APPENDIX C

Geotechnical Reports

GEOTECHNICAL INVESTIGATION

PROPOSED COMMERCIAL DEVELOPMENT
800 & 908 NORTH MAIN STREET
1081 & 1087 NORTH VIGNES STREET
LOS ANGELES, CALIFORNIA

TRACT: CITY LANDS OF LOS ANGELES TRACT, LOT: PT "UNNUMBERED LT", ARB: 352-355 & 402
AND
TRACT: OIL WELL SUPPLY COMPANY TRACT, LOT: FR LT A, ARB: 1 & 2

GEOCON
WEST, INC.
GEOTECHNICAL
ENVIRONMENTAL
MATERIALS

PREPARED FOR

LINC HOUSING CORPORATION
LONG BEACH, CALIFORNIA

PROJECT NO. W1815-06-01
OCTOBER 24, 2023

Project No. W1815-06-01
October 24, 2023

Ms. Cecilia Ngo
LINC Housing
3590 Elm Avenue
Long Beach, CA 90807

Subject: GEOTECHNICAL INVESTIGATION
PROPOSED COMMERCIAL DEVELOPMENT
800 & 908 NORTH MAIN STREET
1081 & 1087 NORTH VIGNES STREET
LOS ANGELES, CALIFORNIA
TRACT: CITY LANS OF LOS ANGELES, LOT: PT "UNNUMBERED LOT",
ARB: 352-355 & 402 AND TRACT: OIL WELL SUPPLY COMPANY,
LOT:FR LT A, ARB: 1&2

Dear Ms. Ngo:

In accordance with your authorization of our proposal dated August 3, 20223, we have performed a geotechnical investigation for the proposed commercial development located at 800 and 908 North Main Street and 1081 and 1087 North Vignes Street in 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 the 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 in 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.

REGISTERED PROFESSIONAL ENGINEER
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GEOTECHNICAL INVESTIGATION

1. PURPOSE AND SCOPE

This report presents the results of a geotechnical investigation for the proposed commercial development located at 800 and 908 North Main Street and 1081 and 1087 North Vignes Street 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 design and construction.

The scope of this investigation included a site reconnaissance, field exploration, laboratory testing, engineering analysis, and the preparation of this report. The site was explored on September 6 and 7, 2023, by excavating two 7-inch diameter borings to between depths of approximately 55½ feet and 66 feet below the ground surface utilizing a truck-mounted hollow-stem auger drilling machine. Two additional 7-inch diameter borings were drilled at the adjacent site on September 7 and 8 to between depths of approximately 55½ feet and 81 feet below the ground surface utilizing a truck-mounted hollow-stem auger drilling machine. The approximate locations of the exploratory borings on both sites are depicted on the Site Plan (see Figure 2). A detailed discussion of the field investigation, including the boring logs, is presented in Appendix A.

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

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

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

2. SITE AND PROJECT DESCRIPTION

The subject site is located at 800 and 908 North Main Street and 1081 and 1087 North Vignes Street in the City of Los Angeles, California. The site is currently occupied by an asphalt paved parking lot that occupies the majority of the site with the exception of the western corner of the site, which is vacant. The ground surface in this portion of the site is covered with angular gravel and sparse vegetation. The vacant portion of the site appears to be the footprint of a former building because portions of the building's foundation remain at the site. The site is bounded by North Vignes Street to the northeast, by Rosabell Street to the southeast, by a parking lot to the southwest, and by North Main Street to the northwest. The paved portion of the site is relatively level, with no pronounced highs or lows. The gravel covered portion of the site is lower than the paved portion with a grade elevation difference of approximately 1-1½ feet. There is also a difference if the existing grade elevations along the southwest portion of the site's perimeter of approximately 1 to 3 feet. Surface water drainage at the site appears to be by sheet flow along the existing ground contours to the city streets.

Based on the information provided by the Client, it is our understanding that the design of the proposed commercial development has not been finalized and that two design options are under consideration. Option A will consist of two commercial structures, a four-story structure, and a two-story structure. The entire site will be underlain by one subterranean parking level. The eastern portion of the site will also be improved with a new structure, but it is not a part of this phase of the development. Option B will also consist of two commercial structures of the same height. However, in the Option B design, two subterranean parking levels are proposed. The first parking level (P1) will underlie the entire site, but the second, lower, parking level (P2) will have a smaller area and will be located beneath the southeastern portion of the site. (see Site Plan, Figures 2A and 2B). It is anticipated that excavations for the proposed structure with one subterranean parking level will extend to depths of approximately 17 feet below the existing ground surface, and 34 feet below the existing ground surface for two subterranean parking levels, including foundation depths and dewatering system.

Based on the preliminary nature of the design at this time, wall and column loads were not available. It is anticipated that the maximum column loads for the proposed structure will be up to 1000 kips, and the maximum wall loads will be up to 10 kips per linear foot.

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

3. GEOLOGIC SETTING

The site is located in the north-central portion of the Los Angeles Basin, a coastal plain bounded by the Santa Monica Mountains on the north, the Elysian Hills and Repetto Hills on the northeast, the Puente Hills and Whittier Fault on the east, the Palos Verdes Peninsula and Pacific Ocean on the west and south, and the Santa Ana Mountains and San Joaquin Hills on the southeast. The basin is underlain by a deep structural depression which has been filled by both marine and continental sedimentary deposits underlain by a basement complex of igneous and metamorphic composition (Yerkes, et al., 1965). Regionally, the site is located within the northern portion of the Peninsular Ranges geomorphic province. This geomorphic province is characterized by northwest-trending physiographic and geologic features such as the Newport-Inglewood Fault Zone located approximately 8.1 miles to the west.

4. SOIL AND GEOLOGIC CONDITIONS

Based on our field investigation and published geologic maps of the area, the site is underlain by artificial fill and Holocene age alluvial deposits consisting of sand and silt with varying amounts of gravel and cobbles (Dibblee, 1991; California Geological Survey, 2012). Detailed stratigraphic profiles of the materials encountered at the site are provided on the boring log in Appendix A.

4.1 Artificial Fill

Artificial fill was encountered in our field explorations to a maximum depth of 2½ feet below existing ground surface. The artificial fill generally consists of dark brown to olive gray or black sand and silt. The artificial fill is characterized as moist and soft or loose to medium dense. The fill is likely the result of past grading or construction activities at the site. Deeper fill may exist between excavations and in other portions of the site that were not directly explored.

4.2 Alluvium

Holocene age alluvial deposits were encountered beneath the fill. The alluvium consists primarily of brown to olive brown to gray interbedded sand and silt, with localized pockets of gravel and cobbles. The alluvium is characterized as moist to wet and loose medium dense to very dense or stiff to hard.

5. GROUNDWATER

A review of the Seismic Hazard Evaluation of the Los Angeles 7.5-Minute Quadrangle (California Division of Mines and Geology [CDMG], 1998), indicates that the historically highest groundwater level in the area is approximately 20 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 borings at depths of approximately 23 to 24 feet below existing ground surface. Based on the depth to groundwater encountered in our boring, and the depth of proposed construction, groundwater may be encountered during construction, based on the deeper proposed site layout. Additionally, it is not uncommon for groundwater levels to vary seasonally or for groundwater seepage conditions to develop where none previously existed, especially in impermeable fine-grained soils or on top of the bedrock, 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 the future performance of the project. Recommendations for drainage are provided in the S*urface Drainage* section of this report (see Section 7.26).

6. GEOLOGIC HAZARDS

6.1 Surface Fault Rupture

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

The site is not within a state-designated Alquist-Priolo Earthquake Fault Zone (CGS, 2021b; CGS, 2017) 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 Holocene-active fault to the site is the Hollywood Fault located approximately 3.9 miles to the north (CGS, 2017). Other nearby Holocene-active faults are the Verdugo Fault, the Newport-Inglewood Fault Zone, the Santa Monica Fault, and the Elsinore Fault located approximately 5.6 miles north, 8.1 miles west, 10½ miles west, and 14½ miles east of the site, respectively. (USGS, 2006; Ziony and Jones, 1989). The active San Andreas Fault Zone is located approximately 34 miles northeast of the site (Ziony and Jones, 1989).

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

6.2 Seismicity

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

LIST OF HISTORIC EARTHQUAKES

Earthquake (Oldest to Youngest)	Date of Earthquake	Magnitude	Distance to Epicenter (Miles)	Direction to Epicenter
Long Beach	March 10, 1933	6.4	34	SE
Tehachapi	July 21, 1952	7.5	79	NW
San Fernando	February 9, 1971	6.6	26	NNW
Whittier Narrows	October 1, 1987	5.9	9	E
Sierra Madre	June 28, 1991	5.8	19	NE
Landers	June 28, 1992	7.3	103	E
Big Bear	June 28, 1992	6.4	81	E
Northridge	January 17, 1994	6.7	20	WNW
Hector Mine	October 16, 1999	7.1	118	ENE
Ridgecrest	July 5, 2022	7.1	123	NNE

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

6.3 Seismic Design Criteria

The following table summarizes summarizes site-specific design criteria obtained from the 2022 California Building Code (CBC; Based on the 2018 International Building Code [IBC] and ASCE 7-16), Chapter 16 Structural Design, Section 1613 Earthquake Loads. The data was calculated using the online application *Seismic Design Maps*, provided by OSHPD. The short spectral response uses a period of 0.2 second. We evaluated the Site Class based on the discussion in Section 1613.2.2 of the 2022 CBC and Table 20.3-1 of ASCE 7-16. The values presented on the following page are for the risk-targeted maximum considered earthquake (MCE_R).

2022 CBC SEISMIC DESIGN PARAMETERS

Parameter	Value	2022 CBC Reference
Site Class	D	Section 1613.2.2
MCE_R Ground Motion Spectral Response Acceleration – Class B (short), S_S	1.995g	Figure 1613.2.1(1)
MCE_R Ground Motion Spectral Response Acceleration – Class B (1 sec), S_1	0.712g	Figure 1613.2.1(2)
Site Coefficient, F_A	1	Table 1613.2.3(1)
Site Coefficient, F_V	1.7*	Table 1613.2.3(2)
Site Class Modified MCE_R Spectral Response Acceleration (short), S_{MS}	1.995g	Section 1613.2.3 (Eqn 16-36)
Site Class Modified MCE_R Spectral Response Acceleration – (1 sec), S_{M1}	1.211g*	Section 1613.2.3 (Eqn 16-37)
5% Damped Design Spectral Response Acceleration (short), S_{DS}	1.33g	Section 1613.2.4 (Eqn 16-38)
5% Damped Design Spectral Response Acceleration (1 sec), S_{D1}	0.807g*	Section 1613.2.4 (Eqn 16-39)
Note: *Per Section 11.4.8 of ASCE/SEI 7-16, a ground motion hazard analysis shall be performed for projects for Site Class "E" sites with Ss greater than or equal to 1.0g and for Site Class "D" and "E" sites with S1 greater than 0.2g. Section 11.4.8 also provides exceptions which indicates that the ground motion hazard analysis may be waived provided the exceptions are followed. Using the code based values presented in the table above, in lieu of a performing a ground motion hazard analysis, requires the exceptions outlined in ASCE 7-16 Section 11.4.8 be followed.		

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

ASCE 7-16 PEAK GROUND ACCELERATION

Parameter	Value	ASCE 7-16 Reference
Mapped MCE_G Peak Ground Acceleration, PGA	0.858g	Figure 22-7
Site Coefficient, F_{PGA}	1.1	Table 11.8-1
Site Class Modified MCE_G Peak Ground Acceleration, PGA_M	0.943g	Section 11.8.3 (Eqn 11.8-1)

Deaggregation of the MCE peak ground acceleration was performed using the USGS online Unified Hazard Tool, 2014 Conterminous U.S. Dynamic edition (v4.2.0). The result of the deaggregation analysis indicates that the predominant earthquake contributing to the MCE peak ground acceleration is characterized as a 6.85 magnitude event occurring at a hypocentral distance of 9.08 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 6.74 magnitude occurring at a hypocentral distance of 12.79 kilometers from the site.

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

6.4 Liquefaction Potential

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

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

The State of California Seismic Hazard Zone Map for the Los Angeles Quadrangle (CDMG, 1999; CGS, 2014) indicates that the site is located within an area identified as having a potential for liquefaction. Also, according to the Los Angeles County Safety Element (Leighton, 1990), 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. In order to supplement the SPT blow count data, California Modified Sampler blow count data were converted to equivalent SPT blow counts based on a correlation factor of 0.55 (Rogers, 2006).

Screening criteria developed by Bray and Sancio (2006) characterize fine-grained soils which are not susceptible to liquefaction as soils with a plasticity index (PI) that is greater than 18 or with a saturated moisture content that is less than 80 percent of the liquid limit. In order to apply the screening criteria, laboratory testing was performed to evaluate the Atterberg Limits of select soil samples. Laboratory test results used for the screening criteria are presented as Figure B26.

The liquefaction analysis for a structure with one subterranean level, extending two a depth of 15 feet below the ground surface, or for a structure with two subterranean levels, extending to a depth of 30 feet below the ground surface, was performed for a Design Earthquake level by using a historic high groundwater table of 20 feet below the ground surface, a magnitude 6.74 earthquake, and a peak horizontal acceleration of 0.629g (⅔PGA_M). The enclosed liquefaction analysis for a structure with one or two subterranean levels, included herein for borings B1 and B4, indicate that the alluvial soils below the historic high groundwater level could be susceptible to up to approximately 0.7 inch of total settlement during Design Earthquake ground motion (see enclosed calculation sheets, Figures 5 through 8).

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 for a structure with one subterranean level, extending two a depth of 15 feet below the ground surface, or for a structure with two subterranean levels, extending to a depth of 30 feet below the ground surface, was also performed for the Maximum Considered Earthquake level by using a historic high groundwater table of 20 feet below the ground surface, a magnitude 6.85 earthquake, and a peak horizontal acceleration of 0.943g (PGA_M). The enclosed liquefaction analysis for a structure with one or two subterranean levels, included herein for borings B1 and B4, indicate that the alluvial soils below the historic high groundwater level could be susceptible to up to approximately 0.7 inch of total settlement during Maximum Considered ground motion (see enclosed calculation sheets, Figures 9 through 12).

6.5 Seismically Induced Dry Settlement

Dynamic compaction of dry and loose sands may occur during a major earthquake. Typically, settlements occur in thick beds of such soils. The seismically-induced settlement calculations were performed in accordance with the American Society of Civil Engineers, Technical Engineering and Design Guides as adapted from the US Army Corps of Engineers, No. 9.

The calculation for a structure with one subterranean level that will extend to a depth of approximately 15 feet below the ground surface. The calculations provided herein for borings B1 and B4, indicate that the soil above the historic high groundwater level of 20 feet could be susceptible to approximately 0.02 inch of settlement as a result of the Design Earthquake peak ground acceleration ($⅔PGA_M$), and is considered negligible. The calculations provided herein for borings B1 and B4, indicate that the soil above the historic high groundwater level of 20 feet could be susceptible to approximately 0.03 inch of settlement as a result of the Maximum Considered Earthquake peak ground acceleration (PGA_M), and is considered negligible.

Dry seismically-induced settlement calculations for a structure with two subterranean levels were not performed because the subterranean excavation will extend to a depth of approximately 30 feet which is below the existing ground water level at the site and the saturated soils would not be prone to seismically-induced dry settlement as a result of the Design Earthquake peak ground acceleration ($⅔PGA_M$) nor as a result of the Maximum Considered Earthquake peak ground acceleration (PGA_M).

6.6 Slope Stability

The topography at the site is relatively level and the topography in the vicinity of the site slopes gently to the west. The site is not located within a City of Los Angeles Hillside Grading Area or a Hillside Ordinance Area (City of Los Angeles, 2022). Additionally, the site is not located within an area identified as having a potential for seismic slope instability (CDMG, 1999; CGS, 2014). There are no known landslides near the site, nor is the site in the path of any known or potential landslides. Therefore, the potential for slope stability hazards to adversely affect the proposed development is considered low.

6.7 Earthquake-Induced Flooding

Earthquake-induced flooding is inundation caused by failure of dams or other water-retaining structures due to earthquakes. Based on a review of the County of Los Angeles Safety Element (Leighton, 1990), the site is located within the Mulholland Dam and Hansen 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.

6.8 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 seismically induced seiche is considered unlikely.

The site is within an area of minimal flooding (Zone X) as defined by the Federal Emergency Management Agency (LACDPW, 2023; FEMA, 2023).

6.9 Oil Fields & Methane Potential

Based on a review of the California Geologic Energy Management Division (CalGEM) Well Finder Website, the site is not located within an oil field and oil or gas wells are not documented in the immediate site vicinity (CalGEM, 2023). However, due to the voluntary nature of record reporting by the oil well drilling companies, wells may be improperly located or not documented on the location map 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 located within the boundaries of a city-designated Methane Buffer Zone (City of Los Angeles, 2023). Should it be determined that a methane study is required for the proposed development, it is recommended that a qualified methane consultant be retained to perform the study and provide mitigation measures as necessary.

6.10 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.

7. CONCLUSIONS AND RECOMMENDATIONS

7.1 General

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

7.1.2 Up to 9 feet of existing artificial fill was encountered during the site investigation. Deeper fill may exist in other areas of the site that were not directly explored. The existing fill encountered is believed to be the result of past grading and construction activities at the site. It is our opinion that the existing fill, in its present condition, is not suitable for direct support of proposed foundations or slabs. The existing fill and site soils are suitable for re-use as engineered fill provided the recommendations in the *Grading* section of this report are followed (see Section 7.5). Excavation for the subterranean level(s) is anticipated to penetrate through the existing artificial fill and expose undisturbed alluvial soils throughout the excavation bottom.

7.1.3 The enclosed seismic settlement analyses indicate that the site soils could be susceptible to up to approximately 0.7 inch of total settlement as a result of a Design Earthquake peak ground acceleration ($\frac{2}{3}PGA_M$). Differential settlement at the foundation level is anticipated to be approximately 0.35 inch over a distance of 20 feet.

7.1.4 Static groundwater was encountered during site exploration at depths of approximately 23 to 24 feet below existing ground surface. Historic high groundwater at the site is approximately 20 feet below the ground surface. Excavation is anticipated to extend to a maximum depth of approximately 17 feet below the ground surface for construction of one subterranean level option, or approximately 34 feet below the ground surface for construction of the two subterranean levels option, including foundation and dewatering system excavations. Based on the conditions encountered at the time of exploration, groundwater is anticipated to be encountered during construction for a structure with two-subterranean levels that extends to a depth of 34 feet below ground surface, including foundation excavation and dewatering system. For a proposed structure with one subterranean level that is 17 feet in depth, the current static groundwater table is sufficiently deep that it not expected to be encountered during construction with the exception of a deep drilled excavation such as for a shoring pile or elevator piston. However, local seepage could be encountered during excavation of the subterranean level, especially if conducted during the rainy season.

7.1.5 The City of Los Angeles Building Code requires that the structure be designed for the historically high groundwater level, which is approximately 20 feet below the existing ground surface. The proposed structure must be designed for hydrostatic pressure for any portion of the structure below a depth of 20 feet. 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. If the proposed structure does not provide sufficient dead load to resist the buoyant forces then uplift mitigation will be required.

7.1.6 Based on these considerations, it is recommended that the proposed structure be supported on a mat foundation system deriving support in competent alluvial soils found at and below a depth of 15 feet below the existing ground surface. In order to minimize differential settlement between the ramp, ramp walls, and basement level, it is recommended that the ramp and ramp walls for the subterranean parking garage be structurally supported on the mat foundation. In addition, the transition area between the one-subterranean level portion to the two-subterranean level portion (Option B) of the structure should be more heavily reinforced to resist differential settlement stresses which could cause cracking. All foundation excavations must be observed and approved by the Geotechnical Engineer (a representative of Geocon), prior to placing steel or concrete. Recommendations for the design of a mat foundation system are provided in Sections 7.7 and 7.8.

7.1.7 Where new foundations are constructed immediately adjacent to existing foundations, the new foundation should be deepened to match the depth of the existing foundation to prevent a surcharge on the existing foundation.

7.1.8 Where proposed foundations will be deeper than an existing foundation, the new foundation must be designed to resist the surcharge imposed by the existing foundation. The surcharge area may be defined by a 1:1 projection down and away from the bottom of the existing foundation.

7.1.9 It should be noted that implementation of the recommendations presented herein is not intended to completely prevent damage to the structure during the occurrence of strong ground shaking as a result of nearby earthquakes. It is intended that the structure be designed in such a way that the amount of damage incurred as a result of strong ground shaking be minimized.

7.1.10 Excavations up to 17 feet in vertical height are anticipated for construction of a structure with one subterranean level or up to 34 feet in vertical height for construction of a structure with two subterranean levels, including foundation depths and dewatering system. Due to the depth of the excavation and the proximity to the property lines, city streets and adjacent offsite structures and improvements, excavation of the proposed subterranean levels will require sloping and/or shoring in order to provide a stable excavation. Where shoring is required, it is recommended that a soldier pile shoring system be utilized. In addition, where the proposed excavation will be deeper than and adjacent to an offsite structure, the proposed shoring should be designed to resist the surcharge imposed by the adjacent offsite structure. Recommendations for shoring are provided in Section 7.21 of this report.

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

7.1.12 Foundations for small outlying structures, such as block walls up to 6 feet high, planter walls or trash enclosures, which will not be tied to the proposed structure, may be supported on conventional foundations bearing on a minimum of 12 inches of newly placed engineered fill which extends laterally at least 12 inches beyond the foundation area. Where excavation and proper compaction cannot be performed, foundations may derive support directly in the undisturbed alluvial soils at and below a depth of 24 inches below the existing ground surface and should be deepened as necessary to maintain a minimum 12-inch embedment into the 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 in writing by a Geocon representative.

7.1.13 Where new paving is to be placed, it is recommended that all existing fill soils and soft alluvial soils be excavated and properly compacted for paving support. The client should be aware that excavation and compaction of all existing fill in the area of new paving is not required; however, paving constructed over existing uncertified fill or unsuitable soils may experience increased settlement and/or cracking and may therefore have a shorter design life and increased maintenance costs. As a minimum, the upper 12 inches of soil should be scarified and properly compacted. Paving recommendations are provided in the *Preliminary Pavement Recommendations* section of this report (see Section 7.14).

7.1.14 Based on the historic and current groundwater levels as well as the potential for liquefaction of the site soils, stormwater infiltration is not recommended for this project. It is suggested that stormwater be retained, filtered and discharged in accordance with the requirements of the local governing agency.

7.1.15 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.

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

7.2 Soil and Excavation Characteristics

7.2.1 The in-situ soils can be excavated with moderate effort using conventional excavation equipment. Some caving should be anticipated in unshored excavations, especially where granular soils are encountered. The contractor should be aware that casing will be required during shoring pile installation.

7.2.2 Please be aware that the use of hollow-stem auger drilling equipment utilized for this investigation does not allow for the identification of the size of rock or abundance of rock being encountered or the visual observation of caving conditions since the drilling method is a small diameter cased excavation. It is recommended that the contractors bidding on excavation and shoring installation for this project perform their own excavations and test borings with the intended earthwork and drilling equipment to verify the presence, abundance, and size of buried rock (cobbles and boulders), potential for caving, as well as the suitability of the proposed excavation and drilling equipment for the safe and efficient earthwork operations and installation of the shoring system.

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

7.2.4 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 or shoring. Excavation recommendations are provided in the *Temporary Excavations* section of this report (see Section 7.20).

7.2.5 The existing site soils encountered at proposed foundation level during this investigation are considered to have a "low" expansive potential (EI = 1); and the soils are classified as "non-expansive" based on the 2022 California Building Code (CBC) Section 1803.5.3. Recommendations presented herein assume that the foundations and slabs will derive support in these materials.

7.3 Minimum Resistivity, pH and Water-Soluble Sulfate

7.3.1 Potential of Hydrogen (pH) and resistivity testing as well as chloride content testing were performed on representative samples of soil to generally evaluate the corrosion potential to surface utilities. The tests were performed in accordance with California Test Method Nos. 643 and 422 and indicate that the soils are considered "moderately" to "severely corrosive" with respect to corrosion of buried ferrous metals on site. The results are presented in Appendix B (Figure B28) and should be considered for design of underground structures. Due to the corrosive potential of the soils, it is recommended that PVC, ABS or other approved plastic piping be utilized in lieu of cast-iron when in direct contact with the site soils.

7.3.2 Laboratory tests were performed on representative samples of the site soils to measure the percentage of water-soluble sulfate content. Results from the laboratory water-soluble sulfate tests are presented in Appendix B (Figure B28) and indicate that the on-site materials possess a sulfate exposure class of "S0" to concrete structures as defined by 2022 CBC Section 1904 and ACI 318-14 Table 19.3.1.1.

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

7.4 Temporary Dewatering

7.4.1 Groundwater was encountered at a depth of approximately 23 to 24 feet below ground surface during site exploration. Based on the conditions encountered at the time of exploration, groundwater is anticipated to be encountered during construction for a structure with two-subterranean levels that extends to a depth of 34 feet below ground surface, including foundation excavation and dewatering system. For a proposed structure with one subterranean level that is 17 feet in depth, the current static groundwater table is sufficiently deep that it not expected to be encountered during construction with the exception of a deep drilled excavation such as for a shoring pile or elevator piston. However, local seepage could be encountered during excavation of the subterranean level, especially if conducted during the rainy season. The depth to groundwater at the time of construction can be further verified during initial dewatering well or shoring pile installations. If groundwater is present above the depth of the subterranean level(s), temporary dewatering will be necessary to maintain a safe working environment during excavation and construction activities.

7.4.2 If dewatering is required, it is recommended the project engage the services of a competent dewatering consultant to develop a dewatering system, calculate the design flow rates required for dewatering, and acquire the NPDES permit for water discharge. Initiating the permit application process well in advance of construction is recommended, as the California State Water Resources Control Board requires adequate time to review and authorize permits. Temporary dewatering typically consists 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 will be determined by qualified dewatering consultant.

7.4.3 Based on prior experiences with the City of Los Angeles Department of Building and Safety, Grading Division, additional engineering analyses be required to evaluate the potential impacts the proposed dewatering at the subject site will have on the adjacent structures and public streets. The additional analyses will determine the anticipated dewatering drawdown curve and resulting settlements that may occur due to the dewatering. If required, the drawdown and settlement analysis will be provided under separate cover.

7.4.4 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 functional on a permanent basis, it must be lined with filter fabric to prevent soil migration into the gravel.

7.5 Grading

7.5.1 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, if applicable, building official in attendance. Special soil handling requirements can be discussed at that time.

7.5.2 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.

7.5.3 Please be aware that the use of hollow-stem auger drilling equipment utilized for this investigation does not allow for the identification of the size of rock or abundance of rock being encountered or the visual observation of caving conditions since the drilling method is a small diameter cased excavation. It is recommended that the contractors bidding on excavation and shoring installation for this project perform their own excavations and test borings with the intended earthwork and drilling equipment to verify the presence, abundance, and size of buried rock (cobbles and boulders), potential for caving, as well as the suitability of the proposed excavation and drilling equipment for the safe and efficient earthwork operations and installation of the shoring system.

7.5.4 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. In accordance with City policy, asphalt and concrete should not be mixed into the structural fill. 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 the City of Los Angeles Inspector.

7.5.5 If subgrade stabilization is required at the excavation bottom, tire equipment should not be allowed in the excavation bottom until it is stabilized or extensive soil disturbance could result. In addition, the use of track equipment should be considered to minimize disturbance to the soils if they become wet at the excavation bottom. Bottom stabilization, if necessary, may be achieved placing 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.).

7.5.6 The City of Los Angeles Department of Building and Safety requires a minimum compactive effort of 95 percent of the laboratory maximum dry density in accordance with ASTM D 1557 (latest edition) where the soils to be utilized in the fill have less than 15 percent finer than 0.005 millimeter. Soils with more than 15 percent finer than 0.005 millimeter may be compacted to 90 percent of the laboratory maximum dry density in accordance with ASTM D 1557 (latest edition). Fill and backfill soils should be placed in horizontal loose layers approximately 6 to 8 inches thick, moisture conditioned to near optimum moisture content and properly compacted in accordance with ASTM D 1557 (latest edition).

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

7.5.8 Where new paving is to be placed, it is recommended that all existing fill and soft alluvial soils be excavated and properly compacted for paving support. The client should be aware that excavation and compaction of all existing fill and soft soils in the area of new paving is not required; however, paving constructed over existing uncertified fill or unsuitable alluvial soil may experience increased settlement and/or cracking, and may therefore have a shorter design life and increased maintenance costs. As a minimum, the upper 12 inches of 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 7.14).

7.5.9 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 20 and corrosivity properties that are equally or less detrimental to that of the existing onsite soils (see Figure B28).

7.5.10 Utility trenches should be properly backfilled in accordance with the following requirements. The pipe should be bedded with clean sands (Sand Equivalent greater than 30) to a depth of at least 1 foot over the pipe, and the bedding material must be inspected and approved in writing by the Geotechnical Engineer (a representative of Geocon). The use of gravel is not acceptable unless used in conjunction with filter fabric to prevent the gravel from having direct contact with soil. 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 as backfill is also acceptable (see Section 7.6). 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).

7.5.11 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 sands, fill, steel, gravel, or concrete.

7.6 Controlled Low Strength Material (CLSM)

7.6.1 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 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.

7.7 Mat Foundation Design – One Subterranean Level

7.7.1 The mat foundation system may derive support in the competent undisturbed alluvial soils at and below a depth of 15 feet below the existing ground surface. Any exposed soft soils should be compacted to a dense state or penetrated by proposed foundations at the direction of the Geotechnical Engineer (a representative of Geocon).

7.7.2 Where new foundations are constructed immediately adjacent to existing foundations, the new foundation should be deepened to match the depth of the existing foundation to prevent a surcharge on the existing foundation.

7.7.3 Where proposed foundations will be deeper than the existing foundation, the proposed foundation must be designed to resist the surcharge imposed by the existing foundation. The surcharge area may be defined by a 1:1 projection up and away from the bottom of an existing foundation.

7.7.4 The recommended maximum allowable bearing value for the design of a reinforced concrete mat foundation is 6,500 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.

7.7.5 It is recommended that a modulus of subgrade reaction of 150 pounds per cubic inch (pci) be utilized for the design of the mat foundation bearing in undisturbed alluvial soils. This value is a unit value 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)

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

7.7.7 For seismic design purposes, a coefficient of friction of 0.45 may be utilized between concrete slab and subgrade soils without a moisture barrier, and 0.15 for slabs underlain by a moisture barrier.

7.7.8 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.

7.7.9 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.

7.8 Mat Foundation Design – Two Subterranean Levels

7.8.1 The mat foundation system may derive support in the competent undisturbed alluvial soils at and below a depth of 30 feet below the existing ground surface. Any exposed soft soils should be compacted to a dense state or penetrated by proposed foundations at the direction of the Geotechnical Engineer (a representative of Geocon). In addition, the transition area between the one-subterranean level portion to the two-subterranean level portion of the structure should be more heavily reinforced to resist differential settlement stresses which could cause cracking.

7.8.2 The City of Los Angeles Building Code requires that the structure be designed for the historically high groundwater level, which is approximately 20 feet below the existing ground surface. The proposed structure must be designed for hydrostatic pressure for any portion of the structure below a depth of 20 feet. 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. If the proposed structure does not provide sufficient dead load to resist the buoyant forces then uplift mitigation will be required.

7.8.3 Where new foundations are constructed immediately adjacent to existing foundations, the new foundation should be deepened to match the depth of the existing foundation to prevent a surcharge on the existing foundation.

7.8.4 Where proposed foundations will be deeper than the existing foundation, the proposed foundation must be designed to resist the surcharge imposed by the existing foundation. The surcharge area may be defined by a 1:1 projection up and away from the bottom of an existing foundation.

7.8.5 The recommended maximum allowable bearing value for the design of a reinforced concrete mat foundation is 3,500 pounds per square foot (psf) (this value have been adjusted for buoyant forces). The allowable bearing pressure may be increased by up to one-third for transient loads due to wind or seismic forces.

7.8.6 It is recommended that a modulus of subgrade reaction of 125 pounds per cubic inch (pci) be utilized for the design of the mat foundation bearing in undisturbed alluvial soils. This value is a unit value 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)

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

7.8.8 For seismic design purposes, a coefficient of friction of 0.45 may be utilized between concrete slab and subgrade soils without a moisture barrier, and 0.15 for slabs underlain by a moisture barrier.

7.8.9 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.

7.8.10 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.

7.9 Foundation Settlement

7.9.1 The enclosed liquefaction settlement analyses indicate that the site soils could be susceptible up to approximately 0.7 inch of total settlement as a result of the Design Earthquake peak ground acceleration (⅔PGA_M). The differential settlement at the foundation level is anticipated to be less than 0.35 inch over a distance of 20 feet. These settlements are in addition to the static settlements indicated below and must be considered in the structural design.

7.9.2 The maximum expected static settlement for on a reinforced concrete mat foundation with a maximum allowable bearing pressure of 6,500 psf deriving support in competent alluvial soils is expected to be approximately than 1¼ inches and occur below the heaviest loaded structural element. Differential settlement is expected to be less than 0.63 inch between the center and corner of the mat foundation. A majority of the settlement of the foundation system is expected to occur on initial application of loading; however, minor additional settlements are expected within the first twelve months. Based on seismic considerations, the proposed structure supported on a mat foundation system should be designed for a combined static and seismically induced differential settlement of approximately 1 inch over a distance of 20 feet.

7.9.3 The maximum expected static settlement for on a reinforced concrete mat foundation with a maximum allowable bearing pressure of 9,500 psf deriving support in competent alluvial soils at and below a depth of 30 feet is expected to be approximately than 1¼ inches and occur below the heaviest loaded structural element. Differential settlement is expected to be less than 0.63 inch between the center and corner of the mat foundation. A majority of the settlement of the foundation system is expected to occur on initial application of loading; however, minor additional settlements are expected within the first twelve months. Based on seismic considerations, the proposed structure supported on a mat foundation system should be designed for a combined static and seismically induced differential settlement of approximately 1 inch over a distance of 20 feet.

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

7.10 Uplift Resistance

7.10.1 Foundation uplift may be resisted by the weight of structure, as well as 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 additional piles constructed within the interior of the structure. In order to maximize capacity it is suggested that post-grouted friction piles be considered. If it is determined that recommendations for uplift resistance are required as a part of this project, the recommendations will be provided under separate cover.

7.11 Miscellaneous Foundations

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

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

7.11.3 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.

7.12 Lateral Design

7.12.1 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 0.45 may be used with the dead load forces in the new placed engineered fill or competent alluvial soils.

7.12.2 Passive earth pressure for the sides of foundations and slabs poured against newly placed engineered fill or the alluvial soils may be computed as an equivalent fluid having a density of 350 pcf with a maximum earth pressure of 3,500 pcf. 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 140 pounds per cubic foot with a maximum earth pressure of 1,400 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.

7.13 Exterior Concrete Slabs-on-Grade

7.13.1 Exterior concrete slabs-on-grade at the ground surface subject to vehicle loading should be designed in accordance with the recommendations in the *Preliminary Pavement Recommendations* section of this report (see Section 7.14).

7.13.2 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 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) and should be installed in general conformance with ASTM E 1643 (latest edition) and the manufacturer's recommendations. A minimum thickness of 15 mils extruded polyolefin plastic is recommended; vapor retarders which contain recycled content or woven materials are not recommended. The vapor retarder should have a permeance of less than 0.01 perms demonstrated by testing before and after mandatory conditioning. The vapor retarder should be installed in direct contact with the concrete slab with proper perimeter seal. If the California Green Building Code requirements apply to this project, the vapor retarder should be underlain by 4 inches of clean aggregate. It is important that the vapor retarder be puncture resistant since it will be in direct contact with angular gravel. As an alternative to the clean aggregate suggested in the Green Building Code, it is our opinion that the concrete slab-on-grade may be underlain by a vapor retarder over 4 inches of clean sand (sand equivalent greater than 30), since the sand will serve a capillary break and will minimize the potential for punctures and damage to the vapor barrier.

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

7.13.4 The moisture content of the slab subgrade should be maintained and sprinkled as necessary to maintain a moist condition as would be expected in any concrete placement.

7.13.5 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.

7.14 Preliminary Pavement Recommendations

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

7.14.2 The following pavement sections are based on an assumed R-Value of 30. Once site grading activities are complete an R-Value should be obtained by laboratory testing to confirm the properties of the soils serving as paving subgrade, prior to placing pavement.

7.14.3 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	10.0

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

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

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

7.15 Retaining Wall Design

7.15.1 The recommendations presented below are generally applicable to the design of rigid concrete or masonry retaining walls having a maximum height of 30 feet. In the event that walls higher than 30 feet are planned, Geocon should be contacted for additional recommendations.

7.15.2 Retaining wall foundations may be designed in accordance with the recommendations provided in the *Mat Foundation Design* sections of this report (see Sections 7.7 and 7.8).

7.15.3 Retaining walls with a level backfill surface that are not restrained at the top should be designed utilizing a triangular distribution of pressure (active pressure). Restrained walls are those that are not allowed to rotate more than 0.001H (where H equals the height of the retaining portion of the wall in feet) at the top of the wall. Where walls are restrained from movement at the top, walls may be designed utilizing a triangular distribution of pressure (at-rest pressure). The table on the following page presents recommended pressures to be used in retaining wall design, assuming that proper drainage will be maintained. The calculations of the retaining wall pressures are presented on Figures 13A and 13B.

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 15	43	52
Between 16 and 30	52	56

7.15.4 The wall pressures provided above assume that the retaining wall will be properly drained preventing the buildup of hydrostatic pressure. If retaining wall drainage is not implemented, the equivalent fluid pressure to be used in design of undrained walls is 90 pcf. The value includes hydrostatic pressures plus buoyant lateral earth pressures. If a partially drained wall is proposed, Geocon should be contacted to provide additional recommendations.

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

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

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

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

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

and

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

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

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

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

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

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

and

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

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

then

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

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

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

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

7.16 Dynamic (Seismic) Lateral Forces

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

7.16.2 A seismic load of 11 pcf should be used for design of walls that support more than 6 feet of backfill in accordance with Section 1803.5.12 of the 2022 CBC. The seismic load is applied as an equivalent fluid pressure along the height of the wall and the calculated loads result in a maximum load exerted at the base of the wall and zero at the top of the wall. This seismic load should be applied in addition to the active earth pressure. The earth pressure is based on half of two thirds of PGA_M calculated from ASCE 7-16 Section 11.8.3.

7.17 Retaining Wall Drainage

7.17.1 Unless designed for hydrostatic pressures, retaining walls should be provided with a drainage system extended at least two-thirds the height of the wall. 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 14). The clean bottom and subdrain pipe, behind a retaining wall, should be observed by the Geotechnical Engineer (a representative of Geocon), prior to placement of gravel or compacting backfill.

7.17.2 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 15). These vertical columns of drainage material would then be connected at the bottom of the wall to a collection panel or a 1-cubic-foot rock pocket drained by a 4-inch subdrain pipe.

7.17.3 Subdrainage pipes at the base of the retaining wall drainage system should outlet to an acceptable location via controlled drainage structures. Drainage should not be allowed to flow uncontrolled over descending slopes.

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

7.18 Elevator Pit Design

7.18.1 The elevator pit slab and retaining wall should be designed by the project structural engineer. Elevator pits may be designed in accordance with the recommendations in the *Mat Foundation Design* and *Retaining Wall Design* sections of this report (see Sections 7.7, 7.8 and 7.15).

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

7.18.3 If retaining wall drainage is to be provided, the drainage system should be designed in accordance with the *Retaining Wall Drainage* section of this report (see Section 7.17).

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

7.19 Elevator Piston

7.19.1 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. Cobble and boulders may be encountered during excavation. Additionally, some of the site soils have little to no cohesion and are prone to excessive caving. The contractor should be prepared for difficult drilling conditions.

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

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

7.20 Temporary Excavations

7.20.1 Excavations on the order of up to 34 feet in height are anticipated for excavation and construction of the proposed subterranean level(s), including the foundation system and dewatering system, depending on final design. The excavations are expected to expose artificial fill and alluvial soils, 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.

7.20.2 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 10 feet. A uniform slope does not have a vertical portion.

7.20.3 If excavations in close proximity to an adjacent property line and/or structure are required, special excavation measures such as shoring may be necessary in order to maintain lateral support of offsite improvements. Shoring recommendation are provided in Section 7.21 of this report.

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

7.21 Shoring – Soldier Pile Design and Installation

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

7.21.2 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. Due to the presence of cobbles the installation of steel soldier piles utilizing high frequency vibration is expected to be difficult. It is recommended that the contractor bidding on shoring installation for this project perform their own test borings and vibratory soldier pile installation with the intended equipment to verify the presence and size of buried rock (cobbles and boulders) as well as the suitability of the proposed equipment for the safe and efficient installation of the soldier piles.

7.21.3 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 grading activities, foundations, and/or adjacent drainage systems.

7.21.4 The proposed soldier piles may also be designed as permanent piles. The required pile depths, dimensions, and spacing should be determined and designed by the project structural and shoring engineers. All piles utilized for shoring can also be incorporated into a permanent retaining wall system (shotcrete wall) and should be designed in accordance with the earth pressure provided in the *Retaining Wall Design* section of this report (see Section 7.15).

7.21.5 Drilled cast-in-place soldier piles should be placed no closer than three diameters on center. The minimum diameter of the piles is 18 inches. Structural concrete should be used for the soldier piles below the excavation; lean-mix concrete may be employed above that level. As an alternative, lean-mix concrete may be used throughout the pile where the reinforcing consists of a wideflange section. The slurry must be of sufficient strength to impart the lateral bearing pressure developed by the wideflange section to the soil. For design purposes, an allowable passive value for the soils below the plane of excavation above groundwater may be assumed to be 280 psf per foot. An allowable passive value for the soils below the plane of excavation below groundwater may be assumed to be 135 psf per foot (value has been reduced for buoyant forces). Where piles are installed by vibration techniques, the passive pressure may be assumed to mobilize across a width equal to the two times the dimension of the beam flange. The allowable passive value may be doubled for isolated piles, spaced a minimum of two times the pile diameter. To develop the full lateral value, provisions should be implemented to assure firm contact between the soldier piles and the undisturbed alluvium.

7.21.6 Groundwater was encountered during site exploration at depths of approximately 23 to 24 feet; however, groundwater levels can fluctuate and may be different at the time of construction. It is not uncommon for groundwater or seepage conditions to develop where none previously existed. Therefore the contractor should be prepared for groundwater during pile installation should the need arise. If more than 6 inches of water is present in the bottom of the excavation, a tremie is required to place the concrete into the bottom of the hole. A tremie should consist of a rigid, water-tight tube having a diameter of not less than 6 inches with a hopper at the top. The tube should be equipped with a device that will close the discharge end and prevent water from entering the tube while it is being charged with concrete. The tremie should be supported so as to permit free movement of the discharge end over the entire top surface of the work and to permit rapid lowering when necessary to retard or stop the flow of concrete. The discharge end should be closed at the start of the work to prevent water entering the tube and should be entirely sealed at all times, except when the concrete is being placed. The tremie tube should be kept full of concrete. The flow should be continuous until the work is completed and the resulting concrete seal should be monolithic and homogeneous. The tip of the tremie tube should always be kept about 5 feet below the surface of the concrete and definite steps and safeguards should be taken to insure that the tip of the tremie tube is never raised above the surface of the concrete.

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

7.21.8 Please be aware that the use of hollow-stem auger drilling equipment utilized for this investigation does not allow for the identification of the size of rocks being encountered or the visual observation of caving conditions since the drilling method is a small diameter cased excavation. It is recommended that the contractor bidding on excavation and shoring installation for this project perform their own test borings with the intended drilling equipment to verify the presence and size of buried rock (cobbles and boulders), potential for caving, as well as the suitability of the proposed drilling equipment for the safe and efficient installation of the shoring system.

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

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

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

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

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

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

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

7.21.16 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 0.45 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 230 psf per foot (value has been reduced for buoyant forces).

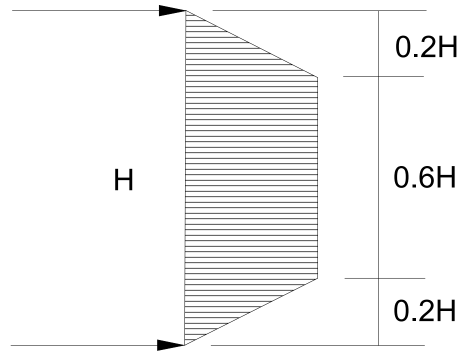
7.21.17 Due to the nature of the site soils, it is expected that continuous lagging between soldier piles will be required. However, it is recommended that the exposed soils be observed by the Geotechnical Engineer (a representative of Geocon West, Inc.), to verify the presence of any competent, cohesive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

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

7.21.19 For the design of unbraced shoring, it is recommended that an equivalent fluid pressure be utilized for design. A trapezoidal distribution of lateral earth pressure may be used where shoring will be restrained by bracing or tie backs. The recommended active and trapezoidal pressures are provided in the following table. A diagram depicting the trapezoidal pressure distribution of lateral earth pressure is provided below the table. Calculations of the shoring pressures are presented on Figures 16A and 16B.

HEIGHT OF SHORING (FEET)	EQUIVALENT FLUID PRESSURE (Pounds Per Cubic Foot) (ACTIVE PRESSURE)	EQUIVALENT FLUID PRESSURE Trapezoidal (Where H is the height of the shoring in feet)
Up to 17	34	22H
Up to 34	44	28H

Trapezoidal Distribution of Pressure



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

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

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

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

and

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

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

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

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

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

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

and

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

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

then

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

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

7.21.23 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 ten feet from the shoring, the traffic surcharge may be neglected.

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

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

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

7.22 Temporary Tie-Back Anchors

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

7.22.2 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 – 600 psf
- 15 feet below the top of the excavation – 650 psf (value has been reduced for buoyant forces)

7.22.3 Depending on the techniques utilized, and the experience of the contractor performing the installation, a maximum allowable friction capacity of 2 kips per linear foot for post-grouted anchors (for a minimum 20-foot length beyond the active wedge) may be assumed for design purposes. Only the frictional resistance developed beyond the active wedge should be utilized in resisting lateral loads.

7.23 Anchor Installation

7.23.1 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.

7.24 Anchor Testing

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

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

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

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

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

7.25 Internal Bracing

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

7.26 Surface Drainage

7.26.1 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.

7.26.2 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 2022 CBC 1804.4 or other applicable standards. In addition, drainage should not be allowed to flow uncontrolled over any descending slope. Discharge from downspouts, roof drains and scuppers are not recommended onto unprotected soils within 5 feet of the building perimeter. Planters which are located adjacent to foundations should be sealed to prevent moisture intrusion into the soils providing foundation support. Landscape irrigation is not recommended within 5 feet of the building perimeter footings except when enclosed in protected planters.

7.26.3 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.

7.26.4 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.

7.27 Plan Review

7.27.1 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.

LIST OF REFERENCES

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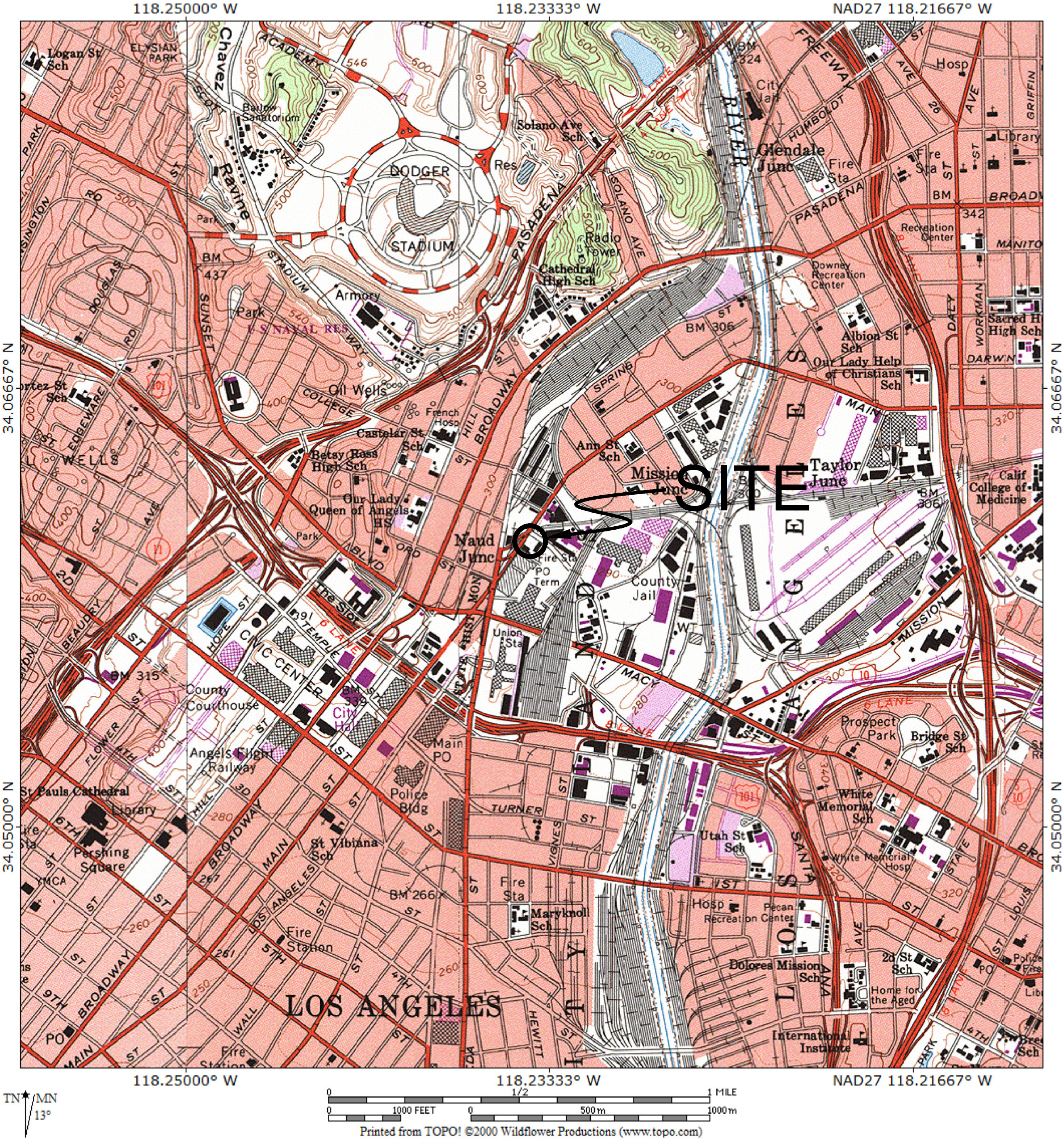
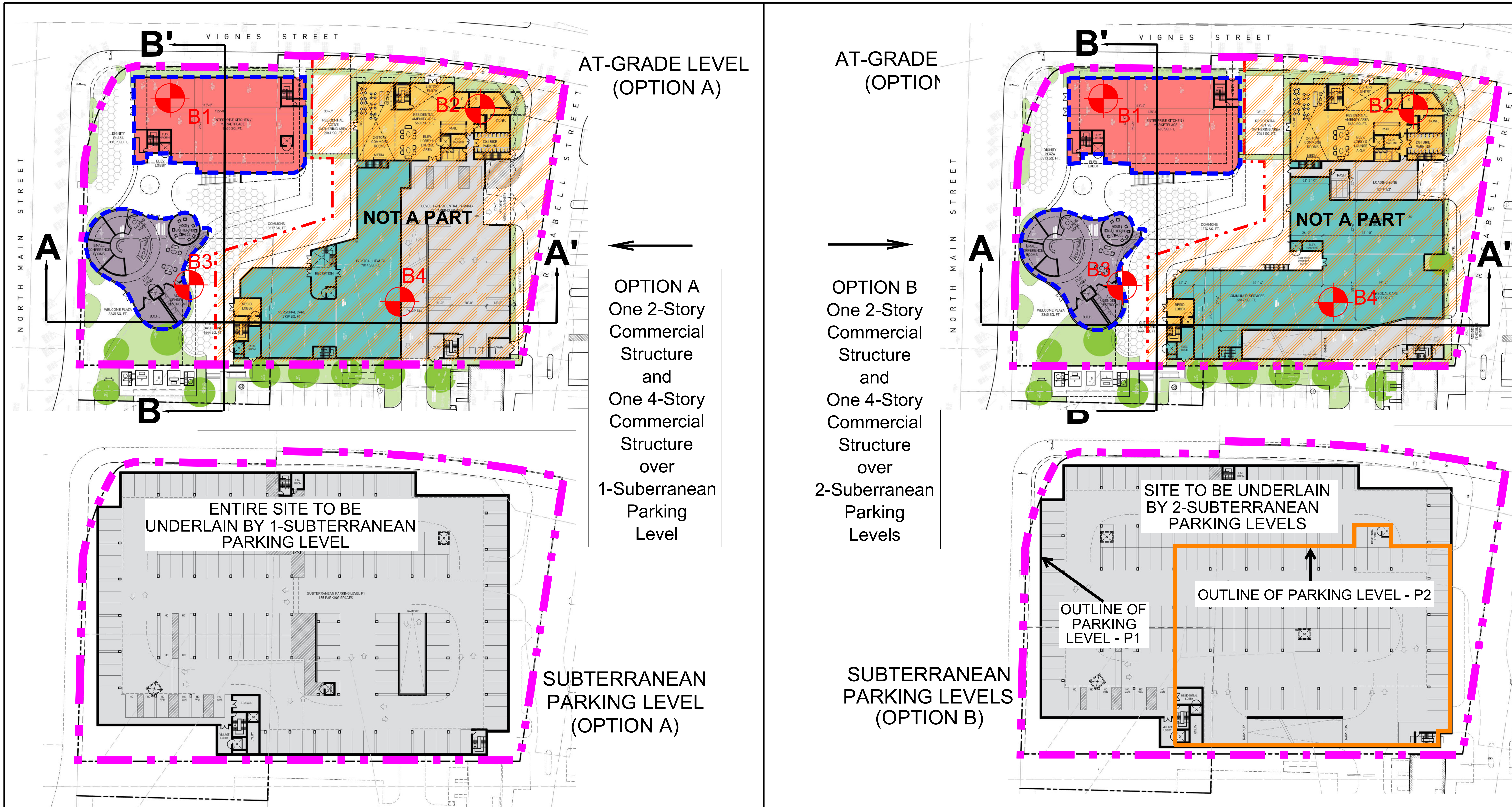


U.S.G.S. TOPOGRAPHIC MAPS, 7.5 MINUTE SERIES, LOS ANGELES AND HOLLYWOOD, CA QUADRANGLES

GEOCON WEST, INC.		VICINITY MAP		
ENVIRONMENTAL GEOTECHNICAL MATERIALS 3303 N. SAN FERNANDO BLVD. - SUITE 100 - BURBANK, CA 91504 PHONE (818) 841-8388 - FAX (818) 841-1704		800 & 908 NORTH MAIN STREET 1081 & 1087 NORTH VIGNES STREET LOS ANGELES, CALIFORNIA		
DRAFTED BY: CB	CHECKED BY: GAK	OCT. 2023	PROJECT NO. W1815-06-01	FIG. 1



Legend

- B4 Boring Locations
- Limit of Proposed Structures
- Property Limits

0 75' 150'

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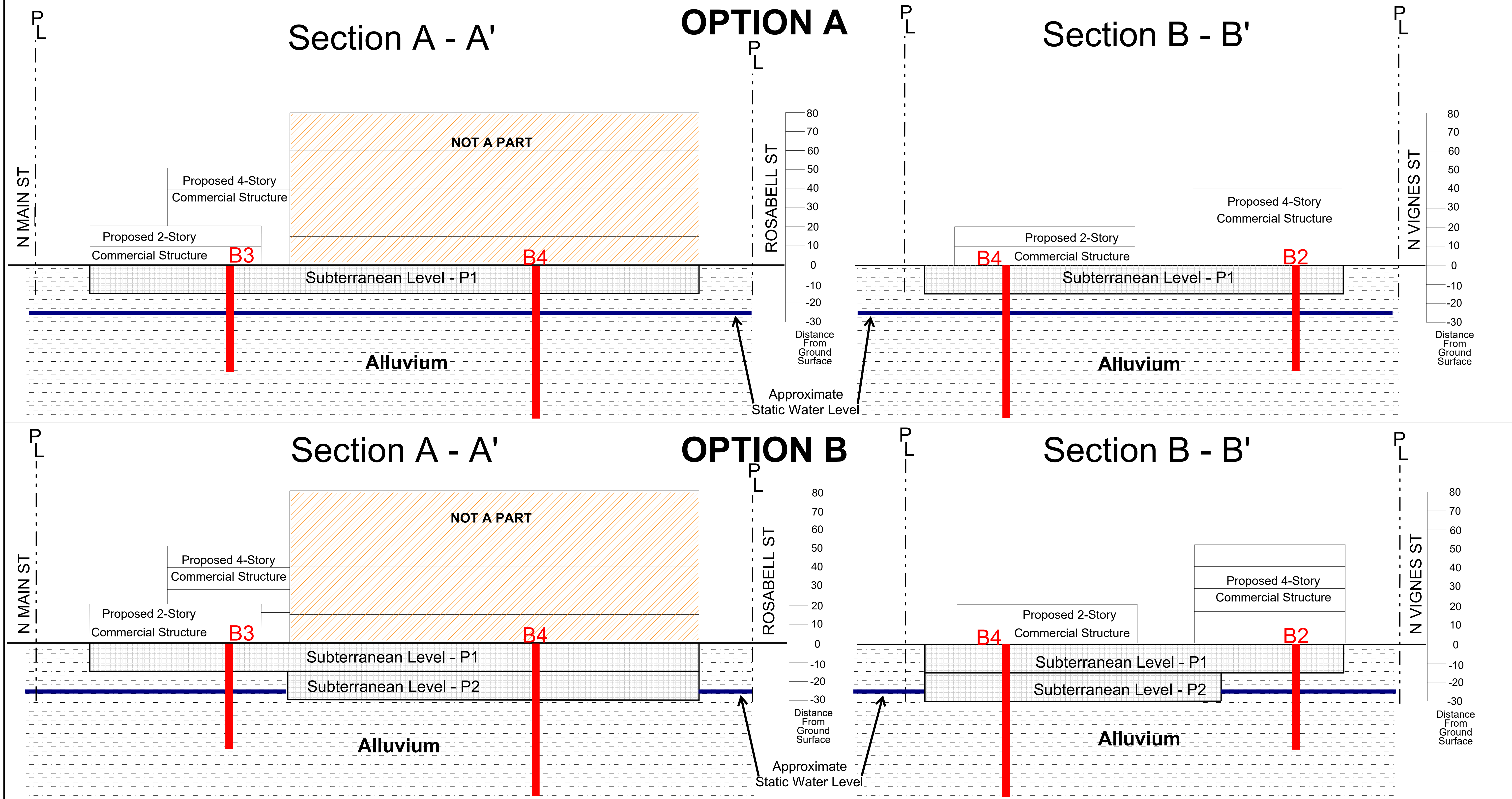
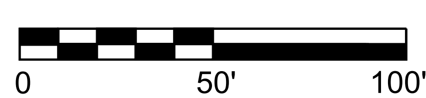
DRAFTED BY: JJK | CHECKED BY: HHD

SITE PLAN

800 & 908 NORTH MAIN STREET
1081 & 1087 NORTH VIGNES STREET STREET
LOS ANGELES, CALIFORNIA

OCT. 2023	PROJECT NO: W1815-06-01	FIG. 2A





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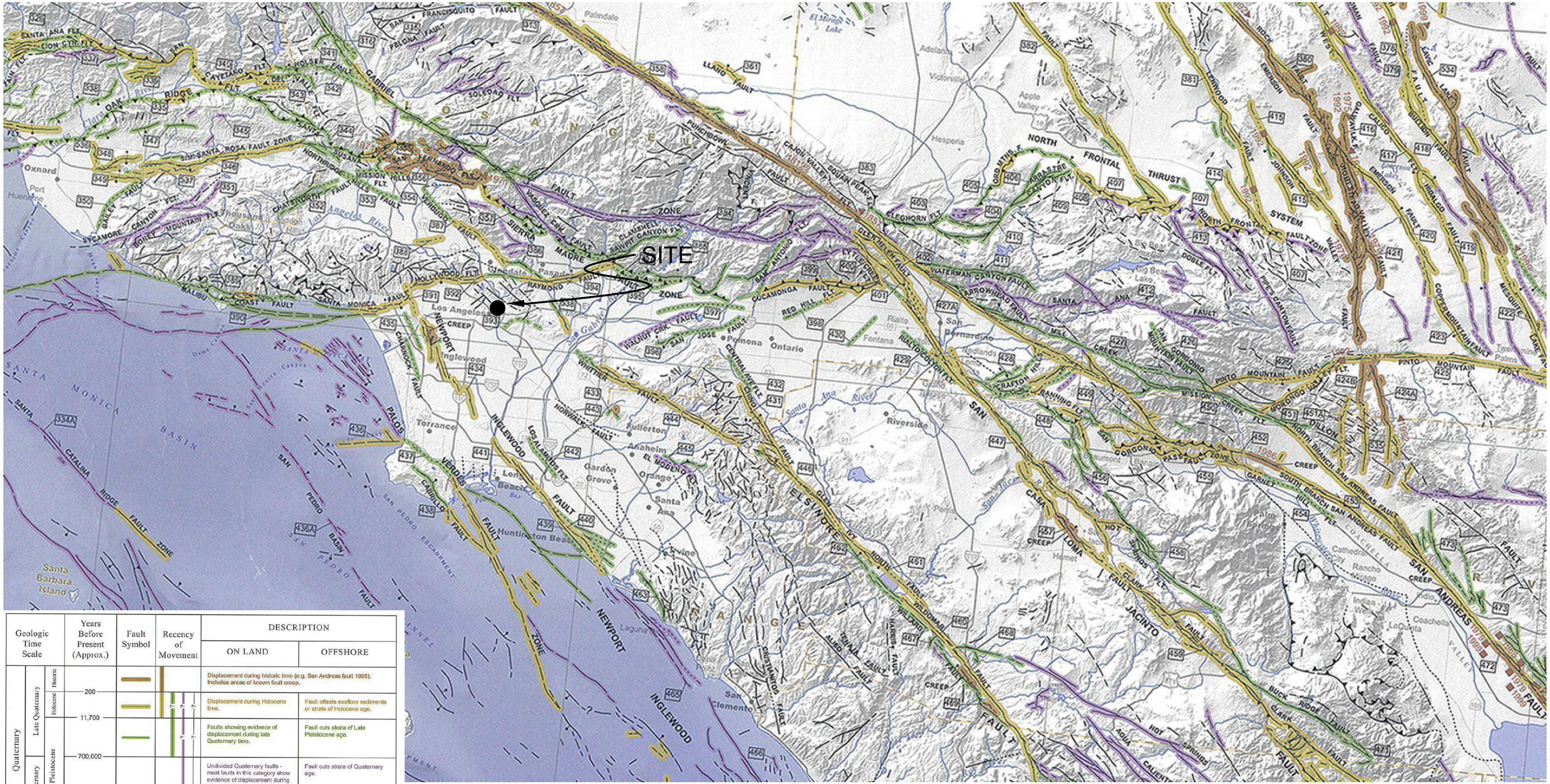
CROSS SECTION

800 & 908 NORTH MAIN STREET
1081 & 1087 NORTH VIGNES STREET STREET
LOS ANGELES, CALIFORNIA

DRAFTED BY: JJK	CHECKED BY: HHD	OCT. 2023	PROJECT NO: W1815-06-01	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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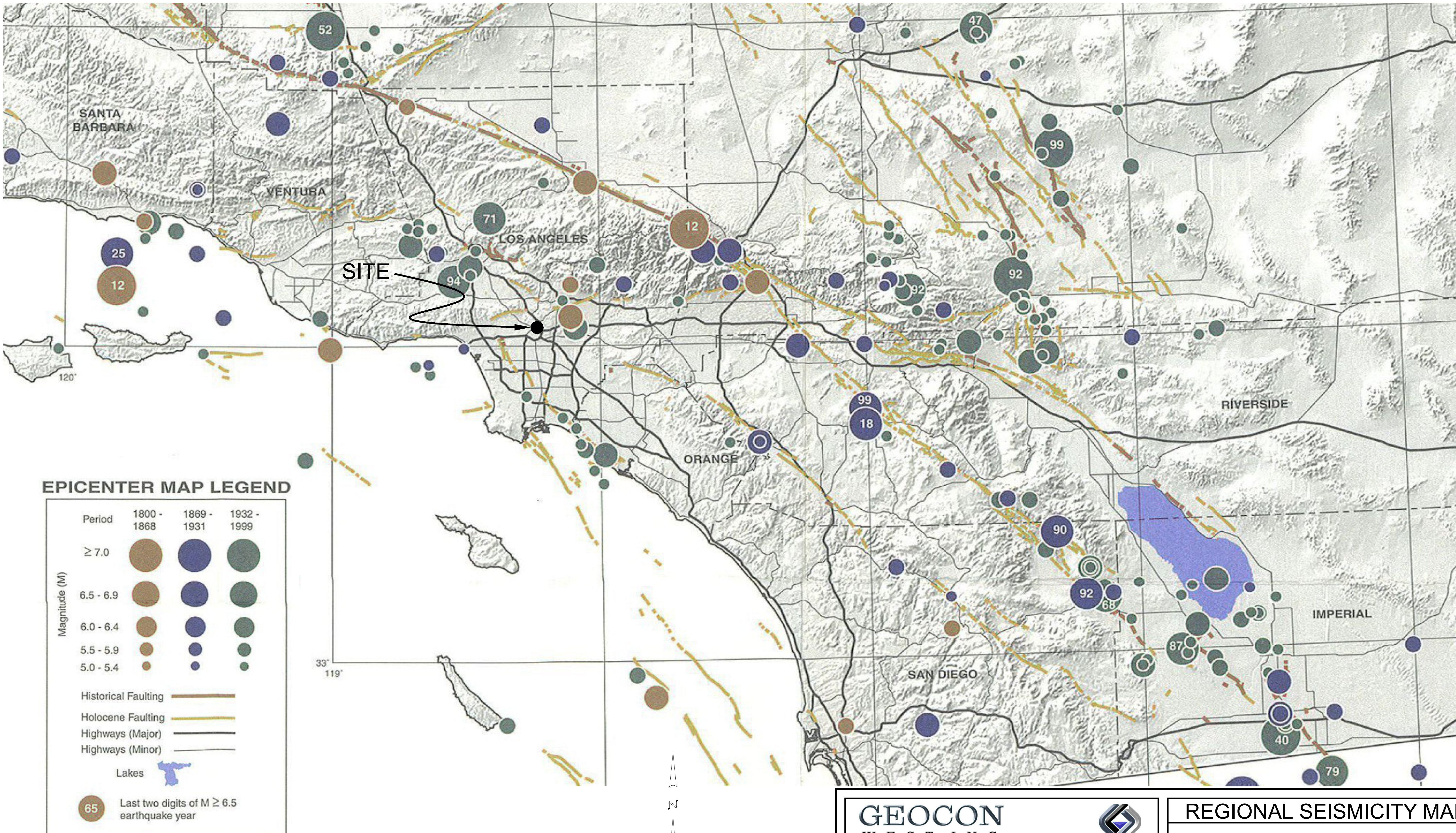
REGIONAL FAULT MAP

800 & 908 NORTH MAIN STREET
1081 & 1087 NORTH VIGNES STREET

LOS ANGELES, CALIFORNIA

OCT. 2023 | PROJECT NO. W1815-06-01 | FIG. 3



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

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REGIONAL SEISMICITY MAP

800 & 908 NORTH MAIN STREET
1081 & 1087 NORTH VIGNES STREET
LOS ANGELES, CALIFORNIA

OCT. 2023 | PROJECT NO. W1815-06-01 | FIG.4

GEOCON

Project Name : Hope Village
Project No : W1815-06-01
Boring : B1

EMPIRICAL ESTIMATION OF LIQUEFACTION POTENTIAL
DESIGN EARTHQUAKE

NCEER (1996) METHOD W 2001 UPDATES

EARTHQUAKE INFORMATION:

Earthquake Magnitude:	6.74
Peak Horiz. Acceleration PGA_M (g):	0.943
2/3 PGA_M (g):	0.629
Magnitude Scaling Factor:	1.314
Historic High Groundwater:	20.0
Groundwater Depth During Exploration:	24.0

ENERGY & ROD CORRECTIONS:

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

LIQUEFACTION CALCULATIONS:

Unit Wt. Water (pcf):	62.4

Depth to Base (ft)	Total Unit Wt. (pcf)	Water (0 or 1)	FIELD SPT (N)	Depth of SPT (ft)	Liq.Sus. (0 or 1)	-200 (%)	Est. Dr (%)	CN Factor	Corrected (N1)60	Eff. Unit Wt. (psf)	Resist. CRR 7.5	rd Factor	Induced CSR	Liquefac. Safe.Fact.
1.0	125.0	0	11.6	1.0	1		82	1.700	22.1	125.0	0.243	1.000	0.409	--
2.0	125.0	0	11.6	2.0	1		80	1.700	22.1	125.0	0.243	0.998	0.408	--
3.0	114.8	0	11.6	2.5	1		78	1.700	22.1	114.8	0.243	0.996	0.407	--
4.0	114.8	0	11.6	2.5	1		78	1.700	22.1	114.8	0.243	0.994	0.406	--
5.0	114.8	0	11.6	2.5	1		78	1.700	22.1	114.8	0.243	0.991	0.405	--
6.0	114.8	0	9.0	5.0	1		65	1.700	17.2	114.8	0.183	0.989	0.404	--
7.0	114.8	0	9.0	5.0	1		65	1.650	16.7	114.8	0.178	0.987	0.403	--
8.0	121.3	0	9.0	5.0	1		65	1.536	15.6	121.3	0.166	0.985	0.403	--
9.0	121.3	0	9.0	5.0	1		65	1.441	14.6	121.3	0.156	0.982	0.402	--
10.0	121.3	0	9.0	5.0	1		65	1.361	13.8	121.3	0.148	0.980	0.401	--
11.0	125.0	0	29.0	10.0	1		106	1.292	42.2	125.0	Infin.	0.978	0.400	--
12.0	125.0	0	29.0	10.0	1		106	1.232	40.2	125.0	Infin.	0.976	0.399	--
13.0	125.0	0	29.0	10.0	1		106	1.180	38.5	125.0	Infin.	0.974	0.398	--
14.0	125.0	0	29.0	10.0	1		106	1.133	37.0	125.0	Infin.	0.972	0.397	--
15.0	125.0	0	29.0	10.0	1		106	1.092	35.6	125.0	Infin.	0.970	0.396	--
16.0	123.4	0	31.0	15.0	1		100	1.055	39.6	123.4	Infin.	0.967	0.396	--
17.0	123.4	0	31.0	15.0	1		100	1.022	38.3	123.4	Infin.	0.965	0.395	--
18.0	123.4	0	31.0	15.0	1		100	0.992	37.2	123.4	Infin.	0.963	0.394	--
19.0	123.4	0	31.0	15.0	1		100	0.964	36.2	123.4	Infin.	0.961	0.393	--
20.0	123.4	0	31.0	15.0	1		100	0.939	35.2	123.4	Infin.	0.958	0.392	--
21.0	123.4	1	50.0	20.0	1		118	0.915	61.4	61.0	Infin.	0.956	0.396	Non-Liq.
22.0	123.4	1	50.0	20.0	1		118	0.894	60.0	61.0	Infin.	0.953	0.404	Non-Liq.
23.0	124.2	1	50.0	20.0	1		118	0.873	58.6	61.8	Infin.	0.950	0.412	Non-Liq.
24.0	124.2	1	50.0	20.0	1		118	0.859	57.6	61.8	Infin.	0.947	0.419	Non-Liq.
25.0	124.2	1	50.0	20.0	1		118	0.849	57.0	61.8	Infin.	0.944	0.426	Non-Liq.
26.0	124.2	1	18.0	25.0	1	36	68	0.841	31.0	61.8	Infin.	0.940	0.432	Non-Liq.
27.0	124.2	1	18.0	25.0	1	36	68	0.832	30.7	61.8	Infin.	0.936	0.438	Non-Liq.
28.0	130.6	1	45.7	27.5	1	3	106	0.823	55.2	68.2	Infin.	0.932	0.443	Non-Liq.
29.0	130.6	1	45.7	27.5	1	3	106	0.814	54.6	68.2	Infin.	0.928	0.447	Non-Liq.
30.0	130.6	1	45.7	27.5	1	3	106	0.805	54.0	68.2	Infin.	0.923	0.451	Non-Liq.
31.0	130.6	1	28.0	30.0	1	3	82	0.797	33.5	68.2	Infin.	0.918	0.455	Non-Liq.
32.0	145.4	1	39.1	32.5	1	3	95	0.788	46.2	83.0	Infin.	0.912	0.457	Non-Liq.
33.0	145.4	1	39.1	32.5	1	3	95	0.778	45.6	83.0	Infin.	0.907	0.459	Non-Liq.
34.0	145.4	1	39.1	32.5	1	3	95	0.769	45.1	83.0	Infin.	0.900	0.461	Non-Liq.
35.0	145.4	1	39.1	32.5	1		95	0.760	44.5	83.0	Infin.	0.894	0.462	Non-Liq.
36.0	145.4	1	50.0	35.0	1		106	0.752	56.4	83.0	Infin.	0.887	0.463	Non-Liq.
37.0	145.4	1	50.0	35.0	1		106	0.743	55.8	83.0	Infin.	0.880	0.463	Non-Liq.
38.0	145.4	1	50.0	35.0	1		106	0.735	55.2	83.0	Infin.	0.872	0.462	Non-Liq.
39.0	145.4	1	50.0	35.0	1		106	0.728	54.6	83.0	Infin.	0.864	0.462	Non-Liq.
40.0	145.4	1	50.0	35.0	1		106	0.720	54.0	83.0	Infin.	0.855	0.460	Non-Liq.
41.0	140.9	1	44.0	40.0	1		95	0.713	47.1	78.5	Infin.	0.846	0.459	Non-Liq.
42.0	140.9	1	44.0	40.0	1		95	0.706	46.6	78.5	Infin.	0.837	0.457	Non-Liq.
43.0	140.9	1	44.0	40.0	1		95	0.700	46.2	78.5	Infin.	0.828	0.455	Non-Liq.
44.0	140.9	1	44.0	40.0	1		95	0.693	45.8	78.5	Infin.	0.818	0.453	Non-Liq.
45.0	140.9	1	44.0	40.0	1		95	0.687	45.4	78.5	Infin.	0.808	0.450	Non-Liq.
46.0	140.9	1	100.0	45.0	1		139	0.681	102.2	78.5	Infin.	0.798	0.447	Non-Liq.
47.0	140.9	1	100.0	45.0	1		139	0.675	101.3	78.5	Infin.	0.788	0.444	Non-Liq.
48.0	140.9	1	100.0	45.0	1		139	0.670	100.4	78.5	Infin.	0.778	0.440	Non-Liq.
49.0	140.9	1	100.0	45.0	1		139	0.664	99.6	78.5	Infin.	0.768	0.437	Non-Liq.
50.0	140.9	1	100.0	45.0	1		139	0.659	98.8	78.5	Infin.	0.757	0.433	Non-Liq.
51.0	127.7	1	23.0	50.0	1	81	65	0.654	32.1	65.3	Infin.	0.747	0.429	Non-Liq.
52.0	127.7	1	23.0	50.0	1	81	65	0.649	31.9	65.3	Infin.	0.737	0.426	Non-Liq.
53.0	127.7	1	37.4	50.0	1	81	83	0.645	48.4	65.3	Infin.	0.727	0.422	Non-Liq.
54.0	127.7	1	37.4	50.0	1	81	83	0.641	48.2	65.3	Infin.	0.717	0.419	Non-Liq.
55.0	127.7	1	37.4	50.0	1	81	83	0.637	47.9	65.3	Infin.	0.708	0.415	Non-Liq.
56.0	127.7	1	74.0	55.0	1		113	0.633	70.3	65.3	Infin.	0.698	0.411	Non-Liq.
57.0	127.7	1	74.0	55.0	1		113	0.629	69.8	65.3	Infin.	0.689	0.408	Non-Liq.
58.0	127.7	1	74.0	55.0	1		113	0.625	69.4	65.3	Infin.	0.680	0.404	Non-Liq.
59.0	127.7	1	74.0	55.0	1		113	0.621	69.0	65.3	Infin.	0.671	0.401	Non-Liq.
60.0	127.7	1	74.0	55.0	1		113	0.618	68.6	65.3	Infin.	0.663	0.397	Non-Liq.
61.0	129.0	1	76.0	60.0	1		112	0.614	70.0	66.6	Infin.	0.655	0.394	Non-Liq.
62.0	129.0	1	76.0	60.0	1		112	0.610	69.6	66.6	Infin.	0.647	0.391	Non-Liq.
63.0	129.0	1	76.0	60.0	1		112	0.607	69.2	66.6	Infin.	0.639	0.388	Non-Liq.
64.0	129.0	1	76.0	60.0	1		112	0.603	68.8	66.6	Infin.	0.632	0.385	Non-Liq.
65.0	129.0	1	76.0	60.0	1		112	0.600	68.4	66.6	Infin.	0.625	0.382	Non-Liq.

