GEOTECHNICAL INVESTIGATION

ELLIS AVENUE CONDOS
PROPOSED MULTI-FAMILY
RESIDENTIAL DEVELOPMENT
8041 ELLIS AVENUE
HUNTINGTON BEACH, CALIFORNIA
APN: 157-341-04, -07 & -08

GEOCON
WEST, INC.

GEOTECHNICAL
ENVIRONMENTAL
MATERIALS

PREPARED FOR

THDT INVESTMENTS
CORONA, CALIFORNIA

PROJECT NO. A9764-88-01

MAY 11, 2018

GEOCON
WEST, INC.
GEOTECHNICAL ■ ENVIRONMENTAL ■ MATERIALS

Project No. A9764-88-01
May 11, 2018

Tahir Salim
THDT Investments
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SUBJECT: GEOTECHNICAL INVESTIGATION
ELLIS AVENUE CONDOS
PROPOSED MULTI-FAMILY RESIDENTIAL DEVELOPMENT
8041 ELLIS AVENUE
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Dear Mr. Salim:

In accordance with your authorization of our proposal dated March 27, 2018, we have prepared this geotechnical investigation report for the proposed multi-family residential development at 8041 Ellis Avenue in the City of Huntington Beach, 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.

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

(PDF) Addressee

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GEOTECHNICAL INVESTIGATION

1. PURPOSE AND SCOPE

This report presents the results of a geotechnical investigation for the multi-family residential development at 8041 Ellis Avenue in the City of Huntington Beach, California (see Vicinity Map, Figure 1). The purpose of the investigation was to evaluate subsurface soil and geologic conditions underlying the site and based on conditions encountered to provide conclusions and recommendations pertaining to the geotechnical aspects of design and construction.

The scope of this investigation included a site reconnaissance, field exploration, laboratory testing, engineering analysis, and the preparation of this report. The site was explored on April 10, 2018, by excavating three 8-inch diameter borings utilizing a truck-mounted hollow-stem auger drilling machine. The borings were advanced to depths between approximately 40½ to 60½ 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 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 CONDITIONS & PROJECT DESCRIPTION

The subject site is located at 8041 Ellis Avenue in the City of Huntington Beach, California. The site is currently occupied by a car wash, a commercial store, and a single-family residence. The property is bounded by a three-story motel to the north, single-story residential structures to the east, Ellis Avenue to the south, and Beach Boulevard to the west. Surface water drainage at the site appears to be by sheet flow along the existing ground contours to the city streets. Vegetation onsite consists of shrubs and trees, which are located in isolated planter areas.

Based on the information provided by the Client, it is our understanding that the proposed development consists of a 51-unit multi-family residential complex. The facility will be three stories above grade and underlain by two levels of subterranean parking with a partial third level of subterranean parking. The proposed development is depicted on the Site Plan and Cross Sections (see Figures 2 and 3). It is assumed that the proposed subterranean parking levels and partial third level of subterranean parking will extend approximately 25 and 35 feet, respectively, below the existing ground surface including foundation depths.

Due to the preliminary nature of the design at this time, wall and column loads were not made available. It is estimated that wall loads for the proposed structure may be up to 6 kips per linear foot, and column loads may be up to 600 kips.

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

The southeastern portion of the Newport-Inglewood Fault Zone is marked by three coastal mesas: Bolsa Chico Mesa, Huntington Beach Mesa and Newport Mesa. The site is located in the northern portion of Huntington Beach Mesa and is underlain by Pleistocene age sediments (California Division of Mines and Geology, 2012).

4. GEOLOGIC MATERIALS

Based on our field investigation and published geologic maps of the area, the soils underlying the site consist of undocumented artificial fill material over Pleistocene age Old Paralic Deposits consisting of interbedded sand, silt, and clay (California Geological Survey, 2012). Detailed stratigraphic profiles are provided on the boring logs in Appendix A.

4.1 Artificial Fill

Artificial fill was encountered in our borings to a depth of approximately 5 feet beneath the existing ground surface. The fill materials generally consist of brown to dark brown, fine-grained sandy clay. The artificial fill is characterized as slightly moist and firm. The fill is likely 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 Old Paralic Deposits

The artificial fill is underlain by Pleistocene age Old Paralic Deposits consisting primarily of interbedded light to dark brown, gray to dark gray, and yellowish brown clay, sandy clay, sandy silt, silty sand, sand with silt and poorly graded sand. The soils are generally slightly moist to moist and firm to stiff or loose to very dense.

5. GROUNDWATER

According to the California Division of Mines and Geology (CDMG, 2001), the historic high groundwater level beneath the site is at a depth greater than 30 feet below the existing ground surface. The historic high groundwater level is based on available groundwater records from the early 1900's to 2000. Based on current groundwater basin management practices, it is unlikely that the groundwater levels will ever exceed the historic high levels.

Groundwater was encountered in boring B2 at a depth of approximately 56 feet beneath the existing ground surface. Based on the historic high groundwater levels in the site vicinity, the depth to groundwater encountered in our boring, and the depth of proposed construction, groundwater is neither expected to be encountered during construction, nor have a detrimental effect on the project. However, groundwater seepage may be encountered during construction. It is not uncommon for groundwater levels to vary seasonally or for groundwater seepage conditions to develop where none previously existed, especially in impermeable fine-grained soils which are heavily irrigated or after seasonal rainfall. In addition, recent requirements for stormwater infiltration could result in shallower seepage conditions in the immediate site vicinity. Proper surface drainage of irrigation and precipitation will be critical for future performance of the project. Recommendations for drainage are provided in the S*urface Drainage* section of this report (see Section 7.23).

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, 2018a). By definition, an active fault is one that has had surface displacement within Holocene time (about the last 11,700 years). A potentially active fault has demonstrated surface displacement during Quaternary time (approximately the last 1.6 million years), but has had no known Holocene movement. Faults that have not moved in the last 1.6 million years are considered inactive.

The site is not within a state-designated Alquist-Priolo Earthquake Fault Zone for surface fault rupture hazards (CGS, 2018b; CDMG, 1986). No active or potentially active faults with the potential for surface fault rupture are known to pass directly beneath the site. Therefore, the potential for surface rupture due to faulting occurring beneath the site during the design life of the proposed development is considered low. However, the site is located in the seismically active Southern California region, and could be subjected to moderate to strong ground shaking in the event of an earthquake on one of the many active Southern California faults. The faults in the vicinity of the site are shown in Figure 4, Regional Fault Map.

The closest near-surface trace of an active fault to the site is the Newport-Inglewood Fault Zone located approximately 1.2 miles to the southwest (Ziony and Jones, 1989; CDMG, 1986). Other nearby active faults include the Palos Verdes Hills Fault Zone, the Cabrillo Fault and the Whittier Fault located approximately 10.8 miles southwest, 13 miles west, and 19 miles northeast of the site, respectively (Ziony and Jones, 1989). The active San Andreas Fault Zone is located approximately 49 miles northeast of the site.

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

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 5, Regional Seismicity Map. A partial list of moderate to major magnitude earthquakes that have occurred in the Southern California area within the last 100 years is included in the following table.

LIST OF HISTORIC EARTHQUAKES

Earthquake (Oldest to Youngest)	Date of Earthquake	Magnitude	Distance to Epicenter (Miles)	Direction to Epicenter
San Jacinto-Hemet area	April 21, 1918	6.8	57	E
Near Redlands	July 23, 1923	6.3	47	ENE
Long Beach	March 10, 1933	6.4	5	SSE
Tehachapi	July 21, 1952	7.5	107	NW
San Fernando	February 9, 1971	6.6	55	NNW
Whittier Narrows	October 1, 1987	5.9	26	NNW
Sierra Madre	June 28, 1991	5.8	39	N
Landers	June 28, 1992	7.3	95	ENE
Big Bear	June 28, 1992	6.4	75	ENE
Northridge	January 17, 1994	6.7	48	NW

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 Parameters

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

2016 CBC SEISMIC DESIGN PARAMETERS

Parameter	Value	2016 CBC Reference
Site Class	D	Table 1613.3.2
MCE_R Ground Motion Spectral Response Acceleration – Class B (short), S_S	1.535g	Figure 1613.3.1(1)
MCE_R Ground Motion Spectral Response Acceleration – Class B (1 sec), S_1	0.574g	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}	1.535g	Section 1613.3.3 (Eqn 16-37)
Site Class Modified MCE_R Spectral Response Acceleration – (1 sec), S_{M1}	0.861g	Section 1613.3.3 (Eqn 16-38)
5% Damped Design Spectral Response Acceleration (short), S_{DS}	1.023g	Section 1613.3.4 (Eqn 16-39)
5% Damped Design Spectral Response Acceleration (1 sec), S_{D1}	0.574g	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	0.598g	Figure 22-7
Site Coefficient, F_{PGA}	1.0	Table 11.8-1
Site Class Modified MCE_G Peak Ground Acceleration, PGA_M	0.598g	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 6.74 magnitude event occurring at a hypocentral distance of 8.56 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 6.65 magnitude occurring at a hypocentral distance of 18.4 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.

A review of the State of California Seismic Hazard Zones Map for Newport Beach Quadrangle (CDMG, 1998) indicates that the site is not located in an area designated as "liquefiable." Additionally, the Orange County Safety Element (2004) and the Huntington Beach General Plan (1996) indicate the site is not located within an area identified as having a potential for liquefaction. As stated previously, the soils encountered during our exploration are Pleistocene age Old Paralic Deposits that are generally dense or stiff to hard and are not prone to liquefaction. Also, the groundwater has historically been at a depth of 30 feet or greater. Based on these considerations, it is our opinion that the site is not susceptible to liquefaction.

6.5 Slope Stability

The site is relatively level to gently sloping to the north and is not in an area identified as having a potential for slope instability (City of Huntington Beach, 1996). According to the State of California Seismic Hazard Zones Map, Newport Beach Quadrangle (CDMG, 1998), the site is not located within an area identified as having a potential for seismic 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 instability or landslides adversely affecting the proposed project 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. A review of the Orange County Safety Element (2004) indicates that the site is not located within the inundation boundaries of up-gradient dams or reservoirs. Therefore, the probability of earthquake-induced flooding is considered very low.

6.7 Tsunamis, Seiches and Flooding

The site is located approximately 2.5 miles from the Pacific Ocean at an elevation of approximately 55 to 60 feet MSL. The City of Huntington Beach (1996) and the State of California (2009) indicate the site is not located within a tsunami inundation area. Therefore, tsunamis, seismic sea waves, are not considered a significant hazard at the site.

Seiches are large waves generated in enclosed bodies of water in response to ground shaking. No major water-retaining structures are located immediately up gradient from the project site. Therefore, flooding 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, 2018).

6.8 Oil Fields & Methane

Based on a review of the California Division of Oil, Gas and Geothermal Resources (DOGGR) Well Finder Website, the site is located within the limits of the Huntington Beach Oil Field (DOGGR, 2018). There are no oil or gas wells identified within the site boundaries (DOGGR, 2018). However, there are two plugged oil/gas wells located within 500 feet of the site (Dragon Oil Company Well Number One-Jauman and Estate of Chas. W. Camp Well Number 4). 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.

According to the Environmental Hazard Element of the City of Huntington Beach General Plan (1996), the site is not located in a methane overlay district. 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. According to the Environmental Hazard Element of the City of Huntington Beach General Plan (1996), the site is located within an area of documented ground subsidence associated with petroleum and gas withdrawal in the Huntington Beach Oil Field. During the monitoring period of 1976 to 1986, approximately 1.8 inches of subsidence occurred in the general site vicinity related to oil field operations. Re-pressurization by water injection has been used to stabilize this vertical movement and in most areas subsidence has been arrested. However, there remains a potential for subsidence of the ground surface in the area, particularly if the water injection program ceases.

7. CONCLUSIONS AND RECOMMENDATIONS

7.1 General

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

7.1.2 Up to 5 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. The existing fill and site soils are suitable for re-use as engineered fill, if needed, provided the recommendations in the *Grading* section of this report are followed (see Section 7.5). Excavations for the proposed subterranean levels are expected to penetrate through the existing fill and expose competent old paralic deposits throughout the excavation bottom.

7.1.3 Based on these considerations, the proposed structure may be supported on a conventional foundation system deriving support in the competent old paralic deposits found at or below a depth of 20 feet. Foundations should be deepened as necessary to penetrate through soft or unsuitable soils at the direction of the Geotechnical Engineer. All foundation excavations must be observed and approved in writing by the Geotechnical Engineer (a representative of Geocon), prior to placing steel or concrete. Recommendations for the design of a conventional foundation system are provided in Section 7.6.

7.1.4 The subterranean portion of the structure which extends to depths greater than 30 feet below existing ground surface should be designed for full hydrostatic pressure. Alternatively, a permanent dewatering system may be implemented to relieve and mitigate the water pressure. The historic high groundwater may be assumed at a depth of 30 feet for design. Recommendations for permanent dewatering are discussed in Sections 7.4 of this report.

