GEOTECHNICAL INVESTIGATION

PROPOSED PUBLIC WORKS YARD IMPROVEMENTS (CIP-19-050)
12158 BASELINE ROAD
RANCHO CUCAMONGA, CALIFORNIA

APN: 108903113; 108903139; 108903138

GEOCON
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GEOTECHNICAL
ENVIRONMENTAL
MATERIALS

PREPARED FOR

SAN BERNARDINO REAL EASTATE SERVICES DEPARTMENT – PROJECT MANAGEMENTS DIVISION
SAN BERNARDINO, CA

PROJECT NO. A9816-99-01

MARCH 8, 2019

GEOCON
WEST, INC.
GEOTECHNICAL ■ ENVIRONMENTAL ■ MATERIALS

Project No. A9816-99-01
March 8, 2019

Ms. Dani Fox
San Bernardino County Real Estate Services Department
Project Management Division
825 East Third Street
San Bernardino, California 92415

Subject: GEOTECHNICAL INVESTIGATION
PROPOSED PUBLIC WORKS YARD IMPROVEMENTS (CIP-19-050)
12158 BASELINE ROAD, RANCHO CUCAMONGA, CALIFORNIA
APN: 108903113; 108903139; 108903138

Dear Ms. Fox:

In accordance with your authorization of our proposal dated January 4, 2019, we have prepared this geotechnical investigation report for the proposed public works yard improvements located at 12158 Baseline Road in the City of Rancho Cucamonga, California. The accompanying report presents the findings of our study, and our conclusions and recommendations pertaining to the geotechnical aspects of proposed design and construction. Based on the results of our investigation, it is our opinion that the site can be developed as proposed, provided the recommendations of this report are followed and implemented during design and construction.

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

Very truly yours,

GEOCON WEST, INC.

John Stapleton	Jelisa Thomas Adams	Susan F. Kirkgard
EIT 164283	GE 3092	CEG 1754

(EMAIL) Addressee

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GEOTECHNICAL INVESTIGATION

1. PURPOSE AND SCOPE

This report presents the results of a geotechnical investigation for the proposed public works yard improvements located at 12158 Baseline Road in the City of Rancho Cucamonga, California (see Vicinity Map, Figure 1). The purpose of the investigation was to evaluate subsurface soil and geologic conditions underlying the site and, based on conditions encountered, to provide conclusions and recommendations pertaining to the geotechnical aspects of design and construction.

The scope of this investigation included a review of published documents for the site, a site reconnaissance, field exploration, laboratory testing, engineering analysis, and the preparation of this report. The site was explored on February 4, 2019, by excavating five 8-inch diameter borings to depths between 5½ and 25½ feet below the existing ground surface using a truck-mounted hollow-stem auger drilling machine. The approximate locations of the exploratory borings are depicted on the Site Plan (see Figure 2). A detailed discussion of the field investigation, including the boring logs, is presented in Appendix A.

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

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

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

2. SITE AND PROJECT DESCRIPTION

The subject site is located at 12158 Baseline Road, in the City of Rancho Cucamonga, California. The site consists of a relatively level pad, which has not been developed. Currently, the site is occupied by a storage yard and single-story maintenance buildings and storage containers. The site is bounded by an additional storage yard to the west, by a shopping center and Day Creek Boulevard to the east, by Pacific Electric Bike Trail to the north, and by Baseline Road to the south. The site topography is roughly level to gently sloping to the south. Surface water drainage at the site appears to have no discernable pattern. The site is paved with gravel and has no vegetation.

Based on the information provided by the Client, it is our understanding that the proposed development will consist of a 3,300-square-foot metal structure, as well as site improvements such as utility connections, CMU perimeter walls, a heavy equipment yard, lighting, landscaping, and pavement. The existing and proposed 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 structure will be up to 100 kips, and wall loads will be up to 1 kip per linear foot.

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

3. GEOLOGIC SETTING

The site is located in the northern portion of the Chino Basin in San Bernardino County, California. The Chino Basin encompasses a broad area of coalescing alluvial fans that extend southward from the San Gabriel Mountains and overlies a down-dropped structural block which is bounded by the Elsinore Fault and the Chino Fault to the southwest, the Red Hill-Etiwanda Avenue Fault to the northwest, the San Gabriel Mountains and Sierra Madre Fault to the north, by the Rialto-Colton Fault to the northeast, and the La Sierra Hills and Jurupa Hills to the southeast. The alluvial deposits within the Chino Basin consist of Holocene age (last 11,700 years old) and Pleistocene age (11,000 to 2 million years old) alluvial sediments. A thin veneer of eolian sand mantles portions of the Chino Basin.

Locally, the site is located on one of the alluvial fans that extends southward from the San Gabriel Mountains, located approximately 3.0 miles north of the site. Day Creek, a southerly flowing drainage bounds the site on the west. Regionally, the Chino Basin is located within the Peninsular Ranges geomorphic province. This province comprises the northwesterly-trending mountains, valleys, and geologic structures extending from the southern Baja Peninsula to the Transverse Ranges in Southern California.

4. SOIL AND GEOLOGIC CONDITIONS

Based on our field investigation and published geologic maps of the area, the site is underlain by previously placed fill and Holocene age alluvial fan deposits consisting predominately of sand and gravel (CGS, 2010). 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 field explorations to a maximum depth of 4 feet below existing ground surface. The fill generally consists of light brown and brown poorly graded sand and silty sand with varying amounts of gravel. The fill is characterized as dry to moist and loose to medium dense. The fill is the result of past grading and 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 Alluvial Fan Deposits

Holocene age alluvial deposits were encountered beneath the fill. The alluvium was generally light brown to dark brown or dark yellowish brown poorly graded to well-graded sand with gravel, gravel with sand or silty sand with various amounts of cobbles. Although not directly observed in our borings, boulders are common in this geologic environment. The alluvium is characterized as fine- to coarse-grained, dry to moist, and loose to very dense.

5. GROUNDWATER

The site is located in the Chino Basin of the Upper Santa Ana Valley Groundwater Basin (Chino Basin Water Master [CBWM] 2017). A review of groundwater contour maps published by the California Division of Mines and Geology (CDMG, 1976) and the U. S. Geological Survey (Mendenhall, 1904) indicate that the groundwater level in the immediate site vicinity has historically been greater than 250 feet beneath the ground surface since 1904.

Review of the California Department of Water Resources Data Library (CDWR, 2019) indicates the closest groundwater monitoring well to the site is Well Number 341217N1175119W001, located approximately 0.9 mile east of the site. The highest groundwater level recorded for this well for the monitoring period between 2011 and 2018, was in 2012 when groundwater was at a depth of approximately 574 feet beneath the existing ground surface. The most recent groundwater level measurement indicates the depth to water was approximately 582 feet below the surface on November 14, 2018. Based on current groundwater basin management practices, it is unlikely that groundwater levels will ever exceed the historic high levels.

Groundwater was not encountered in the borings drilled to a maximum depth of 25½ feet beneath the existing ground surface. Based on the lack of groundwater observed in our borings, the depth to groundwater as recorded in nearby wells (CDWR, 2019; CBWM, 2017), and the depth of the proposed construction, groundwater is neither expected to be encountered during construction, nor have a detrimental effect on the project. However, it is common for groundwater levels to vary seasonally or for perched groundwater conditions to develop where none previously existed, especially in impermeable fine-grained soils which are subjected to irrigation or precipitation. In addition, recent requirements for stormwater infiltration could result in shallower seepage conditions in the region. Proper surface drainage of irrigation and precipitation will be critical to 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 active, potentially active, and inactive faults. The criteria for these major groups are based on criteria developed by the California Geological Survey (CGS, formerly known as CDMG) for the Alquist-Priolo Earthquake Fault Zone Program (CGS, 2018). By definition, an active fault is one that has had surface displacement within Holocene time (about the last 11,700 years). A potentially active fault has demonstrated surface displacement during Quaternary time (approximately the last 1.6 million years) but has had no known Holocene movement. Faults that have not moved in the last 1.6 million years are considered inactive.

The site is not within a state-designated Alquist-Priolo Earthquake Fault Zone (CGS, 2019a; CGS, 2019b) for surface fault rupture hazards. No active or potentially active faults with the potential for surface fault rupture are known to pass directly beneath the site. Therefore, the potential for surface rupture due to faulting occurring beneath the site during the design life of the proposed development is considered low. However, the site is located in the seismically active Southern California region, and could be subjected to moderate to strong ground shaking in the event of an earthquake on one of the many active Southern California faults. The faults in the vicinity of the site are shown in Figure 3, Regional Fault Map.

The closest active fault to the site is the Red Hill Fault located approximately 1.2 miles to the northwest (Ziony and Jones, 1989; USGS, 2006; CDMG, 1995). Other nearby active faults are the Cucamonga Fault, the San Jacinto Fault Zone, the San Andreas Fault Zone, and the Chino Fault located approximately 3.1 miles north, 7.5 miles northeast, 10.9 miles northeast, and 14.5 miles southwest of the site, respectively (Ziony and Jones, 1989).

