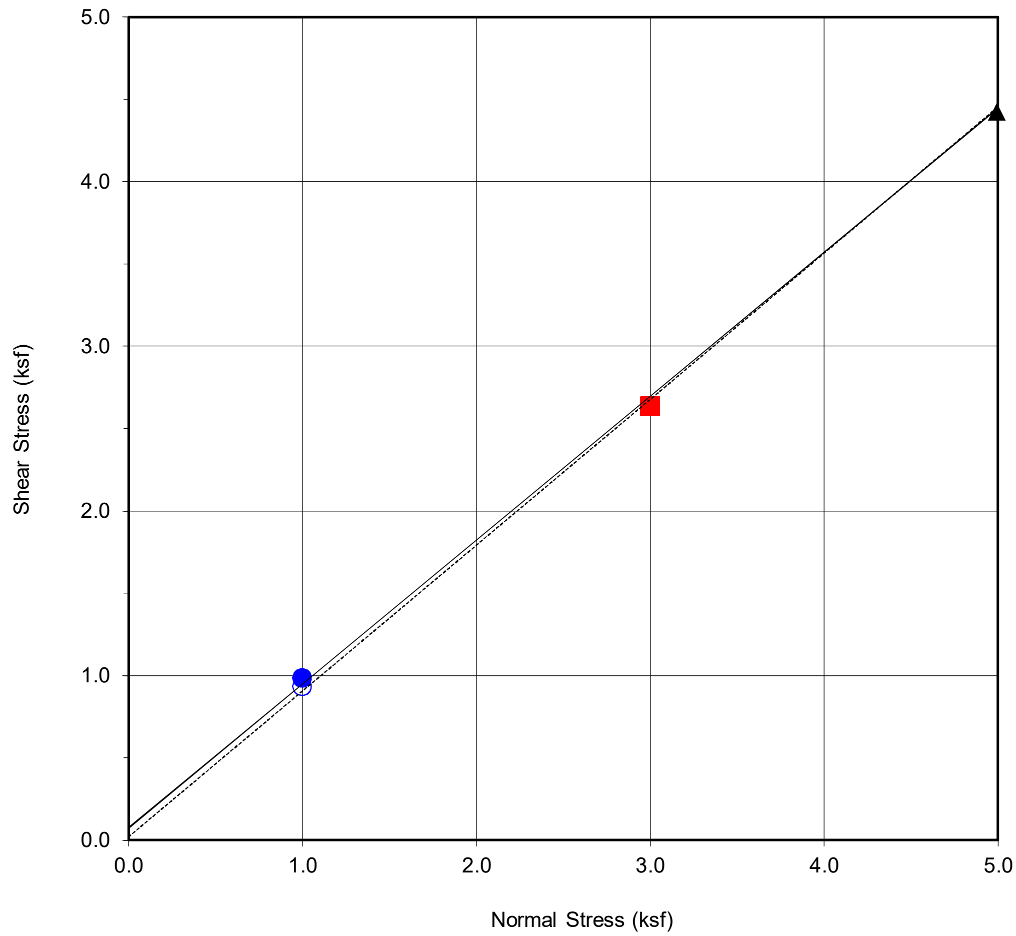
Figure B14



Boring No.	**B4**
Sample No.	**B4@32.5'**
Depth (ft)	**32.5**
Sample Type:	Ring

Soil Identification:		
Brown Silty Clay (CL)		
Strength Parameters		
	C (psf)	ϕ (°)
Peak	74	41.2
Ultimate	20	41.6

Normal Stress (kip/ft2)	1	3	5
Peak Shear Stress (kip/ft²)	● 0.98	■ 2.63	▲ 4.74
Shear Stress @ End of Test (ksf)	○ 0.93	□ 2.63	△ 4.74
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 (%)	31.1	30.6	30.1
Initial Dry Density (pcf)	91.8	91.9	94.5
Initial Degree of Saturation (%)	100.4	98.8	103.6
Soil Height Before Shearing (in.)	1.2	1.2	1.2
Final Moisture Content (%)	26.0	25.7	24.3

GEOCON

DIRECT SHEAR TEST RESULTS

Consolidated Drained ASTM D-3080

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Project No.: W1208-06-01

12805-12825 WEST VENTURA BOULEVARD
STUDIO CITY, CALIFORNIA

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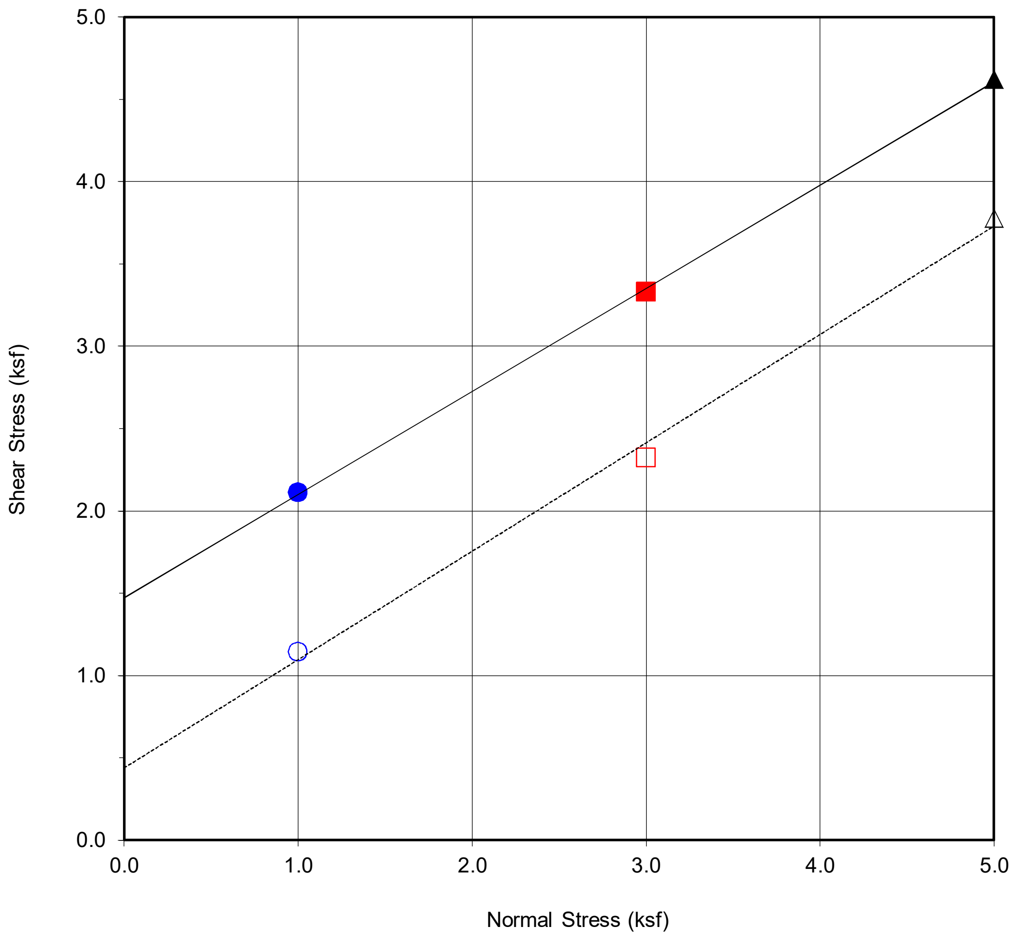
Figure B15



Boring No.	**B4**
Sample No.	**B4@60'**
Depth (ft)	**60**
Sample Type:	Ring

Soil Identification:		
Dark Gray Bedrock (BDRX)		
Strength Parameters		
	C (psf)	ϕ (°)
Peak	1472	32.1
Ultimate	438	33.4

Normal Stress (kip/ft2)	1	3	5
Peak Shear Stress (kip/ft²)	● 2.11	■ 3.33	▲ 4.62
Shear Stress @ End of Test (ksf)	○ 1.14	□ 2.32	△ 3.78
Deformation Rate (in./min.)	0.05	0.05	0.05
Initial Sample Height (in.)	1.0	1.0	1.0
Ring Inside Diameter (in.)	2.375	2.375	2.375
Initial Moisture Content (%)	39.5	36.6	36.9
Initial Dry Density (pcf)	79.4	83.7	81.9
Initial Degree of Saturation (%)	94.8	97.4	94.1
Soil Height Before Shearing (in.)	1.2	1.2	1.2
Final Moisture Content (%)	44.3	38.9	38.9

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DIRECT SHEAR TEST RESULTS

Consolidated Drained ASTM D-3080

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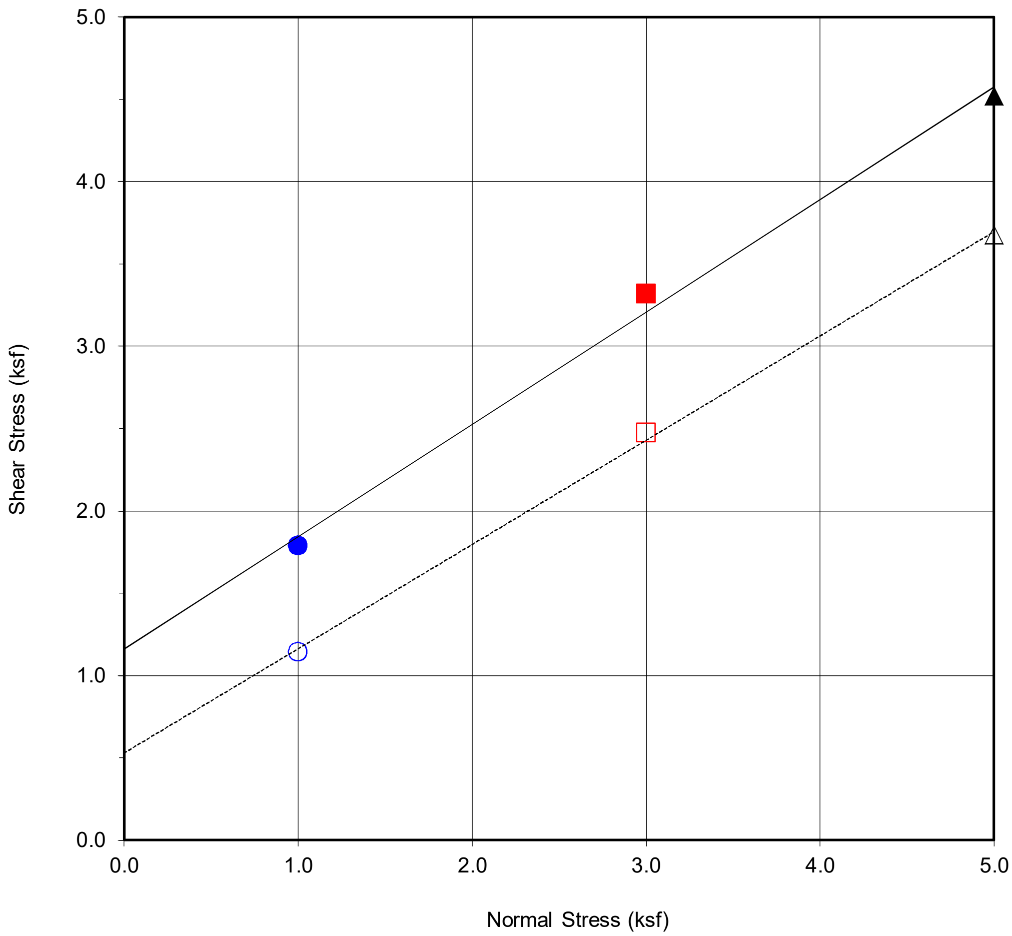
Project No.: W1208-06-01

12805-12825 WEST VENTURA BOULEVARD
STUDIO CITY, CALIFORNIA

JUNE 2021

Figure B16



Boring No.	B4
Sample No.	B4@65'
Depth (ft)	65
Sample Type:	Ring

Soil Identification:		
Dark Gray Bedrock (BDRX)		
Strength Parameters		
	C (psf)	ϕ (°)
Peak	1159	34.3
Ultimate	530	32.3

Normal Stress (kip/ft2)	1	3	5
Peak Shear Stress (kip/ft²)	● 1.79	■ 3.32	▲ 4.52
Shear Stress @ End of Test (ksf)	○ 1.14	□ 2.47	△ 3.68
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 (%)	36.0	34.6	35.5
Initial Dry Density (pcf)	82.5	85.2	82.9
Initial Degree of Saturation (%)	93.1	95.6	92.9
Soil Height Before Shearing (in.)	1.2	1.2	1.2
Final Moisture Content (%)	40.8	37.9	37.9

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DIRECT SHEAR TEST RESULTS

Consolidated Drained ASTM D-3080

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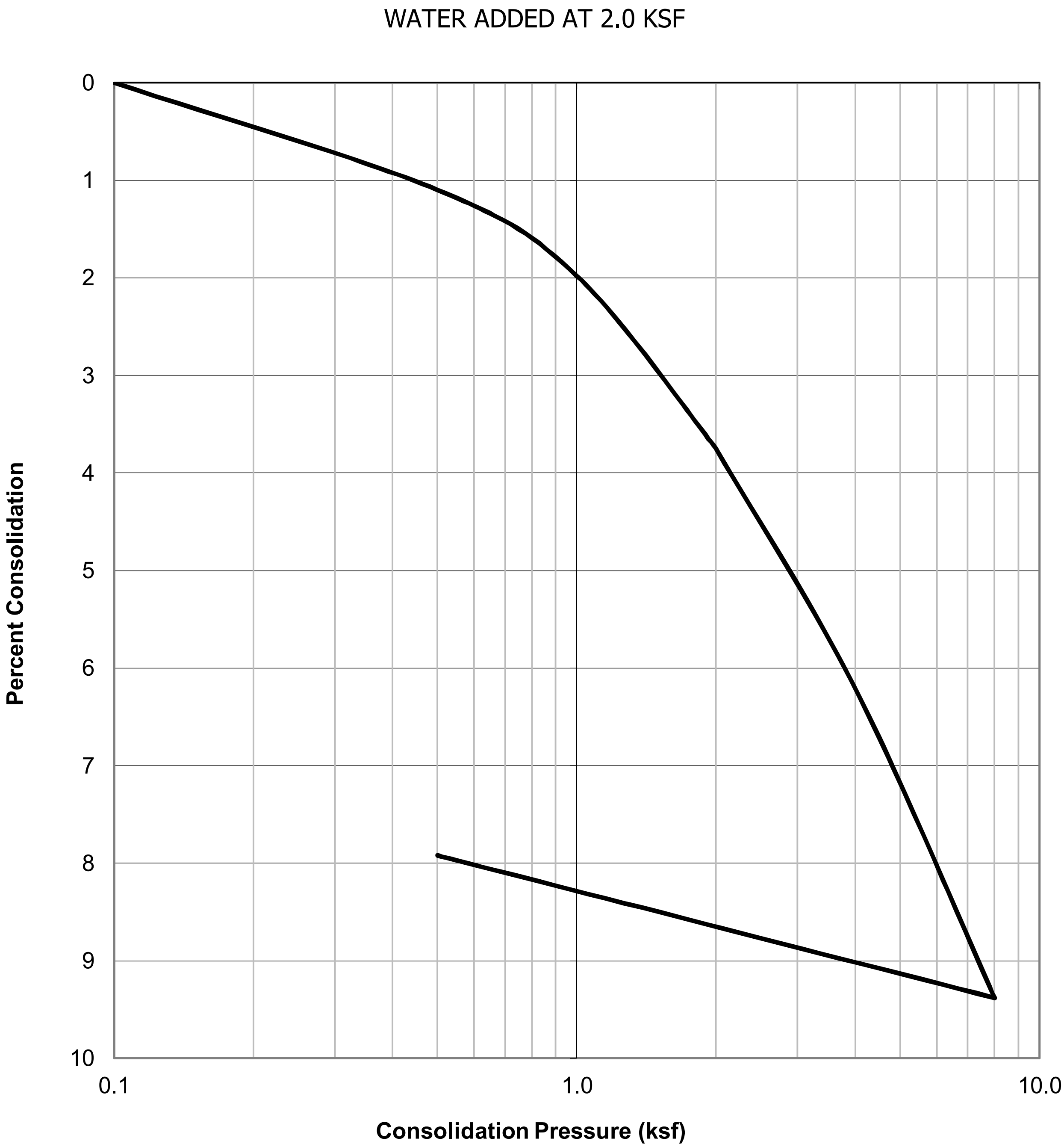
12805-12825 WEST VENTURA BOULEVARD
STUDIO CITY, CALIFORNIA

JUNE 2021

Figure B17



SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B1@37.5	Brown Silty Clay (CL)	81.1	39.3	33.0

