APPENDIX C

GEOTECHNICAL REPORT

Project No. W1249-88-01
Revised June 28, 2023

Red Oak Investments, LLC
4199 Campus Drive, Suite 200
Irvine, CA 92612

Attention: Mr. Alex Wong

Subject: UPDATE OF GEOTECHNICAL INVESTIGATION
PROPOSED RESIDENTIAL DEVELOPMENT
6821 FAIRLYNN BOULEVARD
YORBA LINDA, CALIFORNIA

Reference: *Preliminary Geotechnical Investigation,* by Geocon West, Inc., November 20, 2020;
Percolation Test Results, by Geocon West, Inc., dated March 31, 2021.

Dear Mr. Wong:

This letter has been prepared to update the referenced geotechnical investigation report prepared by Geocon West, Inc. Based on our understanding of the project, the scope remains essentially unchanged. The geotechnical recommendations presented in the referenced report remains applicable to the project as presently proposed. The recommendations presented herein are intended to update the referenced report for compliance with current building code requirements. The soils report follows the requirements of the 2022 California Building Code (CBC), which meets or exceeds the requirements of the 2022 California Residential Code (CRC). Where differing, the recommendations herein supersede the previous recommendations.

Seismic Design Criteria

The following table summarizes the site-specific design criteria obtained from the 2022 California Building Code (CBC; Based on the 2021 International Building Code [IBC] and ASCE 7-16), Chapter 16 Structural Design, Section 1613 Earthquake Loads. The data was calculated using the online application *U.S. Seismic Design Maps*, provided by the Structural Engineers Association of California (SEAOC). The short spectral response uses a period of 0.2 second. We evaluated the Site Class based on the discussion in Section 1613.2.2 of the 2022 CBC and Table 20.3-1 of ASCE 7-16. The values presented on the following page are for the risk-targeted maximum considered earthquake (MCE_R).

2022 CBC SEISMIC DESIGN PARAMETERS

Parameter	Value	2022 CBC Reference
Site Class	D	Section 1613.2.2
MCE_R Ground Motion Spectral Response Acceleration – Class B (short), S_S	1.74g	Figure 1613.2.1(1)
MCE_R Ground Motion Spectral Response Acceleration – Class B (1 sec), S_1	0.614g	Figure 1613.2.1(3)
Site Coefficient, F_A	1	Table 1613.2.3(1)
Site Coefficient, F_V	1.7	Table 1613.2.3(2)
Site Class Modified MCE_R Spectral Response Acceleration (short), S_{MS}	1.74g	Section 1613.2.3 (Eqn 16-20)
Site Class Modified MCE_R Spectral Response Acceleration – (1 sec), S_{M1}	1.043g*	Section 1613.2.3 (Eqn 16-21)
5% Damped Design Spectral Response Acceleration (short), S_{DS}	1.16g	Section 1613.2.4 (Eqn 16-22)
5% Damped Design Spectral Response Acceleration (1 sec), S_{D1}	0.696g*	Section 1613.2.4 (Eqn 16-23)

*Per Supplement 3 of ASCE 7-16, a ground motion hazard analysis (GMHA) shall be performed for projects on Site Class “D” sites with 1-second spectral acceleration (S_1) greater than or equal to 0.2g, which is true for this site. However, Supplement 3 of ASCE 7-16 provides an exception stating that that the GMHA may be waived provided that the parameter S_{M1} is increased by 50% for all applications of S_{M1}. The values for parameters S_{M1} and S_{D1} presented above have **not** been increased in accordance with Supplement 3 of ASCE 7-16.

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

ASCE 7-16 PEAK GROUND ACCELERATION

Parameter	Value	ASCE 7-16 Reference
Mapped MCE_G Peak Ground Acceleration, PGA	0.737g	Figure 22-9
Site Coefficient, F_{PGA}	1.1	Table 11.8-1
Site Class Modified MCE_G Peak Ground Acceleration, PGA_M	0. 811g	Section 11.8.3 (Eqn 11.8-1)

Deaggregation of the MCE peak ground acceleration was performed using the USGS online Unified Hazard Tool, 2014 Conterminous U.S. Dynamic edition (v4.2.0). The result of the deaggregation analysis indicates that the mean earthquake contributing to the MCE peak ground acceleration is characterized as a 6.71 magnitude event occurring at a hypocentral distance of 9.5 kilometers from the site.

Deaggregation was also performed for the Design Earthquake (DE) peak ground acceleration, corresponding to two-thirds of the MCE peak ground acceleration. The result of the analysis indicates that the mean earthquake contributing to the DE peak ground acceleration is characterized as a 6.62 magnitude occurring at a hypocentral distance of 14.55 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.

Plan Review

Grading, foundation, and shoring plans (if required) 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.

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

Very truly yours,

GEOCON WEST, INC.

Petrina Zen
GE 3217

(Email) Addressee

PRELIMINARY GEOTECHNICAL INVESTIGATION

PROPOSED RESIDENTIAL DEVELOPMENT
6821 FAIRLYNN BOULEVARD, YORBA LINDA, CALIFORNIA

GEOCON
WEST, INC.

GEOTECHNICAL
ENVIRONMENTAL
MATERIALS

PREPARED FOR
RED OAK INVESTMENTS, LLC
IRVINE, CALIFORNIA

PROJECT NO. W1249-88-01

NOVEMBER 20, 2020

GEOCON
WEST, INC.
GEOTECHNICAL ■ ENVIRONMENTAL ■ MATERIALS

Project No. W1249-88-01
November 20, 2020

Red Oak Investments, LLC
4199 Campus Drive, Suite 200
Irvine, CA 92612

Attention: Mr. Joseph Flanagan

Subject: PRELIMINARY GEOTECHNICAL INVESTIGATION
PROPOSED RESIDENTIAL DEVELOPMENT
6821 FAIRLYNN BOULEVARD
YORBA LINDA, CALIFORNIA

Dear Mr. Flanagan:

In accordance with your authorization of our proposal dated October 8, 2020, we have prepared this preliminary geotechnical investigation report for the proposed residential development located at 6821 Fairlynn Boulevard in the City of Yorba Linda, California. The accompanying report presents the findings of our study and our conclusions and recommendations pertaining to the geotechnical aspects of proposed design and construction. Based on the results of our investigation, it is our opinion that the site can be developed as proposed, provided the recommendations of this report are followed and implemented during design and construction.

The primary intent of this study was to address potential geologic hazards and geotechnical conditions that could impact the project. A design level geotechnical study will be required once a conceptual site plan is available in order to provide updated geotechnical recommendations for design and construction.

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

Very truly yours,

GEOCON WEST, INC.

Petrina Zen	Jelisa Thomas Adams	Susan F. Kirkgard
PE 87489	GE 3092	CEG 1754

(EMAIL) Addressee

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PRELIMINARY GEOTECHNICAL INVESTIGATION

1. PURPOSE AND SCOPE

This report presents the results of a preliminary geotechnical investigation for the proposed residential development located at 6821 Fairlynn Boulevard in the City of Yorba Linda, 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. A design level geotechnical study will be required once a conceptual site plan is available in order to provide updated geotechnical recommendations for design and construction.

The scope of this investigation included a site reconnaissance, field exploration, laboratory testing, engineering analysis, and the preparation of this report. The site was explored on October 29, 2020 by excavating six 8-inch diameter borings using a truck-mounted hollow-stem auger drilling machine. The borings were excavated to depths between 10½ and 24 feet below the existing ground surface. The approximate locations of the exploratory borings are depicted on the Site Plan (see Figure 2). A detailed discussion of the field investigation, including logs of the borings, is presented in Appendix A.

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

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

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

2. SITE AND PROJECT DESCRIPTION

The subject site is located at 6821 Fairlynn Boulevard in the City of Yorba Linda, California. The site is currently occupied by a commercial shopping center with single-story on-grade structures and associated parking lots. The site is bounded by Fairgreen Avenue to north, by Fairlynn Boulevard to the east, by Esperanza Road to the south, and by residential housing to the west. A graded 2:1 (H:V) slope, ranging from two to ten feet in height, is present along the northern and western property boundaries. Overall, the site slopes gently to the south and surface water drainage at the site appears to flow to the city streets. Onsite vegetation consists of lawn, shrubs and trees in planters throughout the site.

Based on the information provided by the Client, it is our understanding that the proposed development will consist of 3-story townhome structures constructed at or near present grade. Due to the preliminary nature of the project, formal plans depicting the proposed development are not available for inclusion in this report. The existing site conditions are depicted on the Site Plan (see Figure 2).

Based on the preliminary nature of the design at this time, wall and column loads were not available. It is anticipated that column loads for the proposed structures will be up to 200 kips, and wall loads will be up to 3 kips per linear foot.

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

3. GEOLOGIC SETTING

The site is situated in the northeastern portion of the Orange County. Locally, the site is located along the southern flank of the Chino Hills, approximately 0.3 mile north of the Santa Ana River channel. Regionally, the site is located within the northeastern portion of the Peninsular Ranges geomorphic province. This geomorphic province is characterized by northwest-trending physiographic and geologic features such as the nearby Whittier and Chino faults located approximately 2.7 miles northeast and 8 miles northeast, respectively.

4. SOIL AND GEOLOGIC CONDITIONS

Based on our field investigation and published geologic maps of the area, the site is underlain by artificial fill and Pleistocene age alluvium. Detailed stratigraphic profiles of the materials encountered at the site are provided on the boring logs in Appendix A.

4.1 Artificial Fill

Artificial fill was encountered in our explorations to a maximum depth of 1½ feet below existing ground surface. The artificial fill generally consists of light brown to brown silty sand and sandy silt. The artificial fill is characterized as dry and loose or firm. 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

Late Pleistocene age older alluvial deposits were encountered beneath the fill. The alluvium generally consists of light brown to dark brown or reddish brown sandy silt, silty sand, and poorly graded sand with varying amounts of gravel. The alluvial soils are characterized as dry to wet and firm to hard or medium dense to very dense.

5. GROUNDWATER

Based on a review of the Seismic Hazard Zone Report for the Orange 7.5 Minute Quadrangle, Orange County, California (California Division of Mines and Geology [CDMG], 1997, revised 2001), the historically highest groundwater level in the area is approximately 20 feet beneath the existing ground surface. Groundwater information presented in this document is generated from data collected in the early 1900's to the late 1990s. Based on current groundwater basin management practices, it is unlikely that groundwater levels will ever exceed the historic high levels.

Groundwater was encountered in boring B3 at a depth of 21 feet beneath the existing ground surface. Considering the depth to groundwater in boring B3, the reported historic high groundwater level (CDMG, 2001), and the depth of the proposed construction, it is unlikely that groundwater will be encountered during the proposed construction or have an impact on the project. However, it is not uncommon for groundwater levels to vary seasonally or for groundwater seepage conditions to develop where none previously existed, especially in impermeable fine-grained soils which are heavily irrigated or after seasonal rainfall. In addition, recent requirements for stormwater infiltration could result in shallower seepage conditions in the immediate site vicinity. Proper surface drainage of irrigation and precipitation will be critical for future performance of the project. Recommendations for drainage are provided in the S*urface Drainage* section of this report (see Section 7.16).

6. GEOLOGIC HAZARDS

6.1 Surface Fault Rupture

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

The site is not located within a state-designated Alquist-Priolo Earthquake Fault Zone for surface fault rupture hazards (CGS, 2020a; CGS, 2020b). No Holocene-active or pre-Holocene faults with the potential for surface fault rupture are known to pass directly beneath the site. Therefore, the potential for surface rupture due to faulting occurring beneath the site during the design life of the proposed development is considered low. However, the site is located in the seismically active Southern California region and could be subjected to moderate to strong ground shaking in the event of an earthquake on one of the many active Southern California faults. The faults in the vicinity of the site are shown in Figure 3, Regional Fault Map.

The closest surface trace of an active fault to the site is the Whittier Fault located approximately 2.7 miles to the northeast (USGS, 2006; Ziony and Jones, 1989). Other nearby active faults include the Chino Fault, the Elsinore Fault Zone, the Newport-Inglewood Fault Zone, the Duarte Fault, and the Cucamonga Fault located approximately 8 miles northeast, 8.5 miles east, 17.5 miles southwest, 19.5 miles north-northwest, and 20 miles north-northeast of the site, respectively (USGS, 2006; Ziony and Jones, 1989). The active San Andreas Fault Zone is located approximately 33 miles northeast of the site.

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

6.2 Seismicity

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

LIST OF HISTORIC EARTHQUAKES

Earthquake (Oldest to Youngest)	Date of Earthquake	Magnitude	Distance to Epicenter (Miles)	Direction to Epicenter
Near Redlands	July 23, 1923	6.3	32	ENE
Long Beach	March 10, 1933	6.4	20	SW
Tehachapi	July 21, 1952	7.5	105	NW
San Fernando	February 9, 1971	6.6	52	NW
Whittier Narrows	October 1, 1987	5.9	22	NW
Sierra Madre	June 28, 1991	5.8	30	NNW
Landers	June 28, 1992	7.3	81	ENE
Big Bear	June 28, 1992	6.4	60	ENE
Northridge	January 17, 1994	6.7	49	WNW
Hector Mine	October 16, 1999	7.1	100	ENE
Ridgecrest	July 5, 2019	7.1	132	N

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 the site-specific design criteria obtained from the 2019 California Building Code (CBC; Based on the 2018 International Building Code [IBC] and ASCE 7-16), Chapter 16 Structural Design, Section 1613 Earthquake Loads. The data was calculated using the online application *Seismic Design Maps*, provided by OSHPD. The short spectral response uses a period of 0.2 second. We evaluated the Site Class based on the discussion in Section 1613.2.2 of the 2019 CBC and Table 20.3-1 of ASCE 7-16. The values presented below are for the risk-targeted maximum considered earthquake (MCE_R).

2019 CBC SEISMIC DESIGN PARAMETERS

Parameter	Value	2019 CBC Reference
Site Class	D	Section 1613.2.2
MCE_R Ground Motion Spectral Response Acceleration – Class B (short), S_S	1.74g	Figure 1613.2.1(1)
MCE_R Ground Motion Spectral Response Acceleration – Class B (1 sec), S_1	0.614g	Figure 1613.2.1(2)
Site Coefficient, F_A	1	Table 1613.2.3(1)
Site Coefficient, F_V	1.7*	Table 1613.2.3(2)
Site Class Modified MCE_R Spectral Response Acceleration (short), S_{MS}	1.74g	Section 1613.2.3 (Eqn 16-36)
Site Class Modified MCE_R Spectral Response Acceleration – (1 sec), S_{M1}	1.043g*	Section 1613.2.3 (Eqn 16-37)
5% Damped Design Spectral Response Acceleration (short), S_{DS}	1.16g	Section 1613.2.4 (Eqn 16-38)
5% Damped Design Spectral Response Acceleration (1 sec), S_{D1}	0.696g*	Section 1613.2.4 (Eqn 16-39)

Note:

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

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

ASCE 7-16 PEAK GROUND ACCELERATION

Parameter	Value	ASCE 7-16 Reference
Mapped MCE_G Peak Ground Acceleration, PGA	0.737g	Figure 22-7
Site Coefficient, F_{PGA}	1.1	Table 11.8-1
Site Class Modified MCE_G Peak Ground Acceleration, PGA_M	0.811g	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 2019 California Building Code and ASCE 7-16, the MCE is to be utilized for the evaluation of liquefaction, lateral spreading, seismic settlements, and it is our understanding that the intent of the Building code is to maintain “Life Safety” during a MCE event. The Design Earthquake Ground Motion (DE) is the level of ground motion that has a 10 percent chance of exceedance in 50 years, with a statistical return period of 475 years.

Deaggregation of the MCE peak ground acceleration was performed using the USGS online Unified Hazard Tool, 2014 Conterminous U.S. Dynamic edition (v4.2.0). The result of the deaggregation analysis indicates that the predominant earthquake contributing to the MCE peak ground acceleration is characterized as a 6.71 magnitude event occurring at a hypocentral distance of 9.5 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 14.55 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 Orange Quadrangle (CDMG, 1998) indicates that the site is not located within an area designated as having a potential for liquefaction. Also, the City of Yorba Linda General Plan (2016) indicates that the site is not located within an area with a potential for liquefaction. The site is underlain by Pleistocene age alluvial deposits that are hard or dense at a depth of approximately 4 to 10 feet beneath the existing ground surface. Based on these considerations, it is our opinion that the potential for liquefaction to occur beneath the site is considered low.

6.5 Slope Stability

The topography at the site and in the site vicinity slopes gently to the south. According to the City of Yorba Linda General Plan (2016), the site is not located within an area with a potential for slope instability. Additionally, the site is not located within an area identified as having of potential for seismic slope instability (CDMG, 1998). There are no known landslides near the site, nor is the site in the path of any known or potential landslides. Therefore, the potential for slope stability hazards to adversely affect the proposed development is considered low.