Figure 5

GEOCON

Project Name : Hope Village
Project No : W1815-06-01
Boring : B4

EMPIRICAL ESTIMATION OF LIQUEFACTION POTENTIAL
DESIGN EARTHQUAKE

NCEER (1996) METHOD W 2001 UPDATES

EARTHQUAKE INFORMATION:

Earthquake Magnitude:	6.74
Peak Horiz. Acceleration PGA_M (g):	0.943
2/3 PGA_M (g):	0.629
Magnitude Scaling Factor:	1.314
Historic High Groundwater:	20.0
Groundwater Depth During Exploration:	24.0

ENERGY & ROD CORRECTIONS:

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

LIQUEFACTION CALCULATIONS:

Unit Wt. Water (pcf):	62.4

Depth to Base (ft)	Total Unit Wt. (pcf)	Water (0 or 1)	FIELD SPT (N)	Depth of SPT (ft)	Liq.Sus. (0 or 1)	-200 (%)	Est. Dr (%)	CN Factor	Corrected (N1)60	Eff. Unit Wt. (psf)	Resist. CRR 7.5	rd Factor	Induced CSR	Liquefac. Safe.Fact.
1.0	125.0	0	4.4	1.0	1		51	1.700	8.4	125.0	0.099	1.000	0.409	--
2.0	125.0	0	4.4	2.0	1		49	1.700	8.4	125.0	0.099	0.998	0.408	--
3.0	111.0	0	4.4	2.5	1		48	1.700	8.4	111.0	0.099	0.996	0.407	--
4.0	111.0	0	4.4	2.5	1		48	1.700	8.4	111.0	0.099	0.994	0.406	--
5.0	111.0	0	4.4	2.5	1		48	1.700	8.4	111.0	0.099	0.991	0.405	--
6.0	111.0	0	3.0	5.0	1		38	1.700	5.7	111.0	0.078	0.989	0.404	--
7.0	111.0	0	3.0	5.0	1		38	1.669	5.6	111.0	0.077	0.987	0.403	--
8.0	113.1	0	26.4	7.5	1		106	1.557	46.2	113.1	Infin.	0.985	0.403	--
9.0	113.1	0	26.4	7.5	1		106	1.464	43.5	113.1	Infin.	0.982	0.402	--
10.0	113.1	0	26.4	7.5	1		106	1.385	41.1	113.1	Infin.	0.980	0.401	--
11.0	113.1	0	32.0	10.0	1		112	1.319	47.5	113.1	Infin.	0.978	0.400	--
12.0	113.1	0	32.0	10.0	1		112	1.261	45.4	113.1	Infin.	0.976	0.399	--
13.0	120.7	0	32.0	10.0	1		112	1.208	43.5	120.7	Infin.	0.974	0.398	--
14.0	120.7	0	32.0	10.0	1		112	1.160	41.8	120.7	Infin.	0.972	0.397	--
15.0	120.7	0	32.0	10.0	1		112	1.117	40.2	120.7	Infin.	0.970	0.396	--
16.0	120.7	0	32.0	15.0	1		103	1.079	41.8	120.7	Infin.	0.967	0.396	--
17.0	120.7	0	32.0	15.0	1		103	1.045	40.4	120.7	Infin.	0.965	0.395	--
18.0	116.2	0	32.0	15.0	1		103	1.014	39.2	116.2	Infin.	0.963	0.394	--
19.0	116.2	0	32.0	15.0	1		103	0.986	38.2	116.2	Infin.	0.961	0.393	--
20.0	116.2	0	32.0	15.0	1		103	0.960	37.2	116.2	Infin.	0.958	0.392	--
21.0	116.2	1	56.0	20.0	1		126	0.937	70.4	53.8	Infin.	0.956	0.396	Non-Liq.
22.0	116.2	1	56.0	20.0	1		126	0.914	68.7	53.8	Infin.	0.953	0.405	Non-Liq.
23.0	139.0	1	56.0	20.0	1		126	0.892	67.1	76.6	Infin.	0.950	0.413	Non-Liq.
24.0	139.0	1	56.0	20.0	1		126	0.874	65.7	76.6	Infin.	0.947	0.420	Non-Liq.
25.0	139.0	1	56.0	20.0	1		126	0.862	64.8	76.6	Infin.	0.944	0.427	Non-Liq.
26.0	139.0	1	72.0	25.0	1		137	0.851	87.8	76.6	Infin.	0.940	0.433	Non-Liq.
27.0	139.0	1	72.0	25.0	1		137	0.840	86.6	76.6	Infin.	0.936	0.439	Non-Liq.
28.0	158.4	1	72.0	25.0	1		137	0.828	85.4	96.0	Infin.	0.932	0.443	Non-Liq.
29.0	158.4	1	72.0	25.0	1		137	0.815	84.1	96.0	Infin.	0.928	0.447	Non-Liq.
30.0	158.4	1	72.0	25.0	1		137	0.803	82.8	96.0	Infin.	0.923	0.451	Non-Liq.
31.0	158.4	1	27.0	30.0	1	5	80	0.791	32.0	96.0	Infin.	0.918	0.453	Non-Liq.
32.0	158.4	1	27.0	30.0	1	5	80	0.780	31.6	96.0	Infin.	0.912	0.456	Non-Liq.
33.0	131.5	1	38.0	32.5	1	5	93	0.771	43.9	69.1	Infin.	0.907	0.458	Non-Liq.
34.0	131.5	1	38.0	32.5	1	5	93	0.763	43.4	69.1	Infin.	0.900	0.459	Non-Liq.
35.0	131.5	1	38.0	32.5	1	5	93	0.756	43.0	69.1	Infin.	0.894	0.461	Non-Liq.
36.0	131.5	1	23.0	35.0	1	4	71	0.749	25.8	69.1	0.309	0.887	0.462	0.72
37.0	131.5	1	23.0	35.0	1	4	71	0.742	25.6	69.1	0.304	0.880	0.462	0.70
38.0	129.0	1	55.0	37.5	1	4	108	0.736	60.7	66.6	Infin.	0.872	0.462	Non-Liq.
39.0	129.0	1	55.0	37.5	1	4	108	0.729	60.2	66.6	Infin.	0.864	0.462	Non-Liq.
40.0	129.0	1	55.0	37.5	1	4	108	0.723	59.7	66.6	Infin.	0.855	0.461	Non-Liq.
41.0	135.6	1	22.0	40.0	1	7	68	0.717	24.0	73.2	0.273	0.846	0.460	0.62
42.0	135.6	1	22.0	40.0	1	7	68	0.711	23.8	73.2	0.269	0.837	0.459	0.61
43.0	135.6	1	55.0	42.5	1	7	105	0.704	58.7	73.2	Infin.	0.828	0.457	Non-Liq.
44.0	135.6	1	55.0	42.5	1	7	105	0.698	58.2	73.2	Infin.	0.818	0.454	Non-Liq.
45.0	135.6	1	55.0	42.5	1	7	105	0.692	57.7	73.2	Infin.	0.808	0.452	Non-Liq.
46.0	135.6	1	30.0	45.0	1		77	0.687	30.9	73.2	Infin.	0.798	0.449	Non-Liq.
47.0	135.6	1	30.0	45.0	1		77	0.681	30.6	73.2	Infin.	0.788	0.446	Non-Liq.
48.0	156.1	1	30.0	45.0	1		77	0.675	30.4	93.7	Infin.	0.778	0.442	Non-Liq.
49.0	156.1	1	30.0	45.0	1		77	0.668	30.1	93.7	Infin.	0.768	0.438	Non-Liq.
50.0	156.1	1	30.0	45.0	1		77	0.661	29.8	93.7	0.452	0.757	0.434	1.00
51.0	146.7	1	58.0	50.0	1		103	0.655	57.0	84.3	Infin.	0.747	0.430	Non-Liq.
52.0	146.7	1	58.0	50.0	1		103	0.650	56.5	84.3	Infin.	0.737	0.426	Non-Liq.
53.0	146.7	1	58.0	50.0	1		103	0.644	56.1	84.3	Infin.	0.727	0.422	Non-Liq.
54.0	146.7	1	58.0	50.0	1		103	0.639	55.6	84.3	Infin.	0.717	0.418	Non-Liq.
55.0	146.7	1	58.0	50.0	1		103	0.634	55.1	84.3	Infin.	0.708	0.414	Non-Liq.
56.0	146.7	1	79.0	55.0	1		117	0.629	74.5	84.3	Infin.	0.698	0.410	Non-Liq.
57.0	146.7	1	79.0	55.0	1		117	0.624	73.9	84.3	Infin.	0.689	0.406	Non-Liq.
58.0	146.7	1	41.0	57.5	1		83	0.619	38.1	84.3	Infin.	0.680	0.402	Non-Liq.
59.0	146.7	1	41.0	57.5	1		83	0.614	37.8	84.3	Infin.	0.671	0.398	Non-Liq.
60.0	146.7	1	41.0	57.5	1		83	0.610	37.5	84.3	Infin.	0.663	0.394	Non-Liq.
61.0	146.7	1	76.0	60.0	1		111	0.605	69.0	84.3	Infin.	0.655	0.390	Non-Liq.
62.0	146.7	1	76.0	60.0	1		111	0.601	68.5	84.3	Infin.	0.647	0.387	Non-Liq.
63.0	146.7	1	76.0	60.0	1		111	0.596	68.0	84.3	Infin.	0.639	0.383	Non-Liq.
64.0	146.7	1	76.0	60.0	1		111	0.592	67.5	84.3	Infin.	0.632	0.380	Non-Liq.
65.0	146.7	1	76.0	60.0	1		111	0.588	67.0	84.3	Infin.	0.625	0.377	Non-Liq.
66.0	146.7	1	76.0	65.0	1		108	0.584	66.6	84.3	Infin.	0.618	0.374	Non-Liq.
67.0	146.7	1	76.0	65.0	1		108	0.580	66.1	84.3	Infin.	0.612	0.371	Non-Liq.
68.0	146.7	1	76.0	65.0	1		108	0.576	65.7	84.3	Infin.	0.606	0.368	Non-Liq.
69.0	146.7	1	76.0	65.0	1		108	0.572	65.2	84.3	Infin.	0.600	0.365	Non-Liq.
70.0	146.7	1	76.0	65.0	1		108	0.568	64.8	84.3	Infin.	0.594	0.363	Non-Liq.
71.0	146.7	1	43.0	70.0	1		79	0.565	36.4	84.3	Infin.	0.589	0.360	Non-Liq.
72.0	146.7	1	43.0	70.0	1		79	0.561	36.2	84.3	Infin.	0.584	0.358	Non-Liq.
73.0	146.7	1	43.0	70.0	1		79	0.558	36.0	84.3	Infin.	0.579	0.355	Non-Liq.
74.0	146.7	1	43.0	70.0	1		79	0.554	35.7	84.3	Infin.	0.574	0.353	Non-Liq.
75.0	146.7	1	43.0	70.0	1		79	0.551	35.5	84.3	Infin.	0.570	0.351	Non-Liq.
76.0	146.7	1	92.0	75.0	1		113	0.547	75.5	84.3	Infin.	0.565	0.349	Non-Liq.
77.0	146.7	1	92.0	75.0	1		113	0.544	75.1	84.3	Infin.	0.561	0.347	Non-Liq.
78.0	146.7	1	92.0	75.0	1		113	0.541	74.6	84.3	Infin.	0.557	0.345	Non-Liq.
79.0	146.7	1	92.0	75.0	1		113	0.538	74.2	84.3	Infin.	0.553	0.344	Non-Liq.
80.0	146.7	1	92.0	75.0	1		113	0.535	73.8	84.3	Infin.	0.550	0.342	Non-Liq.

GEOCON

Project Name : Hope Village
Project No : W1815-06-01
Boring : B1

LIQUEFACTION SETTLEMENT ANALYSIS
DESIGN EARTHQUAKE

NCEER (1996) METHOD W 2001 UPDATES
EARTHQUAKE INFORMATION:

Earthquake Magnitude:	6.74
PGA_M (g):	0.943
2/3 PGA_M (g):	0.63
Magnitude Scaling Factor:	1.314
Historic High Groundwater:	20.0
Groundwater @ Exploration:	24.0

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	VOL. STRAIN [e_{15}] (%)	EQ. SETTLE. Pe (in.)
1.0	12	125.0	0.031	0.031	82	22	0.409	--	0.00	0.00
2.0	12	125.0	0.094	0.094	80	22	0.409	--	0.00	0.00
3.0	12	114.8	0.154	0.154	78	22	0.409	--	0.00	0.00
4.0	12	114.8	0.211	0.211	78	22	0.409	--	0.00	0.00
5.0	12	114.8	0.269	0.269	78	22	0.409	--	0.00	0.00
6.0	9	114.8	0.326	0.326	65	17	0.409	--	0.00	0.00
7.0	9	114.8	0.383	0.383	65	17	0.409	--	0.00	0.00
8.0	9	121.3	0.442	0.442	65	16	0.409	--	0.00	0.00
9.0	9	121.3	0.503	0.503	65	15	0.409	--	0.00	0.00
10.0	9	121.3	0.564	0.564	65	14	0.409	--	0.00	0.00
11.0	29	125.0	0.625	0.625	106	42	0.409	--	0.00	0.00
12.0	29	125.0	0.688	0.688	106	40	0.409	--	0.00	0.00
13.0	29	125.0	0.750	0.750	106	38	0.409	--	0.00	0.00
14.0	29	125.0	0.813	0.813	106	37	0.409	--	0.00	0.00
15.0	29	125.0	0.875	0.875	106	36	0.409	--	0.00	0.00
16.0	31	123.4	0.937	0.937	100	40	0.409	--	0.00	0.00
17.0	31	123.4	0.999	0.999	100	38	0.409	--	0.00	0.00
18.0	31	123.4	1.061	1.061	100	37	0.409	--	0.00	0.00
19.0	31	123.4	1.122	1.122	100	36	0.409	--	0.00	0.00
20.0	31	123.4	1.184	1.184	100	35	0.409	--	0.00	0.00
21.0	50	123.4	1.246	1.230	118	61	0.414	Non-Liq.	0.00	0.00
22.0	50	123.4	1.308	1.261	118	60	0.424	Non-Liq.	0.00	0.00
23.0	50	124.2	1.369	1.291	118	59	0.434	Non-Liq.	0.00	0.00
24.0	50	124.2	1.432	1.322	118	58	0.443	Non-Liq.	0.00	0.00
25.0	50	124.2	1.494	1.353	118	57	0.451	Non-Liq.	0.00	0.00
26.0	18	124.2	1.556	1.384	68	31	0.460	Non-Liq.	0.00	0.00
27.0	18	124.2	1.618	1.415	68	31	0.467	Non-Liq.	0.00	0.00
28.0	46	130.6	1.682	1.448	106	55	0.475	Non-Liq.	0.00	0.00
29.0	46	130.6	1.747	1.482	106	55	0.482	Non-Liq.	0.00	0.00
30.0	46	130.6	1.812	1.516	106	54	0.489	Non-Liq.	0.00	0.00
31.0	28	130.6	1.877	1.550	82	33	0.495	Non-Liq.	0.00	0.00
32.0	39	145.4	1.946	1.588	95	46	0.501	Non-Liq.	0.00	0.00
33.0	39	145.4	2.019	1.629	95	46	0.507	Non-Liq.	0.00	0.00
34.0	39	145.4	2.092	1.671	95	45	0.512	Non-Liq.	0.00	0.00
35.0	39	145.4	2.165	1.712	95	45	0.517	Non-Liq.	0.00	0.00
36.0	50	145.4	2.237	1.754	106	56	0.522	Non-Liq.	0.00	0.00
37.0	50	145.4	2.310	1.795	106	56	0.526	Non-Liq.	0.00	0.00
38.0	50	145.4	2.383	1.837	106	55	0.530	Non-Liq.	0.00	0.00
39.0	50	145.4	2.455	1.878	106	55	0.534	Non-Liq.	0.00	0.00
40.0	50	145.4	2.528	1.920	106	54	0.538	Non-Liq.	0.00	0.00
41.0	44	140.9	2.600	1.960	95	47	0.542	Non-Liq.	0.00	0.00
42.0	44	140.9	2.670	1.999	95	47	0.546	Non-Liq.	0.00	0.00
43.0	44	140.9	2.740	2.038	95	46	0.550	Non-Liq.	0.00	0.00
44.0	44	140.9	2.811	2.078	95	46	0.553	Non-Liq.	0.00	0.00
45.0	44	140.9	2.881	2.117	95	45	0.556	Non-Liq.	0.00	0.00
46.0	100	140.9	2.952	2.156	139	102	0.560	Non-Liq.	0.00	0.00
47.0	100	140.9	3.022	2.195	139	101	0.563	Non-Liq.	0.00	0.00
48.0	100	140.9	3.093	2.235	139	100	0.566	Non-Liq.	0.00	0.00
49.0	100	140.9	3.163	2.274	139	100	0.569	Non-Liq.	0.00	0.00
50.0	100	140.9	3.234	2.313	139	99	0.572	Non-Liq.	0.00	0.00
51.0	23	127.7	3.301	2.349	65	32	0.574	Non-Liq.	0.00	0.00
52.0	23	127.7	3.365	2.382	65	32	0.578	Non-Liq.	0.00	0.00
53.0	37	127.7	3.428	2.414	83	48	0.581	Non-Liq.	0.00	0.00
54.0	37	127.7	3.492	2.447	83	48	0.583	Non-Liq.	0.00	0.00
55.0	37	127.7	3.556	2.480	83	48	0.586	Non-Liq.	0.00	0.00
56.0	74	127.7	3.620	2.512	113	70	0.589	Non-Liq.	0.00	0.00
57.0	74	127.7	3.684	2.545	113	70	0.592	Non-Liq.	0.00	0.00
58.0	74	127.7	3.748	2.578	113	69	0.594	Non-Liq.	0.00	0.00
59.0	74	127.7	3.812	2.610	113	69	0.597	Non-Liq.	0.00	0.00
60.0	74	127.7	3.875	2.643	113	69	0.599	Non-Liq.	0.00	0.00
61.0	76	129.0	3.940	2.676	112	70	0.602	Non-Liq.	0.00	0.00
62.0	76	129.0	4.004	2.709	112	70	0.604	Non-Liq.	0.00	0.00
63.0	76	129.0	4.069	2.743	112	69	0.607	Non-Liq.	0.00	0.00
64.0	76	129.0	4.133	2.776	112	69	0.609	Non-Liq.	0.00	0.00
65.0	76	129.0	4.198	2.809	112	68	0.611	Non-Liq.	0.00	0.00

TOTAL SETTLEMENT = 0.0 INCHES

GEOCON

Project Name : Hope Village
Project No : W1815-06-01
Boring : B4

LIQUEFACTION SETTLEMENT ANALYSIS
DESIGN EARTHQUAKE

NCEER (1996) METHOD W 2001 UPDATES
EARTHQUAKE INFORMATION:

Earthquake Magnitude:	6.74
PGA_M (g):	0.943
2/3 PGA_M (g):	0.63
Magnitude Scaling Factor:	1.314
Historic High Groundwater:	20.0
Groundwater @ Exploration:	24.0

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	VOL. STRAIN [e_{15}] (%)	EQ. SETTLE. Pe (in.)
1.0	4	125.0	0.031	0.031	51	8	0.409	--	0.00	0.00
2.0	4	125.0	0.094	0.094	49	8	0.409	--	0.00	0.00
3.0	4	111.0	0.153	0.153	48	8	0.409	--	0.00	0.00
4.0	4	111.0	0.208	0.208	48	8	0.409	--	0.00	0.00
5.0	4	111.0	0.264	0.264	48	8	0.409	--	0.00	0.00
6.0	3	111.0	0.319	0.319	38	6	0.409	--	0.00	0.00
7.0	3	111.0	0.375	0.375	38	6	0.409	--	0.00	0.00
8.0	26	113.1	0.431	0.431	106	46	0.409	--	0.00	0.00
9.0	26	113.1	0.487	0.487	106	43	0.409	--	0.00	0.00
10.0	26	113.1	0.544	0.544	106	41	0.409	--	0.00	0.00
11.0	32	113.1	0.600	0.600	112	47	0.409	--	0.00	0.00
12.0	32	113.1	0.657	0.657	112	45	0.409	--	0.00	0.00
13.0	32	120.7	0.715	0.715	112	43	0.409	--	0.00	0.00
14.0	32	120.7	0.776	0.776	112	42	0.409	--	0.00	0.00
15.0	32	120.7	0.836	0.836	112	40	0.409	--	0.00	0.00
16.0	32	120.7	0.896	0.896	103	42	0.409	--	0.00	0.00
17.0	32	120.7	0.957	0.957	103	40	0.409	--	0.00	0.00
18.0	32	116.2	1.016	1.016	103	39	0.409	--	0.00	0.00
19.0	32	116.2	1.074	1.074	103	38	0.409	--	0.00	0.00
20.0	32	116.2	1.132	1.132	103	37	0.409	--	0.00	0.00
21.0	56	116.2	1.190	1.175	126	70	0.414	Non-Liq.	0.00	0.00
22.0	56	116.2	1.248	1.202	126	69	0.425	Non-Liq.	0.00	0.00
23.0	56	139.0	1.312	1.234	126	67	0.435	Non-Liq.	0.00	0.00
24.0	56	139.0	1.382	1.273	126	66	0.444	Non-Liq.	0.00	0.00
25.0	56	139.0	1.451	1.311	126	65	0.453	Non-Liq.	0.00	0.00
26.0	72	139.0	1.521	1.349	137	88	0.461	Non-Liq.	0.00	0.00
27.0	72	139.0	1.590	1.387	137	87	0.469	Non-Liq.	0.00	0.00
28.0	72	158.4	1.665	1.431	137	85	0.476	Non-Liq.	0.00	0.00
29.0	72	158.4	1.744	1.479	137	84	0.482	Non-Liq.	0.00	0.00
30.0	72	158.4	1.823	1.527	137	83	0.488	Non-Liq.	0.00	0.00
31.0	27	158.4	1.902	1.575	80	32	0.494	Non-Liq.	0.00	0.00
32.0	27	158.4	1.981	1.623	80	32	0.499	Non-Liq.	0.00	0.00
33.0	38	131.5	2.054	1.664	93	44	0.505	Non-Liq.	0.00	0.00
34.0	38	131.5	2.120	1.698	93	43	0.510	Non-Liq.	0.00	0.00
35.0	38	131.5	2.185	1.733	93	43	0.516	Non-Liq.	0.00	0.00
36.0	23	131.5	2.251	1.768	71	26	0.521	0.72	1.10	0.13
37.0	23	131.5	2.317	1.802	71	26	0.526	0.70	1.10	0.13
38.0	55	129.0	2.382	1.836	108	61	0.530	Non-Liq.	0.00	0.00
39.0	55	129.0	2.447	1.869	108	60	0.535	Non-Liq.	0.00	0.00
40.0	55	129.0	2.511	1.903	108	60	0.540	Non-Liq.	0.00	0.00
41.0	22	135.6	2.577	1.938	68	24	0.544	0.62	1.30	0.16
42.0	22	135.6	2.645	1.974	68	24	0.548	0.61	1.30	0.16
43.0	55	135.6	2.713	2.011	105	59	0.552	Non-Liq.	0.00	0.00
44.0	55	135.6	2.781	2.047	105	58	0.555	Non-Liq.	0.00	0.00
45.0	55	135.6	2.848	2.084	105	58	0.559	Non-Liq.	0.00	0.00
46.0	30	135.6	2.916	2.121	77	31	0.562	Non-Liq.	0.00	0.00
47.0	30	135.6	2.984	2.157	77	31	0.566	Non-Liq.	0.00	0.00
48.0	30	156.1	3.057	2.199	77	30	0.568	Non-Liq.	0.00	0.00
49.0	30	156.1	3.135	2.246	77	30	0.571	Non-Liq.	0.00	0.00
50.0	30	156.1	3.213	2.293	77	30	0.573	1.00	0.75	0.09
51.0	58	146.7	3.289	2.337	103	57	0.575	Non-Liq.	0.00	0.00
52.0	58	146.7	3.362	2.379	103	57	0.578	Non-Liq.	0.00	0.00
53.0	58	146.7	3.435	2.421	103	56	0.580	Non-Liq.	0.00	0.00
54.0	58	146.7	3.509	2.464	103	56	0.582	Non-Liq.	0.00	0.00
55.0	58	146.7	3.582	2.506	103	55	0.584	Non-Liq.	0.00	0.00
56.0	79	146.7	3.655	2.548	117	74	0.587	Non-Liq.	0.00	0.00
57.0	79	146.7	3.729	2.590	117	74	0.589	Non-Liq.	0.00	0.00
58.0	41	146.7	3.802	2.632	83	38	0.591	Non-Liq.	0.00	0.00
59.0	41	146.7	3.875	2.674	83	38	0.592	Non-Liq.	0.00	0.00
60.0	41	146.7	3.949	2.716	83	37	0.594	Non-Liq.	0.00	0.00
61.0	76	146.7	4.022	2.759	111	69	0.596	Non-Liq.	0.00	0.00
62.0	76	146.7	4.096	2.801	111	68	0.598	Non-Liq.	0.00	0.00
63.0	76	146.7	4.169	2.843	111	68	0.600	Non-Liq.	0.00	0.00
64.0	76	146.7	4.242	2.885	111	67	0.601	Non-Liq.	0.00	0.00
65.0	76	146.7	4.316	2.927	111	67	0.603	Non-Liq.	0.00	0.00
66.0	76	146.7	4.389	2.969	108	67	0.604	Non-Liq.	0.00	0.00
67.0	76	146.7	4.462	3.011	108	66	0.606	Non-Liq.	0.00	0.00
68.0	76	146.7	4.536	3.054	108	66	0.607	Non-Liq.	0.00	0.00
69.0	76	146.7	4.609	3.096	108	65	0.609	Non-Liq.	0.00	0.00
70.0	76	146.7	4.682	3.138	108	65	0.610	Non-Liq.	0.00	0.00
71.0	43	146.7	4.756	3.180	79	36	0.611	Non-Liq.	0.00	0.00
72.0	43	146.7	4.829	3.222	79	36	0.613	Non-Liq.	0.00	0.00
73.0	43	146.7	4.902	3.264	79	36	0.614	Non-Liq.	0.00	0.00
74.0	43	146.7	4.976	3.307	79	36	0.615	Non-Liq.	0.00	0.00
75.0	43	146.7	5.049	3.349	79	36	0.616	Non-Liq.	0.00	0.00
76.0	92	146.7	5.122	3.391	113	76	0.618	Non-Liq.	0.00	0.00
77.0	92	146.7	5.196	3.433	113	75	0.619	Non-Liq.	0.00	0.00
78.0	92	146.7	5.269	3.475	113	75	0.620	Non-Liq.	0.00	0.00
79.0	92	146.7	5.342	3.517	113	74	0.621	Non-Liq.	0.00	0.00
80.0	92	146.7	5.416	3.559	113	74	0.622	Non-Liq.	0.00	0.00

TOTAL SETTLEMENT = 0.7 INCHES

EMPIRICAL ESTIMATION OF LIQUEFACTION POTENTIAL
MAXIMUM CONSIDERED EARTHQUAKE

NCEER (1996) METHOD W 2001 UPDATES

EARTHQUAKE INFORMATION:

Earthquake Magnitude:	6.85
Peak Horiz. Acceleration PGA_M (g):	0.943
Magnitude Scaling Factor:	1.261
Historic High Groundwater:	20.0
Groundwater Depth During Exploration:	24.0

ENERGY & ROD CORRECTIONS:

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

LIQUEFACTION CALCULATIONS:

Unit Wt. Water (pcf):	62.4

Depth to Base (ft)	Total Unit Wt. (pcf)	Water (0 or 1)	Field SPT (N)	Depth of SPT (ft)	Liq.Sus. (0 or 1)	-200 (%)	Est. Dr (%)	CN Factor	Corrected (N1)60cs	Eff. Unit Wt. (psf)	Resist. CRR 7.5	rd Factor	Induced CSR	Liquefac. Safe.Fact.
1.0	125.0	0	11.6	1.0	1		82	1.700	22.1	125.0	0.243	1.000	0.613	--
2.0	125.0	0	11.6	2.0	1	0	80	1.700	22.1	125.0	0.243	0.998	0.612	--
3.0	114.8	0	11.6	2.5	1	0	78	1.700	22.1	114.8	0.243	0.996	0.611	--
4.0	114.8	0	11.6	2.5	1	0	78	1.700	22.1	114.8	0.243	0.994	0.609	--
5.0	114.8	0	11.6	2.5	1	0	78	1.700	22.1	114.8	0.243	0.991	0.608	--
6.0	114.8	0	9.0	5.0	1	0	65	1.700	17.2	114.8	0.183	0.989	0.606	--
7.0	114.8	0	9.0	5.0	1	0	65	1.650	16.7	114.8	0.178	0.987	0.605	--
8.0	121.3	0	9.0	5.0	1	0	65	1.536	15.6	121.3	0.166	0.985	0.603	--
9.0	121.3	0	9.0	5.0	1	0	65	1.441	14.6	121.3	0.156	0.982	0.602	--
10.0	121.3	0	9.0	5.0	1	0	65	1.361	13.8	121.3	0.148	0.980	0.601	--
11.0	125.0	0	29.0	10.0	1	0	106	1.292	42.2	125.0	Infin.	0.978	0.600	--
12.0	125.0	0	29.0	10.0	1	0	106	1.232	40.2	125.0	Infin.	0.976	0.598	--
13.0	125.0	0	29.0	10.0	1	0	106	1.180	38.5	125.0	Infin.	0.974	0.597	--
14.0	125.0	0	29.0	10.0	1	0	106	1.133	37.0	125.0	Infin.	0.972	0.596	--
15.0	125.0	0	29.0	10.0	1	0	106	1.092	35.6	125.0	Infin.	0.970	0.594	--
16.0	123.4	0	31.0	15.0	1	0	100	1.055	39.6	123.4	Infin.	0.967	0.593	--
17.0	123.4	0	31.0	15.0	1	0	100	1.022	38.3	123.4	Infin.	0.965	0.592	--
18.0	123.4	0	31.0	15.0	1	0	100	0.992	37.2	123.4	Infin.	0.963	0.590	--
19.0	123.4	0	31.0	15.0	1	0	100	0.964	36.2	123.4	Infin.	0.961	0.589	--
20.0	123.4	0	31.0	15.0	1	0	100	0.939	35.2	123.4	Infin.	0.958	0.587	--
21.0	123.4	1	50.0	20.0	1	0	118	0.915	61.4	61.0	Infin.	0.956	0.593	Non-Liq.
22.0	123.4	1	50.0	20.0	1	0	118	0.894	60.0	61.0	Infin.	0.953	0.606	Non-Liq.
23.0	124.2	1	50.0	20.0	1	0	118	0.873	58.6	61.8	Infin.	0.950	0.617	Non-Liq.
24.0	124.2	1	50.0	20.0	1	0	118	0.859	57.6	61.8	Infin.	0.947	0.628	Non-Liq.
25.0	124.2	1	50.0	20.0	1	0	118	0.849	57.0	61.8	Infin.	0.944	0.638	Non-Liq.
26.0	124.2	1	18.0	25.0	1	36	68	0.841	31.0	61.8	Infin.	0.940	0.648	Non-Liq.
27.0	124.2	1	18.0	25.0	1	36	68	0.832	30.7	61.8	Infin.	0.936	0.656	Non-Liq.
28.0	130.6	1	45.7	27.5	1	3	106	0.823	55.2	68.2	Infin.	0.932	0.664	Non-Liq.
29.0	130.6	1	45.7	27.5	1	3	106	0.814	54.6	68.2	Infin.	0.928	0.670	Non-Liq.
30.0	130.6	1	45.7	27.5	1	3	106	0.805	54.0	68.2	Infin.	0.923	0.676	Non-Liq.
31.0	130.6	1	28.0	30.0	1	3	82	0.797	33.5	68.2	Infin.	0.918	0.682	Non-Liq.
32.0	145.4	1	39.1	32.5	1	3	95	0.788	46.2	83.0	Infin.	0.912	0.686	Non-Liq.
33.0	145.4	1	39.1	32.5	1	3	95	0.778	45.6	83.0	Infin.	0.907	0.689	Non-Liq.
34.0	145.4	1	39.1	32.5	1	3	95	0.769	45.1	83.0	Infin.	0.900	0.691	Non-Liq.
35.0	145.4	1	39.1	32.5	1	0	95	0.760	44.5	83.0	Infin.	0.894	0.693	Non-Liq.
36.0	145.4	1	50.0	35.0	1	0	106	0.752	56.4	83.0	Infin.	0.887	0.694	Non-Liq.
37.0	145.4	1	50.0	35.0	1	0	106	0.743	55.8	83.0	Infin.	0.880	0.694	Non-Liq.
38.0	145.4	1	50.0	35.0	1	0	106	0.735	55.2	83.0	Infin.	0.872	0.693	Non-Liq.
39.0	145.4	1	50.0	35.0	1	0	106	0.728	54.6	83.0	Infin.	0.864	0.692	Non-Liq.
40.0	145.4	1	50.0	35.0	1	0	106	0.720	54.0	83.0	Infin.	0.855	0.690	Non-Liq.
41.0	140.9	1	44.0	40.0	1	0	95	0.713	47.1	78.5	Infin.	0.846	0.688	Non-Liq.
42.0	140.9	1	44.0	40.0	1	0	95	0.706	46.6	78.5	Infin.	0.837	0.685	Non-Liq.
43.0	140.9	1	44.0	40.0	1	0	95	0.700	46.2	78.5	Infin.	0.828	0.682	Non-Liq.
44.0	140.9	1	44.0	40.0	1	0	95	0.693	45.8	78.5	Infin.	0.818	0.678	Non-Liq.
45.0	140.9	1	44.0	40.0	1	0	95	0.687	45.4	78.5	Infin.	0.808	0.674	Non-Liq.
46.0	140.9	1	100.0	45.0	1	0	139	0.681	102.2	78.5	Infin.	0.798	0.670	Non-Liq.
47.0	140.9	1	100.0	45.0	1	0	139	0.675	101.3	78.5	Infin.	0.788	0.665	Non-Liq.
48.0	140.9	1	100.0	45.0	1	0	139	0.670	100.4	78.5	Infin.	0.778	0.660	Non-Liq.
49.0	140.9	1	100.0	45.0	1	0	139	0.664	99.6	78.5	Infin.	0.768	0.655	Non-Liq.
50.0	140.9	1	100.0	45.0	1	0	139	0.659	98.8	78.5	Infin.	0.757	0.649	Non-Liq.
51.0	127.7	1	23.0	50.0	1	81	65	0.654	32.1	65.3	Infin.	0.747	0.644	Non-Liq.
52.0	127.7	1	23.0	50.0	1	81	65	0.649	31.9	65.3	Infin.	0.737	0.638	Non-Liq.
53.0	127.7	1	37.4	50.0	1	81	83	0.645	48.4	65.3	Infin.	0.727	0.633	Non-Liq.
54.0	127.7	1	37.4	50.0	1	81	83	0.641	48.2	65.3	Infin.	0.717	0.628	Non-Liq.
55.0	127.7	1	37.4	50.0	1	81	83	0.637	47.9	65.3	Infin.	0.708	0.622	Non-Liq.
56.0	127.7	1	74.0	55.0	1	0	113	0.633	70.3	65.3	Infin.	0.698	0.617	Non-Liq.
57.0	127.7	1	74.0	55.0	1	0	113	0.629	69.8	65.3	Infin.	0.689	0.611	Non-Liq.
58.0	127.7	1	74.0	55.0	1	0	113	0.625	69.4	65.3	Infin.	0.680	0.606	Non-Liq.
59.0	127.7	1	74.0	55.0	1	0	113	0.621	69.0	65.3	Infin.	0.671	0.601	Non-Liq.
60.0	127.7	1	74.0	55.0	1	0	113	0.618	68.6	65.3	Infin.	0.663	0.596	Non-Liq.
61.0	129.0	1	76.0	60.0	1	0	112	0.614	70.0	66.6	Infin.	0.655	0.591	Non-Liq.
62.0	129.0	1	76.0	60.0	1	0	112	0.610	69.6	66.6	Infin.	0.647	0.586	Non-Liq.
63.0	129.0	1	76.0	60.0	1	0	112	0.607	69.2	66.6	Infin.	0.639	0.581	Non-Liq.
64.0	129.0	1	76.0	60.0	1	0	112	0.603	68.8	66.6	Infin.	0.632	0.577	Non-Liq.
65.0	129.0	1	76.0	60.0	1	0	112	0.600	68.4	66.6	Infin.	0.625	0.573	Non-Liq.

Project Name : Hope Village
Project No : W1815-06-01
Boring : B4

GEOCON

EMPIRICAL ESTIMATION OF LIQUEFACTION POTENTIAL
MAXIMUM CONSIDERED EARTHQUAKE

NCEER (1996) METHOD W 2001 UPDATES

EARTHQUAKE INFORMATION:

Earthquake Magnitude:	6.85
Peak Horiz. Acceleration PGA_M (g):	0.943
Magnitude Scaling Factor:	1.261
Historic High Groundwater:	20.0
Groundwater Depth During Exploration:	24.0

ENERGY & ROD CORRECTIONS:

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

LIQUEFACTION CALCULATIONS:

Unit Wt. Water (pcf):	62.4

Depth to Base (ft)	Total Unit Wt. (pcf)	Water (0 or 1)	Field SPT (N)	Depth of SPT (ft)	Liq.Sus. (0 or 1)	-200 (%)	Est. Dr (%)	CN Factor	Corrected (N1)60cs	Eff. Unit Wt. (psf)	Resist. CRR 7.5	rd Factor	Induced CSR	Liquefac. Safe.Fact.
1.0	125.0	0	4.4	1.0	1		51	1.700	8.4	125.0	0.099	1.000	0.613	--
2.0	125.0	0	4.4	2.0	1	0	49	1.700	8.4	125.0	0.099	0.998	0.612	--
3.0	111.0	0	4.4	2.5	1	0	48	1.700	8.4	111.0	0.099	0.996	0.611	--
4.0	111.0	0	4.4	2.5	1	0	48	1.700	8.4	111.0	0.099	0.994	0.609	--
5.0	111.0	0	4.4	2.5	1	0	48	1.700	8.4	111.0	0.099	0.991	0.608	--
6.0	111.0	0	3.0	5.0	1	0	38	1.700	5.7	111.0	0.078	0.989	0.606	--
7.0	111.0	0	3.0	5.0	1	0	38	1.669	5.6	111.0	0.077	0.987	0.605	--
8.0	113.1	0	26.4	7.5	1	0	106	1.557	46.2	113.1	Infin.	0.985	0.603	--
9.0	113.1	0	26.4	7.5	1	0	106	1.464	43.5	113.1	Infin.	0.982	0.602	--
10.0	113.1	0	26.4	7.5	1	0	106	1.385	41.1	113.1	Infin.	0.980	0.601	--
11.0	113.1	0	32.0	10.0	1	0	112	1.319	47.5	113.1	Infin.	0.978	0.600	--
12.0	113.1	0	32.0	10.0	1	0	112	1.261	45.4	113.1	Infin.	0.976	0.598	--
13.0	120.7	0	32.0	10.0	1	0	112	1.208	43.5	120.7	Infin.	0.974	0.597	--
14.0	120.7	0	32.0	10.0	1	0	112	1.160	41.8	120.7	Infin.	0.972	0.596	--
15.0	120.7	0	32.0	10.0	1	0	112	1.117	40.2	120.7	Infin.	0.970	0.594	--
16.0	120.7	0	32.0	15.0	1	0	103	1.079	41.8	120.7	Infin.	0.967	0.593	--
17.0	120.7	0	32.0	15.0	1	0	103	1.045	40.4	120.7	Infin.	0.965	0.592	--
18.0	116.2	0	32.0	15.0	1	0	103	1.014	39.2	116.2	Infin.	0.963	0.590	--
19.0	116.2	0	32.0	15.0	1	0	103	0.986	38.2	116.2	Infin.	0.961	0.589	--
20.0	116.2	0	32.0	15.0	1	0	103	0.960	37.2	116.2	Infin.	0.958	0.587	--
21.0	116.2	1	56.0	20.0	1	0	126	0.937	70.4	53.8	Infin.	0.956	0.593	Non-Liq.
22.0	116.2	1	56.0	20.0	1	0	126	0.914	68.7	53.8	Infin.	0.953	0.607	Non-Liq.
23.0	139.0	1	56.0	20.0	1	0	126	0.892	67.1	76.6	Infin.	0.950	0.619	Non-Liq.
24.0	139.0	1	56.0	20.0	1	0	126	0.874	65.7	76.6	Infin.	0.947	0.630	Non-Liq.
25.0	139.0	1	56.0	20.0	1	0	126	0.862	64.8	76.6	Infin.	0.944	0.640	Non-Liq.
26.0	139.0	1	72.0	25.0	1	0	137	0.851	87.8	76.6	Infin.	0.940	0.649	Non-Liq.
27.0	139.0	1	72.0	25.0	1	0	137	0.840	86.6	76.6	Infin.	0.936	0.658	Non-Liq.
28.0	158.4	1	72.0	25.0	1	0	137	0.828	85.4	96.0	Infin.	0.932	0.665	Non-Liq.
29.0	158.4	1	72.0	25.0	1	0	137	0.815	84.1	96.0	Infin.	0.928	0.671	Non-Liq.
30.0	158.4	1	72.0	25.0	1	0	137	0.803	82.8	96.0	Infin.	0.923	0.676	Non-Liq.
31.0	158.4	1	27.0	30.0	1	5	80	0.791	32.0	96.0	Infin.	0.918	0.680	Non-Liq.
32.0	158.4	1	27.0	30.0	1	5	80	0.780	31.6	96.0	Infin.	0.912	0.683	Non-Liq.
33.0	131.5	1	38.0	32.5	1	5	93	0.771	43.9	69.1	Infin.	0.907	0.686	Non-Liq.
34.0	131.5	1	38.0	32.5	1	5	93	0.763	43.4	69.1	Infin.	0.900	0.689	Non-Liq.
35.0	131.5	1	38.0	32.5	1	5	93	0.756	43.0	69.1	Infin.	0.894	0.691	Non-Liq.
36.0	131.5	1	23.0	35.0	1	4	71	0.749	25.8	69.1	0.309	0.887	0.692	0.46
37.0	131.5	1	23.0	35.0	1	4	71	0.742	25.6	69.1	0.304	0.880	0.693	0.45
38.0	129.0	1	55.0	37.5	1	4	108	0.736	60.7	66.6	Infin.	0.872	0.693	Non-Liq.
39.0	129.0	1	55.0	37.5	1	4	108	0.729	60.2	66.6	Infin.	0.864	0.693	Non-Liq.
40.0	129.0	1	55.0	37.5	1	4	108	0.723	59.7	66.6	Infin.	0.855	0.692	Non-Liq.
41.0	135.6	1	22.0	40.0	1	7	68	0.717	24.0	73.2	0.273	0.846	0.690	0.40
42.0	135.6	1	22.0	40.0	1	7	68	0.711	23.8	73.2	0.269	0.837	0.688	0.39
43.0	135.6	1	55.0	42.5	1	7	105	0.704	58.7	73.2	Infin.	0.828	0.685	Non-Liq.
44.0	135.6	1	55.0	42.5	1	7	105	0.698	58.2	73.2	Infin.	0.818	0.681	Non-Liq.
45.0	135.6	1	55.0	42.5	1	7	105	0.692	57.7	73.2	Infin.	0.808	0.677	Non-Liq.
46.0	135.6	1	30.0	45.0	1	0	77	0.687	30.9	73.2	Infin.	0.798	0.673	Non-Liq.
47.0	135.6	1	30.0	45.0	1	0	77	0.681	30.6	73.2	Infin.	0.788	0.668	Non-Liq.
48.0	156.1	1	30.0	45.0	1	0	77	0.675	30.4	93.7	Infin.	0.778	0.663	Non-Liq.
49.0	156.1	1	30.0	45.0	1	0	77	0.668	30.1	93.7	Infin.	0.768	0.657	Non-Liq.
50.0	156.1	1	30.0	45.0	1	0	77	0.661	29.8	93.7	0.452	0.757	0.651	0.64
51.0	146.7	1	58.0	50.0	1	0	103	0.655	57.0	84.3	Infin.	0.747	0.645	Non-Liq.
52.0	146.7	1	58.0	50.0	1	0	103	0.650	56.5	84.3	Infin.	0.737	0.639	Non-Liq.
53.0	146.7	1	58.0	50.0	1	0	103	0.644	56.1	84.3	Infin.	0.727	0.632	Non-Liq.
54.0	146.7	1	58.0	50.0	1	0	103	0.639	55.6	84.3	Infin.	0.717	0.626	Non-Liq.
55.0	146.7	1	58.0	50.0	1	0	103	0.634	55.1	84.3	Infin.	0.708	0.620	Non-Liq.
56.0	146.7	1	79.0	55.0	1	0	117	0.629	74.5	84.3	Infin.	0.698	0.614	Non-Liq.
57.0	146.7	1	79.0	55.0	1	0	117	0.624	73.9	84.3	Infin.	0.689	0.608	Non-Liq.
58.0	146.7	1	41.0	57.5	1	0	83	0.619	38.1	84.3	Infin.	0.680	0.602	Non-Liq.
59.0	146.7	1	41.0	57.5	1	0	83	0.614	37.8	84.3	Infin.	0.671	0.596	Non-Liq.
60.0	146.7	1	41.0	57.5	1	0	83	0.610	37.5	84.3	Infin.	0.663	0.591	Non-Liq.
61.0	146.7	1	76.0	60.0	1	0	111	0.605	69.0	84.3	Infin.	0.655	0.585	Non-Liq.
62.0	146.7	1	76.0	60.0	1	0	111	0.601	68.5	84.3	Infin.	0.647	0.580	Non-Liq.
63.0	146.7	1	76.0	60.0	1	0	111	0.596	68.0	84.3	Infin.	0.639	0.575	Non-Liq.
64.0	146.7	1	76.0	60.0	1	0	111	0.592	67.5	84.3	Infin.	0.632	0.570	Non-Liq.
65.0	146.7	1	76.0	60.0	1	0	111	0.588	67.0	84.3	Infin.	0.625	0.565	Non-Liq.
66.0	146.7	1	76.0	65.0	1	0	108	0.584	66.6	84.3	Infin.	0.618	0.560	Non-Liq.
67.0	146.7	1	76.0	65.0	1	0	108	0.580	66.1	84.3	Infin.	0.612	0.556	Non-Liq.
68.0	146.7	1	76.0	65.0	1	0	108	0.576	65.7	84.3	Infin.	0.606	0.552	Non-Liq.
69.0	146.7	1	76.0	65.0	1	0	108	0.572	65.2	84.3	Infin.	0.600	0.548	Non-Liq.
70.0	146.7	1	76.0	65.0	1	0	108	0.568	64.8	84.3	Infin.	0.594	0.544	Non-Liq.
71.0	146.7	1	43.0	70.0	1	0	79	0.565	36.4	84.3	Infin.	0.589	0.540	Non-Liq.
72.0	146.7	1	43.0	70.0	1	0	79	0.561	36.2	84.3	Infin.	0.584	0.536	Non-Liq.
73.0	146.7	1	43.0	70.0	1	0	79	0.558	36.0	84.3	Infin.	0.579	0.533	Non-Liq.
74.0	146.7	1	43.0	70.0	1	0	79	0.554	35.7	84.3	Infin.	0.574	0.530	Non-Liq.
75.0	146.7	1	43.0	70.0	1	0	79	0.551	35.5	84.3	Infin.	0.570	0.526	Non-Liq.
76.0	146.7	1	92.0	75.0	1	0	113	0.547	75.5	84.3	Infin.	0.565	0.523	Non-Liq.
77.0	146.7	1	92.0	75.0	1	0	113	0.544	75.1	84.3	Infin.	0.561	0.521	Non-Liq.
78.0	146.7	1	92.0	75.0	1	0	113	0.541	74.6	84.3	Infin.	0.557	0.518	Non-Liq.
79.0	146.7	1	92.0	75.0	1	0	113	0.538	74.2	84.3	Infin.	0.553	0.515	Non-Liq.
80.0	146.7	1	92.0	75.0	1	0	113	0.535	73.8	84.3	Infin.	0.550	0.513	Non-Liq.

GEOCON

Project Name : Hope Village
Project No : W1815-06-01
Boring : B1

LIQUEFACTION SETTLEMENT ANALYSIS
MAXIMUM CONSIDERED EARTHQUAKE

NCEER (1996) METHOD W 2001 UPDATES
EARTHQUAKE INFORMATION:

Earthquake Magnitude:	6.85
PGA_M (g):	0.943
Magnitude Scaling Factor:	1.261
Historic High Groundwater:	20.0
Groundwater @ Exploration:	24.0

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	VOL. STRAIN [e_{15}} (%)	EQ. SETTLE. Pe (in.)
1.0	12	125.0	0.031	0.031	82	22	0.613	--	0.00	0.00
2.0	12	125.0	0.094	0.094	80	22	0.613	--	0.00	0.00
3.0	12	114.8	0.154	0.154	78	22	0.613	--	0.00	0.00
4.0	12	114.8	0.211	0.211	78	22	0.613	--	0.00	0.00
5.0	12	114.8	0.269	0.269	78	22	0.613	--	0.00	0.00
6.0	9	114.8	0.326	0.326	65	17	0.613	--	0.00	0.00
7.0	9	114.8	0.383	0.383	65	17	0.613	--	0.00	0.00
8.0	9	121.3	0.442	0.442	65	16	0.613	--	0.00	0.00
9.0	9	121.3	0.503	0.503	65	15	0.613	--	0.00	0.00
10.0	9	121.3	0.564	0.564	65	14	0.613	--	0.00	0.00
11.0	29	125.0	0.625	0.625	106	42	0.613	--	0.00	0.00
12.0	29	125.0	0.688	0.688	106	40	0.613	--	0.00	0.00
13.0	29	125.0	0.750	0.750	106	38	0.613	--	0.00	0.00
14.0	29	125.0	0.813	0.813	106	37	0.613	--	0.00	0.00
15.0	29	125.0	0.875	0.875	106	36	0.613	--	0.00	0.00
16.0	31	123.4	0.937	0.937	100	40	0.613	--	0.00	0.00
17.0	31	123.4	0.999	0.999	100	38	0.613	--	0.00	0.00
18.0	31	123.4	1.061	1.061	100	37	0.613	--	0.00	0.00
19.0	31	123.4	1.122	1.122	100	36	0.613	--	0.00	0.00
20.0	31	123.4	1.184	1.184	100	35	0.613	--	0.00	0.00
21.0	50	123.4	1.246	1.230	118	61	0.621	Non-Liq.	0.00	0.00
22.0	50	123.4	1.308	1.261	118	60	0.636	Non-Liq.	0.00	0.00
23.0	50	124.2	1.369	1.291	118	59	0.650	Non-Liq.	0.00	0.00
24.0	50	124.2	1.432	1.322	118	58	0.664	Non-Liq.	0.00	0.00
25.0	50	124.2	1.494	1.353	118	57	0.677	Non-Liq.	0.00	0.00
26.0	18	124.2	1.556	1.384	68	31	0.689	Non-Liq.	0.00	0.00
27.0	18	124.2	1.618	1.415	68	31	0.701	Non-Liq.	0.00	0.00
28.0	46	130.6	1.682	1.448	106	55	0.712	Non-Liq.	0.00	0.00
29.0	46	130.6	1.747	1.482	106	55	0.723	Non-Liq.	0.00	0.00
30.0	46	130.6	1.812	1.516	106	54	0.733	Non-Liq.	0.00	0.00
31.0	28	130.6	1.877	1.550	82	33	0.743	Non-Liq.	0.00	0.00
32.0	39	145.4	1.946	1.588	95	46	0.751	Non-Liq.	0.00	0.00
33.0	39	145.4	2.019	1.629	95	46	0.760	Non-Liq.	0.00	0.00
34.0	39	145.4	2.092	1.671	95	45	0.767	Non-Liq.	0.00	0.00
35.0	39	145.4	2.165	1.712	95	45	0.775	Non-Liq.	0.00	0.00
36.0	50	145.4	2.237	1.754	106	56	0.782	Non-Liq.	0.00	0.00
37.0	50	145.4	2.310	1.795	106	56	0.789	Non-Liq.	0.00	0.00
38.0	50	145.4	2.383	1.837	106	55	0.795	Non-Liq.	0.00	0.00
39.0	50	145.4	2.455	1.878	106	55	0.801	Non-Liq.	0.00	0.00
40.0	50	145.4	2.528	1.920	106	54	0.807	Non-Liq.	0.00	0.00
41.0	44	140.9	2.600	1.960	95	47	0.813	Non-Liq.	0.00	0.00
42.0	44	140.9	2.670	1.999	95	47	0.819	Non-Liq.	0.00	0.00
43.0	44	140.9	2.740	2.038	95	46	0.824	Non-Liq.	0.00	0.00
44.0	44	140.9	2.811	2.078	95	46	0.829	Non-Liq.	0.00	0.00
45.0	44	140.9	2.881	2.117	95	45	0.834	Non-Liq.	0.00	0.00
46.0	100	140.9	2.952	2.156	139	102	0.839	Non-Liq.	0.00	0.00
47.0	100	140.9	3.022	2.195	139	101	0.844	Non-Liq.	0.00	0.00
48.0	100	140.9	3.093	2.235	139	100	0.848	Non-Liq.	0.00	0.00
49.0	100	140.9	3.163	2.274	139	100	0.853	Non-Liq.	0.00	0.00
50.0	100	140.9	3.234	2.313	139	99	0.857	Non-Liq.	0.00	0.00
51.0	23	127.7	3.301	2.349	65	32	0.861	Non-Liq.	0.00	0.00
52.0	23	127.7	3.365	2.382	65	32	0.866	Non-Liq.	0.00	0.00
53.0	37	127.7	3.428	2.414	83	48	0.870	Non-Liq.	0.00	0.00
54.0	37	127.7	3.492	2.447	83	48	0.875	Non-Liq.	0.00	0.00
55.0	37	127.7	3.556	2.480	83	48	0.879	Non-Liq.	0.00	0.00
56.0	74	127.7	3.620	2.512	113	70	0.883	Non-Liq.	0.00	0.00
57.0	74	127.7	3.684	2.545	113	70	0.887	Non-Liq.	0.00	0.00
58.0	74	127.7	3.748	2.578	113	69	0.891	Non-Liq.	0.00	0.00
59.0	74	127.7	3.812	2.610	113	69	0.895	Non-Liq.	0.00	0.00
60.0	74	127.7	3.875	2.643	113	69	0.899	Non-Liq.	0.00	0.00
61.0	76	129.0	3.940	2.676	112	70	0.902	Non-Liq.	0.00	0.00
62.0	76	129.0	4.004	2.709	112	70	0.906	Non-Liq.	0.00	0.00
63.0	76	129.0	4.069	2.743	112	69	0.909	Non-Liq.	0.00	0.00
64.0	76	129.0	4.133	2.776	112	69	0.913	Non-Liq.	0.00	0.00
65.0	76	129.0	4.198	2.809	112	68	0.916	Non-Liq.	0.00	0.00

TOTAL SETTLEMENT = 0.0 INCHES

GEOCON

Project Name : Hope Village
Project No : W1815-06-01
Boring : B4

LIQUEFACTION SETTLEMENT ANALYSIS
DESIGN EARTHQUAKE

NCEER (1996) METHOD W 2001 UPDATES
EARTHQUAKE INFORMATION:

Earthquake Magnitude:	6.74
PGA_M (g):	0.943
2/3 PGA_M (g):	0.63
Magnitude Scaling Factor:	1.314
Historic High Groundwater:	20.0
Groundwater @ Exploration:	24.0

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	VOL. STRAIN $[e_{15}\}$ (%)	EQ. SETTLE. Pe (in.)
1.0	4	125.0	0.031	0.031	51	8	0.409	--	0.00	0.00
2.0	4	125.0	0.094	0.094	49	8	0.409	--	0.00	0.00
3.0	4	111.0	0.153	0.153	48	8	0.409	--	0.00	0.00
4.0	4	111.0	0.208	0.208	48	8	0.409	--	0.00	0.00
5.0	4	111.0	0.264	0.264	48	8	0.409	--	0.00	0.00
6.0	3	111.0	0.319	0.319	38	6	0.409	--	0.00	0.00
7.0	3	111.0	0.375	0.375	38	6	0.409	--	0.00	0.00
8.0	26	113.1	0.431	0.431	106	46	0.409	--	0.00	0.00
9.0	26	113.1	0.487	0.487	106	43	0.409	--	0.00	0.00
10.0	26	113.1	0.544	0.544	106	41	0.409	--	0.00	0.00
11.0	32	113.1	0.600	0.600	112	47	0.409	--	0.00	0.00
12.0	32	113.1	0.657	0.657	112	45	0.409	--	0.00	0.00
13.0	32	120.7	0.715	0.715	112	43	0.409	--	0.00	0.00
14.0	32	120.7	0.776	0.776	112	42	0.409	--	0.00	0.00
15.0	32	120.7	0.836	0.836	112	40	0.409	--	0.00	0.00
16.0	32	120.7	0.896	0.896	103	42	0.409	--	0.00	0.00
17.0	32	120.7	0.957	0.957	103	40	0.409	--	0.00	0.00
18.0	32	116.2	1.016	1.016	103	39	0.409	--	0.00	0.00
19.0	32	116.2	1.074	1.074	103	38	0.409	--	0.00	0.00
20.0	32	116.2	1.132	1.132	103	37	0.409	--	0.00	0.00
21.0	56	116.2	1.190	1.175	126	70	0.414	Non-Liq.	0.00	0.00
22.0	56	116.2	1.248	1.202	126	69	0.425	Non-Liq.	0.00	0.00
23.0	56	139.0	1.312	1.234	126	67	0.435	Non-Liq.	0.00	0.00
24.0	56	139.0	1.382	1.273	126	66	0.444	Non-Liq.	0.00	0.00
25.0	56	139.0	1.451	1.311	126	65	0.453	Non-Liq.	0.00	0.00
26.0	72	139.0	1.521	1.349	137	88	0.461	Non-Liq.	0.00	0.00
27.0	72	139.0	1.590	1.387	137	87	0.469	Non-Liq.	0.00	0.00
28.0	72	158.4	1.665	1.431	137	85	0.476	Non-Liq.	0.00	0.00
29.0	72	158.4	1.744	1.479	137	84	0.482	Non-Liq.	0.00	0.00
30.0	72	158.4	1.823	1.527	137	83	0.488	Non-Liq.	0.00	0.00
31.0	27	158.4	1.902	1.575	80	32	0.494	Non-Liq.	0.00	0.00
32.0	27	158.4	1.981	1.623	80	32	0.499	Non-Liq.	0.00	0.00
33.0	38	131.5	2.054	1.664	93	44	0.505	Non-Liq.	0.00	0.00
34.0	38	131.5	2.120	1.698	93	43	0.510	Non-Liq.	0.00	0.00
35.0	38	131.5	2.185	1.733	93	43	0.516	Non-Liq.	0.00	0.00
36.0	23	131.5	2.251	1.768	71	26	0.521	0.72	1.10	0.13
37.0	23	131.5	2.317	1.802	71	26	0.526	0.70	1.10	0.13
38.0	55	129.0	2.382	1.836	108	61	0.530	Non-Liq.	0.00	0.00
39.0	55	129.0	2.447	1.869	108	60	0.535	Non-Liq.	0.00	0.00
40.0	55	129.0	2.511	1.903	108	60	0.540	Non-Liq.	0.00	0.00
41.0	22	135.6	2.577	1.938	68	24	0.544	0.62	1.30	0.16
42.0	22	135.6	2.645	1.974	68	24	0.548	0.61	1.30	0.16
43.0	55	135.6	2.713	2.011	105	59	0.552	Non-Liq.	0.00	0.00
44.0	55	135.6	2.781	2.047	105	58	0.555	Non-Liq.	0.00	0.00
45.0	55	135.6	2.848	2.084	105	58	0.559	Non-Liq.	0.00	0.00
46.0	30	135.6	2.916	2.121	77	31	0.562	Non-Liq.	0.00	0.00
47.0	30	135.6	2.984	2.157	77	31	0.566	Non-Liq.	0.00	0.00
48.0	30	156.1	3.057	2.199	77	30	0.568	Non-Liq.	0.00	0.00
49.0	30	156.1	3.135	2.246	77	30	0.571	Non-Liq.	0.00	0.00
50.0	30	156.1	3.213	2.293	77	30	0.573	1.00	0.75	0.09
51.0	58	146.7	3.289	2.337	103	57	0.575	Non-Liq.	0.00	0.00
52.0	58	146.7	3.362	2.379	103	57	0.578	Non-Liq.	0.00	0.00
53.0	58	146.7	3.435	2.421	103	56	0.580	Non-Liq.	0.00	0.00
54.0	58	146.7	3.509	2.464	103	56	0.582	Non-Liq.	0.00	0.00
55.0	58	146.7	3.582	2.506	103	55	0.584	Non-Liq.	0.00	0.00
56.0	79	146.7	3.655	2.548	117	74	0.587	Non-Liq.	0.00	0.00
57.0	79	146.7	3.729	2.590	117	74	0.589	Non-Liq.	0.00	0.00
58.0	41	146.7	3.802	2.632	83	38	0.591	Non-Liq.	0.00	0.00
59.0	41	146.7	3.875	2.674	83	38	0.592	Non-Liq.	0.00	0.00
60.0	41	146.7	3.949	2.716	83	37	0.594	Non-Liq.	0.00	0.00
61.0	76	146.7	4.022	2.759	111	69	0.596	Non-Liq.	0.00	0.00
62.0	76	146.7	4.096	2.801	111	68	0.598	Non-Liq.	0.00	0.00
63.0	76	146.7	4.169	2.843	111	68	0.600	Non-Liq.	0.00	0.00
64.0	76	146.7	4.242	2.885	111	67	0.601	Non-Liq.	0.00	0.00
65.0	76	146.7	4.316	2.927	111	67	0.603	Non-Liq.	0.00	0.00
66.0	76	146.7	4.389	2.969	108	67	0.604	Non-Liq.	0.00	0.00
67.0	76	146.7	4.462	3.011	108	66	0.606	Non-Liq.	0.00	0.00
68.0	76	146.7	4.536	3.054	108	66	0.607	Non-Liq.	0.00	0.00
69.0	76	146.7	4.609	3.096	108	65	0.609	Non-Liq.	0.00	0.00
70.0	76	146.7	4.682	3.138	108	65	0.610	Non-Liq.	0.00	0.00
71.0	43	146.7	4.756	3.180	79	36	0.611	Non-Liq.	0.00	0.00
72.0	43	146.7	4.829	3.222	79	36	0.613	Non-Liq.	0.00	0.00
73.0	43	146.7	4.902	3.264	79	36	0.614	Non-Liq.	0.00	0.00
74.0	43	146.7	4.976	3.307	79	36	0.615	Non-Liq.	0.00	0.00
75.0	43	146.7	5.049	3.349	79	36	0.616	Non-Liq.	0.00	0.00
76.0	92	146.7	5.122	3.391	113	76	0.618	Non-Liq.	0.00	0.00
77.0	92	146.7	5.196	3.433	113	75	0.619	Non-Liq.	0.00	0.00
78.0	92	146.7	5.269	3.475	113	75	0.620	Non-Liq.	0.00	0.00
79.0	92	146.7	5.342	3.517	113	74	0.621	Non-Liq.	0.00	0.00
80.0	92	146.7	5.416	3.559	113	74	0.622	Non-Liq.	0.00	0.00

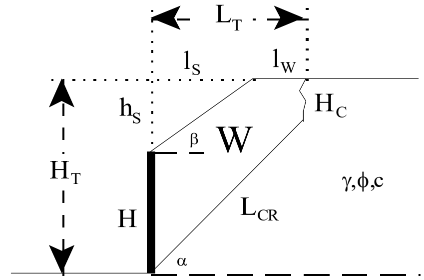
TOTAL SETTLEMENT = 0.7 INCHES

Figure 12

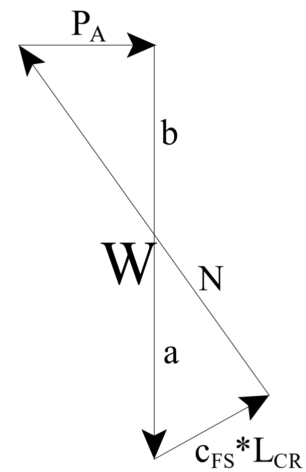
Retaining Wall Design with Transitioned Backfill
(Vector Analysis)

Input:

Retaining Wall Height	(H)	15.00	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)	15.0	feet
Unit Weight of Retained Soils	(g)	125.0	pcf
Friction Angle of Retained Soils	(f)	39.0	degrees
Cohesion of Retained Soils	(c)	29.0	psf
Factor of Safety	(FS)	1.50	
Factored Parameters	(f_{FS})	28.4	degrees
	(c_{FS})	19.3	psf



Failure Angle (α) 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	0.7	112	14034	20	1204	12830	3834
46	0.6	108	13555	20	1120	12434	3953
47	0.6	105	13091	20	1046	12044	4062
48	0.6	101	12641	19	981	11661	4161
49	0.6	98	12206	19	922	11284	4250
50	0.6	94	11783	19	869	10914	4329
51	0.6	91	11372	19	821	10550	4400
52	0.6	88	10972	18	778	10194	4462
53	0.5	85	10583	18	739	9844	4515
54	0.5	82	10204	18	703	9501	4560
55	0.5	79	9834	18	670	9164	4597
56	0.5	76	9474	17	640	8833	4625
57	0.5	73	9121	17	613	8508	4646
58	0.5	70	8777	17	587	8189	4659
59	0.5	68	8440	17	564	7875	4664
60	0.5	65	8109	17	542	7567	4662
61	0.5	62	7786	17	522	7263	4652
62	0.5	60	7468	16	504	6965	4634
63	0.5	57	7156	16	486	6670	4608
64	0.5	55	6850	16	470	6380	4574
65	0.5	52	6549	16	455	6094	4532
66	0.5	50	6253	16	441	5812	4482
67	0.6	48	5961	16	427	5533	4423
68	0.6	45	5673	16	415	5258	4356
69	0.6	43	5390	15	403	4987	4280
70	0.6	41	5110	15	392	4718	4194



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

$P_{A,\,max}$ 4664 lbs/lineal foot

Equivalent Fluid Pressure (per lineal foot of wall)

$EFP = 2*P_A/H^2$

EFP 41.5 pcf 46.3 pcf

Design Wall for an Equivalent Fluid Pressure **41 pcf** **47** pcf

Active At-Rest

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500 N. VICTORY BOULEVARD - BURBANK, CA 91502

