Appendix E Preliminary Geotechnical Investigation

Appendices

This page intentionally left blank.

PRELIMINARY GEOTECHNICAL INVESTIGATION

PROPOSED MULTI-FAMILY RESIDENTIAL DEVELOPMENT
1683 SUNFLOWER AVENUE
COSTA MESA, CALIFORNIA

DRAFT

GEOCON
WEST, INC.
GEOTECHNICAL
ENVIRONMENTAL
MATERIALS

PREPARED FOR

ROSE EQUITIES
BEVERLY HILLS, CALIFORNIA

PROJECT NO. A9933-88-01

JULY 24, 2019

GEOCON
WEST, INC.
GEOTECHNICAL ■ ENVIRONMENTAL ■ MATERIALS

Project No. A9933-88-01
July 24, 2019

Rose Equities
8383 Wilshire Boulevard, Suite 632
Beverly Hills, California 90211

Attention: Mr. Brent Stoll

Subject: PRELIMINARY GEOTECHNICAL INVESTIGATION
PROPOSED MULTI-FAMILY RESIDENTIAL DEVELOPMENT
1683 SUNFLOWER AVENUE
COSTA MESA, CALIFORNIA

Dear Mr. Stoll,

In accordance with your authorization of our proposal with a revised date of February 4, 2019, we have prepared a preliminary geotechnical investigation for the proposed multi-family residential development located at 1683 Sunflower Avenue in the City of Costa Mesa, California. The accompanying report presents the findings of our study, and our conclusions and recommendations pertaining to the geotechnical aspects of proposed design and construction. Based on the results of our investigation, it is our opinion that the site can be developed as proposed, provided the recommendations of this report are followed and implemented during design and construction.

The primary intent of this study was to address potential geologic hazards and geotechnical conditions that could impact the project and to provide preliminary design recommendations. As the project design progresses, updated geotechnical recommendations should be provided for 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

Jamie K. Fink CEG 2636	Jelisa Thomas Adams GE 3092	John Hoobs CEG 1524

(Email) Addressee

DRAFT

TABLE OF CONTENTS

DRAFT

TABLE OF CONTENTS (Continued)

DRAFT

PRELIMINARY GEOTECHNICAL INVESTIGATION

1. PURPOSE AND SCOPE

This report presents the results of a preliminary geotechnical investigation for the proposed multi-family residential development located at 1683 Sunflower Avenue in the City of Costa Mesa, California (see Vicinity Map, Figure 1). The purpose of the investigation was to evaluate subsurface soil and geologic conditions underlying the site and, based on conditions encountered, to provide preliminary conclusions and recommendations pertaining to the geotechnical aspects of design and construction. As the project design progresses, updated geotechnical recommendations should be provided for design and construction.

The scope of this investigation included a site reconnaissance, field exploration, laboratory testing, engineering analysis, and the preparation of this report. The site was explored on February 12, 25, and 26, 2019, by excavating a total of 13 8-inch diameter borings using a hollow stem auger drilling machine and by advancing five cone penetrometer tests (CPTs). The borings were excavated to depths ranging from 6 to 50½ feet below the ground surface. The CPTs were advanced to depths ranging from approximately 60 to 64 feet below the ground surface. It should be noted that the numbers CPT-3 and CPT-4 were not used. The approximate locations of the borings and CPTs are depicted on the Site Plan (see Figure 2). A detailed discussion of the field investigation, including boring logs and CPT soundings, is presented in Appendix A.

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

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

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

2. SITE AND PROJECT DESCRIPTION

The subject site is located at 1683 Sunflower Avenue in the City of Costa Mesa, California. The site is approximately 15.75 acres and is currently occupied by a warehouse structure and on-grade parking. The site is bounded by Sunflower Avenue and one and two-story commercial structures to the north, by the 405 San Diego Freeway to the south, by a one and two-story commercial shopping center to the east, and by a one-story commercial structure and associated parking to the west. The site is relatively level, with no pronounced highs or lows. Surface water drainage at the site appears to be by sheet flow along the existing ground contours to the city streets. Vegetation onsite consists of grass and trees, which are located in isolated landscape areas.

DRAFT

Based on the information provided by the Client, it is our understanding that the proposed development will include three multi-family residential buildings and a commercial building on the subject site. The multi-family residential buildings are further described as 'Building A', a wrap style six-story residential apartment building to be constructed at or near present grade; 'Building B', a podium style seven-story residential apartment building underlain by one level subterranean parking; and 'Building C', a wrap style seven-story residential apartment building to be constructed at or near present grade. A four-story creative office building, comprised of 3 office levels over one level of parking to be constructed at or near present grade, is planned at the southwest portion of the site. Additional site improvements will include parking areas, courtyards, an in-ground swimming pool, landscape areas, and fire access driveways. The proposed development is depicted on the Site Plan (see Figure 2).

Based on the preliminary nature of the design at this time, wall and column loads were not available. Column loads and wall loads for the proposed parking structure are estimated be up to 650 kips and 35 kips per linear foot, respectively. Column loads and wall loads for the proposed apartment building are estimated be up to 175 kips and 6 kips per linear foot, respectively.

We understand that final design of the project has not been completed, hence, once the design phase 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 reviewed a prior report provided to us by the Client:

> *Geotechnical Engineering Investigation, Proposed Project, 1683 Sunflower Avenue, Costa Mesa, California,* prepared by Moore Twining Associates, Inc., dated October 3, 2013.

A prior geotechnical investigation of the subject site was performed in 2013 by Moore Twining Associates, Inc., (MTA). The prior investigation included the excavation and logging of ten boring to depths ranging from 10 to 51½ feet below the ground surface. Additionally, four Cone Penetrometer Tests (CPTs) were advanced to depths of approximately 50 feet below the ground surface. The locations of the prior borings and CPTs are indicated on the Site Plan (Figure 2). Perched groundwater was encountered in one boring at a depth of 10 feet, and groundwater was encountered in another boring at a depth of 18 feet. A copy of the report prepared by Geotechnologies is provided in Appendix C.

Geocon West, Inc. has reviewed the referenced report by MTA, and the recommendations presented herein are based on analysis of the subsurface and laboratory data obtained from the prior investigation by MTA, as well as our own subsurface and laboratory data. Furthermore, we assume responsibility for the utilization of the exploration and laboratory data presented within the geotechnical report by

DRAFT

MTA. Geocon West, Inc. is the Geotechnical Consultant of Record and will be providing all necessary geotechnical consultation, plan review, design recommendations, inspection and testing services for this project. Where differing, the recommendations presented herein supersede all previous recommendations.

4. GEOLOGIC SETTING

The subject site is located in the central portion of the Orange County Coastal Plain, a relatively flat-lying alluviated surface with an average slope of less than 20 feet per mile. The lowland surface is bounded by hills and mountains on the north and east and by the Pacific Ocean to the south and southwest. Prominent structural features within the Orange County Coastal Plain include the central lowland plain, the northwest trending line of low hills and mesas near the coast underlain by the Newport-Inglewood Fault Zone (Newport Mesa, Huntington Beach Mesa, Bolsa Chica Mesa, and Landing Hill), and the San Joaquin Hills to the southeast (Department of Water Resources, 1967).

5. SOIL AND GEOLOGIC CONDITIONS

Based on our field investigation and published geologic maps of the area, the site is underlain by artificial fill and unconsolidated Holocene age alluvial fan deposits consisting of sand, silt and clay (California Geological Survey [CGS], 2012). Detailed stratigraphic profiles of the materials encountered at the site are provided on the boring logs and CPT soundings in Appendix A.

5.1 Artificial Fill

Artificial fill was encountered in our field explorations to a maximum depth of 5½ feet below existing ground surface. The artificial fill generally consists of light brown to brown to gray brown silty sand and sandy silt and sandy clay. The artificial fill is characterized as slightly moist to moist and soft to firm or medium dense. The fill is likely the result of past grading or construction activities at the site. Deeper fill may exist between excavations and in other portions of the site that were not directly explored.

5.2 Alluvial Fan Deposits

The artificial fill is underlain by Holocene age alluvial fan deposits that generally consists of brown to olive and gray brown sandy clay, sandy silt, clay, and both poorly graded and well graded sands. In general, the upper 20 to 25 feet of alluvial deposits generally consist of relatively soft to firm clay and silt which is underlain by approximately 10 to 15 feet of medium dense to dense sand.

Based on review of a published geologic map showing the distribution of localized peat deposits in the Orange County area (CDMG, 1976), the subject site is situated along the northern and western boundaries of a 'T-shaped' area identified as having a strong probability of peat deposits. As discussed in the text accompanying the geologic map, the boundaries of the identified areas are generalized because of lack of subsurface data and the maps do not establish the distribution or thicknesses of the peat deposits.

DRAFT

The current and prior subsurface exploration recorded the presence of organic odor in several borings (Geocon borings B-2, B-5, and B-7 and MTA Borings B2, B3, B5, B6) at depths ranging from 8.5 to 17 feet below ground surface (See Figure B20). The presence of roots and/or organics were observed in Geocon borings B-1 through B-7 ranging from 6 to 20 feet below ground surface.

6. GROUNDWATER

Review of the Seismic Hazard Zone Report for the Anaheim and Newport Beach 7.5-Minute Quadrangle (California Division of Mines and Geology [CDMG], 2001) indicates the historically highest groundwater level in the area is approximately 10 feet beneath the ground surface.

Groundwater was encountered in our borings at depths ranging from approximately 10 to 20 feet below the existing ground surface. Seepage was also noted in boring B5 at 7 feet. Considering the historic high groundwater level and the depth to groundwater observed 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. 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 8.23).

DRAFT

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 (formerly known as CDMG) for the Alquist-Priolo Earthquake Fault Zone Program (CGS, 2018a). 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 for surface fault rupture hazards (CGS, 2018b). 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 3, Regional Fault Map.

The closest surface trace of an active fault to the site is the Newport-Inglewood Fault Zone located approximately 3.7 miles to the southwest (Ziony and Jones, 1989). Other nearby active faults are the Whittier Fault, the Elsinore Fault Zone, and the Palos Verdes Fault (Offshore Segment) located approximately 16 miles northeast, 20½ miles northeast, and 14½ miles southwest of the site, respectively (Ziony and Jones, 1989). The active San Andreas Fault Zone is located approximately 48 miles northeast of the site (Ziony and Jones, 1989).

Several buried thrust faults, commonly referred to as blind thrusts, underlie the Los Angeles Basin (including the Orange County 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. The San Joaquin Thrust underlies the site at depth. This thrust fault and others in the greater Los Angeles/Orange County area are not exposed at the surface and do not present a potential surface fault rupture hazard at the site; however, these deep thrust faults are considered active features capable of generating future earthquakes that could result in moderate to significant ground shaking at the site.

7.2 Seismicity

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

DRAFT

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	44	ENE
Long Beach	March 10, 1933	6.4	6	SSW
Tehachapi	July 21, 1952	7.5	109	NW
San Fernando	February 9, 1971	6.6	56	NW
Whittier Narrows	October 1, 1987	5.9	27	NNW
Sierra Madre	June 28, 1991	5.8	39	N
Landers	June 28, 1992	7.3	92	ENE
Big Bear	June 28, 1992	6.4	72	ENE
Northridge	January 17, 1994	6.7	50	NW
Hector Mine	October 16, 1999	7.1	114	ENE

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).

DRAFT

2016 CBC SEISMIC DESIGN PARAMETERS

Parameter	Value	2016 CBC Reference
Site Class	E	Table 1613.3.2
MCE_R Ground Motion Spectral Response Acceleration – Class B (short), S_S	1.549g	Figure 1613.3.1(1)
MCE_R Ground Motion Spectral Response Acceleration – Class B (1 sec), S_1	0.576g	Figure 1613.3.1(2)
Site Coefficient, F_A	0.9	Table 1613.3.3(1)
Site Coefficient, F_V	2.4	Table 1613.3.3(2)
Site Class Modified MCE_R Spectral Response Acceleration (short), S_{MS}	1.394g	Section 1613.3.3 (Eqn 16-37)
Site Class Modified MCE_R Spectral Response Acceleration – (1 sec), S_{M1}	1.381g	Section 1613.3.3 (Eqn 16-38)
5% Damped Design Spectral Response Acceleration (short), S_{DS}	0.929g	Section 1613.3.4 (Eqn 16-39)
5% Damped Design Spectral Response Acceleration (1 sec), S_{D1}	0.921g	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	0.601g	Figure 22-7
Site Coefficient, F_{PGA}	1.0	Table 11.8-1
Site Class Modified MCE_G Peak Ground Acceleration, PGA_M	0.601g	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 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 6.7 magnitude event occurring at a hypocentral distance of 8.79 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 6.67 magnitude occurring at a hypocentral distance of 19.19 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.

DRAFT

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

The State of California Seismic Hazard Zone Map for the Anaheim and Newport Beach Quadrangle (CDMG, 2001) indicates that the site is located in an area identified as having a potential for liquefaction. Also, according to the Safety Element of the Costa Mesa General Plan (2015), the site is located within an area identified as having a potential for liquefaction.

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

Screening criteria presented by Bray and Sancio (2006) was used to evaluate the liquefaction susceptibility of the fine-grained soils encountered in the boring. Based on these screening criteria, fine-grained soils with a plasticity index of greater than 18 and fine-grained soils with a plasticity index of greater than 12 or a saturated water content of less than 85 percent of the liquid limit are considered not susceptible to liquefaction. Laboratory test results used for the screening criteria are presented as Figures B17 and B18.

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 6.67 earthquake, and a peak horizontal acceleration of 0.401g (⅔PGA_M). The enclosed liquefaction analyses, included herein for borings B3 and B7, indicate that the alluvial soils below the historic high groundwater could be prone to between 0.8 and 1.2 inches of liquefaction induced settlement during Design Earthquake ground motion (see enclosed calculation sheets, Figures 5 through 8).

A comparative analysis was also performed by using select CPTs and the program CLiq (Version 2.2). This program utilizes the Boulanger & Idriss (2014) method of analysis, and the same values for the historic high water table, earthquake magnitude, and peak ground acceleration as indicated above.

Based on the analyses of CPT-1, CPT-2, and CPT-5 through CPT-7, the alluvial soils below the historic high groundwater depth may be susceptible to approximately ½ inch of settlement during Design Earthquake ground motion (see enclosed settlement report, Figure 9).

DRAFT

Given that the CPTs generate a continuous soil profile, and that the driven samples in the borings may not capture thin layers of soils between the samples, the boring and CPT analyses appear to be in agreement regarding the general magnitude of potential liquefaction settlement during Design Earthquake ground motion. It is recommended that the proposed project be designed for up to ½ inch of differential liquefaction induced settlement during Design Earthquake ground motion.

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

The liquefaction analysis was also performed for the Maximum Considered Earthquake level by using a historic high groundwater table of 10 feet below the ground surface, a magnitude 6.7 earthquake, and a peak horizontal acceleration of 0.601g (PGA_M). The enclosed liquefaction analyses, included herein for borings B3 and B7, indicate that the alluvial soils below the historic high groundwater could be prone to between 0.9 and 1.3 inches of liquefaction induced settlement during Maximum Considered Earthquake ground motion (see enclosed calculation sheets, Figures 10 through 13).

Based on the analyses of CPT-1, CPT-2, and CPT-5 through CPT-7, the alluvial soils below the historic high groundwater depth may be susceptible to less than 0.6 inches of settlement during Maximum Considered Earthquake ground motion (see enclosed settlement report, Figure 14).

7.5 Slope Stability

The topography at the site is relatively level and the site is not located within an area identified as having a potential for slope instability. Additionally, the site is not located within an area identified as having a potential for earthquake-induced landslides (CDMG, 1998). There are no known landslides near the site, nor is the site in the path of any known or potential landslides. Therefore, the potential for slope stability hazards to adversely affect the proposed development is considered low.

7.6 Earthquake-Induced Flooding

Earthquake-induced flooding is inundation caused by failure of dams or other water-retaining structures due to earthquakes. Review of the Safety Element of the Costa Mesa General Plan (2015) indicates that the site is located within the inundation boundary of the Prado Dam and Santiago Reservoirs. However, these dams, 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.

DRAFT

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 resulting from a seismically-induced seiche is considered unlikely.

The site is within an area of reduced flood risk due to levee (Zone X) as defined by the Federal Emergency Management Agency (FEMA, 2018). Review of the Safety Element of the Costa Mesa General Plan (2015) indicates the areas is identified as being within a 500-year flood zone.

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 active oil or gas wells are not located in the immediate site vicinity (DOGGR, 2018). The closest oil/gas field is the Talbert (ABD) field located approximately 8,400 feet southwest of the site. The closest well to the site is the Chevron USA, Inc. Well NC-41, a plugged oil/gas well located approximately 3,500 feet to the south/southwest. 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.

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 large-scale extraction of groundwater, gas, oil, or geothermal energy is occurring or planned at the site or in the general site vicinity. There appears to be little or no potential for ground subsidence due to withdrawal of fluids or gases at the site.

DRAFT

8. CONCLUSIONS AND RECOMMENDATIONS

8.1 General

8.1.1 It is our opinion that neither soil nor geologic conditions were encountered during this investigation that would preclude the construction of the proposed development provided the recommendations presented herein are followed and implemented during design and construction. As the project design progresses, updated geotechnical recommendations should be provided for design and construction.

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

8.1.3 Based on the enclosed liquefaction induced settlement calculations, it is recommended that the proposed project be designed for ½ inch of differential settlement as a result the Design Earthquake peak ground acceleration. The grading and foundation recommendations presented herein are intended to minimize and design for the effects of liquefaction settlement on proposed structures.

8.1.4 The results of our laboratory testing indicate that the existing alluvium could yield excessive static and differential settlements upon application of foundation loads. Additionally, as discussed in Section 5.2, there is a zone of alluvial soils which contains a relatively high percentage organic deposits which may be subject to settlement. There are no accepted methods to perform an analysis of the rate of decomposition and potential volume loss due to decomposition of the organic soils; therefore it is recommended that proposed structures utilize a design which eliminates permanent reliance on these soils.

8.1.5 Based on these considerations, it is recommended that soil modification be used below proposed structures and site improvements (including swimming pools). Due to the soft clays and organic soils underlying the site, the soil improvement system must consider the potential for bulging or loss of confinement within these soils. Therefore, it is recommended that a grouted Rammed Aggregate Pier (RAP) system be considered and recommendations are provided in Section 8.8. Other methods of ground improvement may be feasible and can be discussed with a specialty design-build contractor.

DRAFT

8.1.6 Subsequent to construction of the RAP, the proposed structures and improvements may be supported on conventional shallow spread foundations deriving support in the improved soils. Consideration should also be given to the use of a reinforced mat foundation for structures with subterranean levels. It is anticipated that a mat foundation system will be allow for more efficient construction when performed in conjunction with subgrade stabilization and waterproofing (if required). All foundations should be designed to derive vertical support from the RAP and may develop lateral resistance at the foundation perimeter, as well as by friction beneath the foundations, if necessary. Since the foundations will not be structurally connected to the RAP, the piers cannot contribute any lateral capacity to the foundation system.

8.1.7 Where supported on ground improvement, it is recommended that the upper 3 feet of existing site soils within the footprint of the proposed parking structure be excavated and properly compacted for foundation and slab support. The excavation should be deepened as-needed to extend to the bottom of proposed foundations and also to completely remove all existing artificial fill. The engineered fill blanket should extend at least 3 feet beyond the edge of foundations, including building appurtenances, or for a distance equal to the depth of fill below the foundations, whichever is greater. Recommendations for earthwork are provided in the *Grading* section of this report (see Section 8.5).

8.1.8 Soft alluvium is anticipated to be exposed throughout the excavation bottoms and these soils will likely be very moist to wet and subject to excessive pumping. Operation of rubber tire equipment on these subgrade soils may cause excessive disturbance of the soils, and equipment may sink and become stuck in the soft soils. Excavation activities to establish the finished subgrade elevation must be conducted carefully and methodically to avoid excessive disturbance to the subgrade. Track-mounted equipment should be considered. Stabilization of the bottom of the excavation may be required in order to provide a firm working surface upon which heavy equipment can operate. Recommendations for bottom stabilization and earthwork are provided in the *Grading* section of this report (see Section 8.5).

8.1.9 The upper alluvial soils as encountered during site exploration were very moist and the grading contractor should be aware that the existing soils are currently near or slightly above optimum moisture content. Conditions could change seasonally. If the soils are more than 3 percent above the optimum moisture content at the time of construction the soils will likely require some spreading and drying activities in order to achieve proper compaction.

8.1.10 Soil additives, like lime or cement, can also be considered to reduce the moisture content, reduce the expansion potential, and stabilize the upper soils. Recommendations for soil stabilization are discussed in the Grading section of this report (see Section 8.5).

DRAFT

8.1.11 Groundwater was encountered at depths of approximately 10 to 20 feet during the field investigation at the subject site. The depth to groundwater at the time of construction may be different. We expect groundwater would be encountered during the installation of rammed aggregate piers.

8.1.12 The historic high groundwater level beneath the site is reported as 10 feet below the existing ground surface. If the subterranean portion of the structure extends below the historic high groundwater level, that portion of the structure should be designed for full hydrostatic pressure.

8.1.13 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. In addition, a waterproofing inspector should be retained to check proper installation of the system during construction.

8.1.14 It is anticipated that stable excavations for the recommended grading associated with the proposed structures can be achieved with sloping measures. However, if excavations in close proximity to an adjacent property line and/or structure are required, special excavation measures may be necessary in order to maintain lateral support of offsite improvements. Excavation recommendations are provided in the *Temporary Excavations* section of this report (Section 8.21).

8.1.15 Improvements which are not supported on deepened foundations, such as walkways, paving, pool decks, and utilities, may still be subject to seismic and/or static settlement. Furthermore, the upper portion of existing site soils have a medium expansive potential and could be subject to heave and settlement if the soil is subjected to repeated wetting and drying. The client should consider the flexibility of the products and pavements being installed. It is recommended that all utilities traversing through existing site soils utilize flexible connections in order to minimize the damage to underground installations caused by potential soil movements.

8.1.16 Foundations for small outlying structures, such as block walls less than 6 feet high, planter walls or trash enclosures, which will not be tied to the proposed structure, may be supported on conventional foundations deriving support on a minimum of 12 inches of newly placed engineered fill which extends laterally at least 12 inches beyond the foundation area. Where excavation and proper compaction cannot be performed or is undesirable, foundations may derive support directly in the undisturbed alluvial soils found at or below a depth of 24 inches

DRAFT

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

8.1.17 Where new paving is to be placed, it is recommended that all existing fill and soft alluvial soils be excavated and properly compacted for paving support. The client should be aware that excavation and compaction of all existing fill and soft soils in the area of new paving is not required; however, paving constructed over existing uncertified fill or unsuitable alluvium may experience increased settlement and/or cracking, and may therefore have a shorter design life and increased maintenance costs. As a minimum, the upper 12 inches of soil should be scarified and properly compacted for paving support. Paving recommendations are provided in *Preliminary Pavement Recommendations* section of this report (see Section 8.14).

8.1.18 Based on the results of percolation testing performed at the site, as well as the relatively shallow groundwater table, a stormwater infiltration system is not recommended for this project. It is recommended that stormwater be retained, filtered, and discharged in accordance with the requirements of the local governing agency.

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

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

8.2 Soil and Excavation Characteristics

8.2.1 The in-situ soils can be excavated with light to moderate effort using conventional excavation equipment. Moderate caving and slumping should be anticipated in unshored excavations, especially where granular or saturated soil is 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.

DRAFT

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

8.2.4 Based on laboratory test results, the near surface soils encountered during the field investigation are considered to have a "very low" to "medium" expansive potential (expansion index of 90 or less) and are classified as "expansive" in accordance with the 2016 California Building Code (CBC) Section 1803.5.3. We expect a majority of the soil encountered will possess a "medium" expansion potential (expansion index of 90 or less) and the recommendations presented herein assume that the building foundations, slabs, and paving will derive support in these materials.

8.3 Minimum Resistivity, pH, and Water-Soluble Sulfate

8.3.1 Potential of Hydrogen (pH) and resistivity testing, as well as chloride content testing, were performed on representative samples of on-site material to generally evaluate the corrosion potential to surface utilities. The tests were performed in accordance with California Test Method Nos. 643 and 422 and indicate that the soils are considered "moderately" to "severely" corrosive with respect to corrosion of buried ferrous metals on site. The results are presented in Appendix B (Figure B21) and should be considered for design of underground structures.

8.3.2 Laboratory tests were performed on representative samples of the on-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 B21) and indicate that the on-site materials possess a "negligible" or "S0" sulfate exposure to concrete structures as defined by 2016 CBC Section 1904 and ACI 318-11 Sections 4.2 and 4.3.

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 observed at depths between 10 and 20 feet below ground surface. The depth to groundwater at the time of construction can be further verified during initial dewatering well or shoring pile installation. If groundwater is present above the proposed excavation bottom, temporary dewatering will be necessary to maintain a safe working environment during excavation and construction activities.

DRAFT

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

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

8.5 Grading

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

8.5.2 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 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.).

8.5.4 It is recommended that proposed structures and site improvements (including swimming pools) been supported on soils improved with Rammed Aggregate Piers (RAP) with rigid inclusions.

DRAFT

8.5.5 Where supported on ground improvement, it is recommended that the upper 3 feet of existing site soils within the footprint of the proposed parking structure be excavated and properly compacted for foundation and slab support. The excavation should be deepened as-needed to extend to the bottom of proposed foundations and also to completely remove all existing artificial fill. The engineered fill blanket should extend at least 3 feet beyond the edge of foundations, including building appurtenances, or for a distance equal to the depth of fill below the foundations, whichever is greater.

8.5.6 Outside of the proposed building footprint areas, where surface improvements such as walkways, paving, and pool decks are planned, the upper 3 feet of finish grade soil should be excavated and blended with cement to create a stabilized engineered fill, or be replaced by select granular import material. Deeper removals should be performed where we encounter deeper artificial fill. Where engineered fill is placed for support of miscellaneous foundations, excavation and placement of engineered fill should extend at least 3 feet beyond the edge of proposed foundations or for a distance equal to the depth of fill below the foundations, whichever is greater.

8.5.7 For preliminary budgeting purposes, the cement content for the required stabilization should be at least approximately 5 percent by dry weight of the combined soil mixture. Laboratory analyses must be performed to confirm that this percentage achieves the desired requirements. The moisture content of the site soil should be evaluated at the time of construction to evaluate if the cement may be blended directly into the soil, or if alternative recommendations for processing the cement into the soil are necessary.

8.5.8 The blending of cement into the onsite soil requires the use of large construction equipment and a large open area to spread and mix the materials. Once construction of the proposed structures has started, the onsite space to perform blending operations will be limited. Therefore, the contractor should consider creating a stockpile of blended cement/soil material during mass grading for future use during smaller grading operations, such as shallow utility trench backfill within the upper 3 feet of finish grade. Trench backfill in excess of the 3 feet deep can use the native soils; however, the contractor should be aware that some drying of the soil may be necessary to achieve proper compaction.

8.5.9 If select import material will be brought onsite for placement and compaction as engineered fill within proposed surface improvement areas, the import material should have a minimum cohesion of 100 pounds per square foot (psf), a minimum friction angle of 30 degrees, an expansion index 30 or less, and corrosivity properties that are equally or less detrimental to that of the existing onsite soil. The imported fill should be observed, tested, and approved by Geocon prior to bringing soil to the site. Rocks larger than 6 inches in diameter should not be used in the fill.

DRAFT

8.5.10 The upper alluvial soils as encountered during site exploration were very moist and the grading contractor should be aware that the existing soils are currently near or slightly above optimum moisture content. Conditions could change seasonally. If the soils are more than 3 percent above the optimum moisture content at the time of construction the soils will likely require some spreading and drying activities in order to achieve proper compaction.

8.5.11 Prior to placing fill, a stable excavation bottom must be established. In areas where the subgrade is saturated or soft, proper compaction will likely not be possible or achieved in a timely manner without introducing stabilization measures. Based on the typical construction schedule and necessity to avoid delays, the implementation of stabilization measures may be warranted. If subgrade stabilization is required at the excavation bottom, rubber tire equipment should not be allowed in the excavation bottom until it is stabilized or extensive soil disturbance could result. It is suggested that excavation and grading be performed during the summer season to promote moisture control of the soils. In addition, the use of track equipment should be considered to minimize disturbance to the soils at the excavation bottom.

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

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

8.5.14 It is anticipated that stable excavations for construction of the proposed subterranean level, as well as for grading and excavation associated with construction of surface improvements, can be achieved with sloping measures. However, shoring may be required where excavations are in close proximity to existing improvements that must be protected in place. Excavation recommendations are provided in the *Temporary Excavations* section of this report (Section 8.21).

8.5.15 Where new paving is to be placed, it is recommended that all existing fill and soft alluvium be excavated and properly compacted for paving support. As a minimum, the upper 12 inches of soil should be scarified, moisture conditioned to optimum moisture content, and compacted to at least 92 percent relative compaction, as determined by ASTM Test Method D 1557 (latest edition). Paving recommendations are provided in *Preliminary Pavement Recommendations* section of this report (see Section 8.14).

DRAFT

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

8.5.17 It is recommended that flexible utility connections be utilized for all rigid utilities to minimize or prevent damage to utilities from minor differential soil movements and subsidence. Utility trenches should be properly backfilled in accordance with the requirements of the Green Book (latest edition). The pipe should be bedded with clean sands (Sand Equivalent greater than 30) to a depth of at least 1 foot over the pipe, and the bedding material must be inspected and approved in writing by the Geotechnical Engineer (a representative of Geocon). The use of gravel is not acceptable unless used in conjunction with filter fabric to prevent the gravel from having direct contact with soil. The remainder of the trench backfill may be derived from onsite soil or approved import soil, compacted as necessary, until the required compaction is obtained. The use of minimum 2-sack slurry is also acceptable as backfill. 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).

8.5.18 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 B21).

8.5.19 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.

DRAFT

8.6 Shrinkage

8.6.1 Shrinkage results when a volume of material removed at one density is compacted to a higher density. A shrinkage factor of between 5 and 15 percent should be anticipated when excavating and compacting the upper 5 feet of existing earth materials on the site to an average relative compaction of 92 percent. This number does not consider the addition of cement should chemical soil stabilization be used onsite.

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

8.7 Foundation Design – General

8.7.1 Subsequent to construction of the RAP, the proposed structures and improvements (including swimming pools) may be supported on conventional shallow spread foundations deriving support in the improved soils. Consideration should also be given to the use of a reinforced mat foundation for structures with subterranean levels. It is anticipated that a mat foundation system will be allow for more efficient construction when performed in conjunction with subgrade stabilization and waterproofing (if required). All foundations should be designed to derive vertical support from the RAP and may develop lateral resistance at the foundation perimeter, as well as by friction beneath the foundations, if necessary. Since the foundations will not be structurally connected to the RAP, the piers cannot contribute any lateral capacity to the foundation system.

8.7.2 Due to the expansive nature of the on-site soils, the moisture content of untreated subgrade soils should be maintained at 2 to 3 percent above optimum moisture content prior to and at the time of concrete placement. If the subgrade is allowed to dry out, presaturation and/or moisture conditioning and recompacting will be required.

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

8.7.4 Waterproofing of subterranean walls and slabs is recommended for this project for any portions of the structure that will be constructed below the groundwater table. 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

DRAFT

and inspection of the waterproofing is not the responsibility of the geotechnical engineer. A waterproofing consultant should be retained in order to recommend a product or method, which would provide protection to subterranean walls, floor slabs and foundations.

8.7.5 It is recommended that a seismic separation or flexible connection be utilized where the adjacent structures abut. The design of the connection is at the discretion of the project structural engineer and should take into account potential differential settlements between structures.

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

8.7.7 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.

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

8.8 Rammed Aggregate Piers (RAP)

8.8.1 It is recommended that proposed structures and site improvements be supported on soils improved with grouted Rammed Aggregate Piers (RAP). Due to the soft clays and organic soils underlying the site, the soil improvement system must consider the potential for bulging or loss of confinement within these soils. It is anticipated that the RAP will extend to the dense, granular layer generally found at depths of 20 to 30 feet below the ground surface.

8.8.2 The RAP system is based on soil improvement that consists of installing densified, aggregate columns to depths typically ranging up to about 25 feet below the proposed foundation elevation. The system increases density and lateral stress in the surrounding soil and claims improvement in bearing capacity and settlement potential. Grouted RAP elements are constructed by creating shafts (commonly 30 inches in diameter) by drilling or displacement methods, and backfilling the open shaft with grout and specially rammed/compacted, open graded crushed rock and Class 2 AB in 10- to 12-inch lifts. It should be noted that creating the shaft using the displacement method, advancing the shaft with a displacement mandrel, reduces the soil cuttings generated during the creation of the shaft.

8.8.3 The pattern and depth of ground improvements may vary depending upon the purposes of mitigation and stratigraphic conditions. The contractor should design the RAP to incorporate allowable static and seismic settlements in accordance with the recommendations of the

DRAFT

DRAFT

project structural engineer. The RAP contractor should evaluate the post-installation static and dynamic settlement within the remediation zone of the RAP. In addition, the project structural engineer should evaluate if the planned structures can tolerate the planned settlements after the installation of the RAP.

8.8.4 Spacing and diameter should be selected by the specialty contractor to obtain the necessary remediation as outlined herein. The RAP mitigation should extend at least 15 feet laterally outside the edge of planned building structures, where practical.

8.8.5 RAP design should be based on settlement criterial of a maximum combined static and seismic differential settlement of 1 inch between adjacent columns. The anticipated seismic induced differential settlement should be evaluated once the depth of the RAP ground improvement is established, as the ground improvement may mitigate some of the potentially liquefiable soil layers.

8.8.6 The RAP design package should be submitted to Geocon West, Inc. for review at least two weeks prior to mobilization for construction. Within the design package, the specialty contractor should outline a performance and load testing program to verify the effectiveness of the ground improvement and to confirm the bearing capacity of the improved soils with a full-scale load test. During the load testing, a representative of Geocon should be present to observe RAP installation and testing. The information obtained from the load testing should be used to modify the depth necessary to achieve design capacities, as well as develop installation criteria that can be used during construction.

8.9 Conventional Foundation Design

8.9.1 The proposed structures may be supported on a conventional spread foundation system deriving support on the RAP ground improvement. All foundation excavations must be observed and approved in writing by the Geotechnical Engineer (a representative of Geocon), prior to placing steel or concrete.

