APPENDIX J

GEOTECHNICAL INVESTIGATION

GEOTECHNICAL INVESTIGATION

PROPOSED MULTI-FAMILY
RESIDENTIAL STRUCTURES
ROSE HILL COURTS REDEVELOPMENT
4400-4486 E FLORIZEL STREET, 3531-3825
& 3501-3521 N MCKENZIE AVENUE, AND
4401-4496 E MERCURY AVENUE
LOS ANGELES, CALIFORNIA
TRACT: 13089 LOTS: 1 & 3

GEOCON
WEST, INC.

GEOTECHNICAL
ENVIRONMENTAL
MATERIALS

PREPARED FOR

THE RELATED COMPANIES OF CALIFORNIA, LLC
IRVINE, CALIFORNIA

PROJECT NO. A9514-06-01

MAY 16, 2018

Project No. A9514-06-01
May 16, 2018

The Related Companies of California, LLC
18201 Von Karman Avenue, Suite 900
Irvine, California 92612

Attention: Ms. Rose Olson

Subject: GEOTECHNICAL INVESTIGATION
PROPOSED MULTI-FAMILY RESIDENTIAL STRUCTURES
ROSE HILL COURTS REDEVELOPMENT
4400-4486 E FLORIZEL STREET, 3531-3825 & 3501-3521 N MCKENZIE AVENUE, AND 4401-4496 E MERCURY AVENUE
LOS ANGELES, CALIFORNIA
TRACT: 13089, LOTS: 1 & 3

Dear Ms. Olson,

In accordance with your authorization of our proposal dated March 27, 2018, we have performed a geotechnical investigation for the proposed multi-family residential structures located at 4400-4486 East Florizel Street, 3531-3825 & 3501-3521 North McKenzie Avenue, and 4401-4496 East Mercury Avenue, in the City of Los Angeles, California. The accompanying report presents the findings of our study, and our conclusions and recommendations pertaining to the geotechnical aspects of proposed design and construction. Based on the results of our investigation, it is our opinion that the site can be developed as proposed, provided the recommendations of this report are followed and implemented during design and construction.

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

Very truly yours,

GEOCON WEST, INC.

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

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GEOTECHNICAL INVESTIGATION

1. PURPOSE AND SCOPE

This report presents the results of a geotechnical investigation for the proposed multi-family residential structures located at 4400-4486 East Florizel Street, 3531-3825 & 3501-3521 North McKenzie Avenue, and 4401-4496 East Mercury Avenue, in the City of Los Angeles, California (see Vicinity Map, Figure 1). The purpose of the investigation was to evaluate subsurface soil and geologic conditions underlying the site and, based on conditions encountered, to provide conclusions and recommendations pertaining to the geotechnical aspects of design and construction.

The scope of this investigation included a site reconnaissance, field exploration, laboratory testing, engineering analysis, and the preparation of this report. The site was explored on November 17 through 22, 2016, by excavating two 8-inch diameter borings to depths of approximately 26½ and 56½ feet below the existing ground surface utilizing a truck-mounted hollow-stem auger drilling machine. In addition, eleven 4-inch diameter boring to depths 5 to 11 feet below existing ground surface were excavating using hand tools. The site was also explored on April 12 and 13, 2018 by excavating four 8-inch diameter holes to depths of approximately 20½ and 30½ below existing ground surface utilizing a limited access hollow-stem auger-drilling machine. Also, three 4-inch diameter borings were excavated utilizing hand auger equipment to depths of approximately 5 to 15 feet below existing ground surface for percolation testing. The approximate locations of the exploratory borings are depicted on the Site Plan (see Figure 2). A detailed discussion of the field investigation, including boring logs, is presented in Appendix A.

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

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

If project details vary significantly from those described 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 (designated as 4466 Florizel Street), is located at 4400-4486 East Florizel Street, 3531-3825 & 3501-3521 North McKenzie Avenue, and 4401-4496 East Mercury Avenue in the City of Los Angeles, California. The site consists of two rectangular parcels and is currently occupied by 14 two-story residential structures, and administration building and an asphalt paved parking lot. The site is bounded by Boundary Avenue to the west, by Mercury Avenue to the south, by Mackenzie Avenue to the east, and by Florizel Street to the north. The site slopes downward from the west to east. Surface water drainage at the site appears to be by sheet flow along the existing ground contours to the city streets. Vegetation consists of areas of native grasses and trees between the buildings across the site.

It is our understanding that the proposed improvements will consist of a complete redevelopment of the site. All of the existing buildings will be demolished and 191 affordable housing units in 32 residential structures, up to 4-stories high, will be constructed (see Site Plan, Figure 2). The improvements will also include parking areas for 176 vehicles. No subterranean levels are anticipated. The proposed buildings will be constructed at or near existing grade or will be tucked into the hillside and will retain up to 10 feet.

Due to preliminary nature of the design at this time, wall and column loads were not available. It is anticipated that column loads for the proposed structures will be up to 400 kips, and wall loads will be up to 4 kips per linear foot.

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

3. GEOLOGIC SETTING

Locally the site is located the central portion of the Repetto Hills (Lamar, 1970). The Repetto Hills trend northwest-southeast along the northeastern edge of the Los Angeles Basin and are composed of folded and faulted Miocene age marine sedimentary bedrock of the Puente Formation that has been uplifted and incised by elevated flood plains and uplifted alluvial valley deposits. Regionally, the site is located within the Peninsular Ranges geomorphic province that is characterized by elongate northwest-trending mountain ridges separated by straight-sided sediment-filled valleys (Yerkes et al., 1965). The northwest trend is further reflected in the direction of the dominant geologic structural features of the province that are northwest to west-northwest trending folds and faults including the nearby Whittier Fault Zone.

4. SOIL AND GEOLOGIC CONDITIONS

Based on our field investigation and published geologic maps of the area, the site is underlain by artificial fill, Pleistocene age alluvial valley deposits, and Miocene age sedimentary bedrock of the Puente Formation (Lamar, 1970). Detailed stratigraphic profiles of the materials encountered in our borings are provided on the boring logs in Appendix A.

4.1 Artificial Fill

Artificial fill was encountered in our field explorations to a maximum depth of 6 feet below existing ground surface. The artificial fill generally consists of light brown to dark brown and dark yellowish brown silty sand, sandy silt, and clayey silt. The artificial fill is characterized as fine to medium-grained, dry to slightly moist and loose to medium dense or soft to hard. The fill is likely the result of past grading or construction activities at the site. Deeper fill may exist between excavations and in other portions of the site that were not directly explored.

4.2 Older Alluvium

Pleistocene age old alluvial valley deposits were encountered beneath the fill and consist primarily of light to dark brown, yellowish brown, reddish brown, and olive brown to olive gray clayey silt, silt, sandy silt, silt with sand, sand with silt, and silty sand. The alluvial soils are mostly fine-grained and characterized as dry to wet, firm to hard or medium dense to very dense.

4.3 Puente Formation

Miocene age sedimentary bedrock of the Puente Formation was in encountered in B1, B2, B14 and B17 at depths of 14.5, 47, 11.5, and 15 feet respectively. The bedrock is identified as olive gray to yellowish brown sandstone, siltstone, and sandy siltstone and is characterized as being soft to moderately hard, slightly moist, massive to thinly bedded, and completely to moderately weathered.

5. GROUNDWATER

Based on a review of the Seismic Hazard Evaluation Report for the Los Angeles 7.5-Minute Quadrangle (California Division of Mines and Geology [CDMG], 1998), the historically highest groundwater level in the area is approximately 20 feet beneath the ground surface. Groundwater information presented in this document is generated from data collected in the early 1900's to the late 1990s. Based on current groundwater basin management practices, it is unlikely that groundwater levels will ever exceed the historic high levels.

Perched groundwater was encountered in borings B1, B2, and B17 at depths of 15, 40, and 15 feet below ground surface respectively. The groundwater is interpreted to be perched on top of the less permeable Puente Formation bedrock. Based on the presence of only perched groundwater in our borings, the reported historic high ground water level in the area (CDMG, 1998), and the depth of the proposed construction, it is unlikely that groundwater will be encountered during construction. However, it is common for groundwater to seasonally occur in the area or for groundwater conditions to develop where none previously existed, especially in impermeable fine-grained soils which are heavily irrigated or after seasonal rainfall. In addition, recent requirements for stormwater infiltration could result in shallower seepage conditions in the immediate site vicinity. Proper surface drainage of irrigation and precipitation will be critical for future performance of the project. Recommendations for drainage are provided in the S*urface Drainage* section of this report (see Section 7.21).

6. GEOLOGIC HAZARDS

6.1 Surface Fault Rupture

The numerous faults in Southern California include active, potentially active, and inactive faults. The criteria for these major groups is based on criteria developed by the California Geological Survey (CGS, 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 11,000 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 (CDMG, 1977; CGS, 2017; CGS, 2018b) or a city-designated Preliminary Fault Rupture Study Area (City of Los Angeles, 2018) for surface fault rupture hazards. No active or potentially active faults with the potential for surface fault rupture are known to pass directly beneath the site. Therefore, the potential for surface rupture due to faulting occurring beneath the site during the design life of the proposed development is considered low. However, the site is located in the seismically active Southern California region, and could be subjected to moderate to strong ground shaking in the event of an earthquake on one of the many active Southern California faults. The faults in the site vicinity are shown in Figure 3, Regional Fault Map.

The closest active fault to the site is the Raymond Fault located approximately 2.3 miles to the north (CDMG, 1977). Other nearby active faults are the Eagle Rock Fault, the Verdugo Fault, the Hollywood Fault, the Whittier Fault, and the Sierra Madre Fault located approximately 4.0 miles northwest, 4.8 miles northwest, 4.9 miles west, 6.0 miles southeast, and 7.6 miles northeast of the site, respectively (Ziony and Jones, 1989). The active San Andreas Fault Zone is located approximately 30 miles northeast of the site.

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

6.2 Seismicity

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

LIST OF HISTORIC EARTHQUAKES

Earthquake (Oldest to Youngest)	Date of Earthquake	Magnitude	Distance to Epicenter (Miles)	Direction to Epicenter
San Jacinto-Hemet area	April 21, 1918	6.8	72	ESE
Near Redlands	July 23, 1923	6.3	54	E
Long Beach	March 10, 1933	6.4	35	SSE
Tehachapi	July 21, 1952	7.5	79	NW
San Fernando	February 9, 1971	6.6	25	NW
Whittier Narrows	October 1, 1987	5.9	7	ESE
Sierra Madre	June 28, 1991	5.8	16	NE
Landers	June 28, 1992	7.3	101	E
Big Bear	June 28, 1992	6.4	78	E
Northridge	January 17, 1994	6.7	22	WNW
Hector Mine	October 16, 1999	7.1	115	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.

6.3 Seismic Design Criteria

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

2016 CBC SEISMIC DESIGN PARAMETERS

Parameter	Value	2016 CBC Reference
Site Class	D	Table 1613.3.2
MCE_R Ground Motion Spectral Response Acceleration – Class B (short), S_S	2.760g	Figure 1613.3.1(1)
MCE_R Ground Motion Spectral Response Acceleration – Class B (1 sec), S_1	0.958g	Figure 1613.3.1(2)
Site Coefficient, F_A	1.0	Table 1613.3.3(1)
Site Coefficient, F_V	1.5	Table 1613.3.3(2)
Site Class Modified MCE_R Spectral Response Acceleration (short), S_{MS}	2.760g	Section 1613.3.3 (Eqn 16-37)
Site Class Modified MCE_R Spectral Response Acceleration – (1 sec), S_{M1}	1.436g	Section 1613.3.3 (Eqn 16-38)
5% Damped Design Spectral Response Acceleration (short), S_{DS}	1.840g	Section 1613.3.4 (Eqn 16-39)
5% Damped Design Spectral Response Acceleration (1 sec), S_{D1}	0.958g	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	1.053g	Figure 22-7
Site Coefficient, F_{PGA}	1.0	Table 11.8-1
Site Class Modified MCE_G Peak Ground Acceleration, PGA_M	1.053g	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 6.61 magnitude event occurring at a hypocentral distance of 5.74 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 6.62 magnitude occurring at a hypocentral distance of 8.46 kilometers from the site.

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

6.4 Liquefaction Potential

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

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

The State of California Seismic Hazard Zone Map for the Los Angeles Quadrangle (CDMG, 1999; CGS, 2017) indicates that the majority of the site is located within a zone of required investigation 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 developed by Bray and Sancio (2006) characterize fine-grained soils which are not susceptible to liquefaction as soils with a plasticity index (PI) that is greater than 18 or with a saturated moisture content that is less than 80 percent of the liquid limit. In order to apply the screening criteria, laboratory testing was performed to evaluate the Atterberg Limits and saturated moisture content of select soil samples. Laboratory test results used for the screening criteria are presented as Figure B6.

In addition, the Boulanger and Idriss method was utilized to analyze clayey soil layers from 30 to 35 feet and 45 to 46 feet below the existing ground surface in boring B2. Based on the results of the analyses (see enclosed calculation sheet, Figure 5), the clayey soil layer at approximately depth of 30 feet in boring B2 has a calculated sensitivity (S_t) of 1.5 and is not subject to liquefaction. At a depth of 45 feet, the clayey soil layer in boring B2 has a calculated sensitivity (S_t) of 3, and is not subject to liquefaction. Based on the results of the analyses, it is our opinion that the clayey layers at the site indicated above are not subject to liquefaction.

The liquefaction analysis was performed for a Design Earthquake level by using a historic high groundwater table of 20 feet below the ground surface, a magnitude 6.62 earthquake, and a peak horizontal acceleration of 0.702g (⅔PGAM). The enclosed liquefaction analyses, included herein for borings B1 and B2, indicate that the alluvial soils below the historic high groundwater level are not susceptible to liquefaction settlement during Design Earthquake ground motion (see enclosed calculation sheets, Figures 6 through 9).

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

The liquefaction analysis was also performed for the Maximum Considered Earthquake level by using a historic high groundwater table of 20 feet below the ground surface, a magnitude 6.61 earthquake, and a peak horizontal acceleration of 1.053g (PGA_M). The enclosed liquefaction analyses, included herein for borings B1 and B2, indicate that the alluvial soils below the historic high groundwater level are not susceptible to liquefaction settlement during Maximum Considered Earthquake ground motion (see enclosed calculation sheets, Figures 10 through 13).

6.5 Seismically-Induced Settlement

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

The calculations provided herein for borings B1 and B2 indicate that the soil above the historic high groundwater level of 20 feet could be susceptible to approximately 0.11 and 0.14 inch, respectively, of settlement as a result of the Design Earthquake peak ground acceleration ($\frac{2}{3}PGA_M$).

The calculations provided herein for borings B1 and B2 indicate that the soil above the historic high groundwater level of 20 feet could be susceptible to approximately 0.39 and 0.38 inches, respectively, of settlements as a result of the Maximum Considered Earthquake peak ground acceleration (PGA_M).

Calculation of the anticipated seismically-induced settlements is provided as Figures 14 through 17.

6.6 Slope Stability

Topography at the site slopes to the southeast at a gradient flatter than 5:1 (H:V). The site is located within a City of Los Angeles Hillside Grading Area and a Hillside Ordinance Area (City of Los Angeles, 2018). However, the site is not located within an area identified as having a potential for seismic slope instability by the state of California (CDMG, 1999; CGS, 2017). There are no known landslides near the site, nor is the site in the path of any known or potential landslides. Therefore, the probability of slope stability hazards affecting the site is considered very low.

6.7 Earthquake-Induced Flooding

Earthquake-induced flooding is inundation caused by failure of dams or other water-retaining structures due to earthquakes. The Los Angeles County Safety Element (Leighton, 1990) indicates that the site is not located within a designated dam inundation area. Therefore, the potential for inundation at the site, as a result of an earthquake-induced dam failure, is considered low.

6.8 Tsunamis, Seiches, and Flooding

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

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

The site is within an area of minimal flooding (Zone X) as defined by the Federal Emergency Management Agency (FEMA, 2018; LACDPW, 2018b).

6.9 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 (DOGGR, 2018). In addition, there are no active or inactive oil or gas well within a 1-mile radius of the site. 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 will need to be properly abandoned in accordance with the current requirements of the DOGGR.

The site is not located within the boundaries of a city-designated Methane Zone or a Methane Buffer Zone (City of Los Angeles, 2018). Therefore, the potential for the presence of methane or other volatile gases at the site is considered low. However, should it be determined that a methane study is required for the proposed development it is recommended that a qualified methane consultant be retained to perform the study and provide mitigation measures as necessary.

6.10 Subsidence

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

7. CONCLUSIONS AND RECOMMENDATIONS

7.1 General

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

7.1.2 The depth of artificial fill encountered during site exploration was observed to be variable, between 2½ feet and 6 feet. The existing fill encountered is believed to be the result of past grading and construction activities at the site. Deeper fill may exist in other areas of the site that were not directly explored. It is our opinion that the existing fill, in its present condition, is not suitable for direct support of proposed foundations or slabs. The existing fill and site soils are suitable for re-use as engineered fill provided the recommendations in the *Grading* section of this report are followed (see Section 7.4).

7.1.3 The enclosed seismically induced settlement analyses indicate that the site soils could be prone to approximately 0.14 inch of total settlement as a result of the Design Earthquake peak ground acceleration (⅔PGA_M). Differential settlement at the foundation level is anticipated to be less than 0.1 inch over a distance of 20 feet (see Section 7.9).

7.1.4 It is recommended that the upper 5 feet of existing earth materials in the building footprint areas be excavated and properly compacted for foundation and slab support. Deeper excavations should be conducted as needed to remove any encountered fill or soft soils as necessary at the direction of the Geotechnical Engineer (a representative of Geocon). Proposed building foundations should be underlain by a minimum of 3 feet of newly placed engineered fill. The excavation should extend laterally a minimum distance of 3 feet beyond the building footprint areas, including building appurtenances, or a distance equal to the depth of fill below the foundation, whichever is greater. The limits of existing fill and/or soft soil removal will be verified by the Geocon representative during site grading activities. Recommendations for earthwork are provided in the *Grading* section of this report (see Section 7.4).

7.1.5 Subsequent to the recommended grading, the proposed structures may be supported on a conventional foundation system deriving support in newly placed engineered fill. Recommendations for the design of a conventional foundation system are provided in Section 7.7.

7.1.6 The Building Code requires that proposed foundations be sufficiently setback from an ascending or descending slope. Recommendations for the proposed foundation setbacks are provided in the *Foundation Setback* section of this report (Section 7.4).

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

7.1.8 Excavations up to 12 feet in vertical height may be required for construction of structures tucked into existing slopes, including foundation. Due to the depth of the excavation and the proximity to the property lines, city streets, and adjacent offsite structures, excavation of the proposed tucked in portions will require sloping and/or shoring measures in order to provide a stable excavation. Where shoring is required it is recommended that a soldier pile shoring system be utilized. Recommendations for *Shoring* are provided in Section 8.18.

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

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

7.1.11 Where new paving is to be placed, it is recommended that all existing fill soils and soft alluvial soils be excavated and properly compacted for paving support. The client should be aware that excavation and compaction of all existing fill in the area of new paving is not required, however, paving constructed over existing uncertified fill or unsuitable soils may experience increased settlement and/or cracking, and may therefore have a shorter design life and increased maintenance costs. As a minimum, the upper 12 inches of soil should be scarified and properly compacted. Paving recommendations are provided in the *Preliminary Pavement Recommendations* section of this report (see Section 7.12).

7.1.12 Based on the results of the percolation testing performed at the site, a stormwater infiltration system is not considered feasible for this project. A discussion of the test results is provided in the *Stormwater Infiltration* section of this report (see Section 7.20).

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

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

7.2 Soil and Excavation Characteristics

7.2.1 The in-situ soils can be excavated with moderate effort using conventional excavation equipment. Minor caving should be anticipated in unshored excavations, especially where granular soils are encountered.

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

7.2.3 All onsite excavations must be conducted in such a manner that potential surcharges from existing structures, construction equipment, and vehicle loads are resisted. The surcharge area may be defined by a 1:1 projection down and away from the bottom of an existing foundation or vehicle load. Penetrations below this 1:1 projection will require special excavation measures such as sloping or shoring. Excavation recommendations are provided in the *Temporary Excavations* section of this report (see Section 7.15).

7.2.4 The upper 5 feet of soils encountered during the investigation are considered to have a “low” to “moderate” (EI = 37 and 69) expansive potential and are classified as “expansive” based on the 2016 California Building Code (CBC) Section 1803.5.3. Recommendations presented herein assume that the building foundations and slabs will derive support in these materials.

7.3 Minimum Resistivity, pH, and Water-Soluble Sulfate

7.3.1 Potential of Hydrogen (pH) and resistivity testing as well as chloride content testing were performed on representative samples of soil to generally evaluate the corrosion potential to surface utilities. The tests were performed in accordance with California Test Method Nos. 643 and 422 and indicate that the soils are considered “corrosive” to “severely corrosive” with respect to corrosion of buried ferrous metals on site. The results are presented in Appendix B (Figure B9) and should be considered for design of underground structures.

7.3.2 Laboratory tests were performed on representative samples of the on-site soil to measure the percentage of water-soluble sulfate content. Results from the laboratory water-soluble sulfate tests are presented in Appendix B (Figure B9) and indicate that the on-site soil possess a “negligible” to “moderate” sulfate exposure to concrete structures as defined by 2016 CBC Section 1904.3 and ACI 318-11 Section 4.2 and 4.3. The following table presents a summary of concrete requirements set forth by 2016 CBC Section 1904.3 and ACI 318. The presence of water-soluble sulfates is not a visually discernible characteristic; therefore, other soil samples from the site could yield different concentrations. Additionally, over time landscaping activities (i.e., addition of fertilizers and other soil nutrients) may affect the concentration.

REQUIREMENTS FOR CONCRETE EXPOSED TO SULFATE-CONTAINING SOLUTIONS

Sulfate Exposure	Water-Soluble Sulfate Percent by Weight	Cement Type	Maximum Water to Cement Ratio by Weight	Minimum Compressive Strength (psi)
Negligible	0.00-0.10	--	--	--
Moderate	**0.10-0.20**	**II**	**0.50**	**4000**
Severe	0.20-2.00	V	0.45	4500
Very Severe	> 2.00	V	0.45	4500

7.3.3 It is recommended that after finish pad grades have been achieved, laboratory testing of the subgrade soil be performed to confirm the corrosivity characteristics of the soils.

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

7.4 Grading

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

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

7.4.3 Grading should commence with the removal of existing vegetation and existing improvements from the area to be graded. Deleterious debris such as wood and root structures should be exported from the site and should not be mixed with the fill soils. Asphalt and concrete should not be mixed with the fill soils unless approved in writing 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.).

7.4.4 As a minimum, it is recommended that the upper 5 feet of existing earth materials within the proposed building footprint areas be excavated and properly compacted for foundation and slab support. Deeper excavations should be conducted as necessary to remove existing artificial fill or soft alluvial soil at the direction of the Geotechnical Engineer (a representative of Geocon). Proposed building foundations should be underlain by a minimum of 3 feet of newly placed engineered fill. The excavation should extend laterally a minimum distance of 3 feet beyond the building footprint areas, including building appurtenances, or a distance equal to the depth of fill below the foundation, whichever is greater.

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

7.4.6 The City of Los Angeles Department of Building and Safety requires a minimum compactive effort of 95 percent of the laboratory maximum dry density in accordance with ASTM D 1557 (latest edition) where the soils to be utilized in the fill have less than 15 percent finer than 0.005 millimeters. Soils with more than 15 percent finer than 0.005 millimeters may be compacted to 90 percent of the laboratory maximum dry density in accordance with ASTM D 1557 (latest edition). Based on the soils encountered during this investigation, it is anticipated that 90 percent relative compaction will be required. 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 the required degree of compaction in accordance with ASTM D 1557 (latest edition).

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

7.4.8 Import may be necessary to bring the site to finished grade elevations. 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. In order to achieve the geotechnical design values provided herein, the imported soil must meet the following soil parameters:

Cohesion: minimum of 150 psf (unless Phi Angle is greater than 33 degrees)
Phi Angle: minimum of 27 degrees
Expansion Index: less than 40
Water-Soluble Sulfate content: less than 0.10 percent

Import soils placed in the building area should be placed uniformly across the building pad or in a manner that is approved by the Geotechnical Engineer (a representative of Geocon).

7.4.9 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 (see Section 7.5). Prior to placing any bedding materials or pipes, the trench excavation bottom must be observed and approved in writing by the Geotechnical Engineer (a representative of Geocon).

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

7.5 Controlled Low Strength Material (CLSM)

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

Standard Requirements

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

Requirements for CLSM that will be used for support of footings

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

7.6 Shrinkage

7.6.1 Shrinkage results when a volume of material removed at one density is compacted to a higher density. A shrinkage factor of between 10 and 20 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.

7.4.2 If import soils will be utilized in building pad areas, 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.

7.7 Foundation Setback

7.7.1 The Building Code requires that foundations be sufficiently setback from an ascending or descending slope. The required horizontal setback from a descending slope is 1/3 the height of the descending slope with a minimum of 5 feet and a maximum of 40 feet measured horizontally from the exterior face of the foundation to the slope face. The required horizontal setback from an ascending slope is 1/2 the height of the ascending slope with a maximum of 15 feet measured horizontally from the exterior face of the structure to the toe of the slope. Foundations may be deepened as necessary to achieve the required setback be penetrating the projected setback projection measured from the toe of the slope to the setback location at the ground surface.

7.7.2 The required setbacks should be understood and implemented into the orientation and location of the proposed structure by the project architect.

7.8 Conventional Foundation Design

7.8.1 Subsequent to the recommended grading, a conventional shallow spread foundation system may be utilized for support of the proposed structure and retaining walls provided foundations derive support in newly placed engineered. Foundations should be underlain by a minimum of 3 feet of newly placed engineered fill.

7.8.2 Continuous footings may be designed for an allowable bearing capacity of 2,200 pounds per square foot (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.

7.8.3 Isolated spread foundations may be designed for an allowable bearing capacity of 2,450 psf, and 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.

7.8.4 The allowable soil bearing pressure above may be increased by 50 psf and 250 psf for each additional foot of foundation width and depth, respectively, up to a maximum allowable soil bearing pressure of 3,450 psf.

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

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

7.8.7 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.

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

7.8.9 Due to the expansive potential of the subgrade soils, the moisture content in the slab and foundation subgrade should be maintained at 2 percent above optimum moisture content prior to and at the time of concrete placement.

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

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

7.9 Foundation Settlement

7.9.1 The enclosed seismically induced settlement analyses indicate that the site soils could be prone to approximately 0.14 inches of total settlement as a result of the Design Earthquake peak ground acceleration ($\frac{2}{3}PGA_M$). Differential settlement at the foundation level is anticipated to be less than 0.1 inches over a distance of 20 feet. These settlements are in addition to the static settlements indicated below and must be considered in the structural design.

7.9.2 The maximum expected static settlement for a structure supported on a conventional foundation system deriving support in the recommended bearing materials and designed with a maximum bearing pressure of 3,450 psf is estimated to be less than 1 inch and occur below the heaviest loaded structural element. Settlement of the foundation system is expected to occur on initial application of loading. Differential settlement is not expected to exceed ½ inch over a distance of 20 feet.

7.9.3 Based on seismic considerations, the proposed structure supported on a conventional foundation system should be designed for a combined static and seismically induced total settlement of approximately 1.14 inches, and a combined static and seismically induced differential settlement of 0.6 inches over a distance of 20 feet.

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

7.10 Miscellaneous Foundations

7.10.1 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, and should be deepened as necessary to maintain a minimum 12-inch embedment into the recommended bearing materials.

7.10.2 If the soils exposed in the excavation bottom are loose, compaction of the soils will be required prior to placing steel or concrete. Compaction of the foundation excavation bottom is typically accomplished with a compaction wheel or mechanical whacker and must be observed and approved by a Geocon representative. 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.

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

7.11 Lateral Design

7.11.1 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 0.25 may be used with the dead load forces in properly compacted engineered fill and undisturbed alluvial soils.

7.11.2 Passive earth pressure for the sides of foundations and slabs poured against the alluvial soils or properly compacted engineered fill may be computed as an equivalent fluid having a density of 175 pounds per cubic foot (pcf) with a maximum earth pressure of 1,750 pcf. When combining passive and friction for lateral resistance, the passive component should be reduced by one-third.

7.12 Concrete Slabs-on-Grade

7.12.1 Concrete slabs-on-grade subject to vehicle loading should be designed in accordance with the recommendations in the *Pavement Recommendations* section of this report (Section 7.12).

7.12.2 Subsequent to the recommended grading, concrete slabs-on-grade for structures, not subject to vehicle loading, should be a minimum of 4 inches thick and minimum slab reinforcement should consist of No. 4 steel reinforcing bars placed 16 inches on center in both horizontal directions. Steel reinforcing should be positioned vertically near the slab midpoint.

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

7.12.4 For seismic design purposes, a coefficient of friction of 0.25 may be utilized between concrete slabs and subgrade soils without a moisture barrier, and 0.15 for slabs underlain by a moisture barrier.