PHONE: 818-841-8388 FAX: 818-841-1704

RETAINING WALL PRESSURE CALCULATION

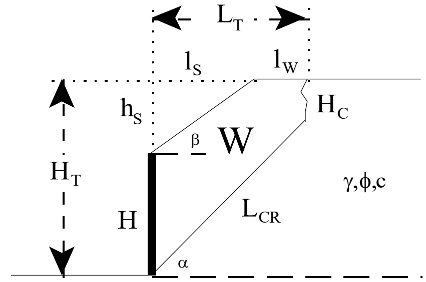
800 & 908 NORTH MAIN STREET
1081 & 1087 NORTH VIGNES STREET
LOS ANGELES, CALIFORNIA

DRAFTED BY: JJK	CHECKED BY: HHD	OCT. 2023	PROJECT NO. W1815-06-01	FIG. 13A

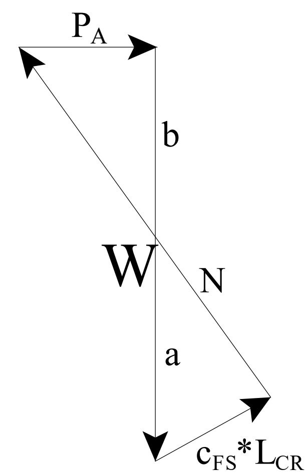
Retaining Wall Design with Transitioned Backfill
(Vector Analysis)

Input:

Retaining Wall Height	(H)	30.00	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)	30.0	feet
Unit Weight of Retained Soils	(g)	125.0	pcf
Friction Angle of Retained Soils	(f)	34.0	degrees
Cohesion of Retained Soils	(c)	6.0	psf
Factor of Safety	(FS)	1.50	
Factored Parameters	(f_{FS})	24.2	degrees
	(c_{FS})	4.0	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	0.1	450	56249	42	434	55815	21189
46	0.1	435	54319	42	408	53911	21550
47	0.1	420	52453	41	385	52068	21875
48	0.1	405	50647	40	364	50283	22165
49	0.1	391	48897	40	345	48552	22422
50	0.1	378	47199	39	327	46872	22646
51	0.1	364	45550	38	311	45238	22840
52	0.1	352	43947	38	297	43650	23002
53	0.1	339	42387	37	284	42103	23135
54	0.1	327	40868	37	271	40596	23238
55	0.1	315	39386	37	260	39126	23313
56	0.1	304	37941	36	250	37691	23358
57	0.1	292	36529	36	240	36289	23376
58	0.1	281	35149	35	231	34917	23364
59	0.1	270	33798	35	223	33575	23325
60	0.1	260	32476	35	215	32260	23256
61	0.1	249	31180	34	208	30971	23159
62	0.1	239	29908	34	202	29707	23033
63	0.1	229	28660	34	195	28465	22877
64	0.1	219	27435	33	190	27245	22690
65	0.1	210	26229	33	184	26045	22472
66	0.1	200	25044	33	179	24865	22222
67	0.1	191	23876	32	174	23702	21939
68	0.1	182	22726	32	170	22556	21621
69	0.1	173	21592	32	166	21426	21268
70	0.1	164	20473	32	162	20311	20878



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

$P_{A,\ max}$ 23376 lbs/lineal foot

Equivalent Fluid Pressure (per lineal foot of wall)

$EFP = 2*P_A/H^2$

EFP 51.9 pcf 55.1 pcf

Design Wall for an Equivalent Fluid Pressure **52 pcf** (Active) **56** pcf (At-Rest)

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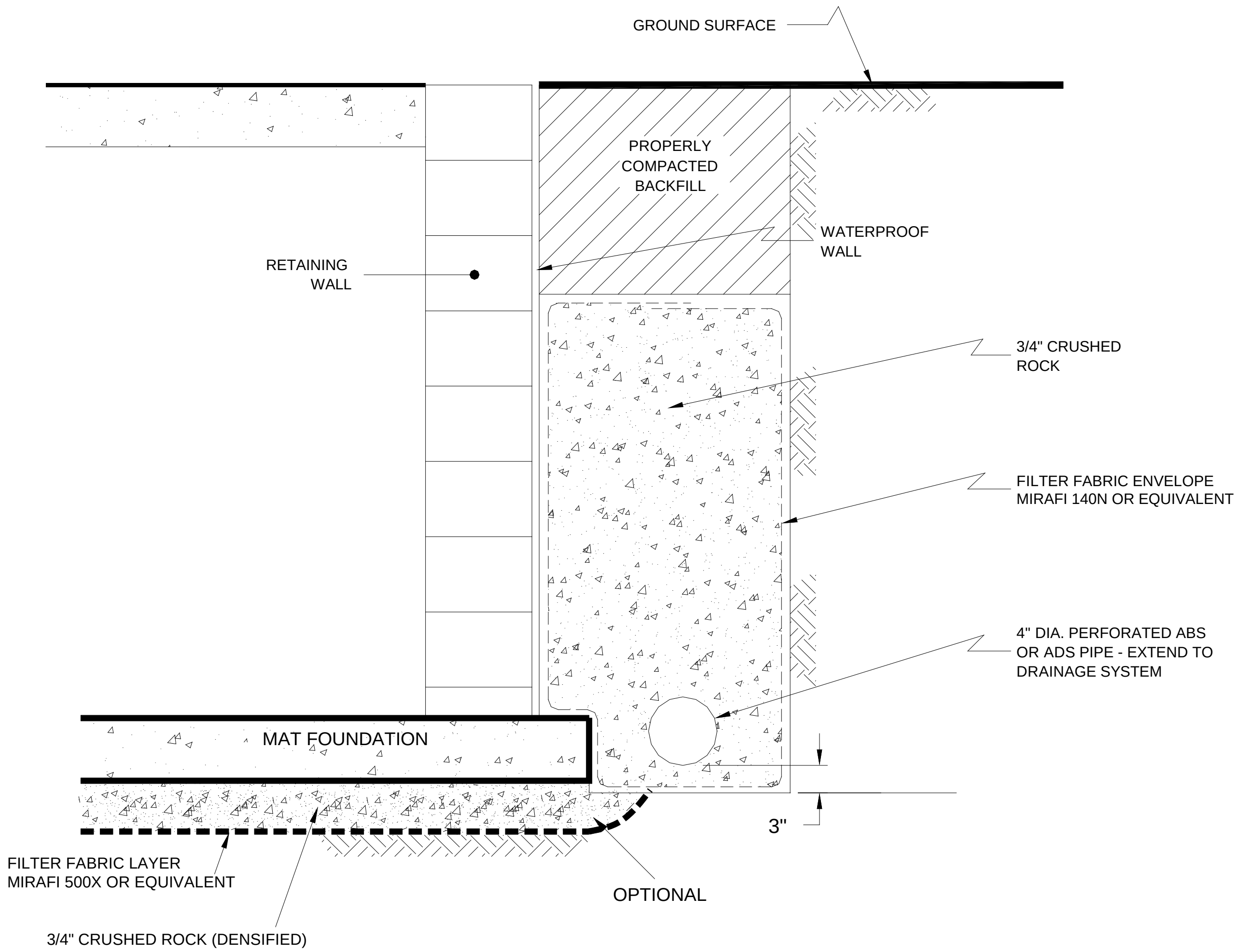
RETAINING WALL PRESSURE CALCULATION

800 & 908 NORTH MAIN STREET
1081 & 1087 NORTH VIGNES STREET
LOS ANGELES, CALIFORNIA

DRAFTED BY: JJK	CHECKED BY: HHD	OCT. 2023	PROJECT NO. W1815-06-01	FIG. 13B



NO SCALE

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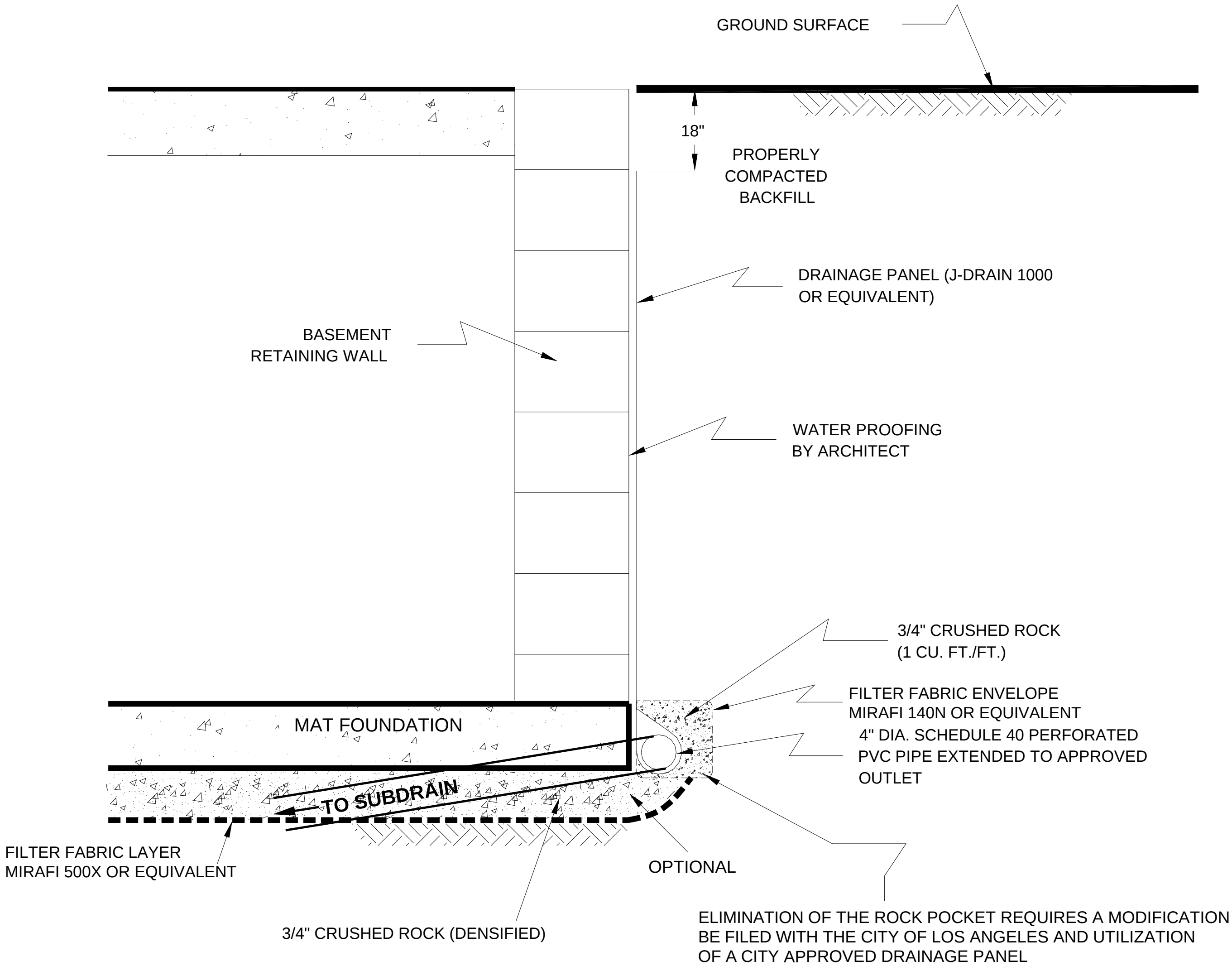
RETAINING WALL DRAIN DETAIL

800 & 908 NORTH MAIN STREET
1081 & 1087 NORTH VIGNES STREET
LOS ANGELES, CALIFORNIA

DRAFTED BY: JJK	CHECKED BY: HHD	OCT. 2023	PROJECT NO. W1815-06-01	FIG. 14



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

NO SCALE

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RETAINING WALL DRAIN DETAIL

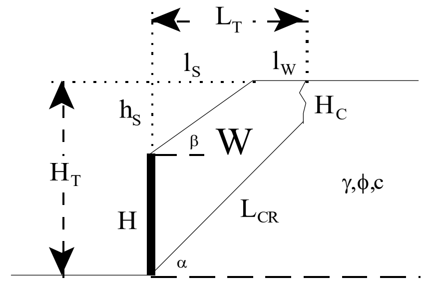
800 & 908 NORTH MAIN STREET
1081 & 1087 NORTH VIGNES STREET
LOS ANGELES, CALIFORNIA

DRAFTED BY: JJK	CHECKED BY: HHD	OCT. 2023	PROJECT NO. W1815-06-01	FIG. 15

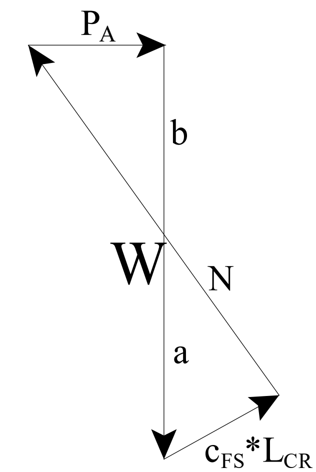
Shoring Design with Transitioned Backfill
(Vector Analysis)

Input:

Shoring Height	(H)	17.00 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)	17.0 feet
Unit Weight of Retained Soils	(g)	125.0 pcf
Friction Angle of Retained Soils	(f)	39.0 degrees
Cohesion of Retained Soils	(c)	29.0 psf
Factor of Safety	(FS)	1.25
Factored Parameters	(f_{FS})	32.9 degrees
	(c_{FS})	23.2 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	1.1	144	17993	23	2101	15892	3396
46	1.0	139	17383	22	1917	15466	3589
47	0.9	134	16792	22	1760	15033	3766
48	0.9	130	16218	22	1624	14595	3928
49	0.9	125	15661	21	1505	14156	4076
50	0.8	121	15120	21	1401	13719	4211
51	0.8	117	14594	21	1309	13285	4333
52	0.8	113	14083	21	1227	12855	4442
53	0.8	109	13584	20	1155	12430	4540
54	0.7	105	13098	20	1089	12009	4625
55	0.7	101	12625	20	1030	11595	4700
56	0.7	97	12162	20	977	11185	4763
57	0.7	94	11710	19	928	10782	4815
58	0.7	90	11268	19	884	10384	4856
59	0.7	87	10835	19	843	9992	4887
60	0.7	83	10411	19	806	9605	4908
61	0.7	80	9996	19	772	9224	4918
62	0.7	77	9588	18	741	8848	4917
63	0.7	74	9188	18	712	8477	4907
64	0.7	70	8795	18	685	8110	4885
65	0.7	67	8409	18	660	7749	4854
66	0.7	64	8028	18	637	7392	4812
67	0.7	61	7654	18	615	7039	4759
68	0.7	58	7284	18	595	6690	4695
69	0.7	55	6920	17	576	6344	4620
70	0.8	52	6561	17	558	6003	4534



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

$P_{A,\ max}$ 4918 lbs/lineal foot

Equivalent Fluid Pressure (per lineal foot of shoring)

$EFP = 2*P_A/H^2$

EFP 34.0 pcf

Design Shoring for an Equivalent Fluid Pressure **34 pcf**

Active

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SHORING WALL PRESSURE CALCULATION

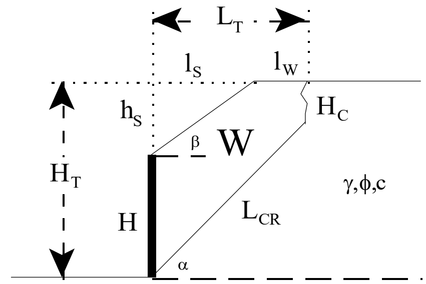
800 & 908 NORTH MAIN STREET
1081 & 1087 NORTH VIGNES STREET
LOS ANGELES, CALIFORNIA

DRAFTED BY: JJK	CHECKED BY: HHD	OCT. 2023	PROJECT NO. W1815-06-01	FIG. 16A

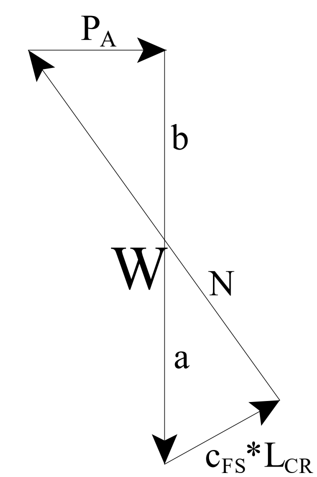
Shoring Design with Transitioned Backfill
(Vector Analysis)

Input:

Shoring Height	(H)	34.00 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)	34.0 feet
Unit Weight of Retained Soils	(g)	125.0 pcf
Friction Angle of Retained Soils	(f)	34.0 degrees
Cohesion of Retained Soils	(c)	6.0 psf
Factor of Safety	(FS)	1.25
Factored Parameters	(f_{FS})	28.4 degrees
	(c_{FS})	4.8 psf



Failure Angle (a)	Height of Tension Crack (H_C)	Area of Wedge (A)	Weight of Wedge (W)	Length of Failure Plane (L_{CR})	a	b	Active Pressure (P_A)
degrees	feet	feet^2	lbs/lineal foot	feet	lbs/lineal foot	lbs/lineal foot	lbs/lineal foot
45	0.2	578	72248	48	705	71543	21394
46	0.2	558	69769	47	655	69114	21989
47	0.2	539	67373	46	611	66761	22531
48	0.2	520	65053	46	572	64481	23022
49	0.1	502	62805	45	537	62267	23465
50	0.1	485	60624	44	506	60118	23861
51	0.1	468	58506	44	478	58028	24212
52	0.1	452	56447	43	453	55994	24520
53	0.1	436	54443	42	430	54014	24785
54	0.1	420	52492	42	409	52083	25008
55	0.1	405	50589	41	389	50200	25191
56	0.1	390	48733	41	372	48361	25334
57	0.1	375	46919	40	356	46563	25438
58	0.1	361	45146	40	341	44805	25503
59	0.1	347	43412	40	327	43084	25529
60	0.1	334	41713	39	315	41398	25517
61	0.1	320	40048	39	303	39745	25465
62	0.1	307	38415	38	292	38123	25375
63	0.1	295	36813	38	282	36530	25246
64	0.1	282	35238	38	273	34965	25077
65	0.1	270	33690	37	264	33426	24868
66	0.1	257	32167	37	256	31911	24618
67	0.1	245	30668	37	249	30419	24325
68	0.1	234	29190	37	242	28949	23990
69	0.1	222	27734	36	235	27499	23609
70	0.1	210	26296	36	229	26067	23183



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

$P_{A,\,max}$ 25529 lbs/lineal foot

Equivalent Fluid Pressure (per lineal foot of shoring)

$EFP = 2*P_A/H^2$

EFP 44.2 pcf

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

Active

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SHORING WALL PRESSURE CALCULATION

800 & 908 NORTH MAIN STREET
1081 & 1087 NORTH VIGNES STREET
LOS ANGELES, CALIFORNIA

DRAFTED BY: JJK	CHECKED BY: HHD	OCT. 2023	PROJECT NO. W1815-06-01	FIG. 16B

APPENDIX A

APPENDIX A

FIELD INVESTIGATION

The site was explored on September 6 through 8, 2023, by excavating four 7-inch diameter borings to between depths of approximately 50½ and 80½ feet below the existing ground surface using a truck-mounted hollow-stem auger drilling machine. Representative and relatively undisturbed samples were obtained by driving a 3-inch O. D., California Modified Sampler into the "undisturbed" soil mass with blows from a 140-pound auto-hammer falling 30 inches. The California Modified Sampler was equipped with 1-inch high by $2^{3}/_{8}$-inch diameter brass rings to facilitate soil removal and testing. Standard Penetration Tests were performed, and bulk samples were obtained.

The soil conditions encountered in the boring was 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 interpretations of the conditions between sampling intervals. Therefore, the logs contain both observed and interpreted data. We determined the lines designating the interface between soil materials on the log using visual observations, penetration rates, excavation characteristics and other factors. The transition between materials may be abrupt or gradual. Where applicable, the logs were revised based on subsequent laboratory testing. The approximate locations of the borings are shown on Figure 2.

PROJECT NO. W1815-06-01

BORING 1

ELEV. (MSL.) - - DATE COMPLETED 09/08/2023

EQUIPMENT HOLLOW STEM AUGER BY: JJK

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0					**AC: 4" BASE: 6"** **ARTIFICIAL FILL** Sandy Silt, medium dense, moist, black, fine-grained.			
2	B1@2 1/2'					21	104.7	9.6
4								
	B1@5'				- melted metal or glass	9		
6								
8	B1@7 1/2'				Silt with Sand, soft, moist to very moist, brown and reddish brown, some fine- to medium-grained.	8	100.2	21.1
10	B1@10' BULK 10-20'			SP	**ALLUVIUM** Sand, poorly graded, coarse gravel fragments.	29		
12								
14								
16	B1@15'			SP-SM	Sand with Silt, poorly graded, dense, moist, olive gray, fine-grained, some medium- to coarse-grained and fine gravel, trace silt.	31		
18	B1@17 1/2'				- very dense, trace coarse gravel	50 (6")	119.0	3.7
20	B1@20'				- brown, increase in coarse gravel	50 (3")		
22	B1@22 1/2'			ML	Sandy Silt, very moist, gray to bluish gray, some fine-grained sand.	14	105.1	18.2
24			▼					
	B1@25'			SM	Silty Sand, medium dense, very moist to wet, gray, fine-grained, some medium-grained.	18		
26				SP	Sand, poorly graded, very dense, wet, gray, fine-grained.			
28	B1@27 1/2'					50 (5")	115.3	13.3

W1815-06-01 BORING LOGS.GPJ

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.

GEOCON

PROJECT NO. W1815-06-01

BORING 1

ELEV. (MSL.) - - DATE COMPLETED 09/08/2023

EQUIPMENT HOLLOW STEM AUGER BY: JJK

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'			SP	- medium dense, trace coarse-grained	28		
32	B1@32 1/2'					71	132.5	10.1
34				SM	Silty Sand, dense, wet, gray, fine-grained, some medium- to coarse-grained and fine gravel. (2" Sandy Silt lense)			
36	B1@35'					50		
38	B1@37 1/2'					50 (4")		
40	B1@40'				- fine to coarse gravel	44		
42	B1@42 1/2'				- trace coarse-grained, increase in silt, no fine to coarse gravel	50 (4")	125.6	12.4
44	B1@45'					50 (6")		
46								
48				ML	Silt with Sand, stiff, moist, gray.			
50	B1@50'					23		
52	B1@52 1/2'				- hard	50 (6")	100.7	26.8
54	B1@55'				- interbedded, brown, black, and gray	50 (5")		
56				ML	Silt, hard, slightly moist to moist, dark brown and dark olive gray.			
58	B1@57 1/2'					50 (4")	100.1	24.6

W1815-06-01 BORING LOGS.GPJ

Log of Boring 1, Page 2 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 1

ELEV. (MSL.) - - DATE COMPLETED 09/08/2023

EQUIPMENT HOLLOW STEM AUGER BY: JJK

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'					50 (5")		
62	B1@62 1/2			ML	- dark olive gray	50 (4")	102.3	26.1
64								
66	B1@65'					50 (3")		

Total depth of boring: 66 feet
Fill to 9 feet.
Groundwater encountered at 24 feet.
Backfilled with soil cuttings and tamped.
Patched with cold patch asphalt.

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

W1815-06-01 BORING LOGS.GPJ

Log of Boring 1, Page 3 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 2

ELEV. (MSL.) - - DATE COMPLETED 09/08/2023

EQUIPMENT HOLLOW STEM AUGER BY: JJK

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0					**AC: 4" BASE: 10"** **ARTIFICIAL FILL** Sandy Silt, soft, moist, dark brown, fine-grained.			
2	B2@2.5'			ML	**ALLUVIUM** Sandy Silt, soft, moist, olive brown, fine-grained.	8	89.4	26.5
4	B2@5'			SM	Silty Sand, loose, moist, olive brown and light reddish brown, fine-grained, some medium- to coarse-grained, trace fine gravel.	15	107.4	9.3
6								
8					Sand, poorly graded, medium dense, moist, light brown, fine-grained, some medium- to coarse-grained and fine gravel, trace coarse gravel.			
10	B2@10'					33	105.3	5.3
12								
14				SP				
16	B2@15' BULK 15-20'				- dense, some medium-grained, increase in sand and fine to coarse gravel, trace cobbles	50 (4")	121.3	3.2
18	B2@17 1/2'				- light gray and light brown, decrease in coarse-grained, fine to coarse gravel	65	102.0	7.7
20	B2@20'				- very dense, light brown, medium- to coarse-grained, some fine-grained and fine gravel	50 (5")	122.0	8.3
22								
24			▼					
26	B2@25'				- very moist to wet, gray and light reddish brown, fine-grained, no fine- to coarse-grained or gravel	50 (5")	121.0	12.0
28				SM	Silty Sand, very dense, wet, gray, fine-grained, some coarse-grained and fine gravel.			

W1815-06-01 BORING LOGS.GPJ

Log of Boring 2, Page 1 of 2

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

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BORING 2

ELEV. (MSL.) - - DATE COMPLETED 09/08/2023

EQUIPMENT HOLLOW STEM AUGER BY: JJK

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'					50 (5")	142.7	2.9
32	B2@32 1/2				- dense, no coarse-grained or fine gravel, oil, hydrocarbon	60	116.9	19.1
34								
	B2@35'				- very dense, very moist	50 (5")	117.4	16.9
36								
38				SM				
40	B2@40'				- wet	50 (2")	114.8	20.7
42								
44								
	B2@45'				- no recovery	50 (3")		
46								
48								
50	B2@50'				- no recovery	50 (4")		

Total depth of boring: 50 1/2 feet
Fill to 2 feet.
Groundwater encountered at 24 feet.
Backfilled with soil cuttings and tamped.
Patched with cold patch asphalt.

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

W1815-06-01 BORING LOGS.GPJ

Log of Boring 2, Page 2 of 2

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

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PROJECT NO. W1815-06-01

BORING 3

ELEV. (MSL.) - - DATE COMPLETED 09/08/2023

EQUIPMENT HOLLOW STEM AUGER BY: JJK

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0					**ARTIFICIAL FILL** Silty Sand, medium dense, moist, olive brown, fine-grained, trace fine gravel.			
4								
5	B3@5'			SP	**ALLUVIUM** Sand, poorly graded, medium dense, slightly moist, light brown, fine-grained, some medium-grained.	21	273.7	10.0
8								
10	B3@10'			SP-SM	Sand with Silt, poorly graded, medium dense, moist, olive brown and reddish brown, fine-grained, some medium- to coarse-grained, trace fine gravel.	45	98.2	1.7
12								
15	B3@15'			SP	Sand, poorly graded, dense, moist, fine-grained, some medium- to coarse-grained and fine to coarse gravel.	50 (5")	130.4	2.4
17					- abundant fine to coarse gravel			
20	B3@20'				- medium dense, very moist, light reddish brown and olive gray, fine-grained, no medium- to coarse-grained or fine to coarse gravel	49	119.2	23.1
21				SP-SM	Sand with Silt, dense, wet, gray, some medium- to coarse-grained.			
24			▼					
25	B3@25'					62	114.8	15.6
28								

W1815-06-01 BORING LOGS.GPJ

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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GEOCON

BORING 3

ELEV. (MSL.) - - DATE COMPLETED 09/08/2023

EQUIPMENT HOLLOW STEM AUGER BY: JJK

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
30	B3@30'			SP-SM	- increase in coarse-grained	59	123.0	16.0
32								
34								
	B3@35'			SM	Silty Sand, very dense, wet, gray, some coarse-grained and fine to coarse gravel, sulfur odor.	50 (4")	134.9	9.7
36								
38					Silt, hard, moist, bluish gray.			
40	B3@40'					52	251.9	22.9
42				ML				
44								
	B3@45'				- slightly moist to moist	50 (4")	95.4	30.7
46								
48								
50	B3@50'				- dark olive gray and dark brown	50 (5")	102.5	24.2
52								
				ML	Silt with Sand, hard, moist, dark brown and gray.			
54								
	B3@55'					50 (5")	96.6	28.2

Total depth of boring: 55 1/2 feet
Fill to 4 1/2 feet.
Groundwater encountered at 24 feet.
Backfilled with soil cuttings and tamped.

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

W1815-06-01 BORING LOGS.GPJ

Log of Boring 3, Page 2 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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GEOCON

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	BORING 3 ELEV. (MSL.) - - DATE COMPLETED 09/08/2023 EQUIPMENT HOLLOW STEM AUGER BY: JJK	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
					MATERIAL DESCRIPTION			
					NOTE: The stratification lines presented herein represent the approximate boundary between earth types; the transitions may be gradual.			

W1815-06-01 BORING LOGS.GPJ

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

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BORING 4

ELEV. (MSL.) **- -** DATE COMPLETED **09/07/2023**

EQUIPMENT **HOLLOW STEM AUGER** BY: **JJK**

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: 5" BASE: 6"** **ARTIFICIAL FILL** Silty Sand, loose, moist, olive gray, fine-grained.			
2								
	B4@2 1/2'			ML	**ALLUVIUM** Sandy Silt, soft, moist, olive brown, fine-grained.	8	94.8	17.1
4								
	B4@5'			SP-SM	Sand with Silt, poorly graded, moist, brown, fine-grained.	3		
6								
				SP	Sand, poorly graded, medium dense, slightly moist to moist, light brown, fine-grained, some medium-grained, trace coarse-grained.			
8	B4@7 1/2'					48	108.6	4.1
10	B4@10'				- dense, slightly moist, some medium- to coarse-grained and fine gravel, trace coarse gravel	32		
12								
	B4@12 1/2'				- no coarse-grained	36	117.1	3.1
14								
	B4@15'				- slightly moist to moist, brown	32		
16								
				SP-SM	Sand with Silt, poorly graded, dense, slightly moist to moist, light olive brown, fine-grained, trace medium - to coarse-grained.			
18	B4@17 1/2'					50 (5")	106.9	8.7
20	B4@20'				- moist, light olive brown and light reddish brown, fine-grained, medium- to coarse-grained	59		
22				SP	Sand, poorly graded, very dense, very moist to wet, light grayish brown and reddish brown, some medium- to coarse-grained, trace fine gravel.			
	B4@22 1/2'		▼			50 (3)	127.1	9.4
24								
	B4@25'				- very moist to wet, light brown, reddish brown, and gray, some fine-grained, trace coarse gravel.	70		
26				SP	Sand, poorly graded, very dense, wet, gray, fine-grained, medium- to coarse-grained.			
28	B4@27 1/2'					50 (5")	148.7	6.5

W1815-06-01 BORING LOGS.GPJ

Log of Boring 4, 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 4

ELEV. (MSL.) - - DATE COMPLETED 09/07/2023

EQUIPMENT HOLLOW STEM AUGER BY: JJK

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
30	B4@30' BULK 30-35'			SP	- medium dense, some medium- to coarse-grained, some fine gravel, trace coarse gravel	27		
32	B4@32 1/2'				- dense, increase in fine to coarse gravel	69	110.7	12.7
34								
	B4@35'				- medium dense, wet, gray, no coarse gravel	23		
36								
38	B4@37 1/2'				- dense	50 (4")	114.2	13.0
40	B4@40'			SP-SM	Sand with Silt, poorly graded medium dense, wet, gray, fine-grained, some medium-grained and fine gravel.	22		
42	B4@42 1/2'					50 (5")	117.0	15.0
44								
46	B4@45'				- dense, gray to dark gray, fine-grained, trace medium-grained and fine gravel, sulfur odor	30		
48	B4@47 1/2'			SP	Sand, poorly graded, medium dense, wet, gray, medium-grained, some fine- to coarse-grained and coarse gravel, sulfur odor.	50 (5")	142.6	9.5
50	B4@50'			SM	Silty Sand, very dense, wet, gray and light gray, fine-grained, medium- to coarse-grained, trace fine to coarse gravel.	58		
52	B4@52 1/2'				- no coarse-grained, fine to coarse gravel	50 (5")	129.4	13.4
54								
56	B4@55'				- fine-grained with coarse-grained, some medium-grained and fine gravel	79		
58	B4@57 1/2'			ML	Silt with Sand, hard, slightly moist, dark grayish brown and light brown.	41		

W1815-06-01 BORING LOGS.GPJ

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

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GEOCON

BORING 4

ELEV. (MSL.) - - DATE COMPLETED 09/07/2023

EQUIPMENT HOLLOW STEM AUGER BY: JJK

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'			ML		76		
65	B4@65'					76		
68				ML	Sandy Silt, hard, moist, dark brown and gray, fine-grained.			
70	B4@70'					43		
75	B4@75'				- slightly moist to moist	50 (6")		
78				ML	Silt with Sand, hard, slightly moist to moist, dark brown and gray.			
80	B4@80'					38		

Total depth of boring: 81 feet
Fill to 2 1/2 feet.
Groundwater encountered at 23 feet.
Backfilled with soil cuttings and tamped.
Patched with cold patch asphalt.

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

W1815-06-01 BORING LOGS.GPJ

Log of Boring 4, Page 3 of 3

SAMPLE SYMBOLS
- ... SAMPLING UNSUCCESSFUL
- ... STANDARD PENETRATION TEST
- ... DRIVE SAMPLE (UNDISTURBED)
- ... DISTURBED OR BAG SAMPLE
- ... CHUNK SAMPLE
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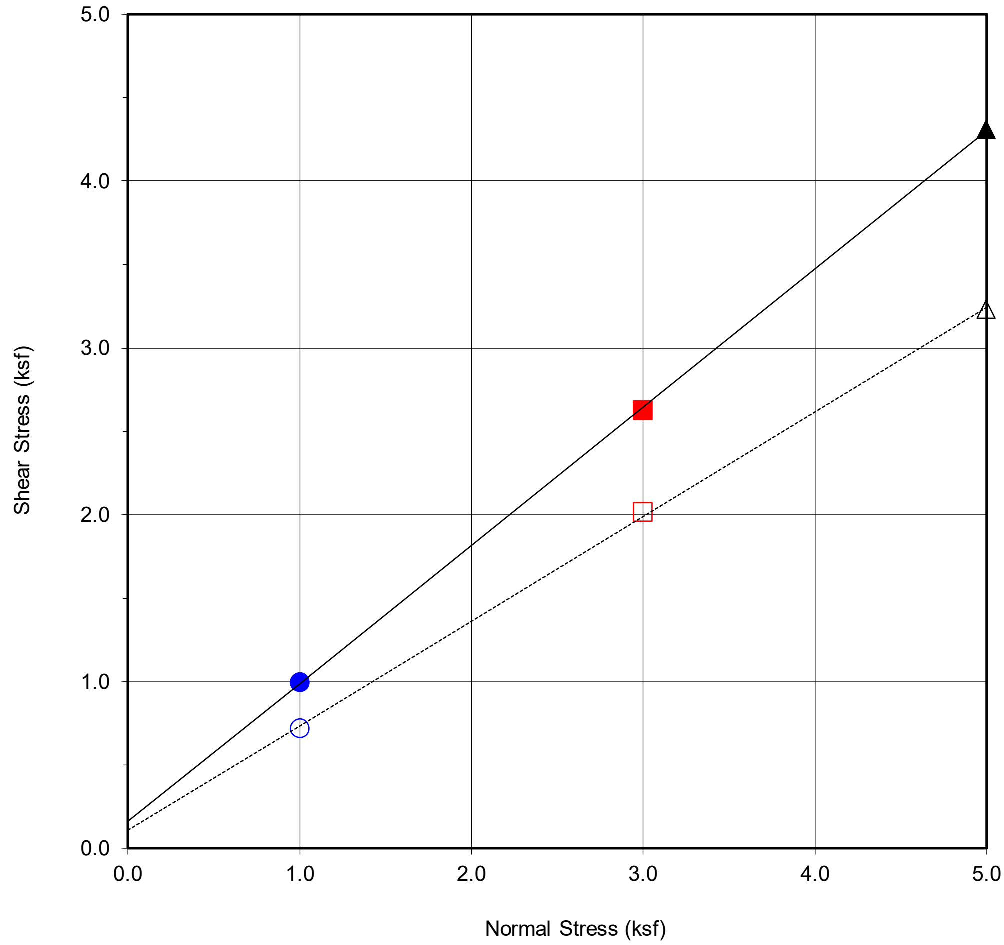
APPENDIX B

APPENDIX B

LABORATORY TESTING

Laboratory tests were performed in accordance with generally accepted test methods of the "American Society for Testing and Materials (ASTM)", or other suggested procedures. Selected samples were tested for direct shear strength, consolidation and expansion characteristics, plasticity indices, grain size analysis, optimum moisture and maximum dry density relationships, corrosivity and in-place dry density and moisture content. The results of the laboratory tests are summarized in Figures B1 through B28. The in-place dry density and moisture content of the samples tested are presented on the boring logs, Appendix A.



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

Soil Identification:		
Sand (SP)		
Strength Parameters		
	C (psf)	ϕ (^{o})
Peak	160	40
Ultimate	107	32

Normal Stress (kip/ft²)	1	3	5
Peak Shear Stress (kip/ft²)	● 1.00	■ 2.63	▲ 4.31
Shear Stress @ End of Test (ksf)	○ 0.72	□ 2.02	△ 3.23
Deformation Rate (in./min.)	0.05	0.05	0.05
Initial Sample Height (in.)	1.0	1.0	1.0
Ring Inside Diameter (in.)	2.375	2.375	2.375
Initial Moisture Content (%)	5.3	5.5	6.2
Initial Dry Density (pcf)	101.8	102.2	103.8
Initial Degree of Saturation (%)	21.9	23.0	26.7
Soil Height Before Shearing (in.)	1.2	1.2	1.2
Final Moisture Content (%)	20.8	20.1	20.2

GEOCON

DIRECT SHEAR TEST RESULTS

Consolidated Drained ASTM D-3080

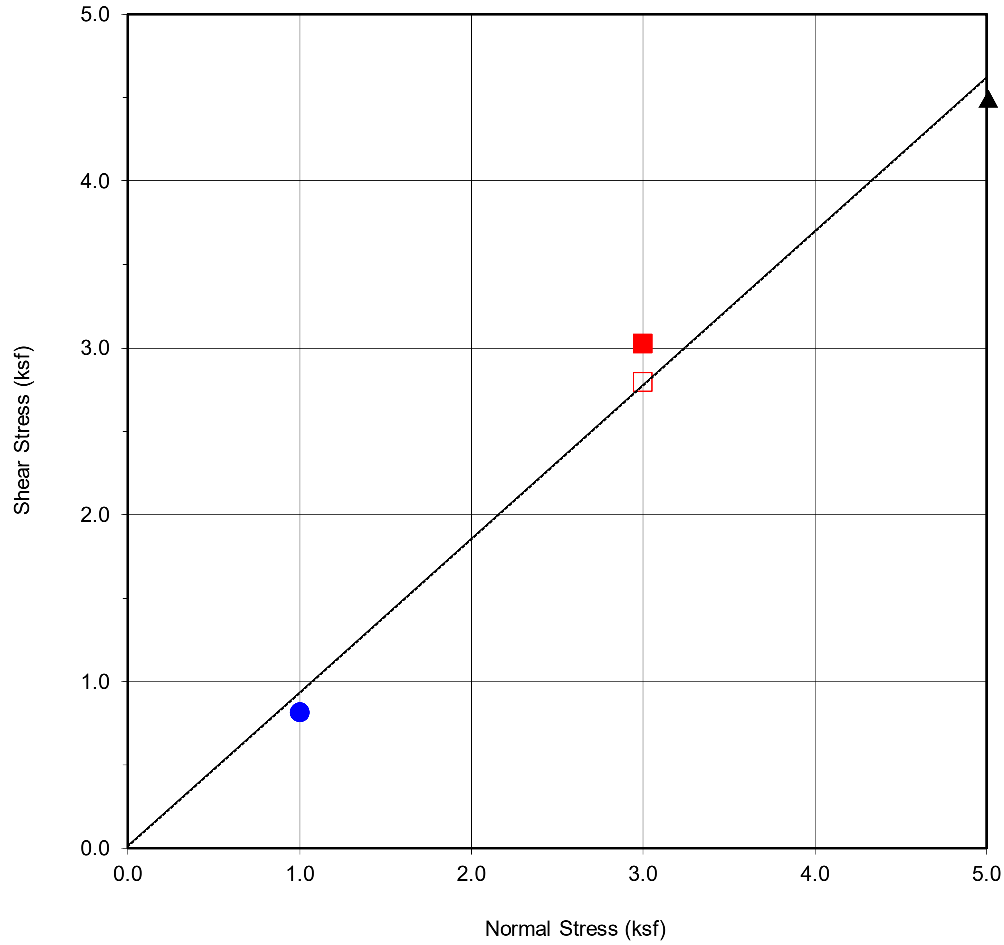
Checked by: JJK

Project No.: W1815-06-01

800 & 908 NORTH MAIN STREET
1081 & 1087 NORTH VIGNES STREET
LOS ANGELES, CALIFORNIA

OCT. 2023

Figure B1



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

Soil Identification:		
Sand (SP)		
Strength Parameters		
	C (psf)	ϕ (°)
Peak	17	43
Ultimate	9	43

Normal Stress (kip/ft²)	1	3	5
Peak Shear Stress (kip/ft²)	● 0.82	■ 3.02	▲ 4.50
Shear Stress @ End of Test (ksf)	○ 0.82	□ 2.80	△ 4.50
Deformation Rate (in./min.)	0.05	0.05	0.05
Initial Sample Height (in.)	1.0	1.0	1.0
Ring Inside Diameter (in.)	2.375	2.375	2.375
Initial Moisture Content (%)	7.8	2.5	8.9
Initial Dry Density (pcf)	101.3	108.0	105.2
Initial Degree of Saturation (%)	31.7	12.2	40.1
Soil Height Before Shearing (in.)	1.2	1.2	1.2
Final Moisture Content (%)	18.2	12.5	15.3

GEOCON

DIRECT SHEAR TEST RESULTS

Consolidated Drained ASTM D-3080

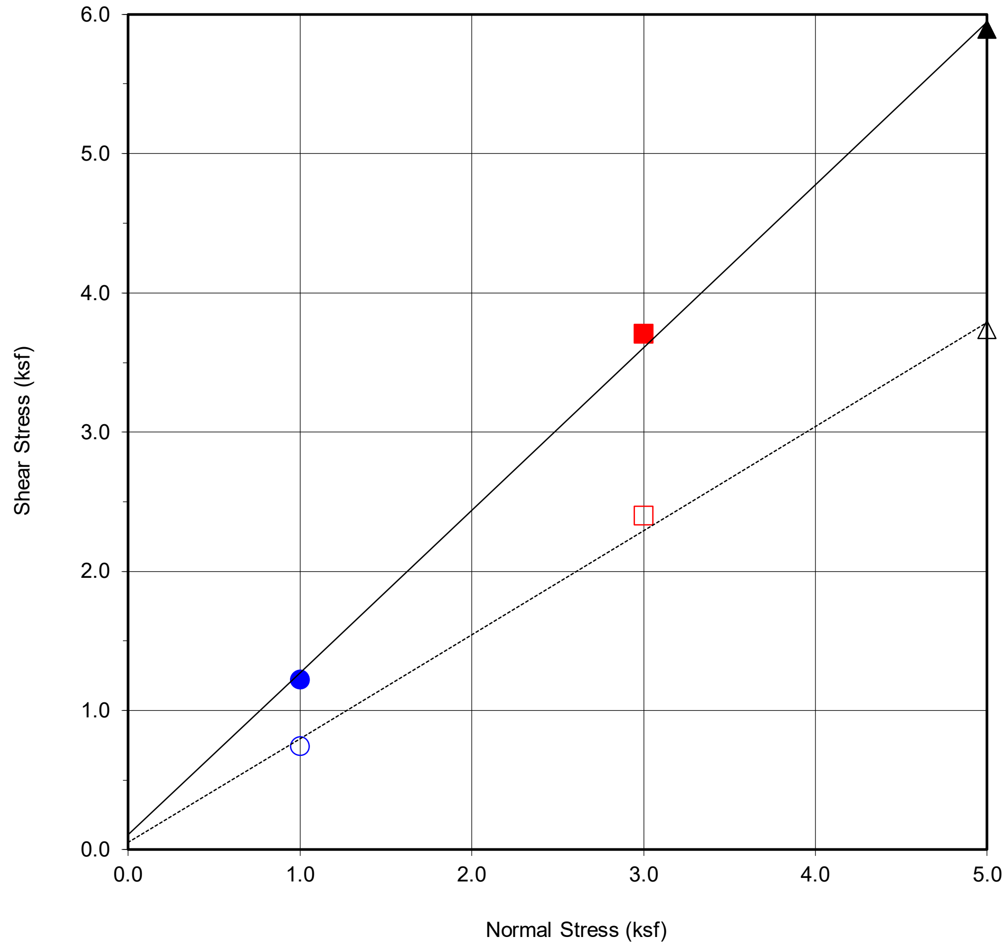
Checked by: JJK

Project No.: W1815-06-01

800 & 908 NORTH MAIN STREET
1081 & 1087 NORTH VIGNES STREET
LOS ANGELES, CALIFORNIA

OCT. 2023

Figure B2



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

Soil Identification:		
Sand w/ Silt (SP-SM)		
Strength Parameters		
	C (psf)	ϕ (^{o})
Peak	107	49
Ultimate	51	37

Normal Stress (kip/ft²)	1	3	5
Peak Shear Stress (kip/ft²)	● 1.22	■ 3.71	▲ 5.89
Shear Stress @ End of Test (ksf)	○ 0.74	□ 2.40	△ 3.73
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 (%)	18.4	15.0	15.8
Initial Dry Density (pcf)	111.5	119.8	118.9
Initial Degree of Saturation (%)	96.8	99.3	102.5
Soil Height Before Shearing (in.)	1.2	1.2	1.2
Final Moisture Content (%)	20.2	15.0	16.4

GEOCON

DIRECT SHEAR TEST RESULTS

Consolidated Drained ASTM D-3080

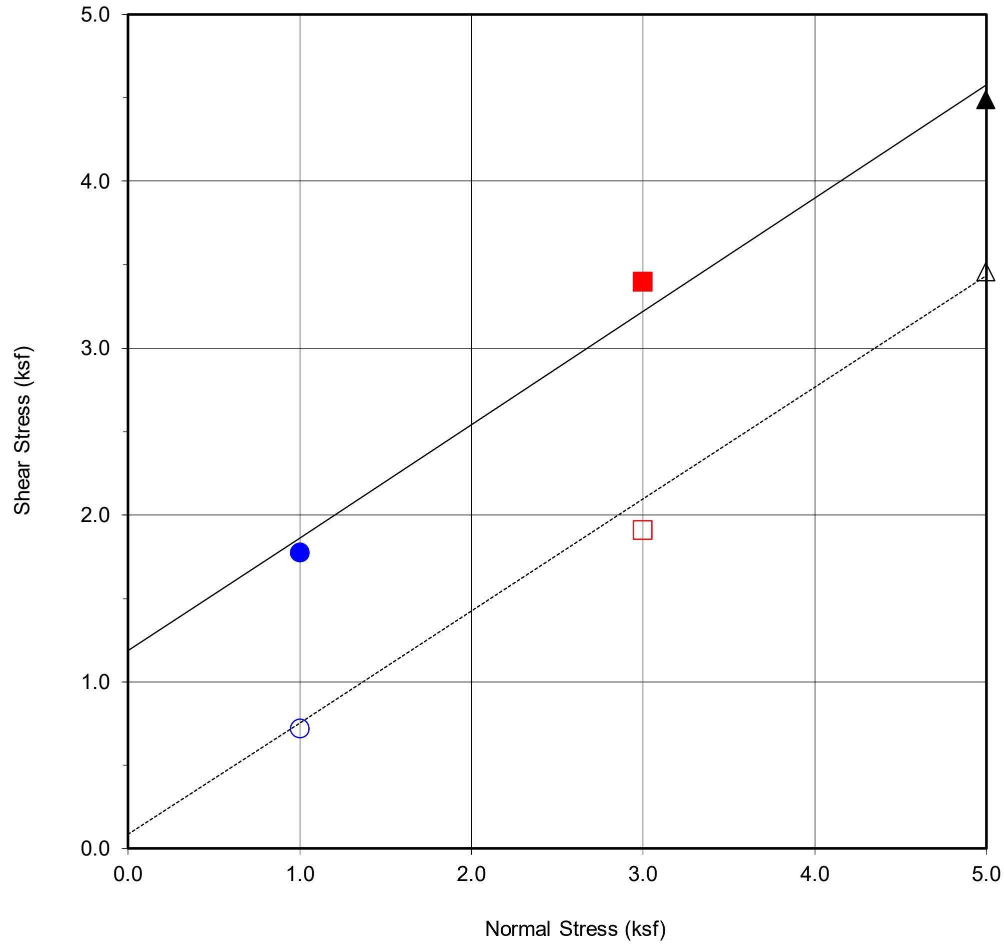
Checked by: JJK

Project No.: W1815-06-01

800 & 908 NORTH MAIN STREET
1081 & 1087 NORTH VIGNES STREET
LOS ANGELES, CALIFORNIA

OCT. 2023

Figure B3



Boring No.	B3
Sample No.	B3@55
Depth (ft)	55
Sample Type:	Ring

Soil Identification:		
Silt w/ Sand (ML)		
Strength Parameters		
	C (psf)	ϕ (°)
Peak	1186	34
Ultimate	86	34

Normal Stress (kip/ft²)	1	3	5
Peak Shear Stress (kip/ft²)	● 1.78	■ 3.40	▲ 4.49
Shear Stress @ End of Test (ksf)	○ 0.72	□ 1.91	△ 3.46
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 (%)	28.2	28.6	26.0
Initial Dry Density (pcf)	94.1	94.3	99.3
Initial Degree of Saturation (%)	96.2	98.0	100.6
Soil Height Before Shearing (in.)	1.2	1.2	1.2
Final Moisture Content (%)	33.3	32.4	28.7

GEOCON

DIRECT SHEAR TEST RESULTS

Consolidated Drained ASTM D-3080

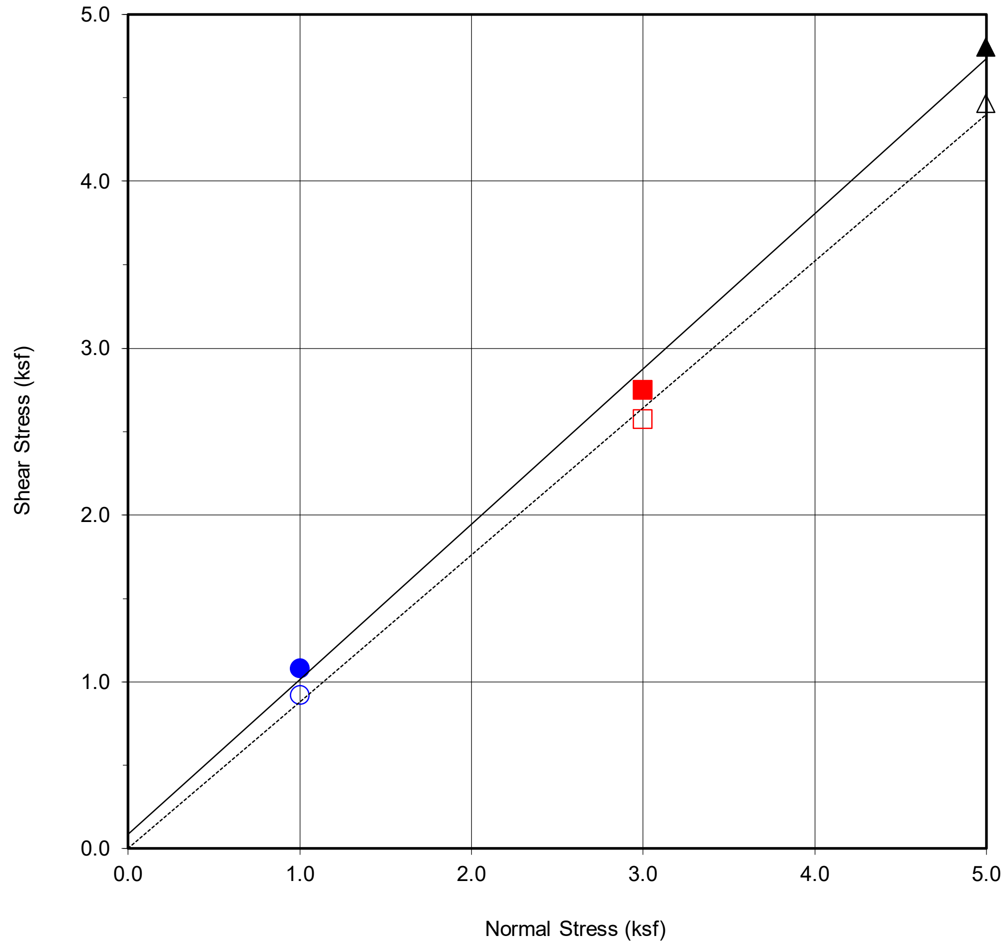
Checked by: JJK

Project No.: W1815-06-01

800 & 908 NORTH MAIN STREET
1081 & 1087 NORTH VIGNES STREET
LOS ANGELES, CALIFORNIA

OCT. 2023

Figure B4



Boring No.	**B4**
Sample No.	**B4@7.5**
Depth (ft)	**7.5**
Sample Type:	Ring

Soil Identification:		
Sand (SP)		
Strength Parameters		
	C (psf)	ϕ (°)
Peak	86	43
Ultimate	0	41

Normal Stress (kip/ft²)	1	3	5
Peak Shear Stress (kip/ft²)	● 1.08	■ 2.75	▲ 4.80
Shear Stress @ End of Test (ksf)	○ 0.90	□ 2.59	△ 4.42
Deformation Rate (in./min.)	0.05	0.05	0.05
Initial Sample Height (in.)	1.0	1.0	1.0
Ring Inside Diameter (in.)	2.375	2.375	2.375
Initial Moisture Content (%)	4.1	0.4	12.2
Initial Dry Density (pcf)	104.2	110.9	104.4
Initial Degree of Saturation (%)	17.8	1.9	53.6
Soil Height Before Shearing (in.)	1.2	1.2	1.2
Final Moisture Content (%)	19.3	16.3	17.7

GEOCON

DIRECT SHEAR TEST RESULTS

Consolidated Drained ASTM D-3080

Checked by: JJK

Project No.: W1815-06-01

800 & 908 NORTH MAIN STREET
1081 & 1087 NORTH VIGNES STREET
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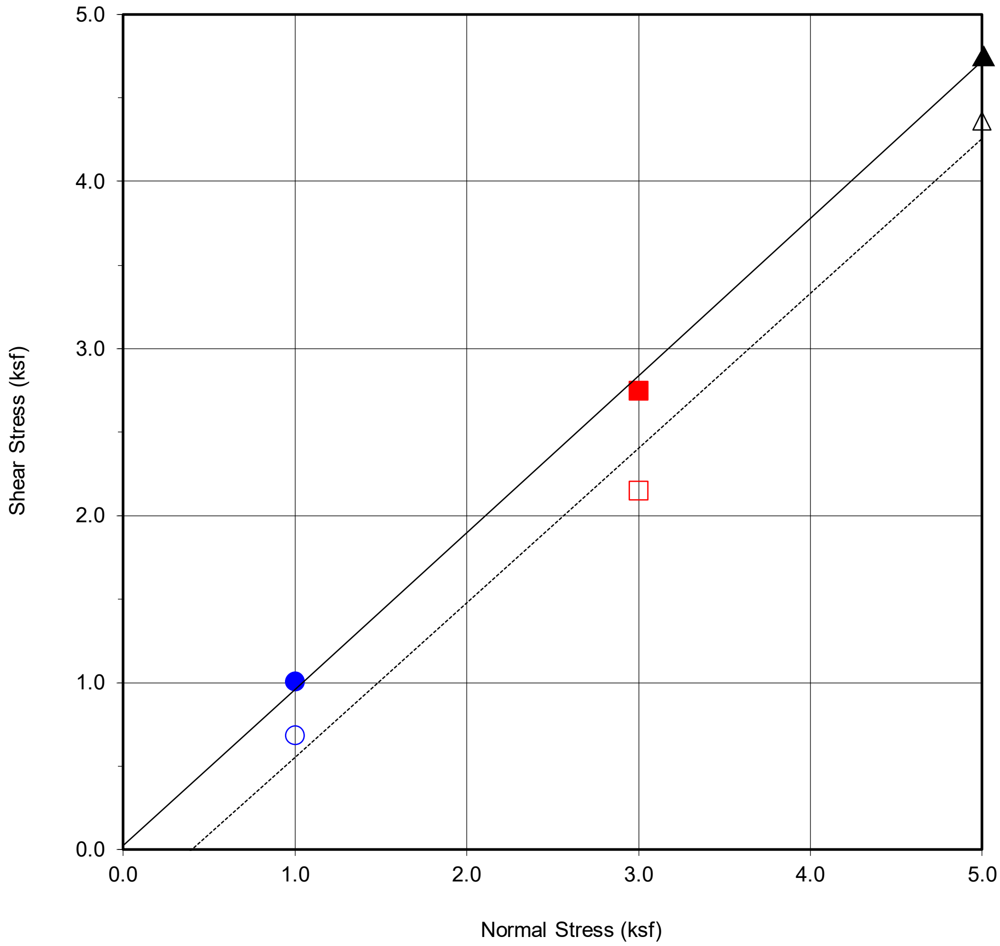
OCT. 2023

Figure B5



Boring No.	B4
Sample No.	B4@17.5
Depth (ft)	17.5
Sample Type:	Ring

Normal Stress (kip/ft²)	1	3	5
Peak Shear Stress (kip/ft²)	● 1.01	■ 2.75	▲ 4.76
Shear Stress @ End of Test (ksf)	○ 0.68	□ 2.15	△ 4.38
Deformation Rate (in./min.)	0.05	0.05	0.05
Initial Sample Height (in.)	1.0	1.0	1.0
Ring Inside Diameter (in.)	2.375	2.375	2.375
Initial Moisture Content (%)	8.7	7.2	8.5
Initial Dry Density (pcf)	105.3	108.1	107.8
Initial Degree of Saturation (%)	39.2	34.7	40.9
Soil Height Before Shearing (in.)	1.2	1.2	1.2
Final Moisture Content (%)	19.0	17.3	16.7

Soil Identification:

Sand w/ Silt (SP-SM)

Strength Parameters

	C (psf)	ϕ (^{o})
Peak	23	43
Ultimate	0	43

GEOCON

DIRECT SHEAR TEST RESULTS

Consolidated Drained ASTM D-3080

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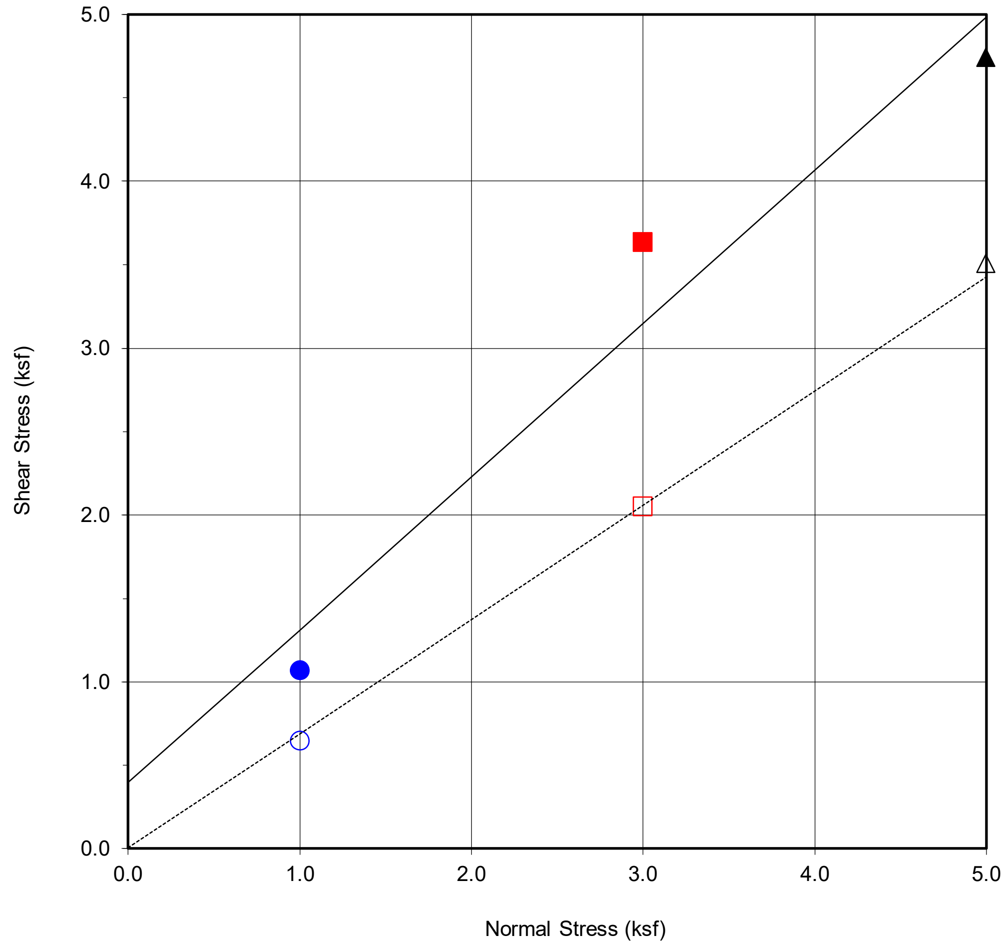
Project No.: W1815-06-01

800 & 908 NORTH MAIN STREET
1081 & 1087 NORTH VIGNES STREET
LOS ANGELES, CALIFORNIA

OCT. 2023

Figure B6



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

Soil Identification:		
Sand (SP)		
Strength Parameters		
	C (psf)	ϕ (°)
Peak	394	43
Ultimate	6	34

Normal Stress (kip/ft²)	1	3	5
Peak Shear Stress (kip/ft²)	1.07	3.64	4.74
Shear Stress @ End of Test (ksf)	0.65	2.05	3.50
Deformation Rate (in./min.)	0.05	0.05	0.05
Initial Sample Height (in.)	1.0	1.0	1.0
Ring Inside Diameter (in.)	2.375	2.375	2.375
Initial Moisture Content (%)	12.7	21.1	25.0
Initial Dry Density (pcf)	108.4	102.2	100.4
Initial Degree of Saturation (%)	61.8	88.0	99.5
Soil Height Before Shearing (in.)	1.2	1.2	1.2
Final Moisture Content (%)	21.4	21.5	23.8

GEOCON

DIRECT SHEAR TEST RESULTS

Consolidated Drained ASTM D-3080

Checked by: JJK

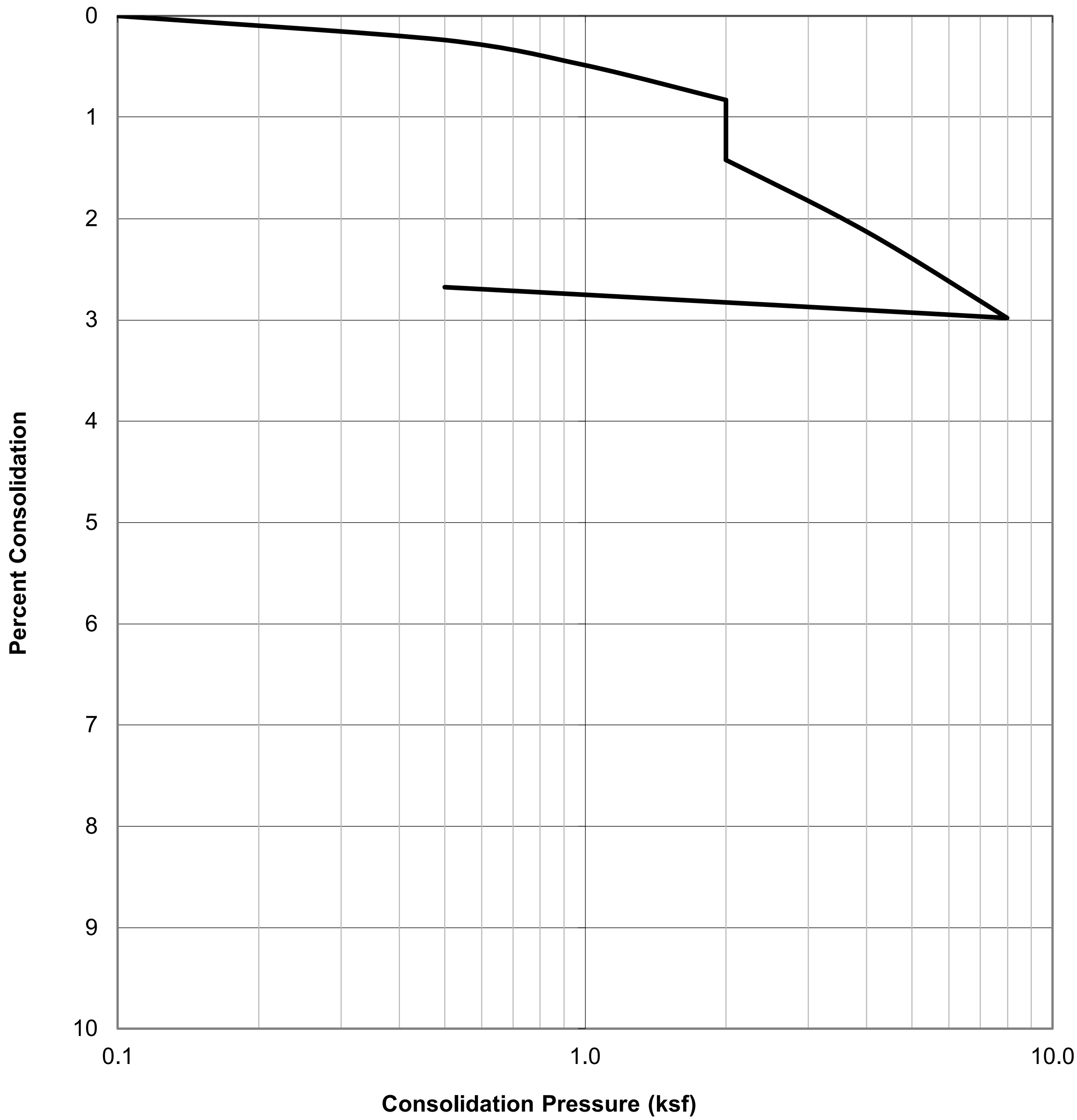
Project No.: W1815-06-01

800 & 908 NORTH MAIN STREET
1081 & 1087 NORTH VIGNES STREET
LOS ANGELES, CALIFORNIA

OCT. 2023

Figure B7

WATER ADDED AT 2.0 KSF



SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B1@17.5	Sand w/ Silt (SP-SM)	107.5	3.6	15.7

GEOCON

CONSOLIDATION TEST RESULTS

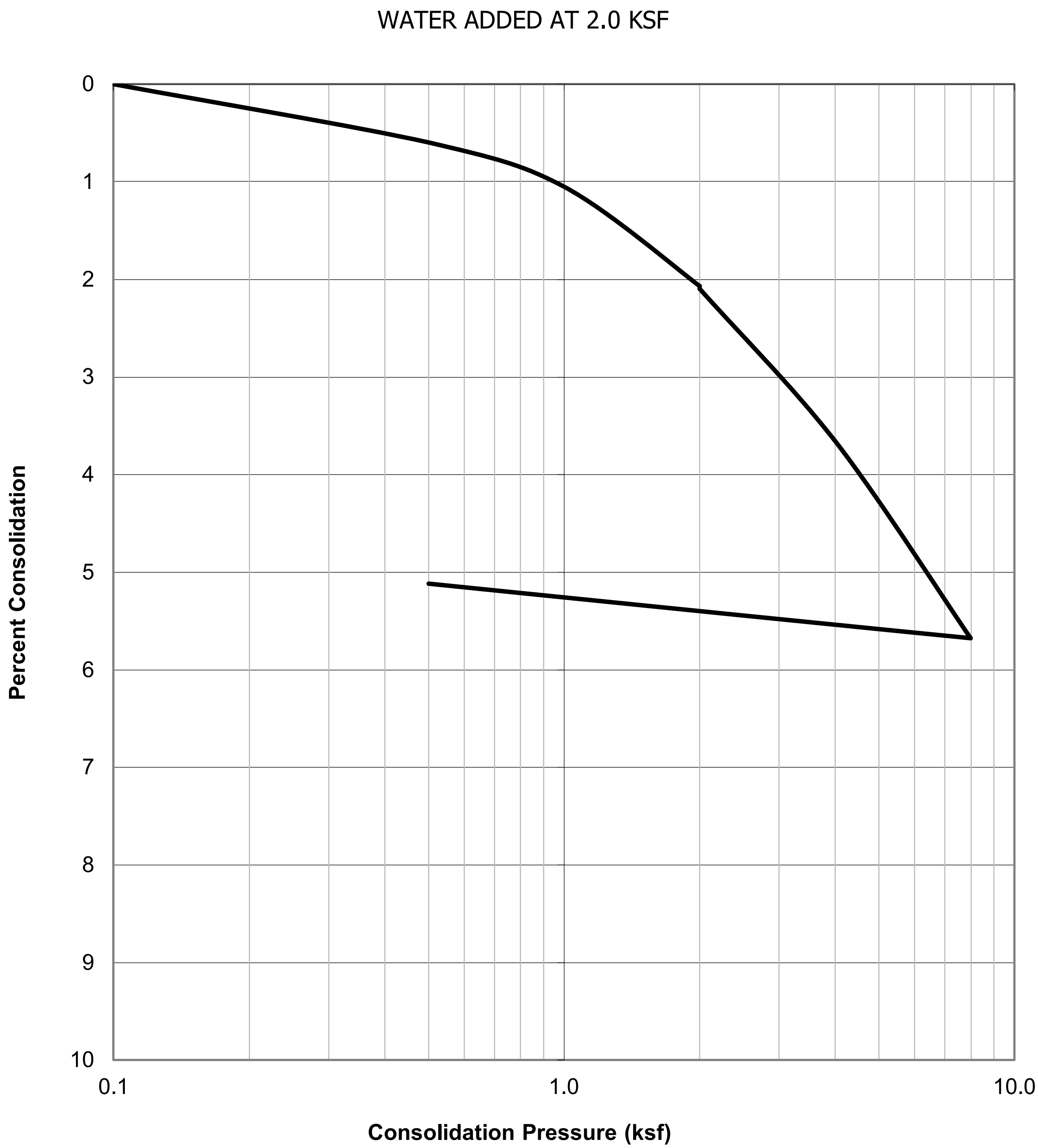
ASTM D-2435

Checked by: JJK

Project No.: W1815-06-01

800 & 908 NORTH MAIN STREET
1081 & 1087 NORTH VIGNES STREET
LOS ANGELES, CALIFORNIA

OCT. 2023



SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B1@22.5	Sandy Silt (ML)	109.8	18.1	15.9