GEOCON

CONSOLIDATION TEST RESULTS

ASTM D-2435

Checked by: JJK

Project No.: W1208-06-01

12805-12825 WEST VENTURA BOULEVARD
STUDIO CITY, CALIFORNIA

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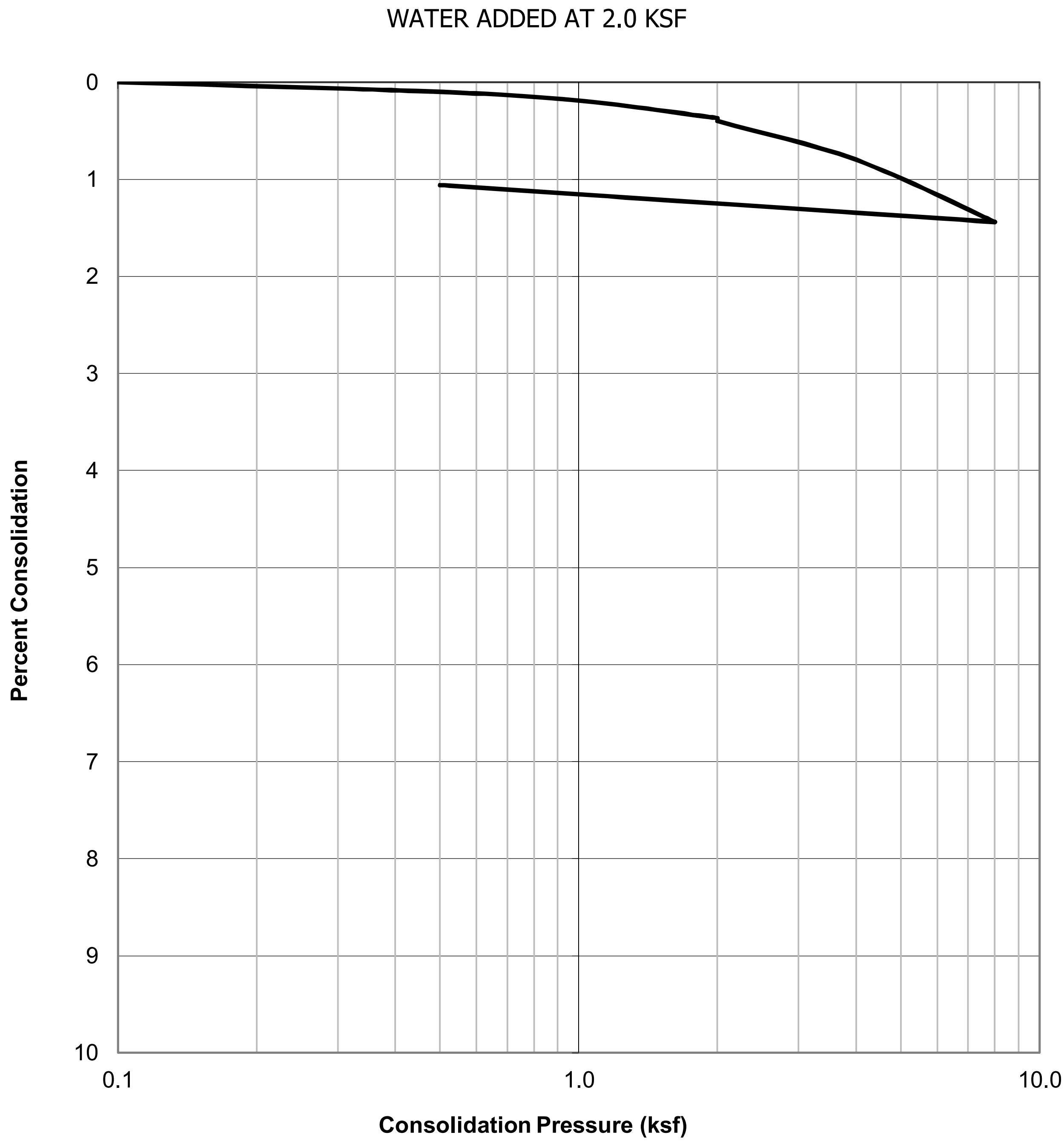
Figure B18



SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B1@42.5	Light Brown Silty Sand (SM)	109.9	15.4	17.6

GEOCON

CONSOLIDATION TEST RESULTS

ASTM D-2435

Checked by: JJK

Project No.: W1208-06-01

12805-12825 WEST VENTURA BOULEVARD
STUDIO CITY, CALIFORNIA

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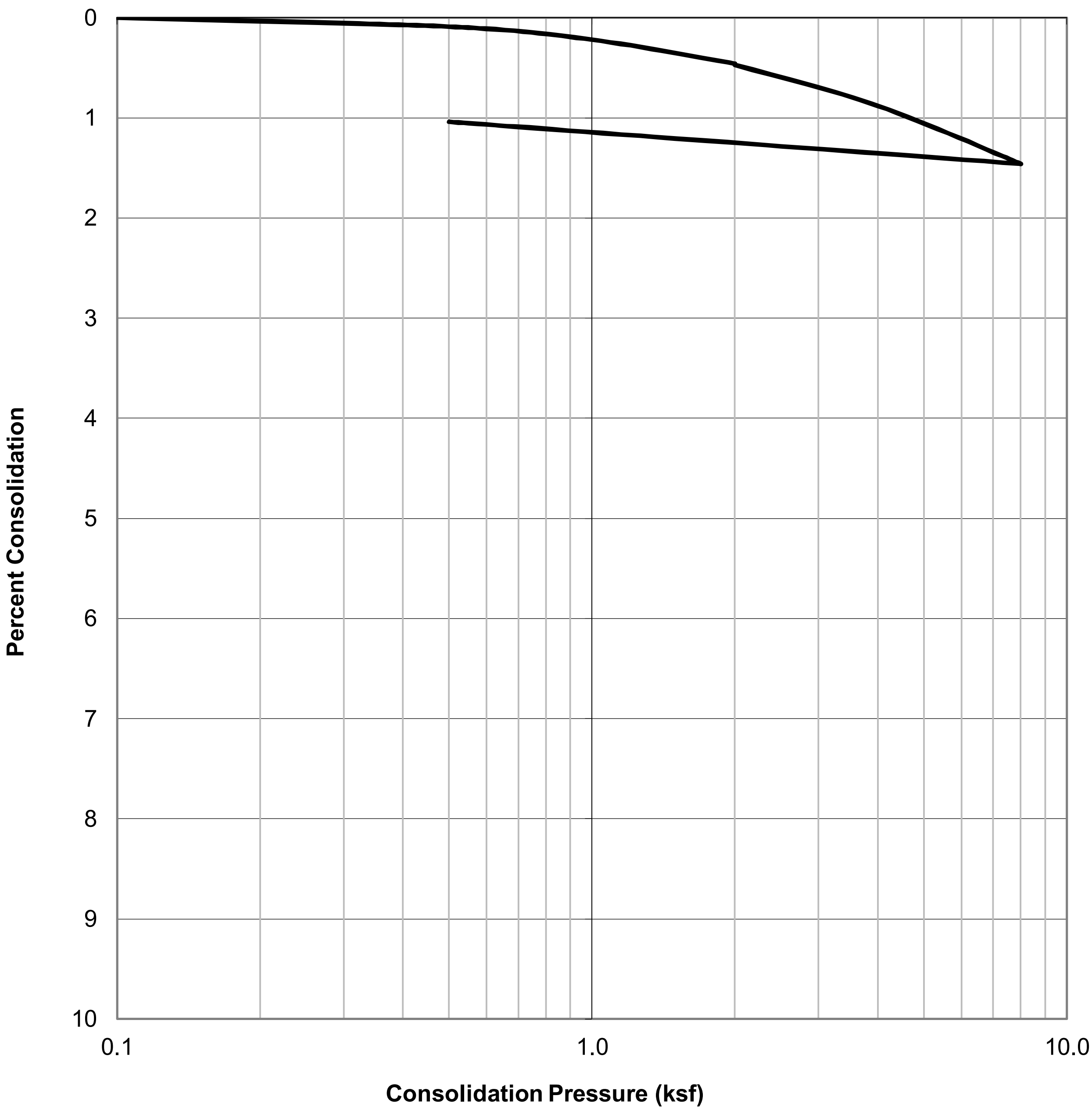
Figure B19

WATER ADDED AT 2.0 KSF



SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B1@47.5	Grayish Brown Poorly Graded Sand (SP)	118.7	12.8	13.4

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CONSOLIDATION TEST RESULTS

ASTM D-2435

Checked by: JJK

Project No.: W1208-06-01

12805-12825 WEST VENTURA BOULEVARD
STUDIO CITY, CALIFORNIA

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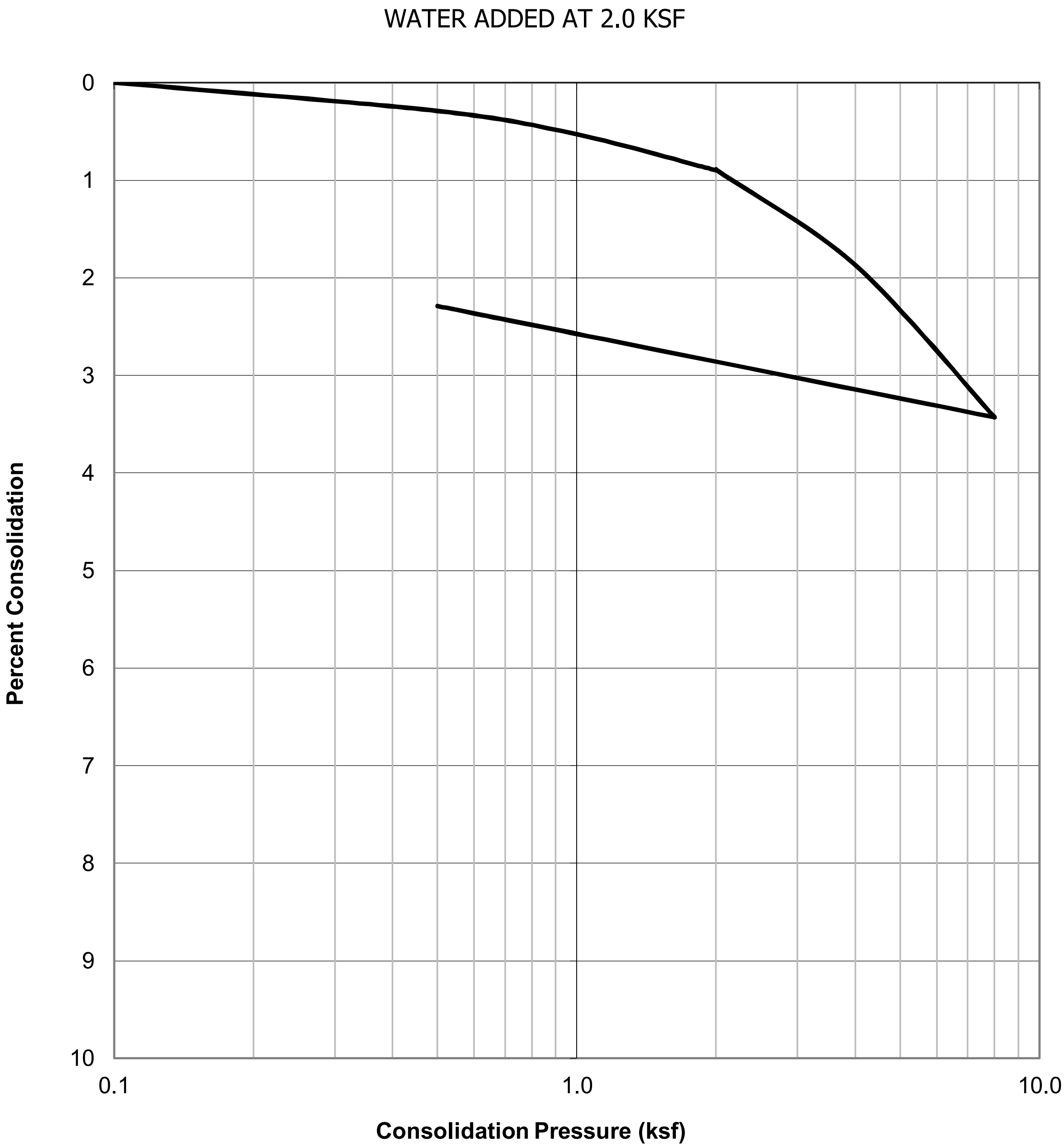
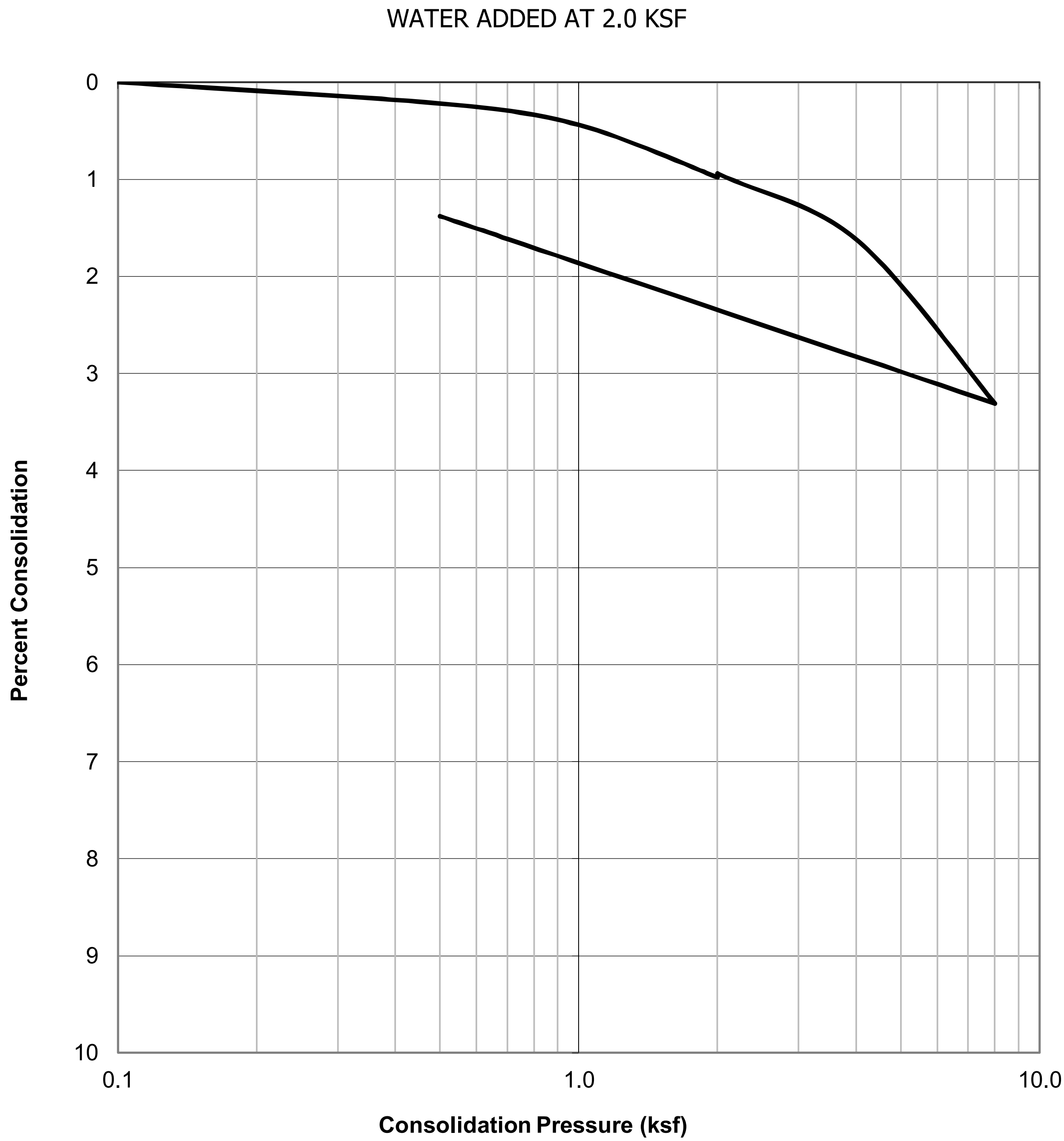
Figure B20



SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B1@52.5	Olive Brown & Gray Bedrock (BDRX)	77.1	40.6	42.1

GEOCON	CONSOLIDATION TEST RESULTS ASTM D-2435	Project No.: W1208-06-01 12805-12825 WEST VENTURA BOULEVARD STUDIO CITY, CALIFORNIA
	Checked by: JJK	JUNE 2021 Figure B21



SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B2@15	Brown Clayey Silt (ML)	94.2	25.6	26.7