7.12.5 Exterior slabs for walkways or flatwork, not subject to traffic loads, should be at least 4 inches thick and reinforced with No. 3 steel reinforcing bars placed 18 inches on center in both horizontal directions, positioned near the slab midpoint. Prior to construction of slabs, the upper 12 inches of subgrade should be moistened to 2 percent above optimum moisture content and properly compacted to at least 92 percent relative compaction, as determined by ASTM Test Method D 1557 (latest edition). Crack control joints should be spaced at intervals not greater than 8 feet and should be constructed using saw-cuts or other methods as soon as practical following concrete placement. Crack control joints should extend a minimum depth of one-fourth the slab thickness. The project structural engineer should design construction joints as necessary.

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

7.13 Preliminary Pavement Recommendations

7.13.1 Where new paving is to be placed, it is recommended that all existing fill and soft alluvium materials be excavated and properly compacted for paving support. The client should be aware that excavation and compaction of all existing artificial fill and soft alluvium in the area of new paving is not required; however, paving constructed over existing uncertified fill or unsuitable alluvium material may experience increased settlement and/or cracking, and may therefore have a shorter design life and increased maintenance costs. As a minimum, the upper 12 inches of paving subgrade should be scarified, moisture conditioned to 2 percent above optimum moisture content, and properly compacted to at least 92 percent relative compaction, as determined by ASTM Test Method D 1557 (latest edition).

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

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

PRELIMINARY PAVEMENT DESIGN SECTIONS

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

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

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

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

7.14 Retaining Walls Design

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

7.14.2 Retaining wall foundations may be designed in accordance with the recommendations provided in the *Foundation Design* section of this report (see Section 7.7).

7.14.3 Retaining walls with select backfill (EI less than 20) with a level backfill surface that are not restrained at the top should be designed utilizing a triangular distribution of pressure (active pressure) of 30 pcf. 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 with a level backfill surface may be designed utilizing a triangular distribution of pressure (at-rest pressure) of 50 pcf.

7.14.4 Retaining walls with select backfill (EI less than 20) with a 2:1 (H:V) slope backfill that are not restrained at the top should be designed utilizing a triangular distribution of pressure (active pressure) of 45 pcf. 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 with a 2:1 (H:V) slope backfill surface may be designed utilizing a triangular distribution of pressure (at-rest pressure) of 65 pcf.

7.14.5 Retaining walls backfilled with relatively undisturbed alluvial soils or engineered fill derived from onsite soils 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. 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 with a level backfill surface may be designed utilizing a triangular distribution of pressure (at-rest pressure) of 60 pcf.

7.14.6 Retaining walls backfilled with relatively undisturbed alluvial soils or engineered fill derived from onsite soils with a 2:1 (H:V) slope backfill that are not restrained at the top should be designed utilizing a triangular distribution of pressure (active pressure) of 55 pcf. 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 with a 2:1 (H:V) slope backfill surface may be designed utilizing a triangular distribution of pressure (at-rest pressure) of 75 pcf.

7.14.7 If import soil will be used to backfill proposed retaining walls, revised earth pressures may be required to account for the geotechnical properties of the import soil used as engineered fill. This should be evaluated once the use of import soil is established. All imported fill shall be observed, tested, and approved by Geocon West, Inc. prior to bringing soil to the site.

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

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

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

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

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

and

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

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

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

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

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

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

and

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

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

then

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

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

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

7.15 Dynamic (Seismic) Lateral Forces

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

7.15.2 A seismic load of 10 pcf should be used for design on walls that support more than 6 feet of backfill in accordance with Section 1803.5.12 of the 2016 CBC. The seismic load is applied as an equivalent fluid pressure along the height of the wall and the calculated loads result in a maximum load exerted at the base of the wall and zero at the top of the wall. This seismic load should be applied in addition to the active earth pressure. The earth pressure is based on half of two thirds of PGA_M calculated from ASCE 7-10 Section 11.8.3.

7.16 Retaining Wall Drainage

7.16.1 Retaining walls should be provided with a drainage system extended at least two-thirds the height of the wall. At the base of the drain system, a subdrain covered with a minimum of 12 inches of gravel should be installed, and a compacted fill blanket or other seal placed at the surface (see Figure 18). The clean bottom and subdrain pipe, behind a retaining wall, should be observed by the Geotechnical Engineer (a representative of Geocon), prior to placement of gravel or compacting backfill.

7.16.2 As an alternative, a plastic drainage composite such as Miradrain or equivalent may be installed in continuous, 4-foot-wide columns along the entire back face of the wall, at 8 feet on center. The top of these drainage composite columns should terminate approximately 18 inches below the ground surface, where either hardscape or a minimum of 18 inches of relatively cohesive material should be placed as a cap (see Figure 19).

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

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

7.17 Temporary Excavations

7.17.1 Excavations up to 12 feet in height may be required for the tucked in portions. Excavation is anticipated to expose artificial fill and alluvium, which are considered suitable for vertical excavations up to 5 feet in height where loose soils or caving sands are not present and where excavations are not surcharged by adjacent traffic or structures.

7.17.2 Vertical excavations greater than 5 feet or where surcharged by existing structures will require sloping or shoring measures in order to provide a stable excavation. Where sufficient space is available, temporary unsurcharged embankments could be sloped back at a uniform 1:1 slope gradient or flatter. A uniform slope does not have a vertical portion.

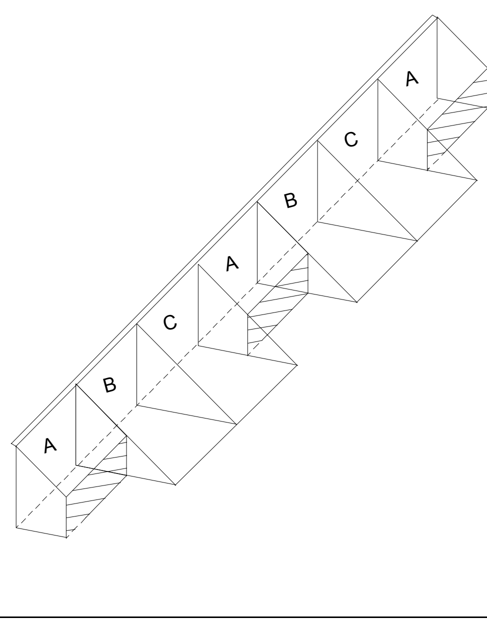
7.17.3 If excavations in close proximity to an adjacent property line and/or structure are required, special excavation measures such as slot-cutting or shoring may be necessary in order to maintain lateral support of offsite improvements. Recommendations for slot cutting are provided in Sections 7.18 of this report.

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

7.18 Slot Cutting

7.18.1 The slot-cutting method employs the earth as a buttress and allows the earth excavation to proceed in phases. Where slot-cutting is used for foundation construction, the proposed construction techniques should be discussed with the structural engineer so that appropriate modifications can be made to the foundation design; such as additional reinforcing or details for doweling.

7.18.2 It is recommended that the initial temporary excavation along the property line or adjacent foundation be sloped back at a uniform 1:1 (H:V) slope gradient or flatter for excavation of the existing soils to the necessary depth. The temporary excavation should not extend below the surcharge area of any adjacent foundations. The surcharge area may be defined by a 1:1 projection down and away from the bottom of an existing foundation. The temporary slope may then be excavated using the slot-cutting (see illustration below).



7.18.3 Alternate "A" slots of 8 feet in width may be worked. The remaining earth buttresses ("B" and "C" slots) should also be 8 feet in width. The wall, foundation, or backfill should be completed in the "A" slots to a point where support of the offsite property and/or any existing structures is restored before the "B" slots are excavated. After completing the wall, foundation, or backfill in the "B" slots, finally the "C" slots may be excavated. Slot-cutting is not recommended for vertical excavations greater than 12 feet in height, or for backslopes steeper than 1:1 (H:V). The slot-cut calculation should be revised as needed for each surcharge condition as the project progresses. A slot-cut calculation is provided below.

Slot Cut Calculation

Input:

Height of Slots	(H)	12.0	feet
Unit Weight of Soils	(γ)	120.0	pcf
Friction Angle of Soils	(ϕ)	28.0	degrees
Cohesion of Soils	(c)	170.0	psf
Factor of Safety	(FS)	1.25	

Factor of Safety = Resistance Force/Driving Force

Design Equations

$b = H/(\tan \alpha)$

$A = 0.5*H*b$

$W = 0.5*H*b*\gamma$ *(per lineal foot of slot width)*

$F_1 = d*W*(\sin \alpha)$

$R_1 = d*[W*(\cos\alpha)*(\tan \phi)+(c*b)]$

$R_2 = 2*[(0.5*H*b)*c]$

FS = Resistance Force/Driving Force

$FS = (R_1+R_2)/(F_1)$

Surcharge Pressure:

Line Load	(q_L)	0.0	plf
Distance Away from Edge of Excavation	(X)	0.0	feet

Failure Angle (α) degrees	Width of Failure Wedge (b) feet	Area of Failure Wedge (A) feet^2	Weight of Failure Wedge (W) lbs/lineal foot	**Driving Force** Wedge + Surcharge per lineal foot of Slot Wdith	**Resisting Force** Failure Wedge per lineal foot of Slot Width	**Resisting Force** Side Resistance Force lbs	**Allowable Width of Slots*** (d) feet
45	12.0	72	8640.0	6109.4	6133.4	24480.0	8.0
46	11.6	70	8343.6	6001.8	5917.7	23640.1	8.0
47	11.2	67	8056.9	5892.5	5711.0	22828.0	8.0
48	10.8	65	7779.5	5781.3	5512.9	22041.9	8.0
49	10.4	63	7510.6	5668.4	5323.0	21280.1	8.0
50	10.1	60	7249.8	5553.7	5140.8	20541.2	8.0
51	9.7	58	6996.5	5437.3	4966.1	19823.5	8.0
52	9.4	56	6750.3	5319.3	4798.5	19125.9	8.0
53	9.0	54	6510.7	5199.7	4637.7	18447.0	8.0
54	8.7	52	6277.3	5078.5	4483.4	17785.8	8.0
55	8.4	50	6049.8	4955.7	4335.4	17141.1	8.0
56	8.1	49	5827.8	4831.4	4193.4	16512.0	8.0
57	7.8	47	5610.9	4705.7	4057.3	15897.5	8.0
58	7.5	45	5398.9	4578.5	3926.7	15296.8	8.0
59	7.2	43	5191.4	4449.9	3801.6	14709.1	8.0
60	6.9	42	4988.3	4320.0	3681.8	14133.5	8.0
61	6.7	40	4789.2	4188.8	3567.0	13569.5	8.0
62	6.4	38	4594.0	4056.2	3457.2	13016.2	8.0
63	6.1	37	4402.3	3922.5	3352.2	12473.2	8.0
64	5.9	35	4214.0	3787.5	3251.9	11939.7	8.0
65	5.6	34	4028.9	3651.4	3156.2	11415.2	8.0
66	5.3	32	3846.8	3514.2	3065.0	10899.2	8.0
67	5.1	31	3667.5	3375.9	2978.1	10391.1	8.0
68	4.8	29	3490.8	3236.6	2895.5	9890.6	8.0
69	4.6	28	3316.6	3096.3	2817.1	9397.0	8.0
70	4.4	26	3144.7	2955.1	2742.8	8910.0	8.0

* Width of Slots to achieve a minimum of 1.25 Factor of Safety, with a Maximum Allowable Slot Width of 8-feet.

Critical Slot Width with Factor of Safety equal or exceeding 1.25:

d_{allow} = 8.0 feet

7.19 Shoring – Soldier Pile Design and Installation

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

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

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

7.19.4 The proposed soldier piles may also be designed as permanent piles. The required pile depths, dimensions, and spacing should be determined and designed by the project structural and shoring engineers. All piles utilized for shoring can also be incorporated into a permanent retaining wall system (shotcrete wall) and should be designed in accordance with the earth pressure provided in the *Retaining Wall Design* section of this report (see Section 8.13).

7.19.5 Drilled cast-in-place soldier piles should be placed no closer than three diameters on center. The minimum diameter of the piles is 18 inches. Structural concrete should be used for the soldier piles below the excavation; lean-mix concrete may be employed above that level. As an alternative, lean-mix concrete may be used throughout the pile where the reinforcing consists of a wideflange section. The slurry must be of sufficient strength to impart the lateral bearing pressure developed by the wideflange section to the soil. For design purposes, an allowable passive value for the soils below the bottom plane of excavation may be assumed to be 175 psf per foot. The allowable passive value may be doubled for isolated piles, spaced a minimum of three times the pile diameter. To develop the full lateral value, provisions should be implemented to assure firm contact between the soldier piles and the undisturbed bedrock.

7.19.6 Local seepage may be encountered during excavations for the proposed soldier piles, especially if conducted during the rainy season. If more than 6 inches of water is present in the bottom of the excavation, a tremie is required to place the concrete into the bottom of the hole. A tremie should consist of a rigid, water-tight tube having a diameter of not less than 6 inches with a hopper at the top. The tube should be equipped with a device that will close the discharge end and prevent water from entering the tube while it is being charged with concrete. The tremie should be supported so as to permit free movement of the discharge end over the entire top surface of the work and to permit rapid lowering when necessary to retard or stop the flow of concrete. The discharge end should be closed at the start of the work to prevent water entering the tube and should be entirely sealed at all times, except when the concrete is being placed. The tremie tube should be kept full of concrete. The flow should be continuous until the work is completed and the resulting concrete seal should be monolithic and homogeneous. The tip of the tremie tube should always be kept about 5 feet below the surface of the concrete and definite steps and safeguards should be taken to insure that the tip of the tremie tube is never raised above the surface of the concrete.

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

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

7.19.9 The frictional resistance between the soldier piles and retained earth may be used to resist a vertical component load. The coefficient of friction may be taken as 0.25 based on uniform contact between the steel beam and lean-mix concrete and retained earth. The portion of soldier piles below the plane of excavation may also be employed to resist the downward loads. The downward capacity may be determined using a frictional resistance of 200 psf.

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

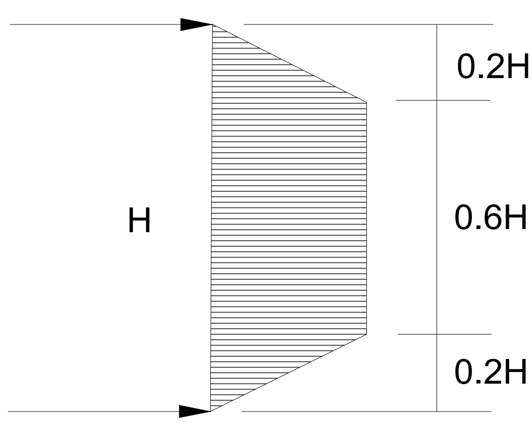
7.19.11 The time between lagging excavation and lagging placement should be as short as possible. Soldier piles should be designed for the full-anticipated pressures. Due to arching in the soils, the pressure on the lagging will be less. It is recommended that the lagging be designed for the full design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

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

HEIGHT OF SHORING (FEET)	EQUIVALENT FLUID PRESSURE - Level Backfill Surface (Pounds Per Cubic Foot) (ACTIVE PRESSURE)	EQUIVALENT FLUID PRESSURE (Pounds Per Square Foot per Foot) Trapezoidal –Active (Where H is the height of the shoring in feet)
Up to 12	25	16H

HEIGHT OF SHORING (FEET)	EQUIVALENT FLUID PRESSURE - 2:1 (H:V) Slope Backfill (Pounds Per Cubic Foot) (ACTIVE PRESSURE)	EQUIVALENT FLUID PRESSURE (Pounds Per Square Foot per Foot) Trapezoidal –Active (Where H is the height of the shoring in feet)
Up to 12	26	17H

Trapezoidal Distribution of Pressure



7.19.13 It is very important to note that active pressures can only be achieved when movement in the soil (earth wall) occurs. If movement in the soil is not acceptable, such as adjacent to an existing structure, an at-rest pressure of 46 pcf should be considered for design purposes.

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

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

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

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

and

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

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

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

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

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

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

and

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

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

then

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

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

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

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

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

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

7.20 Stormwater Infiltration

7.20.1 During the April 13, 2018, site exploration, borings B18, B19, and B20 were utilized to perform percolation testing. The borings were advanced to the depths listed in the table below. Slotted casing was placed in the borings, and the annular space between the casing and excavation was filled with gravel. The borings were then filled with water to pre-saturate the soils. The casings were refilled with water and percolation test readings were performed after repeated flooding of the cased excavation. Based on the test results, the measured percolation rate and design infiltration rate, for the earth materials encountered, are provided in the following table. These values have been calculated in accordance with the Boring Percolation Test Procedure in the County of Los Angeles Department of Public Works GMED *Guidelines for Geotechnical Investigation and Reporting, Low Impact Development Stormwater Infiltration* (June 2017).

Boring	Soil Type	Infiltration Depth (ft)	Measured Percolation Rate (in / hour)	Design Infiltration Rate (in / hour)
B18	Silty Sand (SM)	2-5	0.13	0.03
B19	Silty Sand (SM)	2-5	0.15	0.04
B20	Silty Sand (SM)	2-5	0.61	0.15

7.20.2 The results of the percolation testing indicate that soils at the locations and depths listed in the table above are minimally conductive to infiltration. The infiltration rate of the soils is less than 0.3 inches per hour (the typical minimal infiltration rate required). Based on these considerations, a stormwater infiltration system is likely not feasible for this project; however, the project civil engineer should evaluate these results.

7.21 Surface Drainage

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

7.21.2 All site drainage should be collected and controlled in non-erosive drainage devices. Drainage should not be allowed to pond anywhere on the site, and especially not against any foundation or retaining wall. The site should be graded and maintained such that surface drainage is directed away from structures in accordance with 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. If planters adjacent to foundations consist of draught tolerant plants irrigated with a drip system, and saturating of the soil is not anticipated, then sealed planters are not necessary.

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

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

7.22 Plan Review

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

LIMITATIONS AND UNIFORMITY OF CONDITIONS

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

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

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

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

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GEOCON WEST, INC.

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DRAFTED BY: MDS	CHECKED BY: SFK

VICINITY MAP

ROSE HILL COURTS REDEVELOPMENT

4446 FLORIZEL STREET

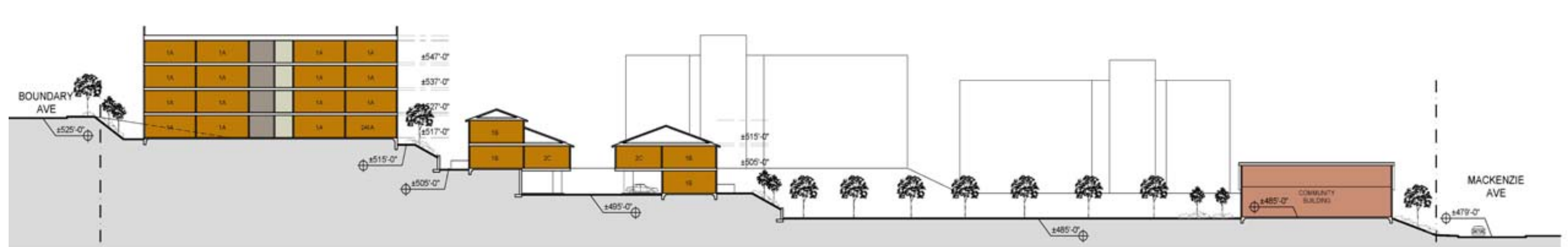
LOS ANGELES, CALIFORNIA

MAY 2018	PROJECT NO. A9514-06-01	FIG. 1

LEGEND

B20 Approximate Location of Boring

0 60' 120'



A - A'
NO SCALE

GEOCON
W E S T, I N C.
ENVIRONMENTAL GEOTECHNICAL MATERIALS
3303 N. SAN FERNANDO BLVD. - SUITE 100 - BURBANK, CA 91504
PHONE (818) 841-8388 - FAX (818) 841-1704

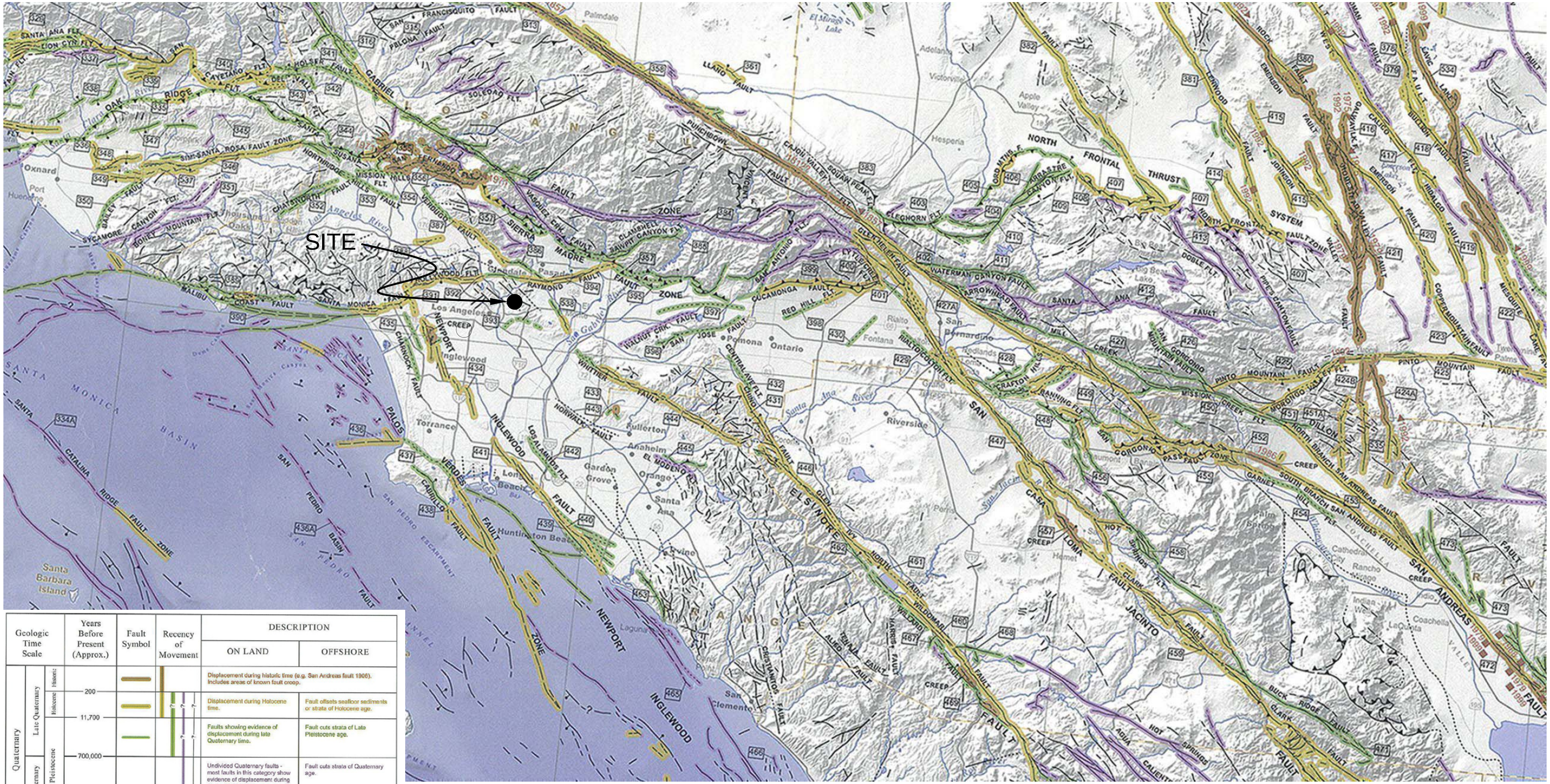
DRAFTED BY: MDS	CHECKED BY: SFK

SITE PLAN

ROSE HILL COURTS REDEVELOPMENT
4446 FLORIZEL STREET LOS ANGELES, CALIFORNIA

MAY 2018	PROJECT NO. A9514-06-01	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				Displacement during historic time (e.g. San Andreas fault 1906). Includes areas of known fault creep.	
			200				
		Holocene			? ? ?	Displacement during Holocene time.	Fault offsets seafloor sediments or strata of Holocene age.
			11,700				
		Pleistocene			? ?	Faults showing evidence of displacement during late Quaternary time.	Fault cuts strata of Late Pleistocene age.
			700,000				
	Early Quaternary				?	Undivided Quaternary faults - most faults in this category show evidence of displacement during the last 1,600,000 years; possible exceptions are faults which displace rocks of undifferentiated Plio-Pleistocene age.	Fault cuts strata of Quaternary age.
			1,600,000*				
Pre-Quaternary			4.5 billion (Age of Earth)			Faults without recognized Quaternary displacement or showing evidence of no displacement during Quaternary time. Not necessarily inactive.	Fault cuts strata of Pliocene or older age.

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

0 12 24 Miles

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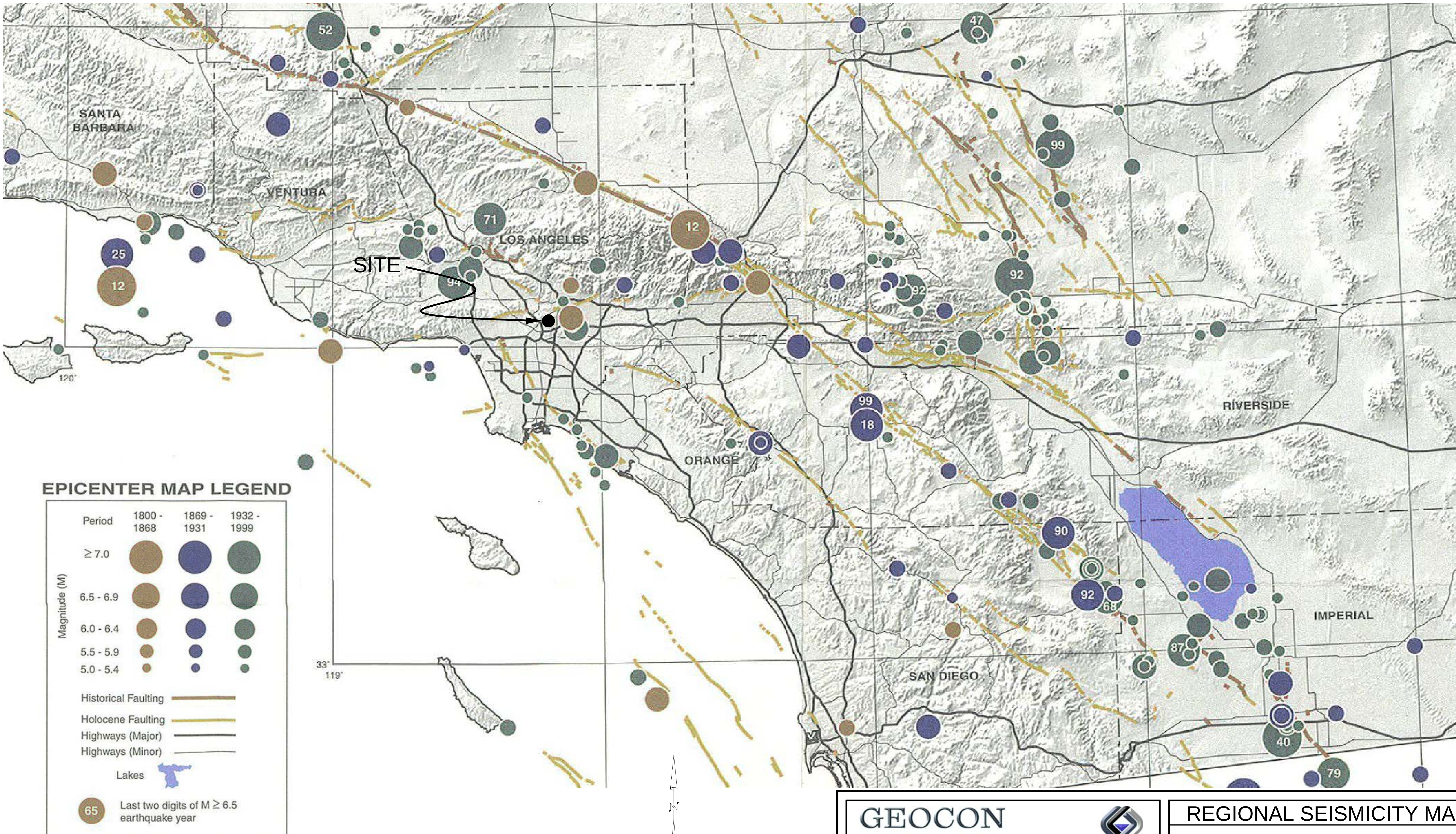
DRAFTED BY: MDS | CHECKED BY: SFK

REGIONAL FAULT MAP

ROSE HILL COURTS REDEVELOPMENT
4446 FLORIZEL STREET
LOS ANGELES, CALIFORNIA

MAY 2018 | PROJECT NO. A9514-06-01 | FIG. 3



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

N

0 20 40 Miles

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DRAFTED BY: MDS | CHECKED BY: SFK

REGIONAL SEISMICITY MAP

ROSE HILL COURTS REDEVELOPMENT
4446 FLORIZEL STREET
LOS ANGELES, CALIFORNIA

MAY 2018 | PROJECT NO. A9514-06-01 | FIG. 4

B2 @ 30 (Layer from 30' - 35')

$$\text{Liquidity Index} = \frac{W_n - PL}{PI}$$

$W_n = 20.7\%$
$LL = 29$
$PL = 19$
$PI = 10$

$$LI_{@30} = \frac{20.7 - 19}{10} = 0.17$$

$$\sigma'_v = (101.8 \text{ pcf})(30\text{ft}) = 3054 \text{ psf} = 1.49 \text{ kg/cm}^2$$

From Figure 2-20 (Report No. UCD/CGM-04/01)

Sensitivity $(S_t) = 1.5$ (low sensitive)

The layer is not subject to liquefaction.