7.1.5 Based on the depth of proposed construction and potential hydrostatic pressures, the proposed structure may also be supported on a reinforced concrete mat foundation system. A mat foundation system could be a very cost-effective foundation system for this project since the pad can remain relatively flat which allows for more efficient construction of waterproofing, saving time and labor. If desired, recommendations for a reinforced concrete mat foundation system can be provided under separate cover.

7.1.6 Excavations up to 35 feet in vertical height are anticipated for construction of the subterranean levels, including foundation depths. Due to the depth of the excavation and the proximity to the property lines, city streets and adjacent offsite structures, excavation of the proposed subterranean levels will require sloping and shoring measures in order to provide a stable excavation. Where shoring is required, it is recommended that a soldier pile shoring system be utilized. In addition, where the proposed excavation will be deeper than and adjacent to an offsite structure, the proposed shoring should be designed to resist the surcharge imposed by the adjacent offsite structure. Recommendations for shoring are provided in Section 7.17 of this report.

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

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 old paralic deposits, 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. The design team and contractor should be aware that the depth to undisturbed old paralic deposits may be on the order of 5 feet; recommendations for the design and construction of miscellaneous foundations should be reevaluated once formal plans are available.

7.1.9 Based on the results of percolation testing performed at the site, a stormwater infiltration system is considered feasible for this project. Recommendations for infiltration are provided in the *Stormwater Infiltration* section of this report (see Section 7.22).

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

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

7.2 Soil and Excavation Characteristics

7.2.1 The in-situ soils can be excavated with moderate effort using conventional excavation equipment. Due to the granular nature of the soils, moderate to excessive caving is anticipated in unshored excavations. The contractor should be aware that casing may be required during shoring pile installation and formwork may be required to prevent caving of shallow spread foundation excavations.

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

7.2.3 All onsite excavations must be conducted in such a manner that potential surcharges from existing structures, construction equipment, and vehicle loads are resisted. The surcharge area may be defined by a 1:1 projection down and away from the bottom of an existing foundation or vehicle load. Penetrations below this 1:1 projection will require special excavation measures such as sloping and possibly shoring. Excavation recommendations are provided in the *Temporary Excavations* section of this report (see Section 7.16).

7.2.4 The existing site soils encountered during the field investigation near the ground surface are considered to have a "high" (EI = 105) expansive potential and are classified as "expansive" in accordance with the 2016 California Building Code (CBC) Section 1803.5.3. However, the proposed subterranean levels are expected to penetrate through these soils into material which is primarily granular in nature and are considered to be "non-expansive". The recommendations presented in this report assume that foundations and slabs will derive support in materials with a "non-expansive" potential.

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 "moderately corrosive" with respect to corrosion of buried ferrous metals on site. The results are presented in Appendix B (Figure B9) and should be considered for design of underground structures.

7.3.2 Laboratory tests were performed on representative samples of the 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 B9) and indicate that the on-site materials possess "negligible" sulfate exposure to concrete structures as defined by 2016 CBC Section 1904 and ACI 318-11 Sections 4.2 and 4.3.

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 Permanent Dewatering

7.4.1 If any portion of the proposed structure extends below the historic high groundwater depth and is not designed for full hydrostatic pressure and buoyancy, a permanent dewatering system will be required to relieve and mitigate the water pressure. If permanent dewatering is to be utilized, a sub-slab drainage system consisting of perforated pipes placed in gravel-filled trenches may be installed beneath the subterranean slab-on-grade to intercept and control groundwater. A separate retaining wall drainage system is also required around the perimeter of the structure. The sub-slab drainage system can be combined with the perimeter retaining wall drainage system provided backflow valves are installed at the base of the wall drainage system.

7.4.2 A typical permanent sub-slab drainage system would consist of a 12-inch thick layer of ¾-inch gravel that is placed upon a layer of filter fabric (Miami 500X or equivalent), and vibrated to a dense state. Subdrain pipes leading to sump areas, provided with automatic pumping units, should drain the gravel layer. The drain lines should consist of perforated pipe, placed with perforations down, in trenches that are at least 6 inches below the gravel layer. The excavation bottom, as well as the trench bottoms should be lined with filter fabric prior to placing and compacting gravel. The trenches should be spaced approximately

40 feet apart at most, within the interior, and should extend along to the perimeter of the building. Subsequent to the installation of the drainage system, the waterproofing system and building slab may then be placed on the densified gravel. A mud- or rat-slab may be placed below and over the waterproofing system for protection during placement of rebar and slab construction.

7.4.3 Recommendations for design flow rates for the permanent dewatering system should be determined by a qualified contractor or dewatering consultant.

7.5 Grading

7.5.1 Grading is anticipated to include excavation of site soils for the subterranean levels, foundations, and utility trenches, as well as placement of backfill for walls, ramps, and trenches.

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

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

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

7.5.5 The foundation system for the proposed structure may derive support in the competent undisturbed paralic deposits found at and below a depth of 20 feet.

7.5.6 All fill and backfill soils should be placed in horizontal loose layers approximately 6 to 8 inches thick, moisture conditioned, and properly compacted. If soils are granular and confirmed to be non-expansive by the geotechnical engineer, soils should be moisture conditioned to optimum moisture content. If soils are fine-grained or expansive, soils should be moisture conditioned to 2 to 3 percent above optimum moisture content. All fill shall be compacted to a minimum 90 percent of the maximum dry density per ASTM D 1557 (latest edition).

7.5.7 Prior to construction of exterior slabs, the upper 12 inches of the subgrade should be moisture conditioned to recommended moisture content and properly compacted to at least 92 percent relative compaction, as determined by ASTM Test Method D1557 (latest edition).

7.5.8 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 old paralic deposits, 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.

7.5.9 Although not anticipated for this project, all imported fill shall be observed, tested, and approved by Geocon West, Inc. prior to bringing soil to the site. Rocks larger than 6 inches in diameter shall not be used in the fill. If necessary, import soils used as structural fill should have an expansion index less than 20 and corrosivity properties that are equally or less detrimental to that of the existing onsite soils (see Figure B9).

7.5.10 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 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).

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

7.6 Foundation Design

7.6.1 The proposed structure may be supported on a conventional foundation system deriving support in the competent old paralic deposits found at and below a depth 20 feet. Foundations should be deepened as necessary to penetrate through soft or unsuitable soils at the direction of the Geotechnical Engineer. All foundation excavations must be observed and approved in writing by the Geotechnical Engineer (a representative of Geocon), prior to placing steel or concrete.

7.6.2 Continuous footings may be designed for an allowable bearing capacity of 3,000 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 3,500 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 6,000 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 exterior wall footings, this office should be provided a copy of the final construction plans so that the excavation recommendations presented herein could be properly reviewed and revised if necessary.

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 If the portion of the proposed structure which extends below the historic high groundwater table is to be designed for full hydrostatic pressure, the recommended floor slab uplift pressure to be used in design would be 62.4(H) in units of psf, where "H" is the height of the water above the bottom of the mat foundation in feet. If a permanent dewatering system is not implemented then the structure must be designed for hydrostatic pressure based on the historic high groundwater of 30 feet below ground surface.

7.6.11 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.

7.6.12 Waterproofing of subterranean walls and slabs is recommended for this project. 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.

7.6.13 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 6,000 psf is estimated to be less than ¾ 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 Miscellaneous Foundations

7.8.1 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 old paralic deposits, and should be deepened as necessary to maintain a minimum 12-inch embedment into the recommended bearing materials. The design team and contractor should be aware that the depth to undisturbed old paralic deposits may be on the order of 5 feet; recommendations for the design and construction of miscellaneous foundations should be reevaluated once formal plans are available.

7.8.2 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.

7.8.3 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.

7.9 Lateral Design

7.9.1 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 0.4 may be used with the dead load forces in the properly compacted engineered fill and competent old paralic deposits.

7.9.2 Passive earth pressure for the sides of foundations and slabs poured against properly compacted engineered fill or competent old paralic deposits may be computed as an equivalent fluid having a density of 250 pcf with a maximum earth pressure of 2,500 psf. When combining passive and friction for lateral resistance, the passive component should be reduced by one-third.

7.10 Concrete Slabs-on-Grade

7.10.1 Unless specifically evaluated and designed by a qualified structural engineer, the slab-on-grade subject to vehicle loading should be a minimum of 5 inches of concrete reinforced with No. 3 steel reinforcing bars placed 18 inches on center in both horizontal directions and positioned vertically near the slab midpoint. The concrete slab-on-grade and ramp may derive support directly on the undisturbed old paralic deposits at the excavation bottom as well as compacted soils, if necessary. Any artificial fill or disturbed soils should be properly compacted for slab support. Soil placed and compacted for ramp and slab support should be 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) for ramp support.

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

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

7.10.4 For seismic design purposes, a coefficient of friction of 0.4 may be utilized between concrete slabs and subgrade soils without a moisture barrier, and 0.15 for slabs underlain by a moisture barrier.

7.10.5 Exterior slabs for walkways or flatwork, not subject to traffic loads, should be at least 4 inches thick and reinforced with No. 3 steel reinforcing bars placed 18 inches on center in both horizontal directions, positioned near the slab midpoint. Prior to construction of slabs, the upper 12 inches of subgrade should be moistened to 2 to 3 percent above optimum moisture content and properly compacted to at least 92 percent relative compaction, as determined by ASTM Test Method D 1557 (latest edition). Crack control joints should be spaced at intervals not greater than 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.

7.10.6 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.

7.11 Retaining Wall Design

7.11.1 The recommendations presented below are generally applicable to the design of rigid concrete or masonry retaining walls having a maximum height of 35 feet. In the event that walls higher than 35 feet are planned, Geocon should be contacted for additional recommendations.

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

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

RETAINING WALL WITH LEVEL BACKFILL SURFACE

HEIGHT OF RETAINING WALL (Feet)	ACTIVE PRESSURE EQUIVALENT FLUID PRESSURE (Pounds Per Cubic Foot)	AT-REST PRESSURE EQUIVALENT FLUID PRESSURE (Pounds Per Cubic Foot)
Up to 25	40	60
Up to 35	42	62

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

7.11.5 The wall pressures provided above assume that the proposed retaining walls will support relatively undisturbed old paralic deposits. If sloping techniques are to be utilized for construction of proposed walls, which would result in a wedge of engineered fill behind the retaining walls, revised earth pressures may be required, especially if the wall backfill does not consist of the existing onsite soils. This should be evaluated once the use of sloping measures is established and once the geotechnical characteristics of the engineered backfill soils can be further evaluated.

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

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

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

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

and

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

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

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

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

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

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

and

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

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

then

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

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

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

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

7.12 Dynamic (Seismic) Lateral Forces

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

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

7.13 Retaining Wall Drainage

7.13.1 Retaining walls should be provided with a drainage system. At the base of the drain system, a subdrain covered with a minimum of 12 inches of gravel should be installed, and a compacted fill blanket or other seal placed at the surface (see Figure 6). 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.

7.13.2 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 7). 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.

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

7.13.4 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.

7.14 Elevator Pit Design

7.14.1 The elevator pit slab and retaining wall should be designed by the project structural engineer. Elevator pits may be designed in accordance with the recommendations in the *Foundation Design* and *Retaining Wall Design* section of this report (see Sections 7.6 and 7.11).

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

7.14.3 If retaining wall drainage is to be provided, the drainage system should be designed in accordance with the *Retaining Wall Drainage* section of this report (see Section 7.13).

7.14.4 It is suggested that the exterior walls and slab be waterproofed to prevent excessive moisture inside of the elevator pit. Waterproofing design and installation is not the responsibility of the geotechnical engineer.

7.15 Elevator Piston

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

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

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

7.16 Temporary Excavations

7.16.1 Excavations up to 35 feet in height are anticipated for excavation and construction of the proposed subterranean levels and foundation system. The excavations are expected to expose artificial fill and old paralic deposits, which are suitable for vertical excavations up to 5 feet where loose soils or caving sands are not present or where not surcharged by adjacent traffic or structures.

7.16.2 Vertical excavations greater than 5 feet will require sloping and/or shoring measures in order to provide a stable excavation. Where sufficient space is available, temporary unsurcharged embankments 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. Where space is limited, shoring measures will be required. *Shoring* data is provided in Section 7.17 of this report.

7.16.3 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.

7.17 Shoring – Soldier Pile Design and Installation

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

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

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

7.17.4 The proposed soldier piles may also be designed as permanent piles. The required pile depths, dimensions, and spacing should be determined and designed by the project structural and shoring engineers. All piles utilized for shoring can also be incorporated into a permanent retaining wall system (shotcrete wall) and should be designed in accordance with the earth pressure provided in the *Retaining Wall Design* section of this report (see Section 7.11).

7.17.5 Drilled cast-in-place soldier piles should be placed no closer than three diameters on center. The minimum diameter of the piles is 18 inches. Structural concrete should be used for the soldier piles below the excavation; lean-mix concrete may be employed above that level. As an alternative, lean-mix concrete may be used throughout the pile where the reinforcing consists of a wideflange section. The slurry must be of sufficient strength to impart the lateral bearing pressure developed by the wideflange section to the soil. For design purposes, an allowable passive value for the soils below the bottom plane of excavation may be assumed to be 250 psf per foot. Where piles are installed by vibration techniques, the passive pressure may be assumed to mobilize across a width equal to the two times the dimension of the beam flange. The allowable passive value may be doubled for isolated piles spaced a minimum of three times the pile diameter. To develop the full lateral value, provisions should be implemented to assure firm contact between the soldier piles and the undisturbed alluvium.

