Appendix D
Geotechnical Study

ESA

UPDATED GEOTECHNICAL INVESTIGATION

PROPOSED LONG BEACH APARTMENTS
MULTI-FAMILY RESIDENTIAL DEVELOPMENT
3RD STREET AND PACIFIC AVENUE
LONG BEACH, CALIFORNIA

GEOCON
WEST, INC.

GEOTECHNICAL
ENVIRONMENTAL
MATERIALS

PREPARED FOR

ENSEMBLE INVESTMENTS
LONG BEACH, CALIFORNIA

PROJECT NO. A9542-06-01

JANUARY 31, 2017

Project No. A9542-06-01
January 31, 2017

Jason Muller
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Subject: UPDATED GEOTECHNICAL INVESTIGATION
PROPOSED LONG BEACH APARTMENTS
MULTI-FAMILY RESIDENTIAL DEVELOPMENT
3RD STREET AND PACIFIC AVENUE
LONG BEACH, CALIFORNIA

Dear Mr. Muller:

In accordance with your authorization of our proposal dated October 11, 2016, we have prepared this updated geotechnical investigation for the proposed multi-family residential development located at 3rd Street and Pacific Avenue in the City of Long 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 grading 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.

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UPDATED GEOTECHNICAL INVESTIGATION

1. PURPOSE AND SCOPE

This report presents the results of an updated geotechnical investigation for the proposed multi-family residential development located, known as the Long Beach Apartments project, at 3rd Street and Pacific Avenue in the City of Long Beach, California (see Vicinity Map, Figure 1). The purpose of the investigation was to evaluate subsurface soil and geologic conditions underlying the property, and based on conditions encountered, to provide conclusions and recommendations pertaining to the geotechnical aspects of proposed 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 previously explored on May 25, 2010 by conducting five 8-inch diameter borings utilizing a truck-mounted hollow-stem auger drilling machine. The borings were advanced to depths between 20½ and 30½ feet below the existing ground surface. Additional site exploration was performed on December 27, 2016 by excavating three 8-inch diameter borings to depths of 20 and 60½ feet below the existing ground surface using a truck-mounted hollow-stem auger drilling machine. The approximate locations of the exploratory borings are depicted on the Site Plan (Figure 2A). 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 the northeast corner of 3rd Street and Pacific Avenue in the City of Long Beach, California. The site is an approximately 51,150 square-foot rectangular-shaped parcel and is bisected by West Roble Way. The property is currently occupied by two asphalt paved parking lots. The site is bounded by 4th Street to the north, by Pacific Avenue to the east, by 3rd Street to the south, and by Solano Court to the west. The site is relatively level and surface water drainage at the site appears to be by sheet flow along the existing ground contours to the city streets. Vegetation is not existent due to the developed nature of the site.

Based on the information provided to us by the Client, it is our understanding that the proposed multi-family residential development will consist of a 7-story tower and an 18-story tower to be constructed over two levels of subterranean parking. It is anticipated that excavation for the subterranean level will extend to depths of 25 feet below the ground surface, including foundation depths. The proposed development is depicted on the Site Plan and Cross Sections (see Figures 2A and 2B).

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 15 kips per linear foot, and column loads may be up to 1,000 kips.

Once the design phase proceeds to a more finalized plan, the recommendations within this report should be reviewed and revised, if necessary. Geocon should be contacted to determine the necessity for review and possible revision of this report.

3. GEOLOGIC SETTING

The site is located in the southern edge of the Los Angeles Basin. The Los Angeles Basin is a coastal plain between the Santa Monica Mountains, Elysian Hills, and Repetto Hills to the north, the Puente Hills and Whittier Fault to the east, the Palos Verdes Hills and Pacific Ocean to the west and south, and the Santa Ana Mountains and San Joaquin Hills to the southeast. The Los Angeles Basin is located in the northern portion of the Peninsular Ranges geomorphic province and is a northwest-trending alluviated lowland plain, sometimes called the Coastal Plain of Los Angeles. The basin is underlain by a deep structural depression which has been filled by both marine and continental sedimentary deposits, which overlie a basement complex of igneous and metamorphic composition (Yerkes, et al., 1965). The prominent structural features within the Los Angeles Basin include the central lowland plain, the uplifted Palos Verdes Hills, and the northwest-trending line of low hills and mesas (underlain by the Newport-Inglewood Fault Zone).

4. GEOLOGIC MATERIALS

Based on our field investigation and published geologic maps of the area, the soils underlying the site consist of undocumented fill over Pleistocene Age old paralic deposits which are comprised of continental alluvial sediments and marine terrace deposits (CGS, 2010). The soil and geologic units encountered at the site are discussed below. General soil profiles are provided on the boring logs in Appendix A.

4.1 Artificial Fill

Various amounts of artificial fill were encountered during site exploration. The maximum depth of fill encountered was 6½ feet below existing ground surface in boring B1 near the northwest corner of the property. The 6½ feet of existing artificial fill encountered in Boring 1 consisted of 2 feet of soil underlain by 4½ feet of concrete underlain by undisturbed native soils. The remaining explorations, Borings 2 through 8, encountered less than 3 feet of artificial fill. The fill generally consists of light brown to dark brown or reddish brown silty sand or olive brown clay with varied amounts of construction debris. The artificial fill is characterized as slightly moist and medium dense or soft. 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.

Based on the depth and contents of the artificial fill encountered in Boring 1, it is our opinion that the deeper fill, including the concrete, is likely isolated within the northwestern portion of the site. If additional information regarding prior site conditions associated with this portion of the site become available, they should be provided to Geocon for consideration and possible incorporation into grading and foundation recommendations. Excavation of the subterranean level is expected to penetrate through the deeper fill and expose competent old paralic deposits throughout the excavation bottom.

4.2 Old Paralic Deposits

The artificial fill is underlain by Pleistocene Age old paralic deposits that consist of interbedded shallow marine terrace deposits and continental alluvial sediments. The old paralic deposits consist primarily of light brown to brown or reddish brown to yellowish brown, poorly graded sand, silty sand, and sandy silt with minor interbeds of silt and clay. The soils are generally slightly moist to wet, and loose to very dense or stiff to hard.

5. GROUNDWATER

The City of Long Beach General Plan (2004) and the City of Long Beach Safety Element (1988) indicate the historic high groundwater level at the site is reported to be approximately 25 feet and 30 feet below the ground surface, respectively. A review of the Seismic Hazard Zone Report for the Long Beach 7.5 Minute Quadrangle (California Division of Mines and Geology [CDMG], 1998) indicates there is no reported historic groundwater level information for the immediate site vicinity. Based on current groundwater basin management practices, it is unlikely that groundwater levels will ever exceed the historic high levels.

The Los Angeles County Department of Public Works (LADPW) maintains various wells in the vicinity of the subject site. The closest LADPW water well to the site is Well No. 403E, which is located approximately 0.2 miles southwest of the site (LADPW, 2017). Although Well No. 403E is located in close proximity to the site, the City of Long Beach (2004) indicates this well is located within an area where the groundwater levels have historically been shallower than what is reported in

the site vicinity. Therefore, the data from this well is not representative of the historic groundwater conditions at the site.

Groundwater was encountered in borings B2, B6, and B7 at depths of approximately 29 feet, 32 feet, and 32 feet below existing ground surface respectively. It is not uncommon for groundwater levels to vary seasonally or for groundwater conditions to develop where none previously existed, especially in impermeable fine-grained soils which are subjected to irrigation or precipitation. In addition, recent requirements for stormwater infiltration could result in shallower seepage conditions in the region. Proper surface drainage of irrigation and precipitation will be critical to future performance of the project. Recommendations for drainage are provided in the S*urface Drainage* section of this report (see Section 7.24).

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 (Bryant and Hart, 2007). By definition, an active fault is one that has had surface displacement within Holocene time (about the last 11,000 years). A potentially active fault has demonstrated surface displacement during Quaternary time (approximately the last 1.6 million years), but has had no known Holocene movement. Faults that have not moved in the last 1.6 million years are considered inactive.

The site is not within a state-designated Alquist-Priolo Earthquake Fault Zone (CDMG, 1986; CGS, 2016) for surface fault rupture hazards. No active or potentially active faults with the potential for surface fault rupture are known to pass directly beneath the site. Therefore, the potential for surface rupture due to faulting occurring beneath the site during the design life of the proposed development is considered low. However, the site is located in the seismically active Southern California region, and could be subjected to moderate to strong ground shaking in the event of an earthquake on one of the many active Southern California faults. The faults in the vicinity of the site are shown in Figure 3, Regional Fault Map.

The nearest active surface fault rupture to the site is the Newport-Inglewood Fault Zone located approximately 2.3 miles to the northeast (CDMG, 1986). Other nearby active faults are the Palos Verdes Fault, the Cabrillo Fault and the Redondo Canyon Fault located approximately 5.0 miles southwest, 7.0 miles southwest and 12½ miles west-northwest of the site, respectively (Ziony and Jones, 1989). The active San Andreas Fault Zone is located approximately 50 miles northeast of the site.

The closest potentially active faults to the site are the Los Alamitos Fault and the Norwalk Fault located approximately 6.1 miles to the northeast and 11.5 miles to the northeast, respectively (Ziony and Jones, 1989).

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

6.2 Seismicity

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

LIST OF HISTORIC EARTHQUAKES

Earthquake (Oldest to Youngest)	Date of Earthquake	Magnitude	Distance to Epicenter (Miles)	Direction to Epicenter
San Jacinto-Hemet area	April 21, 1918	6.8	68	E
Near Redlands	July 23, 1923	6.3	56	ENE
Long Beach	March 10, 1933	6.4	17	SE
Tehachapi	July 21, 1952	7.5	97	NW
San Fernando	February 9, 1971	6.6	46	NNW
Whittier Narrows	October 1, 1987	5.9	21	NNE
Sierra Madre	June 28, 1991	5.8	36	NNE
Landers	June 28, 1992	7.3	105	ENE
Big Bear	June 28, 1992	6.4	84	ENE
Northridge	January 17, 1994	6.7	36	NW
Hector Mine	October 16, 1999	7.1	124	ENE

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

6.3 Seismic Design Criteria

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

2016 CBC SEISMIC DESIGN PARAMETERS

Parameter	Value	2016 CBC Reference
Site Class	D	Table 1613.5.2
Spectral Response – Class B (short), S_S	1.614g	Figure 1613.5(3)
Spectral Response – Class B (1 sec), S_1	0.607g	Figure 1613.5(4)
Site Coefficient, F_a	1.0	Table 1613.5.3(1)
Site Coefficient, F_v	1.5	Table 1613.5.3(2)
Maximum Considered Earthquake Spectral Response Acceleration (short), S_{MS}	1.614g	Section 1613.5.3 (Eqn 16-37)
Maximum Considered Earthquake Spectral Response Acceleration – (1 sec), S_{M1}	0.911g	Section 1613.5.3 (Eqn 16-38)
5% Damped Design Spectral Response Acceleration (short), S_{DS}	1.076g	Section 1613.5.4 (Eqn 16-39)
5% Damped Design Spectral Response Acceleration (1 sec), S_{D1}	0.607g	Section 1613.5.4 (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	0.630g	Figure 22-7
Site Coefficient, F_{PGA}	1.0	Table 11.8-1
Site Class Modified MCE_G Peak Ground Acceleration, PGA_M	0.63g	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 2008 Probabilistic Seismic Hazard Analysis (PSHA) Interactive Deaggregation online tool. The result of the deaggregation analysis indicates that the predominant earthquake contributing to the MCE peak ground acceleration is characterized as a 6.76 magnitude event occurring at a hypocentral distance of 9.8 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 18.9 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" 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 Zone Map for the Long Beach Quadrangle (CDMG, 1999) indicates that the site is not located in an area designated as having a potential for liquefaction. In addition, according to the County of Los Angeles Safety Element (Leighton, 1990), and the City of Long Beach General Plan (2004), the site is not located within an area identified as having a potential for liquefaction. As discussed in Section 4.2, the soils encountered during exploration consist of Pleistocene age old paralic deposits which are generally stiff to hard or dense. Based on the dense and well consolidated nature of the old paralic deposits, it is our opinion that the potential for liquefaction of the site soils is very low. Further, no surface manifestations of liquefaction, including settlement and lateral spread, are expected at the subject site.

6.5 Landslides

According to the Los Angeles County Safety Element (Leighton, 1990), the site is not located within an area identified as having a potential for slope instability. Additionally, according to the California Geological Survey (1999), the site is not located within an area identified as having a potential for seismic slope instability. The site and surrounding vicinity generally slopes gently to the south and southwest. There are no known landslides near the site, nor is the site in the path of any known or potential landslides. Therefore, the potential for slope stability hazards at the site are 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 Los Angeles County Safety Element (Leighton, 1990) indicates that the site is not located within the inundation boundaries of upgradient dams or reservoirs. Therefore, the potential for earthquake-induced flooding to adversely impact the site is considered very low.

6.7 Tsunamis, Seiches and Flooding

The site is located approximately 0.7 mile from the coastline at an elevation of approximately 27 to 30 feet above mean sea level (MSL). The site is not within a state-designated tsunami inundation area (CGS, 2009). Additionally, the City of Long Beach (2004) indicates the potential for the site to be inundated by a tsunami is very low. Therefore, tsunamis are not considered a significant hazard at the site.

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

The site is in an area of minimal flooding potential (Zone X) as defined by the Federal Emergency Management Agency (FEMA, 2017; Los Angeles County Department of Public Works, 2017b).

6.8 Oil Fields & Methane

Based on a review of the California Division of Oil, Gas and Geothermal Resources (DOGGR) Oil and Gas Well Location Map W1-6, Sheet 131, the site is located within the boundaries of the Long Beach Downtown Oil Field. No oil wells are located in the immediate vicinity of the site. However, due to the voluntary nature of record reporting by the oil well drilling companies, wells may be improperly located or not shown on the location map. Other wells could be encountered during construction. Any wells encountered will need to be properly abandoned in accordance with the current requirements of the DOGGR.