6.6 Earthquake-Induced Flooding

Earthquake-induced flooding is inundation caused by failure of dams or other water-retaining structures due to earthquakes. Review of the City of Yorba Linda General Plan (2016) indicates that the site is not located within a dam inundation area. Therefore, the potential for inundation at the site to occur as a result of an earthquake-induced dam failure is considered low.

6.7 Tsunamis, Seiches, and Flooding

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

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

The site is within an area of minimal flooding (Zone X) as defined by the Federal Emergency Management Agency (FEMA, 2020).

6.8 Oil Fields & Methane Potential

Based on a review of the California Geologic Energy Management Division (CalGEM) Well Finder Website, the site is not located within an oil field and active oil or gas wells are not documented in the immediate site vicinity (CalGEM, 2020). However, due to the voluntary nature of record reporting by the oil well drilling companies, wells may be improperly located or not shown on the location map and undocumented wells could be encountered during construction. Any wells encountered during construction will need to be properly abandoned in accordance with the current requirements of the CalGEM.

Since the site is not located within the boundaries of a known oil field, the potential for the presence of methane or other volatile gases at the site is considered low. However, should it be determined that a methane study is required for the proposed development it is recommended that a qualified methane consultant be retained to perform the study and provide mitigation measures as necessary.

6.9 Subsidence

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

7. CONCLUSIONS AND RECOMMENDATIONS

7.1 General

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

7.1.2 Up to 1½ feet of existing artificial fill was encountered during the site investigation. The existing fill encountered is believed to be the result of past grading and construction activities at the site. Deeper fill may exist in other areas of the site that were not directly explored. Future demolition of the existing structures which occupy the site will likely disturb the upper few feet of soil. 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 Based on these considerations, it is recommended that the upper five feet of existing earth materials within 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). The excavation should extend laterally a minimum distance of three 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).

7.1.4 Subsequent to the recommended grading, the proposed structures may be supported on conventional shallow spread foundation systems or post-tensioned foundation systems deriving support in newly placed engineered fill. Recommendations for the design of a conventional foundation system and a post-tensioned foundation system are provided in Sections 7.6 and 7.7.

7.1.5 All excavations must be observed and approved in writing by the Geotechnical Engineer (a representative of Geocon). Prior to placing any fill, the upper 12 inches of the excavation bottom must be scarified, moistened, and proof-rolled with heavy equipment in the presence of the Geotechnical Engineer (a representative of Geocon West, Inc.).

7.1.6 It is anticipated that stable excavations for the recommended grading associated with the proposed structure can be achieved with sloping measures. However, if excavations in close proximity to an adjacent property line and/or structure are required, special excavation measures may be necessary in order to maintain lateral support of offsite improvements. Excavation recommendations are provided in the *Temporary Excavations* section of this report (Section 7.15).

7.1.7 For the purposes of this preliminary report, it is assumed that proposed site grading will not include the existing slopes along the northern and western property lines. As the project progresses, we should review the project plans and provide additional recommendations regarding these property lines, if needed.

7.1.8 Foundations for small outlying structures, such as block walls up to 6 feet in height, planter walls or trash enclosures, which will not be tied to the proposed structure, may be supported on conventional foundations deriving support on a minimum of 12 inches of newly placed engineered fill which extends laterally at least 12 inches beyond the foundation area. Where excavation and compaction cannot be performed or is undesirable, foundations may derive support directly in the competent undisturbed alluvial soils at or below a depth of 18 inches, and should be deepened as necessary to maintain a minimum 12 inch embedment into the recommended bearing materials. If the soils exposed in the excavation bottom are soft or loose, compaction of the soils will be required prior to placing steel or concrete. Compaction of the foundation excavation bottom is typically accomplished with a compaction wheel or mechanical whacker and must be observed and approved by a Geocon representative.

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

7.1.10 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.

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

7.2 Soil and Excavation Characteristics

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

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 existing adjacent 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 existing site soils encountered during this investigation are considered to have a “low” expansive potential (EI = 24); and are classified as “expansive” based on the 2019 California Building Code (CBC) Section 1803.5.3. Recommendations presented herein assume that the proposed 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” with respect to corrosion of buried ferrous metals on site. The results are presented in Appendix B (Figure B15) and should be considered for design of underground structures. Due to the corrosive potential of the soils, it is recommended that corrosion-resistant ABS pipes (or equivalent) be utilized in lieu of cast-iron for subdrains and retaining wall drains beneath the structure.

7.3.2 Laboratory tests were performed on representative samples of the site materials to measure the percentage of water-soluble sulfate content. Results from the laboratory water-soluble sulfate tests are presented in Appendix B (Figure B15) and indicate that the on-site materials possess a sulfate exposure class of “S0” to concrete structures as defined by 2019 CBC Section 1904 and ACI 318-14 Table 19.3.1.1.

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

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

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

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 deeper artificial fill or soft alluvial soil at the direction of the Geotechnical Engineer (a representative of Geocon). The excavation should extend laterally a minimum distance of 3 feet beyond the building footprint area, 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 alluvial soils removal will be verified by the Geocon representative during site grading activities.

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 upper 12 inches of the excavation bottom must be scarified, moistened, and proof-rolled with heavy equipment in the presence of the Geotechnical Engineer (a representative of Geocon West, Inc.).

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

7.4.7. Where new paving is to be placed, it is recommended that all existing fill and soft alluvium be excavated and properly compacted for paving support. As a minimum, the upper 12 inches of soil should be scarified, moisture conditioned to optimum moisture content, and compacted to at least 95 percent relative compaction, as determined by ASTM Test Method D 1557 (latest edition). Paving recommendations are provided in *Preliminary Pavement Recommendations* section of this report (see Section 7.12).

7.4.8 It is anticipated that stable excavations for the recommended grading can be achieved with sloping measures. However, if excavations in close proximity to an adjacent property line and/or structure are required, special excavation measures may be necessary in order to maintain lateral support of the existing offsite improvements. Excavation recommendations are provided in the *Temporary Excavations* section of this report (Section 7.15).

7.4.9 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 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 proper compaction cannot be performed or is undesirable, foundations may derive support directly in the undisturbed alluvial soils found at or below a depth of 18 inches, and should be deepened as necessary to maintain a minimum 12 inch embedment into the recommended bearing materials. If the soils exposed in the excavation bottom are soft or loose, compaction of the soils will be required prior to placing steel or concrete. Compaction of the foundation excavation bottom is typically accomplished with a compaction wheel or mechanical whacker and must be observed and approved by a Geocon representative.

7.4.10 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 as backfill is also acceptable. Prior to placing any bedding materials or pipes, the excavation bottom must be observed and approved in writing by the Geotechnical Engineer (a representative of Geocon).

7.4.11 All imported fill shall be observed, tested, and approved by Geocon West, Inc. prior to bringing soil to the site. Rocks larger than 6 inches in diameter shall not be used in the fill. If necessary, import soils used as structural fill should have an expansion index less than 20 and corrosivity properties that are equally or less detrimental to that of the existing onsite soils (see Figure B15). 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).

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

7.5 Shrinkage

7.5.1 Shrinkage results when a volume of material removed at one density is compacted to a higher density. A shrinkage factor of between 5 and 10 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 the building pad, the soils must be placed uniformly and at equal thickness at the direction of the Geotechnical Engineer (a representative of Geocon West, Inc.). Soils can be borrowed from non-building pad areas and later replaced with imported soils.

7.6 Conventional Foundation Design

7.6.1 Subsequent to the recommended grading, a conventional shallow spread foundation system may be utilized for support of the proposed structures provided foundations derive support in newly placed engineered fill.

7.6.2 Continuous footings may be designed for an allowable bearing capacity of 2,250 pounds per square foot (psf), and should be a minimum of 12 inches in width, 18 inches in depth below the lowest adjacent grade, and 12 inches into the recommended bearing material.

7.6.3 Isolated spread foundations may be designed for an allowable bearing capacity of 2,500 psf, and should be a minimum of 24 inches in width, 18 inches in depth below the lowest adjacent grade, and 12 inches into the recommended bearing material.

7.6.4 The allowable soil bearing pressure above may be increased by 250 psf and 500 psf for each additional foot of foundation width and depth, respectively, up to a maximum allowable soil bearing pressure of 4,000 psf.

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

7.6.6 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.

7.6.7 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.

7.6.8 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.

7.6.9 No special subgrade presaturation is required prior to placement of concrete. However, the slab and foundation subgrade should be sprinkled as necessary; to maintain a moist condition as would be expected in any concrete placement.

7.6.10 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.

7.6.11 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.

7.7 Post-Tensioned Foundation Recommendations

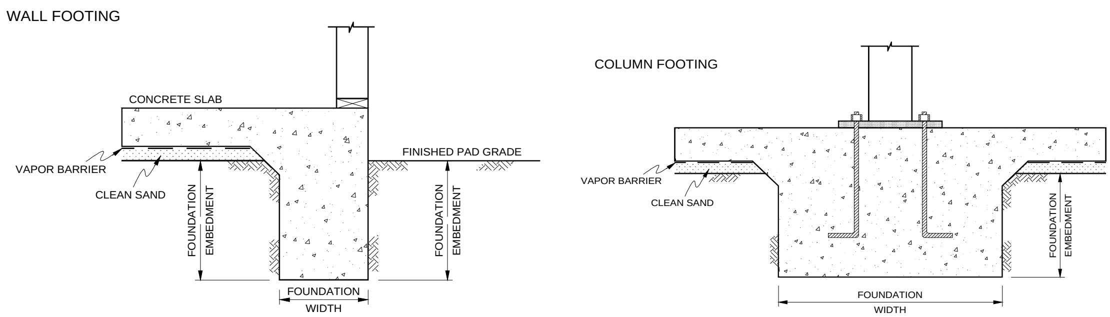
7.7.1 As an alternative, the proposed structures may be supported on a post-tensioned foundation system. Proposed post-tensioned foundations should derive support exclusively in newly placed engineered fill.

7.7.2 The post-tensioned system should be designed by a structural engineer experienced in post-tensioned slab design and design criteria of the Post-Tensioning Institute (PTI) DC 10.5-12 *Standard Requirements for Design and Analysis of Shallow Post-Tensioned Concrete Foundations on Expansive Soils* or *WRI/CRSI Design of Slab-on-Ground Foundations*, as required by the 2019 California Building Code (CBC Section 1808.6.2). Although this procedure was developed for expansive soil conditions, we understand it can also be used to reduce the potential for foundation distress due to differential settlement. The post-tensioned design should incorporate the geotechnical parameters presented in the following table, which are based on the guidelines presented in the PTI, Third Edition design manual.

POST-TENSIONED FOUNDATION SYSTEM DESIGN PARAMETERS

Post-Tensioning Institute (PTI) DC 10.5-12 Design Parameters	Value
Thornthwaite Index	-20
Equilibrium Suction	3.9
Edge Lift Moisture Variation Distance, e_M (Feet)	5.3
Edge Lift, y_M (Inches)	0.42
Center Lift Moisture Variation Distance, e_M (Feet)	9.0
Center Lift, y_M (Inches)	0.18

7.7.3 The foundations for the post-tensioned slabs should be embedded in accordance with the recommendations of the structural engineer. For a post-tensioned mat foundation system, the foundation should possess a thickened edge with a minimum width of 12 inches and extend below the clean sand or crushed rock layer. A graphic depicting the foundation embedment is provided below.



7.7.4 If the structural engineer proposes a post-tensioned foundation design method other than PTI DC 10.5:

- The criteria presented in the above table are still applicable.
- Interior stiffener beams should be used.
- The width of the perimeter foundations should be at least 12 inches.
- The perimeter footing embedment depths should be at least 12 inches. The embedment depths should be measured from the lowest adjacent pad grade.

7.7.5 Our experience indicates post-tensioned slabs may be susceptible to excessive edge lift, regardless of the underlying soil conditions. Placing reinforcing steel at the bottom of the perimeter footings and the interior stiffener beams may mitigate this potential. The structural engineer should design the foundation system to reduce the potential of edge lift occurring for the proposed structures.

7.7.6 During the construction of the post-tension foundation system, the concrete should be placed monolithically. Under no circumstances should cold joints form between the footings/grade beams and the slab during the construction of the post-tension foundation system unless specifically designed by the structural engineer.

7.7.7 The post-tensioned mat foundations may be designed for an allowable soil bearing pressure of 4.000 pounds per square foot (psf) for the proposed structures. This bearing pressure may be increased by one-third for transient loads due to wind or seismic forces.

7.7.8 It is recommended that a modulus of subgrade reaction of 150 pounds per cubic inch (pci) be utilized for the design of a post-tensioned mat foundation bearing in newly placed engineered fill. This value is a unit value for use with a 1-foot square footing. The modulus should be reduced in accordance with the following equation when used with larger foundations:

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

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

7.7.9 Isolated footings, if present, should have a minimum embedment depth and width of 24 inches. The use of isolated footings, which are located beyond the perimeter of the building and support structural elements connected to the building, are not recommended. If this condition cannot be avoided, the isolated footings should be connected to the building foundation system with grade beams. In addition, consideration should be given to connecting patio slabs, which exceed 5 feet in width, to the building foundation to reduce the potential for future separation to occur.

7.7.10 Special subgrade presaturation is not deemed necessary prior to placing concrete; however, the exposed foundation and slab subgrade soil should be moisture conditioned, as necessary, to maintain a moist condition as would be expected in any such concrete placement.

7.7.11 The recommendations of this report are intended to reduce the potential for cracking of slabs and foundations due to expansive soil (if present), differential settlement of fill soil with varying thicknesses. However, even with the incorporation of the recommendations presented herein, foundations, stucco walls, and slabs-on-grade placed on such conditions may still exhibit some cracking due to soil movement and/or shrinkage. The occurrence of concrete shrinkage cracks is independent of the supporting soil characteristics. Their occurrence may be reduced 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.

7.7.12 Foundation excavations should be observed and approved in writing by the Geotechnical Engineer (a representative of Geocon) prior to the placement of reinforcing steel and concrete to check that the exposed soil conditions are consistent with those expected and have been extended to appropriate bearing strata. If unexpected soil conditions are encountered, foundation modifications may be required.

7.7.13 Slabs that may receive moisture-sensitive floor coverings or may be used to store moisture-sensitive materials should be underlain by a vapor retarder. The vapor retarder design should be consistent with the guidelines presented in the American Concrete Institute's (ACI) *Guide for Concrete Slabs that Receive Moisture-Sensitive Flooring Materials* (ACI 302.2R-06). The vapor retarder used should be specified by the project architect or developer based on the type of floor covering that will be installed and if the structure will possess a humidity controlled environment.

7.7.14 The bedding sand thickness should be determined by the project foundation engineer, architect, and/or developer. However, we should be contacted to provide recommendations if the bedding sand is thicker than 6 inches. It is common to see 3 inches and 4 inches of sand below the concrete slab-on-grade for 5- and 4-inch thick slabs, respectively, in the southern California area. The foundation design engineer should provide appropriate concrete mix design criteria and curing measures to assure proper curing of the slab by reducing the potential for rapid moisture loss and subsequent cracking and/or slab curl. We suggest that the foundation design engineer present the concrete mix design and proper curing methods on the foundation plans. It is critical that the foundation contractor understands and follows the recommendations presented on the foundation plans.

7.7.15 Geocon should be consulted to provide additional design parameters as required by the structural engineer.

7.8 Foundation Settlement

7.8.1 The maximum expected static settlement for a structure supported on a conventional foundation system or a post-tensioned foundation deriving support in the recommended bearing materials and designed with a maximum bearing pressure of 4,000 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.

7.8.2 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.

7.9 Miscellaneous Foundations

7.9.1 Foundations for small outlying structures, such as block walls up to 6 feet in height, planter walls or trash enclosures which will not be tied to the proposed structure may be supported on conventional foundations bearing on a minimum of 12 inches of newly placed engineered fill which extends laterally at least 12 inches beyond the foundation area. Where excavation and compaction cannot be performed or is undesirable, such as adjacent to property lines, foundations may derive support in the undisturbed alluvial soils at or below a depth of 18 inches, and should be deepened as necessary to maintain a minimum 12 inch embedment into the recommended bearing materials.

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

7.9.3 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.

7.10 Lateral Design

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

7.10.2 Passive earth pressure for the sides of foundations and slabs poured against properly compacted engineered fill or competent alluvial soils may be computed as an equivalent fluid having a density of 230 pcf with a maximum earth pressure of 2,300 psf. When combining passive and friction for lateral resistance, the passive component should be reduced by one-third.

7.11 Concrete Slabs-on-Grade

7.11.1 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).

7.11.2 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.

7.11.3 Slabs-on-grade 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.

7.11.4 For seismic design purposes, a coefficient of friction of 0.35 may be utilized between concrete slabs and subgrade soils without a moisture barrier, and 0.15 for slabs underlain by a moisture barrier.