8.9.2 Continuous footings should be a minimum of 12 inches in width, 24 inches in depth below the lowest adjacent grade, and 12 inches into the recommended bearing material. Isolated spread foundations should be a minimum of 24 inches in width, 24 inches in depth below the lowest adjacent grade, and 12 inches into the recommended bearing material. Foundations constructed over RAP ground improvement can achieve relatively high bearing pressures. For preliminary design purposes, a bearing pressure of 7,000 psf may be assumed; however, the design bearing pressure should be provided by the RAP contractor.

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

8.9.4 For preliminary design purposes, a modulus of subgrade reaction of 150 pounds per cubic inch (pci) may be utilized for design of the foundations where directly underlain by improved soil. However, the RAP contractor should provide the structural engineer a revised modulus value incorporating the planned improvement techniques. This value is a unit value for use with a 1-foot square footing. The modulus should be reduced in accordance with the following equation when used with larger foundations:

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

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

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

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

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

8.9.8 Due to the expansive nature of the onsite soils, the moisture content of untreated subgrade soils should be maintained at 2 to 3 percent above optimum moisture content prior to and at the time of concrete placement. If the subgrade is allowed to dry out, presaturation and/or moisture conditioning and recompacting will be required.

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

8.9.10 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.

DRAFT

8.10 Mat Foundation Design

8.10.1 Based on the depth of proposed construction and potential hydrostatic pressures, consideration should be given to the use of a reinforced mat foundation for structures with subterranean levels. It is anticipated that a mat foundation system will be allow for more efficient construction when performed in conjunction with subgrade stabilization and waterproofing (if required). Foundations should derive support on the RAP ground improvement, and must be observed and approved in writing by the Geotechnical Engineer (a representative of Geocon West, Inc.).

8.10.2 It is anticipated that the mat foundation will impart an average pressure of less than 4,000 psf, with locally higher pressures up to 7,000 psf. For preliminary design purposes, a bearing pressure of 7,000 psf may be assumed; however, the design bearing pressure should be provided by the RAP contractor. The allowable bearing pressure may be increased by up to one-third for transient loads due to wind or seismic forces.

8.10.3 For preliminary design purposes, a modulus of subgrade reaction of 150 pci may be utilized for design of the foundations where directly underlain by improved soil. However, the RAP contractor should provide the structural engineer a revised modulus value incorporating the planned improvement techniques. This value is a unit value for use with a 1-foot square footing. The modulus should be reduced in accordance with the following equation when used with larger foundations:

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

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

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

8.10.5 The historic high groundwater level beneath the site is reported as 10 feet below the existing ground surface. If the subterranean portion of the structure extends below the historic high groundwater level, that portion of the structure should 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 pounds per square foot, where “H” is the height of the water above the bottom of the mat foundation in feet.

8.10.6 For seismic design purposes, a coefficient of friction of 0.3 may be utilized between the concrete mat without a moisture barrier; and 0.15 for slabs underlain by a moisture barrier.

DRAFT

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 0.3 may be used with the dead load forces in the competent alluvial soils or in properly compacted engineered fill.

8.11.2 Passive earth pressure for the sides of foundations poured against undisturbed alluvium may be computed as an equivalent fluid having a density of 200 pounds per cubic foot (pcf) with a maximum earth pressure of 2,000 psf. Below the water table, passive pressure may be computed as 110 pcf with a maximum earth pressure of 1,100 pcf (these values have been adjusted for buoyant forces). When combining passive and friction for lateral resistance, the passive component should be reduced by one-third.

8.12 Miscellaneous Foundations

8.12.1 Foundations for small outlying structures, such as block walls up to 6 feet in height, planter walls or trash enclosures, which will not be structurally supported by the proposed building, may be supported on conventional foundations deriving support on a minimum of 12 inches of newly placed engineered fill which extends laterally at least 12 inches beyond the foundation area. Where excavation and compaction cannot be performed, such as adjacent to property lines, foundations may derive support in the undisturbed alluvial soils found at or below a depth of 3 feet, and should be deepened as necessary to maintain a minimum 12-inch embedment into the recommended bearing materials.

8.12.2 If the soils exposed in the excavation bottom are soft, compaction of the soft soils will be required prior to placing steel or concrete. Compaction of the foundation excavation bottom is typically accomplished with a compaction wheel or mechanical whacker and must be observed and approved by a Geocon representative. Miscellaneous foundations may be designed for a bearing value of 1,500 psf, and should be a minimum of 12 inches in width, 24 inches in depth below the lowest adjacent grade and 12 inches into the recommended bearing material. The allowable bearing pressure may be increased by up to one-third for transient loads due to wind or seismic forces.

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

DRAFT

8.13 Concrete Slabs-on-Grade

8.13.1 Unless designed by the project structural engineer, where supported on a conventional foundation system underlain by RAP ground improvement, concrete slabs-on-grade for structures not subject to vehicle loading should be a minimum 5 inches of concrete reinforced with No. 4 steel reinforcing bars placed 16 inches on center in both horizontal directions. Steel reinforcing should be positioned vertically near the slab midpoint. The slab-on-grade may derive support in the newly placed engineered fill.

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

8.13.3 For seismic design purposes, a coefficient of friction of 0.3 may be utilized between concrete slabs and subgrade soils without a moisture barrier, and 0.15 for slabs underlain by a moisture barrier.

8.13.4 Exterior slabs including pool decks, not subject to traffic loads, 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. Prior to construction of slabs, the upper 3 feet of existing site soil should be removed and replaced with either cement stabilized engineered fill or properly compacted select import granular material (expansion index of 30 or less).

DRAFT

8.13.5 Crack control joints should be spaced at intervals not greater than 10 feet and should be constructed using saw-cuts or other methods as soon as practical following concrete placement. Crack control joints should extend a minimum depth of one-fourth the slab thickness. The project structural engineer should design construction joints as necessary.

8.12.6 Due to the expansive potential of the anticipated subgrade soils, the moisture content of the slab subgrade should be maintained and sprinkled as necessary to maintain a moist condition as would be expected in any concrete placement. Furthermore, consideration should be given to doweling slabs into adjacent curbs and foundations to minimize movements and offsets which could lead to a potential tripping hazard. As an alternative, the upper 18 inches of soil could be replaced with granular, non-expansive soils which will reduce the potential for movements and offsets.

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

8.14 Preliminary Pavement Recommendations

8.14.1 Prior to construction of pavement, the upper 3 feet of existing site soil should be removed and replaced with either cement stabilized engineered fill or properly compacted select import granular material (expansion index of 50 or less).

8.14.2 The following pavement sections are based on an R-Value of 20 (cement stabilized engineered fill or properly compacted select import material). 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.

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

DRAFT

PRELIMINARY PAVEMENT DESIGN SECTIONS

Location	Estimated Traffic Index (TI)	Asphalt Concrete (inches)	Class 2 Aggregate Base (inches)
Automobile Parking And Driveways	5.0	3.0	4.0
Trash Truck & Fire Lanes	7.0	4.0	12.0

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

8.14.5 Unless specifically designed and evaluated by the project structural engineer, where concrete paving will be utilized for support of vehicles, we recommend that the concrete be a minimum of 6 inches thick and 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 at least 92 percent and 95 percent relative compaction, respectively, as determined by ASTM Test Method D 1557 (latest edition).

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

8.15 Retaining Walls Design

8.15.1 The recommendations presented below are generally applicable to the design of rigid concrete or masonry retaining walls having a maximum height of 12 feet. In the event that walls significantly higher than 12 feet are planned, Geocon should be contacted for additional recommendations.

8.15.2 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) of 40 pcf.

DRAFT

8.15.3 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) of 60 pcf.

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

8.14.5 The soil pressures above assume that the backfill material within an area bounded by the wall and a 1:1 plane extending upward from the base of the wall will be comprised of engineered fill derived from the onsite, expansive soils. If select import soil will be used to backfill proposed retaining walls, reduced earth pressures may be feasible based on the geotechnical properties of the import soil used as engineered fill. This should be evaluated once the use of import soil is established. All imported fill shall be observed, tested, and approved by Geocon West, Inc. prior to bringing soil to the site.

8.15.5 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. Recommendations for the incorporation of surcharges are provided in section 8.22 of this report.

8.15.6 In addition to the recommended earth pressure, the upper 10 feet of the subterranean 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 subterranean walls, the traffic surcharge may be neglected.

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

8.16 Dynamic (Seismic) Lateral Forces

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

8.16.2 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

DRAFT

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.

8.17 Retaining Wall Drainage

8.17.1 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.

8.17.2 As an alternative, a plastic drainage composite such as Miradrain or equivalent may be installed in continuous, 4-foot wide columns along the entire back face of the wall, at 8 feet on center. The top of these drainage composite columns should terminate approximately 18 inches below the ground surface, where either hardscape or a minimum of 18 inches of relatively cohesive material should be placed as a cap (see Figure 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.

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

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

DRAFT

8.18 Swimming Pool

8.18.1 The proposed swimming pools should be designed as free-standing structures deriving support in the improved soils. Swimming pool foundations and walls may be designed in accordance with the *Foundation Design* and *Retaining Wall Design* sections of this report (See Sections 8.7 through 8.10 and 8.15). The proposed pools should be constructed utilizing an expansive soils design and a hydrostatic relief valve should be considered as part of the swimming pool design unless a gravity drain system can be placed beneath the pool shell.

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

8.19 Elevator Pit Design

8.19.1 The elevator pit slab and retaining wall should be designed by the project structural engineer. As a minimum the slab-on-grade 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. The elevator pit should be structurally supported either indirectly or directly by the ground improvement system. Elevator pit walls may be designed in accordance with the recommendations in the *Retaining Wall Design* section of this report (see Section 8.15).

8.19.2 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.

8.19.3 If retaining wall drainage is to be provided, the drainage system should be designed in accordance with the *Retaining Wall Drainage* section of this report (see Section 8.17).

8.19.4 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.

8.20 Elevator Piston

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

8.20.2 Casing may be required if caving is encountered in the drilled excavation. 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.

8.20.3 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.

8.21 Temporary Excavations

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

DRAFT

8.21.2 Vertical excavations greater than 5 feet or where surcharged by existing structures will require sloping or shoring measures in order to provide a stable excavation. Where sufficient space is available, temporary unsurcharged embankments which do not extend below the water table may be sloped back at a uniform 1:1 slope gradient or flatter up to maximum height of 15 feet. A uniform slope does not have a vertical portion.

8.21.3 If excavations in close proximity to an adjacent property line and/or structure are required, special excavation measures such as slot-cutting or shoring may be necessary in order to maintain lateral support of offsite improvements. Recommendations for special temporary excavation measures can be provided under separate cover, as needed.

8.21.4 Where temporary slopes are utilized, the top of the slope should be barricaded to prevent vehicles and storage loads at the top of the slope within a horizontal distance equal to the height of the slope. If the temporary construction slopes are to be maintained during the rainy season, berms are suggested along the tops of the slopes where necessary to prevent runoff water from entering the excavation and eroding the slope faces. The soils exposed in the slopes should be inspected during excavation by our personnel 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.

8.22 Surcharge from Adjacent Structures and Improvements

8.22.1 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.

DRAFT

8.22.2 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.

8.22.3 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.

DRAFT

8.23 Surface Drainage

8.23.1 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.

8.23.2 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.

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

8.23.4 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.

8.24 Plan Review

8.24.1 Grading, foundation, and shoring plans should be reviewed by the Geotechnical Engineer prior to finalization to check that the plans have been prepared in substantial conformance with the recommendations of this report and to provide additional analyses or recommendations, if necessary.

DRAFT

LIMITATIONS AND UNIFORMITY OF CONDITIONS

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

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

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

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

DRAFT

LIST OF REFERENCES

Bray, J. D., and Sancio, R. B., 2006, *Assessment of the Liquefaction Susceptibility of Fine-Grained Soils,* American Society of Civil Engineers, Journal of the Geotechnical and Geoenvironmental Engineering, Vol. 132, No. 9, pp. 1165-1177.

California Department of Water Resources, 1967, *Groundwater Geology of the Coastal Plain of Orange County.*

California Division of Mines and Geology, 1976, *Environmental Geology of Orange County, California,* Open File Report 79-8 LA.

California Division of Mines and Geology, 1998, *Seismic Hazard Zone, Newport Beach Quadrangle,* April 15, 1998.

California Division of Mines and Geology, 2001, *Seismic Hazard Zone Report for the Anaheim and Newport Beach 7.5-minute Quadrangle, Los Angeles and Orange Counties, California, Seismic Hazard Zone Report 003*, 1997 (Revised 2001).

California Division of Oil, Gas and Geothermal Resources, 2018, Division of Oil, Gas, and Geothermal Resources Well Finder, http://maps.conservation.ca.gov.doggr/index Accessed March 12, 2018.

California Geological Survey, 2008, *Guidelines for Evaluating and Mitigating Seismic Hazards in California*, Special Publication 117.

California Geological Survey, 2012, *Geologic Compilation of Quaternary Surficial Deposits in Southern California, Santa Ana 30' X 60' Quadrangle*, A Project for the Department of Water Resources by the California Geological Survey, Compiled from existing sources by Trinda L. Bedrossian, CEG and Peter D. Roffers, CGS Special Report 217, Plate 16, Scale 1:100,000.

California Geological Survey, 2018a, *Earthquake Fault Zones, A Guide for Government Agencies, Property Owners/Developers, and Geoscience Practitioners for Assessing Fault Rupture Hazards in California,* Special Publication 42, Revised 2018.

California Geological Survey, 2018b, CGS Information Warehouse, Regulatory Map Portal, http://maps.conservation.ca.gov/cgs/informationwarehouse/index.html?map=regulatorymaps.

Costa Mesa, City of, 2015, *Safety Element of the General Plan.*

FEMA, 2018, Online Flood Hazard Maps, http://www.esri.com/hazards/index.html.

Grant, L, et al., 1999, *Late Quaternary Uplift and Earthquake Potential of the San Joaquin Hills, Southern Los Angeles Basin, California*, Geology, Vol. 27, No. 11, November 1999.

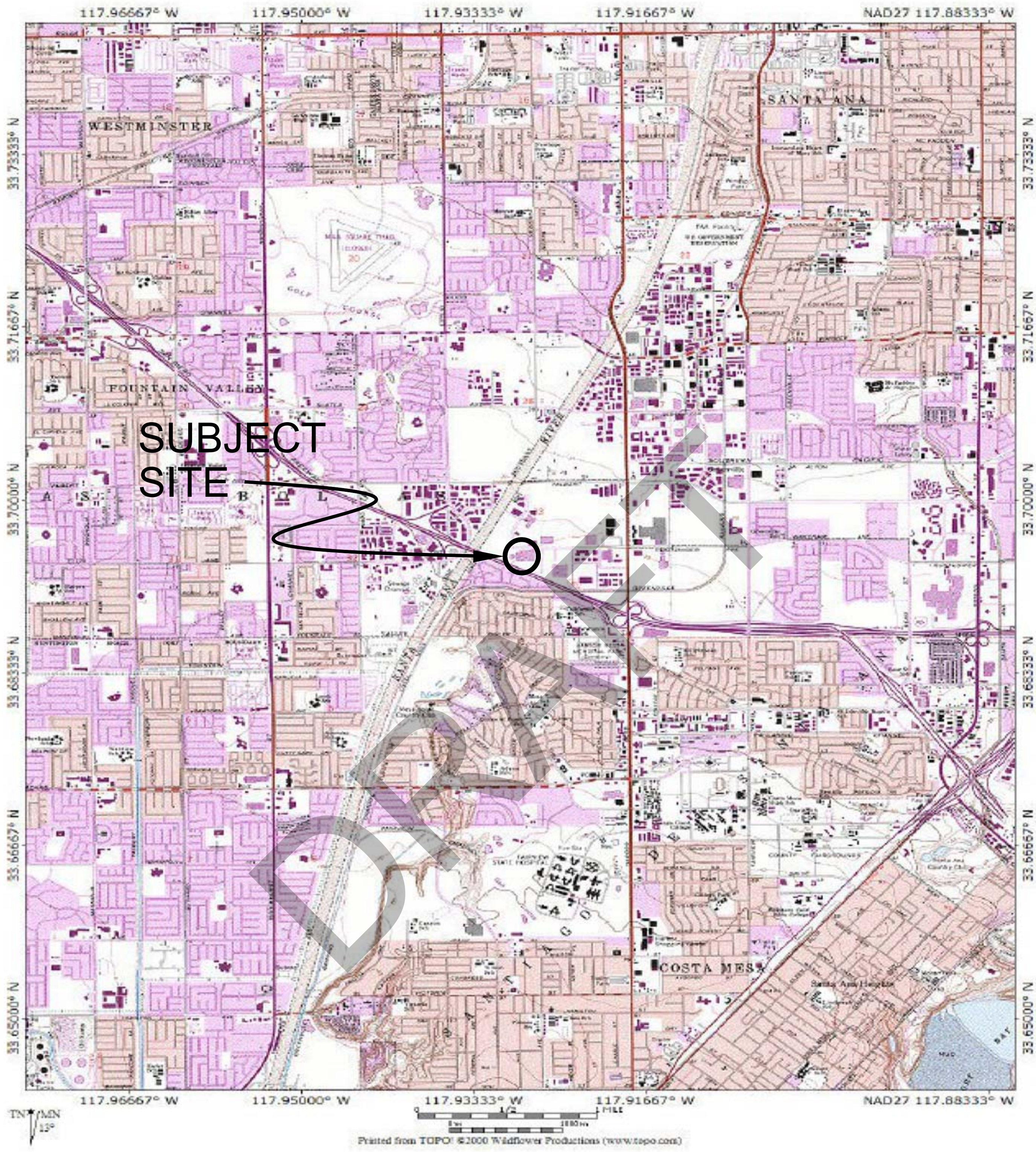
Jennings, C. W. and Bryant, W. A., 2010, *Fault Activity Map of California,* California Geological Survey Geologic Data Map No. 6.

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

Youd, L. T. and Idriss, I. M., editors, 1997, "Proceedings of NCEER Workshop on Evaluation of Liquefaction Resistance of Soils", Technical Report NCEER-97-0022, December 31, 1997.

Ziony, J. I., and Jones, L. M., 1989, *Map Showing Late Quaternary Faults and 1978–1984 Seismicity of the Los Angeles Region, California*, U.S. Geological Survey Miscellaneous Field Studies Map MF-1964.

DRAFT



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

GEOCON WEST, INC.

ENVIRONMENTAL GEOTECHNICAL MATERIALS

15520 ROCKFIELD BOULEVARD, SUITE J, IRVINE, CA 92618
PHONE (949) 491-6570

DRAFTED BY: JS	CHECKED BY: JTA

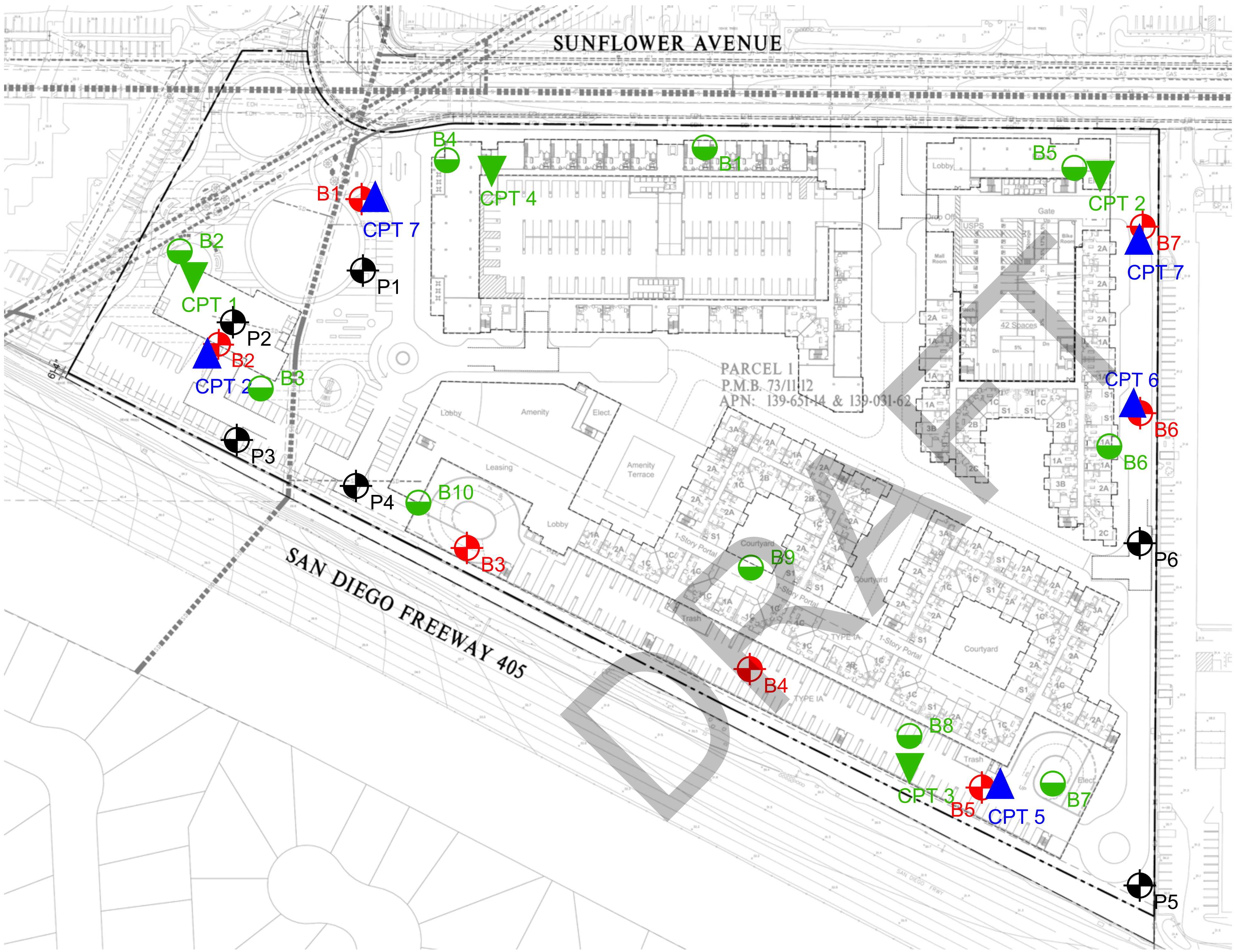
VICINITY MAP

ROSE EQUITIES

1683 SUNFLOWER AVENUE

COSTA MESA, CALIFORNIA

JULY 2019	PROJECT NO. A9933-88-01	FIG. 1



LEGEND

B7 — Location of Boring (Geocon, 2019)

P6 — Location of Percolation Boring (Geocon, 2019)

CPT 7 — Location of CPT (Geocon, 2019)

B10 — Location of Boring (MTA, 2013)

CPT 4 — Location of CPT (MTA, 2013)

0 125' 250'

GEOCON
WEST, INC.

ENVIRONMENTAL GEOTECHNICAL MATERIALS
15520 ROCKFIELD BOULEVARD, SUITE J, IRVINE, CA 92618
PHONE (949) 491-6570 FAX (949) 299-4550

DRAFTED BY: JS	CHECKED BY: JTA

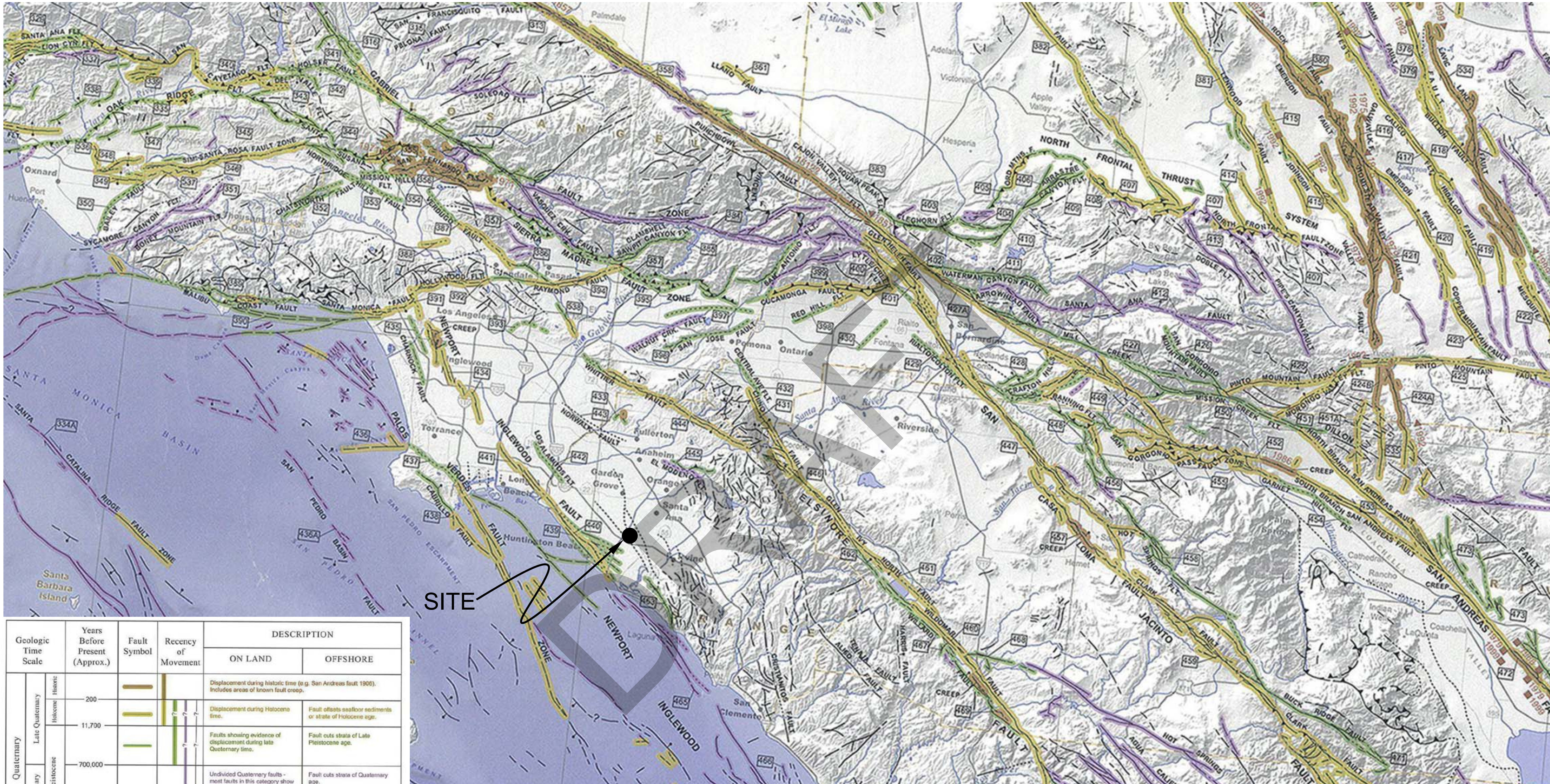
SITE PLAN

ROSE EQUITIES

1683 SUNFLOWER AVENUE
COSTA MESA, CALIFORNIA

JULY 2019	PROJECT NO. A9933-88-01	FIG. 2

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



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

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

0 12 24 Miles

GEOCON WEST, INC.

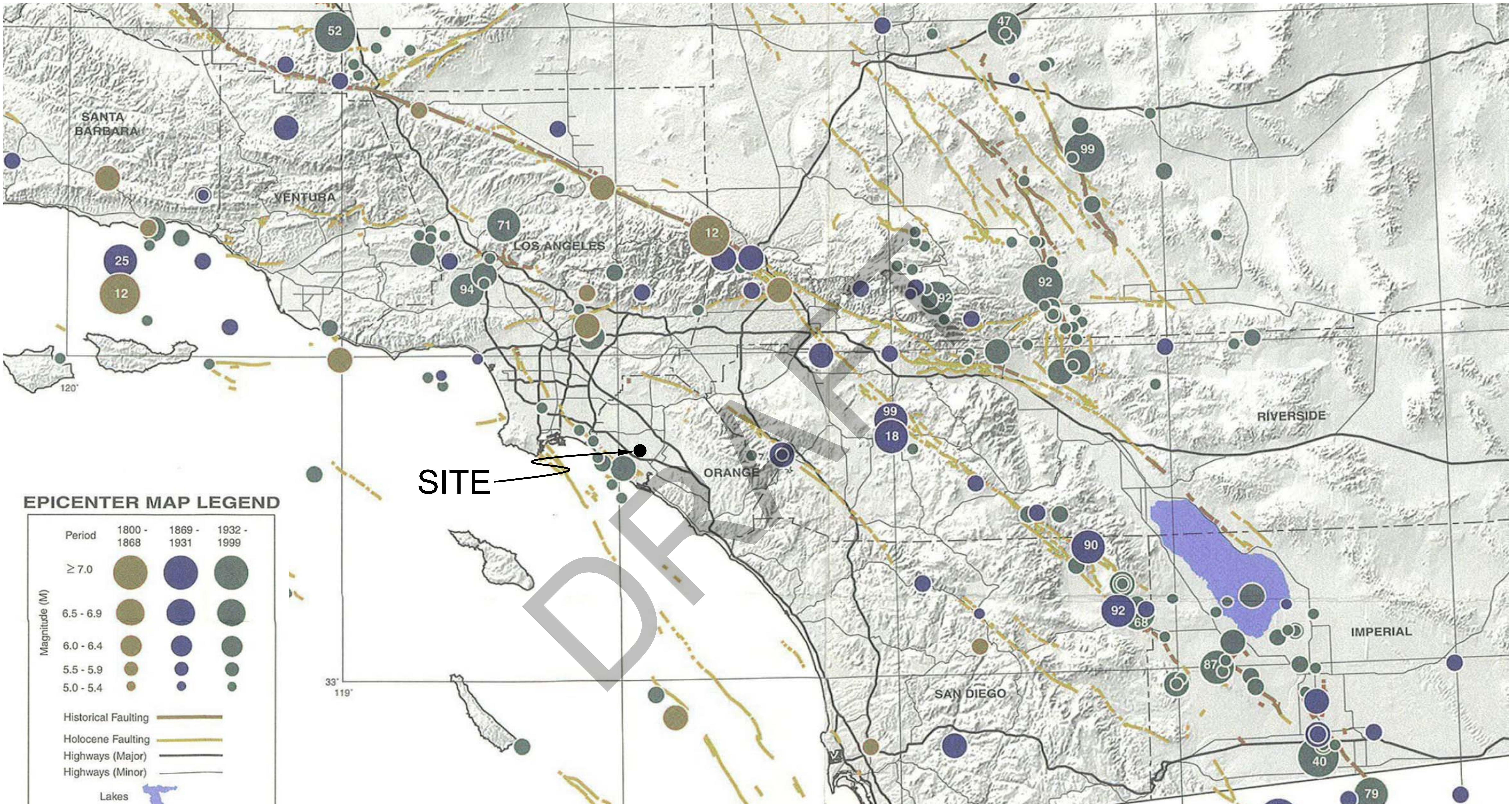
ENVIRONMENTAL GEOTECHNICAL MATERIALS
15520 ROCKFIELD BOULEVARD, SUITE J, IRVINE, CA 92618
PHONE (949) 491-6570

DRAFTED BY: JA | CHECKED BY: JTA

REGIONAL FAULT MAP

ROSE EQUITIES
1683 SUNFLOWER AVENUE
COSTA MESA, CALIFORNIA

JULY 2019	PROJECT NO. A9933-88-01	FIG. 3



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

0 20 40 Miles

GEOCON
W E S T, I N C.