Since the site is located within the boundaries of the Long Beach Oil Field, there is a potential for methane and other volatile gases to occur at the site which could require a permanent methane gas control system beneath the proposed buildings. 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. Subsidence commonly occurs in such small magnitudes and over such large areas that is it generally imperceptible at an individual locality. Accordingly, it affects only regionally extensive structures sensitive to slight elevation changes, such as canals and pipelines. The rate of elevation change is usually uniform over a large enough area that it does not result in differential settlements that would cause damage to individual buildings.

Within the Long Beach area, a substantial level of subsidence has occurred between 1926 through 1967 due to petroleum production from the Wilmington Oil Field and as much as 30 feet of subsidence has been recorded near the Navy drydock on Terminal Island (City of Long Beach, 2004). The site is located on the northeastern edge of this area of documented subsidence.

As of 1958 local agencies began full-scale-water injection operations to impede further subsidence within the within the Long Beach area. In addition, subsidence is continually monitored by a network of 5 microearthquake monitoring stations that have been in operation since 1971 (City of Long Beach, 2004). As a result, no further manifestation of subsidence has occurred in the area since the implementation of this system. As long as the water injection operations are maintained and the ground surface is monitored to control changes, the potential for subsidence to impact the proposed development is low.

7. CONCLUSIONS AND RECOMMENDATIONS

7.1 General

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

7.1.2 Up to 6½ 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 7.6). 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, a conventional foundation system (shallow spread foundations) may be utilized for support of the proposed structure, provided foundations derive support in the competent old paralic deposits found at and below a depth of 20 feet below the existing ground surface. All foundation excavations must be observed and approved by the Geotechnical Engineer (a representative of Geocon), prior to placing steel or concrete.

7.1.4 Due to the preliminary nature of the project at this time, wall and column loads were not available. Once building loads become available and elevations are established, additional analyses may be necessary to reevaluate the anticipated total and differential settlements between the foundation elements, particularly across the transition from the 7-story to the 18-story tower. Updated foundation design recommendations will be provided as necessary in an addendum report.

7.1.5 Groundwater was encountered during current and prior site explorations at depths of approximately 29 and 32 feet below existing ground surface, and may be encountered near the excavation bottom. If the excavation bottom is determined to be excessively soft, stabilization of the bottom of the excavation may be required in order to provide a firm working surface upon which engineered fill can be placed and heavy equipment can operate. Recommendations for earthwork and bottom stabilization are provided in the *Grading* section of this report (see Section 8.6).

7.1.6 Due to the subterranean nature of the proposed structure and the potential for seasonal fluctuation in the groundwater level, temporary dewatering measures may be required to mitigate groundwater seepage during deep excavation. Furthermore, if the subterranean

portion of the structure extends to depths greater than 25 feet below existing ground surface and is not designed for full hydrostatic pressure, a permanent dewatering system will be required to relieve and mitigate the water pressure. The historic high groundwater may be assumed at a depth of 25 feet for design. Recommendations for temporary and permanent dewatering are discussed in Sections 7.4 and 7.5 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 Excavations up to 25 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 and improvements, excavation of the proposed subterranean level will require sloping and/or shoring 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 7.18 of this report.

7.1.9 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-in to the proposed structure, may be supported on conventional foundations bearing on a minimum of 12 inches of newly placed engineered fill. Where excavation and compaction cannot be performed, foundations may derive support in the 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, 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.

7.1.10 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 7.23).

7.1.11 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.

7.1.12 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. Some caving should be anticipated in unshored excavations, especially where granular soils are encountered.

7.2.2 It is the responsibility of the contractor to ensure that all excavations and trenches are properly shored and maintained in accordance with applicable OSHA rules and regulations to maintain safety and maintain the stability of 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 7.17).

7.2.4 The upper 5 feet of existing site soils encountered during this investigation are considered to have a “low” expansive potential (EI = 30 and 43); and the soils are classified as “expansive” based on the 2016 California Building Code (CBC) Section 1803.5.3. Furthermore, soils with a “high” expansive potential (EI = 144) were encountered at a depth of 16 feet below the ground surface. 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 “low” 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 a potential for corrosion of buried ferrous metals exists on site. The results are presented in Appendix B (Figure B16) 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 B16) and indicate that the on-site materials possess "negligible" sulfate exposure to concrete structures as defined by 2016 CBC Section 1904and 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 Temporary Dewatering

7.4.1 Groundwater was encountered during site exploration at depths of 29 and 32 feet below existing ground surface. Based on the conditions encountered at the time of exploration, groundwater may be encountered during construction activities. The depth to groundwater present at the time of construction can be further verified during initial dewatering well or shoring pile installation. If groundwater is present above the depth of the subterranean level, temporary dewatering will be necessary to maintain a safe working environment during excavation and construction activities.

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

7.4.3 The embedment of perimeter shoring piles should be deepened as necessary to take into account any required excavations necessary to place an adjacent french drain system, or sub-slab drainage system, should it be deemed necessary. It is anticipated that a perimeter french drain will 24 inches in depth, or less, below the proposed excavation bottom. If a french drain is to remain on a permanent basis, it must be lined with filter fabric to prevent soil migration into the gravel.

7.5 Permanent Dewatering

7.5.1 If any portion of the structure extends below the historic high groundwater depth of 25 feet below the ground surface, such as elevator pits, 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.

7.5.2 A typical permanent sub-slab drainage system would consist of a twelve-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 six 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 over the waterproofing system for protection during placement of rebar and slab construction.

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

7.6 Grading

7.6.1 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.

7.6.2 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.

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

7.6.4 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.).

7.6.5 If construction is performed during the rainy season and the excavation bottom becomes saturated, stabilization measures may have to be implemented to prevent excessive disturbance the excavation bottom. Should this condition exist, rubber tire equipment should not be allowed in the excavation bottom until it is stabilized or extensive soil disturbance could result.

7.6.6 If a permanent dewatering system is to be installed, subgrade stabilization may be accomplished by placing a one-foot thick layer of washed, angular 3/4-inch gravel atop a stabilization fabric (Mirafi 500X or equivalent), subsequent to subgrade approval. This procedure should be conducted in sections until the entire excavation bottom has been blanketed by fabric and gravel. Heavy equipment may operate upon the gravel once it has been placed. The gravel should be compacted to a dense state utilizing a vibratory drum roller. The placement of gravel at the subgrade level should be coordinated with the temporary or permanent dewatering of the site. The gravel and fabric system will function as both a permeable material for any necessary dewatering procedures as well as a stable material upon which heavy equipment may operate. It is recommended that the contractor meet with the Geotechnical Engineer to discuss this procedure in more detail.

7.6.7 Where temporary or permanent dewatering is not required, an alternative method of subgrade stabilization would consist of introducing a thin lift of three to six-inch diameter crushed angular rock into the soft excavation bottom. The use of crushed concrete will also be acceptable. The crushed rock should be spread thinly across the excavation bottom and pressed into the soils by track rolling or wheel rolling with heavy equipment. It is very important that voids between the rock fragments are not created so the rock must be thoroughly pressed or blended into the soils. All subgrade soils must be properly compacted and proof-rolled in the presence of the Geotechnical Engineer (a representative of Geocon West, Inc.).

7.6.8 The foundation system for the proposed structure may derive support in the competent old paralic deposits found at and below a depth of 20 feet. Foundations should be deepened as necessary to extend into satisfactory soils and must be observed and approved in writing by the Geotechnical Engineer (a representative of Geocon West, Inc.).

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

7.6.10 Foundations for small outlying structures, such as block walls up to 6 feet in height, planter walls or trash enclosures, which will not be tied to a proposed structure, may be supported on a minimum of 12 inches of newly placed engineered fill that extends at least 12 inches beyond the foundation footprint area. Where excavation and compaction cannot be performed, such as adjacent to existing improvements, foundations may derive support in the competent old paralic deposits, and should be deepened as necessary to maintain a minimum 12-inch embedment into the recommended bearing materials. Should the soils exposed in the excavation bottom be 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 should be performed at the direction of the Geocon representative.

7.6.11 Although not anticipated for this project, all imported fill shall be observed, tested and approved by Geocon West, Inc. prior to importing to the site. Rocks larger than 6 inches in diameter shall not be used in the fill. If necessary, import soils to be used in the building pad area 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 B16).

7.6.12 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 one 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).

7.6.13 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.

7.7 Foundation Design

7.7.1 The proposed structure may be supported on a conventional foundation system deriving support in the undisturbed old paralic deposits found at and below a depth of 20 feet. All foundation excavations must be observed and approved in writing by the Geotechnical Engineer (a representative of Geocon), prior to placing steel or concrete.

7.7.2 Once building loads become available and elevations are established, additional analyses may be necessary to reevaluate the anticipated total and differential settlements between the foundation elements, particularly across the transition from the seven-story to the 18-story tower. Updated foundation design recommendations will be provided as necessary in an addendum report.

7.7.3 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.

7.7.4 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.

7.7.5 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.

7.7.6 Continuous footings should be reinforced with a minimum of four No. 4 steel reinforcing bars, two placed near the top of the footing and two near the bottom. The reinforcement for isolated spread footings should be designed by the project structural engineer.

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

7.7.8 If the proposed structure 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 slab in feet. For design purposes the water table may be assumed at 25 feet below the existing ground surface.

7.7.9 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.

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

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

7.8 Foundation Settlement

7.8.1 The maximum expected static settlement for a structure supported on a conventional foundation system deriving support in the competent old paralic deposits found at or below a depth of 20 feet is estimated to be less than 1 inch and occur below the heaviest loaded structural element. Settlement of the foundation system is expected to occur on initial application of loading. Differential settlement is not expected to exceed ½ inch over a distance of 20 feet.

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

7.9 Miscellaneous Foundations

7.9.1 Small outlying miscellaneous structures, such as property line walls up to 6 feet in height, planter walls and trash enclosures, which will not be tied to a proposed structure, may be supported on a minimum of 12 inches of newly placed engineered fill. Where excavation and compaction cannot be performed, foundations may bear in the competent old paralic deposits, and should be deepened as necessary to maintain a minimum 12-inch embedment into the recommended bearing materials.

7.9.2 Miscellaneous foundations may be designed for a bearing value of 1,500 psf, and should be a minimum of 12 inches in width, 24 inches in depth below the lowest adjacent grade and 12 inches into the recommended bearing material. The allowable bearing pressure may be increased by up to one-third for transient loads due to wind or seismic forces. Should the soils exposed in the excavation bottom be 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. As an alternative, excavations should be deepened as necessary to extend into satisfactory soils.

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

7.10 Lateral Design

7.10.1 Resistance to lateral loading may be provided by friction acting at the base of foundations, slabs and by passive earth pressure. An allowable coefficient of friction of 0.45 may be used with the dead load forces in the properly compacted engineered fill and competent old paralic deposits.

7.10.2 Passive earth pressure for the sides of foundations and slabs poured against competent old paralic deposits may be computed as an equivalent fluid having a density of 300 pcf with a maximum earth pressure of 3,000 pcf. Below a depth of 25 feet, passive earth pressure for the sides of foundations and slabs poured against competent old paralic deposits may be computed as an equivalent fluid having a density of 150 pcf with a maximum earth pressure of 1,500 pcf (these values have been adjusted for buoyant forces). When combining passive and friction for lateral resistance, the passive component should be reduced by one-third.

7.11 Concrete Slabs-on-Grade

7.11.1 Unless specifically evaluated and designed by a qualified structural engineer, the slab and ramp for the subterranean parking garage should be a minimum of 5 inches thick and reinforced with No. 3 steel reinforcing bars placed 18 inches on center in both horizontal directions, positioned vertically near the slab midpoint. The concrete slab-on-grade for the parking garage may derive support directly in the competent old paralic deposits found at the excavation bottom as well as compacted soils, if necessary. Any disturbed soils should be properly compacted for slab support. Soil placed and compacted for ramp 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.

7.11.2 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.

7.11.3 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.

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

7.11.5 Exterior slabs, not subject to traffic loads, should be at least 4 inches thick and reinforced with No. 3 steel reinforcing bars placed 24 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 moisture conditioned to near content and properly compacted to at least 95 percent relative compaction, as determined by ASTM Test Method D 1557 (latest edition). Crack control joints should be spaced at intervals not greater than 10 feet and should be constructed using saw-cuts or other methods as soon as practical following

concrete placement. Crack control joints should extend a minimum depth of one-fourth the slab thickness. The project structural engineer should design construction joints as necessary.

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

7.12 Retaining Wall Design

7.12.1 The recommendations presented below are generally applicable to the design of rigid concrete or masonry retaining walls having a maximum height of 25 feet. In the event that walls significantly higher than 25 feet are planned, Geocon should be contacted for additional recommendations.

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

7.12.3 Retaining walls with a level backfill surface that are not restrained at the top should be designed utilizing a triangular distribution of pressure (active pressure) of 35 pcf.

7.12.4 Restrained walls are those that are not allowed to rotate more than 0.001H (where H equals the height of the retaining portion of the wall in feet) at the top of the wall. Where walls are restrained from movement at the top or no movement is desired, walls may be designed utilizing a triangular distribution of pressure (at-rest pressure) of 55 pcf.

7.12.5 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.

7.12.6 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.

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

7.12.8 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:

$$For \quad x/H \leq 0.4$$

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

and

$$For \quad x/H > 0.4$$

$$\sigma_H(x,z) = \frac{1.26\left(\frac{x}{H}\right)^2\left(\frac{z}{H}\right)}{\left[\left(\frac{x}{H}\right)^2 + \left(\frac{z}{H}\right)^2\right]^2} \frac{Q_L}{H}$$

where x is the distance from the face of the excavation to the vertical line-load, H is the distance from the bottom of the footing to the bottom of excavation, z is the depth at which the horizontal pressure is desired, QL is the vertical line-load and σH is the horizontal pressure at depth z.