GEOCON

CONSOLIDATION TEST RESULTS

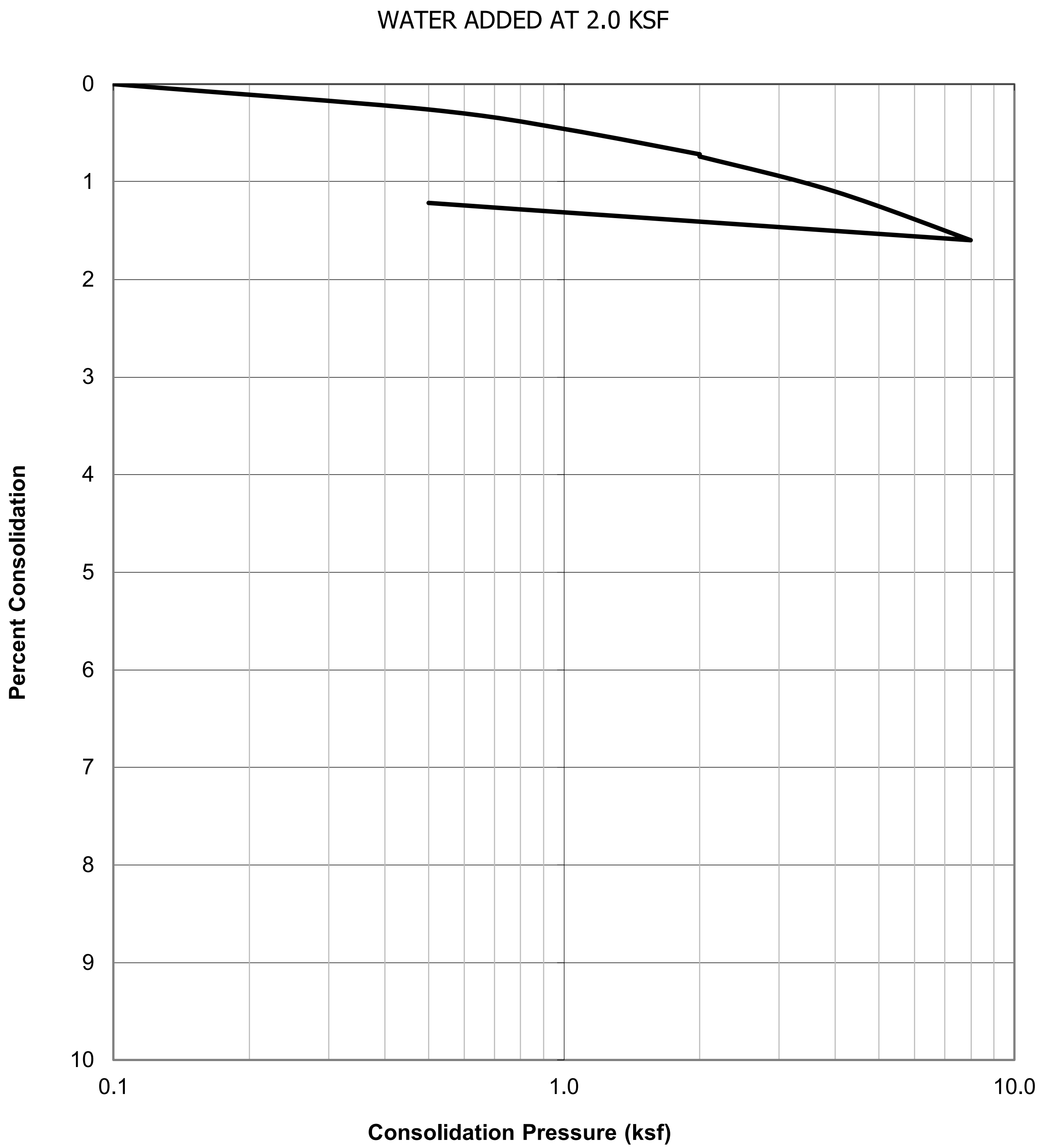
ASTM D-2435

Checked by: JJK

Project No.: W1815-06-01

800 & 908 NORTH MAIN STREET
1081 & 1087 NORTH VIGNES STREET
LOS ANGELES, CALIFORNIA

OCT. 2023



SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B1@27.5	Sand (SP)	111.3	13.4	15.4

GEOCON

CONSOLIDATION TEST RESULTS

ASTM D-2435

Checked by: JJK

Project No.: W1815-06-01

800 & 908 NORTH MAIN STREET
1081 & 1087 NORTH VIGNES STREET
LOS ANGELES, CALIFORNIA

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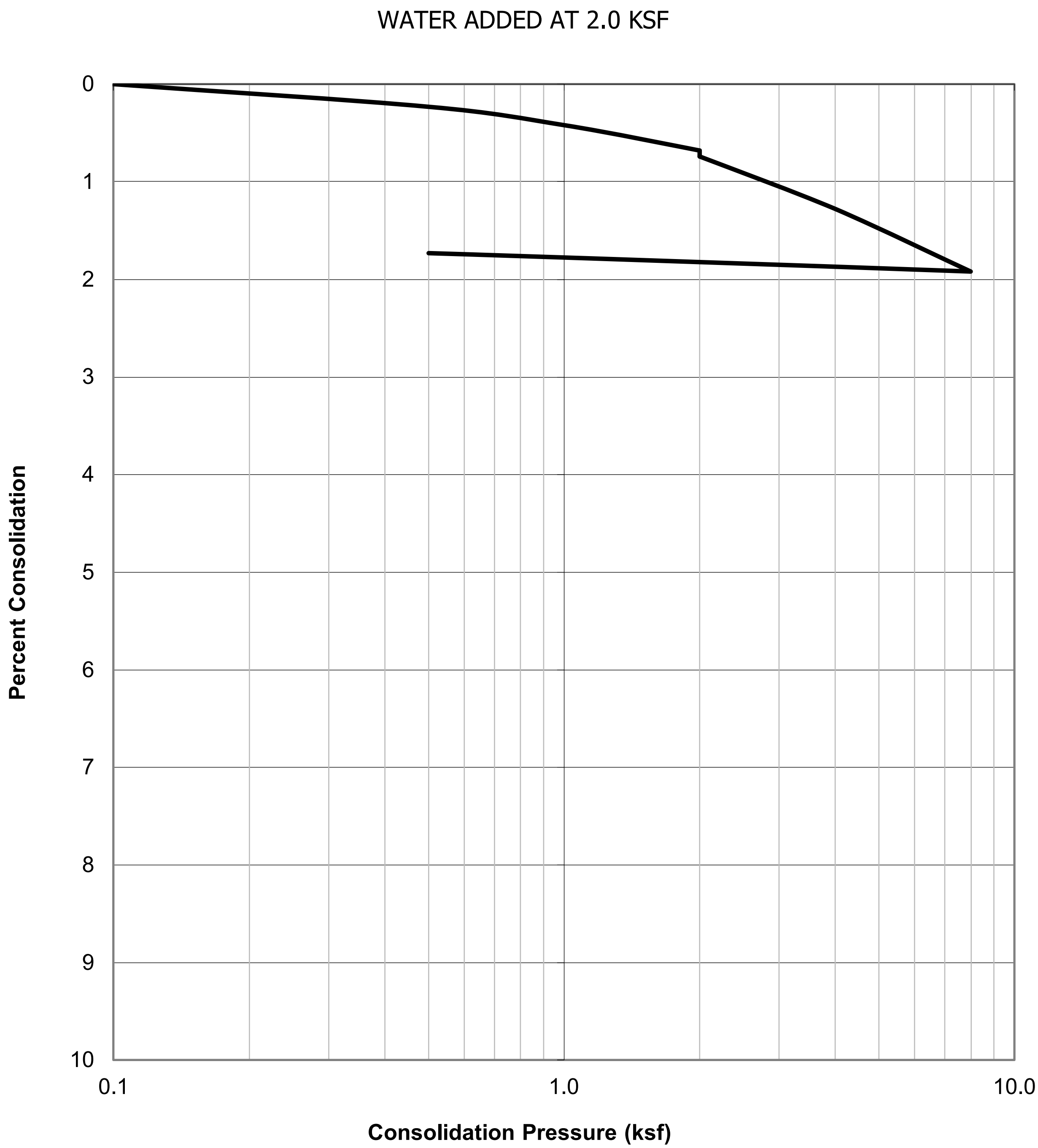
Figure B10



SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B1@32.5	Sand (SP)	124.9	10.0	11.8

GEOCON

CONSOLIDATION TEST RESULTS

ASTM D-2435

Checked by: JJK

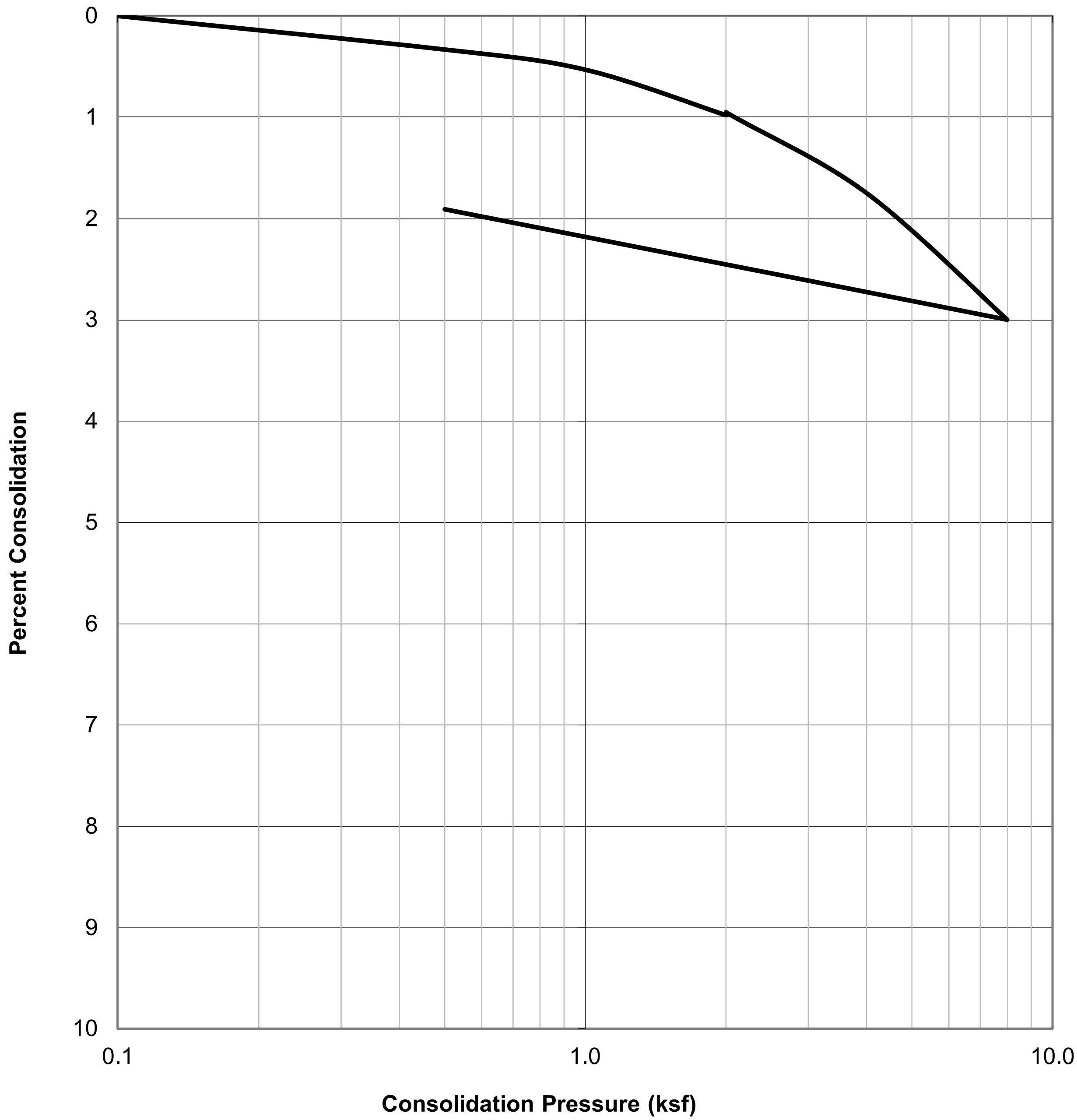
Project No.: W1815-06-01

800 & 908 NORTH MAIN STREET
1081 & 1087 NORTH VIGNES STREET
LOS ANGELES, CALIFORNIA

OCT. 2023

Figure B11

WATER ADDED AT 2.0 KSF



SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B1@52.5	Silt w/ Sand (ML)	97.4	26.8	26.9

GEOCON

CONSOLIDATION TEST RESULTS

ASTM D-2435

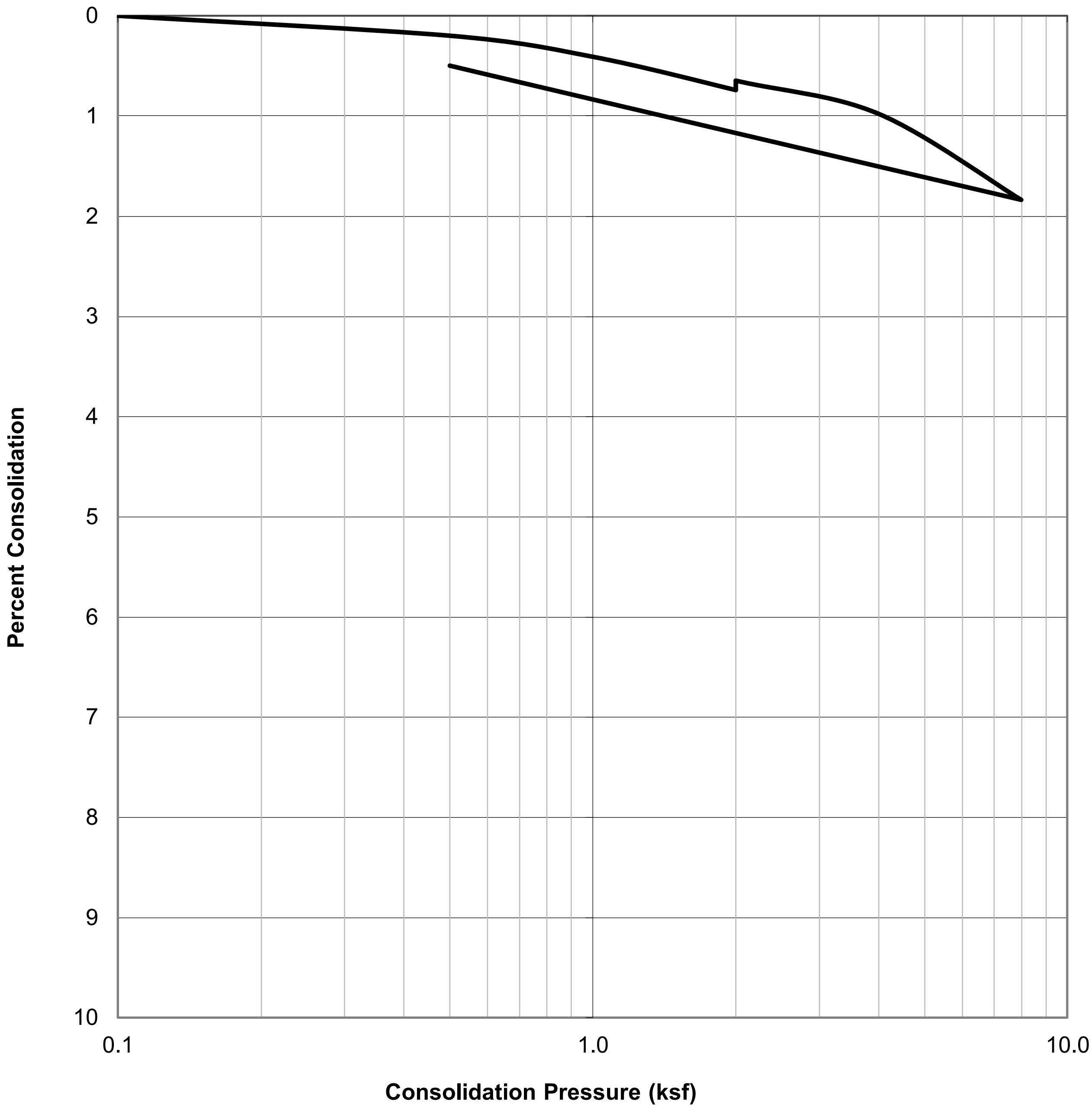
Checked by: JJK

Project No.: W1815-06-01

800 & 908 NORTH MAIN STREET
1081 & 1087 NORTH VIGNES STREET
LOS ANGELES, CALIFORNIA

OCT. 2023

WATER ADDED AT 2.0 KSF



SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B1@57.5'	Silt (ML)	97.4	24.7	27.2

GEOCON

CONSOLIDATION TEST RESULTS

ASTM D-2435

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Project No.: W1815-06-01

800 & 908 NORTH MAIN STREET
1081 & 1087 NORTH VIGNES STREET
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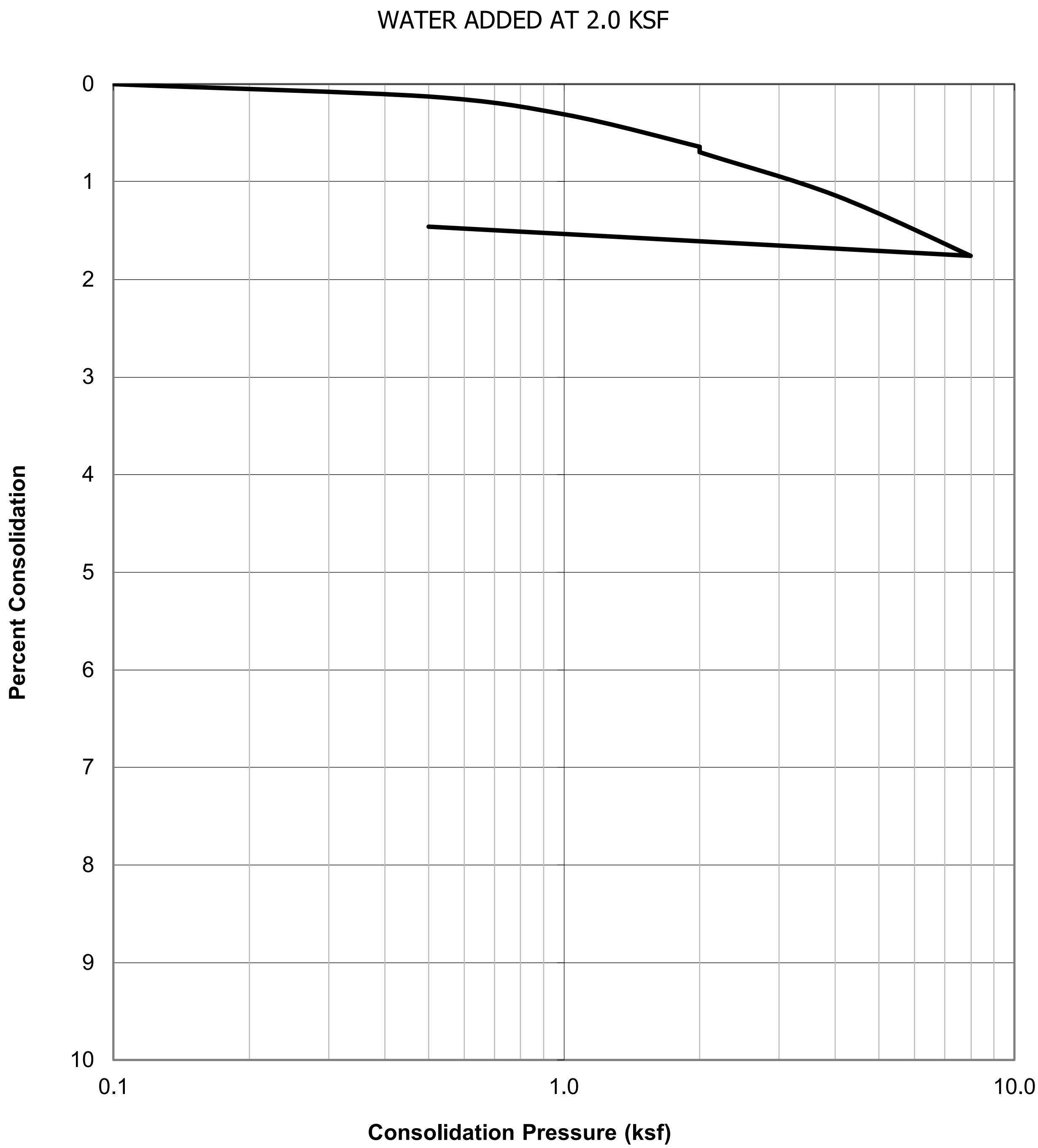
OCT. 2023

Figure B13



SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B2@25	Sand (SP)	114.7	12.0	14.2

GEOCON

CONSOLIDATION TEST RESULTS

ASTM D-2435

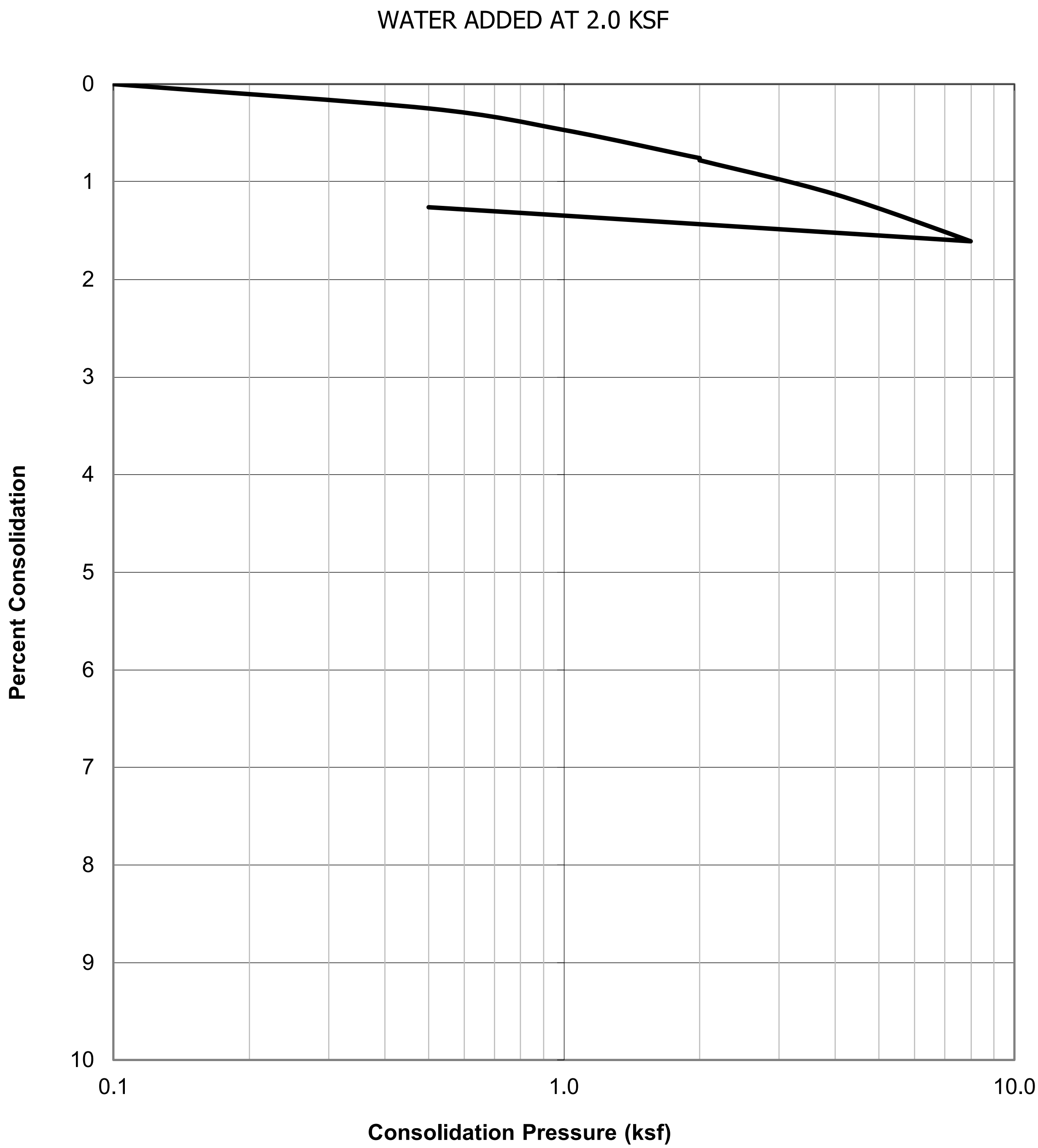
Checked by: JJK

Project No.: W1815-06-01

800 & 908 NORTH MAIN STREET
1081 & 1087 NORTH VIGNES STREET
LOS ANGELES, CALIFORNIA

OCT. 2023

Figure B14



SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B2@32.5	Silty Sand (SM)	113.2	19.1	18.9

GEOCON

CONSOLIDATION TEST RESULTS

ASTM D-2435

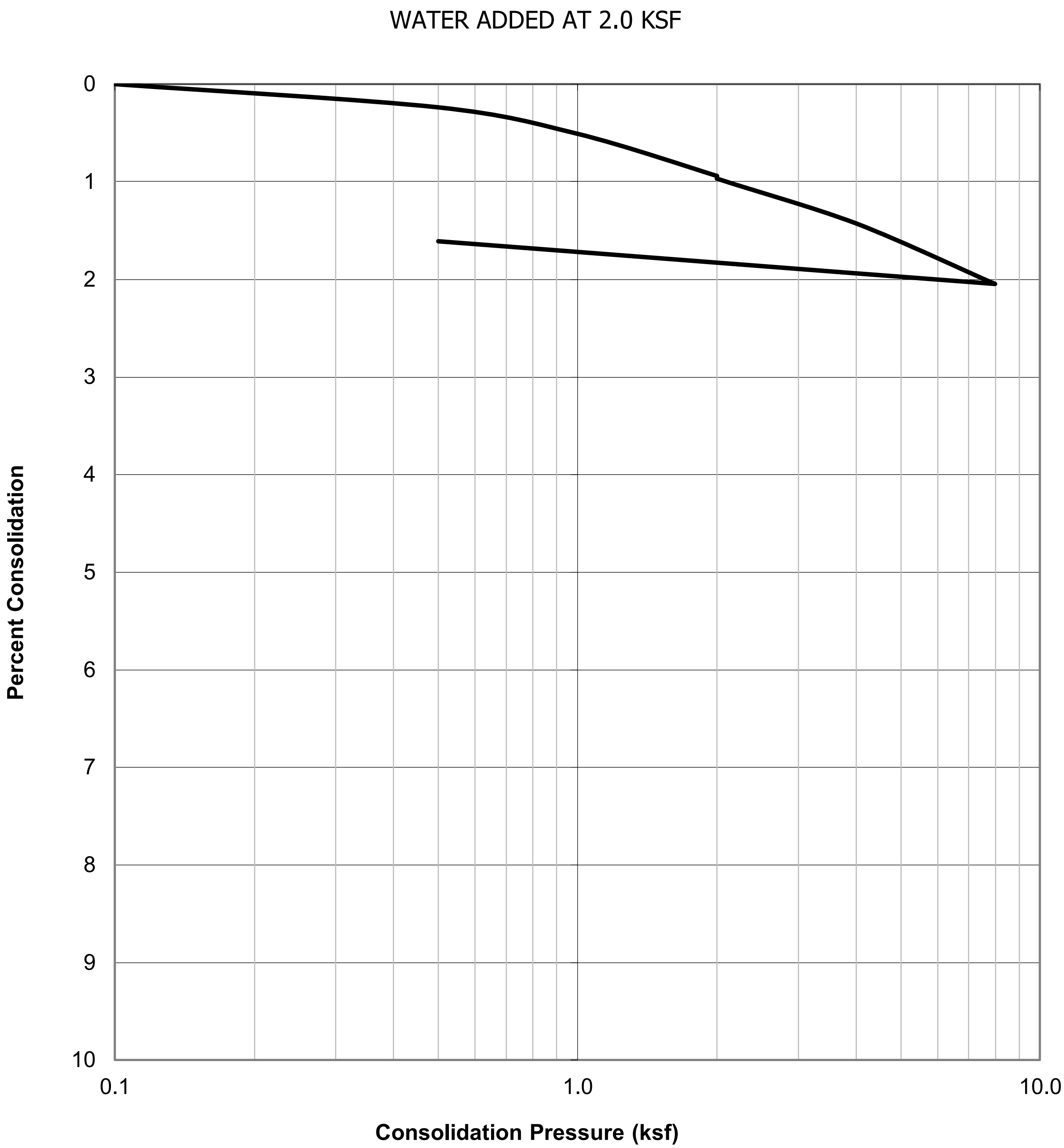
Checked by: JJK

Project No.: W1815-06-01

800 & 908 NORTH MAIN STREET
1081 & 1087 NORTH VIGNES STREET
LOS ANGELES, CALIFORNIA

OCT. 2023

Figure B15



SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B3@20'	Sand (SP)	98.2	23.0	24.8

GEOCON

CONSOLIDATION TEST RESULTS

ASTM D-2435

Checked by: JJK

Project No.: W1815-06-01

800 & 908 NORTH MAIN STREET
1081 & 1087 NORTH VIGNES STREET
LOS ANGELES, CALIFORNIA

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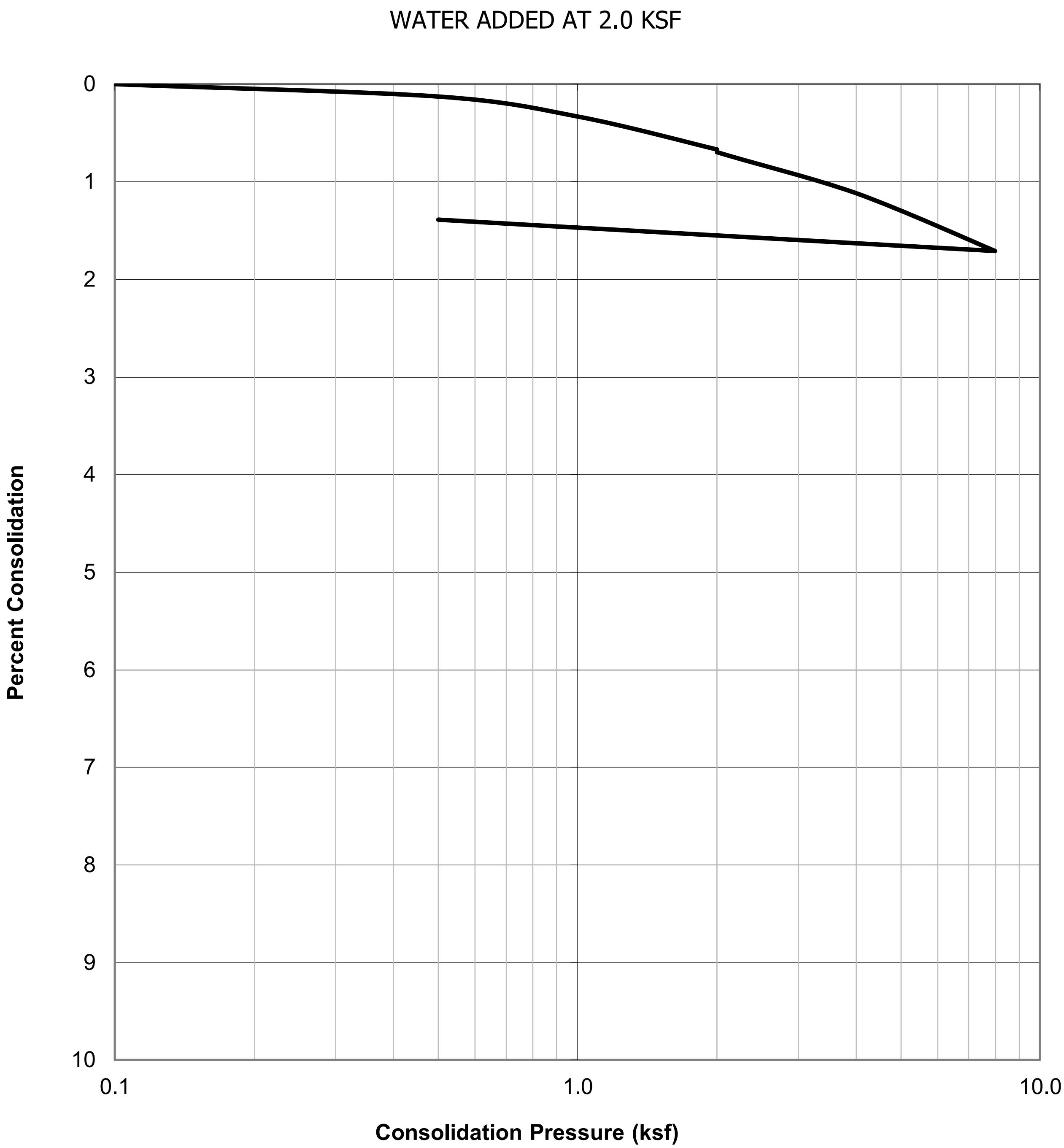
Figure B16



SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B3@25'	Sand w/ Silt (SP-SM)	114.1	15.6	16.3

GEOCON

CONSOLIDATION TEST RESULTS

ASTM D-2435

Checked by: JJK

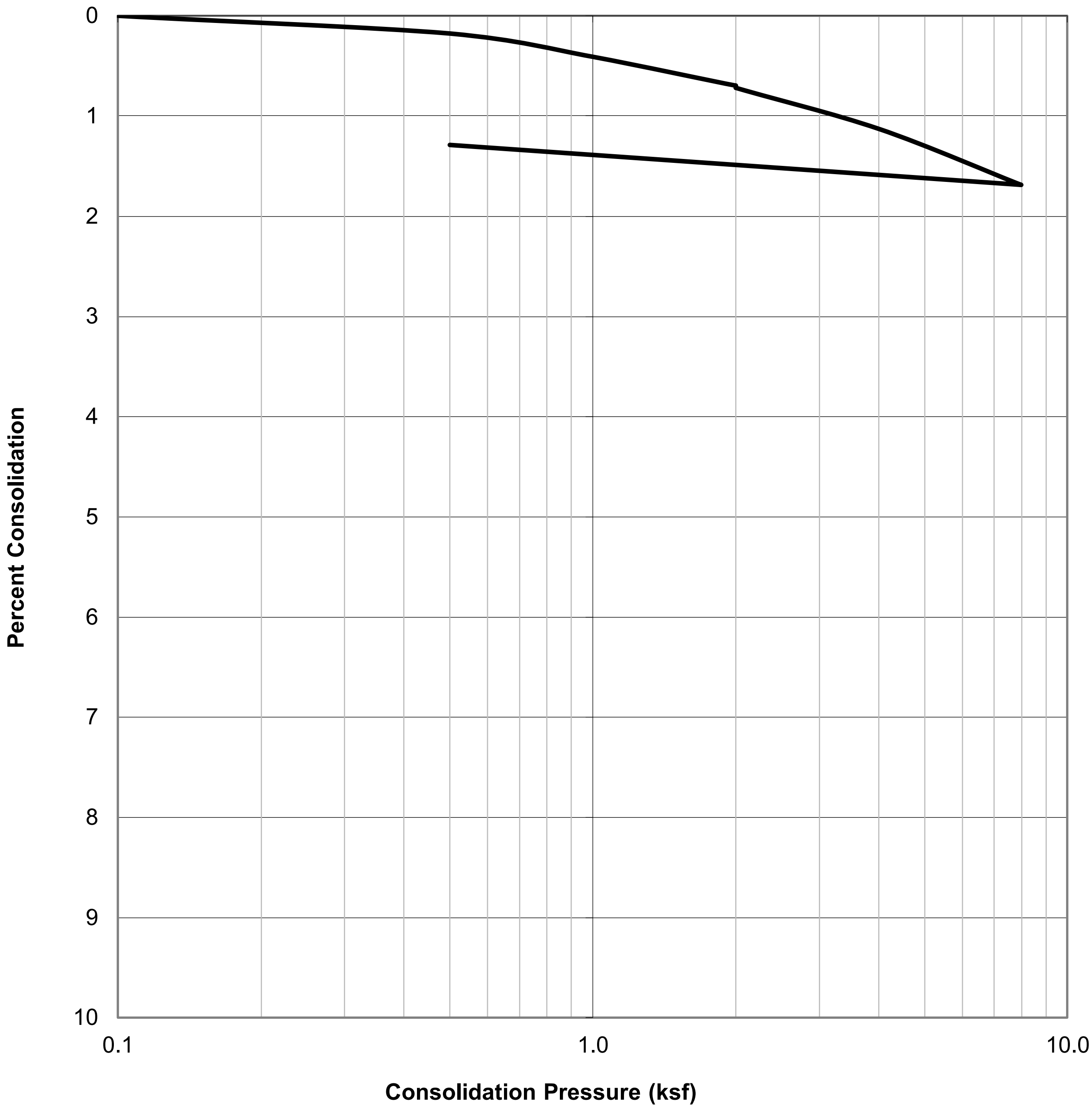
Project No.: W1815-06-01

800 & 908 NORTH MAIN STREET
1081 & 1087 NORTH VIGNES STREET
LOS ANGELES, CALIFORNIA

OCT. 2023

Figure B17

WATER ADDED AT 2.0 KSF



SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B3@30'	Sand w/ Silt (SP-SM)	110.7	16.0	18.4

GEOCON

CONSOLIDATION TEST RESULTS

ASTM D-2435

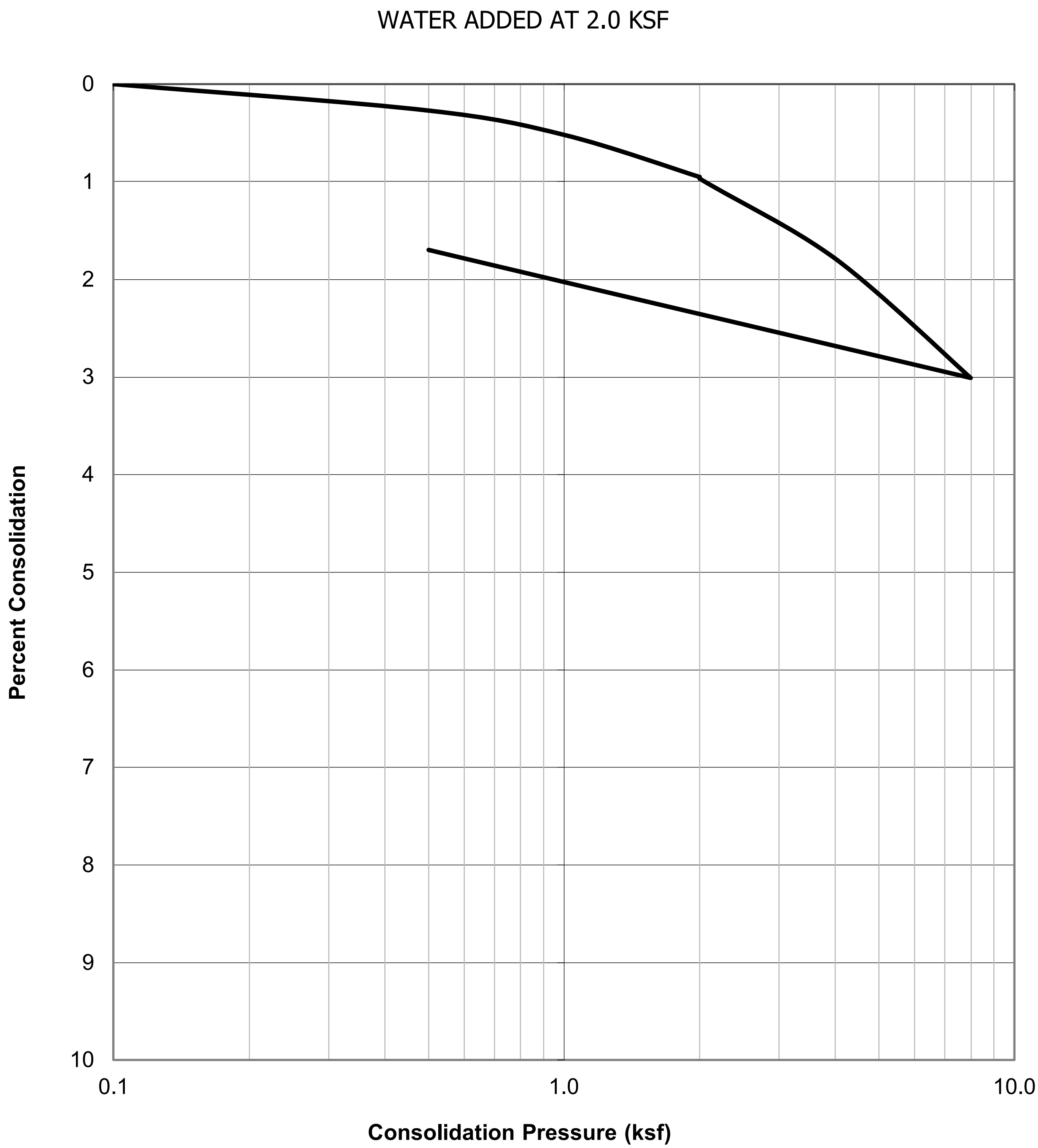
Checked by: JJK

Project No.: W1815-06-01

800 & 908 NORTH MAIN STREET
1081 & 1087 NORTH VIGNES STREET
LOS ANGELES, CALIFORNIA

OCT. 2023

Figure B18



SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B3@40	Silt (ML)	96.2	22.9	26.2

GEOCON

CONSOLIDATION TEST RESULTS

ASTM D-2435

Checked by: JJK

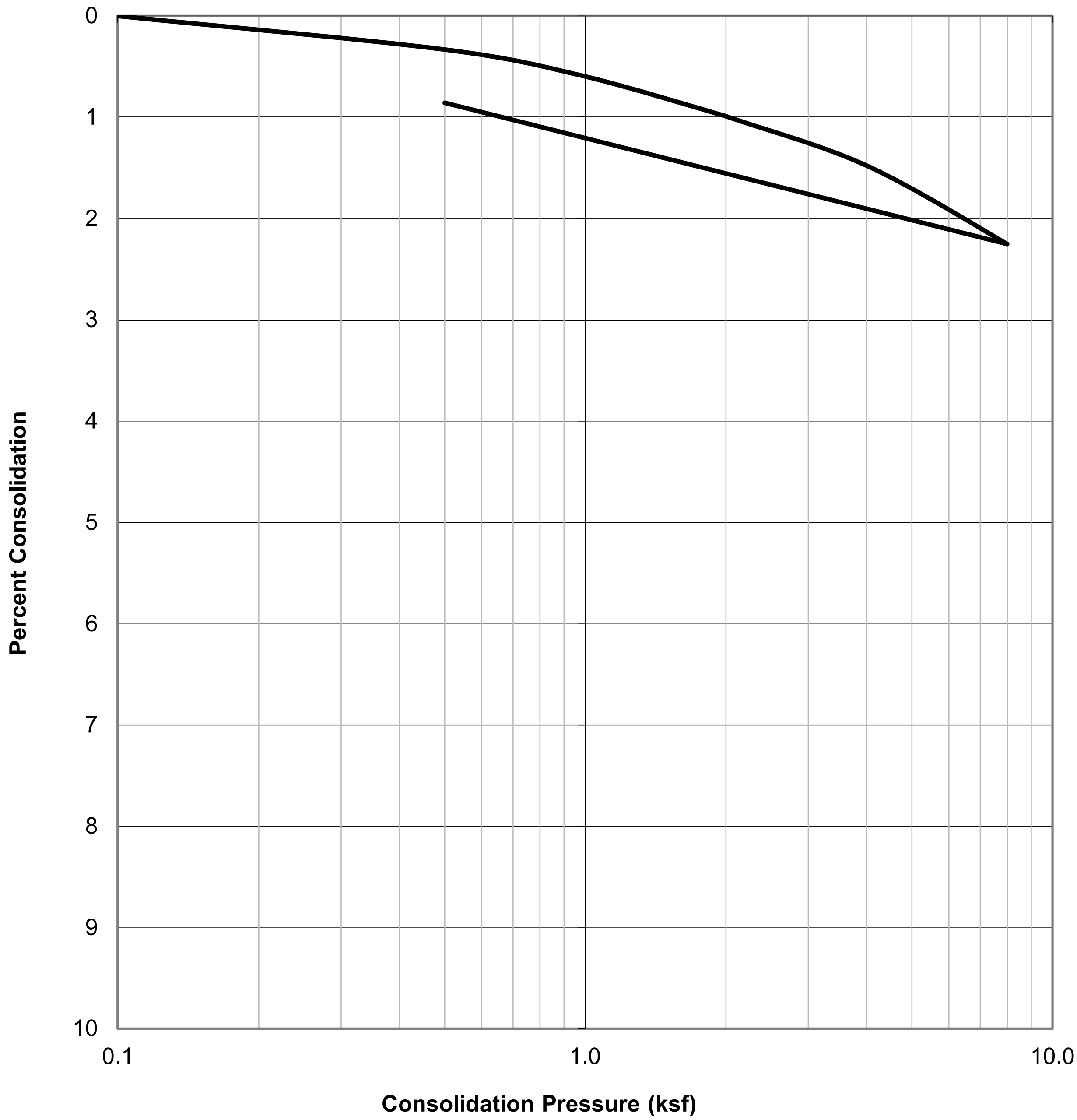
Project No.: W1815-06-01

800 & 908 NORTH MAIN STREET
1081 & 1087 NORTH VIGNES STREET
LOS ANGELES, CALIFORNIA

OCT. 2023

Figure B19

WATER ADDED AT 2.0 KSF



SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B3@45	Silt (ML)	92.8	30.7	30.7

GEOCON

CONSOLIDATION TEST RESULTS

ASTM D-2435

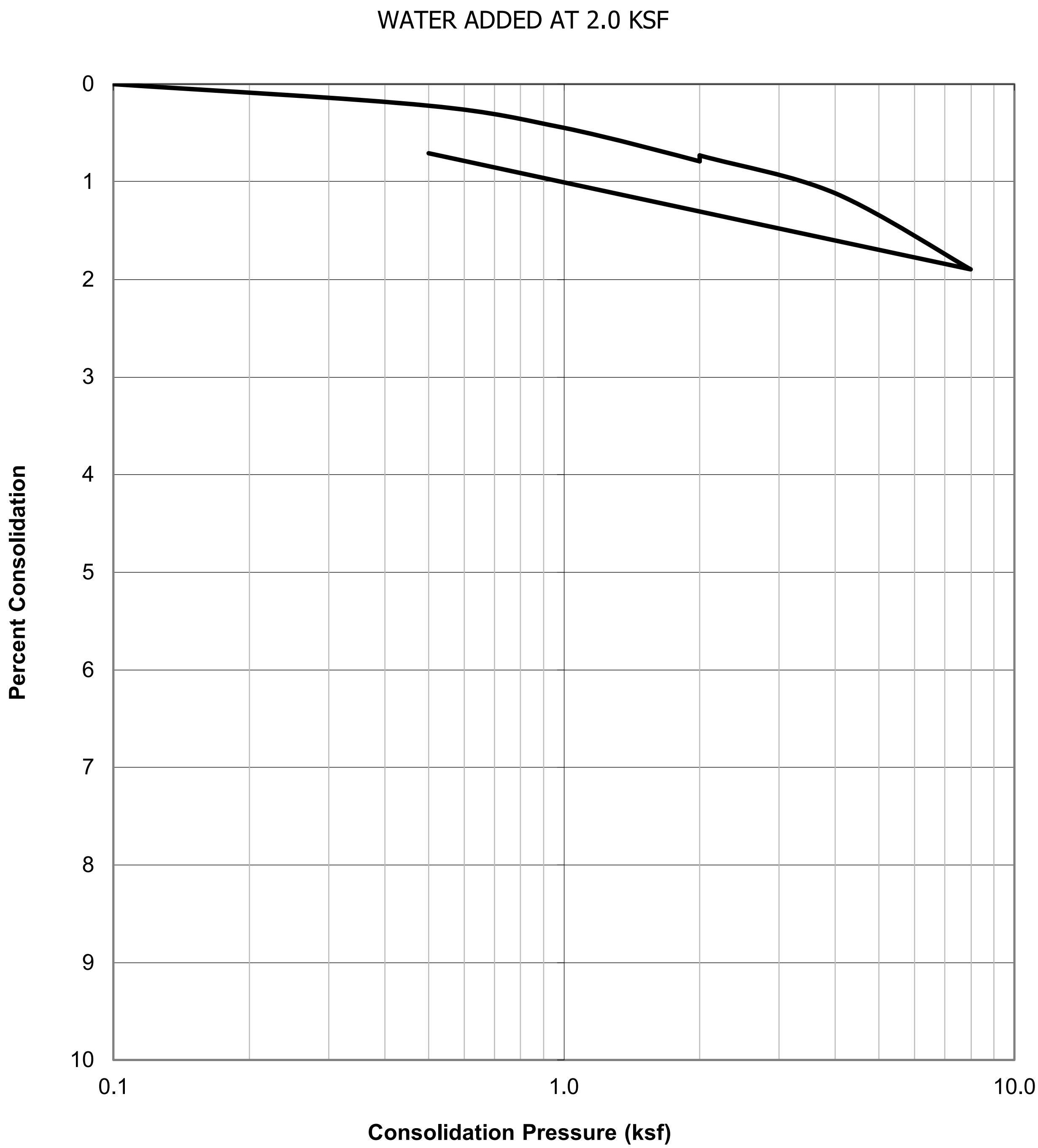
Checked by: JJK

Project No.: W1815-06-01

800 & 908 NORTH MAIN STREET
1081 & 1087 NORTH VIGNES STREET
LOS ANGELES, CALIFORNIA

OCT. 2023

Figure B20



SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B3@50	Silt (ML)	98.2	24.2	26.8

GEOCON

CONSOLIDATION TEST RESULTS

ASTM D-2435

Checked by: JJK

Project No.: W1815-06-01

800 & 908 NORTH MAIN STREET
1081 & 1087 NORTH VIGNES STREET
LOS ANGELES, CALIFORNIA

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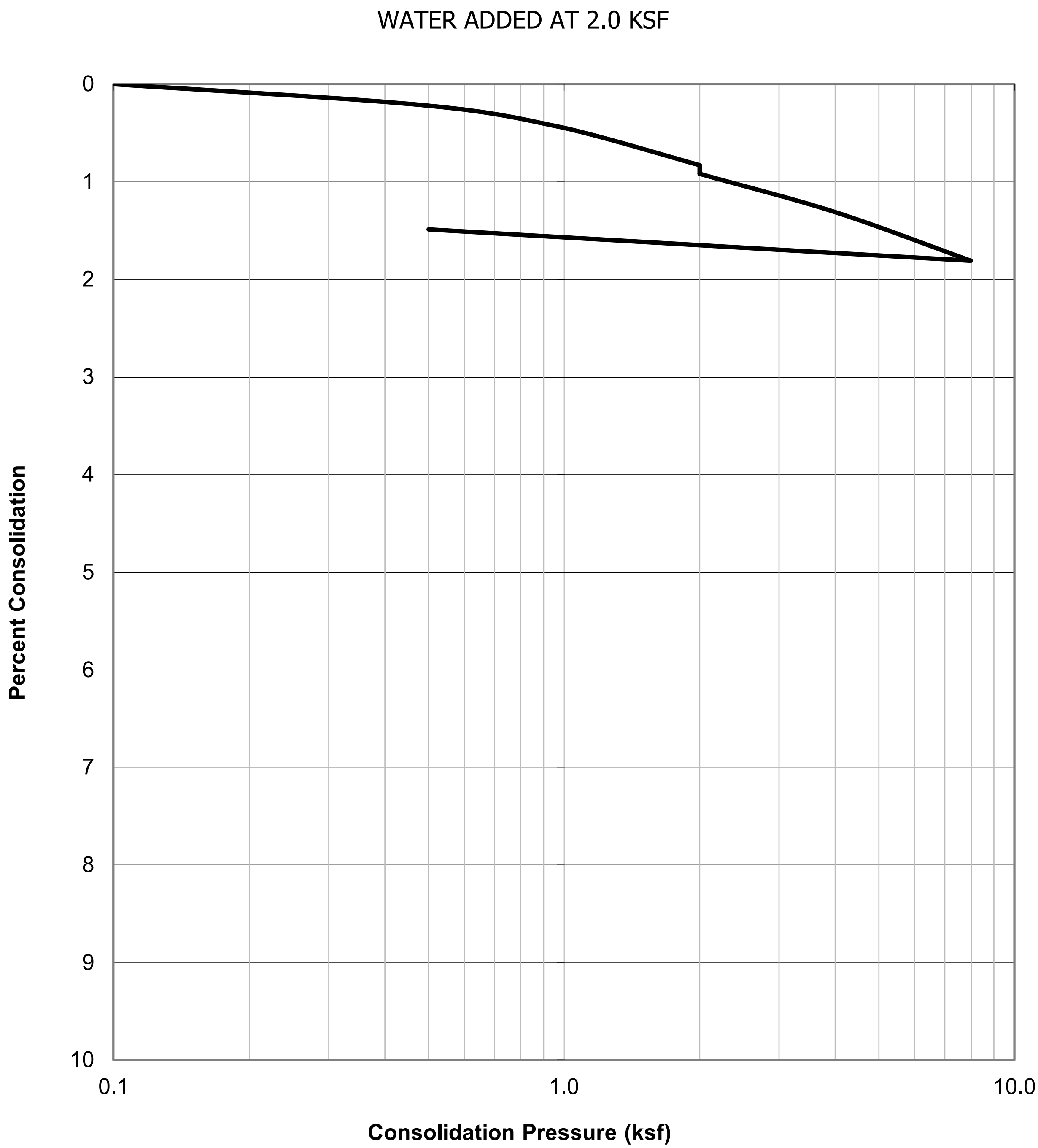
Figure B21



SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B4@32.5	Sand (SP)	130.5	10.1	10.5

GEOCON

CONSOLIDATION TEST RESULTS

ASTM D-2435

Checked by: JJK

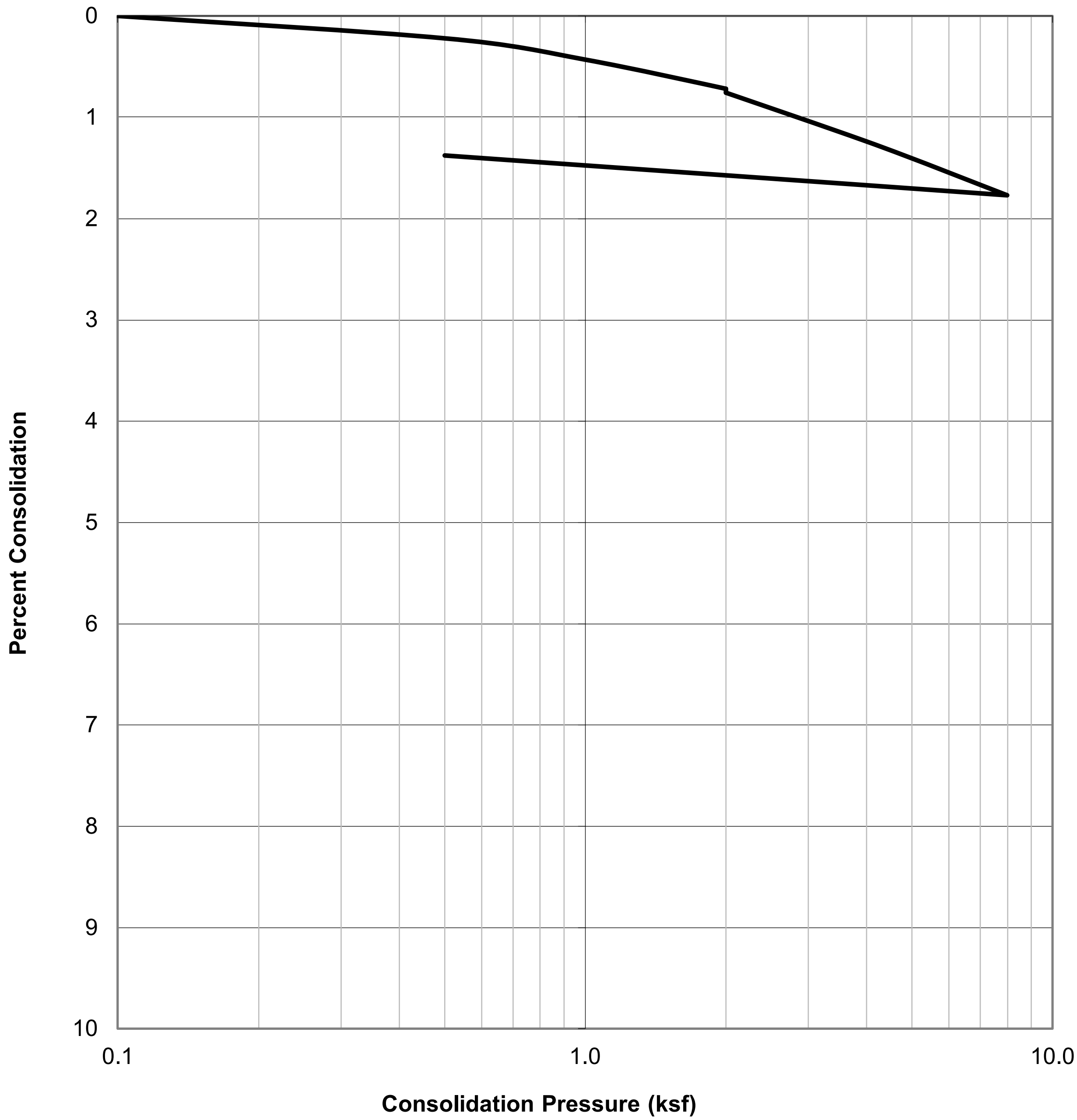
Project No.: W1815-06-01

800 & 908 NORTH MAIN STREET
1081 & 1087 NORTH VIGNES STREET
LOS ANGELES, CALIFORNIA

OCT. 2023

Figure B22

WATER ADDED AT 2.0 KSF



SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B4@47.5	Sand (SP)	133.2	9.5	10.7

GEOCON

CONSOLIDATION TEST RESULTS

ASTM D-2435

Checked by: JJK

Project No.: W1815-06-01

800 & 908 NORTH MAIN STREET
1081 & 1087 NORTH VIGNES STREET
LOS ANGELES, CALIFORNIA

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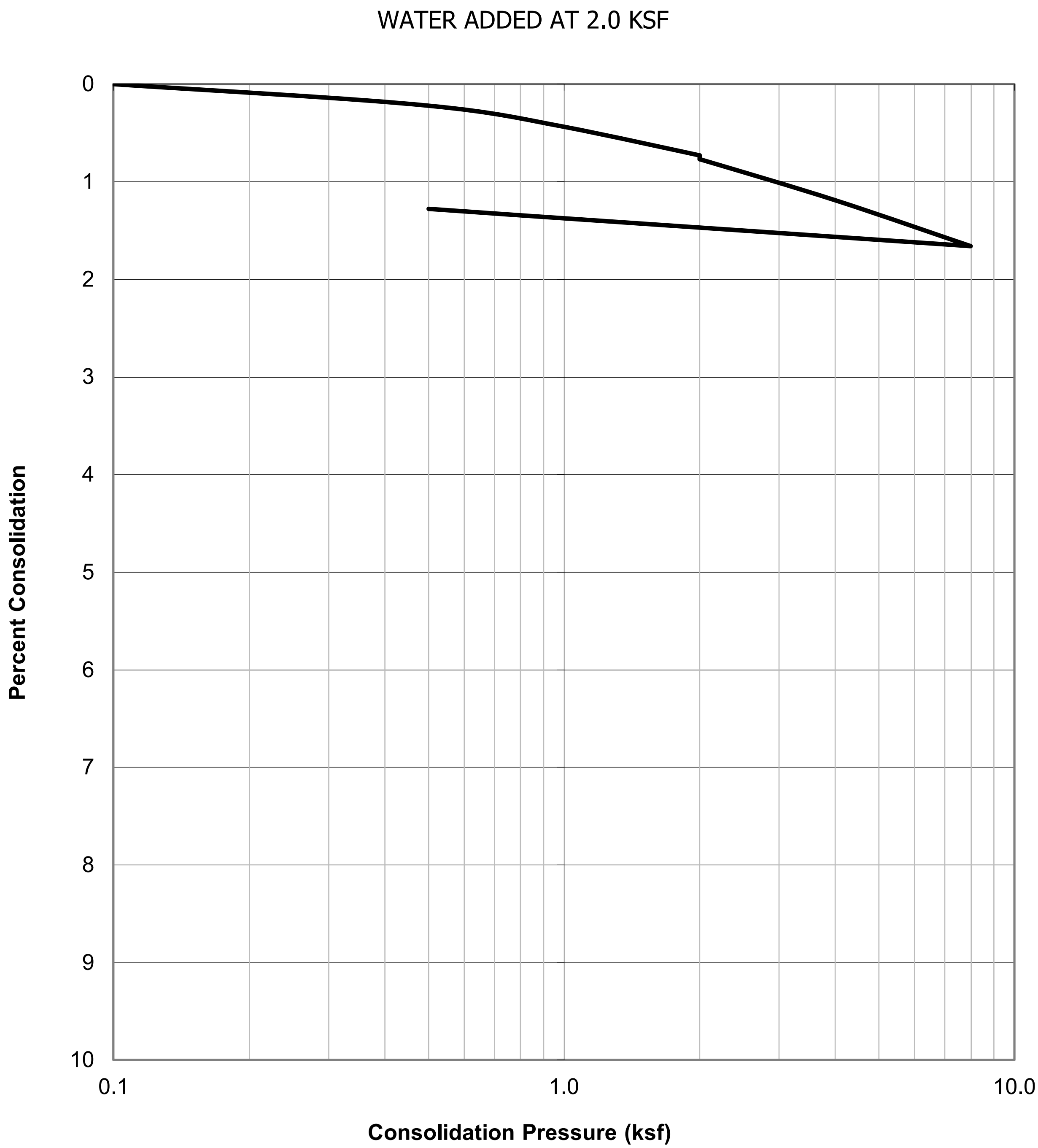

SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B4@52.5	Silty Sand (SM)	121.3	13.4	14.7

GEOCON

CONSOLIDATION TEST RESULTS

ASTM D-2435

Checked by: JJK

Project No.: W1815-06-01

800 & 908 NORTH MAIN STREET
1081 & 1087 NORTH VIGNES STREET
LOS ANGELES, CALIFORNIA

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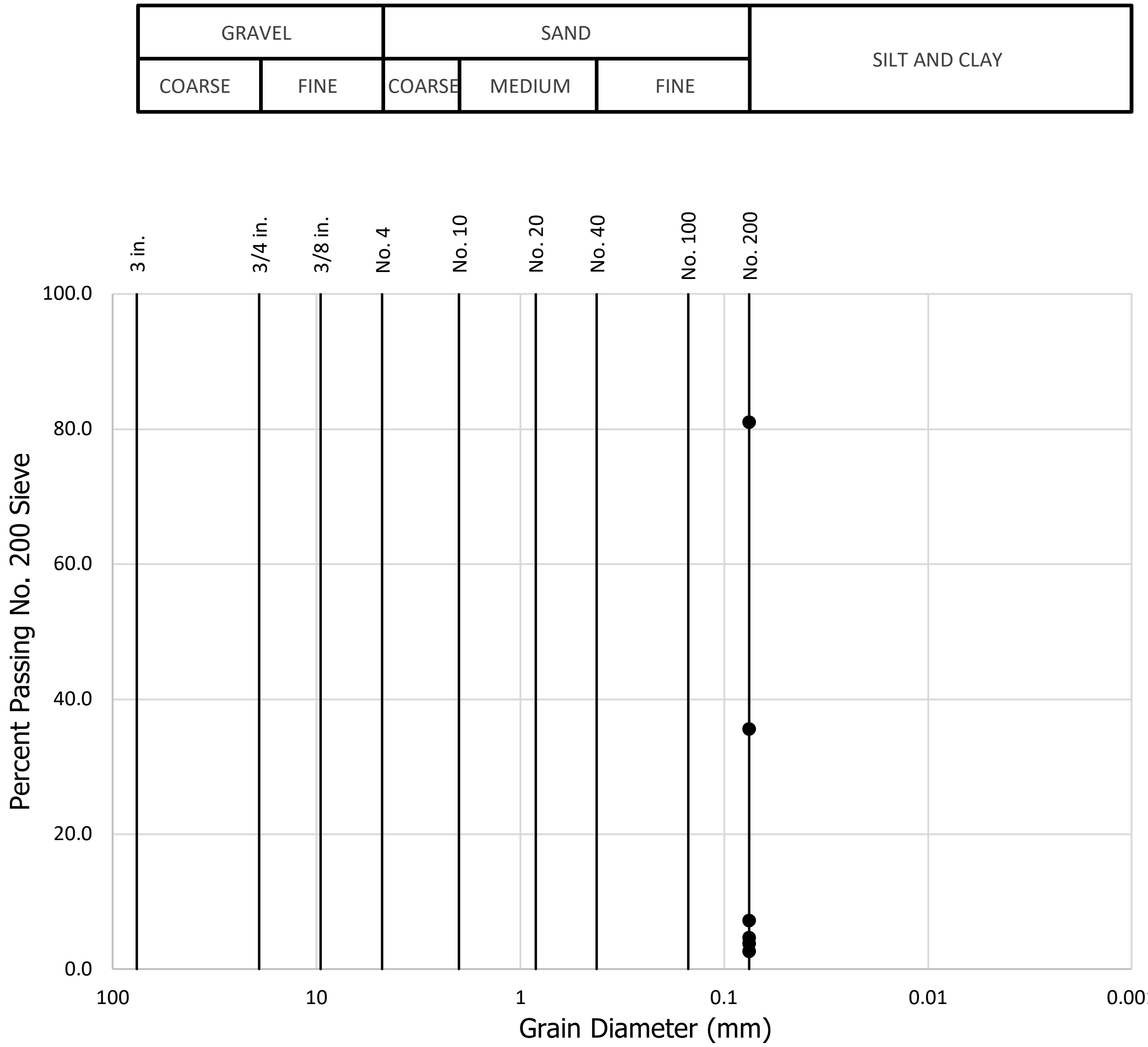
Figure B24



Sample No.	Percent Passing No. 200 Sieve
B1 @ 25'	35.6
B1 @ 30'	2.6
B1 @ 50'	81.0
B4 @ 30'	4.6
B4 @ 35'	3.8
B4 @ 40'	7.2

GEOCON

GRAIN SIZE ANALYSIS

ASTM D-1140

Checked by: JJK

Project No.: W1815-06-01

800 & 908 NORTH MAIN STREET
1081 & 1087 NORTH VIGNES STREET
LOS ANGELES, CALIFORNIA

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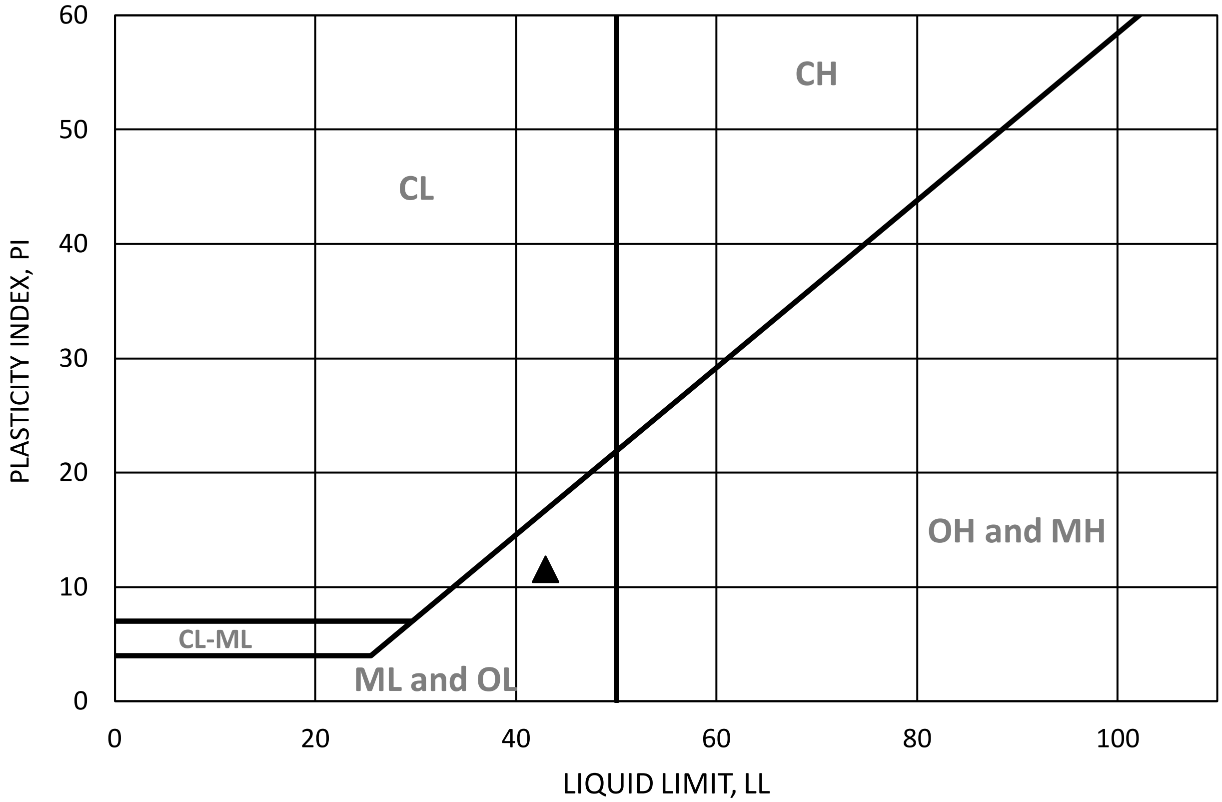
Figure B25



SYMBOL	BORING	DEPTH (ft)	LL	PL	PI	MOISTURE CONTENT AT SATURATION	SOIL BEHAVIOR
■	B1	25	N/P	N/P	N/P		N/P
◆	B1	30	N/P	N/P	N/P		N/P
▲	B1	50	43	31	12		ML
●	B4	30	N/P	N/P	N/P		N/P
□	B4	35	N/P	N/P	N/P		N/P
◇	B4	40	N/P	N/P	N/P		N/P
△							
○							

N/P = Non-Plastic

GEOCON

ATTERBERG LIMITS

ASTM D-4318

Checked by: JJK

Project No.: W1815-06-01

800 & 908 NORTH MAIN STREET
1081 & 1087 NORTH VIGNES STREET
LOS ANGELES, CALIFORNIA

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Figure B26

B1+B2@15-20'

MOLDED SPECIMEN		BEFORE TEST	AFTER TEST
Specimen Diameter	(in.)	4.0	4.0
Specimen Height	(in.)	1.0	1.0
Wt. Comp. Soil + Mold	(gm)	778.1	797.9
Wt. of Mold	(gm)	367.6	367.6
Specific Gravity	(Assumed)	2.7	2.7
Wet Wt. of Soil + Cont.	(gm)	491.8	797.9
Dry Wt. of Soil + Cont.	(gm)	468.6	378.7
Wt. of Container	(gm)	191.8	367.6
Moisture Content	(%)	8.4	13.6
Wet Density	(pcf)	123.8	129.6
Dry Density	(pcf)	114.2	114.1
Void Ratio		0.5	0.5
Total Porosity		0.3	0.3
Pore Volume	(cc)	66.7	66.8
Degree of Saturation	(%) [S_{meas}]	48.1	77.2

Date	Time	Pressure (psi)	Elapsed Time (min)	Dial Readings (in.)
9/19/2023	10:00	1.0	0	0.329
9/19/2023	10:10	1.0	10	0.3285
Add Distilled Water to the Specimen				
9/20/2023	10:00	1.0	1430	0.329
9/20/2023	11:00	1.0	1490	0.329

Expansion Index (EI meas) =	0.5
Expansion Index (Report) =	**1**

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

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

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EXPANSION INDEX TEST RESULTS

ASTM D-4829

Checked by: JJK

Project No.: W1815-06-01

800 & 908 NORTH MAIN STREET
1081 & 1087 NORTH VIGNES STREET
LOS ANGELES, CALIFORNIA

OCT. 2023

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

Sample No.	pH	Resistivity (ohm centimeters)
B1+B2@15-20	8.8	6700 (Moderately Corrosive)
B4@30-35	5.9	1000 (Severely Corrosive)

SUMMARY OF LABORATORY CHLORIDE CONTENT TEST RESULTS
AASHTO T291 ASTM C1218

Sample No.	Chloride Ion Content (%)
B1+B2@15-20	0.006
B4@30-35	0.002

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

Sample No.	Water Soluble Sulfate (% SO_4)	Sulfate Exposure
B1+B2@15-20	0.000	S0
B4@30-35	0.068	S0

GEOCON

CORROSIVITY TEST RESULTS

Checked by: JJK

Project No.: W1815-06-01

800 & 908 NORTH MAIN STREET
1081 & 1087 NORTH VIGNES STREET
LOS ANGELES, CALIFORNIA

OCT. 2023

Figure B28

GEOTECHNICAL INVESTIGATION

PROPOSED MIXED-USE AFFORDABLE HOUSING DEVELOPMENT
800 & 908 NORTH MAIN STREET
1081 & 1087 NORTH VIGNES STREET
LOS ANGELES, CALIFORNIA

TRACT: CITY LANDS OF LOS ANGELES TRACT, LOT: PT "UNNUMBERED LT", ARB: 352-355 & 402
AND
TRACT: OIL WELL SUPPLY COMPANY TRACT, LOT: FR LT A, ARB: 1 & 2

GEOCON
WEST, INC.

