APPENDIX E.1

Geotechnical Investigation

GEOTECHNICAL INVESTIGATION

CRENSHAW CROSSING MIXED-USE DEVELOPMENT

3606 W. EXPOSITION BLVD
3633 W. OBAMA BLVD
3642-3630 S CRENSHAW BLVD
3502-3510 W. EXPOSITION BLVD
3501-3519 W. OBAMA BLVD
631-3645 S. BRONSON BLVD
LOS ANGELES, CALIFORNIA
TRACT 11393, LOT 1; PM 3201, LOTS A, B, & C;
AND PM 2647, LOTS A & B

GEOCON
WEST, INC.

GEOTECHNICAL
ENVIRONMENTAL
MATERIALS

PREPARED FOR

WIP EXPO CRENSHAW, LLC
SANTA MONICA, CALIFORNIA

PROJECT NO. A9930-06-01

AUGUST 14, 2019

GEOCON
WEST, INC.
GEOTECHNICAL ■ ENVIRONMENTAL ■ MATERIALS

Project No. A9930-06-01
August 14, 2019

WIP Expo Crenshaw, LLC
2716 Ocean Park Boulevard, Suite 2025
Santa Monica, California 90405

Attention: Mr. Max Levenstein

Subject: GEOTECHNICAL INVESTIGATION
CRENSHAW CROSSING MIXED-USE DEVELOPMENT
3606 WEST EXPOSITION BOULEVARD
3633 WEST OBAMA BOULEVARD
3642-3630 SOUTH CRENSHAW BOULEVARD
3502-3510 WEST EXPOSITION BOULEVARD
3501-3519 WEST OBAMA BOULEVARD
3631-3645 SOUTH BRONSON BOULEVARD, LOS ANGELES, CALIFORNIA
TRACT 11393, LOT 1; PM 3201, LOTS A, B, & C; AND PM 2647, LOTS A & B

Dear Mr. Levenstein:

In accordance with your authorization of our proposal dated January 25, 2019, we have performed a geotechnical investigation for the proposed Crenshaw Crossing mixed-use development located at the southwest and southeast corners of the intersection of Exposition Boulevard and Crenshaw Boulevard 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 proposed design and construction. Based on the results of our investigation, it is our opinion that the sites can be developed as proposed, provided the recommendations of this report are followed and implemented during design and construction.

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

Very truly yours,

GEOCON WEST, INC.

DRAFT

Rex Panoy	Jelisa Thomas Adams	Susan F. Kirkgard
Staff Engineer	GE 3092	CEG 1754

(EMAIL) Addressee

TABLE OF CONTENTS

1. PURPOSE AND SCOPE 1
2. SITE AND PROJECT DESCRIPTION 1
3. BACKGROUND REVIEW 2
4. GEOLOGIC SETTING 3
5. SOIL AND GEOLOGIC CONDITIONS 3
    5.1 Artificial Fill 3
    5.2 Alluvium 4
6. GROUNDWATER 4
7. GEOLOGIC HAZARDS 5
    7.1 Surface Fault Rupture 5
    7.2 Seismicity 6
    7.3 Seismic Design Criteria 6
    7.4 Liquefaction Potential 8
    7.5 Slope Stability 9
    7.6 Earthquake-Induced Flooding 10
    7.7 Tsunamis, Seiches, and Flooding 10
    7.8 Oil Fields & Methane Potential 10
    7.9 Subsidence 11
8. CONCLUSIONS AND RECOMMENDATIONS 12
    8.1 General 12
    8.2 Soil and Excavation Characteristics 15
    8.3 Minimum Resistivity, pH, and Water-Soluble Sulfate *(In-Progress)* 15
    8.4 Temporary Dewatering 16
    8.5 Grading 16
    8.6 Controlled Low Strength Material (CLSM) 19
    8.7 Foundation Design – General 20
    8.8 Friction Pile Design 20
    8.9 End-Bearing Caissons 22
    8.10 Deepened Foundation Installation 24
    8.11 Lateral Design 25
    8.12 Concrete Slabs-on-Grade 27
    8.13 Preliminary Pavement Recommendations 28
    8.14 Retaining Walls Design 29
    8.15 Dynamic (Seismic) Lateral Forces 31
    8.16 Retaining Wall Drainage 31
    8.17 Elevator Pit Design 32
    8.18 Elevator Piston 32
    8.19 Temporary Excavations 33
    8.20 Shoring – Soldier Pile Design and Installation 33
    8.21 Temporary Tie-Back Anchors 38
    8.22 Anchor Installation 39
    8.23 Anchor Testing 39
    8.24 Internal Bracing 40
    8.25 Surcharge from Adjacent Structures and Improvements 40
    8.26 Surface Drainage 42
    8.27 Plan Review 42

LIMITATIONS AND UNIFORMITY OF CONDITIONS

LIST OF REFERENCES

TABLE OF CONTENTS (Continued)

MAPS, TABLES, AND ILLUSTRATIONS
  Figure 1, Vicinity Map
  Figure 2, Site Plan
  Figure 3, Cross Sections
  Figure 4, Regional Fault Map
  Figure 5, Regional Seismicity Map
  Figures 6 through 9, DE Empirical Estimation of Liquefaction Potential
  Figures 10 through 13, MCE Empirical Estimation of Liquefaction Potential
  Figure 14, Retaining Wall Calculation
  Figures 15 and 16, Retaining Wall Drain Detail
  Figure 17, Shoring Wall Calculation

APPENDIX A
  FIELD INVESTIGATION
  Figures A1 through A12, Boring Logs

APPENDIX B
  LABORATORY TESTING
  Figures B1 through B11, Direct Shear Test Results
  Figures B12 through B35, Consolidation Test Results
  Figures B36 and B37, Atterberg Limits
  Figures B38 and 39, Grain Size Analysis
  Figures B40 and B41, Expansion Index Test Results
  Figure B42, Modified Compaction Test of Soils
  Figure B43, Corrosivity Test Results

GEOTECHNICAL INVESTIGATION

1. PURPOSE AND SCOPE

This report presents the results of a geotechnical investigation for the proposed Crenshaw Crossing mixed-use development located at the southwest and southeast corners of the intersection of Exposition Boulevard and Crenshaw Boulevard in the City of Los Angeles, California (see Vicinity Map, Figure 1). The purpose of the investigation was to evaluate subsurface soil and geologic conditions underlying the sites 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 review of published geologic information and in-house information, a site reconnaissance, field exploration, laboratory testing, engineering analysis, and the preparation of this report. The sites were explored on June 27, 2019, and July 6, 2019, by excavating twelve 8-inch-diameter borings utilizing a truck-mounted hollow-stem auger drilling machine. The borings were drilled to depths ranging from approximately 10½ to 50½ feet below existing ground surface. The locations of the borings are depicted on the Site Plan (see Figure 2). A detailed discussion of the field investigation, including boring logs, is presented in Appendix A.

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

The recommendations presented herein are based on analyses of the data obtained during our 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 proposed development consists of two properties located on the southwest corner (Site A) and the southeast corner (Site B) of the intersection of Exposition Boulevard and Crenshaw Boulevard. The addresses associated with Site A are 3606 W. Exposition Boulevard and 3633 West Obama Boulevard. The addresses associated with Site B are 3501 and 3505 West Obama Boulevard; 3631, 3633, 3635, 3639, and 3645 South Bronson Avenue; 3502 and 3510 West Exposition Boulevard; and 3630 South Crenshaw Boulevard.

Site A is bounded by the existing Metro Expo line and Exposition Boulevard to the north, Crenshaw Boulevard to the east, an existing gas station to the southeast, Obama Boulevard to the south, and South Victoria Avenue to the west. Site B is bounded by the existing Metro Expo Line and Exposition Boulevard to the north, South Bronson Avenue to the east, Obama Boulevard to the south, and Crenshaw Boulevard to the west. Site A is currently occupied by a one-story office building and parking lot operated by Los Angeles County. Site B is an active construction site for the Metro Transit Authority (MTA) Crenshaw/Exposition Station. The sites and surrounding vicinity are relatively level to gently sloping to the west. Surface water drainage at the site appears to be by sheet flow along the existing ground contours towards to the city streets.

Based on our review of not-for-construction set of MTA Crenshaw/Exposition Station plans, the station box extends to depths on the order of 60 to 70 feet below the existing ground surface. The actual depth of the station box should be requested from and confirmed by Metro. Because the proposed structures will be adjacent to the MTA station box, it is anticipated that the geotechnical report and foundation design will require MTA review and approval. The proposed development must be designed in a manner that will prevent or minimize surcharges on the MTA substructures.

Based on preliminary architectural drawings provided to us for review, development of Site A will include a podium-style structure consisting of five levels of residential housing over three parking levels constructed at or near present site grade. Development of Site B will consist of an eight-story mixed-use structure wrapped around three levels of above grade parking constructed over one subterranean parking level (see Figures 2 and 3).

Based on the preliminary nature of the design at this time, wall and column loads were not available. It is anticipated that column loads for the proposed structures will be up to 950 kips, and 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. BACKGROUND REVIEW

As a part of the preparation of this report, we researched and reviewed prior reports performed for the adjacent property located at 3670 South Crenshaw Boulevard. The reports were obtained from the public database at the City of Los Angeles Department of Building and Safety:

> *Report of Geotechnical Investigation, Proposed Mixed-Use Building Project, Lots 20, 21, 23, 27035 and 41 of Tract 11754, 3670 South Crenshaw Boulevard, Los Angeles, California 90018*, prepared by AES, Project No. 18-332-02, Dated May 9, 2018.

The referenced report prepared by AES indicates that the subject site was previously investigated by Irvine Engineering Group (IEG), Geotechnical Professionals (GPI), and Tetra Tech (Tetra) between 2008 and 2016. The prior reports by IEG, GPI, and Tetra are appended to the AES the report. Within the borings performed by AES in March 2018, groundwater was not encountered and the borings were drilled to a maximum depth of 51½ feet. Similarly, the borings conducted by Tetra in 2016 also did not encounter groundwater. Both AES and Tetra attribute the lack of groundwater to the dewatering activity associated with the adjacent MTA Crenshaw/Exposition Station construction. Groundwater was encountered by IEG and GPI at depths of approximately 11 to 13 feet below the ground surface during their borings conducted in 2008 and 2010, respectively. Within the various borings conducted at this site, organic deposits were noted between depths of 15 feet to 25 feet.

Geocon has reviewed the information contained within the referenced report prepared by AES, including the appended reports by Irvine Engineering Group, Geotechnical Professionals, and Tetra Tech, and as this information pertains to an adjacent site, we are using this data for reference only. The recommendations presented herein are based on the results of our borings and laboratory testing conducted for the subject sites.

4. 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, the Elysian Hills and the Repetto Hills to the north and northeast, the Puente Hills and Whittier Fault to the east, the Palos Verdes Peninsula and Pacific Ocean to the west and south, and the Santa Ana Mountains and San Joaquin Hills to the southeast. The basin is underlain by a deep structural depression which has been filled by both marine and continental sedimentary deposits. Regionally, the site is located within the northern portion of the Peninsular Ranges geomorphic province that is characterized by northwest-trending geologic structures and physiographic features such as the Newport-Inglewood Fault Zone located approximately 1.2 miles west-southwest of the site (California Geological Survey, 2014).

5. SOIL AND GEOLOGIC CONDITIONS

Based on our field investigation and published geologic maps of the area, the site is underlain by artificial fill that is in turn underlain by Holocene age alluvial fan deposits consisting of gravel, sand, silt, and clay with some localized zones of cobbles and boulders (Dibblee, 1991). Detailed stratigraphic profiles are provided on the boring logs in Appendix A.

5.1 Artificial Fill

Artificial fill was encountered in our borings to a maximum depth of 5½ feet below existing ground surface. As encountered in our explorations, the fill consists of brown to dark brown, olive brown or yellowish brown silt, sandy silt and silty sand with localized beds containing some fine gravel. The artificial fill is characterized as slightly moist, and soft to firm or loose. The fill is the result of past grading and construction activities at the site.

5.2 Alluvium

The artificial fill is underlain by Holocene age alluvial fan deposits. The alluvium generally consists of olive brown or grayish brown to brown interbedded silt, sandy silt, clay, sandy clay, silty sand, sand with silt, and poorly-graded sand. The alluvium is slightly moist to wet, and very soft to stiff or loose to medium dense. Organic materials were encountered in several borings at depths of 20 to 25 feet below the ground surface. In general, below depths of 27 to 30 feet, the alluvium consists predominately of poorly graded to well-graded sand and silty sand and is dense to very dense. Organic materials were not observed within this layer of alluvial sands.

6. GROUNDWATER

Based on a review of the Seismic Hazard Zone Report for the Hollywood Quadrangle (California Division of Mines and Geology [CDMG], 1998), the historically highest groundwater level in the area is approximately 10 feet beneath the ground surface. Groundwater level information in the CDMG publication is based on data collected from the early 1900’s to the late 1990’s. Based on current groundwater basin management practices, it is unlikely that the groundwater levels will ever exceed the historic high levels.

Groundwater was encountered in borings B-1, B-3, and B-7 at depths ranging from 18 to 20 feet below existing ground surface. Several drilled borings, B-2, B-8, and B-9, extended below the anticipated groundwater depth but groundwater was not encountered in those borings. The variable depths of groundwater encountered in our borings may be a result of the on-going dewatering for the construction of the MTA station box. Furthermore, site exploration performed in 2008 and 2010 at 3670 South Crenshaw (immediately south of Site B and Obama Boulevard) encountered water at depths ranging from 11 to 13 feet.

Considering the historic high groundwater level and the depth to groundwater encountered in our borings, groundwater may be encountered during construction. It is not uncommon for groundwater levels to vary seasonally or for groundwater seepage conditions to develop where none previously existed, especially in impermeable fine-grained soils which are heavily irrigated or after seasonal rainfall. In addition, recent requirements for stormwater infiltration could result in shallower seepage conditions in the immediate site vicinity. Proper surface drainage of irrigation and precipitation will be critical for future performance of the project. Recommendations for drainage are provided in the S*urface Drainage* section of this report (see Section 8.26).

7. GEOLOGIC HAZARDS

7.1 Surface Fault Rupture

The numerous faults in Southern California include active, potentially active, 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, an active fault is one that has had surface displacement within Holocene time (about the last 11,700 years). A potentially active 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, 2014) or a city-designated Preliminary Fault Rupture Study Area (City of Los Angeles, 2019) for surface fault rupture hazards. No active or potentially active 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 4, Regional Fault Map.

The closest active fault to the site is the Newport-Inglewood Fault Zone located approximately 1.2 miles to the west-southwest (CGS, 2014). Other nearby active faults are the Santa Monica Fault, the Hollywood Fault, the Raymond Fault, the Verdugo Fault, and the Palos Verde Fault Zone (offshore segment) located approximately 4.9 miles north-northwest, 5.4 miles north, 8.8 miles northeast, 10.5 miles northeast, and 12 miles south-southwest of the site, respectively (USGS, 2006; Ziony and Jones, 1989). The active San Andreas Fault Zone is located approximately 38 miles northeast of the site.

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

7.2 Seismicity

As with all of Southern California, the site has experienced historic earthquakes from various regional faults. The seismicity of the region surrounding the site was formulated based on research of an electronic database of earthquake data. The epicenters of recorded earthquakes with magnitudes equal to or greater than 5.0 in the site vicinity are depicted on Figure 5, Regional Seismicity Map. A partial list of moderate to major magnitude earthquakes that have occurred in the Southern California area within the last 100 years is included in the following table.

LIST OF HISTORIC EARTHQUAKES

Earthquake (Oldest to Youngest)	Date of Earthquake	Magnitude	Distance to Epicenter (Miles)	Direction to Epicenter
Near Redlands	July 23, 1923	6.3	62	E
Long Beach	March 10, 1933	6.4	35	SE
Tehachapi	July 21, 1952	7.5	78	NW
San Fernando	February 9, 1971	6.6	27	N
Whittier Narrows	October 1, 1987	5.9	15	E
Sierra Madre	June 28, 1991	5.8	25	NE
Landers	June 28, 1992	7.3	109	E
Big Bear	June 28, 1992	6.4	87	E
Northridge	January 17, 1994	6.7	17	NW
Hector Mine	October 16, 1999	7.1	124	ENE
Ridgecrest	July 5, 2019	7.1	127	NNE

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

7.3 Seismic Design Criteria

The following table summarizes summarizes site-specific design criteria obtained from the 2016 California Building Code (CBC; Based on the 2015 International Building Code [IBC] and ASCE 7-10), Chapter 16 Structural Design, Section 1613 Earthquake Loads. The data was calculated using the computer program *U.S. Seismic Design Maps*, provided by the USGS. The short spectral response uses a period of 0.2 second. We evaluated the Site Class based on the discussion in Section 1613.3.2 of the 2016 CBC and Table 20.3-1 of ASCE 7-10. The values presented below are for the risk-targeted maximum considered earthquake (MCE_R).

2016 CBC SEISMIC DESIGN PARAMETERS

Parameter	Value	2016 CBC Reference
Site Class	D	Table 1613.3.2
MCE_R Ground Motion Spectral Response Acceleration – Class B (short), S_S	2.013g	Figure 1613.3.1(1)
MCE_R Ground Motion Spectral Response Acceleration – Class B (1 sec), S_1	0.725g	Figure 1613.3.1(2)
Site Coefficient, F_A	1.0	Table 1613.3.3(1)
Site Coefficient, F_V	1.5	Table 1613.3.3(2)
Site Class Modified MCE_R Spectral Response Acceleration (short), S_{MS}	2.013g	Section 1613.3.3 (Eqn 16-37)
Site Class Modified MCE_R Spectral Response Acceleration – (1 sec), S_{M1}	1.087g	Section 1613.3.3 (Eqn 16-38)
5% Damped Design Spectral Response Acceleration (short), S_{DS}	1.342g	Section 1613.3.4 (Eqn 16-39)
5% Damped Design Spectral Response Acceleration (1 sec), S_{D1}	0.725g	Section 1613.3.4 (Eqn 16-40)

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 7-10.

ASCE 7-10 PEAK GROUND ACCELERATION

Parameter	Value	ASCE 7-10 Reference
Mapped MCE_G Peak Ground Acceleration, PGA	0.733g	Figure 22-7
Site Coefficient, F_{PGA}	1.0	Table 11.8-1
Site Class Modified MCE_G Peak Ground Acceleration, PGA_M	0.733g	Section 11.8.3 (Eqn 11.8-1)

The Maximum Considered Earthquake Ground Motion (MCE) is the level of ground motion that has a 2 percent chance of exceedance in 50 years, with a statistical return period of 2,475 years. According to the 2016 California Building Code and ASCE 7-10, the MCE is to be utilized for the evaluation of liquefaction, lateral spreading, seismic settlements, and it is our understanding that the intent of the Building code is to maintain “Life Safety” during a MCE event. The Design Earthquake Ground Motion (DE) is the level of ground motion that has a 10 percent chance of exceedance in 50 years, with a statistical return period of 475 years.

Deaggregation of the MCE peak ground acceleration was performed using the USGS online BETA Unified Hazard Tool, 2008 Conterminous U.S. Dynamic edition. The result of the deaggregation analysis indicates that the predominant earthquake contributing to the MCE peak ground acceleration is characterized as a 6.76 magnitude event occurring at a hypocentral distance of 9.0 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 6.71 magnitude occurring at a hypocentral distance of 14.29 kilometers from the site.

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

7.4 Liquefaction Potential

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

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

Based on review of geologic maps of the area and the geologic units encountered in the borings, the site is underlain by Holocene age alluvial deposits. A review of the Seismic Hazard Zone Map for the Hollywood Quadrangle (CGS, 2014; CDMG, 1999) indicates that the site is located in an area designated as having a potential for liquefaction.

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

Screening criteria presented by Bray and Sancio (2006) was used to evaluate the liquefaction susceptibility of the fine-grained soils encountered in the boring. Based on these screening criteria, fine-grained soils with a plasticity index of greater than 18 and fine-grained soils with a plasticity index of greater than 12 and a saturated water content of less than 80 percent of the liquid limit are considered not susceptible to liquefaction. Laboratory test results used for the screening criteria are presented as Figures B36 and B39.

The liquefaction analysis was performed for a Design Earthquake level by using a historic high groundwater table of 10 feet below the ground surface, a magnitude 6.71 earthquake, and a peak horizontal acceleration of 0.489g (⅔PGA_{M}). The enclosed liquefaction analyses, included herein for borings B1 and B7, indicate that the alluvial soils below the historic high groundwater could be prone to up to approximately 2.6 inches of liquefaction induced settlement during Design Earthquake ground motion (see enclosed calculation sheets, Figures 6 through 9).

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.

Liquefaction analyses was also performed for the Maximum Considered Earthquake level by using an assumed groundwater level of 10 feet below the ground surface, a magnitude 6.76 earthquake, and a peak horizontal acceleration of 0.733g (PGA_{M}). The enclosed liquefaction analyses, included herein for borings B1 and B7, indicate that the alluvial soils below the historic high groundwater could be prone to up to approximately 2.6 inches of liquefaction induced settlement during Maximum Considered Earthquake ground motion (see enclosed calculation sheets, Figures 10 through 13).

7.5 Slope Stability

Topography at the site is relatively level. The site is not located within a City of Los Angeles Hillside Ordinance Area or a Hillside Grading Area (City of Los Angeles, 2019). The County of Los Angeles Safety Element (Leighton, 1990), indicates the site is not located within an area identified as a "Hillside Area" or an area having a potential for slope instability. Additionally, the site is not located within an area identified as having a potential for seismic slope instability (CGS, 2014; CDMG, 1999). The closest slope to the site is an ascending slope on the north side of the Baldwin Hills, located over 1 mile to the southwest. There are no known landslides near the site, nor is the site in the path of any known or potential landslides. Therefore, the probability of slope stability hazards affecting the site is considered very low.

7.6 Earthquake-Induced Flooding

Earthquake-induced flooding is inundation caused by failure of dams or other water-retaining structures due to earthquakes. The Los Angeles County Safety Element (Leighton, 1990) indicates that the site is located within the Hansen Dam inundation area. However, this reservoir, as well as others in California, are continually monitored by various governmental agencies (such as the State of California Division of Safety of Dams and the U.S. Army Corps of Engineers) to guard against the threat of dam failure. Current design, construction practices, and ongoing programs of review, modification, or total reconstruction of existing dams are intended to ensure that all dams are capable of withstanding the maximum considered earthquake (MCE) for the site. Therefore, the potential for inundation at the site as a result of an earthquake-induced dam failure is considered low.

7.7 Tsunamis, Seiches, and Flooding

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

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

The site is within a Flood Zone X (0.2%) as defined by the Federal Emergency Management Agency (FEMA, 2019; LACDPW, 2019b). Zone X (0.2%) is defined as an area with a 0.2% chance of flooding on an annual basis (LACDPW, 2019b).

7.8 Oil Fields & Methane Potential

Based on a review of the California Division of Oil, Gas and Geothermal Resources (DOGGR) Well Finder Website, the site is not located within the limits of an oilfield and oil or gas wells are not located in the immediate site vicinity (DOGGR, 2019). However, due to the voluntary nature of record reporting by the oil well drilling companies, wells may be improperly located or not shown on the location map and undocumented wells could be encountered during construction. Any wells encountered during construction will need to be properly abandoned in accordance with the current requirements of the DOGGR.

The site is not located within the boundaries of a city-designated Methane Zone or Methane Buffer Zone (City of Los Angeles, 2019). Since the site is not located within the boundaries of a known oil field, the potential for the presence of methane or other volatile gases at the site is considered low. However, should it be determined that a methane study is required for the proposed development it is recommended that a qualified methane consultant be retained to perform the study and provide mitigation measures as necessary.

7.9 Subsidence

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

8. CONCLUSIONS AND RECOMMENDATIONS

8.1 General

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

8.1.2 Up to 5½ feet of existing artificial fill was encountered during the site investigation. The existing fill encountered is believed to be the result of past grading and construction activities at the site. Deeper fill may exist in other areas of the site that were not directly explored. Future demolition of the existing structures which occupy the site will likely disturb the upper few feet of existing site soils. It is our opinion that the existing artificial fill, in its present condition, is not considered suitable for direct support of proposed new foundations or slabs; however, the existing site soils are suitable for re-use as engineered fill provided the recommendations in the *Grading* section of this report are followed (see Section 8.5).

8.1.3 The liquefaction analyses indicate that the alluvial soils below the historic high groundwater depth could be prone to approximately 2.6 inches of settlement as a result of the Design Earthquake peak ground acceleration ($\frac{2}{3}\mathrm{PGA}_\mathrm{M}$). The resulting differential settlement at the ground surface is anticipated to be approximately half of the total settlement, or 1.3 inches of settlement over a distance of 20 feet. The grading and foundation recommendations presented herein are intended to reduce the effects of settlement on proposed improvements.

8.1.4 The upper 25 to 30 feet of alluvial soils consist of very soft to soft silts and clays and loose to medium dense sand layers. Based on laboratory testing (see Figures B12 to B35), the alluvium is moderately to highly compressible. Additionally, organic deposits were noted in the borings between the depths of 20 to 25 feet. Based on the presence of the compressible soils and organic deposits, as well as the potential for liquefaction, the use of a conventional foundation system and a mat foundation system is not considered feasible.

8.1.5 It is recommended that the proposed structures be supported on a deepened foundation system deriving support in the competent alluvial soils found at and below a depth of 30 feet. Recommendations for the design and construction of drilled, cast-in-place friction piles and end-bearing piles are provided in Sections 8.7 through 8.10. Alternate deep foundation systems, such as Auger-Cast Pressure Grouted Displacement (APGD) piles may also be feasible. The APGD system has the benefit of not generating soil spoils; however, the City of Los Angeles will require a comprehensive load testing program, as well as complete removal of one APGD test pile. As the design progresses, recommendations regarding alternate deep foundation design can be provided under separate cover, if desired.

8.1.6 Portions of the proposed structures will be located within the MTA Crenshaw/Exposition Station box surcharge zone. The surcharge zone may be defined by a 1:1 projection up and away from the bottom of the MTA station box foundation (see Section A-A', Figure 2). Where located within the MTA surcharge influence zone, the proposed piles may need to be deepened to extend below the surcharge influence zone. Furthermore, where located in very close proximity to existing MTA structures, the lateral surcharge imposed by proposed piles will need to be evaluated. It is suggested that a meeting with MTA engineers be requested to discuss the foundation design and any allowable vertical or horizontal surcharge loads.

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

8.1.8 Groundwater was encountered during site exploration at depths ranging from 18 to 20 feet below existing ground surface; however, the current depth to groundwater may not be representative of static conditions due to the dewatering operations associated with the Metro construction. Groundwater was previously encountered at an adjacent site at depths ranging from 11 to 13 feet below the ground surface. Based on these considerations, it is likely that groundwater will be encountered during construction. Temporary dewatering measures may be required to control groundwater seepage during excavation and construction. Recommendations for a *Temporary Dewatering* system are provided in Section 8.4

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

8.1.10 Excavation for construction of the proposed subterranean levels is anticipated to extend to depths of 12 feet, including foundation excavations. Due to the depth of the excavation and the proximity to the property lines, city streets, substructures, and adjacent offsite structures, excavation will likely require sloping and/or shoring measures in order to provide a stable excavation. Where shoring is required, it is recommended that a soldier pile shoring system be utilized. In addition, where the proposed excavation will be deeper than and adjacent to 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 8.20 of this report.

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

8.1.12 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.

8.1.13 It is suggested that flexible utility connections be considered for all rigid utilities tied into pile supported structures in order to minimize damage to utilities from minor differential soil movements, or potentially larger movements caused by an earthquake event.

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

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

8.1.16 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.

8.2 Soil and Excavation Characteristics

8.2.1 The in-situ soils can be excavated with moderate effort using conventional excavation equipment. Caving should be anticipated in unshored excavations, especially where granular or saturated soils are encountered.

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

8.2.3 All onsite excavations must be conducted in such a manner that potential surcharges from existing structures, construction equipment, and existing foundation supports 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. Excavation recommendations are provided in the *Temporary Excavations* section of this report (see Section 8.19).

8.2.4 The soils encountered for the upper 5 feet of site soils have a "low to medium" expansive potential (EI = 39 to 75), which are classified as "expansive" in accordance with the 2016 California Building Code (CBC) Section 1803.5.3. Based on the depth of the proposed subterranean level for Site B, the proposed structure would not be prone to the effects of expansive soils. The recommendations presented herein assume that near-surface foundations and slabs will derive support in materials with a "medium" expansive potential.

8.3 Minimum Resistivity, pH, and Water-Soluble Sulfate *(In-Progress)*

8.3.1 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 corrosive" with respect to corrosion of buried ferrous metals on site. The results are presented in Appendix B (Figure B43) and should be considered for design of underground structures. Due to the corrosive potential of the soils, it is suggested that ABS pipes be considered in lieu of cast-iron for subdrains and retaining wall drains beneath the structure.

8.3.2 Laboratory tests were performed on representative samples of the near-surface site materials to measure the percentage of water-soluble sulfate content. Results from the laboratory water-soluble sulfate tests are presented in Appendix B (Figure B43) and indicate that the near-surface materials possess a sulfate exposure class of "S0" to concrete structures as defined by 2016 CBC Section 1904 and ACI 318-14 Table 19.3.1.1.

8.3.3 Geocon West, Inc. does not practice in the field of corrosion engineering 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.

8.4 Temporary Dewatering

8.4.1 Groundwater was encountered during site exploration at depths ranging from 18 to 20 feet below existing ground surface; however, the current depth to groundwater may not be representative of static conditions due to the dewatering operations associated with the Metro construction. Groundwater was previously encountered at an adjacent site at depths ranging from 11 to 13 feet below the ground surface. The depth to groundwater at the time of construction can be further verified during shoring pile installation. If groundwater is present above the depth of the proposed excavation, temporary dewatering will be necessary to maintain a safe and efficient working environment during excavation and construction activities

8.2.2 It is recommended that a qualified dewatering consultant be retained to design the dewatering system and determine the design flow rates for dewatering. The dewatering consultant should also provide the minimum depth that the temporary dewatering be effective to, and also the potential effects of temporary dewatering on adjacent structures and the public right of way. Temporary dewatering may consist of perimeter wells with interior well points as well as gravel filled trenches (French drains) placed adjacent to the shoring system and interior of the site. The number and locations of the wells or French drains can be adjusted during excavation activities as necessary to collect and control any encountered seepage. The French drains will then direct the collected seepage to a sump where it will be pumped out of the excavation.

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

8.5 Grading

8.5.1 Grading is anticipated to include preparation of the subgrade, the excavation of site soils for the subterranean level on Site B, excavation for proposed foundations and utility trenches, as well as placement of backfill for walls, ramps, and trenches.

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

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

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

8.5.5 The proposed structures for Site A and Site B may be supported on a deepened foundation system that penetrates through the compressible alluvial soils and organic deposits and derives support in the competent alluvial soils found at or below a depth of 30 feet. The concrete slab for the pile-supported structures should be designed as a structural slab that derives all support from the pile, eliminating permanent reliance on the underlying soil. It is recommended that the upper 12 inches of slab subgrade be compacted to provide a suitable temporary surface upon which concrete can be poured and placed. Any disturbed soils should be properly compacted prior to slab construction. All foundation excavations must be observed and approved by the Geotechnical Engineer (a representative of Geocon), prior to placing steel or concrete.

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

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

8.5.8 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 placed as fill have less than 15 percent finer than 0.005 millimeters. Soils with more than 15 percent finer than 0.005 millimeters may be compacted to 90 percent of the laboratory maximum dry density in accordance with ASTM D 1557 (latest edition). All fill and backfill soils should be placed in horizontal loose layers approximately 6 to 8 inches thick, moisture conditioned to at least 2 percent above optimum moisture content, and properly compacted to the required degree of compaction in accordance with ASTM D 1557 (latest edition).

8.5.9 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 50 and soil corrosivity properties that are equally or less detrimental to that of the existing onsite soils (see Figure B43).

8.5.10 It is suggested that flexible utility connections be considered for all rigid utilities tied into pile supported structures in order to minimize or prevent damage to utilities from minor differential soil movements, or potentially larger movements caused by an earthquake event. Utility trenches should be properly backfilled in accordance with the requirements of the Green Book (latest edition). The pipe should be bedded with clean sands (Sand Equivalent greater than 30) to a depth of at least 1 foot over the pipe, and the bedding material must be observed and approved in writing by the Geotechnical Engineer (a representative of Geocon). The use of gravel is not acceptable unless used in conjunction with filter fabric to prevent the gravel from having direct contact with soil. The remainder of the trench backfill may be derived from onsite soil or approved import soil, compacted as necessary, until the required compaction is obtained. The use of minimum 2-sack slurry is also acceptable as backfill (see Section 8.6). Prior to placing any bedding materials or pipes, the excavation bottom must be observed and approved in writing by the Geotechnical Engineer (a representative of Geocon).

8.5.11 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 materials, fill, steel, gravel or concrete.

8.6 Controlled Low Strength Material (CLSM)

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

Standard Requirements

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

Requirements for CLSM that will be used for support of footings

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

8.7 Foundation Design – General

8.7.1 Portions of the proposed structures will be located within the MTA Crenshaw/Exposition Station box surcharge zone. The surcharge zone may be defined by a 1:1 projection up and away from the bottom of the MTA station box foundation (see Section A-A', Figure 2). Where located within the MTA surcharge influence zone, proposed foundations may need to be deepened to extend below the surcharge influence zone. Furthermore, where located in very close proximity to existing MTA structures, the lateral surcharge imposed by proposed piles will need to be evaluated. It is suggested that a meeting with MTA engineers be requested to discuss the foundation design and any allowable vertical or horizontal surcharge loads.

8.7.2 If the portion of the proposed structure which extends below the historic high groundwater table is to be designed for full hydrostatic pressure, the recommended floor slab uplift pressure to be used in design would be 62.4(H) in units of psf, where "H" is the height of the water above the bottom of the mat foundation in feet. If a permanent dewatering system is not implemented then the structure must be designed for hydrostatic pressure based on the historic high groundwater of 10 feet below ground surface.

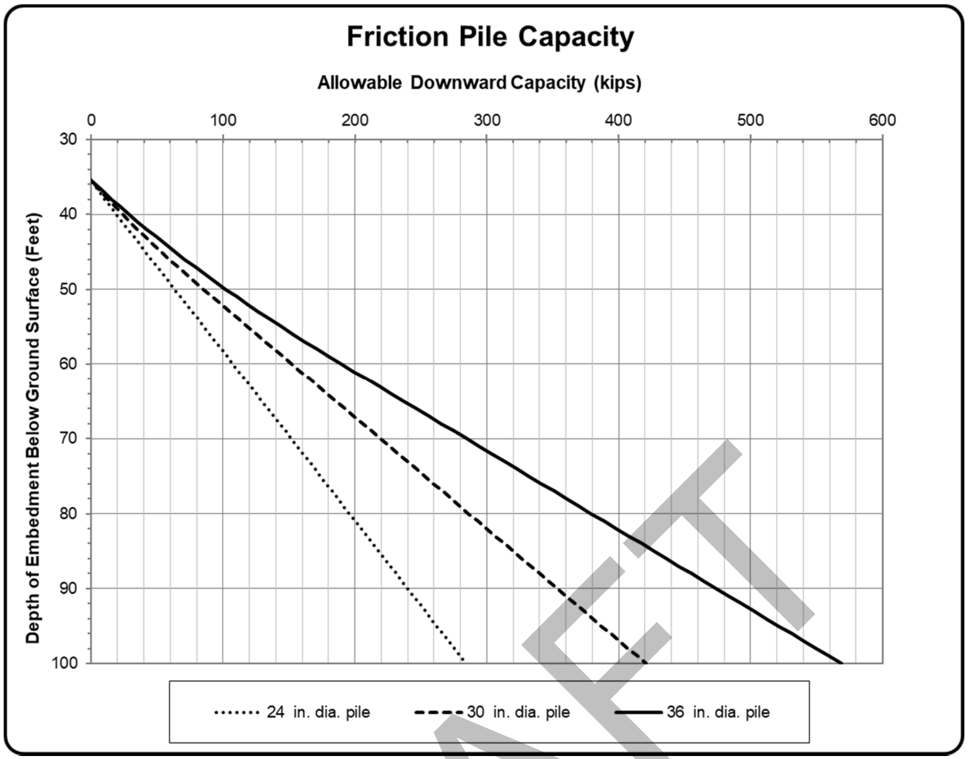
8.7.3 Once proposed foundation depths and building loads are available, additional analyses may be required to evaluate the anticipated total and differential settlements between the foundation elements. Updated foundation design recommendations will be provided as necessary in an addendum report.

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

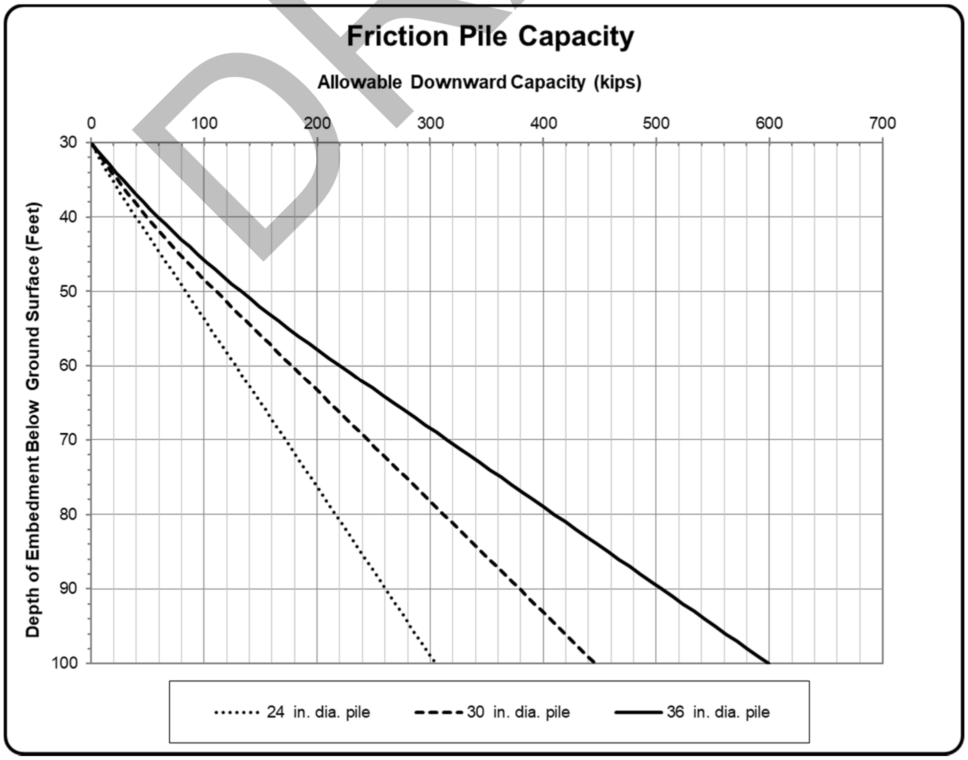
8.8 Friction Pile Design

8.8.1 For preliminary design purposes 24-, 30-, and 36-inch diameter drilled cast-in-place friction piles have been evaluated. Friction piles should be embedded a minimum of 15 feet into the competent alluvium found at and below a depth of 30 feet. The allowable axial capacities for pile embedment into the competent alluvial soils are provided in the charts below. The axial capacities are based on skin friction; end-bearing capacity is not being considered. The axial capacities also include consideration of downdrag forces due to consolidation of the overlying compressible soils as well as downdrag from liquefiable soils.

8.8.2 Friction piles supporting the proposed on-grade structure at Site A may use the capacities presented in the chart on the following page.



8.8.3 Friction piles supporting the proposed subterranean structure at Site B may use the capacities presented in the following chart.



8.8.4 All drilled pile excavations should be continuously observed by personnel of this firm to verify adequate penetration into the recommended bearing materials. The capacity presented is based on the strength of the soils. The compressive and tensile strength of the pile sections should be checked to verify the structural capacity of the piles.

8.8.5 Single pile uplift capacity can be taken as 60 percent of the allowable downward capacity.

8.8.6 The allowable downward capacity and allowable uplift capacity may be increased by one-third when considering transient wind or seismic loads.

8.8.7 The maximum expected static settlement for the structure supported on friction piles is estimated to be less than ½ inch. Differential settlement between adjacent pile foundations is not expected to exceed ¼ inch. The majority of the foundation settlement is expected to occur on initial application of loading and during construction.

8.8.8 For increased resistance to differential foundation movement and lateral drift, the pile tops should be interconnected in two horizontal directions with grade beams or tied with a structural slab. The project structural engineer should provide slab and grade beam design, reinforcement and spacing dependent on anticipated loading. However, for grade beams we recommend a minimum embedment depth below lowest adjacent pad grade of 24 inches and a minimum width of 12 inches. In addition, minimum reinforcement should consist of four No. 4 steel reinforcing bars; two placed near the top of the grade beam and two near the bottom.

8.8.9 If pile spacing is at least three times the maximum dimension of the pile, no reduction in axial capacity is considered necessary for group effects. If pile spacing is closer than three pile diameters, an evaluation for group effects including appropriate reductions should be performed by Geocon based on pile dimension and spacing.

8.9 End-Bearing Caissons

8.9.1 Drilled, cast-in-placed end-bearing caissons may also be used to support proposed improvements provided the foundations derive support in the competent alluvium found at or below a depth of 30 feet. Drilled, cast-in-place end-bearing concrete caissons should be a minimum of 18 inches in diameter. For preliminary design purposes 18-, 24-, and 30-inch diameter drilled cast-in-place end-bearing piles have been evaluated. Piles should be embedded a minimum of 5 feet into the competent alluvial soils. The allowable axial capacities for pile embedment into the competent alluvial soils are provided in the chart on the following page. The axial capacities are based on skin friction; end-bearing capacity is not being considered. The axial capacities also include consideration of downdrag forces due to consolidation of the overlying compressible soils as well as downdrag from liquefiable soils.

8.9.2 End-bearing caissons supporting the proposed on-grade structure at Site A may use the capacities presented in the following table.

Pile Diameter (inches)	Depth below Ground Surface (ft)	Pile Capacity (kips)
18	35	105
24	35	230
	40	250
30	35	350
	50	500

8.9.3 End-bearing caissons supporting the proposed subterranean level at Site B may use the capacities presented in the following table.

Pile Diameter (inches)	Depth below Ground Surface (ft)	Pile Capacity (kips)
18	35	120
24	35	250
	40	280
30	35	390
	50	540

8.9.4 All drilled pile excavations should be continuously observed by personnel of this firm to verify adequate penetration into the recommended bearing materials. The capacity presented is based on the strength of the soils. The compressive and tensile strength of the pile sections should be checked to verify the structural capacity of the piles.

8.9.5 Single pile uplift capacity can be taken as 60 percent of the allowable downward frictional capacity (see Section 8.8).

8.9.6 The allowable downward capacity and allowable uplift capacity may be increased by one-third when considering transient wind or seismic loads.

8.9.7 The maximum expected static settlement for the structure supported on end-bearing piles is estimated to be less than 1 inch. Differential settlement between adjacent pile foundations is not expected to exceed ½ inch. The majority of the foundation settlement is expected to occur on initial application of loading and during construction.

8.9.8 For increased resistance to differential foundation movement and lateral drift, the pile tops should be interconnected in two horizontal directions with grade beams or tied with a structural slab. The project structural engineer should provide slab and grade beam design, reinforcement and spacing dependent on anticipated loading. However, for grade beams we recommend a minimum embedment depth below lowest adjacent pad grade of 24 inches and a minimum width of 12 inches. In addition, minimum reinforcement should consist of four No. 4 steel reinforcing bars; two placed near the top of the grade beam and two near the bottom.

8.9.9 If pile spacing is at least three times the maximum dimension of the pile, no reduction in axial capacity is considered necessary for group effects. If pile spacing is closer than three pile diameters, an evaluation for group effects including appropriate reductions should be performed by Geocon based on pile dimension and spacing.

8.9.10 All loose soils must be completely removed from the bottom of all end-bearing foundation excavations and approved in writing by the Geotechnical Engineer (a representative of Geocon West, Inc.).