B2@45 (layer from 45'-46')

$W_n = 22.3\%$
$LL = 29$
$PL = 19$
$PI = 10$

$$LI_{@45} = \frac{22.3 - 19}{10} = 0.33$$

$$\sigma'_v = (109.1 \text{ pcf})(45 \text{ ft}) - 62.4 \text{ pcf}(5\text{ft}) = 4597.5 \text{ psf}$$
$$= 2.24 \text{ kg/cm}^2$$

From Figure 2-20 (Report No. UCD/CGM-04/01)

sensitivity $(S_t) = 3$ (low sensitive)

The layer is not subject to liquefaction.

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DRAFTED BY: PZ	CHECKED BY: NDB

LIQUEFACTION SENSITIVITY ANALYSIS

ROSE HILL COURTS REDEVELOPMENT

4446 FLORIZEL STREET
LOS ANGELES, CALIFORNIA

DEC. 2016	PROJECT NO. A9514-06-01	FIG. 5

GEOCON

Client : Rose Hills Courts
File No. : A9514-06-01
Boring : 1

EMPIRICAL ESTIMATION OF LIQUEFACTION POTENTIAL
DESIGN EARTHQUAKE

NCEER (1996) METHOD

By Thomas F. Blake (1994-1996)

EARTHQUAKE INFORMATION:

Earthquake Magnitude:	6.62
Peak Horiz. Acceleration PGA_M (g):	1.053
2/3 PGA_M (g):	0.702
Calculated Mag.Wtg.Factor:	0.730
Historic High Groundwater:	20.0
Groundwater Depth During Exploration:	20.0

ENERGY & ROD CORRECTIONS:

Energy Correction (CE) for N60:	1.25
Rod Len.Corr.(CR)(0-no or 1-yes):	1.0
Bore Dia. Corr. (CB):	1.15
Sampler Corr. (CS):	1.20
Use Ksigma (0 or 1):	1.0

LIQUEFACTION CALCULATIONS:

Unit Wt. Water (pcf):	62.4

Depth to Base (ft)	Total Unit Wt. (pcf)	Water (0 or 1)	FIELD SPT (N)	Depth of SPT (ft)	Liq.Sus. (0 or 1)	-200 (%)	Est. Dr (%)	CN Factor	Corrected (N1)60	Eff. Unit Wt. (psf)	Resist. CRR	rd Factor	Induced CSR	Liquefac. Safe.Fact.
1.0	119.7	0	8.0	7.5	1		58	2.000	20.7	119.7	0.226	0.998	0.333	--
2.0	119.7	0	8.0	7.5	1	0	58	2.000	20.7	119.7	0.226	0.993	0.331	--
3.0	119.7	0	8.0	7.5	1	0	58	2.000	20.7	119.7	0.226	0.989	0.329	--
4.0	119.7	0	8.0	7.5	1	0	58	2.000	20.7	119.7	0.226	0.984	0.328	--
5.0	119.7	0	8.0	7.5	1	0	58	1.969	20.4	119.7	0.222	0.979	0.326	--
6.0	119.7	0	8.0	7.5	1	0	58	1.781	18.4	119.7	0.200	0.975	0.325	--
7.0	119.7	0	8.0	7.5	1	0	58	1.638	17.0	119.7	0.184	0.970	0.323	--
8.0	119.7	0	8.0	7.5	1	0	58	1.525	15.8	119.7	0.172	0.966	0.322	--
9.0	119.7	0	8.0	7.5	1	0	58	1.432	14.8	119.7	0.162	0.961	0.320	--
10.0	116.0	0	26.0	12.5	1	0	95	1.356	45.6	116.0	Infin.	0.957	0.319	--
11.0	116.0	0	26.0	12.5	1	0	95	1.292	43.4	116.0	Infin.	0.952	0.317	--
12.0	116.0	0	26.0	12.5	1	0	95	1.236	41.6	116.0	Infin.	0.947	0.316	--
13.0	116.0	0	26.0	12.5	1	0	95	1.186	39.9	116.0	Infin.	0.943	0.314	--
14.5	116.0	0	26.0	12.5	1	0	95	1.132	38.1	116.0	Infin.	0.937	0.312	--
15.0	129.6	0	43.0	17.5	0	0		1.111	70.5	129.6	~	0.933	0.311	~
16.0	129.6	0	43.0	17.5	0	0		1.062	67.4	129.6	~	0.929	0.310	~
17.0	129.6	0	43.0	17.5	0	0		1.026	65.1	129.6	~	0.925	0.308	~
18.0	129.6	0	43.0	17.5	0	0		0.994	63.1	129.6	~	0.920	0.307	~
19.0	129.6	0	43.0	17.5	0	0		0.965	61.3	129.6	~	0.915	0.305	~
20.0	133.4	1	43.0	17.5	0	0		0.944	59.9	71.0	~	0.911	0.308	~
21.0	133.4	1	43.0	17.5	0	0		0.930	59.0	71.0	~	0.906	0.314	~
22.0	133.4	1	43.0	17.5	0	0		0.917	58.2	71.0	~	0.902	0.319	~
23.0	133.4	1	43.0	17.5	0	0		0.904	57.4	71.0	~	0.897	0.325	~
24.0	133.4	1	43.0	17.5	0	0		0.892	56.6	71.0	~	0.893	0.329	~
25.0	133.4	1	32.0	25.0	0	0		0.880	46.4	71.0	~	0.888	0.334	~
26.0	133.4	1	32.0	25.0	0	0		0.869	45.8	71.0	~	0.883	0.338	~
27.0	133.4	1	32.0	25.0	0	0		0.858	45.2	71.0	~	0.879	0.341	~
28.0	133.4	1	32.0	25.0	0	0		0.847	44.7	71.0	~	0.874	0.345	~
29.0	133.4	1	32.0	25.0	0	0		0.837	44.1	71.0	~	0.870	0.348	~
30.0	133.4	1	32.0	25.0	0	0		0.827	43.6	71.0	~	0.865	0.350	~
31.0	133.4	1	32.0	25.0	0	0		0.818	43.1	71.0	~	0.861	0.353	~
32.0	133.4	1	32.0	25.0	0	0		0.809	42.6	71.0	~	0.856	0.355	~
33.0	133.4	1	32.0	25.0	0	0		0.800	42.2	71.0	~	0.851	0.357	~
34.0	133.4	1	32.0	25.0	0	0		0.791	41.7	71.0	~	0.847	0.359	~
35.0	133.4	1	32.0	25.0	0	0		0.783	41.3	71.0	~	0.842	0.360	~
36.0	133.4	1	32.0	25.0	0	0		0.775	40.9	71.0	~	0.838	0.362	~
37.0	133.4	1	32.0	25.0	0	0		0.767	40.5	71.0	~	0.833	0.363	~
38.0	133.4	1	32.0	25.0	0	0		0.760	40.1	71.0	~	0.829	0.364	~
39.0	133.4	1	32.0	25.0	0	0		0.752	39.7	71.0	~	0.824	0.365	~
40.0	133.4	1	32.0	25.0	0	0		0.745	39.3	71.0	~	0.819	0.366	~
41.0	133.4	1	32.0	25.0	0	0		0.738	38.9	71.0	~	0.815	0.367	~
42.0	133.4	1	32.0	25.0	0	0		0.731	38.6	71.0	~	0.810	0.367	~
43.0	133.4	1	32.0	25.0	0	0		0.725	38.2	71.0	~	0.806	0.368	~
44.0	133.4	1	32.0	25.0	0	0		0.718	37.9	71.0	~	0.801	0.368	~
45.0	133.4	1	32.0	25.0	0	0		0.712	37.6	71.0	~	0.797	0.368	~
46.0	133.4	1	32.0	25.0	0	0		0.706	37.2	71.0	~	0.792	0.368	~
47.0	133.4	1	32.0	25.0	0	0		0.700	36.9	71.0	~	0.787	0.368	~
48.0	133.4	1	32.0	25.0	0	0		0.694	36.6	71.0	~	0.783	0.368	~
49.0	133.4	1	32.0	25.0	0	0		0.689	36.3	71.0	~	0.778	0.368	~
50.0	133.4	1	32.0	25.0	0	0		0.683	36.0	71.0	~	0.774	0.368	~

GEOCON

Client : Rose Hills Courts
File No. : A9514-06-01
Boring : 1

LIQUEFACTION SETTLEMENT ANALYSIS
DESIGN EARTHQUAKE

(SATURATED SAND AT INITIAL LIQUEFACTION CONDITION)

NCEER (1996) METHOD
EARTHQUAKE INFORMATION:

Earthquake Magnitude:	6.62
PGAM (g):	1.053
2/3 PGAM (g):	0.70
Calculated Mag.Wtg.Factor:	0.730
Historic High Groundwater:	20.0
Groundwater @ Exploration:	20.0

DEPTH TO BASE	BLOW COUNT N	WET DENSITY (PCF)	TOTAL STRESS O (TSF)	EFFECT STRESS O' (TSF)	REL. DEN. Dr (%)	ADJUST BLOWS (N1)60	Tav/σ'o	LIQUEFACTION SAFETY FACTOR	Volumetric Strain [e15} (%)	EQ. SETTLE. Pe (in.)
1	8	119.7325	0.030	0.030	58	21	0.457	--	0.00	0.00
2	8	119.7325	0.090	0.090	58	21	0.457	--	0.00	0.00
3	8	119.7325	0.150	0.150	58	21	0.457	--	0.00	0.00
4	8	119.7325	0.210	0.210	58	21	0.457	--	0.00	0.00
5	8	119.7325	0.269	0.269	58	20	0.457	--	0.00	0.00
6	8	119.7325	0.329	0.329	58	18	0.457	--	0.00	0.00
7	8	119.7325	0.389	0.389	58	17	0.457	--	0.00	0.00
8	8	119.7325	0.449	0.449	58	16	0.457	--	0.00	0.00
9	8	119.7325	0.509	0.509	58	15	0.457	--	0.00	0.00
10	26	116.0415	0.568	0.568	95	46	0.457	--	0.00	0.00
11	26	116.0415	0.626	0.626	95	43	0.457	--	0.00	0.00
12	26	116.0415	0.684	0.684	95	42	0.457	--	0.00	0.00
13	26	116.0415	0.742	0.742	95	40	0.457	--	0.00	0.00
14.5	26	116.0415	0.814	0.814	95	38	0.457	--	0.00	0.00
15	43	129.577	0.845	0.845		71	0.457	~	0.00	0.00
16	43	129.577	0.926	0.926		67	0.457	~	0.00	0.00
17	43	129.577	0.991	0.991		65	0.457	~	0.00	0.00
18	43	129.577	1.056	1.056		63	0.457	~	0.00	0.00
19	43	129.577	1.120	1.120		61	0.457	~	0.00	0.00
20	43	133.4032	1.186	1.171		60	0.463	~	0.00	0.00
21	43	133.4032	1.253	1.206		59	0.474	~	0.00	0.00
22	43	133.4032	1.320	1.242		58	0.485	~	0.00	0.00
23	43	133.4032	1.386	1.277		57	0.496	~	0.00	0.00
24	43	133.4032	1.453	1.313		57	0.505	~	0.00	0.00
25	32	133.4032	1.520	1.348		46	0.515	~	0.00	0.00
26	32	133.4032	1.586	1.384		46	0.523	~	0.00	0.00
27	32	133.4032	1.653	1.419		45	0.532	~	0.00	0.00
28	32	133.4032	1.720	1.455		45	0.540	~	0.00	0.00
29	32	133.4032	1.787	1.490		44	0.547	~	0.00	0.00
30	32	133.4032	1.853	1.526		44	0.555	~	0.00	0.00
31	32	133.4032	1.920	1.561		43	0.561	~	0.00	0.00
32	32	133.4032	1.987	1.597		43	0.568	~	0.00	0.00
33	32	133.4032	2.053	1.632		42	0.574	~	0.00	0.00
34	32	133.4032	2.120	1.668		42	0.580	~	0.00	0.00
35	32	133.4032	2.187	1.703		41	0.586	~	0.00	0.00
36	32	133.4032	2.253	1.739		41	0.592	~	0.00	0.00
37	32	133.4032	2.320	1.774		40	0.597	~	0.00	0.00
38	32	133.4032	2.387	1.810		40	0.602	~	0.00	0.00
39	32	133.4032	2.454	1.845		40	0.607	~	0.00	0.00
40	32	133.4032	2.520	1.881		39	0.612	~	0.00	0.00
41	32	133.4032	2.587	1.916		39	0.616	~	0.00	0.00
42	32	133.4032	2.654	1.952		39	0.621	~	0.00	0.00
43	32	133.4032	2.720	1.987		38	0.625	~	0.00	0.00
44	32	133.4032	2.787	2.023		38	0.629	~	0.00	0.00
45	32	133.4032	2.854	2.058		38	0.633	~	0.00	0.00
46	32	133.4032	2.920	2.094		37	0.637	~	0.00	0.00
47	32	133.4032	2.987	2.129		37	0.640	~	0.00	0.00
48	32	133.4032	3.054	2.165		37	0.644	~	0.00	0.00
49	32	133.4032	3.121	2.200		36	0.648	~	0.00	0.00
50	32	133.4032	3.187	2.236		36	0.651	~	0.00	0.00

TOTAL SETTLEMENT = 0.0 INCHES

Figure 7

GEOCON

Client : Rose Hills Courts
File No. : A9514-06-01
Boring : 2

EMPIRICAL ESTIMATION OF LIQUEFACTION POTENTIAL
DESIGN EARTHQUAKE

NCEER (1996) METHOD

By Thomas F. Blake (1994-1996)

EARTHQUAKE INFORMATION:

Earthquake Magnitude:	6.62
Peak Horiz. Acceleration PGA_M (g):	1.053
2/3 PGA_M (g):	0.702
Calculated Mag.Wtg.Factor:	0.730
Historic High Groundwater:	20.0
Groundwater Depth During Exploration:	40.0

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):	1.15
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	134.6	0	17.0	5.0	1		89	2.000	44.0	134.6	Infin.	0.998	0.333	--
2.0	134.6	0	17.0	5.0	1	0	89	2.000	44.0	134.6	Infin.	0.993	0.331	--
3.0	134.6	0	17.0	5.0	1	0	89	2.000	44.0	134.6	Infin.	0.989	0.329	--
4.0	134.6	0	17.0	5.0	1	0	89	2.000	44.0	134.6	Infin.	0.984	0.328	--
5.0	134.6	0	17.0	5.0	1	0	89	1.857	40.8	134.6	Infin.	0.979	0.326	--
6.0	135.7	0	17.0	5.0	1	0	89	1.679	36.9	135.7	Infin.	0.975	0.325	--
7.0	135.7	0	17.0	5.0	1	0	89	1.544	33.9	135.7	Infin.	0.970	0.323	--
8.0	135.7	0	17.0	5.0	1	0	89	1.436	31.6	135.7	Infin.	0.966	0.322	--
9.0	135.7	0	17.0	5.0	1	0	89	1.349	29.7	135.7	0.434	0.961	0.320	--
10.0	135.7	0	12.0	10.0	1	0	66	1.276	19.8	135.7	0.216	0.957	0.319	--
11.0	135.7	0	12.0	10.0	1	0	66	1.213	18.8	135.7	0.205	0.952	0.317	--
12.0	135.7	0	12.0	10.0	1	0	66	1.159	18.0	135.7	0.196	0.947	0.316	--
13.0	135.7	0	12.0	10.0	1	0	66	1.111	17.3	135.7	0.188	0.943	0.314	--
14.5	135.7	0	12.0	10.0	1	0	66	1.060	16.4	135.7	0.179	0.937	0.312	--
15.0	141.1	0	27.0	15.0	1	61	91	1.040	46.1	141.1	Infin.	0.933	0.311	--
16.0	141.1	0	27.0	15.0	1	61	91	0.996	44.4	141.1	Infin.	0.929	0.310	--
17.0	141.1	0	27.0	15.0	1	61	91	0.964	43.2	141.1	Infin.	0.925	0.308	--
18.5	141.1	0	27.0	15.0	1	61	91	0.928	41.9	141.1	Infin.	0.919	0.306	--
19.0	141.1	0	27.0	15.0	1	61	91	0.915	41.4	141.1	Infin.	0.914	0.305	--
20.0	137.7	1	18.0	20.0	0	61		0.885	31.6	75.3	~	0.911	0.307	~
21.0	137.7	1	18.0	20.0	0	61		0.863	31.0	75.3	~	0.906	0.312	~
22.0	137.7	1	18.0	20.0	0	61		0.842	30.4	75.3	~	0.902	0.317	~
23.0	137.7	1	18.0	20.0	0	61		0.823	29.9	75.3	~	0.897	0.322	~
24.0	137.7	1	18.0	20.0	0	61		0.805	29.4	75.3	~	0.893	0.326	~
25.0	137.7	1	18.0	20.0	0	61		0.789	28.9	75.3	~	0.888	0.330	~
26.0	122.4	1	12.0	25.0	0	58		0.774	22.3	60.0	~	0.883	0.333	~
27.0	122.4	1	12.0	25.0	0	58		0.761	22.0	60.0	~	0.879	0.337	~
28.5	122.4	1	12.0	25.0	0	58		0.745	21.7	60.0	~	0.873	0.340	~
29.0	122.4	1	12.0	25.0	0	58		0.739	21.6	60.0	~	0.869	0.341	~
30.0	126.3	1	5.0	30.0	0	59		0.724	13.2	63.9	~	0.865	0.345	~
31.0	126.3	1	5.0	30.0	0	59		0.713	13.2	63.9	~	0.861	0.348	~
32.0	126.3	1	5.0	30.0	0	59		0.703	13.1	63.9	~	0.856	0.350	~
33.0	126.3	1	5.0	30.0	0	59		0.692	13.0	63.9	~	0.851	0.352	~
34.0	126.3	1	5.0	30.0	0	59		0.682	12.9	63.9	~	0.847	0.354	~
35.0	126.3	1	4.0	35.0	0	54		0.673	11.6	63.9	~	0.842	0.355	~
36.0	126.3	1	4.0	35.0	0	54		0.664	11.6	63.9	~	0.838	0.357	~
37.0	126.3	1	4.0	35.0	0	54		0.655	11.5	63.9	~	0.833	0.358	~
38.0	126.3	1	4.0	35.0	0	54		0.647	11.5	63.9	~	0.829	0.359	~
39.0	126.3	1	4.0	35.0	0	54		0.639	11.4	63.9	~	0.824	0.360	~
40.0	133.4	1	4.0	40.0	0	52		0.633	11.4	71.0	~	0.819	0.361	~
41.0	133.4	1	4.0	40.0	0	52		0.629	11.3	71.0	~	0.815	0.362	~
42.0	133.4	1	4.0	40.0	0	52		0.624	11.3	71.0	~	0.810	0.362	~
43.0	133.4	1	4.0	40.0	0	52		0.620	11.3	71.0	~	0.806	0.363	~
44.0	133.4	1	4.0	40.0	0	52		0.616	11.3	71.0	~	0.801	0.363	~
45.0	133.4	1	7.0	45.0	0	42		0.612	14.4	71.0	~	0.797	0.363	~
46.0	133.4	1	7.0	45.0	0	42		0.608	14.3	71.0	~	0.792	0.363	~
47.0	133.4	1	7.0	45.0	0	42		0.605	14.3	71.0	~	0.787	0.363	~
48.0	125.8	1	23.0	50.0	0	42		0.601	30.9	63.4	~	0.783	0.363	~
49.0	125.8	1	23.0	50.0	0	42		0.598	30.7	63.4	~	0.778	0.363	~
50.0	125.8	1	23.0	50.0	0	42		0.595	30.6	63.4	~	0.774	0.363	~

Client : Rose Hills Courts
File No. : A9514-06-01
Boring : 2

GEOCON

LIQUEFACTION SETTLEMENT ANALYSIS
DESIGN EARTHQUAKE

(SATURATED SAND AT INITIAL LIQUEFACTION CONDITION)

NCEER (1996) METHOD
EARTHQUAKE INFORMATION:

Earthquake Magnitude:	6.62
PGAM (g):	1.053
2/3 PGAM (g):	0.70
Calculated Mag.Wtg.Factor:	0.730
Historic High Groundwater:	20.0
Groundwater @ Exploration:	40.0

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	17	134.5591	0.034	0.034	89	44	0.457	--	0.00	0.00
2	17	134.5591	0.101	0.101	89	44	0.457	--	0.00	0.00
3	17	134.5591	0.168	0.168	89	44	0.457	--	0.00	0.00
4	17	134.5591	0.235	0.235	89	44	0.457	--	0.00	0.00
5	17	134.5591	0.303	0.303	89	41	0.457	--	0.00	0.00
6	17	135.6784	0.370	0.370	89	37	0.457	--	0.00	0.00
7	17	135.6784	0.438	0.438	89	34	0.457	--	0.00	0.00
8	17	135.6784	0.506	0.506	89	32	0.457	--	0.00	0.00
9	17	135.6784	0.574	0.574	89	30	0.457	--	0.00	0.00
10	12	135.6784	0.642	0.642	66	20	0.457	--	0.00	0.00
11	12	135.6784	0.710	0.710	66	19	0.457	--	0.00	0.00
12	12	135.6784	0.777	0.777	66	18	0.457	--	0.00	0.00
13	12	135.6784	0.845	0.845	66	17	0.457	--	0.00	0.00
14.5	12	135.6784	0.930	0.930	66	16	0.457	--	0.00	0.00
15	27	141.056	0.965	0.965	91	46	0.457	--	0.00	0.00
16	27	141.056	1.053	1.053	91	44	0.457	--	0.00	0.00
17	27	141.056	1.123	1.123	91	43	0.457	--	0.00	0.00
18.5	27	141.056	1.211	1.211	91	42	0.457	--	0.00	0.00
19	27	141.056	1.247	1.247	91	41	0.457	--	0.00	0.00
20	18	137.71	1.334	1.318		32	0.462	~	0.00	0.00
21	18	137.71	1.403	1.356		31	0.472	~	0.00	0.00
22	18	137.71	1.472	1.394		30	0.482	~	0.00	0.00
23	18	137.71	1.541	1.431		30	0.491	~	0.00	0.00
24	18	137.71	1.609	1.469		29	0.500	~	0.00	0.00
25	18	137.71	1.678	1.507		29	0.509	~	0.00	0.00
26	12	122.3636	1.743	1.541		22	0.517	~	0.00	0.00
27	12	122.3636	1.804	1.570		22	0.525	~	0.00	0.00
28.5	12	122.3636	1.881	1.608		22	0.534	~	0.00	0.00
29	12	122.3636	1.912	1.623		22	0.538	~	0.00	0.00
30	5	126.2977	1.989	1.661		13	0.547	~	0.00	0.00
31	5	126.2977	2.052	1.693		13	0.553	~	0.00	0.00
32	5	126.2977	2.115	1.725		13	0.560	~	0.00	0.00
33	5	126.2977	2.178	1.757		13	0.566	~	0.00	0.00
34	5	126.2977	2.242	1.789		13	0.572	~	0.00	0.00
35	4	126.2977	2.305	1.821		12	0.578	~	0.00	0.00
36	4	126.2977	2.368	1.853		12	0.583	~	0.00	0.00
37	4	126.2977	2.431	1.885		12	0.589	~	0.00	0.00
38	4	126.2977	2.494	1.917		11	0.594	~	0.00	0.00
39	4	126.2977	2.557	1.949		11	0.599	~	0.00	0.00
40	4	133.4293	2.622	1.983		11	0.604	~	0.00	0.00
41	4	133.4293	2.689	2.018		11	0.608	~	0.00	0.00
42	4	133.4293	2.756	2.054		11	0.613	~	0.00	0.00
43	4	133.4293	2.822	2.089		11	0.617	~	0.00	0.00
44	4	133.4293	2.889	2.125		11	0.621	~	0.00	0.00
45	7	133.4293	2.956	2.160		14	0.625	~	0.00	0.00
46	7	133.4293	3.023	2.196		14	0.628	~	0.00	0.00
47	7	133.4293	3.089	2.231		14	0.632	~	0.00	0.00
48	23	125.8378	3.154	2.265		31	0.636	~	0.00	0.00
49	23	125.8378	3.217	2.297		31	0.639	~	0.00	0.00
50	23	125.8378	3.280	2.328		31	0.643	~	0.00	0.00

TOTAL SETTLEMENT = 0.0 INCHES

Figure 9

GEOCON

Client : Rose Hills Courts
File No. : A9514-06-01
Boring : 1

EMPIRICAL ESTIMATION OF LIQUEFACTION POTENTIAL
MAXIMUM CONSIDERED EARTHQUAKE

NCEER (1996) METHOD

By Thomas F. Blake (1994-1996)

EARTHQUAKE INFORMATION:

Earthquake Magnitude:	6.60
Peak Horiz. Acceleration PGA_M (g):	1.053
Calculated Mag.Wtg.Factor:	0.724
Historic High Groundwater:	20.0
Groundwater Depth During Exploration:	20.0

ENERGY & ROD CORRECTIONS:

Energy Correction (CE) for N60:	1.25
Rod Len.Corr.(CR)(0-no or 1-yes):	1.0
Bore Dia. Corr. (CB):	1.15
Sampler Corr. (CS):	1.20
Use Ksigma (0 or 1):	1.0

LIQUEFACTION CALCULATIONS:

Unit Wt. Water (pcf):	62.4

Depth to Base (ft)	Total Unit Wt. (pcf)	Water (0 or 1)	FIELD SPT (N)	Depth of SPT (ft)	Liq.Sus. (0 or 1)	-200 (%)	Est. Dr (%)	CN Factor	Corrected (N1)60	Eff. Unit Wt. (psf)	Resist. CRR	rd Factor	Induced CSR	Liquefac. Safe.Fact.
1.0	119.7	0	8.0	7.5	1	0	58	2.000	20.7	119.7	0.226	0.998	0.495	--
2.0	119.7	0	8.0	7.5	1	0	58	2.000	20.7	119.7	0.226	0.993	0.492	--
3.0	119.7	0	8.0	7.5	1	0	58	2.000	20.7	119.7	0.226	0.989	0.490	--
4.0	119.7	0	8.0	7.5	1	0	58	2.000	20.7	119.7	0.226	0.984	0.488	--
5.0	119.7	0	8.0	7.5	1	0	58	1.969	20.4	119.7	0.222	0.979	0.486	--
6.0	119.7	0	8.0	7.5	1	0	58	1.781	18.4	119.7	0.200	0.975	0.483	--
7.0	119.7	0	8.0	7.5	1	0	58	1.638	17.0	119.7	0.184	0.970	0.481	--
8.0	119.7	0	8.0	7.5	1	0	58	1.525	15.8	119.7	0.172	0.966	0.479	--
9.0	119.7	0	8.0	7.5	1	0	58	1.432	14.8	119.7	0.162	0.961	0.477	--
10.0	116.0	0	26.0	12.5	1	0	95	1.356	45.6	116.0	Infin.	0.957	0.474	--
11.0	116.0	0	26.0	12.5	1	0	95	1.292	43.4	116.0	Infin.	0.952	0.472	--
12.0	116.0	0	26.0	12.5	1	0	95	1.236	41.6	116.0	Infin.	0.947	0.470	--
13.0	116.0	0	26.0	12.5	1	0	95	1.186	39.9	116.0	Infin.	0.943	0.467	--
14.5	116.0	0	26.0	12.5	1	0	95	1.132	38.1	116.0	Infin.	0.937	0.465	--
15.0	129.6	0	43.0	17.5	0	0		1.111	70.5	129.6	~	0.933	0.462	~
16.0	129.6	0	43.0	17.5	0	0		1.062	67.4	129.6	~	0.929	0.461	~
17.0	129.6	0	43.0	17.5	0	0		1.026	65.1	129.6	~	0.925	0.458	~
18.0	129.6	0	43.0	17.5	0	0		0.994	63.1	129.6	~	0.920	0.456	~
19.0	129.6	0	43.0	17.5	0	0		0.965	61.3	129.6	~	0.915	0.454	~
20.0	133.4	1	43.0	17.5	0	0		0.944	59.9	71.0	~	0.911	0.458	~
21.0	133.4	1	43.0	17.5	0	0		0.930	59.0	71.0	~	0.906	0.467	~
22.0	133.4	1	43.0	17.5	0	0		0.917	58.2	71.0	~	0.902	0.475	~
23.0	133.4	1	43.0	17.5	0	0		0.904	57.4	71.0	~	0.897	0.483	~
24.0	133.4	1	43.0	17.5	0	0		0.892	56.6	71.0	~	0.893	0.490	~
25.0	133.4	1	32.0	25.0	0	0		0.880	46.4	71.0	~	0.888	0.496	~
26.0	133.4	1	32.0	25.0	0	0		0.869	45.8	71.0	~	0.883	0.502	~
27.0	133.4	1	32.0	25.0	0	0		0.858	45.2	71.0	~	0.879	0.508	~
28.0	133.4	1	32.0	25.0	0	0		0.847	44.7	71.0	~	0.874	0.513	~
29.0	133.4	1	32.0	25.0	0	0		0.837	44.1	71.0	~	0.870	0.517	~
30.0	133.4	1	32.0	25.0	0	0		0.827	43.6	71.0	~	0.865	0.521	~
31.0	133.4	1	32.0	25.0	0	0		0.818	43.1	71.0	~	0.861	0.525	~
32.0	133.4	1	32.0	25.0	0	0		0.809	42.6	71.0	~	0.856	0.528	~
33.0	133.4	1	32.0	25.0	0	0		0.800	42.2	71.0	~	0.851	0.531	~
34.0	133.4	1	32.0	25.0	0	0		0.791	41.7	71.0	~	0.847	0.534	~
35.0	133.4	1	32.0	25.0	0	0		0.783	41.3	71.0	~	0.842	0.536	~
36.0	133.4	1	32.0	25.0	0	0		0.775	40.9	71.0	~	0.838	0.538	~
37.0	133.4	1	32.0	25.0	0	0		0.767	40.5	71.0	~	0.833	0.540	~
38.0	133.4	1	32.0	25.0	0	0		0.760	40.1	71.0	~	0.829	0.542	~
39.0	133.4	1	32.0	25.0	0	0		0.752	39.7	71.0	~	0.824	0.543	~
40.0	133.4	1	32.0	25.0	0	0		0.745	39.3	71.0	~	0.819	0.544	~
41.0	133.4	1	32.0	25.0	0	0		0.738	38.9	71.0	~	0.815	0.545	~
42.0	133.4	1	32.0	25.0	0	0		0.731	38.6	71.0	~	0.810	0.546	~
43.0	133.4	1	32.0	25.0	0	0		0.725	38.2	71.0	~	0.806	0.547	~
44.0	133.4	1	32.0	25.0	0	0		0.718	37.9	71.0	~	0.801	0.547	~
45.0	133.4	1	32.0	25.0	0	0		0.712	37.6	71.0	~	0.797	0.548	~
46.0	133.4	1	32.0	25.0	0	0		0.706	37.2	71.0	~	0.792	0.548	~
47.0	133.4	1	32.0	25.0	0	0		0.700	36.9	71.0	~	0.787	0.548	~
48.0	133.4	1	32.0	25.0	0	0		0.694	36.6	71.0	~	0.783	0.548	~
49.0	133.4	1	32.0	25.0	0	0		0.689	36.3	71.0	~	0.778	0.547	~
50.0	133.4	1	32.0	25.0	0	0		0.683	36.0	71.0	~	0.774	0.547	~