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

6.2 Seismicity

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

LIST OF HISTORIC EARTHQUAKES

Earthquake (Oldest to Youngest)	Date of Earthquake	Magnitude	Distance to Epicenter (Miles)	Direction to Epicenter
San Jacinto-Hemet area	April 21, 1918	6.8	40	SE
Near Redlands	July 23, 1923	6.3	19	ESE
Long Beach	March 10, 1933	6.4	43	SW
Tehachapi	July 21, 1952	7.5	103	NW
San Fernando	February 9, 1971	6.6	53	WNW
Whittier Narrows	October 1, 1987	5.9	31	W
Sierra Madre	June 28, 1991	5.8	28	WNW
Landers	June 28, 1992	7.3	63	E
Big Bear	June 28, 1992	6.4	41	E
Northridge	January 17, 1994	6.7	57	W
Hector Mine	October 16, 1999	7.1	80	ENE

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

6.3 Seismic Design Criteria

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

2016 CBC SEISMIC DESIGN PARAMETERS

Parameter	Value	2016 CBC Reference
Site Class	D	Table 1613.3.2
MCE_R Ground Motion Spectral Response Acceleration – Class B (short), S_S	1.648g	Figure 1613.3.1(1)
MCE_R Ground Motion Spectral Response Acceleration – Class B (1 sec), S_1	0.604g	Figure 1613.3.1(2)
Site Coefficient, F_A	1.0	Table 1613.3.3(1)
Site Coefficient, F_V	1.5	Table 1613.3.3(2)
Site Class Modified MCE_R Spectral Response Acceleration (short), S_{MS}	1.648g	Section 1613.3.3 (Eqn 16-37)
Site Class Modified MCE_R Spectral Response Acceleration – (1 sec), S_{M1}	0.906g	Section 1613.3.3 (Eqn 16-38)
5% Damped Design Spectral Response Acceleration (short), S_{DS}	1.099g	Section 1613.3.4 (Eqn 16-39)
5% Damped Design Spectral Response Acceleration (1 sec), S_{D1}	0.604g	Section 1613.3.4 (Eqn 16-40)

The table below presents the mapped maximum considered geometric mean (MCE_G) seismic design parameters for projects located in Seismic Design Categories of D through F in accordance with ASCE 7-10.

ASCE 7-10 PEAK GROUND ACCELERATION

Parameter	Value	ASCE 7-10 Reference
Mapped MCE_G Peak Ground Acceleration, PGA	0.621g	Figure 22-7
Site Coefficient, F_{PGA}	1.0	Table 11.8-1
Site Class Modified MCE_G Peak Ground Acceleration, PGA_M	0.621g	Section 11.8.3 (Eqn 11.8-1)

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

Deaggregation of the MCE peak ground acceleration was performed using the USGS online Unified Hazard Tool, 2008 Conterminous U.S. Dynamic edition. The result of the deaggregation analysis indicates that the predominant earthquake contributing to the MCE peak ground acceleration is characterized as a 6.85 magnitude event occurring at a hypocentral distance of 9.85 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 6.90 magnitude occurring at a hypocentral distance of 11.98 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.

According to the County of San Bernardino (2010a) and the City of Rancho Cucamonga Safety Element of the General Plan (2010), the site is not located within an area identified as having a potential for liquefaction. Also, the groundwater level in the immediate site vicinity has been greater than 250 feet since 1904 and is currently greater than 500 feet beneath the site. 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 surrounding is relatively level to sloping gently to the south. According to the City of Rancho Cucamonga Safety Element (2010), the site is not within an area identified as having a potential for slope instability. 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. The City of Rancho Cucamonga Safety Element (City of Rancho Cucamonga, 2010) and the County of San Bernardino (2010b) indicate that the site is not located within a dam or debris basin inundation area or flood boundary from any such reservoirs. Therefore, the potential for inundation at the site 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, 2019; City of Rancho Cucamonga, 2010).

6.8 Oil Fields & Methane Potential

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

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 City of Rancho Cucamonga (2010) indicates that regional subsidence is possible within the general area of the site due to the low density of the subsurface soils. However, in the 1970s, the County of San Bernardino initiated a groundwater recharge program that has minimized subsidence in the area. 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. As long as the County maintains the groundwater recharge program, the potential for ground subsidence due to withdrawal of fluids or gases at the site is considered low.

7. CONCLUSIONS AND RECOMMENDATIONS

7.1 General

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

7.1.2 Up to 4 feet of existing artificial fill was encountered during the site investigation. 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. It is our opinion that the existing fill, in its present condition, is not suitable for direct support of proposed foundations or slabs. The existing fill and site soils are suitable for re-use as engineered fill provided the recommendations in the Grading section of this report are followed (see Section 7.4).

7.1.3 The results of the laboratory testing indicate that some of the alluvial soils may be subject to excessive hydro-consolidation upon saturation (see Figure B3). Hydro-consolidation is the tendency of a soil structure to collapse upon saturation, resulting in the overall settlement of the effected soils and any overlying soils or foundations supported therein. The recommendations provided herein are intended to minimize the effects of hydro-consolidation on proposed improvements.

7.1.4 Based on the potential for hydro-consolidation, maintaining proper surface drainage is critical to future performance of foundations. Recommendations for drainage are provided in the S*urface Drainage* section of this report (see Section 7.16).

7.1.5 It is recommended that the upper 4 feet of existing earth materials within the building footprint area be excavated and properly compacted for foundation and slab support. Deeper excavations should be conducted to remove all existing artificial 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 3 feet beyond the building footprint areas, including building appurtenances, or a distance equal to the depth of fill below the foundation, whichever is greater. The limits of existing fill and/or soft soil removal will be verified by the Geocon representative during site grading activities. Recommendations for earthwork are provided in the *Grading* section of this report (see Section 7.4).

7.1.6 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.).

7.1.7 Subsequent to grading of the site, the proposed structures may be supported on a conventional foundation system deriving support in newly placed engineered fill. All foundation excavations must be observed and approved in writing by the Geotechnical Engineer prior to placement of steel or concrete. Recommendations for the design of a conventional foundation system are provided in Section 7.6.

7.1.8 Where new exterior concrete slab-on-grade is to be constructed, it is recommended that all existing artificial fill and any soils disturbed during construction activities be properly compacted for slab support. Recommendations for earthwork are provided in the *Grading* section of this report (see Section 7.4).

7.1.9 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 an existing 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 7.14).

7.1.10 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. It is essential that proper drainage be maintained in order to minimize settlements in the soils and any foundations supported therein. 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. Where excavation and compaction cannot be performed or is undesirable, and due to the depth of previously placed fill at the site, Geocon should be contacted for additional recommendations.

7.1.11 Where new paving is to be placed, it is recommended that unsuitable or soft existing fill and 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 7.13).

7.1.12 Based on the results of percolation testing performed in the upper 10 feet of site soils, a stormwater infiltration system is considered feasible for this project. Additional discussion is provided in the *Stormwater Infiltration* section of this report (see Section 7.15).

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

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

7.1.15 The most recent ASTM standards apply to this project and must be utilized, even if older ASTM standards are indicated in 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. Due to the presence of granular soils, caving should be anticipated in unshored vertical excavations and the contractor should be prepared for caving conditions. Formwork may be required to prevent caving of foundation excavations. In addition, due to the presence of cobbles, the contractor should be prepared for difficult excavation conditions during earthwork activities.

7.2.2 Screening of the earth materials may be required to remove oversize (greater than 6 inches) rock, prior to placement and compaction. Oversized materials should be managed in accordance with the recommendations provided herein.

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

7.2.4 All onsite excavations must be conducted in such a manner that potential surcharges from existing structures, construction equipment, and vehicle loads are resisted. The surcharge area may be defined by a 1:1 projection down and away from the bottom of an existing foundation or vehicle load. Penetrations below this 1:1 projection will require special excavation measures such as sloping and shoring. Excavation recommendations are provided in the *Temporary Excavations* section of this report (see Section 7.14).

7.2.5 The upper 5 feet of existing site soils encountered during the investigation are considered to have a “very low” expansive potential (EI = 0) and are classified as “non-expansive” in accordance with the 2016 California Building Code (CBC) Section 1803.5.3. The recommendations presented herein assume that the building foundations and slabs will derive support in these materials.

7.3 Minimum Resistivity, pH, and Water-Soluble Sulfate

7.3.1 Potential of Hydrogen (pH) and resistivity testing, as well as chloride content testing, were performed on representative samples of on-site material to generally evaluate the corrosion potential to surface utilities. The tests were performed in accordance with California Test Method Nos. 643 and 422 and indicate that the soils are considered “mildly corrosive” with respect to corrosion of buried ferrous metals on site. The results are presented in Appendix B (Figure B7) and should be considered for design of underground structures.