7.12.9 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:

$$For \quad x/H \le 0.4$$

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

and

$$For \quad x/H > 0.4$$

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

then

$$\sigma_H^{'}(z) = \sigma_H(z)\cos^2(1.1\theta)$$

where x is the distance from the face of the excavation 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, σ is the vertical pressure at depth z, Θ is the angle between a line perpendicular to the bulkhead and a line from the point-load to half the pile spacing at the bulkhead, and σ_H is the horizontal pressure at depth z.

7.12.10 In addition to the recommended earth pressure, the upper ten feet of the subterranean wall adjacent to the street 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 ten feet from the subterranean walls, the traffic surcharge may be neglected.

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

7.13 Dynamic (Seismic) Lateral Forces

7.13.1 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).

7.13.2 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.

7.14 Retaining Wall Drainage

7.14.1 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 5). The clean bottom and subdrain pipe, behind a retaining wall, should be observed by the Geotechnical Engineer (a representative of Geocon), prior to placement of gravel or compacting backfill.

7.14.2 As an alternative, a plastic drainage composite such as Miradrain or equivalent may be installed in continuous, 4-foot wide columns along the entire back face of the wall, at 8 feet on center. The top of these drainage composite columns should terminate approximately 18 inches below the ground surface, where either hardscape or a minimum of 18 inches of relatively cohesive material should be placed as a cap (see Figure 6). These vertical columns of drainage material would then be connected at the bottom of the wall to a one-cubic-foot rock pocket or collection strip drained by a 4-inch subdrain pipe.

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

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

7.15 Elevator Pit Design

7.15.1 The elevator pit slab and retaining wall should be designed by the project structural engineer. Elevator pit walls may be designed in accordance with the recommendations in the *Foundation Design* and *Retaining Wall Design* sections of this report (see Sections 7.7 and 7.12).

7.15.2 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.

7.15.3 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 7.14).

7.15.4 It is suggested that the elevator pit 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.

7.16 Elevator Piston

7.16.1 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 support or pile support, especially if the drilling is performed subsequent to the foundation or pile construction.

7.16.2 Casing may be required if caving is experienced in the drilled excavation, especially if the excavation is conducted below the groundwater level. 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.

7.16.3 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.

7.17 Temporary Excavations

7.17.1 Excavations up to 25 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.

7.17.2 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 12 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 7.18 of this report.

7.17.3 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. Our personnel should inspect the soils exposed in the cut slopes during excavation so that modifications of the slopes can be made if variations in the soil conditions occur. All excavations should be stabilized within 30 days of initial excavation.

7.18 Shoring – Soldier Pile Design and Installation

7.18.1 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.

7.18.2 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.

7.18.3 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 subgrade stabilization activities, foundations, and/ or adjacent drainage systems.

7.18.4 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 7.12).

7.18.5 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 360 psf per foot for the portion of the pile above the water table, and 180 psf per foot for the portion of the pile below the water table (value has been reduced for buoyant forces). 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 capacity may be doubled for isolated piles spaced more than twice the diameter. To develop the full lateral value, provisions should be implemented to assure firm contact between the soldier piles and the undisturbed soils.

7.18.6 Groundwater was encountered during exploration at depths of 29 and 32 feet, and the contractor should be prepared for groundwater during pile installation. The depth to groundwater at the time of construction may be further verified during initial shoring pile installation. Piles placed below the water level require the use of a tremie 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.

7.18.7 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.

7.18.8 Casing will likely be required since caving is expected in the saturated and/or granular soils. If casing is used, extreme care should be employed so that the pile is not pulled apart as the casing is withdrawn. At no time should the distance between the surface of the concrete and the bottom of the casing be less than five 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.

7.18.9 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.

7.18.10 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.

7.18.11 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.

7.18.12 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.

7.18.13 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.

7.18.14 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.

7.18.15 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 0.45 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 340 psf (value has been reduced for buoyant forces).

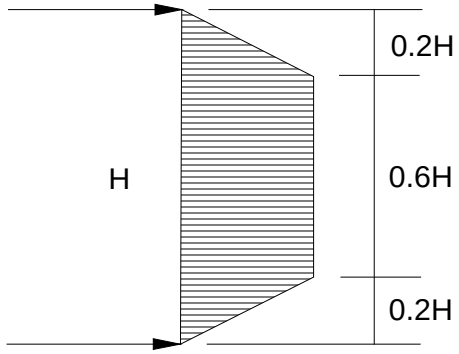
7.18.16 Continuous lagging between soldier piles will be required. It is recommended that the exposed soils be observed by the Geotechnical Engineer (a representative of Geocon West, Inc.), to verify the presence of any cohesive soils and the areas where lagging may potentially be omitted. The time between lagging excavation and lagging placement should be as short as possible. As a minimum, the upper 5 feet of lagging should be backfilled with minimum two-sack slurry.

7.18.17 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.

7.18.18 For the design of shoring, it is recommended that an equivalent fluid pressure based on the following table, be utilized for design. A diagram depicting the trapezoidal pressure distribution of lateral earth pressure is provided below the table.

HEIGHT OF CANTILEVERED 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	25	16H

Trapezoidal Distribution of Pressure



7.18.19 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, or the pile is restrained from movement by bracing or a tie back anchor, an at-rest pressure of 45 pcf should be considered for design purposes.

7.18.20 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.

7.18.21 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:

$$For \quad x/H \leq 0.4$$

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

and

$$For \quad x/H > 0.4$$

$$\sigma_H(x,z) = \frac{1.26\left(\frac{x}{H}\right)^2\left(\frac{z}{H}\right)}{\left[\left(\frac{x}{H}\right)^2 + \left(\frac{z}{H}\right)^2\right]^2} \frac{Q_L}{H}$$

where x is the distance from the face of the excavation to the vertical line-load, H is the distance from the bottom of the footing to the bottom of excavation, z is the depth at which the horizontal pressure is desired, QL is the vertical line-load and σH is the horizontal pressure at depth z.

7.18.22 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:

$$For \quad x/H \le 0.4$$

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

and

$$For \quad x/H > 0.4$$

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

then

$$\sigma_H^{'}(z) = \sigma_H(z)\cos^2(1.1\theta)$$

where x is the distance from the face of the excavation 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, σ is the vertical pressure at depth z, Θ is the angle between a line perpendicular to the bulkhead and a line from the point-load to half the pile spacing at the bulkhead, and σ_H is the horizontal pressure at depth z.

7.18.23 In addition to the recommended earth pressure, the upper ten feet of the shoring adjacent to the street 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 ten feet from the shoring, the traffic surcharge may be neglected.

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

7.18.25 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.

7.18.26 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 sot 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.

7.19 Temporary Tie-Back Anchors

7.19.1 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.

7.19.2 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 – 1,700 pounds per square foot

7.19.3 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.

7.20 Anchor Installation

7.20.1 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.

7.21 Anchor Testing

7.21.1 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.

7.21.2 At least ten 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.

7.21.3 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.

7.21.4 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.

7.21.5 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.

7.22 Internal Bracing

7.22.1 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 1,000 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.

7.23 Stormwater Infiltration

7.23.1 During the December 27, 2017 site exploration, boring B8 was utilized to perform percolation testing. The boring was advanced to the depths listed in the table below. Slotted casing was placed in the boring, and the annular space between the casing and excavation was filled with gravel. The boring was then filled with water to pre-saturate the soils. On December 28, 2016, 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 Reduction Factor (Rf), to convert the field-measured percolation rate to an infiltration rate, is also shown in the table below. This value has been calculated in accordance with the Boring Percolation Test Procedure in the County of Los Angeles Department of Public Works GMED *Guidelines for Design, Investigation, and Reporting Low Impact Development Stormwater Infiltration* (December 2014). Calculation of the percolation rate, reduction factor, and infiltration rate are provided Figure 7.

Boring	Infiltration Depth (ft)	Measured Percolation Rate (in / hour)	Reduction Factor (Rf)	Design Infiltration Rate (in / hour)
B8	15-20	42.0	13.4	3.1

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

7.23.3 It is our further opinion that infiltration of stormwater and 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.

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

7.23.5 Where the 10-foot horizontal setback cannot be maintained between the infiltration system and an adjacent footing, and the infiltration system penetrates below the foundation influence line, the proposed stormwater infiltration system must be designed to resist the surcharge from the adjacent foundation. The foundation surcharge line may be assumed to project down away from the bottom of the foundation at a 1:1 gradient. The stormwater infiltration system must still be sufficiently deep to maintain the 10-foot vertical offset between the bottom of the footing and the zone of saturation.

7.23.6 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.

7.23.7 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).

7.24 Surface Drainage

7.24.1 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.

7.24.2 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 five 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.

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

7.25 Plan Review

7.25.1 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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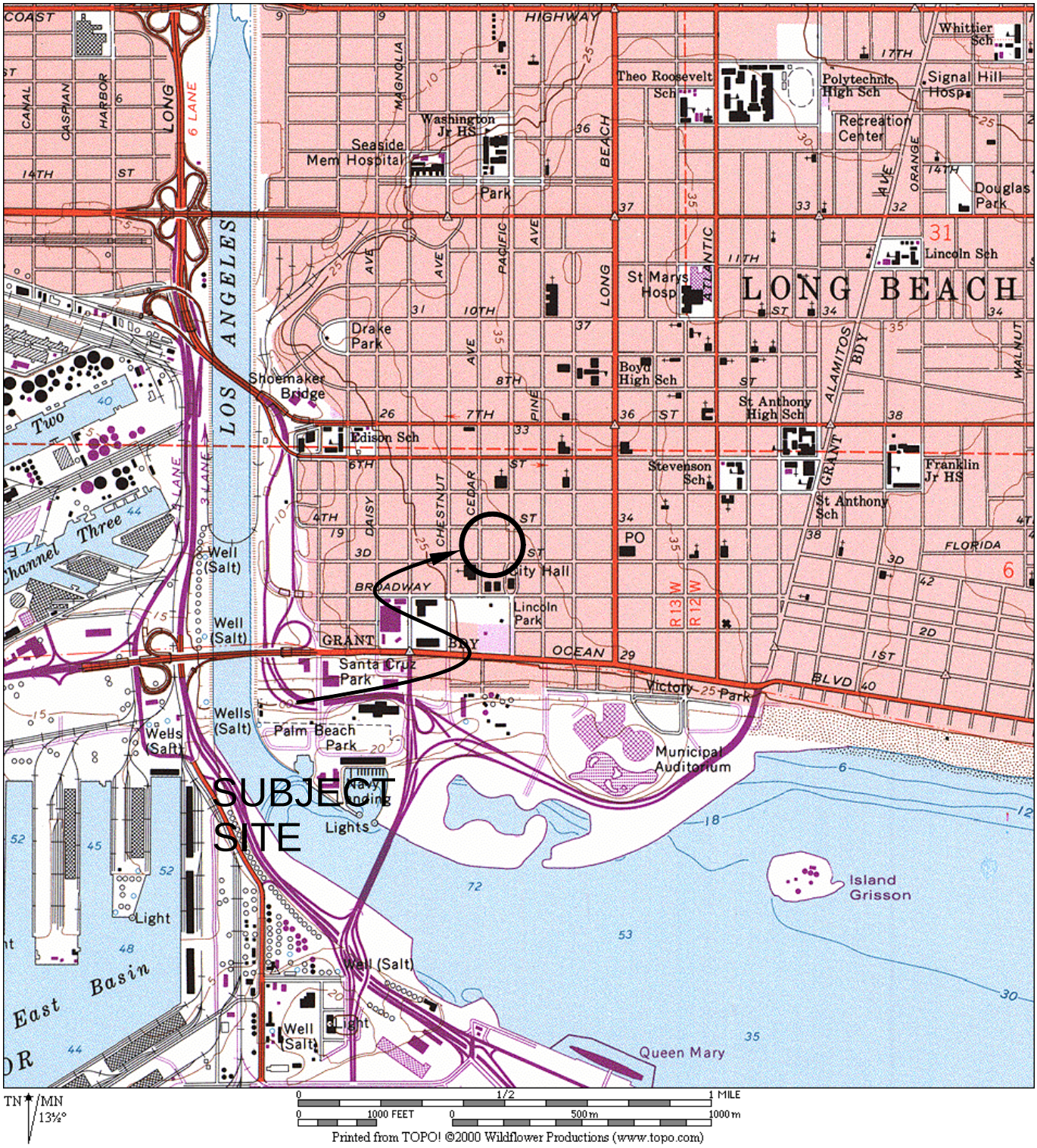