GEOCON

Client : Rose Hills Courts
File No. : A9514-06-01
Boring : 1

LIQUEFACTION SETTLEMENT ANALYSIS
MAXIMUM CONSIDERED EARTHQUAKE

(SATURATED SAND AT INITIAL LIQUEFACTION CONDITION)

NCEER (1996) METHOD
EARTHQUAKE INFORMATION:

Earthquake Magnitude:	6.60
PGA_M (g):	1.053
Calculated Mag.Wtg.Factor:	0.724
Historic High Groundwater:	20.0
Groundwater @ Exploration:	20.0

DEPTH TO BASE	BLOW COUNT N	WET DENSITY (PCF)	TOTAL STRESS O (TSF)	EFFECT STRESS O' (TSF)	REL. DEN. Dr (%)	ADJUST BLOWS (N1)60	Tav/σ'_o	LIQUEFACTION SAFETY FACTOR	Volumetric Strain $[e_{15}]$ (%)	EQ. SETTLE. Pe (in.)
1	8	119.7325	0.030	0.030	58	21	0.684	--	0.00	0.00
2	8	119.7325	0.090	0.090	58	21	0.684	--	0.00	0.00
3	8	119.7325	0.150	0.150	58	21	0.684	--	0.00	0.00
4	8	119.7325	0.210	0.210	58	21	0.684	--	0.00	0.00
5	8	119.7325	0.269	0.269	58	20	0.684	--	0.00	0.00
6	8	119.7325	0.329	0.329	58	18	0.684	--	0.00	0.00
7	8	119.7325	0.389	0.389	58	17	0.684	--	0.00	0.00
8	8	119.7325	0.449	0.449	58	16	0.684	--	0.00	0.00
9	8	119.7325	0.509	0.509	58	15	0.684	--	0.00	0.00
10	26	116.0415	0.568	0.568	95	46	0.684	--	0.00	0.00
11	26	116.0415	0.626	0.626	95	43	0.684	--	0.00	0.00
12	26	116.0415	0.684	0.684	95	42	0.684	--	0.00	0.00
13	26	116.0415	0.742	0.742	95	40	0.684	--	0.00	0.00
14.5	26	116.0415	0.814	0.814	95	38	0.684	--	0.00	0.00
15	43	129.577	0.845	0.845		71	0.684	~	0.00	0.00
16	43	129.577	0.926	0.926		67	0.684	~	0.00	0.00
17	43	129.577	0.991	0.991		65	0.684	~	0.00	0.00
18	43	129.577	1.056	1.056		63	0.684	~	0.00	0.00
19	43	129.577	1.120	1.120		61	0.684	~	0.00	0.00
20	43	133.4032	1.186	1.171		60	0.694	~	0.00	0.00
21	43	133.4032	1.253	1.206		59	0.711	~	0.00	0.00
22	43	133.4032	1.320	1.242		58	0.727	~	0.00	0.00
23	43	133.4032	1.386	1.277		57	0.743	~	0.00	0.00
24	43	133.4032	1.453	1.313		57	0.758	~	0.00	0.00
25	32	133.4032	1.520	1.348		46	0.772	~	0.00	0.00
26	32	133.4032	1.586	1.384		46	0.785	~	0.00	0.00
27	32	133.4032	1.653	1.419		45	0.797	~	0.00	0.00
28	32	133.4032	1.720	1.455		45	0.809	~	0.00	0.00
29	32	133.4032	1.787	1.490		44	0.821	~	0.00	0.00
30	32	133.4032	1.853	1.526		44	0.831	~	0.00	0.00
31	32	133.4032	1.920	1.561		43	0.842	~	0.00	0.00
32	32	133.4032	1.987	1.597		43	0.852	~	0.00	0.00
33	32	133.4032	2.053	1.632		42	0.861	~	0.00	0.00
34	32	133.4032	2.120	1.668		42	0.870	~	0.00	0.00
35	32	133.4032	2.187	1.703		41	0.879	~	0.00	0.00
36	32	133.4032	2.253	1.739		41	0.887	~	0.00	0.00
37	32	133.4032	2.320	1.774		40	0.895	~	0.00	0.00
38	32	133.4032	2.387	1.810		40	0.903	~	0.00	0.00
39	32	133.4032	2.454	1.845		40	0.910	~	0.00	0.00
40	32	133.4032	2.520	1.881		39	0.917	~	0.00	0.00
41	32	133.4032	2.587	1.916		39	0.924	~	0.00	0.00
42	32	133.4032	2.654	1.952		39	0.931	~	0.00	0.00
43	32	133.4032	2.720	1.987		38	0.937	~	0.00	0.00
44	32	133.4032	2.787	2.023		38	0.943	~	0.00	0.00
45	32	133.4032	2.854	2.058		38	0.949	~	0.00	0.00
46	32	133.4032	2.920	2.094		37	0.955	~	0.00	0.00
47	32	133.4032	2.987	2.129		37	0.960	~	0.00	0.00
48	32	133.4032	3.054	2.165		37	0.966	~	0.00	0.00
49	32	133.4032	3.121	2.200		36	0.971	~	0.00	0.00
50	32	133.4032	3.187	2.236		36	0.976	~	0.00	0.00

TOTAL SETTLEMENT = 0.0 INCHES

Figure 11

GEOCON

Client : Rose Hills Courts
File No. : A9514-06-01
Boring : 2

EMPIRICAL ESTIMATION OF LIQUEFACTION POTENTIAL
MAXIMUM CONSIDERED EARTHQUAKE

NCEER (1996) METHOD

By Thomas F. Blake (1994-1996)

EARTHQUAKE INFORMATION:

Earthquake Magnitude:	6.60
Peak Horiz. Acceleration PGA_M (g):	1.053
Calculated Mag.Wtg.Factor:	0.724
Historic High Groundwater:	20.0
Groundwater Depth During Exploration:	40.0

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):	1.15
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	134.6	0	17.0	5.0	1	0	89	2.000	44.0	134.6	Infin.	0.998	0.495	--
2.0	134.6	0	17.0	5.0	1	0	89	2.000	44.0	134.6	Infin.	0.993	0.492	--
3.0	134.6	0	17.0	5.0	1	0	89	2.000	44.0	134.6	Infin.	0.989	0.490	--
4.0	134.6	0	17.0	5.0	1	0	89	2.000	44.0	134.6	Infin.	0.984	0.488	--
5.0	134.6	0	17.0	5.0	1	0	89	1.857	40.8	134.6	Infin.	0.979	0.486	--
6.0	135.7	0	17.0	5.0	1	0	89	1.679	36.9	135.7	Infin.	0.975	0.483	--
7.0	135.7	0	17.0	5.0	1	0	89	1.544	33.9	135.7	Infin.	0.970	0.481	--
8.0	135.7	0	17.0	5.0	1	0	89	1.436	31.6	135.7	Infin.	0.966	0.479	--
9.0	135.7	0	17.0	5.0	1	0	89	1.349	29.7	135.7	0.434	0.961	0.477	--
10.0	135.7	0	12.0	10.0	1	0	66	1.276	19.8	135.7	0.216	0.957	0.474	--
11.0	135.7	0	12.0	10.0	1	0	66	1.213	18.8	135.7	0.205	0.952	0.472	--
12.0	135.7	0	12.0	10.0	1	0	66	1.159	18.0	135.7	0.196	0.947	0.470	--
13.0	135.7	0	12.0	10.0	1	0	66	1.111	17.3	135.7	0.188	0.943	0.467	--
14.5	135.7	0	12.0	10.0	1	0	66	1.060	16.4	135.7	0.179	0.937	0.465	--
15.0	141.1	0	27.0	15.0	1	61	91	1.040	46.1	141.1	Infin.	0.933	0.462	--
16.0	141.1	0	27.0	15.0	1	61	91	0.996	44.4	141.1	Infin.	0.929	0.461	--
17.0	141.1	0	27.0	15.0	1	61	91	0.964	43.2	141.1	Infin.	0.925	0.458	--
18.5	141.1	0	27.0	15.0	1	61	91	0.928	41.9	141.1	Infin.	0.919	0.456	--
19.0	141.1	0	27.0	15.0	1	61	91	0.915	41.4	141.1	Infin.	0.914	0.453	--
20.0	137.7	1	18.0	20.0	0	61		0.885	31.6	75.3	~	0.911	0.457	~
21.0	137.7	1	18.0	20.0	0	61		0.863	31.0	75.3	~	0.906	0.465	~
22.0	137.7	1	18.0	20.0	0	61		0.842	30.4	75.3	~	0.902	0.472	~
23.0	137.7	1	18.0	20.0	0	61		0.823	29.9	75.3	~	0.897	0.479	~
24.0	137.7	1	18.0	20.0	0	61		0.805	29.4	75.3	~	0.893	0.485	~
25.0	137.7	1	18.0	20.0	0	61		0.789	28.9	75.3	~	0.888	0.490	~
26.0	122.4	1	12.0	25.0	0	58		0.774	22.3	60.0	~	0.883	0.496	~
27.0	122.4	1	12.0	25.0	0	58		0.761	22.0	60.0	~	0.879	0.501	~
28.5	122.4	1	12.0	25.0	0	58		0.745	21.7	60.0	~	0.873	0.506	~
29.0	122.4	1	12.0	25.0	0	58		0.739	21.6	60.0	~	0.869	0.507	~
30.0	126.3	1	5.0	30.0	0	59		0.724	13.2	63.9	~	0.865	0.514	~
31.0	126.3	1	5.0	30.0	0	59		0.713	13.2	63.9	~	0.861	0.517	~
32.0	126.3	1	5.0	30.0	0	59		0.703	13.1	63.9	~	0.856	0.520	~
33.0	126.3	1	5.0	30.0	0	59		0.692	13.0	63.9	~	0.851	0.523	~
34.0	126.3	1	5.0	30.0	0	59		0.682	12.9	63.9	~	0.847	0.526	~
35.0	126.3	1	4.0	35.0	0	54		0.673	11.6	63.9	~	0.842	0.529	~
36.0	126.3	1	4.0	35.0	0	54		0.664	11.6	63.9	~	0.838	0.531	~
37.0	126.3	1	4.0	35.0	0	54		0.655	11.5	63.9	~	0.833	0.533	~
38.0	126.3	1	4.0	35.0	0	54		0.647	11.5	63.9	~	0.829	0.535	~
39.0	126.3	1	4.0	35.0	0	54		0.639	11.4	63.9	~	0.824	0.536	~
40.0	133.4	1	4.0	40.0	0	52		0.633	11.4	71.0	~	0.819	0.537	~
41.0	133.4	1	4.0	40.0	0	52		0.629	11.3	71.0	~	0.815	0.538	~
42.0	133.4	1	4.0	40.0	0	52		0.624	11.3	71.0	~	0.810	0.539	~
43.0	133.4	1	4.0	40.0	0	52		0.620	11.3	71.0	~	0.806	0.540	~
44.0	133.4	1	4.0	40.0	0	52		0.616	11.3	71.0	~	0.801	0.540	~
45.0	133.4	1	7.0	45.0	0	42		0.612	14.4	71.0	~	0.797	0.540	~
46.0	133.4	1	7.0	45.0	0	42		0.608	14.3	71.0	~	0.792	0.541	~
47.0	133.4	1	7.0	45.0	0	42		0.605	14.3	71.0	~	0.787	0.541	~
48.0	125.8	1	23.0	50.0	0	42		0.601	30.9	63.4	~	0.783	0.541	~
49.0	125.8	1	23.0	50.0	0	42		0.598	30.7	63.4	~	0.778	0.541	~
50.0	125.8	1	23.0	50.0	0	42		0.595	30.6	63.4	~	0.774	0.540	~

GEOCON

Client : Rose Hills Courts
File No. : A9514-06-01
Boring : 2

LIQUEFACTION SETTLEMENT ANALYSIS
MAXIMUM CONSIDERED EARTHQUAKE

(SATURATED SAND AT INITIAL LIQUEFACTION CONDITION)

NCEER (1996) METHOD
EARTHQUAKE INFORMATION:

Earthquake Magnitude:	6.60
PGA_M (g):	1.053
Calculated Mag.Wtg.Factor:	0.724
Historic High Groundwater:	20.0
Groundwater @ Exploration:	40.0

DEPTH TO BASE	BLOW COUNT N	WET DENSITY (PCF)	TOTAL STRESS O (TSF)	EFFECT STRESS O' (TSF)	REL. DEN. Dr (%)	ADJUST BLOWS (N1)60	τ_{av}/σ'_o	LIQUEFACTION SAFETY FACTOR	Volumetric Strain $[e_{15}]$ (%)	EQ. SETTLE. Pe (in.)
1	17	134.5591	0.034	0.034	89	44	0.684	--	0.00	0.00
2	17	134.5591	0.101	0.101	89	44	0.684	--	0.00	0.00
3	17	134.5591	0.168	0.168	89	44	0.684	--	0.00	0.00
4	17	134.5591	0.235	0.235	89	44	0.684	--	0.00	0.00
5	17	134.5591	0.303	0.303	89	41	0.684	--	0.00	0.00
6	17	135.6784	0.370	0.370	89	37	0.684	--	0.00	0.00
7	17	135.6784	0.438	0.438	89	34	0.684	--	0.00	0.00
8	17	135.6784	0.506	0.506	89	32	0.684	--	0.00	0.00
9	17	135.6784	0.574	0.574	89	30	0.684	--	0.00	0.00
10	12	135.6784	0.642	0.642	66	20	0.684	--	0.00	0.00
11	12	135.6784	0.710	0.710	66	19	0.684	--	0.00	0.00
12	12	135.6784	0.777	0.777	66	18	0.684	--	0.00	0.00
13	12	135.6784	0.845	0.845	66	17	0.684	--	0.00	0.00
14.5	12	135.6784	0.930	0.930	66	16	0.684	--	0.00	0.00
15	27	141.056	0.965	0.965	91	46	0.684	--	0.00	0.00
16	27	141.056	1.053	1.053	91	44	0.684	--	0.00	0.00
17	27	141.056	1.123	1.123	91	43	0.684	--	0.00	0.00
18.5	27	141.056	1.211	1.211	91	42	0.684	--	0.00	0.00
19	27	141.056	1.247	1.247	91	41	0.684	--	0.00	0.00
20	18	137.71	1.334	1.318		32	0.693	~	0.00	0.00
21	18	137.71	1.403	1.356		31	0.708	~	0.00	0.00
22	18	137.71	1.472	1.394		30	0.723	~	0.00	0.00
23	18	137.71	1.541	1.431		30	0.737	~	0.00	0.00
24	18	137.71	1.609	1.469		29	0.750	~	0.00	0.00
25	18	137.71	1.678	1.507		29	0.762	~	0.00	0.00
26	12	122.3636	1.743	1.541		22	0.775	~	0.00	0.00
27	12	122.3636	1.804	1.570		22	0.786	~	0.00	0.00
28.5	12	122.3636	1.881	1.608		22	0.801	~	0.00	0.00
29	12	122.3636	1.912	1.623		22	0.806	~	0.00	0.00
30	5	126.2977	1.989	1.661		13	0.819	~	0.00	0.00
31	5	126.2977	2.052	1.693		13	0.829	~	0.00	0.00
32	5	126.2977	2.115	1.725		13	0.839	~	0.00	0.00
33	5	126.2977	2.178	1.757		13	0.849	~	0.00	0.00
34	5	126.2977	2.242	1.789		13	0.858	~	0.00	0.00
35	4	126.2977	2.305	1.821		12	0.866	~	0.00	0.00
36	4	126.2977	2.368	1.853		12	0.875	~	0.00	0.00
37	4	126.2977	2.431	1.885		12	0.883	~	0.00	0.00
38	4	126.2977	2.494	1.917		11	0.891	~	0.00	0.00
39	4	126.2977	2.557	1.949		11	0.898	~	0.00	0.00
40	4	133.4293	2.622	1.983		11	0.905	~	0.00	0.00
41	4	133.4293	2.689	2.018		11	0.912	~	0.00	0.00
42	4	133.4293	2.756	2.054		11	0.918	~	0.00	0.00
43	4	133.4293	2.822	2.089		11	0.925	~	0.00	0.00
44	4	133.4293	2.889	2.125		11	0.931	~	0.00	0.00
45	7	133.4293	2.956	2.160		14	0.937	~	0.00	0.00
46	7	133.4293	3.023	2.196		14	0.942	~	0.00	0.00
47	7	133.4293	3.089	2.231		14	0.948	~	0.00	0.00
48	23	125.8378	3.154	2.265		31	0.953	~	0.00	0.00
49	23	125.8378	3.217	2.297		31	0.959	~	0.00	0.00
50	23	125.8378	3.280	2.328		31	0.964	~	0.00	0.00

TOTAL SETTLEMENT = 0.0 INCHES

GEOCON

Client : Rose Hills Courts
File No. : A9514-06-01
Boring : 1

TECHNICAL ENGINEERING AND DESIGN GUIDES AS ADAPTED FROM THE US ARMY CORPS OF ENGINEERS, NO. 9
EVALUATION OF EARTHQUAKE-INDUCED SETTLEMENTS IN DRY SANDY SOILS
DESIGN EARTHQUAKE

DE EARTHQUAKE INFORMATION:

Earthquake Magnitude:	6.62
Peak Horiz. Acceleration (g):	0.702

Depth of Base of Strata (ft)	Thickness of Layer (ft)	Depth of Mid-point of Layer (ft)	Soil Unit Weight (pcf)	Overburden Pressure at Mid-point (tsf)	Mean Effective Pressure at Mid-point (tsf)	Average Cyclic Shear Stress [Tav]	Field SPT [N]	Correction Factor [Cer]	Fig 4.1 Relative Density [Dr] (%)	Fig 4.2 Correction Factor [Cn]	Corrected [N1]60	rd Factor	Maximum Shear Mod. [Gmax] (tsf)	[yeff]*[Geff] / [Gmax]	Fig 4.4 yeff Shear Strain	[yeff]*100%	Volumetric Strain M7.5 [E15} (%)	Number of Strain Cycles [Nc]	Corrected Vol. Strains [Ec]	Estimated Settlement [S] (inches)
1.0	1.0	0.5	119.7	0.03	0.02	0.014	8	1.25	57.8	2.0	20.7	1.0	173.811	7.78E-05	1.40E-04	0.014	1.34E-02	8.0856	1.02E-02	0.00
2.0	1.0	1.5	119.7	0.09	0.06	0.041	8	1.25	57.8	2.0	20.7	1.0	301.050	1.32E-04	2.30E-04	0.023	2.21E-02	8.0856	1.67E-02	0.00
3.0	1.0	2.5	119.7	0.15	0.10	0.068	8	1.25	57.8	2.0	20.7	1.0	388.654	1.67E-04	1.70E-04	0.017	1.63E-02	8.0856	1.24E-02	0.00
4.0	1.0	3.5	119.7	0.21	0.14	0.096	8	1.25	57.8	2.0	20.7	1.0	459.861	1.94E-04	1.70E-04	0.017	1.63E-02	8.0856	1.24E-02	0.00
5.0	1.0	4.5	119.7	0.27	0.18	0.123	8	1.25	57.8	2.0	20.4	1.0	518.689	2.17E-04	8.10E-04	0.081	7.92E-02	8.0856	6.00E-02	0.01
6.0	1.0	5.5	119.7	0.33	0.22	0.150	8	1.25	57.8	1.8	18.4	1.0	554.570	2.44E-04	4.50E-04	0.045	4.96E-02	8.0856	3.76E-02	0.01
7.0	1.0	6.5	119.7	0.39	0.26	0.177	8	1.25	57.8	1.6	17.0	1.0	586.327	2.67E-04	4.50E-04	0.045	5.49E-02	8.0856	4.16E-02	0.01
8.0	1.0	7.5	119.7	0.45	0.30	0.204	8	1.25	57.8	1.5	15.8	1.0	614.973	2.88E-04	4.50E-04	0.045	5.98E-02	8.0856	4.53E-02	0.01
9.0	1.0	8.5	119.7	0.51	0.34	0.231	8	1.25	57.8	1.4	14.8	1.0	641.173	3.08E-04	1.00E-03	0.100	1.43E-01	8.0856	1.08E-01	0.03
10.0	1.0	9.5	116.0	0.57	0.38	0.257	26	1.25	95.0	1.4	45.6	1.0	985.076	2.19E-04	4.50E-04	0.045	1.67E-02	8.0856	1.27E-02	0.00
11.0	1.0	10.5	116.0	0.63	0.42	0.283	26	1.25	95.0	1.3	43.4	1.0	1017.547	2.30E-04	4.50E-04	0.045	1.77E-02	8.0856	1.34E-02	0.00
12.0	1.0	11.5	116.0	0.68	0.46	0.308	26	1.25	95.0	1.2	41.6	0.9	1048.068	2.40E-04	4.50E-04	0.045	1.87E-02	8.0856	1.42E-02	0.00
13.0	1.0	12.5	116.0	0.74	0.50	0.333	26	1.25	95.0	1.2	39.9	0.9	1076.908	2.48E-04	4.50E-04	0.045	1.96E-02	8.0856	1.49E-02	0.00
14.5	1.5	13.8	116.0	0.81	0.55	0.365	26	1.25	95.0	1.1	38.1	0.9	1110.916	2.59E-04	3.70E-04	0.037	1.71E-02	8.0856	1.29E-02	0.00
15.0	0.5	14.8	129.6	0.87	0.59	0.391	43	1.25	112.4	1.1	70.5	0.9	1413.357	2.14E-04	3.70E-04	0.037	8.15E-03	8.0856	6.17E-03	0.00
16.0	1.0	15.5	129.6	0.92	0.62	0.411	43	1.25	112.4	1.1	67.4	0.9	1430.130	2.21E-04	3.70E-04	0.037	8.61E-03	8.0856	6.52E-03	0.00
17.0	1.0	16.5	129.6	0.99	0.66	0.439	43	1.25	112.4	1.0	65.1	0.9	1462.907	2.27E-04	3.70E-04	0.037	8.97E-03	8.0856	6.79E-03	0.00
18.0	1.0	17.5	129.6	1.05	0.71	0.466	43	1.25	112.4	1.0	63.1	0.9	1494.278	2.33E-04	3.70E-04	0.037	9.32E-03	8.0856	7.06E-03	0.00
19.0	1.0	18.5	129.6	1.12	0.75	0.493	43	1.25	112.4	1.0	61.3	0.9	1524.384	2.38E-04	3.70E-04	0.037	9.66E-03	8.0856	7.31E-03	0.00
20.0	1.0	19.5	133.4	1.18	0.79	0.520	43	1.25	112.4	0.9	59.9	0.9	1557.197	2.43E-04	3.70E-04	0.037	9.91E-03	8.0856	7.51E-03	0.00
21.0	1.0	20.5	133.4	1.25	0.84	0.547	43	1.25	112.4	0.9	59.0	0.9	1592.552	2.47E-04	3.70E-04	0.037	1.01E-02	8.0856	7.64E-03	0.00
22.0	1.0	21.5	133.4	1.32	0.88	0.574	43	1.25	112.4	0.9	58.2	0.9	1626.622	2.50E-04	3.70E-04	0.037	1.03E-02	8.0856	7.78E-03	0.00
23.0	1.0	22.5	133.4	1.38	0.93	0.601	43	1.25	112.4	0.9	57.4	0.9	1659.513	2.54E-04	3.70E-04	0.037	1.04E-02	8.0856	7.91E-03	0.00
24.0	1.0	23.5	133.4	1.45	0.97	0.627	43	1.25	112.4	0.9	56.6	0.9	1691.316	2.57E-04	3.70E-04	0.037	1.06E-02	8.0856	8.04E-03	0.00
25.0	1.0	24.5	133.4	1.52	1.02	0.653	32	1.25	90.7	0.9	46.4	0.9	1619.065	2.76E-04	3.00E-04	0.030	1.09E-02	8.0856	8.27E-03	0.00
26.0	1.0	25.5	133.4	1.58	1.06	0.679	32	1.25	90.7	0.9	45.8	0.9	1647.141	2.79E-04	3.00E-04	0.030	1.11E-02	8.0856	8.40E-03	0.00
27.0	1.0	26.5	133.4	1.65	1.11	0.704	32	1.25	90.7	0.9	45.2	0.9	1674.400	2.81E-04	3.00E-04	0.030	1.13E-02	8.0856	8.53E-03	0.00
28.0	1.0	27.5	133.4	1.72	1.15	0.729	32	1.25	90.7	0.8	44.7	0.9	1700.895	2.83E-04	3.00E-04	0.030	1.14E-02	8.0856	8.66E-03	0.00
29.0	1.0	28.5	133.4	1.78	1.19	0.753	32	1.25	90.7	0.8	44.1	0.9	1726.676	2.86E-04	3.00E-04	0.030	1.16E-02	8.0856	8.79E-03	0.00
30.0	1.0	29.5	133.4	1.85	1.24	0.777	32	1.25	90.7	0.8	43.6	0.9	1751.786	2.87E-04	3.00E-04	0.030	1.18E-02	8.0856	8.91E-03	0.00
31.0	1.0	30.5	133.4	1.92	1.28	0.800	32	1.25	90.7	0.8	43.1	0.9	1776.267	2.89E-04	3.00E-04	0.030	1.19E-02	8.0856	9.03E-03	0.00
32.0	1.0	31.5	133.4	1.98	1.33	0.824	32	1.25	90.7	0.8	42.6	0.9	1800.153	2.91E-04	3.00E-04	0.030	1.21E-02	8.0856	9.16E-03	0.00
33.0	1.0	32.5	133.4	2.05	1.37	0.846	32	1.25	90.7	0.8	42.2	0.9	1823.479	2.92E-04	3.00E-04	0.030	1.23E-02	8.0856	9.28E-03	0.00
34.0	1.0	33.5	133.4	2.12	1.42	0.869	32	1.25	90.7	0.8	41.7	0.8	1846.275	2.94E-04	3.00E-04	0.030	1.24E-02	8.0856	9.40E-03	0.00
35.0	1.0	34.5	133.4	2.18	1.46	0.891	32	1.25	90.7	0.8	41.3	0.8	1868.568	2.95E-04	3.00E-04	0.030	1.26E-02	8.0856	9.52E-03	0.00
36.0	1.0	35.5	133.4	2.25	1.51	0.912	32	1.25	90.7	0.8	40.9	0.8	1890.385	2.96E-04	3.00E-04	0.030	1.27E-02	8.0856	9.64E-03	0.00
37.0	1.0	36.5	133.4	2.32	1.55	0.933	32	1.25	90.7	0.8	40.5	0.8	1911.749	2.97E-04	3.00E-04	0.030	1.29E-02	8.0856	9.76E-03	0.00
38.0	1.0	37.5	133.4	2.38	1.60	0.954	32	1.25	90.7	0.8	40.1	0.8	1932.682	2.98E-04	3.00E-04	0.030	1.30E-02	8.0856	9.87E-03	0.00
39.0	1.0	38.5	133.4	2.45	1.64	0.974	32	1.25	90.7	0.8	39.7	0.8	1953.204	2.99E-04	3.00E-04	0.030	1.32E-02	8.0856	9.99E-03	0.00
40.0	1.0	39.5	133.4	2.52	1.69	0.994	32	1.25	90.7	0.7	39.3	0.8	1973.334	2.99E-04	3.00E-04	0.030	1.33E-02	8.0856	1.01E-02	0.00
41.0	1.0	40.5	133.4	2.58	1.73	1.013	32	1.25	90.7	0.7	38.9	0.8	1993.089	3.00E-04	3.00E-04	0.030	1.35E-02	8.0856	1.02E-02	0.00
42.0	1.0	41.5	133.4	2.65	1.78	1.032	32	1.25	90.7	0.7	38.6	0.8	2012.487	3.00E-04	5.20E-04	0.052	2.36E-02	8.0856	1.79E-02	0.00
43.0	1.0	42.5	133.4	2.72	1.82	1.051	32	1.25	90.7	0.7	38.2	0.8	2031.542	3.01E-04	5.20E-04	0.052	2.39E-02	8.0856	1.81E-02	0.00
44.0	1.0	43.5	133.4	2.78	1.87	1.069	32	1.25	90.7	0.7	37.9	0.8	2050.268	3.01E-04	5.20E-04	0.052	2.42E-02	8.0856	1.83E-02	0.00
45.0	1.0	44.5	133.4	2.85	1.91	1.086	32	1.25	90.7	0.7	37.6	0.8	2068.679	3.01E-04	5.20E-04	0.052	2.44E-02	8.0856	1.85E-02	0.00
46.0	1.0	45.5	133.4	2.92	1.95	1.103	32	1.25	90.7	0.7	37.2	0.8	2086.787	3.01E-04	5.20E-04	0.052	2.47E-02	8.0856	1.87E-02	0.00
47.0	1.0	46.5	133.4	2.98	2.00	1.120	32	1.25	90.7	0.7	36.9	0.8	2104.604	3.02E-04	5.20E-04	0.052	2.49E-02	8.0856	1.89E-02	0.00
48.0	1.0	47.5	133.4	3.05	2.04	1.136	32	1.25	90.7	0.7	36.6	0.8	2122.142	3.02E-04	1.00E-02	1.000	4.84E-01	8.0856	3.66E-01	0.00
49.0	1.0	48.5	133.4	3.12	2.09	1.152	32	1.25	90.7	0.7	36.3	0.8	2139.409	3.02E-04	1.00E-02	1.000	4.89E-01	8.0856	3.70E-01	0.00
50.0	1.0	49.5	133.4	3.18	2.13	1.167	32	1.25	90.7	0.7	36.0	0.8	2156.417	3.02E-04	1.00E-02	1.000	4.93E-01	8.0856	3.74E-01	0.00