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 B7) and indicate that the on-site materials possess a sulfate exposure class of “S0” to concrete structures as defined by 2016 CBC Section 1904 and ACI 318-14 Table 19.3.1.1.

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

7.4 Grading

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

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

7.4.3 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 4 feet of existing earth materials within the proposed building footprint area be excavated and properly compacted for foundation and slab support. Deeper excavations should be conducted as necessary to remove all 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 90 percent of the laboratory maximum dry density in accordance with ASTM D 1557 (latest edition).

7.4.7 It is anticipated that stable excavations for the proposed construction activities 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 7.14).

7.4.8. Where new paving is to be placed, it is recommended that unsuitable or soft existing fill and alluvial soils be excavated and properly compacted for paving support. As a minimum, the upper 12 inches of subgrade soil should be scarified, moisture conditioned to optimum moisture content, and compacted to at least 95 percent relative compaction for paving support. Paving recommendations are provided in *Preliminary Pavement Recommendations* section of this report (see Section 7.13).

7.4.9 Utility trenches should be properly backfilled in accordance with the requirements of the Green Book (latest edition). The pipe should be bedded with clean sands (Sand Equivalent greater than 30) to a depth of at least 1 foot over the pipe, and the bedding material must be inspected and approved in writing by the Geotechnical Engineer (a representative of Geocon). The use of gravel is not acceptable unless used in conjunction with filter fabric to prevent the gravel from having direct contact with soil. The remainder of the trench backfill may be derived from onsite soil or approved import soil, compacted as necessary, until the required compaction is obtained. The use of 2-sack slurry is also acceptable as backfill. Prior to placing any bedding materials or pipes, the excavation bottom must be observed and approved in writing by the Geotechnical Engineer (a representative of Geocon).

7.4.10 Although not anticipated for this project, all imported fill shall be observed, tested, and approved by Geocon West, Inc. prior to bringing soil to the site. Rocks larger than 6 inches in diameter shall not be used in the fill. If necessary, import soils used as structural fill should have an expansion index less than 20 and corrosivity properties that are equally or less detrimental to that of the existing onsite soils (see Figure B7). 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.

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

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 up to 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. In addition, additional shrinkage may occur during the required scarification and compaction of the excavation bottom. The grading contractor should verify shrinkage and earthwork yardage estimates.

7.5.2 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 structure 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 400 psf and 600 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 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 Foundation Settlement

7.7.1 The maximum expected static settlement for a structure supported on a conventional foundation system deriving support in the recommended bearing materials and designed with a maximum bearing pressure of 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.

7.7.2 Once the design and foundation loading configurations for the proposed structure 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.

7.8 Friction Pile Foundations – Light Standards

7.8.1 Typical light standards are between 10 and 15 feet in height and are supported on pile foundations. Cast-in-place friction piles may be utilized for support of proposed light standards provided foundations derive support in the competent alluvium generally found at or below a depth of 2 feet.

7.8.2 Friction piles should be a minimum of 18 inches in diameter and should be embedded a minimum of 6 feet into the recommended bearing materials. Where not protected from erosion or disturbance, the upper 2 feet of soil should be ignored when calculating axial and lateral capacity.

7.8.3 Friction piles may be designed based on a skin friction capacity of 160 psf. Uplift capacity may be assumed to be ⅔ the axial capacity in compression. Friction piles do not require the complete removal of all loose earth materials from the bottom of the excavation since the end-bearing capacity is not being considered for design. However, a cleanout of the excavation bottom will be required. A one-third increase in the capacity may be used for wind or seismic loads.

7.8.4 For design purposes, an allowable passive value for the soils may be assumed to be 290 psf per foot. The allowable passive value may be doubled for isolated piles placed more than twice the diameter. To develop the full lateral value, provisions should be implemented to assure firm contact between the piles and the surrounding soil. The allowable passive pressure may be increased by up to one-third for transient loads due to wind or seismic forces.

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

7.9 Deepened Foundation Installation

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

7.9.2 Friction piles do not require the complete removal of all loose earth materials from the bottom of the excavation since the end-bearing capacity is not being considered for design. However, a cleanout of the excavation bottom will be required.

7.9.3 Groundwater was not encountered in our field explorations, drilled to a maximum depth of 25½ feet below the existing ground surface. However, should groundwater or seepage be encountered during construction, pile excavations with more than 6 inches of standing water level require the use of a tremie to place the concrete into the bottom of the hole. A tremie shall consist of a water-tight tube, with a hopper at the top. The tube shall be equipped with a device that will close the discharge end and prevent water from entering the tube while it is being charged with concrete. The tremie shall be supported so as to permit free movement of the discharge end over the entire top surface of the work and to permit rapid lowering when necessary to retard or stop the flow of concrete. The discharge end shall be closed at the start of the work to prevent water entering the tube and shall be entirely sealed at all times, except when the concrete is being placed. The tremie tube shall be kept full of concrete. The flow shall be continuous until the work is completed and the resulting concrete seal shall be monolithic and homogeneous. The tip of the tremie tube shall always be kept about 5 feet below the surface of the concrete and definite steps and safeguards should be taken to insure that the tip of the tremie tube is never raised above the surface of the concrete.

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

7.9.5 Closely spaced piles should be drilled and filled alternately, with the concrete permitted to set at least 8 hours before drilling an adjacent hole. Pile excavations should be filled with concrete as soon after drilling and inspection as possible; the holes should not be left open overnight unless approved by the Geotechnical Engineer.

7.10 Miscellaneous Foundations

7.10.1 Foundations for small outlying structures, such as block walls up to 6 feet in height, planter walls or trash enclosures which will not be 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. It is essential that proper drainage be maintained in order to minimize settlements in the soils and any foundations supported therein. Where excavation and compaction cannot be performed or is undesirable, and due to the depth of previously placed fill at the site, Geocon should be contacted for additional recommendations.

7.10.2 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.

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

7.11 Lateral Design

7.11.1 Resistance to lateral loading may be provided by friction acting at the base of foundations, slabs and by passive earth pressure. An allowable coefficient of friction of 0.45 may be used with the dead load forces in the undisturbed alluvial soils or newly placed engineered fill.

7.11.2 Passive earth pressure for the sides of foundations and slabs poured against undisturbed alluvial soils or newly placed engineered fill soils may be computed as an equivalent fluid having a density of 290 pounds per cubic foot (pcf) with a maximum earth pressure of 2,900 psf. When combining passive and friction for lateral resistance, the passive component should be reduced by one-third.

7.12 Concrete Slabs-On-Grade

7.12.1 Concrete slabs-on-grade subject to vehicle loading should be designed in accordance with the recommendations in the *Preliminary Pavement Recommendations* section of this report (Section 7.13).

7.12.2 Subsequent to the recommended grading, concrete slabs-on-grade for structures, not subject to vehicle loading, should be a minimum of 4 inches thick and minimum slab reinforcement should consist of No. 4 steel reinforcing bars placed 16 inches on center in both horizontal directions. Steel reinforcing should be positioned vertically near the slab midpoint. The finished subgrade for the concrete slab-on-grade must be observed and approved in writing prior to placement of a vapor retarder, reinforcing steel, or concrete.

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

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

7.12.5 Exterior slabs for walkways or flatwork, not subject to traffic loads, should be at least 4 inches thick and reinforced with No. 3 steel reinforcing bars placed 18 inches on center in both horizontal directions, positioned near the slab midpoint. Prior to construction of slabs, the upper 12 inches of subgrade should be moistened to 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.

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

7.13 Preliminary Pavement Recommendations

7.13.1 Where new paving is to be placed, it is recommended that unsuitable or soft existing fill and 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 unsuitable material may experience increased settlement and/or cracking, and may therefore have a shorter design life and increased maintenance costs. As a minimum, the upper 12 inches of paving subgrade should be scarified, moisture conditioned to optimum moisture content, and properly compacted to at least 95 percent relative compaction, as determined by ASTM Test Method D 1557 (latest edition).

7.13.2 The following pavement sections are based on an assumed R-Value of 40. Once site grading activities are complete, it is recommended that laboratory testing confirm the properties of the soils serving as paving subgrade prior to placing pavement.

7.13.3 The Traffic Indices listed below are estimates. Geocon does not practice in the field of traffic engineering. The actual Traffic Index for each area should be determined by the project civil engineer. If pavement sections for Traffic Indices other than those listed below are required, Geocon should be contacted to provide additional recommendations. Pavement thicknesses were determined following procedures outlined in the *California Highway Design Manual* (Caltrans). It is anticipated that the majority of traffic will consist of automobile and 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	4.0	4.0
Trash Truck & Fire Lanes	7	4.0	7.0

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

7.13.5 Unless specifically designed and evaluated by the project structural engineer, where exterior concrete paving will be utilized for support of vehicles, it is recommended that the concrete be a minimum of 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).