REFERENCE:U.S.G.S. TOPOGRAPHIC MAPS, 7.5 MINUTE SERIES, LONG BEACH, CA QUADRANGLE

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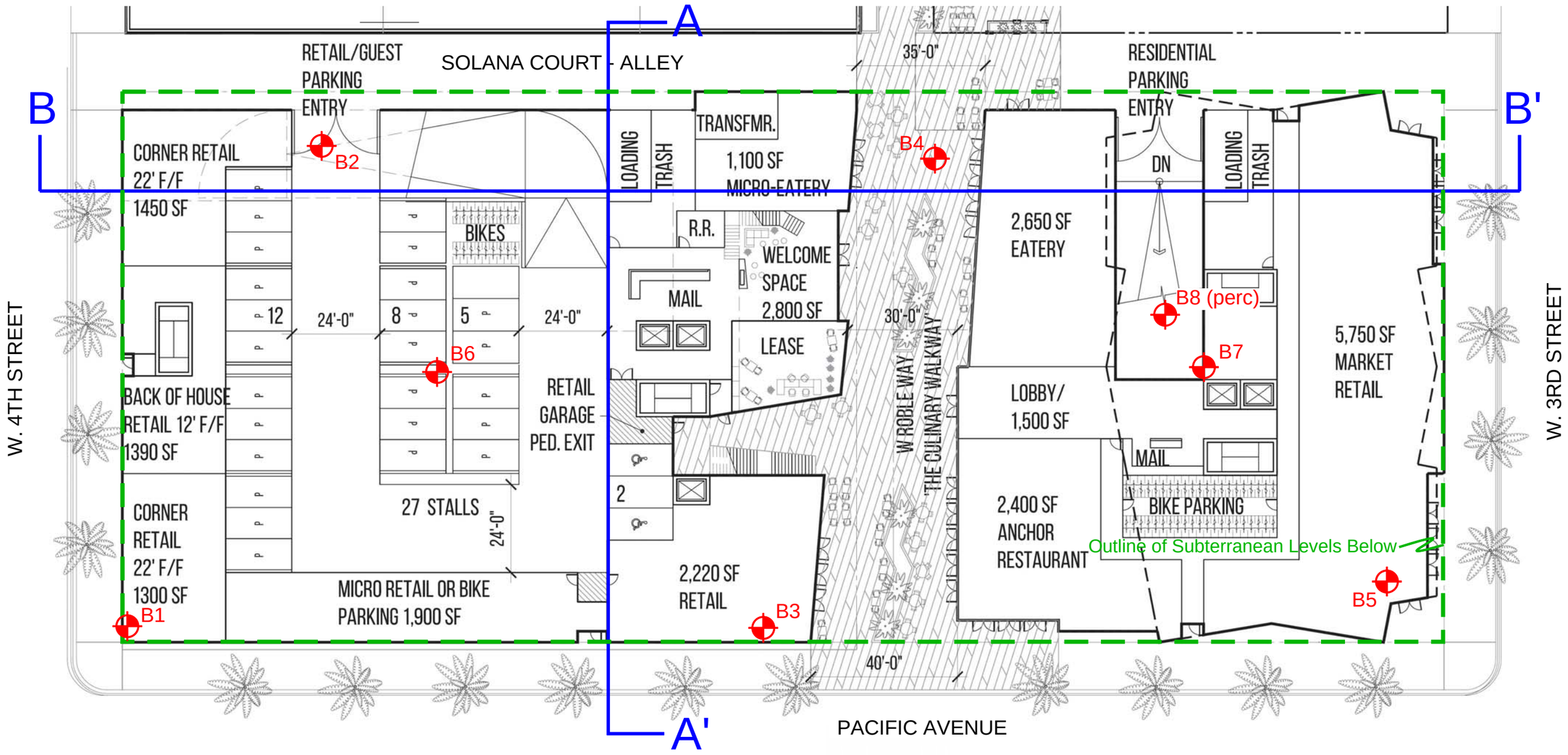
VICINITY MAP

LONG BEACH APARTMENTS
ENSEMBLE INVESTMENTS
3RD STREET & PACIFIC AVENUE
LONG BEACH, CALIFORNIA

DRAFTED BY: MDS	CHECKED BY: SFK	JAN. 2017	PROJECT NO. A9542-06-01	FIG. 1



LEGEND

B8 Approximate Location of Boring

Limits of Proposed Development

0 30' 60'

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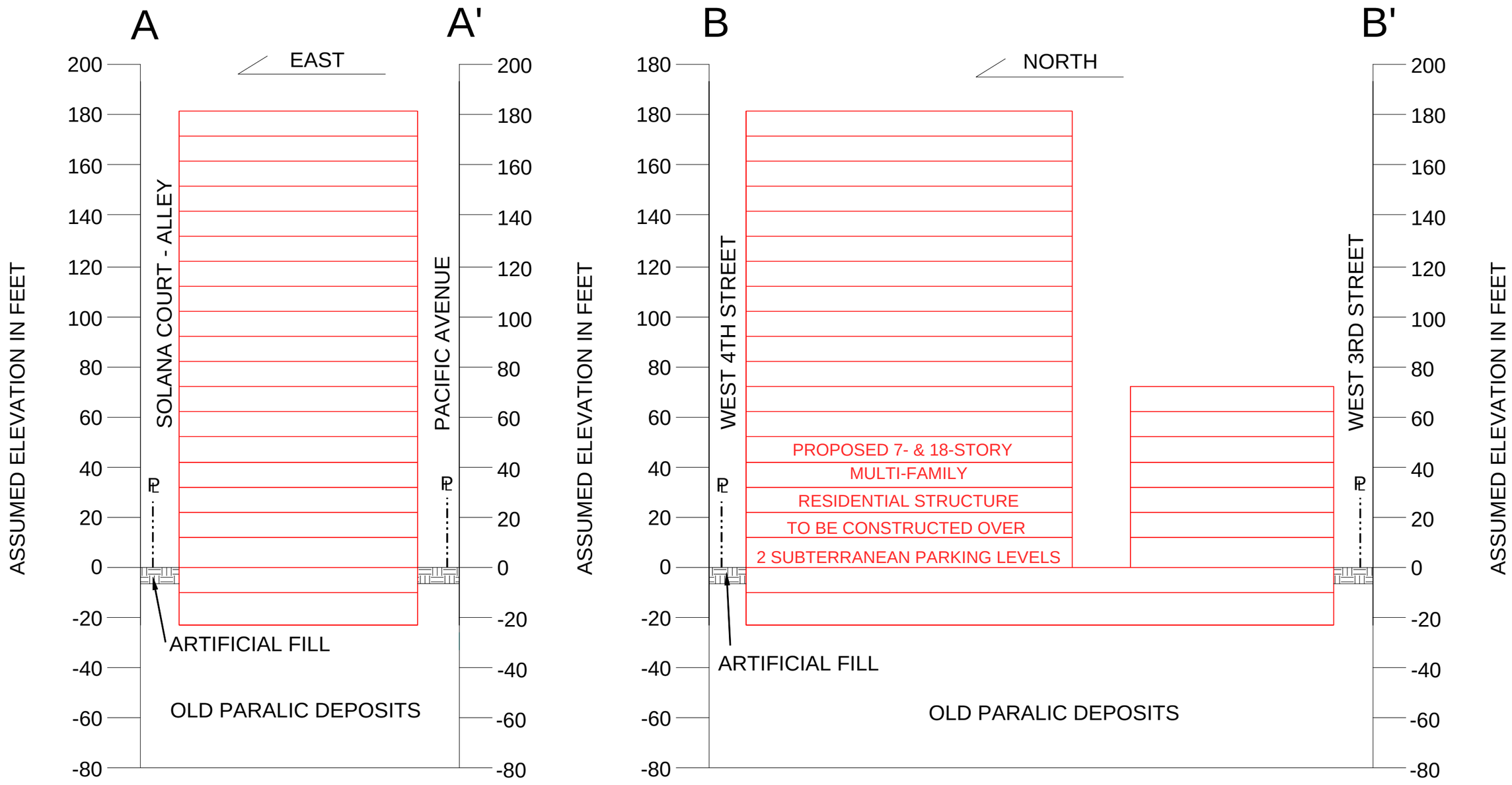
SITE PLAN

LONG BEACH APARTMENTS
ENSEMBLE INVESTMENTS
3RD STREET & PACIFIC AVENUE
LONG BEACH, CALIFORNIA

JAN. 2017 | PROJECT NO. A9542-06-01 | FIG. 2A



NOTE: CROSS-SECTIONS ARE FOR ILLUSTRATION PURPOSES ONLY, NOT FOR CONSTRUCTION

0 40' 80'

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CROSS SECTIONS

LONG BEACH APARTMENTS
ENSEMBLE INVESTMENTS
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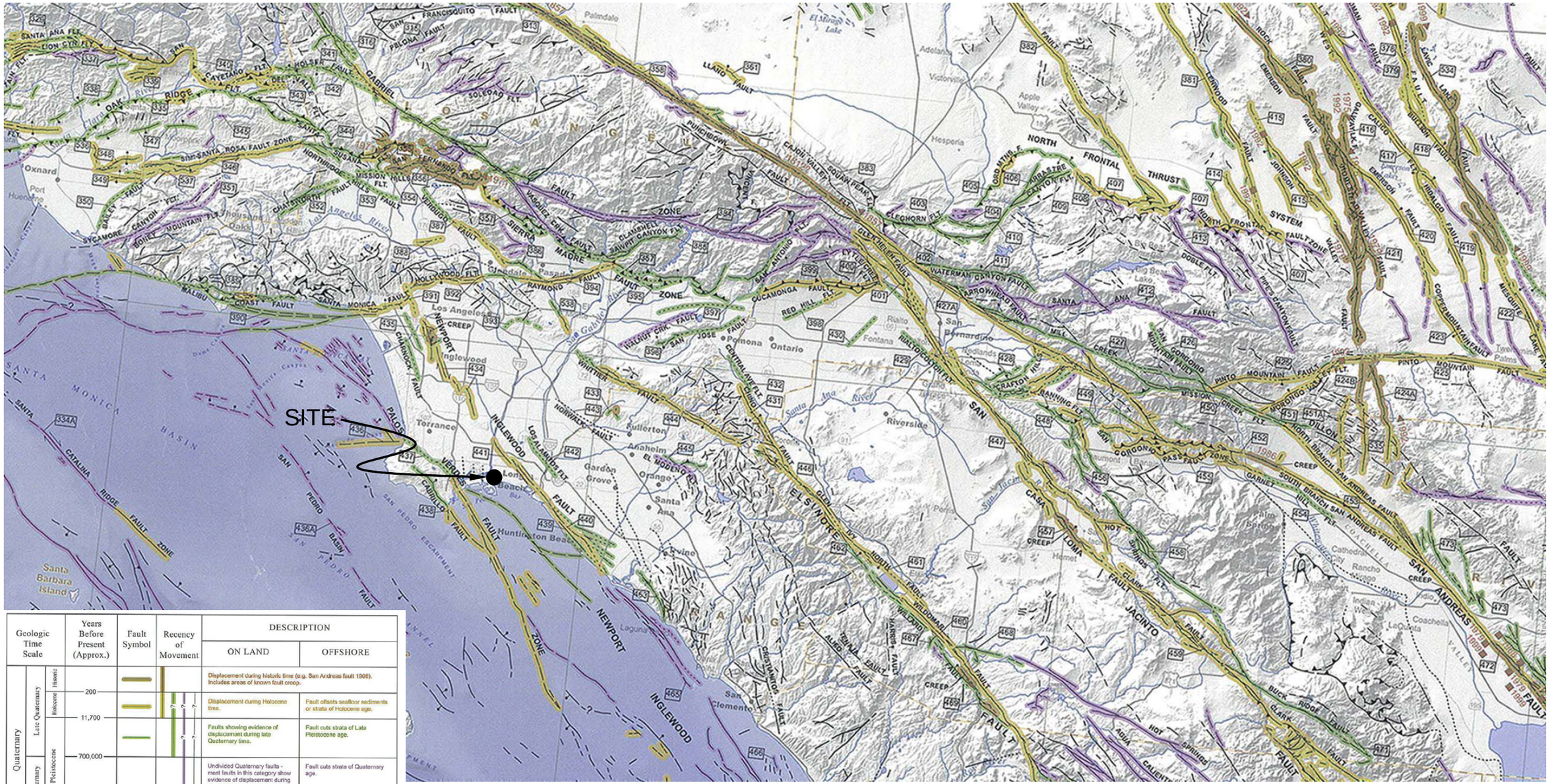
LONG BEACH, CALIFORNIA

JAN. 2017 | PROJECT NO. A9542-06-01 | FIG. 2B

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



Geologic Time Scale			Years Before Present (Approx.)	Fault Symbol	Recency of Movement	DESCRIPTION: ON LAND	DESCRIPTION: OFFSHORE
Quaternary	Late Quaternary	Historic				Displacement during historic time (e.g. San Andreas fault 1906). Includes areas of known fault creep.	
			200				
		Holocene				Displacement during Holocene time.	Fault offsets seafloor sediments or strata of Holocene age.
			11,700				
		Pleistocene				Faults showing evidence of displacement during late Quaternary time.	Fault cuts strata of Late Pleistocene age.
			700,000				
	Early Quaternary					Undivided Quaternary faults - most faults in this category show evidence of displacement during the last 1,600,000 years; possible exceptions are faults which displace rocks of undifferentiated Plio-Pleistocene age	Fault cuts strata of Quaternary age.
			1,600,000*				
Pre-Quaternary						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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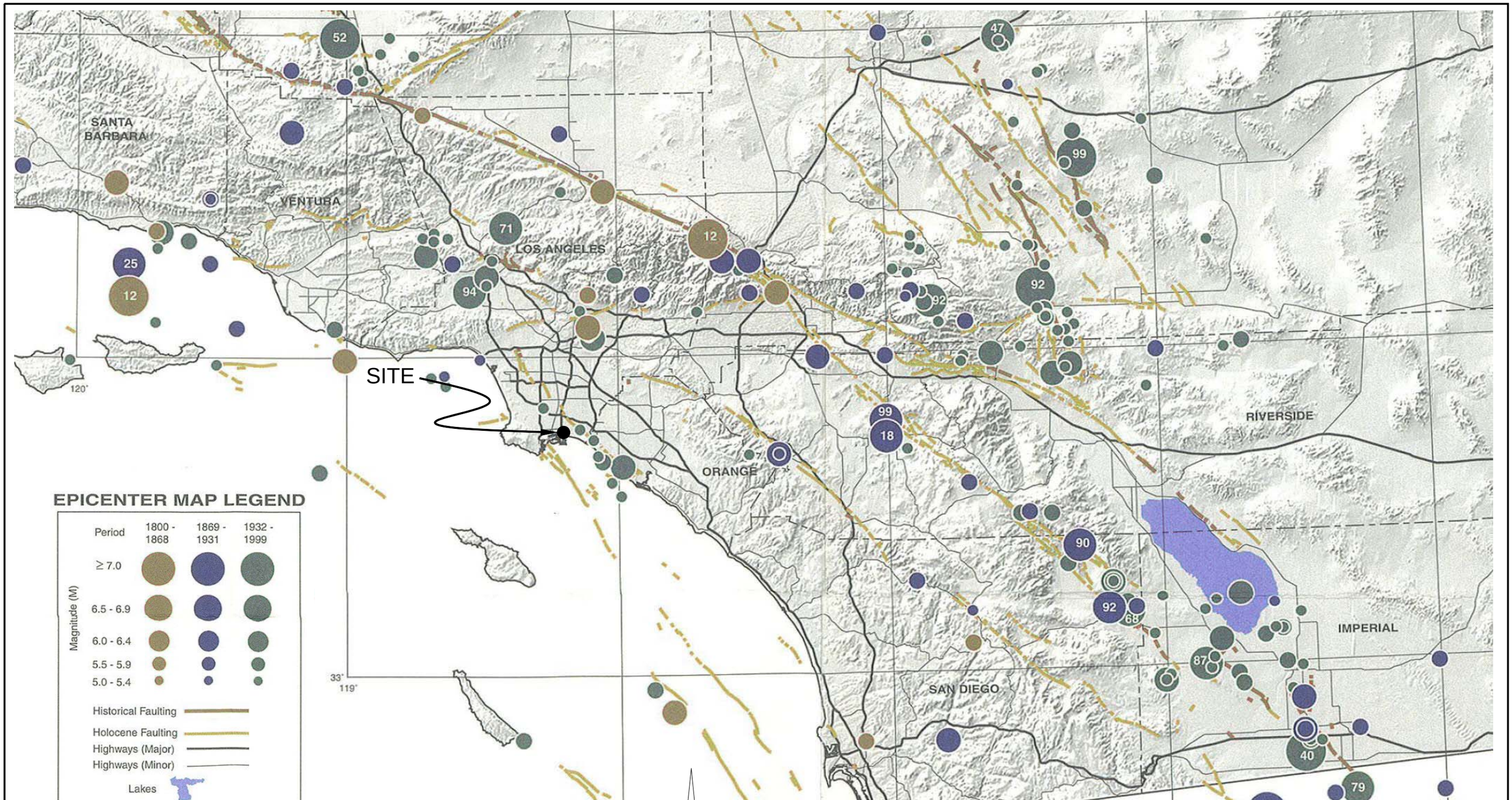
DRAFTED BY: MDS | CHECKED BY: SFK