8.10 Deepened Foundation Installation

8.10.1 Casing may be required if caving occurs in the granular soil layers during deep drilled excavation. The contractor should have casing available and should be prepared to use it. If casing is used, extreme care should be employed so that the pile is not pulled apart as the casing is withdrawn. At no time should the distance between the surface of the concrete and the bottom of the casing be less than 5 feet. Continuous observation of the drilling and pouring of the piles by the Geotechnical Engineer (a representative of Geocon West, Inc.), is required.

8.10.2 Friction piles do not require the complete removal of all loose earth materials from the bottom of the excavation since the end-bearing capacity is not being considered for design. However, a cleanout of the excavation bottom will be required. Where end-bearing caissons are used, all loose soils must be completely removed. 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.

8.10.3 Groundwater was encountered at depths ranging from 18 to 20 feet below existing ground surface. The contractor should be prepared for groundwater during pile installation. Piles placed below the water level require the use of a tremie to place the concrete into the bottom of the hole. A tremie should consist of a rigid, water-tight tube having a diameter of not less than 6 inches with a hopper at the top. The tube should be equipped with a device that will close the discharge end and prevent water from entering the tube while it is being charged with concrete. The tremie should be supported so as to permit free movement of the discharge end over the entire top surface of the work and to permit rapid lowering when necessary to retard or stop the flow of concrete. The discharge end should be closed at the start of the work to prevent water entering the tube and should be entirely sealed at all times, except when the concrete is being placed. The tremie tube should be kept full of concrete. The flow should be continuous until the work is completed and the resulting concrete seal should be monolithic and homogeneous. The tip of the tremie tube should always be kept about 5 feet below the surface of the concrete and definite steps and safeguards should be taken to insure that the tip of the tremie tube is never raised above the surface of the concrete.

8.10.4 A special concrete mix should be used for concrete to be placed below water. The design shall provide for concrete with a strength of 1,000 psi over the initial job specification. An admixture that reduces the problem of segregation of paste/aggregates and dilution of paste shall be included. The slump shall be commensurate to any research report for the admixture, provided that it shall also be the minimum for a reasonable consistency for placing when water is present. Extreme care should be employed so that the pile is not pulled apart as the casing is withdrawn. At no time should the distance between the surface of the concrete and the bottom of the casing be less than 5 feet. Continuous observation of the drilling and pouring of the piles by a representative of this firm is required.

8.10.5 Closely spaced piles should be drilled and filled alternately, with the concrete permitted to set at least eight hours before drilling an adjacent hole. Pile excavations should be filled with concrete as soon after drilling and inspection as possible; the holes should not be left open overnight.

8.11 Lateral Design

8.11.1 Resistance to lateral loading may be provided by friction acting at the base of foundations, slabs and by passive earth pressure. An allowable coefficient of friction of 0.35 may be used with the dead load forces in the undisturbed alluvial soils or properly compacted engineered fill.

8.11.2 Above the historically highest groundwater table of 10 feet, the passive earth pressure for the sides of foundations and slabs poured against undisturbed alluvial soils or properly compacted engineered fill may be taken as an equivalent fluid having a density of 240 pounds per cubic foot (pcf) with a maximum earth pressure of 2,400 pcf. Below the historically highest groundwater table, the passive earth pressure may be computed as an equivalent fluid having a density of 130 pcf with a maximum earth pressure of 1,300 pcf (values have been reduced for buoyancy). When combining passive and friction for lateral resistance, the passive component should be reduced by one-third. When combining passive and friction for lateral resistance, the passive component should be reduced by one-third.

8.11.3 Where foundations are situated adjacent to the subterranean MTA facilities the lateral component of the foundation should be ignored to prevent an appreciable surcharge on the existing facilities. The required lateral capacity can be accounted for by structural connections to other foundations that are outside of the defined surcharge area.

8.11.4 Ultimate lateral capacities for ¼ inch deflection of fixed and free-head drilled cast-in place piles are presented in the table below. No factors of safety have been applied to the lateral load values calculated to induce ¼-inch lateral deflection. Lateral capacities provided are for 24-, 30-, and 36-inch diameter drilled cast-in-place concrete piles, penetrating the earth materials encountered during the course of this investigation. Assumed as part of these lateral capacity calculations are a concrete modulus of elasticity of at least 3,000,000 psi.

LATERAL LOAD CAPACITIES OF DRILLED CAST-IN-PLACE PILES

FIXED HEAD (NO HEAD ROTATION)

PILE NUMBER	PILE DIAMETER (INCHES)	Lateral Load Capacity "P" (KIPS)	Maximum Positive Moment "Mp" (LAT FORCE =P)	Maximum Negative Moment "Mp" (LAT FORCE =P)	Depth to Max Pos. Moment (Feet)	Depth to Zero Moment (Feet)	Depth to Inflection Point (Feet)	MINIMUM PILE LENGTH FOR APPLICABILITY OF LATERAL DESIGN DATA (FEET)
1	24	**43**	**1.4** P	**-5.1** P	**12**	**25**	**6.4**	**25**
2	30	**61**	**1.7** P	**-6.1** P	**15**	**30**	**7.6**	**30**
3	36	**81**	**1.9** P	**-7.1** P	**17**	**35**	**8.8**	**35**

FREE HEAD (HINGED)

PILE NUMBER	PILE DIAMETER (INCHES)	Lateral Load Capacity "P" (KIPS)	Maximum Moment "Mp" (LAT FORCE =P)	Depth to Zero Moment (Feet)	Depth to Maximum Moment (Feet)
1	24	**17**	**4.3** P	**23**	**7**
2	30	**25**	**5.2** P	**27**	**9**
3	36	**33**	**6.0** P	**31**	**10**

Lateral capacities are based on 1/4-inch deflection.
Moment magnitudes are presented as a function of the applied lateral load "P".
"P" is entered in units of kips and the moment magnitude will be in units of kip-feet.
The maximum negative moment is at the rigid, pile to pile cap or grade beam connection at the top of the pile.

8.11.5 Once the project design proceeds to a more finalized state and the foundation system has been selected, an LPile analysis of lateral pile capacity can be performed, if necessary. If piles are spaced at least at least 8 diameters on-center when loaded in-line and at least 3 diameters on-center when loaded in parallel, no reduction in lateral capacity is considered necessary for group effects. If pile spacing is closer, an evaluation for group effects including appropriate reductions should be incorporated into the pile design based on pile dimension, spacing, and the direction of loading.

8.12 Concrete Slabs-on-Grade

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

8.12.2 Slabs-on-grade at the ground surface that may receive moisture-sensitive floor coverings or may be used to store moisture-sensitive materials should be underlain by a vapor retarder placed directly beneath the slab. The vapor retarder and acceptable permeance should be specified by the project architect or developer based on the type of floor covering that will be installed. The vapor retarder 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 Los Angeles Green Building Code requirements apply to this project, the vapor retarder should be underlain by 4 inches of clean aggregate. It is important that the vapor retarder be puncture resistant since it will be in direct contact with angular gravel. As an alternative to the clean aggregate suggested in the Green Building Code, it is our opinion that the concrete slab-on-grade may be underlain by a vapor retarder over 4 inches of clean sand (sand equivalent greater than 30), since the sand will serve a capillary break and will minimize the potential for punctures and damage to the vapor barrier.

8.12.3 For seismic design purposes, an allowable coefficient of friction of 0.35 may be utilized between concrete slabs and subgrade soils; and 0.15 for slabs underlain by a vapor retarder.

8.12.4 Exterior slabs, not subject to traffic loads, should be at least 4 inches thick and reinforced with No. 3 steel reinforcing bars placed 18 inches on center in both horizontal directions, positioned near the slab midpoint. Prior to construction of slabs, the upper 12 inches of subgrade should be moisture conditioned to at least 2 percent above optimum moisture content and properly compacted to at least 95 percent relative compaction, as determined by ASTM Test Method D 1557 (latest edition). Crack control joints should be spaced at intervals not greater than 8 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.

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

8.13 Preliminary Pavement Recommendations

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

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

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

PRELIMINARY PAVEMENT DESIGN SECTIONS

Location	Estimated Traffic Index (TI)	Asphalt Concrete (inches)	Class 2 Aggregate Base (inches)
Automobile Parking And Driveways	4.0	3	4
Trash Truck & Fire Lanes	7.0	4	12

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

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

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

8.14 Retaining Walls Design

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

8.14.2 Retaining walls with a level backfill surface that are not restrained at the top should be designed utilizing a triangular distribution of pressure (active pressure). Restrained walls are those that are not allowed to rotate more than 0.001H (where H equals the height of the retaining portion of the wall in feet) at the top of the wall. Where walls are restrained from movement at the top, walls may be designed utilizing a triangular distribution of pressure (at-rest pressure). The table below presents recommended pressures to be used in retaining wall design, assuming that proper drainage will be maintained. Calculation of the recommended retaining wall pressures are provided as Figure 14.

RETAINING WALL WITH LEVEL BACKFILL SURFACE

HEIGHT OF RETAINING WALL (Feet)	ACTIVE PRESSURE EQUIVALENT FLUID PRESSURE (Pounds Per Cubic Foot)	AT-REST PRESSURE EQUIVALENT FLUID PRESSURE (Pounds Per Cubic Foot)
Up to 10	35	60

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

8.14.4 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.

8.14.5 Additional active pressure should be added for a surcharge condition due to sloping ground, vehicular traffic or adjacent structures and should be designed for each condition as the project progresses. Surcharges may be evaluated using Section 8.25 of this report. Once the design becomes more finalized, an addendum letter can be prepared revising recommendations and addressing specific surcharge conditions throughout the project, if necessary.

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

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

8.15 Dynamic (Seismic) Lateral Forces

8.15.1 The structural engineer should determine the seismic design category for the project in accordance with Section 1613 of the CBC. If the project possesses a seismic design category of D, E, or F, proposed retaining walls in excess of 6 feet in height should be designed with seismic lateral pressure (Section 1803.5.12 of the 2016 CBC).

8.15.2 A seismic load of 10 pcf should be used for design of walls that support more than 6 feet of backfill in accordance with Section 1803.5.12 of the 2016 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 7-10 Section 11.8.3.

8.16 Retaining Wall Drainage

8.16.1 Retaining walls should be provided with a drainage system. At the base of the drain system, a subdrain covered with a minimum of 12 inches of gravel should be installed, and a compacted fill blanket or other seal placed at the surface (see Figure 15). The clean bottom and subdrain pipe, behind a retaining wall, should be observed by the Geotechnical Engineer (a representative of Geocon), prior to placement of gravel or compacting backfill.

8.16.2 As an alternative, a plastic drainage composite such as Miradrain or equivalent may be installed in continuous, 4-foot wide columns along the entire back face of the wall, at 8 feet on center. The top of these drainage composite columns should terminate approximately 18 inches below the ground surface, where either hardscape or a minimum of 18 inches of relatively cohesive material should be placed as a cap (see Figure 16). These vertical columns of drainage material would then be connected at the bottom of the wall to a collection panel or a 1-cubic-foot rock pocket drained by a 4-inch subdrain pipe.

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

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

8.17 Elevator Pit Design

8.17.1 The elevator pit slab and retaining wall should be designed by the project structural engineer. As a minimum the slab-on-grade for the elevator pit bottom should be at least 4 inches thick and reinforced with No. 4 steel reinforcing bars placed 16 inches on center in both horizontal directions, positioned near the slab midpoint. Elevator pit walls may be designed in accordance with the recommendations in the *Deepened Foundation Design* and *Retaining Wall Design* sections of this report (see Section 8.7 through 8.10 and 8.14).

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

8.17.3 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 8.16).

8.17.4 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.

8.18 Elevator Piston

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

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

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

8.19 Temporary Excavations

8.19.1 Excavations on the order of 12 feet in height may be required for the excavation and construction of the proposed subterranean level. The excavations are expected to expose artificial fill and alluvial soils, which may be subject to caving where granular soils are exposed. Vertical excavations up to 5 feet in height may be attempted where not surcharged by adjacent traffic or structures.

8.19.2 Vertical excavations greater than 5 feet will require sloping and/or shoring measures in order to provide a stable excavation. Where sufficient space is available, temporary unsurcharged embankments up to 12 feet in height can be sloped back at a uniform 1:1 slope gradient or flatter. A uniform slope does not have a vertical portion. Where space is limited, shoring measures will be required. *Shoring* recommendations are provided in the following section.

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

8.20 Shoring – Soldier Pile Design and Installation

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

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

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

8.20.4 Drilled cast-in-place soldier piles should be placed no closer than 3 diameters on center. The minimum diameter of the piles is 18 inches. Structural concrete should be used for the soldier piles below the excavation; lean-mix concrete may be employed above that level. As an alternative, lean-mix concrete may be used throughout the pile where the reinforcing consists of a wideflange section. The slurry must be of sufficient strength to impart the lateral bearing pressure developed by the wideflange section to the soil. For design purposes, an allowable passive value for the soils below the bottom plane of excavation may be assumed to be 130 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 three times the pile diameter. To develop the full lateral value, provisions should be implemented to assure firm contact between the soldier piles and the undisturbed alluvium.

8.20.5 Groundwater was encountered during site exploration at depths ranging from 18 to 20 feet below existing ground surface; however, the current depth to groundwater may not be representative of static conditions due to the dewatering operations associated with the Metro construction. Groundwater was previously encountered at an adjacent site at depths ranging from 11 to 13 feet below the ground surface. Should groundwater or seepage be encountered, piles placed below the water level require the use of a tremie to place the concrete into the bottom of the hole. A tremie should consist of a rigid, water-tight tube having a diameter of not less than 6 inches with a hopper at the top. The tube should be equipped with a device that will close the discharge end and prevent water from entering the tube while it is being charged with concrete. The tremie should be supported so as to permit free movement of the discharge end over the entire top surface of the work and to permit rapid lowering when necessary to retard or stop the flow of concrete. The discharge end should be closed at the start of the work to prevent water entering the tube and should be entirely sealed at all times, except when the concrete is being placed. The tremie tube should be kept full of concrete. The flow should be continuous until the work is completed and the resulting concrete seal should be monolithic and homogeneous. The tip of the tremie tube should always be kept about 5 feet below the surface of the concrete and definite steps and safeguards should be taken to insure that the tip of the tremie tube is never raised above the surface of the concrete.

8.20.6 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 psi over the initial job specification. An admixture that reduces the problem of segregation of paste/aggregates and dilution of paste should be included. The slump should be commensurate to any research report for the admixture, provided that it should also be the minimum for a reasonable consistency for placing when water is present.

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

8.20.8 If a vibratory method of solider pile installation is utilized, predrilling may be performed prior to installation of the steel beams. If predrilling is performed, the bore diameter should be no greater than 75 percent of the largest dimension of the pile to prevent excessive loss in the frictional component of the pile capacity. Predrilling should not be conducted below the proposed excavation bottom, and the auger should be backspun out of the pilot holes, leaving the soil in place.

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

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

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

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

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

8.20.14 The frictional resistance between the soldier piles and retained soil may be used to resist the vertical component of the anchor load. The coefficient of friction may be taken as 0.35 based on uniform contact between the steel beam and lean-mix concrete and retained earth. The portion of soldier piles below the plane of excavation may also be employed to resist the downward loads. The downward capacity may be determined using a frictional resistance of 330 psf per foot (value has been reduced for buoyant forces).

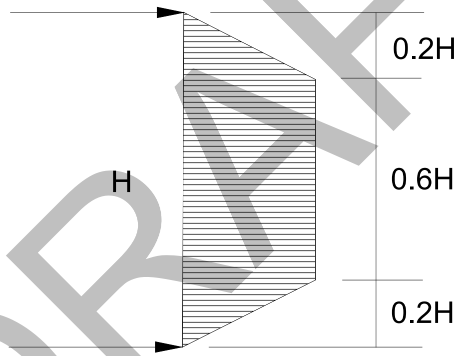
8.20.15 Due to the nature of the site soils, it is expected that continuous lagging between soldier piles will be required. However, it is recommended that the exposed soils be observed by the Geotechnical Engineer (a representative of Geocon West, Inc.), to verify the presence of any cohesive soils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

8.20.16 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.

8.20.17 For the design of unbraced shoring, it is recommended that an equivalent fluid pressure be utilized for design. A trapezoidal distribution of lateral earth pressure may be used where shoring will be restrained by bracing or tiebacks. The recommended active and trapezoidal pressure are provided in the following table. A diagram depicting the trapezoidal pressure distribution of lateral earth pressure is provided below the table. Calculation of the recommended shoring wall pressures are provided as Figure 17.

HEIGHT OF SHORING (FEET)	**EQUIVALENT FLUID PRESSURE (Pounds Per Cubic Foot) (ACTIVE PRESSURE)**	**EQUIVALENT FLUID PRESSURE (Pounds Per Square Foot per Foot) Active Trapezoidal (Where H is the height of the shoring in feet)**
Up to 12	29	18H

Trapezoidal Distribution of Pressure



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

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

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

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

8.20.22 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.

8.21 Temporary Tie-Back Anchors

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

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

- 5 feet below the top of the excavation – 400 pounds per square foot value has been reduced for buoyant forces

8.21.3 Depending on the techniques utilized, and the experience of the contractor performing the installation, a maximum allowable friction capacity of 1.5 kips per linear foot for post-grouted anchors (for a 20 foot length beyond the active wedge) may be assumed for design purposes. Only the frictional resistance developed beyond the active wedge should be utilized in resisting lateral loads.

8.22 Anchor Installation

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

8.23 Anchor Testing

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

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

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

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

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

8.24 Internal Bracing

8.24.1 Rakers may be utilized to brace the soldier piles in lieu of tieback anchors. The raker bracing could be supported laterally by temporary concrete footings (deadmen) or by the permanent, interior footings. For design of such temporary footings or deadmen, poured with the bearing surface normal to rakers inclined at 45 degrees, a bearing value of 2,000psf in the alluvial soils, provided the shallowest point of the footing is at least 1 foot below the lowest adjacent grade. The client should be aware that the utilization of rakers could significantly impact the construction schedule due to their intrusion into the construction site and potential interference with equipment. The structural engineer should review the shoring plan to determine if the raker footings conflict with the structural foundation system.

8.25 Surcharge from Adjacent Structures and Improvements

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

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

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

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

and

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

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

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

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

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

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

and

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

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

then

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

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

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

8.26 Surface Drainage

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

8.26.2 All site drainage should be collected and controlled in non-erosive drainage devices. Drainage should not be allowed to pond anywhere on the site, and especially not against any foundation or retaining wall. The site should be graded and maintained such that surface drainage is directed away from structures in accordance with 2016 CBC 1804.4 or other applicable standards. In addition, drainage should not be allowed to flow uncontrolled over any descending slope. Discharge from downspouts, roof drains and scuppers are not recommended onto unprotected soils within 5 feet of the building perimeter. Planters which are located adjacent to foundations should be sealed to prevent moisture intrusion into the soils providing foundation support. Landscape irrigation is not recommended within 5 feet of the building perimeter footings except when enclosed in protected planters.

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

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

8.27 Plan Review

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

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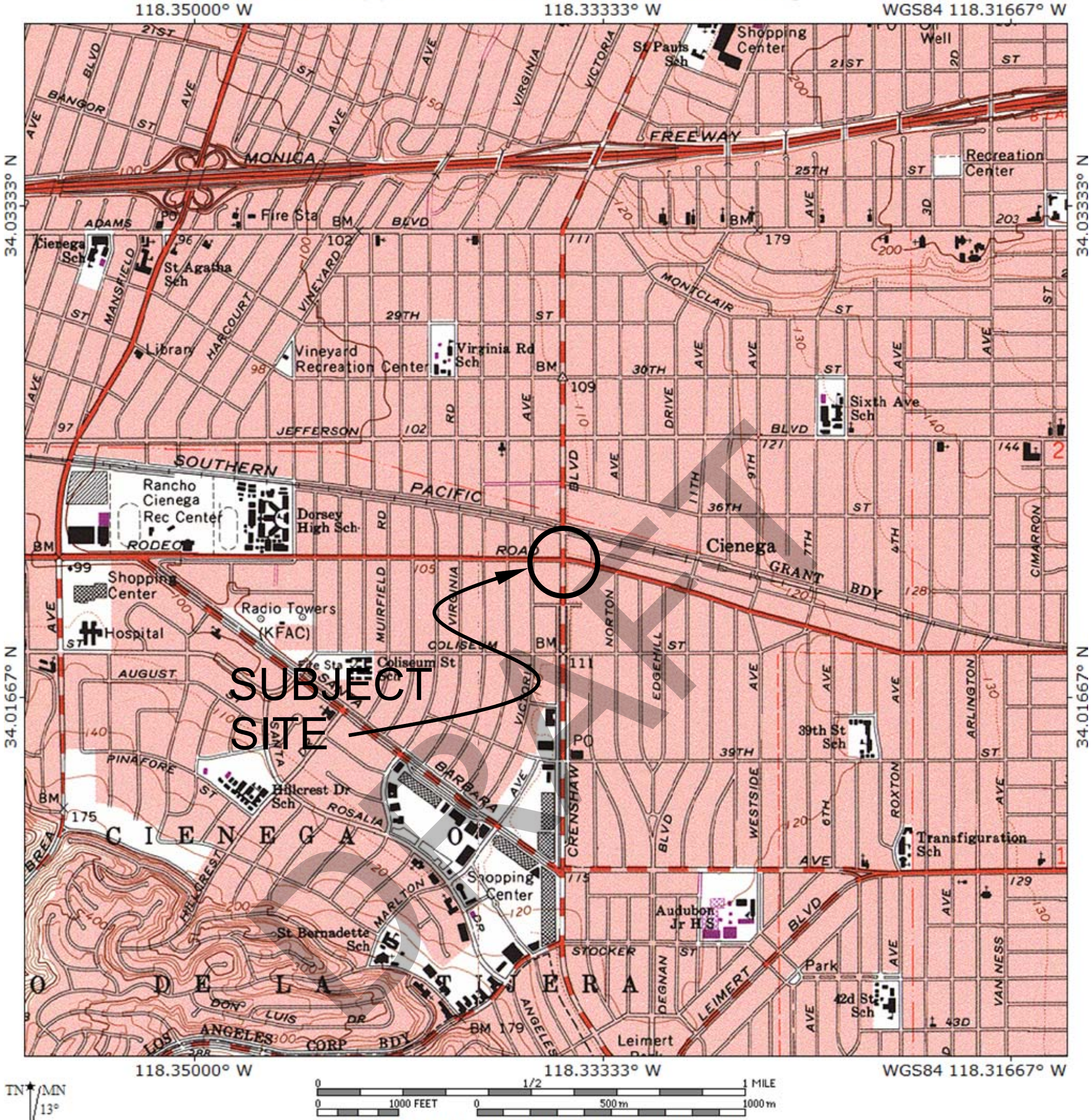

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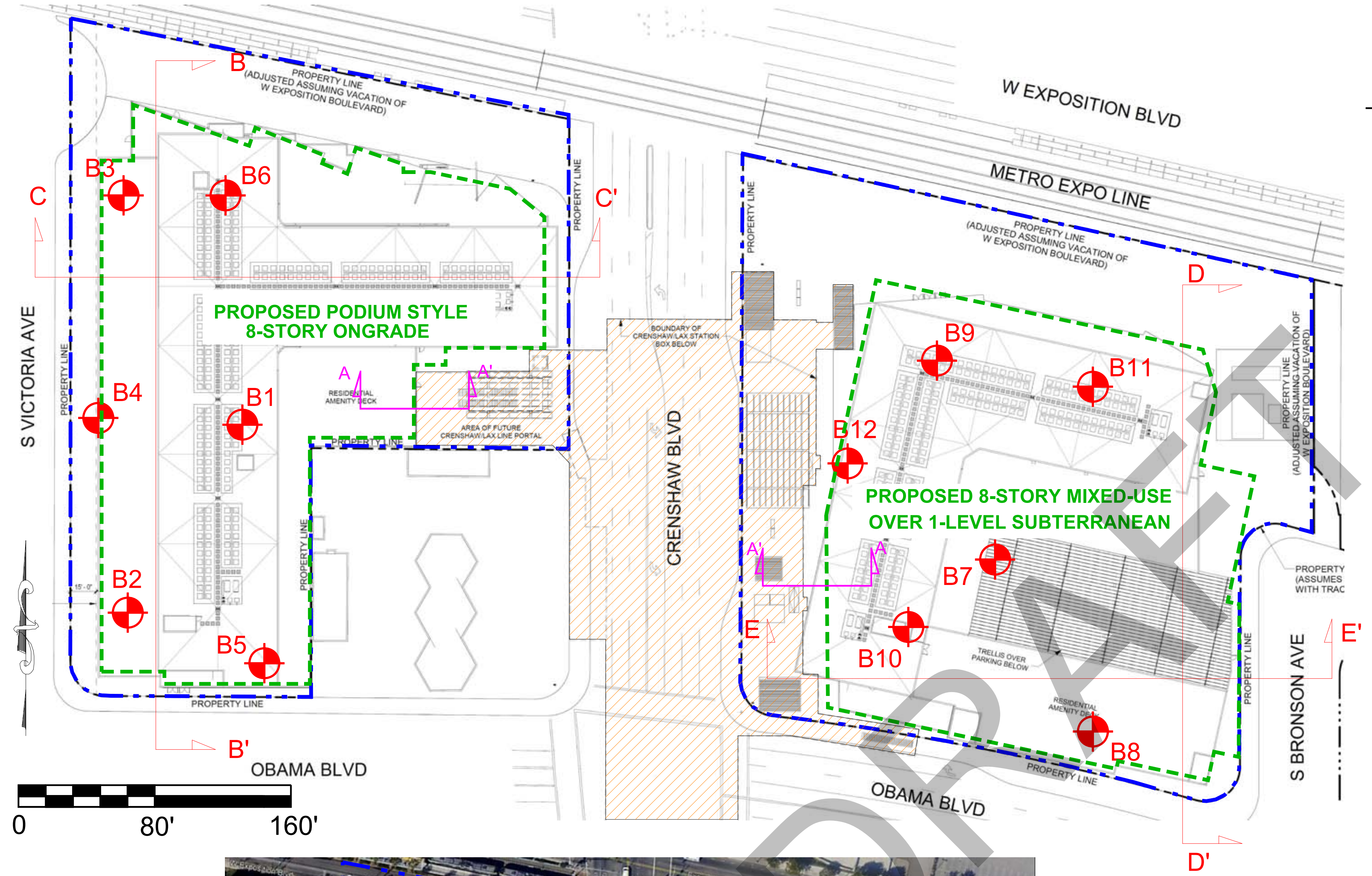
VICINITY MAP

SOUTHWEST AND SOUTHEAST CORNERS OF
EXPOSITION AND CRENSHAW BOULEVARD
LOS ANGELES, CALIFORNIA

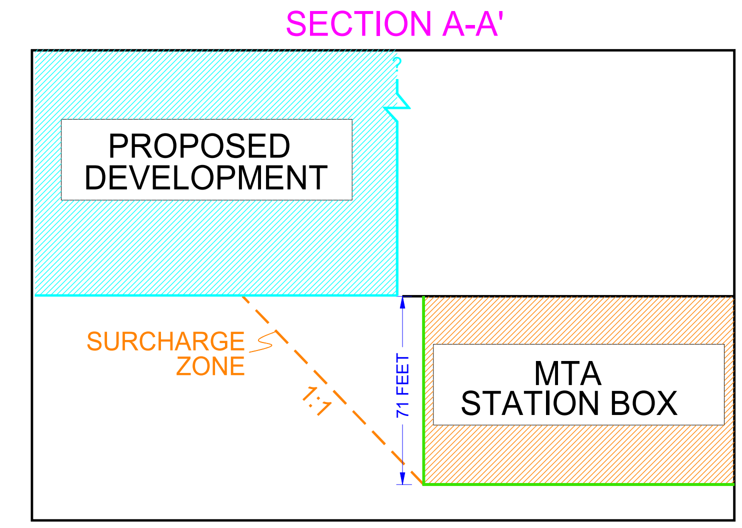
DRAFTED BY: RA	CHECKED BY: SFK	AUGUST 2019	PROJECT NO. A9930-06-01	FIG. 1

Note: Plans by Belzberg Architects and SVA Architects

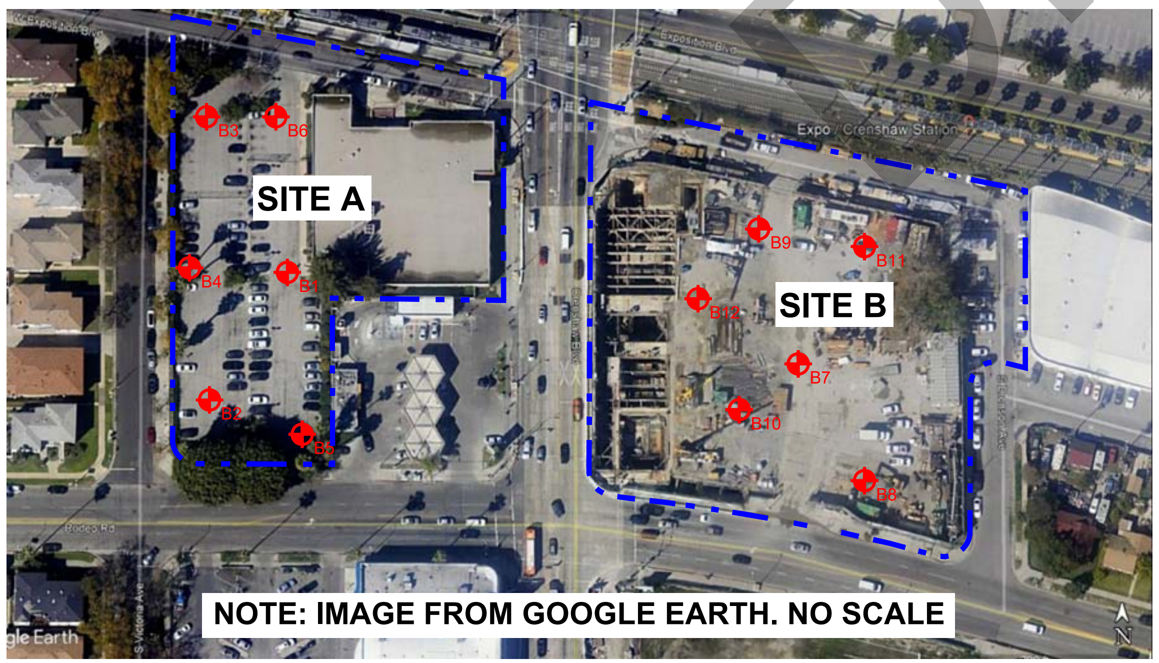


LEGEND

⊕ B12	Location and Number of Boring
▭	Approximate Property Boundary
▨	MTA Crenshaw/Exposition Station Box
▭	Approximate Footprint of Proposed Structure
E ⊔ E'	Cross Sections (See Figure 3)



FOR ILLUSTRATION PURPOSES ONLY
NO SCALE



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SITE PLAN

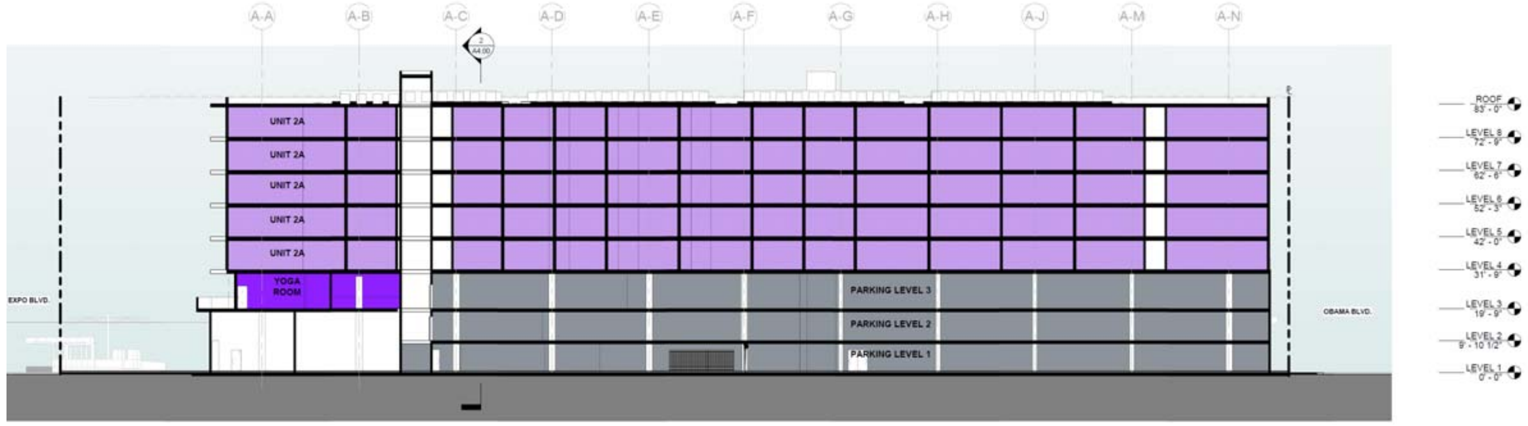
SOUTHWEST AND SOUTHEAST CORNERS OF
EXPOSITION AND CRENSHAW BOULEVARD
LOS ANGELES, CALIFORNIA

AUGUST 2019 | PROJECT NO. A9930-06-01 | FIG. 2

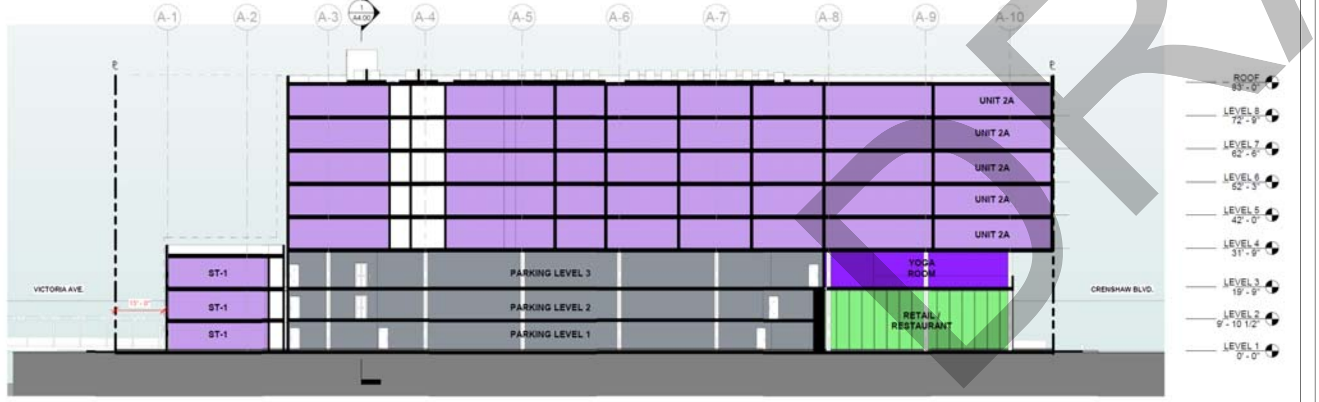
Note: Plans by Belzberg Architects and SVA Architects

SITE A

CROSS SECTION B - B'

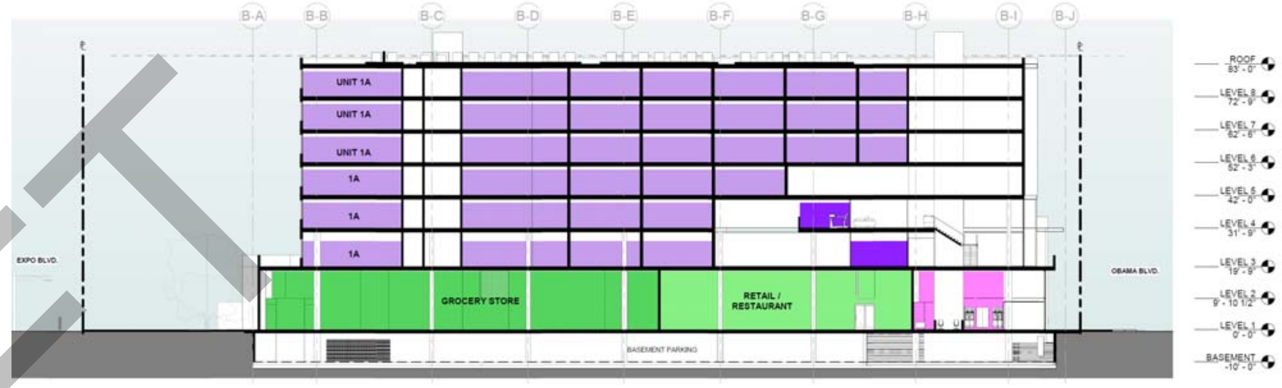


CROSS SECTION C - C'

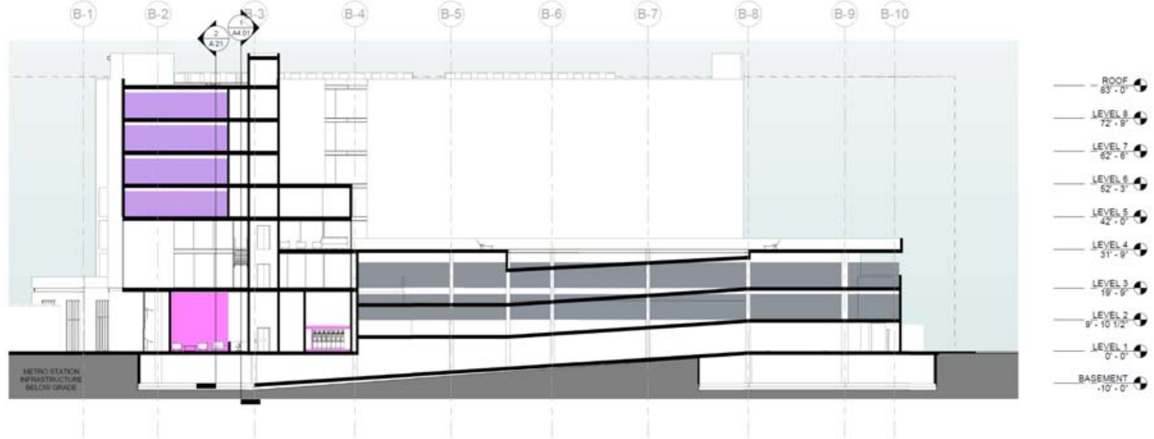


SITE B

CROSS SECTION D - D'



CROSS SECTION E - E'



NOTE: CROSS-SECTIONS ARE FOR ILLUSTRATION PURPOSES ONLY. NOT FOR CONSTRUCTION.

0 60' 120'

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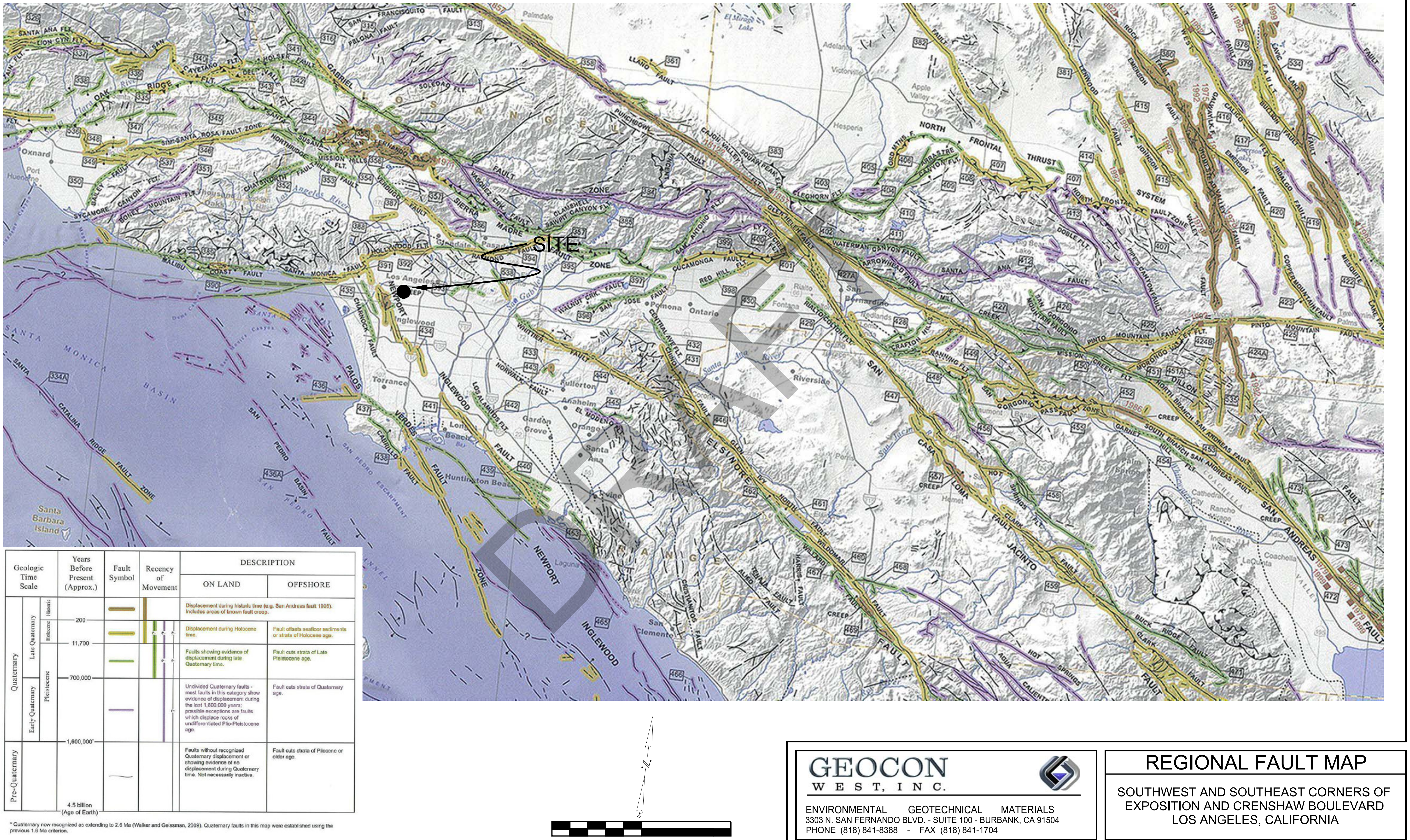
CROSS SECTIONS

SOUTHWEST AND SOUTHEAST CORNERS OF
EXPOSITION AND CRENSHAW BOULEVARD
LOS ANGELES, CALIFORNIA

AUGUST 2019
PROJECT NO. A9930-06-01
FIG. 3

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.	
		Holocene	200	▬	? ? ?	Displacement during Holocene time.	Fault offsets seafloor sediments or strata of Holocene age.
		Pleistocene	11,700	▬	? ?	Faults showing evidence of displacement during late Quaternary time.	Fault cuts strata of Late Pleistocene age.
	Early Quaternary		700,000	▬	?	Undivided Quaternary faults - most faults in this category show evidence of displacement during the last 1,600,000 years; possible exceptions are faults which displace rocks of undifferentiated Plio-Pleistocene age.	Fault cuts strata of Quaternary age.
Pre-Quaternary			1,600,000* 4.5 billion (Age of Earth)	—		Faults without recognized Quaternary displacement or showing evidence of no displacement during Quaternary time. Not necessarily inactive.	Fault cuts strata of Pliocene or older age.