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

7.14 Temporary Excavations

7.14.1 Excavations on the order of 5 feet in height are anticipated during grading operations and construction of the foundation excavations. The excavations are expected to expose fill and alluvial soils, which may be subject to caving. Due to the potential for cobbles, the contractor should be prepared for difficult excavation conditions. Vertical excavations up to 5 feet in height may be attempted where not surcharged; however, the contractor should be prepared for caving, sloughing, and raveling in open excavations. Due to the granular nature of soils and potential for caving, the contractor should also be prepared to form foundation excavations at the excavation bottom.

7.14.2 Vertical excavations greater than 5 feet will require sloping and/or shoring measures in order to provide a stable excavation. Where sufficient space is available, temporary unsurcharged slopes could be sloped back at a uniform 1:1 slope gradient or flatter, up to a maximum of 10 feet in height. A uniform slope does not have a vertical portion.

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

7.15 Stormwater Infiltration

7.15.1 During the February 4, 2019, site exploration, boring B3 was utilized to perform percolation testing. The boring was advanced to the depth listed in the table below. Slotted casing was placed in the 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 February 5, 2019, the casing was refilled with water and percolation test readings were performed after repeated flooding of the cased excavation. Based on the test results, the measured percolation rate and design infiltration rate, for the earth materials encountered, are provided in the following table. 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 San Bernardino *Technical Guidance Document for Water Quality Management Plans* (June 2013). Additional correction factors may be required and should be applied by the engineer in responsible charge of the design of the stormwater infiltration system and based on applicable guidelines. The percolation test data sheet is provided as Figure 5.

Boring	Soil Type	Infiltration Depth (ft)	Average Infiltration Rate (in / hour)
B3	Sand (SP)	5-10	10.1

7.15.2 The results of the percolation testing indicate that soils at the location and depths listed in the table above are conductive to infiltration, and it is our opinion that the site is suitable for infiltration of stormwater at the location tested above. Due to the presence of hydro-collapsible soils located in boring B2, infiltration should not be conducted near this boring. If infiltration is planned for any location other than where the above testing was performed, additional onsite and laboratory testing may be required.

7.15.3 It is our further opinion that infiltration of stormwater and will not induce excessive hydro-consolidation at the location of percolation testing (see Figure B5), will not create a perched groundwater condition, will not affect soil structure interaction of existing or proposed foundations due to expansive soils, will not saturate soils supported by existing retaining walls, and will not increase the potential for liquefaction. Resulting settlements are anticipated to be less than ¼ inch, if any. If infiltration is planned for any location other than where the above testing was performed, additional onsite and laboratory testing may be required.

7.15.4 The infiltration system must be located such that the closest distance between an adjacent foundation is at least 10 feet in all directions from the zone of saturation. The zone of saturation may be assumed to project downward from the discharge of the infiltration facility at a gradient of 1:1. Additional property line or foundation setbacks may be required by the governing jurisdiction and should be incorporated into the stormwater infiltration system design as necessary.

7.15.5 Subsequent to the placement of the infiltration system, it is acceptable to backfill the resulting void space between the excavation sidewalls and the infiltration system with minimum two-sack slurry provided the slurry is not placed in the infiltration zone. It is recommended that pea gravel be utilized adjacent to the infiltration zone so communication of water to the soil is not hindered.

7.15.6 The design drawings should be reviewed and approved by the Geotechnical Engineer. The installation of the stormwater infiltration system should be observed and approved in writing by the Geotechnical Engineer (a representative of Geocon).

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 2016 CBC 1804.4 or other applicable standards. In addition, drainage should not be allowed to flow uncontrolled over any descending slope. Discharge from downspouts, roof drains and scuppers are not recommended onto unprotected soils within 5 feet of the building perimeter. Planters which are located adjacent to foundations should be sealed to prevent moisture intrusion into the soils providing foundation support. Landscape irrigation is not recommended within 5 feet of the building perimeter footings except when enclosed in protected planters.

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

LIMITATIONS AND UNIFORMITY OF CONDITIONS

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

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

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

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

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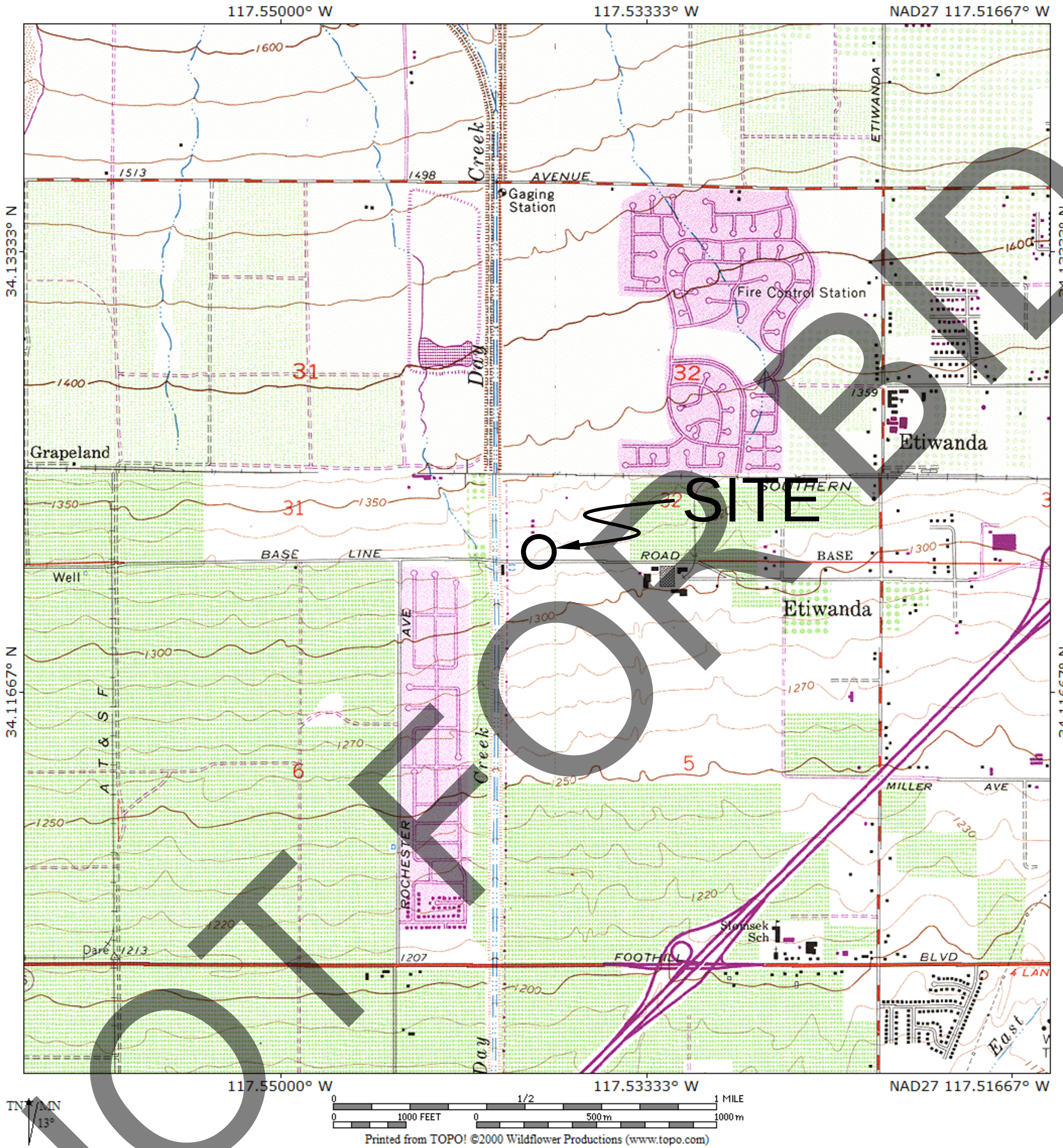

U.S.G.S. TOPOGRAPHIC MAPS, 7.5 MINUTE SERIES, GUASTI, CA QUADRANGLE

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VICINITY MAP

12158 BASELINE ROAD
RANCHO CUCAMONGA, CALIFORNIA

DRAFTED BY: CB	CHECKED BY: SFK

MARCH 2019	PROJECT NO. A9816-99-01	FIG. 1

LEGEND

B5 Approximate Location of Boring

Approximate Limits of Proposed Improvements

Approximate Location of Proposed Structure

0 80' 160'