REGIONAL FAULT MAP

LONG BEACH APARTMENTS
ENSEMBLE INVESTMENTS
3RD STREET & PACIFIC AVENUE
LONG BEACH, CALIFORNIA

JAN. 2017 | PROJECT NO. A9542-06-01 | FIG. 3



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

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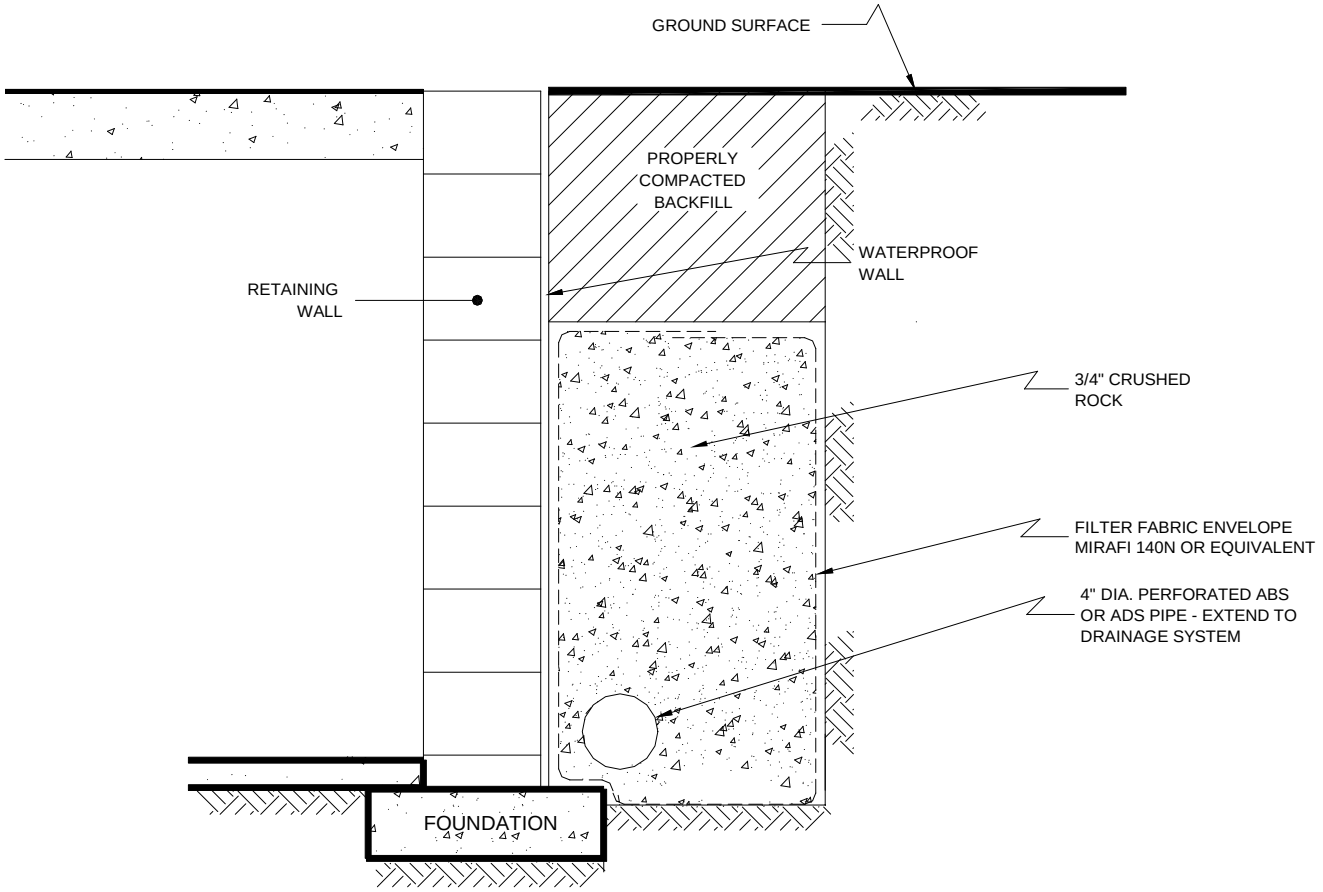
DRAFTED BY: MDS | CHECKED BY: SFK

REGIONAL SEISMICITY MAP

LONG BEACH APARTMENTS
ENSEMBLE INVESTMENTS
3RD STREET & PACIFIC AVENUE
LONG BEACH, CALIFORNIA

JAN. 2017 | PROJECT NO. A9542-06-01 | FIG. 4



NO SCALE

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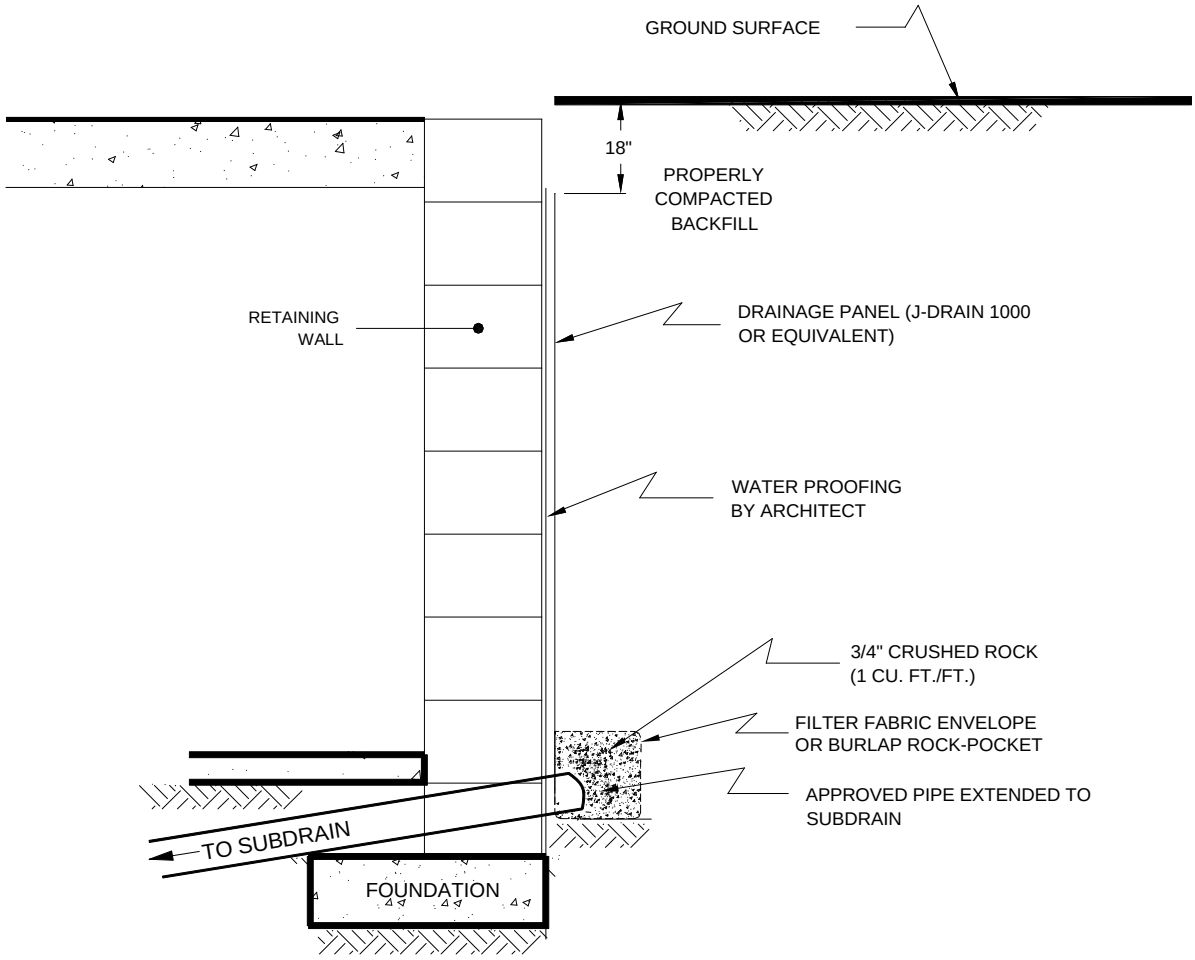
RETAINING WALL DRAIN DETAIL

LONG BEACH APARTMENTS
ENSEMBLE INVESTMENTS
3RD STREET & PACIFIC AVENUE
LONG BEACH, CALIFORNIA

DRAFTED BY: PZ	CHECKED BY: JTA	JAN. 2017	PROJECT NO. A9542-06-01	FIG. 5



NO SCALE

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PHONE (818) 841-8388 - FAX (818) 841-1704

RETAINING WALL DRAIN DETAIL

LONG BEACH APARTMENTS
ENSEMBLE INVESTMENTS
3RD STREET & PACIFIC AVENUE
LONG BEACH, CALIFORNIA

DRAFTED BY: PZ	CHECKED BY: JTA	JAN. 2017	PROJECT NO. A9542-06-01	FIG. 6

PERCOLATION TEST RESULTS

Boring 8

Project No:	A9542-06-01	Boring Diameter, DIA:	8	inches
Project Name:	3rd & Pacific	Boring Depth:	20	feet
Testing Date:	12/28/2016	Boring Depth:	240	inches
Tested By:	PZ			

Reading Number	Initial Water Depth (ft)	Final Water Depth (ft)	Water Drop (ft)	Water Drop (in)	ΔT (min)	Percolation Rate (in/hour)
1	15.00	16.78	1.78	21.4	30	42.7
2	15.00	16.72	1.72	20.6	30	41.3
3	15.00	16.75	1.75	21.0	30	42.0
Average:	15.00	16.75			Preadjusted Perc Rate*	**42.0**

* Based only on Stabilized Readings

Initial Water Depth, d_1 =	60	inches
Final Water Depth, d_2 =	39	inches
Water Level Drop, Δd =	21	inches
Boring Diameter, DIA =	8	inches

$$R_f = \left(\frac{2d_1 - \Delta d}{DIA}\right) + 1$$

Reduction Factor, R_f = **13.4**

Infiltration Rate = **3.1** inches/hour

Figure 7

APPENDIX A

APPENDIX A

FIELD INVESTIGATION

The site was initially explored on May 25, 2010, by conducting five 8-inch diameter borings utilizing a truck-mounted hollow-stem auger drilling machine. The borings were advanced to depths between 20½ and 30½ feet below the existing ground surface. Additional site exploration was performed on December 27, 2016 by excavating three 8-inch diameter borings to depths of 20 and 60½ feet below the existing ground surface using a truck-mounted hollow-stem auger drilling machine. Representative and relatively undisturbed samples were obtained by driving a 3-inch O. D., California Modified Sampler into the "undisturbed" soil mass with blows from a 140-pound hammer falling 30 inches. The California Modified Sampler was equipped with 1-inch high by 2³/₈-inch diameter brass sampler rings to facilitate removal and testing. Bulk samples were also obtained.

The soil conditions encountered in the borings were visually examined, classified and logged in general accordance with the Unified Soil Classification System (USCS). Logs of the borings are presented on Figures A1 through A8. The logs depict the soil and geologic conditions encountered and the depth at which samples were obtained. The approximate locations of the borings are indicated the Site Plan (see Figure 2A).