GEOCON

CONSOLIDATION TEST RESULTS

ASTM D-2435

Checked by: JJK

Project No.: W1208-06-01

12805-12825 WEST VENTURA BOULEVARD
STUDIO CITY, CALIFORNIA

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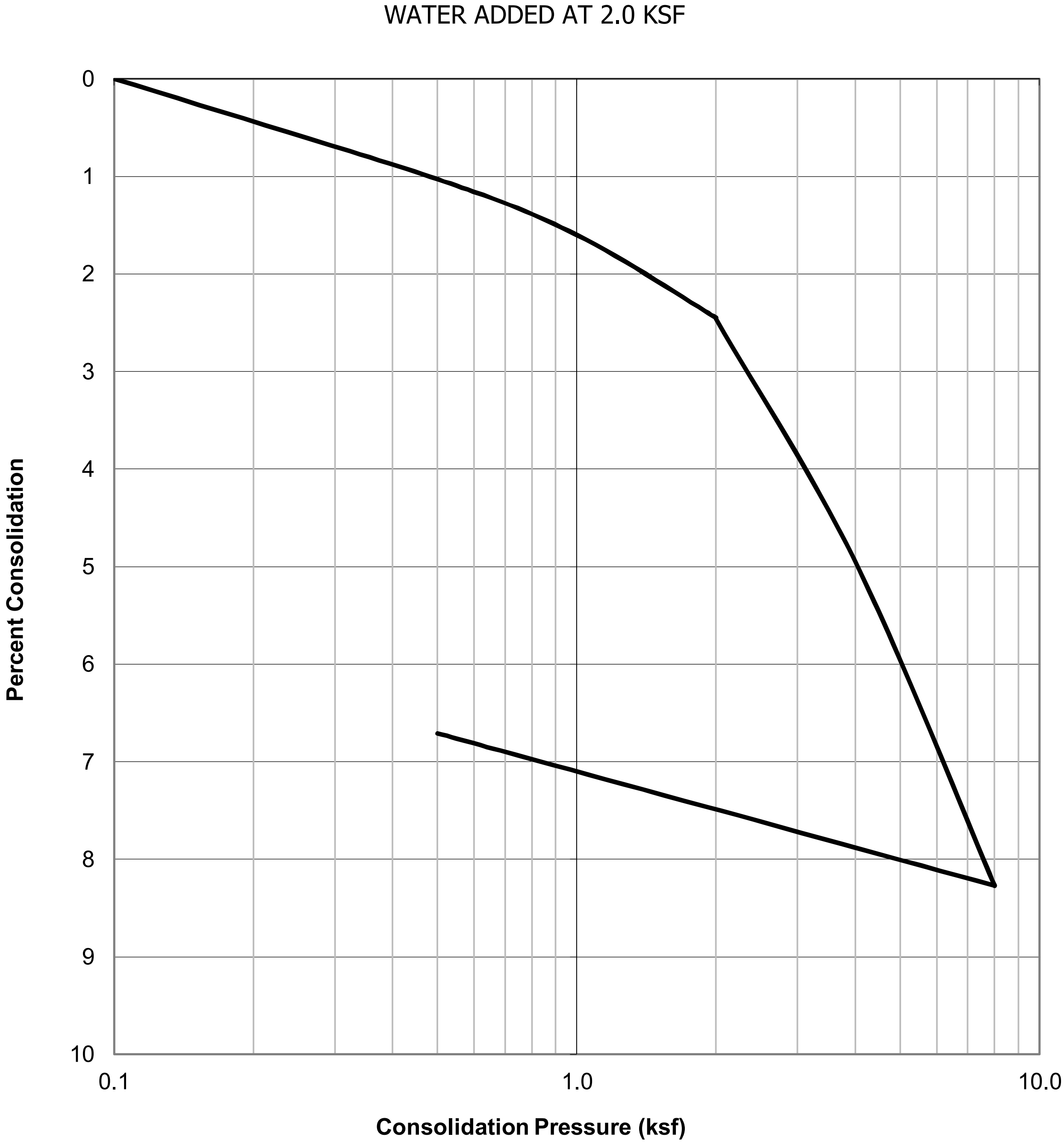
Figure B22



SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B2@20	Brown Sandy Silt (ML)	90.6	31.7	27.7

GEOCON

CONSOLIDATION TEST RESULTS

ASTM D-2435

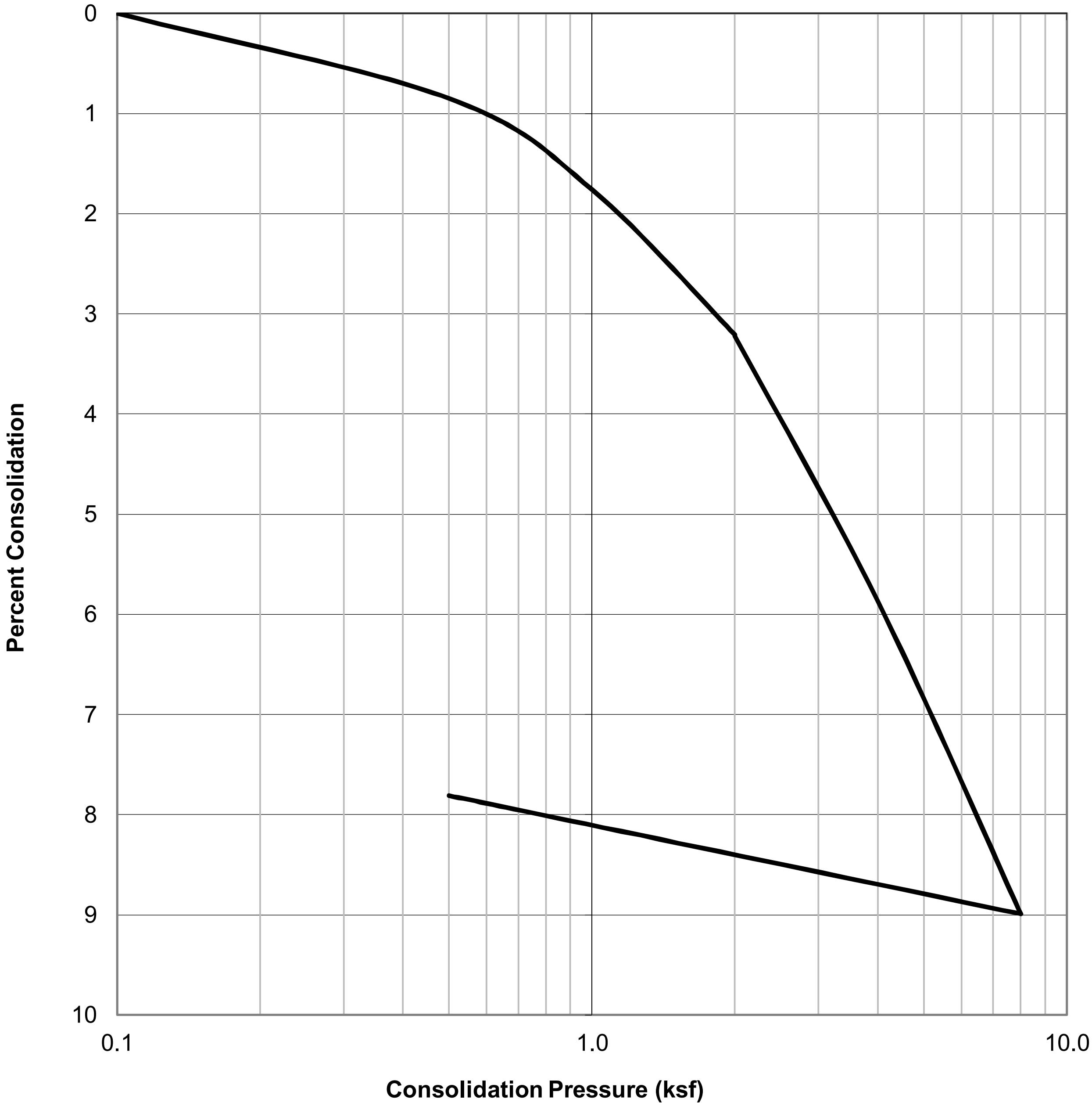
Checked by: JJK

Project No.: W1208-06-01

12805-12825 WEST VENTURA BOULEVARD
STUDIO CITY, CALIFORNIA

JUNE 2021

WATER ADDED AT 2.0 KSF



SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B2@30	Brown Silty Clay (CL)	90.9	30.8	25.2

GEOCON

CONSOLIDATION TEST RESULTS

ASTM D-2435

Checked by: JJK

Project No.: W1208-06-01

12805-12825 WEST VENTURA BOULEVARD
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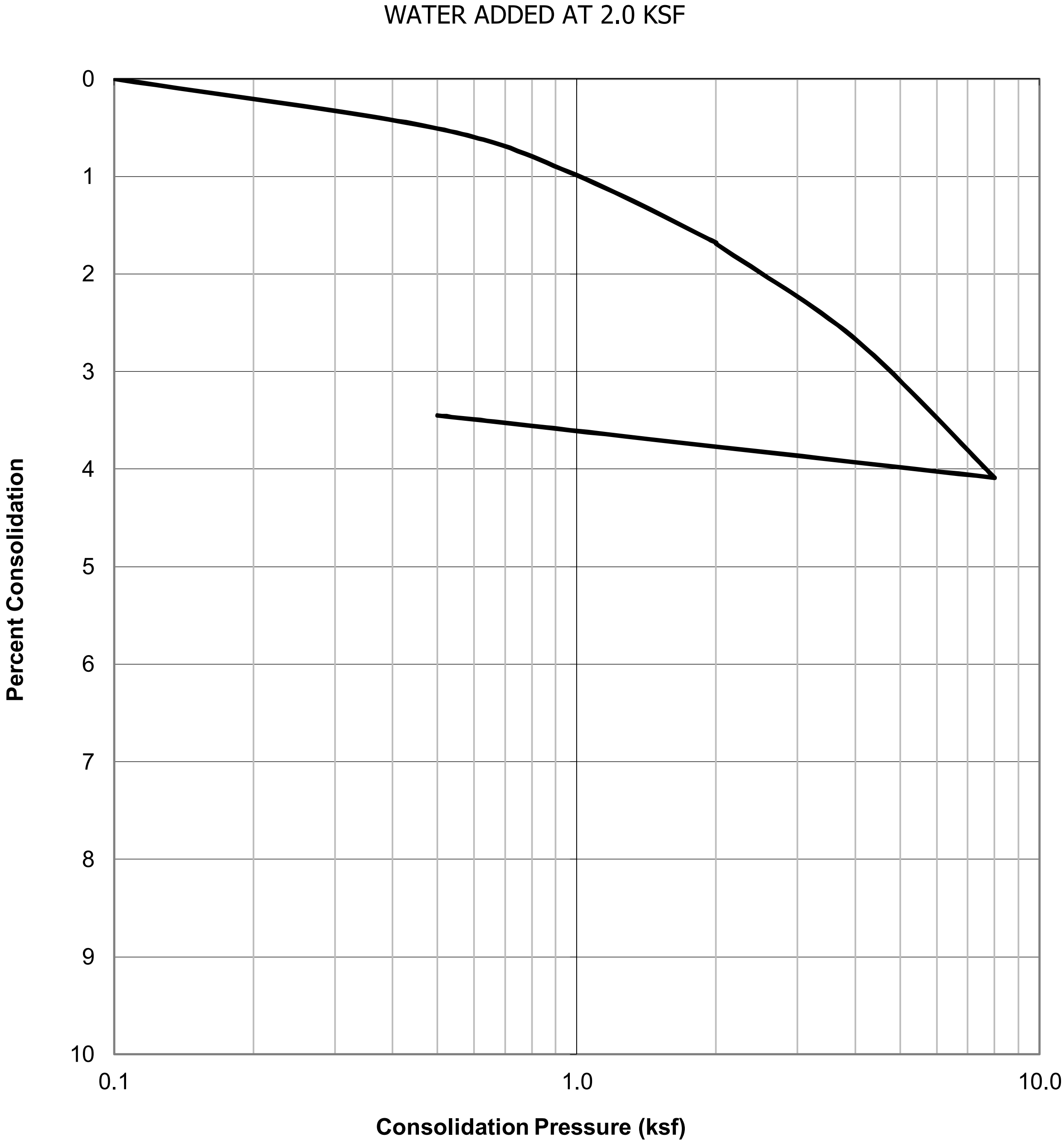
Figure B24



SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B2@40	Brown Silty Sand (SM)	98.3	24.9	23.0

GEOCON

CONSOLIDATION TEST RESULTS

ASTM D-2435

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Project No.: W1208-06-01

12805-12825 WEST VENTURA BOULEVARD
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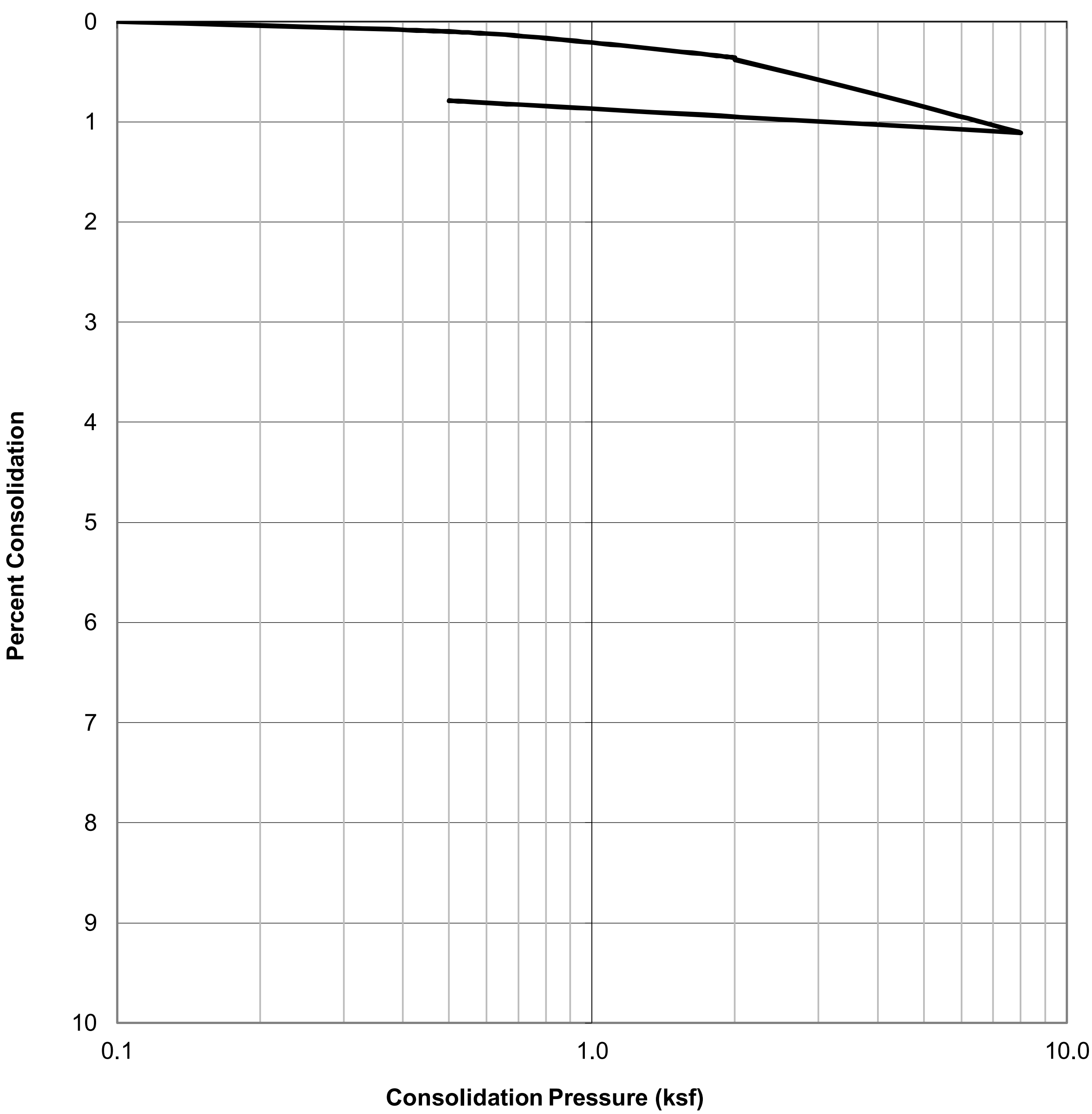
JUNE 2021

Figure B25

WATER ADDED AT 2.0 KSF



SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B2@50	Olive Gray Sand w/ Silt (SP-SM)	104.1	19.2	20.2

GEOCON

CONSOLIDATION TEST RESULTS

ASTM D-2435

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Project No.: W1208-06-01

12805-12825 WEST VENTURA BOULEVARD
STUDIO CITY, CALIFORNIA

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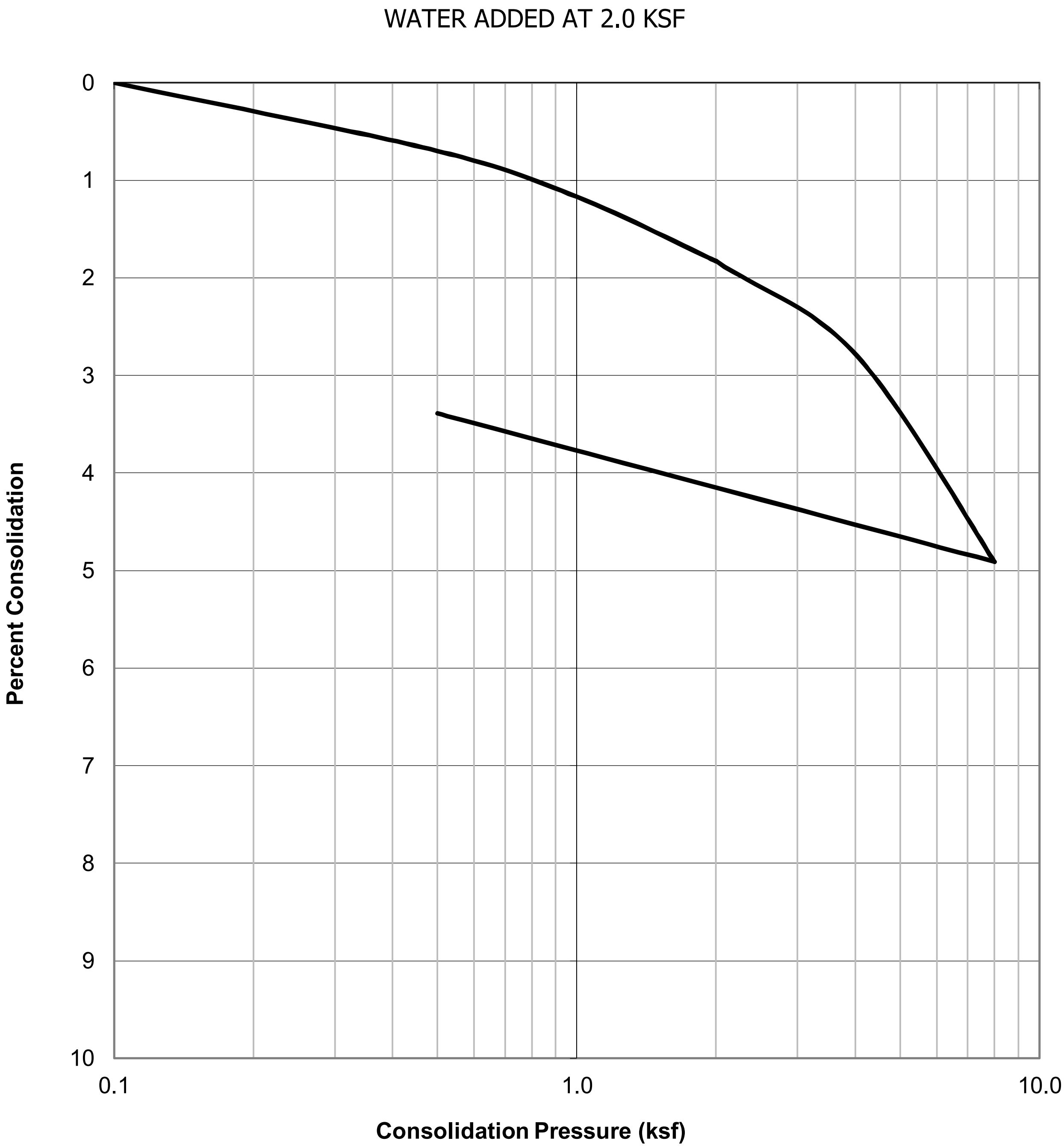
Figure B26



SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B2@60	Brown Silt (ML)	95.0	27.5	26.7

GEOCON

CONSOLIDATION TEST RESULTS

ASTM D-2435

Checked by: JJK

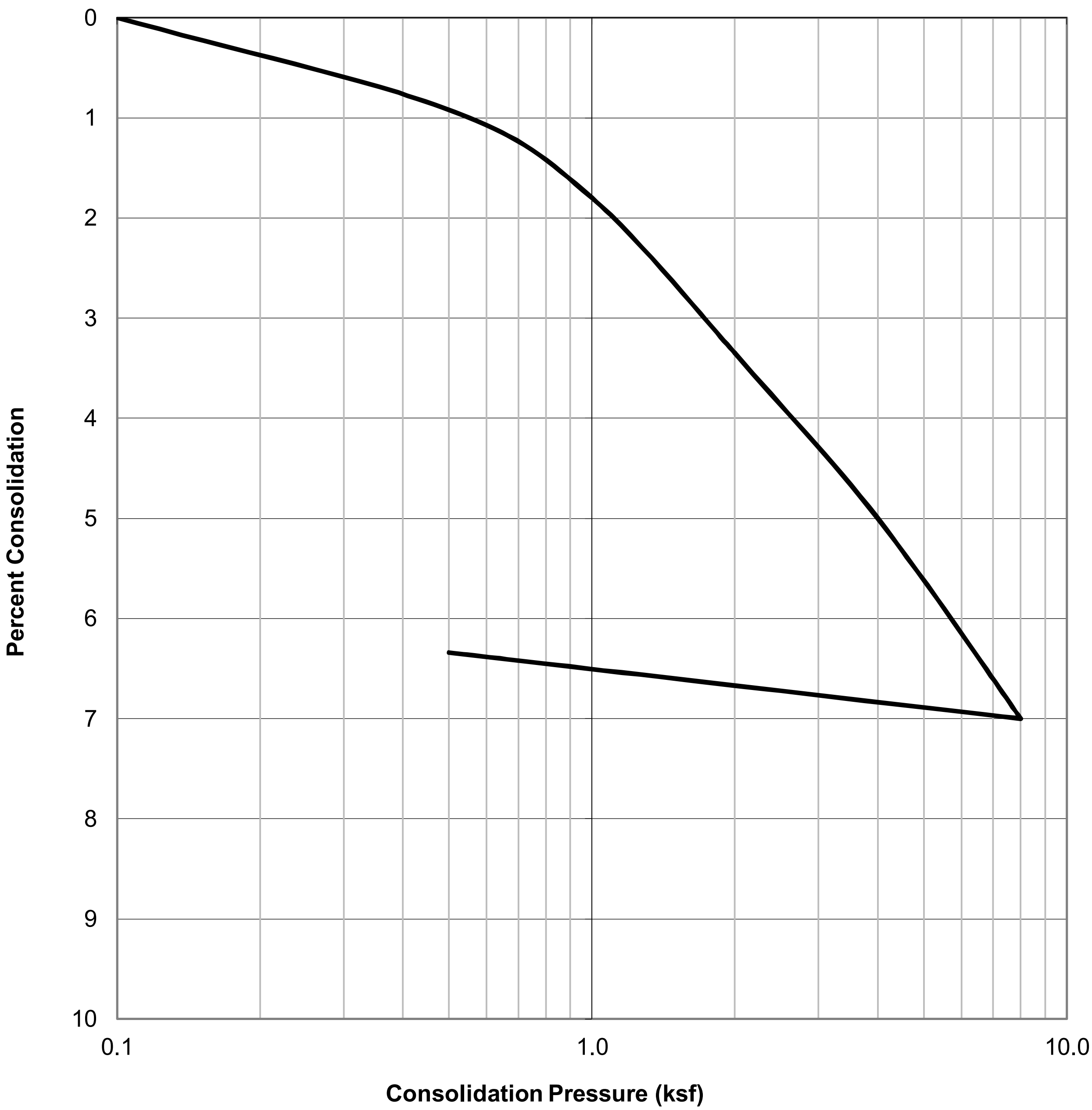
Project No.: W1208-06-01

12805-12825 WEST VENTURA BOULEVARD
STUDIO CITY, CALIFORNIA

JUNE 2021

Figure B27

WATER ADDED AT 2.0 KSF



SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B4@35	Brown Silty Clay (CL)	90.5	31.0	24.3

GEOCON

CONSOLIDATION TEST RESULTS

ASTM D-2435

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Project No.: W1208-06-01

12805-12825 WEST VENTURA BOULEVARD
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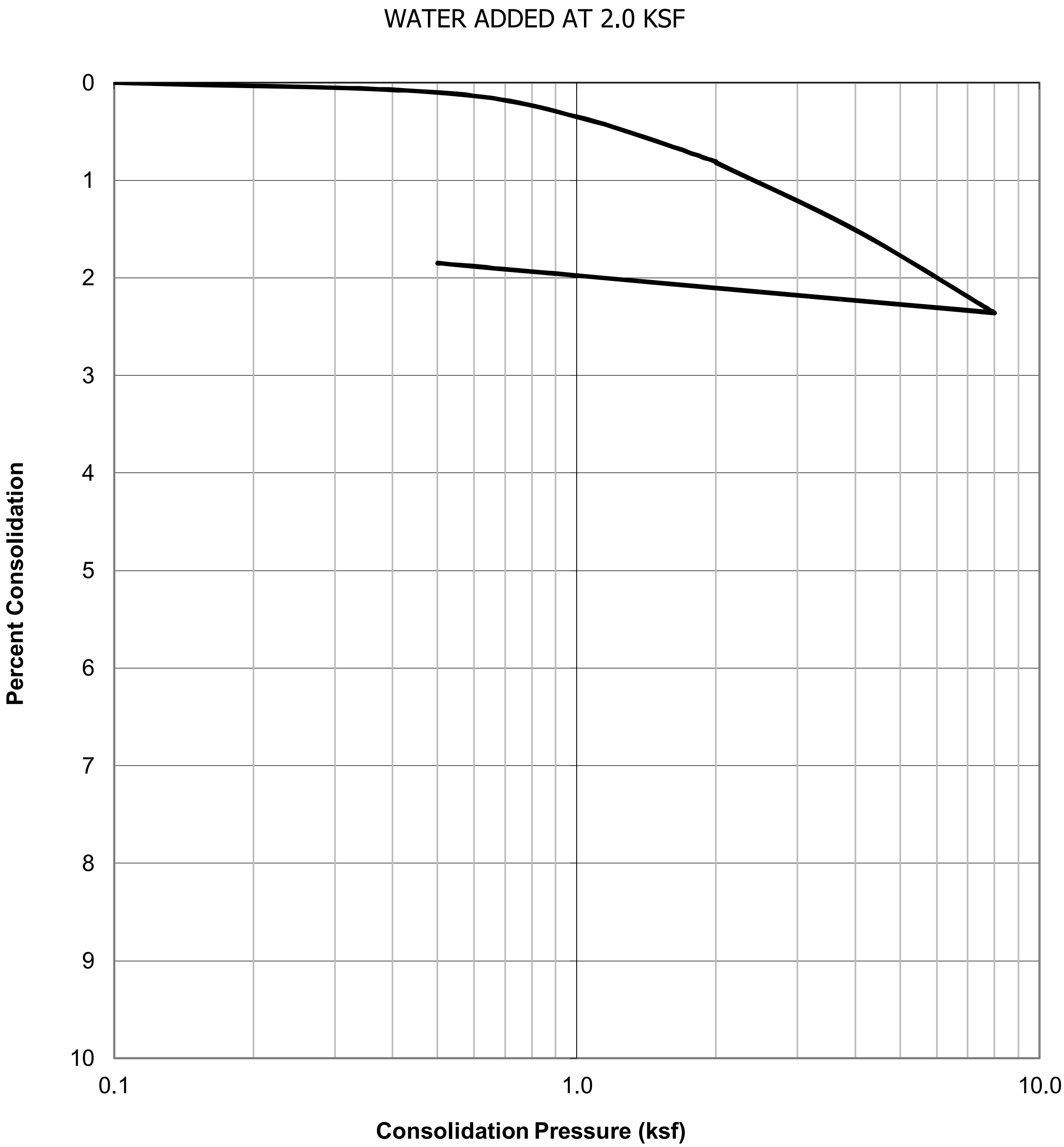
Figure B28



SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B4@40	Brown Silty Sand (SM)	94.2	26.8	25.3

GEOCON

CONSOLIDATION TEST RESULTS

ASTM D-2435

Checked by: JJK

Project No.: W1208-06-01

12805-12825 WEST VENTURA BOULEVARD
STUDIO CITY, CALIFORNIA

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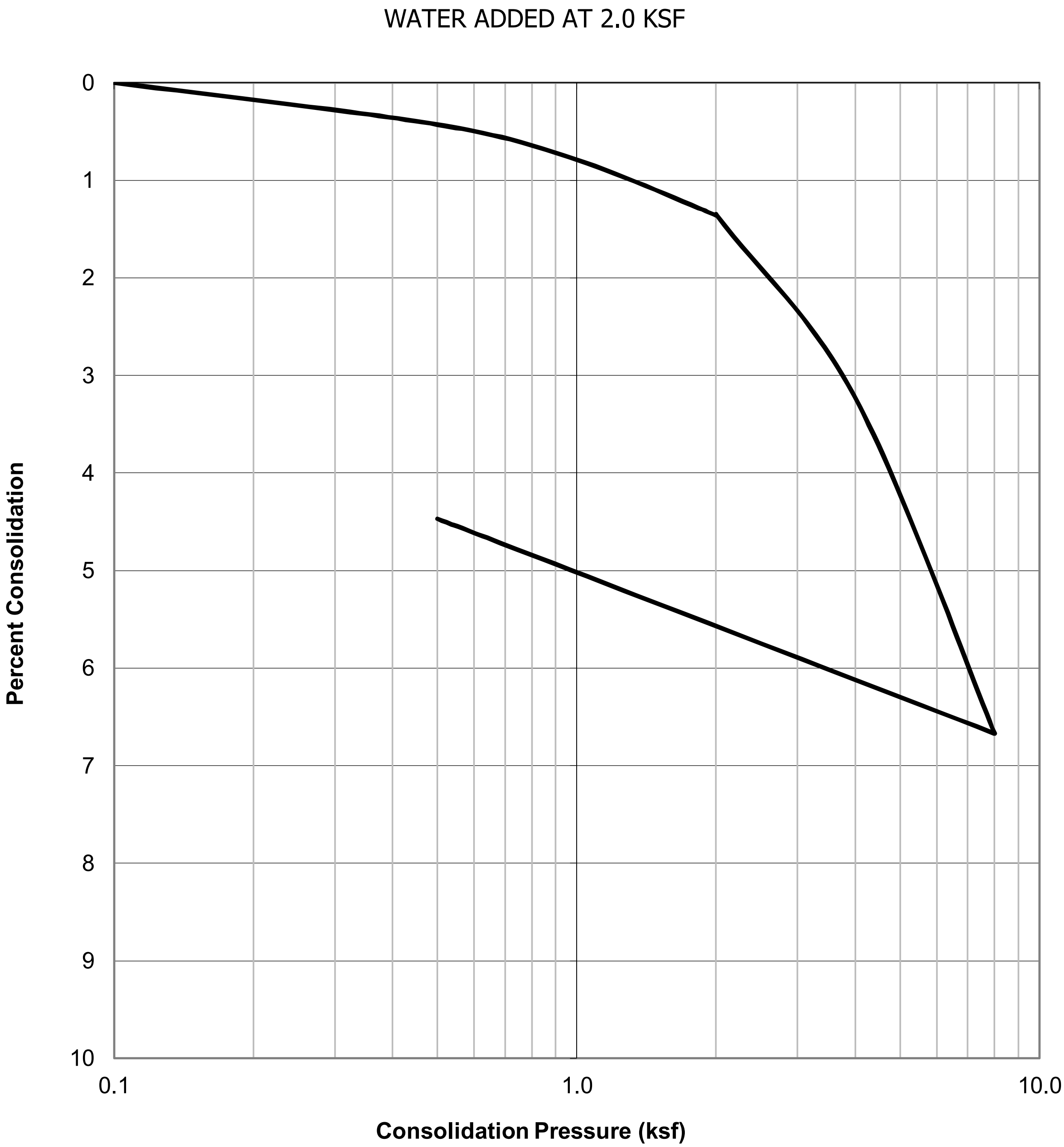
Figure B29



SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B4@50	Gray Clayey Silt (ML)	83.5	36.7	34.3

GEOCON

CONSOLIDATION TEST RESULTS

ASTM D-2435

Checked by: JJK

Project No.: W1208-06-01

12805-12825 WEST VENTURA BOULEVARD
STUDIO CITY, CALIFORNIA

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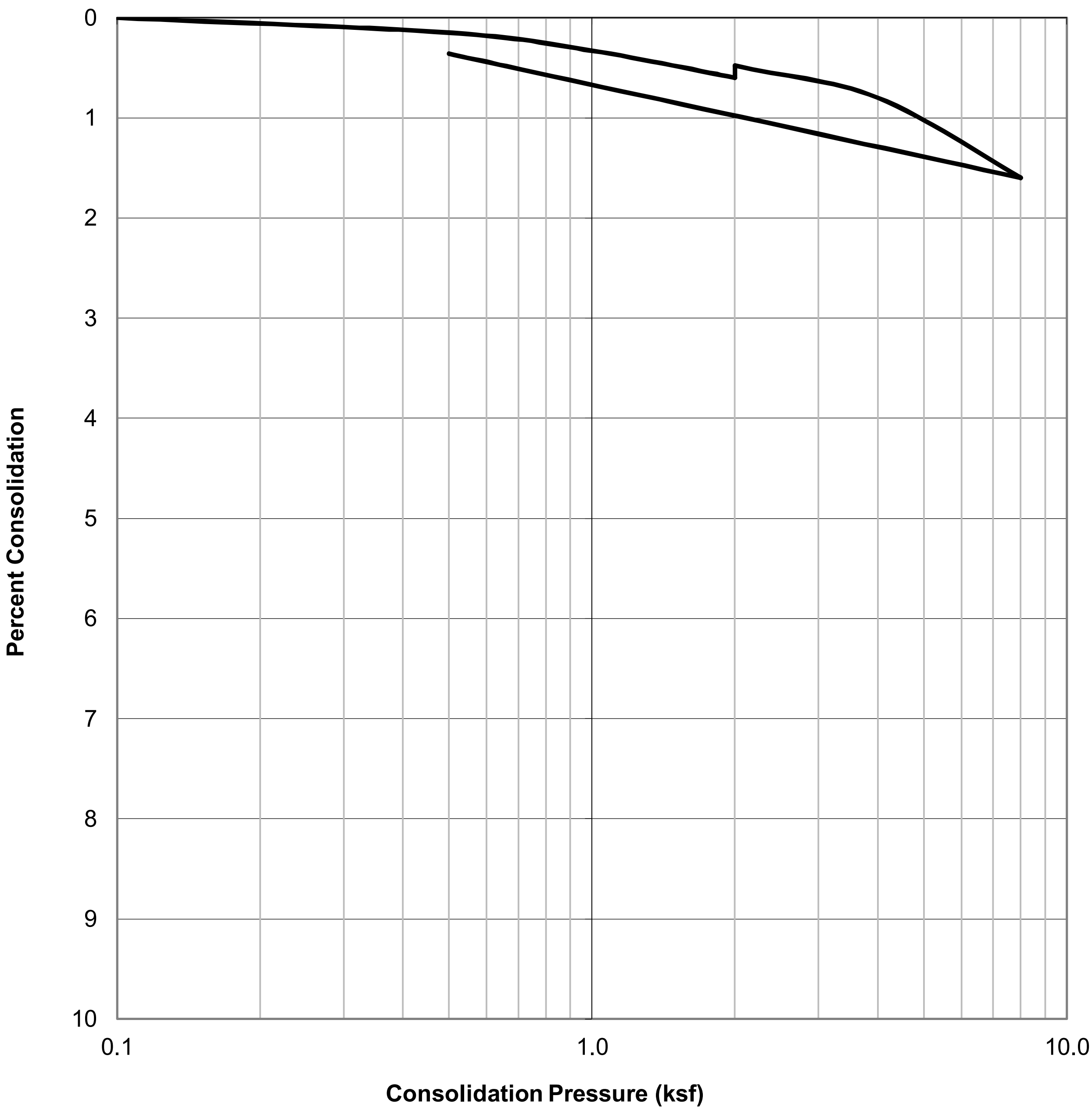
Figure B30