GEOTECHNICAL
ENVIRONMENTAL
MATERIALS

PREPARED FOR

LINC HOUSING CORPORATION
LONG BEACH, CALIFORNIA

PROJECT NO. W1814-06-01
OCTOBER 17, 2023

Project No. W1814-06-01
October 17, 2023

Ms. Cecilia Ngo
LINC Housing
3590 Elm Avenue
Long Beach, CA 90807

Subject: GEOTECHNICAL INVESTIGATION
PROPOSED MIXED-USE AFFORDABLE HOUSING DEVELOPMENT
800 & 908 NORTH MAIN STREET
1081 & 1087 NORTH VIGNES STREET
LOS ANGELES, CALIFORNIA
TRACT: CITY LANS OF LOS ANGELES, LOT: PT "UNNUMBERED LOT", ARB: 352-355 & 402 AND TRACT: OIL WELL SUPPLY COMPANY, LOT:FR LT A, ARB: 1&2

Dear Ms. Ngo:

In accordance with your authorization of our proposal dated August 3, 20223, we have performed a geotechnical investigation for the proposed mixed-use affordable housing development located at 800 and 908 North Main Street and 1081 and 1087 North Vignes Street in 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 the 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 in 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.

REGISTERED PROFESSIONAL ENGINEER
H. HARRY DERKALOUSDIAN
C 79694
CIVIL
STATE OF CALIFORNIA

PROFESSIONAL GEOLOGIST
GERALD KASMAN
NO. 2251
CERTIFIED ENGINEERING GEOLOGIST
STATE OF CALIFORNIA

Joshua Kulas	Harry Derkalousdian	Gerald Kasman
Staff Engineer	PE 79694	CEG 2251

(EMAIL) Addressee

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GEOTECHNICAL INVESTIGATION

1. PURPOSE AND SCOPE

This report presents the results of a geotechnical investigation for the proposed mixed-use affordable housing development located at 800 and 908 North Main Street and 1081 and 1087 North Vignes Street 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 design and construction.

The scope of this investigation included a site reconnaissance, field exploration, laboratory testing, engineering analysis, and the preparation of this report. The site was explored on September 7 and 8, 2023, by excavating two 7-inch diameter borings to between depths of approximately 55½ feet and 81 feet below the ground surface utilizing a truck-mounted hollow-stem auger drilling machine. Two additional 7-inch diameter borings were drilled at the adjacent site on September 6 and 7 to between depths of approximately 55½ feet and 66 feet below the ground surface utilizing a truck-mounted hollow-stem auger drilling machine. The approximate locations of the exploratory borings on both sites are depicted on the Site Plan (see Figure 2). A detailed discussion of the field investigation, including the boring logs, is presented in Appendix A.

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

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

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

2. SITE AND PROJECT DESCRIPTION

The subject site is located at 800 and 908 North Main Street and 1081 and 1087 North Vignes Street in the City of Los Angeles, California. The site is currently occupied by an asphalt paved parking lot that occupies the majority of the site with the exception of the western corner of the site, which is vacant. The ground surface in this portion of the site is covered with angular gravel and sparse vegetation. The vacant portion of the site appears to be the footprint of a former building because portions of the building's foundation remain at the site. The site is bounded by North Vignes Street to the northeast, by Rosabell Street to the southeast, by a parking lot to the southwest, and by North Main Street to the northwest. The paved portion of the site is relatively level, with no pronounced highs or lows. The gravel covered portion of the site is lower than the paved portion with a grade elevation difference of approximately 1-1½ feet. There is also a difference if the existing grade elevations along the southwest portion of the site's perimeter of approximately 1 to 3 feet. Surface water drainage at the site appears to be by sheet flow along the existing ground contours to the city streets.

Based on the information provided by the Client, it is our understanding that the design of the proposed mixed-use affordable housing development has not been finalized and that two design options are under consideration. Option A will consist of a seven-story structure. The first two levels of the proposed structure will consist of podium parking level, community care facilities and residential amenities. The structure's remaining five levels will be comprised of residential units. The entire site will be underlain by one subterranean parking level. The western portion of the site will also be improved with two new structures, but they are not a part of this phase of the development. Option B will also consist of a seven-story structure. However, the first two stories will be comprised of community care facilities and residential amenities (Option B does not have two podium parking levels). The remaining five stories will be residential units. In the Option B design, two subterranean parking levels are proposed. The first parking level (P1) will underlay the entire site, but the second, lower, parking level (P2) will have a smaller area and will be located beneath the southeastern portion of the site. (see Site Plan, Figures 2A and 2B). It is anticipated that excavations for the proposed structure with one subterranean parking level will extend to depths of approximately 17 feet below the existing ground surface, and 34 feet below the existing ground surface for two subterranean parking levels, including foundation depths and dewatering system.

Based on the preliminary nature of the design at this time, wall and column loads were not available. It is anticipated that the maximum column loads for the proposed structure will be up to 1000 kips, and the maximum wall loads will be up to 10 kips per linear foot.

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

3. GEOLOGIC SETTING

The site is located in the north-central portion of the Los Angeles Basin, a coastal plain bounded by the Santa Monica Mountains on the north, the Elysian Hills and Repetto Hills on the northeast, the Puente Hills and Whittier Fault on the east, the Palos Verdes Peninsula and Pacific Ocean on the west and south, and the Santa Ana Mountains and San Joaquin Hills on the southeast. The basin is underlain by a deep structural depression which has been filled by both marine and continental sedimentary deposits underlain by a basement complex of igneous and metamorphic composition (Yerkes, et al., 1965). Regionally, the site is located within the northern portion of the Peninsular Ranges geomorphic province. This geomorphic province is characterized by northwest-trending physiographic and geologic features such as the Newport-Inglewood Fault Zone located approximately 8.1 miles to the west.

4. SOIL AND GEOLOGIC CONDITIONS

Based on our field investigation and published geologic maps of the area, the site is underlain by artificial fill and Holocene age alluvial deposits consisting of sand and silt with varying amounts of gravel and cobbles (Dibblee, 1991; California Geological Survey, 2012). Detailed stratigraphic profiles of the materials encountered at the site are provided on the boring log in Appendix A.

4.1 Artificial Fill

Artificial fill was encountered in our field explorations to a maximum depth of 2½ feet below existing ground surface. The artificial fill generally consists of dark brown to olive gray or black sand and silt. The artificial fill is characterized as moist and soft or loose to medium dense. The fill is likely the result of past grading or construction activities at the site. Deeper fill may exist between excavations and in other portions of the site that were not directly explored.

4.2 Alluvium

Holocene age alluvial deposits were encountered beneath the fill. The alluvium consists primarily of brown to olive brown to gray interbedded sand and silt, with localized pockets of gravel and cobbles. The alluvium is characterized as moist to wet and loose medium dense to very dense or stiff to hard.

5. GROUNDWATER

A review of the Seismic Hazard Evaluation of the Los Angeles 7.5-Minute Quadrangle (California Division of Mines and Geology [CDMG], 1998), indicates that the historically highest groundwater level in the area is approximately 20 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 borings at depths of approximately 23 to 24 feet below existing ground surface. Based on the depth to groundwater encountered in our boring, and the depth of proposed construction, groundwater may be encountered during construction, based on the deeper proposed site layout. Additionally, it is not uncommon for groundwater levels to vary seasonally or for groundwater seepage conditions to develop where none previously existed, especially in impermeable fine-grained soils or on top of the bedrock, 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 the future performance of the project. Recommendations for drainage are provided in the S*urface Drainage* section of this report (see Section 7.26).

6. GEOLOGIC HAZARDS

6.1 Surface Fault Rupture

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

The site is not within a state-designated Alquist-Priolo Earthquake Fault Zone (CGS, 2021b; CGS, 2017) 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 Holocene-active fault to the site is the Hollywood Fault located approximately 3.9 miles to the north (CGS, 2017). Other nearby Holocene-active faults are the Verdugo Fault, the Newport-Inglewood Fault Zone, the Santa Monica Fault, and the Elsinore Fault located approximately 5.6 miles north, 8.1 miles west, 10½ miles west, and 14½ miles east of the site, respectively. (USGS, 2006; Ziony and Jones, 1989). The active San Andreas Fault Zone is located approximately 34 miles northeast of the site (Ziony and Jones, 1989).

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

6.2 Seismicity

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

LIST OF HISTORIC EARTHQUAKES

Earthquake (Oldest to Youngest)	Date of Earthquake	Magnitude	Distance to Epicenter (Miles)	Direction to Epicenter
Long Beach	March 10, 1933	6.4	34	SE
Tehachapi	July 21, 1952	7.5	79	NW
San Fernando	February 9, 1971	6.6	26	NNW
Whittier Narrows	October 1, 1987	5.9	9	E
Sierra Madre	June 28, 1991	5.8	19	NE
Landers	June 28, 1992	7.3	103	E
Big Bear	June 28, 1992	6.4	81	E
Northridge	January 17, 1994	6.7	20	WNW
Hector Mine	October 16, 1999	7.1	118	ENE
Ridgecrest	July 5, 2022	7.1	123	NNE

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

6.3 Seismic Design Criteria

The following table summarizes summarizes site-specific design criteria obtained from the 2022 California Building Code (CBC; Based on the 2018 International Building Code [IBC] and ASCE 7-16), Chapter 16 Structural Design, Section 1613 Earthquake Loads. The data was calculated using the online application *Seismic Design Maps*, provided by OSHPD. The short spectral response uses a period of 0.2 second. We evaluated the Site Class based on the discussion in Section 1613.2.2 of the 2022 CBC and Table 20.3-1 of ASCE 7-16. The values presented on the following page are for the risk-targeted maximum considered earthquake (MCE_R).

2022 CBC SEISMIC DESIGN PARAMETERS

Parameter	Value	2022 CBC Reference
Site Class	D	Section 1613.2.2
MCE_R Ground Motion Spectral Response Acceleration – Class B (short), S_S	1.995g	Figure 1613.2.1(1)
MCE_R Ground Motion Spectral Response Acceleration – Class B (1 sec), S_1	0.712g	Figure 1613.2.1(2)
Site Coefficient, F_A	1	Table 1613.2.3(1)
Site Coefficient, F_V	1.7*	Table 1613.2.3(2)
Site Class Modified MCE_R Spectral Response Acceleration (short), S_{MS}	1.995g	Section 1613.2.3 (Eqn 16-36)
Site Class Modified MCE_R Spectral Response Acceleration – (1 sec), S_{M1}	1.211g*	Section 1613.2.3 (Eqn 16-37)
5% Damped Design Spectral Response Acceleration (short), S_{DS}	1.33g	Section 1613.2.4 (Eqn 16-38)
5% Damped Design Spectral Response Acceleration (1 sec), S_{D1}	0.807g*	Section 1613.2.4 (Eqn 16-39)

Note:

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

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

ASCE 7-16 PEAK GROUND ACCELERATION

Parameter	Value	ASCE 7-16 Reference
Mapped MCE_G Peak Ground Acceleration, PGA	0.858g	Figure 22-7
Site Coefficient, F_{PGA}	1.1	Table 11.8-1
Site Class Modified MCE_G Peak Ground Acceleration, PGA_M	0.943g	Section 11.8.3 (Eqn 11.8-1)

Deaggregation of the MCE peak ground acceleration was performed using the USGS online Unified Hazard Tool, 2014 Conterminous U.S. Dynamic edition (v4.2.0). The result of the deaggregation analysis indicates that the predominant earthquake contributing to the MCE peak ground acceleration is characterized as a 6.85 magnitude event occurring at a hypocentral distance of 9.08 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 6.74 magnitude occurring at a hypocentral distance of 12.79 kilometers from the site.

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

6.4 Liquefaction Potential

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

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

The State of California Seismic Hazard Zone Map for the Los Angeles Quadrangle (CDMG, 1999; CGS, 2014) indicates that the site is located within an area identified as having a potential for liquefaction. Also, according to the Los Angeles County Safety Element (Leighton, 1990), 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. In order to supplement the SPT blow count data, California Modified Sampler blow count data were converted to equivalent SPT blow counts based on a correlation factor of 0.55 (Rogers, 2006).

Screening criteria developed by Bray and Sancio (2006) characterize fine-grained soils which are not susceptible to liquefaction as soils with a plasticity index (PI) that is greater than 18 or with a saturated moisture content that is less than 80 percent of the liquid limit. In order to apply the screening criteria, laboratory testing was performed to evaluate the Atterberg Limits of select soil samples. Laboratory test results used for the screening criteria are presented as Figure B26.

The liquefaction analysis for a structure with one subterranean level, extending two a depth of 15 feet below the ground surface, or for a structure with two subterranean levels, extending to a depth of 30 feet below the ground surface, was performed for a Design Earthquake level by using a historic high groundwater table of 20 feet below the ground surface, a magnitude 6.74 earthquake, and a peak horizontal acceleration of 0.629g (⅔PGA_M). The enclosed liquefaction analysis for a structure with one or two subterranean levels, included herein for borings B1 and B4, indicate that the alluvial soils below the historic high groundwater level could be susceptible to up to approximately 0.7 inch of total settlement during Design Earthquake ground motion (see enclosed calculation sheets, Figures 5 through 8).

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 for a structure with one subterranean level, extending two a depth of 15 feet below the ground surface, or for a structure with two subterranean levels, extending to a depth of 30 feet below the ground surface, was also performed for the Maximum Considered Earthquake level by using a historic high groundwater table of 20 feet below the ground surface, a magnitude 6.85 earthquake, and a peak horizontal acceleration of 0.943g (PGA_M). The enclosed liquefaction analysis for a structure with one or two subterranean levels, included herein for borings B1 and B4, indicate that the alluvial soils below the historic high groundwater level could be susceptible to up to approximately 0.7 inch of total settlement during Maximum Considered ground motion (see enclosed calculation sheets, Figures 9 through 12).

6.5 Seismically Induced Dry Settlement

Dynamic compaction of dry and loose sands may occur during a major earthquake. Typically, settlements occur in thick beds of such soils. The seismically-induced settlement calculations were performed in accordance with the American Society of Civil Engineers, Technical Engineering and Design Guides as adapted from the US Army Corps of Engineers, No. 9.

The calculation for a structure with one subterranean level that will extend to a depth of approximately 15 feet below the ground surface. The calculations provided herein for borings B1 and B4, indicate that the soil above the historic high groundwater level of 20 feet could be susceptible to approximately 0.02 inch of settlement as a result of the Design Earthquake peak ground acceleration (⅔PGA_M), and is considered negligible. The calculations provided herein for borings B1 and B4, indicate that the soil above the historic high groundwater level of 20 feet could be susceptible to approximately 0.03 inch of settlement as a result of the Maximum Considered Earthquake peak ground acceleration (PGA_M), and is considered negligible.

Dry seismically-induced settlement calculations for a structure with two subterranean levels were not performed because the subterranean excavation will extend to a depth of approximately 30 feet which is below the existing ground water level at the site and the saturated soils would not be prone to seismically-induced dry settlement as a result of the Design Earthquake peak ground acceleration (⅔PGA_M) nor as a result of the Maximum Considered Earthquake peak ground acceleration (PGA_M).

6.6 Slope Stability

The topography at the site is relatively level and the topography in the vicinity of the site slopes gently to the west. The site is not located within a City of Los Angeles Hillside Grading Area or a Hillside Ordinance Area (City of Los Angeles, 2022). Additionally, the site is not located within an area identified as having a potential for seismic slope instability (CDMG, 1999; CGS, 2014). There are no known landslides near the site, nor is the site in the path of any known or potential landslides. Therefore, the potential for slope stability hazards to adversely affect the proposed development is considered low.

6.7 Earthquake-Induced Flooding

Earthquake-induced flooding is inundation caused by failure of dams or other water-retaining structures due to earthquakes. Based on a review of the County of Los Angeles Safety Element (Leighton, 1990), the site is located within the Mulholland Dam and Hansen 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.

6.8 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 seismically induced seiche is considered unlikely.

The site is within an area of minimal flooding (Zone X) as defined by the Federal Emergency Management Agency (LACDPW, 2023; FEMA, 2023).

6.9 Oil Fields & Methane Potential

Based on a review of the California Geologic Energy Management Division (CalGEM) Well Finder Website, the site is not located within an oil field and oil or gas wells are not documented in the immediate site vicinity (CalGEM, 2023). However, due to the voluntary nature of record reporting by the oil well drilling companies, wells may be improperly located or not documented on the location map 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 located within the boundaries of a city-designated Methane Buffer Zone (City of Los Angeles, 2023). Should it be determined that a methane study is required for the proposed development, it is recommended that a qualified methane consultant be retained to perform the study and provide mitigation measures as necessary.

6.10 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.

7. CONCLUSIONS AND RECOMMENDATIONS

7.1 General

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

7.1.2 Up to 9 feet of existing artificial fill was encountered during the site investigation. Deeper fill may exist in other areas of the site that were not directly explored. The existing fill encountered is believed to be the result of past grading and construction activities at the site. It is our opinion that the existing fill, in its present condition, is not suitable for direct support of proposed foundations or slabs. The existing fill and site soils are suitable for re-use as engineered fill provided the recommendations in the *Grading* section of this report are followed (see Section 7.5). Excavation for the subterranean level(s) is anticipated to penetrate through the existing artificial fill and expose undisturbed alluvial soils throughout the excavation bottom.

7.1.3 The enclosed seismic settlement analyses indicate that the site soils could be susceptible to up to approximately 0.7 inch of total settlement as a result of a Design Earthquake peak ground acceleration (⅔PGA_M). Differential settlement at the foundation level is anticipated to be approximately 0.35 inch over a distance of 20 feet.

7.1.4 Static groundwater was encountered during site exploration at depths of approximately 23 to 24 feet below existing ground surface. Historic high groundwater at the site is approximately 20 feet below the ground surface. Excavation is anticipated to extend to a maximum depth of approximately 17 feet below the ground surface for construction of one subterranean level option, or approximately 34 feet below the ground surface for construction of the two subterranean levels option, including foundation and dewatering system excavations. Based on the conditions encountered at the time of exploration, groundwater is anticipated to be encountered during construction for a structure with two-subterranean levels that extends to a depth of 34 feet below ground surface, including foundation excavation and dewatering system. For a proposed structure with one subterranean level that is 17 feet in depth, the current static groundwater table is sufficiently deep that it not expected to be encountered during construction with the exception of a deep drilled excavation such as for a shoring pile or elevator piston. However, local seepage could be encountered during excavation of the subterranean level, especially if conducted during the rainy season.

7.1.5 The City of Los Angeles Building Code requires that the structure be designed for the historically high groundwater level, which is approximately 20 feet below the existing ground surface. The proposed structure must be designed for hydrostatic pressure for any portion of the structure below a depth of 20 feet. 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. If the proposed structure does not provide sufficient dead load to resist the buoyant forces then uplift mitigation will be required.

7.1.6 Based on these considerations, it is recommended that the proposed structure be supported on a mat foundation system deriving support in competent alluvial soils found at and below a depth of 15 feet below the existing ground surface. In order to minimize differential settlement between the ramp, ramp walls, and basement level, it is recommended that the ramp and ramp walls for the subterranean parking garage be structurally supported on the mat foundation. In addition, the transition area between the one-subterranean level portion to the two-subterranean level portion (Option B) of the structure should be more heavily reinforced to resist differential settlement stresses which could cause cracking. All foundation excavations must be observed and approved by the Geotechnical Engineer (a representative of Geocon), prior to placing steel or concrete. Recommendations for the design of a mat foundation system are provided in Sections 7.7 and 7.8.

7.1.7 Where new foundations are constructed immediately adjacent to existing foundations, the new foundation should be deepened to match the depth of the existing foundation to prevent a surcharge on the existing foundation.

7.1.8 Where proposed foundations will be deeper than an existing foundation, the new foundation must be designed to resist the surcharge imposed by the existing foundation. The surcharge area may be defined by a 1:1 projection down and away from the bottom of the existing foundation.

7.1.9 It should be noted that implementation of the recommendations presented herein is not intended to completely prevent damage to the structure during the occurrence of strong ground shaking as a result of nearby earthquakes. It is intended that the structure be designed in such a way that the amount of damage incurred as a result of strong ground shaking be minimized.

7.1.10 Excavations up to 17 feet in vertical height are anticipated for construction of a structure with one subterranean level or up to 34 feet in vertical height for construction of a structure with two subterranean levels, including foundation depths and dewatering system. Due to the depth of the excavation and the proximity to the property lines, city streets and adjacent offsite structures and improvements, excavation of the proposed subterranean levels will require sloping and/or shoring in order to provide a stable excavation. Where shoring is required, it is recommended that a soldier pile shoring system be utilized. In addition, where the proposed excavation will be deeper than and adjacent to an offsite structure, the proposed shoring should be designed to resist the surcharge imposed by the adjacent offsite structure. Recommendations for shoring are provided in Section 7.21 of this report.

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

7.1.12 Foundations for small outlying structures, such as block walls up to 6 feet high, planter walls or trash enclosures, which will not be tied to the proposed structure, may be supported on conventional foundations bearing on a minimum of 12 inches of newly placed engineered fill which extends laterally at least 12 inches beyond the foundation area. Where excavation and proper compaction cannot be performed, foundations may derive support directly in the undisturbed alluvial soils at and below a depth of 24 inches below the existing ground surface and should be deepened as necessary to maintain a minimum 12-inch embedment into the 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 in writing by a Geocon representative.

7.1.13 Where new paving is to be placed, it is recommended that all existing fill soils and soft alluvial soils be excavated and properly compacted for paving support. The client should be aware that excavation and compaction of all existing fill in the area of new paving is not required; however, paving constructed over existing uncertified fill or unsuitable soils may experience increased settlement and/or cracking and may therefore have a shorter design life and increased maintenance costs. As a minimum, the upper 12 inches of soil should be scarified and properly compacted. Paving recommendations are provided in the *Preliminary Pavement Recommendations* section of this report (see Section 7.14).

7.1.14 Based on the historic and current groundwater levels as well as the potential for liquefaction of the site soils, stormwater infiltration is not recommended for this project. It is suggested that stormwater be retained, filtered and discharged in accordance with the requirements of the local governing agency.

7.1.15 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.

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

7.2 Soil and Excavation Characteristics

7.2.1 The in-situ soils can be excavated with moderate effort using conventional excavation equipment. Some caving should be anticipated in unshored excavations, especially where granular soils are encountered. The contractor should be aware that casing will be required during shoring pile installation.

7.2.2 Please be aware that the use of hollow-stem auger drilling equipment utilized for this investigation does not allow for the identification of the size of rock or abundance of rock being encountered or the visual observation of caving conditions since the drilling method is a small diameter cased excavation. It is recommended that the contractors bidding on excavation and shoring installation for this project perform their own excavations and test borings with the intended earthwork and drilling equipment to verify the presence, abundance, and size of buried rock (cobbles and boulders), potential for caving, as well as the suitability of the proposed excavation and drilling equipment for the safe and efficient earthwork operations and installation of the shoring system.

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

7.2.4 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 or shoring. Excavation recommendations are provided in the *Temporary Excavations* section of this report (see Section 7.20).

7.2.5 The existing site soils encountered at proposed foundation level during this investigation are considered to have a "low" expansive potential (EI = 1); and the soils are classified as "non-expansive" based on the 2022 California Building Code (CBC) Section 1803.5.3. Recommendations presented herein assume that the foundations and slabs will derive support in these materials.

7.3 Minimum Resistivity, pH and Water-Soluble Sulfate

7.3.1 Potential of Hydrogen (pH) and resistivity testing as well as chloride content testing were performed on representative samples of soil to generally evaluate the corrosion potential to surface utilities. The tests were performed in accordance with California Test Method Nos. 643 and 422 and indicate that the soils are considered "moderately" to "severely corrosive" with respect to corrosion of buried ferrous metals on site. The results are presented in Appendix B (Figure B28) and should be considered for design of underground structures. Due to the corrosive potential of the soils, it is recommended that PVC, ABS or other approved plastic piping be utilized in lieu of cast-iron when in direct contact with the site soils.

7.3.2 Laboratory tests were performed on representative samples of the site soils to measure the percentage of water-soluble sulfate content. Results from the laboratory water-soluble sulfate tests are presented in Appendix B (Figure B28) and indicate that the on-site materials possess a sulfate exposure class of "S0" to concrete structures as defined by 2022 CBC Section 1904 and ACI 318-14 Table 19.3.1.1.

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

7.4 Temporary Dewatering

7.4.1 Groundwater was encountered at a depth of approximately 23 to 24 feet below ground surface during site exploration. Based on the conditions encountered at the time of exploration, groundwater is anticipated to be encountered during construction for a structure with two-subterranean levels that extends to a depth of 34 feet below ground surface, including foundation excavation and dewatering system. For a proposed structure with one subterranean level that is 17 feet in depth, the current static groundwater table is sufficiently deep that it not expected to be encountered during construction with the exception of a deep drilled excavation such as for a shoring pile or elevator piston. However, local seepage could be encountered during excavation of the subterranean level, especially if conducted during the rainy season. The depth to groundwater at the time of construction can be further verified during initial dewatering well or shoring pile installations. If groundwater is present above the depth of the subterranean level(s), temporary dewatering will be necessary to maintain a safe working environment during excavation and construction activities.

7.4.2 If dewatering is required, it is recommended the project engage the services of a competent dewatering consultant to develop a dewatering system, calculate the design flow rates required for dewatering, and acquire the NPDES permit for water discharge. Initiating the permit application process well in advance of construction is recommended, as the California State Water Resources Control Board requires adequate time to review and authorize permits. Temporary dewatering typically consists 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 will be determined by qualified dewatering consultant.

7.4.3 Based on prior experiences with the City of Los Angeles Department of Building and Safety, Grading Division, additional engineering analyses be required to evaluate the potential impacts the proposed dewatering at the subject site will have on the adjacent structures and public streets. The additional analyses will determine the anticipated dewatering drawdown curve and resulting settlements that may occur due to the dewatering. If required, the drawdown and settlement analysis will be provided under separate cover.

7.4.4 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 functional on a permanent basis, it must be lined with filter fabric to prevent soil migration into the gravel.

7.5 Grading

7.5.1 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, if applicable, building official in attendance. Special soil handling requirements can be discussed at that time.

7.5.2 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.

7.5.3 Please be aware that the use of hollow-stem auger drilling equipment utilized for this investigation does not allow for the identification of the size of rock or abundance of rock being encountered or the visual observation of caving conditions since the drilling method is a small diameter cased excavation. It is recommended that the contractors bidding on excavation and shoring installation for this project perform their own excavations and test borings with the intended earthwork and drilling equipment to verify the presence, abundance, and size of buried rock (cobbles and boulders), potential for caving, as well as the suitability of the proposed excavation and drilling equipment for the safe and efficient earthwork operations and installation of the shoring system.

7.5.4 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. In accordance with City policy, asphalt and concrete should not be mixed into the structural fill. 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 the City of Los Angeles Inspector.

7.5.5 If subgrade stabilization is required at the excavation bottom, tire equipment should not be allowed in the excavation bottom until it is stabilized or extensive soil disturbance could result. In addition, the use of track equipment should be considered to minimize disturbance to the soils if they become wet at the excavation bottom. Bottom stabilization, if necessary, may be achieved placing 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.).

7.5.6 The City of Los Angeles Department of Building and Safety requires a minimum compactive effort of 95 percent of the laboratory maximum dry density in accordance with ASTM D 1557 (latest edition) where the soils to be utilized in the fill have less than 15 percent finer than 0.005 millimeter. Soils with more than 15 percent finer than 0.005 millimeter may be compacted to 90 percent of the laboratory maximum dry density in accordance with ASTM D 1557 (latest edition). Fill and backfill soils should be placed in horizontal loose layers approximately 6 to 8 inches thick, moisture conditioned to near optimum moisture content and properly compacted in accordance with ASTM D 1557 (latest edition).

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

7.5.8 Where new paving is to be placed, it is recommended that all existing fill and soft alluvial soils be excavated and properly compacted for paving support. The client should be aware that excavation and compaction of all existing fill and soft soils in the area of new paving is not required; however, paving constructed over existing uncertified fill or unsuitable alluvial soil may experience increased settlement and/or cracking, and may therefore have a shorter design life and increased maintenance costs. As a minimum, the upper 12 inches of 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 7.14).

7.5.9 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 20 and corrosivity properties that are equally or less detrimental to that of the existing onsite soils (see Figure B28).

7.5.10 Utility trenches should be properly backfilled in accordance with the following requirements. The pipe should be bedded with clean sands (Sand Equivalent greater than 30) to a depth of at least 1 foot over the pipe, and the bedding material must be inspected and approved in writing by the Geotechnical Engineer (a representative of Geocon). The use of gravel is not acceptable unless used in conjunction with filter fabric to prevent the gravel from having direct contact with soil. 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 as backfill is also acceptable (see Section 7.6). 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).

7.5.11 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 sands, fill, steel, gravel, or concrete.

7.6 Controlled Low Strength Material (CLSM)

7.6.1 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 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.

7.7 Mat Foundation Design – One Subterranean Level

7.7.1 The mat foundation system may derive support in the competent undisturbed alluvial soils at and below a depth of 15 feet below the existing ground surface. Any exposed soft soils should be compacted to a dense state or penetrated by proposed foundations at the direction of the Geotechnical Engineer (a representative of Geocon).

7.7.2 Where new foundations are constructed immediately adjacent to existing foundations, the new foundation should be deepened to match the depth of the existing foundation to prevent a surcharge on the existing foundation.

7.7.3 Where proposed foundations will be deeper than the existing foundation, the proposed foundation must be designed to resist the surcharge imposed by the existing foundation. The surcharge area may be defined by a 1:1 projection up and away from the bottom of an existing foundation.

7.7.4 The recommended maximum allowable bearing value for the design of a reinforced concrete mat foundation is 6,500 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.

7.7.5 It is recommended that a modulus of subgrade reaction of 150 pounds per cubic inch (pci) be utilized for the design of the mat foundation bearing in undisturbed alluvial soils. This value is a unit value 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)

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

7.7.7 For seismic design purposes, a coefficient of friction of 0.45 may be utilized between concrete slab and subgrade soils without a moisture barrier, and 0.15 for slabs underlain by a moisture barrier.

7.7.8 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.

7.7.9 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.

7.8 Mat Foundation Design – Two Subterranean Levels

7.8.1 The mat foundation system may derive support in the competent undisturbed alluvial soils at and below a depth of 30 feet below the existing ground surface. Any exposed soft soils should be compacted to a dense state or penetrated by proposed foundations at the direction of the Geotechnical Engineer (a representative of Geocon). In addition, the transition area between the one-subterranean level portion to the two-subterranean level portion of the structure should be more heavily reinforced to resist differential settlement stresses which could cause cracking.

7.8.2 The City of Los Angeles Building Code requires that the structure be designed for the historically high groundwater level, which is approximately 20 feet below the existing ground surface. The proposed structure must be designed for hydrostatic pressure for any portion of the structure below a depth of 20 feet. 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. If the proposed structure does not provide sufficient dead load to resist the buoyant forces then uplift mitigation will be required.

7.8.3 Where new foundations are constructed immediately adjacent to existing foundations, the new foundation should be deepened to match the depth of the existing foundation to prevent a surcharge on the existing foundation.

7.8.4 Where proposed foundations will be deeper than the existing foundation, the proposed foundation must be designed to resist the surcharge imposed by the existing foundation. The surcharge area may be defined by a 1:1 projection up and away from the bottom of an existing foundation.

7.8.5 The recommended maximum allowable bearing value for the design of a reinforced concrete mat foundation is 3,500 pounds per square foot (psf) (this value have been adjusted for buoyant forces). The allowable bearing pressure may be increased by up to one-third for transient loads due to wind or seismic forces.

7.8.6 It is recommended that a modulus of subgrade reaction of 125 pounds per cubic inch (pci) be utilized for the design of the mat foundation bearing in undisturbed alluvial soils. This value is a unit value 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)

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

7.8.8 For seismic design purposes, a coefficient of friction of 0.45 may be utilized between concrete slab and subgrade soils without a moisture barrier, and 0.15 for slabs underlain by a moisture barrier.

7.8.9 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.

7.8.10 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.

7.9 Foundation Settlement

7.9.1 The enclosed liquefaction settlement analyses indicate that the site soils could be susceptible up to approximately 0.7 inch of total settlement as a result of the Design Earthquake peak ground acceleration (⅔PGA_M). The differential settlement at the foundation level is anticipated to be less than 0.35 inch over a distance of 20 feet. These settlements are in addition to the static settlements indicated below and must be considered in the structural design.

7.9.2 The maximum expected static settlement for on a reinforced concrete mat foundation with a maximum allowable bearing pressure of 6,500 psf deriving support in competent alluvial soils is expected to be approximately than 1¼ inches and occur below the heaviest loaded structural element. Differential settlement is expected to be less than 0.63 inch between the center and corner of the mat foundation. A majority of the settlement of the foundation system is expected to occur on initial application of loading; however, minor additional settlements are expected within the first twelve months. Based on seismic considerations, the proposed structure supported on a mat foundation system should be designed for a combined static and seismically induced differential settlement of approximately 1 inch over a distance of 20 feet.

7.9.3 The maximum expected static settlement for on a reinforced concrete mat foundation with a maximum allowable bearing pressure of 9,500 psf deriving support in competent alluvial soils at and below a depth of 30 feet is expected to be approximately than 1¼ inches and occur below the heaviest loaded structural element. Differential settlement is expected to be less than 0.63 inch between the center and corner of the mat foundation. A majority of the settlement of the foundation system is expected to occur on initial application of loading; however, minor additional settlements are expected within the first twelve months. Based on seismic considerations, the proposed structure supported on a mat foundation system should be designed for a combined static and seismically induced differential settlement of approximately 1 inch over a distance of 20 feet.

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

7.10 Uplift Resistance

7.10.1 Foundation uplift may be resisted by the weight of structure, as well as 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 additional piles constructed within the interior of the structure. In order to maximize capacity it is suggested that post-grouted friction piles be considered. If it is determined that recommendations for uplift resistance are required as a part of this project, the recommendations will be provided under separate cover.

7.11 Miscellaneous Foundations

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

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

7.11.3 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.

7.12 Lateral Design

7.12.1 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 0.45 may be used with the dead load forces in the new placed engineered fill or competent alluvial soils.

7.12.2 Passive earth pressure for the sides of foundations and slabs poured against newly placed engineered fill or the alluvial soils may be computed as an equivalent fluid having a density of 350 pcf with a maximum earth pressure of 3,500 pcf. 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 140 pounds per cubic foot with a maximum earth pressure of 1,400 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.

7.13 Exterior Concrete Slabs-on-Grade

7.13.1 Exterior concrete slabs-on-grade at the ground surface subject to vehicle loading should be designed in accordance with the recommendations in the *Preliminary Pavement Recommendations* section of this report (see Section 7.14).

7.13.2 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 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) and should be installed in general conformance with ASTM E 1643 (latest edition) and the manufacturer's recommendations. A minimum thickness of 15 mils extruded polyolefin plastic is recommended; vapor retarders which contain recycled content or woven materials are not recommended. The vapor retarder should have a permeance of less than 0.01 perms demonstrated by testing before and after mandatory conditioning. The vapor retarder should be installed in direct contact with the concrete slab with proper perimeter seal. If the California Green Building Code requirements apply to this project, the vapor retarder should be underlain by 4 inches of clean aggregate. It is important that the vapor retarder be puncture resistant since it will be in direct contact with angular gravel. As an alternative to the clean aggregate suggested in the Green Building Code, it is our opinion that the concrete slab-on-grade may be underlain by a vapor retarder over 4 inches of clean sand (sand equivalent greater than 30), since the sand will serve a capillary break and will minimize the potential for punctures and damage to the vapor barrier.

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

7.13.4 The moisture content of the slab subgrade should be maintained and sprinkled as necessary to maintain a moist condition as would be expected in any concrete placement.

7.13.5 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.

7.14 Preliminary Pavement Recommendations

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

7.14.2 The following pavement sections are based on an assumed R-Value of 30. Once site grading activities are complete an R-Value should be obtained by laboratory testing to confirm the properties of the soils serving as paving subgrade, prior to placing pavement.

7.14.3 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	10.0

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

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

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

7.15 Retaining Wall Design

7.15.1 The recommendations presented below are generally applicable to the design of rigid concrete or masonry retaining walls having a maximum height of 30 feet. In the event that walls higher than 30 feet are planned, Geocon should be contacted for additional recommendations.

7.15.2 Retaining wall foundations may be designed in accordance with the recommendations provided in the *Mat Foundation Design* sections of this report (see Sections 7.7 and 7.8).

7.15.3 Retaining walls with a level backfill surface that are not restrained at the top should be designed utilizing a triangular distribution of pressure (active pressure). Restrained walls are those that are not allowed to rotate more than 0.001H (where H equals the height of the retaining portion of the wall in feet) at the top of the wall. Where walls are restrained from movement at the top, walls may be designed utilizing a triangular distribution of pressure (at-rest pressure). The table on the following page presents recommended pressures to be used in retaining wall design, assuming that proper drainage will be maintained. The calculations of the retaining wall pressures are presented on Figures 13A and 13B.

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 15	43	52
Between 16 and 30	52	56

7.15.4 The wall pressures provided above assume that the retaining wall will be properly drained preventing the buildup of hydrostatic pressure. If retaining wall drainage is not implemented, the equivalent fluid pressure to be used in design of undrained walls is 90 pcf. The value includes hydrostatic pressures plus buoyant lateral earth pressures. If a partially drained wall is proposed, Geocon should be contacted to provide additional recommendations.

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

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

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

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

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

and

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

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

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

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

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

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

and

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

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

then

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

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

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

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

7.16 Dynamic (Seismic) Lateral Forces

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

7.16.2 A seismic load of 11 pcf should be used for design of walls that support more than 6 feet of backfill in accordance with Section 1803.5.12 of the 2022 CBC. The seismic load is applied as an equivalent fluid pressure along the height of the wall and the calculated loads result in a maximum load exerted at the base of the wall and zero at the top of the wall. This seismic load should be applied in addition to the active earth pressure. The earth pressure is based on half of two thirds of PGA_M calculated from ASCE 7-16 Section 11.8.3.

7.17 Retaining Wall Drainage

7.17.1 Unless designed for hydrostatic pressures, retaining walls should be provided with a drainage system extended at least two-thirds the height of the wall. 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 14). The clean bottom and subdrain pipe, behind a retaining wall, should be observed by the Geotechnical Engineer (a representative of Geocon), prior to placement of gravel or compacting backfill.

7.17.2 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 15). These vertical columns of drainage material would then be connected at the bottom of the wall to a collection panel or a 1-cubic-foot rock pocket drained by a 4-inch subdrain pipe.

7.17.3 Subdrainage pipes at the base of the retaining wall drainage system should outlet to an acceptable location via controlled drainage structures. Drainage should not be allowed to flow uncontrolled over descending slopes.

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

7.18 Elevator Pit Design

7.18.1 The elevator pit slab and retaining wall should be designed by the project structural engineer. Elevator pits may be designed in accordance with the recommendations in the *Mat Foundation Design* and *Retaining Wall Design* sections of this report (see Sections 7.7, 7.8 and 7.15).

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

7.18.3 If retaining wall drainage is to be provided, the drainage system should be designed in accordance with the *Retaining Wall Drainage* section of this report (see Section 7.17).

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

7.19 Elevator Piston

7.19.1 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. Cobble and boulders may be encountered during excavation. Additionally, some of the site soils have little to no cohesion and are prone to excessive caving. The contractor should be prepared for difficult drilling conditions.

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

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

7.20 Temporary Excavations

7.20.1 Excavations on the order of up to 34 feet in height are anticipated for excavation and construction of the proposed subterranean level(s), including the foundation system and dewatering system, depending on final design. The excavations are expected to expose artificial fill and alluvial soils, 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.

7.20.2 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 10 feet. A uniform slope does not have a vertical portion.

7.20.3 If excavations in close proximity to an adjacent property line and/or structure are required, special excavation measures such as shoring may be necessary in order to maintain lateral support of offsite improvements. Shoring recommendation are provided in Section 7.21 of this report.

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

7.21 Shoring – Soldier Pile Design and Installation

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

7.21.2 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. Due to the presence of cobbles the installation of steel soldier piles utilizing high frequency vibration is expected to be difficult. It is recommended that the contractor bidding on shoring installation for this project perform their own test borings and vibratory soldier pile installation with the intended equipment to verify the presence and size of buried rock (cobbles and boulders) as well as the suitability of the proposed equipment for the safe and efficient installation of the soldier piles.

7.21.3 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 grading activities, foundations, and/or adjacent drainage systems.

7.21.4 The proposed soldier piles may also be designed as permanent piles. The required pile depths, dimensions, and spacing should be determined and designed by the project structural and shoring engineers. All piles utilized for shoring can also be incorporated into a permanent retaining wall system (shotcrete wall) and should be designed in accordance with the earth pressure provided in the *Retaining Wall Design* section of this report (see Section 7.15).

7.21.5 Drilled cast-in-place soldier piles should be placed no closer than three diameters on center. The minimum diameter of the piles is 18 inches. Structural concrete should be used for the soldier piles below the excavation; lean-mix concrete may be employed above that level. As an alternative, lean-mix concrete may be used throughout the pile where the reinforcing consists of a wideflange section. The slurry must be of sufficient strength to impart the lateral bearing pressure developed by the wideflange section to the soil. For design purposes, an allowable passive value for the soils below the plane of excavation above groundwater may be assumed to be 280 psf per foot. An allowable passive value for the soils below the plane of excavation below groundwater may be assumed to be 135 psf per foot (value has been reduced for buoyant forces). Where piles are installed by vibration techniques, the passive pressure may be assumed to mobilize across a width equal to the two times the dimension of the beam flange. The allowable passive value may be doubled for isolated piles, spaced a minimum of two times the pile diameter. To develop the full lateral value, provisions should be implemented to assure firm contact between the soldier piles and the undisturbed alluvium.

7.21.6 Groundwater was encountered during site exploration at depths of approximately 23 to 24 feet; however, groundwater levels can fluctuate and may be different at the time of construction. It is not uncommon for groundwater or seepage conditions to develop where none previously existed. Therefore the contractor should be prepared for groundwater during pile installation should the need arise. If more than 6 inches of water is present in the bottom of the excavation, a tremie is required to place the concrete into the bottom of the hole. A tremie should consist of a rigid, water-tight tube having a diameter of not less than 6 inches with a hopper at the top. The tube should be equipped with a device that will close the discharge end and prevent water from entering the tube while it is being charged with concrete. The tremie should be supported so as to permit free movement of the discharge end over the entire top surface of the work and to permit rapid lowering when necessary to retard or stop the flow of concrete. The discharge end should be closed at the start of the work to prevent water entering the tube and should be entirely sealed at all times, except when the concrete is being placed. The tremie tube should be kept full of concrete. The flow should be continuous until the work is completed and the resulting concrete seal should be monolithic and homogeneous. The tip of the tremie tube should always be kept about 5 feet below the surface of the concrete and definite steps and safeguards should be taken to insure that the tip of the tremie tube is never raised above the surface of the concrete.

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

7.21.8 Please be aware that the use of hollow-stem auger drilling equipment utilized for this investigation does not allow for the identification of the size of rocks being encountered or the visual observation of caving conditions since the drilling method is a small diameter cased excavation. It is recommended that the contractor bidding on excavation and shoring installation for this project perform their own test borings with the intended drilling equipment to verify the presence and size of buried rock (cobbles and boulders), potential for caving, as well as the suitability of the proposed drilling equipment for the safe and efficient installation of the shoring system.

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

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

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

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

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

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

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

7.21.16 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 0.45 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 230 psf per foot (value has been reduced for buoyant forces).

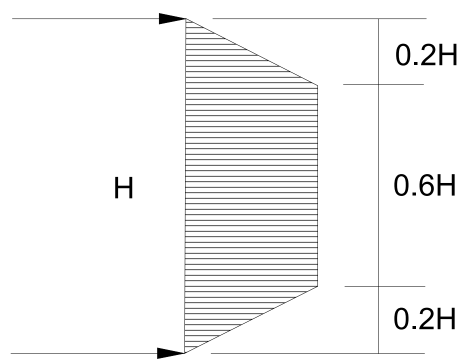
7.21.17 Due to the nature of the site soils, it is expected that continuous lagging between soldier piles will be required. However, it is recommended that the exposed soils be observed by the Geotechnical Engineer (a representative of Geocon West, Inc.), to verify the presence of any competent, cohesive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

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

7.21.19 For the design of unbraced shoring, it is recommended that an equivalent fluid pressure be utilized for design. A trapezoidal distribution of lateral earth pressure may be used where shoring will be restrained by bracing or tie backs. The recommended active and trapezoidal pressures are provided in the following table. A diagram depicting the trapezoidal pressure distribution of lateral earth pressure is provided below the table. Calculations of the shoring pressures are presented on Figures 16A and 16B.

HEIGHT OF SHORING (FEET)	EQUIVALENT FLUID PRESSURE (Pounds Per Cubic Foot) (ACTIVE PRESSURE)	EQUIVALENT FLUID PRESSURE Trapezoidal (Where H is the height of the shoring in feet)
Up to 17	34	22H
Up to 34	44	28H

Trapezoidal Distribution of Pressure



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

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

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

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

and

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

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

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

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

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

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

and

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

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

then

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

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

7.21.23 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 ten feet from the shoring, the traffic surcharge may be neglected.

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

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

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

7.22 Temporary Tie-Back Anchors

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

7.22.2 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 – 600 psf
- 15 feet below the top of the excavation – 650 psf (value has been reduced for buoyant forces)

7.22.3 Depending on the techniques utilized, and the experience of the contractor performing the installation, a maximum allowable friction capacity of 2 kips per linear foot for post-grouted anchors (for a minimum 20-foot length beyond the active wedge) may be assumed for design purposes. Only the frictional resistance developed beyond the active wedge should be utilized in resisting lateral loads.

7.23 Anchor Installation

7.23.1 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.

7.24 Anchor Testing

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

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

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

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

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

7.25 Internal Bracing

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

7.26 Surface Drainage

7.26.1 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.

7.26.2 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 2022 CBC 1804.4 or other applicable standards. In addition, drainage should not be allowed to flow uncontrolled over any descending slope. Discharge from downspouts, roof drains and scuppers are not recommended onto unprotected soils within 5 feet of the building perimeter. Planters which are located adjacent to foundations should be sealed to prevent moisture intrusion into the soils providing foundation support. Landscape irrigation is not recommended within 5 feet of the building perimeter footings except when enclosed in protected planters.

7.26.3 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.

7.26.4 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.

7.27 Plan Review

7.27.1 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.

LIST OF REFERENCES

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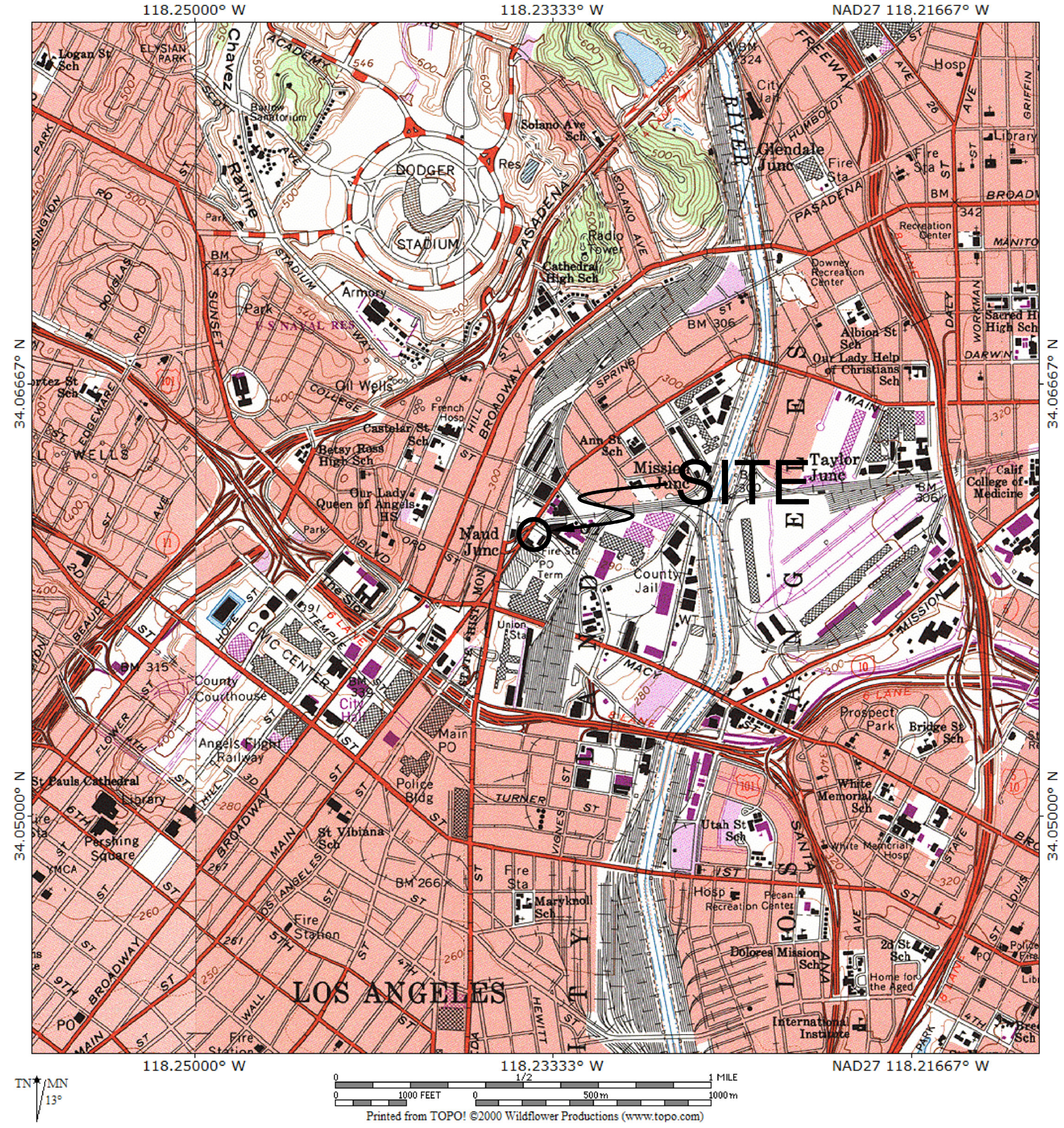
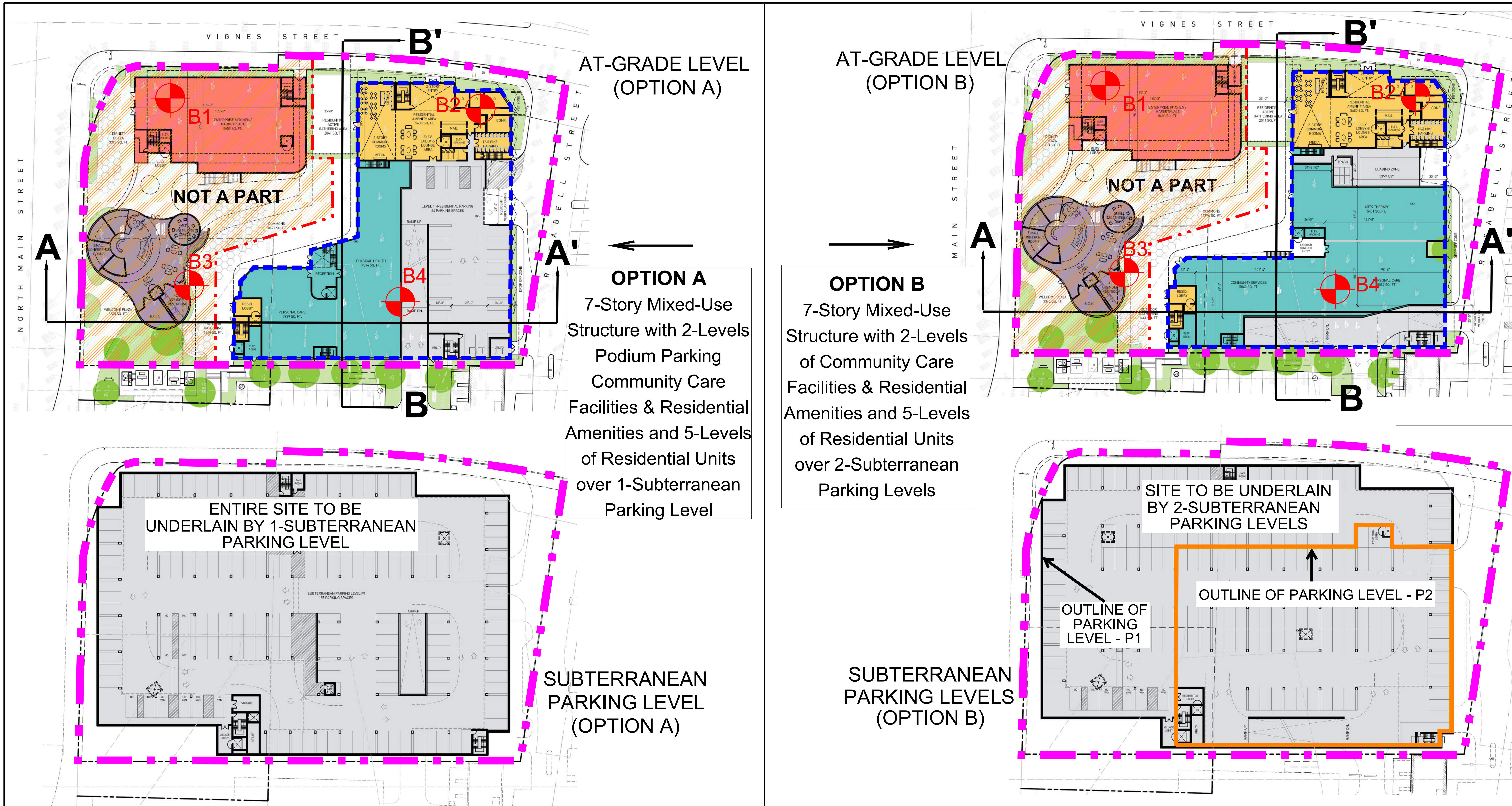


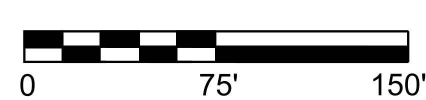
U.S.G.S. TOPOGRAPHIC MAPS, 7.5 MINUTE SERIES, LOS ANGELES AND HOLLYWOOD, CA QUADRANGLES

GEOCON WEST, INC. ENVIRONMENTAL GEOTECHNICAL MATERIALS 3303 N. SAN FERNANDO BLVD. - SUITE 100 - BURBANK, CA 91504 PHONE (818) 841-8388 - FAX (818) 841-1704	VICINITY MAP 800 & 908 NORTH MAIN STREET 1081 & 1087 NORTH VIGNES STREET LOS ANGELES, CALIFORNIA
DRAFTED BY: CB / CHECKED BY: GAK	OCT. 2023 / PROJECT NO. W1814-06-01 / FIG. 1



Legend

B4 Boring Locations
Limit of Mixed-Use Structure
Property Limits



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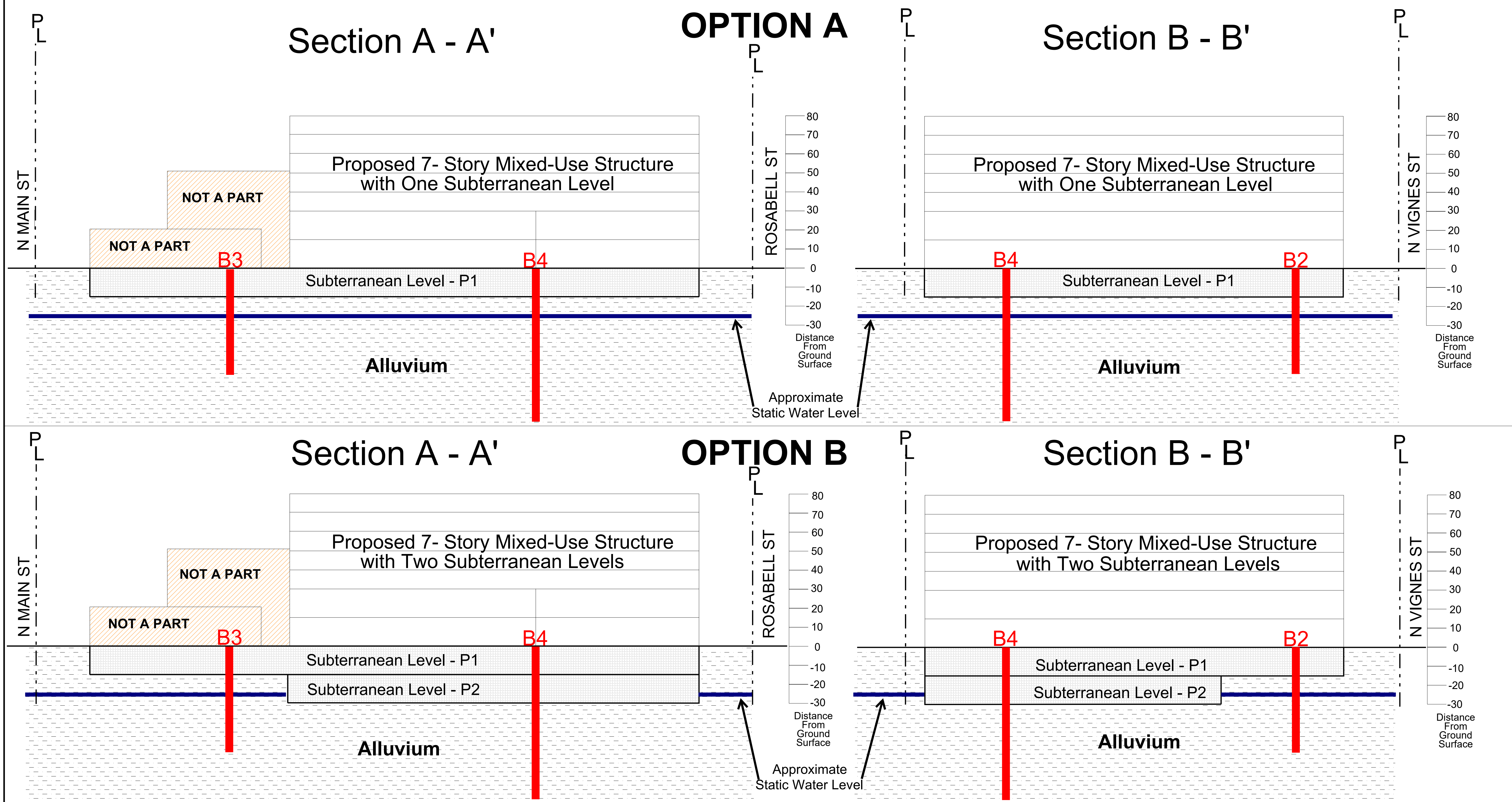
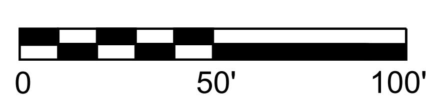
DRAFTED BY: JJK	CHECKED BY: HHD

SITE PLAN

800 & 908 NORTH MAIN STREET
1081 & 1087 NORTH VIGNES STREET STREET
LOS ANGELES, CALIFORNIA

OCT. 2023	PROJECT NO: W1814-06-01	FIG. 2A





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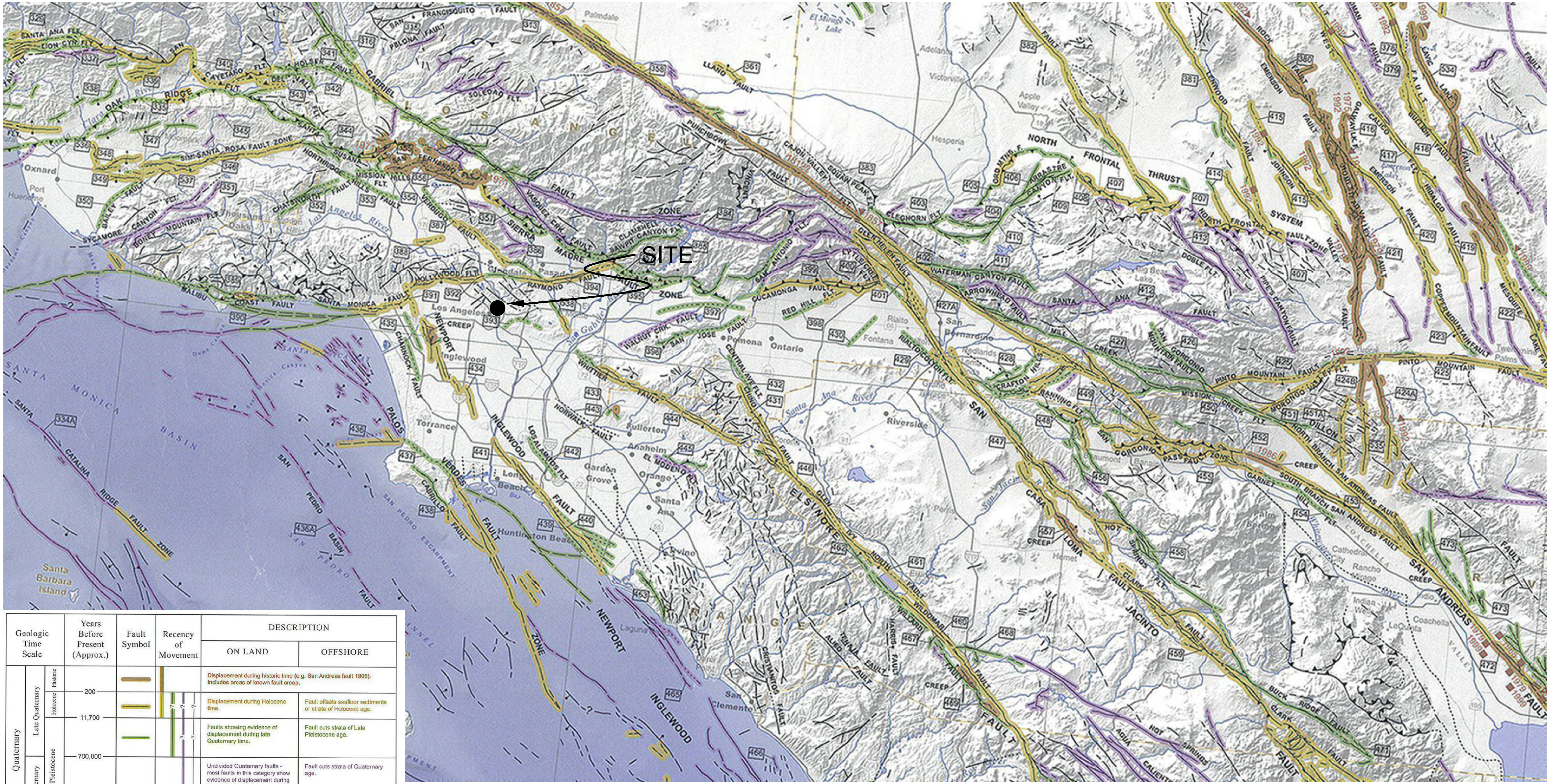
CROSS SECTION

800 & 908 NORTH MAIN STREET
1081 & 1087 NORTH VIGNES STREET STREET
LOS ANGELES, CALIFORNIA

OCT. 2023	PROJECT NO: W1814-06-01	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						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.
			4.5 billion (Age of Earth)				

* 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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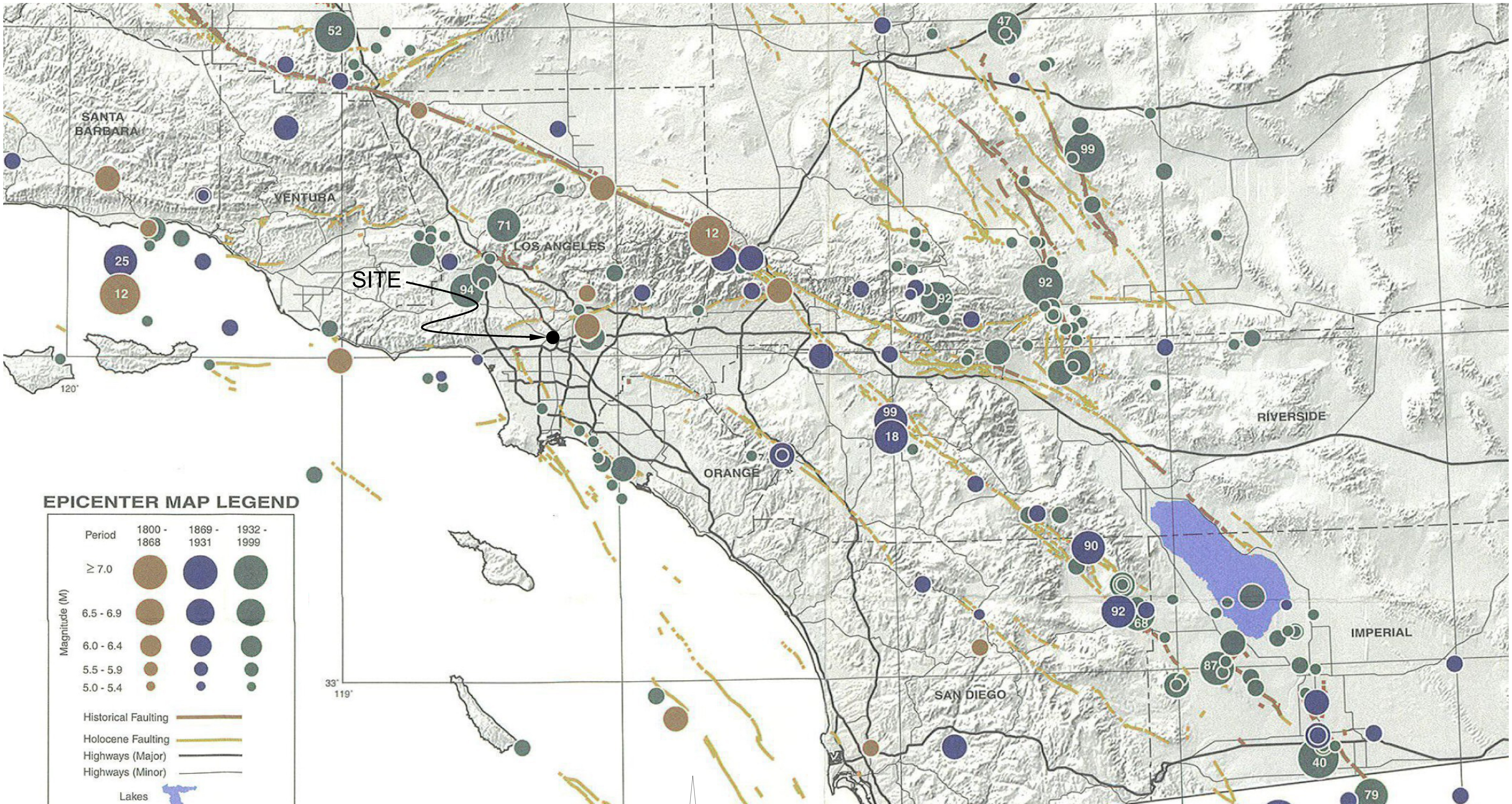
REGIONAL FAULT MAP

800 & 908 NORTH MAIN STREET
1081 & 1087 NORTH VIGNES STREET
LOS ANGELES, CALIFORNIA

OCT. 2023 | PROJECT NO. W1814-06-01 | FIG. 3



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

N

0 20 40 Miles

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REGIONAL SEISMICITY MAP

800 & 908 NORTH MAIN STREET
1081 & 1087 NORTH VIGNES STREET
LOS ANGELES, CALIFORNIA

OCT. 2023 | PROJECT NO. W1814-06-01 | FIG.4

GEOCON

Project Name : Hope Village
Project No : W1814-06-01
Boring : B1

EMPIRICAL ESTIMATION OF LIQUEFACTION POTENTIAL
DESIGN EARTHQUAKE

NCEER (1996) METHOD W 2001 UPDATES

EARTHQUAKE INFORMATION:

Earthquake Magnitude:	6.74
Peak Horiz. Acceleration PGA_M (g):	0.943
2/3 PGA_M (g):	0.629
Magnitude Scaling Factor:	1.314
Historic High Groundwater:	20.0
Groundwater Depth During Exploration:	24.0

ENERGY & ROD CORRECTIONS:

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

LIQUEFACTION CALCULATIONS:

Unit Wt. Water (pcf):	62.4

Depth to Base (ft)	Total Unit Wt. (pcf)	Water (0 or 1)	FIELD SPT (N)	Depth of SPT (ft)	Liq.Sus. (0 or 1)	-200 (%)	Est. Dr (%)	CN Factor	Corrected (N1)60	Eff. Unit Wt. (psf)	Resist. CRR 7.5	rd Factor	Induced CSR	Liquefac. Safe.Fact.
1.0	125.0	0	11.6	1.0	1		82	1.700	22.1	125.0	0.243	1.000	0.409	--
2.0	125.0	0	11.6	2.0	1		80	1.700	22.1	125.0	0.243	0.998	0.408	--
3.0	114.8	0	11.6	2.5	1		78	1.700	22.1	114.8	0.243	0.996	0.407	--
4.0	114.8	0	11.6	2.5	1		78	1.700	22.1	114.8	0.243	0.994	0.406	--
5.0	114.8	0	11.6	2.5	1		78	1.700	22.1	114.8	0.243	0.991	0.405	--
6.0	114.8	0	9.0	5.0	1		65	1.700	17.2	114.8	0.183	0.989	0.404	--
7.0	114.8	0	9.0	5.0	1		65	1.650	16.7	114.8	0.178	0.987	0.403	--
8.0	121.3	0	9.0	5.0	1		65	1.536	15.6	121.3	0.166	0.985	0.403	--
9.0	121.3	0	9.0	5.0	1		65	1.441	14.6	121.3	0.156	0.982	0.402	--
10.0	121.3	0	9.0	5.0	1		65	1.361	13.8	121.3	0.148	0.980	0.401	--
11.0	125.0	0	29.0	10.0	1		106	1.292	42.2	125.0	Infin.	0.978	0.400	--
12.0	125.0	0	29.0	10.0	1		106	1.232	40.2	125.0	Infin.	0.976	0.399	--
13.0	125.0	0	29.0	10.0	1		106	1.180	38.5	125.0	Infin.	0.974	0.398	--
14.0	125.0	0	29.0	10.0	1		106	1.133	37.0	125.0	Infin.	0.972	0.397	--
15.0	125.0	0	29.0	10.0	1		106	1.092	35.6	125.0	Infin.	0.970	0.396	--
16.0	123.4	0	31.0	15.0	1		100	1.055	39.6	123.4	Infin.	0.967	0.396	--
17.0	123.4	0	31.0	15.0	1		100	1.022	38.3	123.4	Infin.	0.965	0.395	--
18.0	123.4	0	31.0	15.0	1		100	0.992	37.2	123.4	Infin.	0.963	0.394	--
19.0	123.4	0	31.0	15.0	1		100	0.964	36.2	123.4	Infin.	0.961	0.393	--
20.0	123.4	0	31.0	15.0	1		100	0.939	35.2	123.4	Infin.	0.958	0.392	--
21.0	123.4	1	50.0	20.0	1		118	0.915	61.4	61.0	Infin.	0.956	0.396	Non-Liq.
22.0	123.4	1	50.0	20.0	1		118	0.894	60.0	61.0	Infin.	0.953	0.404	Non-Liq.
23.0	124.2	1	50.0	20.0	1		118	0.873	58.6	61.8	Infin.	0.950	0.412	Non-Liq.
24.0	124.2	1	50.0	20.0	1		118	0.859	57.6	61.8	Infin.	0.947	0.419	Non-Liq.
25.0	124.2	1	50.0	20.0	1		118	0.849	57.0	61.8	Infin.	0.944	0.426	Non-Liq.
26.0	124.2	1	18.0	25.0	1	36	68	0.841	31.0	61.8	Infin.	0.940	0.432	Non-Liq.
27.0	124.2	1	18.0	25.0	1	36	68	0.832	30.7	61.8	Infin.	0.936	0.438	Non-Liq.
28.0	130.6	1	45.7	27.5	1	3	106	0.823	55.2	68.2	Infin.	0.932	0.443	Non-Liq.
29.0	130.6	1	45.7	27.5	1	3	106	0.814	54.6	68.2	Infin.	0.928	0.447	Non-Liq.
30.0	130.6	1	45.7	27.5	1	3	106	0.805	54.0	68.2	Infin.	0.923	0.451	Non-Liq.
31.0	130.6	1	28.0	30.0	1	3	82	0.797	33.5	68.2	Infin.	0.918	0.455	Non-Liq.
32.0	145.4	1	39.1	32.5	1	3	95	0.788	46.2	83.0	Infin.	0.912	0.457	Non-Liq.
33.0	145.4	1	39.1	32.5	1	3	95	0.778	45.6	83.0	Infin.	0.907	0.459	Non-Liq.
34.0	145.4	1	39.1	32.5	1	3	95	0.769	45.1	83.0	Infin.	0.900	0.461	Non-Liq.
35.0	145.4	1	39.1	32.5	1		95	0.760	44.5	83.0	Infin.	0.894	0.462	Non-Liq.
36.0	145.4	1	50.0	35.0	1		106	0.752	56.4	83.0	Infin.	0.887	0.463	Non-Liq.
37.0	145.4	1	50.0	35.0	1		106	0.743	55.8	83.0	Infin.	0.880	0.463	Non-Liq.
38.0	145.4	1	50.0	35.0	1		106	0.735	55.2	83.0	Infin.	0.872	0.462	Non-Liq.
39.0	145.4	1	50.0	35.0	1		106	0.728	54.6	83.0	Infin.	0.864	0.462	Non-Liq.
40.0	145.4	1	50.0	35.0	1		106	0.720	54.0	83.0	Infin.	0.855	0.460	Non-Liq.
41.0	140.9	1	44.0	40.0	1		95	0.713	47.1	78.5	Infin.	0.846	0.459	Non-Liq.
42.0	140.9	1	44.0	40.0	1		95	0.706	46.6	78.5	Infin.	0.837	0.457	Non-Liq.
43.0	140.9	1	44.0	40.0	1		95	0.700	46.2	78.5	Infin.	0.828	0.455	Non-Liq.
44.0	140.9	1	44.0	40.0	1		95	0.693	45.8	78.5	Infin.	0.818	0.453	Non-Liq.
45.0	140.9	1	44.0	40.0	1		95	0.687	45.4	78.5	Infin.	0.808	0.450	Non-Liq.
46.0	140.9	1	100.0	45.0	1		139	0.681	102.2	78.5	Infin.	0.798	0.447	Non-Liq.
47.0	140.9	1	100.0	45.0	1		139	0.675	101.3	78.5	Infin.	0.788	0.444	Non-Liq.
48.0	140.9	1	100.0	45.0	1		139	0.670	100.4	78.5	Infin.	0.778	0.440	Non-Liq.
49.0	140.9	1	100.0	45.0	1		139	0.664	99.6	78.5	Infin.	0.768	0.437	Non-Liq.
50.0	140.9	1	100.0	45.0	1		139	0.659	98.8	78.5	Infin.	0.757	0.433	Non-Liq.
51.0	127.7	1	23.0	50.0	1	81	65	0.654	32.1	65.3	Infin.	0.747	0.429	Non-Liq.
52.0	127.7	1	23.0	50.0	1	81	65	0.649	31.9	65.3	Infin.	0.737	0.426	Non-Liq.
53.0	127.7	1	37.4	50.0	1	81	83	0.645	48.4	65.3	Infin.	0.727	0.422	Non-Liq.
54.0	127.7	1	37.4	50.0	1	81	83	0.641	48.2	65.3	Infin.	0.717	0.419	Non-Liq.
55.0	127.7	1	37.4	50.0	1	81	83	0.637	47.9	65.3	Infin.	0.708	0.415	Non-Liq.
56.0	127.7	1	74.0	55.0	1		113	0.633	70.3	65.3	Infin.	0.698	0.411	Non-Liq.
57.0	127.7	1	74.0	55.0	1		113	0.629	69.8	65.3	Infin.	0.689	0.408	Non-Liq.
58.0	127.7	1	74.0	55.0	1		113	0.625	69.4	65.3	Infin.	0.680	0.404	Non-Liq.
59.0	127.7	1	74.0	55.0	1		113	0.621	69.0	65.3	Infin.	0.671	0.401	Non-Liq.
60.0	127.7	1	74.0	55.0	1		113	0.618	68.6	65.3	Infin.	0.663	0.397	Non-Liq.
61.0	129.0	1	76.0	60.0	1		112	0.614	70.0	66.6	Infin.	0.655	0.394	Non-Liq.
62.0	129.0	1	76.0	60.0	1		112	0.610	69.6	66.6	Infin.	0.647	0.391	Non-Liq.
63.0	129.0	1	76.0	60.0	1		112	0.607	69.2	66.6	Infin.	0.639	0.388	Non-Liq.
64.0	129.0	1	76.0	60.0	1		112	0.603	68.8	66.6	Infin.	0.632	0.385	Non-Liq.
65.0	129.0	1	76.0	60.0	1		112	0.600	68.4	66.6	Infin.	0.625	0.382	Non-Liq.