7.17.6 Groundwater was encountered during site exploration at a depth of 56 feet; however, groundwater levels can fluctuate and may be different at the time of construction. It is not uncommon for groundwater or seepage conditions to develop where none previously existed. Therefore the contractor should be prepared for groundwater during pile installation should the need arise. If more than 6 inches of water is present in the bottom of the excavation, a tremie is required to place the concrete into the bottom of the hole. A tremie should consist of a rigid, water-tight tube having a diameter of not less than 6 inches with a hopper at the top. The tube should be equipped with a device that will close the discharge end and prevent water from entering the tube while it is being charged with concrete. The tremie should be supported so as to permit free movement of the discharge end over the entire top surface of the work and to permit rapid lowering when necessary to retard or stop the flow of concrete. The discharge end should be closed at the start of the work to prevent water entering the tube and should be entirely sealed at all times, except when the concrete is being placed. The tremie tube should be kept full of concrete. The flow should be continuous until the work is completed and the resulting concrete seal should be monolithic and homogeneous. The tip of the tremie tube should always be kept about 5 feet below the surface of the concrete and definite steps and safeguards should be taken to insure that the tip of the tremie tube is never raised above the surface of the concrete.

7.17.7 A special concrete mix should be used for concrete to be placed below water. The design should provide for concrete with an unconfined compressive strength psi of 1,000 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 should be included. The slump should be commensurate to any research report for the admixture, provided that it should also be the minimum for a reasonable consistency for placing when water is present.

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

7.17.9 If a vibratory method of solider pile installation is utilized, predrilling may be performed prior to installation of the steel beams. If predrilling is performed, it is recommended that the bore diameter be at least 2 inches smaller than the largest dimension of the pile to prevent excessive loss in the frictional component of the pile capacity. Predrilling should not be conducted below the proposed excavation bottom.

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

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

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

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

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

7.17.15 The frictional resistance between the soldier piles and retained soil may be used to resist the vertical component of the anchor load. The coefficient of friction may be taken as 0.4 based on uniform contact between the steel beam and lean-mix concrete and retained earth. The portion of soldier piles below the plane of excavation may also be employed to resist the downward loads. The downward capacity may be determined using a frictional resistance of 520 psf per foot.

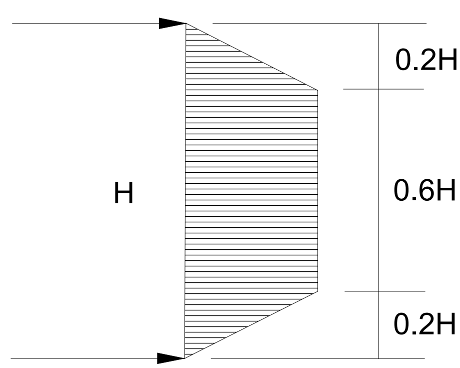
7.17.16 Due to the nature of the site soils, it is expected that continuous lagging between soldier piles will be required. However, it is recommended that the exposed soils be observed by the Geotechnical Engineer (a representative of Geocon West, Inc.), to verify the presence of any competent, cohesive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

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

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

HEIGHT OF SHORING (FEET)	EQUIVALENT FLUID PRESSURE (Pounds Per Cubic Foot) (ACTIVE PRESSURE)	EQUIVALENT FLUID PRESSURE Trapezoidal (Where H is the height of the shoring in feet)
Up to 25	31	19H
Up to 35	34	21H

Trapezoidal Distribution of Pressure



7.17.19 It is very important to note that active pressures can only be achieved when movement in the soil (earth wall) occurs. If movement in the soil is not acceptable, such as adjacent to an existing structure, an at-rest pressure of 51 and 54 pcf should be considered for the design of 25 foot and 35 foot high shoring, respectively.

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

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

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

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

and

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

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

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

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

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

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

and

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

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

then

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

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

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

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

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

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

7.18 Temporary Tie-Back Anchors

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

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

- 7 feet below the top of the excavation – 1,000 pounds per square foot
- 15 feet below the top of the excavation –1,400 pounds per square foot
- 25 feet below the top of the excavation –2,100 pounds per square foot

7.18.3 Depending on the techniques utilized, and the experience of the contractor performing the installation, a maximum allowable friction capacity of 3.5 kips per linear foot for post-grouted anchors (for a minimum 20 foot length beyond the active wedge) may be assumed for design purposes. Only the frictional resistance developed beyond the active wedge should be utilized in resisting lateral loads.

7.19 Anchor Installation

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

7.20 Anchor Testing

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

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

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

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

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

7.21 Internal Bracing

7.21.1 Rakers may be utilized to brace the soldier piles in lieu of tieback anchors. The raker bracing could be supported laterally by temporary concrete footings (deadmen) or by the permanent, interior footings. For design of such temporary footings or deadmen, poured with the bearing surface normal to rakers inclined at 45 degrees, a bearing value of 2,500 psf may be used, provided the shallowest point of the footing is at least 1 foot below the lowest adjacent grade. The structural engineer should review the shoring plans to determine if raker footings conflict with the structural foundation system. The client should be aware that the utilization of rakers could significantly impact the construction schedule due to their intrusion into the construction site and potential interference with equipment.

7.22 Stormwater Infiltration

7.22.1 During the April 10, 2018, 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 April 11, 2018, 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 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). 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. Percolation test results are provided on Figure 8.

Boring	Soil Type	Infiltration Depth (ft)	Average Infiltration Rate (in / hour)
B3	Silty Sand (SM)	35 – 40½	3.95

7.22.2 The results of the percolation testing indicate that the soils are conductive to infiltration. It is our opinion that the soil zones encountered at the depths and locations as listed in the table above are suitable for infiltration of stormwater.

7.22.3 It is our opinion that the introduction of stormwater at the depth and location indicated above will not induce excessive hydro-consolidation, 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 or proposed retaining walls, and will not increase the potential for liquefaction. Resulting settlements are anticipated to be less than ¼ inch, if any.

7.22.4 Where infiltration systems will be utilized, it is recommended that a minimum 10-foot horizontal and vertical setback be maintained from existing or proposed foundations. Additional setbacks may be required by the governing jurisdiction and should be incorporated into the stormwater infiltration system design as necessary.

7.22.5 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.

7.22.6 Due to the preliminary nature of the project at this time, the type of stormwater infiltration system and location of the stormwater infiltration systems has not yet been determined. The design drawings should be reviewed and approved by the Geotechnical Engineer. The installation of the stormwater infiltration system should be observed and approved by the Geotechnical Engineer (a representative of Geocon).

7.23 Surface Drainage

7.23.1 Proper surface drainage is critical to the future performance of the project. Uncontrolled infiltration of irrigation excess and storm runoff into the foundation supporting 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 in building areas should be maintained at all times.

7.23.2 All site drainage should be collected and transferred to the street 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. The proposed structure should be provided with roof gutters. Discharge from downspouts, roof drains and scuppers 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 engineered fill providing foundation support. Landscape irrigation is not recommended within 5 feet of the building perimeter footings except when enclosed in protected planters.

7.23.3 Positive site drainage should be provided away from structures, pavement, and the tops of slopes to swales or other controlled drainage structures.

7.24 Plan Review

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

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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Sprotte, E.C., et al., 1980, *Classification and Mapping of Quaternary Sedimentary Deposits for Purposes of Seismic Zonation, South Coastal Los Angeles Basin, Orange County, California*, California Division Of Mines and Geology Open File Report 80-19LA.

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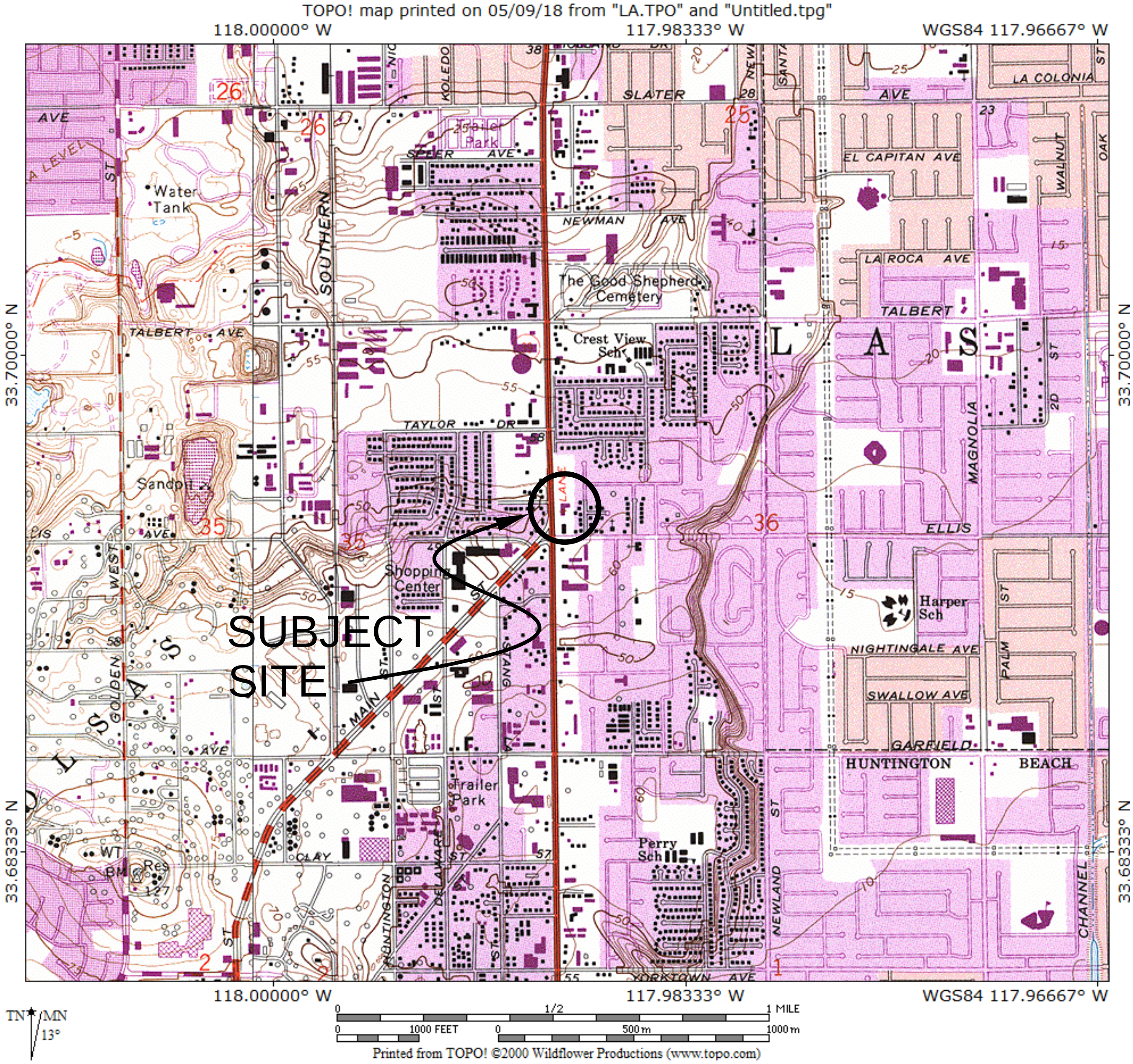
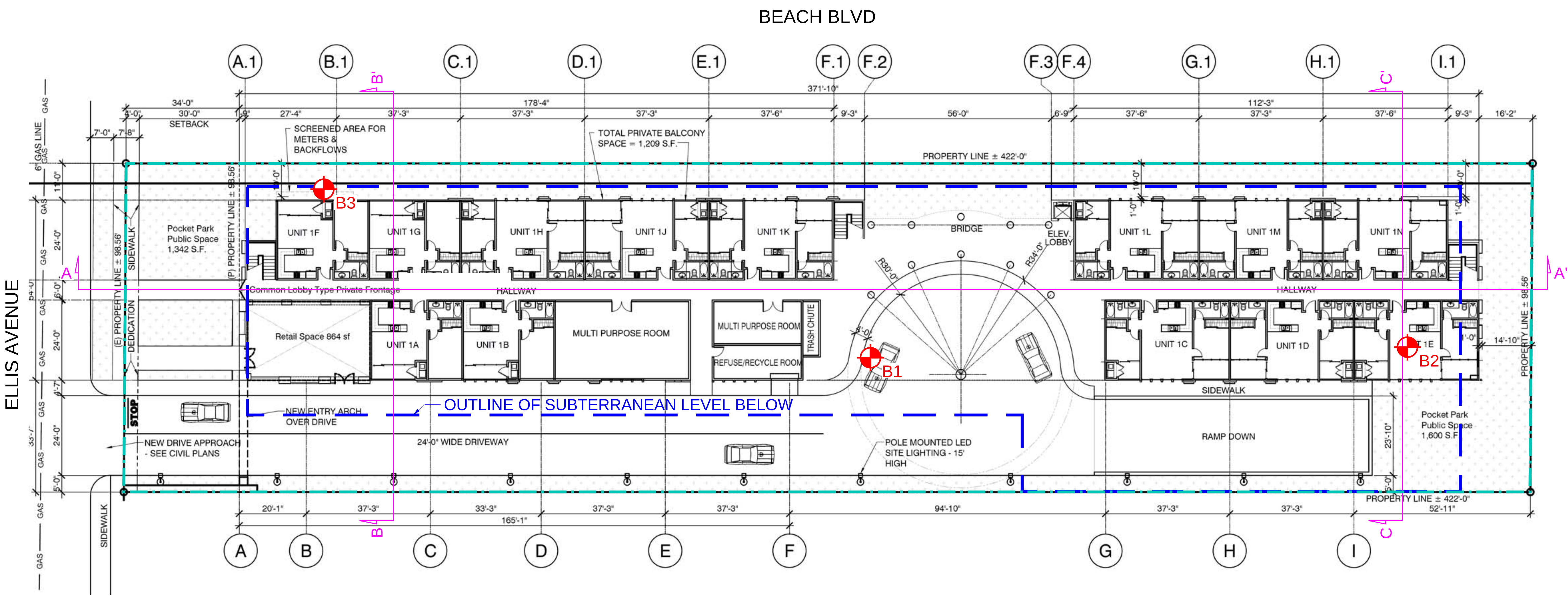
TOPO! map printed on 05/09/18 from "LA.TPO" and "Untitled.tpg"