WATER ADDED AT 2.0 KSF



SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B4@60	Dark Gray Bedrock (BDRX)	83.6	36.3	37.3

GEOCON

CONSOLIDATION TEST RESULTS

ASTM D-2435

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Project No.: W1208-06-01

12805-12825 WEST VENTURA BOULEVARD
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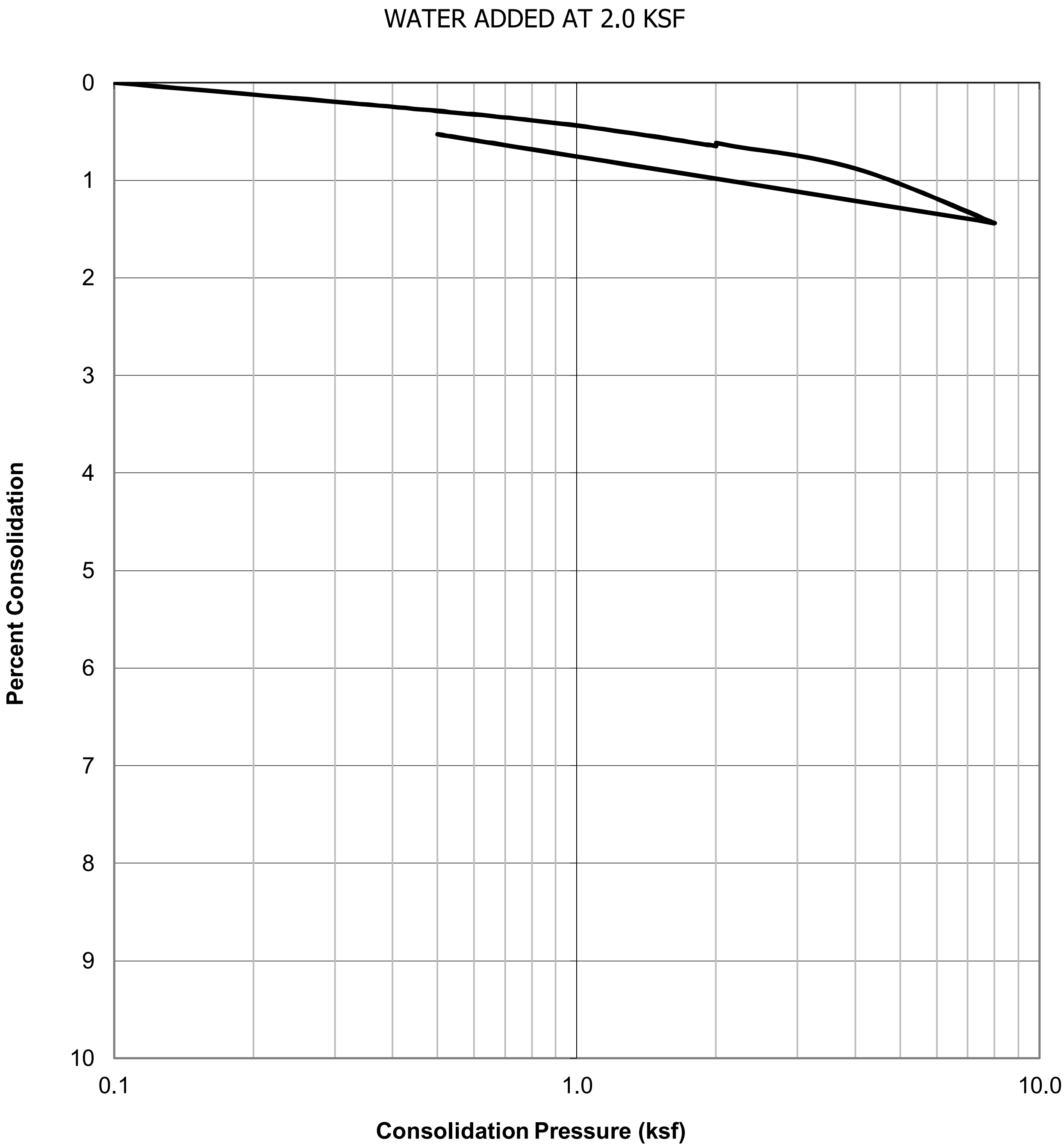
Figure B31



SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B4@65	Dark Gray Bedrock (BDRX)	85.8	32.8	33.7

GEOCON

CONSOLIDATION TEST RESULTS

ASTM D-2435

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12805-12825 WEST VENTURA BOULEVARD
STUDIO CITY, CALIFORNIA

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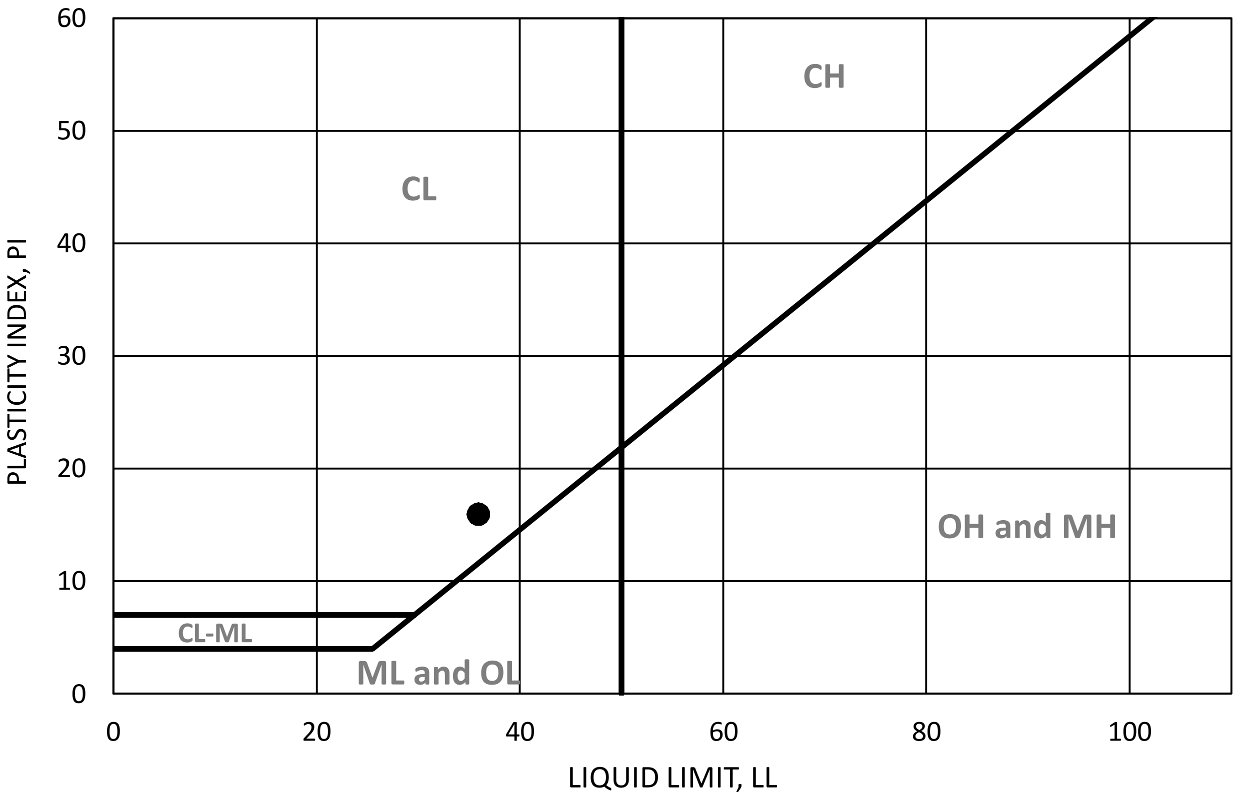
Figure B32



SYMBOL	BORING	DEPTH (ft)	LL	PL	PI	MOISTURE CONTENT AT SATURATION	SOIL BEHAVIOR
■	B1	10	N/P	N/P	N/P		
◆	B1	15	N/P	N/P	N/P		
▲	B1	20	N/P	N/P	N/P		
●	B1	25	36	20	16		CL
□							
◇							
△							
○							

N/P = Non-Plastic

GEOCON

ATTERBERG LIMITS

ASTM D-4318

Checked by: JJK

Project No.: W1208-06-01

12805-12825 WEST VENTURA BOULEVARD
STUDIO CITY, CALIFORNIA

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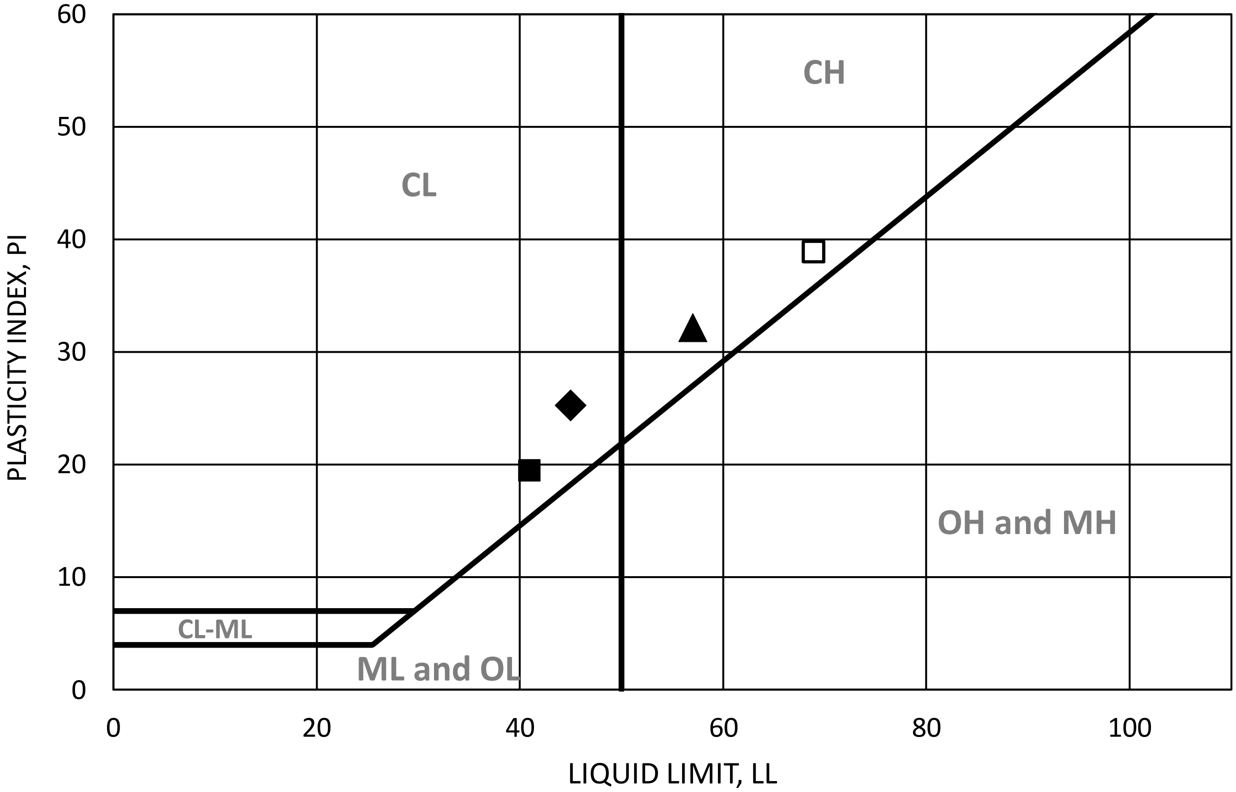
Figure B33



SYMBOL	BORING	DEPTH (ft)	LL	PL	PI	MOISTURE CONTENT AT SATURATION	SOIL BEHAVIOR
■	B1	30	41	22	19		CL
◆	B1	35	45	20	25		CL
▲	B1	40	57	25	32		CH
●	B1	45	N/P	N/P	N/P		
□	B1	50	69	30	39		CH
◇							
△							
○							

N/P = Non-Plastic

GEOCON

ATTERBERG LIMITS

ASTM D-4318

Checked by: JJK

Project No.: W1208-06-01

12805-12825 WEST VENTURA BOULEVARD
STUDIO CITY, CALIFORNIA

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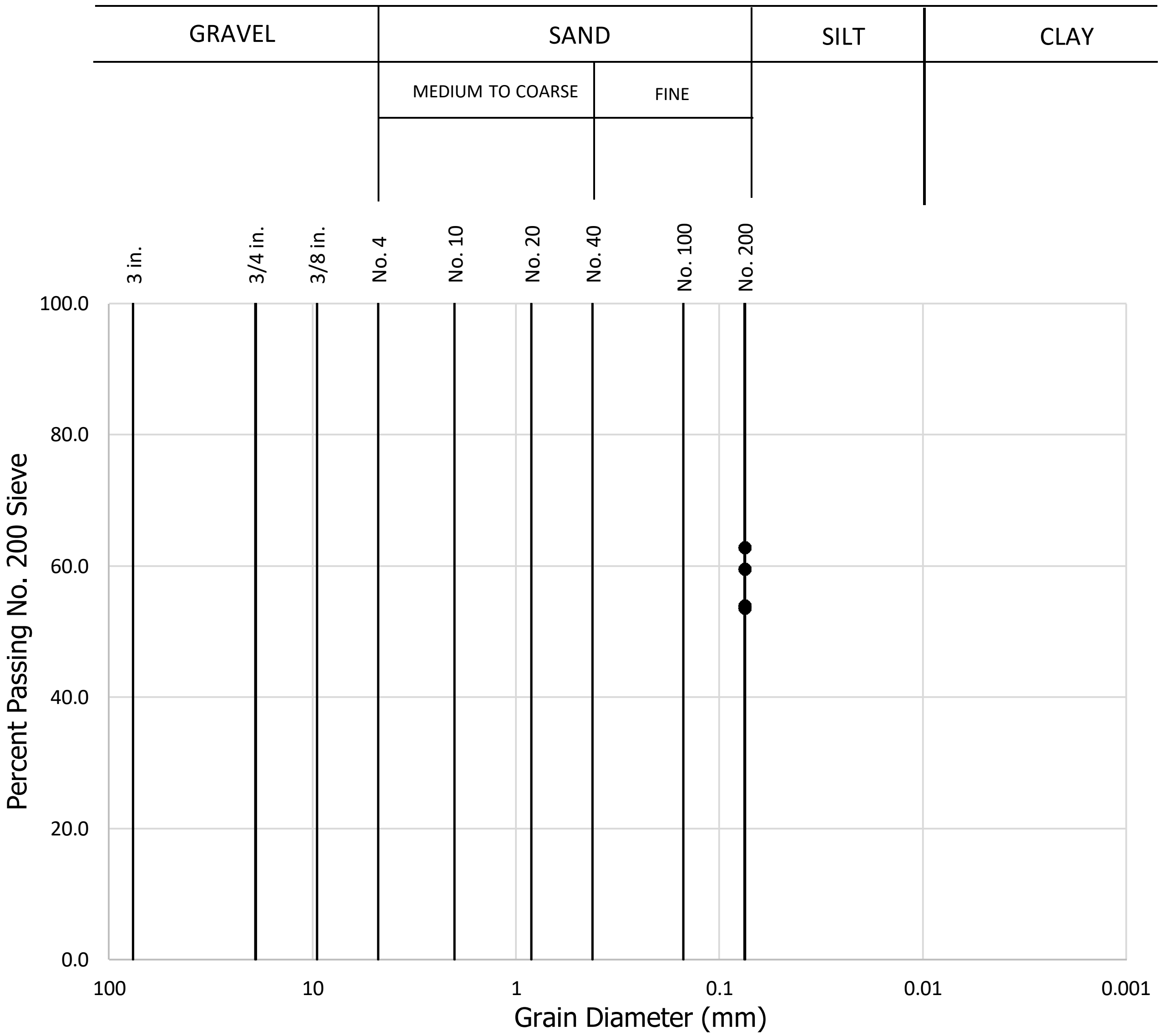
Figure B34



Sample No.	Percent Passing No. 200 Sieve
B1 @ 10'	53.9
B1 @ 15'	59.5
B1 @ 20'	53.6
B1 @ 25'	62.7