ENVIRONMENTAL GEOTECHNICAL MATERIALS
15520 ROCKFIELD BOULEVARD, SUITE J, IRVINE, CA 92618
PHONE (949) 491-6570

DRAFTED BY: JA	CHECKED BY: JTA

REGIONAL SEISMICITY MAP

ROSE EQUITIES
1683 SUNFLOWER AVENUE
COSTA MESA, CALIFORNIA

JULY 2019	PROJECT NO. A9933-88-01	FIG. 4

GEOCON

Client : Rose Equities
File No. : A9933-88-01
Boring : 3

EMPIRICAL ESTIMATION OF LIQUEFACTION POTENTIAL
DESIGN EARTHQUAKE

NCEER (1996) METHOD

By Thomas F. Blake (1994-1996)

EARTHQUAKE INFORMATION:

Earthquake Magnitude:	6.67
Peak Horiz. Acceleration PGA_M (g):	0.601
2/3 PGA_M (g):	0.401
Calculated Mag.Wtg.Factor:	0.744
Historic High Groundwater:	10.0
Groundwater Depth During Exploration:	21.0

ENERGY & ROD CORRECTIONS:

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

LIQUEFACTION CALCULATIONS:

Unit Wt. Water (pcf):	62.4

Depth to Base (ft)	Total Unit Wt. (pcf)	Water (0 or 1)	FIELD SPT (N)	Depth of SPT (ft)	Liq.Sus. (0 or 1)	-200 (%)	Est. Dr (%)	CN Factor	Corrected (N1)60	Eff. Unit Wt. (psf)	Resist. CRR	rd Factor	Induced CSR	Liquefac. Safe.Fact.
1.0	120.0	0	18.0	1.0	1		103	1.700	34.4	120.0	Infin.	0.998	0.193	--
2.5	120.0	0	18.0	2.0	1		99	1.700	34.4	120.0	Infin.	0.992	0.192	--
3.0	120.0	0	18.0	3.0	1		98	1.700	34.4	120.0	Infin.	0.987	0.191	--
4.0	120.0	0	18.0	4.0	1		95	1.700	34.4	120.0	Infin.	0.984	0.191	--
5.0	120.0	0	4.0	5.0	1		44	1.700	7.7	120.0	0.086	0.979	0.190	--
6.0	120.0	0	4.0	6.0	1		43	1.700	7.7	120.0	0.086	0.975	0.189	--
7.0	120.0	0	5.0	7.0	0			1.636	9.2	120.0	~	0.970	0.188	~
8.0	120.0	0	5.0	8.0	0			1.523	8.6	120.0	~	0.966	0.187	~
9.0	120.0	0	5.0	9.0	0			1.431	8.0	120.0	~	0.961	0.186	~
10.0	120.0	0	5.0	10.0	0			1.353	7.6	120.0	~	0.957	0.186	~
11.0	120.0	1	5.0	10.0	0			1.287	7.2	57.6	~	0.952	0.189	~
12.0	120.0	1	8.0	12.5	0			1.230	11.1	57.6	~	0.947	0.197	~
13.0	120.0	1	8.0	12.5	0			1.180	10.6	57.6	~	0.943	0.204	~
14.0	120.0	1	8.0	12.5	0			1.135	10.2	57.6	~	0.938	0.210	~
15.0	120.0	1	8.0	12.5	0			1.095	9.9	57.6	~	0.934	0.216	~
16.5	120.0	1	8.0	12.5	0			1.051	9.5	57.6	~	0.928	0.222	~
17.0	120.0	1	11.0	17.5	0			1.035	14.6	57.6	~	0.923	0.224	~
18.0	120.0	1	11.0	17.5	0			0.997	14.1	57.6	~	0.920	0.230	~
19.0	120.0	1	11.0	17.5	0			0.970	13.7	57.6	~	0.915	0.233	~
20.0	120.0	1	7.0	20.0	0			0.945	8.9	57.6	~	0.911	0.237	~
20.5	120.0	1	7.0	20.0	0			0.927	8.7	57.6	~	0.907	0.239	~
22.0	120.0	1	13.0	22.5	1	50	63	0.903	23.3	57.6	0.260	0.903	0.243	1.07
23.0	120.0	1	13.0	22.5	1	50	63	0.895	23.2	57.6	0.258	0.897	0.245	1.05
24.0	120.0	1	13.0	22.5	1	50	63	0.885	23.0	57.6	0.255	0.893	0.247	1.04
25.0	120.0	1	13.0	22.5	1	50	63	0.876	22.8	57.6	0.253	0.888	0.249	1.02
26.0	120.0	1	13.0	22.5	1	50	63	0.867	22.7	57.6	0.251	0.883	0.250	1.00
27.0	120.0	1	25.0	27.5	1	4	85	0.858	31.5	57.6	Infin.	0.879	0.252	Non-Liq.
28.0	120.0	1	25.0	27.5	1	4	85	0.849	31.2	57.6	Infin.	0.874	0.253	Non-Liq.
29.0	120.0	1	25.0	27.5	1	4	85	0.841	30.9	57.6	Infin.	0.870	0.255	Non-Liq.
30.0	120.0	1	25.0	27.5	1	4	85	0.833	30.6	57.6	Infin.	0.865	0.256	Non-Liq.
31.0	120.0	1	25.0	27.5	1	4	85	0.825	30.3	57.6	Infin.	0.861	0.257	Non-Liq.
32.0	120.0	1	25.0	32.5	1	4	82	0.817	30.7	57.6	Infin.	0.856	0.257	Non-Liq.
33.0	120.0	1	25.0	32.5	1	4	82	0.810	30.4	57.6	Infin.	0.851	0.258	Non-Liq.
34.0	120.0	1	25.0	32.5	1	4	82	0.803	30.1	57.6	Infin.	0.847	0.259	Non-Liq.
35.0	120.0	1	25.0	32.5	1	10	82	0.796	30.9	57.6	Infin.	0.842	0.259	Non-Liq.
36.0	120.0	1	25.0	32.5	1	10	82	0.789	30.7	57.6	Infin.	0.838	0.259	Non-Liq.
37.0	120.0	1	50.0	37.5	1	10	112	0.782	59.8	57.6	Infin.	0.833	0.260	Non-Liq.
38.0	120.0	1	50.0	37.5	1	10	112	0.776	59.3	57.6	Infin.	0.829	0.260	Non-Liq.
39.0	120.0	1	50.0	37.5	1	10	112	0.769	58.8	57.6	Infin.	0.824	0.260	Non-Liq.
40.0	120.0	1	50.0	37.5	1	10	112	0.763	58.3	57.6	Infin.	0.819	0.260	Non-Liq.
41.0	120.0	1	12.0	42.5	0			0.757	13.6	57.6	~	0.815	0.260	~
42.0	120.0	1	12.0	42.5	0			0.751	13.5	57.6	~	0.810	0.260	~
43.0	120.0	1	12.0	42.5	0			0.745	13.4	57.6	~	0.806	0.259	~
44.0	120.0	1	12.0	42.5	0			0.740	13.3	57.6	~	0.801	0.259	~
45.0	120.0	1	12.0	42.5	0	0		0.734	13.2	57.6	~	0.797	0.259	~
46.5	120.0	1	12.0	42.5	0	0		0.727	13.1	57.6	~	0.791	0.258	~
47.0	120.0	1	25.0	47.5	1	54	75	0.725	34.2	57.6	Infin.	0.786	0.257	Non-Liq.
48.0	120.0	1	25.0	47.5	1	54	75	0.718	33.9	57.6	Infin.	0.783	0.258	Non-Liq.
49.0	120.0	1	25.0	47.5	1	54	75	0.713	33.7	57.6	Infin.	0.778	0.257	Non-Liq.
50.0	120.0	1	25.0	47.5	1	54	75	0.708	33.6	57.6	Infin.	0.774	0.256	Non-Liq.

DRAFT

Client : Rose Equities
File No. : A9933-88-01
Boring : 3

LIQUEFACTION SETTLEMENT ANALYSIS
DESIGN EARTHQUAKE

(SATURATED SAND AT INITIAL LIQUEFACTION CONDITION)

NCEER (1996) METHOD
EARTHQUAKE INFORMATION:

Earthquake Magnitude:	6.67
PGAM (g):	0.601
2/3 PGAM (g):	0.40
Calculated Mag.Wtg.Factor:	0.744
Historic High Groundwater:	10.0
Groundwater @ Exploration:	21.0

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	18	120	0.030	0.030	103	34	0.261	--	0.00	0.00
2.5	18	120	0.105	0.105	99	34	0.261	--	0.00	0.00
3	18	120	0.135	0.135	98	34	0.261	--	0.00	0.00
4	18	120	0.210	0.210	95	34	0.261	--	0.00	0.00
5	4	120	0.270	0.270	44	8	0.261	--	0.00	0.00
6	4	120	0.330	0.330	43	8	0.261	--	0.00	0.00
7	5	120	0.390	0.390		9	0.261	~	0.00	0.00
8	5	120	0.450	0.450		9	0.261	~	0.00	0.00
9	5	120	0.510	0.510		8	0.261	~	0.00	0.00
10	5	120	0.570	0.570		8	0.261	~	0.00	0.00
11	5	120	0.630	0.614		7	0.267	~	0.00	0.00
12	8	120	0.690	0.643		11	0.280	~	0.00	0.00
13	8	120	0.750	0.672		11	0.291	~	0.00	0.00
14	8	120	0.810	0.701		10	0.301	~	0.00	0.00
15	8	120	0.870	0.730		10	0.311	~	0.00	0.00
16.5	8	120	0.945	0.766		9	0.322	~	0.00	0.00
17	11	120	0.975	0.780		15	0.326	~	0.00	0.00
18	11	120	1.050	0.816		14	0.335	~	0.00	0.00
19	11	120	1.110	0.845		14	0.342	~	0.00	0.00
20	7	120	1.170	0.874		9	0.349	~	0.00	0.00
20.5	7	120	1.215	0.895		9	0.354	~	0.00	0.00
22	13	120	1.305	0.938	63	23	0.362	1.07	1.20	0.22
23	13	120	1.350	0.960	63	23	0.366	1.05	1.20	0.14
24	13	120	1.410	0.989	63	23	0.372	1.04	1.20	0.14
25	13	120	1.470	1.018	63	23	0.376	1.02	1.30	0.16
26	13	120	1.530	1.046	63	23	0.381	1.00	1.30	0.16
27	25	120	1.590	1.075	85	32	0.385	Non-Liq.	0.00	0.00
28	25	120	1.650	1.104	85	31	0.389	Non-Liq.	0.00	0.00
29	25	120	1.710	1.133	85	31	0.393	Non-Liq.	0.00	0.00
30	25	120	1.770	1.162	85	31	0.397	Non-Liq.	0.00	0.00
31	25	120	1.830	1.190	85	30	0.401	Non-Liq.	0.00	0.00
32	25	120	1.890	1.219	82	31	0.404	Non-Liq.	0.00	0.00
33	25	120	1.950	1.248	82	30	0.407	Non-Liq.	0.00	0.00
34	25	120	2.010	1.277	82	30	0.410	Non-Liq.	0.00	0.00
35	25	120	2.070	1.306	82	31	0.413	Non-Liq.	0.00	0.00
36	25	120	2.130	1.334	82	31	0.416	Non-Liq.	0.00	0.00
37	50	120	2.190	1.363	112	60	0.419	Non-Liq.	0.00	0.00
38	50	120	2.250	1.392	112	59	0.421	Non-Liq.	0.00	0.00
39	50	120	2.310	1.421	112	59	0.424	Non-Liq.	0.00	0.00
40	50	120	2.370	1.450	112	58	0.426	Non-Liq.	0.00	0.00
41	12	120	2.430	1.478		14	0.428	~	0.00	0.00
42	12	120	2.490	1.507		14	0.430	~	0.00	0.00
43	12	120	2.550	1.536		13	0.433	~	0.00	0.00
44	12	120	2.610	1.565		13	0.435	~	0.00	0.00
45	12	120	2.670	1.594		13	0.437	~	0.00	0.00
46.5	12	120	2.745	1.630		13	0.439	~	0.00	0.00
47	25	120	2.775	1.644	75	34	0.440	Non-Liq.	0.00	0.00
48	25	120	2.850	1.680	75	34	0.442	Non-Liq.	0.00	0.00
49	25	120	2.910	1.709	75	34	0.444	Non-Liq.	0.00	0.00
50	25	120	2.970	1.738	75	34	0.445	Non-Liq.	0.00	0.00

TOTAL SETTLEMENT = 0.8 INCHES

DRAFT

GEOCON

Client : Rose Equities
File No. : A9933-88-01
Boring : 7

EMPIRICAL ESTIMATION OF LIQUEFACTION POTENTIAL
DESIGN EARTHQUAKE

NCEER (1996) METHOD

By Thomas F. Blake (1994-1996)

EARTHQUAKE INFORMATION:

Earthquake Magnitude:	6.67
Peak Horiz. Acceleration PGA_M (g):	0.601
2/3 PGA_M (g):	0.401
Calculated Mag.Wtg.Factor:	0.744
Historic High Groundwater:	10.0
Groundwater Depth During Exploration:	17.5

ENERGY & ROD CORRECTIONS:

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

LIQUEFACTION CALCULATIONS:

Unit Wt. Water (pcf): 62.4

Depth to Base (ft)	Total Unit Wt. (pcf)	Water (0 or 1)	FIELD SPT (N)	Depth of SPT (ft)	Liq.Sus. (0 or 1)	-200 (%)	Est. Dr (%)	CN Factor	Corrected (N1)60	Eff. Unit Wt. (psf)	Resist. CRR	rd Factor	Induced CSR	Liquefac. Safe.Fact.
1.0	120.0	0	6.0	1.0	1		59	1.700	11.5	120.0	0.125	0.998	0.193	--
2.0	120.0	0	6.0	2.0	1		58	1.700	11.5	120.0	0.125	0.993	0.193	--
3.0	120.0	0	6.0	3.0	1		56	1.700	11.5	120.0	0.125	0.989	0.192	--
4.0	120.0	0	6.0	4.0	1		55	1.700	11.5	120.0	0.125	0.984	0.191	--
5.5	120.0	0	6.0	5.0	1		53	1.700	11.5	120.0	0.125	0.978	0.190	--
6.0	120.0	0	8.0	6.0	1		61	1.700	15.3	120.0	0.167	0.974	0.189	--
7.0	120.0	0	8.0	7.0	1		59	1.636	14.7	120.0	0.161	0.970	0.188	--
8.0	120.0	0	8.0	8.0	1		58	1.523	13.7	120.0	0.150	0.966	0.187	--
9.0	120.0	0	8.0	9.0	1		57	1.431	12.9	120.0	0.141	0.961	0.186	--
10.0	120.0	0	8.0	10.0	1	50	55	1.353	19.2	120.0	0.209	0.957	0.186	--
11.0	120.0	1	8.0	10.0	1	50	55	1.287	18.6	57.6	0.202	0.952	0.189	1.07
12.0	120.0	1	8.0	10.0	1	50	55	1.230	18.1	57.6	0.196	0.947	0.197	1.00
13.0	120.0	1	2.0	12.5	0			1.180	2.7	57.6	~	0.943	0.204	~
14.0	120.0	1	2.0	12.5	0			1.135	2.6	57.6	~	0.938	0.210	~
15.0	120.0	1	2.0	17.5	0			1.095	2.8	57.6	~	0.934	0.216	~
16.0	120.0	1	2.0	17.5	0			1.060	2.7	57.6	~	0.929	0.221	~
17.0	120.0	1	2.0	17.5	0			1.027	2.6	57.6	~	0.925	0.225	~
18.0	120.0	1	2.0	17.5	0			1.005	2.6	57.6	~	0.920	0.230	~
19.0	120.0	1	2.0	17.5	0			0.991	2.5	57.6	~	0.915	0.233	~
20.0	120.0	1	6.0	20.0	0			0.978	7.9	57.6	~	0.911	0.237	~
21.0	120.0	1	6.0	20.0	0			0.965	7.8	57.6	~	0.906	0.240	~
22.0	120.0	1	6.0	20.0	0			0.953	7.7	57.6	~	0.902	0.242	~
23.0	120.0	1	4.0	22.5	0			0.941	5.2	57.6	~	0.897	0.245	~
24.0	120.0	1	4.0	22.5	1	56	35	0.930	12.2	57.6	0.133	0.893	0.247	0.54
25.0	120.0	1	9.0	22.5	1	54	53	0.919	18.5	57.6	0.201	0.888	0.249	0.81
26.0	120.0	1	9.0	22.5	1	54	53	0.909	18.4	57.6	0.200	0.883	0.250	0.80
27.0	120.0	1	9.0	22.5	1	54	53	0.898	18.3	57.6	0.198	0.879	0.252	0.79
28.0	120.0	1	27.0	27.5	1		88	0.889	35.2	57.6	Infin.	0.874	0.253	Non-Liq.
29.0	120.0	1	44.0	32.5	1		109	0.879	58.0	57.6	Infin.	0.870	0.255	Non-Liq.
30.0	120.0	1	44.0	32.5	1		109	0.870	57.4	57.6	Infin.	0.865	0.256	Non-Liq.
31.0	120.0	1	44.0	32.5	1		109	0.861	56.8	57.6	Infin.	0.861	0.257	Non-Liq.
32.0	120.0	1	44.0	32.5	1		109	0.852	56.2	57.6	Infin.	0.856	0.257	Non-Liq.
33.0	120.0	1	44.0	32.5	1		109	0.844	55.7	57.6	Infin.	0.851	0.258	Non-Liq.
34.0	120.0	1	44.0	32.5	1		109	0.836	55.1	57.6	Infin.	0.847	0.259	Non-Liq.
35.0	120.0	1	49.0	37.5	1		111	0.828	60.8	57.6	Infin.	0.842	0.259	Non-Liq.
36.0	120.0	1	49.0	37.5	1		111	0.820	60.3	57.6	Infin.	0.838	0.259	Non-Liq.
37.0	120.0	1	49.0	37.5	1		111	0.812	59.7	57.6	Infin.	0.833	0.260	Non-Liq.
38.0	120.0	1	49.0	37.5	1		111	0.805	59.2	57.6	Infin.	0.829	0.260	Non-Liq.
39.0	120.0	1	49.0	37.5	1		111	0.798	58.7	57.6	Infin.	0.824	0.260	Non-Liq.
40.0	120.0	1	49.0	37.5	1		111	0.791	58.1	57.6	Infin.	0.819	0.260	Non-Liq.
41.0	120.0	1	49.0	37.5	1		111	0.784	57.7	57.6	Infin.	0.815	0.260	Non-Liq.
42.0	120.0	1	29.0	42.5	1		84	0.778	33.8	57.6	Infin.	0.810	0.260	Non-Liq.
43.5	120.0	1	29.0	42.5	1		84	0.770	33.5	57.6	Infin.	0.805	0.259	Non-Liq.
44.0	120.0	1	27.0	45.0	1		79	0.767	31.1	57.6	Infin.	0.800	0.258	Non-Liq.
45.5	120.0	1	27.0	45.0	1		79	0.758	30.7	57.6	Infin.	0.795	0.259	Non-Liq.
46.0	120.0	1	12.0	47.5	0			0.755	13.6	57.6	~	0.791	0.258	~
47.0	120.0	1	12.0	47.5	0			0.747	13.5	57.6	~	0.787	0.258	~
48.0	120.0	1	12.0	47.5	0			0.742	13.3	57.6	~	0.783	0.258	~
49.0	120.0	1	12.0	47.5	0			0.736	13.2	57.6	~	0.778	0.257	~
50.0	120.0	1	12.0	47.5	0			0.731	13.2	57.6	~	0.774	0.256	~

DRAFT

Client : Rose Equities
File No. : A9933-88-01
Boring : 7

GEOCON

LIQUEFACTION SETTLEMENT ANALYSIS
DESIGN EARTHQUAKE

(SATURATED SAND AT INITIAL LIQUEFACTION CONDITION)

NCEER (1996) METHOD
EARTHQUAKE INFORMATION:

Earthquake Magnitude:	6.67
PGAM (g):	0.601
2/3 PGAM (g):	0.40
Calculated Mag.Wtg.Factor:	0.744
Historic High Groundwater:	10.0
Groundwater @ Exploration:	17.5

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	6	120	0.030	0.030	59	11	0.261	--	0.00	0.00
2	6	120	0.090	0.090	58	11	0.261	--	0.00	0.00
3	6	120	0.150	0.150	56	11	0.261	--	0.00	0.00
4	6	120	0.210	0.210	55	11	0.261	--	0.00	0.00
5.5	6	120	0.285	0.285	53	11	0.261	--	0.00	0.00
6	8	120	0.315	0.315	61	15	0.261	--	0.00	0.00
7	8	120	0.390	0.390	59	15	0.261	--	0.00	0.00
8	8	120	0.450	0.450	58	14	0.261	--	0.00	0.00
9	8	120	0.510	0.510	57	13	0.261	--	0.00	0.00
10	8	120	0.570	0.570	55	19	0.261	--	0.00	0.00
11	8	120	0.630	0.614	55	19	0.267	1.07	1.20	0.14
12	8	120	0.690	0.643	55	18	0.280	1.00	1.70	0.20
13	2	120	0.750	0.672		3	0.291	~	0.00	0.00
14	2	120	0.810	0.701		3	0.301	~	0.00	0.00
15	2	120	0.870	0.730		3	0.311	~	0.00	0.00
16	2	120	0.930	0.758		3	0.320	~	0.00	0.00
17	2	120	0.990	0.787		3	0.328	~	0.00	0.00
18	2	120	1.050	0.816		3	0.335	~	0.00	0.00
19	2	120	1.110	0.845		3	0.342	~	0.00	0.00
20	6	120	1.170	0.874		8	0.349	~	0.00	0.00
21	6	120	1.230	0.902		8	0.355	~	0.00	0.00
22	6	120	1.290	0.931		8	0.361	~	0.00	0.00
23	4	120	1.350	0.960		5	0.366	~	0.00	0.00
24	4	120	1.410	0.989	35	12	0.372	0.54	2.30	0.28
25	9	120	1.470	1.018	53	19	0.376	0.81	1.60	0.19
26	9	120	1.530	1.046	53	18	0.381	0.80	1.70	0.20
27	9	120	1.590	1.075	53	18	0.385	0.79	1.70	0.20
28	27	120	1.650	1.104	88	35	0.389	Non-Liq.	0.00	0.00
29	44	120	1.710	1.133	109	58	0.393	Non-Liq.	0.00	0.00
30	44	120	1.770	1.162	109	57	0.397	Non-Liq.	0.00	0.00
31	44	120	1.830	1.190	109	57	0.401	Non-Liq.	0.00	0.00
32	44	120	1.890	1.219	109	56	0.404	Non-Liq.	0.00	0.00
33	44	120	1.950	1.248	109	56	0.407	Non-Liq.	0.00	0.00
34	44	120	2.010	1.277	109	55	0.410	Non-Liq.	0.00	0.00
35	49	120	2.070	1.306	111	61	0.413	Non-Liq.	0.00	0.00
36	49	120	2.130	1.334	111	60	0.416	Non-Liq.	0.00	0.00
37	49	120	2.190	1.363	111	60	0.419	Non-Liq.	0.00	0.00
38	49	120	2.250	1.392	111	59	0.421	Non-Liq.	0.00	0.00
39	49	120	2.310	1.421	111	59	0.424	Non-Liq.	0.00	0.00
40	49	120	2.370	1.450	111	58	0.426	Non-Liq.	0.00	0.00
41	49	120	2.430	1.478	111	58	0.428	Non-Liq.	0.00	0.00
42	29	120	2.490	1.507	84	34	0.430	Non-Liq.	0.00	0.00
43.5	29	120	2.565	1.543	84	33	0.433	Non-Liq.	0.00	0.00
44	27	120	2.595	1.558	79	31	0.434	Non-Liq.	0.00	0.00
45.5	27	120	2.685	1.601	79	31	0.437	Non-Liq.	0.00	0.00
46	12	120	2.715	1.615		14	0.438	~	0.00	0.00
47	12	120	2.790	1.651		13	0.440	~	0.00	0.00
48	12	120	2.850	1.680		13	0.442	~	0.00	0.00
49	12	120	2.910	1.709		13	0.444	~	0.00	0.00
50	12	120	2.970	1.738		13	0.445	~	0.00	0.00

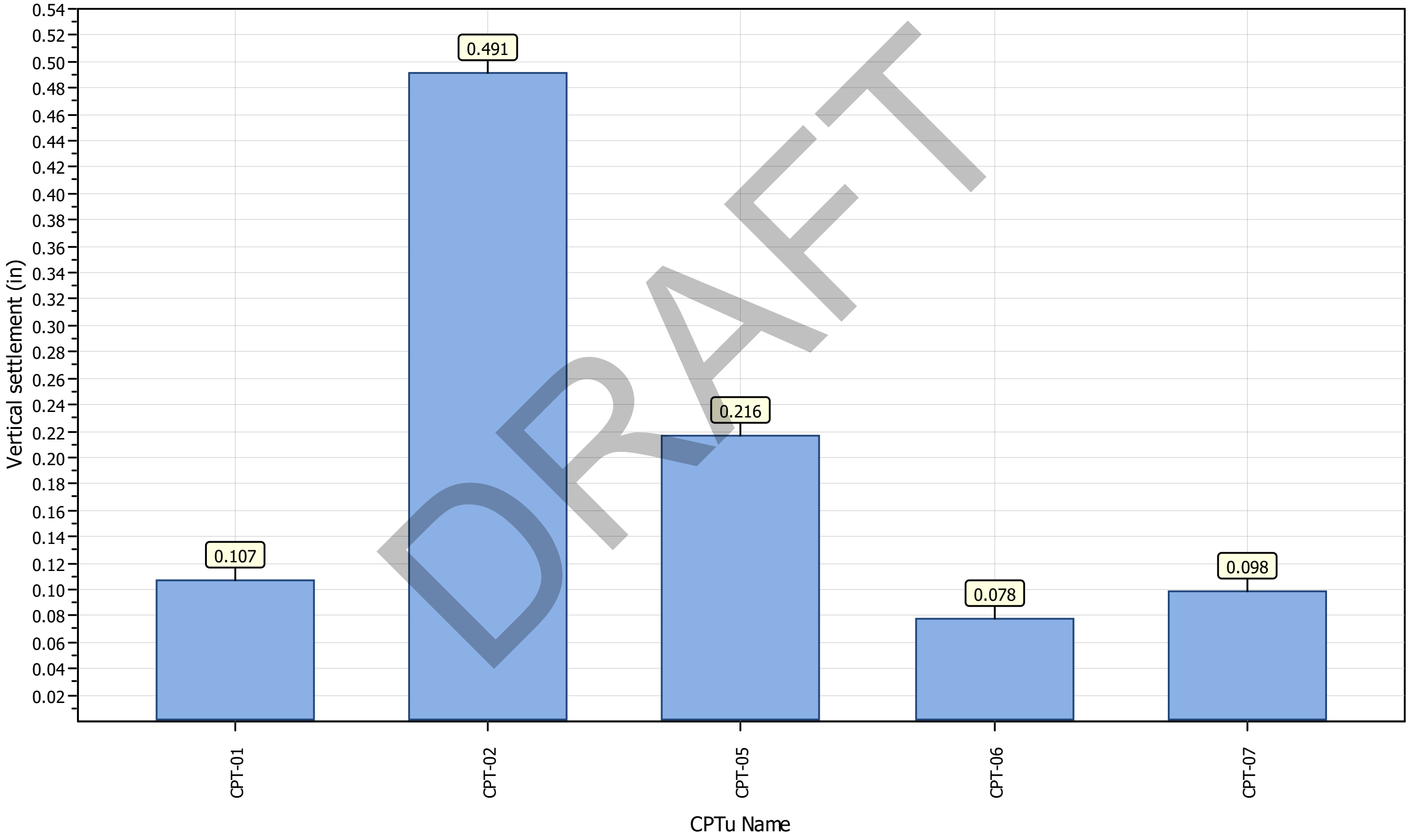
TOTAL SETTLEMENT = 1.2 INCHES

DRAFT

Project title : 1683 Sunflower

Location : A9933-88-01

Overall vertical settlements report



GEOCON

Client : Rose Equities
File No. : A9933-88-01
Boring : 3

EMPIRICAL ESTIMATION OF LIQUEFACTION POTENTIAL
MAXIMUM CONSIDERED EARTHQUAKE

NCEER (1996) METHOD

By Thomas F. Blake (1994-1996)

EARTHQUAKE INFORMATION:

Earthquake Magnitude:	6.70
Peak Horiz. Acceleration PGA_M (g):	0.601
Calculated Mag.Wtg.Factor:	0.753
Historic High Groundwater:	10.0
Groundwater Depth During Exploration:	21.0

ENERGY & ROD CORRECTIONS:

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

LIQUEFACTION CALCULATIONS:

Unit Wt. Water (pcf):	62.4

Depth to Base (ft)	Total Unit Wt. (pcf)	Water (0 or 1)	FIELD SPT (N)	Depth of SPT (ft)	Liq.Sus. (0 or 1)	-200 (%)	Est. Dr (%)	CN Factor	Corrected (N1)60	Eff. Unit Wt. (psf)	Resist. CRR	rd Factor	Induced CSR	Liquefac. Safe.Fact.
1.0	120.0	0	18.0	1.0	1		103	1.700	34.4	120.0	Infin.	0.998	0.293	--
2.5	120.0	0	18.0	2.0	1		99	1.700	34.4	120.0	Infin.	0.992	0.292	--
3.0	120.0	0	18.0	3.0	1		98	1.700	34.4	120.0	Infin.	0.987	0.290	--
4.0	120.0	0	18.0	4.0	1		95	1.700	34.4	120.0	Infin.	0.984	0.289	--
5.0	120.0	0	4.0	5.0	1		44	1.700	7.7	120.0	0.086	0.979	0.288	--
6.0	120.0	0	4.0	6.0	1		43	1.700	7.7	120.0	0.086	0.975	0.287	--
7.0	120.0	0	5.0	7.0	0			1.636	9.2	120.0	~	0.970	0.285	~
8.0	120.0	0	5.0	8.0	0			1.523	8.6	120.0	~	0.966	0.284	~
9.0	120.0	0	5.0	9.0	0			1.431	8.0	120.0	~	0.961	0.283	~
10.0	120.0	0	5.0	10.0	0			1.353	7.6	120.0	~	0.957	0.281	~
11.0	120.0	1	5.0	10.0	0			1.287	7.2	57.6	~	0.952	0.287	~
12.0	120.0	1	8.0	12.5	0			1.230	11.1	57.6	~	0.947	0.299	~
13.0	120.0	1	8.0	12.5	0			1.180	10.6	57.6	~	0.943	0.309	~
14.0	120.0	1	8.0	12.5	0			1.135	10.2	57.6	~	0.938	0.319	~
15.0	120.0	1	8.0	12.5	0			1.095	9.9	57.6	~	0.934	0.327	~
16.5	120.0	1	8.0	12.5	0			1.051	9.5	57.6	~	0.928	0.337	~
17.0	120.0	1	11.0	17.5	0			1.035	14.6	57.6	~	0.923	0.339	~
18.0	120.0	1	11.0	17.5	0			0.997	14.1	57.6	~	0.920	0.348	~
19.0	120.0	1	11.0	17.5	0			0.970	13.7	57.6	~	0.915	0.354	~
20.0	120.0	1	7.0	20.0	0			0.945	8.9	57.6	~	0.911	0.359	~
20.5	120.0	1	7.0	20.0	0			0.927	8.7	57.6	~	0.907	0.362	~
22.0	120.0	1	13.0	22.5	1	50	63	0.903	23.3	57.6	0.260	0.903	0.369	0.70
23.0	120.0	1	13.0	22.5	1	50	63	0.895	23.2	57.6	0.258	0.897	0.371	0.70
24.0	120.0	1	13.0	22.5	1	50	63	0.885	23.0	57.6	0.255	0.893	0.374	0.68
25.0	120.0	1	13.0	22.5	1	50	63	0.876	22.8	57.6	0.253	0.888	0.377	0.67
26.0	120.0	1	13.0	22.5	1	50	63	0.867	22.7	57.6	0.251	0.883	0.380	0.66
27.0	120.0	1	25.0	27.5	1	4	85	0.858	31.5	57.6	Infin.	0.879	0.382	Non-Liq.
28.0	120.0	1	25.0	27.5	1	4	85	0.849	31.2	57.6	Infin.	0.874	0.384	Non-Liq.
29.0	120.0	1	25.0	27.5	1	4	85	0.841	30.9	57.6	Infin.	0.870	0.386	Non-Liq.
30.0	120.0	1	25.0	27.5	1	4	85	0.833	30.6	57.6	Infin.	0.865	0.388	Non-Liq.
31.0	120.0	1	25.0	27.5	1	4	85	0.825	30.3	57.6	Infin.	0.861	0.389	Non-Liq.
32.0	120.0	1	25.0	32.5	1	4	82	0.817	30.7	57.6	Infin.	0.856	0.390	Non-Liq.
33.0	120.0	1	25.0	32.5	1	4	82	0.810	30.4	57.6	Infin.	0.851	0.391	Non-Liq.
34.0	120.0	1	25.0	32.5	1	4	82	0.803	30.1	57.6	Infin.	0.847	0.392	Non-Liq.
35.0	120.0	1	25.0	32.5	1	10	82	0.796	30.9	57.6	Infin.	0.842	0.393	Non-Liq.
36.0	120.0	1	25.0	32.5	1	10	82	0.789	30.7	57.6	Infin.	0.838	0.393	Non-Liq.
37.0	120.0	1	50.0	37.5	1	10	112	0.782	59.8	57.6	Infin.	0.833	0.394	Non-Liq.
38.0	120.0	1	50.0	37.5	1	10	112	0.776	59.3	57.6	Infin.	0.829	0.394	Non-Liq.
39.0	120.0	1	50.0	37.5	1	10	112	0.769	58.8	57.6	Infin.	0.824	0.394	Non-Liq.
40.0	120.0	1	50.0	37.5	1	10	112	0.763	58.3	57.6	Infin.	0.819	0.394	Non-Liq.
41.0	120.0	1	12.0	42.5	0			0.757	13.6	57.6	~	0.815	0.394	~
42.0	120.0	1	12.0	42.5	0			0.751	13.5	57.6	~	0.810	0.394	~
43.0	120.0	1	12.0	42.5	0			0.745	13.4	57.6	~	0.806	0.393	~
44.0	120.0	1	12.0	42.5	0			0.740	13.3	57.6	~	0.801	0.393	~
45.0	120.0	1	12.0	42.5	0			0.734	13.2	57.6	~	0.797	0.393	~
46.5	120.0	1	12.0	42.5	0			0.727	13.1	57.6	~	0.791	0.392	~
47.0	120.0	1	25.0	47.5	1	54	75	0.725	34.2	57.6	Infin.	0.786	0.390	Non-Liq.
48.0	120.0	1	25.0	47.5	1	54	75	0.718	33.9	57.6	Infin.	0.783	0.391	Non-Liq.
49.0	120.0	1	25.0	47.5	1	54	75	0.713	33.7	57.6	Infin.	0.778	0.390	Non-Liq.
50.0	120.0	1	25.0	47.5	1	54	75	0.708	33.6	57.6	Infin.	0.774	0.389	Non-Liq.