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

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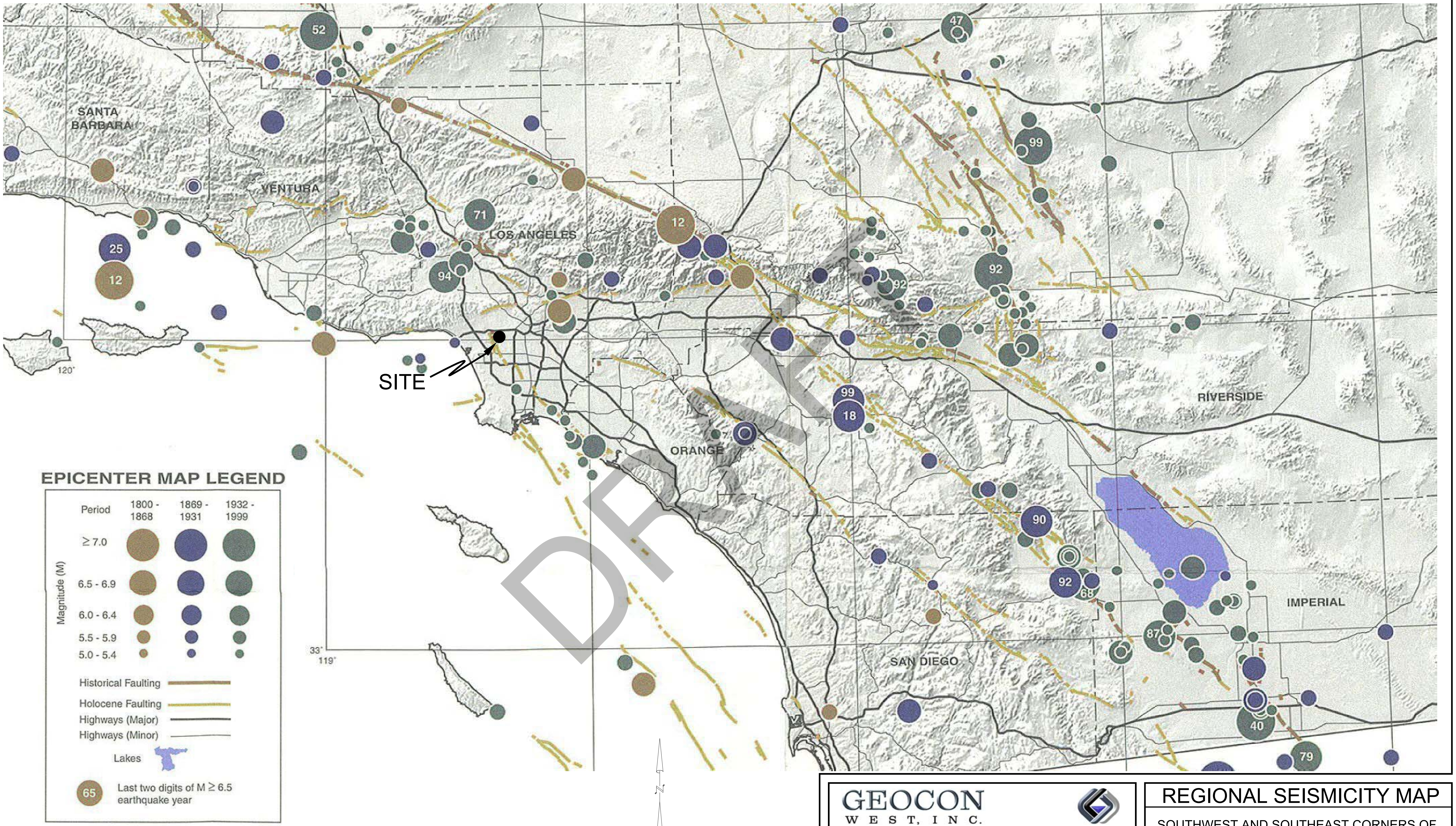
REGIONAL FAULT MAP

SOUTHWEST AND SOUTHEAST CORNERS OF
EXPOSITION AND CRENSHAW BOULEVARD
LOS ANGELES, CALIFORNIA

AUGUST 2019	PROJECT NO. A9930-06-01	FIG. 4



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REGIONAL SEISMICITY MAP

SOUTHWEST AND SOUTHEAST CORNERS OF
EXPOSITION AND CRENSHAW BOULEVARD
LOS ANGELES, CALIFORNIA

AUGUST 2019 | PROJECT NO. A9930-06-01 | FIG. 5

GEOCON

Client : CRENSHAW CROSSINGS
File No. : A9930-06-01
Boring : 1

EMPIRICAL ESTIMATION OF LIQUEFACTION POTENTIAL
DESIGN EARTHQUAKE

NCEER (1996) METHOD

By Thomas F. Blake (1994-1996)

EARTHQUAKE INFORMATION:

Earthquake Magnitude:	6.71
Peak Horiz. Acceleration PGA_M (g):	0.733
2/3 PGA_M (g):	0.489
Calculated Mag.Wtg.Factor:	0.756
Historic High Groundwater:	10.0
Groundwater Depth During Exploration:	18.0

ENERGY & ROD CORRECTIONS:

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

LIQUEFACTION CALCULATIONS:

Unit Wt. Water (pcf): 62.4

Depth to Base (ft)	Total Unit Wt. (pcf)	Water (0 or 1)	FIELD SPT (N)	Depth of SPT (ft)	Liq.Sus. (0 or 1)	-200 (%)	Est. Dr (%)	CN Factor	Corrected (N1)60	Eff. Unit Wt. (psf)	Resist. CRR	rd Factor	Induced CSR	Liquefac. Safe.Fact.
1.0	119.0	0	11.0	2.0	1		78	1.700	21.0	119.0	0.230	0.998	0.240	--
2.0	119.0	0	11.0	2.0	1		78	1.700	21.0	119.0	0.230	0.993	0.239	--
3.0	119.0	0	11.0	2.0	1		78	1.700	21.0	119.0	0.230	0.989	0.237	--
4.0	119.0	0	11.0	2.0	1		78	1.700	21.0	119.0	0.230	0.984	0.236	--
5.0	119.0	0	11.0	2.0	1		78	1.700	21.0	119.0	0.230	0.979	0.235	--
6.5	119.0	0	11.0	2.0	1		78	1.700	21.0	119.0	0.230	0.974	0.234	--
7.0	108.0	0	13.0	7.0	1	0	76	1.679	24.6	108.0	0.278	0.969	0.233	--
8.0	108.0	0	13.0	7.0	1	0	76	1.544	22.6	108.0	0.250	0.966	0.232	--
10.0	108.0	0	13.0	7.0	1	0	76	1.418	20.7	108.0	0.227	0.959	0.230	--
10.0	108.0	0	13.0	7.0	1	84	76	1.418	27.7	108.0	0.342	0.954	0.229	--
11.0	108.0	1	4.0	12.0	1	84	39	1.319	12.9	45.6	0.141	0.952	0.235	0.60
12.0	108.0	1	4.0	12.0	1	84	39	1.263	12.7	45.6	0.139	0.947	0.245	0.57
13.0	108.0	1	4.0	12.0	1	44	39	1.214	12.5	45.6	0.136	0.943	0.254	0.53
14.0	108.0	1	4.0	12.0	1	44	39	1.171	12.3	45.6	0.134	0.938	0.263	0.51
15.0	130.0	1	13.0	17.0	1	44	67	1.127	25.6	67.6	0.296	0.934	0.270	1.09
16.0	130.0	1	13.0	17.0	1	44	67	1.085	24.9	67.6	0.284	0.929	0.277	1.03
17.0	130.0	1	13.0	17.0	1	9	67	1.047	18.2	67.6	0.198	0.925	0.282	0.70
18.0	130.0	1	13.0	17.0	1	9	67	1.021	17.8	67.6	0.193	0.920	0.287	0.67
19.0	130.0	1	13.0	17.0	1	9	67	1.004	17.5	67.6	0.190	0.915	0.291	0.65
20.0	123.0	1	13.0	17.0	0	0		0.989	16.3	60.6	~	0.911	0.295	~
21.5	123.0	1	13.0	17.0	0	0		0.972	16.0	60.6	~	0.905	0.300	~
22.0	123.0	1	2.0	22.0	0	0		0.966	2.7	60.6	~	0.901	0.301	~
23.0	123.0	1	2.0	22.0	0	0		0.950	2.6	60.6	~	0.897	0.305	~
24.0	123.0	1	2.0	22.0	0			0.937	2.6	60.6	~	0.893	0.307	~
25.0	123.0	1	9.0	27.0	1	51	51	0.926	19.2	60.6	0.208	0.888	0.310	0.67
26.0	123.0	1	9.0	27.0	1	51	51	0.914	19.0	60.6	0.206	0.883	0.312	0.66
27.0	123.0	1	9.0	27.0	1	51	51	0.904	18.9	60.6	0.205	0.879	0.314	0.65
28.0	123.0	1	9.0	27.0	1	51	51	0.893	18.7	60.6	0.203	0.874	0.315	0.65
29.0	123.0	1	9.0	27.0	1	51	51	0.883	18.6	60.6	0.202	0.870	0.316	0.64
30.0	135.0	1	50.0	32.0	1	0	116	0.872	65.4	72.6	Infin.	0.865	0.317	Non-Liq.
31.0	135.0	1	50.0	32.0	1	0	116	0.861	64.6	72.6	Infin.	0.861	0.318	Non-Liq.
32.0	133.0	1	50.0	32.0	1	0	116	0.850	63.8	70.6	Infin.	0.856	0.318	Non-Liq.
33.0	133.0	1	50.0	32.0	1	0	116	0.840	63.0	70.6	Infin.	0.851	0.318	Non-Liq.
34.0	133.0	1	50.0	32.0	1	0	116	0.830	62.3	70.6	Infin.	0.847	0.318	Non-Liq.
35.0	133.0	1	50.0	32.0	1	0	116	0.821	61.5	70.6	Infin.	0.842	0.318	Non-Liq.
36.0	133.0	1	50.0	32.0	1	0	116	0.811	60.8	70.6	Infin.	0.838	0.318	Non-Liq.
37.0	133.0	1	50.0	32.0	1	0	116	0.802	60.2	70.6	Infin.	0.833	0.318	Non-Liq.
38.0	133.0	1	80.0	37.0	1	0	142	0.794	95.3	70.6	Infin.	0.829	0.318	Non-Liq.
39.0	135.0	1	80.0	37.0	1	0	142	0.785	94.2	72.6	Infin.	0.824	0.317	Non-Liq.
40.0	135.0	1	80.0	37.0	1	0	142	0.777	93.3	72.6	Infin.	0.819	0.317	Non-Liq.
41.0	135.0	1	80.0	37.0	1	0	142	0.769	92.3	72.6	Infin.	0.815	0.316	Non-Liq.
42.0	135.0	1	80.0	37.0	1	0	142	0.761	91.4	72.6	Infin.	0.810	0.315	Non-Liq.
43.0	135.0	1	90.0	42.0	1	0	145	0.754	101.8	72.6	Infin.	0.806	0.315	Non-Liq.
44.0	135.0	1	90.0	42.0	1	0	145	0.746	100.8	72.6	Infin.	0.801	0.314	Non-Liq.
45.0	135.0	1	90.0	42.0	1	0	145	0.739	99.8	72.6	Infin.	0.797	0.313	Non-Liq.
46.0	135.0	1	90.0	42.0	1	0	145	0.732	98.9	72.6	Infin.	0.792	0.312	Non-Liq.
47.0	135.0	1	90.0	42.0	1	0	145	0.726	98.0	72.6	Infin.	0.787	0.311	Non-Liq.
48.0	135.0	1	90.0	47.0	1	0	140	0.719	97.1	72.6	Infin.	0.783	0.310	Non-Liq.
49.0	135.0	1	90.0	47.0	1	0	140	0.713	96.2	72.6	Infin.	0.778	0.309	Non-Liq.
50.5	135.0	1	90.0	47.0	1	0	140	0.705	95.2	72.6	Infin.	0.773	0.308	Non-Liq.

GEOCON

Client : CRENSHAW CROSSINGS
File No. : A9930-06-01
Boring : 1

LIQUEFACTION SETTLEMENT ANALYSIS
DESIGN EARTHQUAKE

(SATURATED SAND AT INITIAL LIQUEFACTION CONDITION)

NCEER (1996) METHOD
EARTHQUAKE INFORMATION:

Earthquake Magnitude:	6.71
PGAM (g):	0.733
2/3 PGAM (g):	0.49
Calculated Mag.Wtg.Factor:	0.756
Historic High Groundwater:	10.0
Groundwater @ Exploration:	18.0

DEPTH TO BASE	BLOW COUNT N	WET DENSITY (PCF)	TOTAL STRESS O (TSF)	EFFECT STRESS O' (TSF)	REL. DEN. Dr (%)	ADJUST BLOWS (N1)60	Tav/σ'_o	LIQUEFACTION SAFETY FACTOR	Volumetric Strain [e_{15}} (%)	EQ. SETTLE. Pe (in.)
1	11	119	0.030	0.030	78	21	0.318	--	0.00	0.00
2	11	119	0.089	0.089	78	21	0.318	--	0.00	0.00
3	11	119	0.149	0.149	78	21	0.318	--	0.00	0.00
4	11	119	0.208	0.208	78	21	0.318	--	0.00	0.00
5	11	119	0.268	0.268	78	21	0.318	--	0.00	0.00
6.5	11	119	0.342	0.342	78	21	0.318	--	0.00	0.00
7	13	108	0.371	0.371	76	25	0.318	--	0.00	0.00
10	13	108	0.438	0.438	76	23	0.318	--	0.00	0.00
10	13	108	0.519	0.519	76	21	0.318	--	0.00	0.00
10	13	108	0.519	0.519	76	28	0.318	--	0.00	0.00
11	4	108	0.600	0.584	39	13	0.326	0.60	1.80	0.22
12	4	108	0.654	0.607	39	44	0.342	0.57	0.00	0.00
13	4	108	0.708	0.630	39	12	0.357	0.53	2.30	0.28
14	4	108	0.762	0.653	39	12	0.371	0.51	2.30	0.28
15	13	130	0.822	0.681	67	26	0.383	1.09	0.80	0.10
16	13	130	0.887	0.715	67	25	0.394	1.03	1.30	0.16
17	13	130	0.952	0.749	67	18	0.404	0.70	1.70	0.20
18	13	130	1.017	0.783	67	18	0.413	0.67	1.70	0.20
19	13	130	1.082	0.816	67	18	0.421	0.65	1.70	0.20
20	13	123	1.145	0.848		16	0.429	~	0.00	0.00
21.5	13	123	1.222	0.886		16	0.438	~	0.00	0.00
22	2	123	1.252	0.901		3	0.442	~	0.00	0.00
23	2	123	1.329	0.939		3	0.450	~	0.00	0.00
24	2	123	1.391	0.970		3	0.456	~	0.00	0.00
25	9	123	1.452	1.000	51	19	0.462	0.67	1.60	0.19
26	9	123	1.514	1.030	51	19	0.467	0.66	1.60	0.19
27	9	123	1.575	1.060	51	19	0.472	0.65	1.60	0.19
28	9	123	1.637	1.091	51	19	0.477	0.65	1.60	0.19
29	9	123	1.698	1.121	51	19	0.481	0.64	1.60	0.19
30	50	135	1.763	1.154	116	65	0.485	Non-Liq.	0.00	0.00
31	50	135	1.830	1.191	116	65	0.489	Non-Liq.	0.00	0.00
32	50	133	1.897	1.226	116	64	0.492	Non-Liq.	0.00	0.00
33	50	133	1.964	1.262	116	63	0.495	Non-Liq.	0.00	0.00
34	50	133	2.030	1.297	116	62	0.497	Non-Liq.	0.00	0.00
35	50	133	2.097	1.332	116	62	0.500	Non-Liq.	0.00	0.00
36	50	133	2.163	1.368	116	61	0.503	Non-Liq.	0.00	0.00
37	50	133	2.230	1.403	116	60	0.505	Non-Liq.	0.00	0.00
38	80	133	2.296	1.438	142	95	0.507	Non-Liq.	0.00	0.00
39	80	135	2.363	1.474	142	94	0.509	Non-Liq.	0.00	0.00
40	80	135	2.431	1.510	142	93	0.511	Non-Liq.	0.00	0.00
41	80	135	2.498	1.547	142	92	0.513	Non-Liq.	0.00	0.00
42	80	135	2.566	1.583	142	91	0.515	Non-Liq.	0.00	0.00
43	90	135	2.633	1.619	145	102	0.517	Non-Liq.	0.00	0.00
44	90	135	2.701	1.656	145	101	0.518	Non-Liq.	0.00	0.00
45	90	135	2.768	1.692	145	100	0.520	Non-Liq.	0.00	0.00
46	90	135	2.836	1.728	145	99	0.521	Non-Liq.	0.00	0.00
47	90	135	2.903	1.764	145	98	0.523	Non-Liq.	0.00	0.00
48	90	135	2.971	1.801	140	97	0.524	Non-Liq.	0.00	0.00
49	90	135	3.038	1.837	140	96	0.526	Non-Liq.	0.00	0.00
50.5	90	135	3.123	1.882	140	95	0.527	Non-Liq.	0.00	0.00

TOTAL SETTLEMENT = 2.6 INCHES

GEOCON

Client : CRENSHAW CROSSINGS
File No. : A9930-06-01
Boring : 7

EMPIRICAL ESTIMATION OF LIQUEFACTION POTENTIAL
DESIGN EARTHQUAKE

NCEER (1996) METHOD

By Thomas F. Blake (1994-1996)

EARTHQUAKE INFORMATION:

Earthquake Magnitude:	6.70
Peak Horiz. Acceleration PGA_M (g):	0.733
2/3 PGA_M (g):	0.489
Calculated Mag.Wtg.Factor:	0.753
Historic High Groundwater:	10.0
Groundwater Depth During Exploration:	20.0

ENERGY & ROD CORRECTIONS:

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

LIQUEFACTION CALCULATIONS:

Unit Wt. Water (pcf):	62.4

Depth to Base (ft)	Total Unit Wt. (pcf)	Water (0 or 1)	FIELD SPT (N)	Depth of SPT (ft)	Liq.Sus. (0 or 1)	-200 (%)	Est. Dr (%)	CN Factor	Corrected (N1)60	Eff. Unit Wt. (psf)	Resist. CRR	rd Factor	Induced CSR	Liquefac. Safe.Fact.
1.0	128.0	0	8.0	2.0	1		66	1.700	15.3	128.0	0.167	0.998	0.239	--
2.0	128.0	0	8.0	2.0	1		66	1.700	15.3	128.0	0.167	0.993	0.238	--
3.0	128.0	0	8.0	2.0	1		66	1.700	15.3	128.0	0.167	0.989	0.237	--
4.0	128.0	0	8.0	2.0	1		66	1.700	15.3	128.0	0.167	0.984	0.235	--
5.0	128.0	0	8.0	2.0	1		66	1.700	15.3	128.0	0.167	0.979	0.234	--
6.5	128.0	0	8.0	2.0	1		66	1.684	15.2	128.0	0.165	0.974	0.233	--
7.0	122.0	0	13.0	7.0	1	25	75	1.617	28.3	122.0	0.358	0.969	0.232	--
8.0	122.0	0	13.0	7.0	1	25	75	1.482	26.3	122.0	0.309	0.966	0.231	--
9.0	122.0	0	13.0	7.0	1	25	75	1.395	25.0	122.0	0.286	0.961	0.230	--
10.0	122.0	0	13.0	7.0	1	25	75	1.322	24.0	122.0	0.269	0.957	0.229	--
11.5	122.0	1	13.0	7.0	1	25	75	1.245	22.8	59.6	0.253	0.951	0.236	1.07
12.0	122.0	1	6.0	12.0	1	64	47	1.218	15.2	59.6	0.166	0.946	0.240	0.69
13.0	122.0	1	6.0	12.0	1	64	47	1.157	14.8	59.6	0.161	0.943	0.251	0.64
14.0	122.0	1	6.0	12.0	1	64	47	1.114	14.5	59.6	0.158	0.938	0.258	0.61
15.0	122.0	1	6.0	12.0	1	64	47	1.076	14.3	59.6	0.156	0.934	0.265	0.59
16.0	128.0	1	14.0	17.0	1	32	68	1.040	24.7	65.6	0.281	0.929	0.270	1.04
17.0	128.0	1	14.0	17.0	1	32	68	1.007	24.1	65.6	0.271	0.925	0.275	0.99
18.0	128.0	1	14.0	17.0	1	32	68	0.977	23.6	65.6	0.263	0.920	0.280	0.94
19.0	128.0	1	14.0	17.0	1	32	68	0.950	23.1	65.6	0.257	0.915	0.284	0.90
20.0	128.0	1	14.0	17.0	1	32	68	0.931	22.8	65.6	0.252	0.911	0.288	0.88
21.5	128.0	1	14.0	17.0	1	32	68	0.915	22.5	65.6	0.248	0.905	0.292	0.85
22.0	105.0	1	2.0	22.0	0	0		0.910	2.5	42.6	~	0.901	0.293	~
23.0	105.0	1	2.0	22.0	0	0		0.901	2.5	42.6	~	0.897	0.298	~
24.0	105.0	1	2.0	22.0	0	0		0.893	2.5	42.6	~	0.893	0.301	~
25.5	105.0	1	2.0	22.0	0	0		0.884	2.4	42.6	~	0.887	0.305	~
26.0	105.0	1	7.0	25.0	1	0	45	0.881	8.8	42.6	0.098	0.882	0.305	0.32
27.0	105.0	1	7.0	25.0	1	0	45	0.872	8.8	42.6	0.097	0.879	0.309	0.31
28.0	131.0	1	61.0	27.0	1	0	133	0.864	77.0	68.6	Infin.	0.874	0.311	Non-Liq.
29.0	131.0	1	61.0	27.0	1	0	133	0.853	76.1	68.6	Infin.	0.870	0.312	Non-Liq.
30.0	131.0	1	61.0	27.0	1	0	133	0.843	75.2	68.6	Infin.	0.865	0.313	Non-Liq.
31.0	131.0	1	61.0	27.0	1	0	133	0.834	74.3	68.6	Infin.	0.861	0.314	Non-Liq.
32.0	131.0	1	74.0	32.0	1	0	141	0.824	91.5	68.6	Infin.	0.856	0.314	Non-Liq.
33.0	131.0	1	74.0	32.0	1	0	141	0.815	90.5	68.6	Infin.	0.851	0.315	Non-Liq.
34.0	131.0	1	74.0	32.0	1	0	141	0.806	89.5	68.6	Infin.	0.847	0.315	Non-Liq.
35.0	123.0	1	74.0	32.0	1	0	141	0.798	88.6	60.6	Infin.	0.842	0.315	Non-Liq.
36.5	123.0	1	74.0	32.0	1	0	141	0.789	87.6	60.6	Infin.	0.837	0.315	Non-Liq.
37.0	123.0	1	53.0	37.0	1	0	115	0.786	62.5	60.6	Infin.	0.832	0.315	Non-Liq.
38.0	123.0	1	53.0	37.0	1	0	115	0.777	61.8	60.6	Infin.	0.829	0.316	Non-Liq.
39.0	123.0	1	53.0	37.0	1	0	115	0.770	61.2	60.6	Infin.	0.824	0.316	Non-Liq.
40.0	123.0	1	90.0	40.0	1	0	147	0.764	103.1	60.6	Infin.	0.819	0.316	Non-Liq.
41.0	123.0	1	90.0	40.0	1	0	147	0.757	102.3	60.6	Infin.	0.815	0.316	Non-Liq.
42.0	123.0	1	100.0	42.0	1	0	153	0.751	112.7	60.6	Infin.	0.810	0.315	Non-Liq.
43.0	123.0	1	100.0	42.0	1	0	153	0.745	111.8	60.6	Infin.	0.806	0.315	Non-Liq.
44.0	123.0	1	100.0	42.0	1	0	153	0.739	110.9	60.6	Infin.	0.801	0.315	Non-Liq.
45.0	123.0	1	100.0	42.0	1	0	153	0.733	110.0	60.6	Infin.	0.797	0.314	Non-Liq.
46.0	123.0	1	100.0	42.0	1	0	153	0.728	109.2	60.6	Infin.	0.792	0.314	Non-Liq.
47.0	122.0	1	31.0	47.0	1	0	83	0.722	33.6	59.6	Infin.	0.787	0.313	Non-Liq.
48.0	122.0	1	31.0	47.0	1	0	83	0.717	33.3	59.6	Infin.	0.783	0.313	Non-Liq.
49.0	122.0	1	31.0	47.0	1	0	83	0.712	33.1	59.6	Infin.	0.778	0.312	Non-Liq.
50.5	122.0	1	31.0	47.0	1	0	83	0.705	32.8	59.6	Infin.	0.773	0.311	Non-Liq.

Figure 8

GEOCON

Client : CRENSHAW CROSSINGS
File No. : A9930-06-01
Boring : 7

LIQUEFACTION SETTLEMENT ANALYSIS
DESIGN EARTHQUAKE

(SATURATED SAND AT INITIAL LIQUEFACTION CONDITION)

NCEER (1996) METHOD
EARTHQUAKE INFORMATION:

Earthquake Magnitude:	6.70
PGAM (g):	0.733
2/3 PGAM (g):	0.49
Calculated Mag.Wtg.Factor:	0.753
Historic High Groundwater:	10.0
Groundwater @ Exploration:	20.0

DEPTH TO BASE	BLOW COUNT N	WET DENSITY (PCF)	TOTAL STRESS O (TSF)	EFFECT STRESS O' (TSF)	REL. DEN. Dr (%)	ADJUST BLOWS (N1)60	Tav/σ'$_o$	LIQUEFACTION SAFETY FACTOR	Volumetric Strain [e_{15}} (%)	EQ. SETTLE. Pe (in.)
1	8	128	0.032	0.032	66	15	0.318	--	0.00	0.00
2	8	128	0.096	0.096	66	15	0.318	--	0.00	0.00
3	8	128	0.160	0.160	66	15	0.318	--	0.00	0.00
4	8	128	0.224	0.224	66	15	0.318	--	0.00	0.00
5	8	128	0.288	0.288	66	15	0.318	--	0.00	0.00
6.5	8	128	0.368	0.368	66	15	0.318	--	0.00	0.00
7	13	122	0.399	0.399	75	28	0.318	--	0.00	0.00
8	13	122	0.476	0.476	75	26	0.318	--	0.00	0.00
9	13	122	0.537	0.537	75	25	0.318	--	0.00	0.00
10	13	122	0.598	0.598	75	24	0.318	--	0.00	0.00
11.5	13	122	0.674	0.650	75	23	0.329	1.07	1.10	0.20
12	6	122	0.704	0.665	47	15	0.336	0.69	1.70	0.10
13	6	122	0.781	0.703	47	15	0.353	0.64	1.80	0.22
14	6	122	0.842	0.732	47	15	0.365	0.61	1.80	0.22
15	6	122	0.903	0.762	47	14	0.376	0.59	1.80	0.22
16	14	128	0.965	0.793	68	25	0.387	1.04	1.30	0.16
17	14	128	1.029	0.826	68	24	0.396	0.99	1.30	0.16
18	14	128	1.093	0.859	68	24	0.404	0.94	1.30	0.16
19	14	128	1.157	0.892	68	23	0.412	0.90	1.30	0.16
20	14	128	1.221	0.925	68	23	0.420	0.88	1.30	0.16
21.5	14	128	1.301	0.966	68	22	0.428	0.85	1.40	0.25
22	2	105	1.330	0.979		3	0.432	~	0.00	0.00
23	2	105	1.396	1.006		2	0.441	~	0.00	0.00
24	2	105	1.448	1.027		2	0.448	~	0.00	0.00
25.5	2	105	1.514	1.054		2	0.457	~	0.00	0.00
26	7	105	1.540	1.064	45	9	0.460	0.32	2.70	0.16
27	7	105	1.606	1.091	45	9	0.468	0.31	2.70	0.32
28	61	131	1.665	1.119	133	77	0.473	Non-Liq.	0.00	0.00
29	61	131	1.730	1.153	133	76	0.477	Non-Liq.	0.00	0.00
30	61	131	1.796	1.187	133	75	0.481	Non-Liq.	0.00	0.00
31	61	131	1.861	1.222	133	74	0.484	Non-Liq.	0.00	0.00
32	74	131	1.927	1.256	141	91	0.488	Non-Liq.	0.00	0.00
33	74	131	1.992	1.290	141	90	0.491	Non-Liq.	0.00	0.00
34	74	131	2.058	1.325	141	90	0.494	Non-Liq.	0.00	0.00
35	74	123	2.121	1.357	141	89	0.497	Non-Liq.	0.00	0.00
36.5	74	123	2.198	1.395	141	88	0.501	Non-Liq.	0.00	0.00
37	53	123	2.229	1.410	115	62	0.502	Non-Liq.	0.00	0.00
38	53	123	2.306	1.448	115	62	0.506	Non-Liq.	0.00	0.00
39	53	123	2.367	1.478	115	61	0.509	Non-Liq.	0.00	0.00
40	90	123	2.429	1.508	147	103	0.512	Non-Liq.	0.00	0.00
41	90	123	2.490	1.539	147	102	0.514	Non-Liq.	0.00	0.00
42	100	123	2.552	1.569	153	113	0.517	Non-Liq.	0.00	0.00
43	100	123	2.613	1.599	153	112	0.519	Non-Liq.	0.00	0.00
44	100	123	2.675	1.630	153	111	0.522	Non-Liq.	0.00	0.00
45	100	123	2.736	1.660	153	110	0.524	Non-Liq.	0.00	0.00
46	100	123	2.798	1.690	153	109	0.526	Non-Liq.	0.00	0.00
47	31	122	2.859	1.720	83	34	0.528	Non-Liq.	0.00	0.00
48	31	122	2.920	1.750	83	33	0.530	Non-Liq.	0.00	0.00
49	31	122	2.981	1.780	83	33	0.532	Non-Liq.	0.00	0.00
50.5	31	122	3.057	1.817	83	33	0.535	Non-Liq.	0.00	0.00

TOTAL SETTLEMENT = 2.5 INCHES

GEOCON

Client : CRENSHAW CROSSINGS
File No. : A9930-06-01
Boring : 1

EMPIRICAL ESTIMATION OF LIQUEFACTION POTENTIAL
MAXIMUM CONSIDERED EARTHQUAKE

NCEER (1996) METHOD

EARTHQUAKE INFORMATION:

Earthquake Magnitude:	6.76
Peak Horiz. Acceleration PGA_M (g):	0.733
Calculated Mag.Wtg.Factor:	0.770
Historic High Groundwater:	10.0
Groundwater Depth During Exploration:	18.0

By Thomas F. Blake (1994-1996)

ENERGY & ROD CORRECTIONS:

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

LIQUEFACTION CALCULATIONS:

Unit Wt. Water (pcf):	62.4

Depth to Base (ft)	Total Unit Wt. (pcf)	Water (0 or 1)	FIELD SPT (N)	Depth of SPT (ft)	Liq.Sus. (0 or 1)	-200 (%)	Est. Dr (%)	CN Factor	Corrected (N1)60	Eff. Unit Wt. (psf)	Resist. CRR	rd Factor	Induced CSR	Liquefac. Safe.Fact.
1.0	119.0	0	11.0	2.0	1		78	1.700	21.0	119.0	0.230	0.998	0.366	--
2.0	119.0	0	11.0	2.0	1		78	1.700	21.0	119.0	0.230	0.993	0.364	--
3.0	119.0	0	11.0	2.0	1		78	1.700	21.0	119.0	0.230	0.989	0.363	--
4.0	119.0	0	11.0	2.0	1		78	1.700	21.0	119.0	0.230	0.984	0.361	--
5.0	119.0	0	11.0	2.0	1		78	1.700	21.0	119.0	0.230	0.979	0.359	--
6.5	119.0	0	11.0	2.0	1		78	1.700	21.0	119.0	0.230	0.974	0.357	--
7.0	108.0	0	13.0	7.0	1		76	1.679	24.6	108.0	0.278	0.969	0.356	--
8.0	108.0	0	13.0	7.0	1		76	1.544	22.6	108.0	0.250	0.966	0.354	--
9.0	108.0	0	13.0	7.0	1		76	1.457	21.3	108.0	0.233	0.961	0.353	--
10.0	108.0	0	13.0	7.0	1	84	76	1.383	27.2	108.0	0.329	0.957	0.351	--
11.0	108.0	1	4.0	12.0	1	84	39	1.319	12.9	45.6	0.141	0.952	0.359	0.39
12.0	108.0	1	4.0	12.0	1	84	39	1.263	12.7	45.6	0.139	0.947	0.374	0.37
13.0	108.0	1	4.0	12.0	1	84	39	1.214	12.5	45.6	0.136	0.943	0.389	0.35
14.0	108.0	1	4.0	12.0	1	44	39	1.171	12.3	45.6	0.134	0.938	0.402	0.33
15.0	130.0	1	13.0	17.0	1	44	67	1.127	25.6	67.6	0.296	0.934	0.413	0.72
16.0	130.0	1	13.0	17.0	1	44	67	1.085	24.9	67.6	0.284	0.929	0.423	0.67
17.0	130.0	1	13.0	17.0	1	9	67	1.047	18.2	67.6	0.198	0.925	0.431	0.46
18.0	130.0	1	13.0	17.0	1	9	67	1.021	17.8	67.6	0.193	0.920	0.439	0.44
19.0	130.0	1	13.0	17.0	1	9	67	1.004	17.5	67.6	0.190	0.915	0.445	0.43
20.0	123.0	1	13.0	17.0	0			0.989	16.3	60.6	~	0.911	0.451	~
21.5	123.0	1	13.0	17.0	0			0.972	16.0	60.6	~	0.905	0.458	~
22.0	123.0	1	2.0	22.0	0			0.966	2.7	60.6	~	0.901	0.459	~
23.0	123.0	1	2.0	22.0	0			0.950	2.6	60.6	~	0.897	0.466	~
24.0	123.0	1	2.0	22.0	0			0.937	2.6	60.6	~	0.893	0.470	~
25.0	123.0	1	9.0	27.0	1	51	51	0.926	19.2	60.6	0.208	0.888	0.473	0.44
26.0	123.0	1	9.0	27.0	1	51	51	0.914	19.0	60.6	0.206	0.883	0.476	0.43
27.0	123.0	1	9.0	27.0	1	51	51	0.904	18.9	60.6	0.205	0.879	0.479	0.43
28.0	123.0	1	9.0	27.0	1	51	51	0.893	18.7	60.6	0.203	0.874	0.481	0.42
29.0	123.0	1	9.0	27.0	1	51	51	0.883	18.6	60.6	0.202	0.870	0.483	0.42
30.0	135.0	1	50.0	32.0	1		116	0.872	65.4	72.6	Infin.	0.865	0.485	Non-Liq.
31.0	135.0	1	50.0	32.0	1		116	0.861	64.6	72.6	Infin.	0.861	0.485	Non-Liq.
32.0	133.0	1	50.0	32.0	1		116	0.850	63.8	70.6	Infin.	0.856	0.486	Non-Liq.
33.0	133.0	1	50.0	32.0	1		116	0.840	63.0	70.6	Infin.	0.851	0.486	Non-Liq.
34.0	133.0	1	50.0	32.0	1		116	0.830	62.3	70.6	Infin.	0.847	0.486	Non-Liq.
35.0	133.0	1	50.0	32.0	1		116	0.821	61.5	70.6	Infin.	0.842	0.486	Non-Liq.
36.0	133.0	1	50.0	32.0	1		116	0.811	60.8	70.6	Infin.	0.838	0.486	Non-Liq.
37.0	133.0	1	50.0	32.0	1		116	0.802	60.2	70.6	Infin.	0.833	0.486	Non-Liq.
38.0	133.0	1	80.0	37.0	1		142	0.794	95.3	70.6	Infin.	0.829	0.485	Non-Liq.
39.0	135.0	1	80.0	37.0	1		142	0.785	94.2	72.6	Infin.	0.824	0.485	Non-Liq.
40.0	135.0	1	80.0	37.0	1		142	0.777	93.3	72.6	Infin.	0.819	0.484	Non-Liq.
41.0	135.0	1	80.0	37.0	1		142	0.769	92.3	72.6	Infin.	0.815	0.483	Non-Liq.
42.0	135.0	1	80.0	37.0	1		142	0.761	91.4	72.6	Infin.	0.810	0.482	Non-Liq.
43.0	135.0	1	90.0	42.0	1		145	0.754	101.8	72.6	Infin.	0.806	0.481	Non-Liq.
44.0	135.0	1	90.0	42.0	1		145	0.746	100.8	72.6	Infin.	0.801	0.480	Non-Liq.
45.0	135.0	1	90.0	42.0	1		145	0.739	99.8	72.6	Infin.	0.797	0.478	Non-Liq.
46.0	135.0	1	90.0	42.0	1		145	0.732	98.9	72.6	Infin.	0.792	0.477	Non-Liq.
47.0	135.0	1	90.0	42.0	1		145	0.726	98.0	72.6	Infin.	0.787	0.475	Non-Liq.
48.0	135.0	1	90.0	47.0	1		140	0.719	97.1	72.6	Infin.	0.783	0.474	Non-Liq.
49.0	135.0	1	90.0	47.0	1		140	0.713	96.2	72.6	Infin.	0.778	0.472	Non-Liq.
50.5	135.0	1	90.0	47.0	1		140	0.705	95.2	72.6	Infin.	0.773	0.470	Non-Liq.

Figure 10

GEOCON

Client : CRENSHAW CROSSINGS
File No. : A9930-06-01
Boring : 1

LIQUEFACTION SETTLEMENT ANALYSIS MAXIMUM CONSIDERED EARTHQUAKE

(SATURATED SAND AT INITIAL LIQUEFACTION CONDITION)

NCEER (1996) METHOD
EARTHQUAKE INFORMATION:

Earthquake Magnitude:	6.76
PGA_M (g):	0.733
Calculated Mag.Wtg.Factor:	0.770
Historic High Groundwater:	10.0
Groundwater @ Exploration:	18.0

DEPTH TO BASE	BLOW COUNT N	WET DENSITY (PCF)	TOTAL STRESS O (TSF)	EFFECT STRESS O' (TSF)	REL. DEN. Dr (%)	ADJUST BLOWS (N1)60	Tav/σ'$_o$	LIQUEFACTION SAFETY FACTOR	Volumetric Strain [e_{15}} (%)	EQ. SETTLE. Pe (in.)
1	11	119	0.030	0.030	78	21	0.476	--	0.00	0.00
2	11	119	0.089	0.089	78	21	0.476	--	0.00	0.00
3	11	119	0.149	0.149	78	21	0.476	--	0.00	0.00
4	11	119	0.208	0.208	78	21	0.476	--	0.00	0.00
5	11	119	0.268	0.268	78	21	0.476	--	0.00	0.00
6.5	11	119	0.342	0.342	78	21	0.476	--	0.00	0.00
7	13	108	0.371	0.371	76	25	0.476	--	0.00	0.00
8	13	108	0.438	0.438	76	23	0.476	--	0.00	0.00
10	13	108	0.492	0.492	76	21	0.476	--	0.00	0.00
10	13	108	0.546	0.546	76	27	0.476	--	0.00	0.00
11	4	108	0.600	0.584	39	13	0.489	0.39	1.80	0.22
12	4	108	0.654	0.607	39	13	0.513	0.37	1.80	0.22
13	4	108	0.708	0.630	39	44	0.535	0.35	0.00	0.00
14	4	108	0.762	0.653	39	12	0.556	0.33	2.30	0.28
15	13	130	0.822	0.681	67	26	0.575	0.72	1.10	0.13
16	13	130	0.887	0.715	67	25	0.591	0.67	1.30	0.16
17	13	130	0.952	0.749	67	18	0.606	0.46	1.70	0.20
18	13	130	1.017	0.783	67	18	0.619	0.44	1.70	0.20
19	13	130	1.082	0.816	67	18	0.631	0.43	1.70	0.20
20	13	123	1.145	0.848		16	0.643	~	0.00	0.00
21.5	13	123	1.222	0.886		16	0.657	~	0.00	0.00
22	2	123	1.252	0.901		3	0.662	~	0.00	0.00
23	2	123	1.329	0.939		3	0.674	~	0.00	0.00
24	2	123	1.391	0.970		3	0.683	~	0.00	0.00
25	9	123	1.452	1.000	51	19	0.692	0.44	1.60	0.19
26	9	123	1.514	1.030	51	19	0.700	0.43	1.60	0.19
27	9	123	1.575	1.060	51	19	0.708	0.43	1.60	0.19
28	9	123	1.637	1.091	51	19	0.715	0.42	1.60	0.19
29	9	123	1.698	1.121	51	19	0.722	0.42	1.60	0.19
30	50	135	1.763	1.154	116	65	0.728	Non-Liq.	0.00	0.00
31	50	135	1.830	1.191	116	65	0.732	Non-Liq.	0.00	0.00
32	50	133	1.897	1.226	116	64	0.737	Non-Liq.	0.00	0.00
33	50	133	1.964	1.262	116	63	0.742	Non-Liq.	0.00	0.00
34	50	133	2.030	1.297	116	62	0.746	Non-Liq.	0.00	0.00
35	50	133	2.097	1.332	116	62	0.750	Non-Liq.	0.00	0.00
36	50	133	2.163	1.368	116	61	0.754	Non-Liq.	0.00	0.00
37	50	133	2.230	1.403	116	60	0.757	Non-Liq.	0.00	0.00
38	80	133	2.296	1.438	142	95	0.761	Non-Liq.	0.00	0.00
39	80	135	2.363	1.474	142	94	0.764	Non-Liq.	0.00	0.00
40	80	135	2.431	1.510	142	93	0.767	Non-Liq.	0.00	0.00
41	80	135	2.498	1.547	142	92	0.770	Non-Liq.	0.00	0.00
42	80	135	2.566	1.583	142	91	0.772	Non-Liq.	0.00	0.00
43	90	135	2.633	1.619	145	102	0.775	Non-Liq.	0.00	0.00
44	90	135	2.701	1.656	145	101	0.777	Non-Liq.	0.00	0.00
45	90	135	2.768	1.692	145	100	0.780	Non-Liq.	0.00	0.00
46	90	135	2.836	1.728	145	99	0.782	Non-Liq.	0.00	0.00
47	90	135	2.903	1.764	145	98	0.784	Non-Liq.	0.00	0.00
48	90	135	2.971	1.801	140	97	0.786	Non-Liq.	0.00	0.00
49	90	135	3.038	1.837	140	96	0.788	Non-Liq.	0.00	0.00
50.5	90	135	3.123	1.882	140	95	0.790	Non-Liq.	0.00	0.00

TOTAL SETTLEMENT = 2.6 INCHES

GEOCON

Client : CRENSHAW CROSSINGS
File No. : A9930-06-01
Boring : 7

EMPIRICAL ESTIMATION OF LIQUEFACTION POTENTIAL
MAXIMUM CONSIDERED EARTHQUAKE

NCEER (1996) METHOD

By Thomas F. Blake (1994-1996)

EARTHQUAKE INFORMATION:

Earthquake Magnitude:	6.76
Peak Horiz. Acceleration PGA_M (g):	0.733
Calculated Mag.Wtg.Factor:	0.770
Historic High Groundwater:	10.0
Groundwater Depth During Exploration:	20.0

ENERGY & ROD CORRECTIONS:

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

LIQUEFACTION CALCULATIONS:

Unit Wt. Water (pcf):	62.4

Depth to Base (ft)	Total Unit Wt. (pcf)	Water (0 or 1)	FIELD SPT (N)	Depth of SPT (ft)	Liq.Sus. (0 or 1)	-200 (%)	Est. Dr (%)	CN Factor	Corrected (N1)60	Eff. Unit Wt. (psf)	Resist. CRR	rd Factor	Induced CSR	Liquefac. Safe.Fact.
1.0	128.0	0	8.0	2.0	1		66	1.700	15.3	128.0	0.167	0.998	0.366	--
2.0	128.0	0	8.0	2.0	1		66	1.700	15.3	128.0	0.167	0.993	0.364	--
3.0	128.0	0	8.0	2.0	1		66	1.700	15.3	128.0	0.167	0.989	0.363	--
4.0	128.0	0	8.0	2.0	1		66	1.700	15.3	128.0	0.167	0.984	0.361	--
5.0	128.0	0	8.0	2.0	1		66	1.700	15.3	128.0	0.167	0.979	0.359	--
6.5	128.0	0	8.0	2.0	1		66	1.684	15.2	128.0	0.165	0.974	0.357	--
7.0	122.0	0	13.0	7.0	1	25	75	1.617	28.3	122.0	0.358	0.969	0.356	--
8.0	122.0	0	13.0	7.0	1	25	75	1.482	26.3	122.0	0.309	0.966	0.354	--
9.0	122.0	0	13.0	7.0	1	25	75	1.395	25.0	122.0	0.286	0.961	0.353	--
10.0	122.0	0	13.0	7.0	1	25	75	1.322	24.0	122.0	0.269	0.957	0.351	--
11.5	122.0	1	13.0	7.0	1	25	75	1.245	22.8	59.6	0.253	0.951	0.361	0.70
12.0	122.0	1	6.0	12.0	1	64	47	1.218	15.2	59.6	0.166	0.946	0.368	0.45
13.0	122.0	1	6.0	12.0	1	64	47	1.157	14.8	59.6	0.161	0.943	0.384	0.42
14.0	122.0	1	6.0	12.0	1	64	47	1.114	14.5	59.6	0.158	0.938	0.396	0.40
15.0	122.0	1	6.0	12.0	1	64	47	1.076	14.3	59.6	0.156	0.934	0.406	0.38
16.0	128.0	1	14.0	17.0	1	32	68	1.040	24.7	65.6	0.281	0.929	0.415	0.68
17.0	128.0	1	14.0	17.0	1	32	68	1.007	24.1	65.6	0.271	0.925	0.423	0.64
18.0	128.0	1	14.0	17.0	1	32	68	0.977	23.6	65.6	0.263	0.920	0.430	0.61
19.0	128.0	1	14.0	17.0	1	32	68	0.950	23.1	65.6	0.257	0.915	0.436	0.59
20.0	128.0	1	14.0	17.0	1	32	68	0.931	22.8	65.6	0.252	0.911	0.441	0.57
21.5	128.0	1	14.0	17.0	1	32	68	0.915	22.5	65.6	0.248	0.905	0.448	0.55
22.0	105.0	1	2.0	22.0	0			0.910	2.5	42.6	~	0.901	0.449	~
23.0	105.0	1	2.0	22.0	0			0.901	2.5	42.6	~	0.897	0.457	~
24.0	105.0	1	2.0	22.0	0			0.893	2.5	42.6	~	0.893	0.462	~
25.5	105.0	1	2.0	22.0	0			0.884	2.4	42.6	~	0.887	0.468	~
26.0	105.0	1	7.0	25.0	1		45	0.881	8.8	42.6	0.098	0.882	0.469	0.21
27.0	105.0	1	7.0	25.0	1		45	0.872	8.8	42.6	0.097	0.879	0.475	0.20
28.0	131.0	1	61.0	27.0	1		133	0.864	77.0	68.6	Infin.	0.874	0.477	Non-Liq.
29.0	131.0	1	61.0	27.0	1		133	0.853	76.1	68.6	Infin.	0.870	0.479	Non-Liq.
30.0	131.0	1	61.0	27.0	1		133	0.843	75.2	68.6	Infin.	0.865	0.480	Non-Liq.
31.0	131.0	1	61.0	27.0	1		133	0.834	74.3	68.6	Infin.	0.861	0.481	Non-Liq.
32.0	131.0	1	74.0	32.0	1		141	0.824	91.5	68.6	Infin.	0.856	0.482	Non-Liq.
33.0	131.0	1	74.0	32.0	1		141	0.815	90.5	68.6	Infin.	0.851	0.482	Non-Liq.
34.0	131.0	1	74.0	32.0	1		141	0.806	89.5	68.6	Infin.	0.847	0.483	Non-Liq.
35.0	123.0	1	74.0	32.0	1		141	0.798	88.6	60.6	Infin.	0.842	0.483	Non-Liq.
36.5	123.0	1	74.0	32.0	1		141	0.789	87.6	60.6	Infin.	0.837	0.484	Non-Liq.
37.0	123.0	1	53.0	37.0	1		115	0.786	62.5	60.6	Infin.	0.832	0.483	Non-Liq.
38.0	123.0	1	53.0	37.0	1		115	0.777	61.8	60.6	Infin.	0.829	0.484	Non-Liq.
39.0	123.0	1	53.0	37.0	1		115	0.770	61.2	60.6	Infin.	0.824	0.484	Non-Liq.
40.0	123.0	1	90.0	40.0	1		147	0.764	103.1	60.6	Infin.	0.819	0.484	Non-Liq.
41.0	123.0	1	90.0	40.0	1		147	0.757	102.3	60.6	Infin.	0.815	0.484	Non-Liq.
42.0	123.0	1	100.0	42.0	1		153	0.751	112.7	60.6	Infin.	0.810	0.484	Non-Liq.
43.0	123.0	1	100.0	42.0	1		153	0.745	111.8	60.6	Infin.	0.806	0.483	Non-Liq.
44.0	123.0	1	100.0	42.0	1		153	0.739	110.9	60.6	Infin.	0.801	0.483	Non-Liq.
45.0	123.0	1	100.0	42.0	1		153	0.733	110.0	60.6	Infin.	0.797	0.482	Non-Liq.
46.0	123.0	1	100.0	42.0	1		153	0.728	109.2	60.6	Infin.	0.792	0.481	Non-Liq.
47.0	122.0	1	31.0	47.0	1		83	0.722	33.6	59.6	Infin.	0.787	0.480	Non-Liq.
48.0	122.0	1	31.0	47.0	1		83	0.717	33.3	59.6	Infin.	0.783	0.479	Non-Liq.
49.0	122.0	1	31.0	47.0	1		83	0.712	33.1	59.6	Infin.	0.778	0.478	Non-Liq.
50.5	122.0	1	31.0	47.0	1		83	0.705	32.8	59.6	Infin.	0.773	0.477	Non-Liq.