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

GEOCON WEST, INC. ENVIRONMENTAL GEOTECHNICAL MATERIALS 3303 N. SAN FERNANDO BLVD. - SUITE 100 - BURBANK, CA 91504 PHONE (818) 841-8388 - FAX (818) 841-1704	VICINITY MAP ELLIS AVENUE CONDOS 8041 ELLIS AVENUE HUNTINGTON BEACH, CALIFORNIA

DRAFTED BY: RA	CHECKED BY: SFK	MAY 2018	PROJECT NO. A9764-88-01	FIG. 1



LEGEND

B3 Approximate Location of Boring

Approximate Location of Property Line

0 30' 60'

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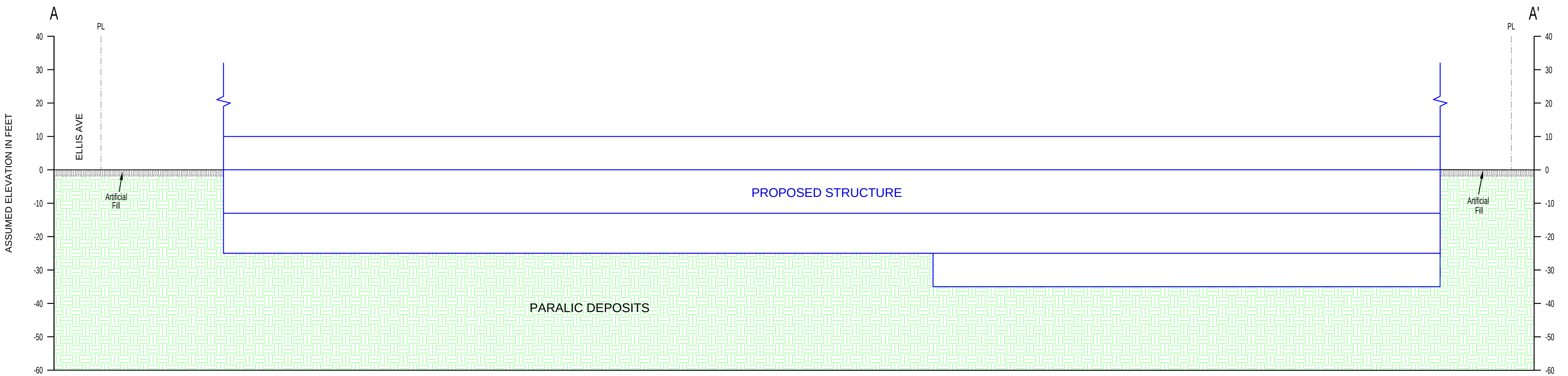
SITE PLAN

ELLIS AVENUE CONDOS

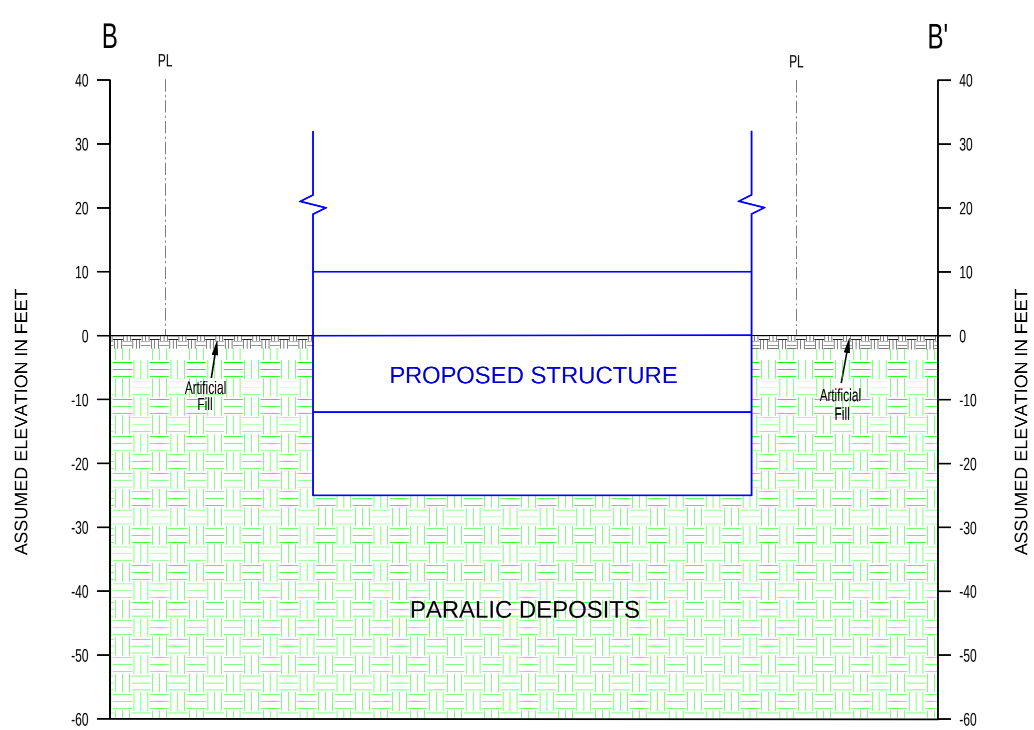
8041 ELLIS AVENUE

HUNTINGTON BEACH, CALIFORNIA

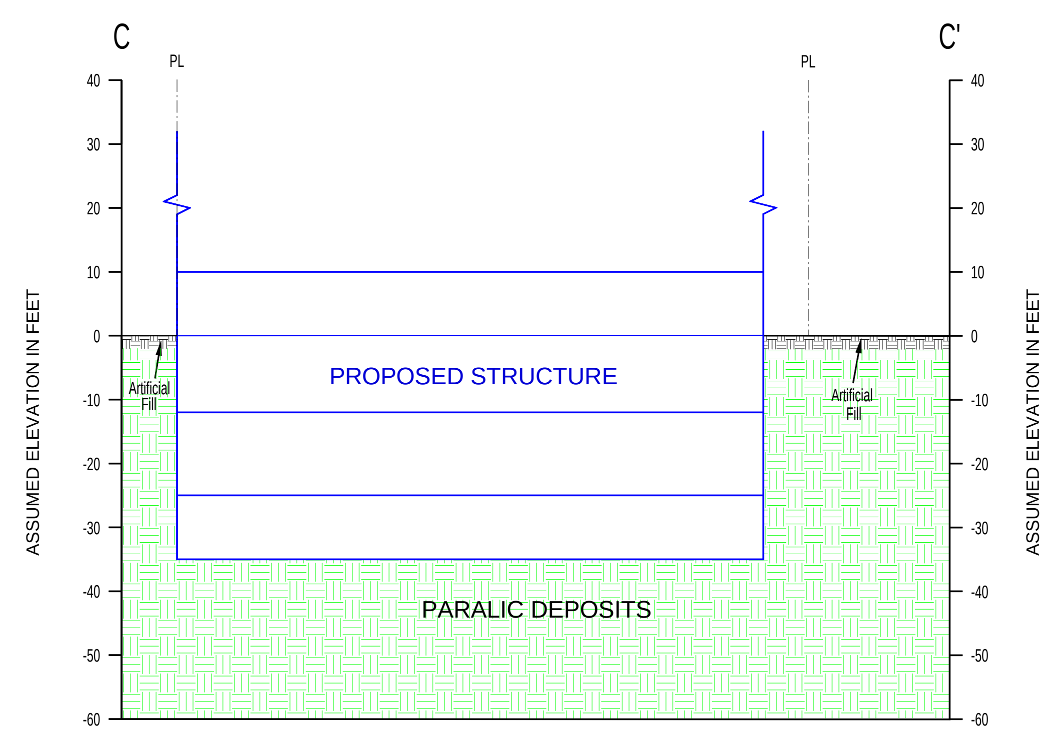
MAY 2018 | PROJECT NO. A9764-88-01 | FIG. 2



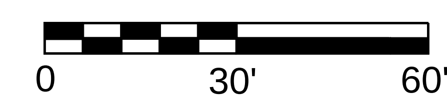
SECTION A - A'



SECTION B - B'



SECTION C - C'



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CROSS SECTIONS

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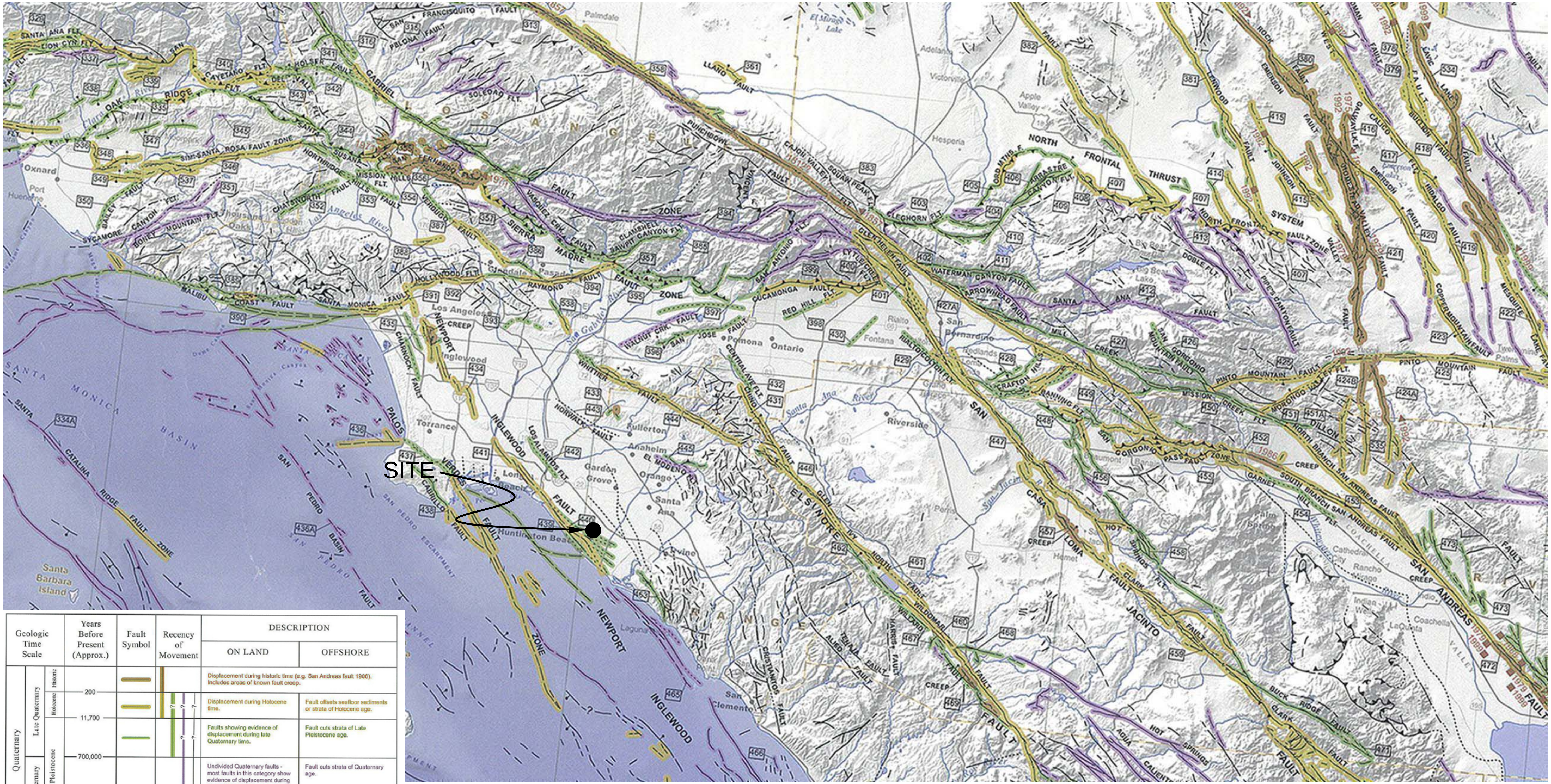
MAY 2018	PROJECT NO. A9764-88-01	FIG. 3

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



Geologic Time Scale			Years Before Present (Approx.)	Fault Symbol	Recency of Movement	DESCRIPTION	
						ON LAND	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*			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.
			4.5 billion (Age of Earth)				

* 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.