DRAFT

GEOCON

Client : Rose Equities
File No. : A9933-88-01
Boring : 3

LIQUEFACTION SETTLEMENT ANALYSIS
MAXIMUM CONSIDERED EARTHQUAKE

(SATURATED SAND AT INITIAL LIQUEFACTION CONDITION)

NCEER (1996) METHOD
EARTHQUAKE INFORMATION:

Earthquake Magnitude:	6.70
PGA_M (g):	0.601
Calculated Mag.Wtg.Factor:	0.753
Historic High Groundwater:	10.0
Groundwater @ Exploration:	21.0

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	18	120	0.030	0.030	103	34	0.391	--	0.00	0.00
2.5	18	120	0.105	0.105	99	34	0.391	--	0.00	0.00
3	18	120	0.135	0.135	98	34	0.391	--	0.00	0.00
4	18	120	0.210	0.210	95	34	0.391	--	0.00	0.00
5	4	120	0.270	0.270	44	8	0.391	--	0.00	0.00
6	4	120	0.330	0.330	43	8	0.391	--	0.00	0.00
7	5	120	0.390	0.390		9	0.391	~	0.00	0.00
8	5	120	0.450	0.450		9	0.391	~	0.00	0.00
9	5	120	0.510	0.510		8	0.391	~	0.00	0.00
10	5	120	0.570	0.570		8	0.391	~	0.00	0.00
11	5	120	0.630	0.614		7	0.401	~	0.00	0.00
12	8	120	0.690	0.643		11	0.419	~	0.00	0.00
13	8	120	0.750	0.672		11	0.436	~	0.00	0.00
14	8	120	0.810	0.701		10	0.452	~	0.00	0.00
15	8	120	0.870	0.730		10	0.466	~	0.00	0.00
16.5	8	120	0.945	0.766		9	0.482	~	0.00	0.00
17	11	120	0.975	0.780		15	0.488	~	0.00	0.00
18	11	120	1.050	0.816		14	0.503	~	0.00	0.00
19	11	120	1.110	0.845		14	0.513	~	0.00	0.00
20	7	120	1.170	0.874		9	0.523	~	0.00	0.00
20.5	7	120	1.215	0.895		9	0.530	~	0.00	0.00
22	13	120	1.305	0.938	63	23	0.543	0.70	1.30	0.23
23	13	120	1.350	0.960	63	23	0.549	0.70	1.30	0.16
24	13	120	1.410	0.989	63	23	0.557	0.68	1.30	0.16
25	13	120	1.470	1.018	63	23	0.564	0.67	1.30	0.16
26	13	120	1.530	1.046	63	23	0.571	0.66	1.30	0.16
27	25	120	1.590	1.075	85	32	0.578	Non-Liq.	0.00	0.00
28	25	120	1.650	1.104	85	31	0.584	Non-Liq.	0.00	0.00
29	25	120	1.710	1.133	85	31	0.590	Non-Liq.	0.00	0.00
30	25	120	1.770	1.162	85	31	0.595	Non-Liq.	0.00	0.00
31	25	120	1.830	1.190	85	30	0.601	Non-Liq.	0.00	0.00
32	25	120	1.890	1.219	82	31	0.606	Non-Liq.	0.00	0.00
33	25	120	1.950	1.248	82	30	0.610	Non-Liq.	0.00	0.00
34	25	120	2.010	1.277	82	30	0.615	Non-Liq.	0.00	0.00
35	25	120	2.070	1.306	82	31	0.619	Non-Liq.	0.00	0.00
36	25	120	2.130	1.334	82	31	0.624	Non-Liq.	0.00	0.00
37	50	120	2.190	1.363	112	60	0.628	Non-Liq.	0.00	0.00
38	50	120	2.250	1.392	112	59	0.631	Non-Liq.	0.00	0.00
39	50	120	2.310	1.421	112	59	0.635	Non-Liq.	0.00	0.00
40	50	120	2.370	1.450	112	58	0.639	Non-Liq.	0.00	0.00
41	12	120	2.430	1.478		14	0.642	~	0.00	0.00
42	12	120	2.490	1.507		14	0.645	~	0.00	0.00
43	12	120	2.550	1.536		13	0.649	~	0.00	0.00
44	12	120	2.610	1.565		13	0.652	~	0.00	0.00
45	12	120	2.670	1.594		13	0.655	~	0.00	0.00
46.5	12	120	2.745	1.630		13	0.658	~	0.00	0.00
47	25	120	2.775	1.644	75	34	0.659	Non-Liq.	0.00	0.00
48	25	120	2.850	1.680	75	34	0.663	Non-Liq.	0.00	0.00
49	25	120	2.910	1.709	75	34	0.665	Non-Liq.	0.00	0.00
50	25	120	2.970	1.738	75	34	0.668	Non-Liq.	0.00	0.00

TOTAL SETTLEMENT = 0.9 INCHES

DRAFT

Figure 11

GEOCON

Client : Rose Equities
File No. : A9933-88-01
Boring : 7

EMPIRICAL ESTIMATION OF LIQUEFACTION POTENTIAL
MAXIMUM CONSIDERED EARTHQUAKE

NCEER (1996) METHOD
By Thomas F. Blake (1994-1996)

EARTHQUAKE INFORMATION:

Earthquake Magnitude:	6.70
Peak Horiz. Acceleration PGA_M (g):	0.601
Calculated Mag.Wtg.Factor:	0.753
Historic High Groundwater:	10.0
Groundwater Depth During Exploration:	17.5

ENERGY & ROD CORRECTIONS:

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

LIQUEFACTION CALCULATIONS:

Unit Wt. Water (pcf):	62.4

Depth to Base (ft)	Total Unit Wt. (pcf)	Water (0 or 1)	FIELD SPT (N)	Depth of SPT (ft)	Liq.Sus. (0 or 1)	-200 (%)	Est. Dr (%)	CN Factor	Corrected (N1)60	Eff. Unit Wt. (psf)	Resist. CRR	rd Factor	Induced CSR	Liquefac. Safe.Fact.
1.0	120.0	0	6.0	1.0	1		59	1.700	11.5	120.0	0.125	0.998	0.293	--
2.0	120.0	0	6.0	2.0	1		58	1.700	11.5	120.0	0.125	0.993	0.292	--
3.0	120.0	0	6.0	3.0	1		56	1.700	11.5	120.0	0.125	0.989	0.291	--
4.0	120.0	0	6.0	4.0	1		55	1.700	11.5	120.0	0.125	0.984	0.289	--
5.5	120.0	0	6.0	5.0	1		53	1.700	11.5	120.0	0.125	0.978	0.288	--
6.0	120.0	0	8.0	6.0	1		61	1.700	15.3	120.0	0.167	0.974	0.286	--
7.0	120.0	0	8.0	7.0	1		59	1.636	14.7	120.0	0.161	0.970	0.285	--
8.0	120.0	0	8.0	8.0	1		58	1.523	13.7	120.0	0.150	0.966	0.284	--
9.0	120.0	0	8.0	9.0	1		57	1.431	12.9	120.0	0.141	0.961	0.283	--
10.0	120.0	0	8.0	10.0	1	50	55	1.353	19.2	120.0	0.209	0.957	0.281	--
11.0	120.0	1	8.0	10.0	1	50	55	1.287	18.6	57.6	0.202	0.952	0.287	0.70
12.0	120.0	1	8.0	10.0	1	50	55	1.230	18.1	57.6	0.196	0.947	0.299	0.66
13.0	120.0	1	2.0	12.5	0			1.180	2.7	57.6	~	0.943	0.309	~
14.0	120.0	1	2.0	12.5	0			1.135	2.6	57.6	~	0.938	0.319	~
15.0	120.0	1	2.0	17.5	0			1.095	2.8	57.6	~	0.934	0.327	~
16.0	120.0	1	2.0	17.5	0			1.060	2.7	57.6	~	0.929	0.335	~
17.0	120.0	1	2.0	17.5	0			1.027	2.6	57.6	~	0.925	0.342	~
18.0	120.0	1	2.0	17.5	0			1.005	2.6	57.6	~	0.920	0.348	~
19.0	120.0	1	2.0	17.5	0			0.991	2.5	57.6	~	0.915	0.354	~
20.0	120.0	1	6.0	20.0	0			0.978	7.9	57.6	~	0.911	0.359	~
21.0	120.0	1	6.0	20.0	0			0.965	7.8	57.6	~	0.906	0.363	~
22.0	120.0	1	6.0	20.0	0			0.953	7.7	57.6	~	0.902	0.367	~
23.0	120.0	1	4.0	22.5	0			0.941	5.2	57.6	~	0.897	0.371	~
24.0	120.0	1	4.0	22.5	1	56	35	0.930	12.2	57.6	0.133	0.893	0.374	0.36
25.0	120.0	1	9.0	22.5	1	54	53	0.919	18.5	57.6	0.201	0.888	0.377	0.53
26.0	120.0	1	9.0	22.5	1	54	53	0.909	18.4	57.6	0.200	0.883	0.380	0.53
27.0	120.0	1	9.0	22.5	1	54	53	0.898	18.3	57.6	0.198	0.879	0.382	0.52
28.0	120.0	1	27.0	27.5	1		88	0.889	35.2	57.6	Infin.	0.874	0.384	Non-Liq.
29.0	120.0	1	44.0	32.5	1		109	0.879	58.0	57.6	Infin.	0.870	0.386	Non-Liq.
30.0	120.0	1	44.0	32.5	1		109	0.870	57.4	57.6	Infin.	0.865	0.388	Non-Liq.
31.0	120.0	1	44.0	32.5	1		109	0.861	56.8	57.6	Infin.	0.861	0.389	Non-Liq.
32.0	120.0	1	44.0	32.5	1		109	0.852	56.2	57.6	Infin.	0.856	0.390	Non-Liq.
33.0	120.0	1	44.0	32.5	1		109	0.844	55.7	57.6	Infin.	0.851	0.391	Non-Liq.
34.0	120.0	1	44.0	32.5	1		109	0.836	55.1	57.6	Infin.	0.847	0.392	Non-Liq.
35.0	120.0	1	49.0	37.5	1		111	0.828	60.8	57.6	Infin.	0.842	0.393	Non-Liq.
36.0	120.0	1	49.0	37.5	1		111	0.820	60.3	57.6	Infin.	0.838	0.393	Non-Liq.
37.0	120.0	1	49.0	37.5	1		111	0.812	59.7	57.6	Infin.	0.833	0.394	Non-Liq.
38.0	120.0	1	49.0	37.5	1		111	0.805	59.2	57.6	Infin.	0.829	0.394	Non-Liq.
39.0	120.0	1	49.0	37.5	1		111	0.798	58.7	57.6	Infin.	0.824	0.394	Non-Liq.
40.0	120.0	1	49.0	37.5	1		111	0.791	58.1	57.6	Infin.	0.819	0.394	Non-Liq.
41.0	120.0	1	49.0	37.5	1		111	0.784	57.7	57.6	Infin.	0.815	0.394	Non-Liq.
42.0	120.0	1	29.0	42.5	1		84	0.778	33.8	57.6	Infin.	0.810	0.394	Non-Liq.
43.5	120.0	1	29.0	42.5	1		84	0.770	33.5	57.6	Infin.	0.805	0.393	Non-Liq.
44.0	120.0	1	27.0	45.0	1		79	0.767	31.1	57.6	Infin.	0.800	0.392	Non-Liq.
45.5	120.0	1	27.0	45.0	1		79	0.758	30.7	57.6	Infin.	0.795	0.392	Non-Liq.
46.0	120.0	1	12.0	47.5	0			0.755	13.6	57.6	~	0.791	0.391	~
47.0	120.0	1	12.0	47.5	0			0.747	13.5	57.6	~	0.787	0.391	~
48.0	120.0	1	12.0	47.5	0			0.742	13.3	57.6	~	0.783	0.391	~
49.0	120.0	1	12.0	47.5	0			0.736	13.2	57.6	~	0.778	0.390	~
50.0	120.0	1	12.0	47.5	0			0.731	13.2	57.6	~	0.774	0.389	~

DRAFT

GEOCON

Client : Rose Equities
File No. : A9933-88-01
Boring : 7

LIQUEFACTION SETTLEMENT ANALYSIS
MAXIMUM CONSIDERED EARTHQUAKE

(SATURATED SAND AT INITIAL LIQUEFACTION CONDITION)

NCEER (1996) METHOD
EARTHQUAKE INFORMATION:

Earthquake Magnitude:	6.70
PGA_M (g):	0.601
Calculated Mag.Wtg.Factor:	0.753
Historic High Groundwater:	10.0
Groundwater @ Exploration:	17.5

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	6	120	0.030	0.030	59	11	0.391	--	0.00	0.00
2	6	120	0.090	0.090	58	11	0.391	--	0.00	0.00
3	6	120	0.150	0.150	56	11	0.391	--	0.00	0.00
4	6	120	0.210	0.210	55	11	0.391	--	0.00	0.00
5.5	6	120	0.285	0.285	53	11	0.391	--	0.00	0.00
6	8	120	0.315	0.315	61	15	0.391	--	0.00	0.00
7	8	120	0.390	0.390	59	15	0.391	--	0.00	0.00
8	8	120	0.450	0.450	58	14	0.391	--	0.00	0.00
9	8	120	0.510	0.510	57	13	0.391	--	0.00	0.00
10	8	120	0.570	0.570	55	19	0.391	--	0.00	0.00
11	8	120	0.630	0.614	55	19	0.401	0.70	1.60	0.19
12	8	120	0.690	0.643	55	18	0.419	0.66	1.70	0.20
13	2	120	0.750	0.672		3	0.436	~	0.00	0.00
14	2	120	0.810	0.701		3	0.452	~	0.00	0.00
15	2	120	0.870	0.730		3	0.466	~	0.00	0.00
16	2	120	0.930	0.758		3	0.479	~	0.00	0.00
17	2	120	0.990	0.787		3	0.491	~	0.00	0.00
18	2	120	1.050	0.816		3	0.503	~	0.00	0.00
19	2	120	1.110	0.845		3	0.513	~	0.00	0.00
20	6	120	1.170	0.874		8	0.523	~	0.00	0.00
21	6	120	1.230	0.902		8	0.532	~	0.00	0.00
22	6	120	1.290	0.931		8	0.541	~	0.00	0.00
23	4	120	1.350	0.960		5	0.549	~	0.00	0.00
24	4	120	1.410	0.989	35	12	0.557	0.36	2.30	0.28
25	9	120	1.470	1.018	53	19	0.564	0.53	1.60	0.19
26	9	120	1.530	1.046	53	18	0.571	0.53	1.70	0.20
27	9	120	1.590	1.075	53	18	0.578	0.52	1.70	0.20
28	27	120	1.650	1.104	88	35	0.584	Non-Liq.	0.00	0.00
29	44	120	1.710	1.133	109	58	0.590	Non-Liq.	0.00	0.00
30	44	120	1.770	1.162	109	57	0.595	Non-Liq.	0.00	0.00
31	44	120	1.830	1.190	109	57	0.601	Non-Liq.	0.00	0.00
32	44	120	1.890	1.219	109	56	0.606	Non-Liq.	0.00	0.00
33	44	120	1.950	1.248	109	56	0.610	Non-Liq.	0.00	0.00
34	44	120	2.010	1.277	109	55	0.615	Non-Liq.	0.00	0.00
35	49	120	2.070	1.306	111	61	0.619	Non-Liq.	0.00	0.00
36	49	120	2.130	1.334	111	60	0.624	Non-Liq.	0.00	0.00
37	49	120	2.190	1.363	111	60	0.628	Non-Liq.	0.00	0.00
38	49	120	2.250	1.392	111	59	0.631	Non-Liq.	0.00	0.00
39	49	120	2.310	1.421	111	59	0.635	Non-Liq.	0.00	0.00
40	49	120	2.370	1.450	111	58	0.639	Non-Liq.	0.00	0.00
41	49	120	2.430	1.478	111	58	0.642	Non-Liq.	0.00	0.00
42	29	120	2.490	1.507	84	34	0.645	Non-Liq.	0.00	0.00
43.5	29	120	2.565	1.543	84	33	0.649	Non-Liq.	0.00	0.00
44	27	120	2.595	1.558	79	31	0.651	Non-Liq.	0.00	0.00
45.5	27	120	2.685	1.601	79	31	0.655	Non-Liq.	0.00	0.00
46	12	120	2.715	1.615		14	0.657	~	0.00	0.00
47	12	120	2.790	1.651		13	0.660	~	0.00	0.00
48	12	120	2.850	1.680		13	0.663	~	0.00	0.00
49	12	120	2.910	1.709		13	0.665	~	0.00	0.00
50	12	120	2.970	1.738		13	0.668	~	0.00	0.00

TOTAL SETTLEMENT = 1.3 INCHES

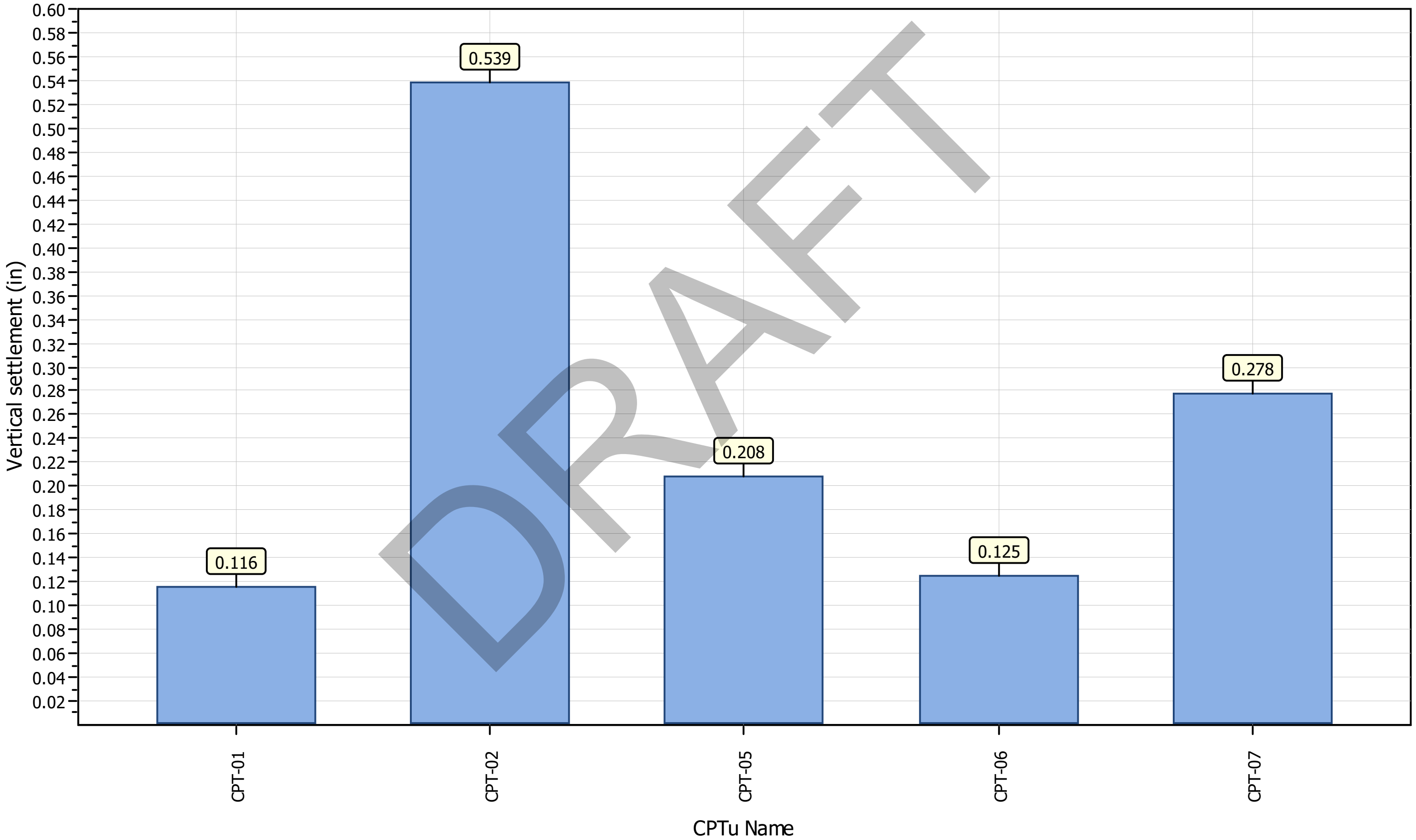
DRAFT

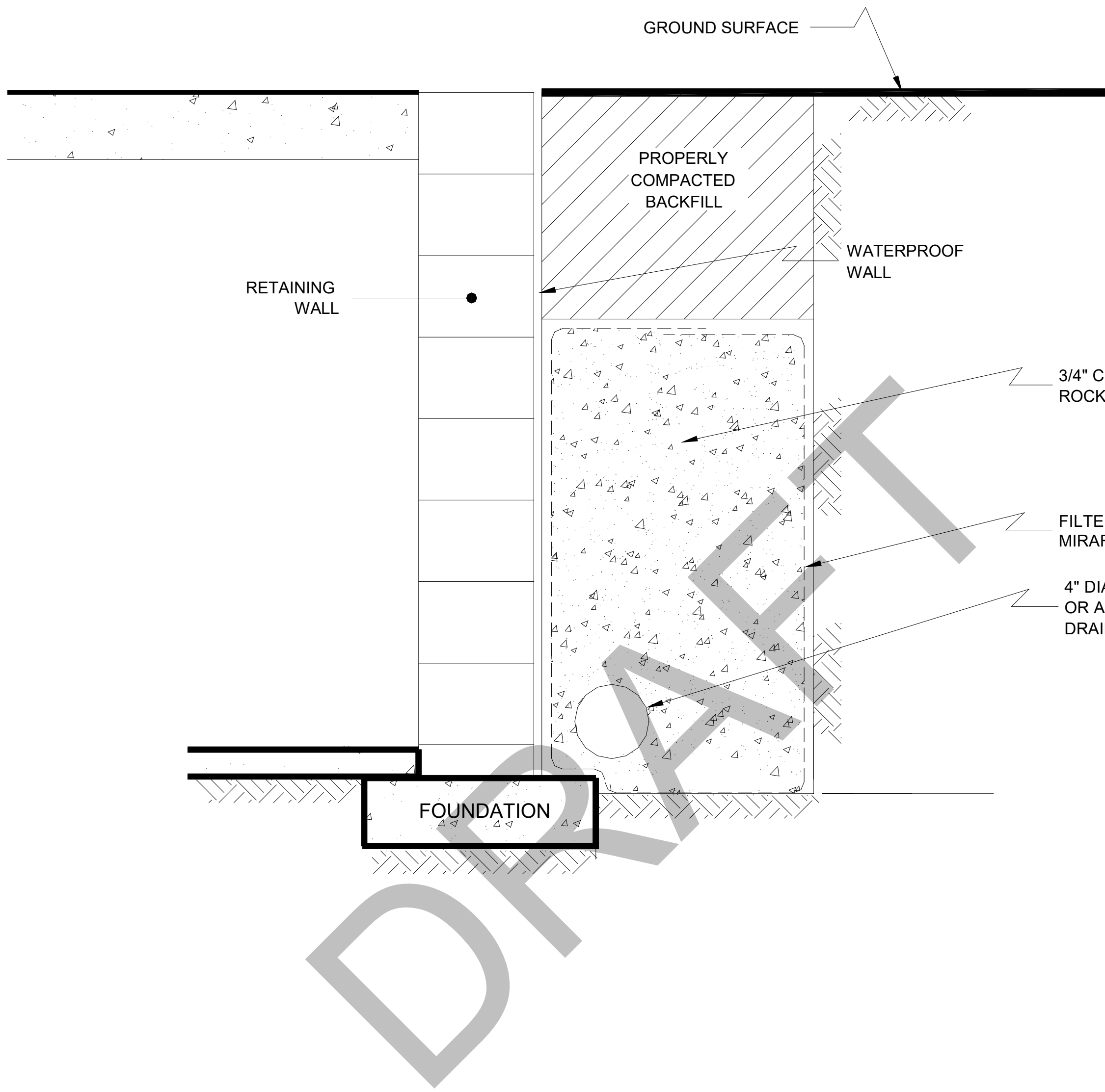
GEOCON

Project title : 1683 Sunflower

Location : A9933-88-01

Overall vertical settlements report





NO SCALE

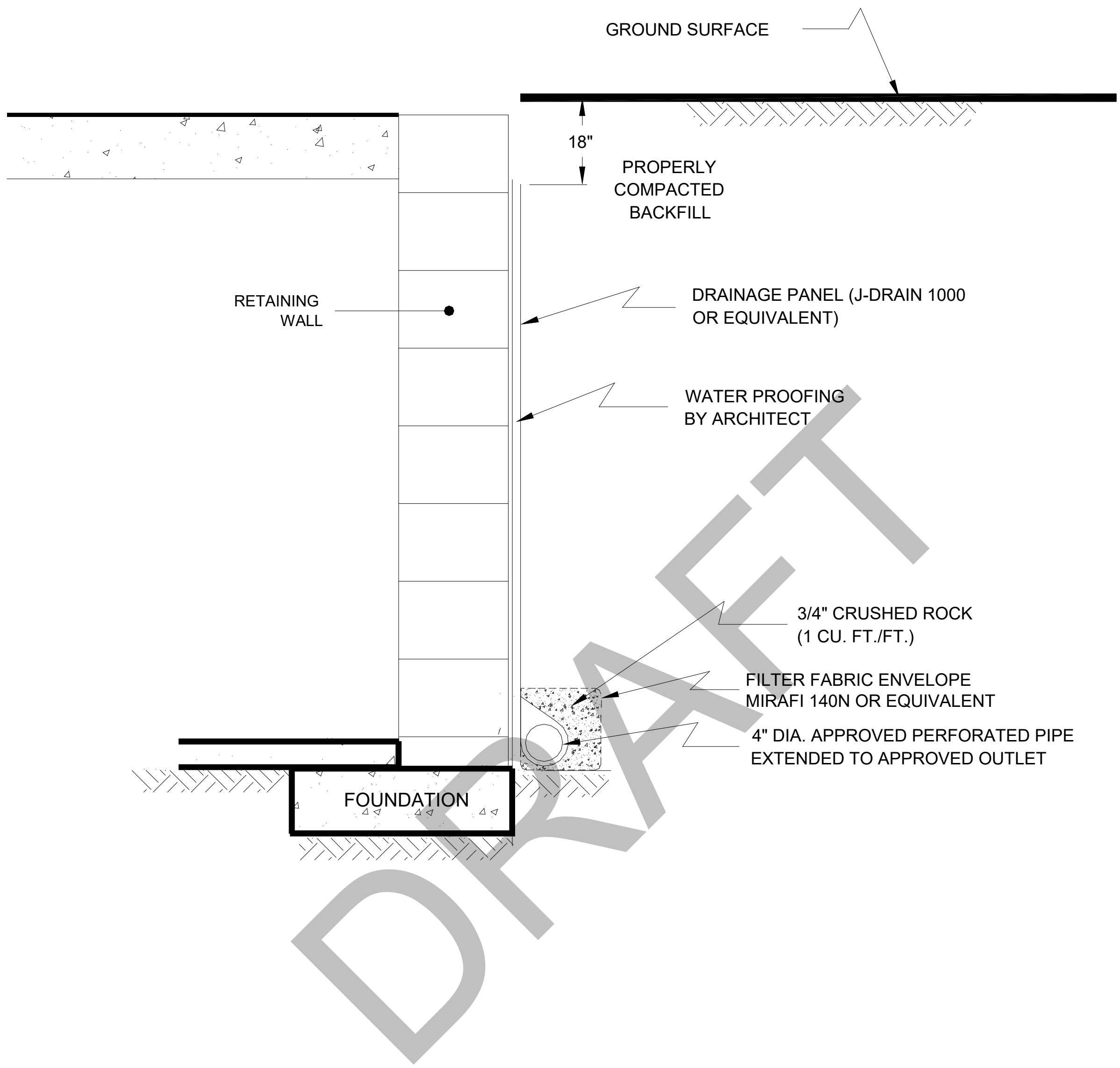
GEOCON WEST, INC.
ENVIRONMENTAL GEOTECHNICAL MATERIALS
15520 ROCKFIELD BOULEVARD, SUITE J, IRVINE, CA 92618
PHONE (949) 491-6570 FAX (949) 299-4550

Drafted by: JTA	Checked by: NDB

RETAINING WALL DRAIN DETAIL

ROSE EQUITIES
1683 SUNFLOWER AVENUE
COSTA MESA, CALIFORNIA

JULY 2019	PROJECT NO. A9933-88-01	FIG. 15



NO SCALE

GEOCON
WEST, INC.
ENVIRONMENTAL GEOTECHNICAL MATERIALS
15520 ROCKFIELD BOULEVARD, SUITE J, IRVINE, CA 92618
PHONE (949) 491-6570 FAX (949) 299-4550

Drafted by: JTA	Checked by: NDB

RETAINING WALL DRAIN DETAIL

ROSE EQUITIES
1683 SUNFLOWER AVENUE
COSTA MESA, CALIFORNIA

JULY 2019	PROJECT NO. A9933-88-01	FIG. 16

PERCOLATION TEST DATA SHEET

Project:	1683 Sunflower		Project No:	A9933-88-01		Date:	2/27/2019
Test Hole No:		P1	Tested By:	JS			
Depth of Test Hole, D_T:		5	USCS Soil Classification:		SP / SM / CL		
Test Hole Dimensions (inches)					Length	Width	
Diameter (if round) =		8	Sides (if rectangular) =		---	---	

Sandy Soil Criteria Test*

Trial No.	Start Time	Stop Time	Δt Time Interval (min)	D_0 Initial Depth to Water (in)	D_f Final Depth to Water (in)	ΔD Change in Water Level (in)	Greater than or Equal to 6"? (y/n)
1	12:07	12:32	25	14.5	39.0	24.5	y
2	12:35	13:00	25	14.6	31.7	17.0	y

*If two consecutive measurements show that six inches of water seeps away in less than 25 minutes, the test shall be run for an additional hour with measurements, taken every 10 minutes. Otherwise, pre-soak (fill) overnight. Obtain at least twelve measurements per hole over at least six hours (approximately 30 minute intervals) with a precision of at least 0.25".

Trial No.	Start Time	Stop Time	Δt Time Interval (min)	D_0 Initial Depth to Water (in)	D_f Final Depth to Water (in)	ΔD Change in Water Level (in)	Percolation Rate (min/in)
1	13:02	13:12	10	16.3	24.1	7.8	1846
2	13:12	13:22	10	14.4	21.8	7.4	1935
3	13:23	13:33	10	14.9	23.9	9.0	1600
4	13:36	13:46	10	14.9	22.1	7.2	2000
5	13:46	13:56	10	14.6	22.7	8.0	1791
6	13:47	13:57	10	15.1	22.6	7.4	1935
7							
8							

Infiltration Rate Calculation:

Time Interval, Δt =	10	minutes	Ho =	43.7	inches
Final Depth to Water, Df =	22.6	inches	Hf =	37.4	inches
Test Hole Radius, r =	4	inches	ΔH =	6.2	inches
Initial Depth to Water, Do =	16.3	inches	Havg =	40.6	inches
Total Depth of Test Hole, DT =	60.0	inches			

$$I_t = \frac{\Delta H(60r)}{\Delta t(r + 2H_{avg})}$$

Infiltration Rate, It = **1.76** inches/hour

DRAFT

PERCOLATION TEST DATA SHEET

Project:	1683 Sunflower	Project No:	A9933-88-01	Date:	2/27/2019
Test Hole No:	P2	Tested By:	JS		
Depth of Test Hole, D_T:	5	USCS Soil Classification:	CL		
Test Hole Dimensions (inches)			Length	Width	
Diameter (if round) =	8	Sides (if rectangular) =	---	---	

Sandy Soil Criteria Test*

Trial No.	Start Time	Stop Time	Δt Time Interval (min)	D_0 Initial Depth to Water (in)	D_f Final Depth to Water (in)	ΔD Change in Water Level (in)	Greater than or Equal to 6"? (y/n)
1	10:08	10:33	25	12.8	30.0	17.2	y
2	10:35	11:00	25	14.2	30.4	16.2	y

*If two consecutive measurements show that six inches of water seeps away in less than 25 minutes, the test shall be run for an additional hour with measurements, taken every 10 minutes. Otherwise, pre-soak (fill) overnight. Obtain at least twelve measurements per hole over at least six hours (approximately 30 minute intervals) with a precision of at least 0.25".

Trial No.	Start Time	Stop Time	Δt Time Interval (min)	D_0 Initial Depth to Water (in)	D_f Final Depth to Water (in)	ΔD Change in Water Level (in)	Percolation Rate (min/in)
1	11:05	11:15	10	13.0	19.9	7.0	2069
2	11:15	11:25	10	14.3	21.8	7.6	1905
3	11:27	11:37	10	13.2	21.6	8.4	1714
4	11:40	11:50	10	13.1	21.5	8.4	1714
5	11:50	12:00	10	12.4	19.6	7.2	2000
6	12:02	12:12	10	12.2	20.5	8.3	1739
7							
8							

Infiltration Rate Calculation:

Time Interval, Δt =	10	minutes	Ho =	47.0	inches
Final Depth to Water, Df =	20.5	inches	Hf =	39.5	inches
Test Hole Radius, r =	4	inches	ΔH =	7.6	inches
Initial Depth to Water, Do =	13.0	inches	Havg =	43.3	inches
Total Depth of Test Hole, DT =	60.0	inches			

$$I_t = \frac{\Delta H(60r)}{\Delta t(r + 2H_{avg})}$$

Infiltration Rate, It = **2.00** inches/hour

DRAFT

Figure 18

PERCOLATION TEST DATA SHEET

Project:	1683 Sunflower	Project No:	A9933-88-01	Date:	2/27/2019
Test Hole No:	P3	Tested By:	JS		
Depth of Test Hole, D_T:	5	USCS Soil Classification:	SM / ML		
Test Hole Dimensions (inches)			Length	Width	
Diameter (if round) =	8	Sides (if rectangular) =	---	---	

Sandy Soil Criteria Test*

Trial No.	Start Time	Stop Time	Δt Time Interval (min)	D_0 Initial Depth to Water (in)	D_f Final Depth to Water (in)	ΔD Change in Water Level (in)	Greater than or Equal to 6"? (y/n)
1	9:23	9:48	25	14.3	24.1	9.8	y
2	9:48	10:13	25	14.5	25.0	10.4	y

*If two consecutive measurements show that six inches of water seeps away in less than 25 minutes, the test shall be run for an additional hour with measurements, taken every 10 minutes. Otherwise, pre-soak (fill) overnight. Obtain at least twelve measurements per hole over at least six hours (approximately 30 minute intervals) with a precision of at least 0.25".

Trial No.	Start Time	Stop Time	Δt Time Interval (min)	D_0 Initial Depth to Water (in)	D_f Final Depth to Water (in)	ΔD Change in Water Level (in)	Percolation Rate (min/in)
1	10:13	10:23	10	14.8	19.8	5.0	2857
2	10:26	10:36	10	14.3	19.8	5.5	2609
3	10:38	10:48	10	14.6	18.4	3.7	3871
4	10:49	10:59	10	14.9	19.3	4.4	3243
5	11:01	11:11	10	14.4	18.4	4.0	3636
6	11:11	11:21	10	14.5	18.5	4.0	3636
7							
8							

Infiltration Rate Calculation:

Time Interval, Δt =	10	minutes	Ho =	45.2	inches
Final Depth to Water, Df =	18.5	inches	Hf =	41.5	inches
Test Hole Radius, r =	4	inches	ΔH =	3.7	inches
Initial Depth to Water, Do =	14.8	inches	Havg =	43.4	inches
Total Depth of Test Hole, DT =	60.0	inches			

$$I_t = \frac{\Delta H(60r)}{\Delta t(r + 2H_{avg})}$$

Infiltration Rate, It = **0.98** inches/hour

DRAFT

PERCOLATION TEST DATA SHEET

Project:	1683 Sunflower	Project No:	A9933-88-01	Date:	2/27/2019
Test Hole No:	P4	Tested By:	JS		
Depth of Test Hole, D_T:	5	USCS Soil Classification:	SM		

Test Hole Dimensions (inches)			Length	Width	
Diameter (if round) =	8	Sides (if rectangular) =	---	---	

Sandy Soil Criteria Test*

Trial No.	Start Time	Stop Time	Δt Time Interval (min)	D_0 Initial Depth to Water (in)	D_f Final Depth to Water (in)	ΔD Change in Water Level (in)	Greater than or Equal to 6"? (y/n)
1	8:45	9:10	25	14.5	19.6	5.0	N
2							

*If two consecutive measurements show that six inches of water seeps away in less than 25 minutes, the test shall be run for an additional hour with measurements, taken every 10 minutes. Otherwise, pre-soak (fill) overnight. Obtain at least twelve measurements per hole over at least six hours (approximately 30 minute intervals) with a precision of at least 0.25".