GEOCON

GRAIN SIZE ANALYSIS

ASTM D-1140

Checked by: JJK

Project No.: W1208-06-01

12805-12825 WEST VENTURA BOULEVARD
STUDIO CITY, CALIFORNIA

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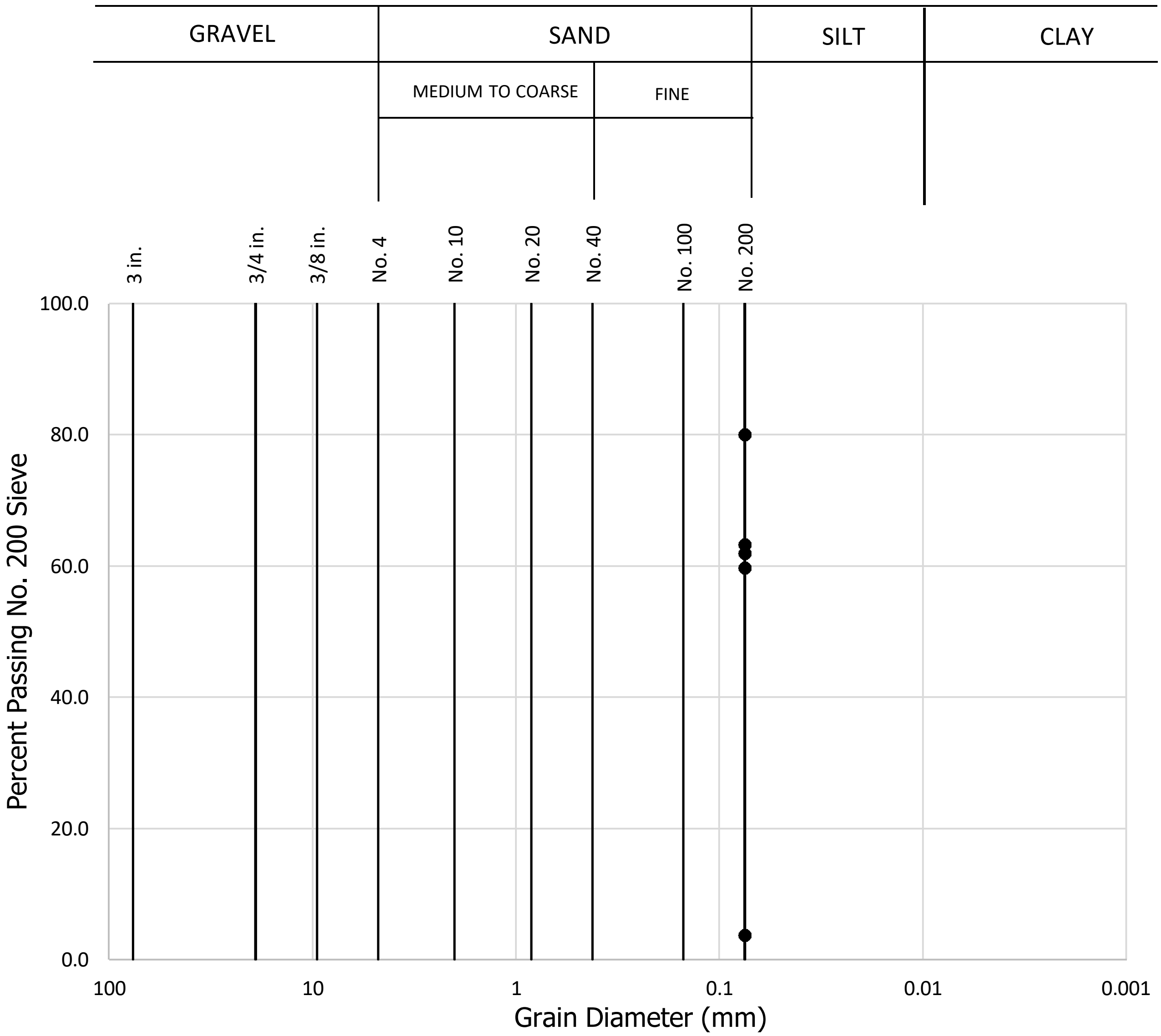
Figure B35



Sample No.	Percent Passing No. 200 Sieve
B1 @ 30'	61.9
B1 @ 35'	59.6
B1 @ 40'	80.0
B1 @ 45'	3.6
B1 @ 50'	63.2

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GRAIN SIZE ANALYSIS

ASTM D-1140

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Project No.: W1208-06-01

12805-12825 WEST VENTURA BOULEVARD
STUDIO CITY, CALIFORNIA

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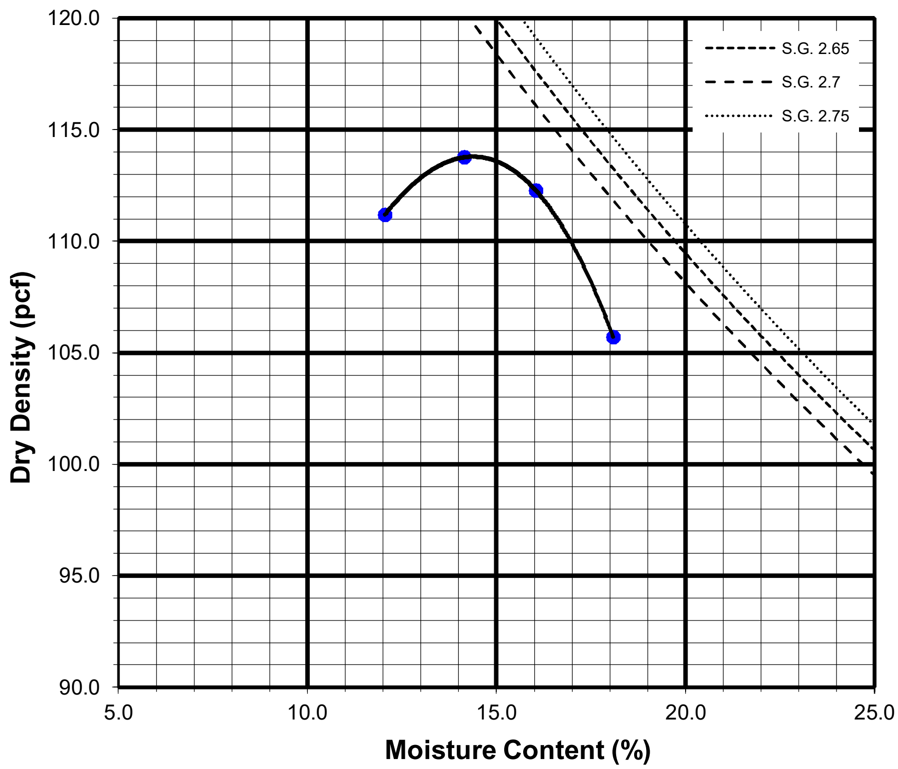
Figure B36

Sample No:

B3@0-5'	Dark Brown Silt (ML)

TEST NO.		1	2	3	4	5	6
Wt. Compacted Soil + Mold	(g)	6020	6100	6107	6024		
Weight of Mold	(g)	4139	4139	4139	4139		
Net Weight of Soil	(g)	1882	1961	1968	1886		
Wet Weight of Soil + Cont.	(g)	638.5	686.7	598.6	673.6		
Dry Weight of Soil + Cont.	(g)	585.7	619.9	536.3	592.6		
Weight of Container	(g)	147.3	147.5	147.8	144.7		
Moisture Content	(%)	12.0	14.1	16.0	18.1		
Wet Density	(pcf)	124.6	129.8	130.3	124.8		
Dry Density	(pcf)	111.2	113.8	112.3	105.7		

Maximum Dry Density (pcf)	**114.0**	**Optimum Moisture Content (%)**	**15.0**



Preparation Method: A

GEOCON	**MODIFIED COMPACTION TEST OF SOILS** ASTM D-1557	Project No.: W1208-06-01
		12805-12825 WEST VENTURA BOULEVARD STUDIO CITY, CALIFORNIA
	Checked by: JJK	JUNE 2021 Figure B37

B2@10-15'

MOLDED SPECIMEN		BEFORE TEST	AFTER TEST
Specimen Diameter	(in.)	4.0	4.0
Specimen Height	(in.)	1.0	1.1
Wt. Comp. Soil + Mold	(gm)	711.5	773.0
Wt. of Mold	(gm)	368.2	368.2
Specific Gravity	(Assumed)	2.7	2.7
Wet Wt. of Soil + Cont.	(gm)	486.6	773.0
Dry Wt. of Soil + Cont.	(gm)	445.4	296.2
Wt. of Container	(gm)	186.6	368.2
Moisture Content	(%)	15.9	36.7
Wet Density	(pcf)	103.6	122.0
Dry Density	(pcf)	89.3	89.2
Void Ratio		0.9	1.1
Total Porosity		0.5	0.5
Pore Volume	(cc)	97.3	118.3
Degree of Saturation	(%) [S_{meas}]	48.8	91.8

Date	Time	Pressure (psi)	Elapsed Time (min)	Dial Readings (in.)
8/26/2020	10:00	1.0	0	0.3075
8/26/2020	10:10	1.0	10	0.3065
Add Distilled Water to the Specimen				
8/27/2020	10:00	1.0	1430	0.408
8/27/2020	11:00	1.0	1490	0.408

Expansion Index (EI meas) =	101.5
Expansion Index (Report) =	**102**

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: JJK

Project No.: W1208-06-01

12805-12825 WEST VENTURA BOULEVARD
STUDIO CITY, CALIFORNIA

JUNE 2021

Figure B38

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

Sample No.	**pH**	**Resistivity (ohm centimeters)**
B1 @ 30-35'	7.8	960 (Severely Corrosive)
B2 @ 10-15'	7.6	780 (Severely Corrosive)

SUMMARY OF LABORATORY CHLORIDE CONTENT TEST RESULTS
EPA NO. 325.3

Sample No.	Chloride Ion Content (%)
B1 @ 30-35'	0.013
B2 @ 10-15'	0.005

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

Sample No.	**Water Soluble Sulfate (% SQ_4)**	**Sulfate Exposure***
B1 @ 30-35'	0.013	S0
B2 @ 10-15'	0.000	S0

GEOCON

CORROSIVITY TEST RESULTS

Checked by: JJK

Project No.: W1208-06-01

12805-12825 WEST VENTURA BOULEVARD
LOS ANGELES, CALIFORNIA

JUNE 2021

APPENDIX C

APPENDIX C

PRIOR BORING LOGS

GEOTECHNICAL BORING LOG

PROJECT NAME WS Development | W.O. NO. 6792
DRILLING COMPANY Choice Drilling | DATE STARTED: 9-16-14 | BORING NO. B-1
TYPE OF DRILL RIG Auto Hammer | LOGGED BY RC | SHEET 1 OF 3
DRILLING METHOD Hollow Stem | HAMMER WEIGHT (LBS) 140 | GROUND ELEVATION (FT)
DIAMETER OF HOLE 8 | DROP (IN) 30 | GW ELEVATION
BORING LOCATION:

DEPTH (FT)	SAMPLE TYPE	BLOWS/ 6 IN.	GEOTECHNICAL DESCRIPTION	MOISTURE CONTENT (%)	DRY DENSITY (pcf)	OTHER TESTS
			0-8", AC			
			8"-17', FILL			
5	Bulk Sample; California Ring	36/29	@ 5-10', Brown, sandy CLAY with rock fragments, moist, stiff	15.2	110.1	
10	Standard Penetration Test	6/6/7		15.3	-----	
15	California Ring	16/20	@ 15', Orange brown, silty SAND with siltstone, moist, dense	23.8	86.7	
			17-60', ALLUVIUM			
20	Standard Penetration Test	1/1/2	@ 20-30', Brown, silty, slightly sandy CLAY, very moist, soft	30.4	-----	
25	California Ring	12/16		28.2	96.3	Cons

LEGEND

- Standard Penetration Test
- California Ring
- Rock Core
- Bulk Sample
- Shelby Tube
- Water Seepage
- Groundwater

SIEVE: GRAIN SIZE ANALYSIS
MAX: MAXIMUM DRY DENSITY
DS: DIRECT SHEAR
CONS: CONSOLIDATION
HYDR: HYDROMETER ANALYSIS
EXPAN: EXPANSION INDEX
CHEM: CHEMICAL TESTS

PLATE A-1

GeoSoils Consultants, Inc.
GEOTECHNICAL * GEOLOGIC * ENVIRONMENTAL

GEOTECHNICAL BORING LOG

PROJECT NAME WS Development
W.O. NO. 6792
DRILLING COMPANY Choice Drilling
DATE STARTED: 9-16-14
BORING NO. B-1
TYPE OF DRILL RIG Auto Hammer
LOGGED BY RC
SHEET 2 OF 3
DRILLING METHOD Hollow Stem
HAMMER WEIGHT (LBS) 140
GROUND ELEVATION (FT)
DIAMETER OF HOLE 8
DROP (IN) 30
GW ELEVATION
BORING LOCATION:

DEPTH (FT)	SAMPLE TYPE	BLOWS/ 6 IN.	GEOTECHNICAL DESCRIPTION	MOISTURE CONTENT (%)	DRY DENSITY (pcf)	OTHER TESTS
	Standard Penetration Test	2/2/3		30.4		Sieve Att
35	California Ring	14/16	@ 35-40', Brown, silty, slightly sandy CLAY, very moist, stiff	22.9	104.0	Cons
40	Standard Penetration Test	10/12/16		28.2	-----	
45	California Ring	15/26	@ 45-50', Brown, slightly silty, very fine to coarse SAND with gravel, moist, dense	9.7	123.3	Cons
50	Standard Penetration Test	14/19/26		17.7	-----	Sieve
55	California Ring	20/35	@ 55', Orange brown, clayey, sandy SILT, moist, stiff	47.3	71.6	

LEGEND

- Standard Penetration Test
- California Ring
- Rock Core
- Bulk Sample
- Shelby Tube
- Water Seepage
- Groundwater

SIEVE: GRAIN SIZE ANALYSIS
MAX: MAXIMUM DRY DENSITY
DS: DIRECT SHEAR
CONS: CONSOLIDATION
HYDR: HYDROMETER ANALYSIS
EXPAN: EXPANSION INDEX
CHEM: CHEMICAL TESTS

PLATE A-2

GeoSoils Consultants, Inc.
GEOTECHNICAL * GEOLOGIC * ENVIRONMENTAL

GEOTECHNICAL BORING LOG

PROJECT NAME: WS Development
W.O. NO. 6792
DRILLING COMPANY: Choice Drilling
DATE STARTED: 9-16-14
BORING NO. B-1
TYPE OF DRILL RIG: Auto Hammer
LOGGED BY: RC
SHEET 3 OF 3
DRILLING METHOD: Hollow Stem
HAMMER WEIGHT (LBS): 140
GROUND ELEVATION (FT):
DIAMETER OF HOLE: 8
DROP (IN): 30
GW ELEVATION:
BORING LOCATION:

DEPTH (FT)	SAMPLE TYPE	BLOWS/ 6 IN.	GEOTECHNICAL DESCRIPTION	MOISTURE CONTENT (%)	DRY DENSITY (pcf)	OTHER TESTS
	Standard Penetration Test	13/18/26	**60-65', BEDROCK** @ 60-65', Gray brown, silty CLAY, moist, stiff	47.3	-----	
65	Standard Penetration Test	14/18/25	TD @ 65' Groundwater @ 21' No caving	54.6	-----	
70						
75						
80						
85						

LEGEND

- Standard Penetration Test
- California Ring
- Rock Core
- Bulk Sample
- Shelby Tube
- Water Seepage
- Groundwater

SIEVE: GRAIN SIZE ANALYSIS
MAX: MAXIMUM DRY DENSITY
DS: DIRECT SHEAR
CONS: CONSOLIDATION
HYDR: HYDROMETER ANALYSIS
EXPAN: EXPANSION INDEX
CHEM: CHEMICAL TESTS

PLATE A-3

GeoSoils Consultants, Inc.
GEOTECHNICAL * GEOLOGIC * ENVIRONMENTAL

GEOTECHNICAL BORING LOG

PROJECT NAME: WS Development
W.O. NO.: 6792
DRILLING COMPANY: Choice Drilling
DATE STARTED: 9-16-14
BORING NO.: B-2
TYPE OF DRILL RIG: Auto Hammer
LOGGED BY: RC
SHEET 1 OF 3
DRILLING METHOD: Hollow Stem
HAMMER WEIGHT (LBS): 140
GROUND ELEVATION (FT):
DIAMETER OF HOLE: 8
DROP (IN): 30
GW ELEVATION:
BORING LOCATION:

DEPTH (FT)	SAMPLE TYPE	BLOWS/ 6 IN.	GEOTECHNICAL DESCRIPTION	MOISTURE CONTENT (%)	DRY DENSITY (pcf)	OTHER TESTS
			0-6", AC; 6-8", FILL SAND; 8-10", FILL			
			10"-17', FILL			
5						
	California Ring	15/22	@ 7.5-12.5', Brown, sandy CLAY with rock fragments, moist, stiff	43.8	85.9	
10	Bulk Sample (10-15)					
	Standard Penetration Test	3/3/4		23.8	-----	
15						
	California Ring	3/5	@ 17.5', Gray brown, slightly sandy CLAY, very moist, soft	22.9	96.2	
20	Bulk Sample (20-22)					
	Standard Penetration Test	1/1/1	@ 22.5', Brown, slightly sandy CLAY, very moist, soft	28.2	-----	Sieve Att
25			@ 25', Brown, silty, sandy CLAY to clayey, sandy SILT with abundant rock fragments, very moist, medium dense			
	California Ring	7/9		35.0	86.8	Cons