N

0 12 24 Miles

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REGIONAL FAULT MAP

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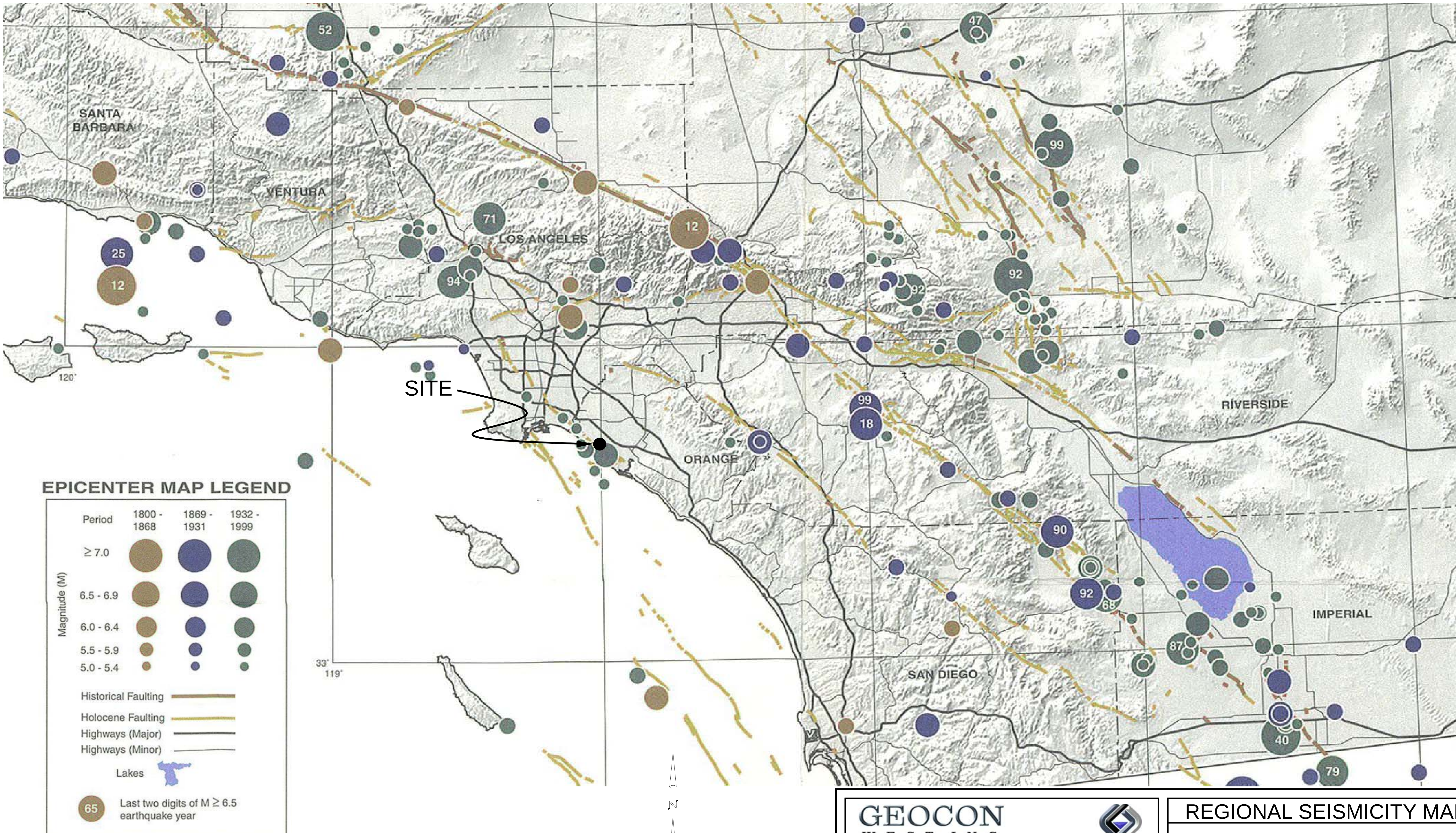
MAY 2018 | PROJECT NO. A9764-88-01 | FIG. 4



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REGIONAL SEISMICITY MAP

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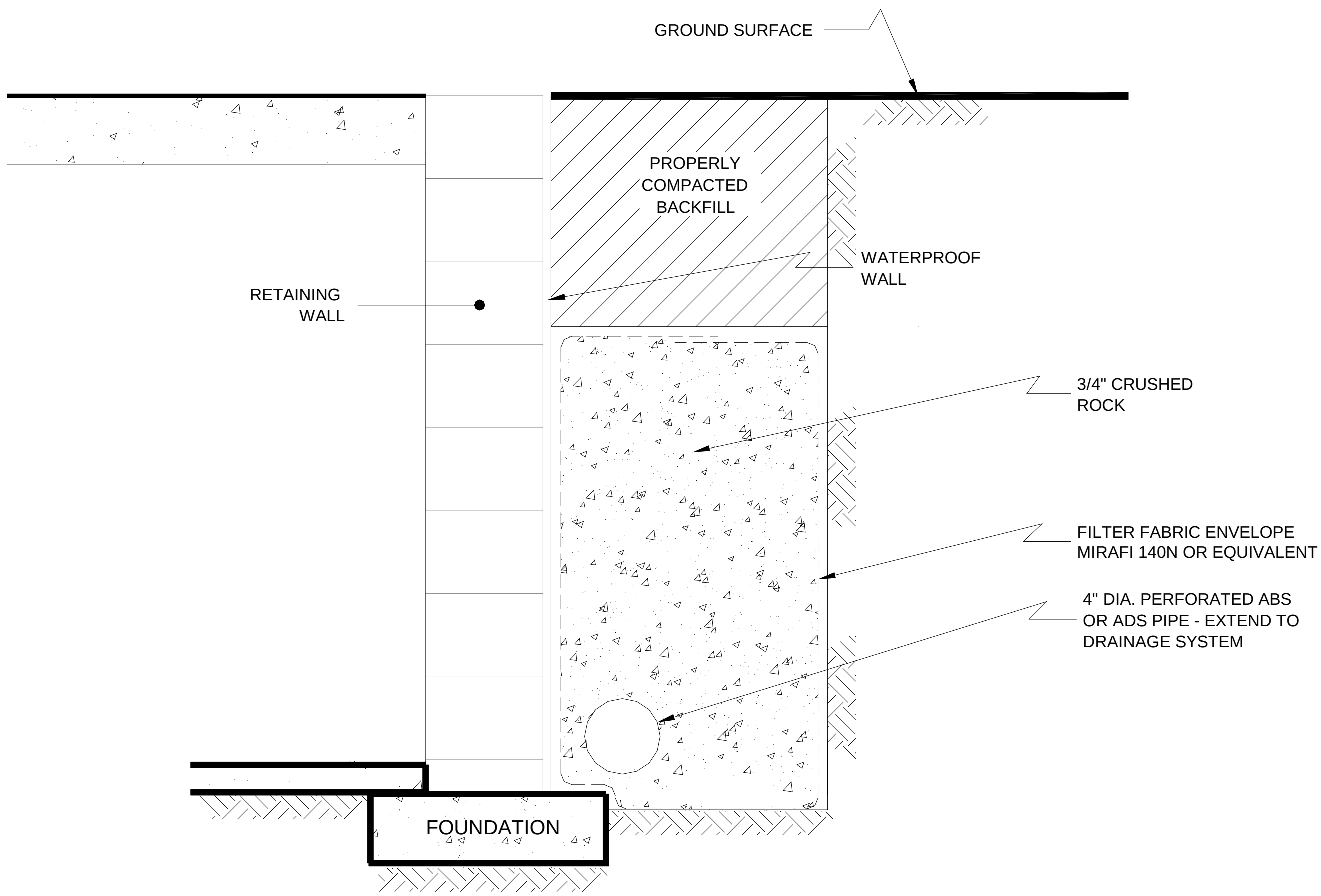
MAY 2018	PROJECT NO. A9764-88-01	FIG. 5



NO SCALE

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RETAINING WALL DRAIN DETAIL

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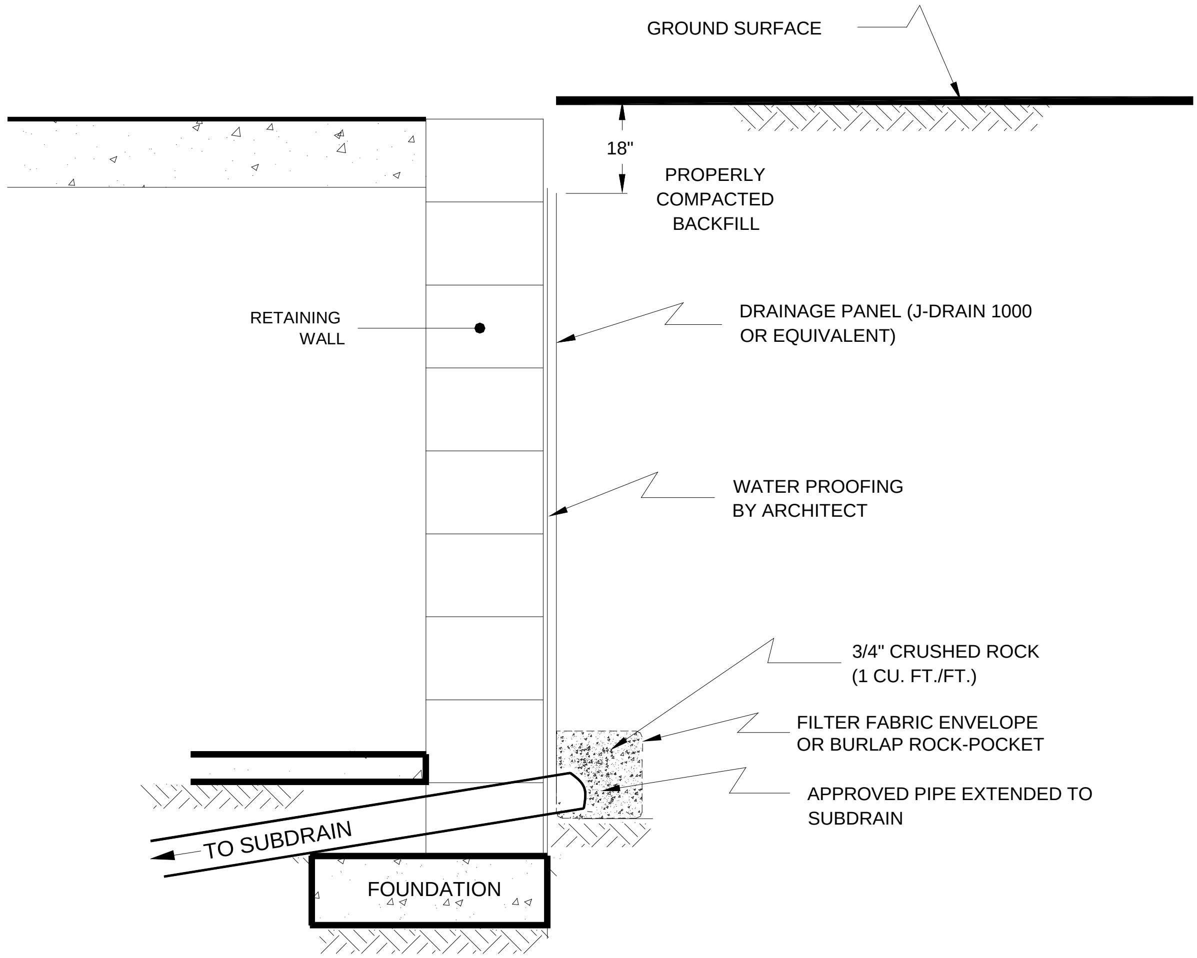
MAY 2018 | PROJECT NO. A9764-88-01 | FIG. 6



NO SCALE

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RETAINING WALL DRAIN DETAIL

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HUNTINGTON BEACH, CALIFORNIA

DRAFTED BY: PZ	CHECKED BY: JTA	MAY 2018	PROJECT NO. A9764-88-01	FIG. 7

PERCOLATION TEST DATA SHEET

Project:	Ellis Avenue		Project No:	A9764-88-01		Date:	4/11/2018
Test Hole No:		B3	Tested By:	PZ			
Depth of Test Hole, D_T:		40.5	USCS Soil Classification:		SM		
Test Hole Dimensions (inches)					Length	Width	
Diameter (if round) =		8	Sides (if rectangular) =		---	---	

Sandy Soil Criteria Test*

Trial No.	Start Time	Stop Time	Δt Time Interval (min)	D_0 Initial Depth to Water (in)	D_f Final Depth to Water (in)	ΔD Change in Water Level (in)	Greater than or Equal to 6"? (y/n)
1	8:47	9:12	25	420.0	481.2	61.2	y
2	9:16	9:41	25	420.0	462.0	42.0	y

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

Trial No.	Start Time	Stop Time	Δt Time Interval (min)	D_0 Initial Depth to Water (in)	D_f Final Depth to Water (in)	ΔD Change in Water Level (in)	Percolation Rate (min/in)
1	9:44	9:54	10	420.0	451.2	31.2	462
2	9:56	10:06	10	420.0	450.0	30.0	480
3	10:09	10:19	10	420.0	442.8	22.8	632
4	10:20	10:30	10	420.0	439.2	19.2	750
5	10:33	10:43	10	420.0	439.2	19.2	750
6	10:45	10:55	10	420.0	439.2	19.2	750
7							
8							

Infiltration Rate Calculation:

Time Interval, Δt =	10	minutes	Ho =	66.0	inches
Final Depth to Water, Df =	439.2	inches	Hf =	46.8	inches
Test Hole Radius, r =	4	inches	ΔH =	19.2	inches
Initial Depth to Water, Do =	420.0	inches	Havg =	56.4	inches
Total Depth of Test Hole, DT =	486.0	inches			

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

Infiltration Rate, It = **3.95** inches/hour

FIGURE 8

APPENDIX A

APPENDIX A

FIELD INVESTIGATION

The site was explored on April 10, 2018, by excavating three 8-inch diameter borings utilizing a truck-mounted hollow-stem auger drilling machine. The borings were advanced to depths between approximately 40½ and 60½ 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 2³/₈-inch diameter brass sampler rings to facilitate soil removal and testing. Bulk samples were also obtained.