GEOCON

Client : CRENSHAW CROSSINGS
File No. : A9930-06-01
Boring : 7

LIQUEFACTION SETTLEMENT ANALYSIS
MAXIMUM CONSIDERED EARTHQUAKE

(SATURATED SAND AT INITIAL LIQUEFACTION CONDITION)

NCEER (1996) METHOD
EARTHQUAKE INFORMATION:

Earthquake Magnitude:	6.76
PGA$_M$ (g):	0.733
Calculated Mag.Wtg.Factor:	0.770
Historic High Groundwater:	10.0
Groundwater @ Exploration:	20.0

DEPTH TO BASE	BLOW COUNT N	WET DENSITY (PCF)	TOTAL STRESS O (TSF)	EFFECT STRESS O' (TSF)	REL. DEN. Dr (%)	ADJUST BLOWS (N1)60	Tav/σ'_o	LIQUEFACTION SAFETY FACTOR	Volumetric Strain [e_{15}} (%)	EQ. SETTLE. Pe (in.)
1	8	128	0.032	0.032	66	15	0.476	--	0.00	0.00
2	8	128	0.096	0.096	66	15	0.476	--	0.00	0.00
3	8	128	0.160	0.160	66	15	0.476	--	0.00	0.00
4	8	128	0.224	0.224	66	15	0.476	--	0.00	0.00
5	8	128	0.288	0.288	66	15	0.476	--	0.00	0.00
6.5	8	128	0.368	0.368	66	15	0.476	--	0.00	0.00
7	13	122	0.399	0.399	75	28	0.476	--	0.00	0.00
8	13	122	0.476	0.476	75	26	0.476	--	0.00	0.00
9	13	122	0.537	0.537	75	25	0.476	--	0.00	0.00
10	13	122	0.598	0.598	75	24	0.476	--	0.00	0.00
11.5	13	122	0.674	0.650	75	23	0.494	0.70	1.30	0.23
12	6	122	0.704	0.665	47	15	0.504	0.45	1.70	0.10
13	6	122	0.781	0.703	47	15	0.529	0.42	1.80	0.22
14	6	122	0.842	0.732	47	15	0.547	0.40	1.80	0.22
15	6	122	0.903	0.762	47	14	0.564	0.38	1.80	0.22
16	14	128	0.965	0.793	68	25	0.579	0.68	1.30	0.16
17	14	128	1.029	0.826	68	24	0.593	0.64	1.30	0.16
18	14	128	1.093	0.859	68	24	0.606	0.61	1.30	0.16
19	14	128	1.157	0.892	68	23	0.618	0.59	1.30	0.16
20	14	128	1.221	0.925	68	23	0.629	0.57	1.30	0.16
21.5	14	128	1.301	0.966	68	22	0.642	0.55	1.40	0.25
22	2	105	1.330	0.979		3	0.647	~	0.00	0.00
23	2	105	1.396	1.006		2	0.661	~	0.00	0.00
24	2	105	1.448	1.027		2	0.672	~	0.00	0.00
25.5	2	105	1.514	1.054		2	0.685	~	0.00	0.00
26	7	105	1.540	1.064	45	9	0.689	0.21	2.70	0.16
27	7	105	1.606	1.091	45	9	0.701	0.20	2.70	0.32
28	61	131	1.665	1.119	133	77	0.709	Non-Liq.	0.00	0.00
29	61	131	1.730	1.153	133	76	0.715	Non-Liq.	0.00	0.00
30	61	131	1.796	1.187	133	75	0.721	Non-Liq.	0.00	0.00
31	61	131	1.861	1.222	133	74	0.726	Non-Liq.	0.00	0.00
32	74	131	1.927	1.256	141	91	0.731	Non-Liq.	0.00	0.00
33	74	131	1.992	1.290	141	90	0.736	Non-Liq.	0.00	0.00
34	74	131	2.058	1.325	141	90	0.740	Non-Liq.	0.00	0.00
35	74	123	2.121	1.357	141	89	0.745	Non-Liq.	0.00	0.00
36.5	74	123	2.198	1.395	141	88	0.751	Non-Liq.	0.00	0.00
37	53	123	2.229	1.410	115	62	0.753	Non-Liq.	0.00	0.00
38	53	123	2.306	1.448	115	62	0.759	Non-Liq.	0.00	0.00
39	53	123	2.367	1.478	115	61	0.763	Non-Liq.	0.00	0.00
40	90	123	2.429	1.508	147	103	0.767	Non-Liq.	0.00	0.00
41	90	123	2.490	1.539	147	102	0.771	Non-Liq.	0.00	0.00
42	100	123	2.552	1.569	153	113	0.775	Non-Liq.	0.00	0.00
43	100	123	2.613	1.599	153	112	0.779	Non-Liq.	0.00	0.00
44	100	123	2.675	1.630	153	111	0.782	Non-Liq.	0.00	0.00
45	100	123	2.736	1.660	153	110	0.785	Non-Liq.	0.00	0.00
46	100	123	2.798	1.690	153	109	0.789	Non-Liq.	0.00	0.00
47	31	122	2.859	1.720	83	34	0.792	Non-Liq.	0.00	0.00
48	31	122	2.920	1.750	83	33	0.795	Non-Liq.	0.00	0.00
49	31	122	2.981	1.780	83	33	0.798	Non-Liq.	0.00	0.00
50.5	31	122	3.057	1.817	83	33	0.802	Non-Liq.	0.00	0.00

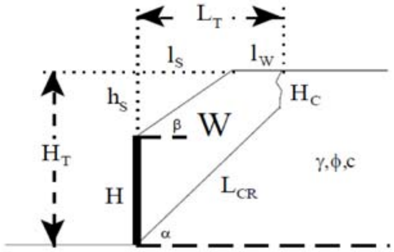
TOTAL SETTLEMENT = 2.5 INCHES

Figure 13

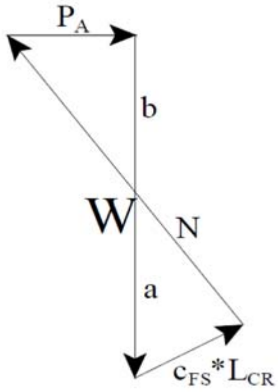
Retaining Wall Design with Transitioned Backfill
(Vector Analysis)

Input:

Retaining Wall Height	(H)	10.00	feet
Slope Angle of Backfill	(b)	0.0	degrees
Height of Slope above Wall	(h_s)	0.0	feet
Horizontal Length of Slope	(l_s)	0.0	feet
Total Height (Wall + Slope)	(H_T)	10.0	feet
Unit Weight of Retained Soils	(g)	120.0	pcf
Friction Angle of Retained Soils	(f)	30.0	degrees
Cohesion of Retained Soils	(c)	130.0	psf
Factor of Safety	(FS)	1.50	
Factored Parameters:	(f_{FS})	21.1	degrees
	(c_{FS})	86.7	psf



Failure Angle (a) degrees	Height of Tension Crack (H_C) feet	Area of Wedge (A) $feet^2$	Weight of Wedge (W) lbs/lineal foot	Length of Failure Plane (L_{CR}) feet	a lbs/lineal foot	b lbs/lineal foot	Active Pressure (P_A) lbs/lineal foot
45	2.3	47	5669.1	10.8	2156.2	3512.9	1560.2
46	2.3	46	5487.5	10.7	2052.5	3435.0	1598.0
47	2.3	44	5309.7	10.6	1956.6	3353.0	1631.6
48	2.2	43	5135.5	10.5	1867.8	3267.7	1661.3
49	2.2	41	4965.1	10.3	1785.4	3179.7	1687.0
50	2.2	40	4798.3	10.2	1708.8	3089.5	1708.9
51	2.1	39	4635.1	10.1	1637.5	2997.6	1727.0
52	2.1	37	4475.3	10.0	1571.0	2904.3	1741.5
53	2.1	36	4318.8	9.9	1508.8	2810.0	1752.3
54	2.1	35	4165.5	9.8	1450.6	2714.8	1759.6
55	2.1	33	4015.2	9.6	1396.1	2619.2	1763.2
56	2.1	32	3867.9	9.5	1344.8	2523.1	1763.3
57	2.1	31	3723.3	9.4	1296.5	2426.8	1759.8
58	2.1	30	3581.3	9.3	1250.9	2330.4	1752.8
59	2.1	29	3441.9	9.2	1207.9	2234.0	1742.1
60	2.1	28	3304.8	9.1	1167.1	2137.7	1727.9
61	2.2	26	3169.9	9.0	1128.4	2041.5	1709.9
62	2.2	25	3037.2	8.8	1091.5	1945.6	1688.2
63	2.2	24	2906.3	8.7	1056.4	1850.0	1662.7
64	2.3	23	2777.4	8.6	1022.7	1754.6	1633.3
65	2.3	22	2650.1	8.5	990.4	1659.7	1599.8
66	2.3	21	2524.4	8.4	959.3	1565.1	1562.3
67	2.4	20	2400.1	8.3	929.1	1471.0	1520.5
68	2.5	19	2277.2	8.1	899.8	1377.3	1474.3
69	2.5	18	2155.4	8.0	871.2	1284.2	1423.7
70	2.6	17	2034.7	7.9	843.1	1191.6	1368.3



Design Equations (Vector Analysis):

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

$b = W-a$

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

$EFP = 2*P_A/H^2$

Maximum Active Pressure Resultant

$P_{A,\,max}$ 1763.3 lbs/lineal foot

Equivalent Fluid Pressure (per lineal foot of wall) At-Rest= γ*(1-sin(ϕ))

$EFP = 2*P_A/H^2$

EFP 35.3 pcf 60.0 pcf

Design Wall for an Equivalent Fluid Pressure: **35** **pcf** **60** **pcf**

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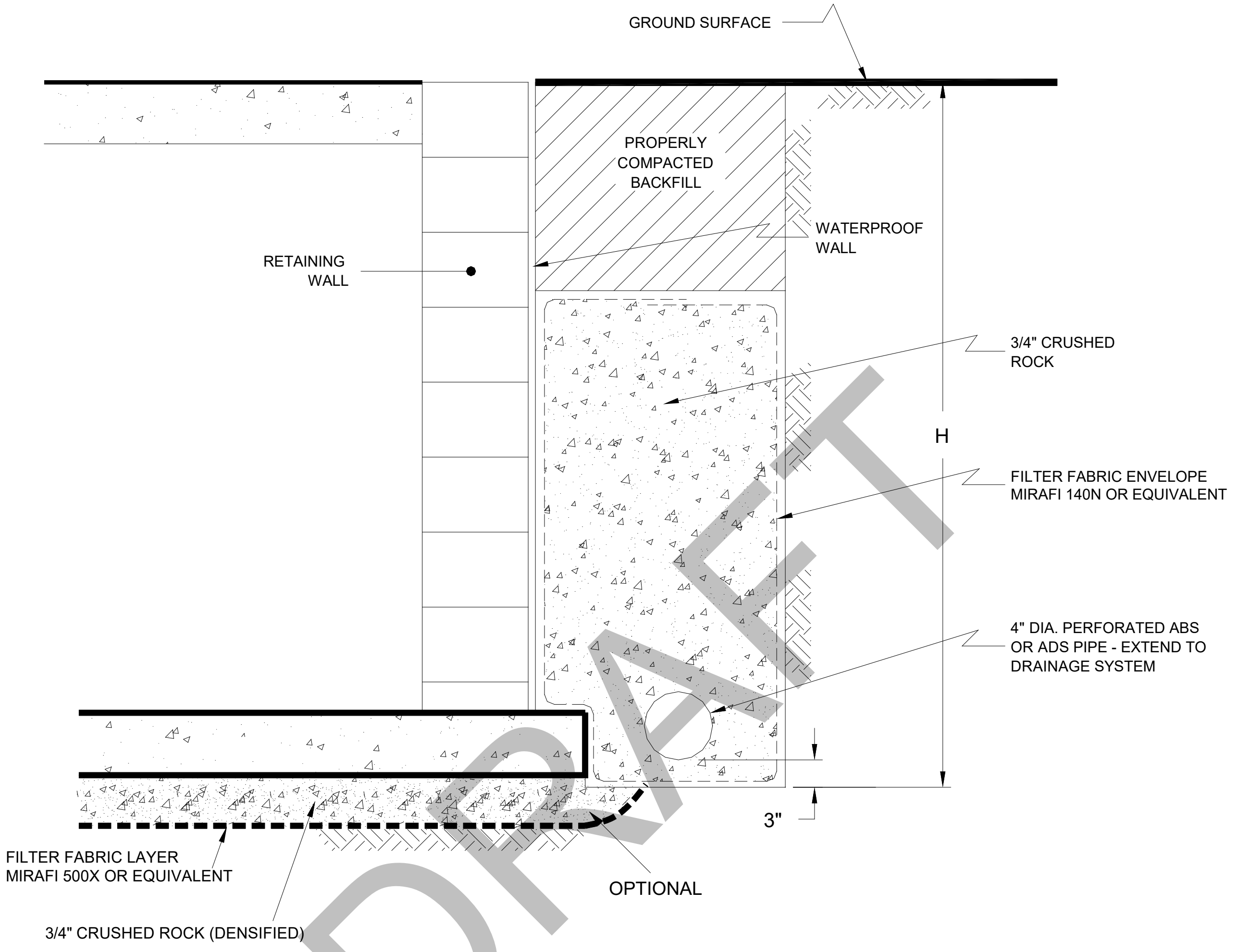
RETAINING WALL CALCULATION

SOUTHWEST AND SOUTHEAST CORNERS OF
EXPOSITION AND CRENSHAW BOULEVARD
LOS ANGELES, CALIFORNIA

AUGUST 2019 PROJECT NO. A9930-06-01 FIG. 14



NO SCALE

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RETAINING WALL DRAIN DETAIL

SOUTHWEST AND SOUTHEAST CORNERS OF
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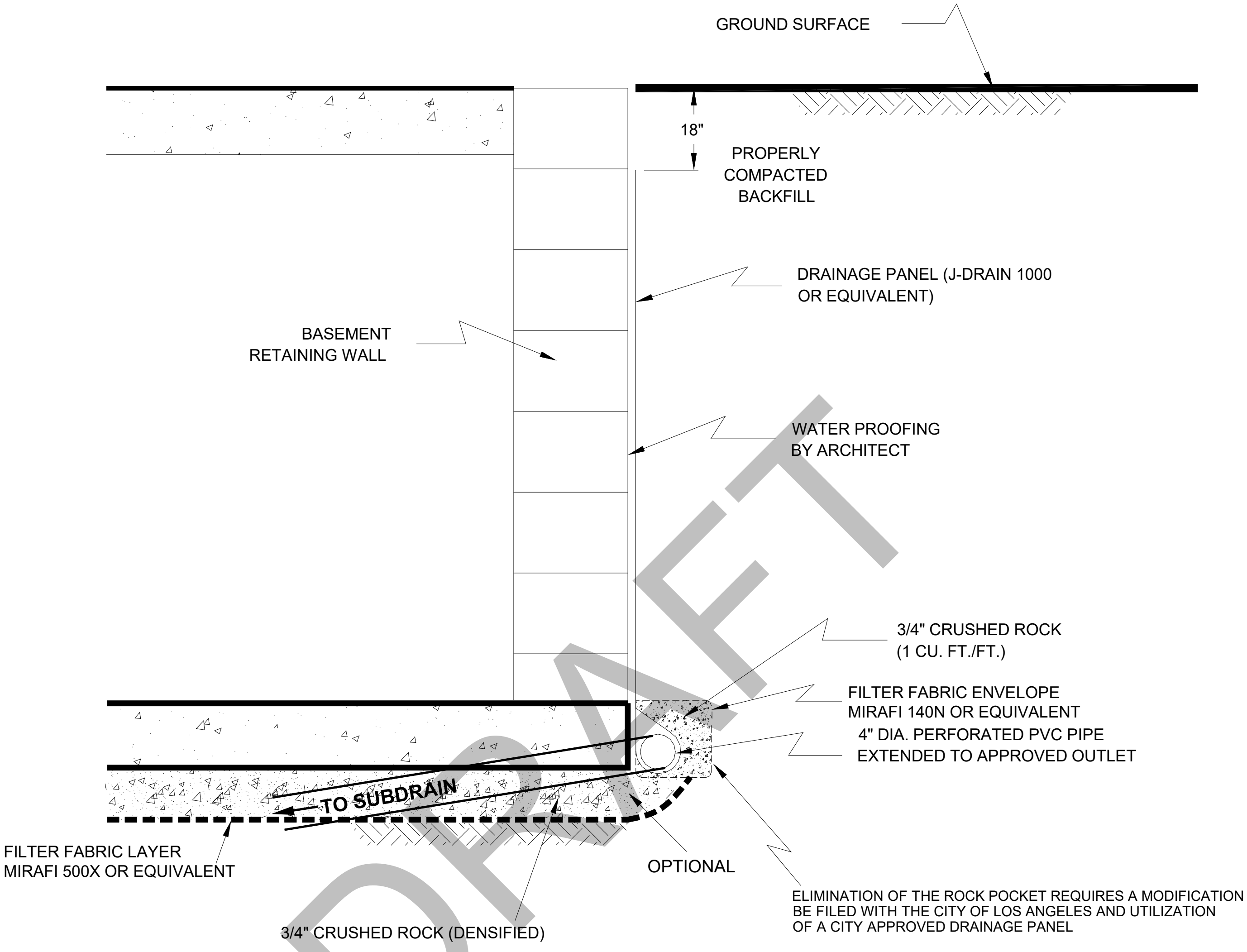
LOS ANGELES, CALIFORNIA

DRAFTED BY: RP	CHECKED BY: JTA	AUGUST 2019	PROJECT NO. A9930-06-01	FIG. 15



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

NO SCALE

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RETAINING WALL DRAIN DETAIL

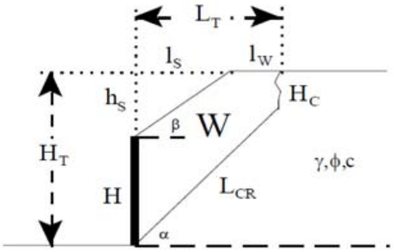
SOUTHWEST AND SOUTHEAST CORNERS OF
EXPOSITION AND CRENSHAW BOULEVARD
LOS ANGELES, CALIFORNIA

DRAFTED BY: RP	CHECKED BY: JTA	AUGUST 2019	PROJECT NO. A9930-06-01	FIG. 16

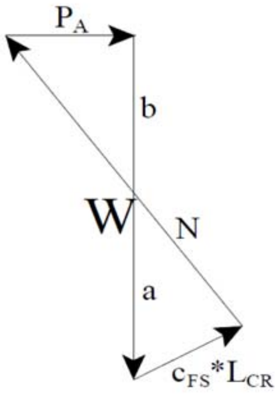
Shoring Design with Transitioned Backfill
(Vector Analysis)

Input:

Shoring Height	(H)	12.00	feet
Slope Angle of Backfill	(b)	0.0	degrees
Height of Slope above Shoring	(h_s)	0.0	feet
Horizontal Length of Slope	(l_s)	0.0	feet
Total Height (Shoring + Slope)	(H_T)	12.0	feet
Unit Weight of Retained Soils	(g)	120.0	pcf
Friction Angle of Retained Soils	(f)	30.0	degrees
Cohesion of Retained Soils	(c)	130.0	psf
Factor of Safety	(FS)	1.25	
Factored Parameters:	(f_{FS})	24.8	degrees
	(c_{FS})	104.0	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	3.2	67	8017.5	12.4	3393.3	4624.2	1702.2
46	3.1	65	7775.6	12.3	3217.8	4557.8	1768.6
47	3.1	63	7535.7	12.2	3056.1	4479.6	1828.9
48	3.0	61	7298.5	12.1	2906.8	4391.8	1883.1
49	2.9	59	7064.5	12.0	2768.7	4295.8	1931.4
50	2.9	57	6834.0	11.9	2640.9	4193.1	1973.9
51	2.8	55	6607.2	11.8	2522.3	4084.9	2010.8
52	2.8	53	6384.1	11.7	2412.1	3972.0	2042.1
53	2.8	51	6164.8	11.6	2309.5	3855.3	2068.0
54	2.7	50	5949.3	11.4	2213.8	3735.5	2088.5
55	2.7	48	5737.5	11.3	2124.4	3613.1	2103.6
56	2.7	46	5529.3	11.2	2040.7	3488.7	2113.6
57	2.7	44	5324.7	11.1	1962.1	3362.6	2118.2
58	2.7	43	5123.3	11.0	1888.3	3235.1	2117.7
59	2.7	41	4925.3	10.8	1818.7	3106.6	2111.9
60	2.7	39	4730.3	10.7	1753.0	2977.3	2100.9
61	2.7	38	4538.2	10.6	1690.9	2847.4	2084.6
62	2.8	36	4348.9	10.5	1631.9	2717.1	2063.0
63	2.8	35	4162.3	10.3	1575.8	2586.5	2036.0
64	2.8	33	3978.1	10.2	1522.3	2455.8	2003.5
65	2.9	32	3796.2	10.1	1471.1	2325.1	1965.5
66	2.9	30	3616.5	9.9	1421.9	2194.6	1921.8
67	3.0	29	3438.7	9.8	1374.5	2064.2	1872.3
68	3.1	27	3262.7	9.6	1328.5	1934.1	1816.8
69	3.1	26	3088.2	9.5	1283.8	1804.4	1755.3
70	3.2	24	2915.2	9.3	1240.0	1675.2	1687.5



Design Equations (Vector Analysis):

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

$b = W-a$

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

$EFP = 2*P_A/H^2$

Maximum Active Pressure Resultant

$P_{A,\ max}$ 2118.2 lbs/lineal foot

Equivalent Fluid Pressure (per lineal foot of shoring)

$EFP = 2*P_A/H^2$

EFP 29.4 pcf

Design Shoring for an Equivalent Fluid Pressure: **29 pcf**

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SHORING WALL CALCULATION

SOUTHWEST AND SOUTHEAST CORNERS OF
EXPOSITION AND CRENSHAW BOULEVARD
LOS ANGELES, CALIFORNIA

AUGUST 2019 | PROJECT NO. A9930-06-01 | FIG. 17

APPENDIX A

APPENDIX A

FIELD INVESTIGATION

The site was explored on June 27, 2019, and July 6, 2019, by excavating twelve 8-inch-diameter borings using a truck-mounted hollow-stem auger drilling machine. The borings were drilled to depths ranging from approximately 10½ to 50½ feet below existing ground surface. Representative and relatively undisturbed samples were obtained from the borings 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 sampler rings to facilitate soil removal and testing. Standard Penetration Tests (SPTs) were also performed. Bulk samples were obtained.

The soil conditions encountered in the borings were visually examined, classified and logged in general accordance with the Unified Soil Classification System (USCS). The logs of the borings are presented on Figures A1 through A12. The logs depict the soil and geologic conditions encountered and the depth at which samples were obtained. The logs also include our interpretation of the conditions between sampling intervals. Therefore, the logs contain both observed and interpreted data. We determined the lines designating the interface between soil materials on the logs using visual observations, penetration rates, excavation characteristics and other factors. The transition between materials may be abrupt or gradual. Where applicable, the logs were revised based on subsequent laboratory testing. The location of the borings are shown on Figure 2.

PROJECT NO. A9930-06-01

BORING 1

ELEV. (MSL.) -- DATE COMPLETED 6/27/19

EQUIPMENT HOLLOW STEM AUGER BY: RP

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0					**ASP: 3.5" BASE: 4"** **ARTIFICIAL FILL** Silt, soft, slightly moist, dark brown, fine-grained.			
2	B1@2'	■			**ALLUVIUM** Silt, firm, slightly moist, olive brown, fine-grained, trace clay, trace oxidation staining, slightly porous.	11	---	---
4				ML				
	B1@5'	■			- stiff, olive brown with light brown mottles - decrease in porosity	26	98.5	21.4
6								
	B1@7'	■		SM	Silty Sand, medium dense, slightly moist, light olive brown, fine-grained, trace silt.	13	---	---
8								
10	B1@10'	■		ML	Silt with Sand, soft, slightly moist, olive brown, some silt, trace interbedded silty sand.	16	85.9	26.4
12	B1@12'	■			- dark olive brown	4	---	---
14				SM	Silty Sand, loose, slightly moist, olive brown, fine-grained, trace medium-grained, slight increase in moisture, some oxidation mottling.			
	B1@15'	■				16	106.6	22.2
16								
	B1@17'	■		SP-SM	Sand with Silt, poorly graded, medium dense, wet, olive brown, fine-grained, trace medium-grained, some oxidation mottling.	13	---	---
18			▼					
20	B1@20'	■		CH	Organic Clay, soft, moist, black to grayish brown, some interbedded organics, high plasticity.	25	97.4	27.1
22	B1@22'	■				2		
24				ML	Sandy Silt, soft, moist, olive brown, fine-grained, trace clay, trace interbedded organics.			
	B1@25'	■				9	---	---
26								
	B1@27'	■			- brown, increase in sand, little to no organics	9	---	---
28								
				SM	Silty Sand with Gravel, very dense, wet, brown, fine- to medium-grained,			

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

A9930-06-01 BORING LOGS.GPJ

SAMPLE SYMBOLS

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

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

PROJECT NO. A9930-06-01

BORING 1

ELEV. (MSL.) -- DATE COMPLETED 6/27/19

EQUIPMENT **HOLLOW STEM AUGER** BY: **RP**

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'			SM	gravel (to 2"), trace clay.	50 (4")	123.5	10.2
32	B1@32'				Sand with Gravel, poorly-graded, very dense, wet, light brown, medium- to coarse-grianed, gravel (to 1").	50 (4")	---	---
34								
	B1@35'					50 (4")	117.0	14.5
36								
	B1@37'					50 (2")	---	---
38								
40	B1@40'			SP		50 (2")	128.1	10.9
42	B1@42'					50 (6")	---	---
44								
	B1@45'				- fine- to medium-grained	50 (2")	134.3	8.9
46								
	B1@47'					50 (4")	---	---
48								
50	B1@50'				- medium- to coarse-grained	50 (5")	128.4	10.1

Total depth of boring: 50.5 feet
Fill to 2 feet.
Groundwater encountered at 18 feet.
Grouted and concrete patched.

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

A9930-06-01 BORING LOGS.GPJ

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

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

PROJECT NO. A9930-06-01

BORING 2

ELEV. (MSL.) -- DATE COMPLETED 6/27/19

EQUIPMENT HOLLOW STEM AUGER BY: RP

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0	BULK 0-5'				**ASP: 3" BASE: 4"** **ARTIFICIAL FILL** Sandy Silt, soft, slightly moist, brown with grayish brown mottles, fine-grained.			
2								
4	B2@3'			CL	**ALLUVIUM** Clay, firm, slightly moist, grayish brown with olive mottles, fine-grained, some silt.	14	85.7	23.3
6	B2@6'					27	110.9	20.7
8				SP-SM	Sand with Silt, medium dense, slightly moist, olive brown, fine-grained, some oxidation staining.			
10	B2@9'				Silt with Sand, firm, slightly moist, olive brown, fine-grained, trace clay.	12	87.3	36.1
12	B2@12'				- soft, trace cemented fragments, slight increase in moisture	8	94.8	25.8
14								
16	B2@15'				- trace rootlets	10	105.1	23.6
18				ML				
20	B2@20'				- firm, grayish brown	16	83.0	40.7
22								
24								
26	B2@25'				- some interbedded organics	12	77.2	41.4
28								
				SP	Sand with Gravel, poorly-graded, very dense, slightly moist, light brown,			

Figure A2,
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.

PROJECT NO. A9930-06-01

BORING 2

ELEV. (MSL.) **--** DATE COMPLETED **6/27/19**

EQUIPMENT **HOLLOW STEM AUGER** BY: **RP**

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' ■			SP	fine-grained, gravel (to 1").	50 (5")	127.3	7.8

Total depth of boring: 30.5 feet
Fill to 3 feet.
No groundwater encountered.
Grouted and concrete patched.

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

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

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

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

PROJECT NO. A9930-06-01

BORING 3

ELEV. (MSL.) -- DATE COMPLETED 6/27/19

EQUIPMENT **HOLLOW STEM AUGER** BY: **RP**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0 2	BULK 0-5'				**ASP: 3" BASE: 4"** **ARTIFICIAL FILL** Sandy Silt, soft, slightly moist, olive brown with brown mottles, fine-grained.			
4	B3@3'			ML	**ALLUVIUM** Silt with Sand, soft, slightly moist, olive brown, fine-grained, trace oxidation staining.	13	88.2	31.0
6	B3@6'			CL	Clay, firm, slightly moist, light olive brown, fine-grained, some silt.	19	96.4	27.9
8	B3@9'			ML	Silt with Sand, soft, slightly moist, olive brown, fine-grained.	8	92.3	26.6
10				CL	Clay, soft, slightly moist, dark grayish brown, fine-grained, slightly plastic.			
12	B3@12'			CL	Sandy Clay, soft, slightly moist, olive brown, fine-grained, trace cemented fragments.	10	106.0	22.3
14 16	B3@15'			SM	Silty Sand, loose, slightly moist, olive brown, fine-grained.	11	106.4	21.7
18			▼	SP-SM	Sand with Silt, poorly-graded, loose, wet, grayish brown, fine-grained.			
20 22 24	B3@20'				- interbedded organics	13	83.8	34.3
26 28	B3@25'				- moist - interbedded organics	10	92.2	31.7
				SP	Sand with Gravel, poorly-graded, very dense, slightly moist, light brown,			

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

A9930-06-01 BORING LOGS.GPJ

SAMPLE SYMBOLS

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

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

PROJECT NO. A9930-06-01

BORING 3

ELEV. (MSL.) **--** DATE COMPLETED **6/27/19**

EQUIPMENT **HOLLOW STEM AUGER** BY: **RP**

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	fine- to medium-grained, gravel (to 3").	50 (6")	132.3	7.9
					Total depth of boring: 30.5 feet Fill to 3 feet. Groundwater encountered at 18 feet. Grouted and concrete patched. *Penetration resistance for 140-pound hammer falling 30 inches by auto-hammer.			

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

A9930-06-01 BORING LOGS.GPJ

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

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GEOCON

PROJECT NO. A9930-06-01

BORING 4

ELEV. (MSL.) **--** DATE COMPLETED **6/27/19**

EQUIPMENT **HOLLOW STEM AUGER** BY: **RP**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0	BULK 0-5'				**ASP: 3.5" BASE: 5"** **ARTIFICIAL FILL** Sandy Silt, soft, slightly moist, olive brown, fine-grained, trace fine gravel.			
2								
	B4@3'			ML	**ALLUVIUM** Silt, soft, slightly moist, olive brown, fine-grained, trace cemented fragments, trace rootlets.	8	90.0	28.5
4				CL	Clay, soft, slightly moist, olive brown, fine-grained, some oxidation staining, some silt.			
6	B4@6'				- firm	21	96.8	26.4
				SP-SM	Sand with Silt, medium dense, slightly moist, olive brown, fine-grained.			
8								
10	B4@10'				- loose	10	84.4	24.4

Total depth of boring: 10.5 feet
Fill to 2.5 feet.
No groundwater encountered.
Backfilled with soil cuttings and tamped.
Asphalt patched.

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

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

A9930-06-01 BORING LOGS.GPJ

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

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

PROJECT NO. A9930-06-01

BORING 5

ELEV. (MSL.) -- DATE COMPLETED 6/27/19

EQUIPMENT HOLLOW STEM AUGER BY: RP

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0 2	BULK 0-5'	☒			**ASP: 4" BASE: 4"** **ARTIFICIAL FILL** Sandy Silt, soft, slightly moist, brown to olive brown, fine-grained.			
4	B5@3'	■		SP-SM	**ALLUVIUM** Sand with Silt, medium dense, slightly moist, olive brown, fine-grained, some oxidation staining.	22	105.7	19.3
6 8	B5@6'	■		ML	Sandy Silt, firm, slightly moist, olive brown with light brown mottles, fine-grained, some oxidation staining.	26	105.2	22.9
10	B5@10'	■		SP-SM	Sand with Silt, loose, slightly moist, olive brown, fine-grained, trace clay.	10	90.5	24.7

Total depth of boring: 10.5 feet
Fill to 3 feet.
No groundwater encountered.
Backfilled with soil cuttings and tamped.
Concrete patched.

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

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

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

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

GEOCON

PROJECT NO. A9930-06-01

BORING 6

ELEV. (MSL.) **--** DATE COMPLETED **6/27/19**

EQUIPMENT **HOLLOW STEM AUGER** BY: **RP**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0 2	BULK 0-5'				**ASP: 3" BASE: 4"** **ARTIFICIAL FILL** Sandy Silt, soft, slightly moist, olive brown with brown mottles, fine-grained.			
4	B6@3'				**ALLUVIUM** Sandy Silt, firm, slightly moist, olive brown with light brown mottles, fine-grained, some oxidation staining.	18	89.3	28.5
6 8	B6@6'			ML	- some clay, slightly porous	18	91.9	28.7
10	B6@10'			ML	Silt with Sand, soft, slightly moist, olive brown, fine-grained.	10	85.0	35.7

Total depth of boring: 10.5 feet
Fill to 3 feet.
No groundwater encountered.
Backfilled with soil cuttings and tamped.
Concrete patched.

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

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

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. A9930-06-01

BORING 7

ELEV. (MSL.) -- DATE COMPLETED **7/6/19**

EQUIPMENT **HOLLOW STEM AUGER** BY: **RP**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0	BULK 0-5'				**ASP: 4" BASE: NONE** **ARTIFICIFIAL FILL** Sandy Silt, soft, slightly moist, brown to dark brown, fine-grained, some clay, trace coarse-grained.			
2	B7@2'			ML	**ALLUVIUM** Silt, soft, slightly moist, olive brown with dark brown mottles, fine-grained, some interbedded clay.	8	---	---
4								
6	B7@5'			ML	Silt with Clay, firm, slightly moist, dark olive brown, fine-grained, trace cemented fragments.	19	101.7	25.6
8	B7@7'			SM	Silty Sand, medium dense, slightly moist, olive brown, fine-grained.	13	---	---
10	B7@10'				- loose, olive brown with light brown mottles, trace clay	9	89.0	37.3
12	B7@12'			CL	Sandy Clay, soft, slightly moist, olive to light brown, fine-grained, trace oxidation staining.	6	---	---
14								
16	B7@15'			SM	Silty Sand, loose, slightly moist to moist, olive brown, fine-grained, some clay.	13	103.5	24.5
18	B7@17'				- medium dense, grayish brown, some oxidation staining	14	---	---
20	B7@20'		▼		- loose, moist, decreased sand	17	92.7	31.6
22	B7@22'			CH	Organic Clay, very soft, slightly moist, olive with black mottles, fine-grained, high plasticity.	2	---	---
24								
26	B7@25'			SM	Silty Sand, loose, slightly moist, olive brown, fine-grained, trace interbedded organics.	13	88.5	19.0
28	B7@27'			SP	Sand with Gravel, poorly graded, dense, slightly moist, light brown, fine-grained, gravel (to 1").	61	---	---

Figure A7,
Log of Boring 7, 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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PROJECT NO. A9930-06-01

BORING 7

ELEV. (MSL.) **--** DATE COMPLETED **7/6/19**

EQUIPMENT **HOLLOW STEM AUGER** BY: **RP**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
30	B7@30'				- very dense, olive brown, fine- to medium-grained, some oxidation staining	50 (6")	121.3	8.3
32	B7@32'			SP	- dense, light brown	74	---	---
34								
	B7@35'				- very dense, no recovery	50 (5")	103.9	8.0
36								
	B7@37'				Sand with Gravel, well-graded, dense, slightly moist, light brown, fine- to coarse-grained, gravel (to 1").	53	---	
38								
40	B7@40'				- very dense - rig chatter	50 (3")	115.0	7.3
42	B7@42'			SW	- no recovery	50 (3")	---	---
44								
	B7@45'				- no recovery	50 (6")	---	---
46								
	B7@47'			SM	Silty Sand, dense, slightly moist, yellowish brown, fine- to medium-grained, trace gravel.	31	---	---
48				SW	Sand with Gravel, well-graded, dense, slightly moist, light brown, fine- to coarse-grained.			
50	B7@50'				- very dense	50 (6")	105.5	16.3

Total depth of boring: 50.5 feet
Fill to 2 feet.
Seepage encountered at 20 feet.
Grouted and surface restored with concrete.

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

Figure A7,
Log of Boring 7, 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. A9930-06-01

BORING 8

ELEV. (MSL.) **--** DATE COMPLETED **7/6/19**

EQUIPMENT **HOLLOW STEM AUGER** BY: **RP**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0	BULK 0-5'				**ARTIFICIAL FILL** Sandy Silt, firm, slightly moist, brown, fine-grained, some clay.			
2								
	B8@3'					15	99.7	23.8
4				CL	**ALLUVIUM** Clay, firm, slightly moist, olive brown to light brown mottles, some silt.			
6	B8@6'			SM	Silty Sand, medium dense, slightly moist, olive brown, fine-grained, trace clay.	35	112.1	17.5
8								
	B8@9'					9	88.9	26.41
10					Silt with Sand, soft, slightly moist, olive brown, fine-grained, some clay.			
12	B8@12'				- firm, brown to dark olive brown	12	95.2	29.4
14								
	B8@15'				- olive brown, slight increase in silt	18	100.4	26.0
16								
18				ML				
20	B8@20'				- stiff, dark olive	27	96.8	29.1
22								
24								
	B8@25'				- soft, dark olive brown - some interbedded clay organics	13	87.9	35.2
26								
28								

A9930-06-01 BORING LOGS.GPJ

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

SAMPLE SYMBOLS

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

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PROJECT NO. A9930-06-01

BORING 8

ELEV. (MSL.) -- DATE COMPLETED 7/6/19

EQUIPMENT **HOLLOW STEM AUGER** BY: **RP**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
30	B8@30' ■			SM	Silty Sand, dense, slightly moist, brown, fine-grained, trace oxidation staining, trace fine gravel.	67	126.8	10.9
					Total depth of boring: 30.5 feet Fill to 3.5 feet. No groundwater encountered. Grouted and surface restored. *Penetration resistance for 140-pound hammer falling 30 inches by auto-hammer.			

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

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

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PROJECT NO. A9930-06-01

BORING 9

ELEV. (MSL.) **--** DATE COMPLETED **7/6/19**

EQUIPMENT **HOLLOW STEM AUGER** BY: **RP**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0	BULK 0-5'				**ARTIFICIAL FILL** Sandy Silt, firm, slightly moist, brown, fine-grained, some fine gravel.			
2								
4				ML	**ALLUVIUM** Silt with Clay, firm, slightly moist, olive brown with dark olive mottles, fine-grained.			
6	B9@5'					18	97.6	25.2
8				SM	Silty Sand, loose, slightly moist, yellowish brown, fine-grained. - olive brown to gray, slight decrease in silt			
10	B9@10'					17	95.4	18.4
12				ML	Silt with Sand, firm, slightly moist, olive brown.			
14				SP-SM	Sand with Silt, poorly graded, loose, slightly moist to moist, brown to yellowish brown, fine-grained.			
16	B9@15'					13	105.4	20.9
18								
20	B9@20'				- medium dense, dark olive brown, slight decrease in sand - increase in sand	28	95.4	28.5
22								
24				CL	Clay, soft, slightly moist, dark olive brown with black mottles, fine-grained, some interbedded organics, some silt.			
26	B9@25'				- decrease in organics	10	71.5	49.0
28				SW	Sand with Gravel, poorly graded, dense, slightly moist, reddish brown, fine-grained, gravel (to 1.5").			