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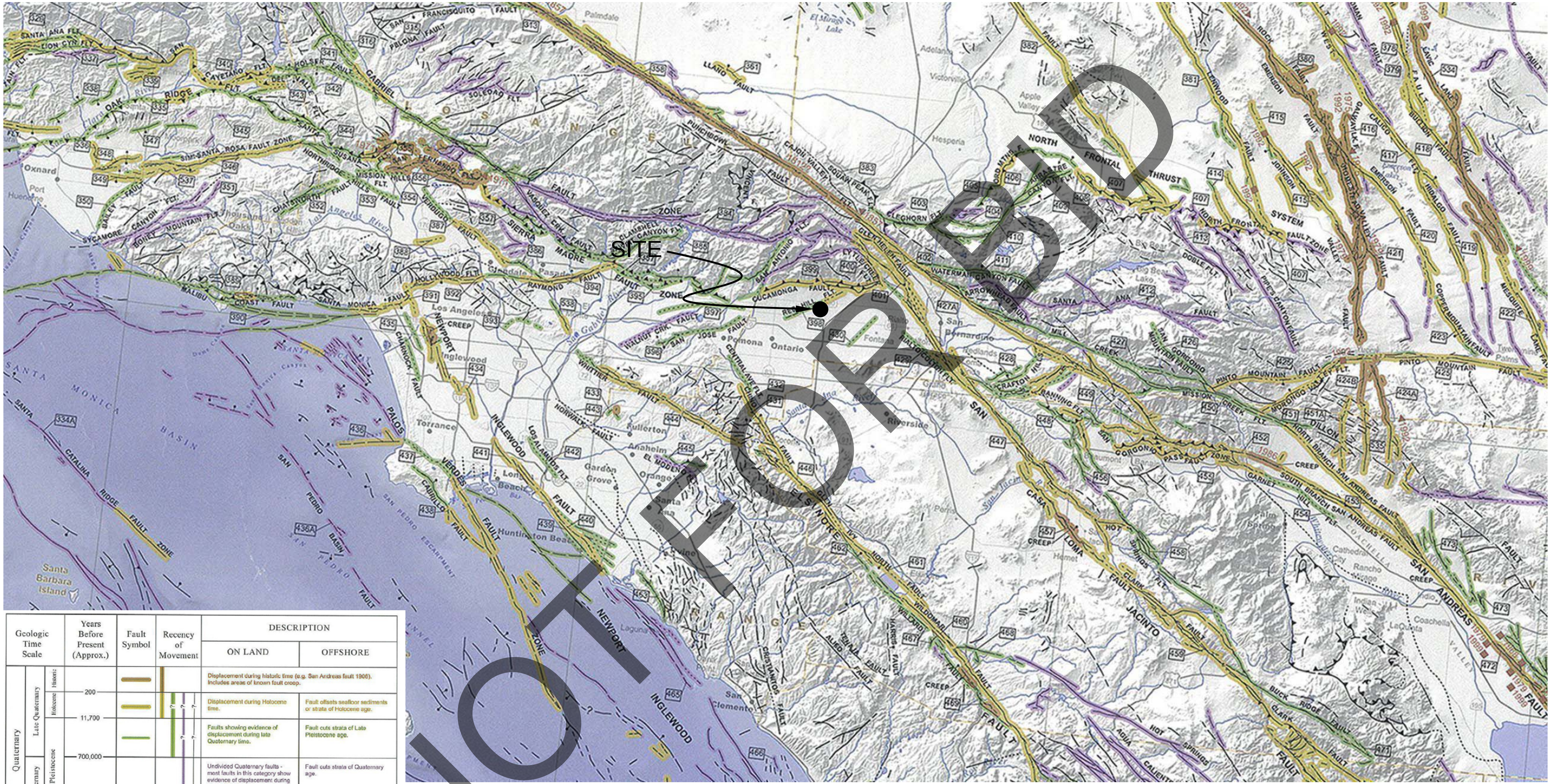
DRAFTED BY: JS | CHECKED BY: JTA

SITE PLAN

12158 BASELINE ROAD
RANCHO CUCAMONGA, CALIFORNIA

MARCH 2019 | PROJECT NO. A9816-99-01 | FIG. 2

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



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

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

0 12 24 Miles

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REGIONAL FAULT MAP

12158 BASELINE ROAD
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MARCH 2019 | PROJECT NO. A9816-99-01 | FIG. 3

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

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REGIONAL SEISMICITY MAP

12158 BASELINE ROAD
RANCHO CUCAMONGA, CALIFORNIA

MARCH 2019 | PROJECT NO. A9916-99-01 | FIG.4

PERCOLATION TEST DATA SHEET

Project:	Rancho Public Works Yard	Project No:	A9816-99-01	Date:	2/5/2019
Test Hole No:	B3	Tested By:	JS		
Depth of Test Hole, D_T:	10	USCS Soil Classification:	SP with gravel		

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

Sandy Soil Criteria Test*

Trial No.	Start Time	Stop Time	Δt Time Interval (min)	D_0 Initial Depth to Water (in)	D_f Final Depth to Water (in)	ΔD Change in Water Level (in)	Greater than or Equal to 6"? (y/n)
1	8:09	8:34	25	71.4	111.4	40.0	yes
2	8:38	9:03	25	63.6	109.6	46.0	yes

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

Trial No.	Start Time	Stop Time	Δt Time Interval (min)	D_0 Initial Depth to Water (in)	D_f Final Depth to Water (in)	ΔD Change in Water Level (in)	Percolation Rate (min/in)
1	9:05	9:15	10	56.8	99.0	42.2	341
2	9:17	9:27	10	64.6	99.8	35.3	408
3	9:30	9:40	10	64.0	99.4	35.4	407
4	9:42	9:52	10	65.4	98.9	33.5	430
5	9:55	10:05	10	63.2	98.9	35.6	404
6	10:07	10:17	10	63.2	98.4	35.2	410

Infiltration Rate Calculation:

Time Interval, Δt =	10	minutes	Ho =	54.6	inches
Final Depth to Water, Df =	98.9	inches	Hf =	21.1	inches
Test Hole Radius, r =	4	inches	ΔH =	33.5	inches
Initial Depth to Water, Do =	65.4	inches	Havg =	37.9	inches
Total Depth of Test Hole, DT =	120.0	inches			

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

Infiltration Rate, It = **10.1** inches/hour

FIGURE 5

APPENDIX A

APPENDIX A

FIELD INVESTIGATION

The site was explored on February 4, 2019, by excavating five 8-inch diameter borings to depths between 5½ and 25½ feet below the existing ground surface utilizing a truck-mounted hollow-stem auger drilling machine. Representative and relatively undisturbed samples were obtained by driving a 3-inch O. D., California Modified Sampler into the "undisturbed" soil mass with blows from a 140-pound auto-hammer falling 30 inches. The California Modified Sampler was equipped with 1-inch high by 2³/₈-inch diameter brass rings to facilitate soil removal and testing. Bulk samples were also obtained.

The soil conditions encountered in the borings were visually examined, classified and logged in general accordance with the Unified Soil Classification System (USCS). The logs of the borings are presented on Figures A1 though A5. 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.

PROJECT NO. A9816-99-01

BORING 1

ELEV. (MSL.) **--** DATE COMPLETED **02/04/2019**

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

DEPTH IN FEET	SAMPLE NO.	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT)*	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0	BULK 0-5'			**UNPAVED, SAND WITH FINE TO COARSE GRAVEL**			
				ARTIFICIAL FILL Sand, poorly graded, medium dense, moist, brown, fine- to medium-grained, trace fine to medium gravel.			
2	B1@2.5'		SP	**ALLUVIUM** Poorly Graded Sand with Gravel, medium dense, brown, moist, medium- to coarse-grained, fine to medium gravel.	48	124.5	3.9
4	B1@5'				26	115.5	4.2
6	BULK 7-11'						
8	B1@7.5'			- light brown	41	116.0	2.0
			GW	Gravel with Sand, dense, slightly moist, light brown, fine- to medium-grained, fine to coarse gravel.			
10	B1@10'				55	105.5	1.3
12	B1@12.5'			- yellowish brown, coarse gravel, cobble fragments	68	124.0	3.2
14	B1@15'			- very dense, light brown, coarse-grained	50(6")	124.2	2.4
16							
18							
20	B1@20'			- no recovery	50(2")		

Total depth of boring: 20.5 feet.
Fill to 1.5 feet.
No groundwater encountered.
Backfilled with soil cuttings.
Penetration resistance for 140-pound hammer falling 30 inches by auto-hammer.

A9816-99-01 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.