TOTAL SETTLEMENT = **0.11**

Figure 14

GEOCON

Client : Rose Hills Courts
File No. : A9514-06-01
Boring : 2

TECHNICAL ENGINEERING AND DESIGN GUIDES AS ADAPTED FROM THE US ARMY CORPS OF ENGINEERS, NO. 9
EVALUATION OF EARTHQUAKE-INDUCED SETTLEMENTS IN DRY SANDY SOILS
DESIGN EARTHQUAKE

DE EARTHQUAKE INFORMATION:

Earthquake Magnitude:	6.62
Peak Horiz. Acceleration (g):	0.702

Depth of Base of Strata (ft)	Thickness of Layer (ft)	Depth of Mid-point of Layer (ft)	Soil Unit Weight (pcf)	Overburden Pressure at Mid-point (tsf)	Mean Effective Pressure at Mid-point (tsf)	Average Cyclic Shear Stress [Tav]	Field SPT [N]	Correction Factor [Cer]	Fig 4.1 Relative Density [Dr] (%)	Fig 4.2 Correction Factor [Cn]	Corrected [N1]60	rd Factor	Maximum Shear Mod. [Gmax] (tsf)	[yeff]*[Geff] [Gmax]	Fig 4.4 yeff Shear Strain	[yeff]*100%	Volumetric Strain M7.5 [E15] (%)	Number of Strain Cycles [Nc]	Corrected Vol. Strains [Ec]	Estimated Settlement [S] (inches)
1.0	1.0	0.5	134.6	0.03	0.02	0.015	17	1.25	88.8	2.0	44.0	1.0	236.891	6.42E-05	1.00E-04	0.010	3.88E-03	8.0856	2.94E-03	0.00
2.0	1.0	1.5	134.6	0.10	0.07	0.046	17	1.25	88.8	2.0	44.0	1.0	410.307	1.09E-04	2.30E-04	0.023	8.93E-03	8.0856	6.76E-03	0.00
3.0	1.0	2.5	134.6	0.17	0.11	0.077	17	1.25	88.8	2.0	44.0	1.0	529.704	1.38E-04	1.70E-04	0.017	6.60E-03	8.0856	5.00E-03	0.00
4.0	1.0	3.5	134.6	0.24	0.16	0.107	17	1.25	88.8	2.0	44.0	1.0	626.754	1.60E-04	1.70E-04	0.017	6.60E-03	8.0856	5.00E-03	0.00
5.0	1.0	4.5	134.6	0.30	0.20	0.138	17	1.25	88.8	1.9	40.8	1.0	693.309	1.83E-04	1.50E-04	0.015	6.37E-03	8.0856	4.82E-03	0.00
6.0	1.0	5.5	135.7	0.37	0.25	0.169	17	1.25	88.8	1.7	36.9	1.0	741.457	2.05E-04	4.50E-04	0.045	2.16E-02	8.0856	1.63E-02	0.00
7.0	1.0	6.5	135.7	0.44	0.29	0.199	17	1.25	88.8	1.5	33.9	1.0	784.220	2.25E-04	4.50E-04	0.045	2.38E-02	8.0856	1.81E-02	0.00
8.0	1.0	7.5	135.7	0.51	0.34	0.230	17	1.25	88.8	1.4	31.6	1.0	822.767	2.43E-04	4.50E-04	0.045	2.60E-02	8.0856	1.97E-02	0.00
9.0	1.0	8.5	135.7	0.57	0.38	0.260	17	1.25	88.8	1.3	29.7	1.0	858.006	2.59E-04	4.50E-04	0.045	2.80E-02	8.0856	2.12E-02	0.01
10.0	1.0	9.5	135.7	0.64	0.43	0.290	12	1.25	66.5	1.3	19.8	1.0	792.946	3.08E-04	1.00E-03	0.100	1.01E-01	8.0856	7.66E-02	0.02
11.0	1.0	10.5	135.7	0.71	0.48	0.320	12	1.25	66.5	1.2	18.8	1.0	819.959	3.24E-04	1.00E-03	0.100	1.07E-01	8.0856	8.14E-02	0.02
12.0	1.0	11.5	135.7	0.78	0.52	0.350	12	1.25	66.5	1.2	18.0	0.9	845.301	3.38E-04	7.10E-04	0.071	8.06E-02	8.0856	6.10E-02	0.01
13.0	1.0	12.5	135.7	0.85	0.57	0.380	12	1.25	66.5	1.1	17.3	0.9	869.208	3.51E-04	7.10E-04	0.071	8.48E-02	8.0856	6.42E-02	0.02
14.5	1.5	13.8	135.7	0.93	0.62	0.417	12	1.25	66.5	1.1	16.4	0.9	897.357	3.66E-04	7.10E-04	0.071	8.98E-02	8.0856	6.80E-02	0.02
15.0	0.5	14.8	141.1	1.00	0.67	0.446	27	1.25	91.1	1.0	46.1	0.9	1310.763	2.64E-04	3.70E-04	0.037	1.36E-02	8.0856	1.03E-02	0.00
16.0	1.0	15.5	141.1	1.05	0.70	0.469	27	1.25	91.1	1.0	44.4	0.9	1328.564	2.71E-04	3.70E-04	0.037	1.42E-02	8.0856	1.08E-02	0.00
17.0	1.0	16.5	141.1	1.12	0.75	0.499	27	1.25	91.1	1.0	43.2	0.9	1359.995	2.77E-04	3.70E-04	0.037	1.47E-02	8.0856	1.11E-02	0.00
18.5	1.5	17.8	141.1	1.21	0.81	0.536	27	1.25	91.1	0.9	41.9	0.9	1397.637	2.85E-04	3.70E-04	0.037	1.52E-02	8.0856	1.15E-02	0.00
19.0	0.5	18.8	141.1	1.28	0.86	0.565	27	1.25	91.1	0.9	41.4	0.9	1432.082	2.90E-04	3.70E-04	0.037	1.55E-02	8.0856	1.17E-02	0.00
20.0	1.0	19.5	137.7	1.33	0.89	0.586	18	1.25	68.5	0.9	31.6	0.9	1335.135	3.19E-04	7.10E-04	0.071	4.10E-02	8.0856	3.11E-02	0.01
21.0	1.0	20.5	137.7	1.40	0.94	0.614	18	1.25	68.5	0.9	31.0	0.9	1360.306	3.24E-04	7.10E-04	0.071	4.20E-02	8.0856	3.18E-02	0.00
22.0	1.0	21.5	137.7	1.47	0.99	0.642	18	1.25	68.5	0.8	30.4	0.9	1384.758	3.28E-04	7.10E-04	0.071	4.30E-02	8.0856	3.25E-02	0.00
23.0	1.0	22.5	137.7	1.54	1.03	0.669	18	1.25	68.5	0.8	29.9	0.9	1408.545	3.32E-04	5.20E-04	0.052	3.21E-02	8.0856	2.43E-02	0.00
24.0	1.0	23.5	137.7	1.61	1.08	0.696	18	1.25	68.5	0.8	29.4	0.9	1431.714	3.36E-04	5.20E-04	0.052	3.28E-02	8.0856	2.48E-02	0.00
25.0	1.0	24.5	137.7	1.68	1.12	0.722	18	1.25	68.5	0.8	28.9	0.9	1454.309	3.40E-04	5.20E-04	0.052	3.34E-02	8.0856	2.53E-02	0.00
26.0	1.0	25.5	122.4	1.74	1.17	0.747	12	1.25	53.8	0.8	22.3	0.9	1359.314	3.71E-04	5.20E-04	0.052	4.56E-02	8.0856	3.45E-02	0.00
27.0	1.0	26.5	122.4	1.80	1.21	0.769	12	1.25	53.8	0.8	22.0	0.9	1377.549	3.73E-04	5.20E-04	0.052	4.63E-02	8.0856	3.50E-02	0.00
28.5	1.5	27.8	122.4	1.88	1.26	0.797	12	1.25	53.8	0.7	21.7	0.9	1399.856	3.76E-04	5.20E-04	0.052	4.71E-02	8.0856	3.56E-02	0.00
29.0	0.5	28.8	122.4	1.94	1.30	0.818	12	1.25	53.8	0.7	21.6	0.9	1419.869	3.76E-04	5.20E-04	0.052	4.74E-02	8.0856	3.59E-02	0.00
30.0	1.0	29.5	126.3	1.99	1.33	0.835	5	1.25	33.7	0.7	13.2	0.9	1220.606	4.43E-04	8.10E-04	0.081	1.33E-01	8.0856	1.01E-01	0.00
31.0	1.0	30.5	126.3	2.05	1.37	0.857	5	1.25	33.7	0.7	13.2	0.9	1236.814	4.44E-04	8.10E-04	0.081	1.34E-01	8.0856	1.01E-01	0.00
32.0	1.0	31.5	126.3	2.11	1.42	0.878	5	1.25	33.7	0.7	13.1	0.9	1252.760	4.46E-04	8.10E-04	0.081	1.35E-01	8.0856	1.02E-01	0.00
33.0	1.0	32.5	126.3	2.18	1.46	0.899	5	1.25	33.7	0.7	13.0	0.9	1268.457	4.46E-04	8.10E-04	0.081	1.36E-01	8.0856	1.03E-01	0.00
34.0	1.0	33.5	126.3	2.24	1.50	0.920	5	1.25	33.7	0.7	12.9	0.8	1283.914	4.47E-04	8.10E-04	0.081	1.37E-01	8.0856	1.04E-01	0.00
35.0	1.0	34.5	126.3	2.30	1.54	0.940	4	1.25	29.3	0.7	11.6	0.8	1258.630	4.62E-04	8.10E-04	0.081	1.55E-01	8.0856	1.17E-01	0.00
36.0	1.0	35.5	126.3	2.37	1.59	0.959	4	1.25	29.3	0.7	11.6	0.8	1273.486	4.62E-04	8.10E-04	0.081	1.56E-01	8.0856	1.18E-01	0.00
37.0	1.0	36.5	126.3	2.43	1.63	0.979	4	1.25	29.3	0.7	11.5	0.8	1288.136	4.62E-04	8.10E-04	0.081	1.57E-01	8.0856	1.19E-01	0.00
38.0	1.0	37.5	126.3	2.49	1.67	0.998	4	1.25	29.3	0.6	11.5	0.8	1302.590	4.62E-04	8.10E-04	0.081	1.58E-01	8.0856	1.20E-01	0.00
39.0	1.0	38.5	126.3	2.56	1.71	1.016	4	1.25	29.3	0.6	11.4	0.8	1316.855	4.62E-04	8.10E-04	0.081	1.59E-01	8.0856	1.20E-01	0.00
40.0	1.0	39.5	133.4	2.62	1.76	1.035	4	1.25	28.5	0.6	11.4	0.8	1331.841	4.62E-04	8.10E-04	0.081	1.60E-01	8.0856	1.21E-01	0.00
41.0	1.0	40.5	133.4	2.69	1.80	1.054	4	1.25	28.5	0.6	11.3	0.8	1347.520	4.61E-04	8.10E-04	0.081	1.60E-01	8.0856	1.21E-01	0.00
42.0	1.0	41.5	133.4	2.75	1.85	1.073	4	1.25	28.5	0.6	11.3	0.8	1362.983	4.61E-04	8.10E-04	0.081	1.61E-01	8.0856	1.22E-01	0.00
43.0	1.0	42.5	133.4	2.82	1.89	1.091	4	1.25	28.5	0.6	11.3	0.8	1378.240	4.60E-04	8.10E-04	0.081	1.61E-01	8.0856	1.22E-01	0.00
44.0	1.0	43.5	133.4	2.89	1.93	1.109	4	1.25	28.5	0.6	11.3	0.8	1393.297	4.60E-04	8.10E-04	0.081	1.62E-01	8.0856	1.22E-01	0.00
45.0	1.0	44.5	133.4	2.95	1.98	1.126	7	1.25	36.6	0.6	14.4	0.8	1529.852	4.22E-04	8.10E-04	0.081	1.20E-01	8.0856	9.10E-02	0.00
46.0	1.0	45.5	133.4	3.02	2.02	1.143	7	1.25	36.6	0.6	14.3	0.8	1545.353	4.22E-04	1.00E-02	1.000	1.49E+00	8.0856	1.13E+00	0.00
47.0	1.0	46.5	133.4	3.09	2.07	1.159	7	1.25	36.6	0.6	14.3	0.8	1560.656	4.21E-04	1.00E-02	1.000	1.50E+00	8.0856	1.13E+00	0.00
48.0	1.0	47.5	125.8	3.15	2.11	1.174	23	1.25	64.6	0.6	30.9	0.8	2037.569	3.25E-04	1.00E-02	1.000	5.94E-01	8.0856	4.50E-01	0.00
49.0	1.0	48.5	125.8	3.22	2.15	1.188	23	1.25	64.6	0.6	30.7	0.8	2054.909	3.24E-04	1.00E-02	1.000	5.97E-01	8.0856	4.52E-01	0.00
50.0	1.0	49.5	125.8	3.28	2.20	1.202	23	1.25	64.6	0.6	30.6	0.8	2072.034	3.23E-04	1.00E-02	1.000	6.00E-01	8.0856	4.55E-01	0.00

TOTAL SETTLEMENT = **0.14**

Figure 15

GEOCON

Client : Rose Hills Courts
File No. : A9514-06-01
Boring : 1

TECHNICAL ENGINEERING AND DESIGN GUIDES AS ADAPTED FROM THE US ARMY CORPS OF ENGINEERS, NO. 9
EVALUATION OF EARTHQUAKE-INDUCED SETTLEMENTS IN DRY SANDY SOILS
MAXIMUM CONSIDERED EARTHQUAKE

MCE EARTHQUAKE INFORMATION:

Earthquake Magnitude:	6.60
Peak Horiz. Acceleration (g):	1.053

Fig 4.1 Fig 4.2 Fig 4.4

Depth of Base of Strata (ft)	Thickness of Layer (ft)	Depth of Mid-point of Layer (ft)	Soil Unit Weight (pcf)	Overburden Pressure at Mid-point (tsf)	Mean Effective Pressure at Mid-point (tsf)	Average Cyclic Shear Stress [Tav]	Field SPT [N]	Correction Factor [Cer]	Relative Density [Dr] (%)	Correction Factor [Cn]	Corrected [N1]60	rd Factor	Maximum Shear Mod. [Gmax] (tsf)	[yeff]*[Geff] [Gmax]	yeff Shear Strain	[yeff]*100%	Volumetric Strain M7.5 [E15} (%)	Number of Strain Cycles [Nc]	Corrected Vol. Strains [Ec]	Estimated Settlement [S] (inches)
1.0	1.0	0.5	119.7	0.03	0.02	0.020	8	1.25	57.8	2.0	20.7	1.0	173.811	1.17E-04	2.30E-04	0.023	2.21E-02	7.9523	1.66E-02	0.00
2.0	1.0	1.5	119.7	0.09	0.06	0.061	8	1.25	57.8	2.0	20.7	1.0	301.050	1.98E-04	2.30E-04	0.023	2.21E-02	7.9523	1.66E-02	0.00
3.0	1.0	2.5	119.7	0.15	0.10	0.102	8	1.25	57.8	2.0	20.7	1.0	388.654	2.51E-04	8.10E-04	0.081	7.77E-02	7.9523	5.84E-02	0.01
4.0	1.0	3.5	119.7	0.21	0.14	0.143	8	1.25	57.8	2.0	20.7	1.0	459.861	2.91E-04	8.10E-04	0.081	7.77E-02	7.9523	5.84E-02	0.01
5.0	1.0	4.5	119.7	0.27	0.18	0.184	8	1.25	57.8	2.0	20.4	1.0	518.689	3.26E-04	5.00E-03	0.500	4.89E-01	7.9523	3.68E-01	0.09
6.0	1.0	5.5	119.7	0.33	0.22	0.225	8	1.25	57.8	1.8	18.4	1.0	554.570	3.65E-04	1.00E-03	0.100	1.10E-01	7.9523	8.29E-02	0.02
7.0	1.0	6.5	119.7	0.39	0.26	0.265	8	1.25	57.8	1.6	17.0	1.0	586.327	4.00E-04	2.70E-03	0.270	3.29E-01	7.9523	2.47E-01	0.06
8.0	1.0	7.5	119.7	0.45	0.30	0.306	8	1.25	57.8	1.5	15.8	1.0	614.973	4.32E-04	2.70E-03	0.270	3.59E-01	7.9523	2.70E-01	0.06
9.0	1.0	8.5	119.7	0.51	0.34	0.346	8	1.25	57.8	1.4	14.8	1.0	641.173	4.61E-04	2.70E-03	0.270	3.87E-01	7.9523	2.91E-01	0.07
10.0	1.0	9.5	116.0	0.57	0.38	0.385	26	1.25	95.0	1.4	45.6	1.0	985.076	3.29E-04	1.00E-03	0.100	3.72E-02	7.9523	2.79E-02	0.01
11.0	1.0	10.5	116.0	0.63	0.42	0.424	26	1.25	95.0	1.3	43.4	1.0	1017.547	3.45E-04	1.00E-03	0.100	3.94E-02	7.9523	2.96E-02	0.01
12.0	1.0	11.5	116.0	0.68	0.46	0.462	26	1.25	95.0	1.2	41.6	0.9	1048.068	3.59E-04	1.00E-03	0.100	4.16E-02	7.9523	3.12E-02	0.01
13.0	1.0	12.5	116.0	0.74	0.50	0.500	26	1.25	95.0	1.2	39.9	0.9	1076.908	3.73E-04	1.00E-03	0.100	4.37E-02	7.9523	3.28E-02	0.01
14.5	1.5	13.8	116.0	0.81	0.55	0.547	26	1.25	95.0	1.1	38.1	0.9	1110.916	3.88E-04	7.10E-04	0.071	3.28E-02	7.9523	2.46E-02	0.01
15.0	0.5	14.8	129.6	0.87	0.59	0.586	43	1.25	112.4	1.1	70.5	0.9	1413.357	3.21E-04	7.10E-04	0.071	1.56E-02	7.9523	1.18E-02	0.00
16.0	1.0	15.5	129.6	0.92	0.62	0.617	43	1.25	112.4	1.1	67.4	0.9	1430.130	3.31E-04	7.10E-04	0.071	1.65E-02	7.9523	1.24E-02	0.00
17.0	1.0	16.5	129.6	0.99	0.66	0.658	43	1.25	112.4	1.0	65.1	0.9	1462.907	3.40E-04	7.10E-04	0.071	1.72E-02	7.9523	1.29E-02	0.00
18.0	1.0	17.5	129.6	1.05	0.71	0.699	43	1.25	112.4	1.0	63.1	0.9	1494.278	3.49E-04	7.10E-04	0.071	1.79E-02	7.9523	1.34E-02	0.00
19.0	1.0	18.5	129.6	1.12	0.75	0.739	43	1.25	112.4	1.0	61.3	0.9	1524.384	3.57E-04	7.10E-04	0.071	1.85E-02	7.9523	1.39E-02	0.00
20.0	1.0	19.5	133.4	1.18	0.79	0.780	43	1.25	112.4	0.9	59.9	0.9	1557.197	3.64E-04	7.10E-04	0.071	1.90E-02	7.9523	1.43E-02	0.00
21.0	1.0	20.5	133.4	1.25	0.84	0.821	43	1.25	112.4	0.9	59.0	0.9	1592.552	3.70E-04	7.10E-04	0.071	1.94E-02	7.9523	1.46E-02	0.00
22.0	1.0	21.5	133.4	1.32	0.88	0.861	43	1.25	112.4	0.9	58.2	0.9	1626.622	3.75E-04	7.10E-04	0.071	1.97E-02	7.9523	1.48E-02	0.00
23.0	1.0	22.5	133.4	1.38	0.93	0.901	43	1.25	112.4	0.9	57.4	0.9	1659.513	3.80E-04	7.10E-04	0.071	2.00E-02	7.9523	1.51E-02	0.00
24.0	1.0	23.5	133.4	1.45	0.97	0.940	43	1.25	112.4	0.9	56.6	0.9	1691.316	3.85E-04	7.10E-04	0.071	2.04E-02	7.9523	1.53E-02	0.00
25.0	1.0	24.5	133.4	1.52	1.02	0.979	32	1.25	90.7	0.9	46.4	0.9	1619.065	4.14E-04	8.10E-04	0.081	2.95E-02	7.9523	2.22E-02	0.00
26.0	1.0	25.5	133.4	1.58	1.06	1.017	32	1.25	90.7	0.9	45.8	0.9	1647.141	4.18E-04	8.10E-04	0.081	3.00E-02	7.9523	2.25E-02	0.00
27.0	1.0	26.5	133.4	1.65	1.11	1.055	32	1.25	90.7	0.9	45.2	0.9	1674.400	4.21E-04	8.10E-04	0.081	3.04E-02	7.9523	2.29E-02	0.00
28.0	1.0	27.5	133.4	1.72	1.15	1.092	32	1.25	90.7	0.8	44.7	0.9	1700.895	4.25E-04	8.10E-04	0.081	3.09E-02	7.9523	2.32E-02	0.00
29.0	1.0	28.5	133.4	1.78	1.19	1.129	32	1.25	90.7	0.8	44.1	0.9	1726.676	4.28E-04	8.10E-04	0.081	3.13E-02	7.9523	2.35E-02	0.00
30.0	1.0	29.5	133.4	1.85	1.24	1.165	32	1.25	90.7	0.8	43.6	0.9	1751.786	4.31E-04	8.10E-04	0.081	3.18E-02	7.9523	2.39E-02	0.00
31.0	1.0	30.5	133.4	1.92	1.28	1.200	32	1.25	90.7	0.8	43.1	0.9	1776.267	4.34E-04	8.10E-04	0.081	3.22E-02	7.9523	2.42E-02	0.00
32.0	1.0	31.5	133.4	1.98	1.33	1.235	32	1.25	90.7	0.8	42.6	0.9	1800.153	4.36E-04	8.10E-04	0.081	3.27E-02	7.9523	2.45E-02	0.00
33.0	1.0	32.5	133.4	2.05	1.37	1.269	32	1.25	90.7	0.8	42.2	0.9	1823.479	4.38E-04	8.10E-04	0.081	3.31E-02	7.9523	2.49E-02	0.00
34.0	1.0	33.5	133.4	2.12	1.42	1.303	32	1.25	90.7	0.8	41.7	0.8	1846.275	4.40E-04	8.10E-04	0.081	3.35E-02	7.9523	2.52E-02	0.00
35.0	1.0	34.5	133.4	2.18	1.46	1.335	32	1.25	90.7	0.8	41.3	0.8	1868.568	4.42E-04	8.10E-04	0.081	3.39E-02	7.9523	2.55E-02	0.00
36.0	1.0	35.5	133.4	2.25	1.51	1.368	32	1.25	90.7	0.8	40.9	0.8	1890.385	4.44E-04	8.10E-04	0.081	3.44E-02	7.9523	2.58E-02	0.00
37.0	1.0	36.5	133.4	2.32	1.55	1.399	32	1.25	90.7	0.8	40.5	0.8	1911.749	4.45E-04	8.10E-04	0.081	3.48E-02	7.9523	2.61E-02	0.00
38.0	1.0	37.5	133.4	2.38	1.60	1.430	32	1.25	90.7	0.8	40.1	0.8	1932.682	4.46E-04	8.10E-04	0.081	3.52E-02	7.9523	2.65E-02	0.00
39.0	1.0	38.5	133.4	2.45	1.64	1.461	32	1.25	90.7	0.8	39.7	0.8	1953.204	4.48E-04	8.10E-04	0.081	3.56E-02	7.9523	2.68E-02	0.00
40.0	1.0	39.5	133.4	2.52	1.69	1.490	32	1.25	90.7	0.7	39.3	0.8	1973.334	4.49E-04	8.10E-04	0.081	3.60E-02	7.9523	2.71E-02	0.00
41.0	1.0	40.5	133.4	2.58	1.73	1.519	32	1.25	90.7	0.7	38.9	0.8	1993.089	4.49E-04	8.10E-04	0.081	3.64E-02	7.9523	2.74E-02	0.00
42.0	1.0	41.5	133.4	2.65	1.78	1.547	32	1.25	90.7	0.7	38.6	0.8	2012.487	4.50E-04	8.10E-04	0.081	3.68E-02	7.9523	2.77E-02	0.00
43.0	1.0	42.5	133.4	2.72	1.82	1.575	32	1.25	90.7	0.7	38.2	0.8	2031.542	4.51E-04	8.10E-04	0.081	3.72E-02	7.9523	2.80E-02	0.00
44.0	1.0	43.5	133.4	2.78	1.87	1.602	32	1.25	90.7	0.7	37.9	0.8	2050.268	4.51E-04	8.10E-04	0.081	3.76E-02	7.9523	2.83E-02	0.00
45.0	1.0	44.5	133.4	2.85	1.91	1.628	32	1.25	90.7	0.7	37.6	0.8	2068.679	4.52E-04	8.10E-04	0.081	3.80E-02	7.9523	2.86E-02	0.00
46.0	1.0	45.5	133.4	2.92	1.95	1.654	32	1.25	90.7	0.7	37.2	0.8	2086.787	4.52E-04	8.10E-04	0.081	3.84E-02	7.9523	2.89E-02	0.00
47.0	1.0	46.5	133.4	2.98	2.00	1.679	32	1.25	90.7	0.7	36.9	0.8	2104.604	4.52E-04	8.10E-04	0.081	3.88E-02	7.9523	2.92E-02	0.00
48.0	1.0	47.5	133.4	3.05	2.04	1.704	32	1.25	90.7	0.7	36.6	0.8	2122.142	4.52E-04	1.00E-02	1.000	4.84E-01	7.9523	3.64E-01	0.00
49.0	1.0	48.5	133.4	3.12	2.09	1.727	32	1.25	90.7	0.7	36.3	0.8	2139.409	4.52E-04	1.00E-02	1.000	4.89E-01	7.9523	3.67E-01	0.00
50.0	1.0	49.5	133.4	3.18	2.13	1.750	32	1.25	90.7	0.7	36.0	0.8	2156.417	4.52E-04	1.00E-02	1.000	4.93E-01	7.9523	3.71E-01	0.00

TOTAL SETTLEMENT = **0.39**

Figure 16

Client : Rose Hills Courts
File No. : A9514-06-01
Boring : 2

TECHNICAL ENGINEERING AND DESIGN GUIDES AS ADAPTED FROM THE US ARMY CORPS OF ENGINEERS, NO. 9
EVALUATION OF EARTHQUAKE-INDUCED SETTLEMENTS IN DRY SANDY SOILS
MAXIMUM CONSIDERED EARTHQUAKE