Trial No.	Start Time	Stop Time	Δt Time Interval (min)	D_0 Initial Depth to Water (in)	D_f Final Depth to Water (in)	ΔD Change in Water Level (in)	Percolation Rate (min/in)
1	9:10	9:40	30	14.9	19.8	4.9	8780
2	9:43	10:13	30	14.6	18.5	3.8	11250
3	10:15	10:45	30	14.6	17.8	3.1	13846
4	10:46	11:16	30	14.8	17.8	3.0	14400
5	11:18	11:48	30	14.6	17.2	2.5	17143
6	11:48	12:18	30	14.0	16.3	2.3	18947
7	12:18	12:48	30	13.9	16.3	2.4	18000
8	12:48	13:18	30	13.4	15.6	2.2	20000
9	13:18	13:48	30	13.1	15.2	2.2	20000
10	13:48	14:18	30	13.7	15.6	1.9	22500
11	14:18	14:48	30	13.2	14.9	1.7	25714
12	14:48	15:18	30	13.4	15.2	1.8	24000

Infiltration Rate Calculation:

Time Interval, Δt =	30	minutes	Ho =	45.1	inches
Final Depth to Water, Df =	15.2	inches	Hf =	44.8	inches
Test Hole Radius, r =	4	inches	ΔH =	0.4	inches
Initial Depth to Water, Do =	14.9	inches	Havg =	44.9	inches
Total Depth of Test Hole, DT =	60.0	inches			

$$I_t = \frac{\Delta H(60r)}{\Delta t(r + 2H_{avg})}$$

Infiltration Rate, It = **0.03** inches/hour

DRAFT

PERCOLATION TEST DATA SHEET

Project:	1683 Sunflower	Project No:	A9933-88-01	Date:	2/27/2019
Test Hole No:	P5	Tested By:	JS		
Depth of Test Hole, D_T:	5	USCS Soil Classification:	SM / CL		
Test Hole Dimensions (inches)			Length	Width	
Diameter (if round) =	8	Sides (if rectangular) =	---	---	

Sandy Soil Criteria Test*

Trial No.	Start Time	Stop Time	Δt Time Interval (min)	D_0 Initial Depth to Water (in)	D_f Final Depth to Water (in)	ΔD Change in Water Level (in)	Greater than or Equal to 6"? (y/n)
1	8:10	8:35	25	13.9	26.2	12.2	y
2	8:37	9:02	25	14.3	25.9	11.6	y

*If two consecutive measurements show that six inches of water seeps away in less than 25 minutes, the test shall be run for an additional hour with measurements, taken every 10 minutes. Otherwise, pre-soak (fill) overnight. Obtain at least twelve measurements per hole over at least six hours (approximately 30 minute intervals) with a precision of at least 0.25".

Trial No.	Start Time	Stop Time	Δt Time Interval (min)	D_0 Initial Depth to Water (in)	D_f Final Depth to Water (in)	ΔD Change in Water Level (in)	Percolation Rate (min/in)
1	9:04	9:14	10	13.7	19.0	5.3	2727
2	9:15	9:25	10	13.9	20.3	6.4	2264
3	9:28	9:38	10	13.8	19.1	5.3	2727
4	9:40	9:50	10	14.4	18.8	4.4	3243
5	9:50	10:00	10	14.0	19.1	5.0	2857
6	10:02	10:12	10	13.8	20.0	6.2	2308
7							
8							

Infiltration Rate Calculation:

Time Interval, Δt =	10	minutes	Ho =	46.3	inches
Final Depth to Water, Df =	20.0	inches	Hf =	40.0	inches
Test Hole Radius, r =	4	inches	ΔH =	6.4	inches
Initial Depth to Water, Do =	13.7	inches	Havg =	43.1	inches
Total Depth of Test Hole, DT =	60.0	inches			

$$I_t = \frac{\Delta H(60r)}{\Delta t(r + 2H_{avg})}$$

Infiltration Rate, It = **1.69** inches/hour

DRAFT

Figure 21

PERCOLATION TEST DATA SHEET

Project:	1683 Sunflower		Project No:	A9933-88-01		Date:	2/27/2019
Test Hole No:		P6	Tested By:	JS			
Depth of Test Hole, D_T:		5	USCS Soil Classification:		SM / ML		
Test Hole Dimensions (inches)					Length	Width	
Diameter (if round) =		8	Sides (if rectangular) =		---	---	

Sandy Soil Criteria Test*

Trial No.	Start Time	Stop Time	Δt Time Interval (min)	D_0 Initial Depth to Water (in)	D_f Final Depth to Water (in)	ΔD Change in Water Level (in)	Greater than or Equal to 6"? (y/n)
1	8:03	8:28	25	14.5	16.2	1.7	n
2							

*If two consecutive measurements show that six inches of water seeps away in less than 25 minutes, the test shall be run for an additional hour with measurements, taken every 10 minutes. Otherwise, pre-soak (fill) overnight. Obtain at least twelve measurements per hole over at least six hours (approximately 30 minute intervals) with a precision of at least 0.25".

Trial No.	Start Time	Stop Time	Δt Time Interval (min)	D_0 Initial Depth to Water (in)	D_f Final Depth to Water (in)	ΔD Change in Water Level (in)	Percolation Rate (min/in)
1	8:30	8:40	10	14.3	15.8	1.6	9231
2	9:00	9:10	10	14.2	15.8	1.7	8571
3	9:30	9:40	10	13.3	15.0	1.7	8571
4	10:00	10:10	10	13.2	14.9	1.7	8571
5	10:30	10:40	10	12.1	14.2	2.0	7059
6	11:00	11:10	10	13.7	15.4	1.7	8571
7	11:30	11:40	10	12.7	14.4	1.7	8571
8	12:00	12:10	10	14.4	15.6	1.2	12000
9	12:30	12:40	10	12.4	14.6	2.3	6316
10	1:00	1:10	10	14.6	15.6	1.0	15000
11	1:30	1:40	10	12.7	14.9	2.2	6667
12	2:00	2:10	10	14.9	15.7	0.8	17143

Infiltration Rate Calculation:

Time Interval, Δt =	10	minutes	Ho =	45.7	inches
Final Depth to Water, Df =	15.4	inches	Hf =	44.6	inches
Test Hole Radius, r =	4	inches	ΔH =	1.1	inches
Initial Depth to Water, Do =	14.3	inches	Havg =	45.2	inches
Total Depth of Test Hole, DT =	60.0	inches			

$$I_t = \frac{\Delta H(60r)}{\Delta t(r + 2H_{avg})}$$

Infiltration Rate, It = **0.27** inches/hour

DRAFT

APPENDIX A

DRAFT

APPENDIX A

FIELD INVESTIGATION

The site was explored on February 12, 25, and 26, 2019, by excavating a total of 13 8-inch diameter borings using a hollow stem auger and advancing five cone penetrometer tests (CPTs). The borings were excavated to depths ranging from 6 to 50½ feet below the ground surface. The CPTs were advanced to depths ranging from approximately 60 to 64 feet below the ground surface. It should be noted that the numbers CPT-3 and CPT-4 were not used. 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 also obtained. The locations of the exploratory borings and CPTs are depicted on the Site Plan (Figure 2).

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 hollow-stem auger borings are presented on Figures A1 through A13. Plots of the CPT logs are presented on Figures A14 through A18. 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 boring logs were revised based on subsequent laboratory testing.

DRAFT

PROJECT NO. A9933-88-01

BORING 1

ELEV. (MSL.) -- DATE COMPLETED 02/12/2019

EQUIPMENT HOLLOW STEM AUGER BY: JS

DEPTH IN FEET	SAMPLE NO.	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT)*	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0	BULK 0-5'		**3" AC / 4" Base** **ARTIFICIAL FILL** Silty Sand, medium dense, moist, light brown to grayish brown, fine-grained.			
2	B1@2.5'			30	102.2	9.7
4	B1@5'	ML	**ALLUVIAL FAN DEPOSITS** Sandy Silt, soft, moist, mottled brown, orange, and gray, fine-grained.	7	92.3	25.8
6			Clay, firm, moist, mottled brown, gray, and dark brown.			
8	B1@7.5'		- dark brown	12	90.4	41.9
10	B1@10'		- soft, grayish brown	11	95.9	36.0
12	B1@12.5'	CL	- dark gray, some mottled light gray, trace fine-grained shells	3	76.2	57.4
14	B1@15'		- moist to wet, gray	7	91.2	44.0
16						
18						
20	B1@20'	CL	Sandy Clay, firm, moist, mottled dark gray and blue, fine- to medium-grained, roots.	21	125.4	19.2
22						
24	B1@25'	SP	Sand, poorly graded, medium dense, wet to saturated, brown, fine- to medium-grained.	42	108.6	19.7
26						
28						

DRAFT

A9933-88-01 BORING LOGS.GPJ

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

SAMPLE SYMBOLS

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

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

GEOCON

PROJECT NO. A9933-88-01

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	BORING 1 ELEV. (MSL.) -- DATE COMPLETED 02/12/2019 EQUIPMENT HOLLOW STEM AUGER BY: JS	PENETRATION RESISTANCE (BLOWS/FT)*	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
					MATERIAL DESCRIPTION			
30	B1@30'				- some coarse-grained	38	107.0	21.3
					Total depth of boring: 30.5 feet. Fill to 5 feet. Groundwater encoutered at 16.3 feet. Backfilled with cement grout. Penetration resistance for 140-pound hammer falling 30 inches by auto-hammer.			

DRAFT

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

A9933-88-01 BORING LOGS.GPJ

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

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

GEOCON

PROJECT NO. A9933-88-01

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	BORING 2 ELEV. (MSL.) -- DATE COMPLETED 02/12/2019 EQUIPMENT HOLLOW STEM AUGER BY: JS MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT)*	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0	BULK 0-5'				**5.5" AC / 3" Base** **ARTIFICIAL FILL** Silty Sand, medium dense, slightly moist, brown mottled with orange, fine-grained, trace clay.			
2								
	B2@3'				Sand and Silt, medium dense and firm, moist, brown mottled with orange, fine-grained.	19	78.8	30.9
4								
6	B2@6'			ML	**ALLUVIAL FAN DEPOSITS** Silt, very soft, moist to wet, brown, water seepage.	2	95.0	37.5
8					Clay, soft, moist, bluish gray.			
	B2@9'					5	71.3	43.2
10								
12	B2@12'					5	80.2	41.8
14								
	B2@15'		▼		- some roots, organic odor	5	50.8	73.6
16				CL				
18								
20	B2@20'				- very dark gray mottled with gray	7	76.3	63.4
22								
24								
	B2@25'			CL	Sandy Clay and Sand, stiff and medium dense, moist, gray, fine- to coarse-grained.	30	98.8	24.5
26								
				SW	Sand, well-graded, dense, moist to wet, light brown.			
28	B2@28'					62	112.6	17.7
					Total depth of boring: 28.5 feet. Fill to 5 feet.			

DRAFT

A9933-88-01 BORING LOGS.GPJ

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.

GEOCON

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	**BORING 2** ELEV. (MSL.) **--** DATE COMPLETED **02/12/2019** EQUIPMENT **HOLLOW STEM AUGER** BY: **JS**	PENETRATION RESISTANCE (BLOWS/FT)*	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
					MATERIAL DESCRIPTION			
					Groundwater encountered at 16 feet. Backfilled with cement grout. Surface restored. Penetration resistance for 140-pound hammer falling 30 inches by auto-hammer.			

DRAFT

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

A9933-88-01 BORING LOGS.GPJ

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

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

BORING 3

ELEV. (MSL.) -- DATE COMPLETED 02/25/2019

EQUIPMENT HOLLOW STEM AUGER BY: JS

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT)*	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0				CL	**GRASS** **ARTIFICIAL FILL** Sandy Clay, soft, moist, grayish brown, fine-grained, rootlets.			
2	B3@2.5'			SC	**ALLUVIAL FAN DEPOSITS** Clayey Sand, medium dense, moist, brown, fine-grained.	18		19.5
4	B3@5'			CL	Silty Clay, soft, moist, brown.	8	83.2	35.7
6				CL	Clay, soft, moist, gray, abundant roots/peat moss.			
8	B3@7.5'					5		50.7
10	B3@10'				- dark grayish brown	8	73.9	47.0
12	B3@12.5'				- trace roots	8		35.5
14	B3@15'				- trace fine gravel or carbonate deposits	10	106.1	18.8
16								
18	B3@17.5'				- some fine-grained sand with trace coarse-grained	11		19.7
20	B3@20'			CL	Sandy Clay, firm, moist, gray mottled with dark gray/brown, fine-grained with medium-grained.	14	117.2	19.9
			▼	ML	Silt, firm, moist to wet, gray mottled with brown.			
22	B3@22.5'					13		27.7
24	B3@25'				- stiff, brown	25	101.2	26.0
26				SP	Sand, poorly graded, medium dense, moist, yellowish brown, fine- to medium-grained.			
28	B3@27.5'					25		20.4

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

A9933-88-01 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.

DRAFT

BORING 3

ELEV. (MSL.) -- DATE COMPLETED 02/25/2019

EQUIPMENT HOLLOW STEM AUGER BY: JS

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'			SW	Sand, well graded, medium dense, moist to wet, yellowish brown.	33	112.7	16.6
32	B3@32.5'				- no recovery, slough at bottom of auger.			
34								
36	B3@35'				- medium dense	39	109.8	15.6
38	B3@37.5'				- some clay, very dense, moist	50 (6")		18.2
40	B3@40'			CH	Clay, hard, moist, brown, some fine-grained sand.	50 (4")	102.8	20.0
42	B3@42.5'				- firm, moist, trace fine-grained sand	12		32.2
44								
46	B3@45'			CL	Clay with Sand, stiff, moist, olive brown, medium-grained.	34	105.6	21.1
48	B3@47.5'			CL	Sandy Clay, stiff, slightly moist, olive brown, fine- to medium-grained, trace carbonate deposits.	25		14.2
50	B3@50'				- firm	17	104.9	22.2

Total depth of boring: 50.5 feet.
Fill to 2.5 feet.
Groundwater encountered at 21 feet.
Backfilled with cement grout.
Surface restored.
Penetration resistance for 140-pound hammer falling 30 inches by auto-hammer.

DRAFT

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

A9933-88-01 BORING LOGS.GPJ

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

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

BORING 4

ELEV. (MSL.) -- DATE COMPLETED 02/25/2019

EQUIPMENT HOLLOW STEM AUGER BY: JS

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT)*	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0					**GRASS** **ALLUVIAL FAN DEPOSITS** Sandy Silt, firm, moist, olive brown, fine-grained, roots.			
3	B4@3'			SM		20	100.5	17.6
6	B4@6'				Clay, soft, moist, dark gray.	9	84.4	31.8
9	B4@9'				- mottled bluish gray/very dark gray, some roots	9	63.5	59.6
12	B4@12'				- dark gray, decrease in root content	9	75.7	48.7
13				CL				
15	B4@15'				- bluish gray, trace sand	10	92.9	28.2
18			▼					
20	B4@20'				- stiff	31	111.9	17.6
20.5					Sand, well graded, dense, moist to wet, olive brown.			
23				SW				
25	B4@25'				Sand and Silt, well graded, dense and hard, moist, olive brown.	66	111.5	20.7
27.5				SW-SM				

DRAFT

A9933-88-01 BORING LOGS.GPJ

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

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

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

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	**BORING 4** ELEV. (MSL.) **--** DATE COMPLETED **02/25/2019** EQUIPMENT **HOLLOW STEM AUGER** BY: **JS** MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT)*	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
30	B4@30'			SP	Sand, poorly graded, loose, wet, olive brown, fine- to medium-grained.	10	97.0	19.9
					Total depth of boring: 30.5 feet. No fill. Groundwater encountered at 18.5 feet. Backfilled with cement grout. Penetration resistance for 140-pound hammer falling 30 inches by auto-hammer.			

DRAFT

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

A9933-88-01 BORING LOGS.GPJ

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

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

BORING 5

ELEV. (MSL.) -- DATE COMPLETED 02/26/2019

EQUIPMENT HOLLOW STEM AUGER BY: JS

DEPTH IN FEET	SAMPLE NO.	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT)*	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0		CL	**GRASS** **ALLUVIAL FAN DEPOSITS** Sandy Clay, firm, moist, olive brown, fine-grained.			
2	B5@2.5'	ML	Sandy Silt, firm, moist, olive brown, fine-grained.	20	102.6	23.8
4	B5@5'	ML	Silt, soft, slightly moist to moist, dark olive brown.	12	91.8	26.2
6			- grounwater seepage			
8	B5@7.5'	SM	Silty Sand, loose, moist to wet, olive brown, fine- to medium-grained.	7	83.9	32.8
		CL-ML	Clayey Silt, soft, olive brown, moist.			
10	B5@10'	CL	Clay, soft, wet, mottled gray/black, abundant roots, moss.	5	59.5	71.2
12	B5@12.5'		- decrease in roots	5	55.6	76.9
14	B5@15'		- increase in roots, organic odor	6	64.7	61.3
16						
18						
20	B5@20'		- gray	5	88.6	33.9
22						
24		SP	Sand, poorly graded, medium dense, moist to wet, grayish brown, fine- to medium-grained, some silt.			
26	B5@25'			33	110.1	19.2
28						

DRAFT

A9933-88-01 BORING LOGS.GPJ

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

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

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

PROJECT NO. A9933-88-01

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	BORING 5 ELEV. (MSL.) -- DATE COMPLETED 02/26/2019 EQUIPMENT HOLLOW STEM AUGER BY: JS	PENETRATION RESISTANCE (BLOWS/FT)*	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
					MATERIAL DESCRIPTION			
30	B5@30'			SP		33	101.8	23.9
					Total depth of boring: 30.5 feet. No fill. Groundwater seepage at 7 feet. Groundwater encountered at 10 feet. Backfilled with grout. Surface restored. Penetration resistance for 140-pound hammer falling 30 inches by auto-hammer.			

DRAFT

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

A9933-88-01 BORING LOGS.GPJ

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

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

GEOCON

BORING 6

ELEV. (MSL.) -- DATE COMPLETED 02/12/2019

EQUIPMENT HOLLOW STEM AUGER BY: JS

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'				**3" AC / 6" Base**			
					ARTIFICIAL FILL			
					Sandy Silt, firm, moist, brown, fine-grained.			
2	B6@2.5'					8	96.8	19.1
4					Clay and Sand, firm to loose, moist, dark brown, coarse-grained sand.			
	B6@5'					12	87.1	38.7
6					**ALLUVIAL FAN DEPOSITS**			
					Clay, firm, moist, dark brown.			
8	B6@7.5'				- dark brown mottled with grayish brown	12	90.3	30.1
10	B6@10'				- soft, abundant roots, grayish brown	11	74.7	39.9
12				CL				
14								
16	B6@15'				- firm, abundant roots, some medium- to coarse-grained sand	16	105.0	19.1
18								
			▼	CL	Sandy Clay, firm, moist, dark grayish brown, medium- to coarse-grained.			
20	B6@20'					16	99.9	22.3
				SP	Sand, poorly graded, loose, moist, dark brown, medium- to coarse-grained.			
22								
24					Silt, firm, moist, light grayish brown, trace hard clasts.			
	B6@25'			ML		15	104.7	21.7
26								
28				SW	Sand, well graded, medium dense, wet to saturated, dark yellowish brown.			

A9933-88-01 BORING LOGS.GPJ

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

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

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

DRAFT

GEOCON

BORING 6

ELEV. (MSL.) -- DATE COMPLETED 02/12/2019

EQUIPMENT HOLLOW STEM AUGER BY: JS

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT)*	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
30	B6@30'					26	101.2	21.3
					Total depth of boring: 30.5 feet. Fill to 5.5 feet. Groundwater encountered at 19 feet. Backfill with cement grout. Surface restored. Penetration resistance for 140-pound hammer falling 30 inches by auto-hammer.			

DRAFT

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

A9933-88-01 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. A9933-88-01

BORING 7

ELEV. (MSL.) **--** DATE COMPLETED **02/12/2019**

EQUIPMENT **HOLLOW STEM AUGER** BY: **JS**

DEPTH IN FEET	SAMPLE NO.	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT)*	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0	BULK 0-5'		**3" AC / 8" Base** **ARTIFICIAL FILL** Sandy Silt, soft to firm, moist, brown, fine-grained.			
2	B7@2.5'	CL	**ALLUVIAL FAN DEPOSITS** Clay, soft, moist, very dark gray.	6		30.3
4		ML	Silt, soft, moist, brown.			
	B7@5'			10	82.0	38.6
6		CL	Clay, soft, slightly moist, brown, some fine- to coarse-grained sand.			
8	B7@7.5'			8		32.9
		ML	Silt, firm, moist, mottled brown/orange.			
10	B7@10'			15	91.6	29.0
12	B7@12.5'	CH	Clay, very soft, moist, very dark gray.	2		60.9
14	B7@15'		- soft, slightly moist, gray	9	91.9	31.6
16						
18	B7@17.5	CL	Clay, very soft, some roots, blush gray, organic odor.	2		27.1
20	B7@20'	CL	Sandy Clay, soft, slightly moist, bluish gray, fine-grained.	11	106.3	21.1
22	B7@22.5'			4		20.0
		SM	Silty Sand, loose, moist, brown, fine-grained, trace medium-grained.			
24	B7@25'	CL	Sandy Clay, firm, moist to wet, brown mottled with gray/orange, trace fine-grained hard clasts, fine- to medium-grained sand.	17	102.6	22.3
26						
28	B7@27.5'	ML	Silt, stiff, moist, brown.	27		26.5
		SW	Sand, well graded, mediumd dense, wet to saturated, light brown. Note: heaving			

DRAFT

A9933-88-01 BORING LOGS.GPJ

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

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. A9933-88-01

BORING 7

ELEV. (MSL.) **--** DATE COMPLETED **02/12/2019**

EQUIPMENT **HOLLOW STEM AUGER** BY: **JS**

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'			SW		25	114.6	15.3
32	B7@32.5'				- decrease in coarse-grained, dense	44		17.4
34								
	B7@35'				- well graded, very dense, moist to wet, yellowish brown	50(6")	111.9	17.5
36								
38	B7@37.5'				- decrease in coarse-grained, dense, olive brown	49		21.5
40	B7@40'					50(4")		20.2
42				SP	Sand, poorly graded, medium dense, moist to wet, olive brown, medium- to coarse-grained.			
	B7@42.5'					29		17.9
44				ML	Silt, moist, olive brown mottled with orange.			
	B7@45'				- hard	50	102.5	22.7
46				CH	Clay, firm, moist, yellowish brown.			
48	B7@47.5'					12		32.9
50	B7@50'				- stiff, some medium- to coarse-grained sand	35	91.5	33.4

Total depth of boring: 50.5 feet.
FIll to 2.5 feet.
Groundwater encountered at 17.5 feet.
Backfilled with grout.
Surface restored.
Penetration resistance for 140-pound hammer falling 30 inches by auto-hammer.

DRAFT

A9933-88-01 BORING LOGS.GPJ

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

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. A9933-88-01

BORING P1

ELEV. (MSL.) -- DATE COMPLETED 02/26/2019

EQUIPMENT HOLLOW STEM AUGER BY: JS

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT)*	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0				SC	**3" AC / 6" Base** **ALLUVIAL FAN DEPOSITS** Clayey Sand, medium dense, slightly moist, dark olive brown, fine-grained.			
2	P1@2.5'			SP	Sand, poorly graded, loose, slightly moist, pale brown, fine-grained.	6		12.3
4	P1@5'			SM	Silty Sand, loose, moist, olive brown, fine-grained.	5		28.3
6				CL	Clay, soft, moist, very dark gray.			

Total depth of boring: 6 feet.
No Fill.
No groundwater encountered.
Percolation testing performed.
Surface resotred.
Penetration resistance for 140-pound hammer falling 30 inches by auto-hammer.

DRAFT

Figure A8,
Log of Boring P1, Page 1 of 1

A9933-88-01 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. A9933-88-01

BORING P2

ELEV. (MSL.) -- DATE COMPLETED 02/26/2019

EQUIPMENT HOLLOW STEM AUGER BY: JS

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					**ARTIFICIAL FILL** Silty Sand, medium dense, slightly moist, olive brown, fine-grained.			
2 – 4	P2@2.5'				Sand, poorly graded, loose, moist, olive brown, fine- to medium-grained, some silt.	8		33.2
4 – 6	P2@5'			CL	**ALLUVIAL FAN DEPOSITS** Clay, soft, moist, mottled grayish brown/very dark gray, roots, oxidation.	5		41.5

Total depth of boring: 6 feet.
Fill to 4 feet.
No groundwater encountered.
Percolation testing performed.
Surface restored.
Penetration resistance for 140-pound hammer falling 30 inches by auto-hammer.

DRAFT

Figure A9,
Log of Boring P2, Page 1 of 1

A9933-88-01 BORING LOGS.GPJ

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

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

GEOCON

PROJECT NO. A9933-88-01

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	BORING P3 ELEV. (MSL.) -- DATE COMPLETED 02/26/2019 EQUIPMENT HOLLOW STEM AUGER BY: JS	PENETRATION RESISTANCE (BLOWS/FT)*	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
					MATERIAL DESCRIPTION			
0					**GRASS** **ALLUVIAL FAN DEPOSITS**			
2				CL	Sandy Clay, firm, moist, dark olive brown, fine-grained.			
	P3@2.5'					10		23.9
4				SM	Silty Sand, loose, moist, dark olive brown, fine-grained.			
	P3@5'			ML	Sandy Silt, soft, moist, dark olive brown mottled with pale brown, trace roots.	8		24.7
6					Total depth of boring: 6 feet. No fill. No goundwater encountered. Percolation testing performed. Surface restored. Penetration resistance for 140-pound hammer falling 30 inches by auto-hammer.			

DRAFT

Figure A10,
Log of Boring P3, Page 1 of 1

A9933-88-01 BORING LOGS.GPJ

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

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

GEOCON

BORING P4

ELEV. (MSL.) -- DATE COMPLETED 02/26/2019

EQUIPMENT HOLLOW STEM AUGER BY: JS

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT)*	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0					**GRASS**			
					ALLUVIAL FAN DEPOSITS			
				ML	Sandy Silt, firm, moist, olive brown, fine-grained.			
2					Silty Sand, loose, moist, olive brown, fine-grained, trace roots.			
	P4@2.5'					9		17.9
4				SM				
	P4@5'				- medium dense	11		24.9
6								

Total depth of boring: 6 feet.
No fill.
No groundwater encountered.
Percolation testing performed.
Surface restored.
Penetration resistance for 140-pound hammer falling 30 inches by auto-hammer.

DRAFT

Figure A11,
Log of Boring P4, Page 1 of 1

A9933-88-01 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. A9933-88-01

BORING P5

ELEV. (MSL.) -- DATE COMPLETED 02/26/2019

EQUIPMENT HOLLOW STEM AUGER BY: JS

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT)*	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0					**3" AC / 6" Base**			
					ARTIFICIAL FILL Silty Sand, very loose, moist, brown, fine- to medium-grained.			
2								
	P5@2.5'					6		25.9
4					**ALLUVIAL FAN DEPOSITS** Clay, soft, moist to wet, mottled gray/black, roots.			
	P5@5'			CL		4		59.0
6								

Total depth of boring: 6 feet.
Fill to 3.5 feet.
No groundwater encountered.
Percolation testing performed.
Surface restored.
Penetration resistance for 140-pound hammer falling 30 inches by auto-hammer.

DRAFT

Figure A12,
Log of Boring P5, Page 1 of 1

A9933-88-01 BORING LOGS.GPJ

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

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

GEOCON

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	**BORING P6** ELEV. (MSL.) **--** DATE COMPLETED **02/26/2019** EQUIPMENT **HOLLOW STEM AUGER** BY: **JS**	PENETRATION RESISTANCE (BLOWS/FT)*	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
					MATERIAL DESCRIPTION			
0					**3" AC / 6" Base** **ARTIFICIAL FILL** Sandy Silt, firm, moist, bronw, fine-grained.			
2	P6@2.5'			ML	**ALLUVIAL FAN DEPOSITS** Silt, soft, moist, mottled gray/brown.	5		36.3
4	P6@5'			CL	Clay, soft, moist, mottled grayish brown/very dark gray, some roots.	8		46.4
6					Total depth of boring: 6 feet. Fill to 2 feet. No groundwater encountered. Percolation testing performed. Surface restored. Penetration resistance for 140-pound hammer falling 30 inches by auto-hammer.			

DRAFT

A9933-88-01 BORING LOGS.GPJ

Figure A13,
Log of Boring P6, 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.

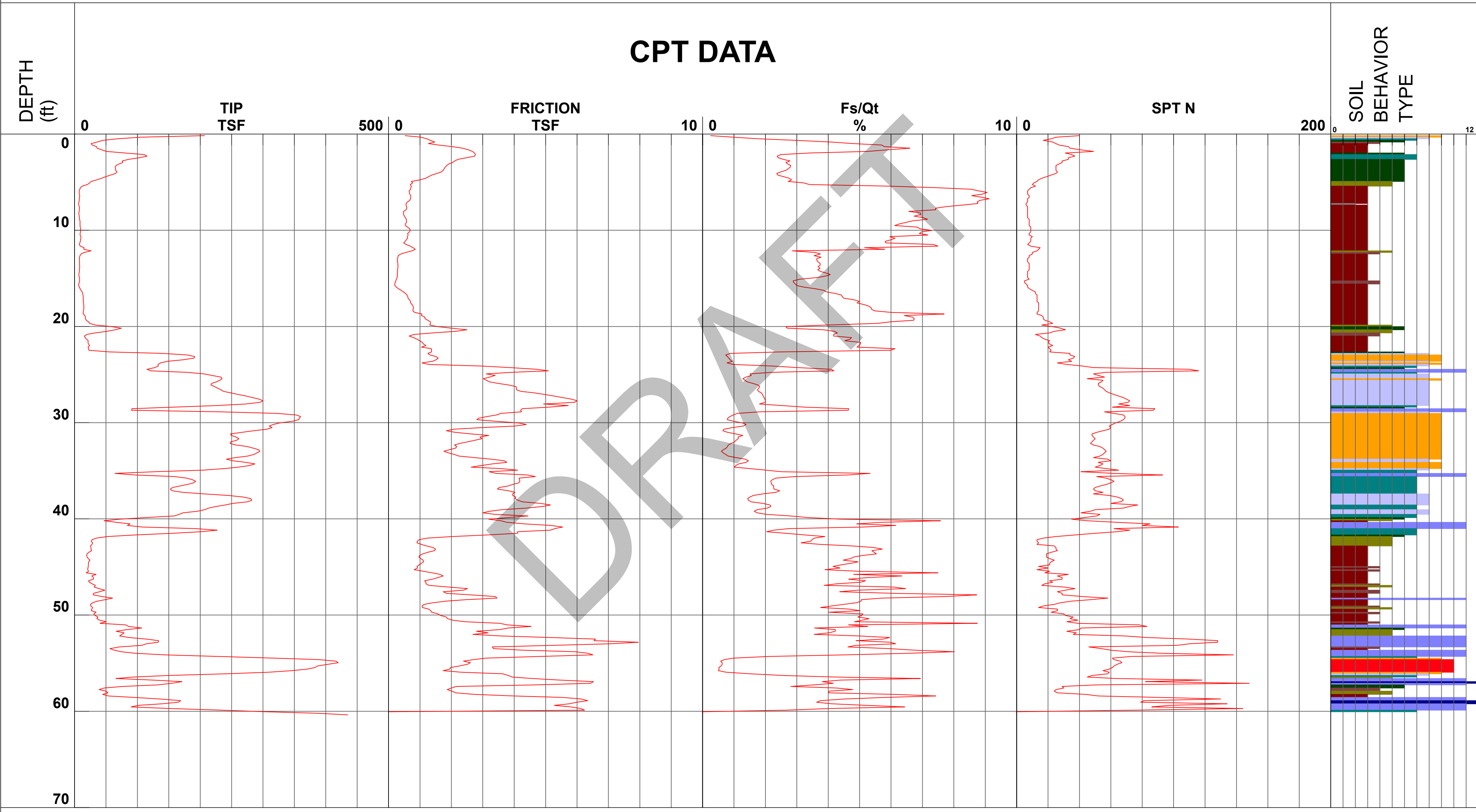
A9933-88-01

Geocon Inc.