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PROJECT NO. A9816-99-01

BORING 2

ELEV. (MSL.) -- DATE COMPLETED **02/04/2019**

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

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT)*	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0	BULK 0-5'				**GRAVEL, FINE TO COARSE GRAINED, SOME COBBLES** **ARTIFICIAL FILL** Sand, poorly graded, medium dense slighty moist, brown, fine- to medium-grained, fine to medium gravel.			
2								
	B2@3'				- loose	7	98.1	6.6
4				SM	**ALLUVIUM** Silty Sand, medium dense, slightly moist to moist, brown, fine-grained.			
6	B2@6'					36	123.6	4.5
					Poorly Graded Sand and Gravel, dense, slightly moist, light brown, coarse-grained sand, fine gravel, cobble fragment.			
8								
	B2@9'					67	125.8	3.4
10								
12	B2@12'			SP	- very dense, some cobble fragments	50(6")		0.9
14								
	B2@15'					50(4")		1.1
16								
	B2@17'				- no recovery	50(3")		
18								
20	B2@20'			SP	Sand, poorly graded, medium dense, moist, brown, fine-grained, fine to coarse gravel, some clay.	32	110.6	16.3

Total depth of boring: 20.5 feet.
Fill to 4 feet.
No groundwater encountered.
Backfilled with soil cuttings.
Penetration resistance for 140-pound hammer falling 30 inches by auto-hammer.

A9816-99-01 BORING LOGS.GPJ

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

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

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

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PROJECT NO. A9816-99-01

BORING 3

ELEV. (MSL.) -- DATE COMPLETED **02/04/2019**

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

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT)*	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0	BULK 0-5'				**GRAVEL, COARSE GRAINED, COBBLES** **ARTIFICIAL FILL** Silty Sand, medium dense, dry, light brown, fine-grained, trace fine gravel.			
2	B3@2.5'			SW	**ALLUVIUM** Well-Graded Sand with Gravel, medium dense, dry to slightly moist, light brown, well-graded sand, fine gravel.	27	111.1	1.2
4	B3@5'			SP	Sand, poorly graded, medium dense, dry to slightly moist, light brown, fine- to medium-grained, some fine gravel.	45	107.9	3.2
10	B3@10'				- very dense, some cobble fragments	50(4")		0.5
15	B3@15'					50(5")		0.3
20	B3@20'				- dense, moist, dark yellowish brown, medium- to coarse-grained	79	121.6	5.7
24				SC	Clayey Sand, dense, moist, dark yellowish brown, fine-grained.			
	B3@25'			SP	Sand, poorly graded, very dense, moist, light brown, coarse-grained.	50(4")	112.2	10.0

Total depth of boring: 25.5 feet.
Fill to 1.5 feet.
No groundwater encountered.
Percolation testing performed.
Backfilled with soil cuttings.
Penetration resistance for 140-pound hammer falling 30 inches by auto-hammer.

A9816-99-01 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.

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BORING 4

ELEV. (MSL.) -- DATE COMPLETED 02/04/2019

EQUIPMENT HOLLOW STEM AUGER BY: JS

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT)*	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0	BULK 0-5'				**2" AC**			
					ARTIFICIALL FILL Silty Sand, medium dense, moist, brown, fine-grained.			
2					**ALLUVIUM**			
	B4@2.5'			SP	Poorly Graded Sand with Gravel, dense, slightly moist, brown, coarse-grained, fine to coarse gravel, trace cobble fragments.	72	105.5	10.5
4								
	B4@5'				- medium- to coarse-grained	57		2.1

Total depth of boring: 5.5 feet.
Fill to 1.5 feet.
No groundwater encountered.
Backfilled with soil cuttings.
Surface restored.
Penetration resistance for 140-pound hammer falling 30 inches by auto-hammer.

A9816-99-01 BORING LOGS.GPJ

Figure A4,
Log of Boring 4, 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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DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	**BORING 5** ELEV. (MSL.) **--** DATE COMPLETED **02/04/2019** EQUIPMENT **HOLLOW STEM AUGER** BY: **JS**	PENETRATION RESISTANCE (BLOWS/FT)*	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
					MATERIAL DESCRIPTION			
0	BULK 0-5'				**COARSE GRAVEL, COBBLES** **ARTIFICIAL FILL** Silty Sand, medium dense, moist, brown, fine- to medium-grained, some fine gravel.			
2	B5@2.5'			SP	**ALLUVIUM** Poorly Graded Sand with Gravel, dense, moist, dark brown, medium- to coarse-grained, fine gravel.	72	110.8	8.2
4	B5@5'			SP	Sand, poorly graded, medium dense, slightly moist, brown, fine- to medium-grained, some fine gravel.	37	115.5	3.2

Total depth of boring: 5.5 feet.
Fill to 1.75 feet.
No groundwater encountered.
Backilled with soil cuttings.
Penetration resistance for 140-pound hammer falling 30 inches by auto-hammer.

A9816-99-01 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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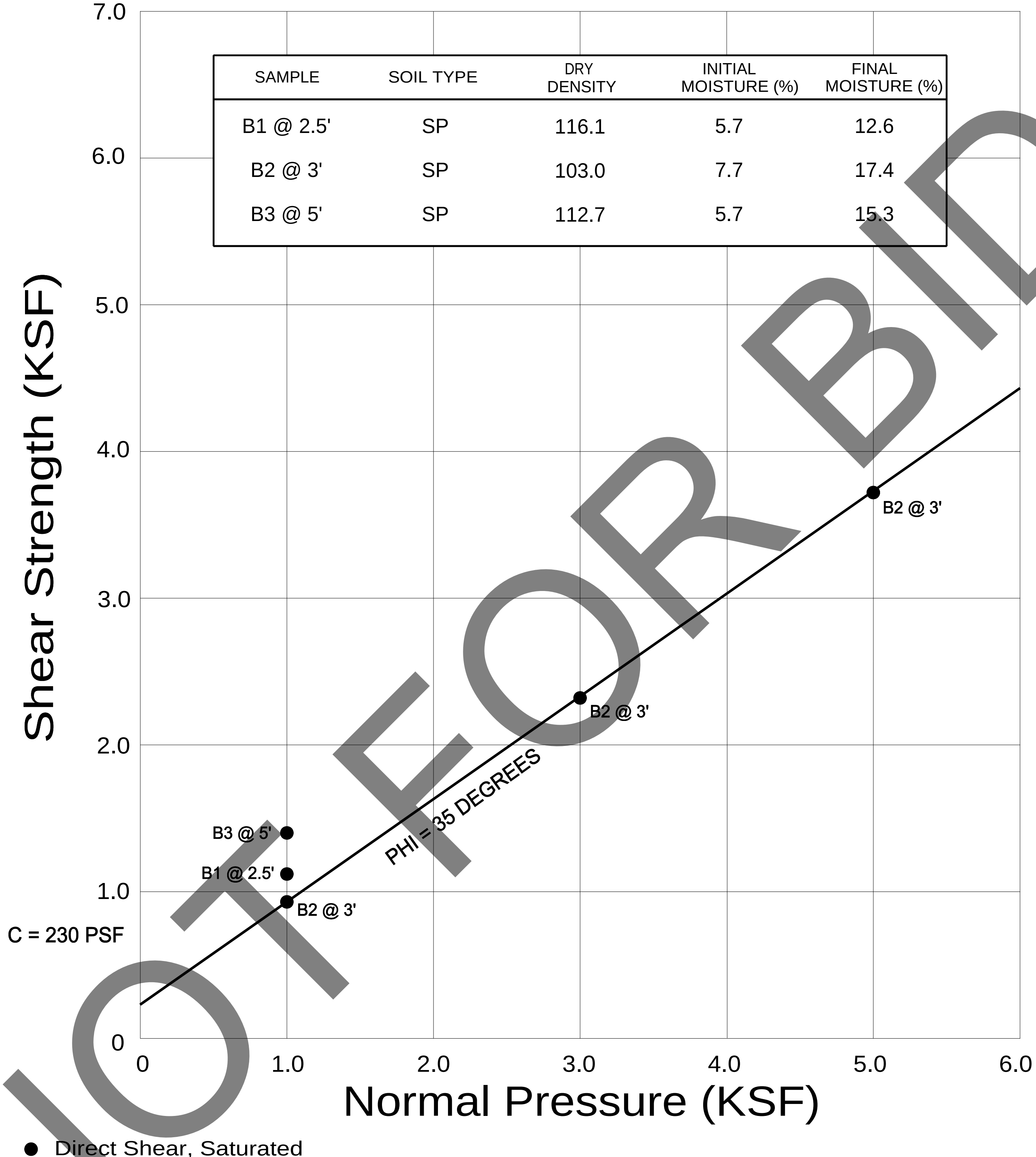
APPENDIX B

APPENDIX B

LABORATORY TESTING

Laboratory tests were performed in accordance with generally accepted test methods of the "American Society for Testing and Materials (ASTM)", or other suggested procedures. Selected samples were tested for direct shear strength, moisture density relationship, corrosivity and in-place dry density and moisture content. The results of the laboratory tests are summarized in Figures B1 through B7. The in-place dry density and moisture content of the samples tested are presented on the boring logs, Appendix A.