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

7.11.6 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.

7.12 Preliminary Pavement Recommendations

7.12.1 Where new paving is to be placed, it is recommended that all existing fill and soft or unsuitable alluvial materials be excavated and properly recompacted 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 unsuitable material may experience increased settlement and/or cracking, and may therefore have a shorter design life and increased maintenance costs. As a minimum, the upper twelve inches of paving subgrade should be scarified, moisture conditioned to optimum moisture content, and properly compacted to at least 95 percent relative compaction, as determined by ASTM Test Method D 1557 (latest edition).

7.12.2 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.

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

PRELIMINARY PAVEMENT DESIGN SECTIONS

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

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

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

7.12.6 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.

7.13 Retaining Wall Design

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

7.13.2 Retaining wall foundations may be designed in accordance with the recommendations provided in the *Conventional Foundation Design* section of this report (see Section 7.6).

7.13.3 Retaining walls with a level backfill surface that are not restrained at the top should be designed utilizing a triangular distribution of pressure (active pressure) of 30 pcf.

7.13.4 Restrained walls are those that are not allowed to rotate more than 0.001H (where H equals the height of the retaining portion of the wall in feet) at the top of the wall. Where walls are restrained from movement at the top, walls may be designed utilizing a triangular distribution of pressure (at-rest pressure) of 70 pcf.

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

7.13.6 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.

7.13.7 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. Once the design becomes more finalized, an addendum letter can be prepared revising recommendations and addressing specific surcharge conditions throughout the project, if necessary.

7.14 Retaining Wall Drainage

7.14.1 If not designed for hydrostatic pressure, 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 5). 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.

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

7.14.3 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.

7.14.4 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.

7.15 Temporary Excavations

7.15.1 Excavations on the order of 5 feet in height may be required during grading operations. The excavations are expected to expose artificial fill and alluvial soils, which are suitable for vertical excavations up to 5 feet in height where loose soils or caving sands are not present, and where not surcharged by adjacent traffic or structures.

7.15.2 Vertical excavations greater than 5 feet or where surcharged by existing structures will require sloping or shoring measures in order to provide a stable excavation. Where sufficient space is available, temporary unsurcharged embankments could be sloped back at a uniform 1:1 slope gradient or flatter up to maximum height of 9 feet. A uniform slope does not have a vertical portion.

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

7.16 Surface Drainage

7.16.1 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.

7.16.2 All site drainage should be collected and controlled in non-erosive drainage devices. Drainage should not be allowed to pond anywhere on the site, and especially not against any foundation or retaining wall. The site should be graded and maintained such that surface drainage is directed away from structures in accordance with 2019 CBC 1804.4 or other applicable standards. In addition, drainage should not be allowed to flow uncontrolled over any descending slope. Discharge from downspouts, roof drains and scuppers are not recommended onto unprotected soils within 5 feet of the building perimeter. Planters which are located adjacent to foundations should be sealed to prevent moisture intrusion into the soils providing foundation support. Landscape irrigation is not recommended within 5 feet of the building perimeter footings except when enclosed in protected planters.

7.16.3 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.

7.16.4 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.

7.17 Plan Review

7.17.1 Grading 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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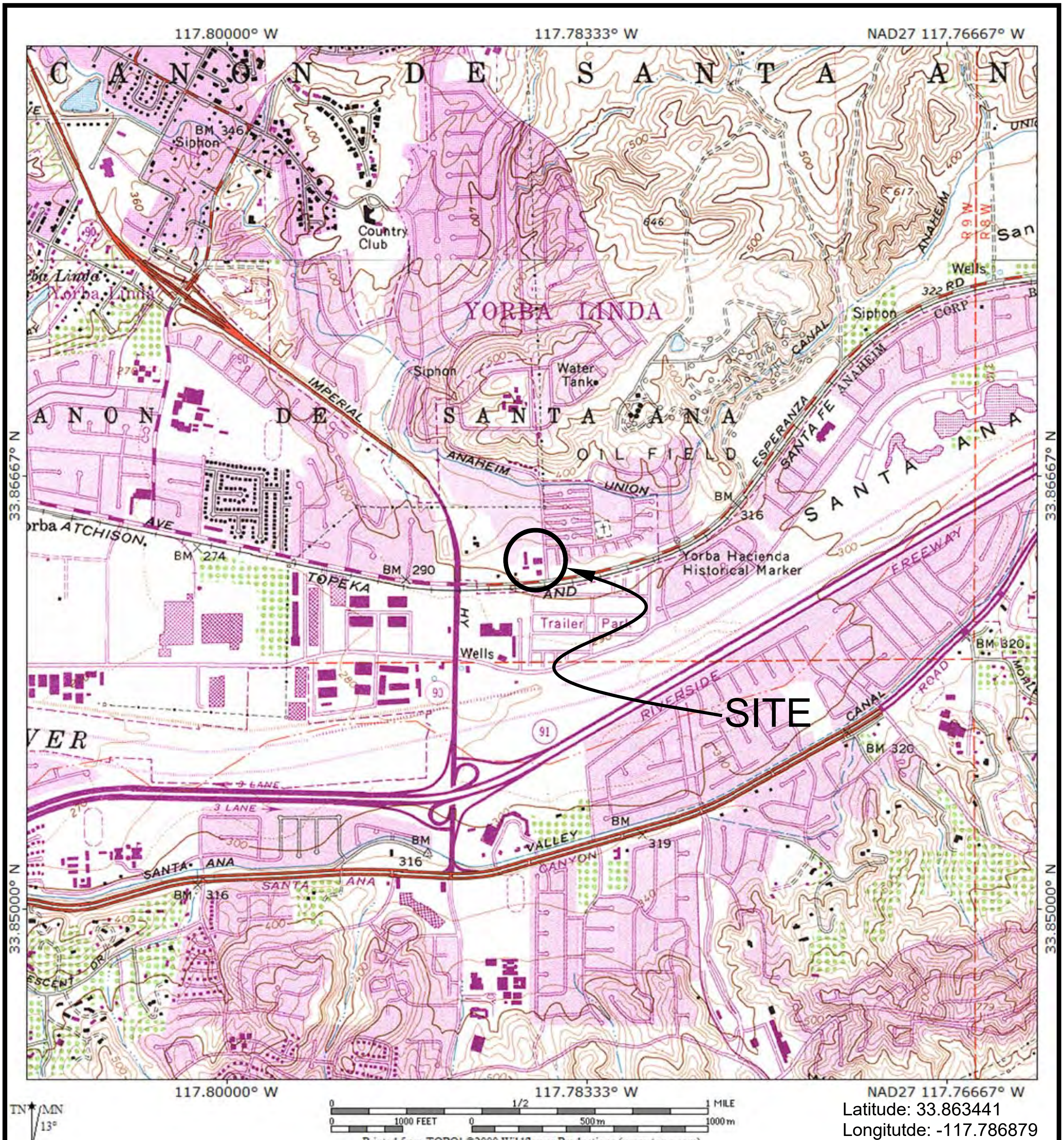

REFERENCE:U.S.G.S. TOPOGRAPHIC MAPS, 7.5 MINUTE SERIES, ORANGE, CA QUADRANGLE

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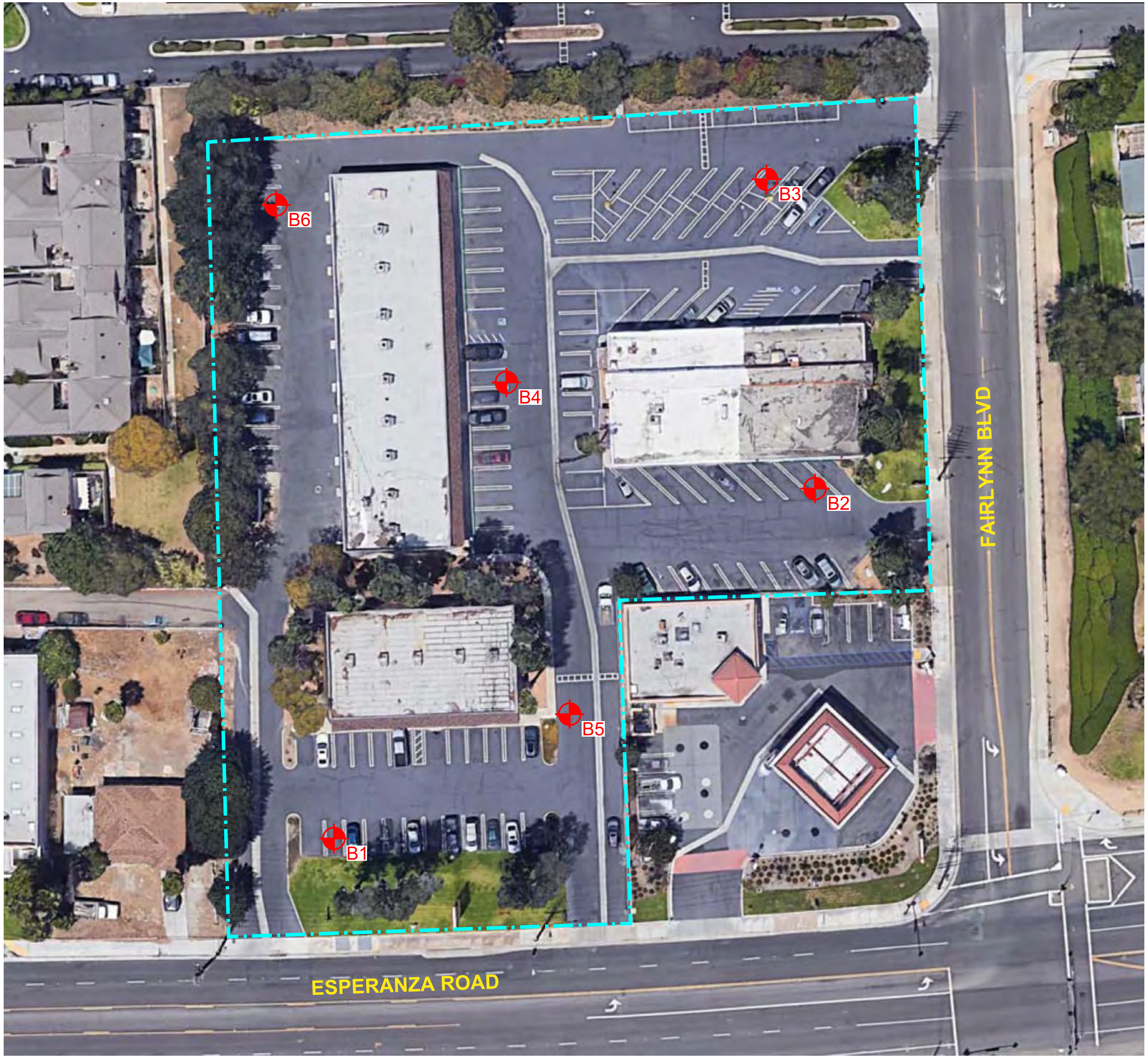
VICINITY MAP

6821 FAIRLYNN BOULEVARD
YORBA LINDA, CALIFORNIA

DRAFTED BY: RA	CHECKED BY: SFK	NOV. 2020	PROJECT NO. W1249-88-01	FIG. 1



LEGEND

B6 Approximate Location of Boring

Approximate Location of Property Line

0 50' 100'

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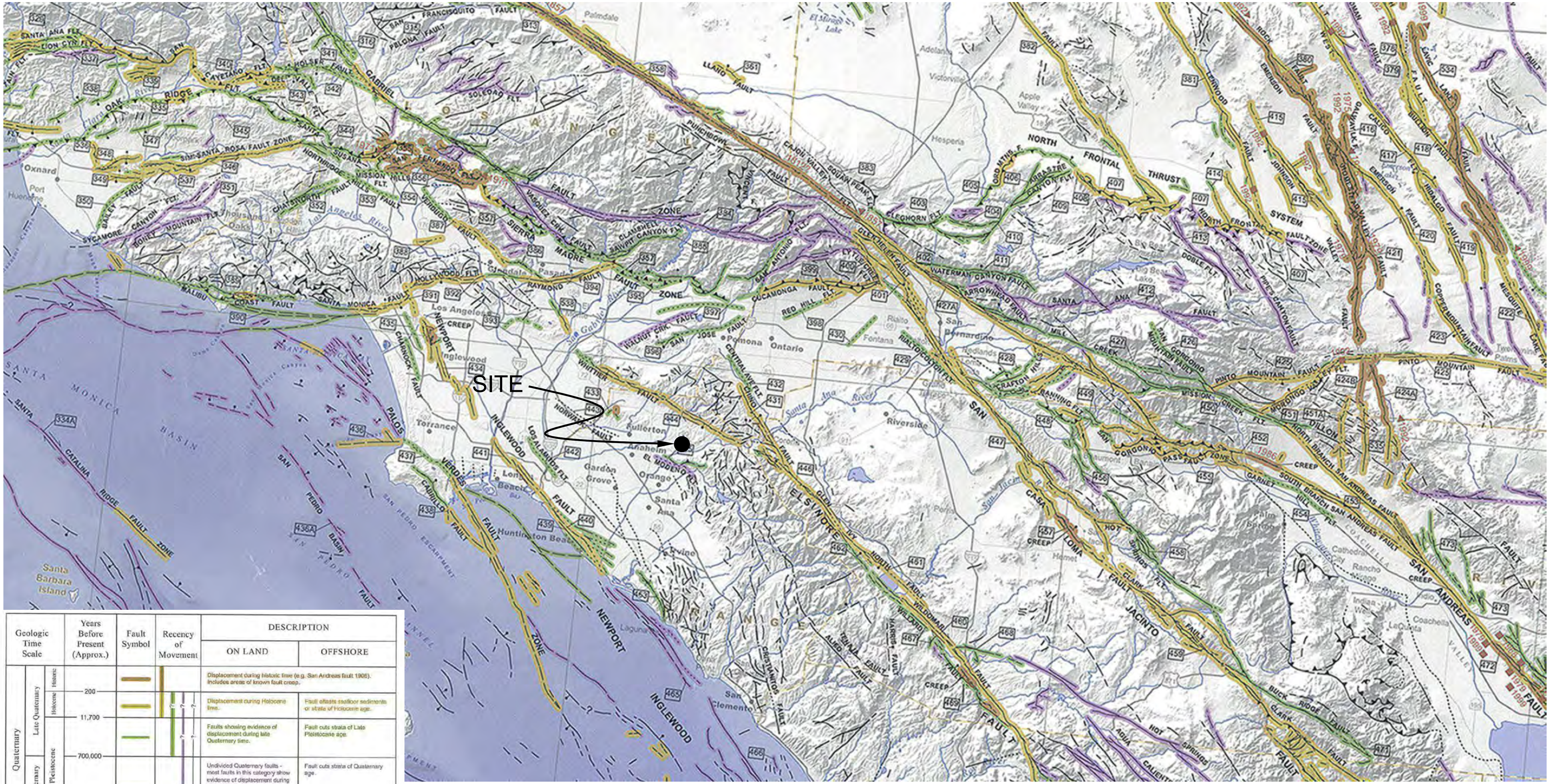
DRAFTED BY: PZ | CHECKED BY: JTA

SITE PLAN

6821 FAIRLYNN BOULEVARD
YORBA LINDA, CALIFORNIA

NOV. 2020 | PROJECT NO. W1249-88-01 | FIG. 2

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



Geologic Time Scale			Years Before Present (Approx.)	Fault Symbol	Recency of Movement	DESCRIPTION: ON LAND	DESCRIPTION: OFFSHORE
Quaternary	Late Quaternary	Historic				Displacement during historic time (e.g. San Andreas fault 1906). Includes areas of known fault creep.	
		Holocene	200			Displacement during Holocene time.	Fault offsets seafloor sediments or strata of Holocene age.
		Pleistocene	11,700			Faults showing evidence of displacement during late Quaternary time.	Fault cuts strata of Late Pleistocene age.
	Early Quaternary		700,000			Undivided Quaternary faults - most faults in this category show evidence of displacement during the last 1,600,000 years; possible exceptions are faults which displace rocks of undifferentiated Plio-Pleistocene age.	Fault cuts strata of Quaternary age.
Pre-Quaternary			1,600,000* – 4.5 billion (Age of Earth)			Faults without recognized Quaternary displacement or showing evidence of no displacement during Quaternary time. Not necessarily inactive.	Fault cuts strata of Pliocene or older age.