Unified Soil Classification System (USCS). The logs of the borings are presented on Figures A1 through A3. The logs depict the soil and geologic conditions encountered and the depth at which samples were obtained. The logs also include our interpretation of the conditions between sampling intervals. Therefore, the logs contain both observed and interpreted data. We determined the lines designating the interface between soil materials on the logs using visual observations, penetration rates, excavation characteristics and other factors. The transition between materials may be abrupt or gradual. Where applicable, the boring logs were revised based on subsequent laboratory testing. The locations of the borings are shown on Figure 2.

PROJECT NO. A9764-88-01

BORING 1

ELEV. (MSL.) -- DATE COMPLETED 04/10/2018

EQUIPMENT HOLLOW STEM AUGER BY: PZ

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT)*	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0	BULK 0-5'				**ARTIFICIAL FILL** Sandy Clay, firm, slightly moist, brown, fine-grained.			
2					**OLD PARALIC DEPOSITS** Clay with Sand, firm, slightly moist, yellowish brown, fine-grained.			
4				CL				
	B1@5'				- increase in sand content	11	115.0	14.2
6								
					Silty Sand, medium dense, yellowish brown, fine-grained.			
8				SM				
10	B1@10'				Sand, poorly graded, medium dense, slightly moist, light brown, fine-grained, trace silt.	21	105.3	3.2
12								
14								
	B1@15'				- dense, no silt, fine- to coarse-grained, trace fine gravel	73	109.6	2.5
16								
18				SP				
20	BULK 20-25' B1@20'					68	110.2	1.2
22	B1@22'					71	119.8	2.2
24								
	B1@25'				- fine- to medium-grained	74	105.0	2.0
26								
	B1@27'				Silty Sand, medium dense, slightly moist, yellowish brown, fine-grained.	52	91.5	17.5
28				SM				

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

A9764-88-01 BORING LOGS.GPJ

SAMPLE SYMBOLS

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

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

GEOCON

BORING 1

ELEV. (MSL.) -- DATE COMPLETED 04/10/2018

EQUIPMENT HOLLOW STEM AUGER BY: PZ

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT)*	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
30	B1@30'				- grayish brown	29	120.8	13.6
32	B1@32'				- dense, yellowish brown	69	118.1	11.2
34								
35	B1@35'					85	97.4	4.1
36				SM				
38								
40	B1@40'				- increase in silt content	54	96.2	21.8
42								
44				CL	Clay, stiff, slightly moist, grayish brown.			
45	B1@45'					29	88.5	31.4

Total depth of boring: 45.5 feet.
Fill to 1.5 feet.
No groundwater encountered.
Backfilled with soil cuttings and tamped.
Asphalt patched.
Penetration resistance for 140-pound hammer falling 30 inches by auto-hammer.

A9764-88-01 BORING LOGS.GPJ

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

SAMPLE SYMBOLS

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

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. A9764-88-01

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	**BORING 2** ELEV. (MSL.) **--** DATE COMPLETED **04/10/2018** EQUIPMENT **HOLLOW STEM AUGER** BY: **PZ** MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT)*	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0					**ARTIFICIAL FILL** Sandy Clay, firm, slightly moist, dark brown, fine-grained.			
5	B2@5'			CL	**OLD PARALIC DEPOSITS** Clay, stiff, slightly moist, brown.	24	122.7	13.5
9				SM	Silty Sand, loose, slightly moist, yellowish brown, fine-grained.			
10	B2@10'					14	102.8	6.5
14				SW	Sand, well graded, dense, slightly moist, light brown, fine- to coarse-grained.			
15	B2@15'					68	120.3	1.3
20	B2@20'				- increase in coarse-grained, some fine gravel	78	109.8	1.5
25	B2@25'				- no gravel	71	117.7	1.6
27	B2@27'				- medium dense	32	115.6	4.9
28				ML	Sandy Silt, stiff, slightly moist, brown, fine-grained.			

A9764-88-01 BORING LOGS.GPJ

Figure A2,
Log of Boring 2, Page 1 of 3

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 04/10/2018

EQUIPMENT HOLLOW STEM AUGER BY: PZ

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT)*	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
30	B2@30'			ML		36	103.7	21.6
				SP-SM	Sand with Silt, poorly graded, dense, slightly moist, light brown, fine-grained.			
32	B2@32'					70	107.0	4.2
34								
	B2@35'					76	102.6	8.9
36								
	B2@37'				- increase in silt content, grayish brown	87	102.3	20.2
38								
40	B2@40'				- yellowish brown	60	109.1	14.4
42								
44								
				CL	Clay, stiff, slightly moist, brown.			
	B2@45'					23	92.0	31.2
46								
48								
50	B2@50'				- firm, dark brown - gray	18	78.9	45.1
52								
54								
	B2@55'			SP-SM	Sand with Silt, poorly graded, very dense, moist, yellowish brown, fine- to medium-grained.	50 (5")	110.1	16.6
56			▼					
58								

A9764-88-01 BORING LOGS.GPJ

Figure A2,
Log of Boring 2, Page 2 of 3

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. A9764-88-01

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	BORING 2 ELEV. (MSL.) -- DATE COMPLETED 04/10/2018 EQUIPMENT HOLLOW STEM AUGER BY: PZ	PENETRATION RESISTANCE (BLOWS/FT)*	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
					MATERIAL DESCRIPTION			
60	B2@60'					46		
					Total depth of boring: 60.5 feet. Fill to 5 feet. Groundwater encountered at 56 feet. Backfilled with soil cuttings and tamped. Asphalt patched. Penetration resistance for 140-pound hammer falling 30 inches by auto-hammer.			

Figure A2,
Log of Boring 2, Page 3 of 3

A9764-88-01 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.

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BORING 3

ELEV. (MSL.) -- DATE COMPLETED 04/10/2018

EQUIPMENT HOLLOW STEM AUGER BY: PZ

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT)*	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0					**ARTIFICIAL FILL** Sandy Clay, firm, slightly moist, brown.			
2					**OLD PARALIC DEPOSITS** Sandy Clay, firm, slightly moist, brown, fine-grained.			
4				CL				
5	B3@5'				- increase in sand content, dark brown	15	121.1	11.3
7					Sand, poorly graded, medium dense, slightly moist, yellowish brown, fine-grained.			
10	B2@10'					22	105.7	3.0
15	B3@15'				- dense, light brown, fine- to coarse-grained	82	116.8	2.5
18				SP				
20	B3@20'				- fine- to medium-grained	83	118.4	3.3
22	B3@22'					79	107.7	2.1
25	B3@25'				- very dense, fine-grained	50 (5")	108.5	3.0
27	B3@27'				- dense, fine- to medium-grained	66	109.9	1.7

A9764-88-01 BORING LOGS.GPJ

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

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

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

BORING 3

ELEV. (MSL.) -- DATE COMPLETED 04/10/2018

EQUIPMENT HOLLOW STEM AUGER BY: PZ

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT)*	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
30	B3@30'			SP		54	119.6	3.8
32	B3@32'			CL	Clay, firm, slightly moist, brown.	18	103.6	26.7
34				SM	Silty Sand, dense, slightly moist, brown, fine-grained.			
36	B3@35'					65	110.4	11.1
38								
40	B3@40'				- medium dense	52	95.5	20.7

Total depth of boring: 40.5 feet.
Fill to 2 feet.
No groundwater encountered.
Percolation testing performed.
Backfilled with soil cuttings.
Asphalt patched.
Penetration resistance for 140-pound hammer falling 30 inches by auto-hammer.

A9764-88-01 BORING LOGS.GPJ

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

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

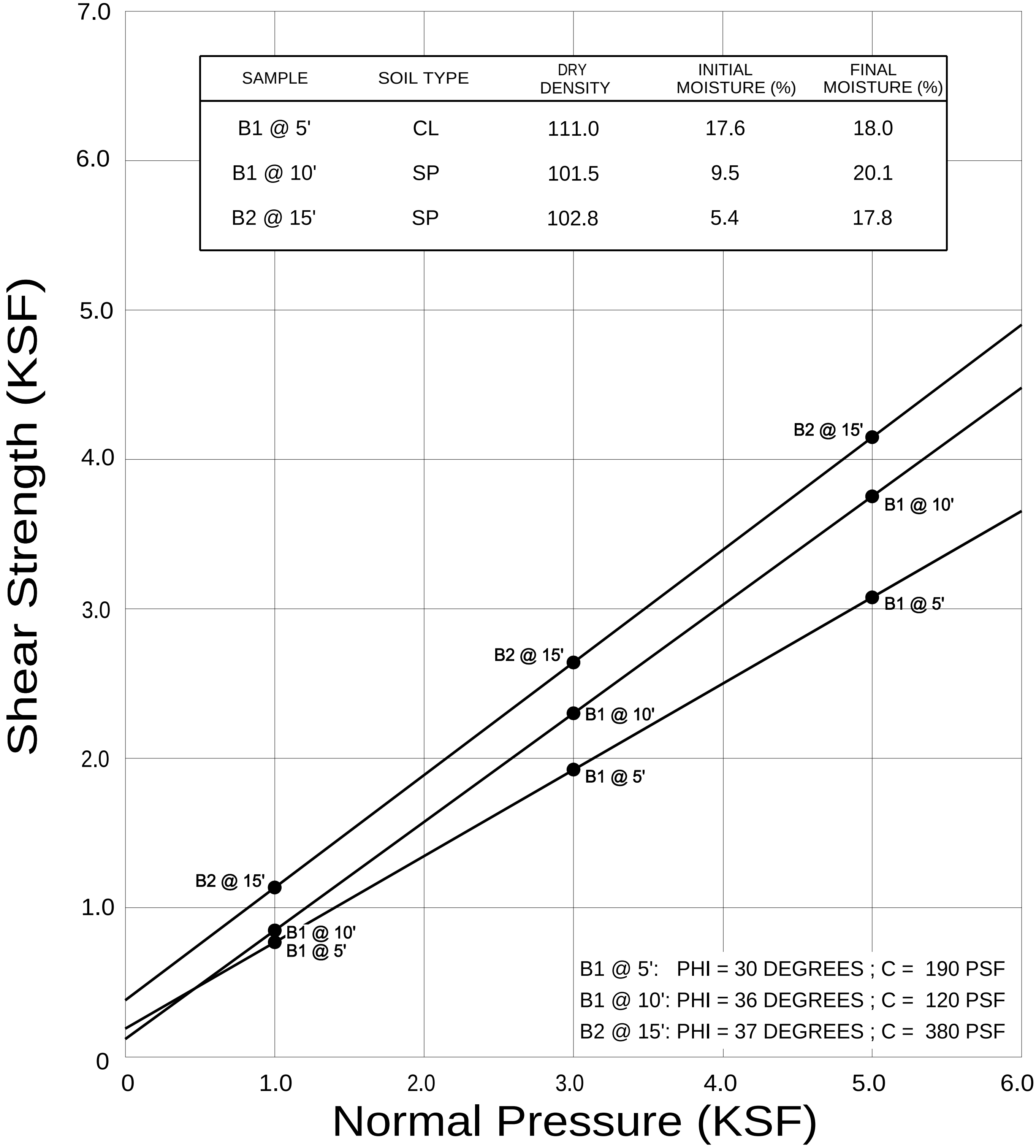
NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