SAMPLE	SOIL TYPE	DRY DENSITY	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B1 @ 2.5'	SP	116.1	5.7	12.6
B2 @ 3'	SP	103.0	7.7	17.4
B3 @ 5'	SP	112.7	5.7	15.3

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DIRECT SHEAR TEST RESULTS

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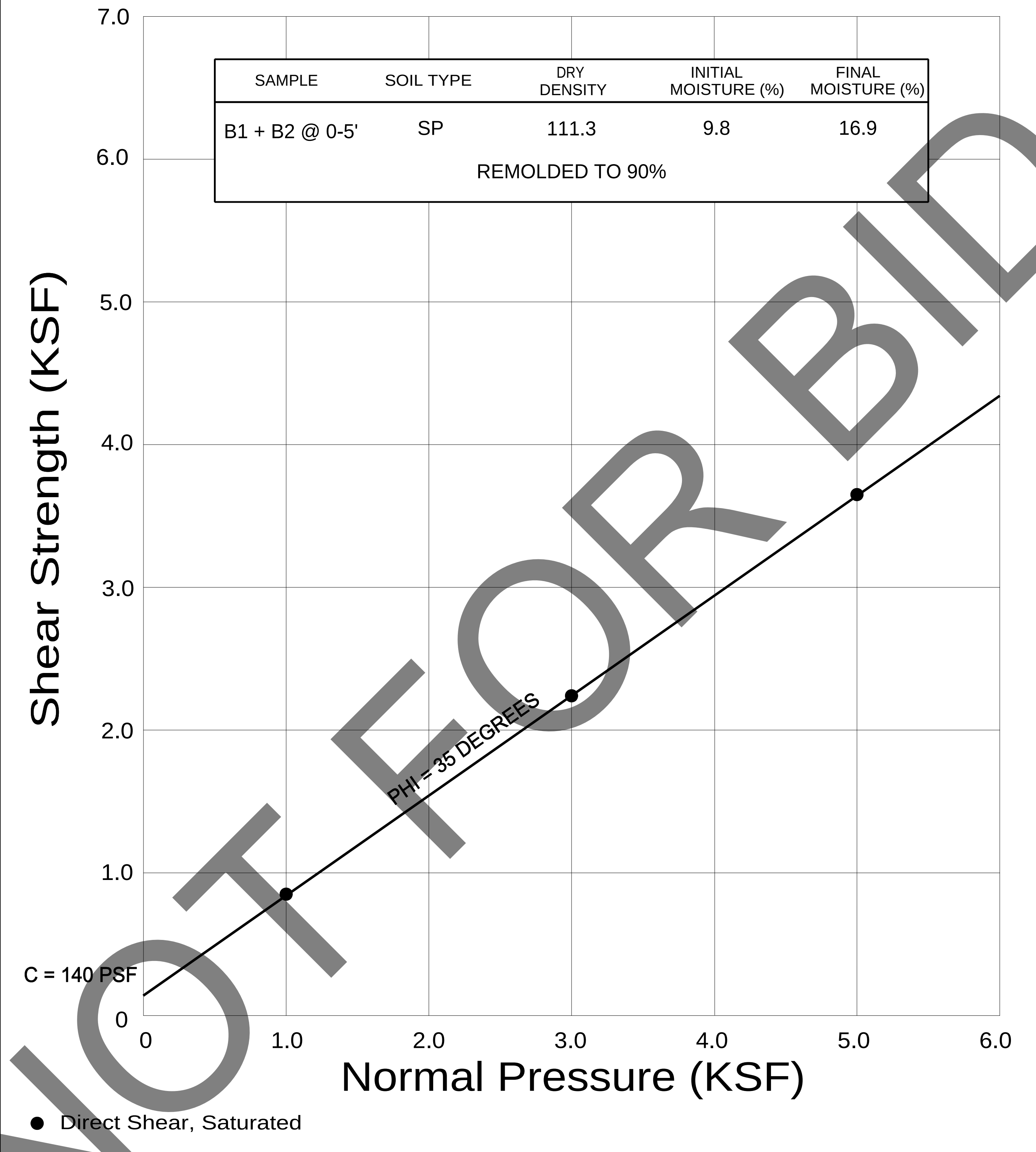
MARCH 2019 | PROJECT NO. A9816-99-01 | FIG. B1



SAMPLE	SOIL TYPE	DRY DENSITY	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B1 + B2 @ 0-5'	SP	111.3	9.8	16.9
REMOLDED TO 90%				

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DIRECT SHEAR TEST RESULTS

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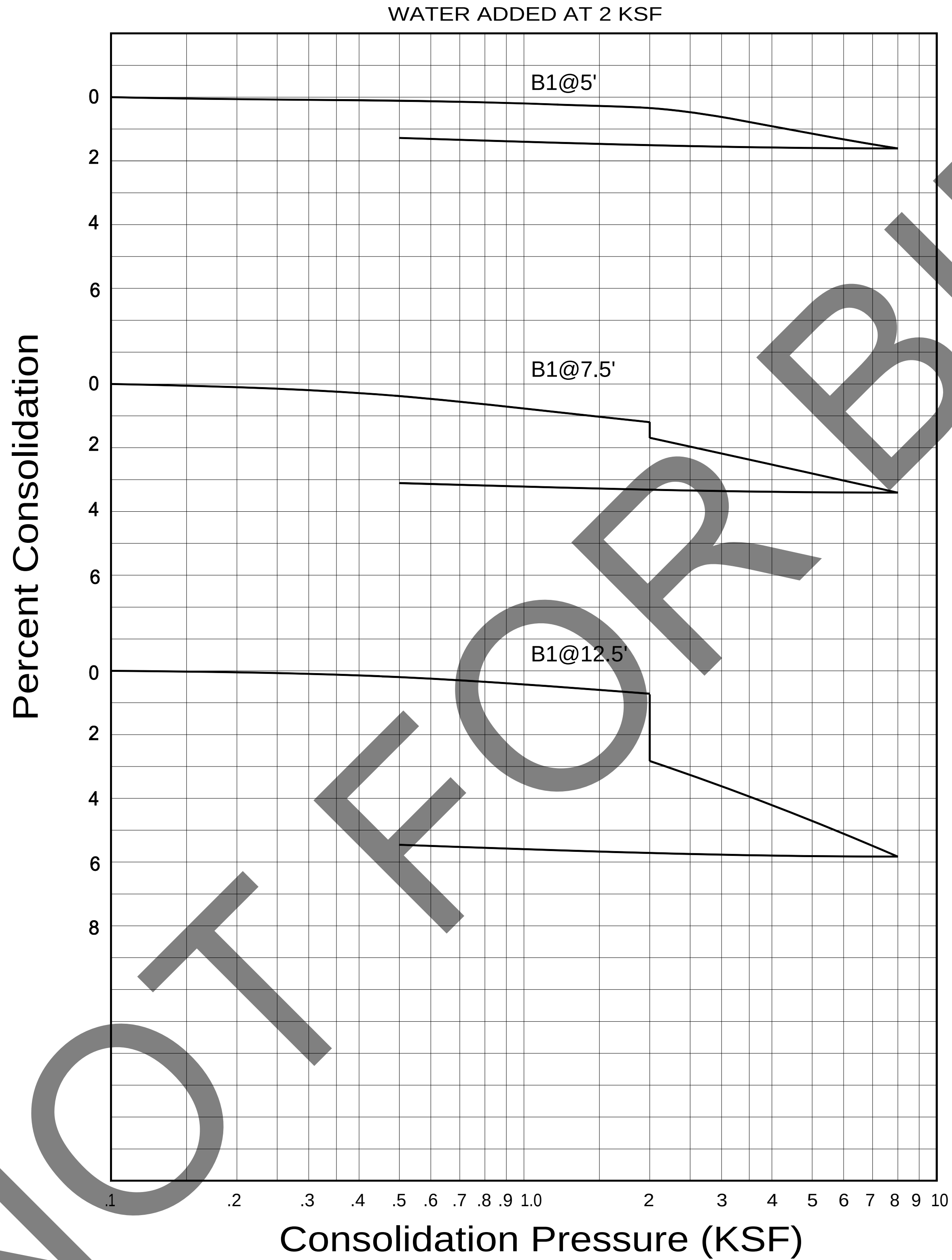