LEGEND

- Standard Penetration Test
- California Ring
- Rock Core
- Bulk Sample
- Shelby Tube
- Water Seepage
- Groundwater

SIEVE: GRAIN SIZE ANALYSIS
MAX: MAXIMUM DRY DENSITY
DS: DIRECT SHEAR
CONS: CONSOLIDATION
HYDR: HYDROMETER ANALYSIS
EXPAN: EXPANSION INDEX
CHEM: CHEMICAL TESTS

PLATE A-4

GeoSoils Consultants, Inc.
GEOTECHNICAL * GEOLOGIC * ENVIRONMENTAL

GEOTECHNICAL BORING LOG

PROJECT NAME: WS Development
W.O. NO. 6792
DRILLING COMPANY: Choice Drilling
DATE STARTED: 9-16-14
BORING NO. B-2
TYPE OF DRILL RIG: Auto Hammer
LOGGED BY: RC
SHEET 2 OF 3
DRILLING METHOD: Hollow Stem
HAMMER WEIGHT (LBS): 140
GROUND ELEVATION (FT)
DIAMETER OF HOLE: 8
DROP (IN): 30
GW ELEVATION
BORING LOCATION:

DEPTH (FT)	SAMPLE TYPE	BLOWS/ 6 IN.	GEOTECHNICAL DESCRIPTION	MOISTURE CONTENT (%)	DRY DENSITY (pcf)	OTHER TESTS
			@ 30', Brown, slightly sandy CLAY, moist, stiff			
	Standard Penetration Test	3/4/5		22.9	-----	
35						
	California Ring	9/29	@ 37.5', Orange brown, slightly silty, fine to medium SAND to sandy CLAY, very moist, dense, stiff	15.6	116.0	Cons
40						
	Standard Penetration Test	7/9/11	@ 42.5', Orange brown, slightly silty, very fine to medium SAND, very moist, dense	23.6	-----	
45						
50	California Ring	19/27	@ 50', Brown, slightly silty, fine to coarse SAND with gravel, very moist, dense	12.6	119.7	Sieve
55	Standard Penetration Test	7/9/17	@ 55', Brown, gravelly, fine to coarse SAND, very moist, dense with sandy clay	31.9	-----	

LEGEND

- Standard Penetration Test
- California Ring
- Rock Core
- Bulk Sample
- Shelby Tube
- Water Seepage
- Groundwater

SIEVE: GRAIN SIZE ANALYSIS
MAX: MAXIMUM DRY DENSITY
DS: DIRECT SHEAR
CONS: CONSOLIDATION
HYDR: HYDROMETER ANALYSIS
EXPAN: EXPANSION INDEX
CHEM: CHEMICAL TESTS

PLATE A-5

GeoSoils Consultants, Inc.
GEOTECHNICAL * GEOLOGIC * ENVIRONMENTAL

GEOTECHNICAL BORING LOG

PROJECT NAME WS Development
DRILLING COMPANY Choice Drilling DATE STARTED: 9-16-14
TYPE OF DRILL RIG Auto Hammer LOGGED BY RC
DRILLING METHOD Hollow Stem HAMMER WEIGHT (LBS) 140
DIAMETER OF HOLE 8 DROP (IN) 30
BORING LOCATION:

W.O. NO. 6792
BORING NO. B-2
SHEET 3 OF 3
GROUND ELEVATION (FT)
GW ELEVATION

DEPTH (FT)	SAMPLE TYPE	BLOWS/ 6 IN.	GEOTECHNICAL DESCRIPTION	MOISTURE CONTENT (%)	DRY DENSITY (pcf)	OTHER TESTS
	California Ring	18/16	**60-65', BEDROCK** @ 60-65', Gray, silty CLAY (layered), very moist, stiff	48.6	72.8	
65	California Ring	17/38	TD @ 65' Groundwater @ 22' No caving	66.9	63.1	
70						
75						
80						
85						

LEGEND

- Standard Penetration Test
- California Ring
- Rock Core
- Bulk Sample
- Shelby Tube
- Water Seepage
- Groundwater

SIEVE: GRAIN SIZE ANALYSIS
MAX: MAXIMUM DRY DENSITY
DS: DIRECT SHEAR
CONS: CONSOLIDATION
HYDR: HYDROMETER ANALYSIS
EXPAN: EXPANSION INDEX
CHEM: CHEMICAL TESTS

PLATE A-6

GeoSoils Consultants, Inc.
GEOTECHNICAL * GEOLOGIC * ENVIRONMENTAL

GEOTECHNICAL BORING LOG

PROJECT NAME WS Development — W.O. NO. 6792
DRILLING COMPANY Choice Drilling — DATE STARTED: 9-16-14 — BORING NO. B-3
TYPE OF DRILL RIG Auto Hammer — LOGGED BY RC — SHEET 1 OF 3
DRILLING METHOD Hollow Stem — HAMMER WEIGHT (LBS) 140 — GROUND ELEVATION (FT)
DIAMETER OF HOLE 8 — DROP (IN) 30 — GW ELEVATION
BORING LOCATION:

DEPTH (FT)	SAMPLE TYPE	BLOWS/ 6 IN.	GEOTECHNICAL DESCRIPTION	MOISTURE CONTENT (%)	DRY DENSITY (pcf)	OTHER TESTS
			0-7", AC			
			7"-17', FILL			
5	Standard Penetration Test	5/6/6	@ 5', Brown, sandy CLAY with some rock fragments, moist, stiff	24.4	-----	
10	California Ring	8/26		2.3	122.3	DS
15	Standard Penetration Test	4/3/6		46.6	-----	
			17-60', ALLUVIUM			
20	California Ring	10/14	@ 20', Orange brown, silty CLAY, moist, medium stiff	26.7	93.1	
25	Standard Penetration Test	1/1/2	@ 25', Brown, slightly sandy, silty CLAY, very moist, soft	26.8	-----	Sieve

LEGEND

- Standard Penetration Test
- California Ring
- Rock Core
- Bulk Sample
- Shelby Tube
- Water Seepage
- Groundwater

SIEVE: GRAIN SIZE ANALYSIS
MAX: MAXIMUM DRY DENSITY
DS: DIRECT SHEAR
CONS: CONSOLIDATION
HYDR: HYDROMETER ANALYSIS
EXPAN: EXPANSION INDEX
CHEM: CHEMICAL TESTS

PLATE A-7

GeoSoils Consultants, Inc.
GEOTECHNICAL * GEOLOGIC * ENVIRONMENTAL

GEOTECHNICAL BORING LOG

PROJECT NAME WS Development | W.O. NO. 6792
DRILLING COMPANY Choice Drilling | DATE STARTED: 9-16-14 | BORING NO. B-3
TYPE OF DRILL RIG Auto Hammer | LOGGED BY RC | SHEET 2 OF 3
DRILLING METHOD Hollow Stem | HAMMER WEIGHT (LBS) 140 | GROUND ELEVATION (FT)
DIAMETER OF HOLE 8 | DROP (IN) 30 | GW ELEVATION
BORING LOCATION:

DEPTH (FT)	SAMPLE TYPE	BLOWS/ 6 IN.	GEOTECHNICAL DESCRIPTION	MOISTURE CONTENT (%)	DRY DENSITY (pcf)	OTHER TESTS
	California Ring	11/14	@ 30-35', Brown, slightly sandy, silty CLAY, very moist, medium stiff	28.5	92.7	
35	Standard Penetration Test	3/4/5	@35', Brown, slightly sandy, silty CLAY, very moist, medium stiff	24.7	-----	Sieve
40	California Ring	15/19	@ 40', Dark brown, silty CLAY, very moist, stiff	20.3	110.9	
45	Standard Penetration Test	7/9/13	@ 45-50', Brown, fine to coarse SAND, very moist, dense	-----	-----	
50	California Ring	28/40	@ 50', No recovery	-----	-----	
55	Standard Penetration Test	15/18/22	@ 55', Orange brown, sandy SILT with sandy clay	21.7	-----	Sieve

LEGEND

- Standard Penetration Test
- California Ring
- Rock Core
- Bulk Sample
- Shelby Tube
- Water Seepage
- Groundwater

SIEVE: GRAIN SIZE ANALYSIS
MAX: MAXIMUM DRY DENSITY
DS: DIRECT SHEAR
CONS: CONSOLIDATION
HYDR: HYDROMETER ANALYSIS
EXPAN: EXPANSION INDEX
CHEM: CHEMICAL TESTS

PLATE A-8

GeoSoils Consultants, Inc.
GEOTECHNICAL * GEOLOGIC * ENVIRONMENTAL

GEOTECHNICAL BORING LOG

PROJECT NAME	WS Development			W.O. NO.	6792
DRILLING COMPANY	Choice Drilling	DATE STARTED:	9-16-14	BORING NO.	B-3
TYPE OF DRILL RIG	Auto Hammer	LOGGED BY	RC	SHEET	3 OF 3
DRILLING METHOD	Hollow Stem	HAMMER WEIGHT (LBS)	140	GROUND ELEVATION (FT)	
DIAMETER OF HOLE	8	DROP (IN)	30	GW ELEVATION	

BORING LOCATION:

DEPTH (FT)	SAMPLE TYPE	BLOWS/ 6 IN.	GEOTECHNICAL DESCRIPTION	MOISTURE CONTENT (%)	DRY DENSITY (pcf)	OTHER TESTS
	California Ring	25/38	<u>60-65', BEDROCK</u> @ 60-65', Gray, silty CLAY	17.4	111.3	DS
65			TD @ 65' Groundwater @ 24' No caving			
70						
75						
80						
85						

LEGEND

- Standard Penetration Test
- California Ring
- Rock Core
- Bulk Sample
- Shelby Tube
- Water Seepage
- Groundwater

SIEVE:	GRAIN SIZE ANALYSIS
MAX:	MAXIMUM DRY DENSITY
DS:	DIRECT SHEAR
CONS:	CONSOLIDATION
HYDR:	HYDROMETER ANALYSIS
EXPAN:	EXPANSION INDEX
CHEM:	CHEMICAL TESTS

PLATE A-9

GeoSoils Consultants, Inc.
GEOTECHNICAL * GEOLOGIC * ENVIRONMENTAL

GEOTECHNICAL BORING LOG

PROJECT NAME: WS Development
W.O. NO.: 6792
DRILLING COMPANY: Choice Drilling
DATE STARTED: 9-16-14
BORING NO.: B-4
TYPE OF DRILL RIG: Auto Hammer
LOGGED BY: RC
SHEET 1 OF 3
DRILLING METHOD: Hollow Stem
HAMMER WEIGHT (LBS): 140
GROUND ELEVATION (FT):
DIAMETER OF HOLE: 8
DROP (IN): 30
GW ELEVATION:
BORING LOCATION:

DEPTH (FT)	SAMPLE TYPE	BLOWS/ 6 IN.	GEOTECHNICAL DESCRIPTION	MOISTURE CONTENT (%)	DRY DENSITY (pcf)	OTHER TESTS
			0-7", AC			
			7"-17', FILL			
5						
7.5	Bulk Sample; California Ring	17/21	@ 7.5', Dark brown, sandy CLAY, moist, stiff	18.2	103.2	
10	Bulk Sample					
12.5	Standard Penetration Test	4/4/6	@ 12.5', Brown, sandy CLAY, moist, stiff	24.7	-----	
15						
17.5	California Ring	6/12	**17-51', ALLUVIUM** @ 17.5-27.5', Brown, silty, slightly sandy CLAY, very moist, medium stiff	49.1	79.8	
20						
22.5	Standard Penetration Test	3/3/3		25.1	-----	Sieve
25						
27.5	California Ring	5/6		31.1	-----	Cons

LEGEND

- Standard Penetration Test
- California Ring
- Rock Core
- Bulk Sample
- Shelby Tube
- Water Seepage
- Groundwater

SIEVE: GRAIN SIZE ANALYSIS
MAX: MAXIMUM DRY DENSITY
DS: DIRECT SHEAR
CONS: CONSOLIDATION
HYDR: HYDROMETER ANALYSIS
EXPAN: EXPANSION INDEX
CHEM: CHEMICAL TESTS

PLATE A-10

GeoSoils Consultants, Inc.
GEOTECHNICAL * GEOLOGIC * ENVIRONMENTAL

GEOTECHNICAL BORING LOG

PROJECT NAME WS Development
W.O. NO. 6792
DRILLING COMPANY Choice Drilling
DATE STARTED: 9-16-14
BORING NO. B-4
TYPE OF DRILL RIG Auto Hammer
LOGGED BY RC
SHEET 2 OF 3
DRILLING METHOD Hollow Stem
HAMMER WEIGHT (LBS) 140
GROUND ELEVATION (FT)
DIAMETER OF HOLE 8
DROP (IN) 30
GW ELEVATION
BORING LOCATION:

DEPTH (FT)	SAMPLE TYPE	BLOWS/ 6 IN.	GEOTECHNICAL DESCRIPTION	MOISTURE CONTENT (%)	DRY DENSITY (pcf)	OTHER TESTS
	Standard Penetration Test	3/5/6	@ 32.5', Brown, slightly sandy CLAY, moist, very stiff	26.1	-----	
35						
	California Ring	6/9	@ 37.5', Brown, slightly sandy CLAY, moist, stiff	28.9	95.4	Cons
40						
	Standard Penetration Test	4/5/7	@ 42.5', Orange brown, sandy, clayey SILT with some silty, very fine SAND, very moist, dense	29.6	-----	Sieve
45						
	California Ring	27/38		14.9	116.3	
50						
			51-57.5', BEDROCK			
	Standard Penetration Test	7/9/11	@ 52.5-62.5', Gray, silty CLAY, moist, dense	34.6	-----	Att
55						
	California Ring	19/25		50.1	74.4	Sieve

LEGEND

- Standard Penetration Test
- California Ring
- Rock Core
- Bulk Sample
- Shelby Tube
- Water Seepage
- Groundwater

SIEVE: GRAIN SIZE ANALYSIS
MAX: MAXIMUM DRY DENSITY
DS: DIRECT SHEAR
CONS: CONSOLIDATION
HYDR: HYDROMETER ANALYSIS
EXPAN: EXPANSION INDEX
CHEM: CHEMICAL TESTS

PLATE A-11

GeoSoils Consultants, Inc.
GEOTECHNICAL * GEOLOGIC * ENVIRONMENTAL

GEOTECHNICAL BORING LOG

PROJECT NAME: WS Development
W.O. NO.: 6792
DRILLING COMPANY: Choice Drilling
DATE STARTED: 9-16-14
BORING NO.: B-4
TYPE OF DRILL RIG: Auto Hammer
LOGGED BY: RC
SHEET 3 OF 3
DRILLING METHOD: Hollow Stem
HAMMER WEIGHT (LBS): 140
GROUND ELEVATION (FT):
DIAMETER OF HOLE: 8
DROP (IN): 30
GW ELEVATION:
BORING LOCATION:

DEPTH (FT)	SAMPLE TYPE	BLOWS/ 6 IN.	GEOTECHNICAL DESCRIPTION	MOISTURE CONTENT (%)	DRY DENSITY (pcf)	OTHER TESTS
	California Ring	18/22	TD @ 62.5' Groundwater @ 25' No caving	49.3	73.5	
65						
70						
75						
80						
85						

LEGEND

- Standard Penetration Test
- California Ring
- Rock Core
- Bulk Sample
- Shelby Tube
- Water Seepage
- Groundwater

SIEVE: GRAIN SIZE ANALYSIS
MAX: MAXIMUM DRY DENSITY
DS: DIRECT SHEAR
CONS: CONSOLIDATION
HYDR: HYDROMETER ANALYSIS
EXPAN: EXPANSION INDEX
CHEM: CHEMICAL TESTS

PLATE A-12

GeoSoils Consultants, Inc.
GEOTECHNICAL * GEOLOGIC * ENVIRONMENTAL

GEOTECHNICAL BORING LOG

PROJECT NAME: WS Development	W.O. NO.: 6792
DRILLING COMPANY: Choice Drilling	DATE STARTED: 9-16-14
BORING NO.: B-5	
TYPE OF DRILL RIG: Auto Hammer	LOGGED BY: RC
SHEET 1 OF 3	
DRILLING METHOD: Hollow Stem	HAMMER WEIGHT (LBS): 140
GROUND ELEVATION (FT):	
DIAMETER OF HOLE: 8	DROP (IN): 30
GW ELEVATION:	
BORING LOCATION:	

DEPTH (FT)	SAMPLE TYPE	BLOWS/ 6 IN.	GEOTECHNICAL DESCRIPTION	MOISTURE CONTENT (%)	DRY DENSITY (pcf)	OTHER TESTS
			0-5.5", AC; 5.5-11.5', FILL SAND			
	Bulk Sample		**11.5"-14', FILL**			
5	Standard Penetration Test	6/9/10	@ 5-10', Dark brown, sandy CLAY with rock fragments, moist, dense	5.2	-----	
10	California Ring	18/28		17.1	107.3	
			14-70', ALLUVIUM			
15	Standard Penetration Test	8/10/12	@ 15', Orange brown, slightly sandy, clayey SILT, moist, stiff	19.5	-----	
20	California Ring	12/18	@ 20', Orange brown, silty CLAY, very moist, medium stiff	26.4	95.4	Cons
25	Standard Penetration Test	4/5/6	@ 25', Orange brown, silty CLAY, very moist, stiff	29.9	-----	