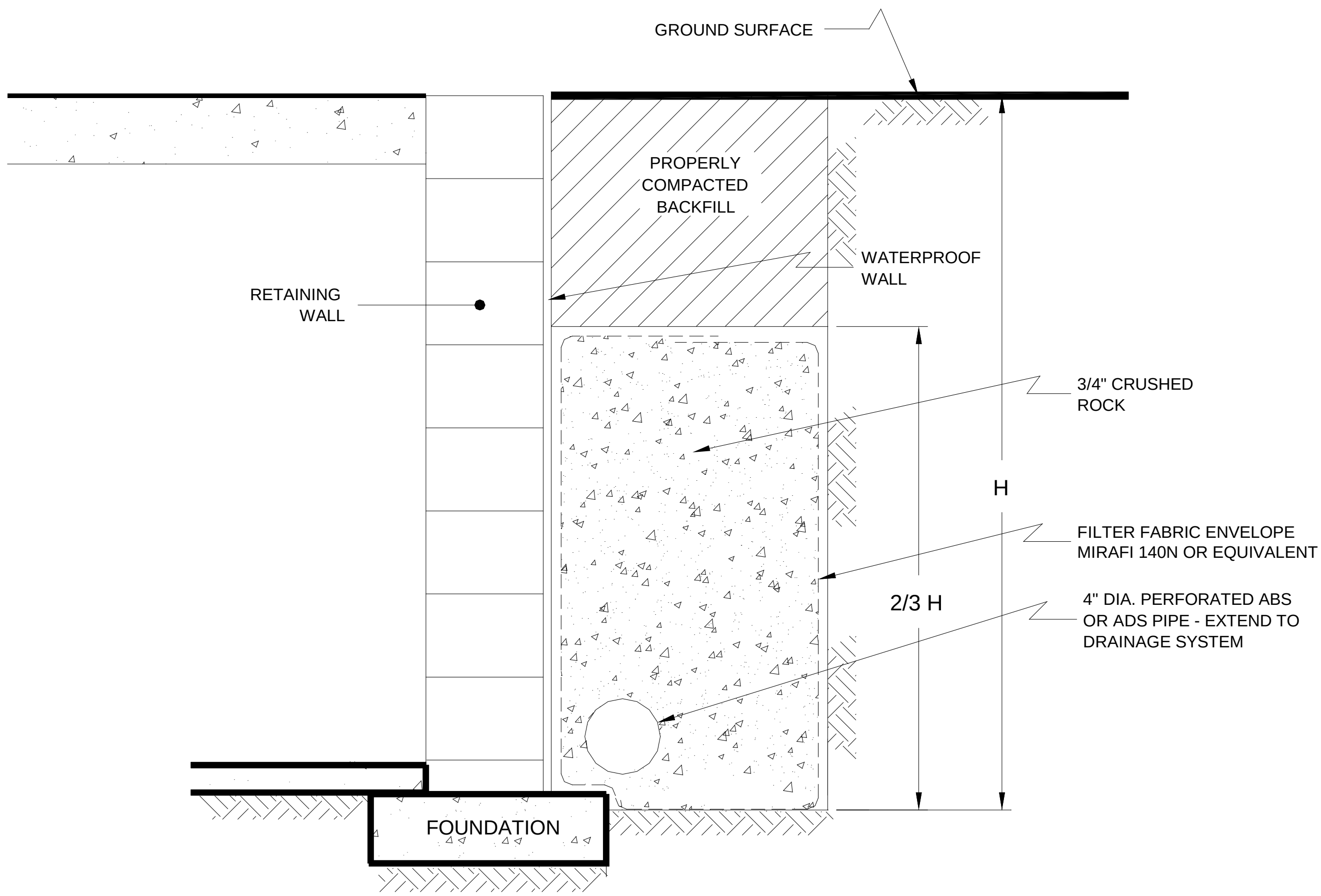
MCE EARTHQUAKE INFORMATION:

Earthquake Magnitude:	6.60
Peak Horiz. Acceleration (g):	1.053

Depth of Base of Strata (ft)	Thickness of Layer (ft)	Depth of Mid-point of Layer (ft)	Soil Unit Weight (pcf)	Overburden Pressure at Mid-point (tsf)	Mean Effective Pressure at Mid-point (tsf)	Average Cyclic Shear Stress [Tav]	Field SPT [N]	Correction Factor [Cer]	Fig 4.1 Relative Density [Dr] (%)	Fig 4.2 Correction Factor [Cn]	Corrected [N1]60	rd Factor	Maximum Shear Mod. [Gmax] (tsf)	[yeff]*[Geff] [Gmax]	Fig 4.4 yeff Shear Strain	[yeff]*100%	Volumetric Strain M7.5 [E15} (%)	Number of Strain Cycles [Nc]	Corrected Vol. Strains [Ec]	Estimated Settlement [S] (inches)
1.0	1.0	0.5	134.6	0.03	0.02	0.023	17	1.25	88.8	2.0	44.0	1.0	236.891	9.62E-05	1.90E-04	0.019	7.38E-03	7.9523	5.55E-03	0.00
2.0	1.0	1.5	134.6	0.10	0.07	0.069	17	1.25	88.8	2.0	44.0	1.0	410.307	1.63E-04	2.30E-04	0.023	8.93E-03	7.9523	6.71E-03	0.00
3.0	1.0	2.5	134.6	0.17	0.11	0.115	17	1.25	88.8	2.0	44.0	1.0	529.704	2.07E-04	8.10E-04	0.081	3.15E-02	7.9523	2.36E-02	0.01
4.0	1.0	3.5	134.6	0.24	0.16	0.161	17	1.25	88.8	2.0	44.0	1.0	626.754	2.40E-04	8.10E-04	0.081	3.15E-02	7.9523	2.36E-02	0.01
5.0	1.0	4.5	134.6	0.30	0.20	0.207	17	1.25	88.8	1.9	40.8	1.0	693.309	2.74E-04	4.50E-04	0.045	1.91E-02	7.9523	1.44E-02	0.00
6.0	1.0	5.5	135.7	0.37	0.25	0.253	17	1.25	88.8	1.7	36.9	1.0	741.457	3.07E-04	1.00E-03	0.100	4.79E-02	7.9523	3.60E-02	0.01
7.0	1.0	6.5	135.7	0.44	0.29	0.299	17	1.25	88.8	1.5	33.9	1.0	784.220	3.37E-04	1.00E-03	0.100	5.30E-02	7.9523	3.98E-02	0.01
8.0	1.0	7.5	135.7	0.51	0.34	0.344	17	1.25	88.8	1.4	31.6	1.0	822.767	3.64E-04	1.00E-03	0.100	5.78E-02	7.9523	4.34E-02	0.01
9.0	1.0	8.5	135.7	0.57	0.38	0.390	17	1.25	88.8	1.3	29.7	1.0	858.006	3.89E-04	1.00E-03	0.100	6.23E-02	7.9523	4.68E-02	0.01
10.0	1.0	9.5	135.7	0.64	0.43	0.435	12	1.25	66.5	1.3	19.8	1.0	792.946	4.62E-04	2.70E-03	0.270	2.73E-01	7.9523	2.05E-01	0.05
11.0	1.0	10.5	135.7	0.71	0.48	0.480	12	1.25	66.5	1.2	18.8	1.0	819.959	4.85E-04	2.70E-03	0.270	2.90E-01	7.9523	2.18E-01	0.05
12.0	1.0	11.5	135.7	0.78	0.52	0.525	12	1.25	66.5	1.2	18.0	0.9	845.301	5.06E-04	2.20E-03	0.220	2.50E-01	7.9523	1.88E-01	0.05
13.0	1.0	12.5	135.7	0.85	0.57	0.570	12	1.25	66.5	1.1	17.3	0.9	869.208	5.26E-04	2.20E-03	0.220	2.63E-01	7.9523	1.97E-01	0.05
14.5	1.5	13.8	135.7	0.93	0.62	0.625	12	1.25	66.5	1.1	16.4	0.9	897.357	5.48E-04	2.20E-03	0.220	2.78E-01	7.9523	2.09E-01	0.08
15.0	0.5	14.8	141.1	1.00	0.67	0.669	27	1.25	91.1	1.0	46.1	0.9	1310.763	3.96E-04	7.10E-04	0.071	2.61E-02	7.9523	1.96E-02	0.00
16.0	1.0	15.5	141.1	1.05	0.70	0.703	27	1.25	91.1	1.0	44.4	0.9	1328.564	4.06E-04	1.20E-03	0.120	4.61E-02	7.9523	3.46E-02	0.01
17.0	1.0	16.5	141.1	1.12	0.75	0.748	27	1.25	91.1	1.0	43.2	0.9	1359.995	4.16E-04	1.20E-03	0.120	4.76E-02	7.9523	3.58E-02	0.01
18.5	1.5	17.8	141.1	1.21	0.81	0.803	27	1.25	91.1	0.9	41.9	0.9	1397.637	4.27E-04	1.20E-03	0.120	4.94E-02	7.9523	3.72E-02	0.01
19.0	0.5	18.8	141.1	1.28	0.86	0.847	27	1.25	91.1	0.9	41.4	0.9	1432.082	4.34E-04	1.20E-03	0.120	5.02E-02	7.9523	3.77E-02	0.00
20.0	1.0	19.5	137.7	1.33	0.89	0.879	18	1.25	68.5	0.9	31.6	0.9	1335.135	4.78E-04	1.20E-03	0.120	6.94E-02	7.9523	5.21E-02	0.01
21.0	1.0	20.5	137.7	1.40	0.94	0.921	18	1.25	68.5	0.9	31.0	0.9	1360.306	4.86E-04	1.20E-03	0.120	7.10E-02	7.9523	5.34E-02	0.00
22.0	1.0	21.5	137.7	1.47	0.99	0.962	18	1.25	68.5	0.8	30.4	0.9	1384.758	4.92E-04	1.20E-03	0.120	7.26E-02	7.9523	5.46E-02	0.00
23.0	1.0	22.5	137.7	1.54	1.03	1.003	18	1.25	68.5	0.8	29.9	0.9	1408.545	4.98E-04	8.10E-04	0.081	5.00E-02	7.9523	3.76E-02	0.00
24.0	1.0	23.5	137.7	1.61	1.08	1.043	18	1.25	68.5	0.8	29.4	0.9	1431.714	5.04E-04	1.30E-03	0.130	8.19E-02	7.9523	6.16E-02	0.00
25.0	1.0	24.5	137.7	1.68	1.12	1.083	18	1.25	68.5	0.8	28.9	0.9	1454.309	5.09E-04	1.30E-03	0.130	8.35E-02	7.9523	6.28E-02	0.00
26.0	1.0	25.5	122.4	1.74	1.17	1.119	12	1.25	53.8	0.8	22.3	0.9	1359.314	5.57E-04	1.30E-03	0.130	1.14E-01	7.9523	8.57E-02	0.00
27.0	1.0	26.5	122.4	1.80	1.21	1.153	12	1.25	53.8	0.8	22.0	0.9	1377.549	5.60E-04	1.30E-03	0.130	1.16E-01	7.9523	8.70E-02	0.00
28.5	1.5	27.8	122.4	1.88	1.26	1.195	12	1.25	53.8	0.7	21.7	0.9	1399.856	5.63E-04	1.30E-03	0.130	1.18E-01	7.9523	8.84E-02	0.00
29.0	0.5	28.8	122.4	1.94	1.30	1.227	12	1.25	53.8	0.7	21.6	0.9	1419.869	5.64E-04	1.30E-03	0.130	1.18E-01	7.9523	8.90E-02	0.00
30.0	1.0	29.5	126.3	1.99	1.33	1.252	5	1.25	33.7	0.7	13.2	0.9	1220.606	6.65E-04	2.00E-03	0.200	3.28E-01	7.9523	2.46E-01	0.00
31.0	1.0	30.5	126.3	2.05	1.37	1.284	5	1.25	33.7	0.7	13.2	0.9	1236.814	6.66E-04	2.00E-03	0.200	3.31E-01	7.9523	2.49E-01	0.00
32.0	1.0	31.5	126.3	2.11	1.42	1.316	5	1.25	33.7	0.7	13.1	0.9	1252.760	6.68E-04	2.00E-03	0.200	3.34E-01	7.9523	2.51E-01	0.00
33.0	1.0	32.5	126.3	2.18	1.46	1.348	5	1.25	33.7	0.7	13.0	0.9	1268.457	6.69E-04	2.00E-03	0.200	3.36E-01	7.9523	2.53E-01	0.00
34.0	1.0	33.5	126.3	2.24	1.50	1.379	5	1.25	33.7	0.7	12.9	0.8	1283.914	6.70E-04	2.00E-03	0.200	3.39E-01	7.9523	2.55E-01	0.00
35.0	1.0	34.5	126.3	2.30	1.54	1.409	4	1.25	29.3	0.7	11.6	0.8	1258.630	6.92E-04	2.00E-03	0.200	3.83E-01	7.9523	2.88E-01	0.00
36.0	1.0	35.5	126.3	2.37	1.59	1.439	4	1.25	29.3	0.7	11.6	0.8	1273.486	6.93E-04	2.00E-03	0.200	3.85E-01	7.9523	2.90E-01	0.00
37.0	1.0	36.5	126.3	2.43	1.63	1.468	4	1.25	29.3	0.7	11.5	0.8	1288.136	6.93E-04	2.00E-03	0.200	3.88E-01	7.9523	2.91E-01	0.00
38.0	1.0	37.5	126.3	2.49	1.67	1.496	4	1.25	29.3	0.6	11.5	0.8	1302.590	6.93E-04	2.00E-03	0.200	3.90E-01	7.9523	2.93E-01	0.00
39.0	1.0	38.5	126.3	2.56	1.71	1.524	4	1.25	29.3	0.6	11.4	0.8	1316.855	6.93E-04	2.00E-03	0.200	3.92E-01	7.9523	2.95E-01	0.00
40.0	1.0	39.5	133.4	2.62	1.76	1.552	4	1.25	28.5	0.6	11.4	0.8	1331.841	6.92E-04	2.00E-03	0.200	3.94E-01	7.9523	2.96E-01	0.00
41.0	1.0	40.5	133.4	2.69	1.80	1.580	4	1.25	28.5	0.6	11.3	0.8	1347.520	6.92E-04	2.00E-03	0.200	3.95E-01	7.9523	2.97E-01	0.00
42.0	1.0	41.5	133.4	2.75	1.85	1.608	4	1.25	28.5	0.6	11.3	0.8	1362.983	6.91E-04	2.00E-03	0.200	3.96E-01	7.9523	2.98E-01	0.00
43.0	1.0	42.5	133.4	2.82	1.89	1.635	4	1.25	28.5	0.6	11.3	0.8	1378.240	6.90E-04	2.00E-03	0.200	3.98E-01	7.9523	2.99E-01	0.00
44.0	1.0	43.5	133.4	2.89	1.93	1.662	4	1.25	28.5	0.6	11.3	0.8	1393.297	6.89E-04	2.00E-03	0.200	3.99E-01	7.9523	3.00E-01	0.00
45.0	1.0	44.5	133.4	2.95	1.98	1.688	7	1.25	36.6	0.6	14.4	0.8	1529.852	6.33E-04	2.00E-03	0.200	2.97E-01	7.9523	2.23E-01	0.00
46.0	1.0	45.5	133.4	3.02	2.02	1.713	7	1.25	36.6	0.6	14.3	0.8	1545.353	6.32E-04	1.00E-02	1.000	1.49E+00	7.9523	1.12E+00	0.00
47.0	1.0	46.5	133.4	3.09	2.07	1.738	7	1.25	36.6	0.6	14.3	0.8	1560.656	6.31E-04	1.00E-02	1.000	1.50E+00	7.9523	1.12E+00	0.00
48.0	1.0	47.5	125.8	3.15	2.11	1.761	23	1.25	64.6	0.6	30.9	0.8	2037.569	4.87E-04	1.00E-02	1.000	5.94E-01	7.9523	4.47E-01	0.00
49.0	1.0	48.5	125.8	3.22	2.15	1.782	23	1.25	64.6	0.6	30.7	0.8	2054.909	4.86E-04	1.00E-02	1.000	5.97E-01	7.9523	4.49E-01	0.00
50.0	1.0	49.5	125.8	3.28	2.20	1.802	23	1.25	64.6	0.6	30.6	0.8	2072.034	4.85E-04	1.00E-02	1.000	6.00E-01	7.9523	4.51E-01	0.00

TOTAL SETTLEMENT = **0.38**

Figure 17



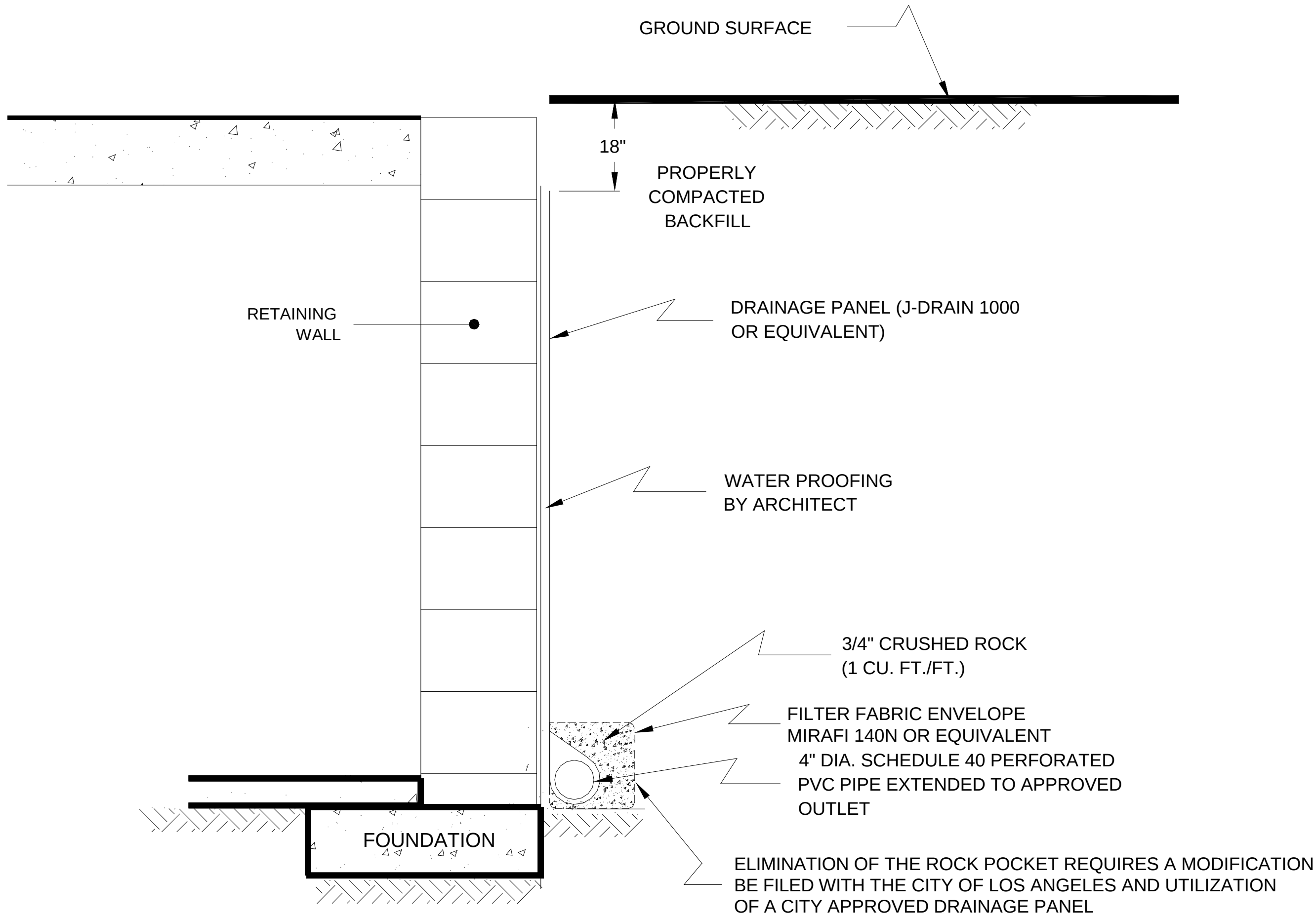
NO SCALE

GEOCON WEST, INC.
ENVIRONMENTAL GEOTECHNICAL MATERIALS
3303 N. SAN FERNANDO BLVD. - SUITE 100 - BURBANK, CA 91504
PHONE (818) 841-8388 - FAX (818) 841-1704

RETAINING WALL DRAIN DETAIL

ROSE HILL COURTS REDEVELOPMENT
4446 FLORIZEL STREET
LOS ANGELES, CALIFORNIA

DRAFTED BY: PZ	CHECKED BY: NDB	DEC. 2016	PROJECT NO. A9514-06-01	FIG. 18



NO SCALE

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LOS ANGELES, CALIFORNIA

DRAFTED BY: PZ	CHECKED BY: NDB	DEC. 2016	PROJECT NO. A9514-06-01	FIG. 19

APPENDIX A

APPENDIX A

FIELD INVESTIGATION

The site was explored on November 17 through 22, 2016, by excavating two 8-inch diameter borings to depths of approximately 26½ and 56½ feet below the existing ground surface utilizing a truck-mounted hollow-stem auger drilling machine. In addition, eleven 4-inch diameter boring to depths 5 to 11 feet below existing ground surface were excavating using hand tools. The site was also explored on April 12 and 13, 2018 by excavating four 8-inch diameter holes to depths of approximately 20½ and 30½ below existing ground surface utilizing a limited access hollow-stem auger-drilling machine. Also, three 4-inch diameter borings were excavated utilizing hand auger equipment to depths of approximately 5 to 15 feet below existing ground surface for percolation testing. Representative and relatively undisturbed samples were obtained by driving a 3-inch O. D., California Modified Sampler into the "undisturbed" soil mass with blows from a 140-pound auto-hammer falling 30 inches (hollow stem auger borings) and a slide hammer (hand auger borings). 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. Bulk samples were also obtained. Standard Penetration Tests were performed in borings B1 and B2.

The soil conditions encountered in the borings were visually examined, classified and logged in general accordance with the Unified Soil Classification System (USCS). The logs of the borings are presented on Figures A1 through A20. The log depicts 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. The locations of the borings are shown on Figure 2.

PROJECT NO. A9514-06-01

BORING 1

ELEV. (MSL.) **486.0** DATE COMPLETED **11/17/16**

EQUIPMENT **HOLLOW STEM AUGER** BY: **MDS**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0				ML	**AC: 4" CONCRETE: 2" BASE: NONE** **ALLUVIUM** Clayey Silt, stiff, slightly moist, dark brown, low plasticity, trace fine-grained sand.			
5	B1@5'					24	101.9	17.5
6				ML	Silt, firm, slightly moist, yellowish brown, trace fine-grained sand and clay.			
7.5	B1@7.5'					8	--	19.38
9				SM	Silty Sand, medium dense, moist, yellowish brown, fine- to medium-grained, oxidized.			
10	B1@10'					18	96.3	20.5
12.5	B1@12.5'				- olive gray with oxidation mottles, increase in silt content	26	--	28.7
15	B1@15'		▼		**PUENTE FORMATION** Sandstone, soft to medium hard, slightly moist, olive gray, highly weathered, thinly bedded, medium-grained, oxidation mottling. - 15.0' groundwater (perched)	50 (6")	107.0	21.1
17.5	B1@17.5'					43	--	24.3
20	B1@20'					50 (6")	110.8	20.4
25	B1@25'					32	--	23.0

Total depth of boring: 26.5 feet; no fill.
Perched groundwater encountered at approximately 15 feet.
Backfilled with soil cuttings and tamped.
*Penetration resistance for 140-pound hammer falling 30 inches by auto-hammer.

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

A9514-06-01 BORING LOGS #1-13.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. A9514-06-01

BORING 2

ELEV. (MSL.) **504.0** DATE COMPLETED **11/17/16**

EQUIPMENT **HOLLOW STEM AUGER** BY: **MDS**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0					**AC: 4" ASPHALT: 2" BASE: NONE** **ALLUVIUM** Silt with Sand, stiff, slightly moist, brown to dark brown, fine-grained, some clay.			
2				ML				
4								
	B2@5'				Clayey Silt with Sand, stiff, slightly moist, dark reddish brown, fine-grained, medium plasticity.	17	--	--
6								
8	B2@7.5'					33	115.7	16.3
10	B2@10'			ML	- firm	12	--	16.9
12	B2@12.5'				- stiff, increase in sand content, low plasticity	35	114.4	18.6
14								
	B2@15'				Sandy Clay, firm, slightly moist, dark brown, fine-grained, trace clay.	27	--	18.3
16								
18	B2@17.5'				- hard, increase in clay content, medium to low plasticity	53	121.6	16.0
20	B2@20'				- stiff	18	--	16.6
22	B2@22.5'			CL	- hard	43	117.5	17.2
24								
	B2@25'				- firm, moist, yellowish brown	12	--	18.8
26								
28	B2@27.5'					19	101.8	20.2

A9514-06-01 BORING LOGS #1-13.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

BORING 2

ELEV. (MSL.) **504.0** DATE COMPLETED **11/17/16**

EQUIPMENT **HOLLOW STEM AUGER** BY: **MDS**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
30	B2@30'			CL	- soft	5	--	25.6
32	B2@32.5'				- firm	12	102.1	23.7
34								
36	B2@35'				- soft, becomes olive brown to yellowish brown, wet	4	--	23.6
38	B2@37.5'				- firm, no recovery	16	--	--
40	B2@40'		▼		- soft, 40.0' groundwater (perched), wet	4	--	24.2
42	B2@42.5'					11	109.1	22.3
44								
46	B2@45'					7	--	22.3
48	B2@47.5'				**PUENTE FORMATION** Sandy Siltstone, soft to medium hard, slightly moist, olive gray with oxidation mottles, fine-grained, moderately weathered.	50 (5")	103.4	21.7
50	B2@50'					23	--	25.1
52								
54								
56								

Total depth of boring: 56.5 feet. No fill.
Perched groundwater encountered at 40 feet.
Backfilled with soil cuttings and tamped.
*Penetration resistance for 140-pound hammer falling 30 inches by auto-hammer.

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

A9514-06-01 BORING LOGS #1-13.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 3** ELEV. (MSL.) **494.0** DATE COMPLETED **11/18/16** EQUIPMENT **HAND AUGER** BY: **MDS**	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
					MATERIAL DESCRIPTION			
0	BULK 0-5'				**ARTIFICIAL FILL** Silty Sand, medium dense, dry, light brown, fine-grained, some medium-grained, trace rootlets, trace fine gravel.			
2	B3@2'				- brick in sampler		--	23.7
4								
	B3@5'						86.0	23.7
6					**ALLUVIUM**			
	B3@7'			SM	Silty Sand, medium dense, dry, yellowish brown, fine- to medium-grained, porous.		80.7	24.2
8					Silt, stiff, slightly moist, dark brown, some fine-grained sand.			
				ML				
10	B3@10'				- dark reddish brown		113.8	16.8
					Total depth of boring: 11 feet Fill to 6 feet. No groundwater encountered. Backfilled with soil cuttings and tamped.			

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

A9514-06-01 BORING LOGS #1-13.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. A9514-06-01

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	BORING 4 ELEV. (MSL.) 488.0 DATE COMPLETED 11/18/16 EQUIPMENT HAND AUGER BY: MDS	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
					MATERIAL DESCRIPTION			
0					**ARTIFICIAL FILL** Silty Sand and Sandy Silt, medium dense to stiff, dry, light yellowish brown, fine- to medium-grained, trace rootlets.			
2					- increase in silt content			
	B4@2.5'						--	--
	B4@3'						111.0	8.7
4								
	B4@5'						99.5	7.8
6					Total depth of boring: 6 feet All fill. No groundwater encountered. Backfilled with soil cuttings and tamped.			

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

A9514-06-01 BORING LOGS #1-13.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 5

ELEV. (MSL.) **480.0** DATE COMPLETED **11/18/16**

EQUIPMENT **HAND AUGER** BY: **MDS**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0					**ALLUVIUM**			
				ML	Silt with Sand, stiff, moist, dark brown, fine-grained.			
2					- bedrock fragment			
4					Clayey Silt, soft to stiff, moist, dark brown, trace fine-grained sand.			
6				ML				
8								
10					Total depth of boring: 10 feet No fill. No groundwater encountered. Backfilled with soil cuttings and tamped.			

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

A9514-06-01 BORING LOGS #1-13.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 6

ELEV. (MSL.) **487.0** DATE COMPLETED **11/18/16**

EQUIPMENT **HAND AUGER** BY: **MDS**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0					**ARTIFICIAL FILL** Silty Sand, medium dense, dry, light brown, fine- to medium-grained, some rootlets.			
2								
	B6@3'			ML	**ALLUVIUM** Sandy Silt, dry, firm, brown, fine-grained.		92.3	10.1
4								
6								
	B6@6.5'			SM/ML	Silty Sand to Sandy Silt, medium dense to stiff, slightly moist, olive brown with oxidation mottles, fine-grained.		116.8	14.0
8								
10	B6@9.5'				- sandy silt, highly oxidized		112.0	19.0

Total depth of boring: 10 feet
Fill to 2.5 feet.
No groundwater encountered.
Backfilled with soil cuttings and tamped.