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

Latitude: 33.863441
Longitude: -117.786879

0 12 24 Miles

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REGIONAL FAULT MAP

6821 FAIRLYNN BOULEVARD

YORBA LINDA, CALIFORNIA

DRAFTED BY: RA	CHECKED BY: SFK	NOV. 2020	PROJECT NO. W1249-88-01	FIG. 3

Latitude: 33.863441
Longitude: -117.786879

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

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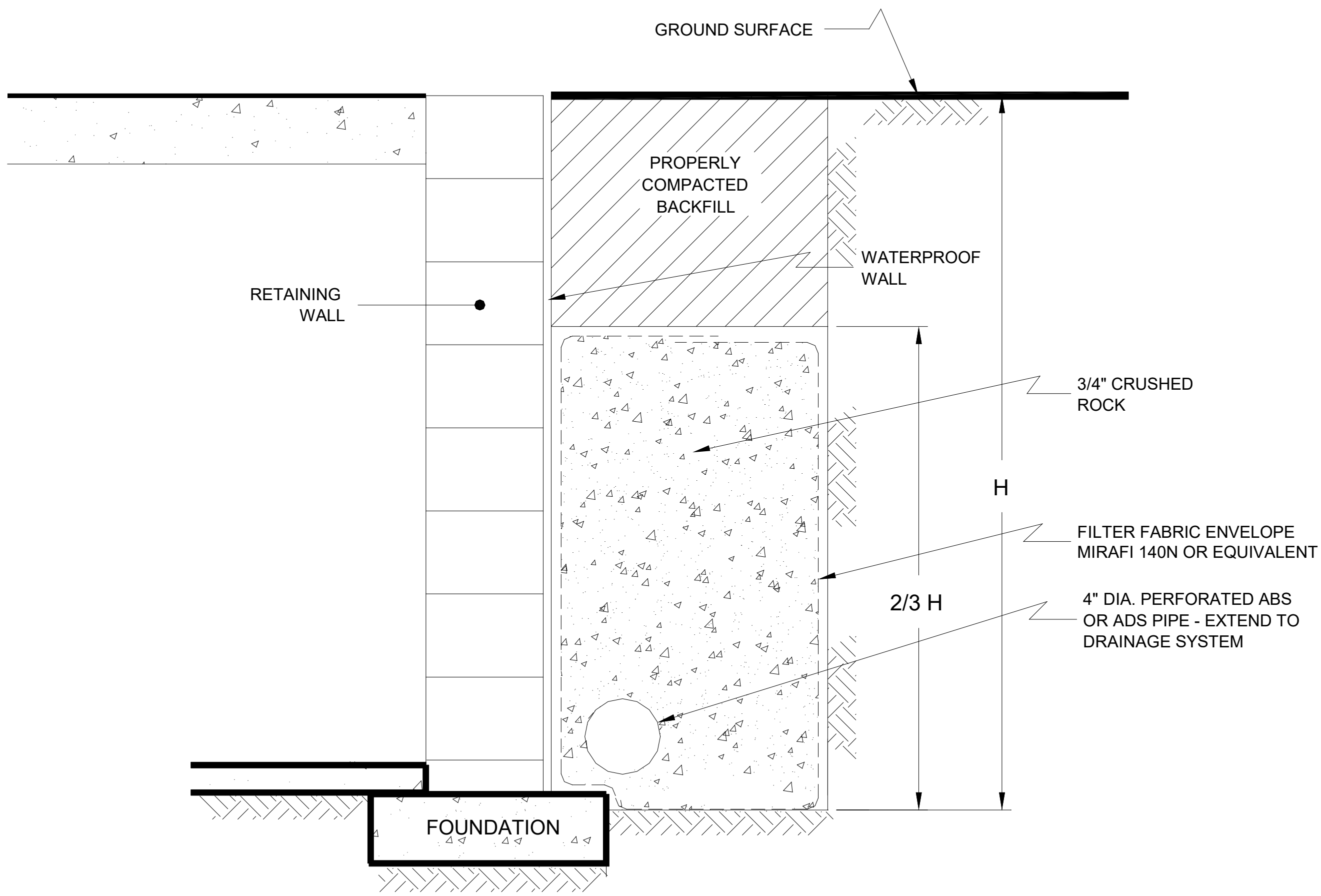
REGIONAL SEISMICITY MAP

6821 FAIRLYNN BOULEVARD
YORBA LINDA, CALIFORNIA

NOV. 2020 | PROJECT NO. W1249-88-01 | FIG. 4



NO SCALE

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RETAINING WALL DRAIN DETAIL

6821 FAIRLYNN BOULEVARD
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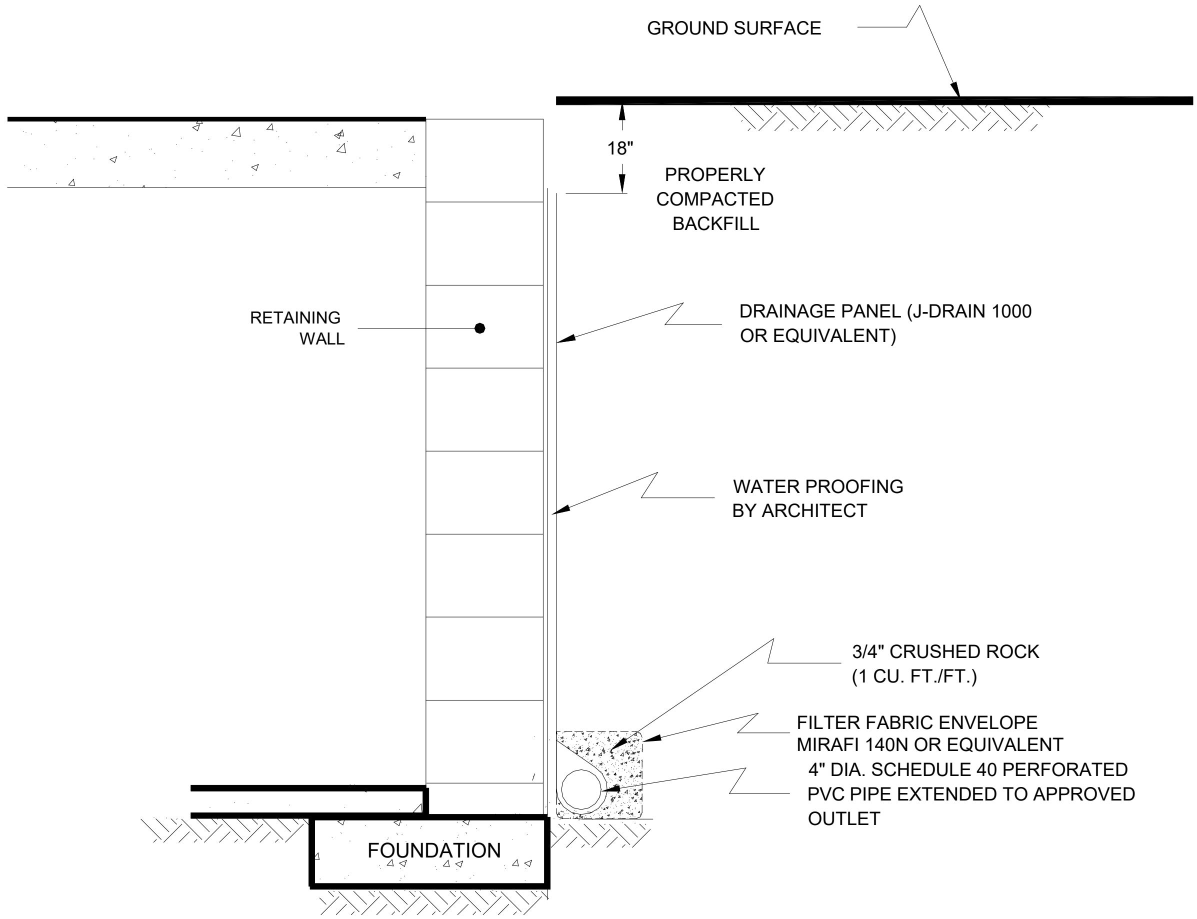
DRAFTED BY: PZ	CHECKED BY: JTA	NOV. 2020	PROJECT NO. W1249-88-01	FIG. 5



NO SCALE

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RETAINING WALL DRAIN DETAIL

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YORBA LINDA, CALIFORNIA

DRAFTED BY: PZ	CHECKED BY: JTA	NOV. 2020	PROJECT NO. W1249-88-01	FIG. 6

APPENDIX

A

APPENDIX A

FIELD INVESTIGATION

The site was explored on October 29, 2020 by excavating six 8-inch diameter borings using a truck-mounted hollow-stem auger drilling machine. The borings were excavated to depths between 10½ and 24 feet below the existing ground surface. Representative and relatively undisturbed samples were obtained by driving a 3-inch, O. D., California Modified Sampler into the "undisturbed" soil mass with blows from a 140-pound auto-hammer falling 30 inches. The California Modified Sampler was equipped with 1-inch high by $2^3/_8$-inch diameter brass sampler rings to facilitate soil removal and testing. Bulk samples were also collected.

The soil conditions encountered in the borings were visually examined, classified and logged in general accordance with the Unified Soil Classification System (USCS). Logs of the borings are presented on Figures A1 through A6. The logs depict the soil and geologic conditions encountered and the depth at which samples were obtained. The logs also include our interpretation of the conditions between sampling intervals. Therefore, the logs contain both observed and interpreted data. We determined the lines designating the interface between soil materials on the logs using visual observations, penetration rates, excavation characteristics and other factors. The transition between materials may be abrupt or gradual. Where applicable, the logs were revised based on subsequent laboratory testing. The locations of the borings are shown on Figure 2.

BORING 1

ELEV. (MSL.) -- DATE COMPLETED 10/29/2020

EQUIPMENT HOLLOW STEM AUGER BY: RA

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0	BULK 0-5'				**AC: 3" BASE: 1"**			
					ARTIFICIAL FILL Sandy Clay, firm, dry, light brown, trace fine to medium gravel.			
2					**ALLUVIUM** Silty Sand, medium dense, slightly moist, fine-grained, brown to reddish brown, trace fine gravel, trace clay.			
	B1@3'			SM		25	121.5	6.5
4								
6	B1@6'					47	122.7	13.9
					Sandy Silt, hard, moist, dark brown, trace coarse-grained.			
8								
	B1@9'			ML		50 (6")	113.5	11.2
10								
12	B1@12'			SM	Silty Sand, very dense, slightly moist, brown, fine- to medium-grained, trace gravel.	50 (4")	127.4	5.7
14								
	B1@15'				Sand, poorly graded, very dense, dry, brown, fine- to medium-grained, some coarse-grained, fine gravel.	50 (5")	110.5	3.1
16								
18				SP				
20	B1@20'				- no recovery	50 (4")		
22								
24								

Total depth of boring: 24 feet (refusal)
Fill to 1 foot.
No groundwater encountered.
Backfilled with soil cuttings and tamped.

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

W1249-88-01 BORING LOGS.GPJ

Figure A1,
Log of Boring 1, 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 2

ELEV. (MSL.) -- DATE COMPLETED 10/29/2020

EQUIPMENT HOLLOW STEM AUGER BY: RA

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0					**AC: 4" BASE: 1.5"**			
					ARTIFICIAL FILL Silty Sand, loose, dry, brown, trace fine gravel.			
2					**ALLUVIUM** Sandy Silt, moist, firm, reddish brown, trace coarse-grained and fine to medium gravel.			
	B2@3'					14	122.5	8.8
4								
6	B2@6'				- stiff	35	117.3	15.0
				ML				
8								
	B2@9'				- hard, decrease in medium-grained	46	120.7	13.9
10								
12	B2@12'				- stiff, decrease in sand	30	123.3	12.0
14								
	B2@15'					38	123.4	9.3

Total depth of boring: 15.5 feet
Fill to 1 foot.
No groundwater encountered.
Backfilled with soil cuttings and tamped.
AC patched.

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

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

W1249-88-01 BORING LOGS.GPJ

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

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

BORING 3

ELEV. (MSL.) -- DATE COMPLETED 10/29/2020

EQUIPMENT HOLLOW STEM AUGER BY: RA

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0	BULK 0-5'				**AC: 4" BASE: 1.5"**			
					ARTIFICIAL FILL Silty Sand, loose, dry, brown, trace fine to medium gravel.			
2					**ALLUVIUM** Sandy Clay, stiff, moist, reddish brown to brown, fine-grained.			
	B3@3'					42	125.2	9.3
4								
6	B3@6'			CL	- hard, trace coarse-grained	50 (5")	123.8	11.6
8								
	B3@9'				- increase in fine- to medium-grained, trace coarse-grained	68	129.5	10.1
10								
12	B3@12'					65	126.5	12.6
14					Sand, poorly graded, medium dense, slightly moist, brown, fine- to medium-grained, trace coarse-grained and silt.			
	B3@15'					37	123.4	5.0
16								
18				SP				
20	B3@20'		▼		- dense, wet, no recovery	50 (5")		
22								
24	B3@23.5'					50 (5")	94.7	20.2

Total depth of boring: 24 feet
Fill to 1 foot.
Groundwater encountered at 21 feet.
Backfilled with soil cuttings and tamped.
AC patched.

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

W1249-88-01 BORING LOGS.GPJ

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

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

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

PROJECT NO. W1249-88-01

BORING 4

ELEV. (MSL.) -- DATE COMPLETED 10/29/2020

EQUIPMENT HOLLOW STEM AUGER BY: RA

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: 3" BASE: 9"**			
					ARTIFICIAL FILL Silty Sand, loose, dry, brown, trace fine gravel.			
2					**ALLUVIUM**			
	B4@2.5'				Sandy Clay, stiff, slightly moist, reddish brown, fine-grained, some fine gravel.	31	126.2	10.7
4								
	B4@5'			CL	- hard, dark brown, trace coarse-grained, decrease in sand	33	123.5	9.0
6								
	B4@7.5'				- hard	58	124.2	13.0
8								
10	B4@10'				- increase in sand	82	123.6	11.8

Total depth of boring: 10.5 feet
Fill to 1.5 feet.
No groundwater encountered.
Backfilled with soil cuttings and tamped.
AC patched.

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

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

W1249-88-01 BORING LOGS.GPJ

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

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

PROJECT NO. W1249-88-01

BORING 5

ELEV. (MSL.) -- DATE COMPLETED 10/29/2020

EQUIPMENT HOLLOW STEM AUGER BY: RA

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: 2.5" BASE: 8"**			
					ARTIFICIAL FILL Silty Sand, loose, dry, trace fine gravel, light brown.			
2	B5@2.5'			SM	**ALLUVIUM** Silty Sand, medium dense, dry, brown, fine- to coarse-grained, trace fine gravel.	41	118.9	8.0
4	B5@5'			ML	Sandy Silt, hard, slightly moist, reddish brown, trace coarse-grained.	60	124.6	8.0
6	B5@7.5'			SM	Silty Sand, very dense, dry to slightly moist, reddish brown, fine- to medium-grained, trace coarse-grained.	50 (5")	117.0	12.0
8								
10	B5@10'					50 (4")	115.3	9.4

Total depth of boring: 10.5 feet
Fill to 1.5 feet.
No groundwater encountered.
Backfilled with soil cuttings and tamped.
AC patched.

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

W1249-88-01 BORING LOGS.GPJ

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

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

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

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BORING 6

ELEV. (MSL.) -- DATE COMPLETED 10/29/2020

EQUIPMENT HOLLOW STEM AUGER BY: RA

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: 2.5" BASE: 8"** **ARTIFICIAL FILL** Silty Sand, loose, dry, light brown, trace gravel.			
2					**ALLUVIUM** Sandy Silt, hard, dry to slightly moist, reddish brown, fine-grained, trace coarse-grained.			
	B6@3'					52	111.2	6.6
4				ML				
6	B6@6'				- dry, fine- to coarse-grained, increase in sand	50 (4")	103.2	7.4
8								
	B6@9'					50 (4")	118.6	7.4
10				SM	Silty Sand, very dense, slightly moist, reddish brown, fine- to medium-grained, trace coarse-grained.			
12	B6@12'					50 (6")	105.2	8.1
14				ML	Sandy Silt, hard, slightly moist, light brown, fine-grained.			
16	B6@15'			SP	Sand, poorly graded, very dense, dry, fine- to medium-grained, trace coarse-grained and fine gravel.	50 (4")	121.3	3.1

Total depth of boring: 16.5 feet
Fill to 1.5 feet.
No groundwater encountered.
Backfilled with soil cuttings and tamped.
AC patched.

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

W1249-88-01 BORING LOGS.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.