LEGEND

- Standard Penetration Test
- California Ring
- Rock Core
- Bulk Sample
- Shelby Tube
- Water Seepage
- Groundwater

SIEVE: GRAIN SIZE ANALYSIS
MAX: MAXIMUM DRY DENSITY
DS: DIRECT SHEAR
CONS: CONSOLIDATION
HYDR: HYDROMETER ANALYSIS
EXPAN: EXPANSION INDEX
CHEM: CHEMICAL TESTS

PLATE A-13

GeoSoils Consultants, Inc.
GEOTECHNICAL * GEOLOGIC * ENVIRONMENTAL

GEOTECHNICAL BORING LOG

PROJECT NAME: WS Development
W.O. NO. 6792
DRILLING COMPANY: Choice Drilling
DATE STARTED: 9-16-14
BORING NO. B-5
TYPE OF DRILL RIG: Auto Hammer
LOGGED BY: RC
SHEET 2 OF 3
DRILLING METHOD: Hollow Stem
HAMMER WEIGHT (LBS): 140
GROUND ELEVATION (FT):
DIAMETER OF HOLE: 8
DROP (IN): 30
GW ELEVATION:
BORING LOCATION:

DEPTH (FT)	SAMPLE TYPE	BLOWS/ 6 IN.	GEOTECHNICAL DESCRIPTION	MOISTURE CONTENT (%)	DRY DENSITY (pcf)	OTHER TESTS
	California Ring	10/14	@ 30', Brown, silty CLAY, very moist, medium stiff	25.9	94.7	Cons
35	Standard Penetration Test	2/3/5	@ 35', Brown, silty CLAY, very moist, soft	28.3	-----	Sieve
40	California Ring	10/13	@ 40', Brown, silty, slightly sandy CLAY, very moist, medium stiff	31.4	90.3	Cons Att
45	Standard Penetration Test	6/8/9	@ 45', No recovery @ 45-50', Orange brown, slightly silty, very fine to medium SAND with gravel, very moist, dense	-----	-----	
50	California Ring	28/48		22.6	104.3	Sieve
55	Standard Penetration Test	15/18/26	@ 55', Gray, sandy, silty CLAY, moist, stiff	26.1	-----	

LEGEND

- Standard Penetration Test
- California Ring
- Rock Core
- Bulk Sample
- Shelby Tube
- Water Seepage
- Groundwater

SIEVE: GRAIN SIZE ANALYSIS
MAX: MAXIMUM DRY DENSITY
DS: DIRECT SHEAR
CONS: CONSOLIDATION
HYDR: HYDROMETER ANALYSIS
EXPAN: EXPANSION INDEX
CHEM: CHEMICAL TESTS

PLATE A-14

GeoSoils Consultants, Inc.
GEOTECHNICAL * GEOLOGIC * ENVIRONMENTAL

GEOTECHNICAL BORING LOG

PROJECT NAME WS Development
W.O. NO. 6792
DRILLING COMPANY Choice Drilling
DATE STARTED: 9-16-14
BORING NO. B-5
TYPE OF DRILL RIG Auto Hammer
LOGGED BY RC
SHEET 3 OF 3
DRILLING METHOD Hollow Stem
HAMMER WEIGHT (LBS) 140
GROUND ELEVATION (FT)
DIAMETER OF HOLE 8
DROP (IN) 30
GW ELEVATION
BORING LOCATION:

DEPTH (FT)	SAMPLE TYPE	BLOWS/ 6 IN.	GEOTECHNICAL DESCRIPTION	MOISTURE CONTENT (%)	DRY DENSITY (pcf)	OTHER TESTS
	California Ring	28/40	@ 60', Orange brown, slightly sandy, clayey SILT, moist, stiff	19.6	109.5	Sieve
65	Standard Penetration Test	14/14/25	@ 65', Orange brown, slightly sandy, silty clay, moist, stiff	32.5	-----	
70	California Ring	28/50	**70-75', BEDROCK** @ 70-75', Gray, silty CLAY, moist, very stiff	37.9	80.9	
75	California Ring	28/45	TD @ 75' Groundwater @ 24' No caving	42.2	75.9	
80						
85						

LEGEND

Standard Penetration Test
California Ring
Rock Core
Bulk Sample
Shelby Tube
Water Seepage
Groundwater

SIEVE: GRAIN SIZE ANALYSIS
MAX: MAXIMUM DRY DENSITY
DS: DIRECT SHEAR
CONS: CONSOLIDATION
HYDR: HYDROMETER ANALYSIS
EXPAN: EXPANSION INDEX
CHEM: CHEMICAL TESTS

PLATE A-15

GeoSoils Consultants, Inc.
GEOTECHNICAL * GEOLOGIC * ENVIRONMENTAL

BORING 1

Date Drilled: April 26, 2007
Equipment Used: Rotary Wash (5-inch diameter)

Depth to Water: Groundwater at 17.5 feet
Driving Weight & Drop: 300 lbs/30" drop
(140 lbs/30" drop for SPT)

SURFACE ELEVATION: 101.4 feet (Assumed datum; see Figure 2)

ELEVATION (feet)	DEPTH (feet)	MOISTURE (% of dry wt.)	DRY DENSITY (pcf)	N VALUE (SPT)	BLOW COUNT (blows / 12-inches)	SAMPLE LOCATION	DESCRIPTION
	0						4" Asphalt Paving; No Base
							FILL CL - SANDY CLAY - some angular gravel, mottled dark brown and light brown
	5	22.1	96		8	⊠	
	10	21.0	102		8	⊠	
	15	27.1	93		7	⊠	
							NATURAL ▽CL - LEAN CLAY - brown GROUNDWATER LEVEL
				6		⧄	[LL=40; PI=16]
	20	28.8	94		6	⊠	Few sandy interbeds
	25	29.6	91		3	⊠	
				6		⧄	[LL=33; PI=12]
	30	26.2	98		5	⊠	
							Some sand
	35	23.0	105		10	⊠	SM - SILTY SAND - fine, some clay, brown [44% Passing No. 200 Sieve]
				22		⧄	[45% Passing No. 200 Sieve]
	40						

(Continued on next page)

LOG OF BORING

The log of subsurface conditions shown hereon applies only at the specific boring location and at the date indicated. It is not warranted to be representative of subsurface conditions at other locations and times.

Job No: 07-018 By: BF Date: 4-27-2007 Checked: ✓ Printed: 5-30-07 [LOG FOR FIELD; 07-018.GPJ]

VAN BEVEREN & BUTELO, INC.

FIGURE A-1.1a

BORING 1
(Continued)

Date Drilled: April 26, 2007
Equipment Used: Rotary Wash (5-inch diameter)
Depth to Water: Groundwater at 17.5 feet
Driving Weight & Drop: 300 lbs/30" drop

SURFACE ELEVATION: 101.4 feet (Assumed datum; see Figure 1)

ELEVATION (feet)	DEPTH (feet)	MOISTURE (% of dry wt.)	DRY DENSITY (pcf)	N VALUE (SPT)	BLOW COUNT (blows / 12-inches)	SAMPLE LOCATION	DESCRIPTION
		24.3	103		6		ML-CL - SILTY CLAY - brown [LL=28; PI=6]
							SP-SM - SAND WITH SILT - fine to coarse, some gravel (up to ½" in size), brown
	45	18.9	107		30		
				73			
							BEDROCK CLAYSTONE - some angular to subrounded gravel, brown
	50	20.1	107		20		
	55	36.3	83		17		Some siltstone and sandstone interbeds, brownish gray
	60				17		
	65	41.0	79		15		
	70	55.4	66		15		
							END OF BORING AT 71 FEET.
	75						
	80						

Notes

1. Fill encountered to depth of 16½ feet.
2. Groundwater monitoring well (and well cover) installed consisting of 2-inch-diameter pipe perforated from a depth of 20 to 50 feet.
3. Groundwater measured at a depth of 17.5 feet on May 7, 2007.

The log of subsurface conditions shown hereon applies only at the specific boring location and at the date indicated. It is not warranted to be representative of subsurface conditions at other locations and times.

Job No: 07-018 By: BF Date: 4-27-2007 Checked: vl Printed: 5-30-07 [LOG FOR FIELD; 07-018.GPJ]

LOG OF BORING

VAN BEVEREN & BUTELO, INC.

FIGURE A-1.1b

BORING 2

Date Drilled: April 27, 2007
Equipment Used: Rotary Wash (5-inch diameter)

Depth to Water: Groundwater at 25.4 feet
Driving Weight & Drop: 300 lbs/30" drop
(140 lbs/30" drop for SPT)

SURFACE ELEVATION: 99.6 feet

ELEVATION (feet)	DEPTH (feet)	MOISTURE (% of dry wt.)	DRY DENSITY (pcf)	N VALUE (SPT)	BLOW COUNT (blows / 12-inches)	SAMPLE LOCATION	Description
	0						4" Asphalt Paving; No Base
							FILL SM - SILTY SAND - fine, some angular gravel (up to 3/4" in size), trace clay, mottled dark brown and light brown
	5	20.2	91		8		
	10	24.5	98		8		Slightly porous
	15	21.9	97		6		
							NATURAL CL - LEAN CLAY - dark brown [LL=37; PI=16]
				10			
	20	24.3	99		5		SM - SILTY SAND - fine, brown [36% Passing No. 200 Sieve]
							CL - LEAN CLAY - dark brown
	25						∇ GROUNDWATER LEVEL
		26.9	98		2		
				1			[LL=29; PI=11]
							SM - SILTY SAND - some angular gravel, brown
	30	21.9	97		5		[48% Passing No. 200 Sieve]
							CL - LEAN CLAY - dark brown
	35	30.4	90		5		
				3			[LL=35; PI=16]
	40						

(Continued on next page)

LOG OF BORING

VAN BEVEREN & BUTELO, INC.

FIGURE A-1.2a

Job No: 07-018 By: BF Date: 4-27-2007 Checked: [signature] Printed: 5-30-07 [LOG FOR FIELD: 07-018.GPJ]

The log of subsurface conditions shown hereon applies only at the specific boring location and at the date indicated. It is not warranted to be representative of subsurface conditions at other locations and times.

BORING 2
(Continued)

Date Drilled: April 27, 2007
Equipment Used: Rotary Wash (5-inch diameter)
Depth to Water: Groundwater at 25.4 feet
Driving Weight & Drop: 300 lbs/30" drop

SURFACE ELEVATION: 99.6 feet

ELEVATION (feet)	DEPTH (feet)	MOISTURE (% of dry wt.)	DRY DENSITY (pcf)	N VALUE (SPT)	BLOW COUNT (blows / 12-inches)	SAMPLE LOCATION	DESCRIPTION
		40.0	83		3	⊠	
	45	31.6	89		3	⊠	Layer of clayey sand [42% Passing No. 200 Sieve]
				14		◻	Some gravel [LL=29; PI=8]
	50						**BEDROCK** INTERBEDDED SANDSTONE AND SILTSTONE - dark gray
		43.5	75		18	⊠	**END OF BORING AT 51 FEET.**
	55						
	60						
	65						
	70						
	75						
	80						

Notes

1. Fill encountered to depth of 16½ feet.
2. Groundwater monitoring well (and well cover) installed consisting of 2-inch-diameter pipe perforated from a depth of 20 to 50 feet.
3. Groundwater measured at a depth of 25.4 feet on May 7, 2007.

The log of subsurface conditions shown hereon applies only at the specific boring location and at the date indicated. It is not warranted to be representative of subsurface conditions at other locations and times.

Job No: 07-018 By: BF Date: 4-27-2007 Checked: JL Printed: 5-30-07 [LOG FOR FIELD; 07-018.GPJ]

LOG OF BORING

VAN BEVEREN & BUTELO, INC.

FIGURE A-1.2b

APPENDIX D

APPENDIX D

APGD PILE SPECIFICATIONS

Piling Equipment

The piling equipment used for the project shall conform to the specifications below.

Piling Rig – The contractor shall use equipment of adequate torque, crowd force, and power, to achieve the design tip elevation. As a minimum, the piling rig shall be capable of providing a minimum torque of 150,000 ft-lbs, and 25 tons of down crowd thrust.

Automated Monitoring Equipment – The drilling rig shall be equipped with an automated monitoring equipment (AME) designed to monitor the pile installation process. During the drilling process, the AME shall record auger depth, drill torque, and elapsed time. During the grouting process, the AME shall record the auger depth, grout pressure, and elapsed time.

Augers – The augers shall be capable of creating a minimum 18-inch diameter pile.

Grouting Equipment – A grout port shall be located near the tip of the displacement auger. A continuous system of grout mixing, pumping, and agitating equipment shall be utilized. Equipment shall be maintained in good working order to maintain a continuous flow of concrete during auger withdrawal. The grout pump shall be capable of developing displacement pressures of 250-psi.

Pile Installation Procedures

The following installation procedures may be followed to install the APGD piles.

1. Contractor is responsible for using equipment of adequate torque, crowd, and power to achieve the design tip elevation. The piling rig and the flight augers used for the production pile installation shall be of identical design to that used for the indicator pile test program.
2. The flight auger is advanced until it reaches the design tip elevation. The grout port in the auger tool shall be closed with a plug that prevents soil and/or water from entering the hollow shaft while the auger is advanced into the ground.
3. The flight auger shall be capable of creating a smooth walled shaft with a minimum of 18 inches in diameter (both test piles and production piles shall be a minimum of 18 inches in diameter).
4. A minimum delivery pressure of 250 psi plus the hydraulic pressure developed by the grout column in the drill stem shall be applied to create the pile. The operator shall maintain positive rotation of the displacement auger continuously throughout the grouting process until the displacement element is completely retracted from the ground.

5. The piling rig shall be equipped with automated monitoring equipment (AME) to record the auger depth, drill torque, grout pressure, and elapsed time. All recorded data shall be provided for review.
6. Once the grouted pile shaft is filled with concrete, the steel reinforcing cage shall be inserted into the wet concrete pile. All reinforcing elements shall be fitted with centralizers or clip spacers.

Indicator Pile Test Program

An indicator pile test program must be performed and approved by the City of Los Angeles prior to installation of the production piles. The number of indicator test piles shall be a minimum of 2 test piles, or equivalent to 1 percent of the total number of production piles, whichever is greater. Pile load tests shall be performed from the proposed subgrade elevation.

Compression load tests will be performed on all indicator test piles. Axial compressive load test shall be performed in accordance with ASTM D1143. The test piles and reaction piles shall be considered sacrificial and shall not be utilized for foundation support of the proposed buildings. The allowable pile capacities and pile lengths presented herein are subject to be confirmed, or altered depending on the results of the indicator pile load test program. Additional foundation piles may be necessary if the actual load tests do not meet the recommended allowable loads presented in this report.

Below is a summary of the indicator pile load test program.

- The number of indicator test piles shall be a minimum of 2 test piles, or equivalent to 1 percent of the total number of production piles, whichever is greater.
- Load tests shall be performed on sacrificial test piles in accordance with ASTM D1143 (Axial Compressive Load). The design load shall be held until the measured creep does not exceed 0.005 inch per hour. Piles with a settlement rate exceeding 0.005 inch/hour under the design load during a pile test will be rejected.
- Pile load tests shall be performed to a minimum load equivalent to the ultimate capacity, which is two times the allowable capacity.
- Test piles and reaction piles shall be sacrificial and shall not be incorporated as foundation piles. Sacrificial test piles and reaction piles shall be cut off 3 feet below the finished grade and abandoned in place following the completion of the testing program.
- Gamma-Gamma density logging (GDL) and Low Strain Pile Integrity Tests (PIT) shall be performed on all test piles and reaction piles. GDL shall be performed in accordance with Caltrans CT 233. PIT shall be performed in accordance with ASTM D5882.
- One test pile shall be exhumed from the ground to physically examine the pile integrity.
- Results of the pile load testing will be submitted as a summary letter to the LADBS Grading Division for review and approval.

Geotechnical Pile Inspections

During pile installation, a City of Los Angeles Deputy Grading Inspector shall record and maintain data for each pile, including the following:

- Pile Number
- Installed pile length
- Auger torque vs. depth
- Head pressure inside the tremie pipe vs. depth
- Drilling rate vs. depth
- Concrete volume vs. depth
- Unanticipated site conditions if any

Non-Destructive Testing

None-destructive testing methods shall be employed to evaluate the integrity of the piles installed to provide quality control and assurance of the pile construction method.

- Gamma-Gamma density logging (GDL) and Low Strain Pile Integrity Tests (PIT) shall be performed on all test piles and reaction piles. GDL shall be performed in accordance with Caltrans CT 233. PIT shall be performed in accordance with ASTM D5882.

- Low Strain Pile Integrity Tests (PIT) shall be performed on 10 percent of the production piles.

- If any PIT test indicates a discontinuity within a tested pile, that pile shall be evaluated by the geotechnical and structural engineers. Unsatisfactory piles may be abandoned in place and shall be replaced with replacement piles.