GEOCON

Project Name : Hope Village
Project No : W1814-06-01
Boring : B4

EMPIRICAL ESTIMATION OF LIQUEFACTION POTENTIAL
DESIGN EARTHQUAKE

NCEER (1996) METHOD W 2001 UPDATES

EARTHQUAKE INFORMATION:

Earthquake Magnitude:	6.74
Peak Horiz. Acceleration PGA_M (g):	0.943
2/3 PGA_M (g):	0.629
Magnitude Scaling Factor:	1.314
Historic High Groundwater:	20.0
Groundwater Depth During Exploration:	24.0

ENERGY & ROD CORRECTIONS:

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

LIQUEFACTION CALCULATIONS:

Unit Wt. Water (pcf):	62.4

Depth to Base (ft)	Total Unit Wt. (pcf)	Water (0 or 1)	FIELD SPT (N)	Depth of SPT (ft)	Liq.Sus. (0 or 1)	-200 (%)	Est. Dr (%)	CN Factor	Corrected (N1)60	Eff. Unit Wt. (psf)	Resist. CRR 7.5	rd Factor	Induced CSR	Liquefac. Safe.Fact.
1.0	125.0	0	4.4	1.0	1		51	1.700	8.4	125.0	0.099	1.000	0.409	--
2.0	125.0	0	4.4	2.0	1		49	1.700	8.4	125.0	0.099	0.998	0.408	--
3.0	111.0	0	4.4	2.5	1		48	1.700	8.4	111.0	0.099	0.996	0.407	--
4.0	111.0	0	4.4	2.5	1		48	1.700	8.4	111.0	0.099	0.994	0.406	--
5.0	111.0	0	4.4	2.5	1		48	1.700	8.4	111.0	0.099	0.991	0.405	--
6.0	111.0	0	3.0	5.0	1		38	1.700	5.7	111.0	0.078	0.989	0.404	--
7.0	111.0	0	3.0	5.0	1		38	1.669	5.6	111.0	0.077	0.987	0.403	--
8.0	113.1	0	26.4	7.5	1		106	1.557	46.2	113.1	Infin.	0.985	0.403	--
9.0	113.1	0	26.4	7.5	1		106	1.464	43.5	113.1	Infin.	0.982	0.402	--
10.0	113.1	0	26.4	7.5	1		106	1.385	41.1	113.1	Infin.	0.980	0.401	--
11.0	113.1	0	32.0	10.0	1		112	1.319	47.5	113.1	Infin.	0.978	0.400	--
12.0	113.1	0	32.0	10.0	1		112	1.261	45.4	113.1	Infin.	0.976	0.399	--
13.0	120.7	0	32.0	10.0	1		112	1.208	43.5	120.7	Infin.	0.974	0.398	--
14.0	120.7	0	32.0	10.0	1		112	1.160	41.8	120.7	Infin.	0.972	0.397	--
15.0	120.7	0	32.0	10.0	1		112	1.117	40.2	120.7	Infin.	0.970	0.396	--
16.0	120.7	0	32.0	15.0	1		103	1.079	41.8	120.7	Infin.	0.967	0.396	--
17.0	120.7	0	32.0	15.0	1		103	1.045	40.4	120.7	Infin.	0.965	0.395	--
18.0	116.2	0	32.0	15.0	1		103	1.014	39.2	116.2	Infin.	0.963	0.394	--
19.0	116.2	0	32.0	15.0	1		103	0.986	38.2	116.2	Infin.	0.961	0.393	--
20.0	116.2	0	32.0	15.0	1		103	0.960	37.2	116.2	Infin.	0.958	0.392	--
21.0	116.2	1	56.0	20.0	1		126	0.937	70.4	53.8	Infin.	0.956	0.396	Non-Liq.
22.0	116.2	1	56.0	20.0	1		126	0.914	68.7	53.8	Infin.	0.953	0.405	Non-Liq.
23.0	139.0	1	56.0	20.0	1		126	0.892	67.1	76.6	Infin.	0.950	0.413	Non-Liq.
24.0	139.0	1	56.0	20.0	1		126	0.874	65.7	76.6	Infin.	0.947	0.420	Non-Liq.
25.0	139.0	1	56.0	20.0	1		126	0.862	64.8	76.6	Infin.	0.944	0.427	Non-Liq.
26.0	139.0	1	72.0	25.0	1		137	0.851	87.8	76.6	Infin.	0.940	0.433	Non-Liq.
27.0	139.0	1	72.0	25.0	1		137	0.840	86.6	76.6	Infin.	0.936	0.439	Non-Liq.
28.0	158.4	1	72.0	25.0	1		137	0.828	85.4	96.0	Infin.	0.932	0.443	Non-Liq.
29.0	158.4	1	72.0	25.0	1		137	0.815	84.1	96.0	Infin.	0.928	0.447	Non-Liq.
30.0	158.4	1	72.0	25.0	1		137	0.803	82.8	96.0	Infin.	0.923	0.451	Non-Liq.
31.0	158.4	1	27.0	30.0	1	5	80	0.791	32.0	96.0	Infin.	0.918	0.453	Non-Liq.
32.0	158.4	1	27.0	30.0	1	5	80	0.780	31.6	96.0	Infin.	0.912	0.456	Non-Liq.
33.0	131.5	1	38.0	32.5	1	5	93	0.771	43.9	69.1	Infin.	0.907	0.458	Non-Liq.
34.0	131.5	1	38.0	32.5	1	5	93	0.763	43.4	69.1	Infin.	0.900	0.459	Non-Liq.
35.0	131.5	1	38.0	32.5	1	5	93	0.756	43.0	69.1	Infin.	0.894	0.461	Non-Liq.
36.0	131.5	1	23.0	35.0	1	4	71	0.749	25.8	69.1	0.309	0.887	0.462	0.72
37.0	131.5	1	23.0	35.0	1	4	71	0.742	25.6	69.1	0.304	0.880	0.462	0.70
38.0	129.0	1	55.0	37.5	1	4	108	0.736	60.7	66.6	Infin.	0.872	0.462	Non-Liq.
39.0	129.0	1	55.0	37.5	1	4	108	0.729	60.2	66.6	Infin.	0.864	0.462	Non-Liq.
40.0	129.0	1	55.0	37.5	1	4	108	0.723	59.7	66.6	Infin.	0.855	0.461	Non-Liq.
41.0	135.6	1	22.0	40.0	1	7	68	0.717	24.0	73.2	0.273	0.846	0.460	0.62
42.0	135.6	1	22.0	40.0	1	7	68	0.711	23.8	73.2	0.269	0.837	0.459	0.61
43.0	135.6	1	55.0	42.5	1	7	105	0.704	58.7	73.2	Infin.	0.828	0.457	Non-Liq.
44.0	135.6	1	55.0	42.5	1	7	105	0.698	58.2	73.2	Infin.	0.818	0.454	Non-Liq.
45.0	135.6	1	55.0	42.5	1	7	105	0.692	57.7	73.2	Infin.	0.808	0.452	Non-Liq.
46.0	135.6	1	30.0	45.0	1		77	0.687	30.9	73.2	Infin.	0.798	0.449	Non-Liq.
47.0	135.6	1	30.0	45.0	1		77	0.681	30.6	73.2	Infin.	0.788	0.446	Non-Liq.
48.0	156.1	1	30.0	45.0	1		77	0.675	30.4	93.7	Infin.	0.778	0.442	Non-Liq.
49.0	156.1	1	30.0	45.0	1		77	0.668	30.1	93.7	Infin.	0.768	0.438	Non-Liq.
50.0	156.1	1	30.0	45.0	1		77	0.661	29.8	93.7	0.452	0.757	0.434	1.00
51.0	146.7	1	58.0	50.0	1		103	0.655	57.0	84.3	Infin.	0.747	0.430	Non-Liq.
52.0	146.7	1	58.0	50.0	1		103	0.650	56.5	84.3	Infin.	0.737	0.426	Non-Liq.
53.0	146.7	1	58.0	50.0	1		103	0.644	56.1	84.3	Infin.	0.727	0.422	Non-Liq.
54.0	146.7	1	58.0	50.0	1		103	0.639	55.6	84.3	Infin.	0.717	0.418	Non-Liq.
55.0	146.7	1	58.0	50.0	1		103	0.634	55.1	84.3	Infin.	0.708	0.414	Non-Liq.
56.0	146.7	1	79.0	55.0	1		117	0.629	74.5	84.3	Infin.	0.698	0.410	Non-Liq.
57.0	146.7	1	79.0	55.0	1		117	0.624	73.9	84.3	Infin.	0.689	0.406	Non-Liq.
58.0	146.7	1	41.0	57.5	1		83	0.619	38.1	84.3	Infin.	0.680	0.402	Non-Liq.
59.0	146.7	1	41.0	57.5	1		83	0.614	37.8	84.3	Infin.	0.671	0.398	Non-Liq.
60.0	146.7	1	41.0	57.5	1		83	0.610	37.5	84.3	Infin.	0.663	0.394	Non-Liq.
61.0	146.7	1	76.0	60.0	1		111	0.605	69.0	84.3	Infin.	0.655	0.390	Non-Liq.
62.0	146.7	1	76.0	60.0	1		111	0.601	68.5	84.3	Infin.	0.647	0.387	Non-Liq.
63.0	146.7	1	76.0	60.0	1		111	0.596	68.0	84.3	Infin.	0.639	0.383	Non-Liq.
64.0	146.7	1	76.0	60.0	1		111	0.592	67.5	84.3	Infin.	0.632	0.380	Non-Liq.
65.0	146.7	1	76.0	60.0	1		111	0.588	67.0	84.3	Infin.	0.625	0.377	Non-Liq.
66.0	146.7	1	76.0	65.0	1		108	0.584	66.6	84.3	Infin.	0.618	0.374	Non-Liq.
67.0	146.7	1	76.0	65.0	1		108	0.580	66.1	84.3	Infin.	0.612	0.371	Non-Liq.
68.0	146.7	1	76.0	65.0	1		108	0.576	65.7	84.3	Infin.	0.606	0.368	Non-Liq.
69.0	146.7	1	76.0	65.0	1		108	0.572	65.2	84.3	Infin.	0.600	0.365	Non-Liq.
70.0	146.7	1	76.0	65.0	1		108	0.568	64.8	84.3	Infin.	0.594	0.363	Non-Liq.
71.0	146.7	1	43.0	70.0	1		79	0.565	36.4	84.3	Infin.	0.589	0.360	Non-Liq.
72.0	146.7	1	43.0	70.0	1		79	0.561	36.2	84.3	Infin.	0.584	0.358	Non-Liq.
73.0	146.7	1	43.0	70.0	1		79	0.558	36.0	84.3	Infin.	0.579	0.355	Non-Liq.
74.0	146.7	1	43.0	70.0	1		79	0.554	35.7	84.3	Infin.	0.574	0.353	Non-Liq.
75.0	146.7	1	43.0	70.0	1		79	0.551	35.5	84.3	Infin.	0.570	0.351	Non-Liq.
76.0	146.7	1	92.0	75.0	1		113	0.547	75.5	84.3	Infin.	0.565	0.349	Non-Liq.
77.0	146.7	1	92.0	75.0	1		113	0.544	75.1	84.3	Infin.	0.561	0.347	Non-Liq.
78.0	146.7	1	92.0	75.0	1		113	0.541	74.6	84.3	Infin.	0.557	0.345	Non-Liq.
79.0	146.7	1	92.0	75.0	1		113	0.538	74.2	84.3	Infin.	0.553	0.344	Non-Liq.
80.0	146.7	1	92.0	75.0	1		113	0.535	73.8	84.3	Infin.	0.550	0.342	Non-Liq.

GEOCON

Project Name : Hope Village
Project No : W1814-06-01
Boring : B1

LIQUEFACTION SETTLEMENT ANALYSIS
DESIGN EARTHQUAKE

NCEER (1996) METHOD W 2001 UPDATES
EARTHQUAKE INFORMATION:

Earthquake Magnitude:	6.74
PGA_M (g):	0.943
2/3 PGA_M (g):	0.63
Magnitude Scaling Factor:	1.314
Historic High Groundwater:	20.0
Groundwater @ Exploration:	24.0

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	VOL. STRAIN [e_{15}} (%)	EQ. SETTLE. Pe (in.)
1.0	12	125.0	0.031	0.031	82	22	0.409	--	0.00	0.00
2.0	12	125.0	0.094	0.094	80	22	0.409	--	0.00	0.00
3.0	12	114.8	0.154	0.154	78	22	0.409	--	0.00	0.00
4.0	12	114.8	0.211	0.211	78	22	0.409	--	0.00	0.00
5.0	12	114.8	0.269	0.269	78	22	0.409	--	0.00	0.00
6.0	9	114.8	0.326	0.326	65	17	0.409	--	0.00	0.00
7.0	9	114.8	0.383	0.383	65	17	0.409	--	0.00	0.00
8.0	9	121.3	0.442	0.442	65	16	0.409	--	0.00	0.00
9.0	9	121.3	0.503	0.503	65	15	0.409	--	0.00	0.00
10.0	9	121.3	0.564	0.564	65	14	0.409	--	0.00	0.00
11.0	29	125.0	0.625	0.625	106	42	0.409	--	0.00	0.00
12.0	29	125.0	0.688	0.688	106	40	0.409	--	0.00	0.00
13.0	29	125.0	0.750	0.750	106	38	0.409	--	0.00	0.00
14.0	29	125.0	0.813	0.813	106	37	0.409	--	0.00	0.00
15.0	29	125.0	0.875	0.875	106	36	0.409	--	0.00	0.00
16.0	31	123.4	0.937	0.937	100	40	0.409	--	0.00	0.00
17.0	31	123.4	0.999	0.999	100	38	0.409	--	0.00	0.00
18.0	31	123.4	1.061	1.061	100	37	0.409	--	0.00	0.00
19.0	31	123.4	1.122	1.122	100	36	0.409	--	0.00	0.00
20.0	31	123.4	1.184	1.184	100	35	0.409	--	0.00	0.00
21.0	50	123.4	1.246	1.230	118	61	0.414	Non-Liq.	0.00	0.00
22.0	50	123.4	1.308	1.261	118	60	0.424	Non-Liq.	0.00	0.00
23.0	50	124.2	1.369	1.291	118	59	0.434	Non-Liq.	0.00	0.00
24.0	50	124.2	1.432	1.322	118	58	0.443	Non-Liq.	0.00	0.00
25.0	50	124.2	1.494	1.353	118	57	0.451	Non-Liq.	0.00	0.00
26.0	18	124.2	1.556	1.384	68	31	0.460	Non-Liq.	0.00	0.00
27.0	18	124.2	1.618	1.415	68	31	0.467	Non-Liq.	0.00	0.00
28.0	46	130.6	1.682	1.448	106	55	0.475	Non-Liq.	0.00	0.00
29.0	46	130.6	1.747	1.482	106	55	0.482	Non-Liq.	0.00	0.00
30.0	46	130.6	1.812	1.516	106	54	0.489	Non-Liq.	0.00	0.00
31.0	28	130.6	1.877	1.550	82	33	0.495	Non-Liq.	0.00	0.00
32.0	39	145.4	1.946	1.588	95	46	0.501	Non-Liq.	0.00	0.00
33.0	39	145.4	2.019	1.629	95	46	0.507	Non-Liq.	0.00	0.00
34.0	39	145.4	2.092	1.671	95	45	0.512	Non-Liq.	0.00	0.00
35.0	39	145.4	2.165	1.712	95	45	0.517	Non-Liq.	0.00	0.00
36.0	50	145.4	2.237	1.754	106	56	0.522	Non-Liq.	0.00	0.00
37.0	50	145.4	2.310	1.795	106	56	0.526	Non-Liq.	0.00	0.00
38.0	50	145.4	2.383	1.837	106	55	0.530	Non-Liq.	0.00	0.00
39.0	50	145.4	2.455	1.878	106	55	0.534	Non-Liq.	0.00	0.00
40.0	50	145.4	2.528	1.920	106	54	0.538	Non-Liq.	0.00	0.00
41.0	44	140.9	2.600	1.960	95	47	0.542	Non-Liq.	0.00	0.00
42.0	44	140.9	2.670	1.999	95	47	0.546	Non-Liq.	0.00	0.00
43.0	44	140.9	2.740	2.038	95	46	0.550	Non-Liq.	0.00	0.00
44.0	44	140.9	2.811	2.078	95	46	0.553	Non-Liq.	0.00	0.00
45.0	44	140.9	2.881	2.117	95	45	0.556	Non-Liq.	0.00	0.00
46.0	100	140.9	2.952	2.156	139	102	0.560	Non-Liq.	0.00	0.00
47.0	100	140.9	3.022	2.195	139	101	0.563	Non-Liq.	0.00	0.00
48.0	100	140.9	3.093	2.235	139	100	0.566	Non-Liq.	0.00	0.00
49.0	100	140.9	3.163	2.274	139	100	0.569	Non-Liq.	0.00	0.00
50.0	100	140.9	3.234	2.313	139	99	0.572	Non-Liq.	0.00	0.00
51.0	23	127.7	3.301	2.349	65	32	0.574	Non-Liq.	0.00	0.00
52.0	23	127.7	3.365	2.382	65	32	0.578	Non-Liq.	0.00	0.00
53.0	37	127.7	3.428	2.414	83	48	0.581	Non-Liq.	0.00	0.00
54.0	37	127.7	3.492	2.447	83	48	0.583	Non-Liq.	0.00	0.00
55.0	37	127.7	3.556	2.480	83	48	0.586	Non-Liq.	0.00	0.00
56.0	74	127.7	3.620	2.512	113	70	0.589	Non-Liq.	0.00	0.00
57.0	74	127.7	3.684	2.545	113	70	0.592	Non-Liq.	0.00	0.00
58.0	74	127.7	3.748	2.578	113	69	0.594	Non-Liq.	0.00	0.00
59.0	74	127.7	3.812	2.610	113	69	0.597	Non-Liq.	0.00	0.00
60.0	74	127.7	3.875	2.643	113	69	0.599	Non-Liq.	0.00	0.00
61.0	76	129.0	3.940	2.676	112	70	0.602	Non-Liq.	0.00	0.00
62.0	76	129.0	4.004	2.709	112	70	0.604	Non-Liq.	0.00	0.00
63.0	76	129.0	4.069	2.743	112	69	0.607	Non-Liq.	0.00	0.00
64.0	76	129.0	4.133	2.776	112	69	0.609	Non-Liq.	0.00	0.00
65.0	76	129.0	4.198	2.809	112	68	0.611	Non-Liq.	0.00	0.00

TOTAL SETTLEMENT = 0.0 INCHES

GEOCON

Project Name : Hope Village
Project No : W1814-06-01
Boring : B4

LIQUEFACTION SETTLEMENT ANALYSIS
DESIGN EARTHQUAKE

NCEER (1996) METHOD W 2001 UPDATES
EARTHQUAKE INFORMATION:

Earthquake Magnitude:	6.74
PGA_M (g):	0.943
2/3 PGA_M (g):	0.63
Magnitude Scaling Factor:	1.314
Historic High Groundwater:	20.0
Groundwater @ Exploration:	24.0

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	VOL. STRAIN [e_{15}} (%)	EQ. SETTLE. Pe (in.)
1.0	4	125.0	0.031	0.031	51	8	0.409	--	0.00	0.00
2.0	4	125.0	0.094	0.094	49	8	0.409	--	0.00	0.00
3.0	4	111.0	0.153	0.153	48	8	0.409	--	0.00	0.00
4.0	4	111.0	0.208	0.208	48	8	0.409	--	0.00	0.00
5.0	4	111.0	0.264	0.264	48	8	0.409	--	0.00	0.00
6.0	3	111.0	0.319	0.319	38	6	0.409	--	0.00	0.00
7.0	3	111.0	0.375	0.375	38	6	0.409	--	0.00	0.00
8.0	26	113.1	0.431	0.431	106	46	0.409	--	0.00	0.00
9.0	26	113.1	0.487	0.487	106	43	0.409	--	0.00	0.00
10.0	26	113.1	0.544	0.544	106	41	0.409	--	0.00	0.00
11.0	32	113.1	0.600	0.600	112	47	0.409	--	0.00	0.00
12.0	32	113.1	0.657	0.657	112	45	0.409	--	0.00	0.00
13.0	32	120.7	0.715	0.715	112	43	0.409	--	0.00	0.00
14.0	32	120.7	0.776	0.776	112	42	0.409	--	0.00	0.00
15.0	32	120.7	0.836	0.836	112	40	0.409	--	0.00	0.00
16.0	32	120.7	0.896	0.896	103	42	0.409	--	0.00	0.00
17.0	32	120.7	0.957	0.957	103	40	0.409	--	0.00	0.00
18.0	32	116.2	1.016	1.016	103	39	0.409	--	0.00	0.00
19.0	32	116.2	1.074	1.074	103	38	0.409	--	0.00	0.00
20.0	32	116.2	1.132	1.132	103	37	0.409	--	0.00	0.00
21.0	56	116.2	1.190	1.175	126	70	0.414	Non-Liq.	0.00	0.00
22.0	56	116.2	1.248	1.202	126	69	0.425	Non-Liq.	0.00	0.00
23.0	56	139.0	1.312	1.234	126	67	0.435	Non-Liq.	0.00	0.00
24.0	56	139.0	1.382	1.273	126	66	0.444	Non-Liq.	0.00	0.00
25.0	56	139.0	1.451	1.311	126	65	0.453	Non-Liq.	0.00	0.00
26.0	72	139.0	1.521	1.349	137	88	0.461	Non-Liq.	0.00	0.00
27.0	72	139.0	1.590	1.387	137	87	0.469	Non-Liq.	0.00	0.00
28.0	72	158.4	1.665	1.431	137	85	0.476	Non-Liq.	0.00	0.00
29.0	72	158.4	1.744	1.479	137	84	0.482	Non-Liq.	0.00	0.00
30.0	72	158.4	1.823	1.527	137	83	0.488	Non-Liq.	0.00	0.00
31.0	27	158.4	1.902	1.575	80	32	0.494	Non-Liq.	0.00	0.00
32.0	27	158.4	1.981	1.623	80	32	0.499	Non-Liq.	0.00	0.00
33.0	38	131.5	2.054	1.664	93	44	0.505	Non-Liq.	0.00	0.00
34.0	38	131.5	2.120	1.698	93	43	0.510	Non-Liq.	0.00	0.00
35.0	38	131.5	2.185	1.733	93	43	0.516	Non-Liq.	0.00	0.00
36.0	23	131.5	2.251	1.768	71	26	0.521	0.72	1.10	0.13
37.0	23	131.5	2.317	1.802	71	26	0.526	0.70	1.10	0.13
38.0	55	129.0	2.382	1.836	108	61	0.530	Non-Liq.	0.00	0.00
39.0	55	129.0	2.447	1.869	108	60	0.535	Non-Liq.	0.00	0.00
40.0	55	129.0	2.511	1.903	108	60	0.540	Non-Liq.	0.00	0.00
41.0	22	135.6	2.577	1.938	68	24	0.544	0.62	1.30	0.16
42.0	22	135.6	2.645	1.974	68	24	0.548	0.61	1.30	0.16
43.0	55	135.6	2.713	2.011	105	59	0.552	Non-Liq.	0.00	0.00
44.0	55	135.6	2.781	2.047	105	58	0.555	Non-Liq.	0.00	0.00
45.0	55	135.6	2.848	2.084	105	58	0.559	Non-Liq.	0.00	0.00
46.0	30	135.6	2.916	2.121	77	31	0.562	Non-Liq.	0.00	0.00
47.0	30	135.6	2.984	2.157	77	31	0.566	Non-Liq.	0.00	0.00
48.0	30	156.1	3.057	2.199	77	30	0.568	Non-Liq.	0.00	0.00
49.0	30	156.1	3.135	2.246	77	30	0.571	Non-Liq.	0.00	0.00
50.0	30	156.1	3.213	2.293	77	30	0.573	1.00	0.75	0.09
51.0	58	146.7	3.289	2.337	103	57	0.575	Non-Liq.	0.00	0.00
52.0	58	146.7	3.362	2.379	103	57	0.578	Non-Liq.	0.00	0.00
53.0	58	146.7	3.435	2.421	103	56	0.580	Non-Liq.	0.00	0.00
54.0	58	146.7	3.509	2.464	103	56	0.582	Non-Liq.	0.00	0.00
55.0	58	146.7	3.582	2.506	103	55	0.584	Non-Liq.	0.00	0.00
56.0	79	146.7	3.655	2.548	117	74	0.587	Non-Liq.	0.00	0.00
57.0	79	146.7	3.729	2.590	117	74	0.589	Non-Liq.	0.00	0.00
58.0	41	146.7	3.802	2.632	83	38	0.591	Non-Liq.	0.00	0.00
59.0	41	146.7	3.875	2.674	83	38	0.592	Non-Liq.	0.00	0.00
60.0	41	146.7	3.949	2.716	83	37	0.594	Non-Liq.	0.00	0.00
61.0	76	146.7	4.022	2.759	111	69	0.596	Non-Liq.	0.00	0.00
62.0	76	146.7	4.096	2.801	111	68	0.598	Non-Liq.	0.00	0.00
63.0	76	146.7	4.169	2.843	111	68	0.600	Non-Liq.	0.00	0.00
64.0	76	146.7	4.242	2.885	111	67	0.601	Non-Liq.	0.00	0.00
65.0	76	146.7	4.316	2.927	111	67	0.603	Non-Liq.	0.00	0.00
66.0	76	146.7	4.389	2.969	108	67	0.604	Non-Liq.	0.00	0.00
67.0	76	146.7	4.462	3.011	108	66	0.606	Non-Liq.	0.00	0.00
68.0	76	146.7	4.536	3.054	108	66	0.607	Non-Liq.	0.00	0.00
69.0	76	146.7	4.609	3.096	108	65	0.609	Non-Liq.	0.00	0.00
70.0	76	146.7	4.682	3.138	108	65	0.610	Non-Liq.	0.00	0.00
71.0	43	146.7	4.756	3.180	79	36	0.611	Non-Liq.	0.00	0.00
72.0	43	146.7	4.829	3.222	79	36	0.613	Non-Liq.	0.00	0.00
73.0	43	146.7	4.902	3.264	79	36	0.614	Non-Liq.	0.00	0.00
74.0	43	146.7	4.976	3.307	79	36	0.615	Non-Liq.	0.00	0.00
75.0	43	146.7	5.049	3.349	79	36	0.616	Non-Liq.	0.00	0.00
76.0	92	146.7	5.122	3.391	113	76	0.618	Non-Liq.	0.00	0.00
77.0	92	146.7	5.196	3.433	113	75	0.619	Non-Liq.	0.00	0.00
78.0	92	146.7	5.269	3.475	113	75	0.620	Non-Liq.	0.00	0.00
79.0	92	146.7	5.342	3.517	113	74	0.621	Non-Liq.	0.00	0.00
80.0	92	146.7	5.416	3.559	113	74	0.622	Non-Liq.	0.00	0.00

TOTAL SETTLEMENT = 0.7 INCHES

GEOCON

Project Name : Hope Village
Project No : W1814-06-01
Boring : B1

EMPIRICAL ESTIMATION OF LIQUEFACTION POTENTIAL
MAXIMUM CONSIDERED EARTHQUAKE

NCEER (1996) METHOD W 2001 UPDATES

EARTHQUAKE INFORMATION:

Earthquake Magnitude:	6.85
Peak Horiz. Acceleration PGA_M (g):	0.943
Magnitude Scaling Factor:	1.261
Historic High Groundwater:	20.0
Groundwater Depth During Exploration:	24.0

ENERGY & ROD CORRECTIONS:

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

LIQUEFACTION CALCULATIONS:

Unit Wt. Water (pcf):	62.4

Depth to Base (ft)	Total Unit Wt. (pcf)	Water (0 or 1)	Field SPT (N)	Depth of SPT (ft)	Liq.Sus. (0 or 1)	-200 (%)	Est. Dr (%)	CN Factor	Corrected (N1)60cs	Eff. Unit Wt. (psf)	Resist. CRR 7.5	rd Factor	Induced CSR	Liquefac. Safe.Fact.
1.0	125.0	0	11.6	1.0	1		82	1.700	22.1	125.0	0.243	1.000	0.613	--
2.0	125.0	0	11.6	2.0	1	0	80	1.700	22.1	125.0	0.243	0.998	0.612	--
3.0	114.8	0	11.6	2.5	1	0	78	1.700	22.1	114.8	0.243	0.996	0.611	--
4.0	114.8	0	11.6	2.5	1	0	78	1.700	22.1	114.8	0.243	0.994	0.609	--
5.0	114.8	0	11.6	2.5	1	0	78	1.700	22.1	114.8	0.243	0.991	0.608	--
6.0	114.8	0	9.0	5.0	1	0	65	1.700	17.2	114.8	0.183	0.989	0.606	--
7.0	114.8	0	9.0	5.0	1	0	65	1.650	16.7	114.8	0.178	0.987	0.605	--
8.0	121.3	0	9.0	5.0	1	0	65	1.536	15.6	121.3	0.166	0.985	0.603	--
9.0	121.3	0	9.0	5.0	1	0	65	1.441	14.6	121.3	0.156	0.982	0.602	--
10.0	121.3	0	9.0	5.0	1	0	65	1.361	13.8	121.3	0.148	0.980	0.601	--
11.0	125.0	0	29.0	10.0	1	0	106	1.292	42.2	125.0	Infin.	0.978	0.600	--
12.0	125.0	0	29.0	10.0	1	0	106	1.232	40.2	125.0	Infin.	0.976	0.598	--
13.0	125.0	0	29.0	10.0	1	0	106	1.180	38.5	125.0	Infin.	0.974	0.597	--
14.0	125.0	0	29.0	10.0	1	0	106	1.133	37.0	125.0	Infin.	0.972	0.596	--
15.0	125.0	0	29.0	10.0	1	0	106	1.092	35.6	125.0	Infin.	0.970	0.594	--
16.0	123.4	0	31.0	15.0	1	0	100	1.055	39.6	123.4	Infin.	0.967	0.593	--
17.0	123.4	0	31.0	15.0	1	0	100	1.022	38.3	123.4	Infin.	0.965	0.592	--
18.0	123.4	0	31.0	15.0	1	0	100	0.992	37.2	123.4	Infin.	0.963	0.590	--
19.0	123.4	0	31.0	15.0	1	0	100	0.964	36.2	123.4	Infin.	0.961	0.589	--
20.0	123.4	0	31.0	15.0	1	0	100	0.939	35.2	123.4	Infin.	0.958	0.587	--
21.0	123.4	1	50.0	20.0	1	0	118	0.915	61.4	61.0	Infin.	0.956	0.593	Non-Liq.
22.0	123.4	1	50.0	20.0	1	0	118	0.894	60.0	61.0	Infin.	0.953	0.606	Non-Liq.
23.0	124.2	1	50.0	20.0	1	0	118	0.873	58.6	61.8	Infin.	0.950	0.617	Non-Liq.
24.0	124.2	1	50.0	20.0	1	0	118	0.859	57.6	61.8	Infin.	0.947	0.628	Non-Liq.
25.0	124.2	1	50.0	20.0	1	0	118	0.849	57.0	61.8	Infin.	0.944	0.638	Non-Liq.
26.0	124.2	1	18.0	25.0	1	36	68	0.841	31.0	61.8	Infin.	0.940	0.648	Non-Liq.
27.0	124.2	1	18.0	25.0	1	36	68	0.832	30.7	61.8	Infin.	0.936	0.656	Non-Liq.
28.0	130.6	1	45.7	27.5	1	3	106	0.823	55.2	68.2	Infin.	0.932	0.664	Non-Liq.
29.0	130.6	1	45.7	27.5	1	3	106	0.814	54.6	68.2	Infin.	0.928	0.670	Non-Liq.
30.0	130.6	1	45.7	27.5	1	3	106	0.805	54.0	68.2	Infin.	0.923	0.676	Non-Liq.
31.0	130.6	1	28.0	30.0	1	3	82	0.797	33.5	68.2	Infin.	0.918	0.682	Non-Liq.
32.0	145.4	1	39.1	32.5	1	3	95	0.788	46.2	83.0	Infin.	0.912	0.686	Non-Liq.
33.0	145.4	1	39.1	32.5	1	3	95	0.778	45.6	83.0	Infin.	0.907	0.689	Non-Liq.
34.0	145.4	1	39.1	32.5	1	3	95	0.769	45.1	83.0	Infin.	0.900	0.691	Non-Liq.
35.0	145.4	1	39.1	32.5	1	0	95	0.760	44.5	83.0	Infin.	0.894	0.693	Non-Liq.
36.0	145.4	1	50.0	35.0	1	0	106	0.752	56.4	83.0	Infin.	0.887	0.694	Non-Liq.
37.0	145.4	1	50.0	35.0	1	0	106	0.743	55.8	83.0	Infin.	0.880	0.694	Non-Liq.
38.0	145.4	1	50.0	35.0	1	0	106	0.735	55.2	83.0	Infin.	0.872	0.693	Non-Liq.
39.0	145.4	1	50.0	35.0	1	0	106	0.728	54.6	83.0	Infin.	0.864	0.692	Non-Liq.
40.0	145.4	1	50.0	35.0	1	0	106	0.720	54.0	83.0	Infin.	0.855	0.690	Non-Liq.
41.0	140.9	1	44.0	40.0	1	0	95	0.713	47.1	78.5	Infin.	0.846	0.688	Non-Liq.
42.0	140.9	1	44.0	40.0	1	0	95	0.706	46.6	78.5	Infin.	0.837	0.685	Non-Liq.
43.0	140.9	1	44.0	40.0	1	0	95	0.700	46.2	78.5	Infin.	0.828	0.682	Non-Liq.
44.0	140.9	1	44.0	40.0	1	0	95	0.693	45.8	78.5	Infin.	0.818	0.678	Non-Liq.
45.0	140.9	1	44.0	40.0	1	0	95	0.687	45.4	78.5	Infin.	0.808	0.674	Non-Liq.
46.0	140.9	1	100.0	45.0	1	0	139	0.681	102.2	78.5	Infin.	0.798	0.670	Non-Liq.
47.0	140.9	1	100.0	45.0	1	0	139	0.675	101.3	78.5	Infin.	0.788	0.665	Non-Liq.
48.0	140.9	1	100.0	45.0	1	0	139	0.670	100.4	78.5	Infin.	0.778	0.660	Non-Liq.
49.0	140.9	1	100.0	45.0	1	0	139	0.664	99.6	78.5	Infin.	0.768	0.655	Non-Liq.
50.0	140.9	1	100.0	45.0	1	0	139	0.659	98.8	78.5	Infin.	0.757	0.649	Non-Liq.
51.0	127.7	1	23.0	50.0	1	81	65	0.654	32.1	65.3	Infin.	0.747	0.644	Non-Liq.
52.0	127.7	1	23.0	50.0	1	81	65	0.649	31.9	65.3	Infin.	0.737	0.638	Non-Liq.
53.0	127.7	1	37.4	50.0	1	81	83	0.645	48.4	65.3	Infin.	0.727	0.633	Non-Liq.
54.0	127.7	1	37.4	50.0	1	81	83	0.641	48.2	65.3	Infin.	0.717	0.628	Non-Liq.
55.0	127.7	1	37.4	50.0	1	81	83	0.637	47.9	65.3	Infin.	0.708	0.622	Non-Liq.
56.0	127.7	1	74.0	55.0	1	0	113	0.633	70.3	65.3	Infin.	0.698	0.617	Non-Liq.
57.0	127.7	1	74.0	55.0	1	0	113	0.629	69.8	65.3	Infin.	0.689	0.611	Non-Liq.
58.0	127.7	1	74.0	55.0	1	0	113	0.625	69.4	65.3	Infin.	0.680	0.606	Non-Liq.
59.0	127.7	1	74.0	55.0	1	0	113	0.621	69.0	65.3	Infin.	0.671	0.601	Non-Liq.
60.0	127.7	1	74.0	55.0	1	0	113	0.618	68.6	65.3	Infin.	0.663	0.596	Non-Liq.
61.0	129.0	1	76.0	60.0	1	0	112	0.614	70.0	66.6	Infin.	0.655	0.591	Non-Liq.
62.0	129.0	1	76.0	60.0	1	0	112	0.610	69.6	66.6	Infin.	0.647	0.586	Non-Liq.
63.0	129.0	1	76.0	60.0	1	0	112	0.607	69.2	66.6	Infin.	0.639	0.581	Non-Liq.
64.0	129.0	1	76.0	60.0	1	0	112	0.603	68.8	66.6	Infin.	0.632	0.577	Non-Liq.
65.0	129.0	1	76.0	60.0	1	0	112	0.600	68.4	66.6	Infin.	0.625	0.573	Non-Liq.

GEOCON

Project Name : Hope Village
Project No : W1814-06-01
Boring : B4

EMPIRICAL ESTIMATION OF LIQUEFACTION POTENTIAL
MAXIMUM CONSIDERED EARTHQUAKE

NCEER (1996) METHOD W 2001 UPDATES

EARTHQUAKE INFORMATION:

Earthquake Magnitude:	6.85
Peak Horiz. Acceleration PGA_M (g):	0.943
Magnitude Scaling Factor:	1.261
Historic High Groundwater:	20.0
Groundwater Depth During Exploration:	24.0

ENERGY & ROD CORRECTIONS:

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

LIQUEFACTION CALCULATIONS:

Unit Wt. Water (pcf):	62.4

Depth to Base (ft)	Total Unit Wt. (pcf)	Water (0 or 1)	Field SPT (N)	Depth of SPT (ft)	Liq.Sus. (0 or 1)	-200 (%)	Est. Dr (%)	CN Factor	Corrected (N1)60cs	Eff. Unit Wt. (psf)	Resist. CRR 7.5	rd Factor	Induced CSR	Liquefac. Safe.Fact.
1.0	125.0	0	4.4	1.0	1		51	1.700	8.4	125.0	0.099	1.000	0.613	--
2.0	125.0	0	4.4	2.0	1	0	49	1.700	8.4	125.0	0.099	0.998	0.612	--
3.0	111.0	0	4.4	2.5	1	0	48	1.700	8.4	111.0	0.099	0.996	0.611	--
4.0	111.0	0	4.4	2.5	1	0	48	1.700	8.4	111.0	0.099	0.994	0.609	--
5.0	111.0	0	4.4	2.5	1	0	48	1.700	8.4	111.0	0.099	0.991	0.608	--
6.0	111.0	0	3.0	5.0	1	0	38	1.700	5.7	111.0	0.078	0.989	0.606	--
7.0	111.0	0	3.0	5.0	1	0	38	1.669	5.6	111.0	0.077	0.987	0.605	--
8.0	113.1	0	26.4	7.5	1	0	106	1.557	46.2	113.1	Infin.	0.985	0.603	--
9.0	113.1	0	26.4	7.5	1	0	106	1.464	43.5	113.1	Infin.	0.982	0.602	--
10.0	113.1	0	26.4	7.5	1	0	106	1.385	41.1	113.1	Infin.	0.980	0.601	--
11.0	113.1	0	32.0	10.0	1	0	112	1.319	47.5	113.1	Infin.	0.978	0.600	--
12.0	113.1	0	32.0	10.0	1	0	112	1.261	45.4	113.1	Infin.	0.976	0.598	--
13.0	120.7	0	32.0	10.0	1	0	112	1.208	43.5	120.7	Infin.	0.974	0.597	--
14.0	120.7	0	32.0	10.0	1	0	112	1.160	41.8	120.7	Infin.	0.972	0.596	--
15.0	120.7	0	32.0	10.0	1	0	112	1.117	40.2	120.7	Infin.	0.970	0.594	--
16.0	120.7	0	32.0	15.0	1	0	103	1.079	41.8	120.7	Infin.	0.967	0.593	--
17.0	120.7	0	32.0	15.0	1	0	103	1.045	40.4	120.7	Infin.	0.965	0.592	--
18.0	116.2	0	32.0	15.0	1	0	103	1.014	39.2	116.2	Infin.	0.963	0.590	--
19.0	116.2	0	32.0	15.0	1	0	103	0.986	38.2	116.2	Infin.	0.961	0.589	--
20.0	116.2	0	32.0	15.0	1	0	103	0.960	37.2	116.2	Infin.	0.958	0.587	--
21.0	116.2	1	56.0	20.0	1	0	126	0.937	70.4	53.8	Infin.	0.956	0.593	Non-Liq.
22.0	116.2	1	56.0	20.0	1	0	126	0.914	68.7	53.8	Infin.	0.953	0.607	Non-Liq.
23.0	139.0	1	56.0	20.0	1	0	126	0.892	67.1	76.6	Infin.	0.950	0.619	Non-Liq.
24.0	139.0	1	56.0	20.0	1	0	126	0.874	65.7	76.6	Infin.	0.947	0.630	Non-Liq.
25.0	139.0	1	56.0	20.0	1	0	126	0.862	64.8	76.6	Infin.	0.944	0.640	Non-Liq.
26.0	139.0	1	72.0	25.0	1	0	137	0.851	87.8	76.6	Infin.	0.940	0.649	Non-Liq.
27.0	139.0	1	72.0	25.0	1	0	137	0.840	86.6	76.6	Infin.	0.936	0.658	Non-Liq.
28.0	158.4	1	72.0	25.0	1	0	137	0.828	85.4	96.0	Infin.	0.932	0.665	Non-Liq.
29.0	158.4	1	72.0	25.0	1	0	137	0.815	84.1	96.0	Infin.	0.928	0.671	Non-Liq.
30.0	158.4	1	72.0	25.0	1	0	137	0.803	82.8	96.0	Infin.	0.923	0.676	Non-Liq.
31.0	158.4	1	27.0	30.0	1	5	80	0.791	32.0	96.0	Infin.	0.918	0.680	Non-Liq.
32.0	158.4	1	27.0	30.0	1	5	80	0.780	31.6	96.0	Infin.	0.912	0.683	Non-Liq.
33.0	131.5	1	38.0	32.5	1	5	93	0.771	43.9	69.1	Infin.	0.907	0.686	Non-Liq.
34.0	131.5	1	38.0	32.5	1	5	93	0.763	43.4	69.1	Infin.	0.900	0.689	Non-Liq.
35.0	131.5	1	38.0	32.5	1	5	93	0.756	43.0	69.1	Infin.	0.894	0.691	Non-Liq.
36.0	131.5	1	23.0	35.0	1	4	71	0.749	25.8	69.1	0.309	0.887	0.692	0.46
37.0	131.5	1	23.0	35.0	1	4	71	0.742	25.6	69.1	0.304	0.880	0.693	0.45
38.0	129.0	1	55.0	37.5	1	4	108	0.736	60.7	66.6	Infin.	0.872	0.693	Non-Liq.
39.0	129.0	1	55.0	37.5	1	4	108	0.729	60.2	66.6	Infin.	0.864	0.693	Non-Liq.
40.0	129.0	1	55.0	37.5	1	4	108	0.723	59.7	66.6	Infin.	0.855	0.692	Non-Liq.
41.0	135.6	1	22.0	40.0	1	7	68	0.717	24.0	73.2	0.273	0.846	0.690	0.40
42.0	135.6	1	22.0	40.0	1	7	68	0.711	23.8	73.2	0.269	0.837	0.688	0.39
43.0	135.6	1	55.0	42.5	1	7	105	0.704	58.7	73.2	Infin.	0.828	0.685	Non-Liq.
44.0	135.6	1	55.0	42.5	1	7	105	0.698	58.2	73.2	Infin.	0.818	0.681	Non-Liq.
45.0	135.6	1	55.0	42.5	1	7	105	0.692	57.7	73.2	Infin.	0.808	0.677	Non-Liq.
46.0	135.6	1	30.0	45.0	1	0	77	0.687	30.9	73.2	Infin.	0.798	0.673	Non-Liq.
47.0	135.6	1	30.0	45.0	1	0	77	0.681	30.6	73.2	Infin.	0.788	0.668	Non-Liq.
48.0	156.1	1	30.0	45.0	1	0	77	0.675	30.4	93.7	Infin.	0.778	0.663	Non-Liq.
49.0	156.1	1	30.0	45.0	1	0	77	0.668	30.1	93.7	Infin.	0.768	0.657	Non-Liq.
50.0	156.1	1	30.0	45.0	1	0	77	0.661	29.8	93.7	0.452	0.757	0.651	0.64
51.0	146.7	1	58.0	50.0	1	0	103	0.655	57.0	84.3	Infin.	0.747	0.645	Non-Liq.
52.0	146.7	1	58.0	50.0	1	0	103	0.650	56.5	84.3	Infin.	0.737	0.639	Non-Liq.
53.0	146.7	1	58.0	50.0	1	0	103	0.644	56.1	84.3	Infin.	0.727	0.632	Non-Liq.
54.0	146.7	1	58.0	50.0	1	0	103	0.639	55.6	84.3	Infin.	0.717	0.626	Non-Liq.
55.0	146.7	1	58.0	50.0	1	0	103	0.634	55.1	84.3	Infin.	0.708	0.620	Non-Liq.
56.0	146.7	1	79.0	55.0	1	0	117	0.629	74.5	84.3	Infin.	0.698	0.614	Non-Liq.
57.0	146.7	1	79.0	55.0	1	0	117	0.624	73.9	84.3	Infin.	0.689	0.608	Non-Liq.
58.0	146.7	1	41.0	57.5	1	0	83	0.619	38.1	84.3	Infin.	0.680	0.602	Non-Liq.
59.0	146.7	1	41.0	57.5	1	0	83	0.614	37.8	84.3	Infin.	0.671	0.596	Non-Liq.
60.0	146.7	1	41.0	57.5	1	0	83	0.610	37.5	84.3	Infin.	0.663	0.591	Non-Liq.
61.0	146.7	1	76.0	60.0	1	0	111	0.605	69.0	84.3	Infin.	0.655	0.585	Non-Liq.
62.0	146.7	1	76.0	60.0	1	0	111	0.601	68.5	84.3	Infin.	0.647	0.580	Non-Liq.
63.0	146.7	1	76.0	60.0	1	0	111	0.596	68.0	84.3	Infin.	0.639	0.575	Non-Liq.
64.0	146.7	1	76.0	60.0	1	0	111	0.592	67.5	84.3	Infin.	0.632	0.570	Non-Liq.
65.0	146.7	1	76.0	60.0	1	0	111	0.588	67.0	84.3	Infin.	0.625	0.565	Non-Liq.
66.0	146.7	1	76.0	65.0	1	0	108	0.584	66.6	84.3	Infin.	0.618	0.560	Non-Liq.
67.0	146.7	1	76.0	65.0	1	0	108	0.580	66.1	84.3	Infin.	0.612	0.556	Non-Liq.
68.0	146.7	1	76.0	65.0	1	0	108	0.576	65.7	84.3	Infin.	0.606	0.552	Non-Liq.
69.0	146.7	1	76.0	65.0	1	0	108	0.572	65.2	84.3	Infin.	0.600	0.548	Non-Liq.
70.0	146.7	1	76.0	65.0	1	0	108	0.568	64.8	84.3	Infin.	0.594	0.544	Non-Liq.
71.0	146.7	1	43.0	70.0	1	0	79	0.565	36.4	84.3	Infin.	0.589	0.540	Non-Liq.
72.0	146.7	1	43.0	70.0	1	0	79	0.561	36.2	84.3	Infin.	0.584	0.536	Non-Liq.
73.0	146.7	1	43.0	70.0	1	0	79	0.558	36.0	84.3	Infin.	0.579	0.533	Non-Liq.
74.0	146.7	1	43.0	70.0	1	0	79	0.554	35.7	84.3	Infin.	0.574	0.530	Non-Liq.
75.0	146.7	1	43.0	70.0	1	0	79	0.551	35.5	84.3	Infin.	0.570	0.526	Non-Liq.
76.0	146.7	1	92.0	75.0	1	0	113	0.547	75.5	84.3	Infin.	0.565	0.523	Non-Liq.
77.0	146.7	1	92.0	75.0	1	0	113	0.544	75.1	84.3	Infin.	0.561	0.521	Non-Liq.
78.0	146.7	1	92.0	75.0	1	0	113	0.541	74.6	84.3	Infin.	0.557	0.518	Non-Liq.
79.0	146.7	1	92.0	75.0	1	0	113	0.538	74.2	84.3	Infin.	0.553	0.515	Non-Liq.
80.0	146.7	1	92.0	75.0	1	0	113	0.535	73.8	84.3	Infin.	0.550	0.513	Non-Liq.

Project Name : Hope Village
Project No : W1814-06-01
Boring : B1

LIQUEFACTION SETTLEMENT ANALYSIS
MAXIMUM CONSIDERED EARTHQUAKE

NCEER (1996) METHOD W 2001 UPDATES
EARTHQUAKE INFORMATION:

Earthquake Magnitude:	6.85
PGA_M (g):	0.943
Magnitude Scaling Factor:	1.261
Historic High Groundwater:	20.0
Groundwater @ Exploration:	24.0

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	VOL. STRAIN $[e_{15}]$ (%)	EQ. SETTLE. Pe (in.)
1.0	12	125.0	0.031	0.031	82	22	0.613	--	0.00	0.00
2.0	12	125.0	0.094	0.094	80	22	0.613	--	0.00	0.00
3.0	12	114.8	0.154	0.154	78	22	0.613	--	0.00	0.00
4.0	12	114.8	0.211	0.211	78	22	0.613	--	0.00	0.00
5.0	12	114.8	0.269	0.269	78	22	0.613	--	0.00	0.00
6.0	9	114.8	0.326	0.326	65	17	0.613	--	0.00	0.00
7.0	9	114.8	0.383	0.383	65	17	0.613	--	0.00	0.00
8.0	9	121.3	0.442	0.442	65	16	0.613	--	0.00	0.00
9.0	9	121.3	0.503	0.503	65	15	0.613	--	0.00	0.00
10.0	9	121.3	0.564	0.564	65	14	0.613	--	0.00	0.00
11.0	29	125.0	0.625	0.625	106	42	0.613	--	0.00	0.00
12.0	29	125.0	0.688	0.688	106	40	0.613	--	0.00	0.00
13.0	29	125.0	0.750	0.750	106	38	0.613	--	0.00	0.00
14.0	29	125.0	0.813	0.813	106	37	0.613	--	0.00	0.00
15.0	29	125.0	0.875	0.875	106	36	0.613	--	0.00	0.00
16.0	31	123.4	0.937	0.937	100	40	0.613	--	0.00	0.00
17.0	31	123.4	0.999	0.999	100	38	0.613	--	0.00	0.00
18.0	31	123.4	1.061	1.061	100	37	0.613	--	0.00	0.00
19.0	31	123.4	1.122	1.122	100	36	0.613	--	0.00	0.00
20.0	31	123.4	1.184	1.184	100	35	0.613	--	0.00	0.00
21.0	50	123.4	1.246	1.230	118	61	0.621	Non-Liq.	0.00	0.00
22.0	50	123.4	1.308	1.261	118	60	0.636	Non-Liq.	0.00	0.00
23.0	50	124.2	1.369	1.291	118	59	0.650	Non-Liq.	0.00	0.00
24.0	50	124.2	1.432	1.322	118	58	0.664	Non-Liq.	0.00	0.00
25.0	50	124.2	1.494	1.353	118	57	0.677	Non-Liq.	0.00	0.00
26.0	18	124.2	1.556	1.384	68	31	0.689	Non-Liq.	0.00	0.00
27.0	18	124.2	1.618	1.415	68	31	0.701	Non-Liq.	0.00	0.00
28.0	46	130.6	1.682	1.448	106	55	0.712	Non-Liq.	0.00	0.00
29.0	46	130.6	1.747	1.482	106	55	0.723	Non-Liq.	0.00	0.00
30.0	46	130.6	1.812	1.516	106	54	0.733	Non-Liq.	0.00	0.00
31.0	28	130.6	1.877	1.550	82	33	0.743	Non-Liq.	0.00	0.00
32.0	39	145.4	1.946	1.588	95	46	0.751	Non-Liq.	0.00	0.00
33.0	39	145.4	2.019	1.629	95	46	0.760	Non-Liq.	0.00	0.00
34.0	39	145.4	2.092	1.671	95	45	0.767	Non-Liq.	0.00	0.00
35.0	39	145.4	2.165	1.712	95	45	0.775	Non-Liq.	0.00	0.00
36.0	50	145.4	2.237	1.754	106	56	0.782	Non-Liq.	0.00	0.00
37.0	50	145.4	2.310	1.795	106	56	0.789	Non-Liq.	0.00	0.00
38.0	50	145.4	2.383	1.837	106	55	0.795	Non-Liq.	0.00	0.00
39.0	50	145.4	2.455	1.878	106	55	0.801	Non-Liq.	0.00	0.00
40.0	50	145.4	2.528	1.920	106	54	0.807	Non-Liq.	0.00	0.00
41.0	44	140.9	2.600	1.960	95	47	0.813	Non-Liq.	0.00	0.00
42.0	44	140.9	2.670	1.999	95	47	0.819	Non-Liq.	0.00	0.00
43.0	44	140.9	2.740	2.038	95	46	0.824	Non-Liq.	0.00	0.00
44.0	44	140.9	2.811	2.078	95	46	0.829	Non-Liq.	0.00	0.00
45.0	44	140.9	2.881	2.117	95	45	0.834	Non-Liq.	0.00	0.00
46.0	100	140.9	2.952	2.156	139	102	0.839	Non-Liq.	0.00	0.00
47.0	100	140.9	3.022	2.195	139	101	0.844	Non-Liq.	0.00	0.00
48.0	100	140.9	3.093	2.235	139	100	0.848	Non-Liq.	0.00	0.00
49.0	100	140.9	3.163	2.274	139	100	0.853	Non-Liq.	0.00	0.00
50.0	100	140.9	3.234	2.313	139	99	0.857	Non-Liq.	0.00	0.00
51.0	23	127.7	3.301	2.349	65	32	0.861	Non-Liq.	0.00	0.00
52.0	23	127.7	3.365	2.382	65	32	0.866	Non-Liq.	0.00	0.00
53.0	37	127.7	3.428	2.414	83	48	0.870	Non-Liq.	0.00	0.00
54.0	37	127.7	3.492	2.447	83	48	0.875	Non-Liq.	0.00	0.00
55.0	37	127.7	3.556	2.480	83	48	0.879	Non-Liq.	0.00	0.00
56.0	74	127.7	3.620	2.512	113	70	0.883	Non-Liq.	0.00	0.00
57.0	74	127.7	3.684	2.545	113	70	0.887	Non-Liq.	0.00	0.00
58.0	74	127.7	3.748	2.578	113	69	0.891	Non-Liq.	0.00	0.00
59.0	74	127.7	3.812	2.610	113	69	0.895	Non-Liq.	0.00	0.00
60.0	74	127.7	3.875	2.643	113	69	0.899	Non-Liq.	0.00	0.00
61.0	76	129.0	3.940	2.676	112	70	0.902	Non-Liq.	0.00	0.00
62.0	76	129.0	4.004	2.709	112	70	0.906	Non-Liq.	0.00	0.00
63.0	76	129.0	4.069	2.743	112	69	0.909	Non-Liq.	0.00	0.00
64.0	76	129.0	4.133	2.776	112	69	0.913	Non-Liq.	0.00	0.00
65.0	76	129.0	4.198	2.809	112	68	0.916	Non-Liq.	0.00	0.00

TOTAL SETTLEMENT = 0.0 INCHES

Project Name : Hope Village
Project No : W1814-06-01
Boring : B4

LIQUEFACTION SETTLEMENT ANALYSIS
DESIGN EARTHQUAKE

NCEER (1996) METHOD W 2001 UPDATES
EARTHQUAKE INFORMATION:

Earthquake Magnitude:	6.74
PGA_M (g):	0.943
2/3 PGA_M (g):	0.63
Magnitude Scaling Factor:	1.314
Historic High Groundwater:	20.0
Groundwater @ Exploration:	24.0

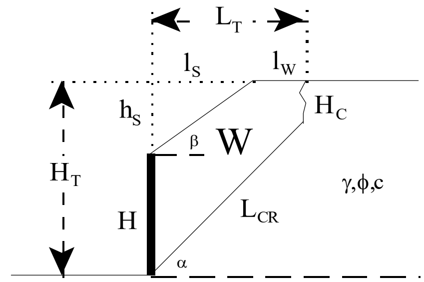
DEPTH TO BASE	BLOW COUNT N	WET DENSITY (PCF)	TOTAL STRESS O (TSF)	EFFECT STRESS O' (TSF)	REL. DEN. Dr (%)	ADJUST BLOWS (N1)60	Tav/σ'$_o$	LIQUEFACTION SAFETY FACTOR	VOL. STRAIN $[e_{15}]$ (%)	EQ. SETTLE. Pe (in.)
1.0	4	125.0	0.031	0.031	51	8	0.409	--	0.00	0.00
2.0	4	125.0	0.094	0.094	49	8	0.409	--	0.00	0.00
3.0	4	111.0	0.153	0.153	48	8	0.409	--	0.00	0.00
4.0	4	111.0	0.208	0.208	48	8	0.409	--	0.00	0.00
5.0	4	111.0	0.264	0.264	48	8	0.409	--	0.00	0.00
6.0	3	111.0	0.319	0.319	38	6	0.409	--	0.00	0.00
7.0	3	111.0	0.375	0.375	38	6	0.409	--	0.00	0.00
8.0	26	113.1	0.431	0.431	106	46	0.409	--	0.00	0.00
9.0	26	113.1	0.487	0.487	106	43	0.409	--	0.00	0.00
10.0	26	113.1	0.544	0.544	106	41	0.409	--	0.00	0.00
11.0	32	113.1	0.600	0.600	112	47	0.409	--	0.00	0.00
12.0	32	113.1	0.657	0.657	112	45	0.409	--	0.00	0.00
13.0	32	120.7	0.715	0.715	112	43	0.409	--	0.00	0.00
14.0	32	120.7	0.776	0.776	112	42	0.409	--	0.00	0.00
15.0	32	120.7	0.836	0.836	112	40	0.409	--	0.00	0.00
16.0	32	120.7	0.896	0.896	103	42	0.409	--	0.00	0.00
17.0	32	120.7	0.957	0.957	103	40	0.409	--	0.00	0.00
18.0	32	116.2	1.016	1.016	103	39	0.409	--	0.00	0.00
19.0	32	116.2	1.074	1.074	103	38	0.409	--	0.00	0.00
20.0	32	116.2	1.132	1.132	103	37	0.409	--	0.00	0.00
21.0	56	116.2	1.190	1.175	126	70	0.414	Non-Liq.	0.00	0.00
22.0	56	116.2	1.248	1.202	126	69	0.425	Non-Liq.	0.00	0.00
23.0	56	139.0	1.312	1.234	126	67	0.435	Non-Liq.	0.00	0.00
24.0	56	139.0	1.382	1.273	126	66	0.444	Non-Liq.	0.00	0.00
25.0	56	139.0	1.451	1.311	126	65	0.453	Non-Liq.	0.00	0.00
26.0	72	139.0	1.521	1.349	137	88	0.461	Non-Liq.	0.00	0.00
27.0	72	139.0	1.590	1.387	137	87	0.469	Non-Liq.	0.00	0.00
28.0	72	158.4	1.665	1.431	137	85	0.476	Non-Liq.	0.00	0.00
29.0	72	158.4	1.744	1.479	137	84	0.482	Non-Liq.	0.00	0.00
30.0	72	158.4	1.823	1.527	137	83	0.488	Non-Liq.	0.00	0.00
31.0	27	158.4	1.902	1.575	80	32	0.494	Non-Liq.	0.00	0.00
32.0	27	158.4	1.981	1.623	80	32	0.499	Non-Liq.	0.00	0.00
33.0	38	131.5	2.054	1.664	93	44	0.505	Non-Liq.	0.00	0.00
34.0	38	131.5	2.120	1.698	93	43	0.510	Non-Liq.	0.00	0.00
35.0	38	131.5	2.185	1.733	93	43	0.516	Non-Liq.	0.00	0.00
36.0	23	131.5	2.251	1.768	71	26	0.521	0.72	1.10	0.13
37.0	23	131.5	2.317	1.802	71	26	0.526	0.70	1.10	0.13
38.0	55	129.0	2.382	1.836	108	61	0.530	Non-Liq.	0.00	0.00
39.0	55	129.0	2.447	1.869	108	60	0.535	Non-Liq.	0.00	0.00
40.0	55	129.0	2.511	1.903	108	60	0.540	Non-Liq.	0.00	0.00
41.0	22	135.6	2.577	1.938	68	24	0.544	0.62	1.30	0.16
42.0	22	135.6	2.645	1.974	68	24	0.548	0.61	1.30	0.16
43.0	55	135.6	2.713	2.011	105	59	0.552	Non-Liq.	0.00	0.00
44.0	55	135.6	2.781	2.047	105	58	0.555	Non-Liq.	0.00	0.00
45.0	55	135.6	2.848	2.084	105	58	0.559	Non-Liq.	0.00	0.00
46.0	30	135.6	2.916	2.121	77	31	0.562	Non-Liq.	0.00	0.00
47.0	30	135.6	2.984	2.157	77	31	0.566	Non-Liq.	0.00	0.00
48.0	30	156.1	3.057	2.199	77	30	0.568	Non-Liq.	0.00	0.00
49.0	30	156.1	3.135	2.246	77	30	0.571	Non-Liq.	0.00	0.00
50.0	30	156.1	3.213	2.293	77	30	0.573	1.00	0.75	0.09
51.0	58	146.7	3.289	2.337	103	57	0.575	Non-Liq.	0.00	0.00
52.0	58	146.7	3.362	2.379	103	57	0.578	Non-Liq.	0.00	0.00
53.0	58	146.7	3.435	2.421	103	56	0.580	Non-Liq.	0.00	0.00
54.0	58	146.7	3.509	2.464	103	56	0.582	Non-Liq.	0.00	0.00
55.0	58	146.7	3.582	2.506	103	55	0.584	Non-Liq.	0.00	0.00
56.0	79	146.7	3.655	2.548	117	74	0.587	Non-Liq.	0.00	0.00
57.0	79	146.7	3.729	2.590	117	74	0.589	Non-Liq.	0.00	0.00
58.0	41	146.7	3.802	2.632	83	38	0.591	Non-Liq.	0.00	0.00
59.0	41	146.7	3.875	2.674	83	38	0.592	Non-Liq.	0.00	0.00
60.0	41	146.7	3.949	2.716	83	37	0.594	Non-Liq.	0.00	0.00
61.0	76	146.7	4.022	2.759	111	69	0.596	Non-Liq.	0.00	0.00
62.0	76	146.7	4.096	2.801	111	68	0.598	Non-Liq.	0.00	0.00
63.0	76	146.7	4.169	2.843	111	68	0.600	Non-Liq.	0.00	0.00
64.0	76	146.7	4.242	2.885	111	67	0.601	Non-Liq.	0.00	0.00
65.0	76	146.7	4.316	2.927	111	67	0.603	Non-Liq.	0.00	0.00
66.0	76	146.7	4.389	2.969	108	67	0.604	Non-Liq.	0.00	0.00
67.0	76	146.7	4.462	3.011	108	66	0.606	Non-Liq.	0.00	0.00
68.0	76	146.7	4.536	3.054	108	66	0.607	Non-Liq.	0.00	0.00
69.0	76	146.7	4.609	3.096	108	65	0.609	Non-Liq.	0.00	0.00
70.0	76	146.7	4.682	3.138	108	65	0.610	Non-Liq.	0.00	0.00
71.0	43	146.7	4.756	3.180	79	36	0.611	Non-Liq.	0.00	0.00
72.0	43	146.7	4.829	3.222	79	36	0.613	Non-Liq.	0.00	0.00
73.0	43	146.7	4.902	3.264	79	36	0.614	Non-Liq.	0.00	0.00
74.0	43	146.7	4.976	3.307	79	36	0.615	Non-Liq.	0.00	0.00
75.0	43	146.7	5.049	3.349	79	36	0.616	Non-Liq.	0.00	0.00
76.0	92	146.7	5.122	3.391	113	76	0.618	Non-Liq.	0.00	0.00
77.0	92	146.7	5.196	3.433	113	75	0.619	Non-Liq.	0.00	0.00
78.0	92	146.7	5.269	3.475	113	75	0.620	Non-Liq.	0.00	0.00
79.0	92	146.7	5.342	3.517	113	74	0.621	Non-Liq.	0.00	0.00
80.0	92	146.7	5.416	3.559	113	74	0.622	Non-Liq.	0.00	0.00

TOTAL SETTLEMENT = 0.7 INCHES

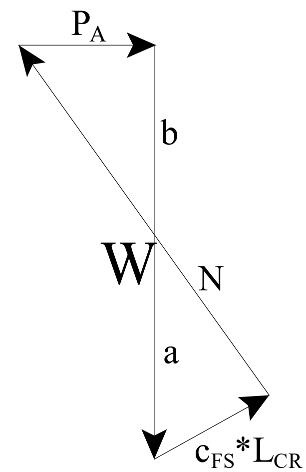
Retaining Wall Design with Transitioned Backfill
(Vector Analysis)

Input:

Retaining Wall Height	(H)	15.00 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)	15.0 feet
Unit Weight of Retained Soils	(g)	125.0 pcf
Friction Angle of Retained Soils	(f)	39.0 degrees
Cohesion of Retained Soils	(c)	29.0 psf
Factor of Safety	(FS)	1.50
Factored Parameters	(f_{FS})	28.4 degrees
	(c_{FS})	19.3 psf



Failure Angle (a)	Height of Tension Crack (H_C)	Area of Wedge (A)	Weight of Wedge (W)	Length of Failure Plane (L_{CR})	a	b	Active Pressure (P_A)
degrees	feet	feet^2	lbs/lineal foot	feet	lbs/lineal foot	lbs/lineal foot	lbs/lineal foot
45	0.7	112	14034	20	1204	12830	3834
46	0.6	108	13555	20	1120	12434	3953
47	0.6	105	13091	20	1046	12044	4062
48	0.6	101	12641	19	981	11661	4161
49	0.6	98	12206	19	922	11284	4250
50	0.6	94	11783	19	869	10914	4329
51	0.6	91	11372	19	821	10550	4400
52	0.6	88	10972	18	778	10194	4462
53	0.5	85	10583	18	739	9844	4515
54	0.5	82	10204	18	703	9501	4560
55	0.5	79	9834	18	670	9164	4597
56	0.5	76	9474	17	640	8833	4625
57	0.5	73	9121	17	613	8508	4646
58	0.5	70	8777	17	587	8189	4659
59	0.5	68	8440	17	564	7875	4664
60	0.5	65	8109	17	542	7567	4662
61	0.5	62	7786	17	522	7263	4652
62	0.5	60	7468	16	504	6965	4634
63	0.5	57	7156	16	486	6670	4608
64	0.5	55	6850	16	470	6380	4574
65	0.5	52	6549	16	455	6094	4532
66	0.5	50	6253	16	441	5812	4482
67	0.6	48	5961	16	427	5533	4423
68	0.6	45	5673	16	415	5258	4356
69	0.6	43	5390	15	403	4987	4280
70	0.6	41	5110	15	392	4718	4194



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

$P_{A,\ max}$ 4664 lbs/lineal foot

Equivalent Fluid Pressure (per lineal foot of wall)

$EFP = 2*P_A/H^2$

	Active	At-Rest
EFP	41.5 pcf	46.3 pcf
Design Wall for an Equivalent Fluid Pressure	**41 pcf**	**47** pcf

GEOCON WEST, INC.