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CONSOLIDATION TEST RESULTS

12158 BASELINE ROAD
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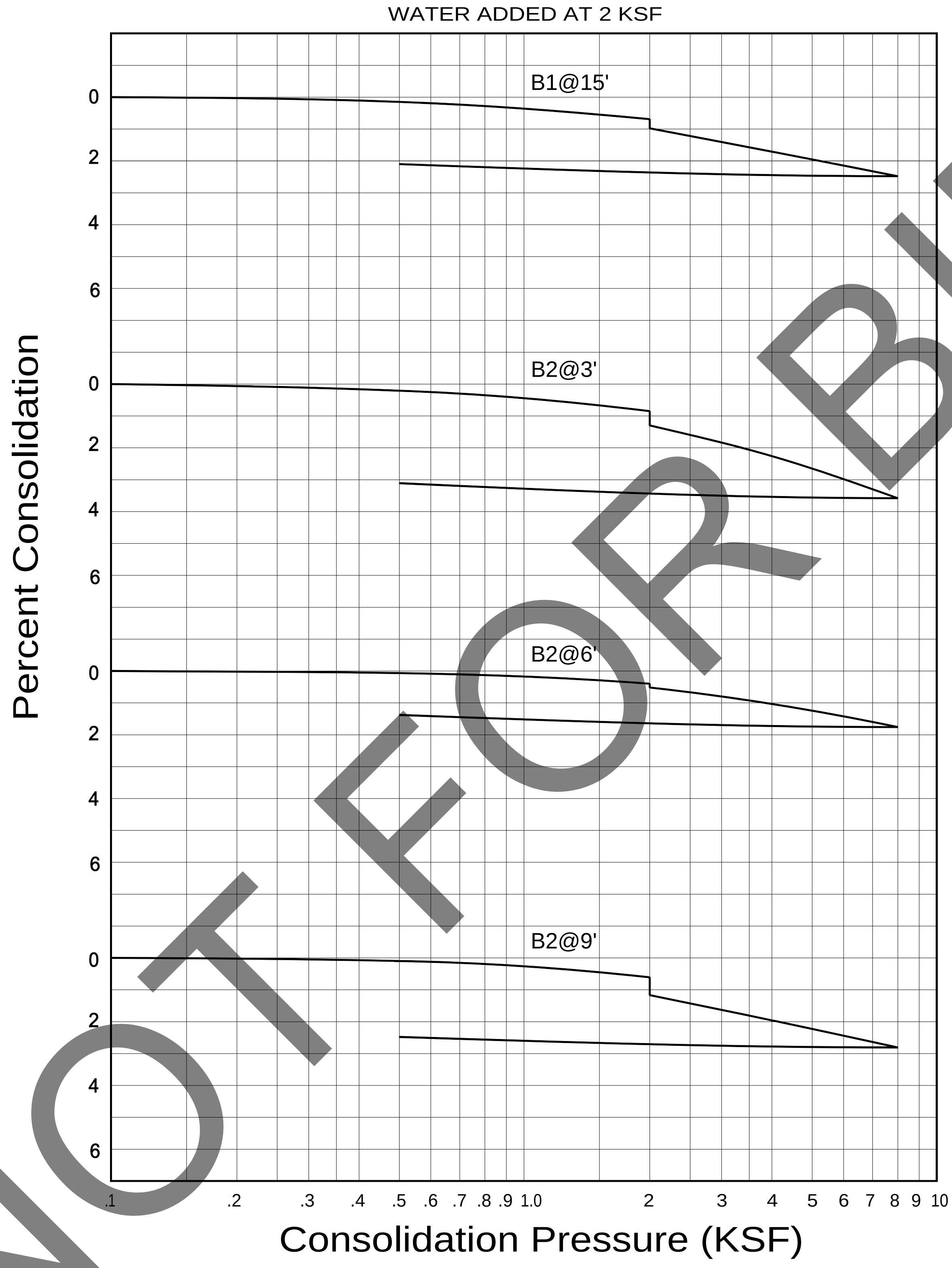
MARCH 2019	PROJECT NO. A9816-99-01	FIG. B3



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CONSOLIDATION TEST RESULTS

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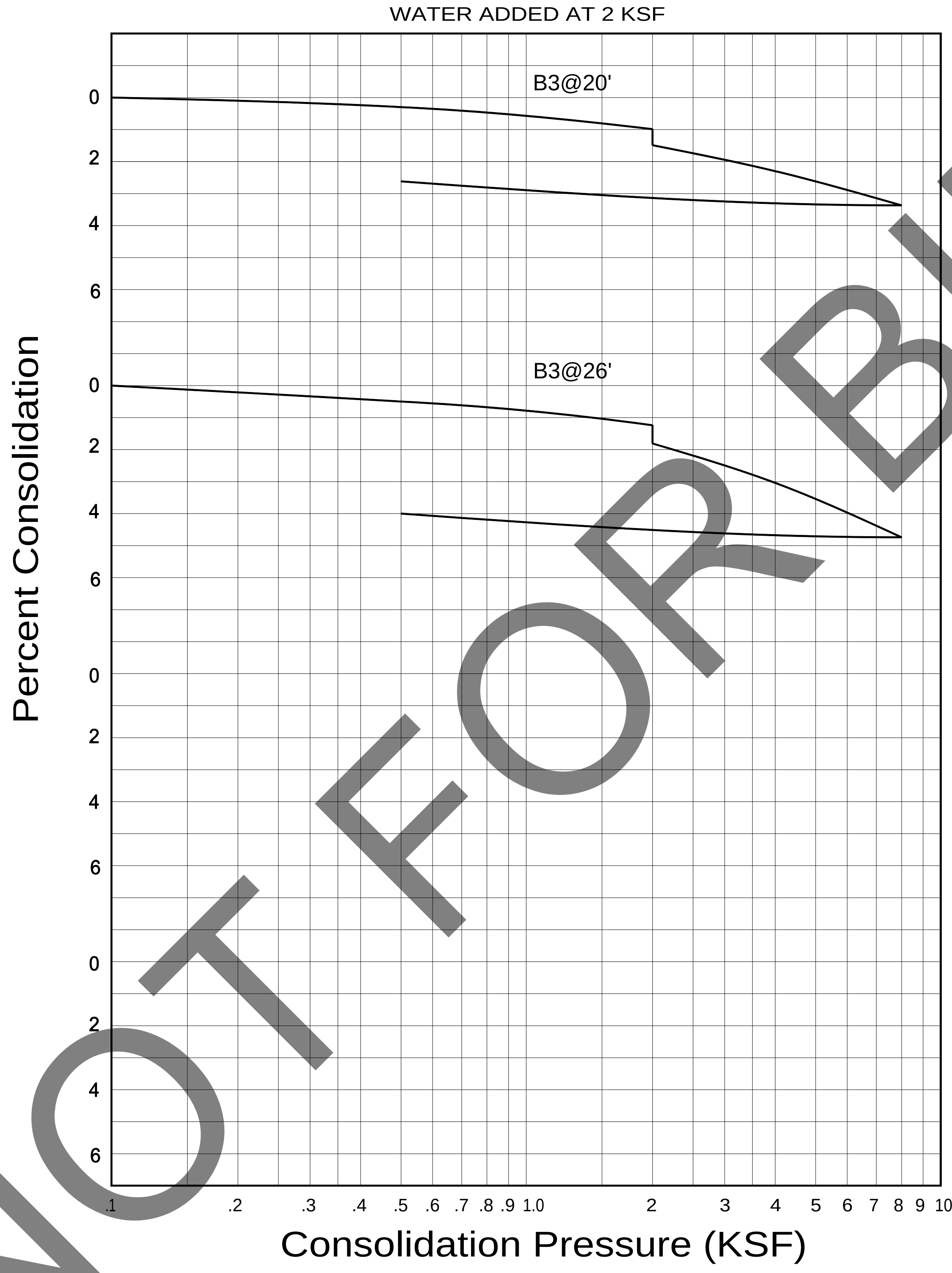

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CONSOLIDATION TEST RESULTS

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MARCH 2019	PROJECT NO. A9816-99-01	FIG. B5

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

Sample No.	Moisture Content (%)		Dry Density (pcf)	Expansion Index	*UBC Classification	**CBC Classification
	Before	After				
B1 & B2 MIX @ 0-5'	7.6	12.7	118.2	0	Very Low	Non-Expansive

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

** Reference: 2016 California Building Code, Section 1803.5.3

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

Sample No.	Soil Description	Maximum Dry Density (pcf)	Optimum Moisture (%)
B1 & B2 MIX @ 0-5'	Olive Brown Poorly Graded Sand with Silt & Gravel	132.5	6.0

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LABORATORY TEST RESULTS

12158 BASELINE ROAD
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SUMMARY OF LABORATORY POTENTIAL OF HYDROGEN (pH) AND RESISTIVITY TEST RESULTS CALIFORNIA TEST NO. 643

Sample No.	pH	Resistivity (ohm centimeters)
B1 & B2 MIX @ 0-5'	7.84	13,000 (Mildly Corrosive)

SUMMARY OF LABORATORY CHLORIDE CONTENT TEST RESULTS EPA NO. 325.3

Sample No.	Chloride Ion Content (%)
B1 & B2 MIX @ 0-5'	0.006

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

Sample No.	Water Soluble Sulfate (% SO_4)	Sulfate Exposure*
B1 & B2 MIX @ 0-5'	0.002	S0

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

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CORROSIVITY TEST RESULTS

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MARCH 2019 | PROJECT NO. A9816-99-01 | FIG. B7