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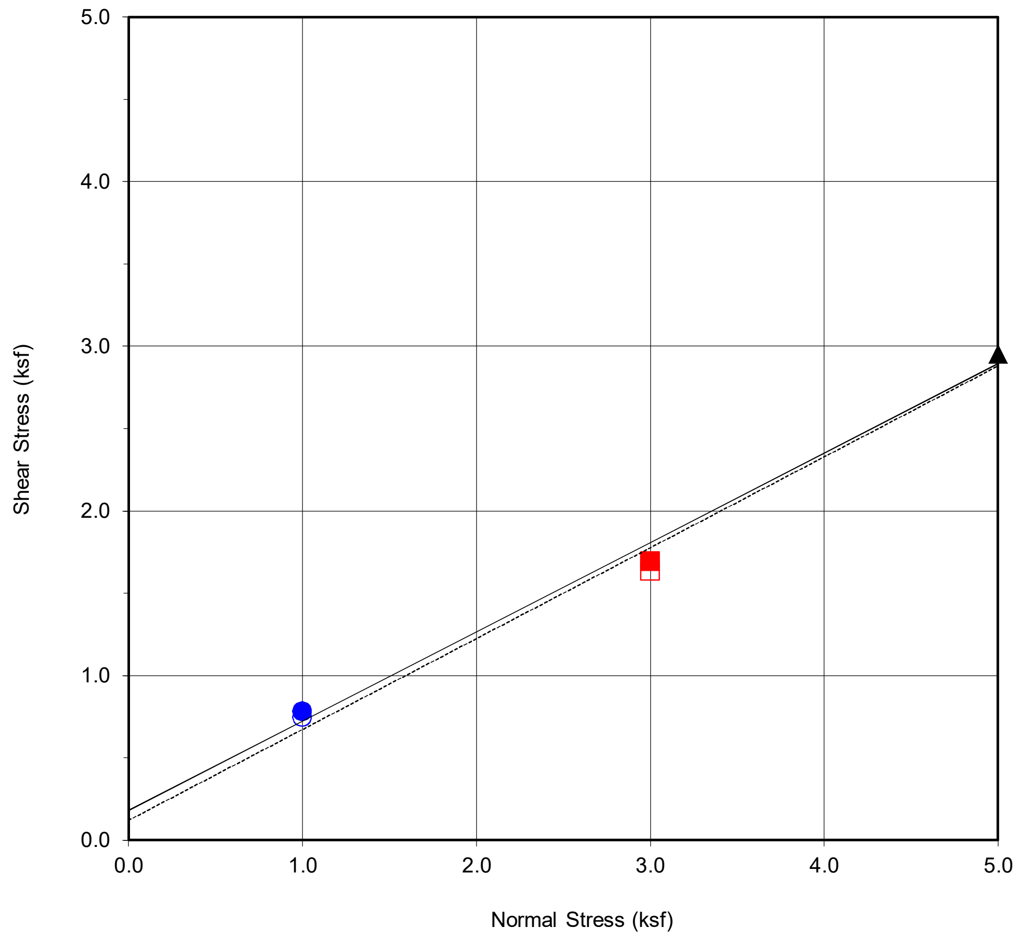
APPENDIX B

APPENDIX B

LABORATORY TESTING

Laboratory tests were performed in accordance with generally accepted test methods of the International ASTM, or other suggested procedures. Selected samples were tested for direct shear strength, consolidation characteristics, plasticity indices, corrosivity, in-place dry density and moisture content. The results of the laboratory tests are summarized in Figures B1 through B16. The in-place dry density and moisture content of the samples tested are presented on the boring logs, Appendix A.



Boring No.	**B1&B3**
Sample No.	**B1&B3@0-5'**
Depth (ft)	**0-5'**
Sample Type:	Ring

Normal Stress (kip/ft2)	1	3	5
Peak Shear Stress (kip/ft²)	● 0.78	■ 1.69	▲ 2.95
Shear Stress @ End of Test (ksf)	○ 0.74	□ 1.63	△ 2.95
Deformation Rate (in./min.)	0.00	0.00	0.00
Initial Sample Height (in.)	1.0	1.0	1.0
Ring Inside Diameter (in.)	2.375	2.375	2.375
Initial Moisture Content (%)	10.3	9.9	9.7
Initial Dry Density (pcf)	116.9	116.7	116.3
Initial Degree of Saturation (%)	62.7	60.2	58.0
Soil Height Before Shearing (in.)	1.2	1.2	1.2
Final Moisture Content (%)	16.3	16.0	17.5

Soil Identification:

Brown Silty Sand (SM)

Strength Parameters

	C (psf)	ϕ (°)
Peak	179	28.5
Ultimate	120	28.9

GEOCON

DIRECT SHEAR TEST RESULTS

Consolidated Drained ASTM D-3080

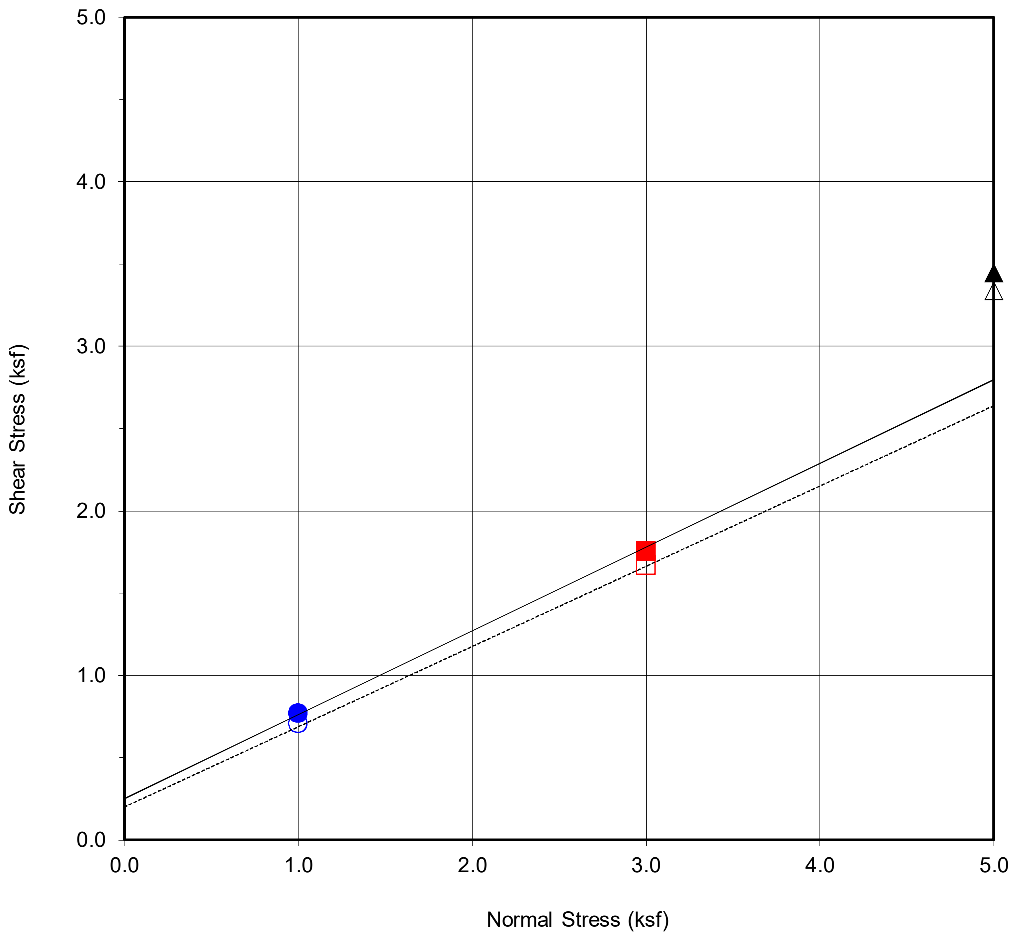
Checked by: PZ

Project No.: W1249-88-01

6821 Fairlynn Boulevard
Yorba Linda, California

Nov. 2020

Figure B1



Boring No.	B5
Sample No.	B5@2.5'
Depth (ft)	2.5'
Sample Type:	Ring

Soil Identification:		
Brown Silty Sand (SM)		
Strength Parameters		
	C (psf)	ϕ (°)
Peak	250	27.0
Ultimate	200	26.0

Normal Stress (kip/ft2)	1	3	5
Peak Shear Stress (kip/ft²)	● 0.77	■ 1.75	▲ 3.44
Shear Stress @ End of Test (ksf)	○ 0.71	□ 1.67	△ 3.34
Deformation Rate (in./min.)	0.00	0.00	0.00
Initial Sample Height (in.)	1.0	1.0	1.0
Ring Inside Diameter (in.)	2.375	2.375	2.375
Initial Moisture Content (%)	14.0	12.7	11.3
Initial Dry Density (pcf)	112.3	115.4	115.8
Initial Degree of Saturation (%)	75.6	74.2	67.3
Soil Height Before Shearing (in.)	1.2	1.2	1.2
Final Moisture Content (%)	19.2	18.0	15.8

GEOCON

DIRECT SHEAR TEST RESULTS

Consolidated Drained ASTM D-3080

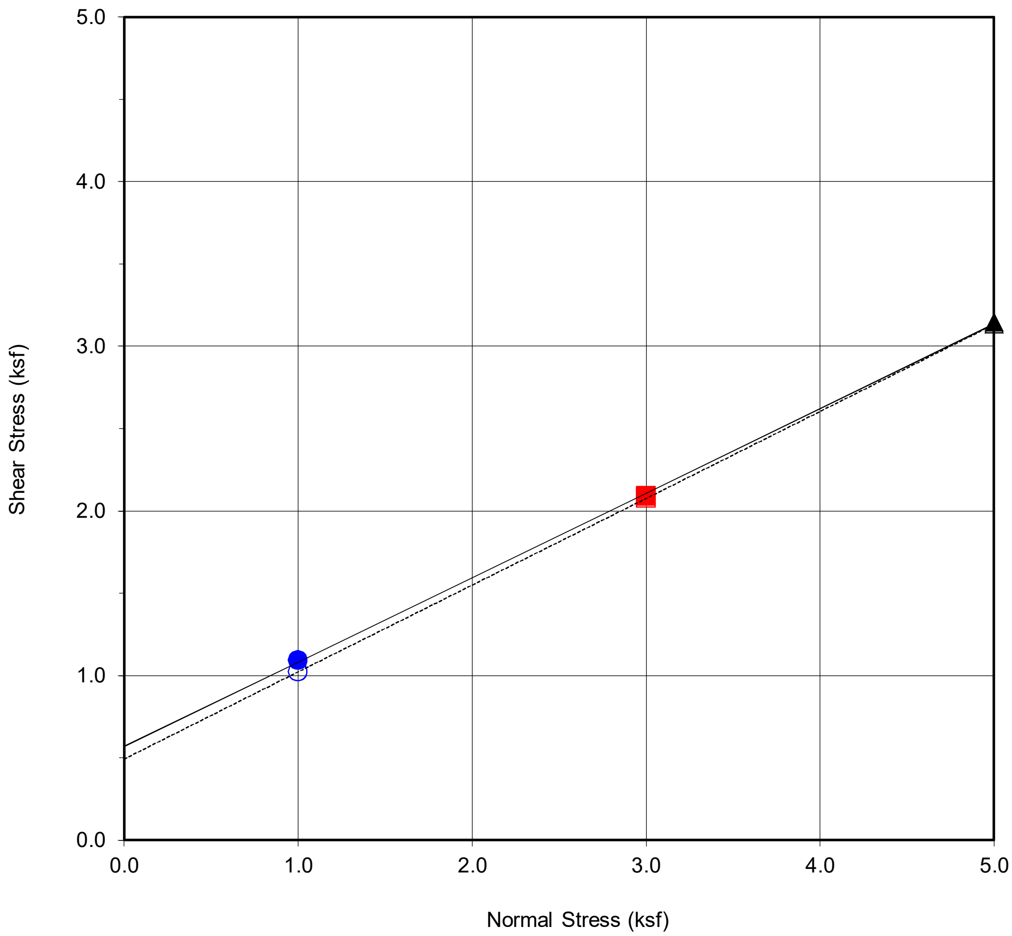
Checked by: PZ

Project No.: W1249-88-01

6821 Fairlynn Boulevard
Yorba Linda, California

Nov. 2020

Figure B2



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

Soil Identification:		
Reddish Brown Sandy Silt (ML)		
Strength Parameters		
	C (psf)	ϕ (°)
Peak	569	27.2
Ultimate	492	27.8

Normal Stress (kip/ft2)	1	3	5
Peak Shear Stress (kip/ft²)	● 1.09	■ 2.09	▲ 3.14
Shear Stress @ End of Test (ksf)	○ 1.02	□ 2.08	△ 3.13
Deformation Rate (in./min.)	0.00	0.00	0.00
Initial Sample Height (in.)	1.0	1.0	1.0
Ring Inside Diameter (in.)	2.375	2.375	2.375
Initial Moisture Content (%)	8.1	9.9	12.5
Initial Dry Density (pcf)	123.3	118.9	118.0
Initial Degree of Saturation (%)	59.7	64.2	79.0
Soil Height Before Shearing (in.)	1.2	1.2	1.2
Final Moisture Content (%)	13.2	14.6	14.5

GEOCON

DIRECT SHEAR TEST RESULTS

Consolidated Drained ASTM D-3080

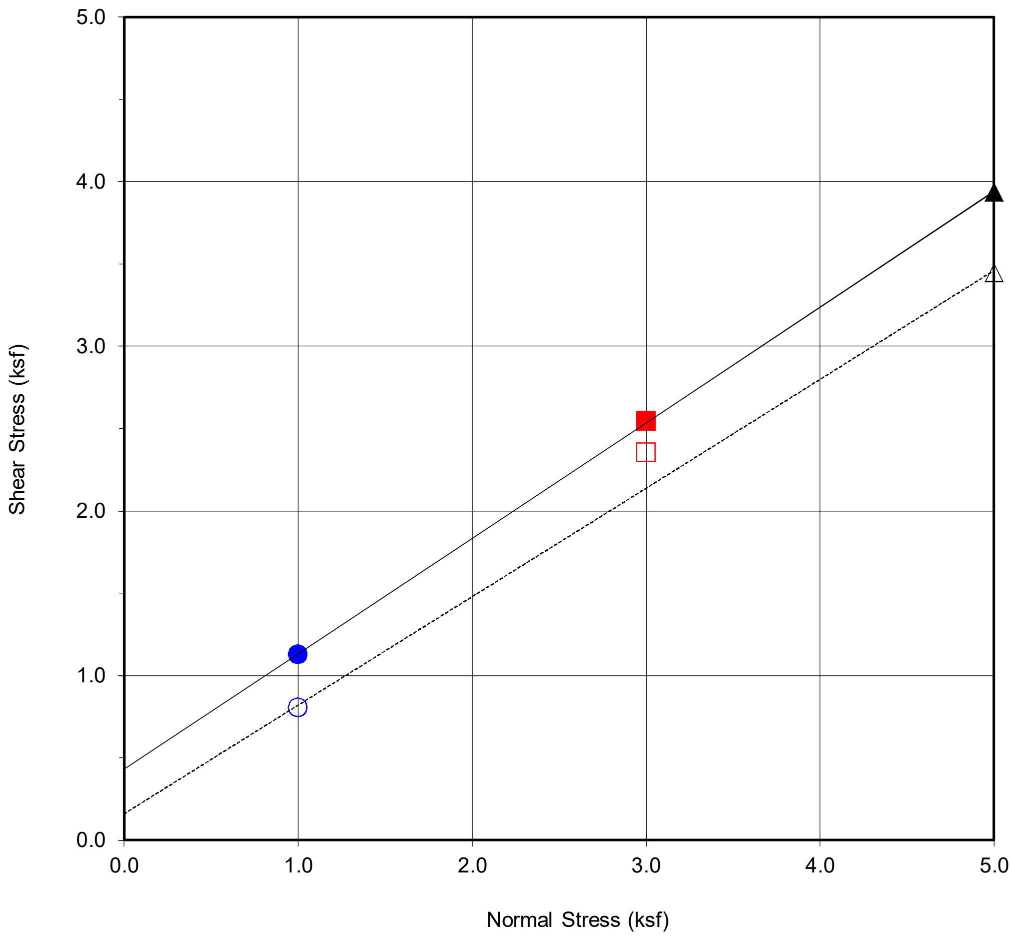
Checked by: PZ

Project No.: W1249-88-01

6821 Fairlynn Boulevard
Yorba Linda, California

Nov. 2020

Figure B3



Boring No.	**B-4**
Sample No.	**B4@5'**
Depth (ft)	**5'**
Sample Type:	Ring

Soil Identification:		
Dark Brown Sandy Clay (CL)		
Strength Parameters		
	C (psf)	ϕ (°)
Peak	430	35.1
Ultimate	160	33.4

Normal Stress (kip/ft2)	1	3	5
Peak Shear Stress (kip/ft²)	● 1.13	■ 2.54	▲ 3.94
Shear Stress @ End of Test (ksf)	○ 0.80	□ 2.35	△ 3.44
Deformation Rate (in./min.)	0.00	0.00	0.00
Initial Sample Height (in.)	1.0	1.0	1.0
Ring Inside Diameter (in.)	2.375	2.375	2.375
Initial Moisture Content (%)	11.3	11.6	12.3
Initial Dry Density (pcf)	120.2	122.3	122.4
Initial Degree of Saturation (%)	76.2	83.0	88.3
Soil Height Before Shearing (in.)	1.2	1.2	1.2
Final Moisture Content (%)	14.5	14.0	14.6