A9930-06-01 BORING LOGS.GPJ

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

SAMPLE SYMBOLS

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

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BORING 9

ELEV. (MSL.) **--** DATE COMPLETED **7/6/19**

EQUIPMENT **HOLLOW STEM AUGER** BY: **RP**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
30	B9@30' ■			SW		80	118.8	7.3
					Total depth of boring: 30.5 feet Fill to 3 feet. No groundwater encountered. Grouted and surface restored. *Penetration resistance for 140-pound hammer falling 30 inches by auto-hammer.			

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

SAMPLE SYMBOLS

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

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BORING 10

ELEV. (MSL.) **--** DATE COMPLETED **7/6/19**

EQUIPMENT **HOLLOW STEM AUGER** BY: **RP**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0	BULK 0-5'				**ARTIFICIAL FILL** Sandy Silt, firm, slightly moist, brown, fine-grained, some fine gravel.			
2								
					Silty Sand, loose, slightly moist, olive brown, fine- to medium-grained.			
4								
	B10@5'					12	106.3	14.4
6					**ALLUVIUM** Clay, firm, slightly moist, olive brown, fine-grained, some silt.			
8				CL				
10	B10@10'				- no recovery	12	---	---

Total depth of boring: 10.5 feet
Fill to 5.5 feet.
No groundwater encountered.
Backfilled with soil cuttings and tamped.
Surface restored.

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

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

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

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

PROJECT NO. A9930-06-01

BORING 11

ELEV. (MSL.) -- DATE COMPLETED 7/6/19

EQUIPMENT **HOLLOW STEM AUGER** BY: **RP**

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** Sandy Silt, soft, slightly moist, dark brown, fine-grained, some clay, trace fine gravel.			
2								
4								
	B11@5'			CL	**ALLUVIUM** Clay, firm, slightly moist, dark olive brown, fine-grained, some silt. - no recovery	20	---	---
6								
8				ML	Silt with Sand, firm, slightly moist, light brown to yellowish brown, some clay.			
10	B11@10'				- no recovery	11	---	---
	B11@10.5				- soft	5	---	---

Total depth of boring: 11.5 feet
Fill to 5 feet.
No groundwater encountered.
Backfilled with soil cuttings and tamped.
Surface restored.

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

Figure A11,
Log of Boring 11, Page 1 of 1

A9930-06-01 BORING LOGS.GPJ

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

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PROJECT NO. A9930-06-01

BORING 12

ELEV. (MSL.) **--** DATE COMPLETED **7/6/19**

EQUIPMENT **HOLLOW STEM AUGER** BY: **RP**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0		▩			**ARTIFICIAL FILL** Silty Sand, medium dense, slightly moist, yellowsh brown, fine- to medium-grained, some fine gravel. - gravelly, light brown, fine- to coarse-grained			
2		▩						
4		▩						
	B12@5'	▨			**ALLUVIUM** Clay, firm, slightly moist, olive brown, some silt, trace oxidation staining.	15	108.5	7.3
6		▨		CL				
8		▨						
		░		SP-SM	Sand with Silt, loose, slightly moist, brown, fine-grained, trace oxidation staining.			
10	B12@10'	░				12	94.0	8.3

Total depth of boring: 10.5 feet
Fill to 5 feet.
No groundwater encountered.
Backfilled with soil cuttings and tamped.
Surface restored.

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

Figure A12,
Log of Boring 12, Page 1 of 1

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

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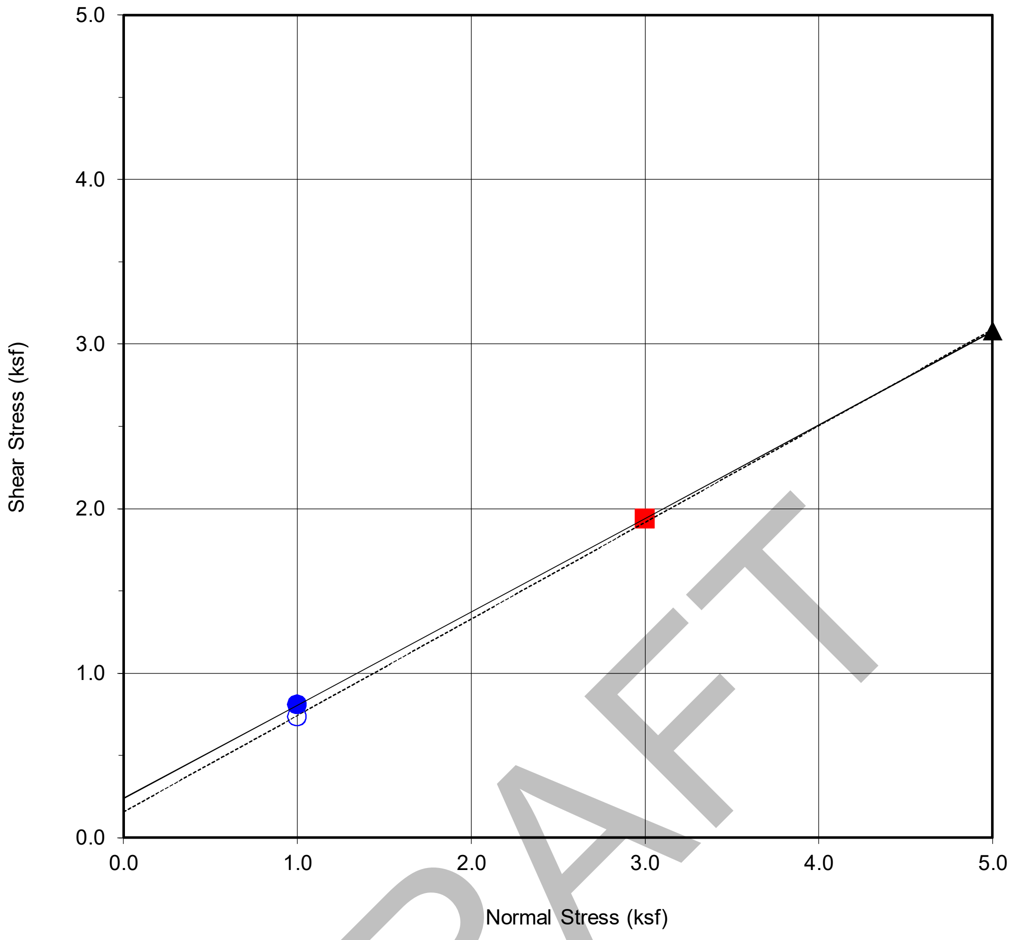
APPENDIX B

APPENDIX B

LABORATORY TESTING

Laboratory tests were performed in accordance with generally accepted test methods of the International ASTM, or other suggested procedures. Selected samples were tested for direct shear strength, consolidation characteristics, plasticity indices, grain size, expansive potential, moisture density relationships, corrosivity, in-place dry density and moisture content. The results of the laboratory tests are summarized in Figures B1 through B43. The in-place dry density and moisture content of the samples tested are presented on the boring logs, Appendix A.



Boring No.	**B7B8**
Sample No.	**B7&B8@0-5**
Depth (ft)	**0-5**
Sample Type:	Ring

Normal Stress (kip/ft2)	1	3	5
Peak Shear Stress (kip/ft²)	● 0.81	🟥 1.94	▲ 3.08
Shear Stress @ End of Test (ksf)	○ 0.73	□ 1.94	△ 3.08
Deformation Rate (in./min.)	0.05	0.05	0.05
Initial Sample Height (in.)	1.0	1.0	1.0
Ring Inside Diameter (in.)	2.375	2.375	2.375
Initial Moisture Content (%)	13.4	13.4	13.5
Initial Dry Density (pcf)	109.0	109.0	109.0
Initial Degree of Saturation (%)	66.4	66.2	66.9
Soil Height Before Shearing (in.)	1.2	1.2	1.2
Final Moisture Content (%)	21.4	18.3	18.9

Soil Identification:

Brown to Drk. Brown Sandy Silt with Clay (ML)

Strength Parameters

	C (psf)	ϕ (°)
Peak	237	29.6
Ultimate	157	30.4

GEOCON

DIRECT SHEAR TEST RESULTS

Consolidated Drained ASTM D-3080

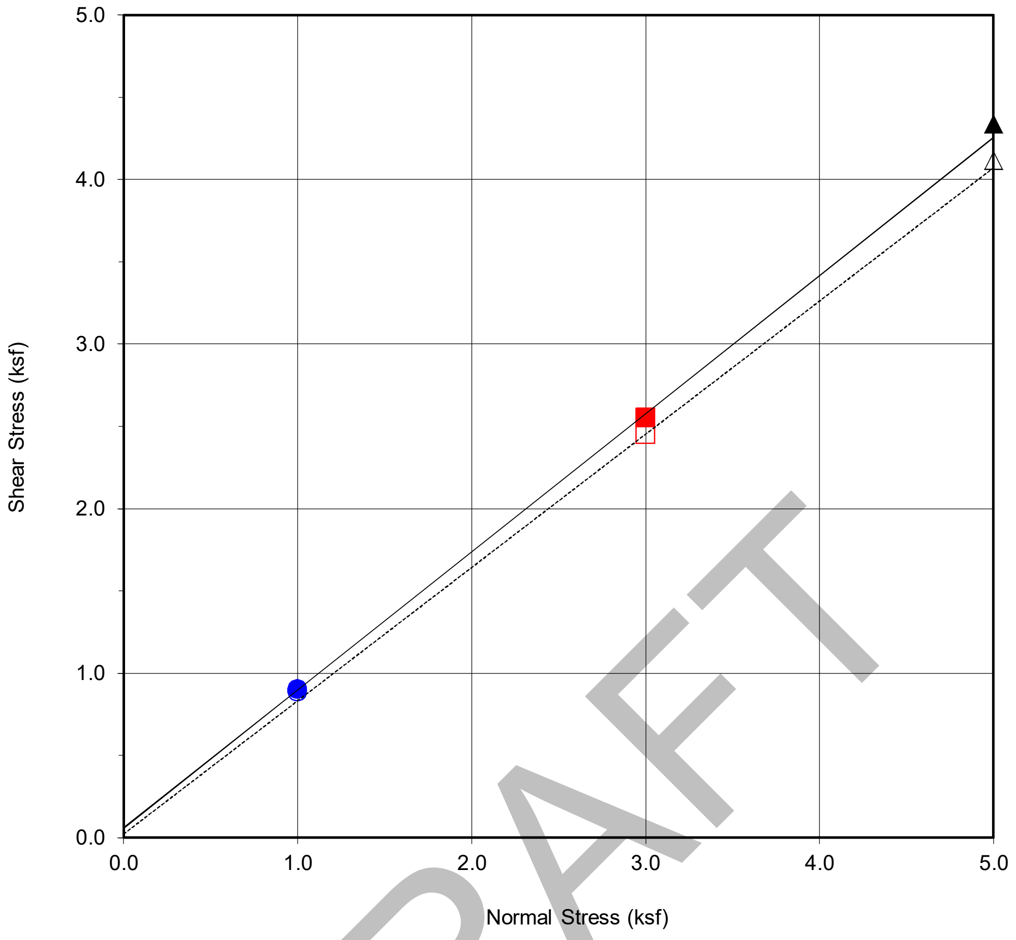
Checked by: RP

Project No.: A9930-06-01

SOUTHWEST AND SOUTHEAST CORNERS OF
EXPOSITION AND CRENSHAW BLVD.
LOS ANGELES, CALIFORNIA

Aug 19

Figure B1



Boring No.	**B1**
Sample No.	**B1@30**
Depth (ft)	**30**
Sample Type:	Ring

Soil Identification:

Brown Silty Sand w/Gravel (SM)

Strength Parameters

	C (psf)	ϕ (°)
Peak	62	41.4
Ultimate	71	39.8

Normal Stress (kip/ft2)	1	3	5
Peak Shear Stress (kip/ft²)	● 0.90	🟥 2.55	▲ 4.33
Shear Stress @ End of Test (ksf)	○ 0.89	□ 2.45	△ 4.12
Deformation Rate (in./min.)	0.05	0.05	0.05
Initial Sample Height (in.)	1.0	1.0	1.0
Ring Inside Diameter (in.)	2.375	2.375	2.375
Initial Moisture Content (%)	9.8	10.8	10.2
Initial Dry Density (pcf)	124.4	120.4	125.9
Initial Degree of Saturation (%)	74.6	73.2	80.9
Soil Height Before Shearing (in.)	1.2	1.2	1.2
Final Moisture Content (%)	13.3	13.6	13.1

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DIRECT SHEAR TEST RESULTS

Consolidated Drained ASTM D-3080

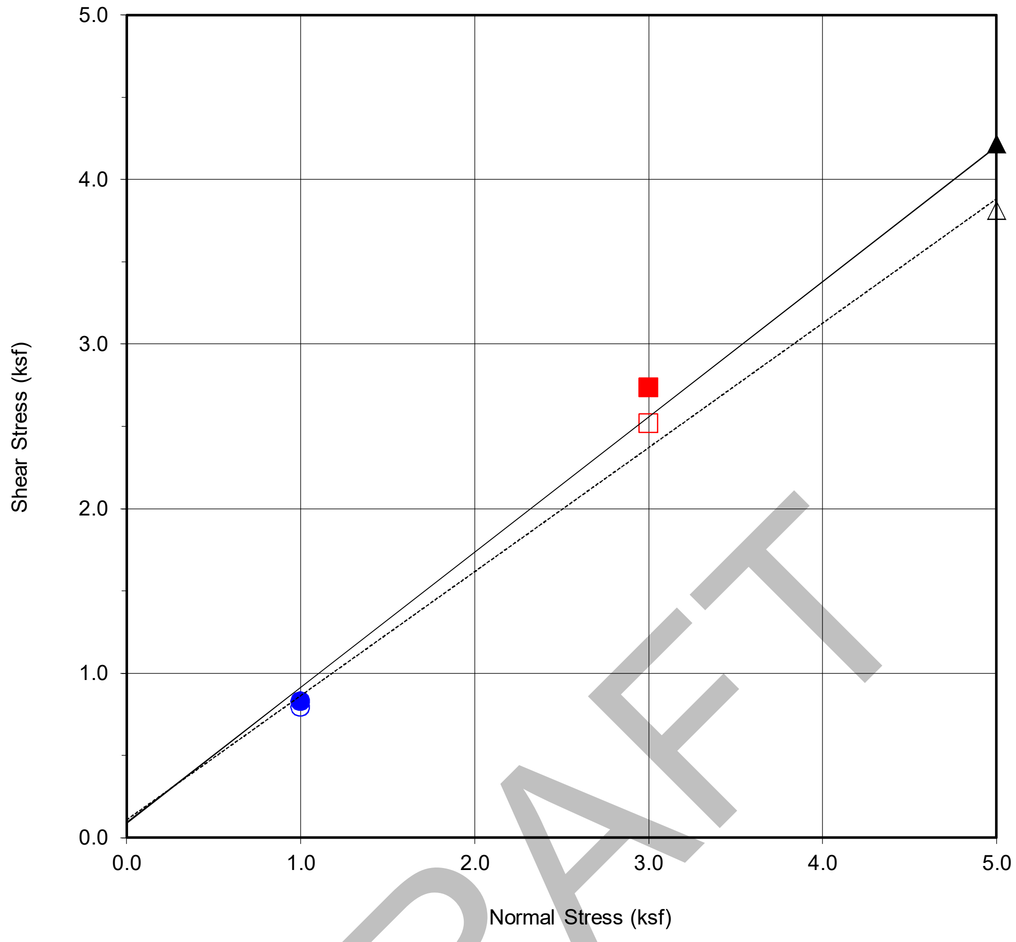
Checked by: RP

Project No.: A9930-06-01

SOUTHWEST AND SOUTHEAST CORNERS OF
EXPOSITION AND CRENSHAW BLVD.
LOS ANGELES, CALIFORNIA

Aug 19

Figure B2



Boring No.	**B1**
Sample No.	**B1@35**
Depth (ft)	**35**
Sample Type:	Ring

Soil Identification:		
Light Brown Poorly Graded Sand w/Gravel (SP)		
Strength Parameters		
	C (psf)	ϕ (°)
Peak	91	39.4
Ultimate	108	37.1

Normal Stress (kip/ft2)	1	3	5
Peak Shear Stress (kip/ft²)	● 0.83	■ 2.73	▲ 4.22
Shear Stress @ End of Test (ksf)	○ 0.79	□ 2.52	△ 3.81
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 (%)	14.5	15.4	14.8
Initial Dry Density (pcf)	118.3	116.6	114.5
Initial Degree of Saturation (%)	92.3	93.3	84.9
Soil Height Before Shearing (in.)	1.2	1.2	1.2
Final Moisture Content (%)	16.7	16.8	17.7

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DIRECT SHEAR TEST RESULTS

Consolidated Drained ASTM D-3080

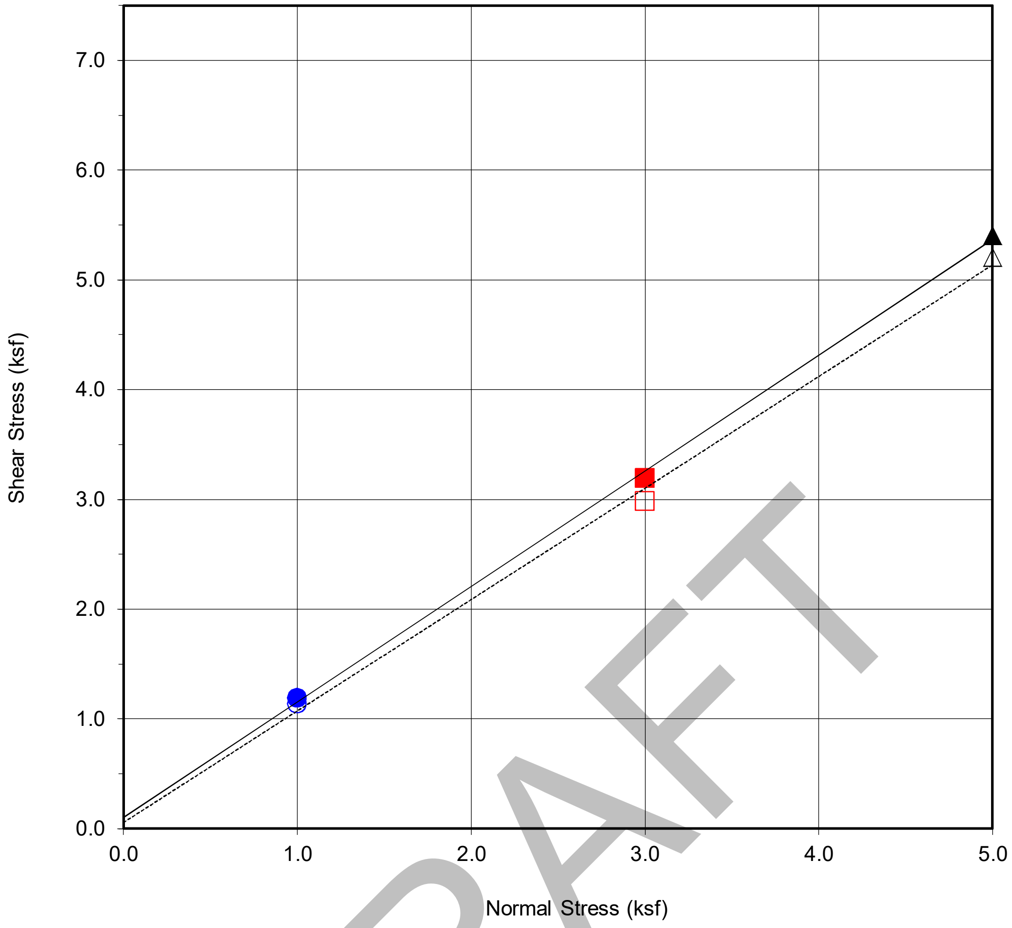
Checked by: RP

Project No.: A9930-06-01

SOUTHWEST AND SOUTHEAST CORNERS OF
EXPOSITION AND CRENSHAW BLVD.
LOS ANGELES, CALIFORNIA

Aug 19

Figure B3



Boring No.	**B1**
Sample No.	**B1@40**
Depth (ft)	**40**
Sample Type:	Ring

Normal Stress (kip/ft2)	1	3	5
Peak Shear Stress (kip/ft²)	● 1.19	■ 3.19	▲ 5.40
Shear Stress @ End of Test (ksf)	○ 1.14	□ 2.98	△ 5.20
Deformation Rate (in./min.)	0.05	0.05	0.05
Initial Sample Height (in.)	1.0	1.0	1.0
Ring Inside Diameter (in.)	2.375	2.375	2.375
Initial Moisture Content (%)	12.9	11.7	10.9
Initial Dry Density (pcf)	124.6	124.6	130.2
Initial Degree of Saturation (%)	98.9	89.1	99.5
Soil Height Before Shearing (in.)	1.2	1.2	1.2
Final Moisture Content (%)	13.2	13.3	11.0

Soil Identification:

Light Brown Poorly Graded Sand w/Gravel (SP)

Strength Parameters

	C (psf)	ϕ (^{o})
Peak	100	46.5
Ultimate	56	45.5

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DIRECT SHEAR TEST RESULTS

Consolidated Drained ASTM D-3080

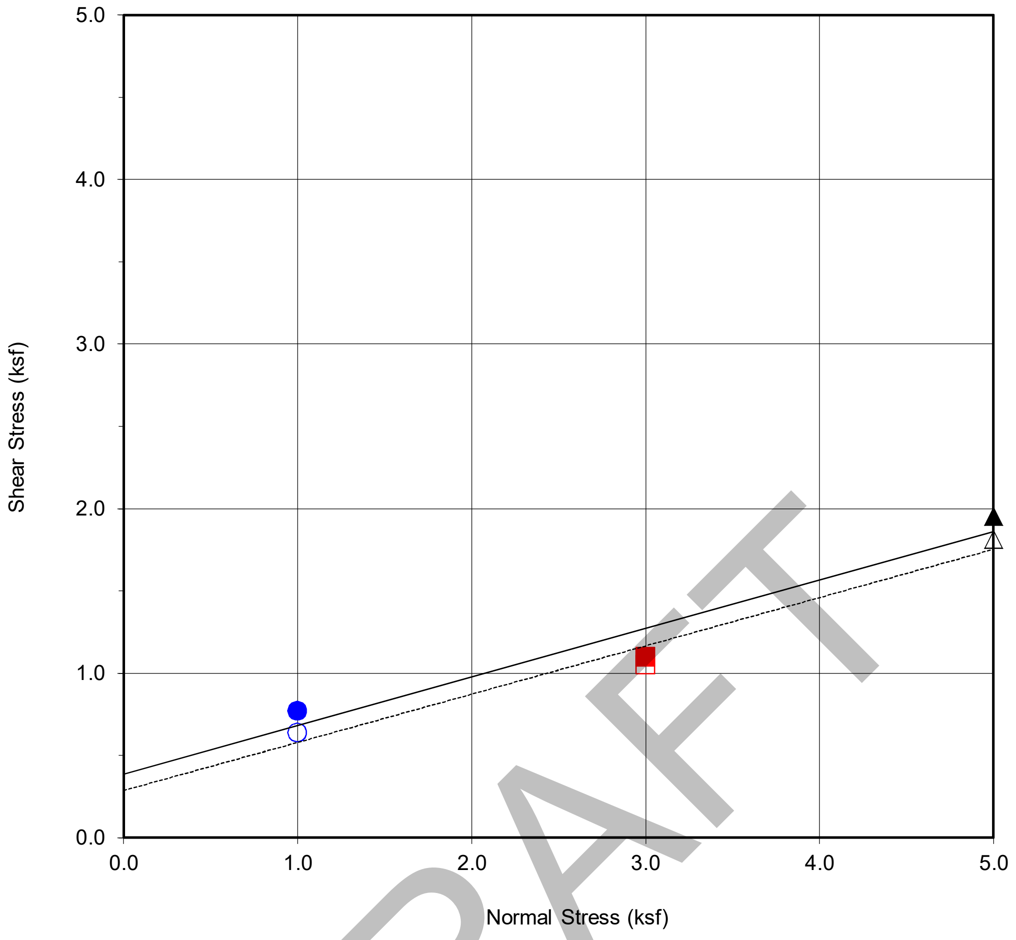
Checked by: RP

Project No.: A9930-06-01

SOUTHWEST AND SOUTHEAST CORNERS OF
EXPOSITION AND CRENSHAW BLVD.
LOS ANGELES, CALIFORNIA

Aug 19

Figure B4



Boring No.	**B2**
Sample No.	**B2@3**
Depth (ft)	**3**
Sample Type:	Ring

Soil Identification:

Grayish Brown Clay (CL)

Strength Parameters

	C (psf)	ϕ (°)
Peak	386	16.4
Ultimate	286	16.3

Normal Stress (kip/ft2)	1	3	5
Peak Shear Stress (kip/ft²)	● 0.77	■ 1.09	▲ 1.95
Shear Stress @ End of Test (ksf)	○ 0.64	□ 1.05	△ 1.81
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 (%)	37.9	39.7	53.3
Initial Dry Density (pcf)	84.4	83.5	78.3
Initial Degree of Saturation (%)	102.4	105.3	124.9
Soil Height Before Shearing (in.)	1.2	1.2	1.2
Final Moisture Content (%)	39.0	36.4	50.4

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DIRECT SHEAR TEST RESULTS

Consolidated Drained ASTM D-3080

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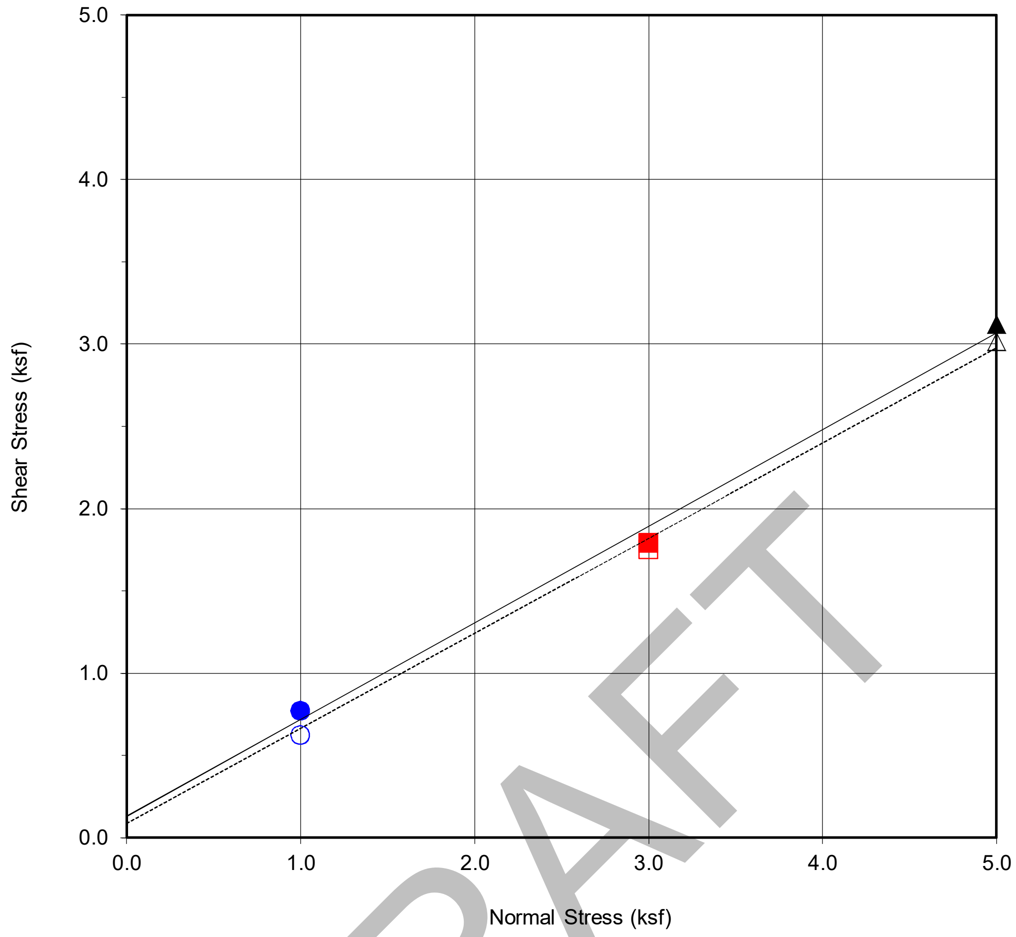
Project No.: A9930-06-01

SOUTHWEST AND SOUTHEAST CORNERS OF
EXPOSITION AND CRENSHAW BLVD.
LOS ANGELES, CALIFORNIA

Aug 19

Figure B5



Boring No.	**B2**
Sample No.	**B2@9**
Depth (ft)	**9**
Sample Type:	Ring

Normal Stress (kip/ft2)	1	3	5
Peak Shear Stress (kip/ft^2)	● 0.77	■ 1.79	▲ 3.12
Shear Stress @ End of Test (ksf)	○ 0.62	□ 1.75	△ 3.01
Deformation Rate (in./min.)	0.05	0.05	0.05
Initial Sample Height (in.)	1.0	1.0	1.0
Ring Inside Diameter (in.)	2.375	2.375	2.375
Initial Moisture Content (%)	36.1	46.7	41.0
Initial Dry Density (pcf)	86.4	77.8	83.2
Initial Degree of Saturation (%)	102.6	107.9	107.8
Soil Height Before Shearing (in.)	1.2	1.2	1.2
Final Moisture Content (%)	35.6	38.5	34.9

Soil Identification:

Olive Brown Silt w/Sand (ML)

Strength Parameters

	C (psf)	ϕ (°)
Peak	130	30.4
Ultimate	87	30.0

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DIRECT SHEAR TEST RESULTS

Consolidated Drained ASTM D-3080

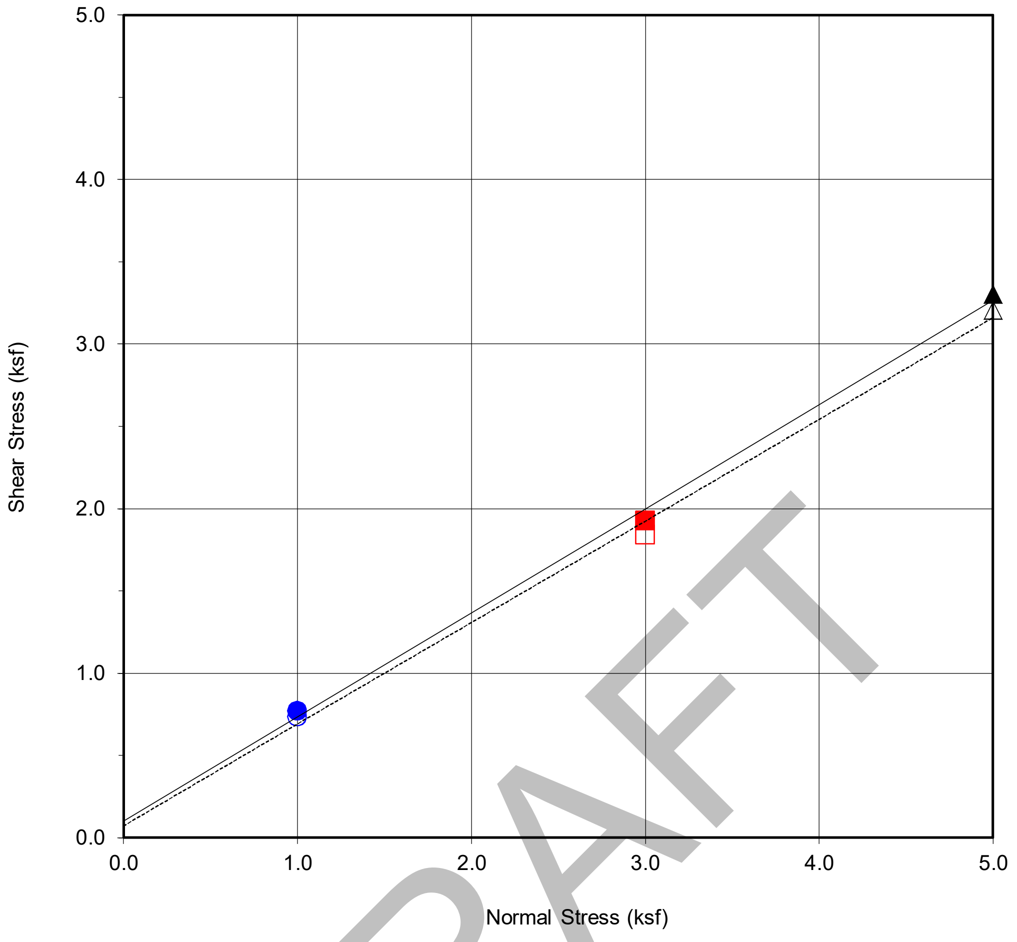
Checked by: RP

Project No.: A9930-06-01

SOUTHWEST AND SOUTHEAST CORNERS OF
EXPOSITION AND CRENSHAW BLVD.
LOS ANGELES, CALIFORNIA

Aug 19

Figure B6



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

Soil Identification:		
Grayish Brown Silt w/Sand (ML)		
Strength Parameters		
	C (psf)	ϕ (°)
Peak	100	32.3
Ultimate	72	31.7

Normal Stress (kip/ft2)	1	3	5
Peak Shear Stress (kip/ft²)	● 0.77	■ 1.93	▲ 3.30
Shear Stress @ End of Test (ksf)	○ 0.73	□ 1.84	△ 3.20
Deformation Rate (in./min.)	0.05	0.05	0.05
Initial Sample Height (in.)	1.0	1.0	1.0
Ring Inside Diameter (in.)	2.375	2.375	2.375
Initial Moisture Content (%)	52.2	106.4	40.7
Initial Dry Density (pcf)	72.2	56.5	83.6
Initial Degree of Saturation (%)	105.5	145.0	108.2
Soil Height Before Shearing (in.)	1.2	1.2	1.2
Final Moisture Content (%)	50.9	54.3	37.0

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DIRECT SHEAR TEST RESULTS

Consolidated Drained ASTM D-3080

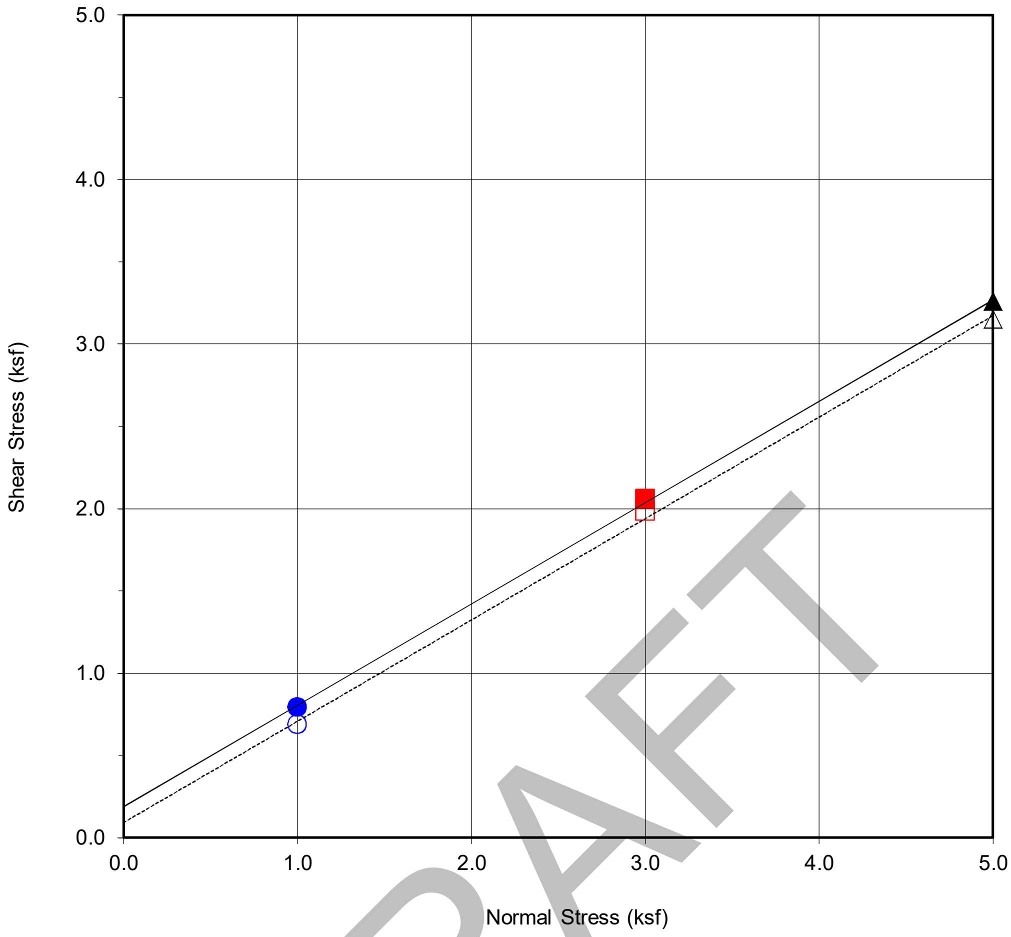
Checked by: RP

Project No.: A9930-06-01

SOUTHWEST AND SOUTHEAST CORNERS OF
EXPOSITION AND CRENSHAW BLVD.
LOS ANGELES, CALIFORNIA

Aug 19

Figure B7



Boring No.	**B3**
Sample No.	**B3@3**
Depth (ft)	**3**
Sample Type:	Ring

Normal Stress (kip/ft2)	1	3	5
Peak Shear Stress (kip/ft²)	● 0.79	■ 2.06	▲ 3.26
Shear Stress @ End of Test (ksf)	○ 0.69	□ 1.98	△ 3.15
Deformation Rate (in./min.)	0.05	0.05	0.05
Initial Sample Height (in.)	1.0	1.0	1.0
Ring Inside Diameter (in.)	2.375	2.375	2.375
Initial Moisture Content (%)	31.0	33.0	32.6
Initial Dry Density (pcf)	88.4	84.0	88.2
Initial Degree of Saturation (%)	92.4	88.5	96.5
Soil Height Before Shearing (in.)	1.2	1.2	1.2
Final Moisture Content (%)	35.3	35.4	33.6

Soil Identification:

Olive Brown Silt with Sand (ML)

Strength Parameters

	C (psf)	ϕ (°)
Peak	188	31.6
Ultimate	92	31.6

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DIRECT SHEAR TEST RESULTS

Consolidated Drained ASTM D-3080

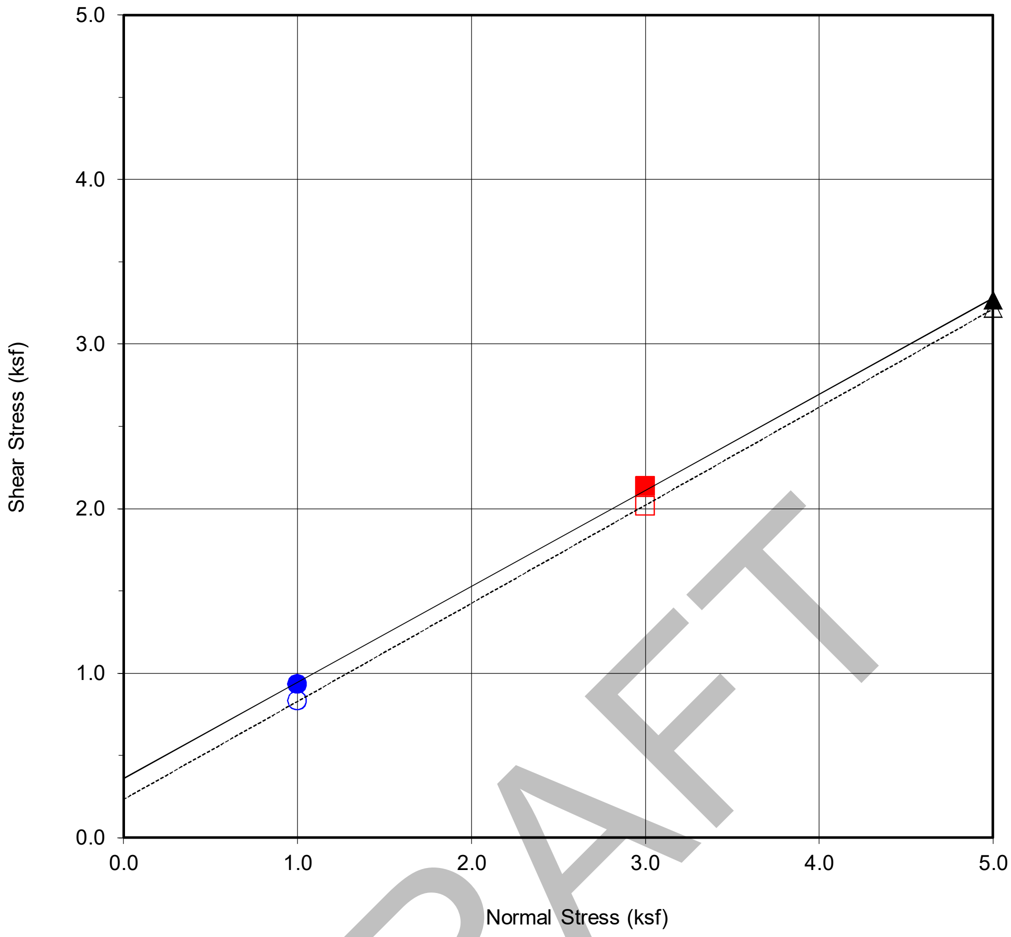
Checked by: RP

Project No.: A9930-06-01

SOUTHWEST AND SOUTHEAST CORNERS OF
EXPOSITION AND CRENSHAW BLVD.
LOS ANGELES, CALIFORNIA

Aug 19

Figure B8



Boring No.	**B3**
Sample No.	**B3@12**
Depth (ft)	**12**
Sample Type:	Ring

Soil Identification:

Olive Brown Sandy Clay (CL)

Strength Parameters

	C (psf)	ϕ (^{o})
Peak	360	30.3
Ultimate	232	30.8

Normal Stress (kip/ft2)	1	3	5
Peak Shear Stress (kip/ft²)	● 0.93	■ 2.14	▲ 3.27
Shear Stress @ End of Test (ksf)	○ 0.83	□ 2.01	△ 3.22
Deformation Rate (in./min.)	0.05	0.05	0.05
Initial Sample Height (in.)	1.0	1.0	1.0
Ring Inside Diameter (in.)	2.375	2.375	2.375
Initial Moisture Content (%)	23.2	25.6	22.3
Initial Dry Density (pcf)	104.1	100.2	105.6
Initial Degree of Saturation (%)	101.1	101.5	100.9
Soil Height Before Shearing (in.)	1.2	1.2	1.2
Final Moisture Content (%)	23.3	23.9	21.0