PROJECT NO. A9542-06-01

BORING 1

ELEV. (MSL.) -- DATE COMPLETED **May 25, 2010**

EQUIPMENT **HOLLOW STEM AUGER** BY: **RG**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0					**Asphalt: 3 inches Base: 3 inches**			
					ARTIFICIAL FILL			
2					Silty Sand, medium dense, slightly moist, dark brown, some concrete and asphalt debris			
					- Concrete debris			
4								
6								
					OLD PARALIC DEPOSITS			
8	B1@8'			ML	Sandy Silt, stiff, slightly moist, brown, fine- to medium-grained - No recovery	31	--	--
10								
12	B1@11'			SM	Silty Sand, medium dense, slightly moist, light brown to brown, fine- to medium-grained, trace carbon deposits	23	112.2	12.2
14	B1@14'			SP	Sand, poorly graded, medium dense, slightly moist, light brown, fine- to medium-grained	35	95.7	5.6
16								
18	B1@18'			ML	Silt, stiff, slightly moist, brown	36	92.4	4.3
20	B1@20'			SM	Silty Sand, dense, slightly moist, light brown, fine-grained, trace carbon deposits	52	93.1	8.0

End at 20.5 feet.
Artificial fill to 6.5 feet.
No groundwater encountered.
Backfilled and tamped with soil cuttings.
Capped with asphalt.

*Penetration resistance for 140 pound hammer falling 30 inches.

Figure A-1,
Log of Boring 1, Page 1 of 1

A9542-06-01 BORING LOGS B1 - B5.GPJ

SAMPLE SYMBOLS

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

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

GEOCON

PROJECT NO. A9542-06-01

BORING 2

ELEV. (MSL.) -- DATE COMPLETED **May 25, 2010**

EQUIPMENT **HOLLOW STEM AUGER** BY: **RG**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0					**Asphalt: 4 inches Base: 4 inches**			
					ARTIFICIAL FILL Silty Sand, medium dense, slightly moist, dark brown, fine- to medium-grained			
2	B2@2'			SM	**OLD PARALIC DEPOSITS** Silty Sand, very loose, slightly moist, dark brown, fine- to medium-grained	4	113.3	11.0
4								
	B2@5'					6	111.6	11.2
6				ML	Sandy Silt, hard, slightly moist, reddish brown, fine-grained, trace carbon deposits			
	B2@7'					72	129.0	10.5
8								
10	B2@10'			SM	Silty Sand, dense, slightly moist, reddish brown, fine- to medium-grained	63	111.8	15.3
12								
	B2@13'			SP	Sand, poorly graded, medium dense, slightly moist, reddish brown to brown, fine- to medium-grained	26	93.3	5.5
14								
16				ML	Silt, stiff, slightly moist, gray, trace fine- to medium-grained sand			
	B2@17'					33	92.6	29.5
18								
20	B2@20'				- Hard, no sand content	85	95.9	28.6
22				SM	Silty Sand, dense, slightly moist, gray to reddish brown, fine- to medium-grained			
	B2@23'					81	113.7	15.6
24								
26	B2@26'			SP	Sand, poorly graded, very dense, slightly moist, light brown, fine- to medium-grained	88	101.8	12.4
28			▽		- Groundwater encoutered at 29 feet			

Figure A-2,
Log of Boring 2, Page 1 of 2

A9542-06-01 BORING LOGS B1 - B5.GPJ

SAMPLE SYMBOLS

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

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

GEOCON

PROJECT NO. A9542-06-01

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	**BORING 2** ELEV. (MSL.) -- DATE COMPLETED **May 25, 2010** EQUIPMENT **HOLLOW STEM AUGER** BY: **RG**	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
					MATERIAL DESCRIPTION			
30	B2@30'					34	105.2	20.6

End at 30.5 feet.
Artificial fill to 2 feet.
Groundwater encountered at 29 feet.
Backfilled and tamped with soil cuttings.
Capped with asphalt.

*Penetration resistance for 140 pound hammer falling 30 inches.

Figure A-2,
Log of Boring 2, Page 2 of 2

A9542-06-01 BORING LOGS B1 - B5.GPJ

SAMPLE SYMBOLS

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

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

GEOCON

PROJECT NO. A9542-06-01

BORING 3

ELEV. (MSL.) -- DATE COMPLETED **May 25, 2010**

EQUIPMENT **HOLLOW STEM AUGER** BY: **RG**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0					**Asphalt: 2.5 inches**			
					ARTIFICIAL FILL			
2	B3@2'			SM	Silty Sand, medium dense, slightly moist, reddish brown to brown, fine- to medium-grained, trace carbon deposits	28	124.7	9.8
					OLD PARALIC DEPOSITS			
4					Silty Sand, medium dense, slightly moist, reddish brown to brown, fine- to medium-grained, trace carbon deposits			
6	B3@5'			ML	Sandy Silt, stiff, slightly moist, reddish brown to brown, fine- to medium-grained	27	114.8	11.3
8	B3@8'					28	115.9	15.8
10					Silty Sand, loose, slightly moist, brown, fine- to medium-grained			
12	B3@11'			SM		11	107.7	7.1
14	B3@14'			SP	Sand, poorly graded, loose, slightly moist, reddish brown to light brown, fine- to medium-grained	17	95.5	8.3
16								
18	B3@17'				- Medium dense, light brown	33	99.4	2.2
20	B3@20'			ML	Silt, hard, slightly moist, gray, trace carbon deposits	81	99.2	25.5
22								
24	B3@23'			SM	Silty Sand, very dense, slightly moist, light brown with reddish brown stains, fine- to medium-grained	50(5")	103.5	13.5
	B3@25'				- Increase in silt content, light brown to reddish brown, trace shell fragments	41	108.1	18.6

End at 25.5 feet.
Artificial fill to 1.5 feet.
No groundwater encountered.
Backfilled and tamped with soil cuttings.
Capped with asphalt.

*Penetration resistance for 140 pound hammer falling 30 inches.

Figure A-3,
Log of Boring 3, Page 1 of 1

A9542-06-01 BORING LOGS B1 - B5.GPJ

SAMPLE SYMBOLS

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

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

GEOCON

PROJECT NO. A9542-06-01

BORING 4

ELEV. (MSL.) -- DATE COMPLETED **May 25, 2010**

EQUIPMENT **HOLLOW STEM AUGER** BY: **RG**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0					**Asphalt: 3.5 inches Base: 1 inches**			
					ARTIFICIAL FILL			
					Silty Sand, medium dense, slightly moist, dark brown to brown, fine- to medium-grained			
2	B4@2'					46	126.6	11.9
					OLD PARALIC DEPOSITS			
					Silty Sand, medium dense, slightly moist, brown, fine-grained			
4	B4@4'			SM	- Dense, increase in sand content	67	128.2	10.3
6	B4@6'			ML	Sandy Silt, hard, slightly moist, reddish brown, fine-grained, trace carbon deposits	47	120.9	13.2
8								
10	B4@6'			SM	Silty Sand, medium dense, slightly moist, brown, fine-grained	38	116.6	11.7
12	B4@12'				Sand, poorly graded, medium dense, slightly moist, light brown, fine- to medium-grained, trace silt	26	97.1	5.7
14								
16	B4@15'			SP	- No silt content	27	97.8	6.2
18	B4@18'				- Dense, fine- to medium-grained	54	105.9	3.2
20	B4@20'				- Medium dense	50	101.1	2.9

End at 20.5 feet.
Artificial fill to 2 feet.
No groundwater encountered.
Backfilled and tamped with soil cuttings.
Capped with asphalt.

*Penetration resistance for 140 pound hammer falling 30 inches.

Figure A-4,
Log of Boring 4, Page 1 of 1

A9542-06-01 BORING LOGS B1 - B5.GPJ

SAMPLE SYMBOLS

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

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

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PROJECT NO. A9542-06-01

BORING 5

ELEV. (MSL.) -- DATE COMPLETED **May 25, 2010**

EQUIPMENT **HOLLOW STEM AUGER** BY: **RG**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0					**Asphalt: 3 inches Base: 4 inches**			
					ARTIFICIAL FILL			
					Silty Sand, medium dense, slightly moist, dark brown, fine- to medium-grained			
2	B5@2'					31	125.5	5.8
					OLD PARALIC DEPOSITS			
					Sandy Silt, stiff, slightly moist, reddish brown, fine-grained			
4								
6	B5@5'			SM	- increase in sand content	36	121.8	15.1
8	B5@7'				- Silt, firm, slightly moist, light brown	19	122.3	10.7
					Silty Sand, loose, slightly moist, brown, fine- to medium-grained			
10	B5@10'			SM		15	105.0	20.1
					Silt, stiff, slightly moist, gray, trace carbon deposits			
12								
14	B5@13'					38	115.3	16.6
16	B5@16'					36	115.4	16.1
18				ML				
20	B5@19'					30	115.9	15.0
22	B5@22'				- Sandy Silt, hard, slightly moist, brown, fine-grained, trace carbon deposits	72	130.2	2.8
24					Silty Sand, medium dense, slightly moist, light brown to brown, fine-grained			
26	B5@25'			SM		47	123.2	9.4
28								
					Sand, poorly graded, medium dense, wet, fine-grained, trace			

A9542-06-01 BORING LOGS B1 - B5.GPJ

Figure A-5,
Log of Boring 5, Page 1 of 2

SAMPLE SYMBOLS

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

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

GEOCON

PROJECT NO. A9542-06-01

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	**BORING 5** ELEV. (MSL.) -- DATE COMPLETED **May 25, 2010** EQUIPMENT **HOLLOW STEM AUGER** BY: **RG**	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
					MATERIAL DESCRIPTION			
30	B5@30'			SP	medium-grained, trace silt	44	119.1	12.6

End at 30.5 feet.
Artificial fill to 2 feet.
No groundwater encountered.
Backfilled and tamped with soil cuttings.
Capped with asphalt.

*Penetration resistance for 140 pound hammer falling 30 inches.

Figure A-5,
Log of Boring 5, Page 2 of 2

A9542-06-01 BORING LOGS B1 - B5.GPJ

SAMPLE SYMBOLS

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

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GEOCON

PROJECT NO. A9542-06-01

BORING 6

ELEV. (MSL.) - - DATE COMPLETED **12/27/16**

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					**ASPHALT: 6" BASE: 16"**			
					ARTIFICIAL FILL			
2					Silty Sand, medium dense, slightly moist, pale brown, fine-grained, some fine gravel and wood fragments.			
					OLD PARALIC DEPOSITS			
4				SM	Silty Sand, medium dense, slightly moist, brown to yellowish brown, fine-grained.			
	B5@5'				Sandy Silt, stiff, slightly moist, yellowish brown, fine-grained, trace medium-grained.	43	125.5	8.8
6								
				ML	- reddish brown			
8								
10	B5@10'					33	123.1	10.5
				SM	Silty Sand, medium dense, slightly moist, brown, fine-grained.			
12								
					Sandy Silt, hard, slightly moist, gray, fine-grained.			
14								
					- some oxidation staining, decrease in sand content			
	B5@15'					61	95.0	29.0
16				ML				
18								
					Silty Sand, very dense, slightly moist, yellowish brown, fine-grained.			
20	B5@20'			SM		90	118.8	11.8
					Sandy Silt, hard, slightly moist, yellowish brown, fine-grained.			
22	B5@22'					55	102.4	23.1
				ML				
24					- grayish brown			
	B5@25'					51	95.4	29.5
26				SP	Sand, poorly graded, dense, slightly moist, yellowish brown, oxidation staining.			
	B5@27'				Silt, hard, slightly moist, grayish brown.	45	93.8	26.8
28				ML				
				SP	Sand, poorly graded, dense, slightly moist, pale brown, fine-grained.			

A9542-06-01 BORING LOGS.GPJ

Figure A6,
Log of Boring 6, 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

PROJECT NO. A9542-06-01

BORING 6

ELEV. (MSL.) - - DATE COMPLETED **12/27/16**

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	B5@30'					71	104.4	2.7
32	B5@32'		▼		- wet	50 (5")	104.5	15.7
34								
36	B5@35'					50	85.3	17.5
38								
40	B5@40'			SP		78	100.6	22.6
42								
44								
46	B5@45'					50 (6")	96.5	22.6
48								
50	B5@50'				- no recovery	50 (6")	--	--
52					Clay, hard, moist, brown, trace fine-grained sand.			
54				CL				
56	B5@55'					50 (5")	115.0	20.2
					Sand, well-graded, very dense, wet, brown, fine- to coarse-grained.			
58				SW				

Figure A6,
Log of Boring 6, Page 2 of 3

A9542-06-01 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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PROJECT NO. A9542-06-01

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	**BORING 6** ELEV. (MSL.) - - DATE COMPLETED **12/27/16** EQUIPMENT **HOLLOW STEM AUGER** BY: **PZ**	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
					MATERIAL DESCRIPTION			
60	B5@60'			SW		52	117.3	16.2

Total depth of boring: 60.5 feet
Fill to 3 feet.
Groundwater encountered at 32 feet.
Backfilled with soil cuttings and tamped.
Asphalt patched.