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DIRECT SHEAR TEST RESULTS

Consolidated Drained ASTM D-3080

Checked by: PZ

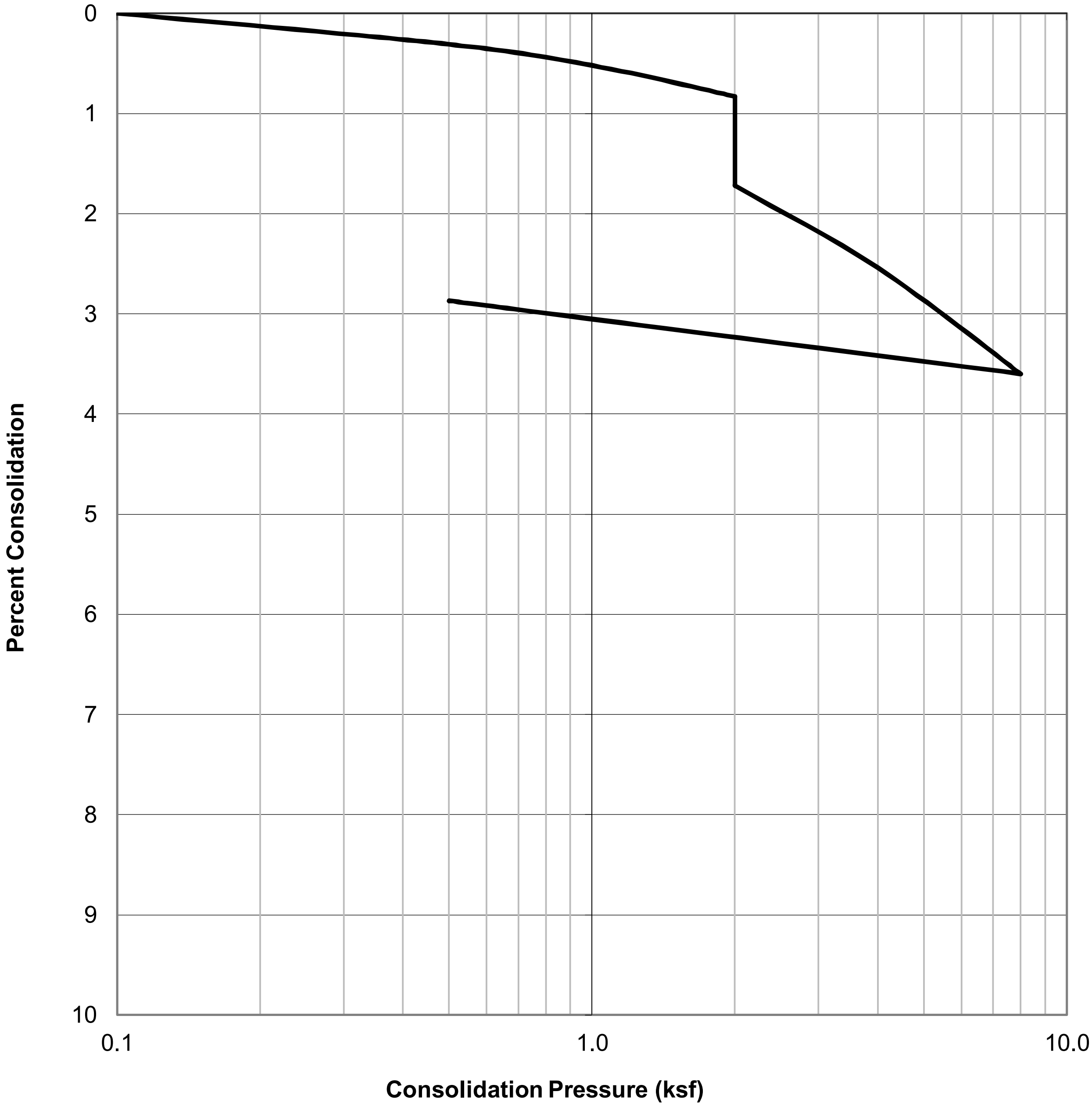
Project No.: W1249-88-01

6821 Fairlynn Boulevard
Yorba Linda, California

Nov. 2020

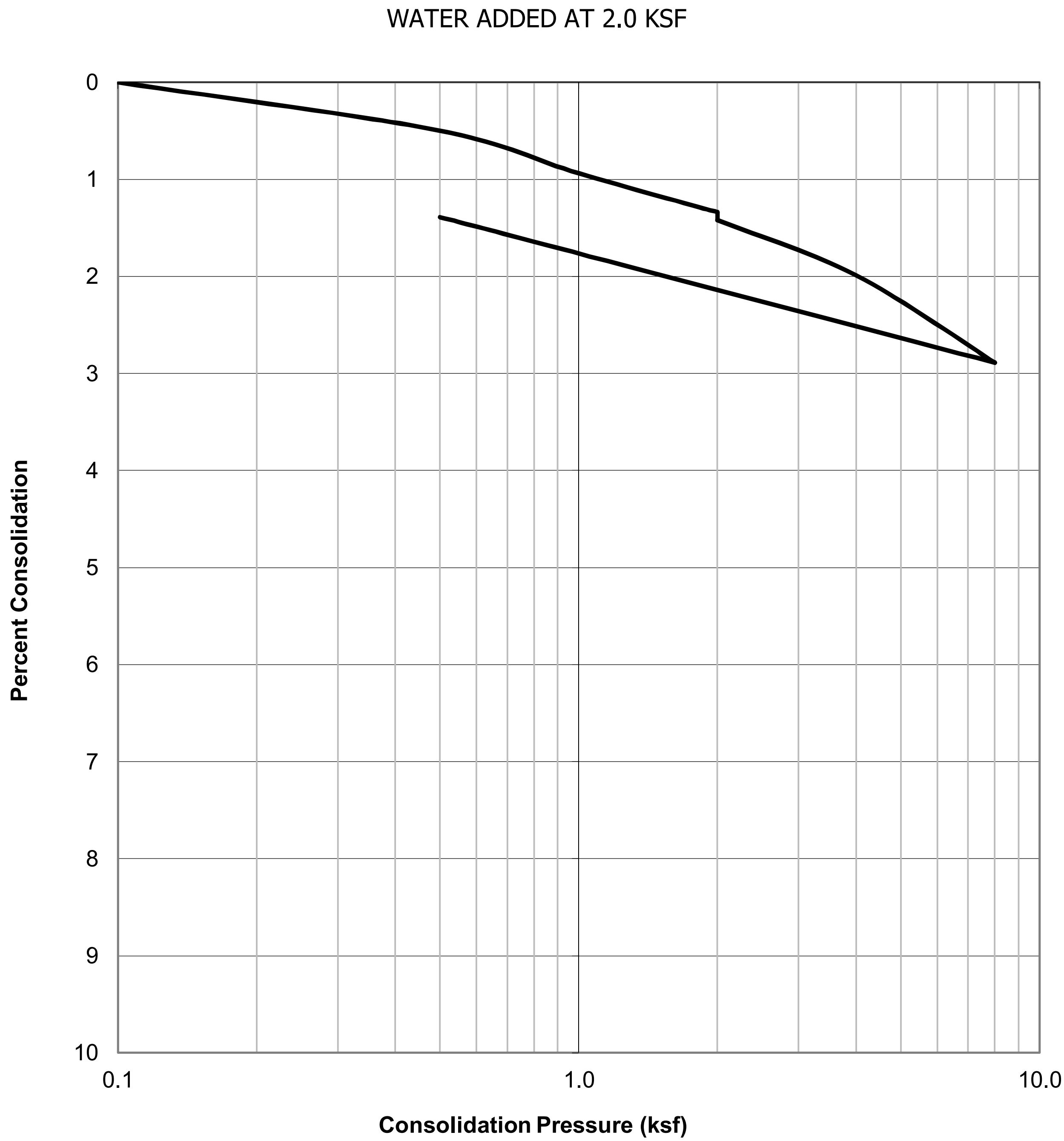
Figure B4

WATER ADDED AT 2.0 KSF



SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B5@5	Reddish Brown Sandy Silt (ML)	119.7	7.3	14.2

GEOCON	**CONSOLIDATION TEST RESULTS** ASTM D-2435	Project No.: W1249-88-01
		6821 Fairlynn Boulevard Yorba Linda, California
	Checked by: PZ	Nov. 2020 Figure B5



SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B1@6	Reddish Brown Silty Sand (SM)	119.5	13.9	15.6

GEOCON

CONSOLIDATION TEST RESULTS

ASTM D-2435

Checked by: PZ

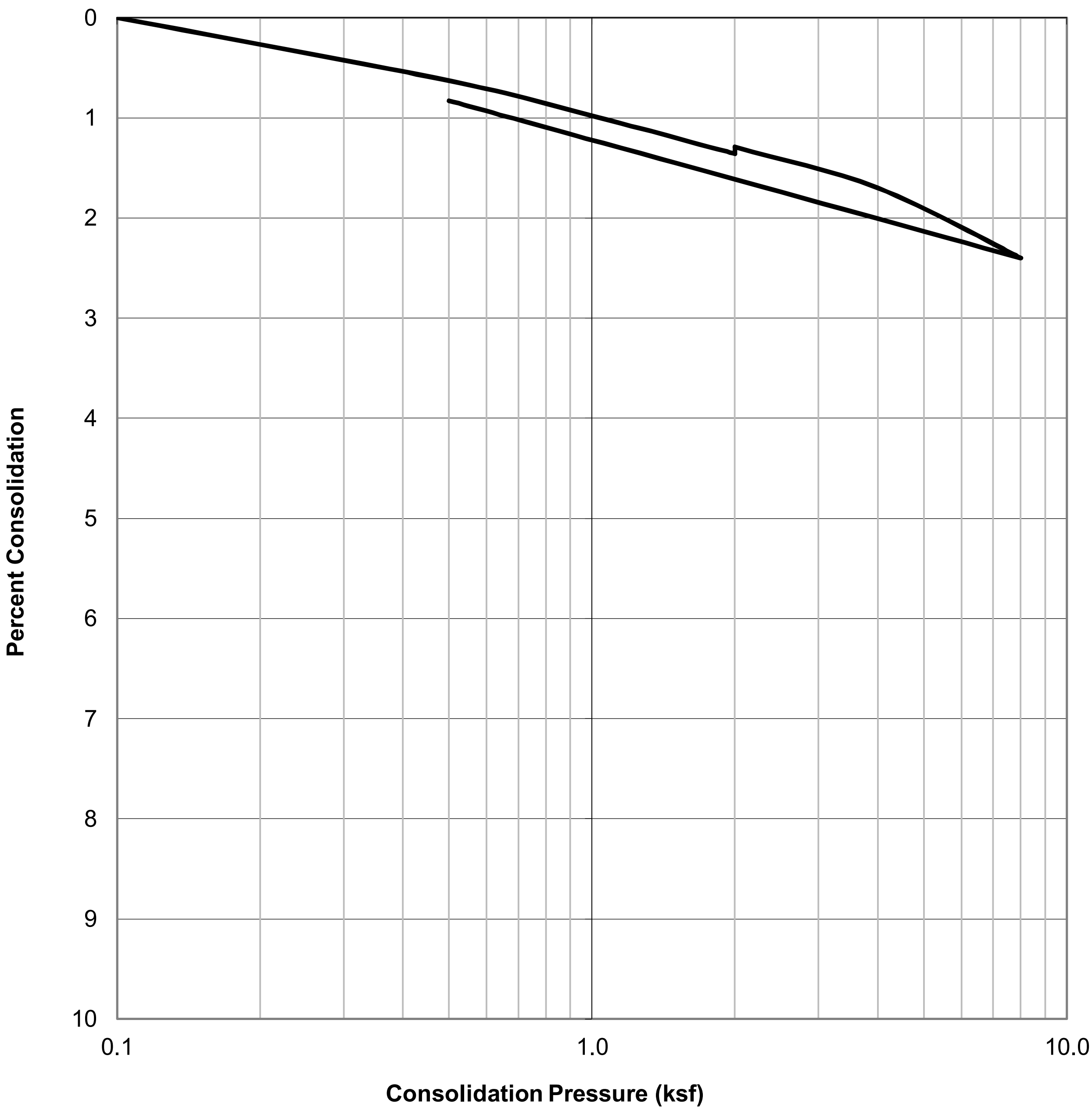
Project No.: W1249-88-01

6821 Fairlynn Boulevard
Yorba Linda, California

Nov. 2020

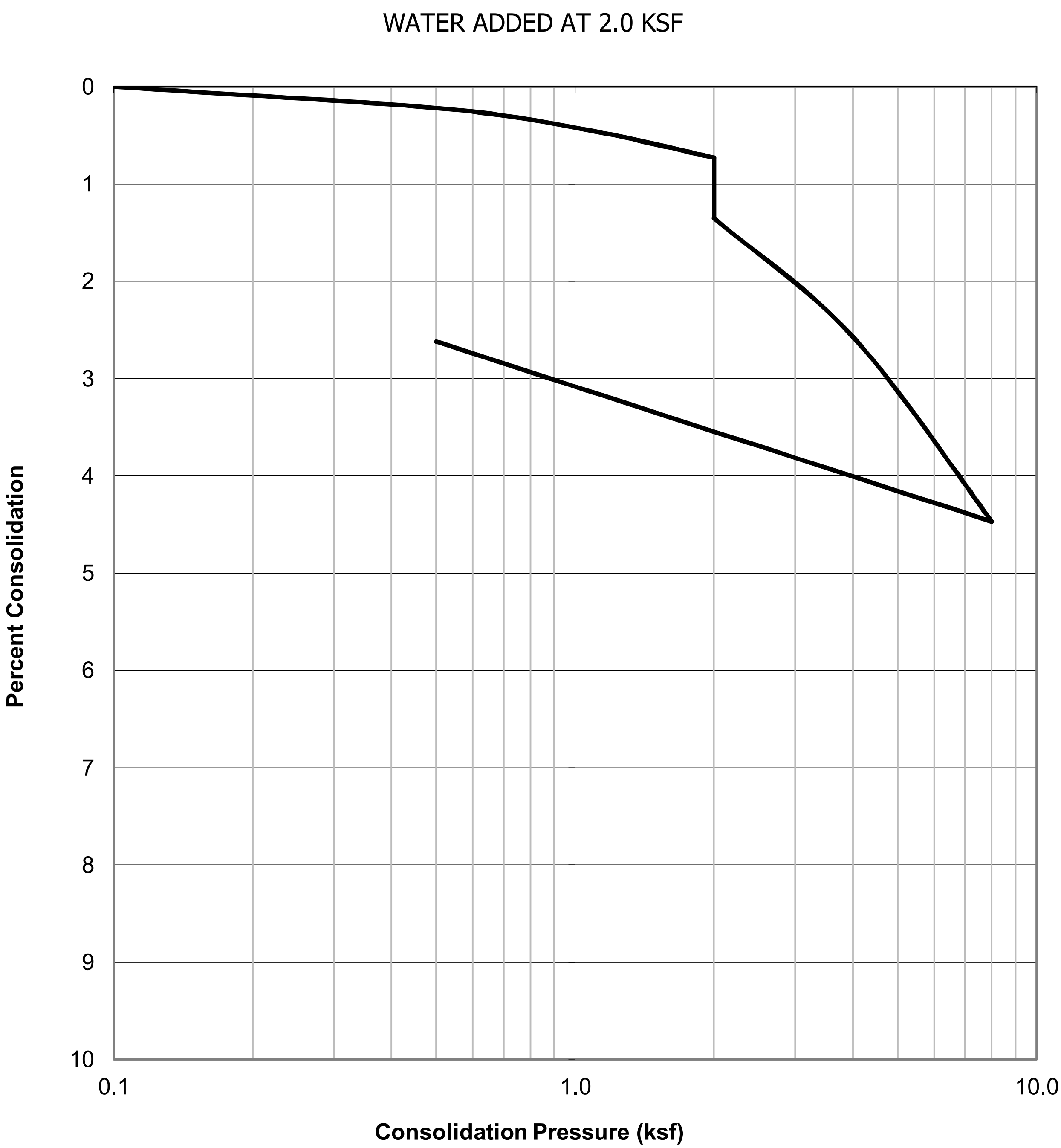
Figure B6

WATER ADDED AT 2.0 KSF



SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B2@6	Reddish Brown Sandy Silt (ML)	115.6	15.1	16.6

GEOCON	**CONSOLIDATION TEST RESULTS** ASTM D-2435	Project No.: W1249-88-01
	Checked by: PZ	6821 Fairlynn Boulevard Yorba Linda, California
		Nov. 2020 Figure B7



SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B5@7.5	Reddish Brown Silty Sand (SM)	110.4	12.0	17.0

GEOCON

CONSOLIDATION TEST RESULTS

ASTM D-2435

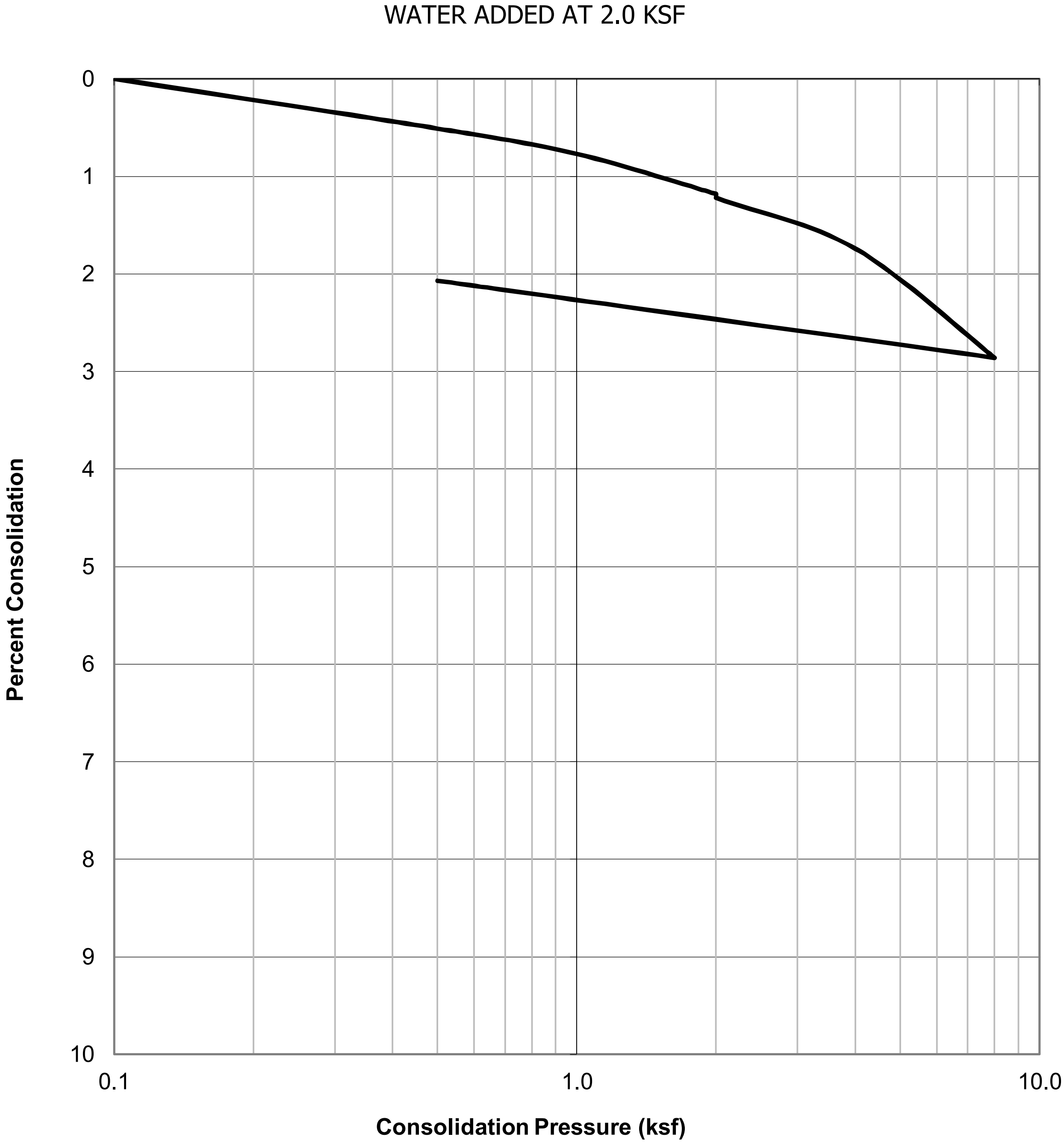
Checked by: PZ

Project No.: W1249-88-01

6821 Fairlynn Boulevard
Yorba Linda, California

Nov. 2020

Figure B8



SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B2@9	Reddish Brown Sandy Silt (ML)	117.1	15.0	15.3

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CONSOLIDATION TEST RESULTS

ASTM D-2435

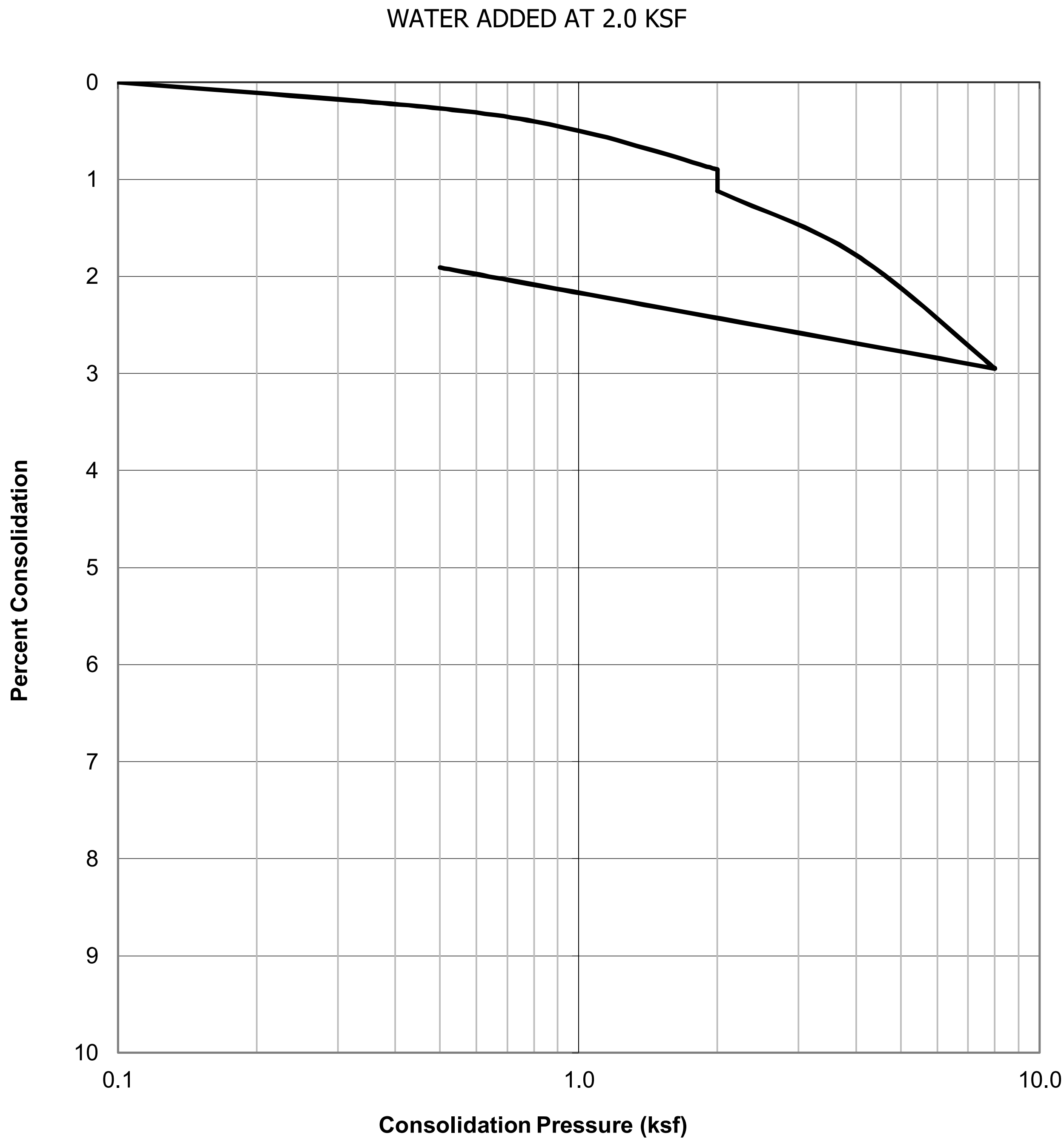
Checked by: PZ

Project No.: W1249-88-01

6821 Fairlynn Boulevard
Yorba Linda, California

Nov. 2020

Figure B9



SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B2@12	Reddish Brown Sandy Silt (ML)	118.2	13.9	15.1

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CONSOLIDATION TEST RESULTS

ASTM D-2435

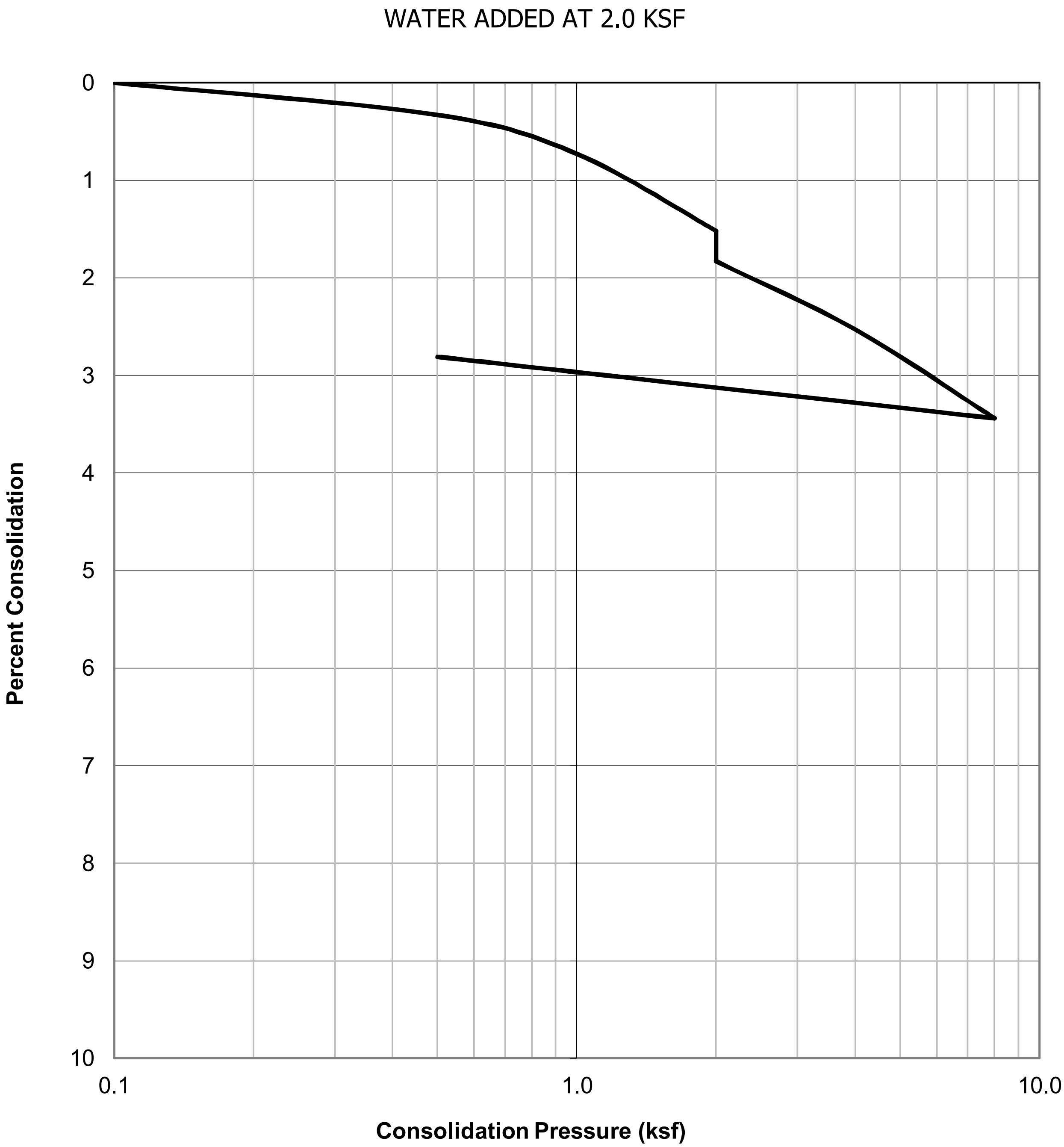
Checked by: PZ

Project No.: W1249-88-01

6821 Fairlynn Boulevard
Yorba Linda, California

Nov. 2020

Figure B10



SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B1@15	Brown Poorly Graded Sand (SP)	101.6	3.1	13.0

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CONSOLIDATION TEST RESULTS

ASTM D-2435

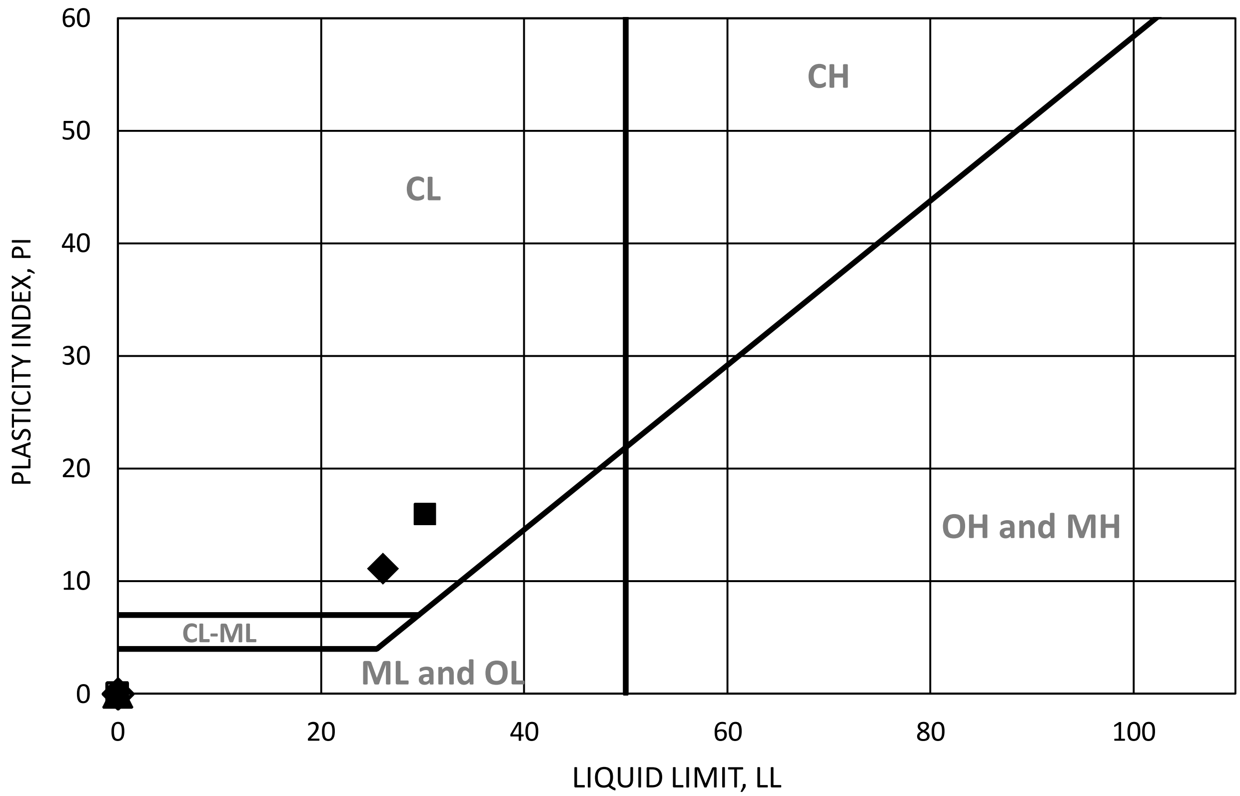
Checked by: PZ

Project No.: W1249-88-01

6821 Fairlynn Boulevard
Yorba Linda, California

Nov. 2020

Figure B11



SYMBOL	BORING	DEPTH (ft)	LL	PL	PI	MOISTURE CONTENT AT SATURATION	SOIL BEHAVIOR
■	B-4	2.5'	30	14	16		CL
◆	B1&B3	0-5'	26	15	11		CL
▲							
●							
□							
◇							
△							
○							

N/P = Non-Plastic

GEOCON

ATTERBERG LIMITS

ASTM D-4318

Checked by: PZ

Project No.: W1249-88-01

6821 Fairlynn Boulevard
Yorba Linda, California

Nov. 2020

Figure B12

B1&B3@0-5'

MOLDED SPECIMEN		BEFORE TEST	AFTER TEST
Specimen Diameter	(in.)	4.0	4.0
Specimen Height	(in.)	1.0	1.0
Wt. Comp. Soil + Mold	(gm)	601.0	634.2
Wt. of Mold	(gm)	176.5	176.5
Specific Gravity	(Assumed)	2.7	2.7
Wet Wt. of Soil + Cont.	(gm)	396.4	634.2
Dry Wt. of Soil + Cont.	(gm)	374.2	393.1
Wt. of Container	(gm)	96.4	176.5
Moisture Content	(%)	8.0	16.4
Wet Density	(pcf)	128.0	137.9
Dry Density	(pcf)	118.6	118.4
Void Ratio		0.4	0.5
Total Porosity		0.3	0.3
Pore Volume	(cc)	61.4	66.4
Degree of Saturation	(%) [S_{meas}]	51.6	97.4

Date	Time	Pressure (psi)	Elapsed Time (min)	Dial Readings (in.)
11/16/2020	15:30	1.0	0	0.454
11/16/2020	15:40	1.0	10	0.4538
Add Distilled Water to the Specimen				
11/17/2020	10:00	1.0	1100	0.4777
11/17/2020	11:00	1.0	1160	0.4777

Expansion Index (EI meas) =	23.9
Expansion Index (Report) =	**24**

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

* Reference: 2019 California Building Code, Section 1803.5.3
** Reference: 1997 Uniform Building Code, Table 18-I-B.