Project	1687 Sunflower	Operator	RC RH	Filename	SDF(416).cpt
Job Number	A9933-88-01	Cone Number	DDG1471	GPS	
Hole Number	CPT-01	Date and Time	2/11/2019 12:15:34 PM	Maximum Depth	60.37 ft
EST GW Depth During Test		11.00 ft			

Net Area Ratio .8

CPT DATA



1 - sensitive fine grained
2 - organic material
3 - clay
4 - silty clay to clay
5 - clayey silt to silty clay
6 - sandy silt to clayey silt
7 - silty sand to sandy silt
8 - sand to silty sand
9 - sand
10 - gravelly sand to sand
11 - very stiff fine grained (*)
12 - sand to clayey sand (*)

Cone Size 15cm squared

S*Soil behavior type and SPT based on data from UBC-1983

Figure A-14

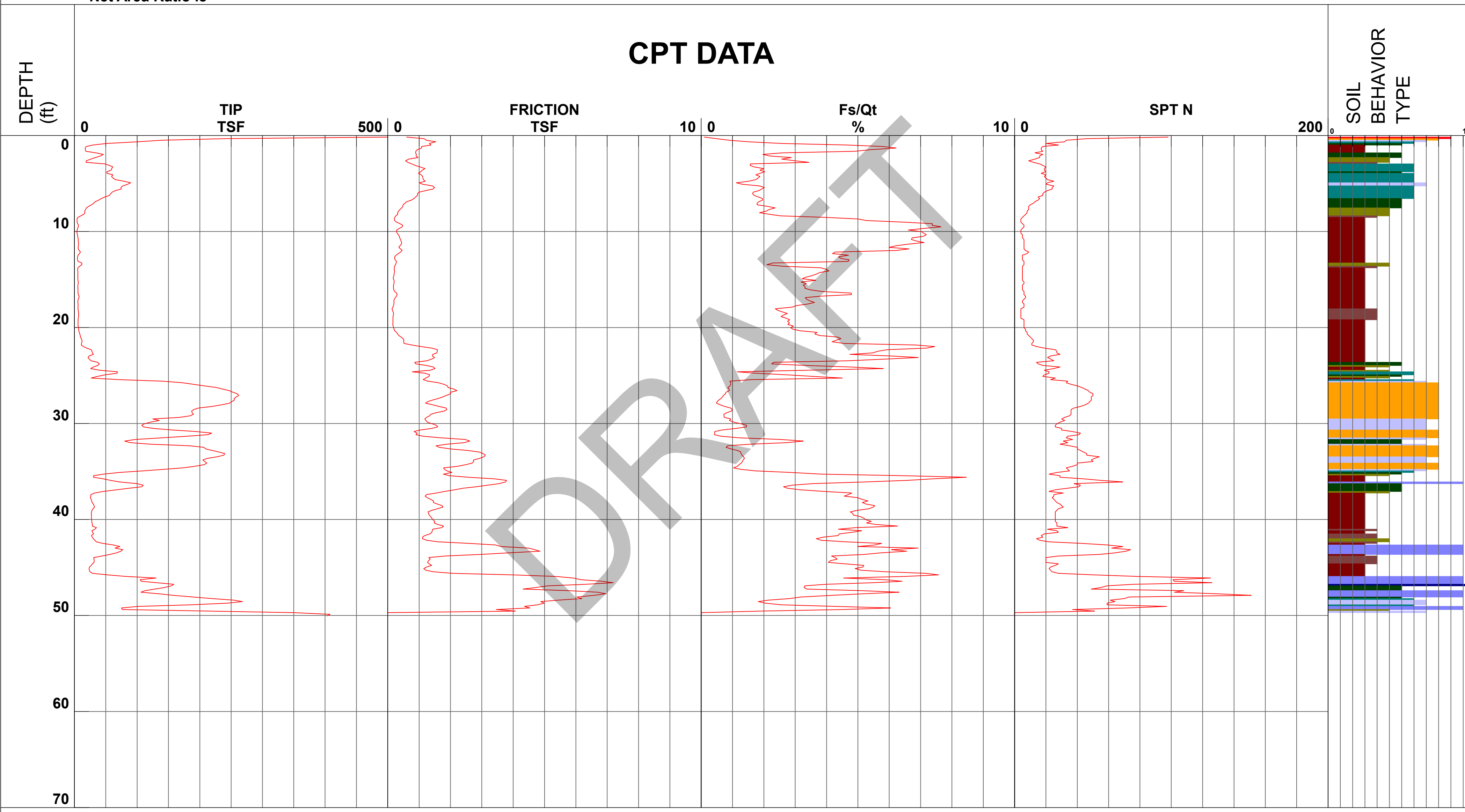
A9933-88-01

Geocon Inc.

Project	1687 Sunflower	Operator	RC RH	Filename	SDF(417).cpt
Job Number	A9933-88-01	Cone Number	DDG1471	GPS	
Hole Number	CPT-02	Date and Time	2/11/2019 1:15:46 PM	Maximum Depth	50.03 ft
EST GW Depth During Test		14.20 ft			

Net Area Ratio .8

CPT DATA



1 - sensitive fine grained
2 - organic material
3 - clay
4 - silty clay to clay
5 - clayey silt to silty clay
6 - sandy silt to clayey silt
7 - silty sand to sandy silt
8 - sand to silty sand
9 - sand
10 - gravelly sand to sand
11 - very stiff fine grained (*)
12 - sand to clayey sand (*)

Cone Size 15cm squared

S*Soil behavior type and SPT based on data from UBC-1983

Figure A-15

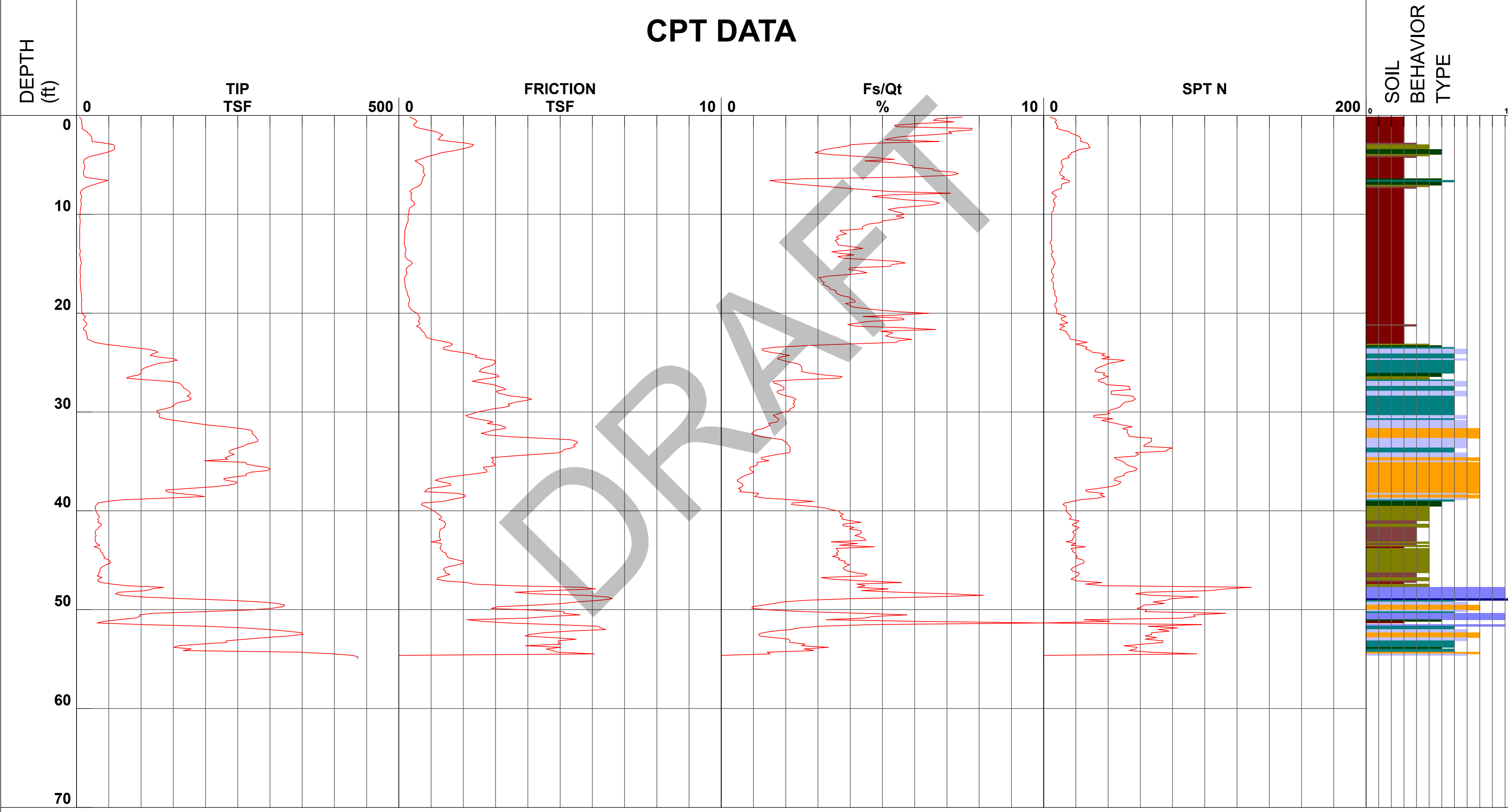
A9933-88-01

Geocon Inc.

Project	1687 Sunflower	Operator	RC RH	Filename	SDF(415).cpt
Job Number	A9933-88-01	Cone Number	DDG1471	GPS	
Hole Number	CPT-05	Date and Time	2/11/2019 10:32:05 AM	Maximum Depth	54.95 ft
EST GW Depth During Test		8.00 ft			

Net Area Ratio .8

CPT DATA



1 - sensitive fine grained
2 - organic material
3 - clay
4 - silty clay to clay
5 - clayey silt to silty clay
6 - sandy silt to clayey silt
7 - silty sand to sandy silt
8 - sand to silty sand
9 - sand
10 - gravelly sand to sand
11 - very stiff fine grained (*)
12 - sand to clayey sand (*)

Cone Size 15cm squared

S*Soil behavior type and SPT based on data from UBC-1983

Figure A-16

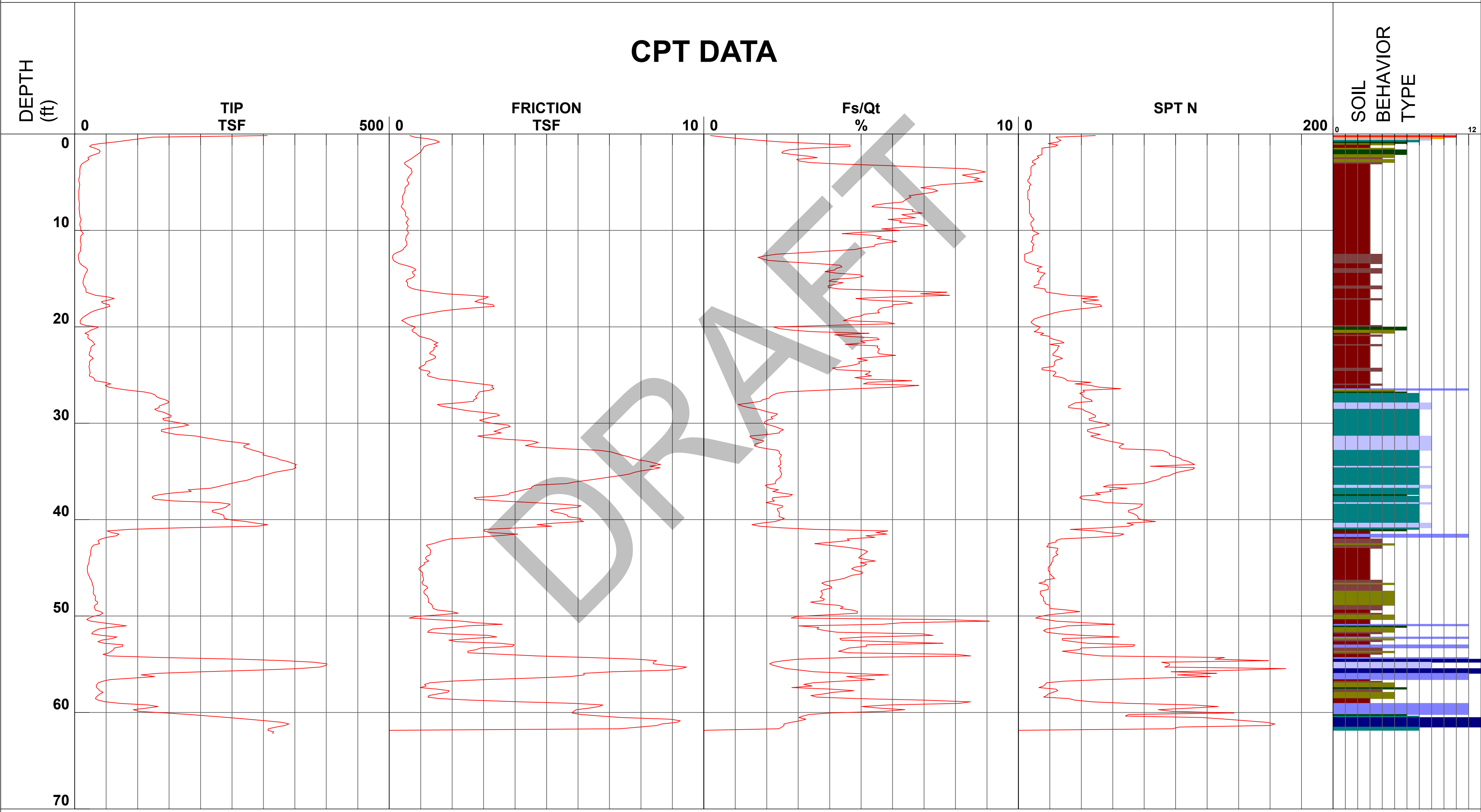
A9933-88-01

Geocon Inc.

Project	1687 Sunflower	Operator	RC RH	Filename	SDF(414).cpt
Job Number	A9933-88-01	Cone Number	DDG1471	GPS	
Hole Number	CPT-06	Date and Time	2/11/2019 8:46:39 AM	Maximum Depth	62.17 ft
EST GW Depth During Test		8.00 ft			

Net Area Ratio .8

CPT DATA



1 - sensitive fine grained
2 - organic material
3 - clay
4 - silty clay to clay
5 - clayey silt to silty clay
6 - sandy silt to clayey silt
7 - silty sand to sandy silt
8 - sand to silty sand
9 - sand
10 - gravelly sand to sand
11 - very stiff fine grained (*)
12 - sand to clayey sand (*)

Cone Size 15cm squared

S*Soil behavior type and SPT based on data from UBC-1983

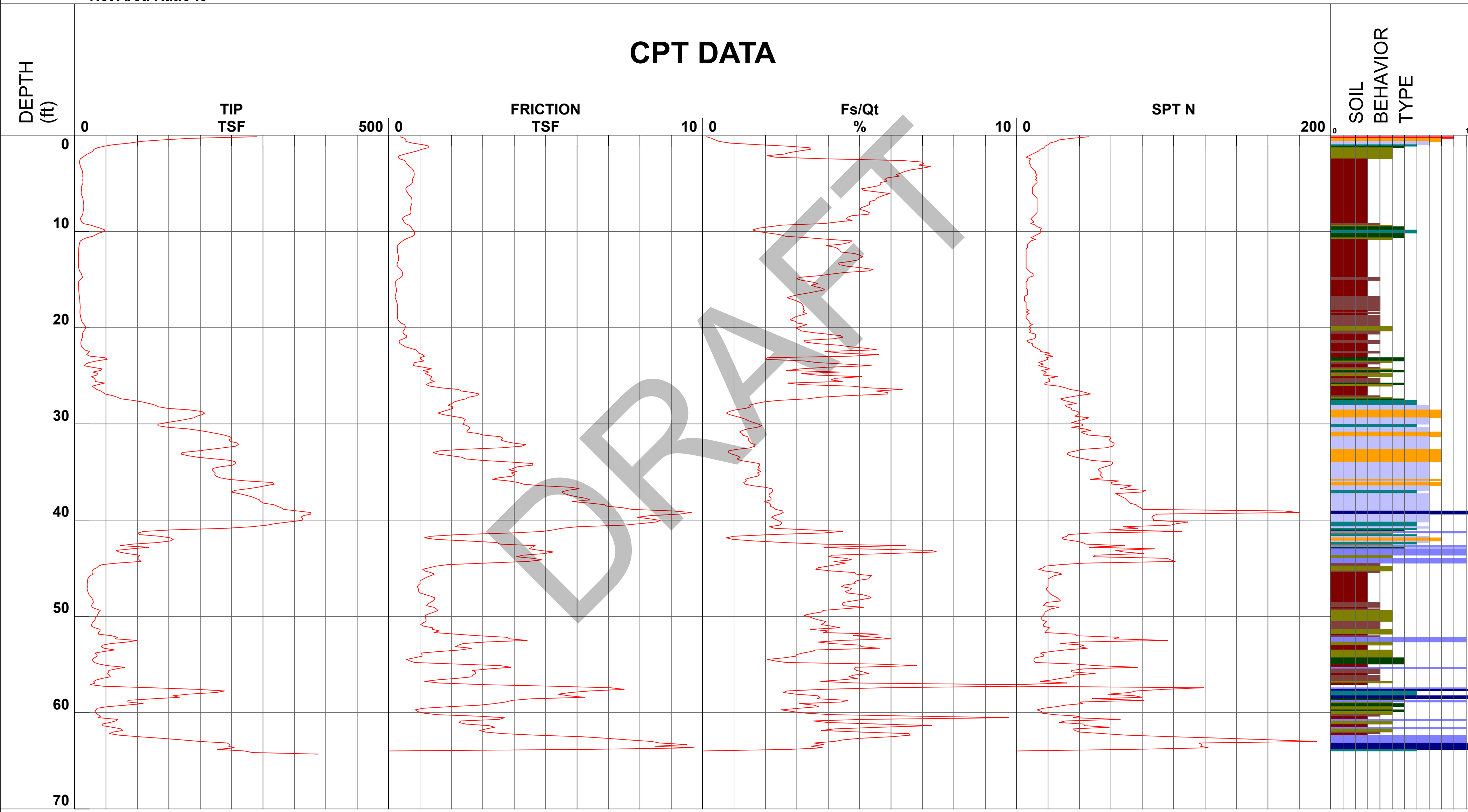
Figure A-17

A9933-88-01

Geocon Inc.

Project	1687 Sunflower	Operator	RC RH	Filename	SDF(413).cpt
Job Number	A9933-88-01	Cone Number	DDG1471	GPS	
Hole Number	CPT-07	Date and Time	2/11/2019 7:38:06 AM	Maximum Depth	64.30 ft
EST GW Depth During Test		15.80 ft			

Net Area Ratio .8



1 - sensitive fine grained	4 - silty clay to clay	7 - silty sand to sandy silt	10 - gravelly sand to sand
2 - organic material	5 - clayey silt to silty clay	8 - sand to silty sand	11 - very stiff fine grained (*)
3 - clay	6 - sandy silt to clayey silt	9 - sand	12 - sand to clayey sand (*)

Cone Size 15cm squared

S*Soil behavior type and SPT based on data from UBC-1983

Figure A-18

APPENDIX B

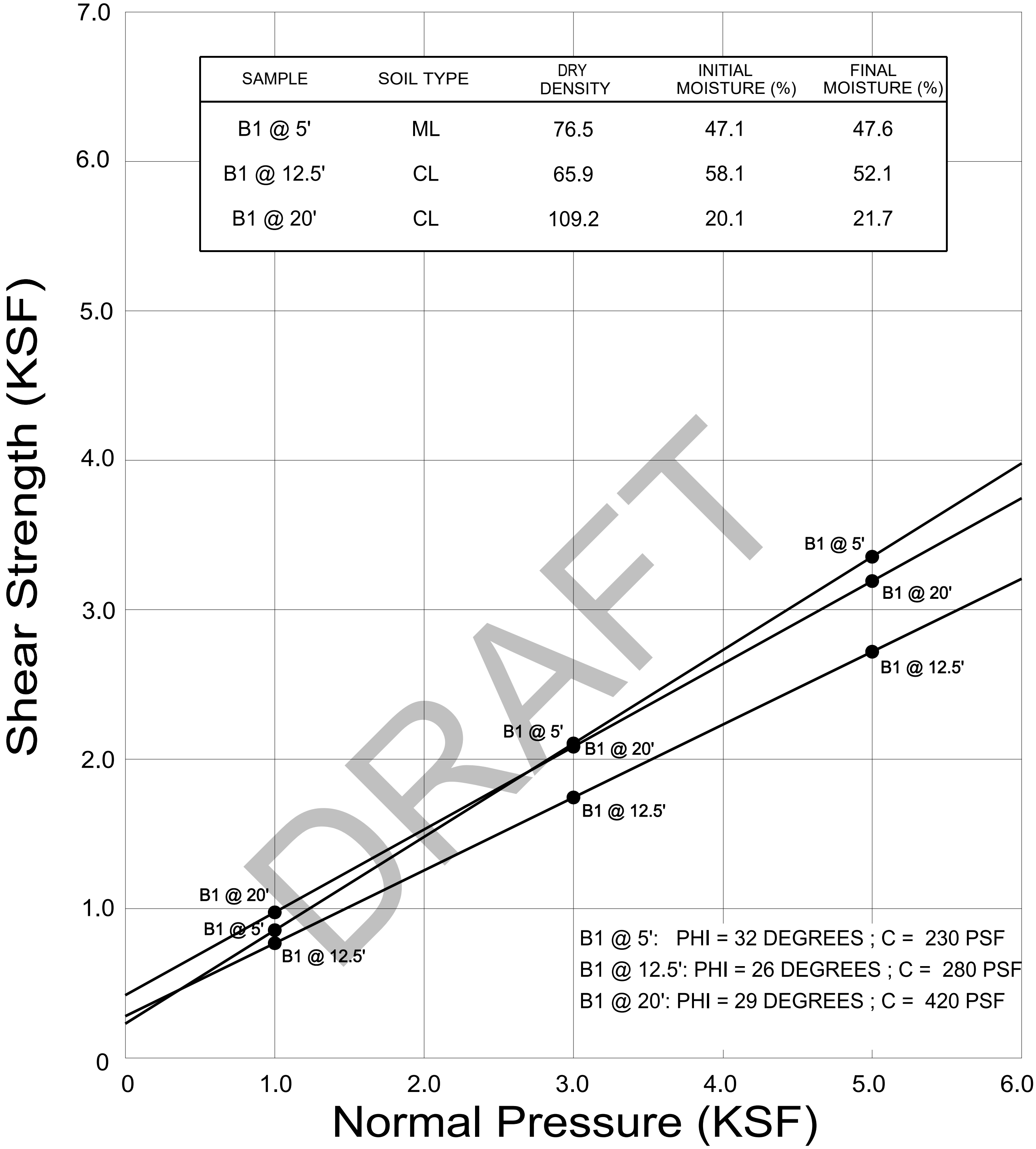
DRAFT

APPENDIX B

LABORATORY TESTING

Laboratory tests were performed in accordance with generally accepted test methods of the "American Society for Testing and Materials (ASTM)", or other suggested procedures. Selected samples were tested for direct shear strength, consolidation, grain size characteristics, Atterberg limits, expansion characteristics, moisture density relationships, corrosivity, and in-place dry density and moisture content. The results of the laboratory tests are summarized in Figures B1 through B21. The in-place dry density and moisture content of the samples tested are presented on the boring logs, Appendix A.

DRAFT



SAMPLE	SOIL TYPE	DRY DENSITY	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B1 @ 5'	ML	76.5	47.1	47.6
B1 @ 12.5'	CL	65.9	58.1	52.1
B1 @ 20'	CL	109.2	20.1	21.7

● Direct Shear, Saturated

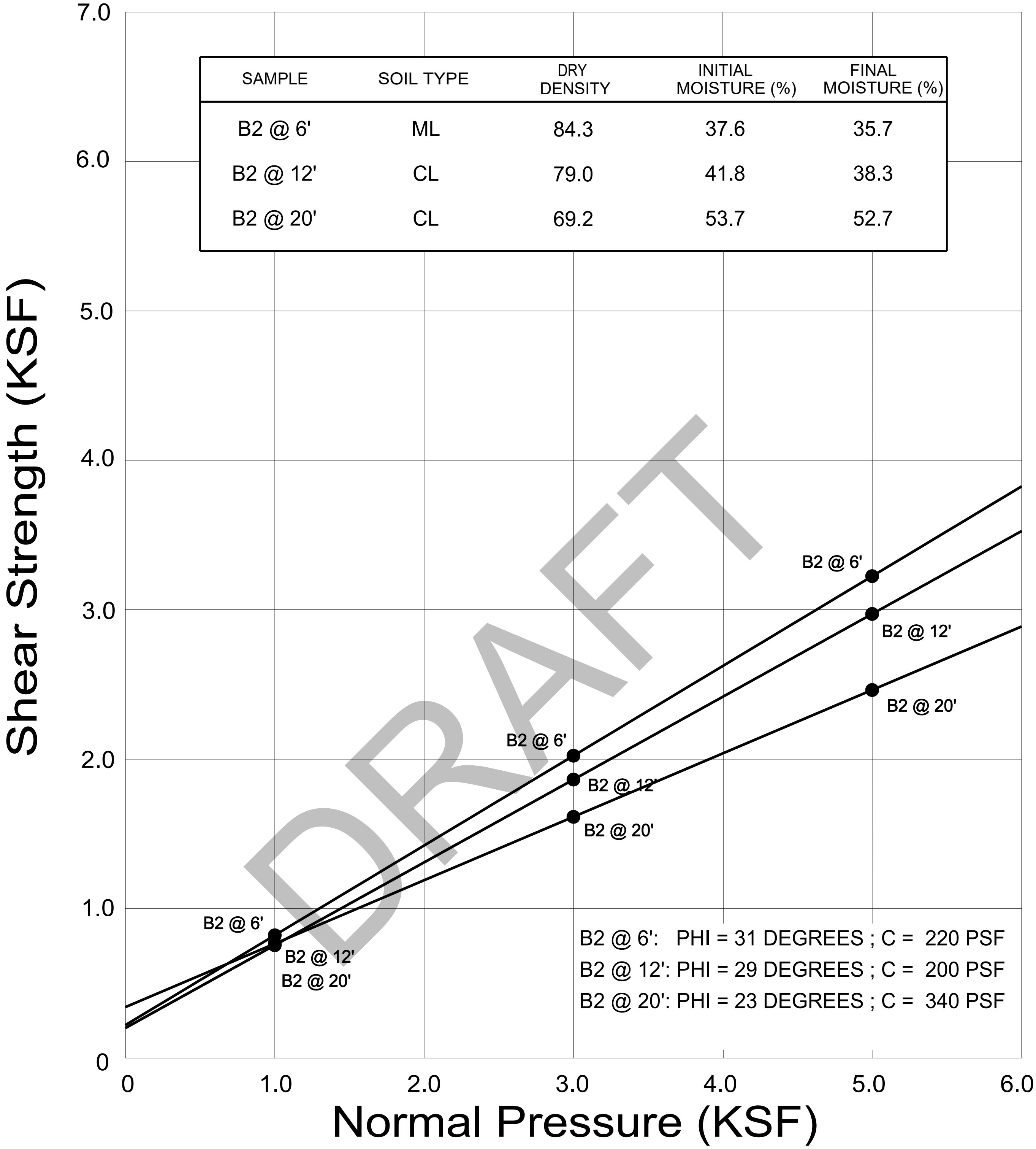
DRAFT

GEOCON
WEST, INC.
ENVIRONMENTAL GEOTECHNICAL MATERIALS
15520 ROCKFIELD BOULEVARD, SUITE J, IRVINE, CA 92618
PHONE (949) 491-6570 FAX (949) 299-4550

DIRECT SHEAR TEST RESULTS

ROSE EQUITIES
1683 SUNFLOWER AVENUE
COSTA MESA, CALIFORNIA

DRAFTED BY: JS	CHECKED BY: JTA	JULY 2019	PROJECT NO. A9933-88-01	FIG. B1



SAMPLE	SOIL TYPE	DRY DENSITY	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B2 @ 6'	ML	84.3	37.6	35.7
B2 @ 12'	CL	79.0	41.8	38.3
B2 @ 20'	CL	69.2	53.7	52.7

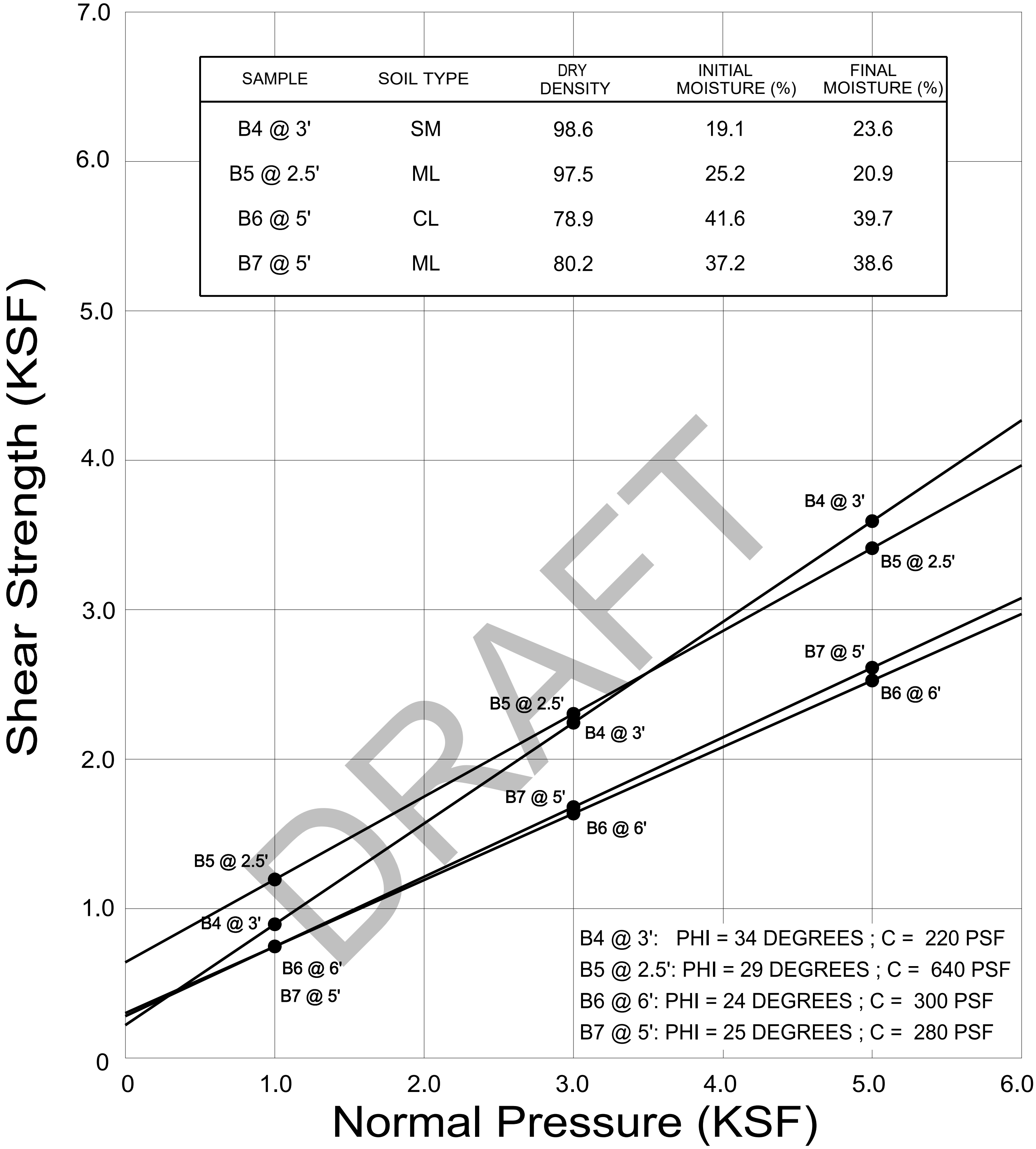
● Direct Shear, Saturated

GEOCON WEST, INC.
ENVIRONMENTAL GEOTECHNICAL MATERIALS
15520 ROCKFIELD BOULEVARD, SUITE J, IRVINE, CA 92618
PHONE (949) 491-6570 FAX (949) 299-4550

DIRECT SHEAR TEST RESULTS

ROSE EQUITIES
1683 SUNFLOWER AVENUE
COSTA MESA, CALIFORNIA

DRAFTED BY: JS	CHECKED BY: JTA	JULY 2019	PROJECT NO. A9933-88-01	FIG. B2



SAMPLE	SOIL TYPE	DRY DENSITY	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B4 @ 3'	SM	98.6	19.1	23.6
B5 @ 2.5'	ML	97.5	25.2	20.9
B6 @ 5'	CL	78.9	41.6	39.7
B7 @ 5'	ML	80.2	37.2	38.6

● Direct Shear, Saturated

GEOCON WEST, INC.