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

Figure A6,
Log of Boring 6, Page 3 of 3

A9542-06-01 BORING LOGS.GPJ

SAMPLE SYMBOLS

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

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PROJECT NO. A9542-06-01

BORING 7

ELEV. (MSL.) - - DATE COMPLETED **12/27/16**

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					**ASPHALT: 2.5" BASE: NONE**			
					ARTIFICIAL FILL			
2					Clay, soft, slightly moist, olive brown.			
					OLD PARALIC DEPOSITS			
4					Sandy Silt, hard, slightly moist, reddish brown, fine-grained.			
6	B7@5'			ML		49	121.9	12.2
					Silty Sand, loose, slightly moist, reddish brown, fine-grained.			
8								
10	B7@10'					13	107.0	6.3
12				SM				
14					- yellowish brown, very fine- to fine-grained			
16	B7@15'					63	108.4	9.6
18					Sand, poorly graded, dense, slightly moist, pale brown, fine-grained.			
20	B7@20'					64		
22	B7@22'			SP	- fine- to medium-grained	55	109.5	2.1
24								
26	B7@25'					70	106.1	2.6
					Silty Sand, medium dense, slightly moist, yellowish brown, fine-grained, some shell fragments.			
28	B7@27'			SM		46	110.3	15.1
				SP	Sand, poorly graded, very dense, moist, yellowish brown, fine-grained, some			

A9542-06-01 BORING LOGS.GPJ

Figure A7,
Log of Boring 7, Page 1 of 3

SAMPLE SYMBOLS

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

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PROJECT NO. A9542-06-01

BORING 7

ELEV. (MSL.) - - DATE COMPLETED **12/27/16**

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	B7@30'				shell fragments.	50 (6")	108.8	18.3
32	B7@32'		▼		- wet, grayish brown, no shell fragments	50 (6")	100.1	23.0
34								
36	B7@35'				- no recovery	50 (5")	--	--
38								
40	B7@40'					50 (5.5")	104.2	23.8
42				SP				
44								
46	B7@45'					50 (5.5")	106.0	19.7
48								
50	B7@50'					50 (6")	103.7	18.5
52								
54					Silty Sand, medium dense, moist, brown, fine-grained, trace medium-grained.			
56	B7@55'			SM		47	124.1	12.6
58								

Figure A7,
Log of Boring 7, Page 2 of 3

A9542-06-01 BORING LOGS.GPJ

SAMPLE SYMBOLS

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

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PROJECT NO. A9542-06-01

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	**BORING 7** ELEV. (MSL.) **- -** DATE COMPLETED **12/27/16** EQUIPMENT **HOLLOW STEM AUGER** BY: **PZ**	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
					MATERIAL DESCRIPTION			
60	B7@60'			SM		52	115.1	18.4

Total depth of boring: 60.5 feet
Fill to 3 feet.
Groundwater encountered at 32 feet.
Backfilled with soil cuttings and tamped.
Asphalt patched.

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

Figure A7,
Log of Boring 7, Page 3 of 3

A9542-06-01 BORING LOGS.GPJ

SAMPLE SYMBOLS

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

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BORING 8

ELEV. (MSL.) - - DATE COMPLETED **12/27/16**

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					**ASPHALT: 2.5"**			
					ARTIFICIAL FILL			
2					Clay, soft, slightly moist, olive brown.			
					OLD PARALIC DEPOSITS			
4					Sandy Silt, firm, slightly moist, reddish brown, fine-grained.			
6				ML				
8					Silty Sand, medium dense, slightly moist, brown, fine-grained.			
10								
12				SM				
14								
16								
					Sand, poorly graded, dense, slightly moist, pale brown, fine-grained.			
18				SP				
20					Total depth of boring: 20 feet Fill to 3 feet. No groundwater encountered. Percolation testing performed. Backfilled with soil cuttings and tamped. Asphalt patched.			

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

A9542-06-01 BORING LOGS.GPJ

SAMPLE SYMBOLS

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

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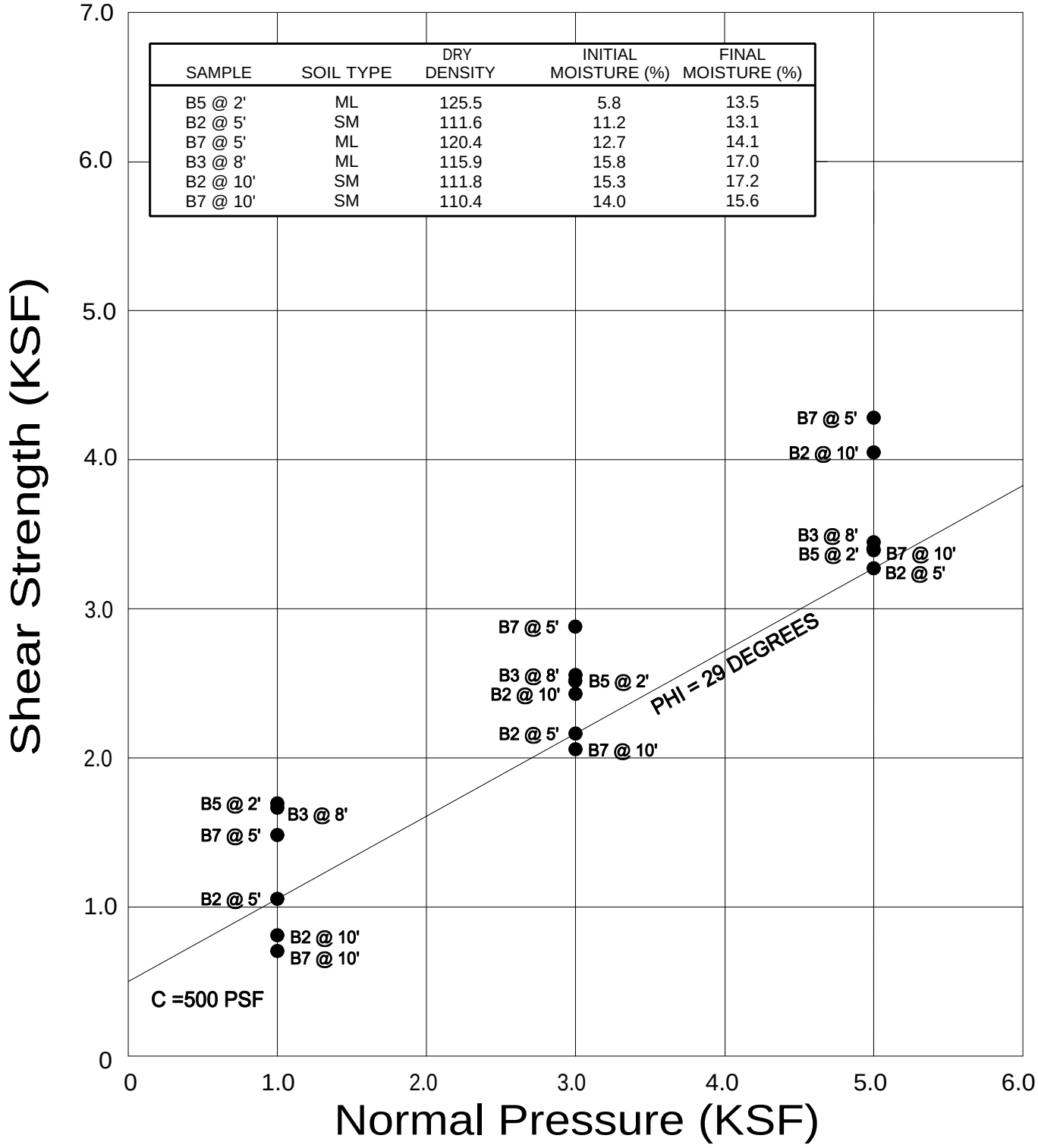
APPENDIX B

APPENDIX B

LABORATORY TESTING

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



SAMPLE	SOIL TYPE	DRY DENSITY	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B5 @ 2'	ML	125.5	5.8	13.5
B2 @ 5'	SM	111.6	11.2	13.1
B7 @ 5'	ML	120.4	12.7	14.1
B3 @ 8'	ML	115.9	15.8	17.0
B2 @ 10'	SM	111.8	15.3	17.2
B7 @ 10'	SM	110.4	14.0	15.6

● Direct Shear, Saturated

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DIRECT SHEAR TEST RESULTS

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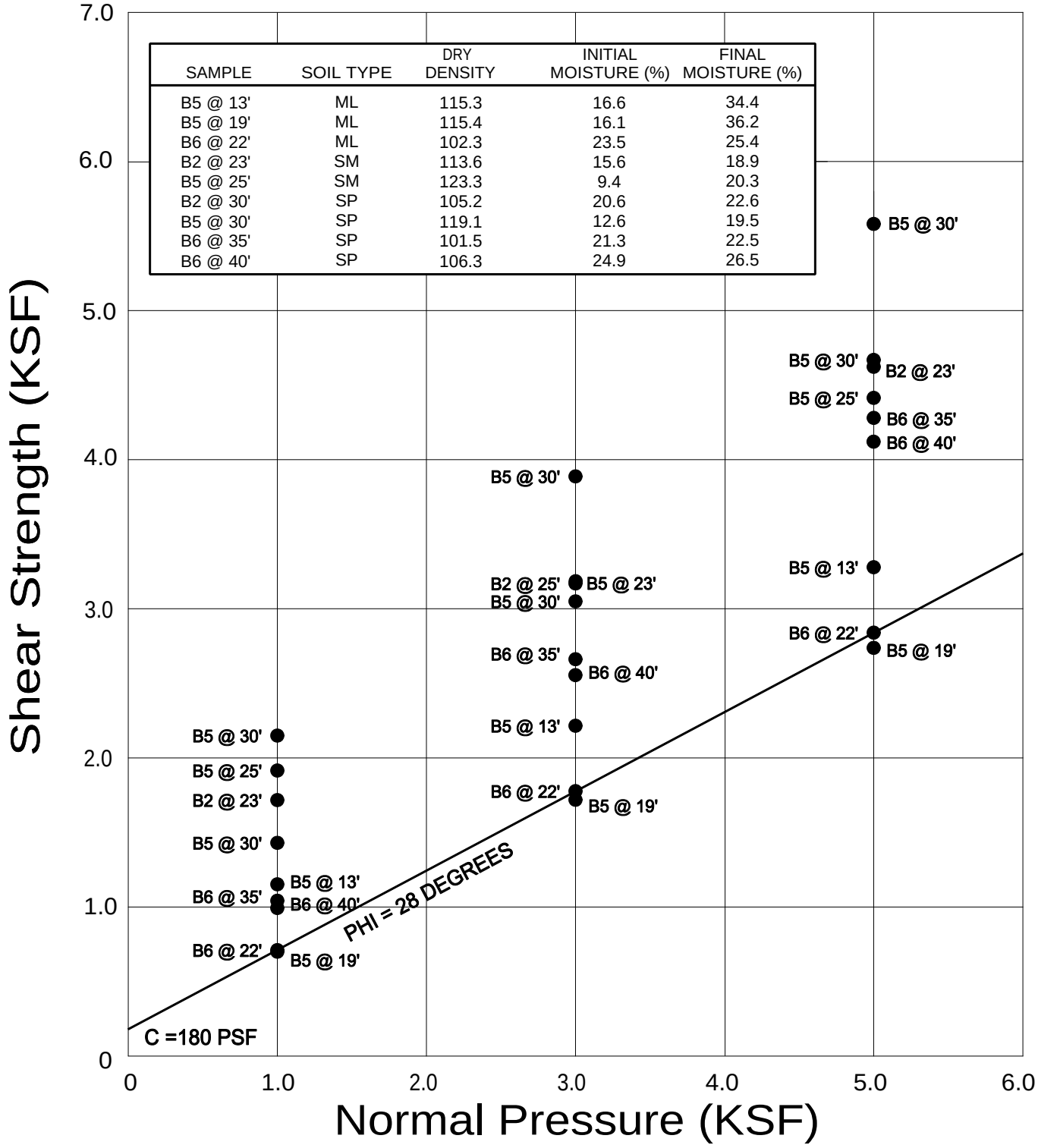
ENSEMBLE INVESTMENTS
3RD STREET & PACIFIC AVENUE
LONG BEACH, CALIFORNIA

DRAFTED BY: PZ	CHECKED BY: JTA	JAN. 2017	PROJECT NO. A9542-06-01	FIG. B1



SAMPLE	SOIL TYPE	DRY DENSITY	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B5 @ 13'	ML	115.3	16.6	34.4
B5 @ 19'	ML	115.4	16.1	36.2
B6 @ 22'	ML	102.3	23.5	25.4
B2 @ 23'	SM	113.6	15.6	18.9
B5 @ 25'	SM	123.3	9.4	20.3
B2 @ 30'	SP	105.2	20.6	22.6
B5 @ 30'	SP	119.1	12.6	19.5
B6 @ 35'	SP	101.5	21.3	22.5
B6 @ 40'	SP	106.3	24.9	26.5

● Direct Shear, Saturated

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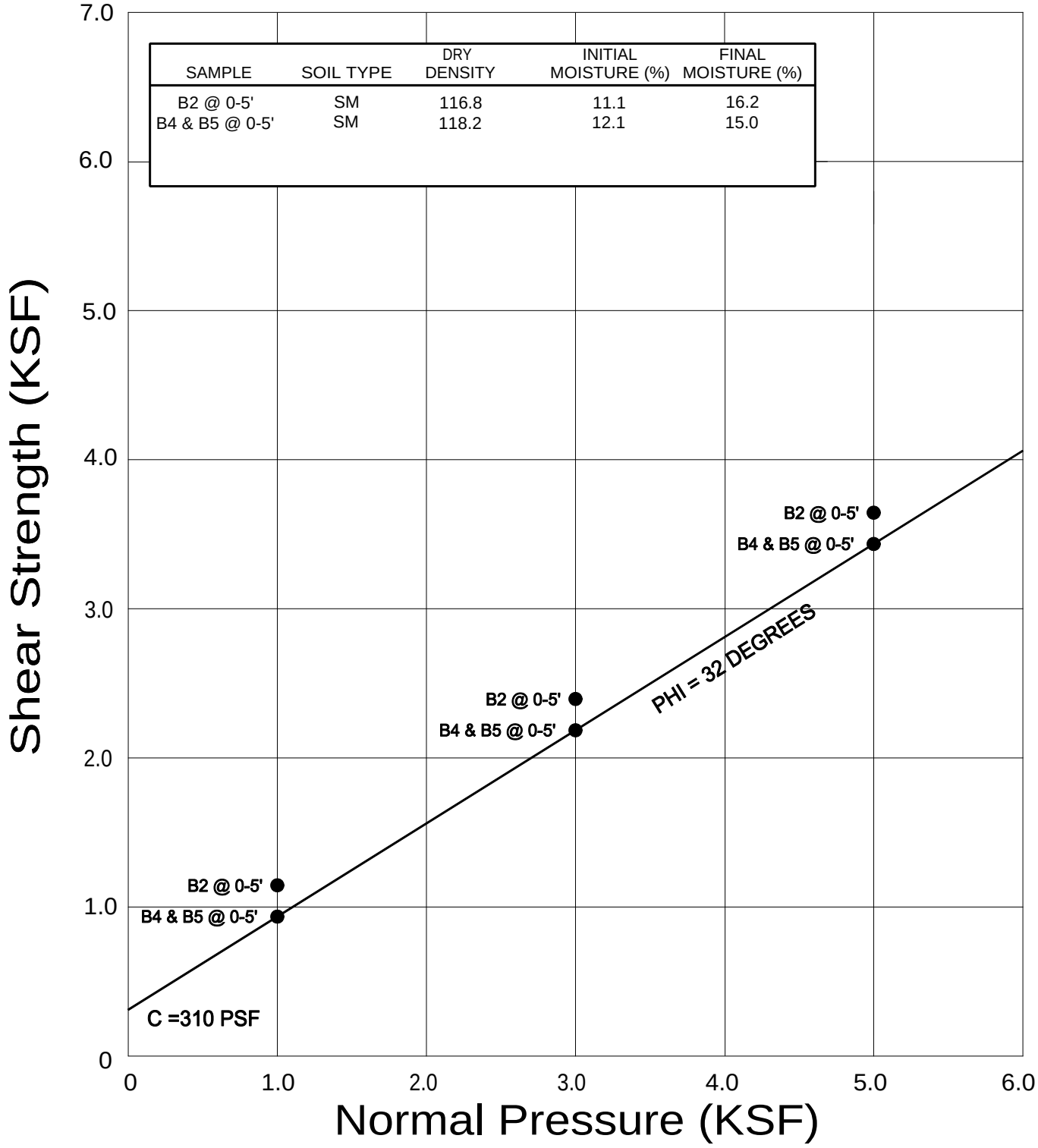
DRAFTED BY: PZ	CHECKED BY: JTA	JAN. 2017	PROJECT NO. A9542-06-01	FIG. B2