GEOCON

DIRECT SHEAR TEST RESULTS

Consolidated Drained ASTM D-3080

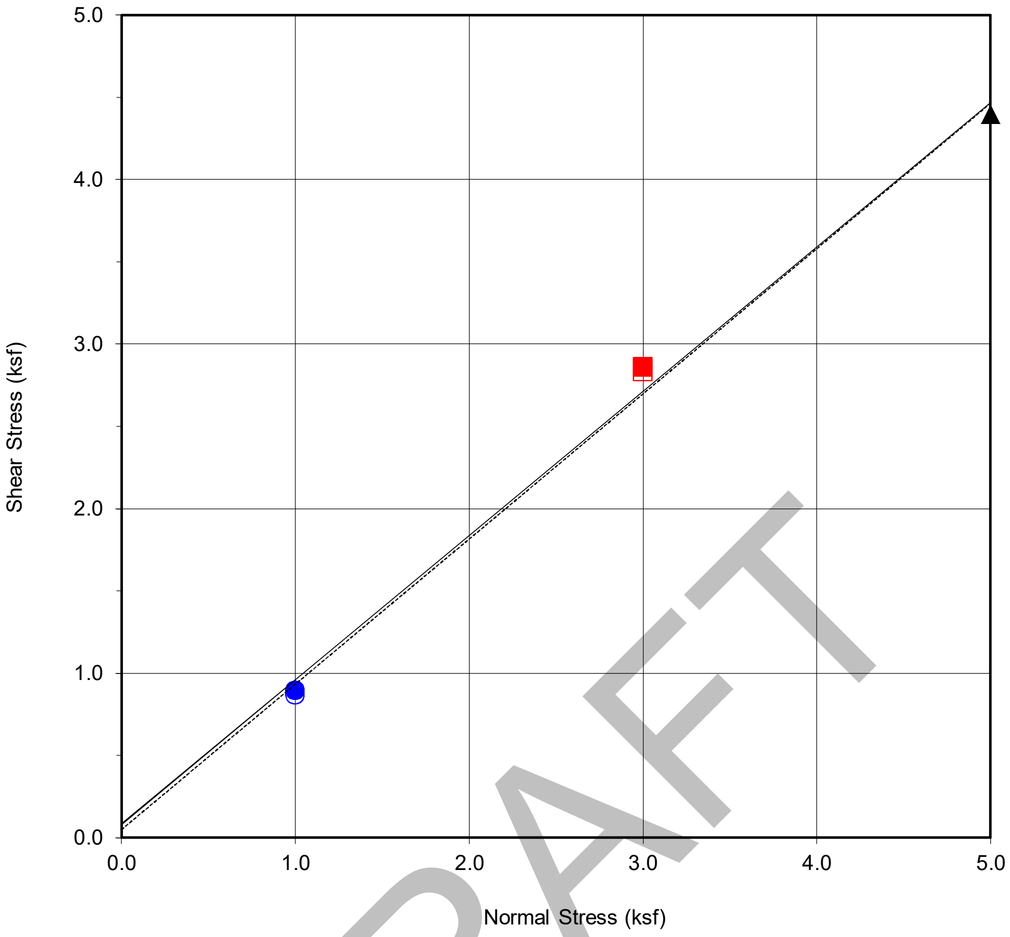
Checked by: RP

Project No.: A9930-06-01

SOUTHWEST AND SOUTHEAST CORNERS OF
EXPOSITION AND CRENSHAW BLVD.
LOS ANGELES, CALIFORNIA

Aug 19

Figure B9



Boring No.	**B7**
Sample No.	**B7@30**
Depth (ft)	**30**
Sample Type:	Ring

Soil Identification:

Olive Brown Poorly Graded Sand w/Gravel (SP)

Strength Parameters

	C (psf)	ϕ (°)
Peak	83	41.2
Ultimate	49	41.4

Normal Stress (kip/ft2)	1	3	5
Peak Shear Stress (kip/ft²)	● 0.89	🟥 2.86	▲ 4.40
Shear Stress @ End of Test (ksf)	○ 0.87	☐ 2.83	△ 4.40
Deformation Rate (in./min.)	0.05	0.05	0.05
Initial Sample Height (in.)	1.0	1.0	1.0
Ring Inside Diameter (in.)	2.375	2.375	2.375
Initial Moisture Content (%)	8.3	8.5	8.3
Initial Dry Density (pcf)	121.3	114.2	112.6
Initial Degree of Saturation (%)	57.3	48.0	45.3
Soil Height Before Shearing (in.)	1.2	1.2	1.2
Final Moisture Content (%)	13.7	14.7	13.4

GEOCON

DIRECT SHEAR TEST RESULTS

Consolidated Drained ASTM D-3080

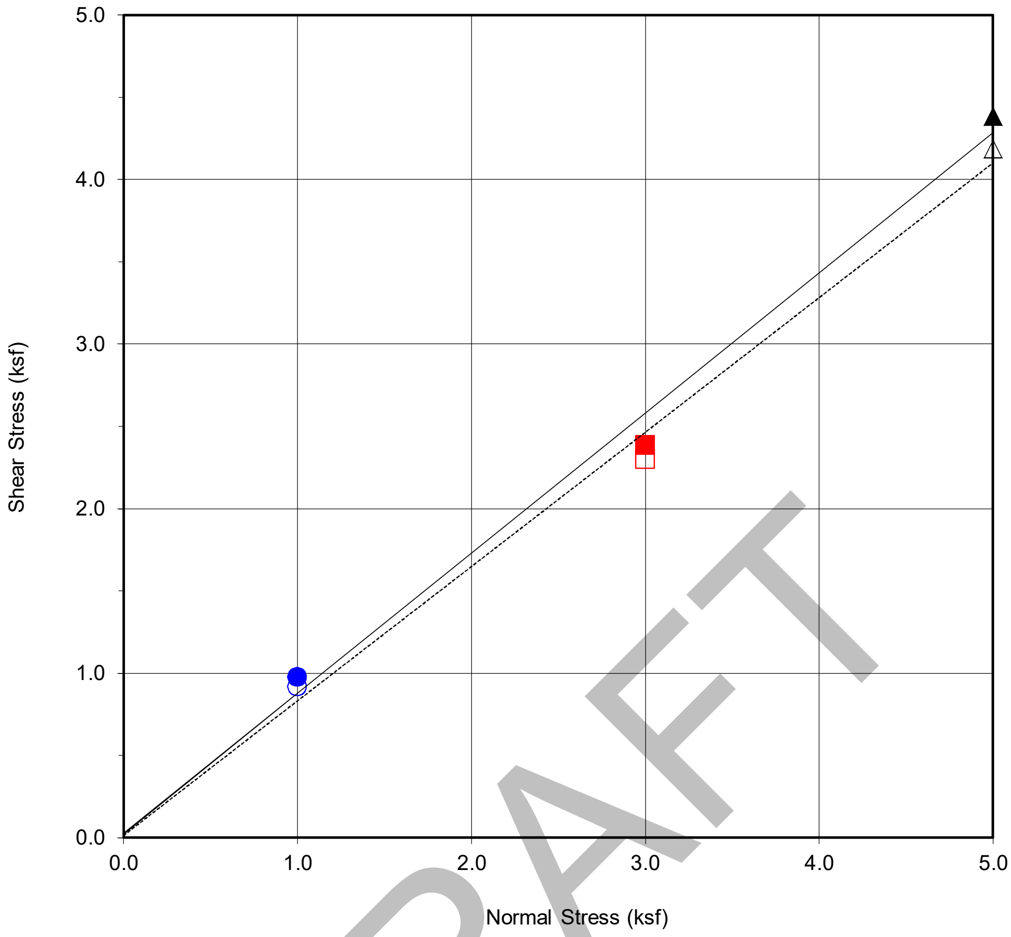
Checked by: RP

Project No.: A9930-06-01

SOUTHWEST AND SOUTHEAST CORNERS OF
EXPOSITION AND CRENSHAW BLVD.
LOS ANGELES, CALIFORNIA

Aug 19

Figure B10



Boring No.	**B7**
Sample No.	**B7@35**
Depth (ft)	**35**
Sample Type:	Ring

Soil Identification:

Light Brown Poorly Graded Sand w/Gravel (SP)

Strength Parameters

	C (psf)	ϕ (°)
Peak	25	40.4
Ultimate	13	39.3

Normal Stress (kip/ft2)	1	3	5
Peak Shear Stress (kip/ft²)	● 0.98	🟥 2.38	▲ 4.38
Shear Stress @ End of Test (ksf)	○ 0.92	□ 2.30	△ 4.19
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 (%)	8.0	7.7	9.3
Initial Dry Density (pcf)	103.9	103.6	101.0
Initial Degree of Saturation (%)	34.5	33.1	37.5
Soil Height Before Shearing (in.)	1.2	1.2	1.2
Final Moisture Content (%)	18.7	19.8	19.9

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DIRECT SHEAR TEST RESULTS

Consolidated Drained ASTM D-3080

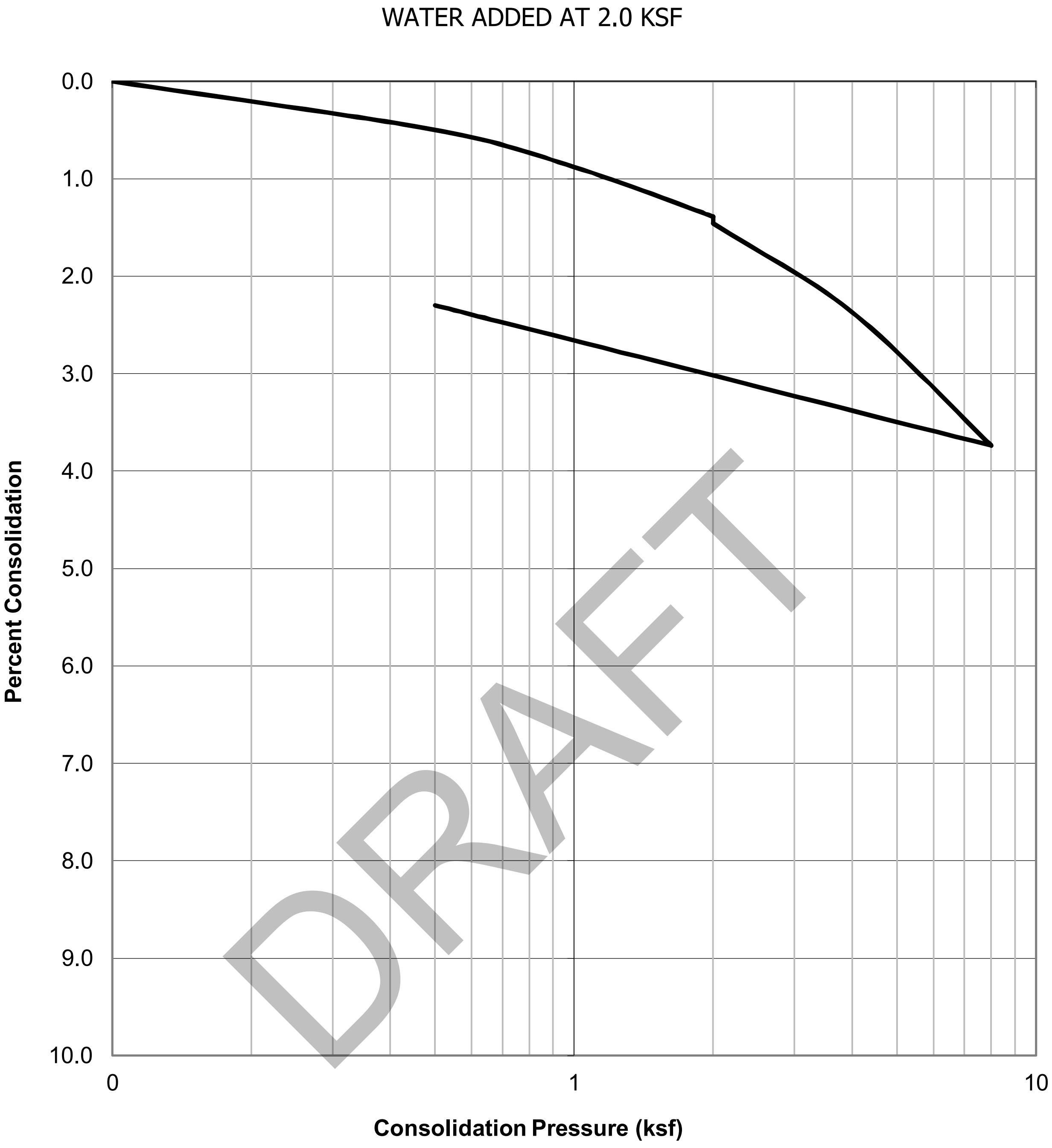
Checked by: RP

Project No.: A9930-06-01

SOUTHWEST AND SOUTHEAST CORNERS OF
EXPOSITION AND CRENSHAW BLVD.
LOS ANGELES, CALIFORNIA

Aug 19

Figure B11



SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B2@6	Grayish Brown Clay (CL)	109.8	16.7	17.3

GEOCON

CONSOLIDATION TEST RESULTS

ASTM D-2435

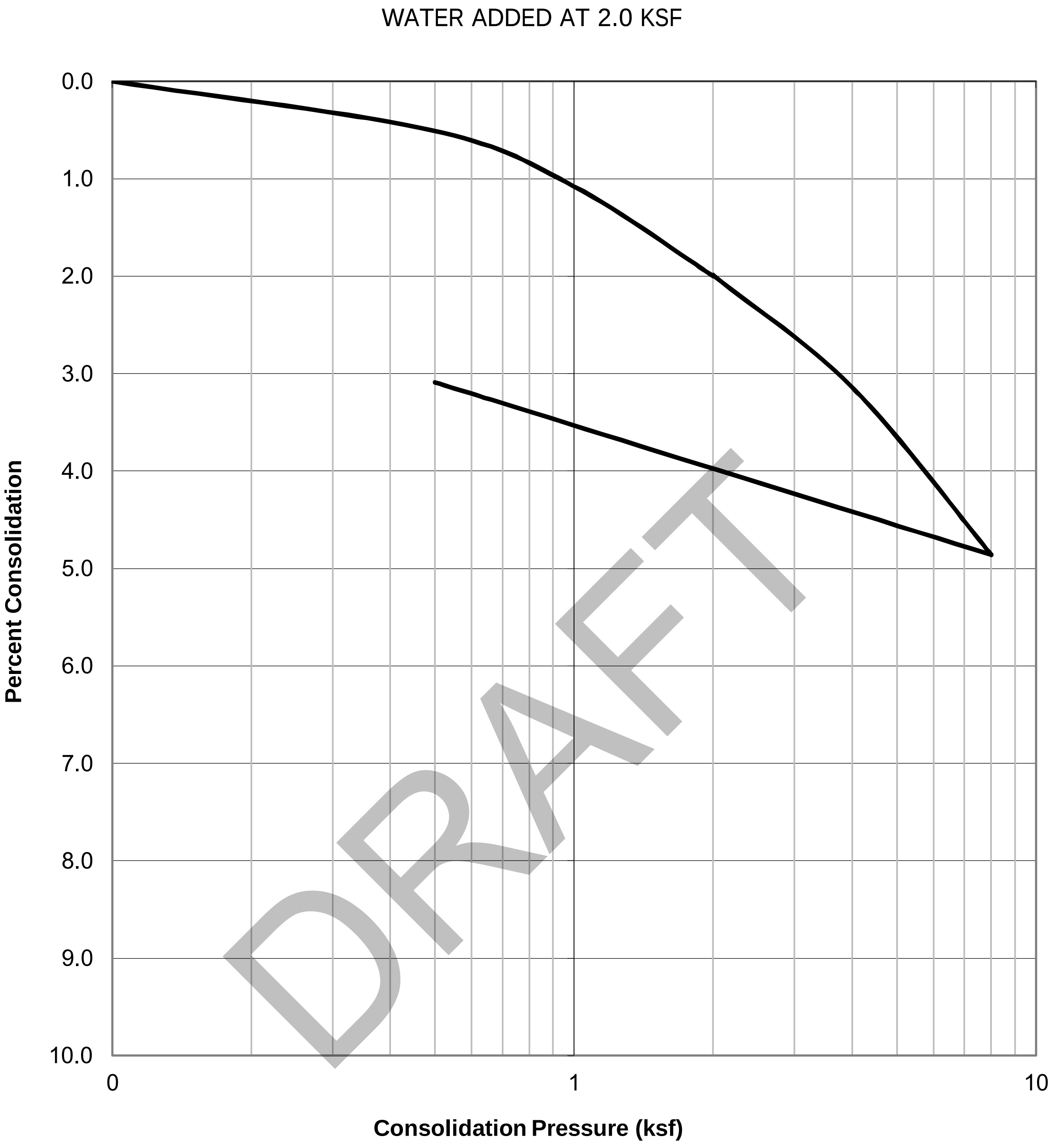
Checked by: RP

Project No.: A9930-06-01

SOUTHWEST AND SOUTHEAST CORNERS OF
EXPOSITION AND CRENSHAW BLVD.
LOS ANGELES, CALIFORNIA

Aug 19

Figure B12



SAMPLE ID.	**SOIL TYPE**	**DRY DENSITY (PCF)**	**INITIAL MOISTURE (%)**	**FINAL MOISTURE (%)**
B2@9	Olive Brown Silt with Sand (ML)	91.6	26.3	29.9

GEOCON

CONSOLIDATION TEST RESULTS

ASTM D-2435

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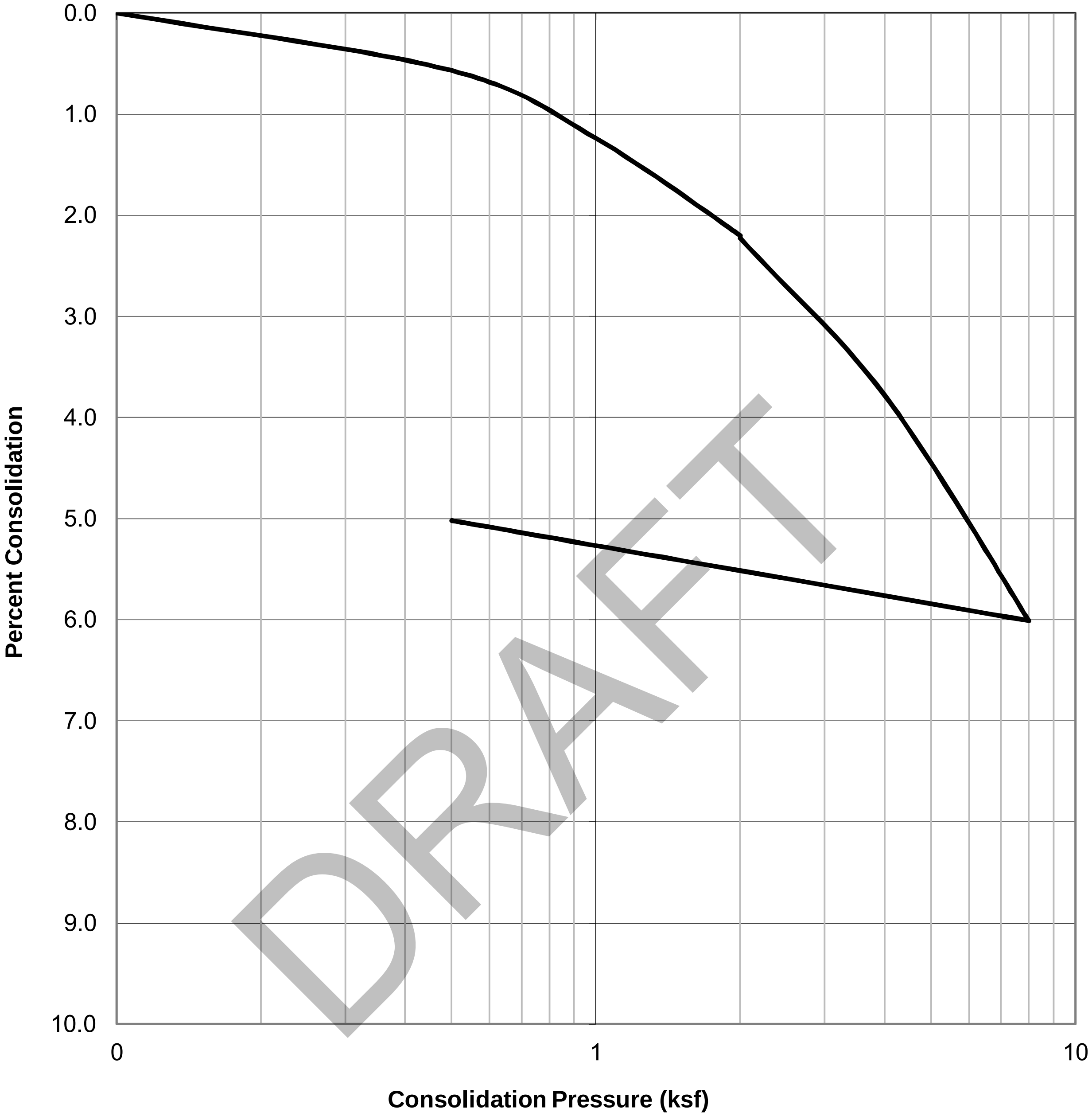
Project No.: A9930-06-01

SOUTHWEST AND SOUTHEAST CORNERS OF
EXPOSITION AND CRENSHAW BLVD.
LOS ANGELES, CALIFORNIA

Aug 19

Figure B13

WATER ADDED AT 2.0 KSF



SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B2@12	Olive Brown Silt with Sand (ML)	90.9	27.1	25.3

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CONSOLIDATION TEST RESULTS

ASTM D-2435

Checked by: RP

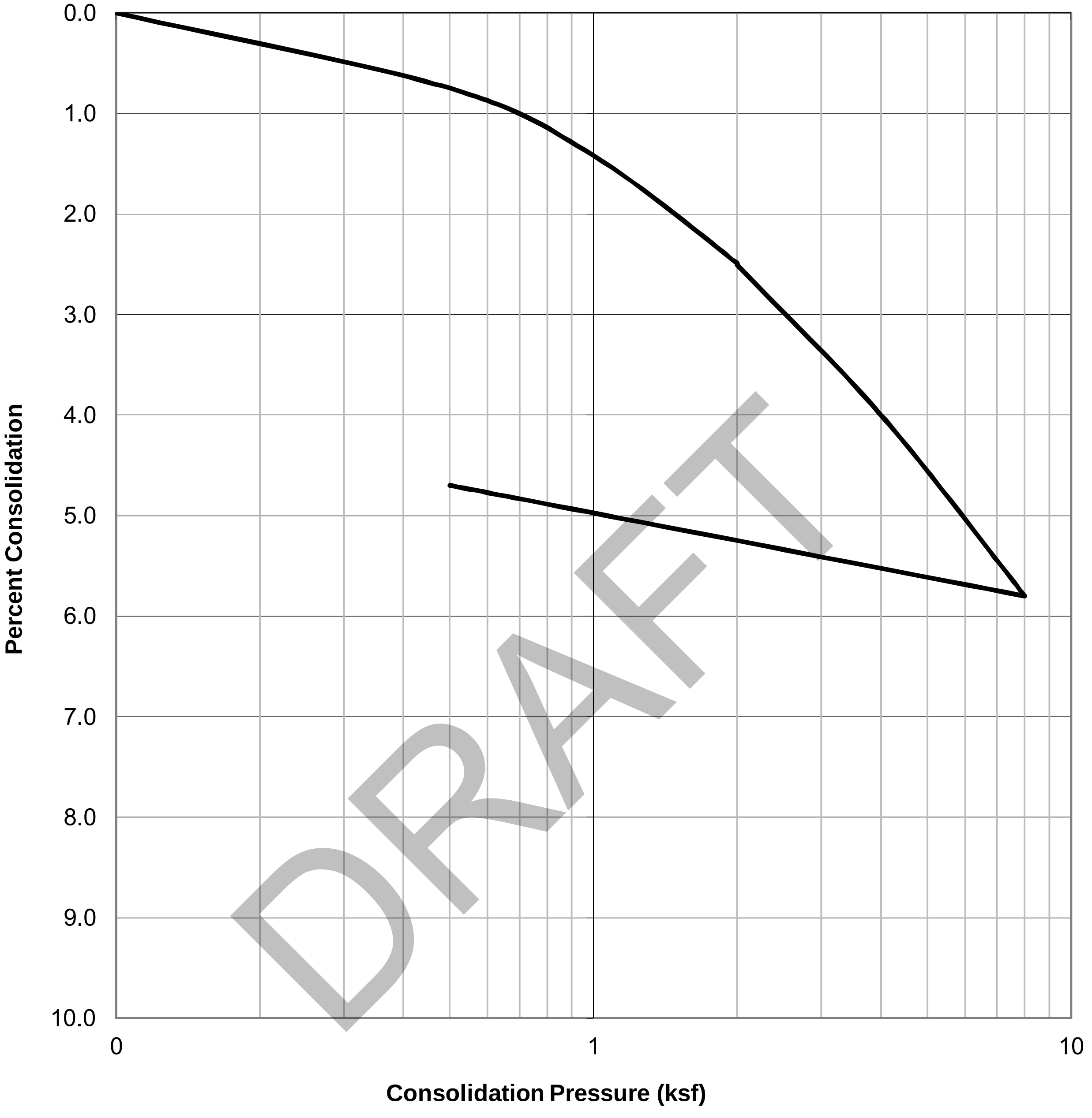
Project No.: A9930-06-01

SOUTHWEST AND SOUTHEAST CORNERS OF
EXPOSITION AND CRENSHAW BLVD.
LOS ANGELES, CALIFORNIA

Aug 19

Figure B14

WATER ADDED AT 2.0 KSF



SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B2@15	Olive Brown Silt with Sand (ML)	103.4	24.3	21.5

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CONSOLIDATION TEST RESULTS

ASTM D-2435

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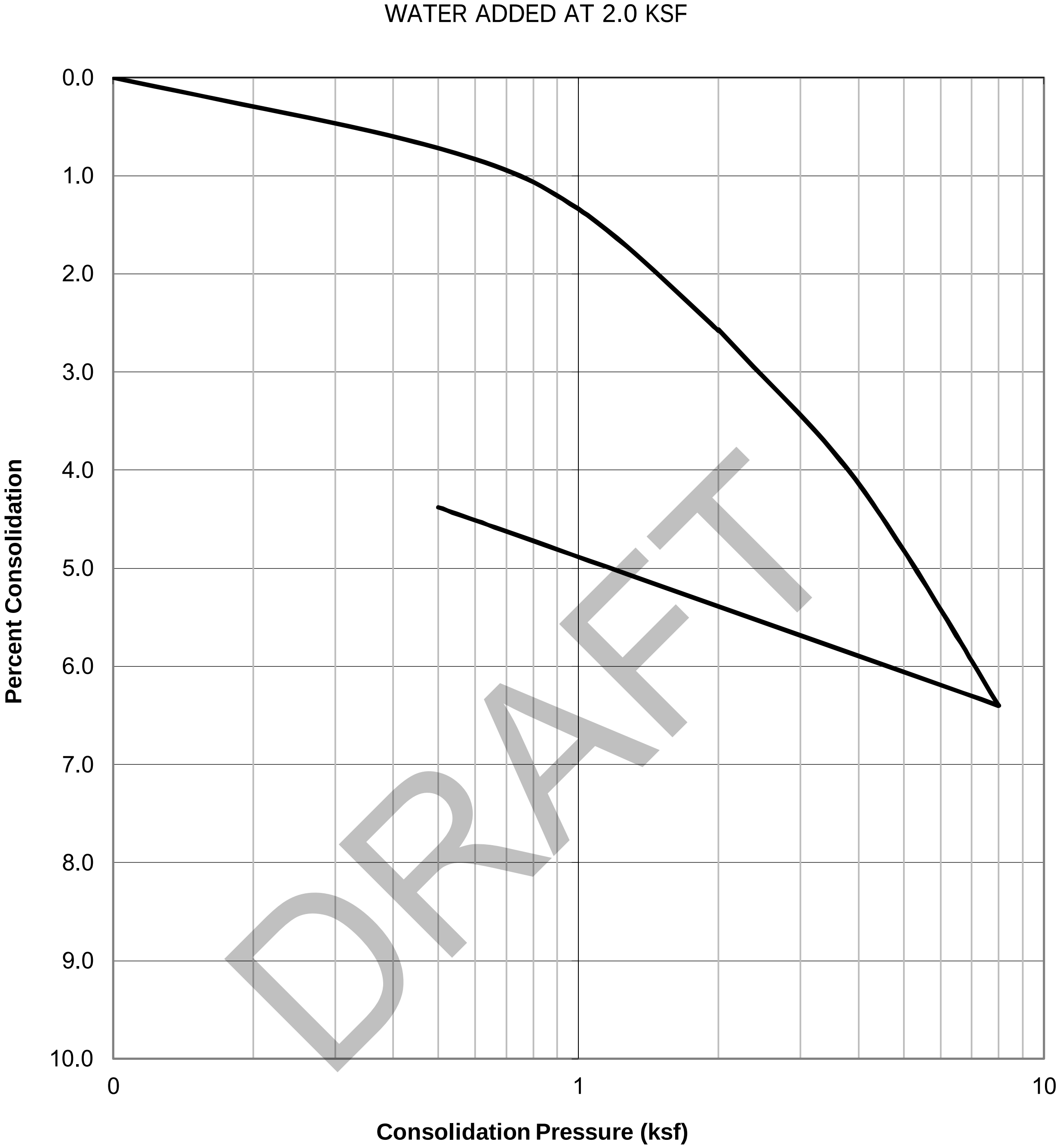
Project No.: A9930-06-01

SOUTHWEST AND SOUTHEAST CORNERS OF
EXPOSITION AND CRENSHAW BLVD.
LOS ANGELES, CALIFORNIA

Aug 19

Figure B15



SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B2@20	Grayish Brown Silt with Sand (ML)	83.6	34.0	31.3

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CONSOLIDATION TEST RESULTS

ASTM D-2435

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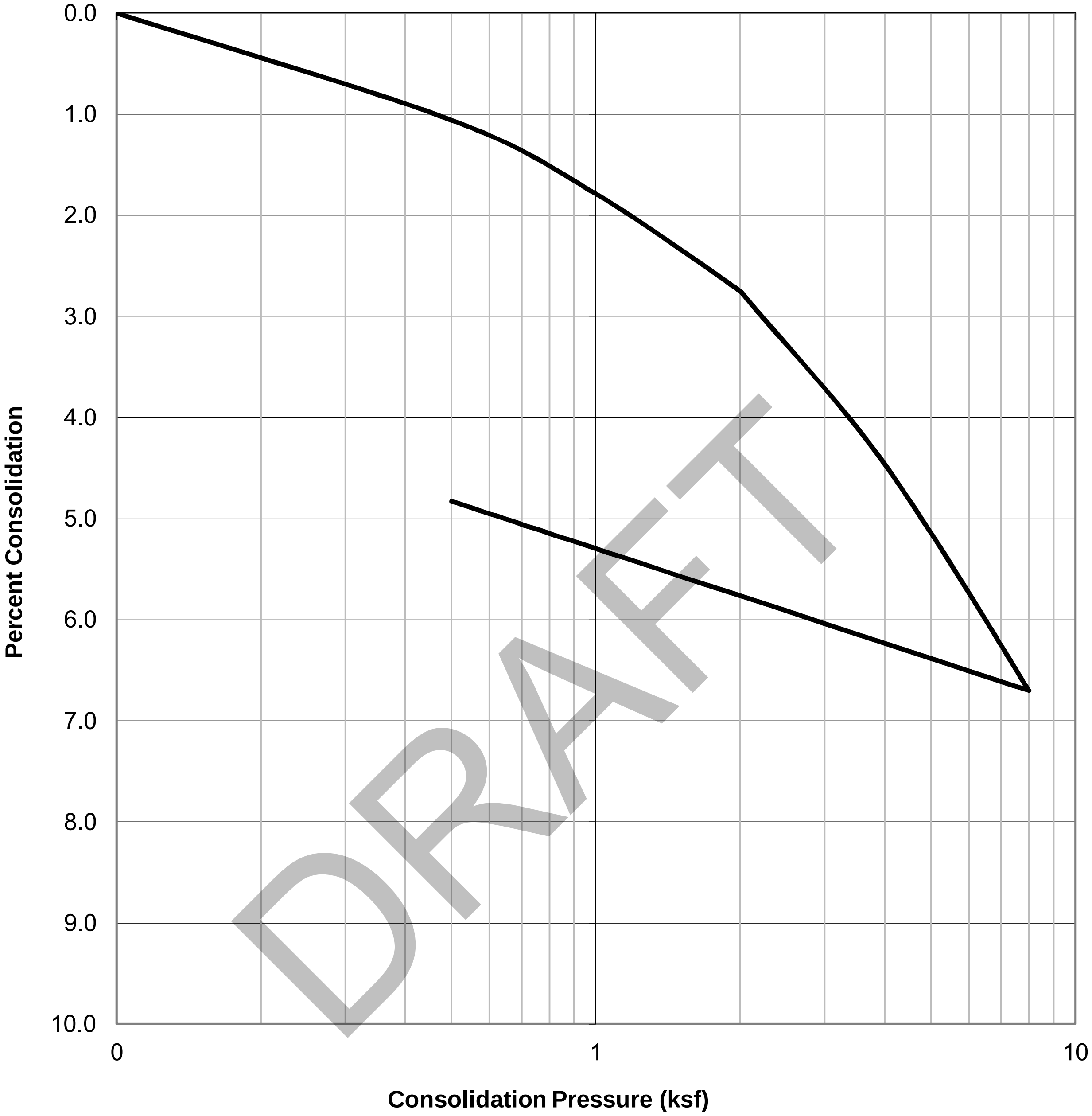
Project No.: A9930-06-01

SOUTHWEST AND SOUTHEAST CORNERS OF
EXPOSITION AND CRENSHAW BLVD.
LOS ANGELES, CALIFORNIA

Aug 19

Figure B16

WATER ADDED AT 2.0 KSF



SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B2@25	Grayish Brown Silt with Sand (ML)	78.8	41.4	39.4

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CONSOLIDATION TEST RESULTS

ASTM D-2435

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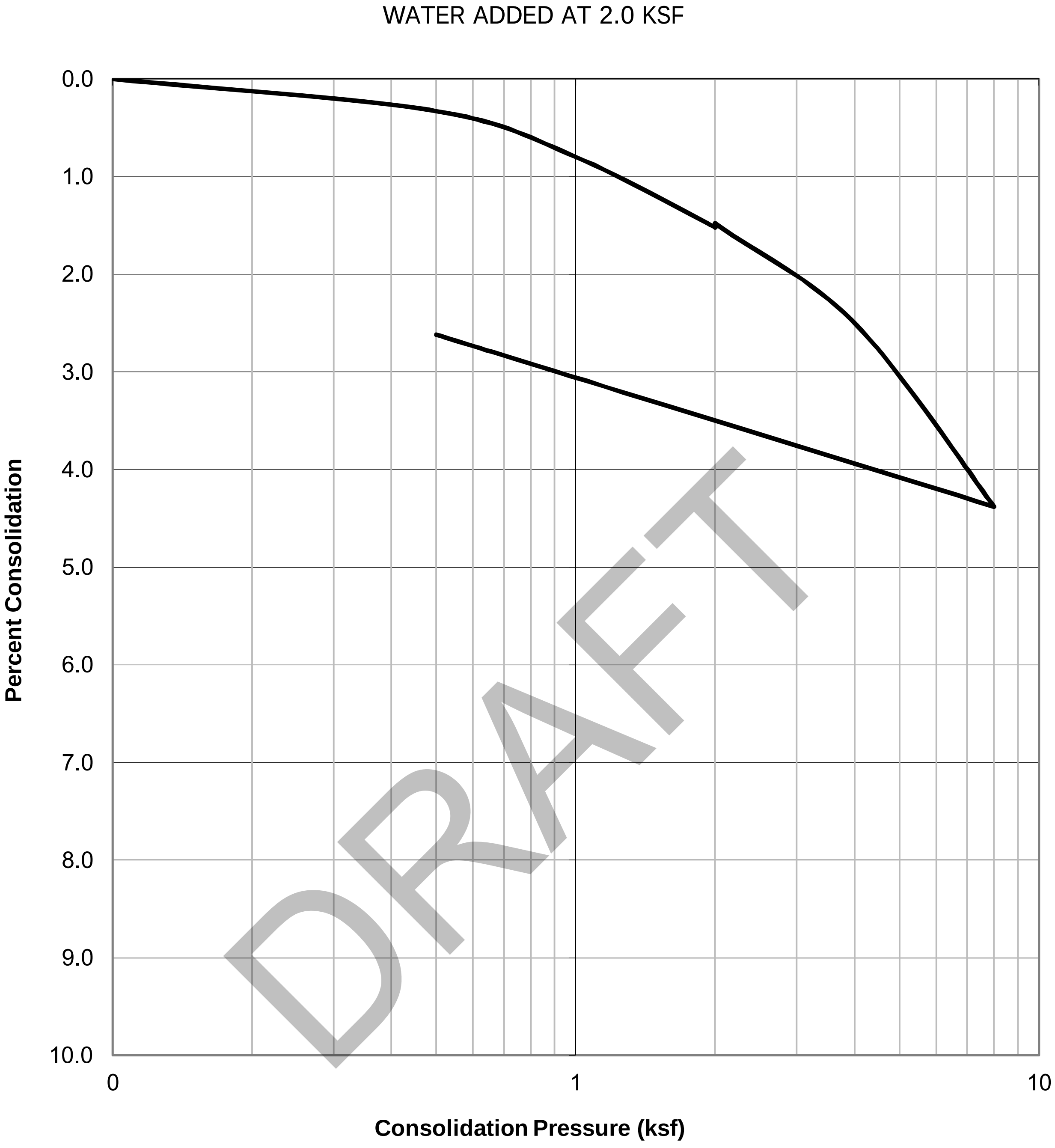
SOUTHWEST AND SOUTHEAST CORNERS OF
EXPOSITION AND CRENSHAW BLVD.
LOS ANGELES, CALIFORNIA

Aug 19



SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B3@6	Light Olive Brown Clay (CL)	95.6	27.0	27.2

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CONSOLIDATION TEST RESULTS

ASTM D-2435

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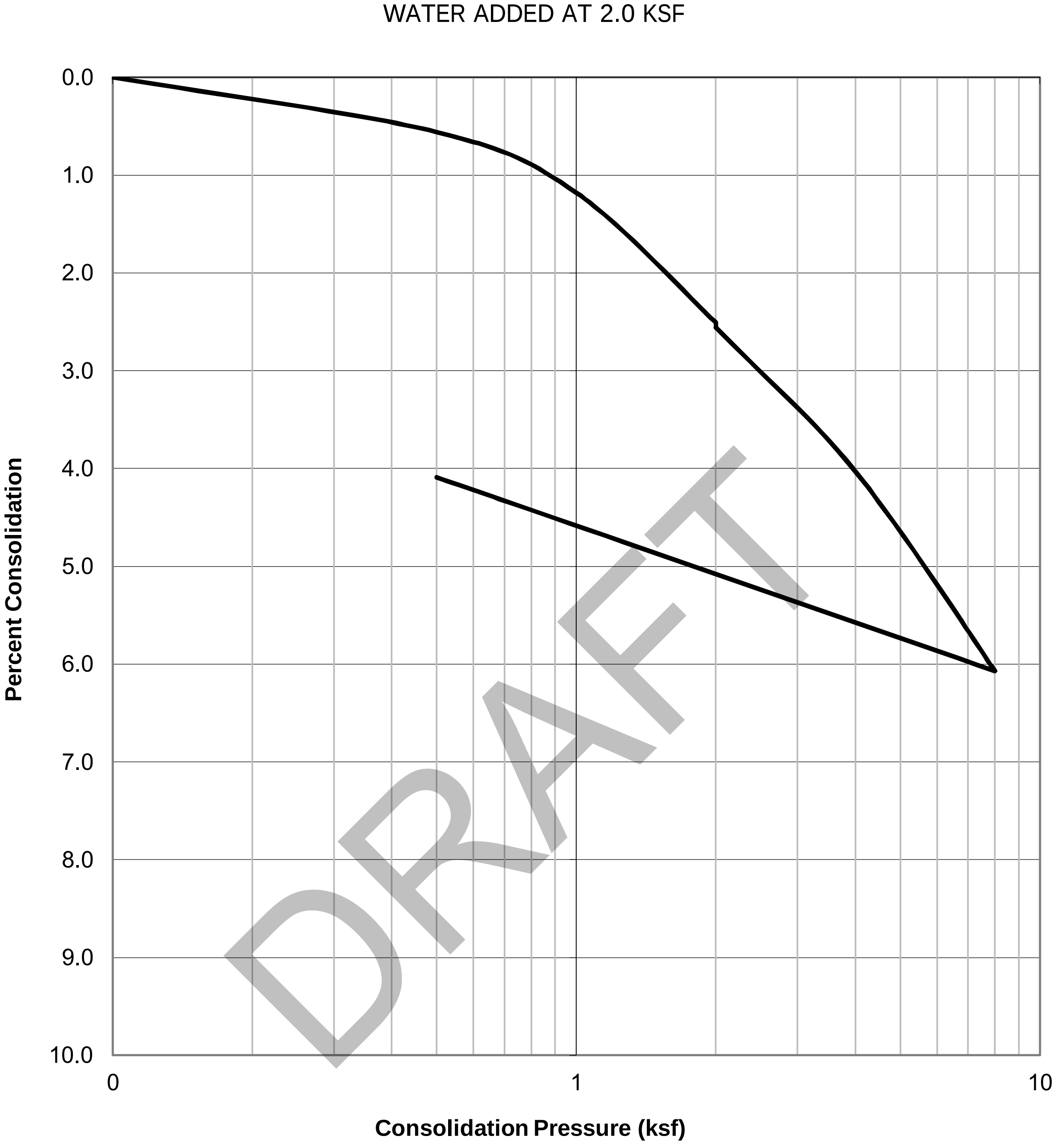
SOUTHWEST AND SOUTHEAST CORNERS OF
EXPOSITION AND CRENSHAW BLVD.
LOS ANGELES, CALIFORNIA

Aug 19

Figure B18



SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B3@9	Olive Brown Silt with Sand (ML)	82.9	29.5	31.1

GEOCON

CONSOLIDATION TEST RESULTS

ASTM D-2435

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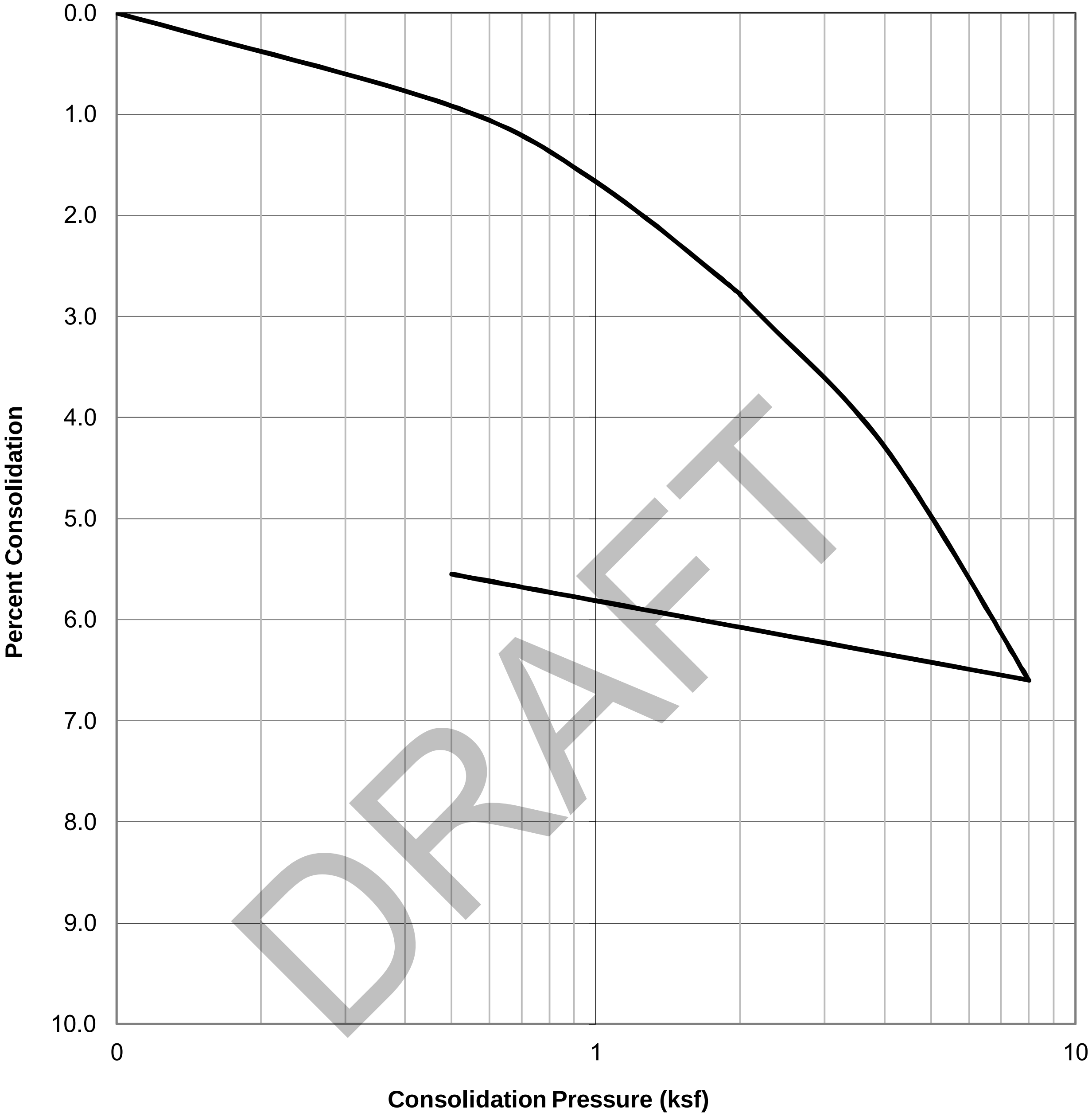
Project No.: A9930-06-01