ENVIRONMENTAL GEOTECHNICAL MATERIALS
500 N. VICTORY BOULEVARD - BURBANK, CA 91502
PHONE: 818-841-8388 FAX: 818-841-1704

RETAINING WALL PRESSURE CALCULATION

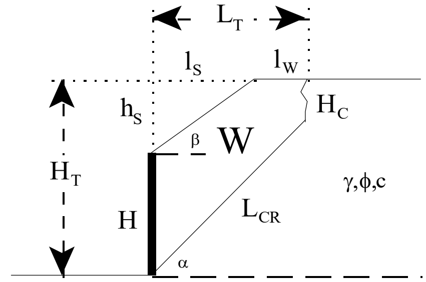
800 & 908 NORTH MAIN STREET
1081 & 1087 NORTH VIGNES STREET
LOS ANGELES, CALIFORNIA

DRAFTED BY: JJK	CHECKED BY: HHD	OCT. 2023	PROJECT NO. W1814-06-01	FIG. 13A

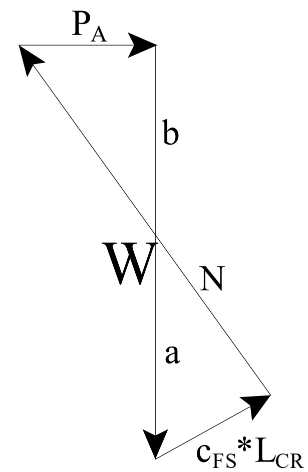
Retaining Wall Design with Transitioned Backfill
(Vector Analysis)

Input:

Retaining Wall Height	(H)	30.00	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)	30.0	feet
Unit Weight of Retained Soils	(g)	125.0	pcf
Friction Angle of Retained Soils	(f)	34.0	degrees
Cohesion of Retained Soils	(c)	6.0	psf
Factor of Safety	(FS)	1.50	
Factored Parameters	(f_{FS})	24.2	degrees
	(c_{FS})	4.0	psf



Failure Angle (a)	Height of Tension Crack (H_C)	Area of Wedge (A)	Weight of Wedge (W)	Length of Failure Plane (L_{CR})	a	b	Active Pressure (P_A)
degrees	feet	feet^2	lbs/lineal foot	feet	lbs/lineal foot	lbs/lineal foot	lbs/lineal foot
45	0.1	450	56249	42	434	55815	21189
46	0.1	435	54319	42	408	53911	21550
47	0.1	420	52453	41	385	52068	21875
48	0.1	405	50647	40	364	50283	22165
49	0.1	391	48897	40	345	48552	22422
50	0.1	378	47199	39	327	46872	22646
51	0.1	364	45550	38	311	45238	22840
52	0.1	352	43947	38	297	43650	23002
53	0.1	339	42387	37	284	42103	23135
54	0.1	327	40868	37	271	40596	23238
55	0.1	315	39386	37	260	39126	23313
56	0.1	304	37941	36	250	37691	23358
57	0.1	292	36529	36	240	36289	23376
58	0.1	281	35149	35	231	34917	23364
59	0.1	270	33798	35	223	33575	23325
60	0.1	260	32476	35	215	32260	23256
61	0.1	249	31180	34	208	30971	23159
62	0.1	239	29908	34	202	29707	23033
63	0.1	229	28660	34	195	28465	22877
64	0.1	219	27435	33	190	27245	22690
65	0.1	210	26229	33	184	26045	22472
66	0.1	200	25044	33	179	24865	22222
67	0.1	191	23876	32	174	23702	21939
68	0.1	182	22726	32	170	22556	21621
69	0.1	173	21592	32	166	21426	21268
70	0.1	164	20473	32	162	20311	20878



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

$P_{A,\ max}$ 23376 lbs/lineal foot

Equivalent Fluid Pressure (per lineal foot of wall)

$EFP = 2*P_A/H^2$

EFP 51.9 pcf 55.1 pcf

Design Wall for an Equivalent Fluid Pressure **52 pcf** **56** pcf

Active At-Rest

GEOCON WEST, INC.

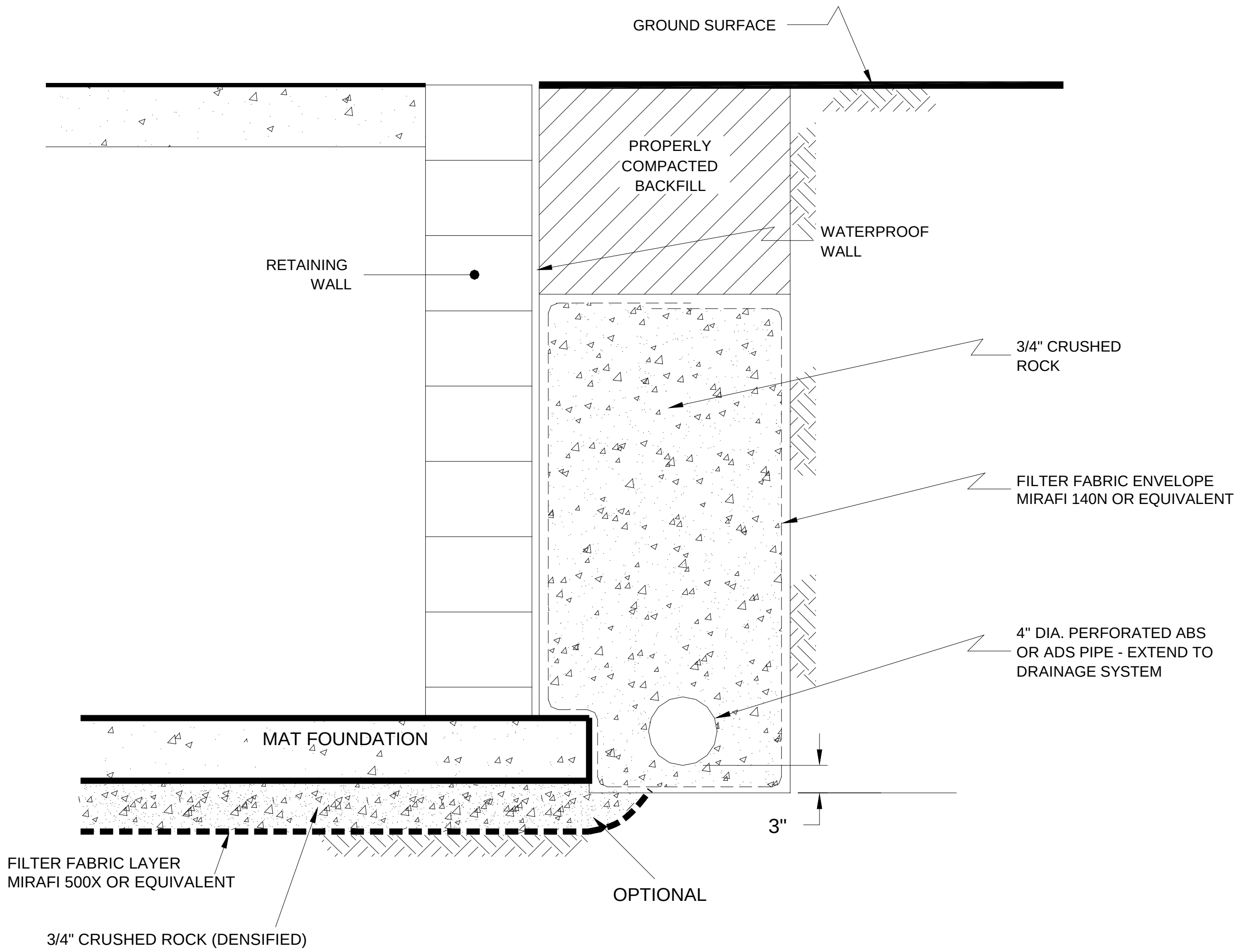
ENVIRONMENTAL GEOTECHNICAL MATERIALS

500 N. VICTORY BOULEVARD - BURBANK, CA 91502
PHONE: 818-841-8388 FAX: 818-841-1704

RETAINING WALL PRESSURE CALCULATION

800 & 908 NORTH MAIN STREET
1081 & 1087 NORTH VIGNES STREET
LOS ANGELES, CALIFORNIA

DRAFTED BY: JJK	CHECKED BY: HHD	OCT. 2023	PROJECT NO. W1814-06-01	FIG. 13B



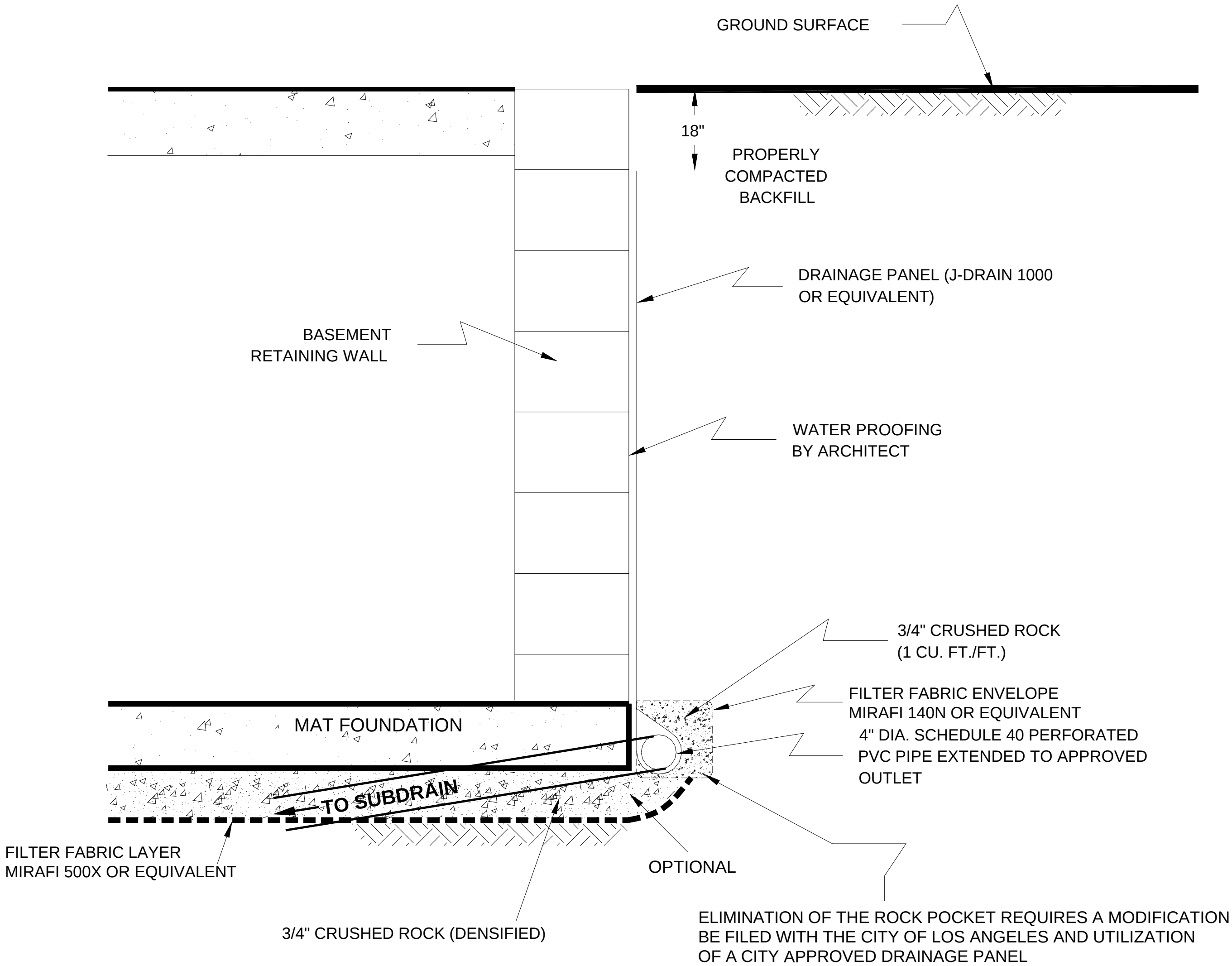
NO SCALE

GEOCON WEST, INC.
ENVIRONMENTAL GEOTECHNICAL MATERIALS
500 NORTH VICTORY BOULEVARD - BURBANK, CA 91502
PHONE (818) 841-8388 - FAX (818) 841-1704

RETAINING WALL DRAIN DETAIL

800 & 908 NORTH MAIN STREET
1081 & 1087 NORTH VIGNES STREET
LOS ANGELES, CALIFORNIA

DRAFTED BY: JJK	CHECKED BY: HHD	OCT. 2023	PROJECT NO. W1814-06-01	FIG. 14



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

NO SCALE

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RETAINING WALL DRAIN DETAIL

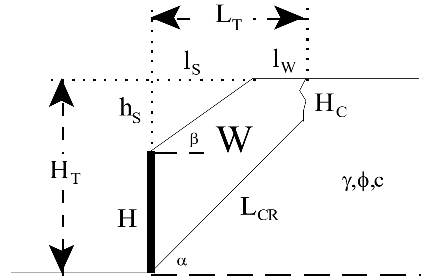
800 & 908 NORTH MAIN STREET
1081 & 1087 NORTH VIGNES STREET
LOS ANGELES, CALIFORNIA

DRAFTED BY: JJK	CHECKED BY: HHD	OCT. 2023	PROJECT NO. W1814-06-01	FIG. 15

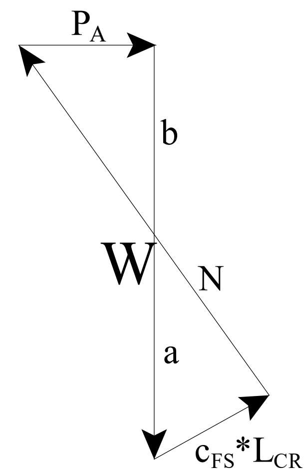
Shoring Design with Transitioned Backfill
(Vector Analysis)

Input:

Shoring Height	(H)	17.00	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)	17.0	feet
Unit Weight of Retained Soils	(g)	125.0	pcf
Friction Angle of Retained Soils	(f)	39.0	degrees
Cohesion of Retained Soils	(c)	29.0	psf
Factor of Safety	(FS)	1.25	
Factored Parameters	(f_{FS})	32.9	degrees
	(c_{FS})	23.2	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	1.1	144	17993	23	2101	15892	3396
46	1.0	139	17383	22	1917	15466	3589
47	0.9	134	16792	22	1760	15033	3766
48	0.9	130	16218	22	1624	14595	3928
49	0.9	125	15661	21	1505	14156	4076
50	0.8	121	15120	21	1401	13719	4211
51	0.8	117	14594	21	1309	13285	4333
52	0.8	113	14083	21	1227	12855	4442
53	0.8	109	13584	20	1155	12430	4540
54	0.7	105	13098	20	1089	12009	4625
55	0.7	101	12625	20	1030	11595	4700
56	0.7	97	12162	20	977	11185	4763
57	0.7	94	11710	19	928	10782	4815
58	0.7	90	11268	19	884	10384	4856
59	0.7	87	10835	19	843	9992	4887
60	0.7	83	10411	19	806	9605	4908
61	0.7	80	9996	19	772	9224	4918
62	0.7	77	9588	18	741	8848	4917
63	0.7	74	9188	18	712	8477	4907
64	0.7	70	8795	18	685	8110	4885
65	0.7	67	8409	18	660	7749	4854
66	0.7	64	8028	18	637	7392	4812
67	0.7	61	7654	18	615	7039	4759
68	0.7	58	7284	18	595	6690	4695
69	0.7	55	6920	17	576	6344	4620
70	0.8	52	6561	17	558	6003	4534



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

$P_{A,\ max}$ 4918 lbs/lineal foot

Equivalent Fluid Pressure (per lineal foot of shoring)

$EFP = 2*P_A/H^2$

EFP 34.0 pcf

Design Shoring for an Equivalent Fluid Pressure **34 pcf**

Active

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PHONE: 818-841-8388 FAX: 818-841-1704

SHORING WALL PRESSURE CALCULATION

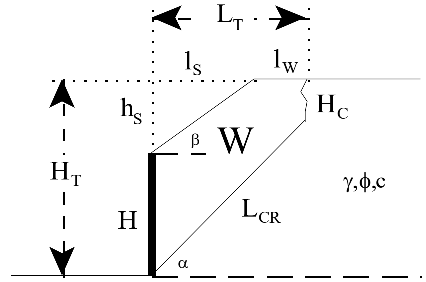
800 & 908 NORTH MAIN STREET
1081 & 1087 NORTH VIGNES STREET
LOS ANGELES, CALIFORNIA

DRAFTED BY: JJK	CHECKED BY: HHD	OCT. 2023	PROJECT NO. W1814-06-01	FIG. 16A

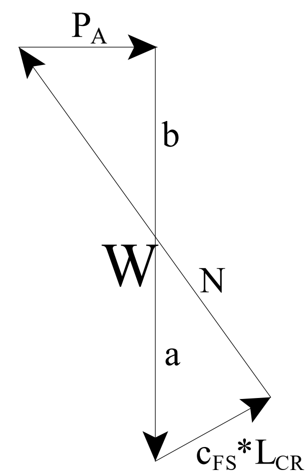
Shoring Design with Transitioned Backfill
(Vector Analysis)

Input:

Shoring Height	(H)	34.00	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)	34.0	feet
Unit Weight of Retained Soils	(g)	125.0	pcf
Friction Angle of Retained Soils	(f)	34.0	degrees
Cohesion of Retained Soils	(c)	6.0	psf
Factor of Safety	(FS)	1.25	
Factored Parameters	(f_{FS})	28.4	degrees
	(c_{FS})	4.8	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	0.2	578	72248	48	705	71543	21394
46	0.2	558	69769	47	655	69114	21989
47	0.2	539	67373	46	611	66761	22531
48	0.2	520	65053	46	572	64481	23022
49	0.1	502	62805	45	537	62267	23465
50	0.1	485	60624	44	506	60118	23861
51	0.1	468	58506	44	478	58028	24212
52	0.1	452	56447	43	453	55994	24520
53	0.1	436	54443	42	430	54014	24785
54	0.1	420	52492	42	409	52083	25008
55	0.1	405	50589	41	389	50200	25191
56	0.1	390	48733	41	372	48361	25334
57	0.1	375	46919	40	356	46563	25438
58	0.1	361	45146	40	341	44805	25503
59	0.1	347	43412	40	327	43084	25529
60	0.1	334	41713	39	315	41398	25517
61	0.1	320	40048	39	303	39745	25465
62	0.1	307	38415	38	292	38123	25375
63	0.1	295	36813	38	282	36530	25246
64	0.1	282	35238	38	273	34965	25077
65	0.1	270	33690	37	264	33426	24868
66	0.1	257	32167	37	256	31911	24618
67	0.1	245	30668	37	249	30419	24325
68	0.1	234	29190	37	242	28949	23990
69	0.1	222	27734	36	235	27499	23609
70	0.1	210	26296	36	229	26067	23183



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

$P_{A,\,max}$ 25529 lbs/lineal foot

Equivalent Fluid Pressure (per lineal foot of shoring)

$EFP = 2*P_A/H^2$

EFP 44.2 pcf

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

Active

GEOCON
WEST, INC.

ENVIRONMENTAL GEOTECHNICAL MATERIALS

500 N. VICTORY BOULEVARD - BURBANK, CA 91502
PHONE: 818-841-8388 FAX: 818-841-1704

DRAFTED BY: JJK | CHECKED BY: HHD

SHORING WALL PRESSURE CALCULATION

800 & 908 NORTH MAIN STREET
1081 & 1087 NORTH VIGNES STREET
LOS ANGELES, CALIFORNIA

OCT. 2023 | PROJECT NO. W1814-06-01 | FIG. 16B

APPENDIX A

APPENDIX A

FIELD INVESTIGATION

The site was explored on September 6 through 8, 2023, by excavating four 7-inch diameter borings to between depths of approximately 50½ and 80½ feet below the existing ground surface using a truck-mounted hollow-stem auger drilling machine. Representative and relatively undisturbed samples were obtained by driving a 3-inch O. D., California Modified Sampler into the “undisturbed” soil mass with blows from a 140-pound auto-hammer falling 30 inches. The California Modified Sampler was equipped with 1-inch high by $2^3/_8$-inch diameter brass rings to facilitate soil removal and testing. Standard Penetration Tests were performed, and bulk samples were obtained.

The soil conditions encountered in the boring was 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 interpretations of the conditions between sampling intervals. Therefore, the logs contain both observed and interpreted data. We determined the lines designating the interface between soil materials on the log using visual observations, penetration rates, excavation characteristics and other factors. The transition between materials may be abrupt or gradual. Where applicable, the logs were revised based on subsequent laboratory testing. The approximate locations of the borings are shown on Figure 2.

PROJECT NO. W1814-06-01

BORING 1

ELEV. (MSL.) - - DATE COMPLETED 09/08/2023

EQUIPMENT HOLLOW STEM AUGER BY: JJK

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0					**AC: 4" BASE: 6"** **ARTIFICIAL FILL** Sandy Silt, medium dense, moist, black, fine-grained.			
2	B1@2 1/2'					21	104.7	9.6
4	B1@5'				- melted metal or glass	9		
6								
8	B1@7 1/2'				Silt with Sand, soft, moist to very moist, brown and reddish brown, some fine- to medium-grained.	8	100.2	21.1
10	B1@10' BULK 10-20'			SP	**ALLUVIUM** Sand, poorly graded, coarse gravel fragments.	29		
12								
14								
16	B1@15'			SP-SM	Sand with Silt, poorly graded, dense, moist, olive gray, fine-grained, some medium- to coarse-grained and fine gravel, trace silt.	31		
18	B1@17 1/2'				- very dense, trace coarse gravel	50 (6")	119.0	3.7
20	B1@20'				- brown, increase in coarse gravel	50 (3")		
22	B1@22 1/2'		▼	ML	Sandy Silt, very moist, gray to bluish gray, some fine-grained sand.	14	105.1	18.2
24	B1@25'			SM	Silty Sand, medium dense, very moist to wet, gray, fine-grained, some medium-grained.	18		
26					Sand, poorly graded, very dense, wet, gray, fine-grained.			
28	B1@27 1/2'			SP		50 (5")	115.3	13.3

W1814-06-01 BORING LOGS.GPJ

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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BORING 1

ELEV. (MSL.) - - DATE COMPLETED 09/08/2023

EQUIPMENT HOLLOW STEM AUGER BY: JJK

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'			SP	- medium dense, trace coarse-grained	28		
	B1@32 1/2'					71	132.5	10.1
				SM	Silty Sand, dense, wet, gray, fine-grained, some medium- to coarse-grained and fine gravel. (2" Sandy Silt lense)			
	B1@35'					50		
	B1@37 1/2'					50 (4")		
40	B1@40'				- fine to coarse gravel	44		
	B1@42 1/2'				- trace coarse-grained, increase in silt, no fine to coarse gravel	50 (4")	125.6	12.4
	B1@45'					50 (6")		
				ML	Silt with Sand, stiff, moist, gray.			
50	B1@50'					23		
	B1@52 1/2'				- hard	50 (6")	100.7	26.8
	B1@55'				- interbedded, brown, black, and gray	50 (5")		
				ML	Silt, hard, slightly moist to moist, dark brown and dark olive gray.			
	B1@57 1/2'					50 (4")	100.1	24.6

W1814-06-01 BORING LOGS.GPJ

Log of Boring 1, Page 2 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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GEOCON

BORING 1

ELEV. (MSL.) - - DATE COMPLETED 09/08/2023

EQUIPMENT HOLLOW STEM AUGER BY: JJK

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'					50 (5")		
62	B1@62 1/2			ML	- dark olive gray	50 (4")	102.3	26.1
64								
66	B1@65'					50 (3")		

Total depth of boring: 66 feet
Fill to 9 feet.
Groundwater encountered at 24 feet.
Backfilled with soil cuttings and tamped.
Patched with cold patch asphalt.

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

W1814-06-01 BORING LOGS.GPJ

Log of Boring 1, Page 3 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 2

ELEV. (MSL.) - - DATE COMPLETED 09/08/2023

EQUIPMENT HOLLOW STEM AUGER BY: JJK

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0					**AC: 4" BASE: 10"** **ARTIFICIAL FILL** Sandy Silt, soft, moist, dark brown, fine-grained.			
2	B2@2.5'			ML	**ALLUVIUM** Sandy Silt, soft, moist, olive brown, fine-grained.	8	89.4	26.5
4	B2@5'			SM	Silty Sand, loose, moist, olive brown and light reddish brown, fine-grained, some medium- to coarse-grained, trace fine gravel.	15	107.4	9.3
8				SP	Sand, poorly graded, medium dense, moist, light brown, fine-grained, some medium- to coarse-grained and fine gravel, trace coarse gravel.			
10	B2@10'					33	105.3	5.3
16	B2@15' BULK 15-20'				- dense, some medium-grained, increase in sand and fine to coarse gravel, trace cobbles	50 (4")	121.3	3.2
18	B2@17 1/2'				- light gray and light brown, decrease in coarse-grained, fine to coarse gravel	65	102.0	7.7
20	B2@20'				- very dense, light brown, medium- to coarse-grained, some fine-grained and fine gravel	50 (5")	122.0	8.3
24			▼					
26	B2@25'				- very moist to wet, gray and light reddish brown, fine-grained, no fine- to coarse-grained or gravel	50 (5")	121.0	12.0
28				SM	Silty Sand, very dense, wet, gray, fine-grained, some coarse-grained and fine gravel.			

W1814-06-01 BORING LOGS.GPJ

Log of Boring 2, Page 1 of 2

SAMPLE SYMBOLS

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

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

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BORING 2

ELEV. (MSL.) - - DATE COMPLETED 09/08/2023

EQUIPMENT HOLLOW STEM AUGER BY: JJK

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'					50 (5")	142.7	2.9
32	B2@32 1/2				- dense, no coarse-grained or fine gravel, oil, hydrocarbon	60	116.9	19.1
34								
	B2@35'				- very dense, very moist	50 (5")	117.4	16.9
36								
38				SM				
40	B2@40'				- wet	50 (2")	114.8	20.7
42								
44								
	B2@45'				- no recovery	50 (3")		
46								
48								
50	B2@50'				- no recovery	50 (4")		

Total depth of boring: 50 1/2 feet
Fill to 2 feet.
Groundwater encountered at 24 feet.
Backfilled with soil cuttings and tamped.
Patched with cold patch asphalt.

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

W1814-06-01 BORING LOGS.GPJ

Log of Boring 2, Page 2 of 2

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.

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PROJECT NO. W1814-06-01

BORING 3

ELEV. (MSL.) **- -** DATE COMPLETED **09/08/2023**

EQUIPMENT **HOLLOW STEM AUGER** BY: **JJK**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0					**ARTIFICIAL FILL** Silty Sand, medium dense, moist, olive brown, fine-grained, trace fine gravel.			
5	B3@5'			SP	**ALLUVIUM** Sand, poorly graded, medium dense, slightly moist, light brown, fine-grained, some medium-grained.	21	273.7	10.0
8.5					Sand with Silt, poorly graded, medium dense, moist, olive brown and reddish brown, fine-grained, some medium- to coarse-grained, trace fine gravel.			
10	B3@10'			SP-SM		45	98.2	1.7
12					Sand, poorly graded, dense, moist, fine-grained, some medium- to coarse-grained and fine to coarse gravel.			
15	B3@15'			SP		50 (5")	130.4	2.4
17.5					- abundant fine to coarse gravel			
20	B3@20'				- medium dense, very moist, light reddish brown and olive gray, fine-grained, no medium- to coarse-grained or fine to coarse gravel	49	119.2	23.1
21					Sand with Silt, dense, wet, gray, some medium- to coarse-grained.			
24			▼					
25	B3@25'			SP-SM		62	114.8	15.6

W1814-06-01 BORING LOGS.GPJ

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

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

GEOCON

BORING 3

ELEV. (MSL.) - - DATE COMPLETED 09/08/2023

EQUIPMENT HOLLOW STEM AUGER BY: JJK

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
30	B3@30'			SP-SM	- increase in coarse-grained	59	123.0	16.0
32								
34				SM	Silty Sand, very dense, wet, gray, some coarse-grained and fine to coarse gravel, sulfur odor.			
	B3@35'					50 (4")	134.9	9.7
36								
38				ML	Silt, hard, moist, bluish gray.			
40	B3@40'					52	251.9	22.9
42								
44								
	B3@45'				- slightly moist to moist	50 (4")	95.4	30.7
46								
48								
50	B3@50'				- dark olive gray and dark brown	50 (5")	102.5	24.2
52								
54				ML	Silt with Sand, hard, moist, dark brown and gray.			
	B3@55'					50 (5")	96.6	28.2

Total depth of boring: 55 1/2 feet
Fill to 4 1/2 feet.
Groundwater encountered at 24 feet.
Backfilled with soil cuttings and tamped.

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

W1814-06-01 BORING LOGS.GPJ

Log of Boring 3, 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.

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	BORING 3 ELEV. (MSL.) - - DATE COMPLETED 09/08/2023 EQUIPMENT HOLLOW STEM AUGER BY: JJK	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
					MATERIAL DESCRIPTION			
					NOTE: The stratification lines presented herein represent the approximate boundary between earth types; the transitions may be gradual.			

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

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PROJECT NO. W1814-06-01

BORING 4

ELEV. (MSL.) - - DATE COMPLETED 09/07/2023

EQUIPMENT HOLLOW STEM AUGER BY: JJK

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: 5" BASE: 6"** **ARTIFICIAL FILL** Silty Sand, loose, moist, olive gray, fine-grained.			
2	B4@2 1/2'			ML	**ALLUVIUM** Sandy Silt, soft, moist, olive brown, fine-grained.	8	94.8	17.1
4	B4@5'			SP-SM	Sand with Silt, poorly graded, moist, brown, fine-grained.	3		
6				SP	Sand, poorly graded, medium dense, slightly moist to moist, light brown, fine-grained, some medium-grained, trace coarse-grained.			
8	B4@7 1/2'					48	108.6	4.1
10	B4@10'				- dense, slightly moist, some medium- to coarse-grained and fine gravel, trace coarse gravel	32		
12	B4@12 1/2'				- no coarse-grained	36	117.1	3.1
14	B4@15'				- slightly moist to moist, brown	32		
16				SP-SM	Sand with Silt, poorly graded, dense, slightly moist to moist, light olive brown, fine-grained, trace medium - to coarse-grained.			
18	B4@17 1/2'					50 (5")	106.9	8.7
20	B4@20'				- moist, light olive brown and light reddish brown, fine-grained, medium- to coarse-grained	59		
22				SP	Sand, poorly graded, very dense, very moist to wet, light grayish brown and reddish brown, some medium- to coarse-grained, trace fine gravel.			
	B4@22 1/2'		▼			50 (3)	127.1	9.4
24	B4@25'				- very moist to wet, light brown, reddish brown, and gray, some fine-grained, trace coarse gravel.	70		
26				SP	Sand, poorly graded, very dense, wet, gray, fine-grained, medium- to coarse-grained.			
28	B4@27 1/2'					50 (5")	148.7	6.5

W1814-06-01 BORING LOGS.GPJ

Log of Boring 4, 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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GEOCON

BORING 4

ELEV. (MSL.) - - DATE COMPLETED 09/07/2023

EQUIPMENT HOLLOW STEM AUGER BY: JJK

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
30	B4@30' BULK 30-35'			SP	- medium dense, some medium- to coarse-grained, some fine gravel, trace coarse gravel	27		
32	B4@32 1/2'				- dense, increase in fine to coarse gravel	69	110.7	12.7
34								
36	B4@35'				- medium dense, wet, gray, no coarse gravel	23		
38	B4@37 1/2'				- dense	50 (4")	114.2	13.0
40	B4@40'			SP-SM	Sand with Silt, poorly graded medium dense, wet, gray, fine-grained, some medium-grained and fine gravel.	22		
42	B4@42 1/2'					50 (5")	117.0	15.0
44								
46	B4@45'				- dense, gray to dark gray, fine-grained, trace medium-grained and fine gravel, sulfur odor	30		
48	B4@47 1/2'			SP	Sand, poorly graded, medium dense, wet, gray, medium-grained, some fine- to coarse-grained and coarse gravel, sulfur odor.	50 (5")	142.6	9.5
50	B4@50'			SM	Silty Sand, very dense, wet, gray and light gray, fine-grained, medium- to coarse-grained, trace fine to coarse gravel.	58		
52	B4@52 1/2'				- no coarse-grained, fine to coarse gravel	50 (5")	129.4	13.4
54								
56	B4@55'				- fine-grained with coarse-grained, some medium-grained and fine gravel	79		
58	B4@57 1/2'			ML	Silt with Sand, hard, slightly moist, dark grayish brown and light brown.	41		

W1814-06-01 BORING LOGS.GPJ

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

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BORING 4

ELEV. (MSL.) - - DATE COMPLETED 09/07/2023

EQUIPMENT HOLLOW STEM AUGER BY: JJK

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'					76		
62								
64				ML				
	B4@65'					76		
66								
68								
					Sandy Silt, hard, moist, dark brown and gray, fine-grained.			
70	B4@70'					43		
72				ML				
74								
	B4@75'				- slightly moist to moist	50 (6")		
76								
78					Silt with Sand, hard, slightly moist to moist, dark brown and gray.			
				ML				
80	B4@80'					38		

Total depth of boring: 81 feet
Fill to 2 1/2 feet.
Groundwater encountered at 23 feet.
Backfilled with soil cuttings and tamped.
Patched with cold patch asphalt.

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

W1814-06-01 BORING LOGS.GPJ

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

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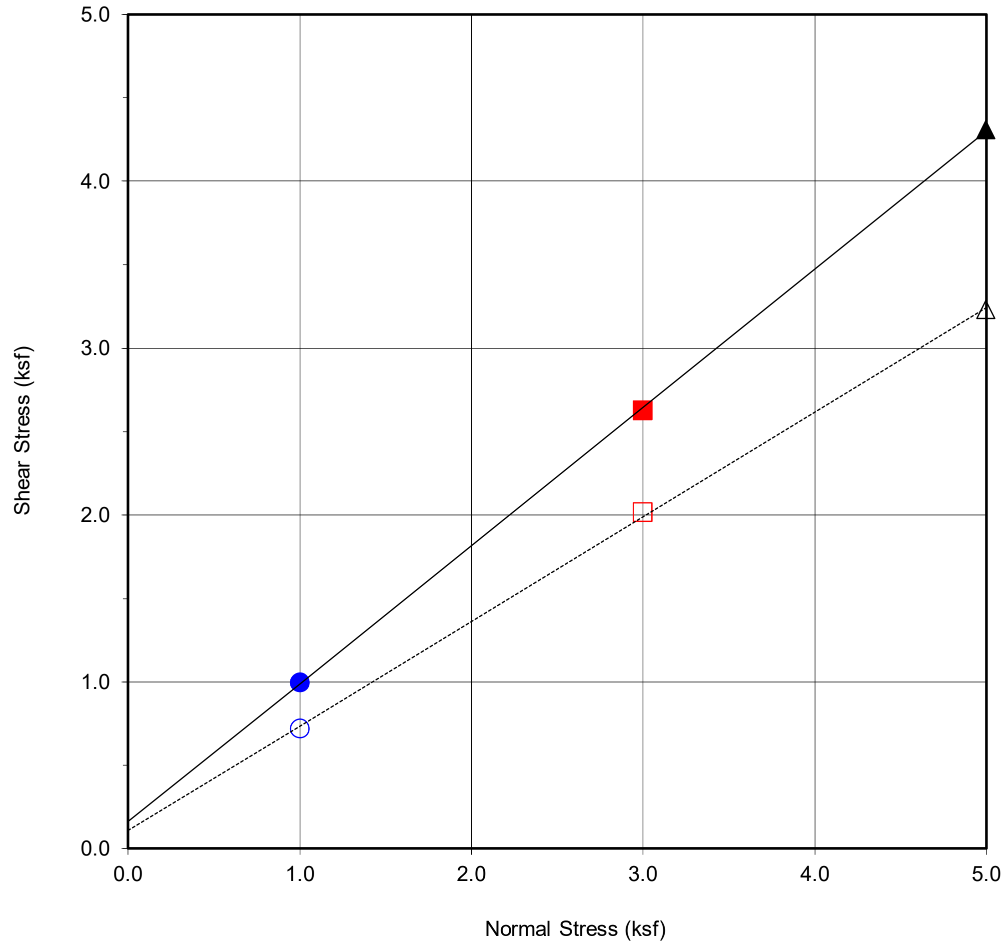
APPENDIX B

APPENDIX B

LABORATORY TESTING

Laboratory tests were performed in accordance with generally accepted test methods of the "American Society for Testing and Materials (ASTM)", or other suggested procedures. Selected samples were tested for direct shear strength, consolidation and expansion characteristics, plasticity indices, grain size analysis, optimum moisture and maximum dry density relationships, corrosivity and in-place dry density and moisture content. The results of the laboratory tests are summarized in Figures B1 through B28. The in-place dry density and moisture content of the samples tested are presented on the boring logs, Appendix A.



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

Soil Identification:		
Sand (SP)		
Strength Parameters		
	C (psf)	ϕ (^{o})
Peak	160	40
Ultimate	107	32

Normal Stress (kip/ft²)	1	3	5
Peak Shear Stress (kip/ft²)	● 1.00	■ 2.63	▲ 4.31
Shear Stress @ End of Test (ksf)	○ 0.72	□ 2.02	△ 3.23
Deformation Rate (in./min.)	0.05	0.05	0.05
Initial Sample Height (in.)	1.0	1.0	1.0
Ring Inside Diameter (in.)	2.375	2.375	2.375
Initial Moisture Content (%)	5.3	5.5	6.2
Initial Dry Density (pcf)	101.8	102.2	103.8
Initial Degree of Saturation (%)	21.9	23.0	26.7
Soil Height Before Shearing (in.)	1.2	1.2	1.2
Final Moisture Content (%)	20.8	20.1	20.2

GEOCON

DIRECT SHEAR TEST RESULTS

Consolidated Drained ASTM D-3080

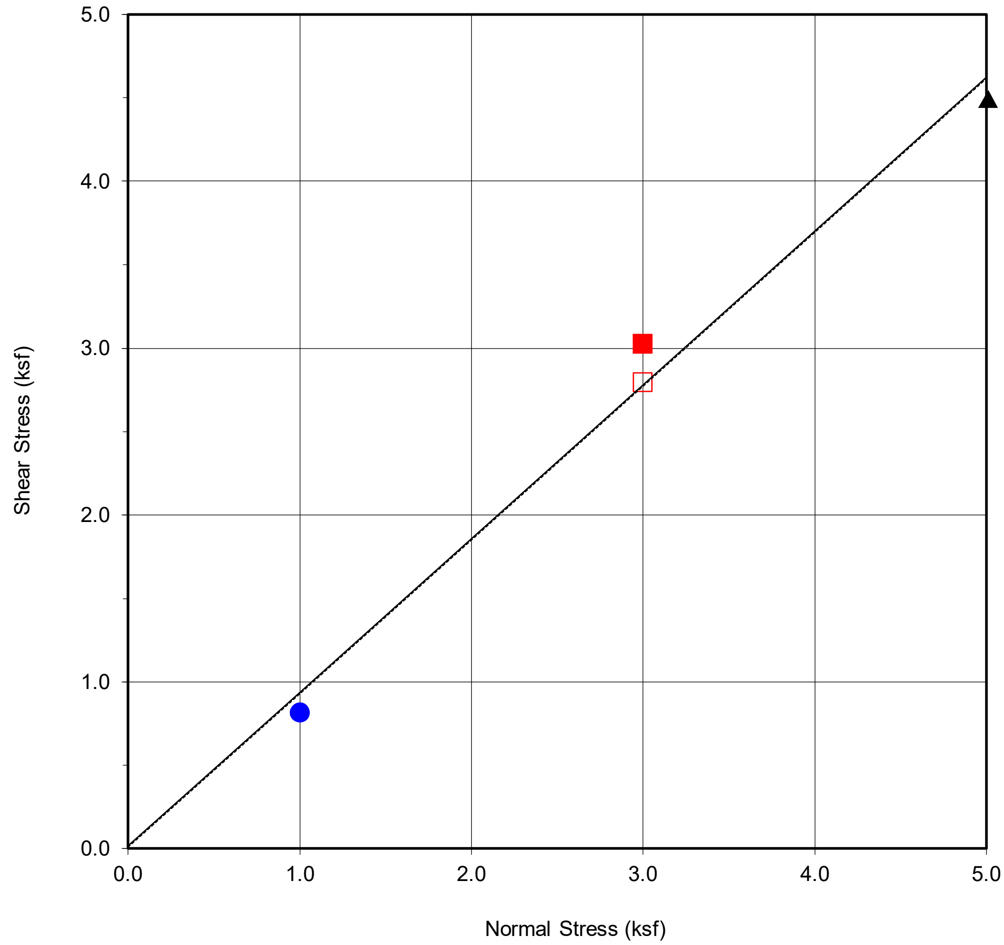
Checked by: JJK

Project No.: W1814-06-01

800 & 908 NORTH MAIN STREET
1081 & 1087 NORTH VIGNES STREET
LOS ANGELES, CALIFORNIA

OCT. 2023

Figure B1



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

Soil Identification:		
Sand (SP)		
Strength Parameters		
	C (psf)	ϕ (°)
Peak	17	43
Ultimate	9	43

Normal Stress (kip/ft²)	1	3	5
Peak Shear Stress (kip/ft²)	● 0.82	■ 3.02	▲ 4.50
Shear Stress @ End of Test (ksf)	○ 0.82	□ 2.80	△ 4.50
Deformation Rate (in./min.)	0.05	0.05	0.05
Initial Sample Height (in.)	1.0	1.0	1.0
Ring Inside Diameter (in.)	2.375	2.375	2.375
Initial Moisture Content (%)	7.8	2.5	8.9
Initial Dry Density (pcf)	101.3	108.0	105.2
Initial Degree of Saturation (%)	31.7	12.2	40.1
Soil Height Before Shearing (in.)	1.2	1.2	1.2
Final Moisture Content (%)	18.2	12.5	15.3

GEOCON

DIRECT SHEAR TEST RESULTS

Consolidated Drained ASTM D-3080

Checked by: JJK

Project No.: W1814-06-01

800 & 908 NORTH MAIN STREET
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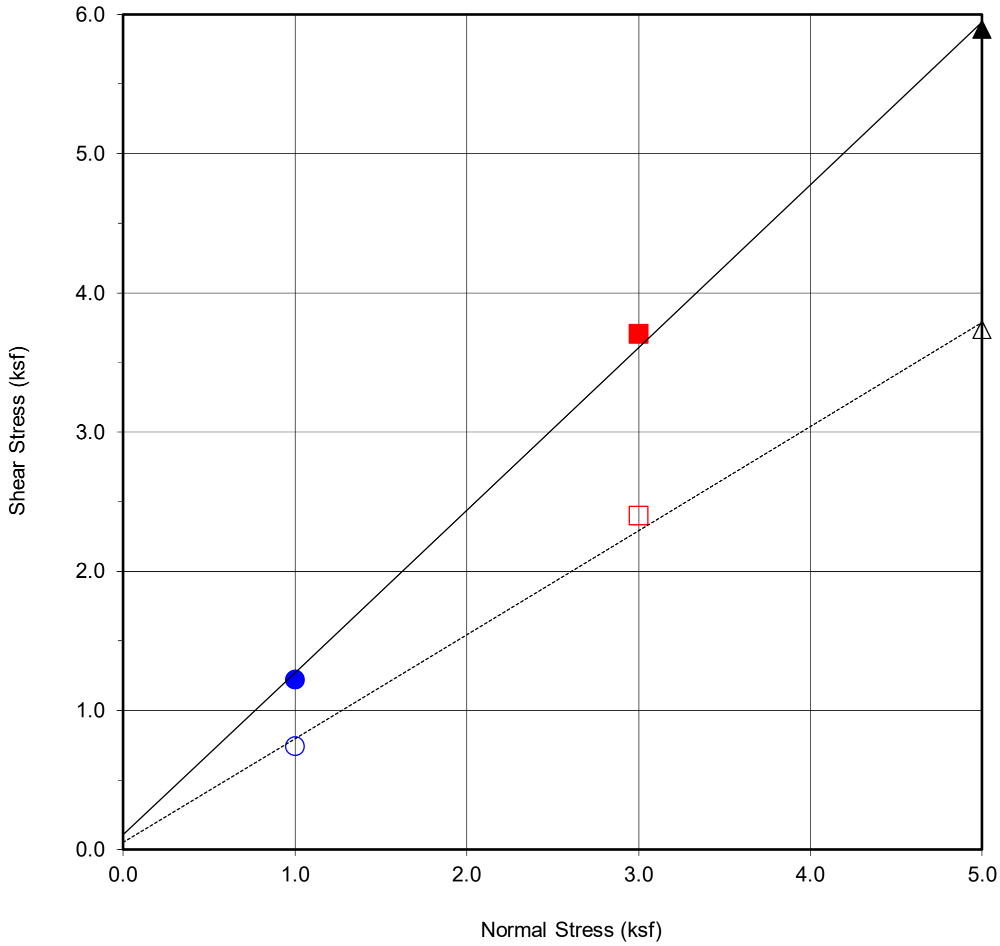
LOS ANGELES, CALIFORNIA

OCT. 2023

Figure B2



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

Soil Identification:		
Sand w/ Silt (SP-SM)		
Strength Parameters		
	C (psf)	ϕ (^{o})
Peak	107	49
Ultimate	51	37

Normal Stress (kip/ft²)	1	3	5
Peak Shear Stress (kip/ft²)	● 1.22	■ 3.71	▲ 5.89
Shear Stress @ End of Test (ksf)	○ 0.74	□ 2.40	△ 3.73
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 (%)	18.4	15.0	15.8
Initial Dry Density (pcf)	111.5	119.8	118.9
Initial Degree of Saturation (%)	96.8	99.3	102.5
Soil Height Before Shearing (in.)	1.2	1.2	1.2
Final Moisture Content (%)	20.2	15.0	16.4

GEOCON

DIRECT SHEAR TEST RESULTS

Consolidated Drained ASTM D-3080

Checked by: JJK

Project No.: W1814-06-01

800 & 908 NORTH MAIN STREET
1081 & 1087 NORTH VIGNES STREET
LOS ANGELES, CALIFORNIA

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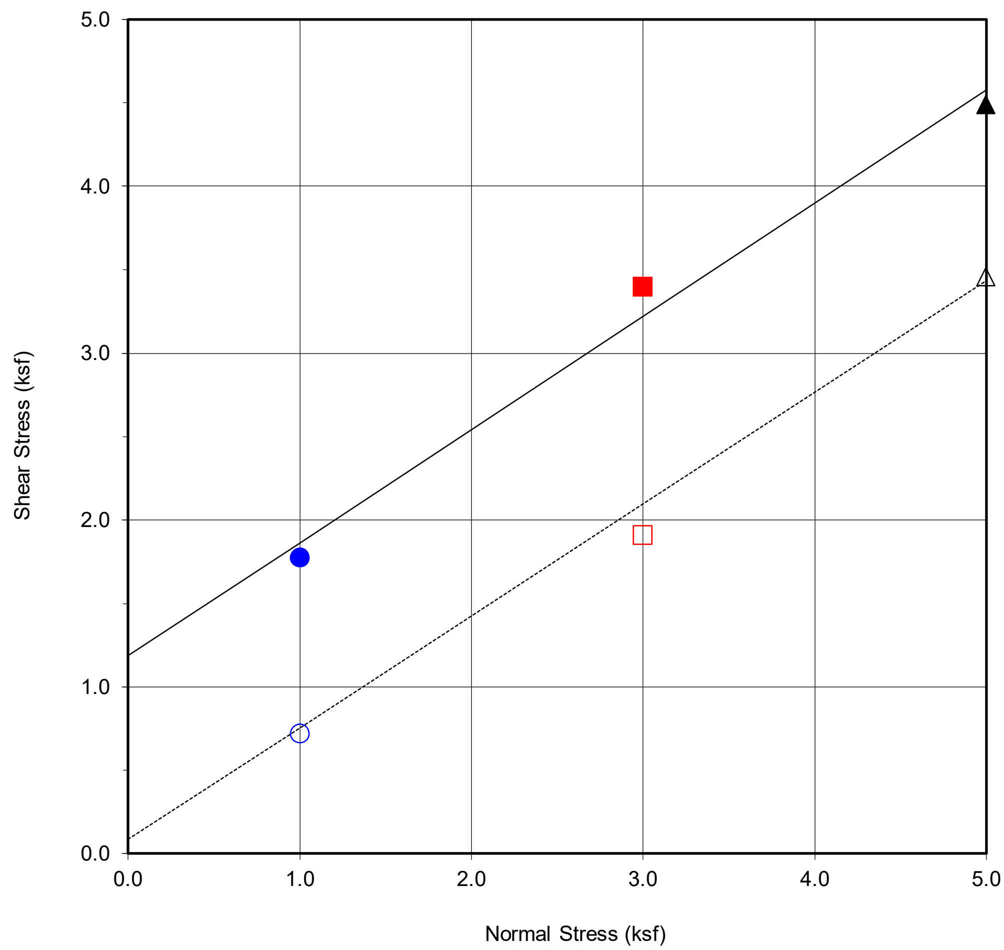
Figure B3



Boring No.	B3
Sample No.	B3@55
Depth (ft)	55
Sample Type:	Ring

Soil Identification:		
Silt w/ Sand (ML)		
Strength Parameters		
	C (psf)	ϕ (°)
Peak	1186	34
Ultimate	86	34

Normal Stress (kip/ft²)	1	3	5
Peak Shear Stress (kip/ft²)	● 1.78	■ 3.40	▲ 4.49
Shear Stress @ End of Test (ksf)	○ 0.72	□ 1.91	△ 3.46
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 (%)	28.2	28.6	26.0
Initial Dry Density (pcf)	94.1	94.3	99.3
Initial Degree of Saturation (%)	96.2	98.0	100.6
Soil Height Before Shearing (in.)	1.2	1.2	1.2
Final Moisture Content (%)	33.3	32.4	28.7

GEOCON

DIRECT SHEAR TEST RESULTS

Consolidated Drained ASTM D-3080

Checked by: JJK

Project No.: W1814-06-01

800 & 908 NORTH MAIN STREET
1081 & 1087 NORTH VIGNES STREET
LOS ANGELES, CALIFORNIA

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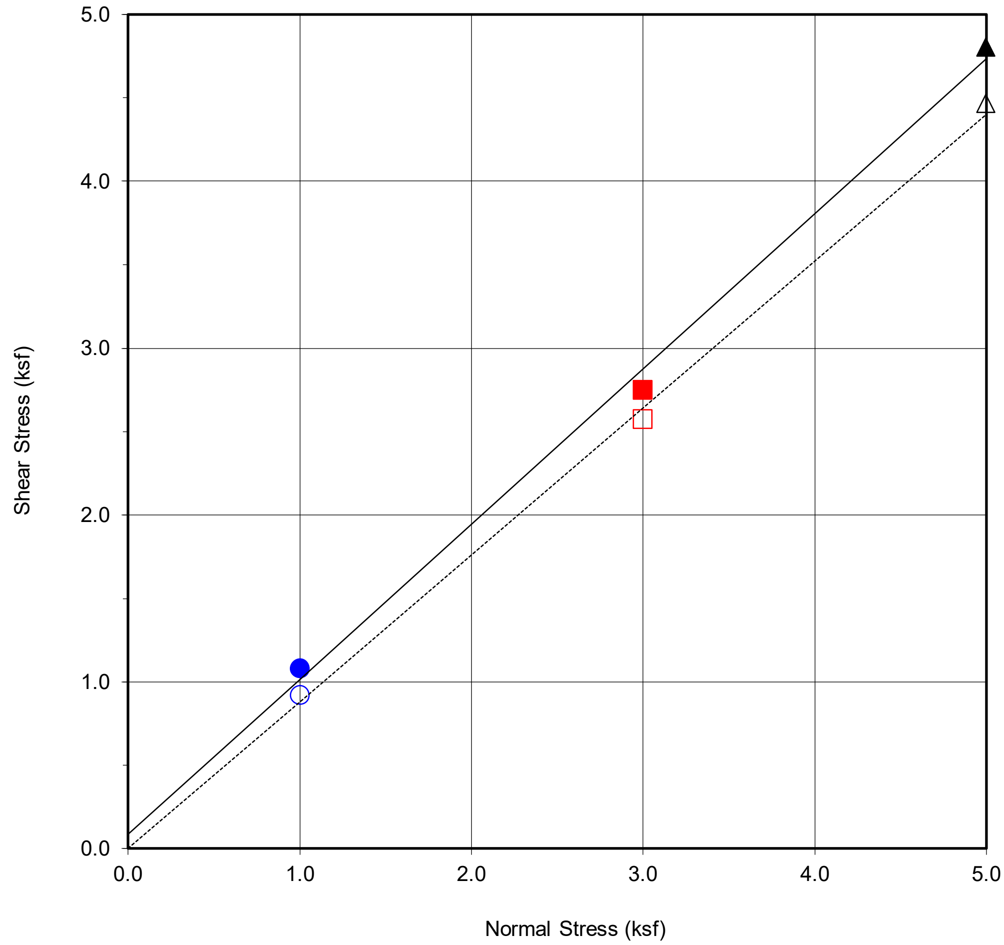
Figure B4



Boring No.	**B4**
Sample No.	**B4@7.5**
Depth (ft)	**7.5**
Sample Type:	Ring

Soil Identification:		
Sand (SP)		
Strength Parameters		
	C (psf)	ϕ (^{o})
Peak	86	43
Ultimate	0	41

Normal Stress (kip/ft²)	1	3	5
Peak Shear Stress (kip/ft²)	1.08	2.75	4.80
Shear Stress @ End of Test (ksf)	0.90	2.59	4.42
Deformation Rate (in./min.)	0.05	0.05	0.05
Initial Sample Height (in.)	1.0	1.0	1.0
Ring Inside Diameter (in.)	2.375	2.375	2.375
Initial Moisture Content (%)	4.1	0.4	12.2
Initial Dry Density (pcf)	104.2	110.9	104.4
Initial Degree of Saturation (%)	17.8	1.9	53.6
Soil Height Before Shearing (in.)	1.2	1.2	1.2
Final Moisture Content (%)	19.3	16.3	17.7

GEOCON

DIRECT SHEAR TEST RESULTS

Consolidated Drained ASTM D-3080

Checked by: JJK

Project No.: W1814-06-01

800 & 908 NORTH MAIN STREET
1081 & 1087 NORTH VIGNES STREET
LOS ANGELES, CALIFORNIA

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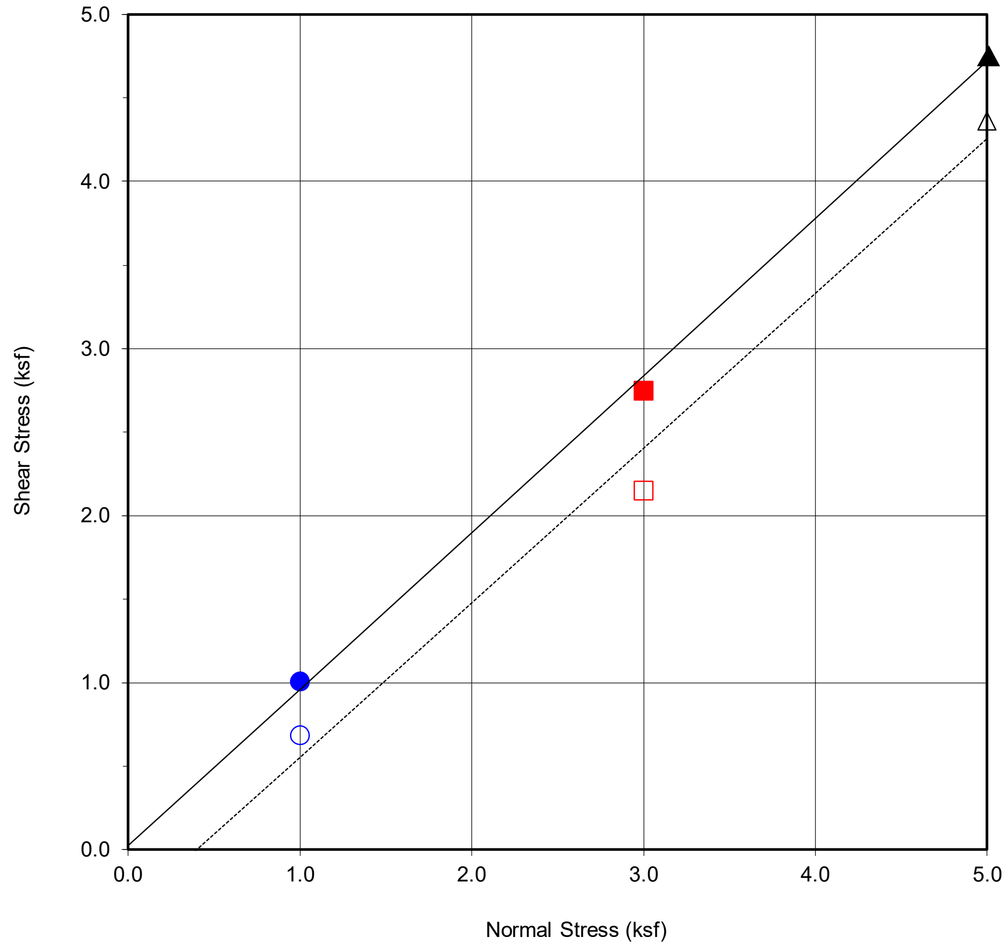
Figure B5



Boring No.	**B4**
Sample No.	**B4@17.5**
Depth (ft)	**17.5**
Sample Type:	Ring

Soil Identification:		
Sand w/ Silt (SP-SM)		
Strength Parameters		
	C (psf)	ϕ (°)
Peak	23	43
Ultimate	0	43

Normal Stress (kip/ft²)	1	3	5
Peak Shear Stress (kip/ft²)	● 1.01	■ 2.75	▲ 4.76
Shear Stress @ End of Test (ksf)	○ 0.68	□ 2.15	△ 4.38
Deformation Rate (in./min.)	0.05	0.05	0.05
Initial Sample Height (in.)	1.0	1.0	1.0
Ring Inside Diameter (in.)	2.375	2.375	2.375
Initial Moisture Content (%)	8.7	7.2	8.5
Initial Dry Density (pcf)	105.3	108.1	107.8
Initial Degree of Saturation (%)	39.2	34.7	40.9
Soil Height Before Shearing (in.)	1.2	1.2	1.2
Final Moisture Content (%)	19.0	17.3	16.7

GEOCON

DIRECT SHEAR TEST RESULTS

Consolidated Drained ASTM D-3080

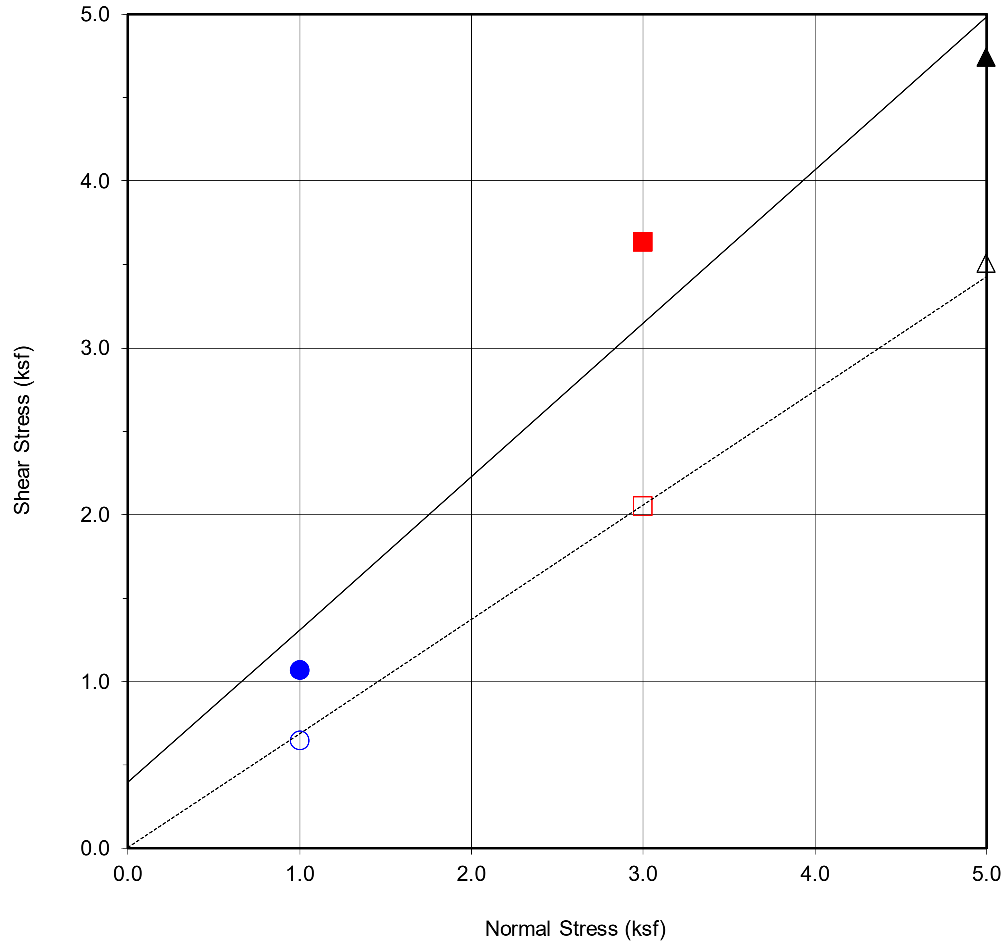
Checked by: JJK

Project No.: W1814-06-01

800 & 908 NORTH MAIN STREET
1081 & 1087 NORTH VIGNES STREET
LOS ANGELES, CALIFORNIA

OCT. 2023

Figure B6



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

Soil Identification:		
Sand (SP)		
Strength Parameters		
	C (psf)	ϕ (^{o})
Peak	394	43
Ultimate	6	34

Normal Stress (kip/ft²)	1	3	5
Peak Shear Stress (kip/ft²)	● 1.07	■ 3.64	▲ 4.74
Shear Stress @ End of Test (ksf)	○ 0.65	□ 2.05	△ 3.50
Deformation Rate (in./min.)	0.05	0.05	0.05
Initial Sample Height (in.)	1.0	1.0	1.0
Ring Inside Diameter (in.)	2.375	2.375	2.375
Initial Moisture Content (%)	12.7	21.1	25.0
Initial Dry Density (pcf)	108.4	102.2	100.4
Initial Degree of Saturation (%)	61.8	88.0	99.5
Soil Height Before Shearing (in.)	1.2	1.2	1.2
Final Moisture Content (%)	21.4	21.5	23.8

GEOCON

DIRECT SHEAR TEST RESULTS

Consolidated Drained ASTM D-3080

Checked by: JJK

Project No.: W1814-06-01

800 & 908 NORTH MAIN STREET
1081 & 1087 NORTH VIGNES STREET
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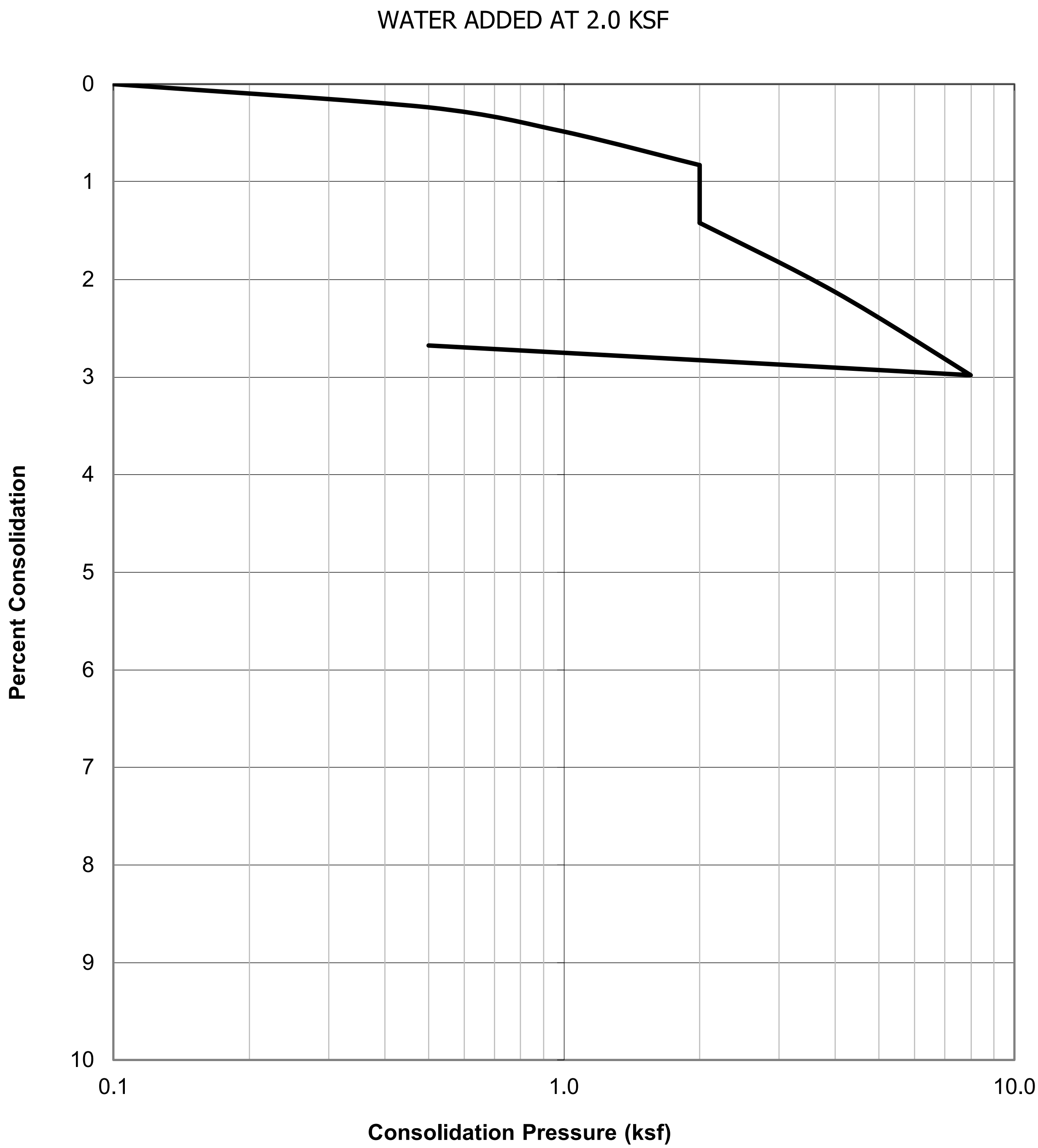
Figure B7



SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B1@17.5	Sand w/ Silt (SP-SM)	107.5	3.6	15.7

GEOCON

CONSOLIDATION TEST RESULTS

ASTM D-2435

Checked by: JJK

Project No.: W1814-06-01

800 & 908 NORTH MAIN STREET
1081 & 1087 NORTH VIGNES STREET
LOS ANGELES, CALIFORNIA

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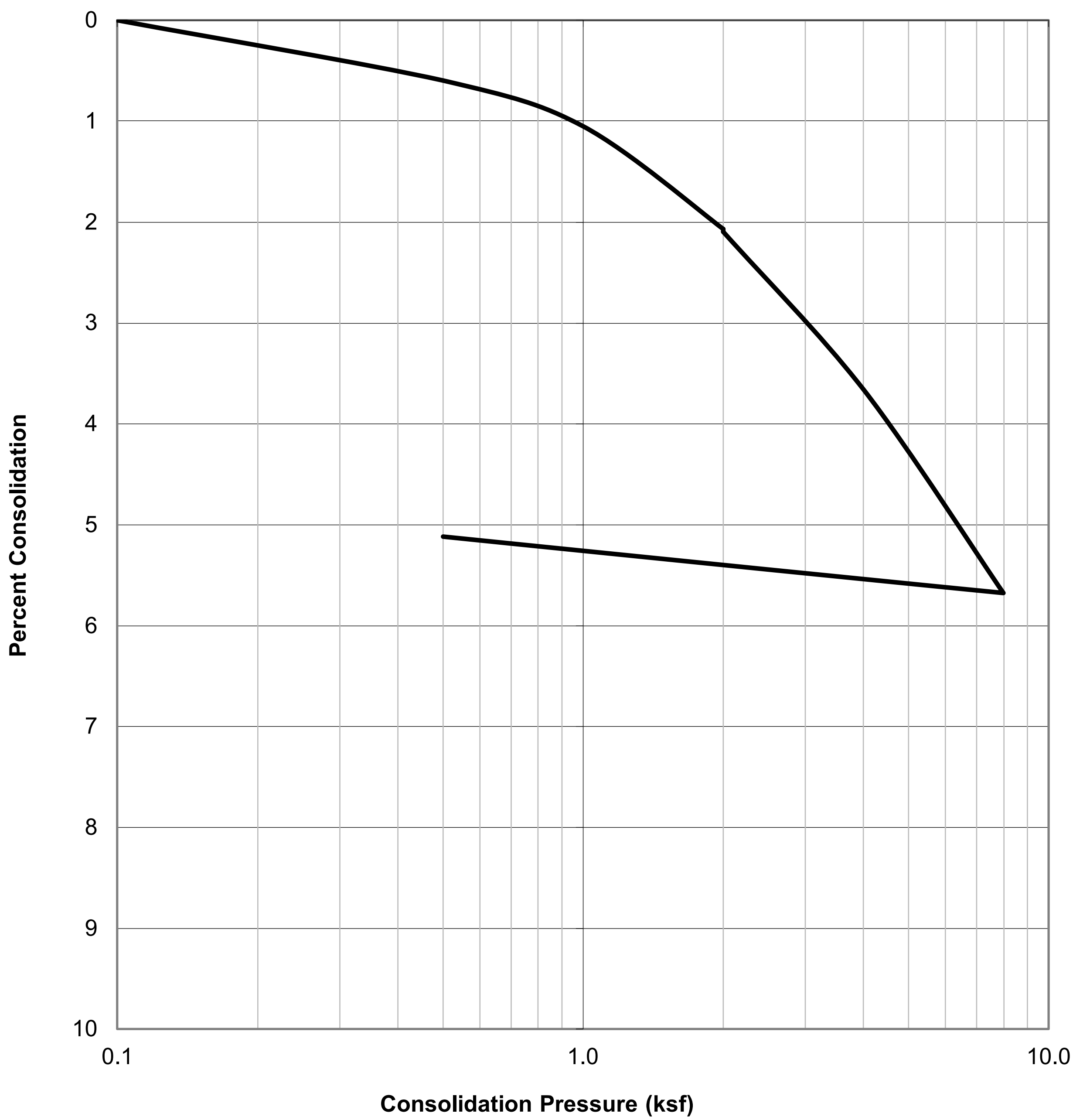
Figure B8

WATER ADDED AT 2.0 KSF



SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B1@22.5	Sandy Silt (ML)	109.8	18.1	15.9