A9514-06-01 BORING LOGS #1-13.GPJ

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

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

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 7

ELEV. (MSL.) **496.0** DATE COMPLETED **11/21/16**

EQUIPMENT **HAND AUGER** BY: **MDS**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0					**ALLUVIUM** Sandy Silt, soft, moist, brown, fine-grained, trace clay.			
2	B7@1.5'			ML			104.5	21.6
4	B7@4'				- firm, olive brown with oxidation mottles, no clay		96.5	13.4
					Total depth of boring: 5 feet No fill. No groundwater encountered. Backfilled with soil cuttings and tamped.			

Figure A7,
Log of Boring 7, Page 1 of 1

A9514-06-01 BORING LOGS #1-13.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 8

ELEV. (MSL.) **501.0** DATE COMPLETED **11/21/16**

EQUIPMENT **HAND AUGER** BY: **MDS**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0					**ALLUVIUM** Sandy Silt, soft, moist to wet, dark brown, fine-grained, abundant rootlets, trace gravel and clay.			
2	B8@1.5'						108.2	17.2
				ML				
4								
	B8@5'						81.8	1.3
6					Total depth of boring: 6 feet. No fill. No groundwater encountered. Backfilled with soil cuttings and tamped.			

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

A9514-06-01 BORING LOGS #1-13.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 9

ELEV. (MSL.) **505.0** DATE COMPLETED **11/21/16**

EQUIPMENT **HAND AUGER** BY: **MDS**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0					**AC: 10"** **ARTIFICIAL FILL** Sandy Silt, soft to stiff, slightly moist, brown, fine-grained, abundant brick fragments.			
2								
	B9@3'						111.5	18.2
4								
					ALLUVIUM Sandy Silt, stiff, moist, brown, fine-grained, some clay, oxidized.			
6	B9@6'						107.2	20.8
				ML				
8								
	B9@9'						112.9	18.5
10								

Total depth of boring: 10 feet
Fill to 5 feet.
No groundwater encountered.
Backfilled with soil cuttings and tamped.
Concrete patched.

A9514-06-01 BORING LOGS #1-13.GPJ

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

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

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

GEOCON

BORING 10

ELEV. (MSL.) **499.0** DATE COMPLETED **11/22/16**

EQUIPMENT **HAND AUGER** BY: **RMA**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0					**ARTIFICIAL FILL** Silty Sand, loose, slightly moist, dark yellowish brown, fine-grained, some medium-grained.			
2	B10@2'						103.8	14.7
4								
	B10@5'						112.1	14.3
					Total depth of boring: 5.5 feet All fill. No groundwater encountered. Backfilled with soil cuttings and tamped.			

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

A9514-06-01 BORING LOGS #1-13.GPJ

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

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

BORING 11

ELEV. (MSL.) **507.0** DATE COMPLETED **11/22/16**

EQUIPMENT **HAND AUGER** BY: **RMA**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0					**ARTIFICIAL FILL** Silty Sand, loose, slightly moist, dark yellowish brown, abundant rootlets.			
2	B11@2'				Clayey Silt, firm to hard, slightly moist, dark yellowish brown, trace fine-grained sand.		--	--
4					**ALLUVIUM** Sandy Silt, hard, slightly moist, brown, fine-grained, trace clay.			
6	B11@5'						118.8	12.4
8	B11@7'			ML			112.1	13.8
10	B11@10'						99.3	14.8

Total depth of boring: 10.5 feet
Fill to 4 feet.
No groundwater encountered.
Backfilled with soil cuttings and tamped.

A9514-06-01 BORING LOGS #1-13.GPJ

Figure A11,
Log of Boring 11, 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.

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PROJECT NO. A9514-06-01

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	BORING 12 ELEV. (MSL.) 508.0 DATE COMPLETED 11/22/16 EQUIPMENT HAND AUGER BY: RMA	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
					MATERIAL DESCRIPTION			
0					**ARTIFICIAL FILL** Silty Sand, medium dense, slightly moist, dark yellowish brown, trace clay.			
2	B12@2'						107.3	16.1
4					**ALLUVIUM** Sand with Silt, poorly graded, dense, slightly moist, yellowish brown, trace clay.			
6				SP-SM				
	B12@7.5'						88.7	11.6
					Total depth of boring: 7.5 feet Fill to 3 feet. No groundwater encountered. Backfilled with soil cuttings and tamped.			

A9514-06-01 BORING LOGS #1-13.GPJ

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

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

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BORING 13

ELEV. (MSL.) **493.0** DATE COMPLETED **11/22/16**

EQUIPMENT **HAND AUGER** BY: **RMA**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0					**ARTIFICIAL FILL** Silty Sand, medium dense to dense, slightly moist, dark yellowish brown, trace clay.			
2								
	B13@3'						97.1	9.9
4					**ALLUVIUM** Sand with Silt, poorly graded, dense, slightly moist, dark yellowish brown, trace clay.			
	B13@5'						105.7	9.5
6	BULK 5-10'			SP-SM	- slightly porous, trace secondary calcium carbonate - very dense			
	B13@7'						104.8	8.9
8								
10				SM	Silty Sand, very dense, slightly moist, dark yellowish brown, fine- to medium-grained, trace to some clay.			
	B13@10'						115.7	14.0

Total depth of boring: 10.5 feet
Fill to 3.5 feet.
No groundwater encountered.
Backfilled with soil cuttings and tamped.

Figure A13,
Log of Boring 13, Page 1 of 1

A9514-06-01 BORING LOGS #1-13.GPJ

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

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

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BORING 14

ELEV. (MSL.) - - DATE COMPLETED 4/13/18

EQUIPMENT LIMITED ACCESS RIG BY: MDS

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0					**ALLUVIUM** Silt, stiff, slightly moist, dark brown, trace fine-grained sand.			
2								
	B1@3'			ML	- brown, increase in fine-grained, trace oxidation	22	108.8	15.6
4								
	B1@5'				- orangish brown, increase in oxidation	22	106.8	16.9
6								
	B1@7'				Silt, firm, slightly moist, orangish brown, oxidized, trace fine-grained sand.	16	98.6	21.0
8				ML				
10	B1@10'			ML	Silt with Sand, firm, slightly moist, light brown, fine-grained.	15	106.7	21.9
12	B1@12'				**PUENTE FORMATION** Siltstone, soft, slightly moist, yellowish brown, oxidized, completely weathered, thinly bedded.	25	94.2	30.8
14								
	B1@15'				- moderately weathered, moderately hard	61	97.4	28.0
16								
18								
20	B1@20'				Sandstone, slightly moist, moderately hard, yellowish brown, oxidized, thinly bedded, moderately weathered, fine-grained.	26	95.7	17.4
22								
24								
	B1@25'				- moderately hard	39	100.2	22.0
26								
28								

Figure A14,
Log of Boring 14, Page 1 of 2

A9514-06-01 BORING LOGS #14-20.GPJ

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

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PROJECT NO. A9514-06-01

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	**BORING 14** ELEV. (MSL.) - - DATE COMPLETED 4/13/18 EQUIPMENT LIMITED ACCESS RIG BY: MDS	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
					MATERIAL DESCRIPTION			
30	B1@30'						103.0	23.3
					Total depth of boring: 30.5 feet No fill. No groundwater encountered. Backfilled with soil cuttings and tamped. *Penetration resistance for 140-pound hammer falling 30 inches by auto-hammer.			

Figure A14,
Log of Boring 14, Page 2 of 2

A9514-06-01 BORING LOGS #14-20.GPJ

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

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PROJECT NO. A9514-06-01

BORING 15

ELEV. (MSL.) - - DATE COMPLETED 4/12/18

EQUIPMENT LIMITED ACCESS RIG BY: MDS

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0				ML	**ALLUVIUM** Sandy Silt, stiff, slightly moist, dark brown, fine-grained.			
3	B2@3'					25	92.5	15.0
5	B2@5'					24	106.7	9.3
7	B2@7'				- hard	43	116.7	12.5
10	B2@10'					53	110.3	13.2
12	B2@12'					35	117.8	13.4
15	B2@15'				- firm	19	115.0	14.7
20	B2@20'				- stiff, reddish brown	37	115.3	14.8
25	B2@25'				- increase in fine-grained	29	113.5	15.0

A9514-06-01 BORING LOGS #14-20.GPJ

Figure A15,
Log of Boring 15, Page 1 of 2

SAMPLE SYMBOLS

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

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DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	BORING 15 ELEV. (MSL.) - - DATE COMPLETED 4/12/18 EQUIPMENT LIMITED ACCESS RIG BY: MDS MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
30	B2@30'			ML		31	115.7	15.2
					Total depth of boring: 30.5 feet No fill. No groundwater encountered. Backfilled with soil cuttings and tamped. *Penetration resistance for 140-pound hammer falling 30 inches by auto-hammer.			

Figure A15,
Log of Boring 15, Page 2 of 2

A9514-06-01 BORING LOGS #14-20.GPJ

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

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

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PROJECT NO. A9514-06-01

BORING 16

ELEV. (MSL.) **- -** DATE COMPLETED **4/12/18**

EQUIPMENT **LIMITED ACCESS RIG** BY: **MDS**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0					**AC: 8"** **ALLUVIUM** Silt, stiff, slightly moist, dark brown, trace fine-grained sand.			
4	B3@4'					31	114.0	15.9
8	B3@8'			ML		29	108.2	17.4
12	B3@12'				- moist, brown	32	107.2	17.2
16	B3@16'				- firm, reddish brown	15	105.2	18.7
20	B3@20'				- stiff	27	109.8	19.7

Total depth of boring: 20.5 feet
No fill.
No groundwater encountered.
Backfilled with soil cuttings and tamped.

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

A9514-06-01 BORING LOGS #14-20.GPJ

Figure A16,
Log of Boring 16, 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.

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BORING 17

ELEV. (MSL.) - - DATE COMPLETED 4/12/18

EQUIPMENT LIMITED ACCESS RIG BY: MDS

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0					**ALLUVIUM** Sandy Silt, stiff, dry, dark brown, fine-grained.			
4	B4@4'					37	112.1	18.2
8	B4@8'			ML	- slightly moist	22	109.3	13.2
12	B4@12'				- increase in sand content	27	110.7	16.7
15			▼		**PUENTE FORMATION**			
16	B4@16'				Siltstone, soft, wet, yellowish brown, oxidized, fine-grained, completely weathered.	33	106.0	21.1
20	B4@20'						78.0	23.3

Total depth of boring: 20.5 feet
No fill.
Groundwater perched at 15 feet.
Backfilled with soil cuttings and tamped.

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

A9514-06-01 BORING LOGS #14-20.GPJ

Figure A17,
Log of Boring 17, Page 1 of 1

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

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PROJECT NO. A9514-06-01

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	BORING 18 ELEV. (MSL.) - - DATE COMPLETED 4/13/18 EQUIPMENT HAND AUGER BY: MDS	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
					MATERIAL DESCRIPTION			
0 2 4				SM	**ALLUVIUM** Silty Sand, stiff, dry to slightly moist, dark brown, very fine- to fine-grained.			
					Total depth of boring: 5 feet No fill. No groundwater encountered. Backfilled with soil cuttings following percolation testing.			

A9514-06-01 BORING LOGS #14-20.GPJ

Figure A18,
Log of Boring 18, Page 1 of 1

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

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DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	BORING 19 ELEV. (MSL.) - - DATE COMPLETED 4/13/18 EQUIPMENT HAND AUGER BY: MDS	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
					MATERIAL DESCRIPTION			
0					**ALLUVIUM** Silty Sand, firm, dry to slightly moist, dark brown, very fine- to fine-grained.			
2				SM				
4					- brown			
					Total depth of boring: 5 feet No fill. No groundwater encountered.			

Figure A19,
Log of Boring 19, Page 1 of 1

A9514-06-01 BORING LOGS #14-20.GPJ

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

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PROJECT NO. A9514-06-01

BORING 20

ELEV. (MSL.) - - DATE COMPLETED 4/13/18

EQUIPMENT HAND AUGER BY: MDS

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0					**ALLUVIUM**			
0–6				SM	Silty Sand, medium dense, dry, dark brown, very fine- to fine-grained, porous.			
4					- brown			
5	B7@5'						94.4	10.3
6–15				ML	Sandy Silt, firm, dry, brown, fine-grained, porous.			
10	B7@10'						97.1	13.0
14.5	B7@14.5'						101.9	12.0

Total depth of boring: 15 feet
No fill.
No groundwater encountered.
Backfilled with soil cuttings following percolation testing.

A9514-06-01 BORING LOGS #14-20.GPJ

Figure A20,
Log of Boring 20, Page 1 of 1

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

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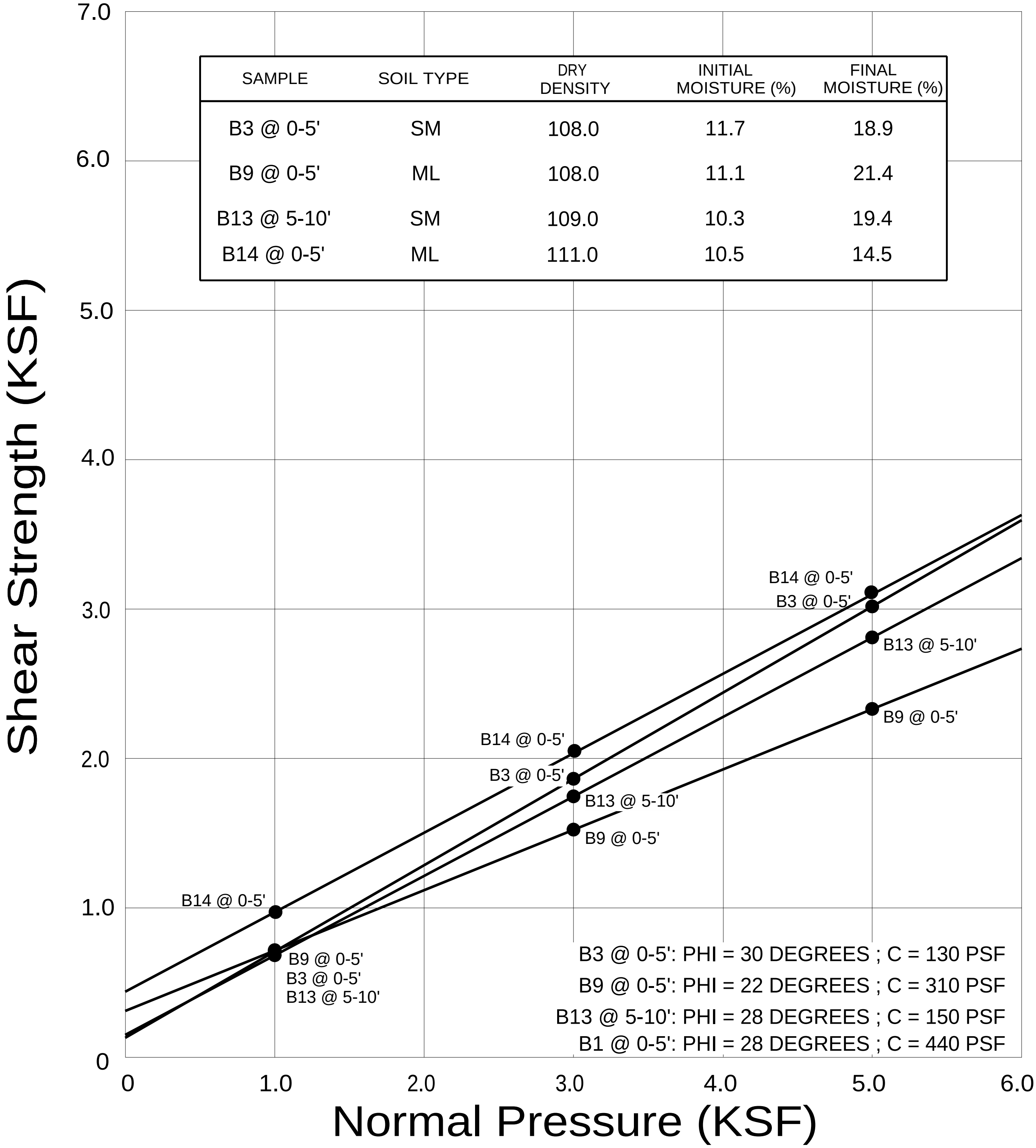
APPENDIX B

APPENDIX B

LABORATORY TESTING

Laboratory tests were performed in accordance with generally accepted test methods of the "American Society for Testing and Materials (ASTM)", or other suggested procedures. Selected samples were tested for direct shear strength, consolidation, expansion characteristics, plasticity indices, grain-size distribution, corrosivity, and in-place dry density and moisture content. The results of the laboratory tests are summarized in Figures B1 through B13. The in-place dry density and moisture content of the samples tested are presented on the boring logs in Appendix A.



SAMPLE	SOIL TYPE	DRY DENSITY	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B3 @ 0-5'	SM	108.0	11.7	18.9
B9 @ 0-5'	ML	108.0	11.1	21.4
B13 @ 5-10'	SM	109.0	10.3	19.4
B14 @ 0-5'	ML	111.0	10.5	14.5

● Direct Shear, Saturated

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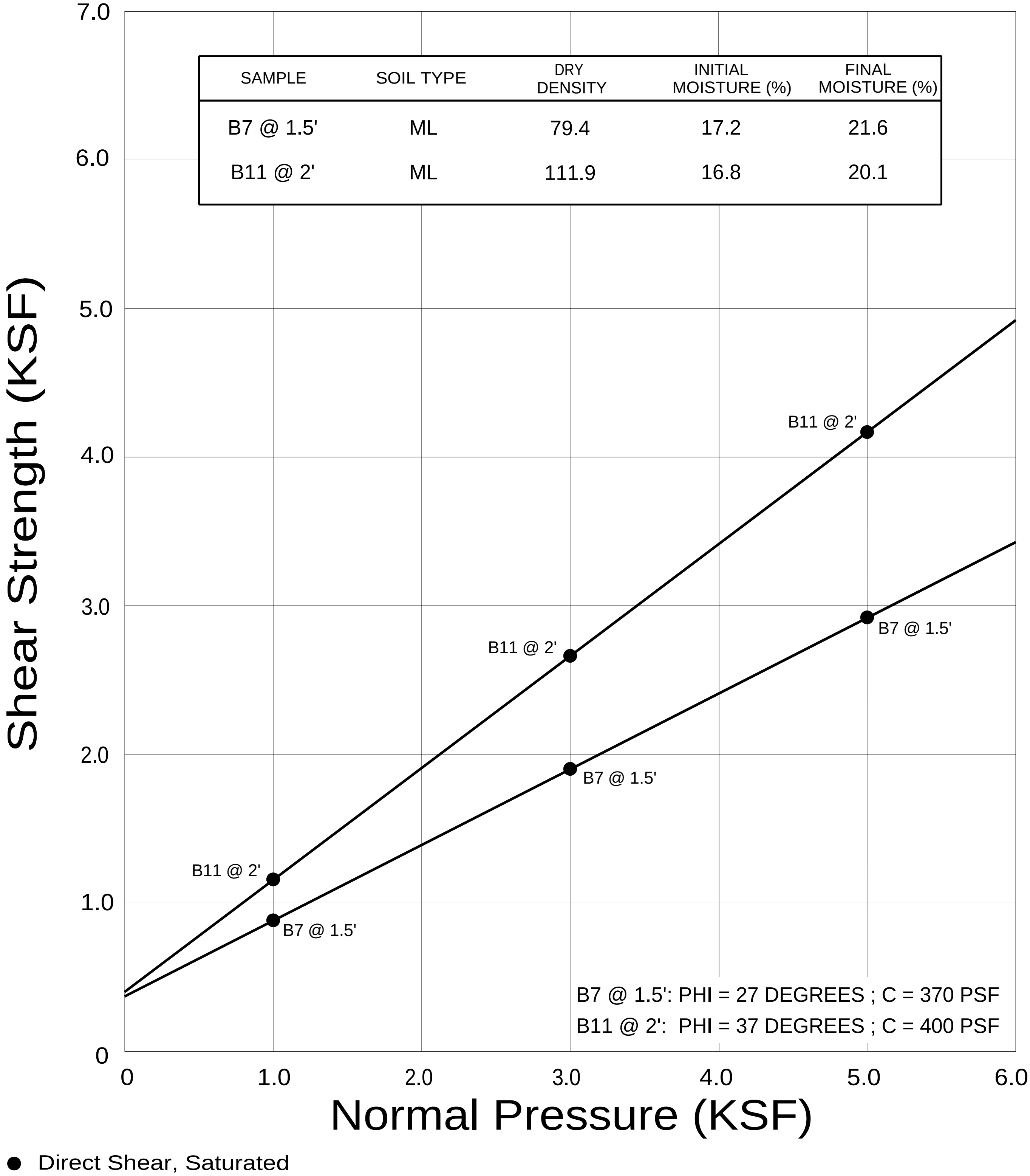
DIRECT SHEAR TEST RESULTS

ROSE HILL COURTS REDEVELOPMENT
4446 FLORIZEL STREET
LOS ANGELES, CALIFORNIA

DRAFTED BY: PZ	CHECKED BY: NDB	MAY 2018	PROJECT NO. A9514-06-01	FIG. B1



SAMPLE	SOIL TYPE	DRY DENSITY	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B7 @ 1.5'	ML	79.4	17.2	21.6
B11 @ 2'	ML	111.9	16.8	20.1

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DIRECT SHEAR TEST RESULTS

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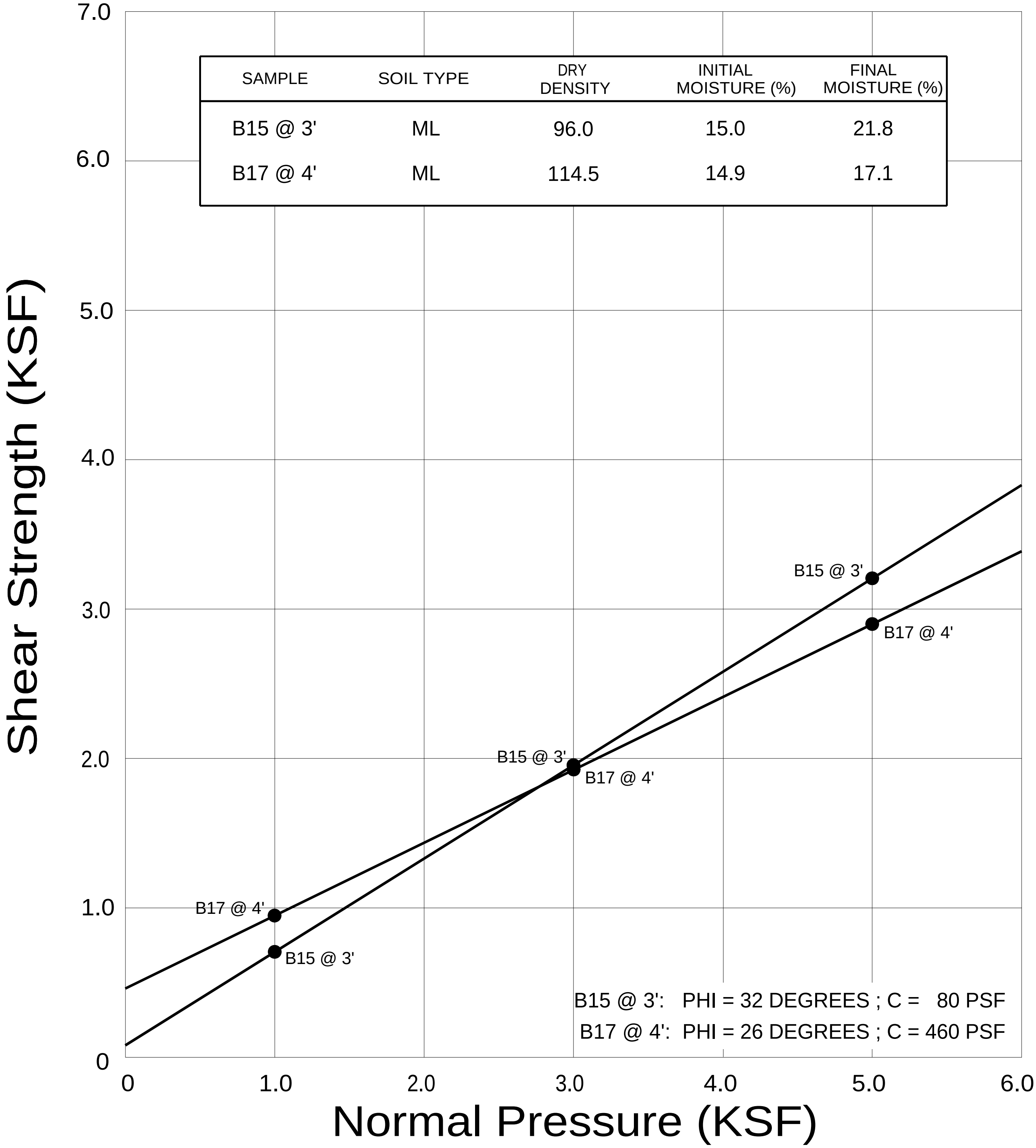
MAY 2018	PROJECT NO. A9514-06-01	FIG. B2

SAMPLE	SOIL TYPE	DRY DENSITY	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B15 @ 3'	ML	96.0	15.0	21.8
B17 @ 4'	ML	114.5	14.9	17.1



● Direct Shear, Saturated

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DIRECT SHEAR TEST RESULTS

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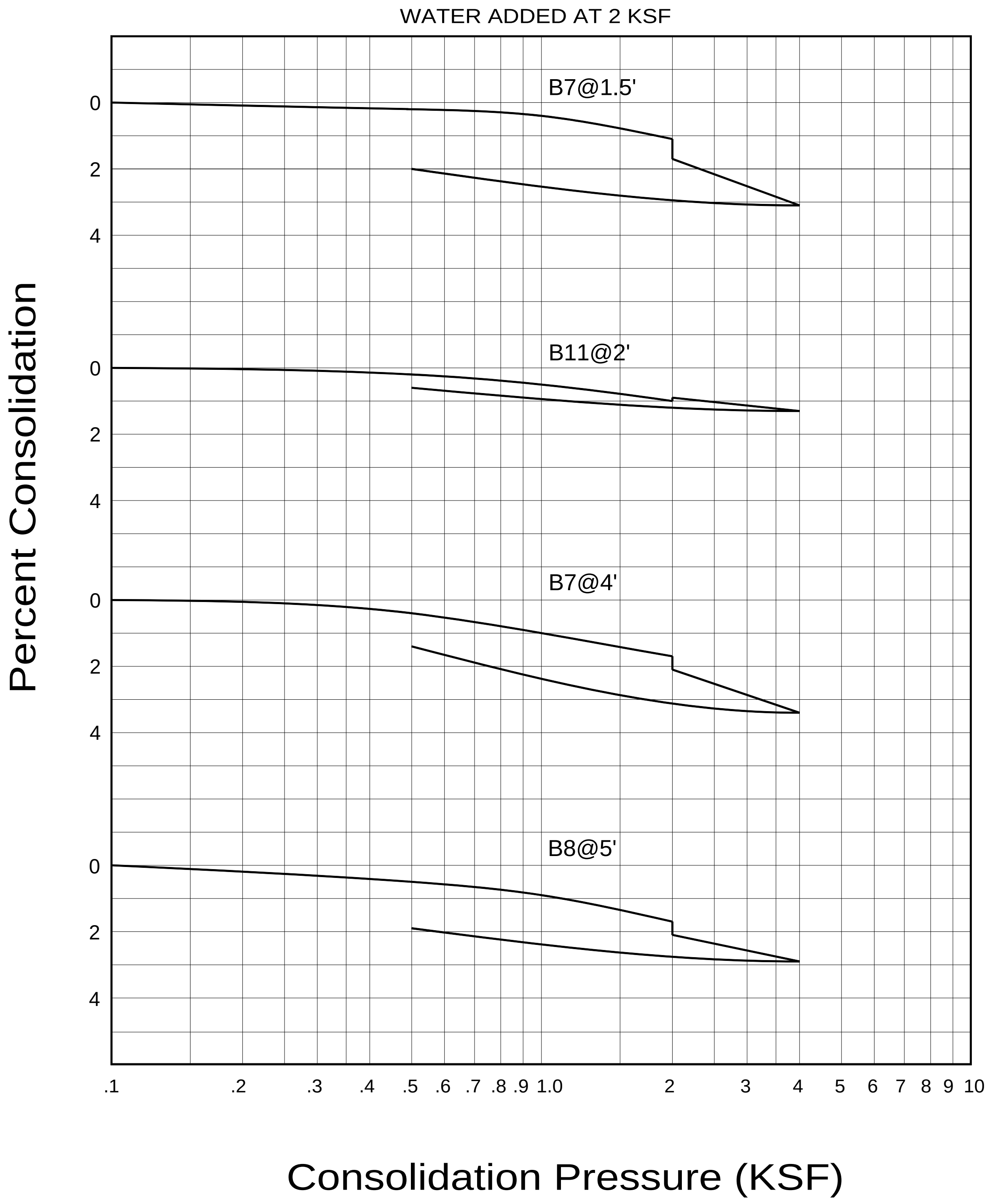
MAY 2018 | PROJECT NO. A9514-06-01 | FIG. B3