SOUTHWEST AND SOUTHEAST CORNERS OF
EXPOSITION AND CRENSHAW BLVD.
LOS ANGELES, CALIFORNIA

Aug 19

Figure B19

WATER ADDED AT 2.0 KSF



SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B3@12	Olive Brown Sandy Clay (CL)	94.9	26.9	23.6

GEOCON

CONSOLIDATION TEST RESULTS

ASTM D-2435

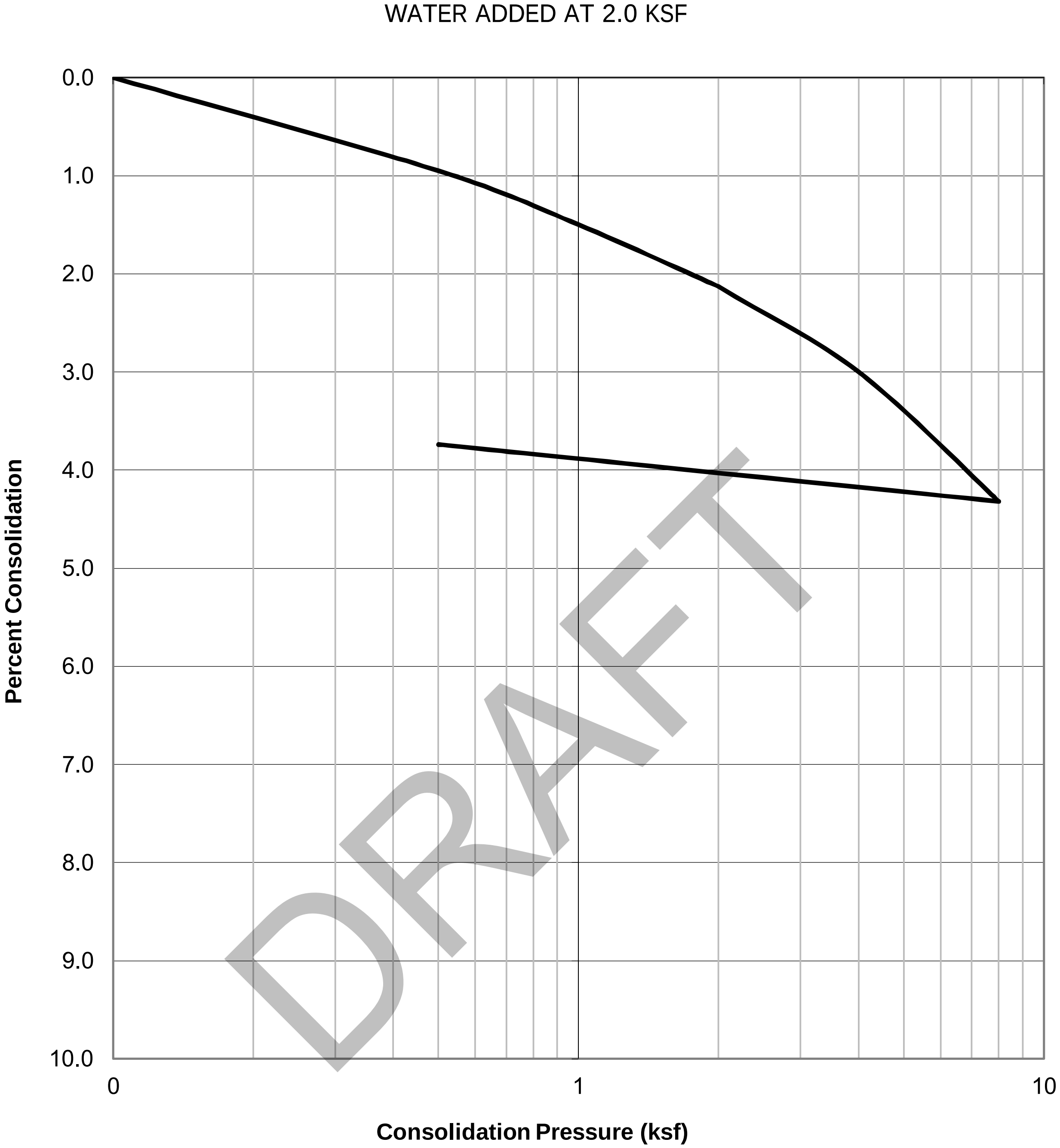
Checked by: RP

Project No.: A9930-06-01

SOUTHWEST AND SOUTHEAST CORNERS OF
EXPOSITION AND CRENSHAW BLVD.
LOS ANGELES, CALIFORNIA

Aug 19

Figure B20



SAMPLE ID.	**SOIL TYPE**	**DRY DENSITY (PCF)**	**INITIAL MOISTURE (%)**	**FINAL MOISTURE (%)**
B3@15	Olive Brown Silty Sand (SM)	105.3	22.6	19.9

GEOCON

CONSOLIDATION TEST RESULTS

ASTM D-2435

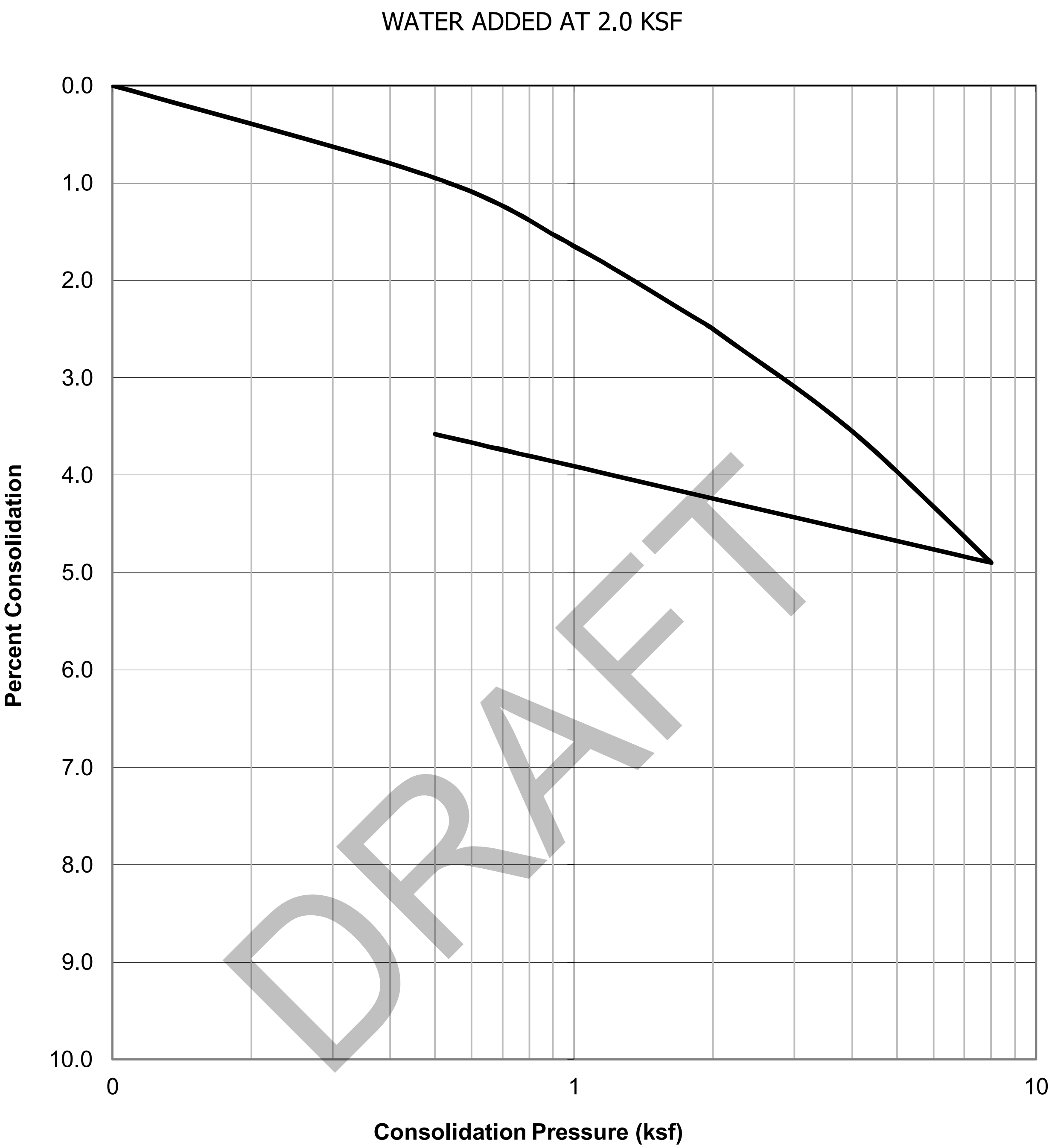
Checked by: RP

Project No.: A9930-06-01

SOUTHWEST AND SOUTHEAST CORNERS OF
EXPOSITION AND CRENSHAW BLVD.
LOS ANGELES, CALIFORNIA

Aug 19

Figure B21



SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B3@20	Grayish Brown Sand with Silt (SP-SM)	85.1	28.6	23.8

GEOCON

CONSOLIDATION TEST RESULTS

ASTM D-2435

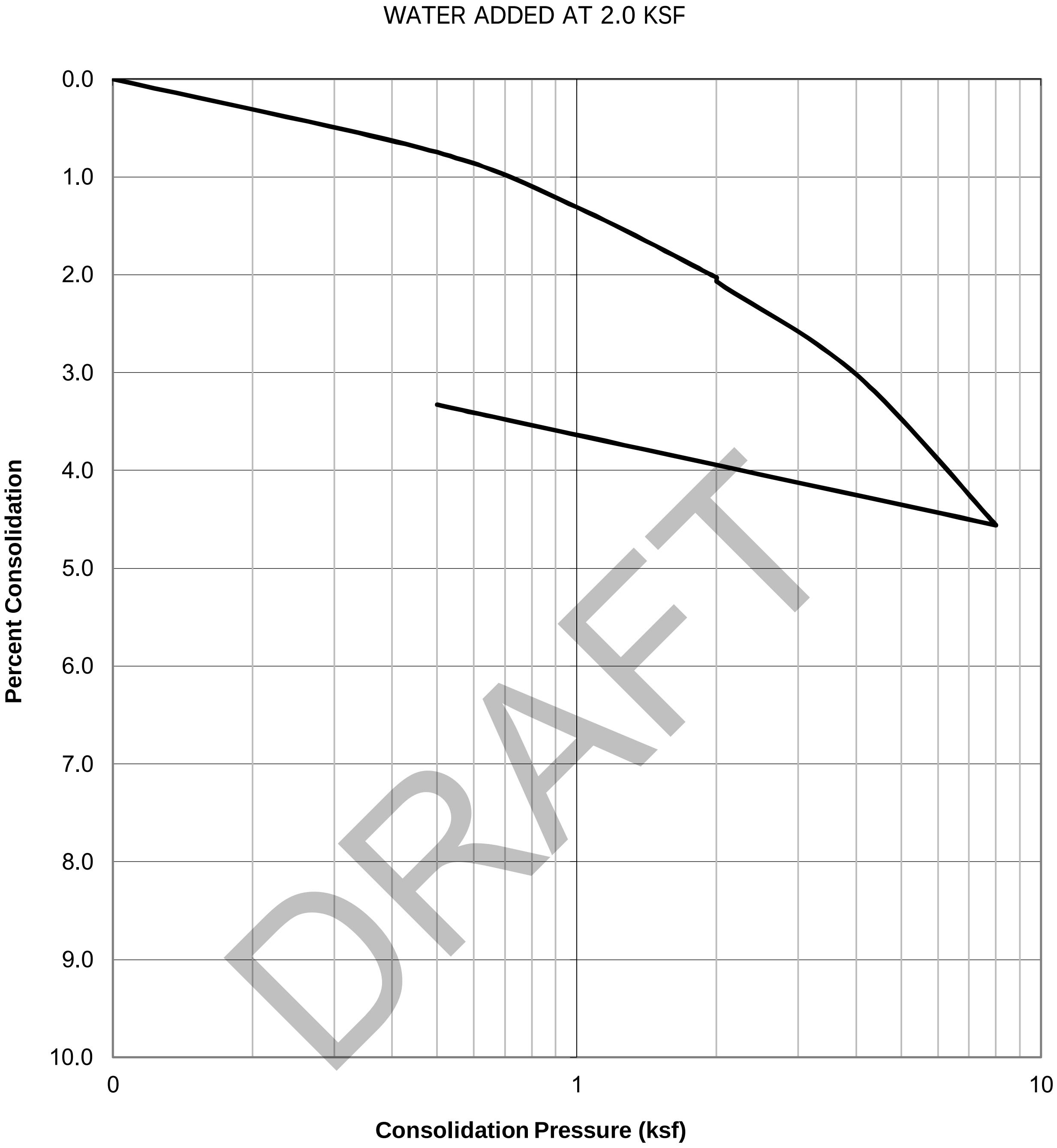
Checked by: RP

Project No.: A9930-06-01

SOUTHWEST AND SOUTHEAST CORNERS OF
EXPOSITION AND CRENSHAW BLVD.
LOS ANGELES, CALIFORNIA

Aug 19

Figure B22



SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B3@25	Grayish Brown Sand with Silt (SP-SM)	90.1	31.7	31.1

GEOCON

CONSOLIDATION TEST RESULTS

ASTM D-2435

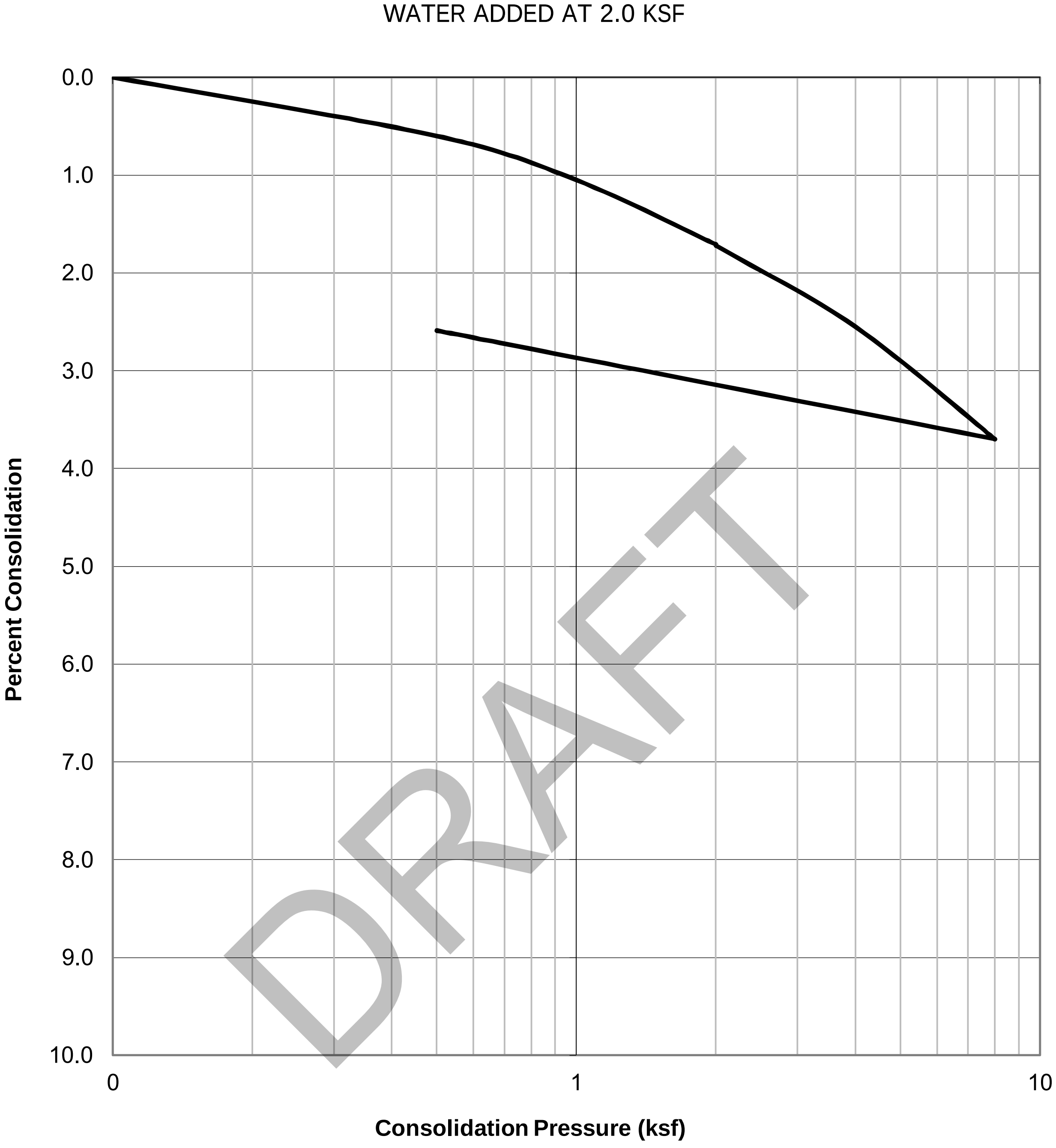
Checked by: RP

Project No.: A9930-06-01

SOUTHWEST AND SOUTHEAST CORNERS OF
EXPOSITION AND CRENSHAW BLVD.
LOS ANGELES, CALIFORNIA

Aug 19

Figure B23



SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B8@6	Olive Brown Silty Sand (SM)	112.1	17.5	18.6

GEOCON

CONSOLIDATION TEST RESULTS

ASTM D-2435

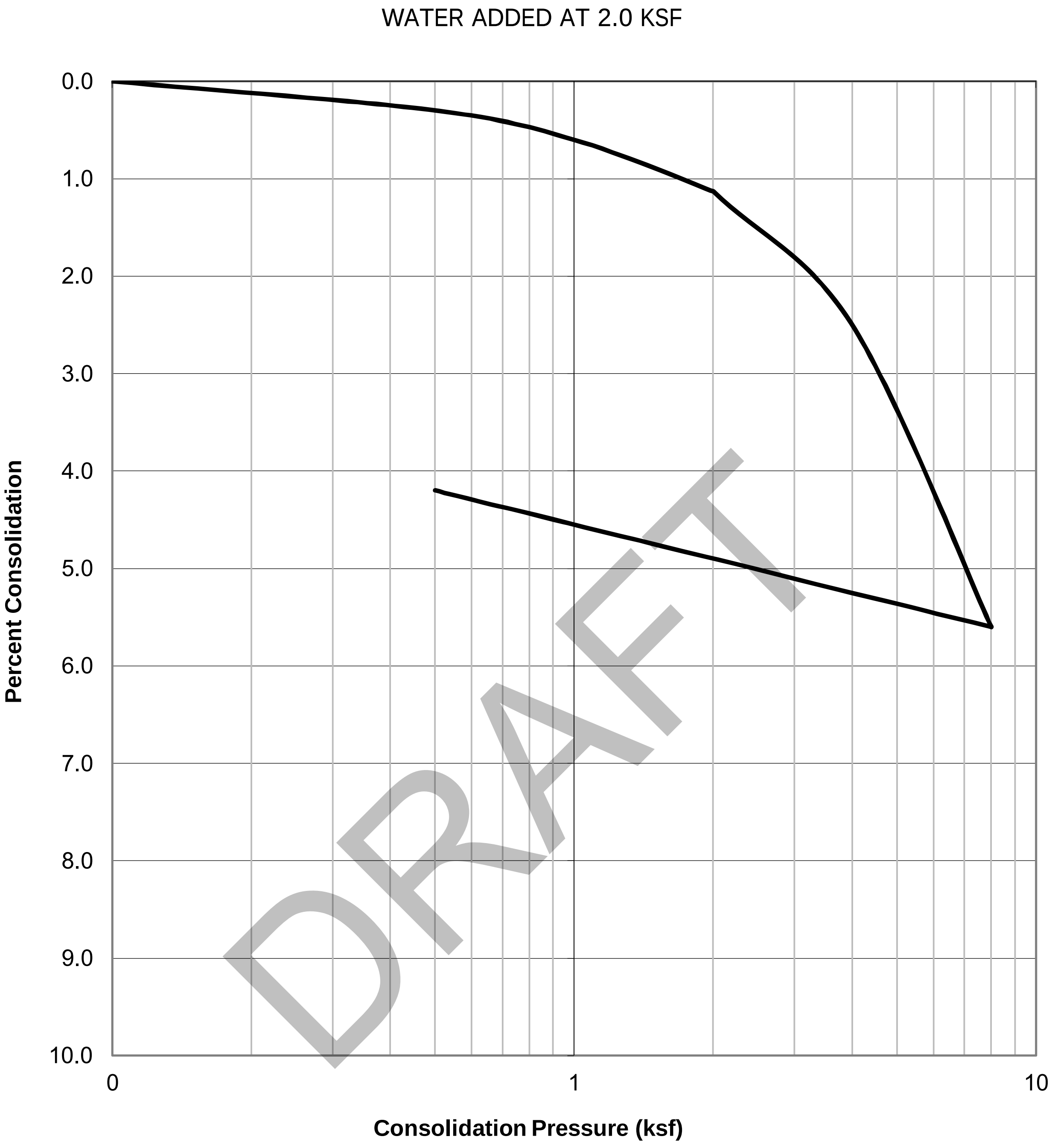
Checked by: RP

Project No.: A9930-06-01

SOUTHWEST AND SOUTHEAST CORNERS OF
EXPOSITION AND CRENSHAW BLVD.
LOS ANGELES, CALIFORNIA

Aug 19

Figure B24



SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B8@9	Olive Brown Silty Sand (SM)	89.0	26.4	32.9

GEOCON

CONSOLIDATION TEST RESULTS

ASTM D-2435

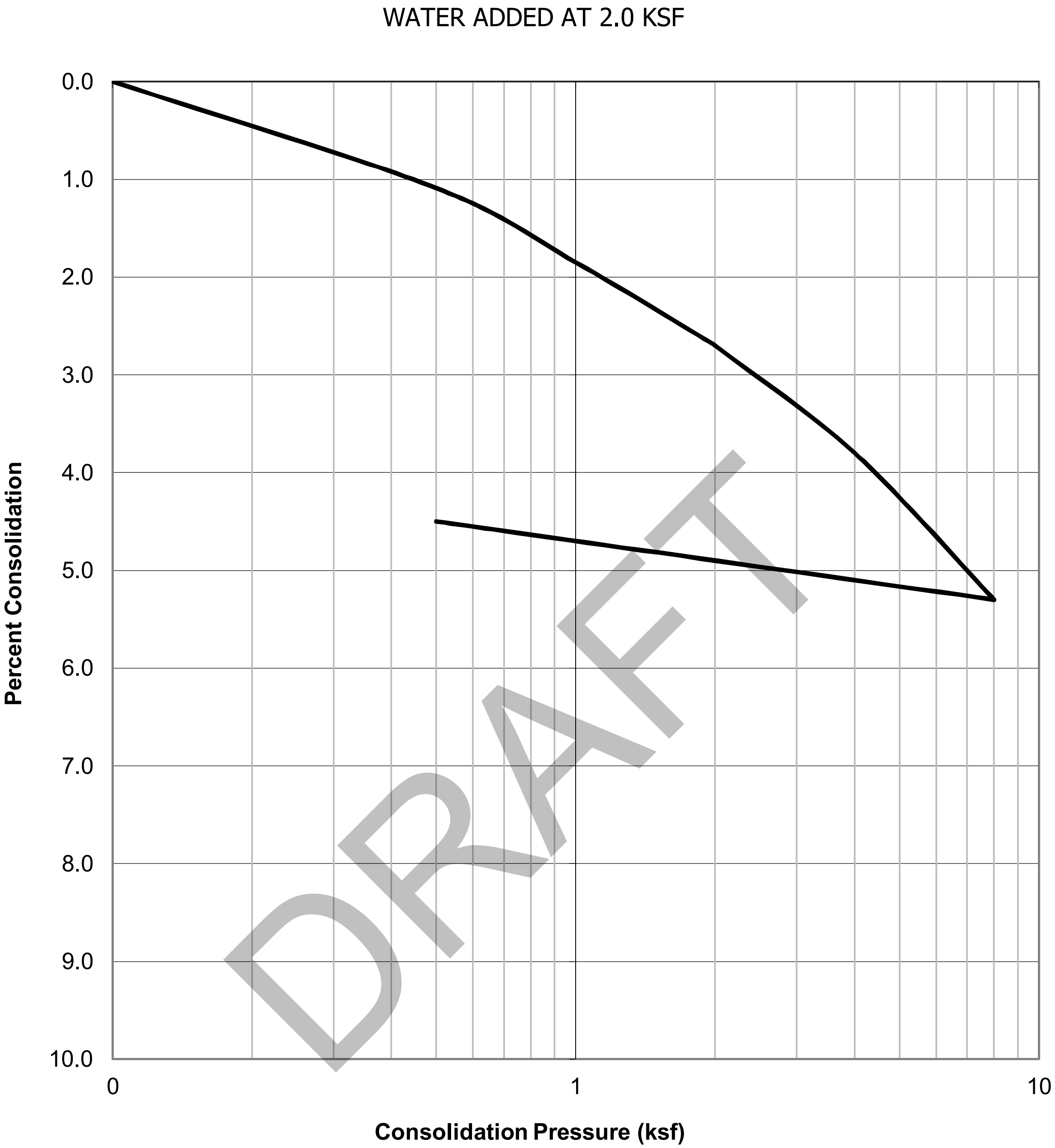
Checked by: RP

Project No.: A9930-06-01

SOUTHWEST AND SOUTHEAST CORNERS OF
EXPOSITION AND CRENSHAW BLVD.
LOS ANGELES, CALIFORNIA

Aug 19

Figure B25



SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B8@12	Drk. Olive Brown Silt w/Sand (ML)	95.2	29.4	25.7

GEOCON

CONSOLIDATION TEST RESULTS

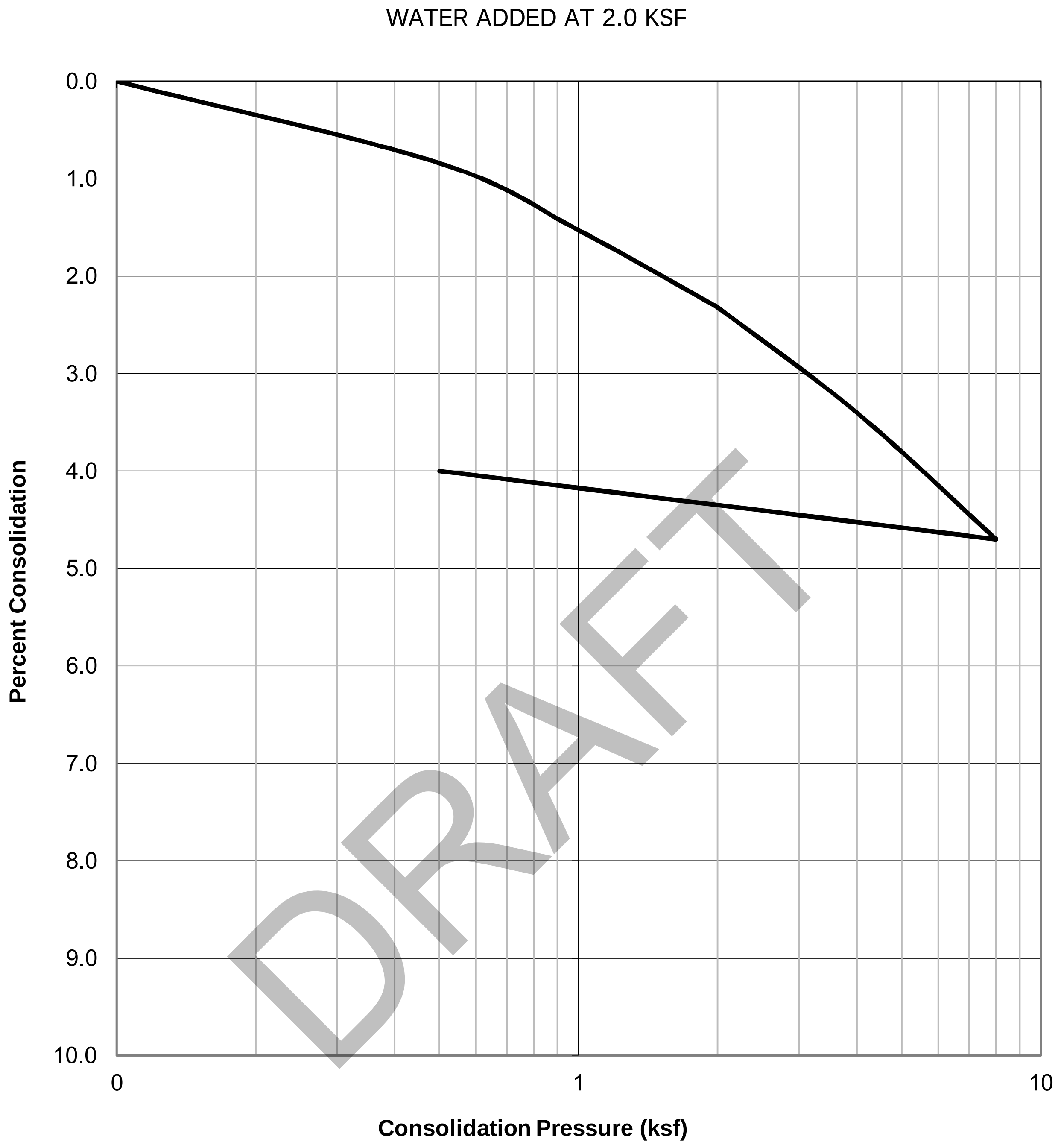
ASTM D-2435

Checked by: RP

Project No.: A9930-06-01

SOUTHWEST AND SOUTHEAST CORNERS OF
EXPOSITION AND CRENSHAW BLVD.
LOS ANGELES, CALIFORNIA

Aug 19



SAMPLE ID.	**SOIL TYPE**	**DRY DENSITY (PCF)**	**INITIAL MOISTURE (%)**	**FINAL MOISTURE (%)**
B8@15	Olive Brown Silt w/Sand (ML)	100.4	26.0	23.6

GEOCON

CONSOLIDATION TEST RESULTS

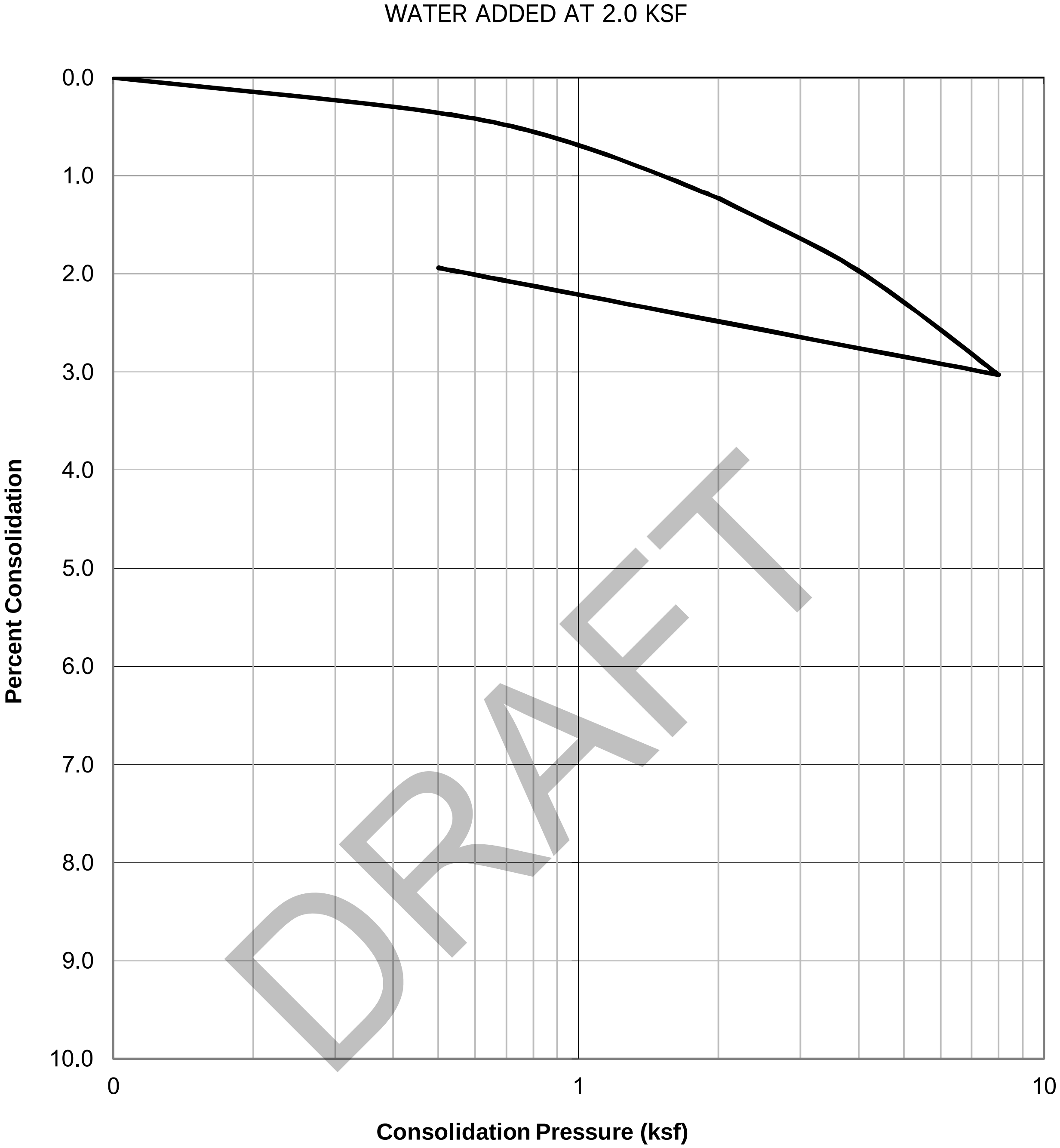
ASTM D-2435

Checked by: RP

Project No.: A9930-06-01

SOUTHWEST AND SOUTHEAST CORNERS OF
EXPOSITION AND CRENSHAW BLVD.
LOS ANGELES, CALIFORNIA

Aug 19



SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B8@20	Dark Olive Silt w/Sand (ML)	96.8	29.1	30.1

GEOCON

CONSOLIDATION TEST RESULTS

ASTM D-2435

Checked by: RP

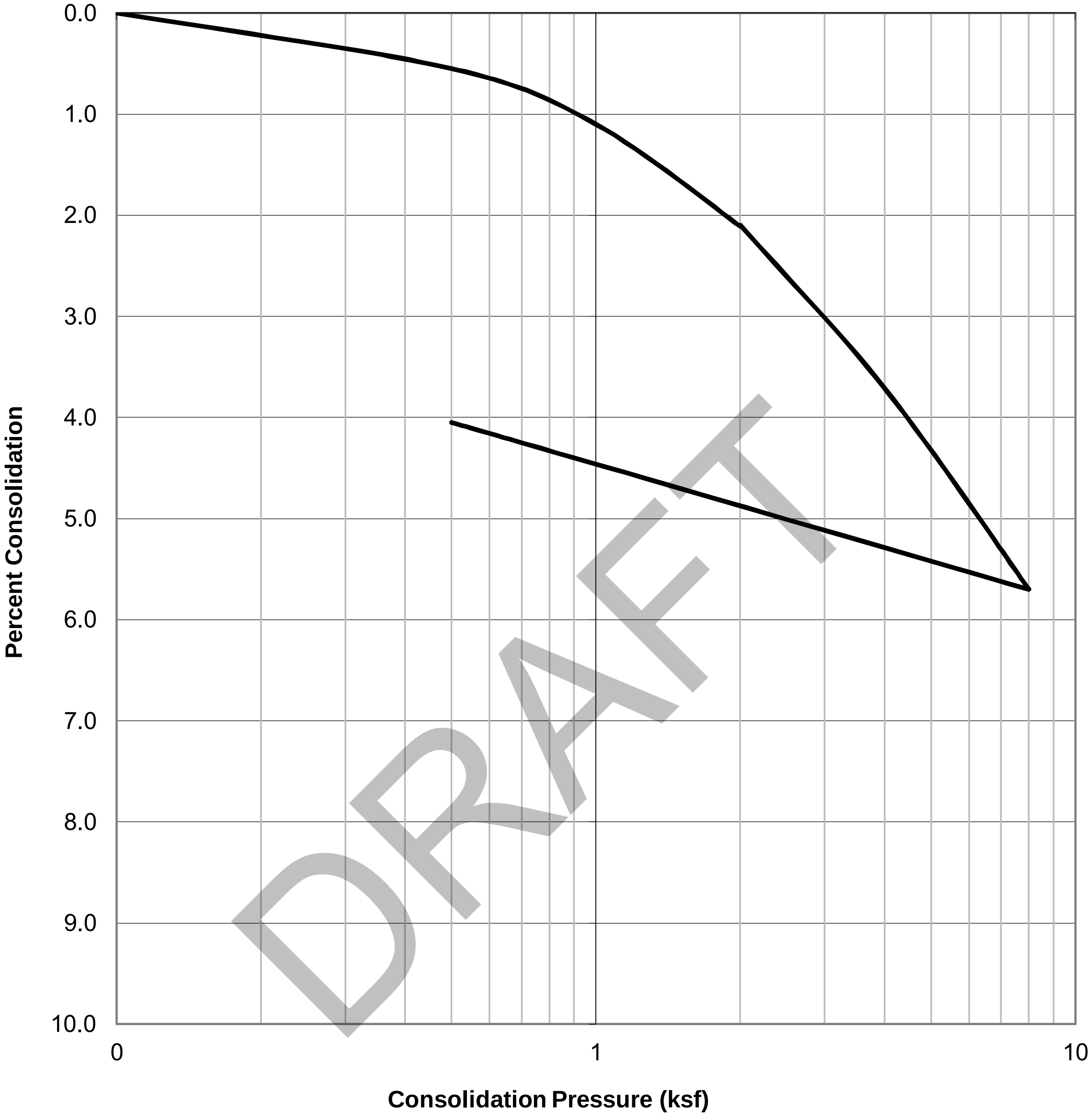
Project No.: A9930-06-01

SOUTHWEST AND SOUTHEAST CORNERS OF
EXPOSITION AND CRENSHAW BLVD.
LOS ANGELES, CALIFORNIA

Aug 19

Figure B28

WATER ADDED AT 2.0 KSF



SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B8@25	Dark Olive Brown Silt with Sand (ML)	87.9	35.2	33.5

GEOCON

CONSOLIDATION TEST RESULTS

ASTM D-2435

Checked by: RP

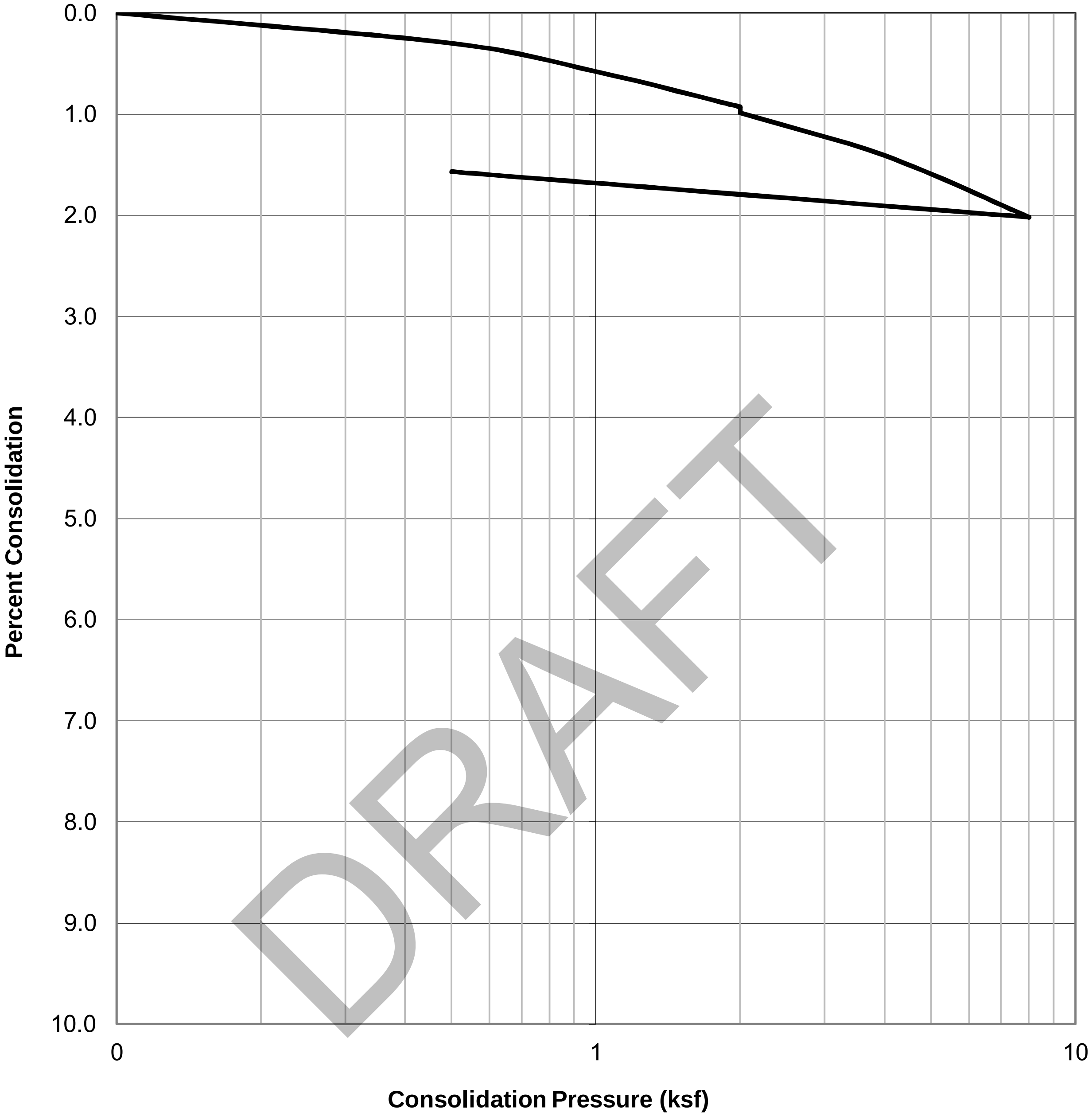
Project No.: A9930-06-01

SOUTHWEST AND SOUTHEAST CORNERS OF
EXPOSITION AND CRENSHAW BLVD.
LOS ANGELES, CALIFORNIA

Aug 19

Figure B29

WATER ADDED AT 2.0 KSF



SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B8@30	Brown Silty Sand w/Gravel (SM)	126.8	10.9	12.5