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 compaction characteristics, direct shear strength, consolidation and expansion characteristics, corrosivity, in-place dry density and moisture content. The results of the laboratory tests are summarized in Figures B1 through B9. 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 @ 5'	CL	111.0	17.6	18.0
B1 @ 10'	SP	101.5	9.5	20.1
B2 @ 15'	SP	102.8	5.4	17.8

B1 @ 5': PHI = 30 DEGREES ; C = 190 PSF
B1 @ 10': PHI = 36 DEGREES ; C = 120 PSF
B2 @ 15': PHI = 37 DEGREES ; C = 380 PSF

● Direct Shear, Saturated

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DIRECT SHEAR TEST RESULTS

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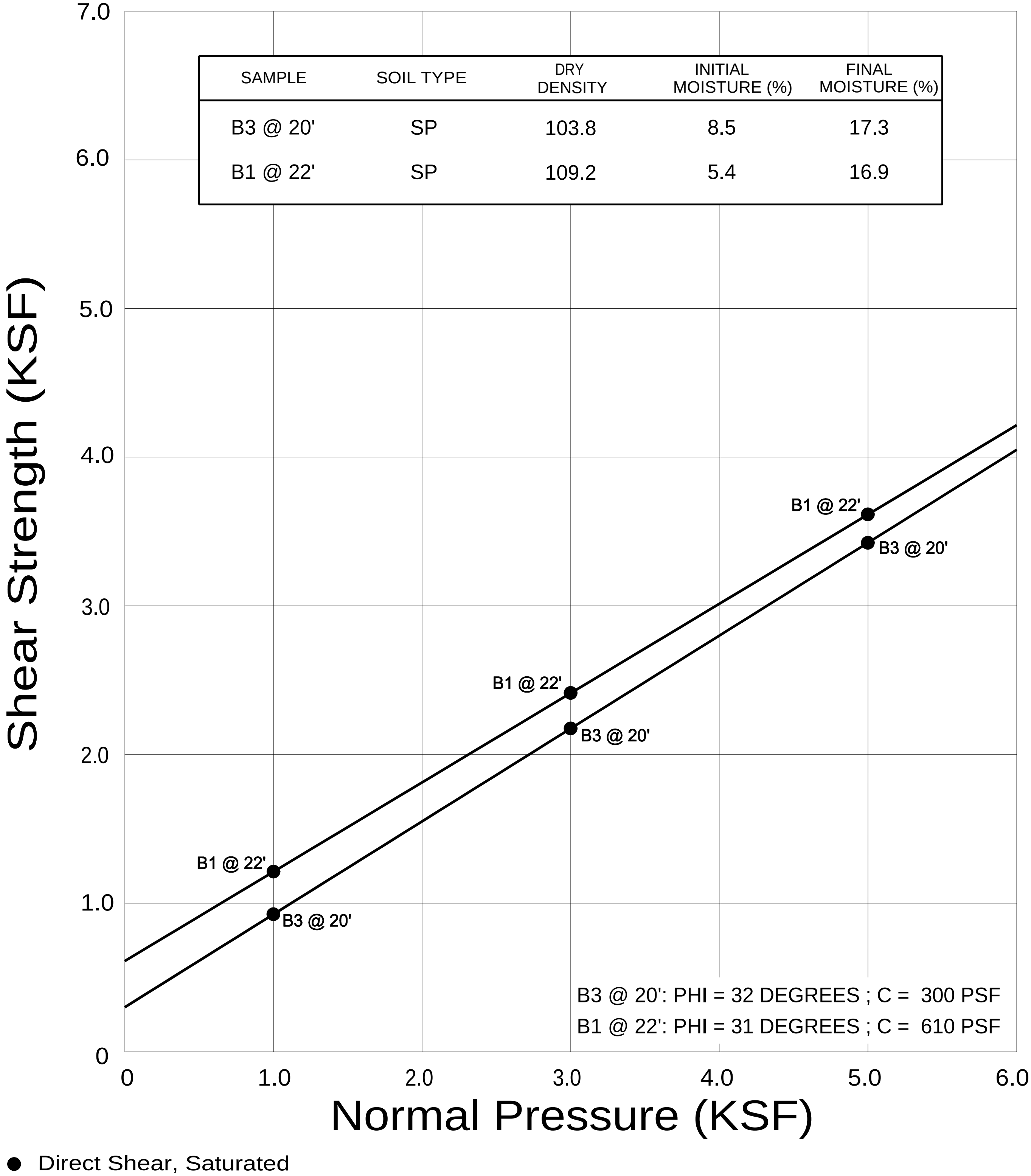
8041 ELLIS AVENUE
HUNTINGTON BEACH, CALIFORNIA

DRAFTED BY: PZ	CHECKED BY: JTA	MAY 2018	PROJECT NO. A9764-88-01	FIG. B1



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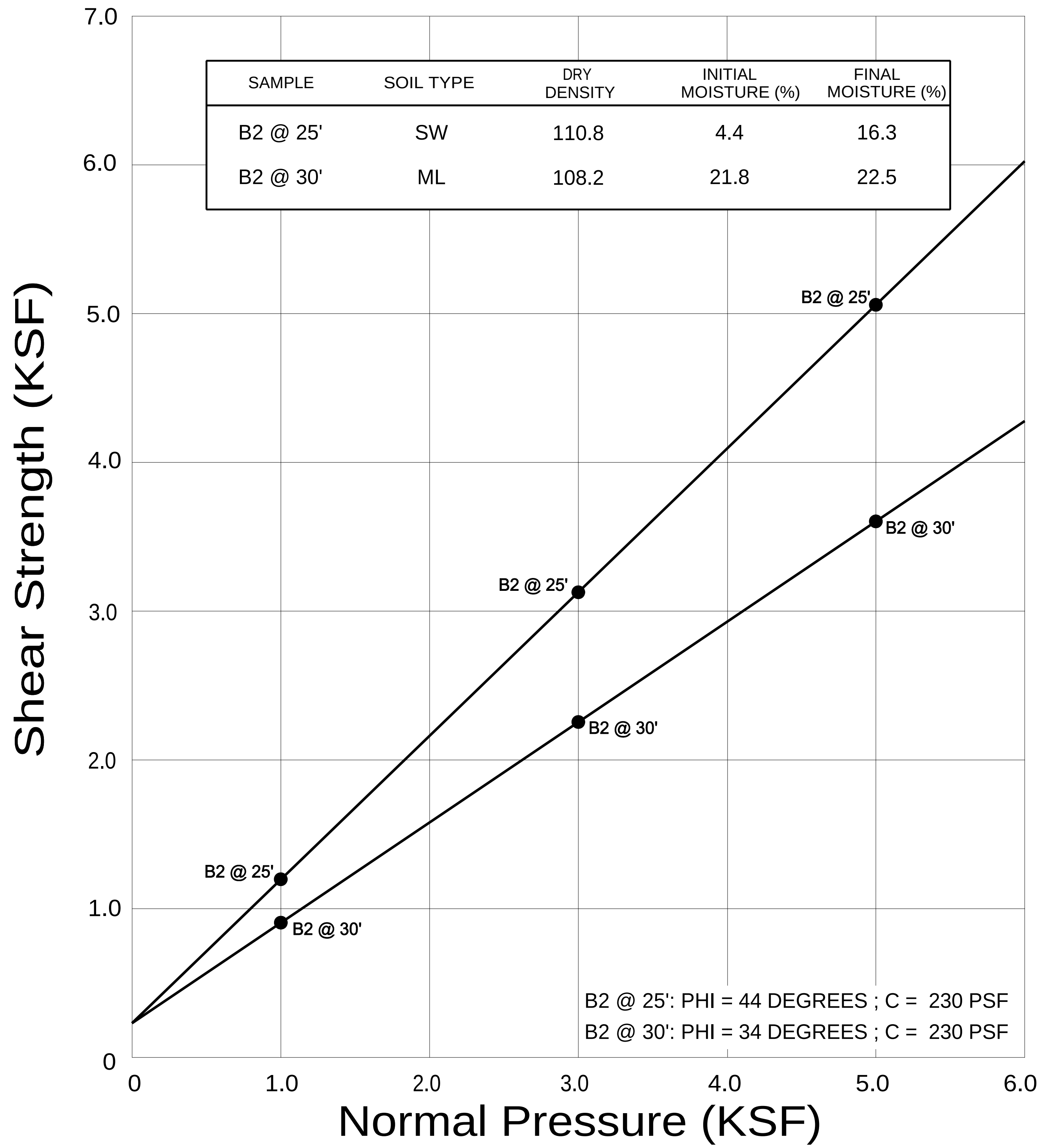
DRAFTED BY: PZ	CHECKED BY: JTA	MAY 2018	PROJECT NO. A9764-88-01	FIG. B2



SAMPLE	SOIL TYPE	DRY DENSITY	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B2 @ 25'	SW	110.8	4.4	16.3
B2 @ 30'	ML	108.2	21.8	22.5

● Direct Shear, Saturated

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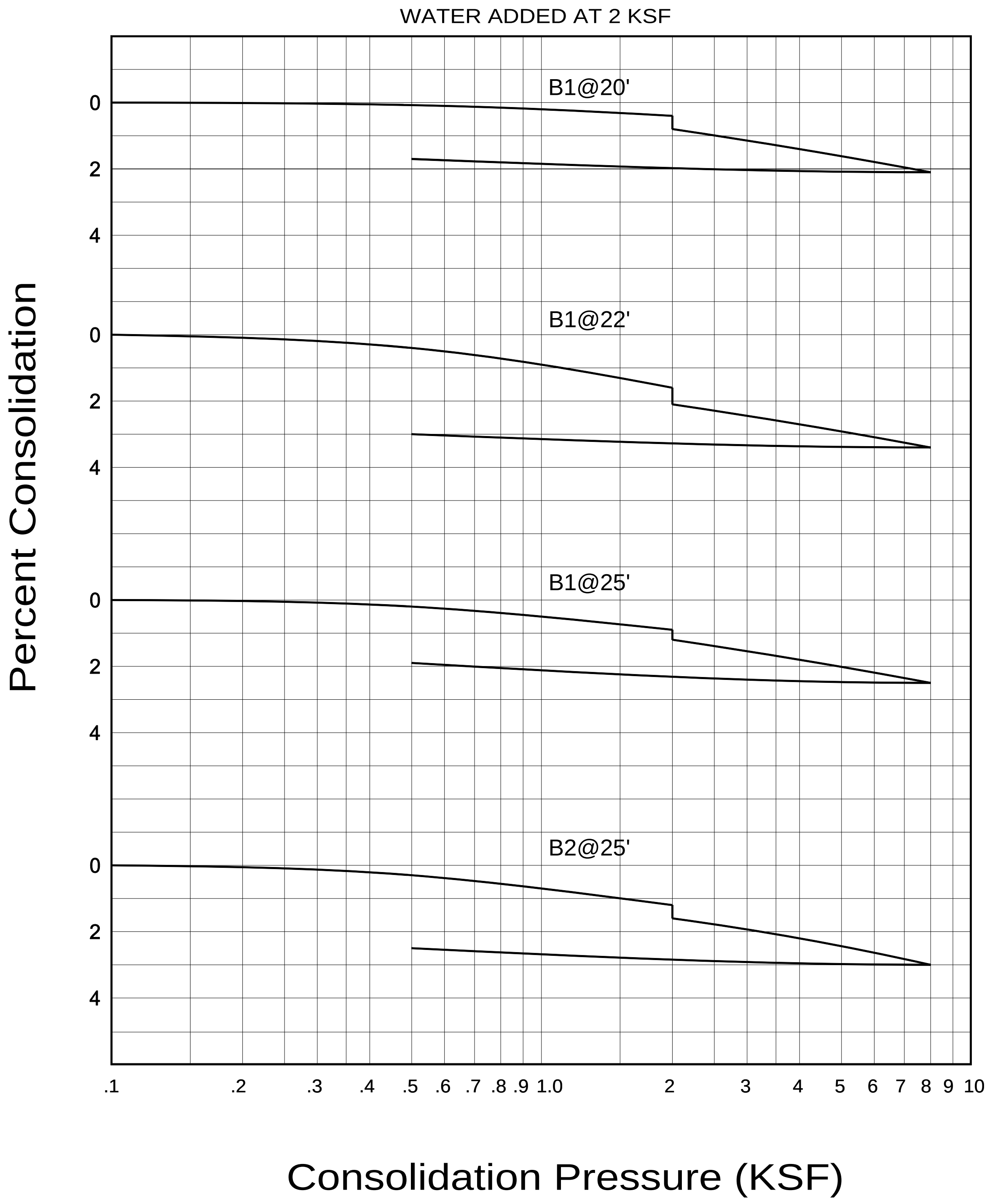
DRAFTED BY: PZ	CHECKED BY: JTA	MAY 2018	PROJECT NO. A9764-88-01	FIG. B3