GEOCON

CONSOLIDATION TEST RESULTS

ASTM D-2435

Checked by: JJK

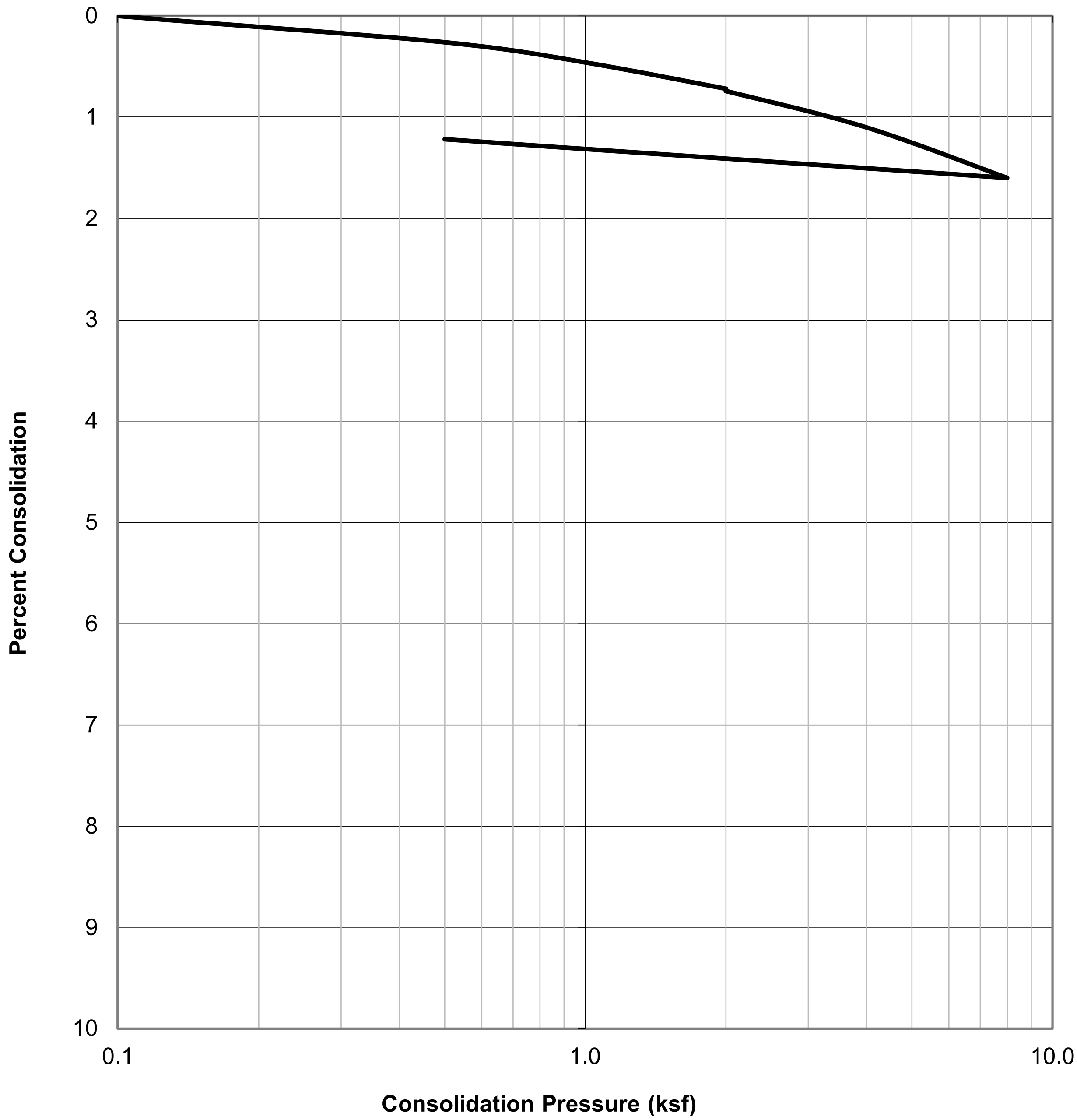
Project No.: W1814-06-01

800 & 908 NORTH MAIN STREET
1081 & 1087 NORTH VIGNES STREET
LOS ANGELES, CALIFORNIA

OCT. 2023

Figure B9

WATER ADDED AT 2.0 KSF



SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B1@27.5	Sand (SP)	111.3	13.4	15.4

GEOCON

CONSOLIDATION TEST RESULTS

ASTM D-2435

Checked by: JJK

Project No.: W1814-06-01

800 & 908 NORTH MAIN STREET
1081 & 1087 NORTH VIGNES STREET
LOS ANGELES, CALIFORNIA

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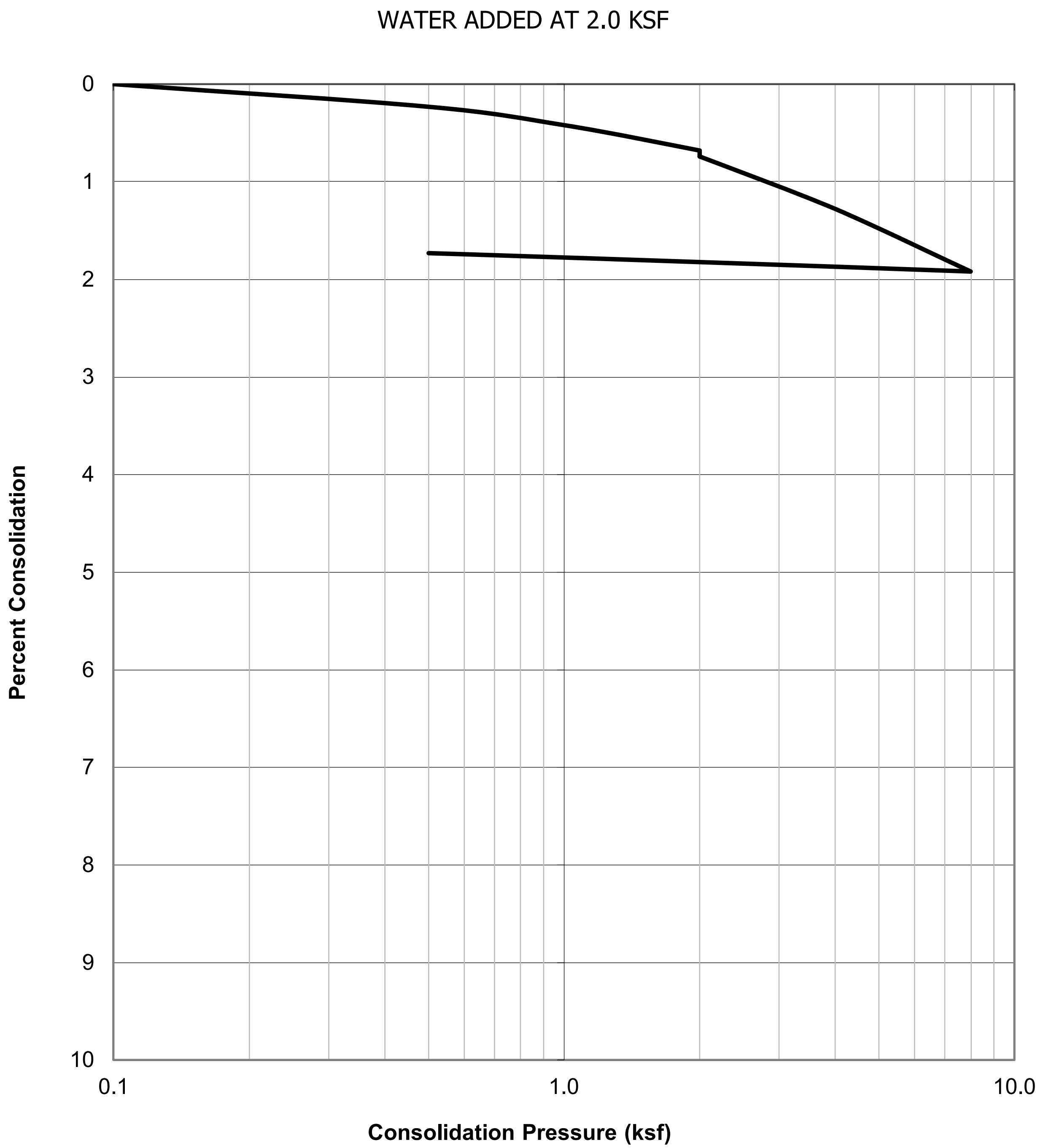

SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B1@32.5	Sand (SP)	124.9	10.0	11.8

GEOCON

CONSOLIDATION TEST RESULTS

ASTM D-2435

Checked by: JJK

Project No.: W1814-06-01

800 & 908 NORTH MAIN STREET
1081 & 1087 NORTH VIGNES STREET
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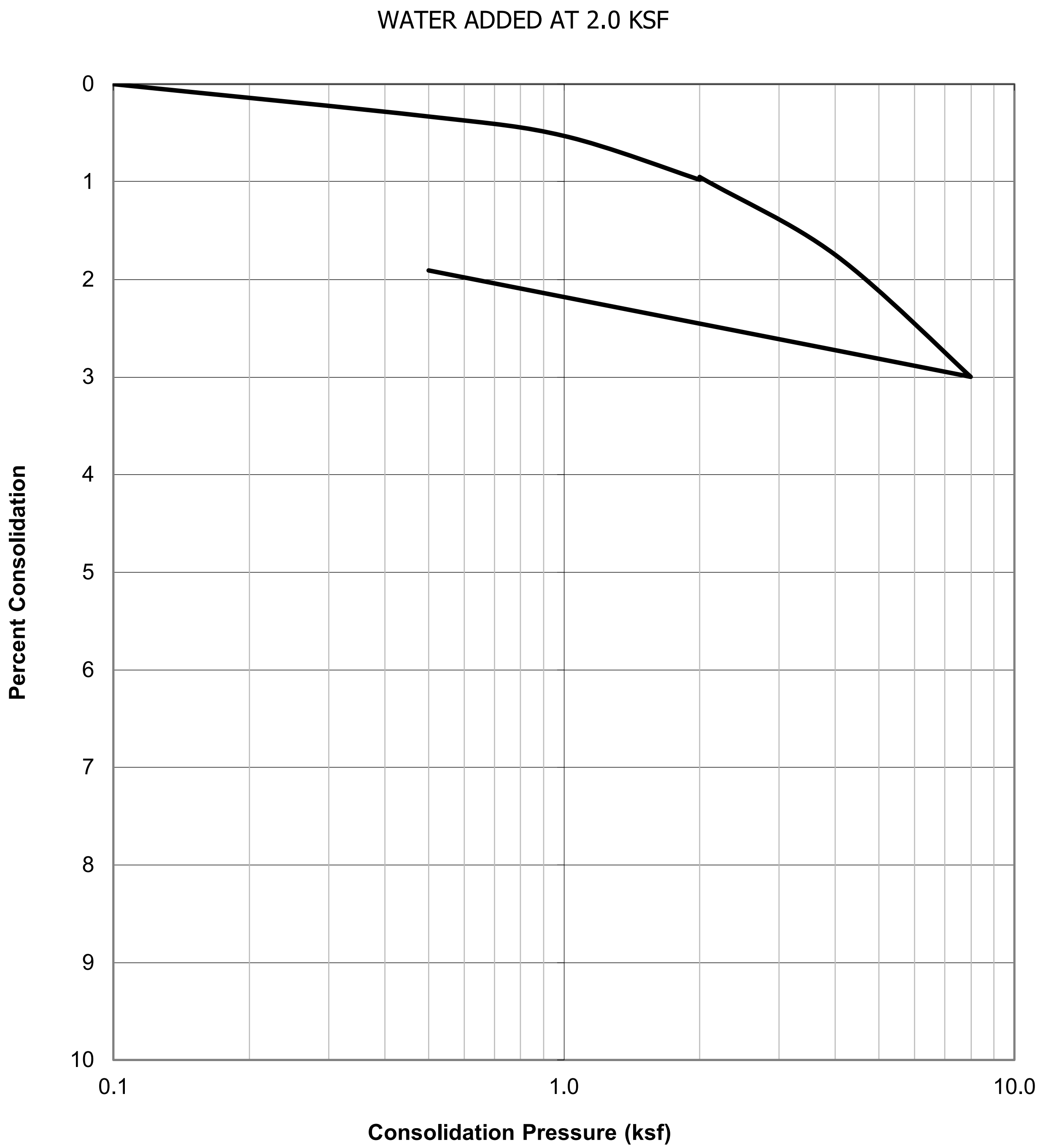
Figure B11



SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B1@52.5	Silt w/ Sand (ML)	97.4	26.8	26.9

GEOCON

CONSOLIDATION TEST RESULTS

ASTM D-2435

Checked by: JJK

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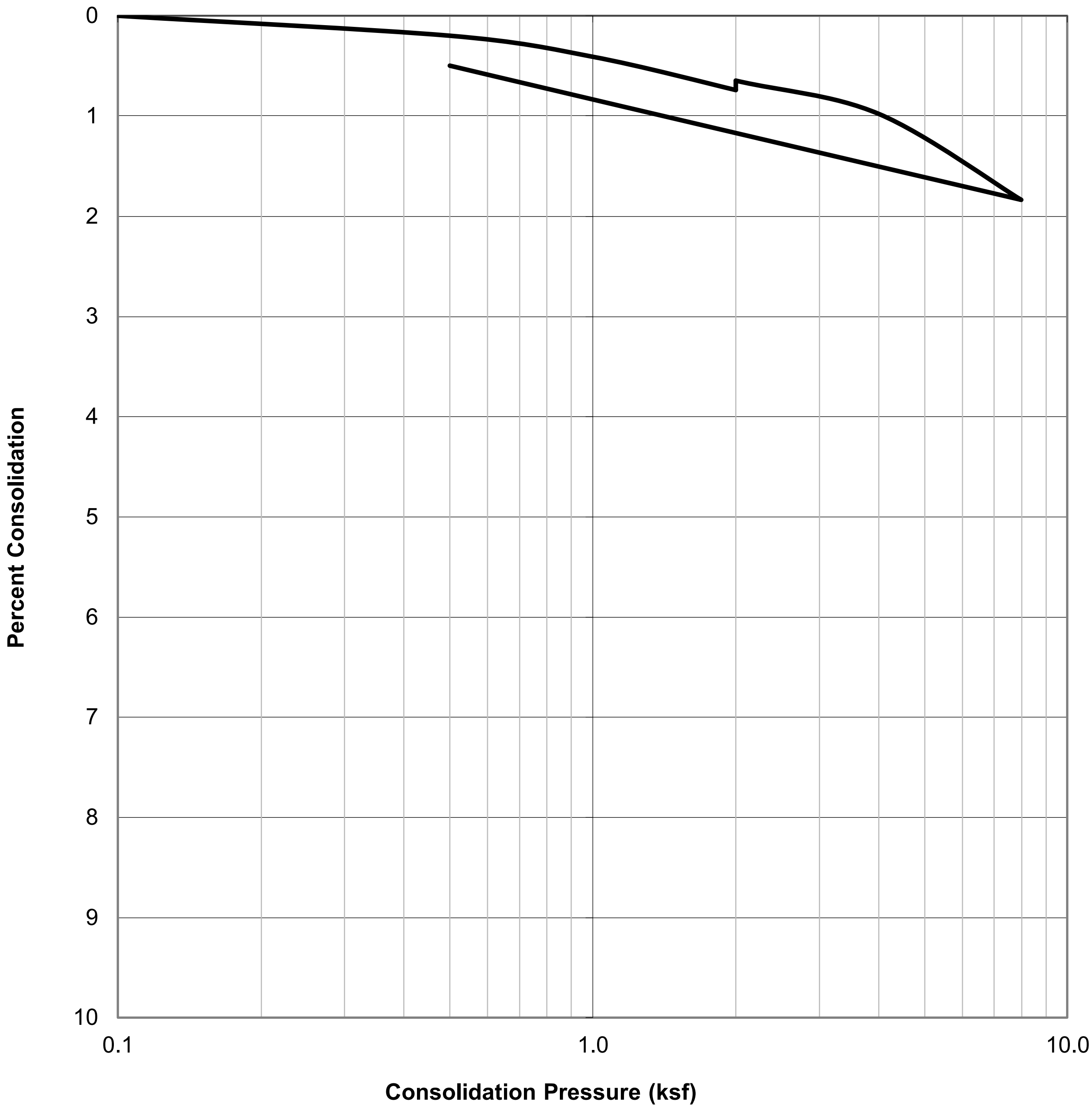
1081 & 1087 NORTH VIGNES STREET
LOS ANGELES, CALIFORNIA

OCT. 2023

Figure B12

WATER ADDED AT 2.0 KSF



SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B1@57.5'	Silt (ML)	97.4	24.7	27.2

GEOCON

CONSOLIDATION TEST RESULTS

ASTM D-2435

Checked by: JJK

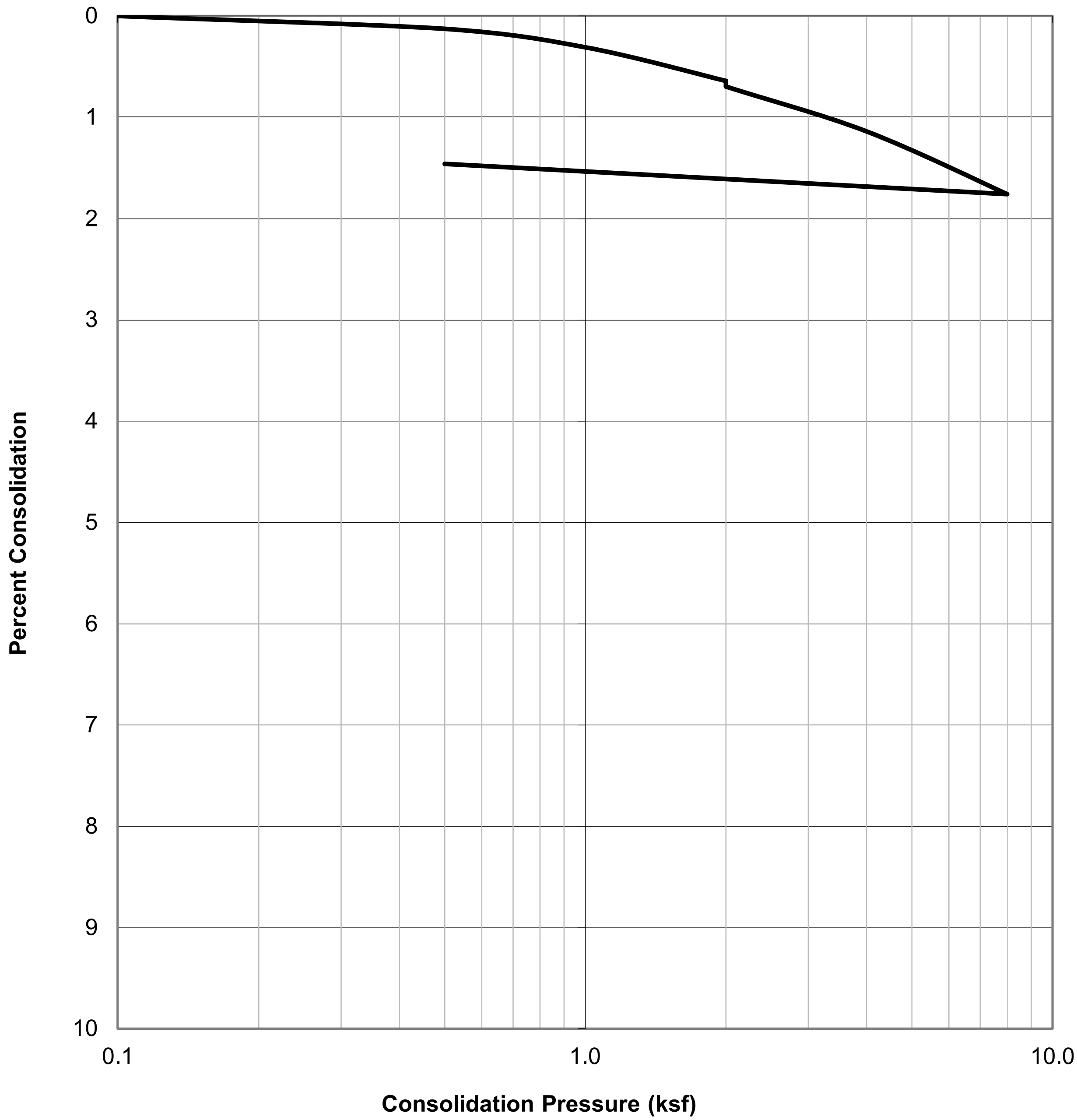
Project No.: W1814-06-01

800 & 908 NORTH MAIN STREET
1081 & 1087 NORTH VIGNES STREET
LOS ANGELES, CALIFORNIA

OCT. 2023

Figure B13

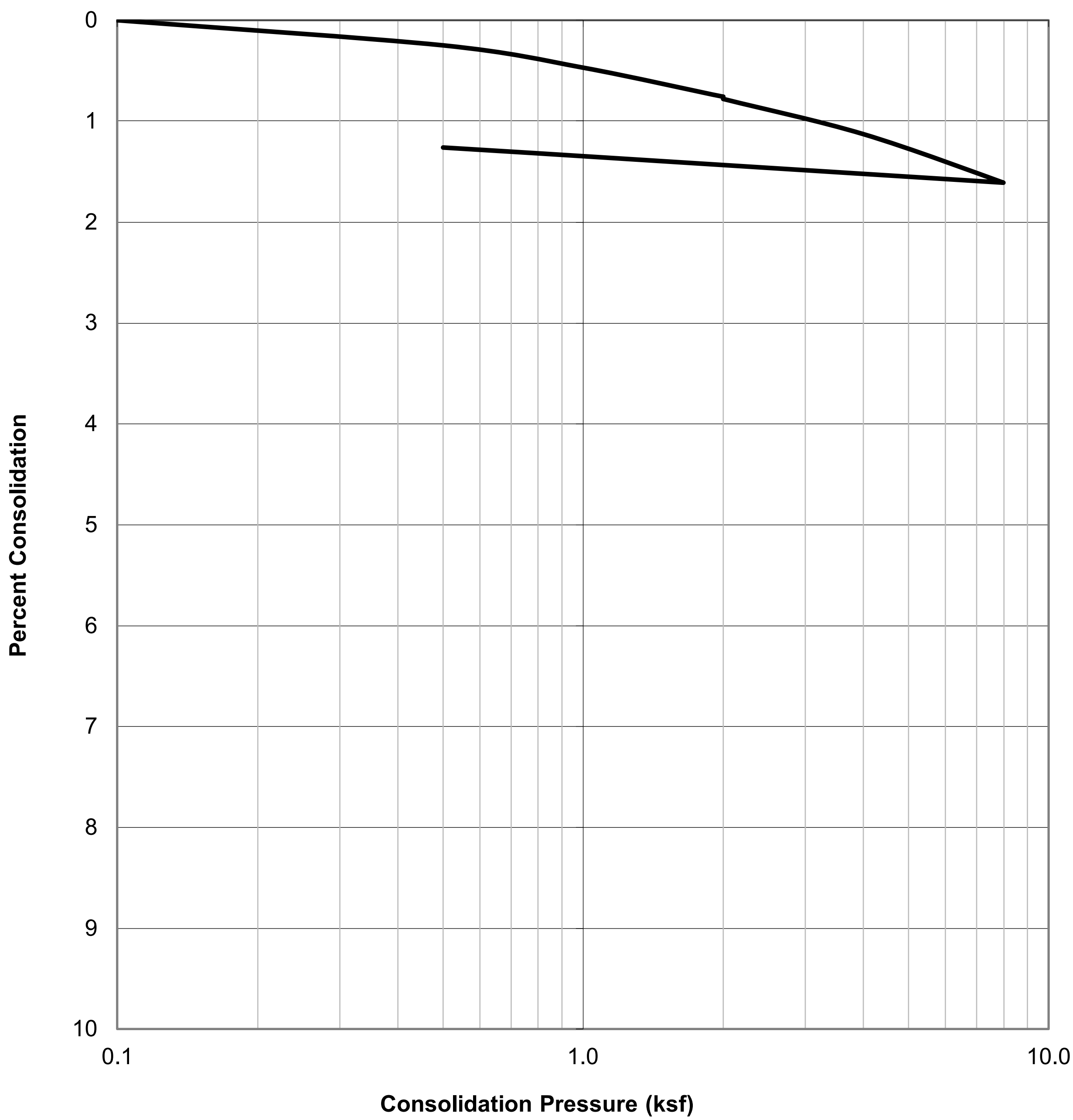
WATER ADDED AT 2.0 KSF



SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B2@25	Sand (SP)	114.7	12.0	14.2

GEOCON	**CONSOLIDATION TEST RESULTS** ASTM D-2435 Checked by: JJK	Project No.: W1814-06-01 800 & 908 NORTH MAIN STREET 1081 & 1087 NORTH VIGNES STREET LOS ANGELES, CALIFORNIA OCT. 2023 Figure B14

WATER ADDED AT 2.0 KSF



SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B2@32.5	Silty Sand (SM)	113.2	19.1	18.9

GEOCON

CONSOLIDATION TEST RESULTS

ASTM D-2435

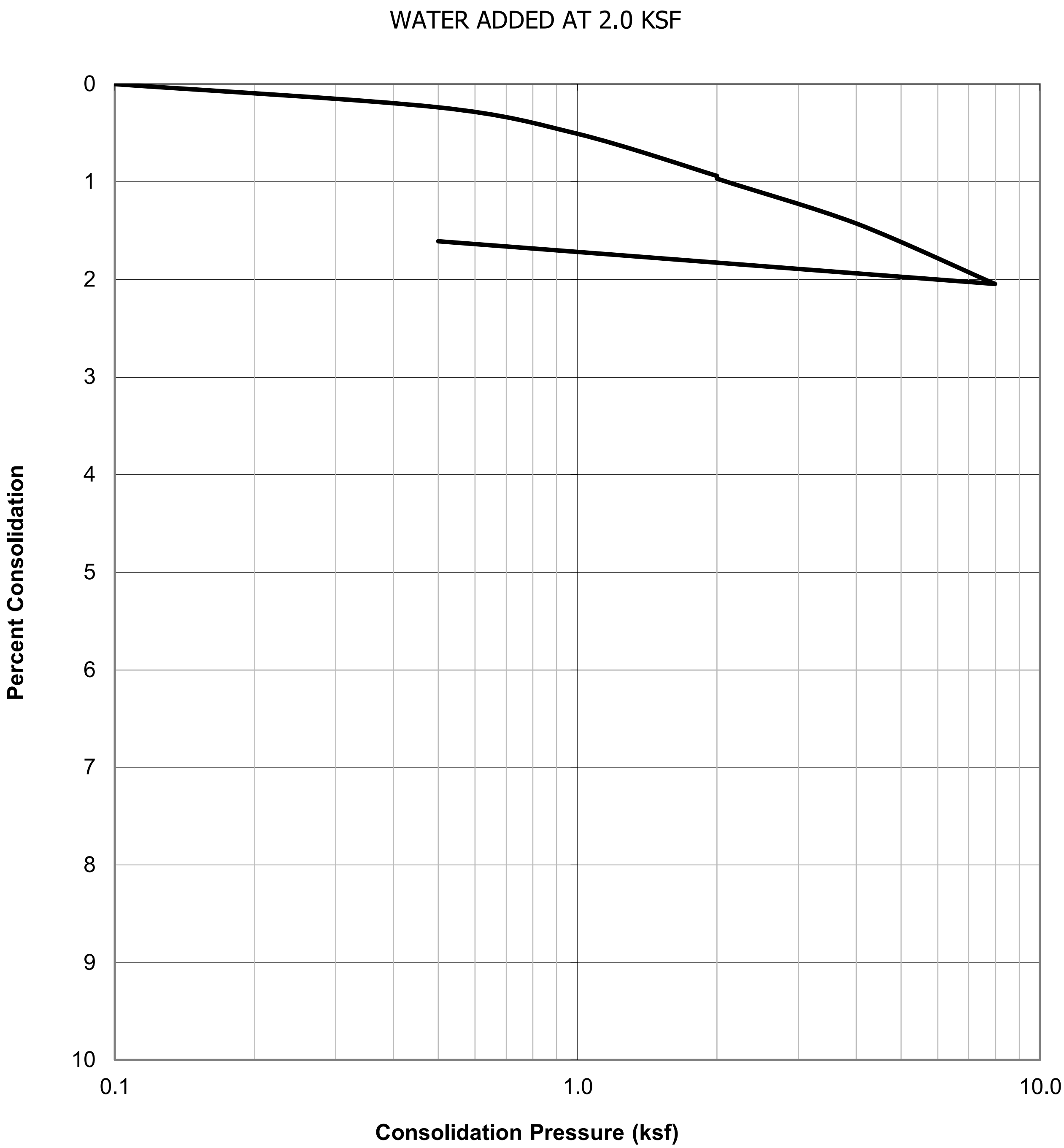
Checked by: JJK

Project No.: W1814-06-01

800 & 908 NORTH MAIN STREET
1081 & 1087 NORTH VIGNES STREET
LOS ANGELES, CALIFORNIA

OCT. 2023

Figure B15



SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B3@20'	Sand (SP)	98.2	23.0	24.8

GEOCON

CONSOLIDATION TEST RESULTS

ASTM D-2435

Checked by: JJK

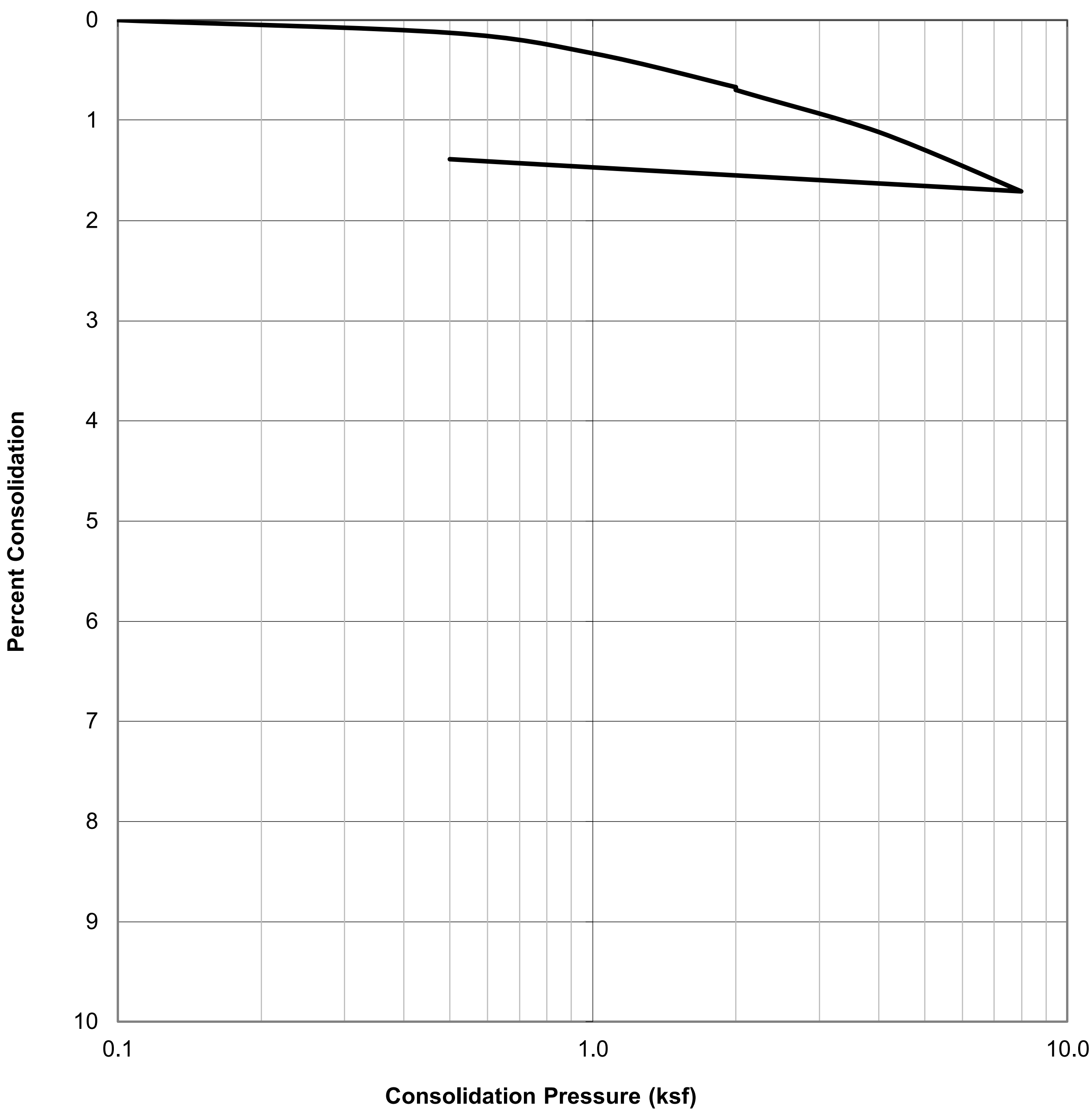
Project No.: W1814-06-01

800 & 908 NORTH MAIN STREET
1081 & 1087 NORTH VIGNES STREET
LOS ANGELES, CALIFORNIA

OCT. 2023

Figure B16

WATER ADDED AT 2.0 KSF



SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B3@25'	Sand w/ Silt (SP-SM)	114.1	15.6	16.3

GEOCON

CONSOLIDATION TEST RESULTS

ASTM D-2435

Checked by: JJK

Project No.: W1814-06-01

800 & 908 NORTH MAIN STREET
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LOS ANGELES, CALIFORNIA

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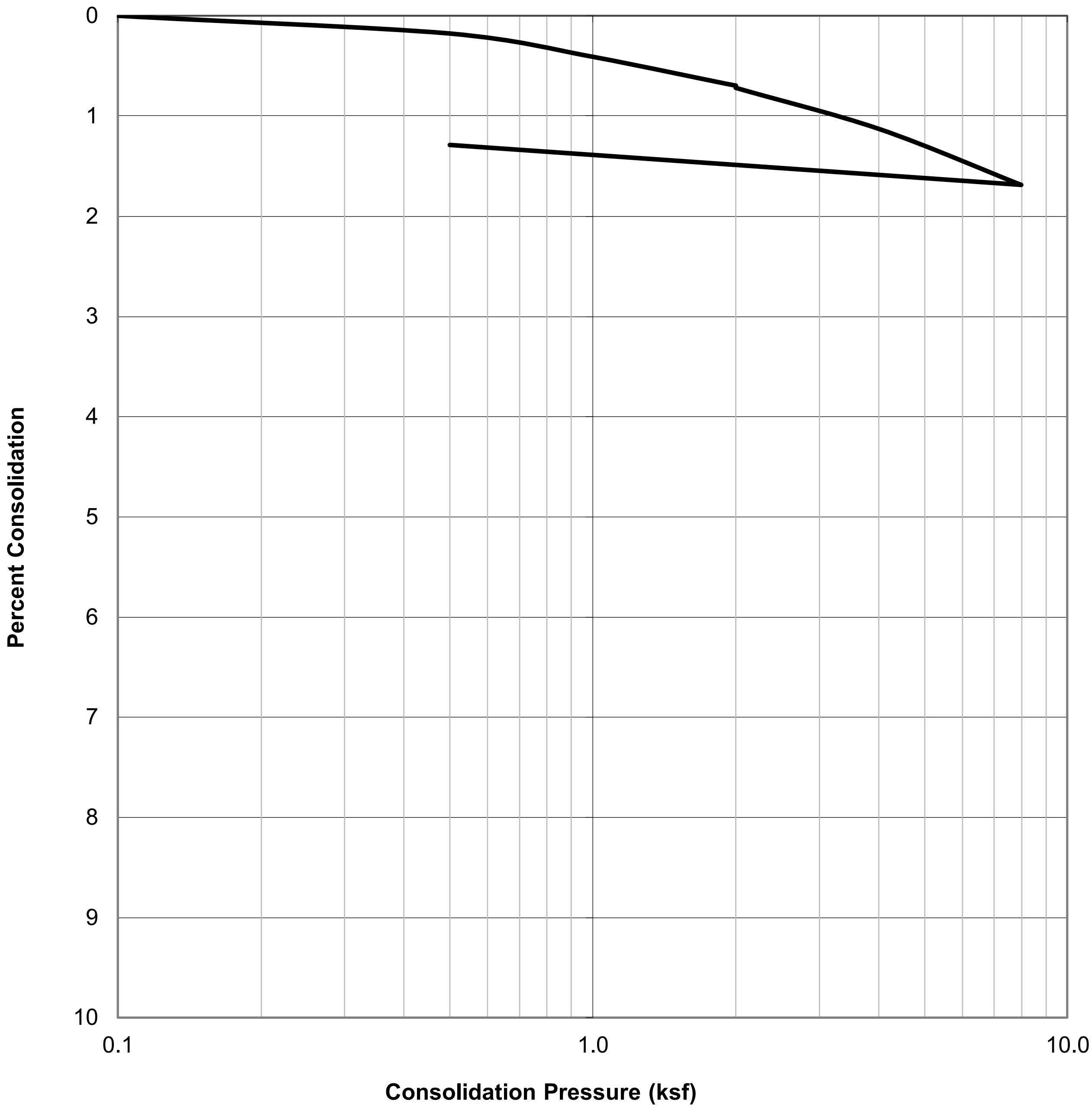
Figure B17

WATER ADDED AT 2.0 KSF



SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B3@30'	Sand w/ Silt (SP-SM)	110.7	16.0	18.4

GEOCON

CONSOLIDATION TEST RESULTS

ASTM D-2435

Checked by: JJK

Project No.: W1814-06-01

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1081 & 1087 NORTH VIGNES STREET
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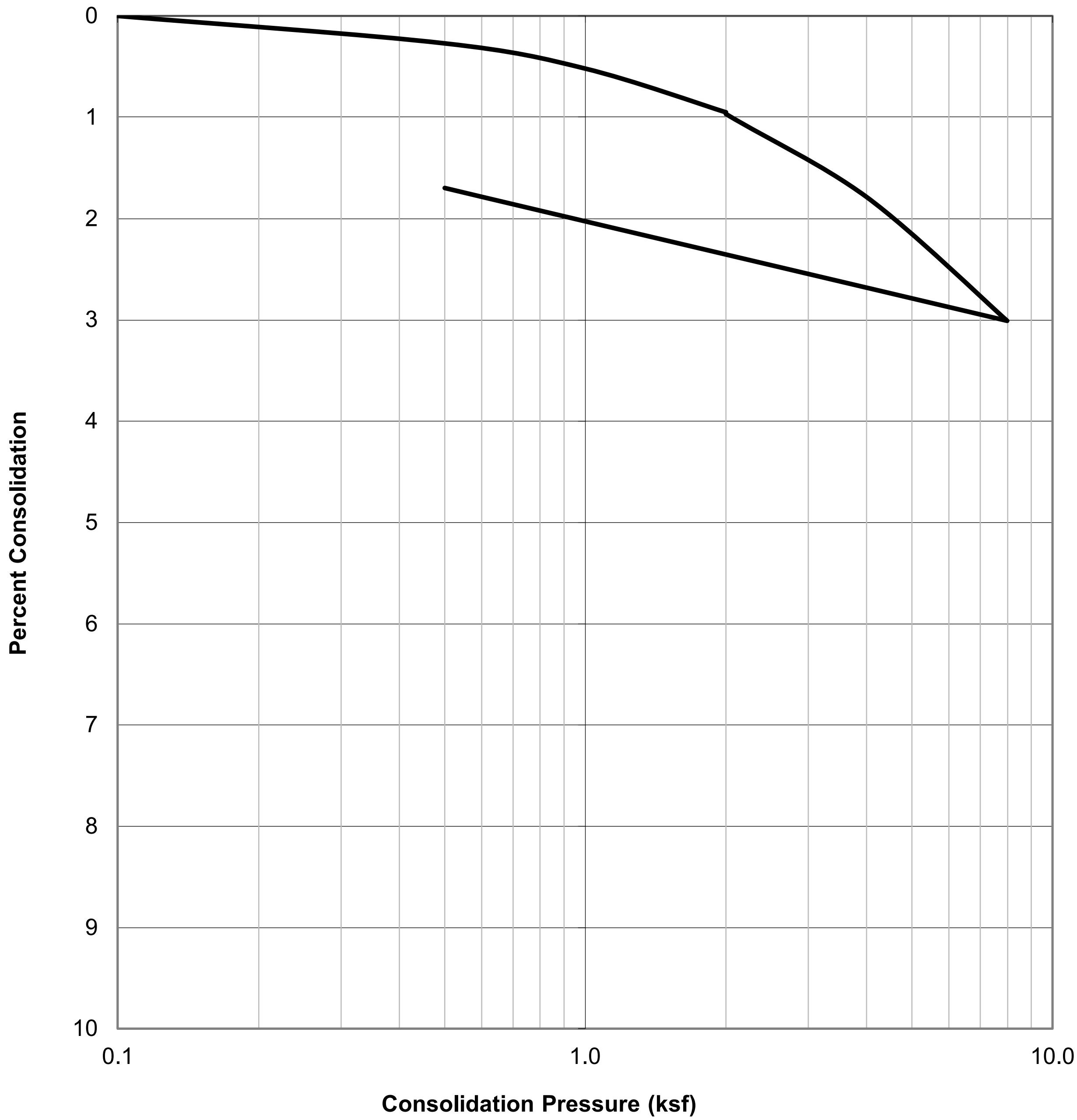
Figure B18

WATER ADDED AT 2.0 KSF



SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B3@40	Silt (ML)	96.2	22.9	26.2

GEOCON

CONSOLIDATION TEST RESULTS

ASTM D-2435

Checked by: JJK

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1081 & 1087 NORTH VIGNES STREET
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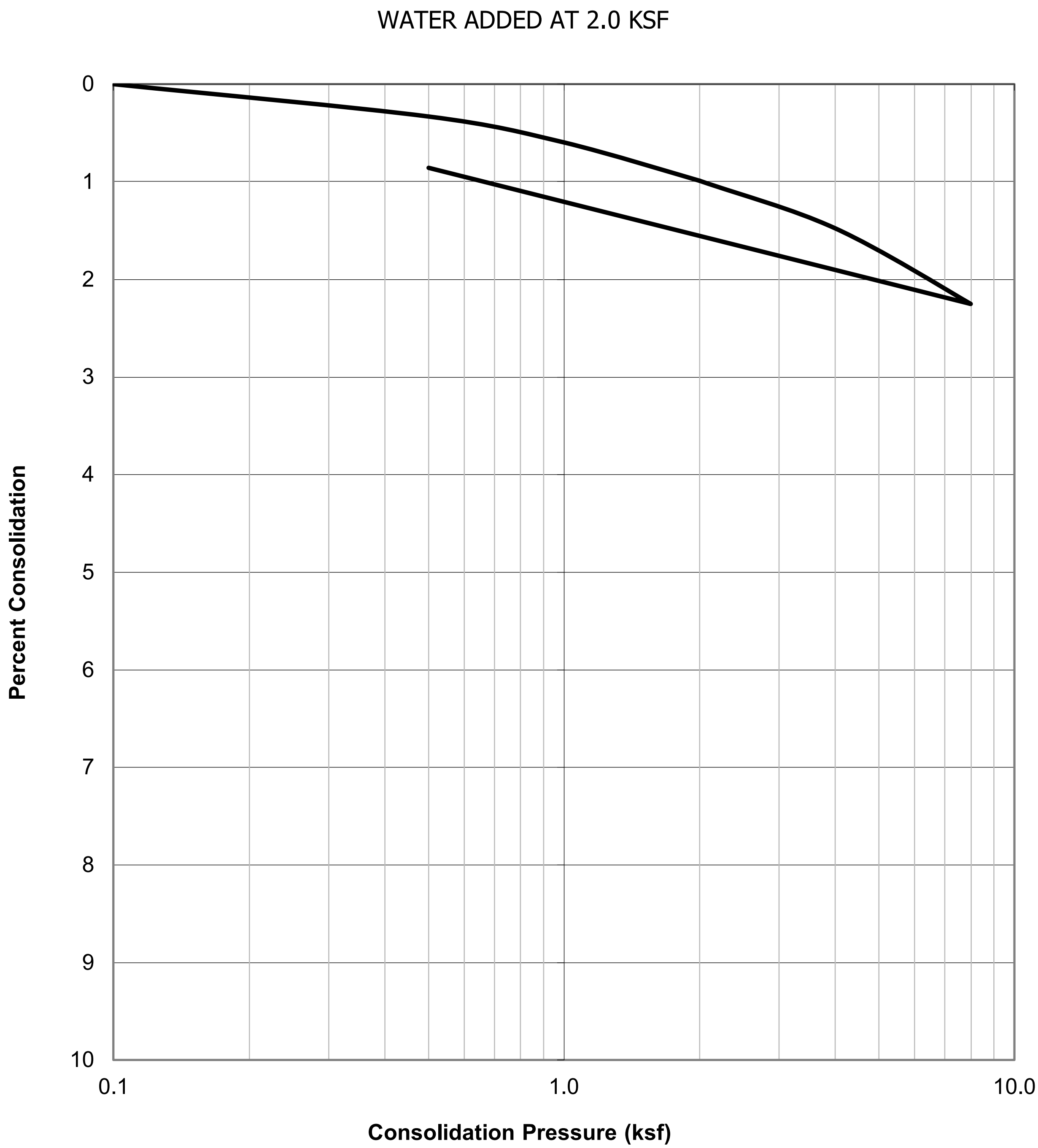
Figure B19



SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B3@45	Silt (ML)	92.8	30.7	30.7

GEOCON

CONSOLIDATION TEST RESULTS

ASTM D-2435

Checked by: JJK

Project No.: W1814-06-01

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1081 & 1087 NORTH VIGNES STREET
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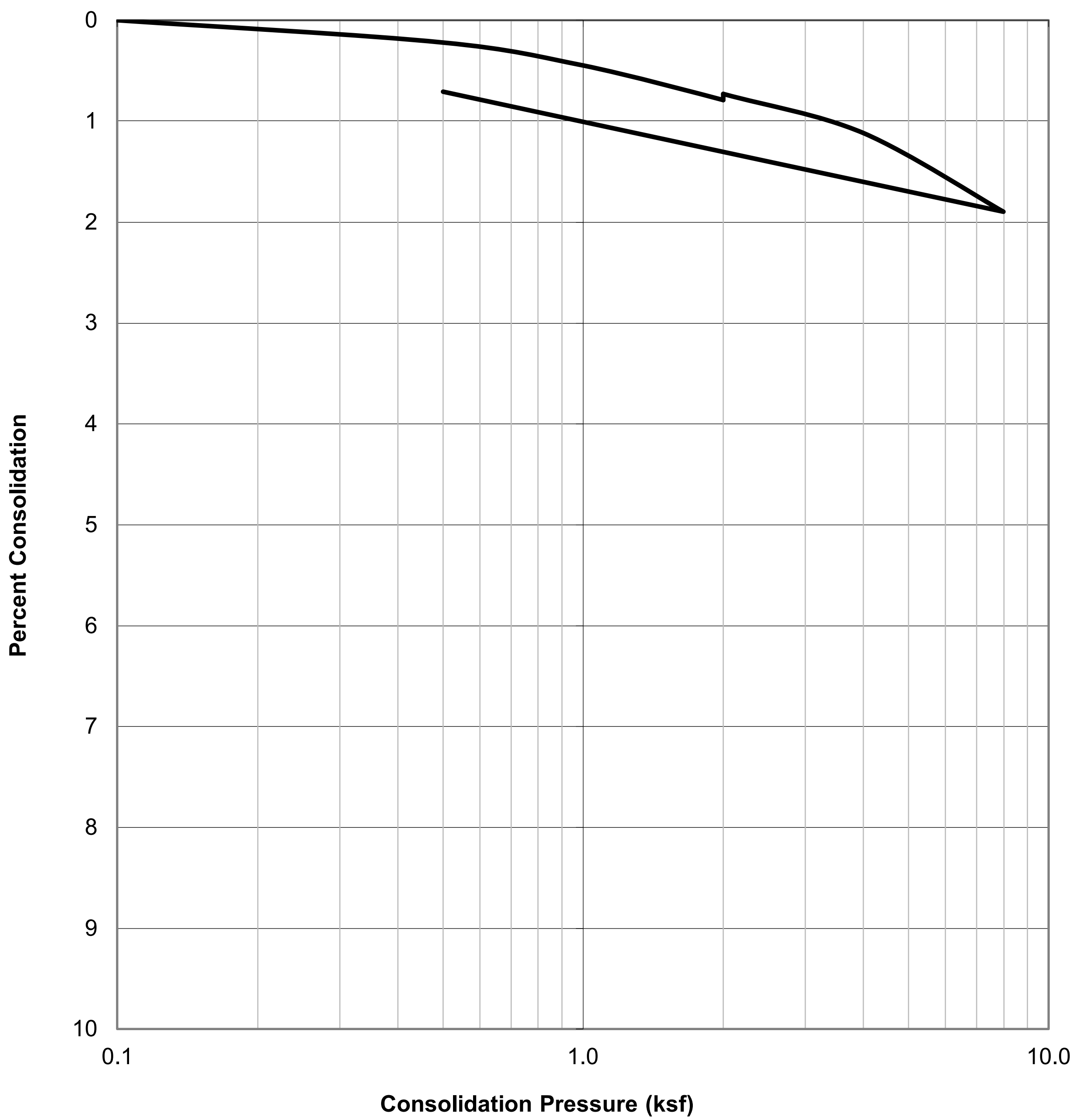
Figure B20

WATER ADDED AT 2.0 KSF



SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B3@50	Silt (ML)	98.2	24.2	26.8

GEOCON

CONSOLIDATION TEST RESULTS

ASTM D-2435

Checked by: JJK

Project No.: W1814-06-01

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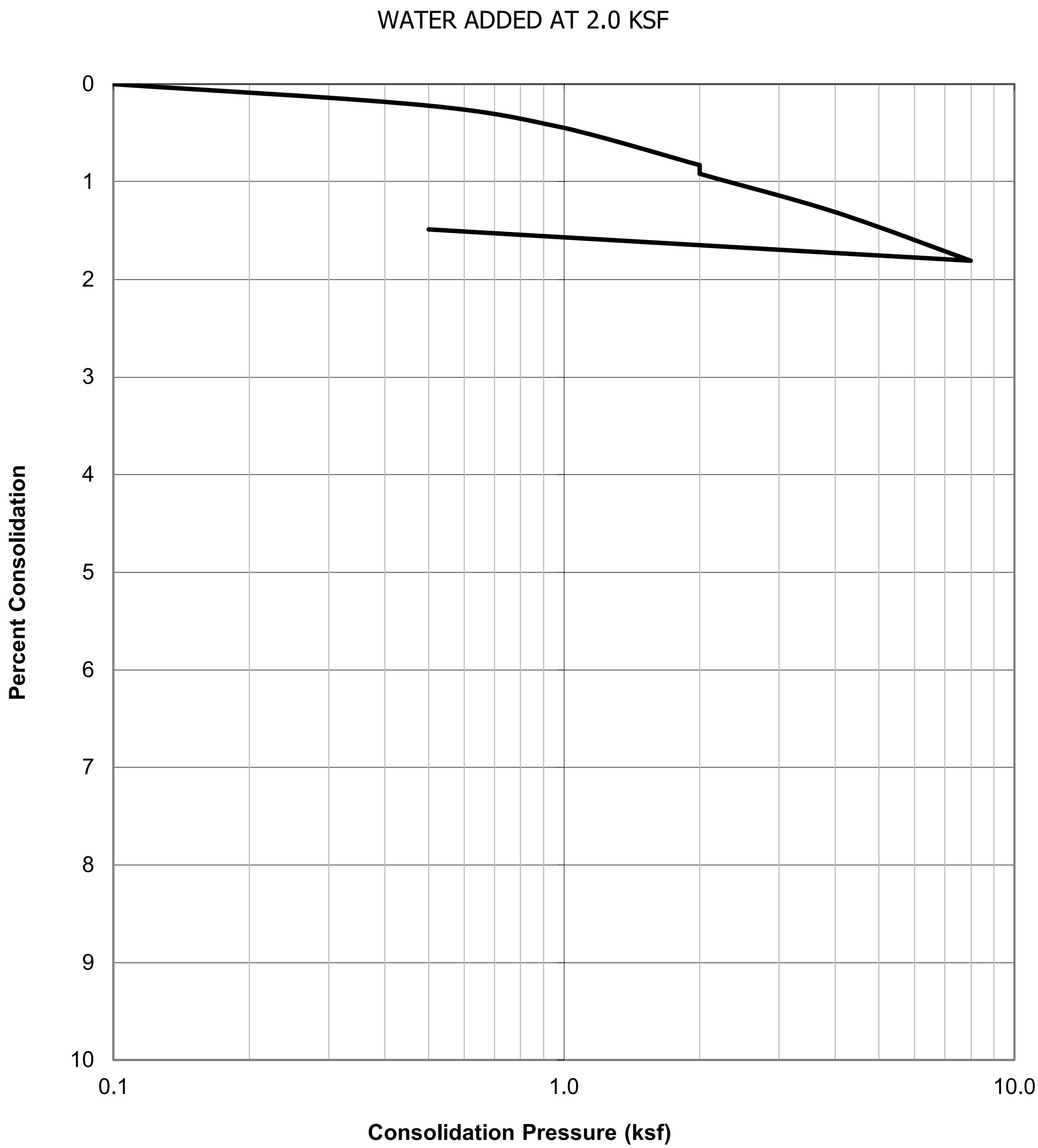
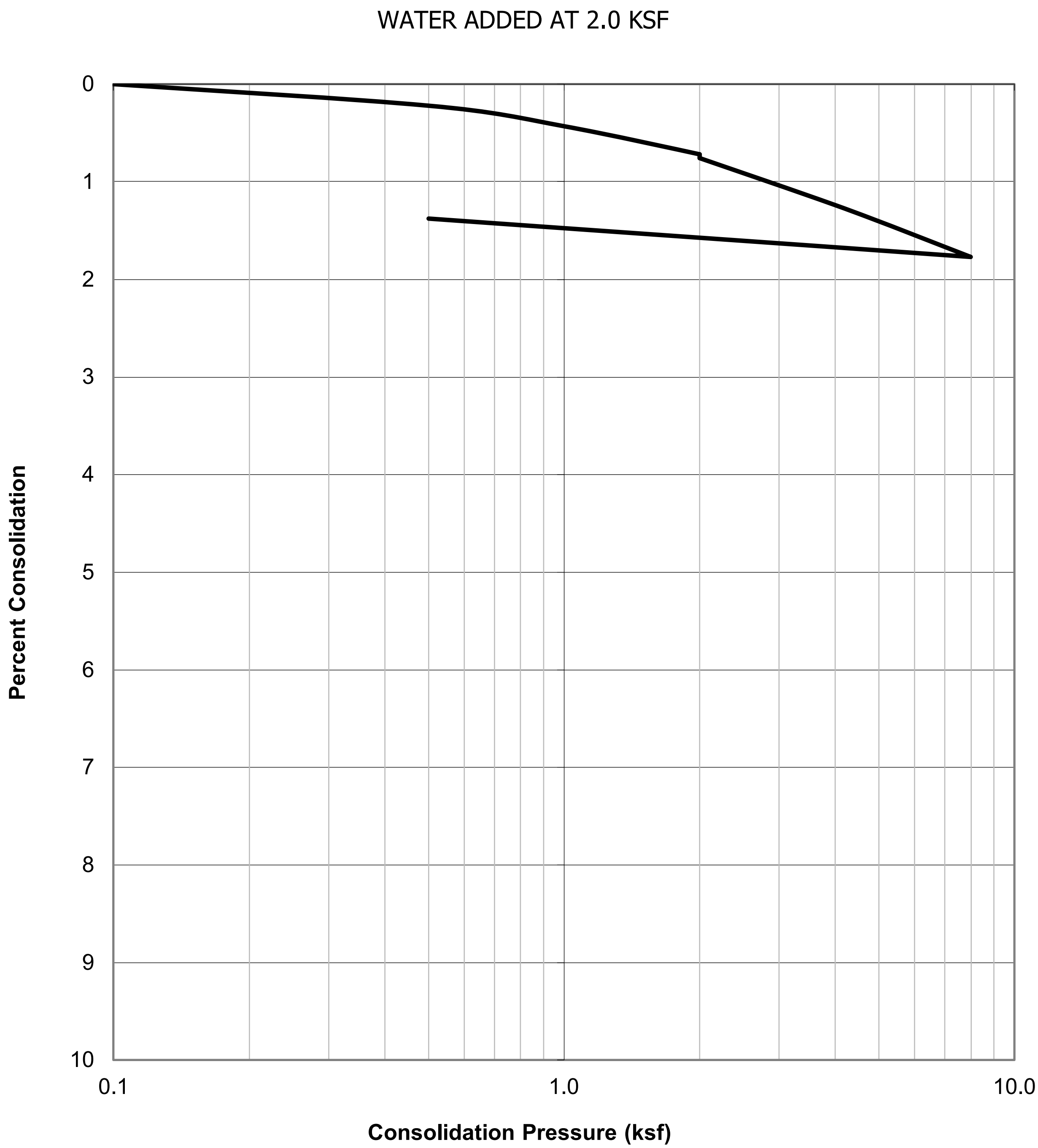
Figure B21



SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B4@32.5	Sand (SP)	130.5	10.1	10.5

GEOCON	**CONSOLIDATION TEST RESULTS** ASTM D-2435 Checked by: JJK	Project No.: W1814-06-01 800 & 908 NORTH MAIN STREET 1081 & 1087 NORTH VIGNES STREET LOS ANGELES, CALIFORNIA OCT. 2023 Figure B22



SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B4@47.5	Sand (SP)	133.2	9.5	10.7

GEOCON

CONSOLIDATION TEST RESULTS

ASTM D-2435

Checked by: JJK

Project No.: W1814-06-01

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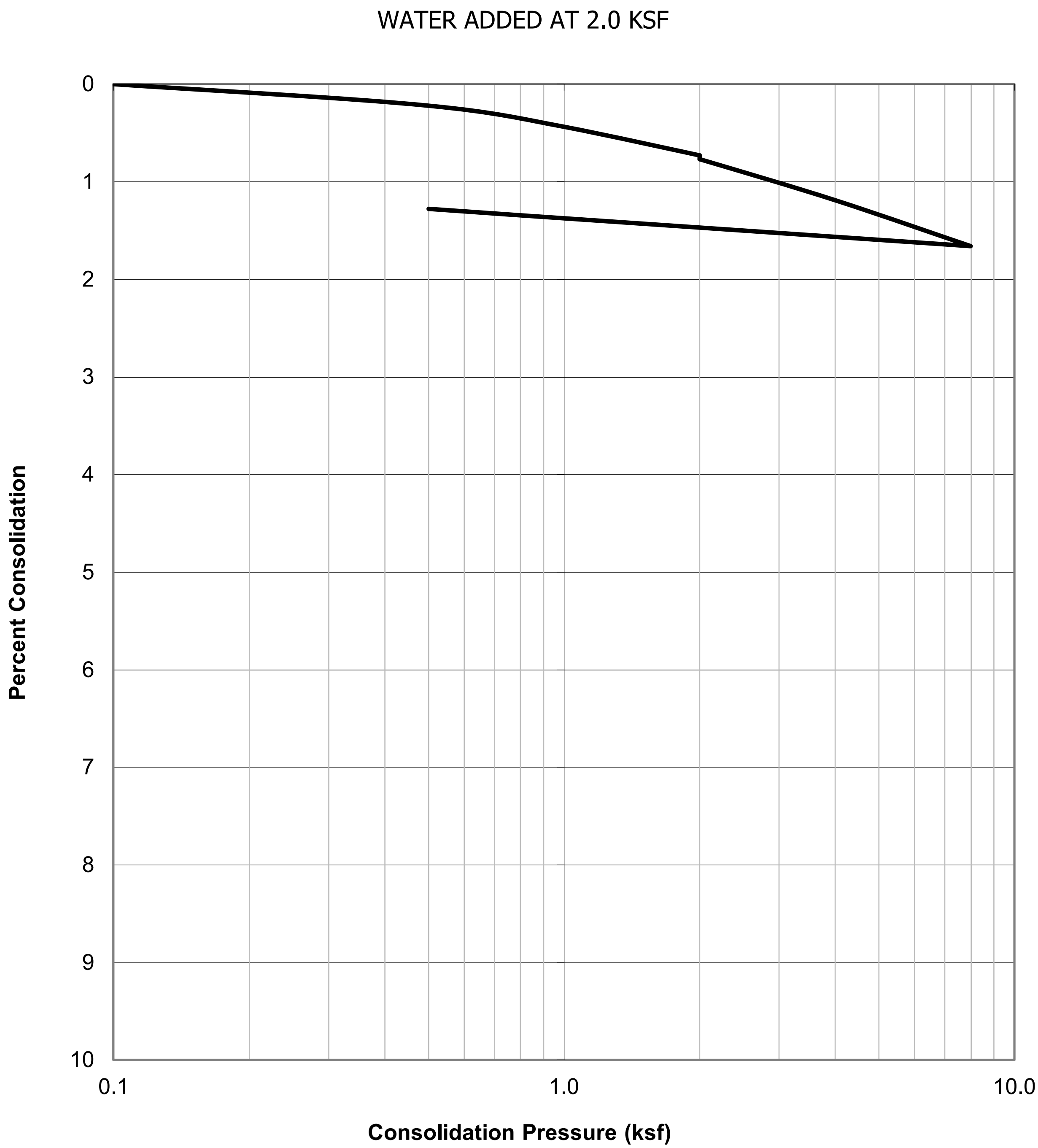

SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B4@52.5	Silty Sand (SM)	121.3	13.4	14.7

GEOCON

CONSOLIDATION TEST RESULTS

ASTM D-2435

Checked by: JJK

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1081 & 1087 NORTH VIGNES STREET
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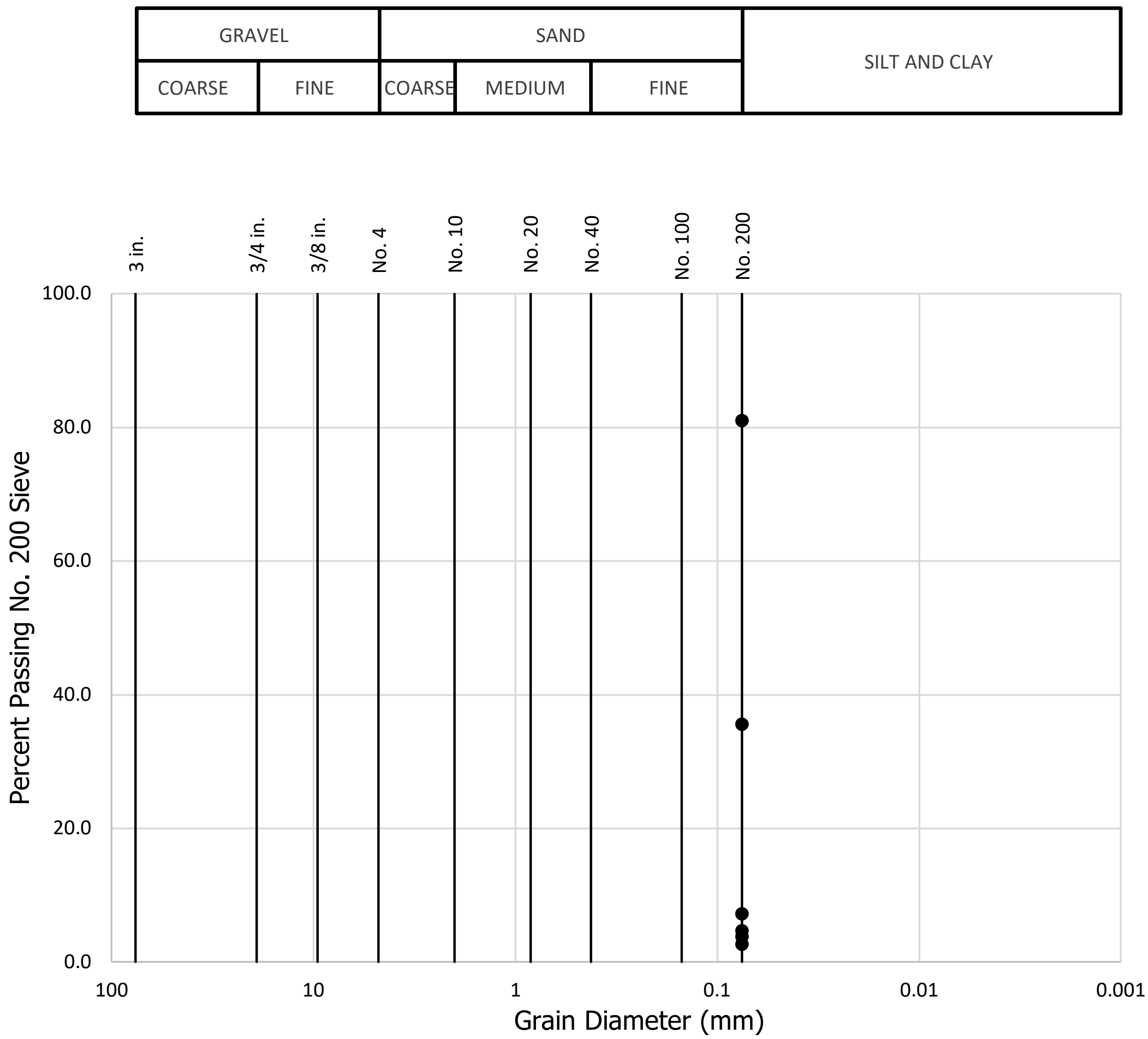

Sample No.	Percent Passing No. 200 Sieve
B1 @ 25'	35.6
B1 @ 30'	2.6
B1 @ 50'	81.0
B4 @ 30'	4.6
B4 @ 35'	3.8
B4 @ 40'	7.2

GEOCON

GRAIN SIZE ANALYSIS

ASTM D-1140

Checked by: JJK

Project No.: W1814-06-01

800 & 908 NORTH MAIN STREET
1081 & 1087 NORTH VIGNES STREET
LOS ANGELES, CALIFORNIA

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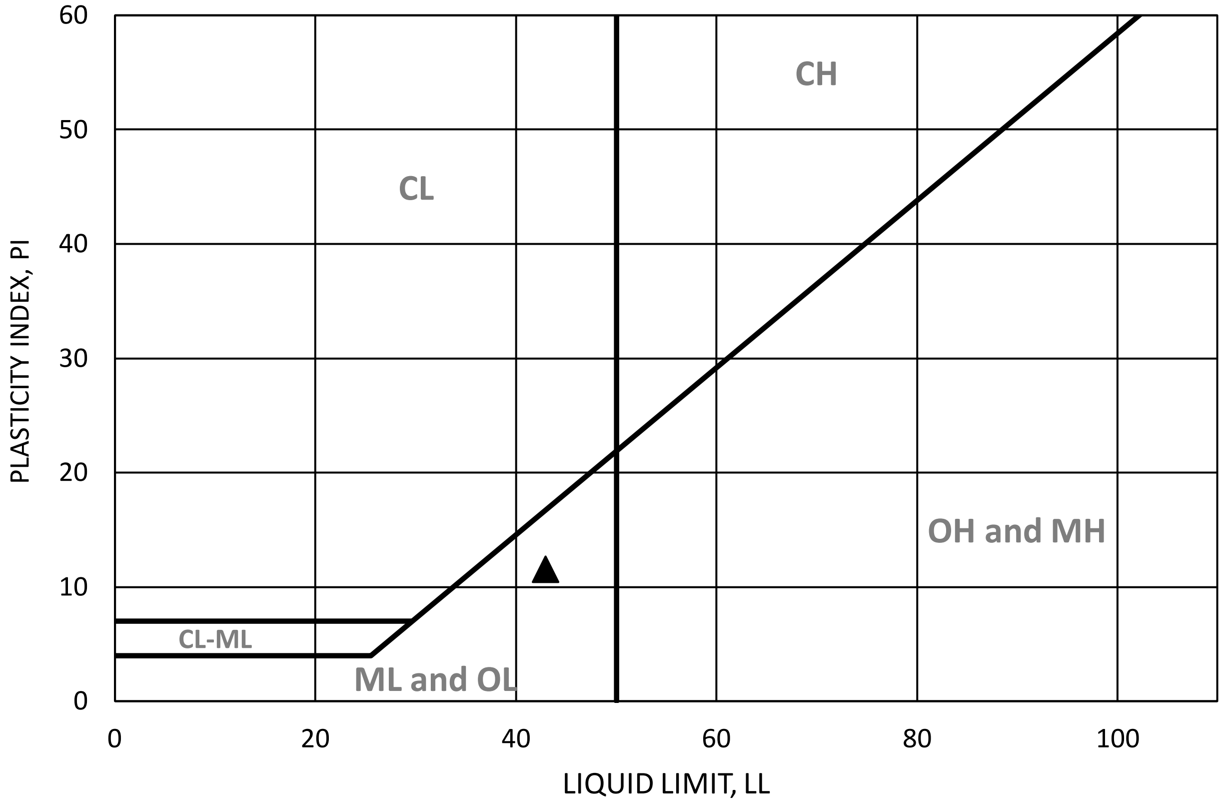
Figure B25



SYMBOL	BORING	DEPTH (ft)	LL	PL	PI	MOISTURE CONTENT AT SATURATION	SOIL BEHAVIOR
■	B1	25	N/P	N/P	N/P		N/P
◆	B1	30	N/P	N/P	N/P		N/P
▲	B1	50	43	31	12		ML
●	B4	30	N/P	N/P	N/P		N/P
□	B4	35	N/P	N/P	N/P		N/P
◇	B4	40	N/P	N/P	N/P		N/P
△							
○							

N/P = Non-Plastic

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ATTERBERG LIMITS

ASTM D-4318

Checked by: JJK

Project No.: W1814-06-01

800 & 908 NORTH MAIN STREET
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Figure B26

B1+B2@15-20'

MOLDED SPECIMEN		BEFORE TEST	AFTER TEST
Specimen Diameter	(in.)	4.0	4.0
Specimen Height	(in.)	1.0	1.0
Wt. Comp. Soil + Mold	(gm)	778.1	797.9
Wt. of Mold	(gm)	367.6	367.6
Specific Gravity	(Assumed)	2.7	2.7
Wet Wt. of Soil + Cont.	(gm)	491.8	797.9
Dry Wt. of Soil + Cont.	(gm)	468.6	378.7
Wt. of Container	(gm)	191.8	367.6
Moisture Content	(%)	8.4	13.6
Wet Density	(pcf)	123.8	129.6
Dry Density	(pcf)	114.2	114.1
Void Ratio		0.5	0.5
Total Porosity		0.3	0.3
Pore Volume	(cc)	66.7	66.8
Degree of Saturation	(%) [S_{meas}]	48.1	77.2

Date	Time	Pressure (psi)	Elapsed Time (min)	Dial Readings (in.)
9/19/2023	10:00	1.0	0	0.329
9/19/2023	10:10	1.0	10	0.3285
Add Distilled Water to the Specimen				
9/20/2023	10:00	1.0	1430	0.329
9/20/2023	11:00	1.0	1490	0.329

Expansion Index (EI meas) =	0.5
Expansion Index (Report) =	**1**

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

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

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EXPANSION INDEX TEST RESULTS

ASTM D-4829

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Project No.: W1814-06-01

800 & 908 NORTH MAIN STREET
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LOS ANGELES, CALIFORNIA

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Figure B27

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

Sample No.	pH	Resistivity (ohm centimeters)
B1+B2@15-20	8.8	6700 (Moderately Corrosive)
B4@30-35	5.9	1000 (Severely Corrosive)

SUMMARY OF LABORATORY CHLORIDE CONTENT TEST RESULTS
AASHTO T291 ASTM C1218

Sample No.	Chloride Ion Content (%)
B1+B2@15-20	0.006
B4@30-35	0.002

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

Sample No.	Water Soluble Sulfate (% SO_4)	Sulfate Exposure
B1+B2@15-20	0.000	S0
B4@30-35	0.068	S0

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CORROSIVITY TEST RESULTS

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Project No.: W1814-06-01

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Figure B28