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CONSOLIDATION TEST RESULTS

ROSE HILL COURTS REDEVELOPMENT

4446 FLORIZEL STREET

LOS ANGELES, CALIFORNIA

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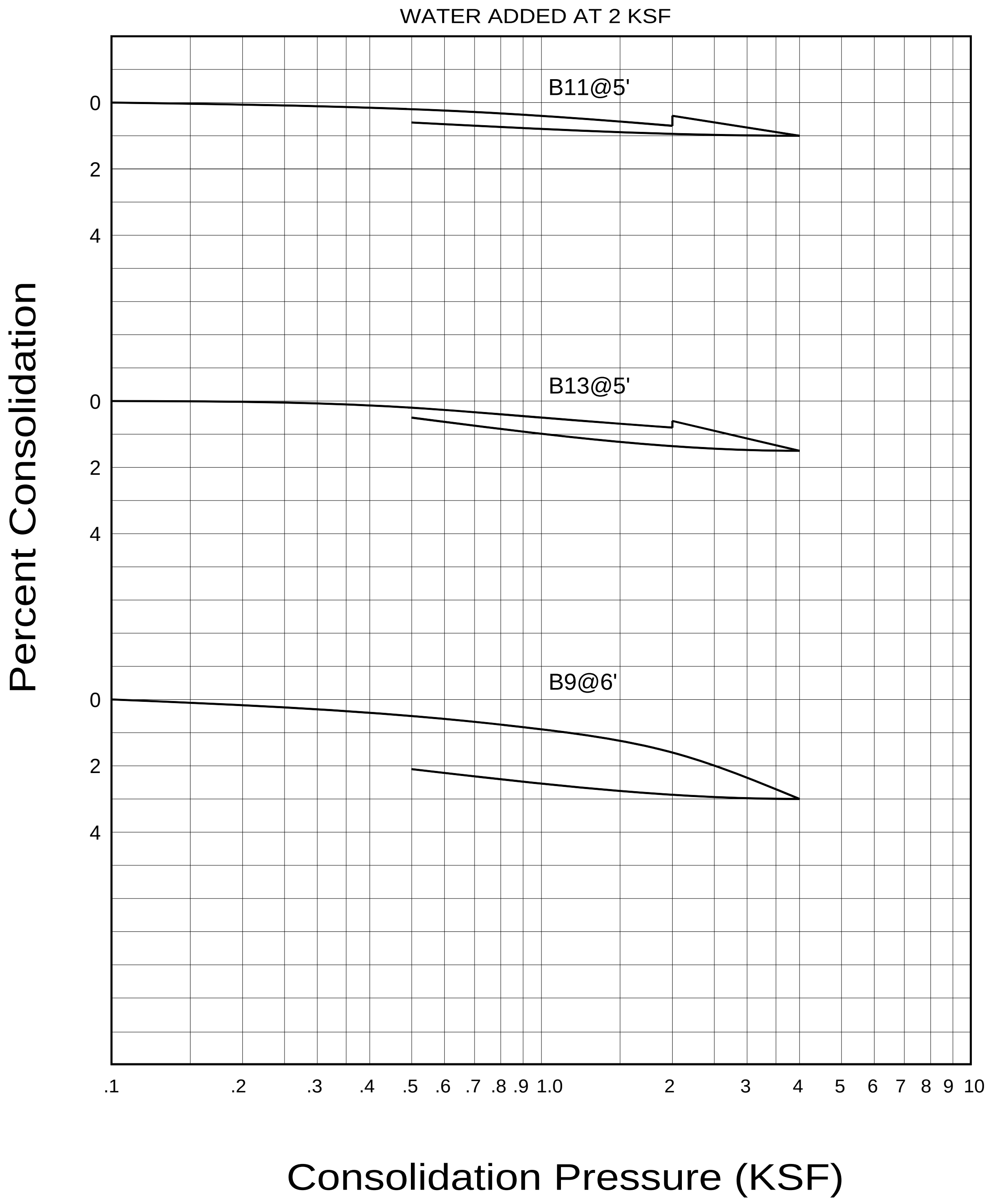
MAY 2018	PROJECT NO. A9514-06-01	FIG. B4



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CONSOLIDATION TEST RESULTS

ROSE HILL COURTS REDEVELOPMENT
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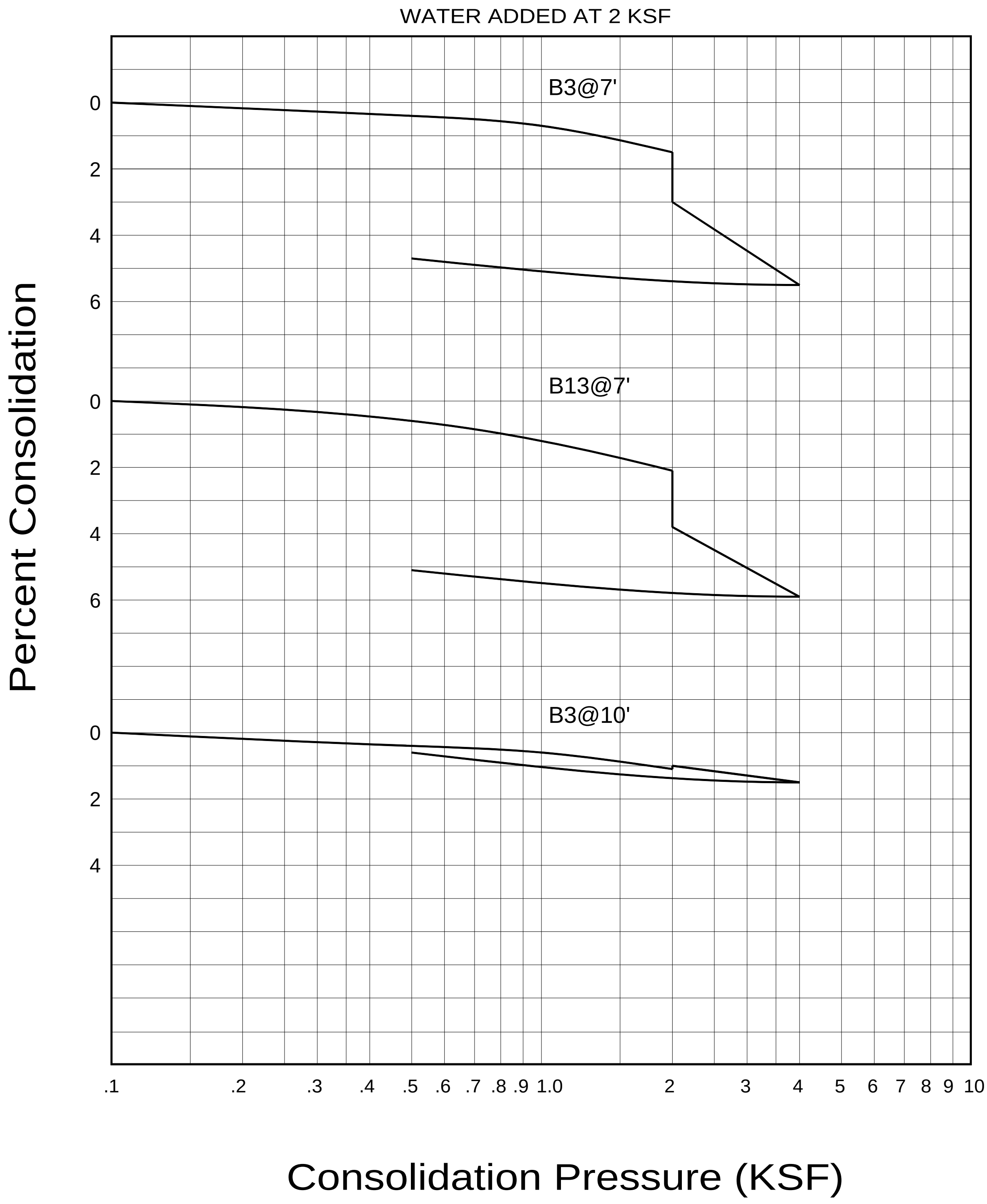
MAY 2018	PROJECT NO. A9514-06-01	FIG. B5



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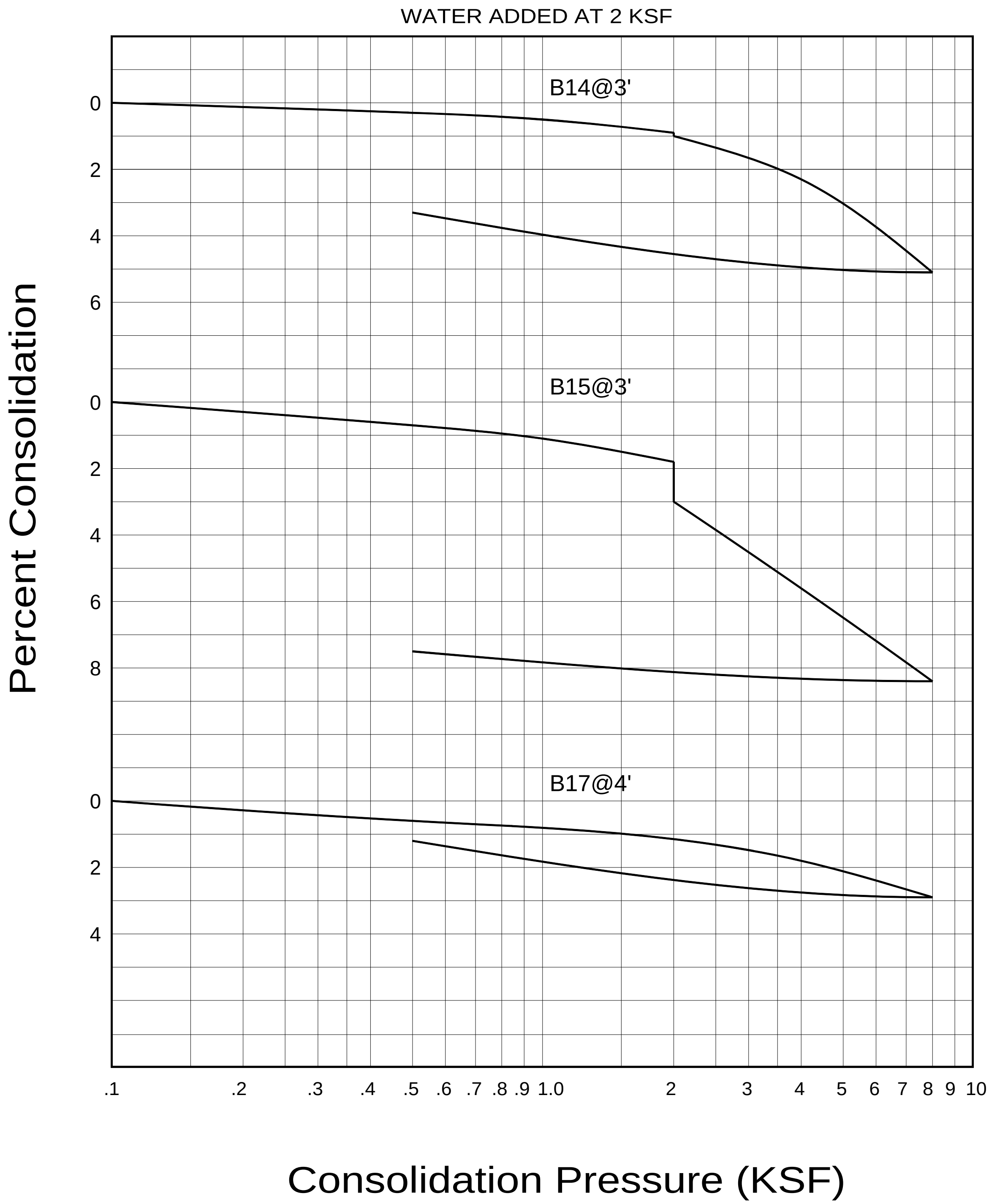
MAY 2018	PROJECT NO. A9514-06-01	FIG. B6



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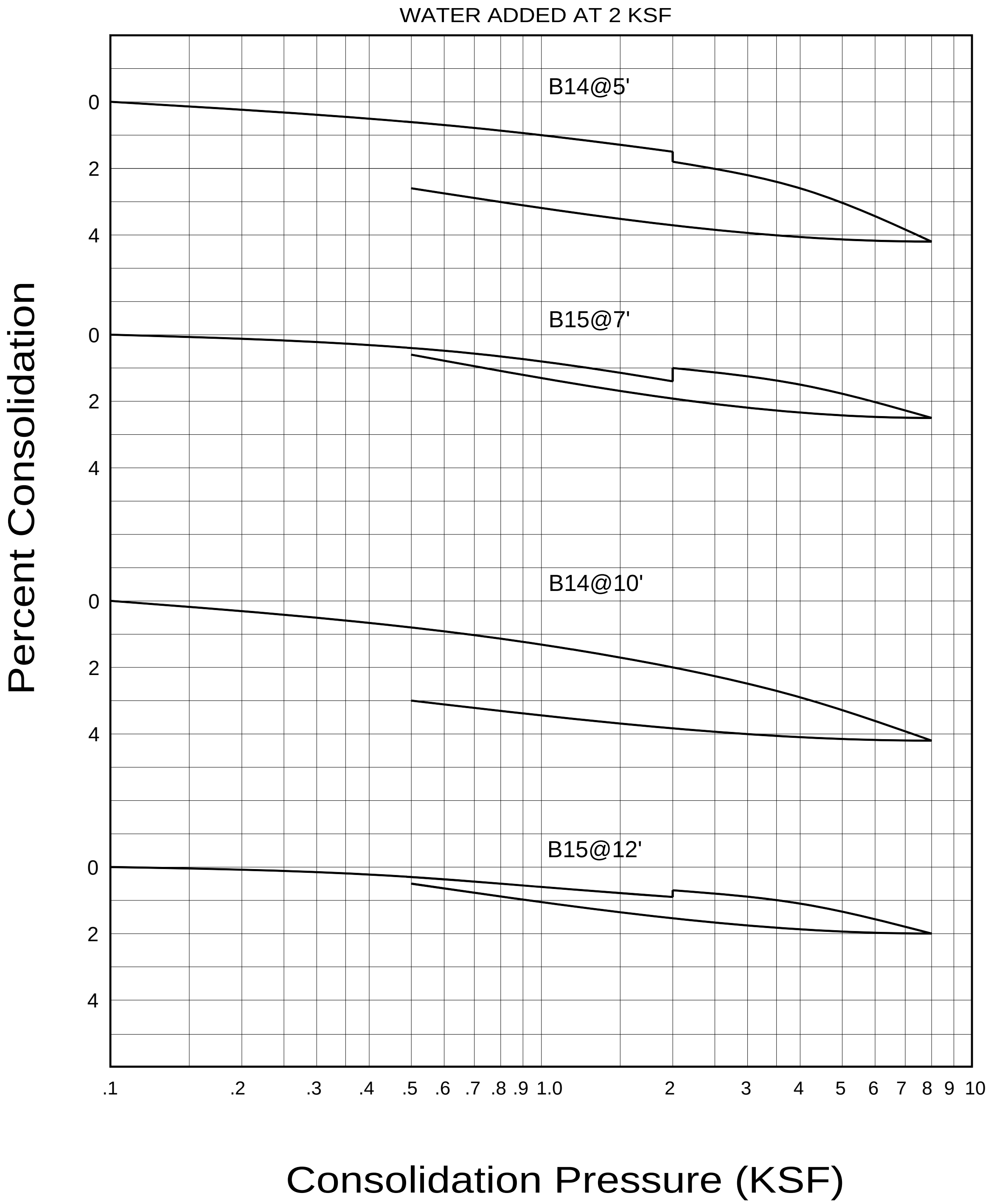

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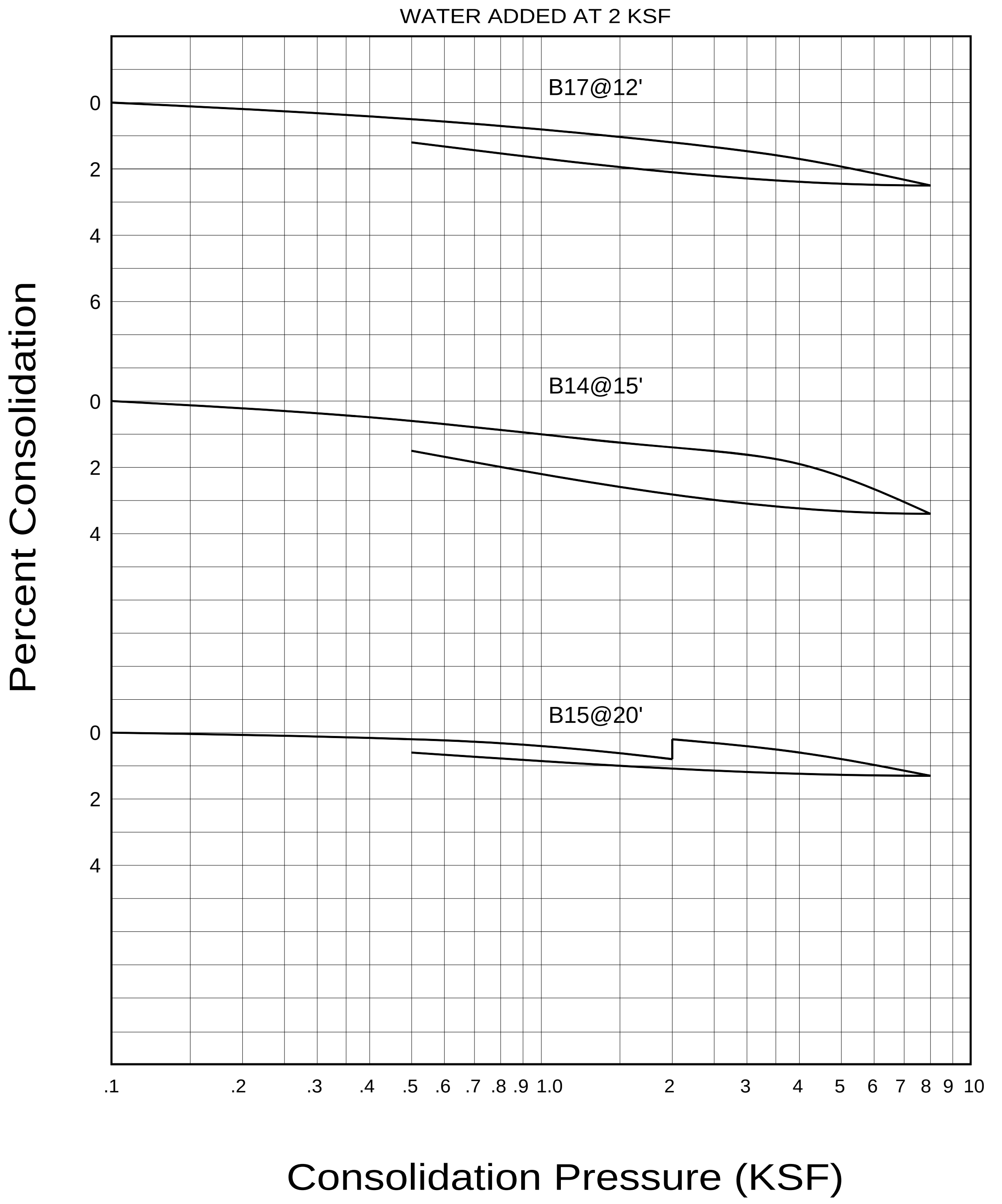

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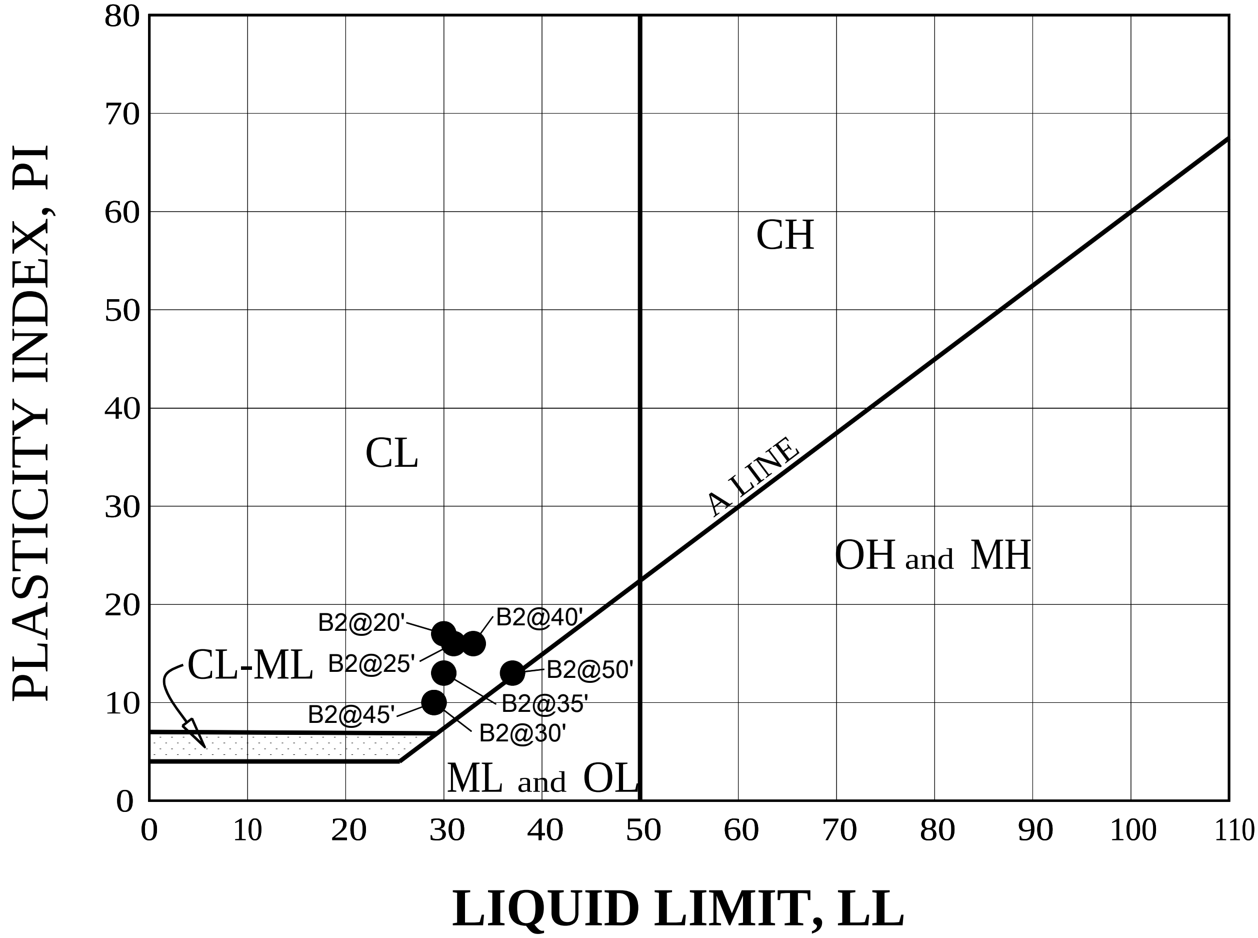

BORING NUMBER	DEPTH (FEET)	LL	PL	PI	MOISTURE CONTENT AT SATURATION	SOIL BEHAVIOR
B2	**20**	**30**	**13**	**17**	**18.3**	**CL**
B2	**25**	**31**	**15**	**16**	**22.1**	**CL**
B2	**30**	**29**	**19**	**10**	**---**	**CL**
B2	**35**	**30**	**17**	**13**	**22.7**	**CL**
B2	**40**	**33**	**17**	**16**	**22.5**	**CL**
B2	**45**	**29**	**19**	**10**	**---**	**CL**
B2	**50**	**37**	**24**	**13**	**---**	**CL**

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ATTERBERG LIMITS

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4446 FLORIZEL STREET
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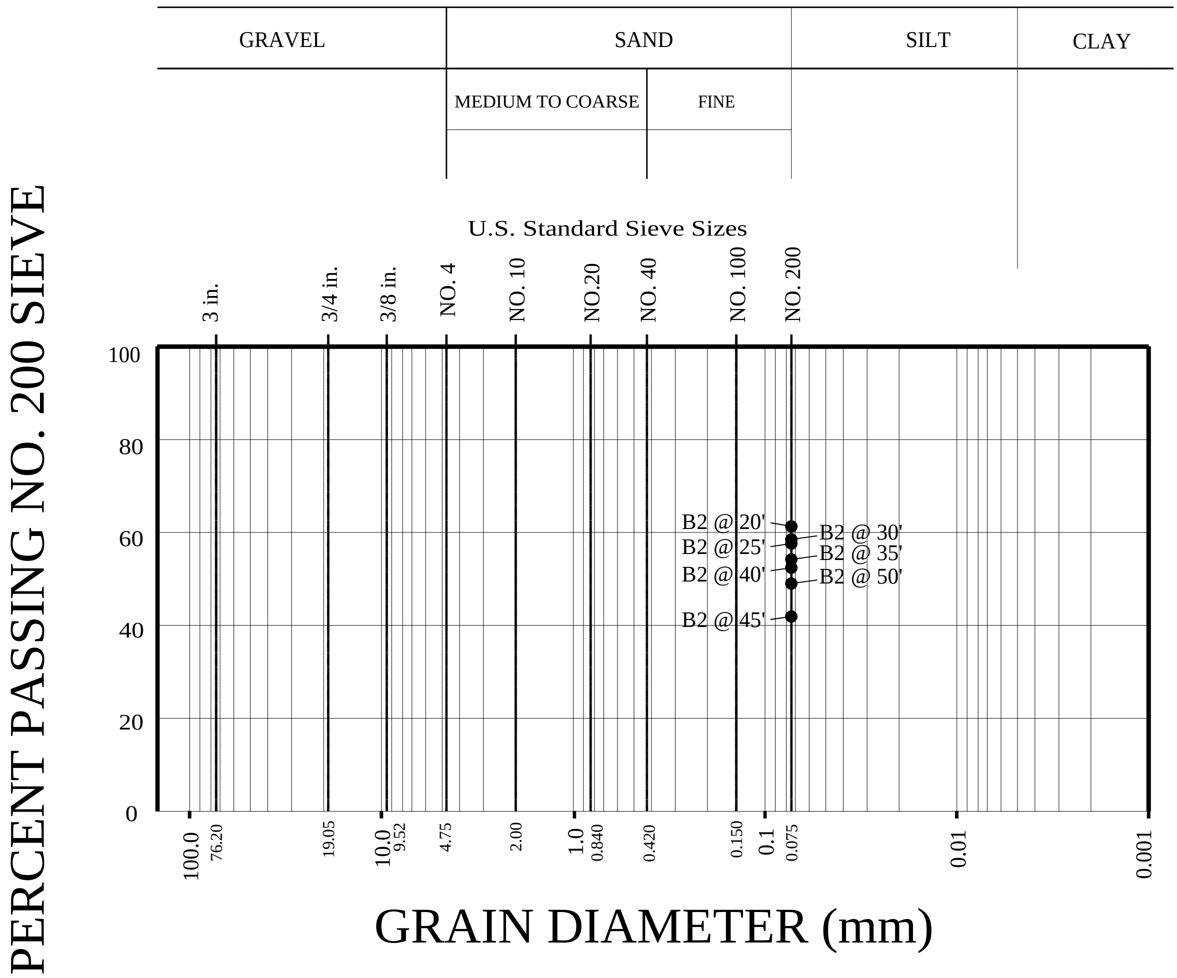

SAMPLE	PERCENT PASSING NO. 200 SIEVE
B2 @ 20'	61.3
B2 @ 25'	57.6
B2 @ 30'	58.5
B2 @ 35'	54.2
B2 @ 40'	52.4
B2 @ 45'	41.9
B2 @ 50'	49.0

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GRAIN SIZE ANALYSIS

ROSE HILL COURTS REDEVELOPMENT

4446 FLORIZEL STREET
LOS ANGELES, CALIFORNIA

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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				
B3 @ 0-5'	10.3	23.0	106.5	37	Low	Expansive
B9 @ 0-5'	12.0	28.5	103.1	69	Moderate	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 (%)
B3 @ 0-5'	Light Brown Silty Sand	119.5	12.0
B9 @ 0-5'	Brown Sandy Silt	120.0	11.5
B13 @ 5-10'	Yellowish Brown Silty Sand	121.0	11.0
B14 @ 0-5'	Dark Brown Silt	123.0	11.0

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LABORATORY TEST RESULTS

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

Sample No.	pH	Resistivity (ohm centimeters)
B3 @ 0-5'	7.63	1100 (Corrosive)
B9 @ 0-5'	7.63	1200 (Corrosive)
B13 @ 5-10'	7.84	680 (Severely Corrosive)
B14 @ 0-5'	7.82	960 (Severely Corrosive)

SUMMARY OF LABORATORY CHLORIDE CONTENT TEST RESULTS EPA NO. 325.3

Sample No.	Chloride Ion Content (%)
B3 @ 0-5'	0.011
B9 @ 0-5'	0.004
B13 @ 5-10'	0.023
B14 @ 0-5'	0.004

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

Sample No.	Water Soluble Sulfate (% SO_4)	Sulfate Exposure*
B3 @ 0-5'	0.030	Negligible
B9 @ 0-5'	0.002	Negligible
B13 @ 5-10'	0.164	Moderate
B15 @ 0-5'	0.001	Negligible

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

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CORROSIVITY TEST RESULTS

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MAY 2018 | PROJECT NO. A9514-06-01 | FIG. B13