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CONSOLIDATION TEST RESULTS

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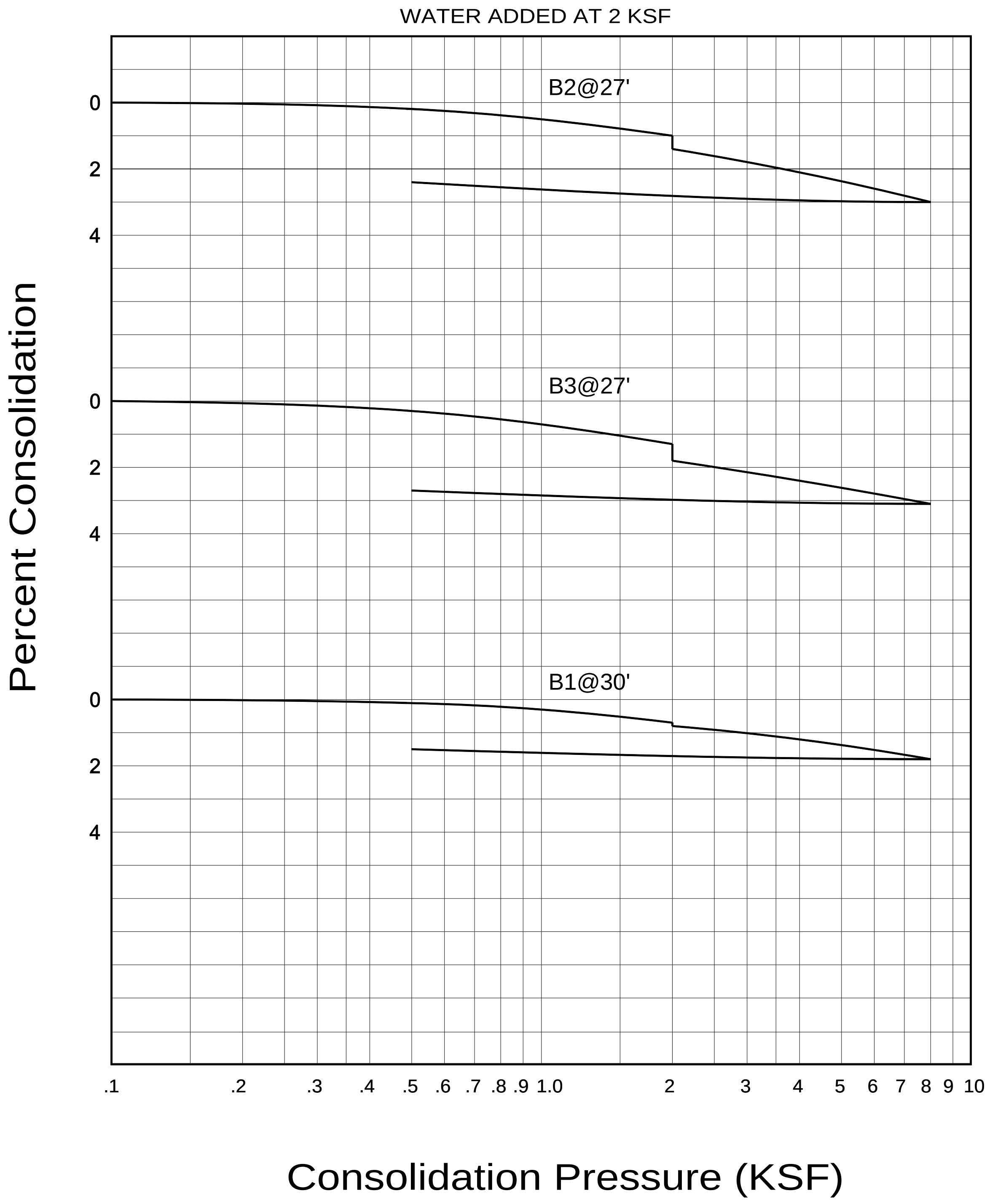
HUNTINGTON BEACH, CALIFORNIA

DRAFTED BY: PZ	CHECKED BY: JTA	MAY 2018	PROJECT NO. A9764-88-01	FIG. B4



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CONSOLIDATION TEST RESULTS

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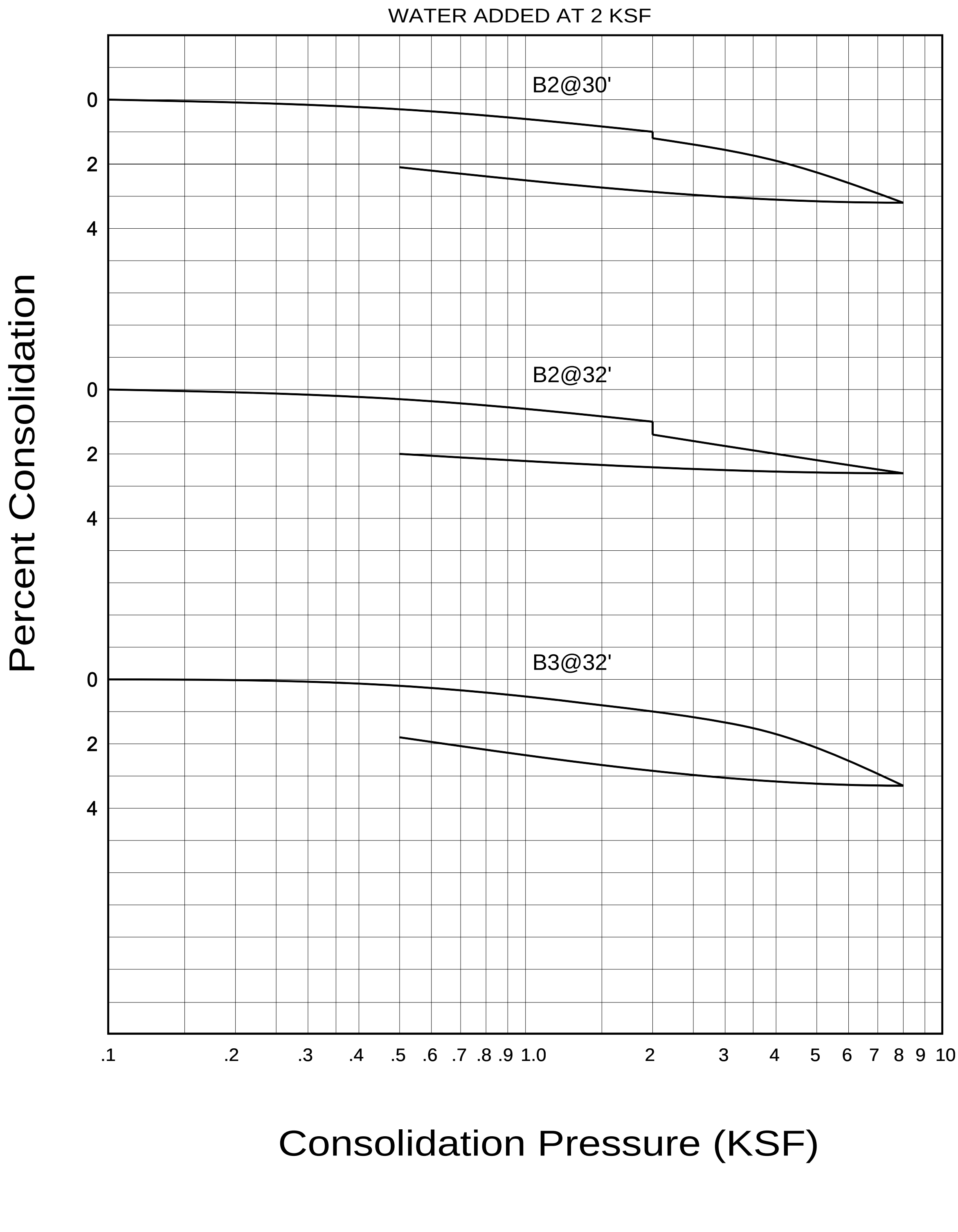
DRAFTED BY: PZ	CHECKED BY: JTA	MAY 2018	PROJECT NO. A9764-88-01	FIG. B5



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CONSOLIDATION TEST RESULTS

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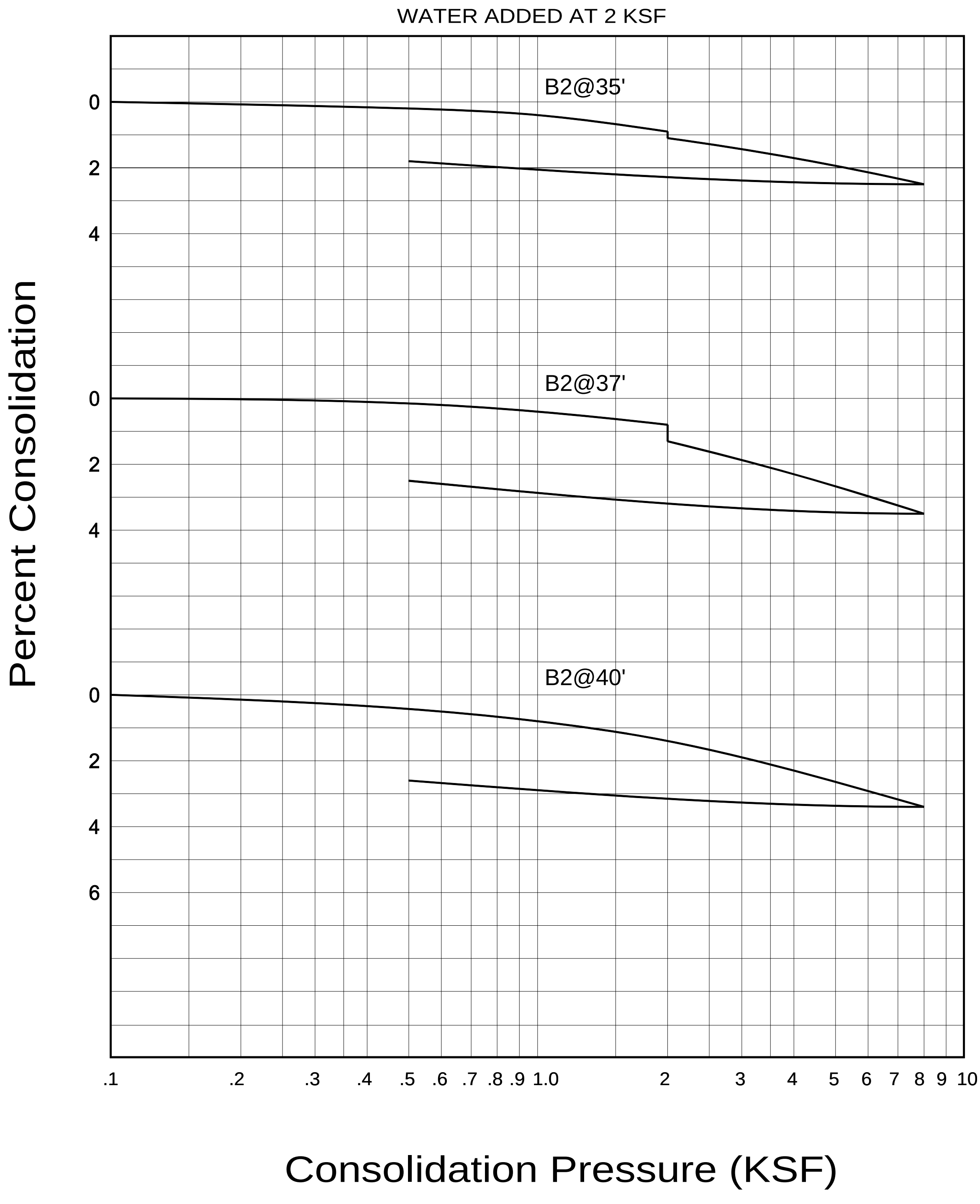
DRAFTED BY: PZ	CHECKED BY: JTA	MAY 2018	PROJECT NO. A9764-88-01	FIG. B6



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CONSOLIDATION TEST RESULTS

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DRAFTED BY: PZ	CHECKED BY: JTA	MAY 2018	PROJECT NO. A9764-88-01	FIG. B7

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

Sample No.	Moisture Content (%)		Dry Density (pcf)	Expansion Index	*UBC Classification	**CBC Classification
	Before	After				
B1 @ 0-5'	9.0	26.2	112.5	105	High	Expansive

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

** Reference: 2016 California Building Code, Section 1803.5.3

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

Sample No.	Soil Description	Maximum Dry Density (pcf)	Optimum Moisture (%)
B1 @ 0-5'	Brown Sandy Clay	122.5	12.3

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LABORATORY TEST RESULTS

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DRAFTED BY: PZ	CHECKED BY: JTA	MAY 2018	PROJECT NO. A9764-88-01	FIG. B8

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

Sample No.	pH	Resistivity (ohm centimeters)
B1 @ 20-25'	8.7	2879 (Moderately Corrosive)

SUMMARY OF LABORATORY CHLORIDE CONTENT TEST RESULTS EPA NO. 325.3

Sample No.	Chloride Ion Content (%)
B1 @ 20-25'	0.003

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

Sample No.	Water Soluble Sulfate (% SO_4)	Sulfate Exposure*
B1 @ 20-25'	0.001	Negligible

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

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CORROSIVITY TEST RESULTS

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MAY 2018 | PROJECT NO. A9764-88-01 | FIG. B9