ENVIRONMENTAL GEOTECHNICAL MATERIALS

15520 ROCKFIELD BOULEVARD, SUITE J, IRVINE, CA 92618

PHONE (949) 491-6570 FAX (949) 299-4550

DRAFTED BY: JS | CHECKED BY: JTA

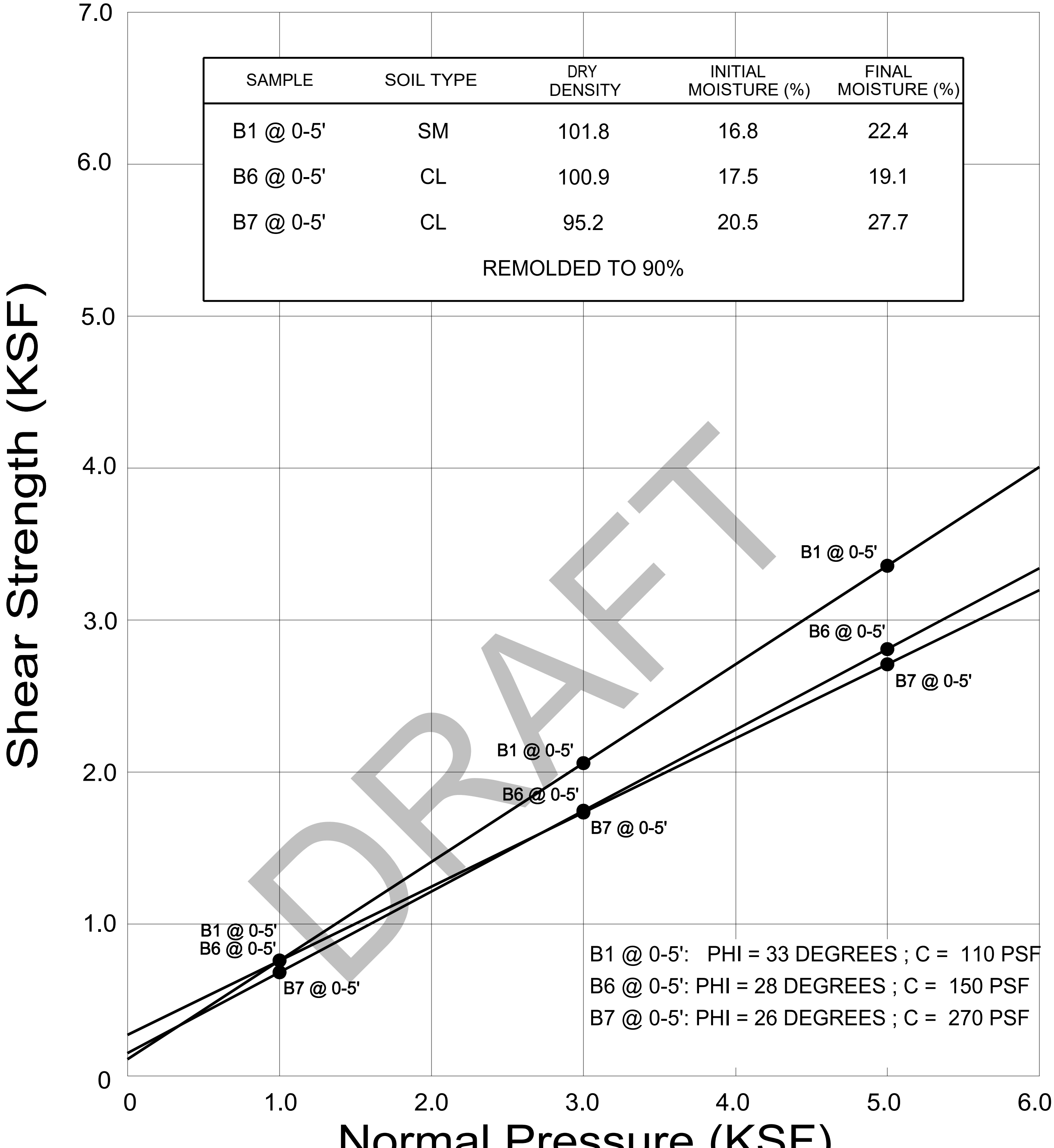
DIRECT SHEAR TEST RESULTS

ROSE EQUITIES

1683 SUNFLOWER AVENUE

COSTA MESA, CALIFORNIA

JULY 2019 | PROJECT NO. A9933-88-01 | FIG. B3



SAMPLE	SOIL TYPE	DRY DENSITY	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B1 @ 0-5'	SM	101.8	16.8	22.4
B6 @ 0-5'	CL	100.9	17.5	19.1
B7 @ 0-5'	CL	95.2	20.5	27.7

REMOLDED TO 90%

● Direct Shear, Saturated

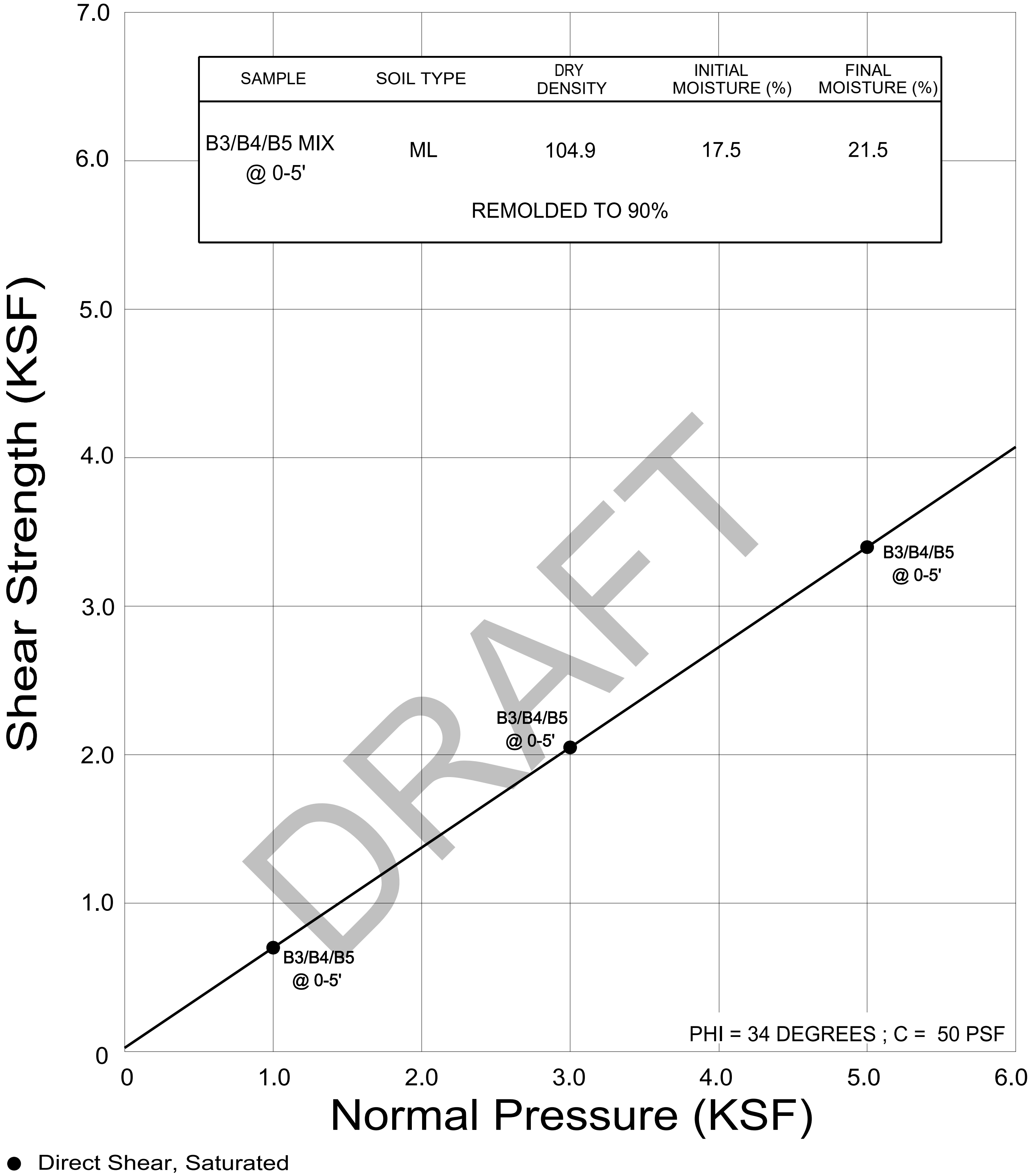
GEOCON WEST, INC.
ENVIRONMENTAL GEOTECHNICAL MATERIALS
15520 ROCKFIELD BOULEVARD, SUITE J, IRVINE, CA 92618
PHONE (949) 491-6570 FAX (949) 299-4550

DRAFTED BY: JS | CHECKED BY: JTA

DIRECT SHEAR TEST RESULTS

ROSE EQUITIES
1683 SUNFLOWER AVENUE
COSTA MESA, CALIFORNIA

JULY 2019 | PROJECT NO. A9933-88-01 | FIG. B4



GEOCON WEST, INC.

ENVIRONMENTAL GEOTECHNICAL MATERIALS

15520 ROCKFIELD BOULEVARD, SUITE J, IRVINE, CA 92618

PHONE (949) 491-6570 FAX (949) 299-4550

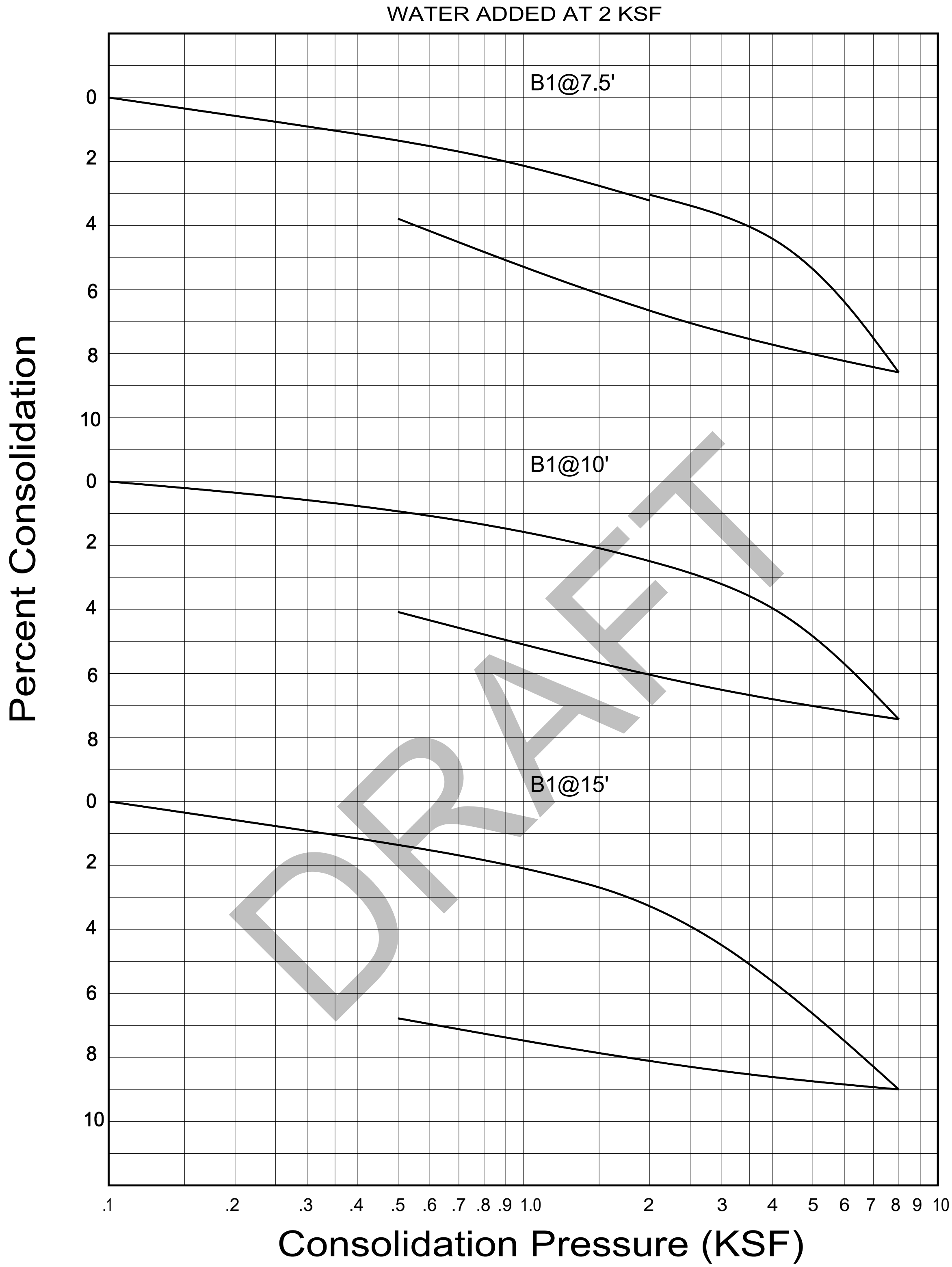
DIRECT SHEAR TEST RESULTS

ROSE EQUITIES

1683 SUNFLOWER AVENUE

COSTA MESA, CALIFORNIA

DRAFTED BY: JS	CHECKED BY: JTA	JULY 2019	PROJECT NO. A9933-88-01	FIG. B5



GEOCON
WEST, INC.
ENVIRONMENTAL GEOTECHNICAL MATERIALS
15520 ROCKFIELD BOULEVARD, SUITE J, IRVINE, CA 92618
PHONE (949) 491-6570 FAX (949) 299-4550

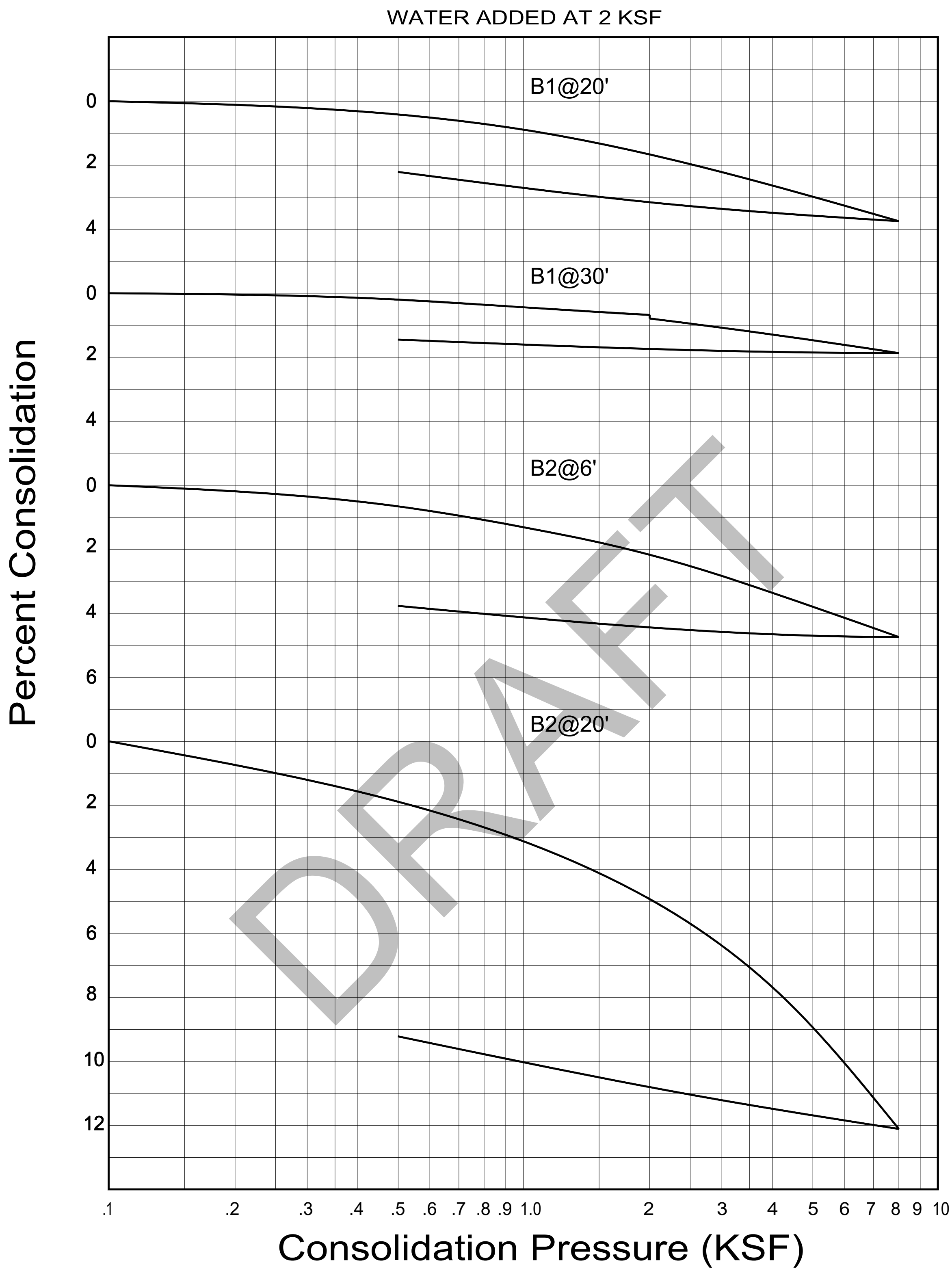
CONSOLIDATION TEST RESULTS

ROSE EQUITIES

1683 SUNFLOWER AVENUE

COSTA MESA, CALIFORNIA

Drafted by: JS	Checked by: JTA	JULY 2019	PROJECT NO. A9933-88-01	FIG. B6



GEOCON WEST, INC.

ENVIRONMENTAL GEOTECHNICAL MATERIALS

15520 ROCKFIELD BOULEVARD, SUITE J, IRVINE, CA 92618

PHONE (949) 491-6570 FAX (949) 299-4550

Drafted by: JS	Checked by: JTA

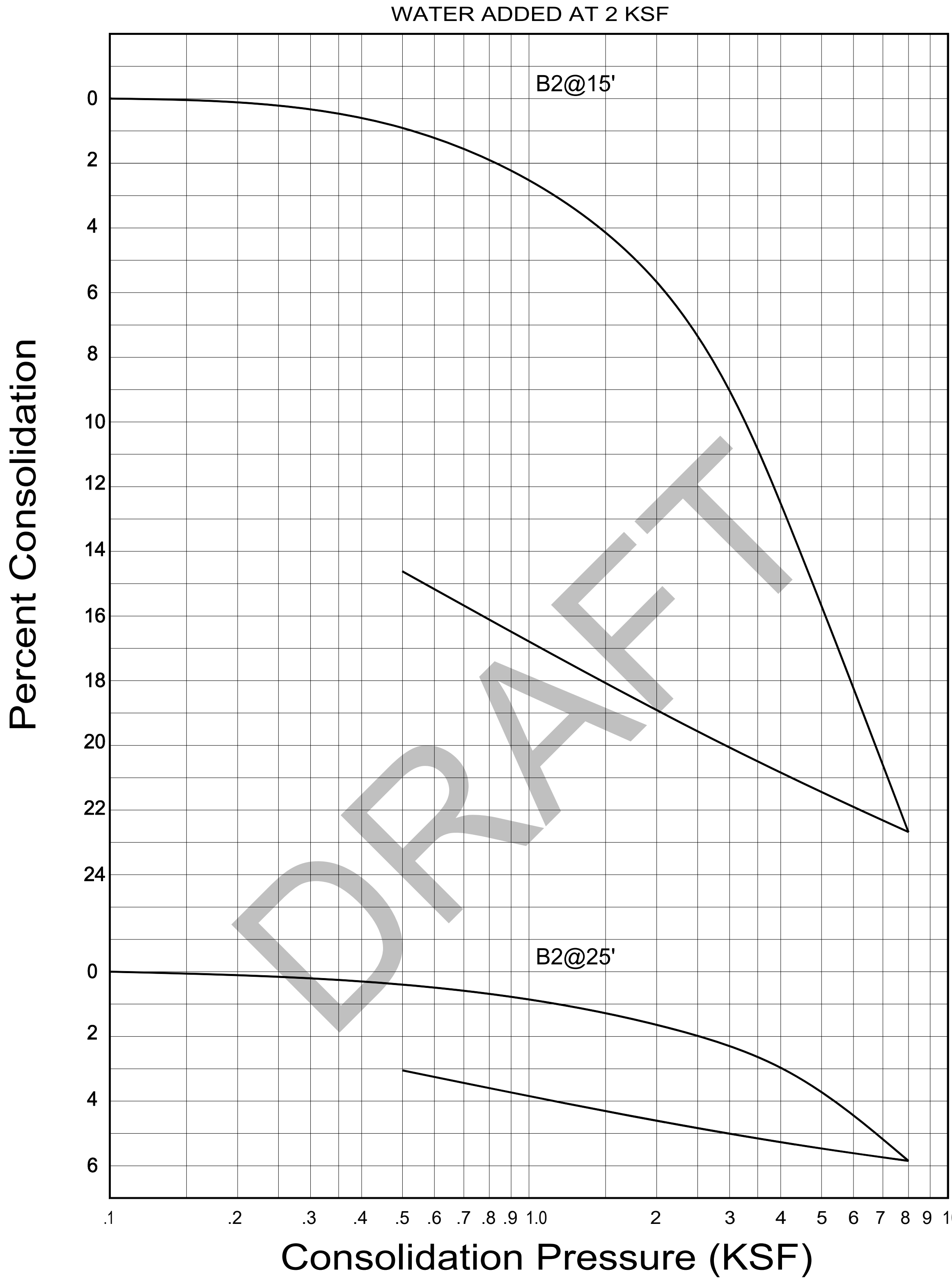
CONSOLIDATION TEST RESULTS

ROSE EQUITIES

1683 SUNFLOWER AVENUE

COSTA MESA, CALIFORNIA

JULY 2019	PROJECT NO. A9933-88-01	FIG. B7



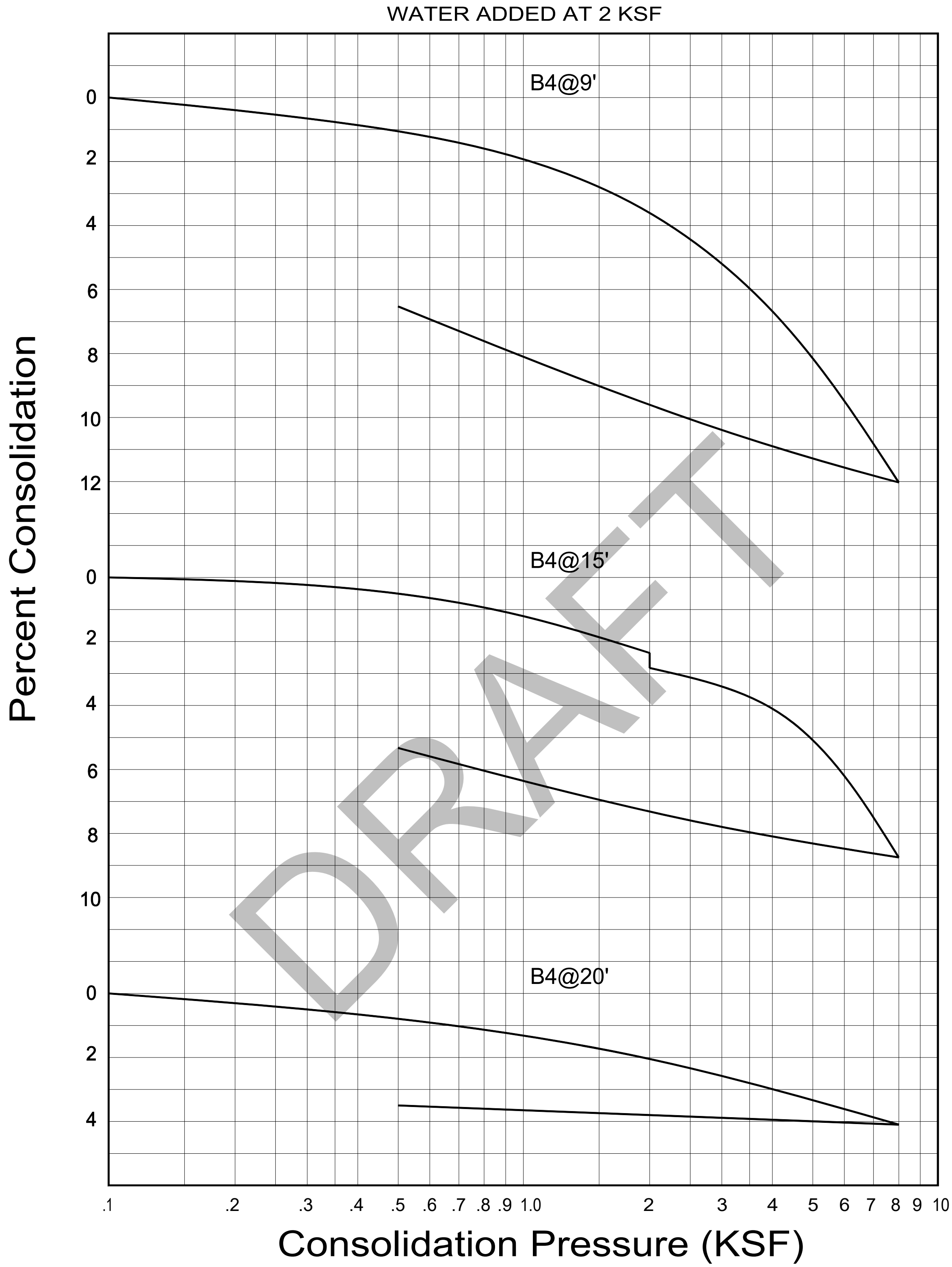
GEOCON WEST, INC.
ENVIRONMENTAL GEOTECHNICAL MATERIALS
15520 ROCKFIELD BOULEVARD, SUITE J, IRVINE, CA 92618
PHONE (949) 491-6570 FAX (949) 299-4550

Drafted by: JS | Checked by: JTA

CONSOLIDATION TEST RESULTS

ROSE EQUITIES
1683 SUNFLOWER AVENUE
COSTA MESA, CALIFORNIA

JULY 2019 | PROJECT NO. A9933-88-01 | FIG. B8



GEOCON
W E S T, I N C.

ENVIRONMENTAL GEOTECHNICAL MATERIALS
15520 ROCKFIELD BOULEVARD, SUITE J, IRVINE, CA 92618
PHONE (949) 491-6570 FAX (949) 299-4550

Drafted by: JS	Checked by: JTA

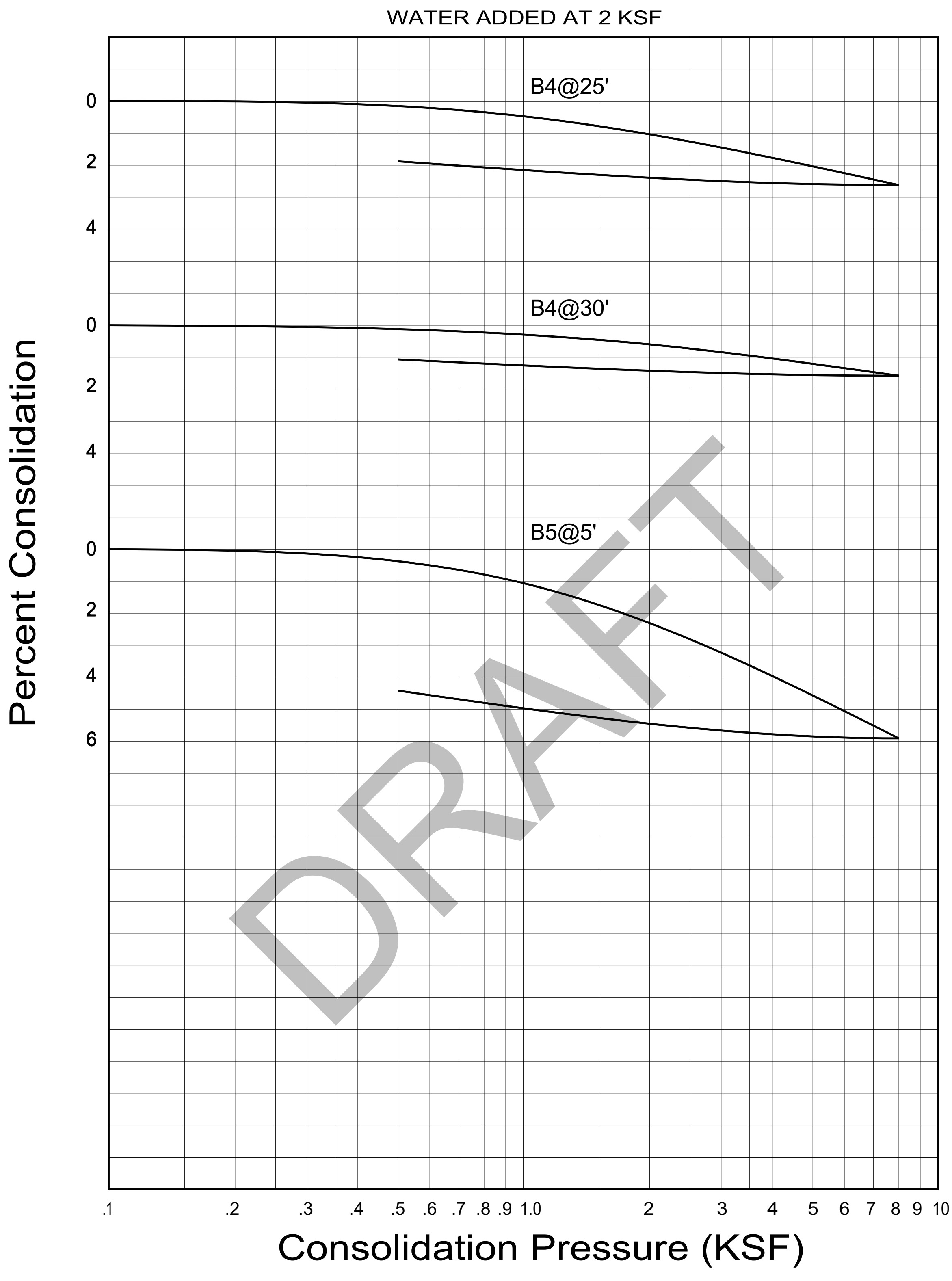
CONSOLIDATION TEST RESULTS

ROSE EQUITIES

1683 SUNFLOWER AVENUE

COSTA MESA, CALIFORNIA

JULY 2019	PROJECT NO. A9933-88-01	FIG. B9



GEOCON WEST, INC.

ENVIRONMENTAL GEOTECHNICAL MATERIALS

15520 ROCKFIELD BOULEVARD, SUITE J, IRVINE, CA 92618
PHONE (949) 491-6570 FAX (949) 299-4550

Drafted by: JS	Checked by: JTA

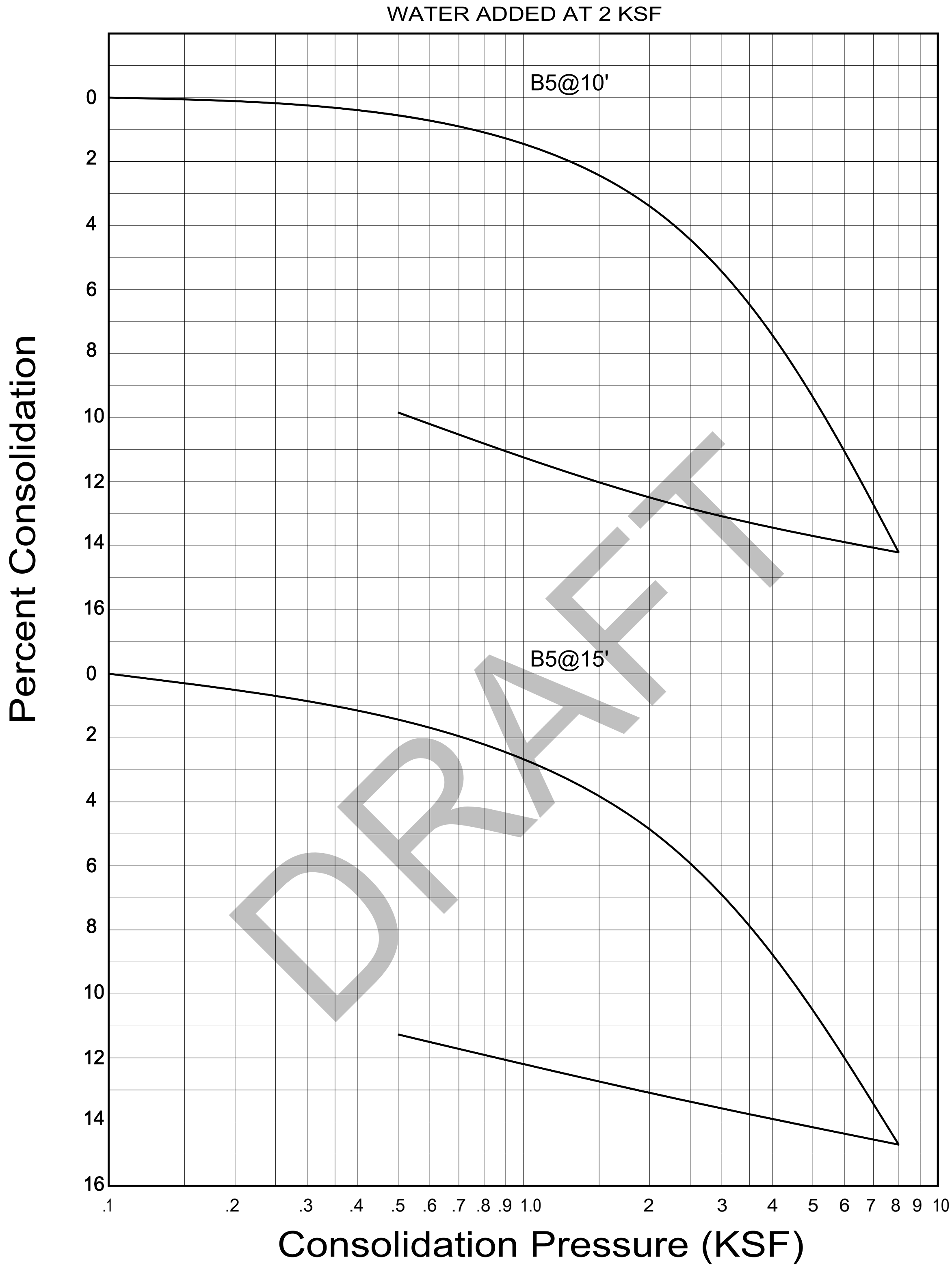
CONSOLIDATION TEST RESULTS

ROSE EQUITIES

1683 SUNFLOWER AVENUE

COSTA MESA, CALIFORNIA

JULY 2019	PROJECT NO. A9933-88-01	FIG. B10



GEOCON
WEST, INC.