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EXPANSION INDEX TEST RESULTS

ASTM D-4829

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Project No.: W1249-88-01

6821 Fairlynn Boulevard
Yorba Linda, California

Nov. 2020

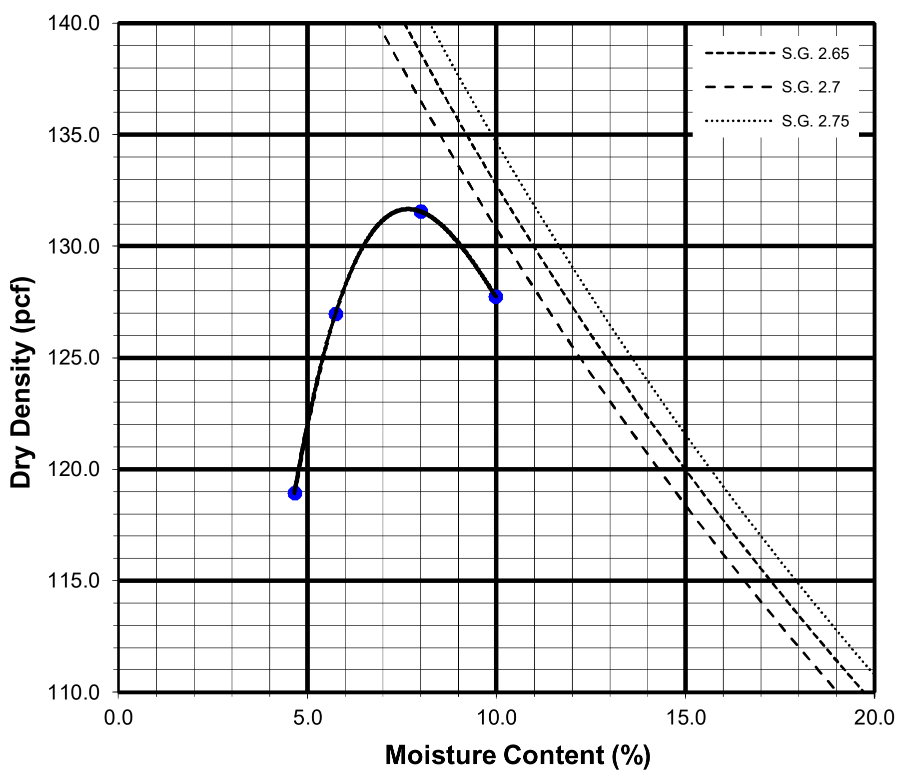
Figure B13

Sample No:

B1&B3@0-5'	Brown Silty Sand (SM)

TEST NO.		1	2	3	4	5	6
Wt. Compacted Soil + Mold	(g)	6438	6414	6320	6172		
Weight of Mold	(g)	4292	4292	4292	4292		
Net Weight of Soil	(g)	2146	2122	2028	1880		
Wet Weight of Soil + Cont.	(g)	1033.6	1021.2	1150.8	402.1		
Dry Weight of Soil + Cont.	(g)	1004.0	987.2	1136.0	388.5		
Weight of Container	(g)	633.6	646.6	878.2	96.4		
Moisture Content	(%)	8.0	10.0	5.7	4.7		
Wet Density	(pcf)	142.1	140.5	134.3	124.5		
Dry Density	(pcf)	131.6	127.7	127.0	118.9		

Maximum Dry Density (pcf)	**131.5**	**Optimum Moisture Content (%)**	**7.0**



Preparation Method: A

GEOCON	**MODIFIED COMPACTION TEST OF SOILS** ASTM D-1557	Project No.:	W1249-88-01
		6821 Fairlynn Boulevard Yorba Linda, California	
	Checked by: PZ	Nov. 2020	Figure B14

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

Sample No.	**pH**	**Resistivity (ohm centimeters)**
B1&B3@0-5'	8.3	1700 (Corrosive)

SUMMARY OF LABORATORY CHLORIDE CONTENT TEST RESULTS
EPA NO. 325.3

Sample No.	Chloride Ion Content (%)
B1&B3@0-5'	0.015

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

Sample No.	**Water Soluble Sulfate (% SQ_4)**	**Sulfate Exposure***
B1&B3@0-5'	0.000	S0

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CORROSIVITY TEST RESULTS

Checked by: PZ

Project No.: W1249-88-01

6821 Fairlynn Boulevard
Yorba Linda, California

Nov. 2020

Figure B15

Project No. W1249-88-01
March 31, 2021

Mr. Alex Wong
Red Oak Investments, LLC
4199 Campus Drive, #200
Irvine, California 92612

Subject: PERCOLATION TEST RESULTS
6821 FAIRLYNN BOULEVARD
YORBA LINDA, CALIFORNIA

References: *Preliminary Geotechnical Investigation, Proposed Residential Development, 6821 Fairlynn Boulevard, Yorba Linda, California,* prepared by Geocon West, Inc. dated November 20, 2020.

Dear Mr. Wong:

In accordance with your authorization of our proposal dated February 23, 2021, this letter has been prepared to present the results of the percolation testing performed at 6821 Fairlynn Boulevard in the City of Yorba Linda, California.

At the request of the project team, we performed percolation testing to evaluate the feasibility of onsite stormwater infiltration at the location provided to us by the civil engineer. Groundwater was encountered during our prior site exploration in boring B3 at a depth of approximately 21 feet below the ground surface. Based on these considerations, the proposed percolation boring was limited to a depth of 10 feet in order to maintain a 10-foot offset from known groundwater elevations.

Supplemental site exploration was performed on March 16, 2021 by excavating two 3¼ inch diameter borings to depths of approximately 4 and 10½ feet below ground surface with a hand auger and manual digging equipment. Boring P1 encountered refusal at a depth of approximately 4 feet. Boring P1A was performed approximately 5 feet north of P1 and excavated to a depth of approximately 10½ feet. The location of the borings are indicated on the Site Plan (see Figure 1) and logs of the borings are provided herein as Figures 2 and 3. Groundwater was encountered not encountered during our supplemental site exploration borings excavated to depth of approximately 10½ feet below the ground surfaces.

Subsequent to the boring excavation, slotted casing was placed in each percolation boring and the annular space between the casing and excavation was filled with filter pack. The boring was then filled with water to pre-saturate the soils. On March 17, 2021, 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 average infiltration rate (adjusted percolation rate), for the earth materials encountered, is provided in the following table. The field-measured percolation rate has been adjusted to infiltration rates in accordance with the *County of Orange Technical Guidance Document for the Preparation of Conceptual/Preliminary and/or Project Water Quality Management Plans (December 2013)*. The percolation test results are provided on Figure 4.

Boring	**Soil Type**	**Infiltration Depth (ft)**	**Average Infiltration Rate (in / hour)**
P1A	SP-SM, CL	5-10	0.1

The results of the percolation testing indicate that the infiltration rate within the alluvial soils is less than the generally accepted minimally required infiltration rate of 0.3 inches per hour. Therefore, based on these considerations, a stormwater infiltration system is not recommended for this development. It is suggested that stormwater be retained, filtered and discharged in accordance with the requirements of the local governing agency.

Should you have any questions regarding this letter, or if we may be of further service, please contact the undersigned at your convenience.

Very truly yours,

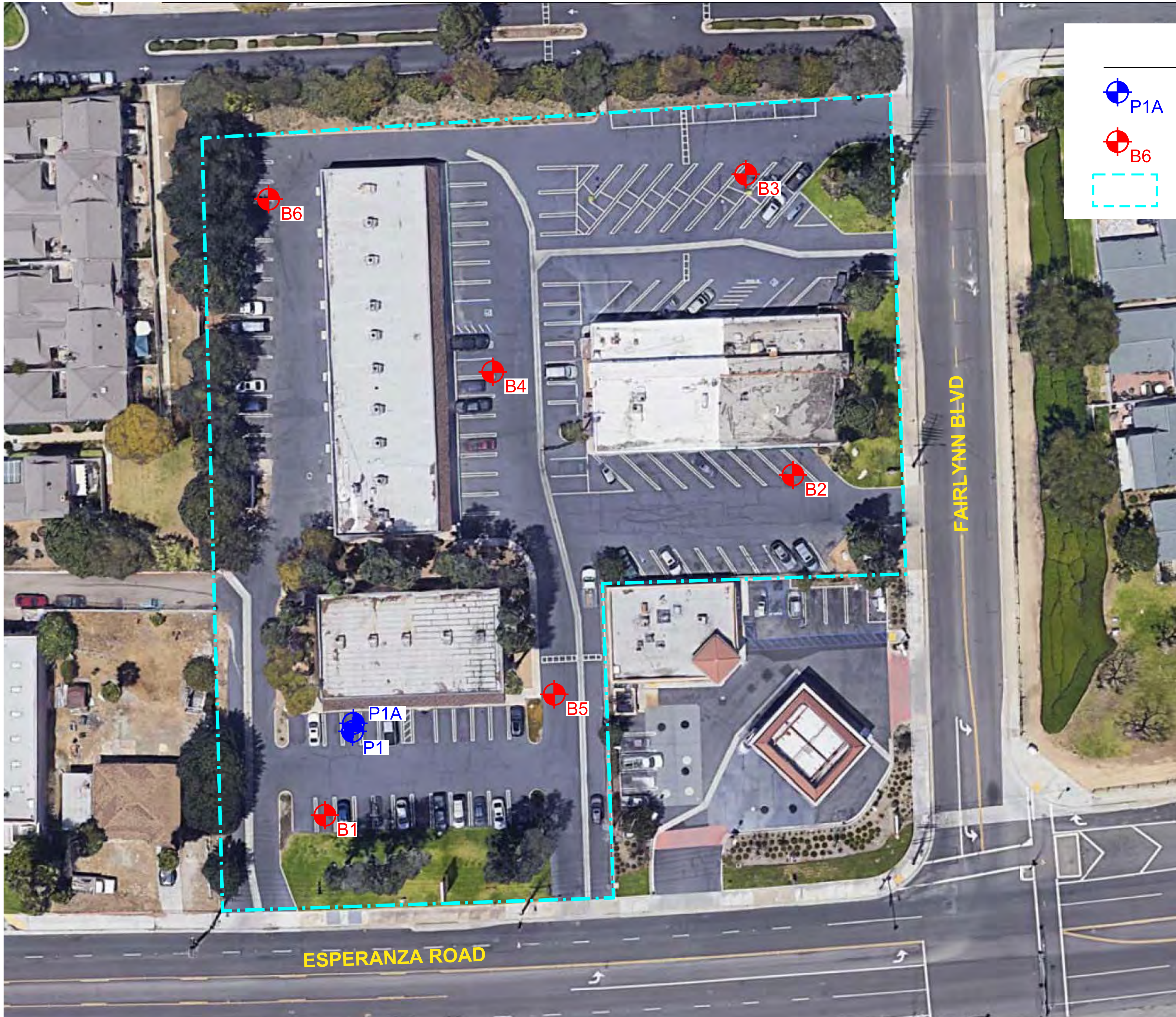
GEOCON WEST, INC.

REGISTERED PROFESSIONAL ENGINEER
JELISA THOMAS ADAMS
GE3092
GEOTECHNICAL
STATE OF CALIFORNIA

John Stapleton
Staff Engineer

Jelisa Thomas Adams
GE 3092

Attachments: Figure 1, Site Plan
Figures 2 and 3, Boring Logs
Figure 4, Percolation Test Data Sheet



LEGEND

P1A — Approximate Location of Percolation Boring (2021)

B6 — Approximate Location of Prior Boring (2020)

Approximate Limits of Proposed Development

0 50' 100'

GEOCON
WEST, INC.

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

DRAFTED BY: JS	CHECKED BY: JTA

SITE PLAN

6821 FAIRLYNN BOULEVARD
YORBA LINDA, CALIFORNIA

MARCH 2021	PROJECT NO. W1249-88-01	FIG. 1

BORING P1

ELEV. (MSL.) -- DATE COMPLETED 3/16/2021

EQUIPMENT HAND AUGER BY: JS

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT)*	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0 – 2					**AC: 3.25" BASE: 8"** **ARTIFICIAL FILL** Sandy Clay, firm, moist, reddish brown, fine- to medium-grained			
2 – 4	P1@2.5			SP	**ALLUVIUM** Sand with Gravel, dense, slightly moist, dark yellowish brown, medium- to course-grained sand, fine- to medium-grained gravel			

Total depth of boring: 4 feet (refusal)
Fill to 2.7 feet
No groundwater encountered
Backfilled with soil cuttings and tamped
Surface Patched

NOTE: The stratification lines presented herein represent the appoximate boundary between earth types; the transitions may be gradual.

W1249-88-01 BORING LOGS.GPJ

Figure 2,
Log of Boring P1, 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.

BORING P1A

ELEV. (MSL.) -- DATE COMPLETED 3/16/2021

EQUIPMENT HAND AUGER BY: JS

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT)*	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0					**AC: 3" BASE: 8"** **ARTIFICIAL FILL** Sandy Clay, firm, moist, reddish brown, fine- to medium-grained			
2								
3	P1A@3'			SP	**ALLUVIUM** Sand with Gravel, dense, slightly moist, dark yellowish brown, medium- to course-grained sand, fine gravel			
4								
5				SP-SM	Sand with Silt, medium dense, moist, brown, fine- to -medium grained			
6				CL	Sandy Clay, firm to stiff, slightly moist, dark yellowish brown, fine-grained			
8								
10	P1A@10'							

Total depth of boring: 10.5 feet
Fill to 2.5 feet
No groundwater encountered
Percolation testing performed
Backfill with soil cuttings and tamped
Surface Patched

NOTE: The stratification lines presented herein represent the appoximate boundary between earth types; the transitions may be gradual.

Figure 3,
Log of Boring P1A, Page 1 of 1

W1249-88-01 BORING LOGS.GPJ

SAMPLE SYMBOLS

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

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

PERCOLATION TEST DATA SHEET

Project:	Red Oak - Esperanza		Project No:	W1249-88-01		Date:	3/17/2021
Test Hole No:		P1A	Tested By:	JS			
Depth of Test Hole, D_T:		10	USCS Soil Classification:		SP-SM/CL		
Test Hole Dimensions (inches)					Length	Width	
Diameter (if round) =		3.25	Sides (if rectangular) =		---	---	

Sandy Soil Criteria Test*

Trial No.	Start Time	Stop Time	Δt Time Interval (min)	D_0 Initial Depth to Water (in)	D_f Final Depth to Water (in)	ΔD Change in Water Level (in)	Greater than or Equal to 6"? (y/n)
1	7:09	7:34	25	67.2	71.5	4.3	n
2	7:35	8:00	25	67.2	70.0	2.8	n

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

Trial No.	Start Time	Stop Time	Δt Time Interval (min)	D_0 Initial Depth to Water (in)	D_f Final Depth to Water (in)	ΔD Change in Water Level (in)	Percolation Rate (min/in)
1	8:01	8:31	30	70.0	75.6	5.6	7660
2	8:31	9:01	30	69.4	74.4	5.0	8571
3	9:01	9:31	30	69.8	74.3	4.4	9730
4	9:31	10:01	30	70.2	74.4	4.2	10286
5	10:01	10:31	30	68.6	73.0	4.3	10000
6	10:31	11:01	30	70.3	73.3	3.0	14400
7							
8							

Infiltration Rate Calculation:

Time Interval, Δt =	30	minutes	Ho =	49.7	inches
Final Depth to Water, Df =	73.3	inches	Hf =	46.7	inches
Test Hole Radius, r =	1.625	inches	ΔH =	3.0	inches
Initial Depth to Water, Do =	70.3	inches	Havg =	48.2	inches
Total Depth of Test Hole, DT =	120.0	inches			

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

Infiltration Rate, It = **0.10** inches/hour