GEOCON

CONSOLIDATION TEST RESULTS

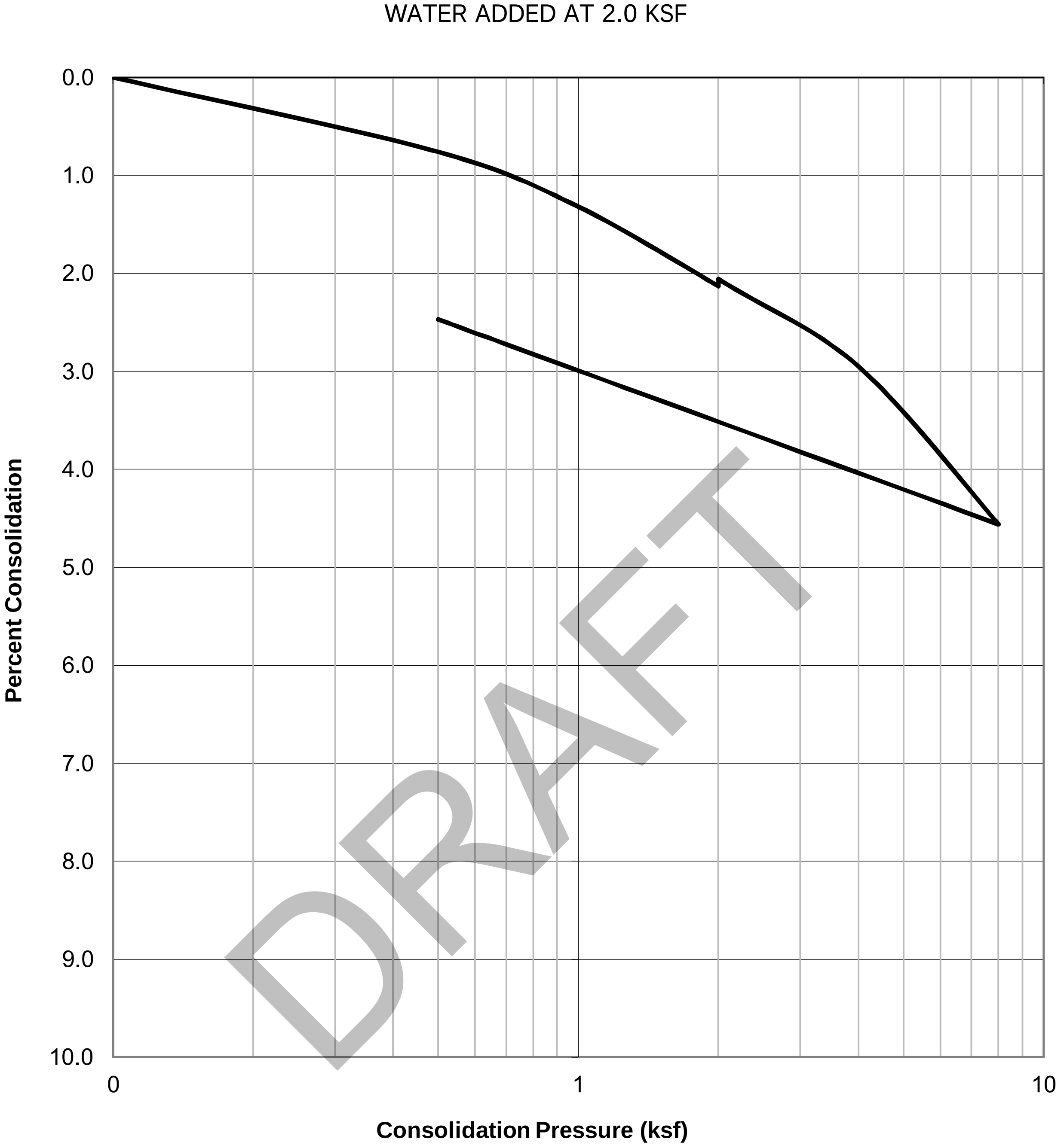
ASTM D-2435

Checked by: RP

Project No.: A9930-06-01

SOUTHWEST AND SOUTHEAST CORNERS OF
EXPOSITION AND CRENSHAW BLVD.
LOS ANGELES, CALIFORNIA

Aug 19



SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B9@5	Olive Brown Silt with Clay (ML)	97.6	25.2	26.3

GEOCON

CONSOLIDATION TEST RESULTS

ASTM D-2435

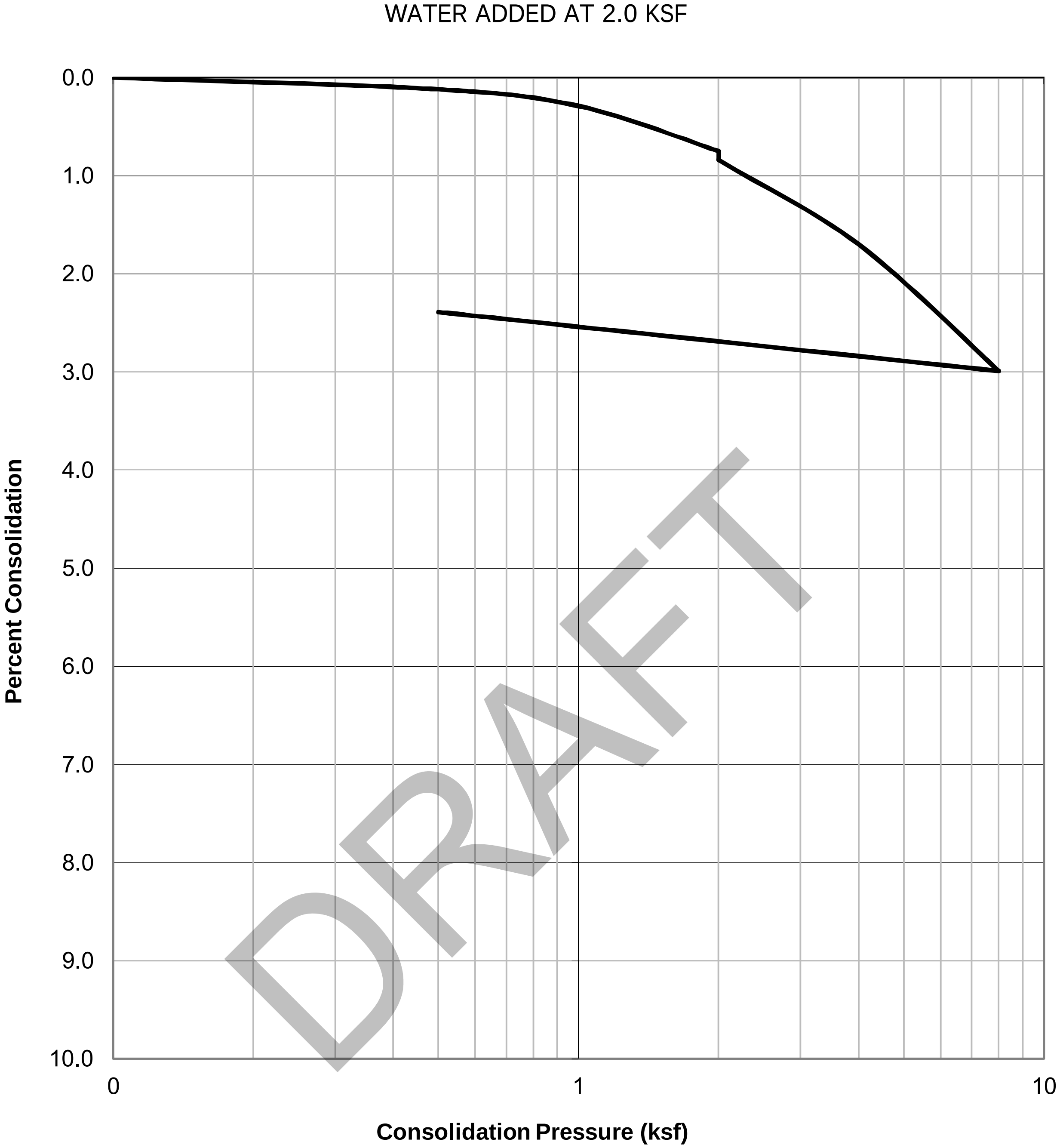
Checked by: RP

Project No.: A9930-06-01

SOUTHWEST AND SOUTHEAST CORNERS OF
EXPOSITION AND CRENSHAW BLVD.
LOS ANGELES, CALIFORNIA

Aug 19

Figure B31



SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B9@10	Yellowish Brown Silty Sand (SM)	95.4	18.4	23.5

GEOCON

CONSOLIDATION TEST RESULTS

ASTM D-2435

Checked by: RP

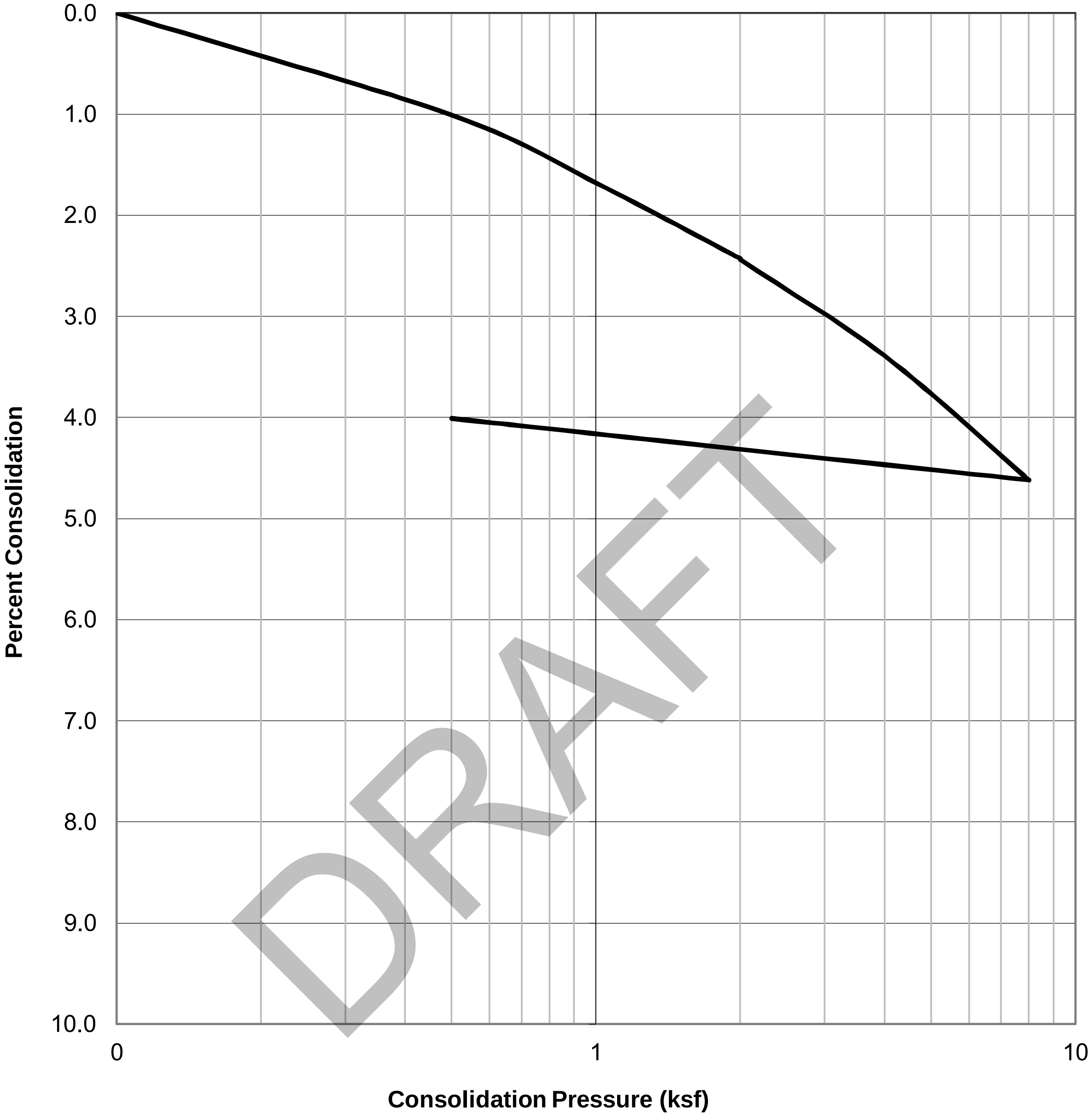
Project No.: A9930-06-01

SOUTHWEST AND SOUTHEAST CORNERS OF
EXPOSITION AND CRENSHAW BLVD.
LOS ANGELES, CALIFORNIA

Aug 19

Figure B32

WATER ADDED AT 2.0 KSF



SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B9@15	Brown to Yellowish Brown Sand with Silt (SP-SM)	105.4	20.9	18.1

GEOCON

CONSOLIDATION TEST RESULTS

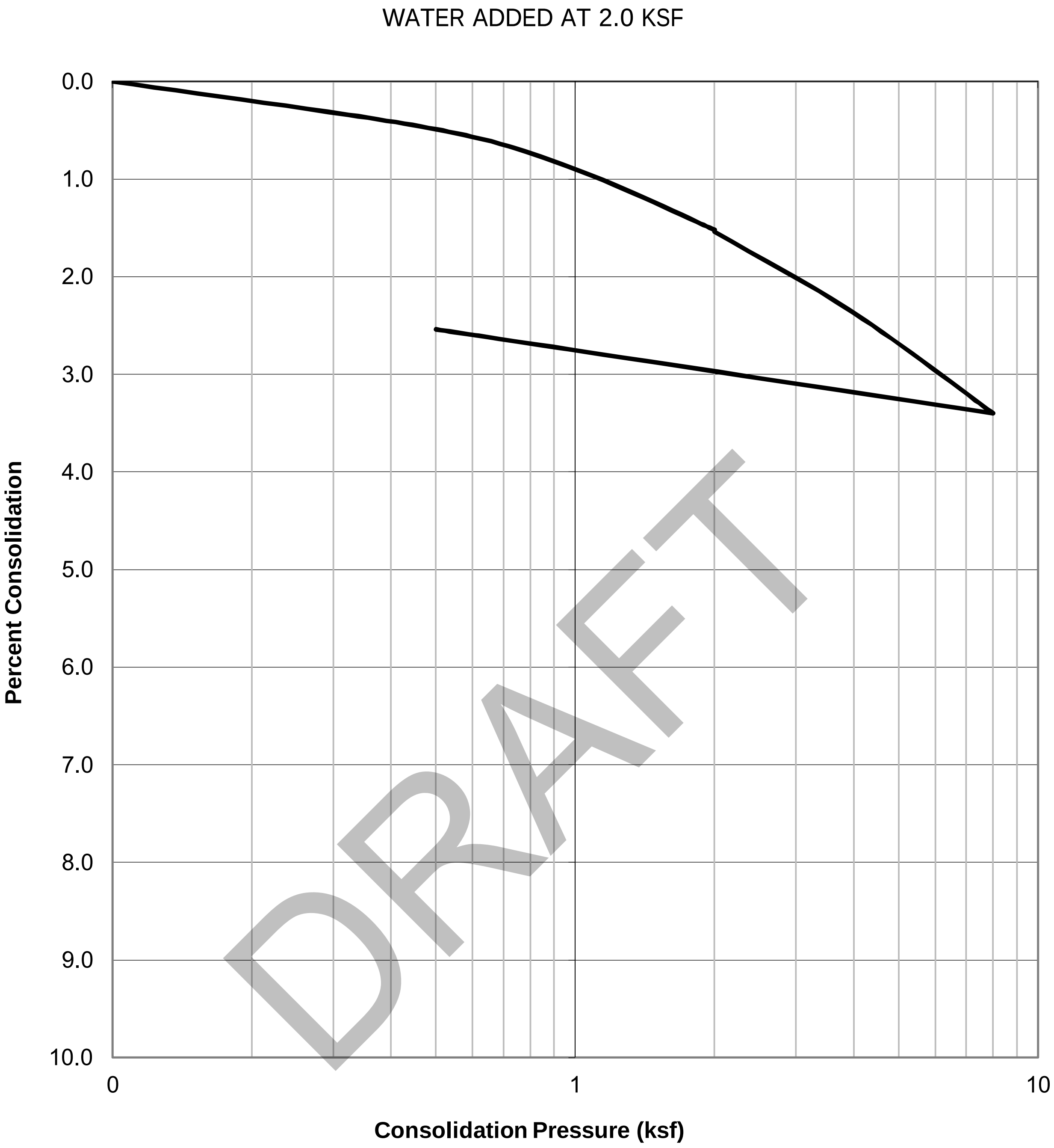
ASTM D-2435

Checked by: RP

Project No.: A9930-06-01

SOUTHWEST AND SOUTHEAST CORNERS OF
EXPOSITION AND CRENSHAW BLVD.
LOS ANGELES, CALIFORNIA

Aug 19



SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B9@20	Dark Olive Sand with Silt (SP-SM)	95.4	28.5	27.8

GEOCON

CONSOLIDATION TEST RESULTS

ASTM D-2435

Checked by: RP

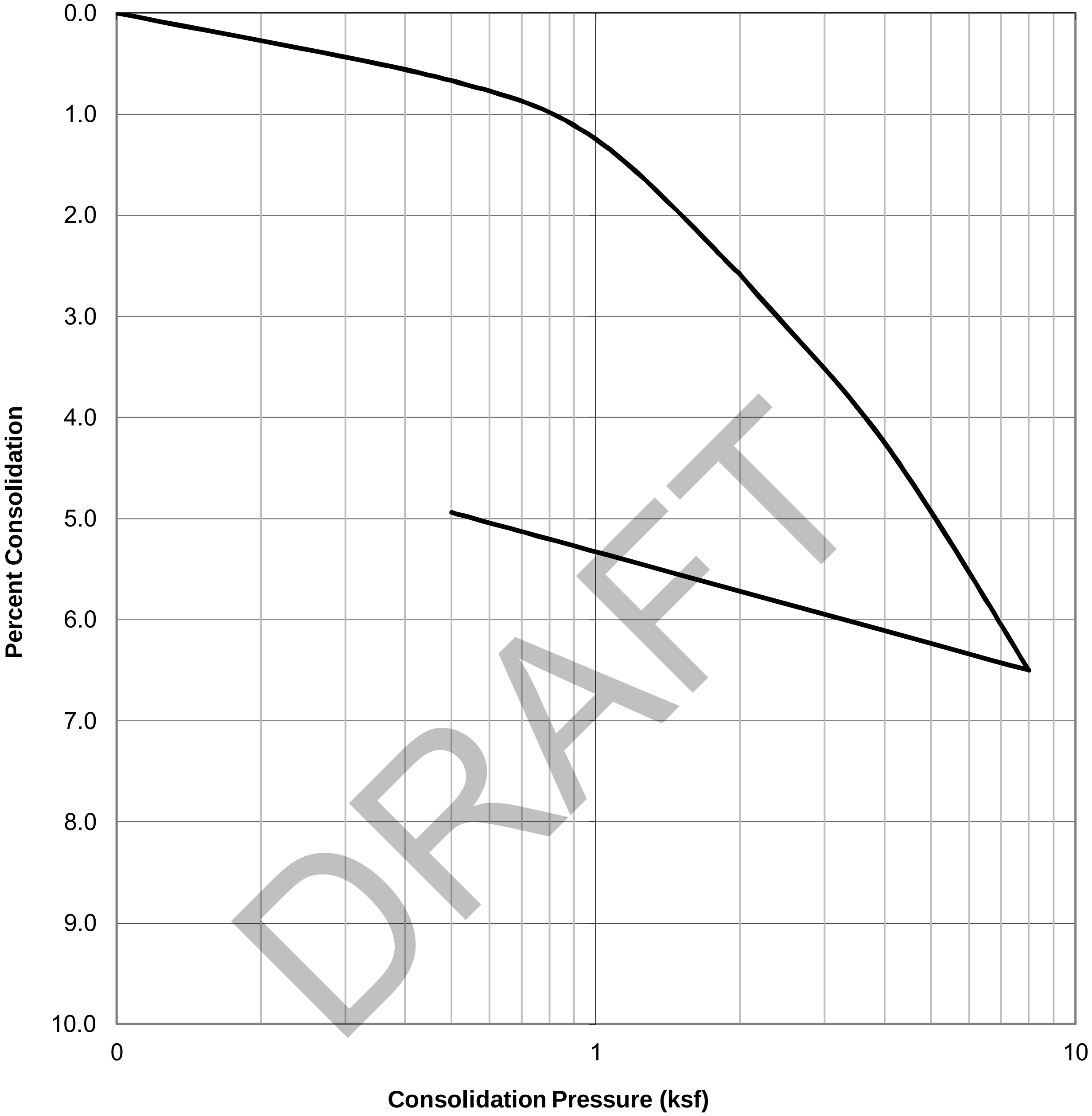
Project No.: A9930-06-01

SOUTHWEST AND SOUTHEAST CORNERS OF
EXPOSITION AND CRENSHAW BLVD.
LOS ANGELES, CALIFORNIA

Aug 19

Figure B34

WATER ADDED AT 2.0 KSF



SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B9@25	Dark Olive Brown Clay (CL)	71.5	49.0	44.9

GEOCON

CONSOLIDATION TEST RESULTS

ASTM D-2435

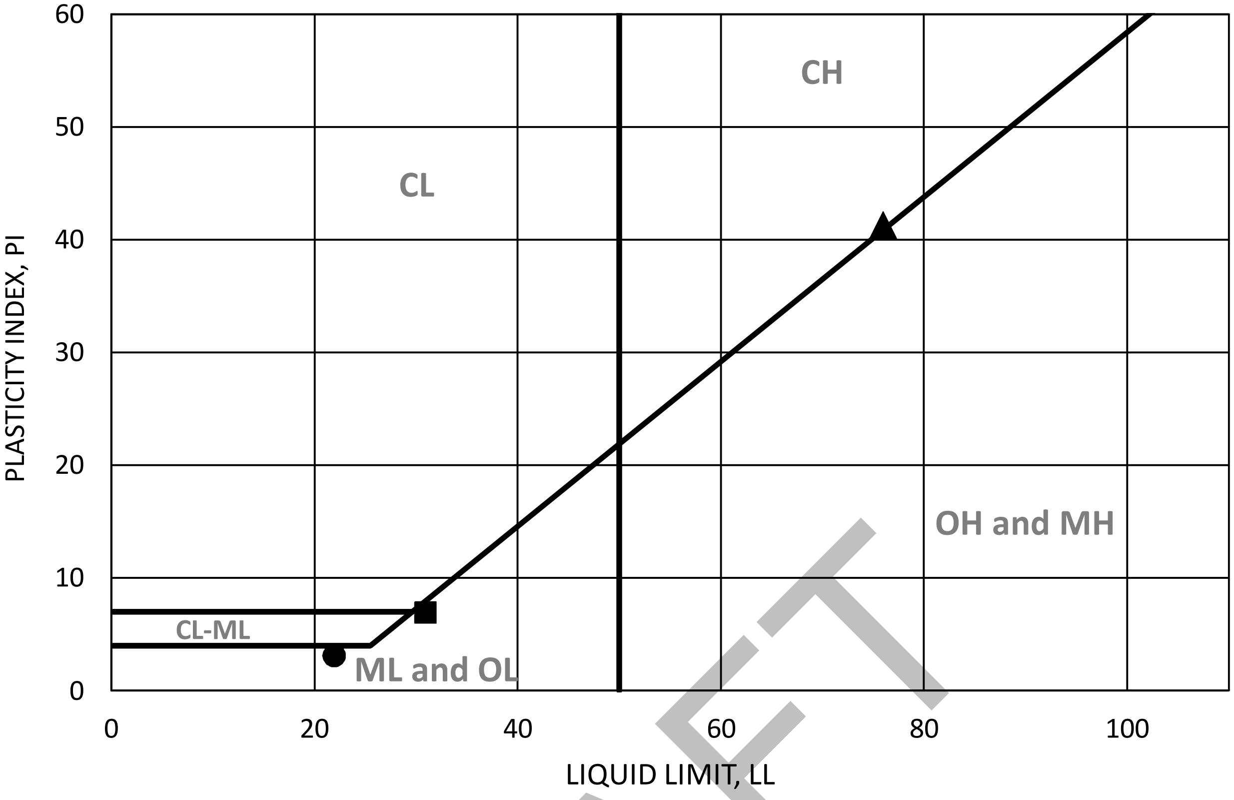
Checked by: RP

Project No.: A9930-06-01

SOUTHWEST AND SOUTHEAST CORNERS OF
EXPOSITION AND CRENSHAW BLVD.
LOS ANGELES, CALIFORNIA

Aug 19

Figure B35



SYMBOL	BORING	DEPTH (ft)	LL	PL	PI	MOISTURE CONTENT AT SATURATION	SOIL BEHAVIOR
■	B1	12	31	24	7	---	ML
◆	B1	17	N/P	N/P	N/P	---	
▲	B1	22	76	35	41	33	CH
●	B1	27	22	19	3	---	ML
□							
◇							
△							
○							

N/P = Non-Plastic

GEOCON

ATTERBERG LIMITS

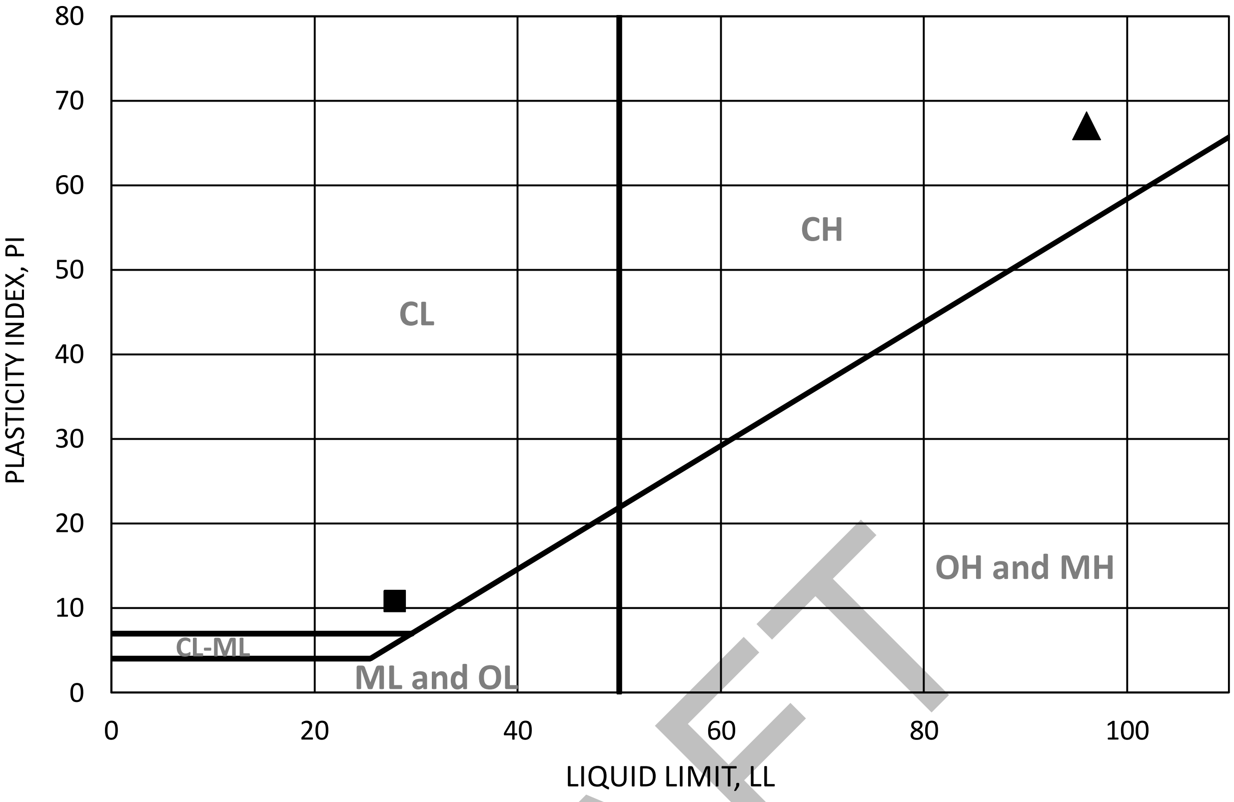
ASTM D-4318

Checked by: RP

Project No.: A9930-06-01

SOUTHWEST AND SOUTHEAST CORNERS OF
EXPOSITION AND CRENSHAW BLVD.
LOS ANGELES, CALIFORNIA

Aug 19



SYMBOL	BORING	DEPTH (ft)	LL	PL	PI	MOISTURE CONTENT AT SATURATION	SOIL BEHAVIOR
■	B7	12'	28	17	11	---	CL
◆	B7	17'	N/P	N/P	N/P	---	
▲	B7	22'	96	29	67	28	CH
●							
□							
◇							
△							
○							

N/P = Non-Plastic

GEOCON

ATTERBERG LIMITS

ASTM D-4318

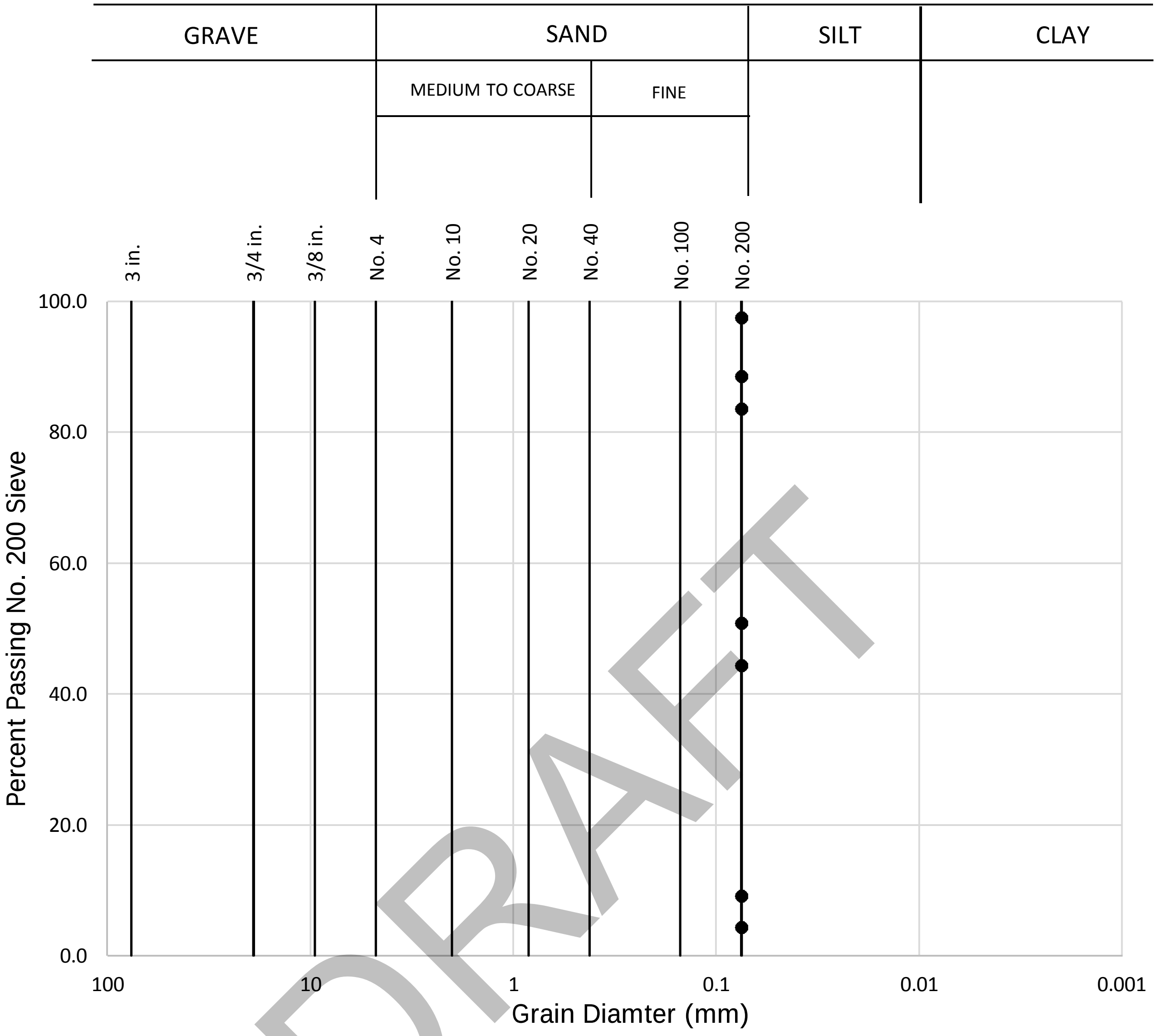
Checked by: RP

Project No.: A9930-06-01

SOUTHWEST AND SOUTHEAST CORNERS OF
EXPOSITION AND CRENSHAW BLVD.
LOS ANGELES, CALIFORNIA

Aug 19

Figure B37



Sample No.	Percent Passing No. 200 Sieve
B1 @ 12'	83.6
B1 @ 15'	44.3
B1 @ 17'	9.1
B1 @ 20'	97.5
B1 @ 22'	88.5
B1 @ 27'	50.8
B1 @ 32'	4.3

GEOCON

GRAIN SIZE ANALYSIS

ASTM D-1140

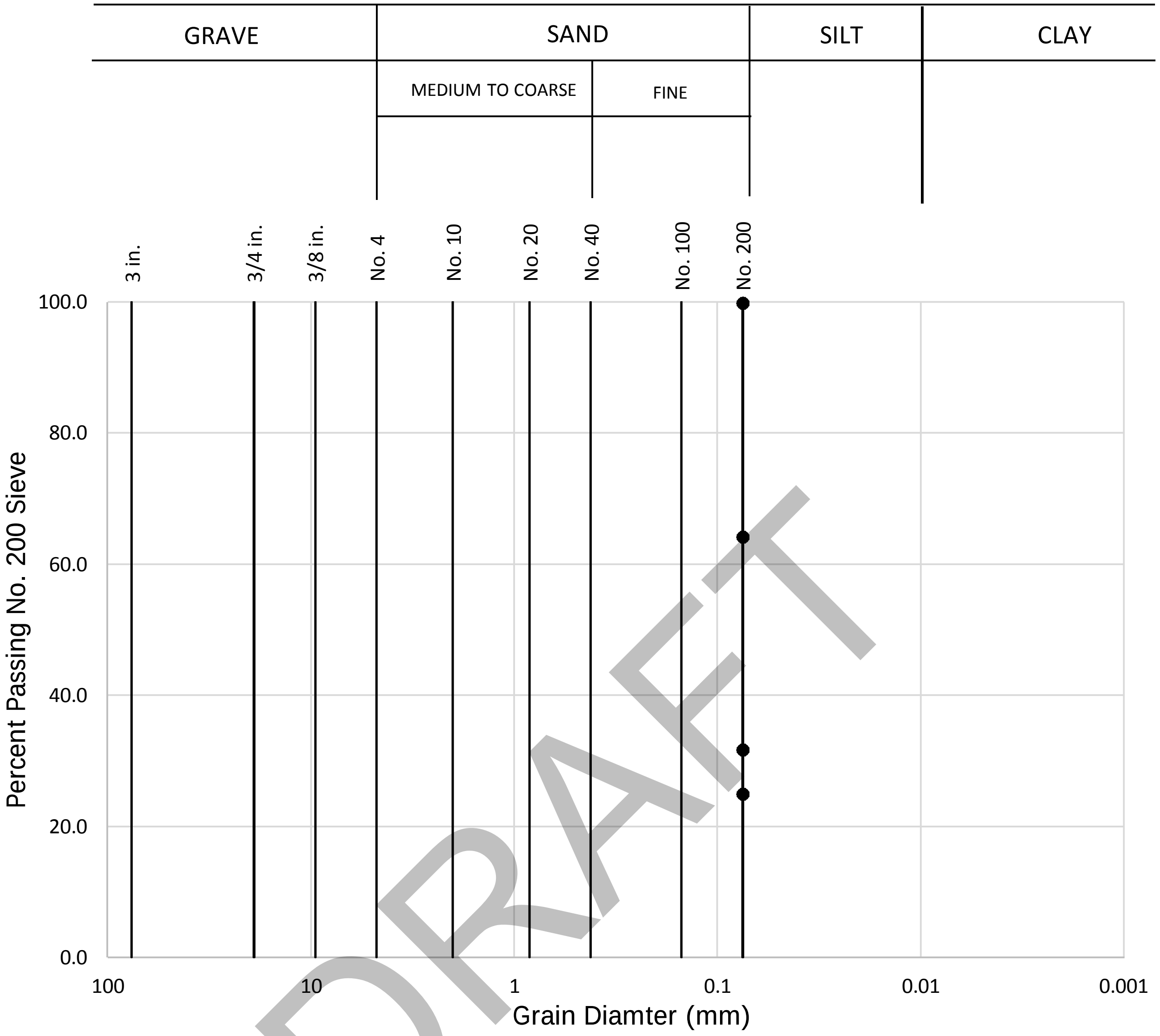
Checked by: RP

Project No.: A9930-06-01

SOUTHWEST AND SOUTHEAST CORNERS OF
EXPOSITION AND CRENSHAW BLVD.
LOS ANGELES, CALIFORNIA

Aug 19

Figure B38



Sample No.	Percent Passing No. 200 Sieve
B7 @ 7'	24.8
B7 @ 12'	64.1
B7 @ 17'	31.6
B7 @ 22'	99.8

GEOCON

GRAIN SIZE ANALYSIS

ASTM D-1140

Checked by: RP

Project No.: A9930-06-01

SOUTHWEST AND SOUTHEAST CORNERS OF
EXPOSITION AND CRENSHAW BLVD.
LOS ANGELES, CALIFORNIA

Aug 19

Figure B39

B7 & B8 @ 0-5

MOLDED SPECIMEN		BEFORE TEST	AFTER TEST
Specimen Diameter	(in.)	4.0	4.0
Specimen Height	(in.)	1.0	1.0
Wt. Comp. Soil + Mold	(gm)	745.4	802.5
Wt. of Mold	(gm)	367.9	367.9
Specific Gravity	(Assumed)	2.7	2.7
Wet Wt. of Soil + Cont.	(gm)	491.9	802.5
Dry Wt. of Soil + Cont.	(gm)	463.7	342.0
Wt. of Container	(gm)	191.9	367.9
Moisture Content	(%)	10.4	27.1
Wet Density	(pcf)	113.9	130.9
Dry Density	(pcf)	103.2	103.0
Void Ratio		0.6	0.7
Total Porosity		0.4	0.4
Pore Volume	(cc)	80.3	88.3
Degree of Saturation	(%) [S_{meas}]	44.6	104.8

Date	Time	Pressure (psi)	Elapsed Time (min)	Dial Readings (in.)
7/11/2019	10:00	1.0	0	0.315
7/11/2019	10:10	1.0	10	0.3145
Add Distilled Water to the Specimen				
7/12/2019	10:00	1.0	1430	0.353
7/12/2019	11:00	1.0	1490	0.353

Expansion Index (EI meas) =	38.5
Expansion Index (Report) =	**39**

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: 2016 California Building Code, Section 1803.5.3

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

GEOCON

EXPANSION INDEX TEST RESULTS

ASTM D 4829

Checked by: RP

Project No.: A9930-06-01

SOUTHWEST AND SOUTHEAST CORNERS OF
EXPOSITION AND CRENSHAW BLVD.
LOS ANGELES, CALIFORNIA

Aug 19

Figure B40

B5@0-5'

MOLDED SPECIMEN		BEFORE TEST	AFTER TEST
Specimen Diameter	(in.)	4.0	4.0
Specimen Height	(in.)	1.0	1.1
Wt. Comp. Soil + Mold	(gm)	742.2	798.0
Wt. of Mold	(gm)	367.9	367.9
Specific Gravity	(Assumed)	2.7	2.7
Wet Wt. of Soil + Cont.	(gm)	491.9	798.0
Dry Wt. of Soil + Cont.	(gm)	457.4	331.2
Wt. of Container	(gm)	191.9	367.9
Moisture Content	(%)	13.0	29.9
Wet Density	(pcf)	112.9	129.6
Dry Density	(pcf)	99.9	99.8
Void Ratio		0.7	0.8
Total Porosity		0.4	0.4
Pore Volume	(cc)	84.3	99.8
Degree of Saturation	(%) [S_{meas}]	51.5	99.1

Date	Time	Pressure (psi)	Elapsed Time (min)	Dial Readings (in.)
7/11/2019	10:00	1.0	0	0.242
7/11/2019	10:10	1.0	10	0.241
Add Distilled Water to the Specimen				
7/12/2019	10:00	1.0	1430	0.316
7/12/2019	11:00	1.0	1490	0.316

Expansion Index (EI meas) =	75
Expansion Index (Report) =	**75**

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: 2016 California Building Code, Section 1803.5.3

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

GEOCON

EXPANSION INDEX TEST RESULTS

ASTM D 4829

Checked by: RP

Project No.: A9930-06-01

SOUTHWEST AND SOUTHEAST CORNERS OF
EXPOSITION AND CRENSHAW BLVD.
LOS ANGELES, CALIFORNIA

Aug 19

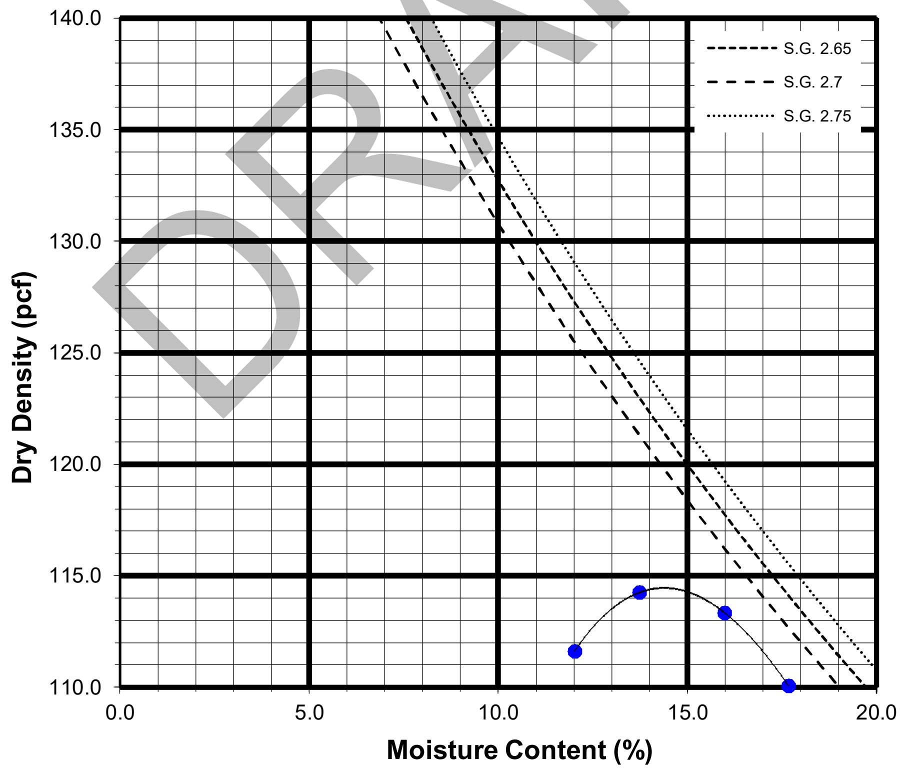
Figure B41

Sample No:

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

TEST NO.		1	2	3	4	5	6
Wt. Compacted Soil + Mold	(g)	6062	6136	6159	6130		
Weight of Mold	(g)	4173	4173	4173	4173		
Net Weight of Soil	(g)	1889	1963	1986	1956		
Wet Weight of Soil + Cont.	(g)	753.8	795.0	686.3	676.1		
Dry Weight of Soil + Cont.	(g)	686.4	715.1	612.0	593.3		
Weight of Container	(g)	125.4	133.4	147.2	124.8		
Moisture Content	(%)	12.0	13.7	16.0	17.7		
Wet Density	(pcf)	125.0	129.9	131.5	129.5		
Dry Density	(pcf)	111.6	114.3	113.3	110.1		

Maximum Dry Density (pcf)	**114.5**	**Optimum Moisture Content (%)**	**14.5**



Preparation Method: A

GEOCON

MODIFIED COMPACTION TEST OF SOILS

ASTM D-1557

Checked by: RP

Project No.: A9930-06-01

SOUTHWEST AND SOUTHEAST CORNERS OF
EXPOSITION AND CRENSHAW BLVD.
LOS ANGELES, CALIFORNIA

Aug 19

Figure B42

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

Sample No.	**pH**	**Resistivity (ohm centimeters)**
B7&B8 @ 0-5	8.7	11000 (Mildly Corrosive)

SUMMARY OF LABORATORY CHLORIDE CONTENT TEST RESULTS
EPA NO. 325.3

Sample No.	Chloride Ion Content (%)
B7&B8@0-5	0.007

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

Sample No.	**Water Soluble Sulfate (% SQ_4)**	**Sulfate Exposure***
B7&B8@0-5	0.026	S0

GEOCON

CORROSIVITY TEST RESULTS

Checked by: RP

Project No.: A9930-06-01

SOUTHWEST AND SOUTHEAST CORNERS OF
EXPOSITION AND CRENSHAW BLVD.
LOS ANGELES, CALIFORNIA

Aug 19

Figure B43