ENVIRONMENTAL GEOTECHNICAL MATERIALS

15520 ROCKFIELD BOULEVARD, SUITE J, IRVINE, CA 92618

PHONE (949) 491-6570 FAX (949) 299-4550

Drafted by: JS	Checked by: JTA

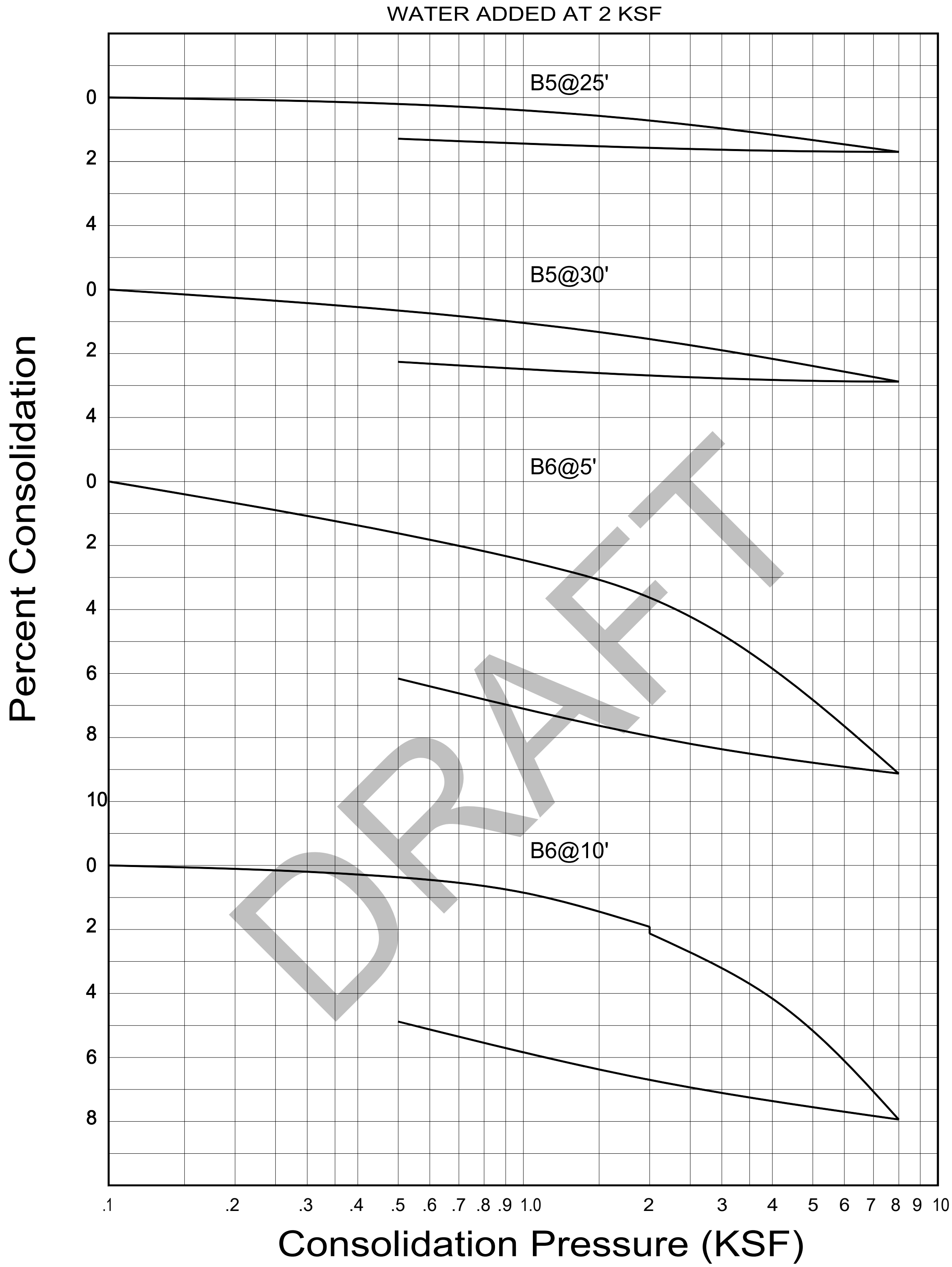
CONSOLIDATION TEST RESULTS

ROSE EQUITIES

1683 SUNFLOWER AVENUE

COSTA MESA, CALIFORNIA

JULY 2019	PROJECT NO. A9933-88-01	FIG. B11



DRAFT

GEOCON
WEST, INC.
ENVIRONMENTAL GEOTECHNICAL MATERIALS
15520 ROCKFIELD BOULEVARD, SUITE J, IRVINE, CA 92618
PHONE (949) 491-6570 FAX (949) 299-4550

Drafted by: JS	Checked by: JTA

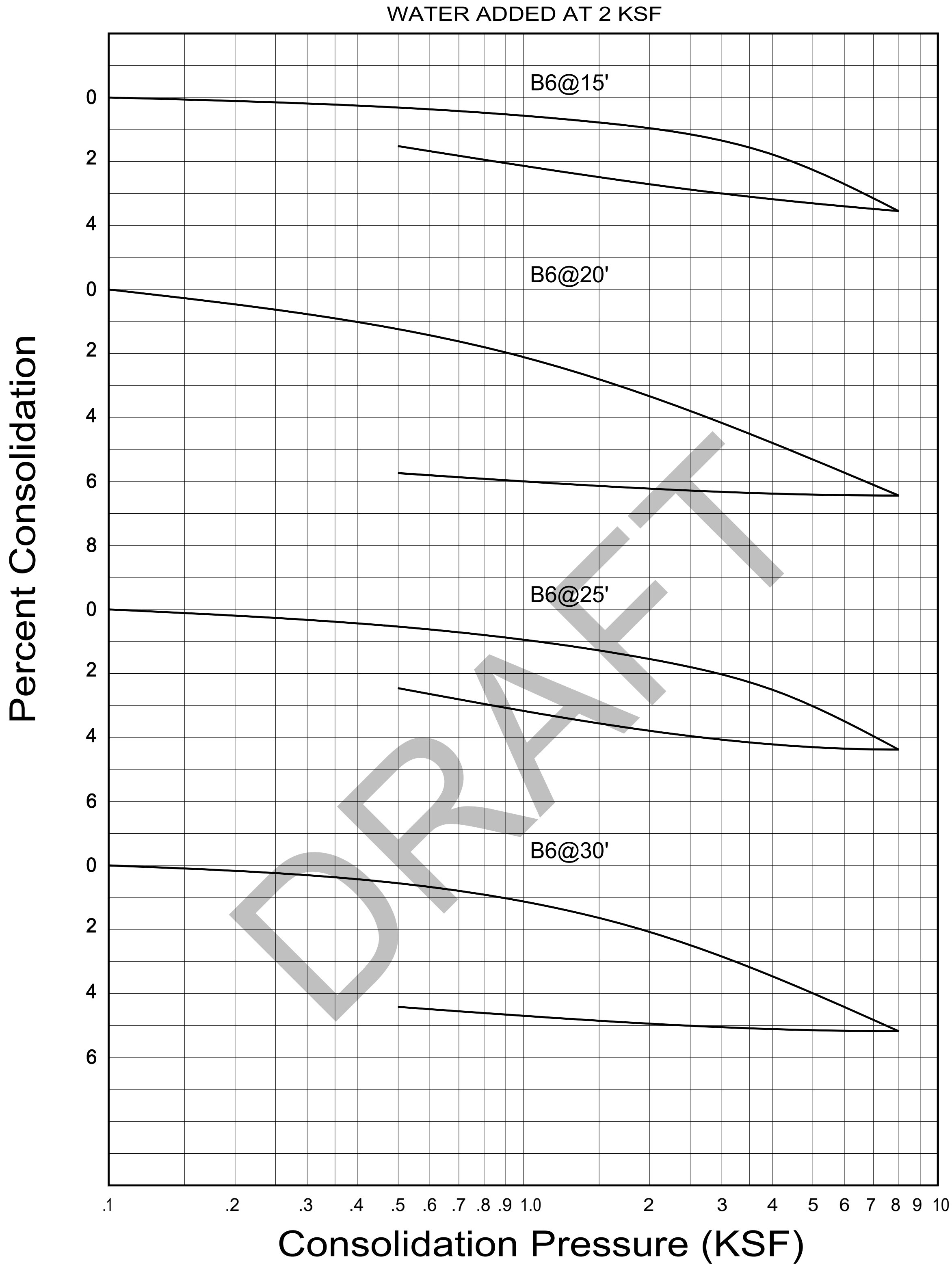
CONSOLIDATION TEST RESULTS

ROSE EQUITIES

1683 SUNFLOWER AVENUE

COSTA MESA, CALIFORNIA

JULY 2019	PROJECT NO. A9933-88-01	FIG. B12



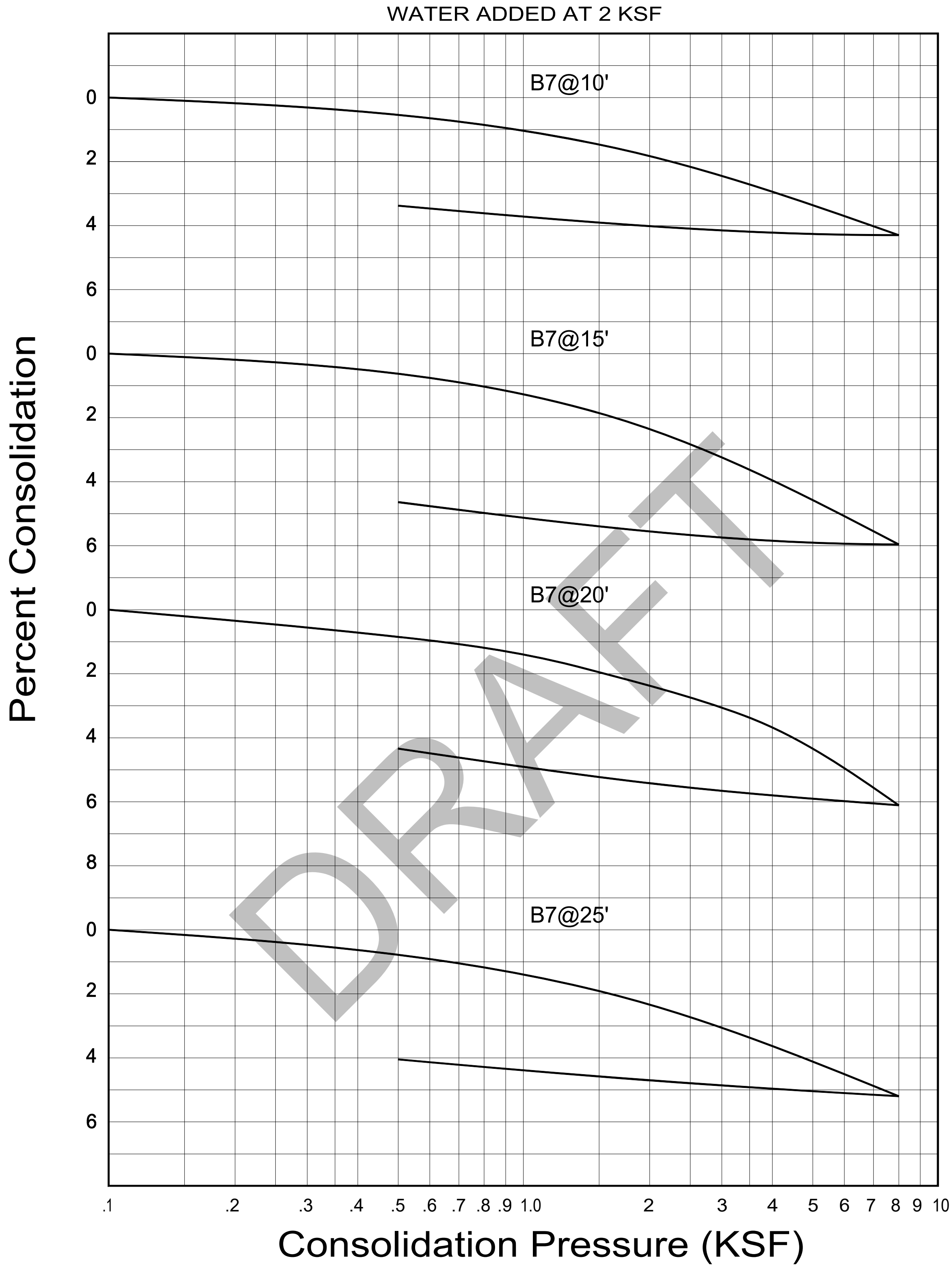
GEOCON
WEST, INC.
ENVIRONMENTAL GEOTECHNICAL MATERIALS
15520 ROCKFIELD BOULEVARD, SUITE J, IRVINE, CA 92618
PHONE (949) 491-6570 FAX (949) 299-4550

Drafted by: JS | Checked by: JTA

CONSOLIDATION TEST RESULTS

ROSE EQUITIES
1683 SUNFLOWER AVENUE
COSTA MESA, CALIFORNIA

JULY 2019 | PROJECT NO. A9933-88-01 | FIG. B13



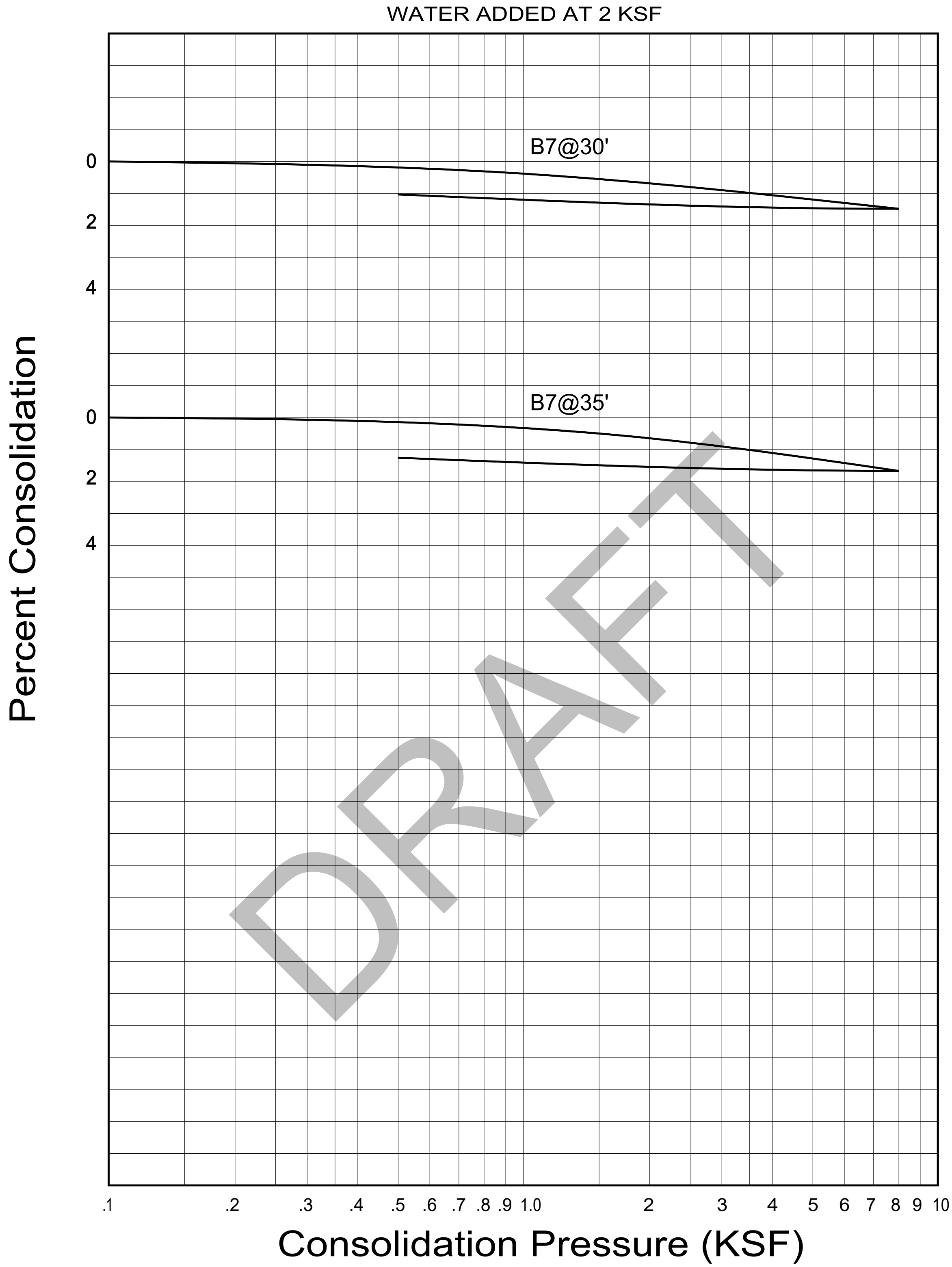
GEOCON WEST, INC.
ENVIRONMENTAL GEOTECHNICAL MATERIALS
15520 ROCKFIELD BOULEVARD, SUITE J, IRVINE, CA 92618
PHONE (949) 491-6570 FAX (949) 299-4550

Drafted by: JS	Checked by: JTA

CONSOLIDATION TEST RESULTS

ROSE EQUITIES
1683 SUNFLOWER AVENUE
COSTA MESA, CALIFORNIA

JULY 2019	PROJECT NO. A9933-88-01	FIG. B14

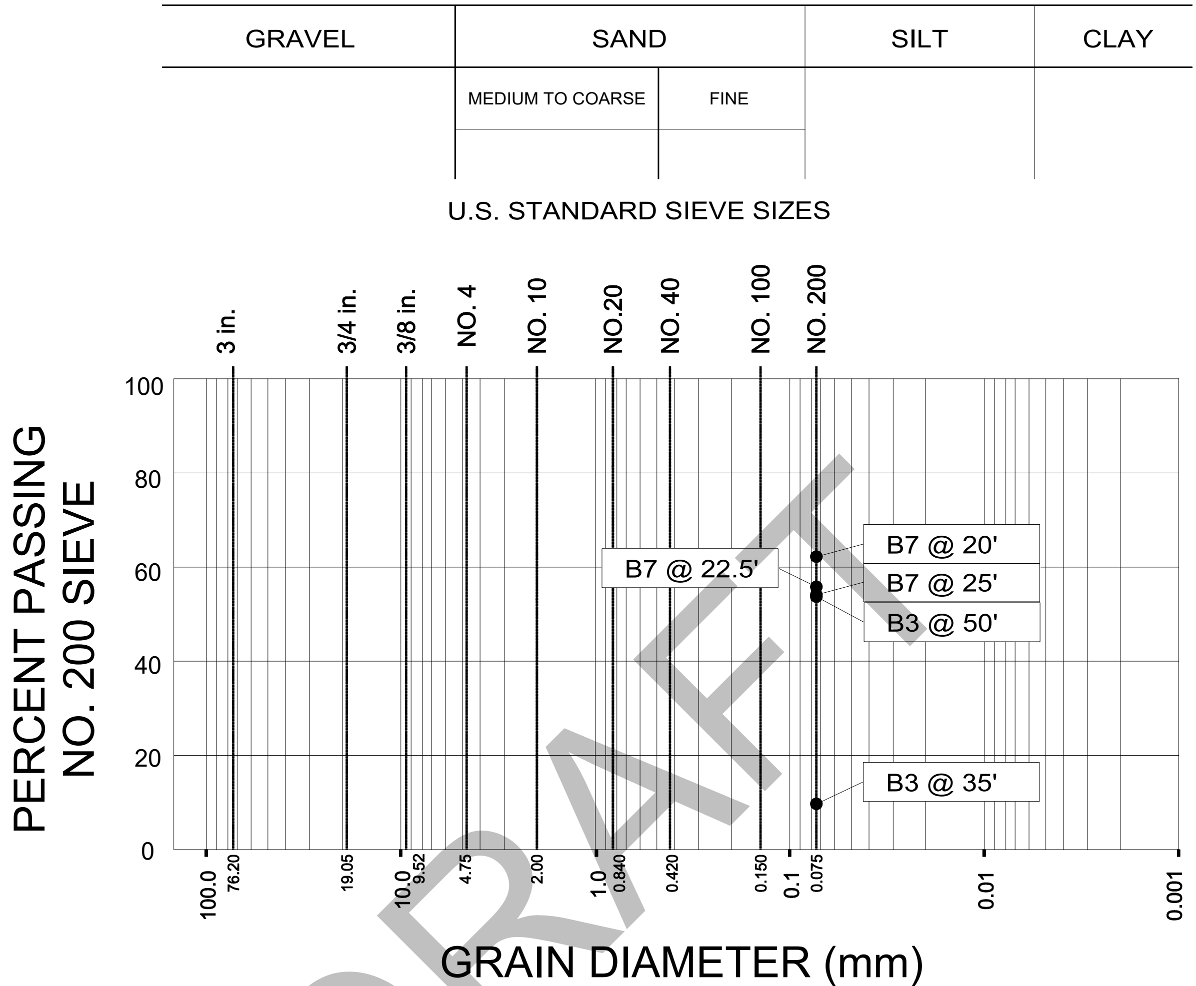


GEOCON
WEST, INC.
ENVIRONMENTAL GEOTECHNICAL MATERIALS
15520 ROCKFIELD BOULEVARD, SUITE J, IRVINE, CA 92618
PHONE (949) 491-6570 FAX (949) 299-4550

CONSOLIDATION TEST RESULTS

ROSE EQUITIES
1683 SUNFLOWER AVENUE
COSTA MESA, CALIFORNIA

Drafted by: JS	Checked by: JTA	JULY 2019	PROJECT NO. A9933-88-01	FIG. B15



SAMPLE	PERCENT PASSING NO. 200 SIEVE
B3 @ 35'	9.7
B3 @ 50'	53.7
B7 @ 20'	62.2
B7 @ 22.5'	55.8
B7 @ 25'	54.1

DRAFT

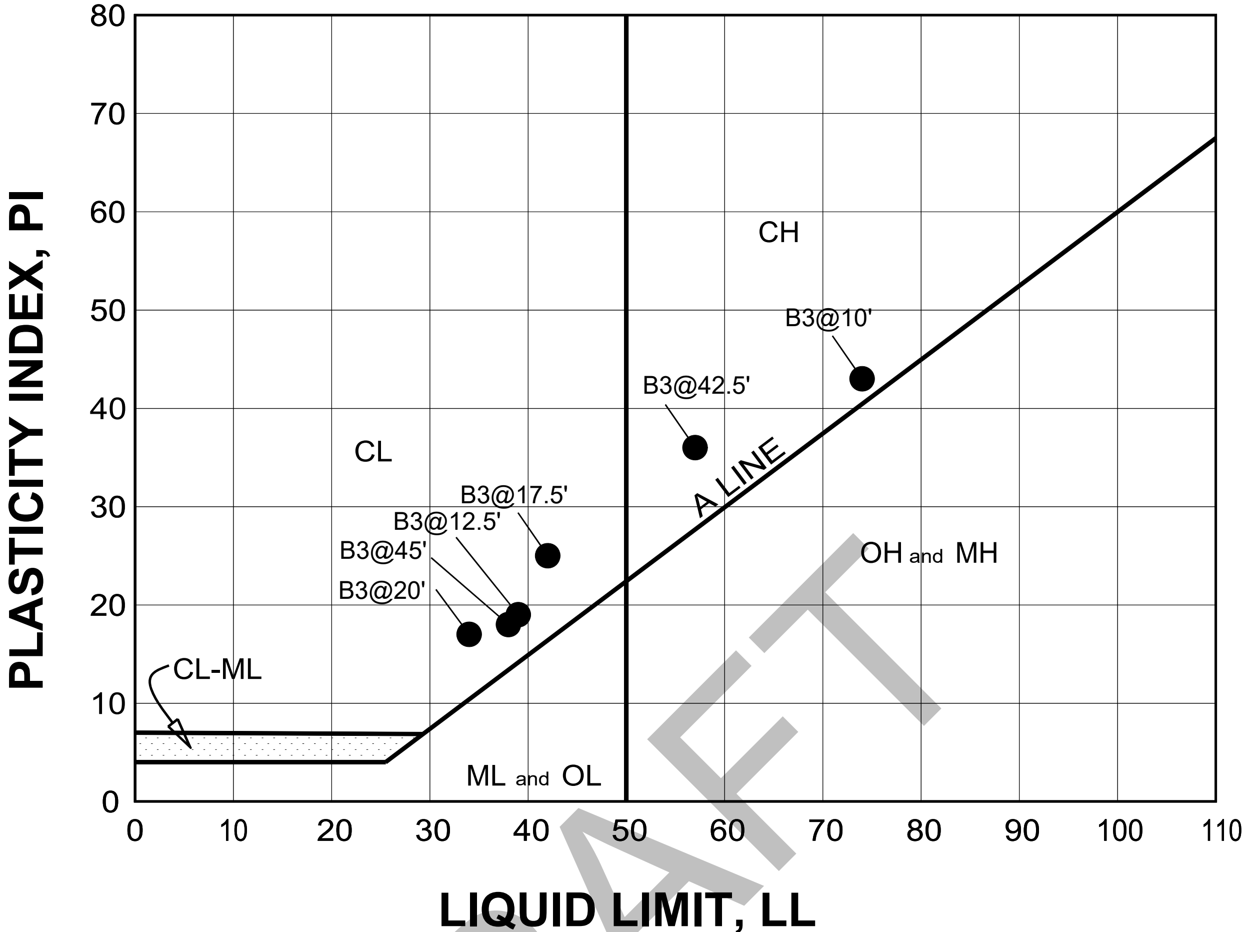
GEOCON WEST, INC.
ENVIRONMENTAL GEOTECHNICAL MATERIALS
15520 ROCKFIELD BOULEVARD, SUITE J, IRVINE, CA 92618
PHONE (949) 491-6570 FAX (949) 299-4550

Drafted by: JS	Checked by: JTA

GRAIN SIZE ANALYSIS

ROSE EQUITIES
1683 SUNFLOWER AVENUE
COSTA MESA, CALIFORNIA

JULY 2019	PROJECT NO. A9933-88-01	FIG. B16



DRAFT

BORING NUMBER	DEPTH (FEET)	LL	PL	PI	MOISTURE CONTENT AT SATURATION	SOIL BEHAVIOR
B3	**10**	**74**	**31**	**43**	**- -**	**CH**
B3	**12.5**	**39**	**20**	**19**	**- -**	**CL**
B3	**17.5**	**42**	**17**	**25**	**- -**	**CL**
B3	**20**	**34**	**17**	**17**	**- -**	**CL**
B3	**22.5**	**N/P**	**N/P**	**N/P**	**- -**	
B3	**42.5**	**57**	**21**	**36**	**- -**	**CH**
B3	**45**	**38**	**20**	**18**	**- -**	**CL**

*N/P indicates Non-Plastic

GEOCON
WEST, INC.
ENVIRONMENTAL GEOTECHNICAL MATERIALS
15520 ROCKFIELD BOULEVARD, SUITE J, IRVINE, CA 92618
PHONE (949) 491-6570 FAX (949) 299-4550

Drafted by: JS | Checked by: JTA

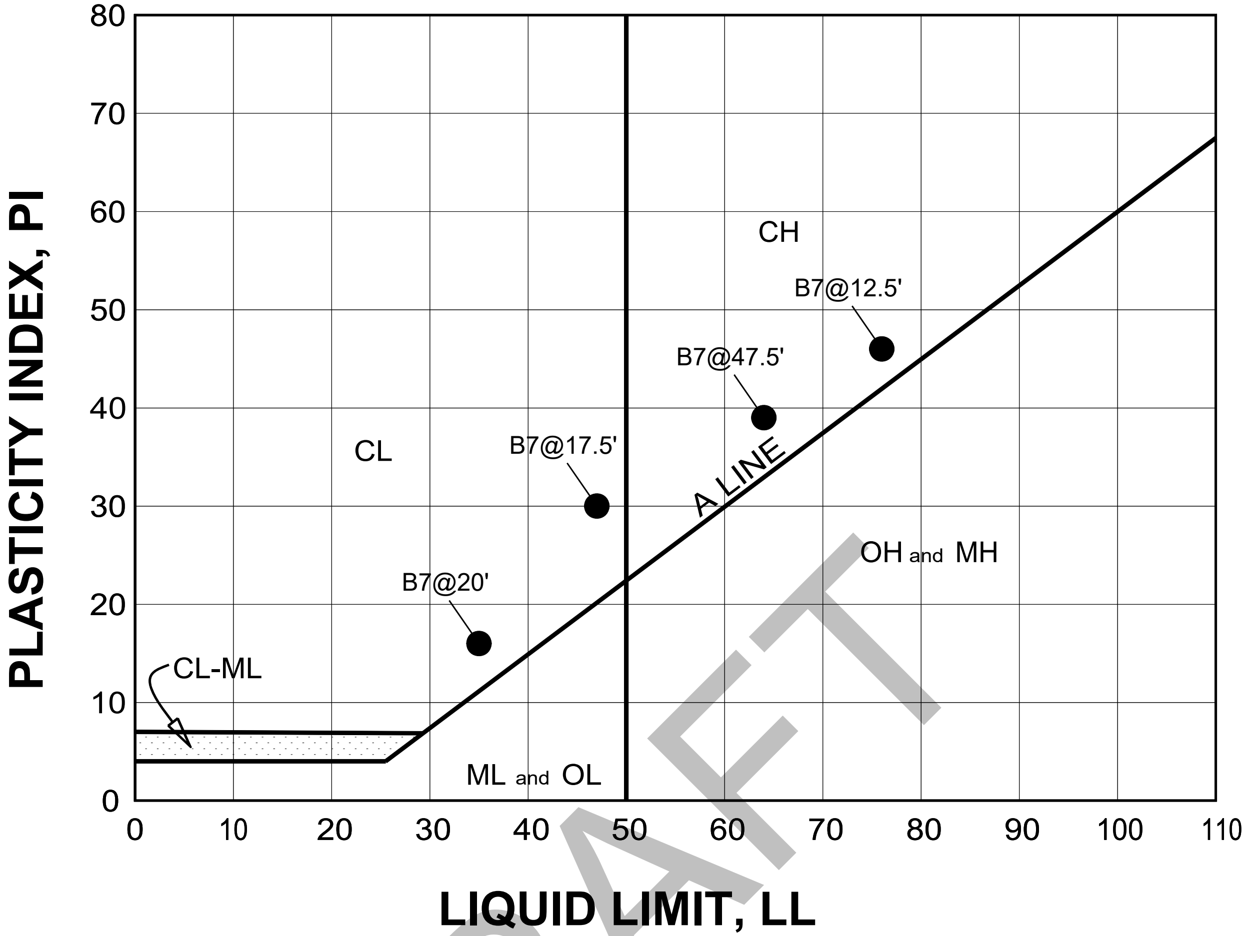
ATTERBERG LIMITS

ROSE EQUITIES

1683 SUNFLOWER AVENUE

COSTA MESA, CALIFORNIA

JULY 2019 | PROJECT NO. A9933-88-01 | FIG. B17



BORING NUMBER	DEPTH (FEET)	LL	PL	PI	MOISTURE CONTENT AT SATURATION	SOIL BEHAVIOR
B7	**10**	**N/P**	**N/P**	**N/P**	**- -**	
B7	**12.5**	**76**	**30**	**46**	**- -**	**CH**
B7	**17.5**	**47**	**17**	**30**	**- -**	**CL**
B7	**20**	**35**	**19**	**16**	**23.7**	**CL**
B7	**27.5**	**N/P**	**N/P**	**N/P**	**- -**	
B7	**47.5**	**64**	**25**	**39**	**- -**	**CH**

*N/P indicates Non-Plastic

DRAFT

GEOCON WEST, INC.
ENVIRONMENTAL GEOTECHNICAL MATERIALS
15520 ROCKFIELD BOULEVARD, SUITE J, IRVINE, CA 92618
PHONE (949) 491-6570 FAX (949) 299-4550

Drafted by: JS	Checked by: JTA

ATTERBERG LIMITS

ROSE EQUITIES
1683 SUNFLOWER AVENUE
COSTA MESA, CALIFORNIA

JULY 2019	PROJECT NO. A9933-88-01	FIG. B18

SUMMARY OF LABORATORY EXPANSION INDEX TEST RESULTS
ASTM D 4829-11

Sample No.	Moisture Content (%)		Dry Density (pcf)	Expansion Index	*UBC Classification	**CBC Classification
	Before	After				
B1 @ 0-5'	11.5	20.4	103.6	0	Very Low	Non-Expansive
B6 @ 0-5'	11.0	24.6	104.0	63	Medium	Expansive
B7 @ 0-5'	15.0	40.0	93.4	88	Medium	Expansive
B3/B4/B5 Mix @ 0-5'	10.8	21.6	107.7	70	Medium	Expansive

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

** Reference: 2016 California Building Code, Section 1803.5.3

SUMMARY OF LABORATORY MAXIMUM DENSITY AND
AND OPTIMUM MOISTURE CONTENT TEST RESULTS
ASTM D 1557-12

Sample No.	Soil Description	Maximum Dry Density (pcf)	Optimum Moisture (%)
B1 @ 0-5'	Brown Silty Sand	116.8	13.1
B6 @ 0-5'	Dark Yellowish Brown Clay	115.4	14.2
B7 @ 0-5'	Dark Olive Brown Clay	109.5	16.4
B3/B4/B5 Mix @ 0-5'	Brown Silty Sand	121.6	12.5

DRAFT

GEOCON
WEST, INC.
ENVIRONMENTAL GEOTECHNICAL MATERIALS
15520 ROCKFIELD BOULEVARD, SUITE J, IRVINE, CA 92618
PHONE (949) 491-6570 FAX (949) 299-4550

DRAFTED BY: JS | CHECKED BY: JTA

LABORATORY TEST RESULTS

ROSE EQUITIES
1683 SUNFLOWER AVENUE
COSTA MESA, CALIFORNIA

ORGANIC CONTENT
ASTM D 2974-14

Sample No.	ORGANICS CONTENT (%)
B1@20'	2.0
B2@15'	20.1
B4@9'	9.3
B4@12'	3.9
B5@10'	6.5
B5@15'	7.8
B6@10'	4.6
B6@15'	2.8
B7@15'	3.1

DRAFT

GEOCON
WEST, INC.
ENVIRONMENTAL GEOTECHNICAL MATERIALS
15520 ROCKFIELD BOULEVARD, SUITE J, IRVINE, CA 92618
PHONE (949) 491-6570 FAX (949) 299-4550

DRAFTED BY: JS	CHECKED BY: JTA

LABORATORY TEST RESULTS

ROSE EQUITIES

1683 SUNFLOWER AVENUE

COSTA MESA, CALIFORNIA

JULY 2019	PROJECT NO. A9933-88-01	FIG. B20

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

Sample No.	pH	Resistivity (ohm centimeters)
B1 @ 0-5'	8.00	3000 (Moderately Corrosive)
B6 @ 0-5'	8.00	1300 (Corrosive)
B7 @ 0-5'	8.24	590 (Severely Corrosive)
B3/B4/B5 MIX @ 0-5'	8.52	1200 (Corrosive)

SUMMARY OF LABORATORY CHLORIDE CONTENT TEST RESULTS EPA NO. 325.3

Sample No.	Chloride Ion Content (%)
B1 @ 0-5'	0.030
B6 @ 0-5'	0.048
B7 @ 0-5'	0.034
B3/B4/B5 MIX @ 0-5'	0.012

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

Sample No.	Water Soluble Sulfate (% SO_4)	Sulfate Exposure*
B1 @ 0-5'	0.001	S0
B6 @ 0-5'	0.000	S0
B7 @ 0-5'	0.045	S0
B3/B4/B5 MIX @ 0-5'	0.001	S0

* Reference: 2016 California Building Code, Section 1904.3 and ACI 318-11 Section 4.3.

DRAFT

GEOCON WEST, INC.
ENVIRONMENTAL GEOTECHNICAL MATERIALS
15520 ROCKFIELD BOULEVARD, SUITE J, IRVINE, CA 92618
PHONE (949) 491-6570 FAX (949) 299-4550

CORROSIVITY TEST RESULTS

ROSE EQUITIES
1683 SUNFLOWER AVENUE
COSTA MESA, CALIFORNIA

DRAFTED BY: JS	CHECKED BY: JTA	JULY 2019	PROJECT NO. A9933-88-01	FIG. B21

APPENDIX C

DRAFT

APPENDIX C

CLIQ LIQUEFACTION ANALYSIS REPORT

(CD Only)

DRAFT