SAMPLE	SOIL TYPE	DRY DENSITY	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B2 @ 0-5'	SM	116.8	11.1	16.2
B4 & B5 @ 0-5'	SM	118.2	12.1	15.0

● Direct Shear, Saturated

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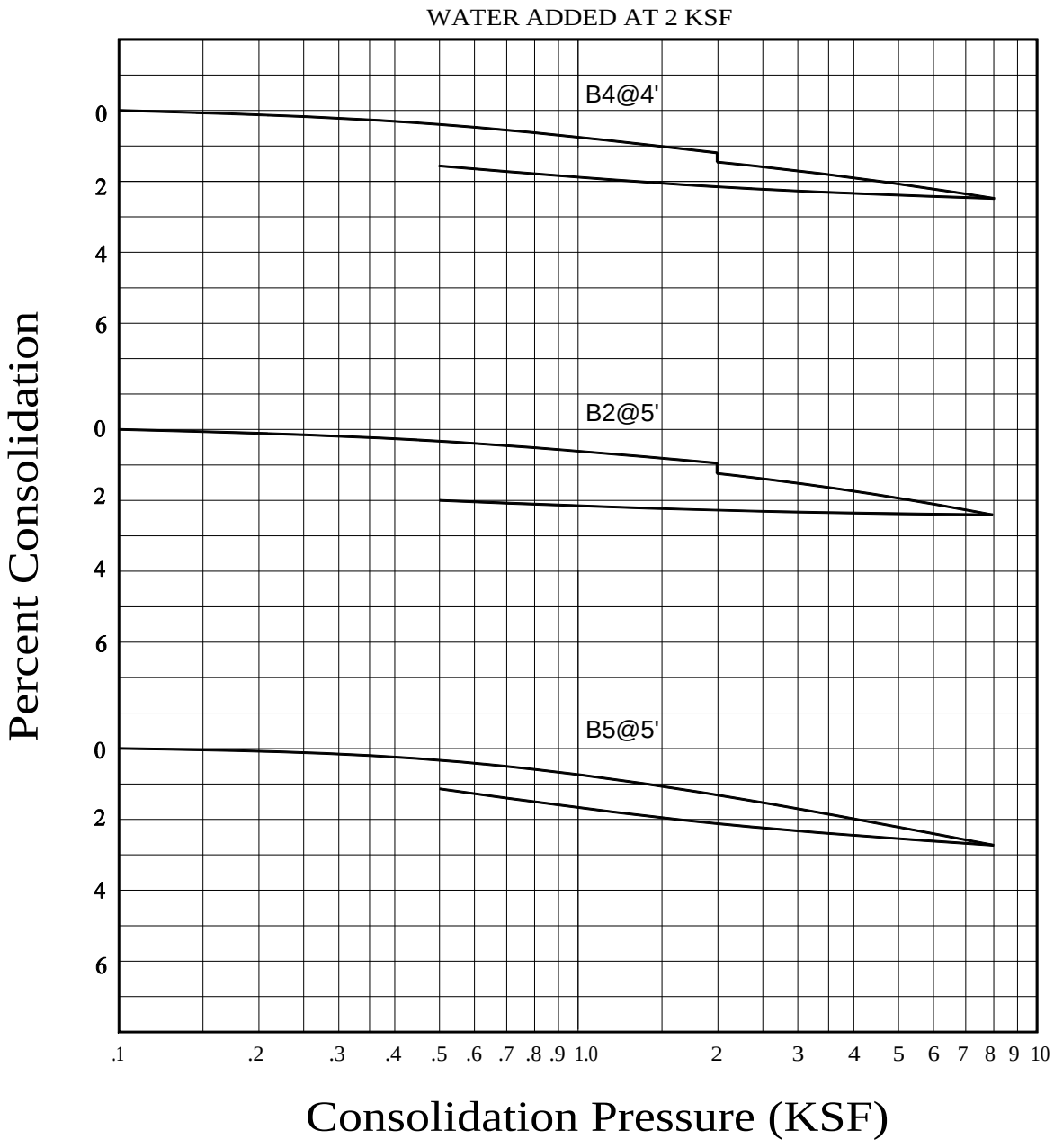
DRAFTED BY: PZ	CHECKED BY: JTA	JAN. 2017	PROJECT NO. A9542-06-01	FIG. B3



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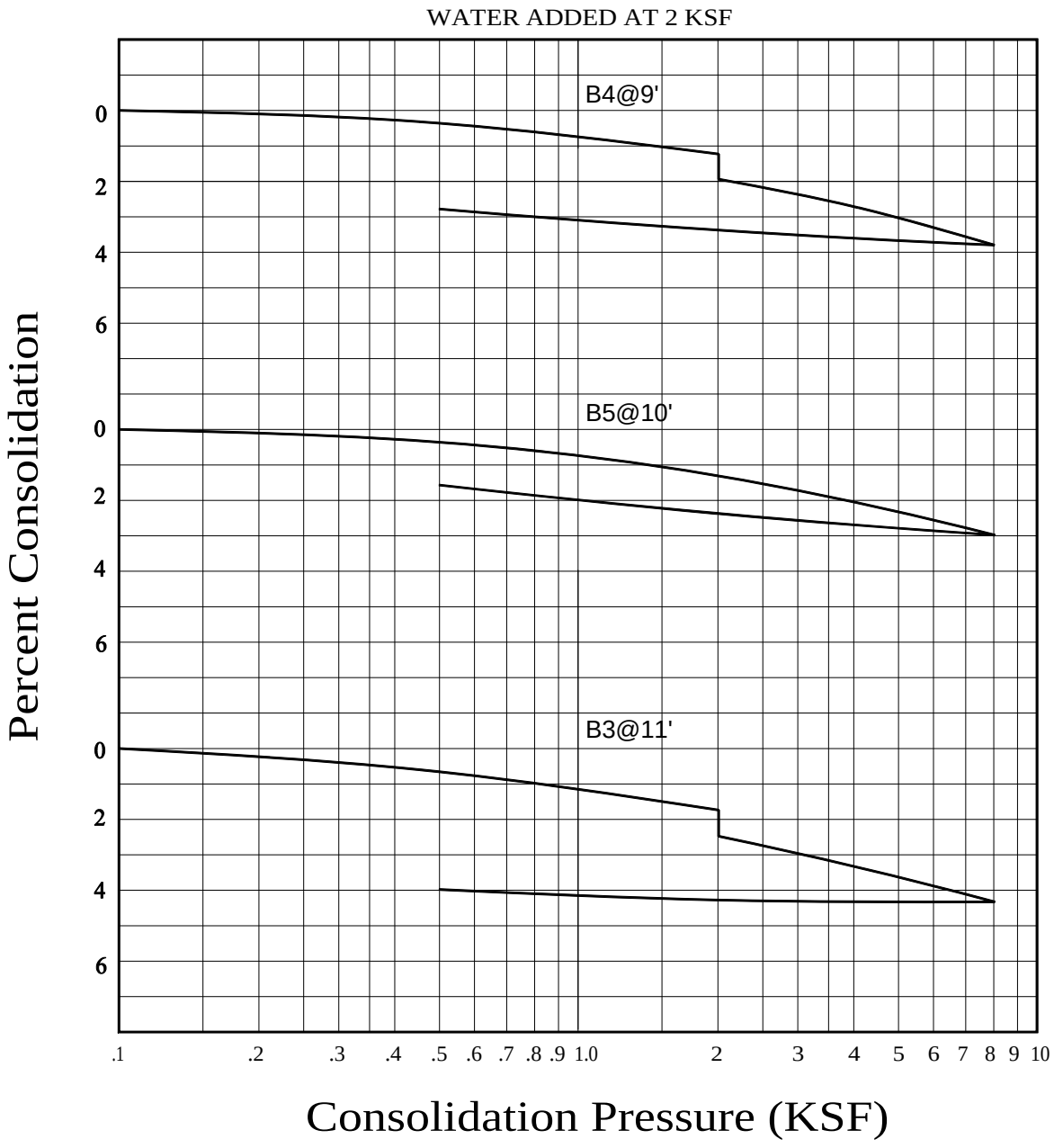
DRAFTED BY: PZ	CHECKED BY: JTA	JAN. 2017	PROJECT NO. A9542-06-01	FIG. B4



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CONSOLIDATION TEST RESULTS

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ENSEMBLE INVESTMENTS
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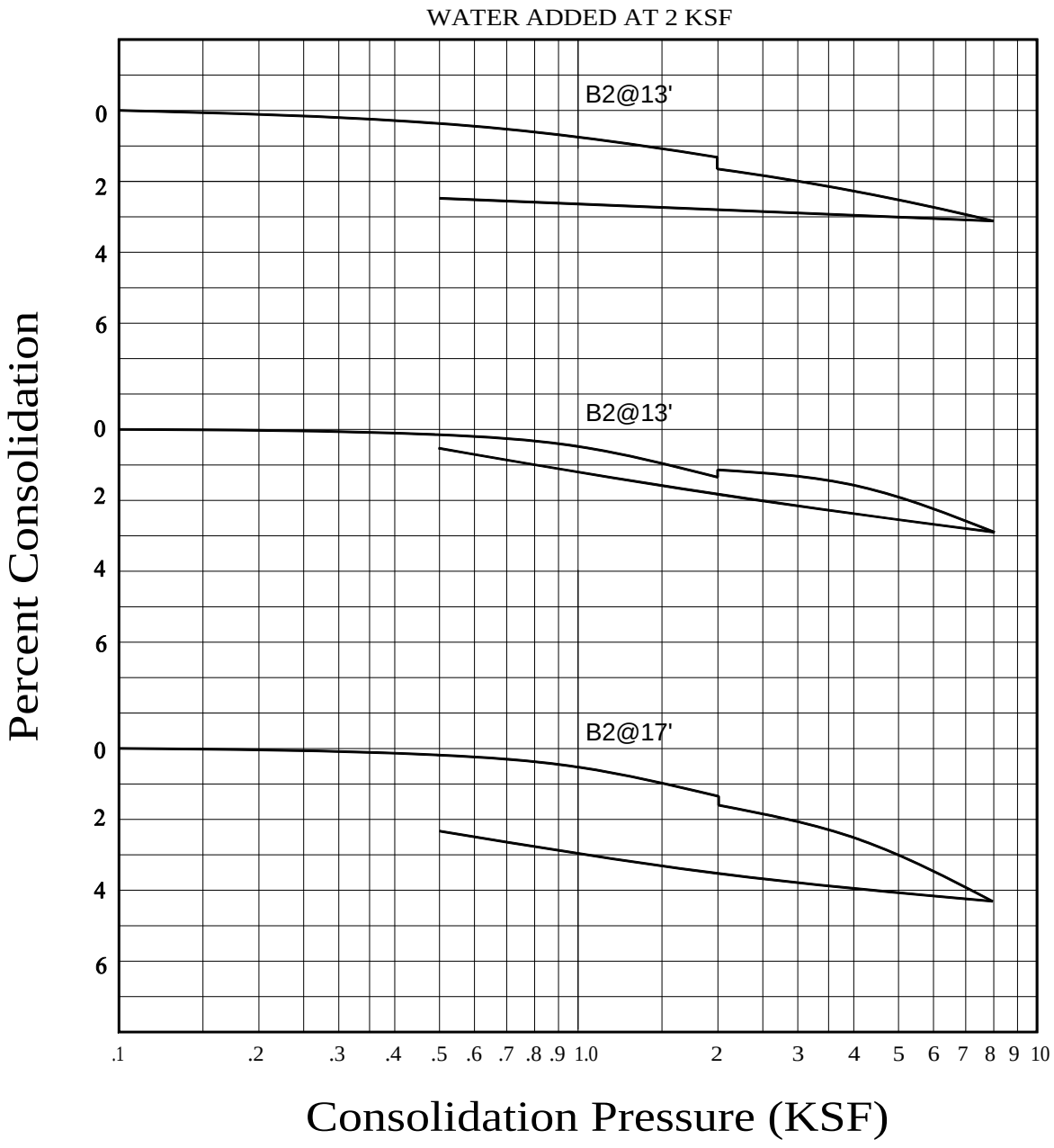
DRAFTED BY: PZ	CHECKED BY: JTA	JAN. 2017	PROJECT NO. A9542-06-01	FIG. B5



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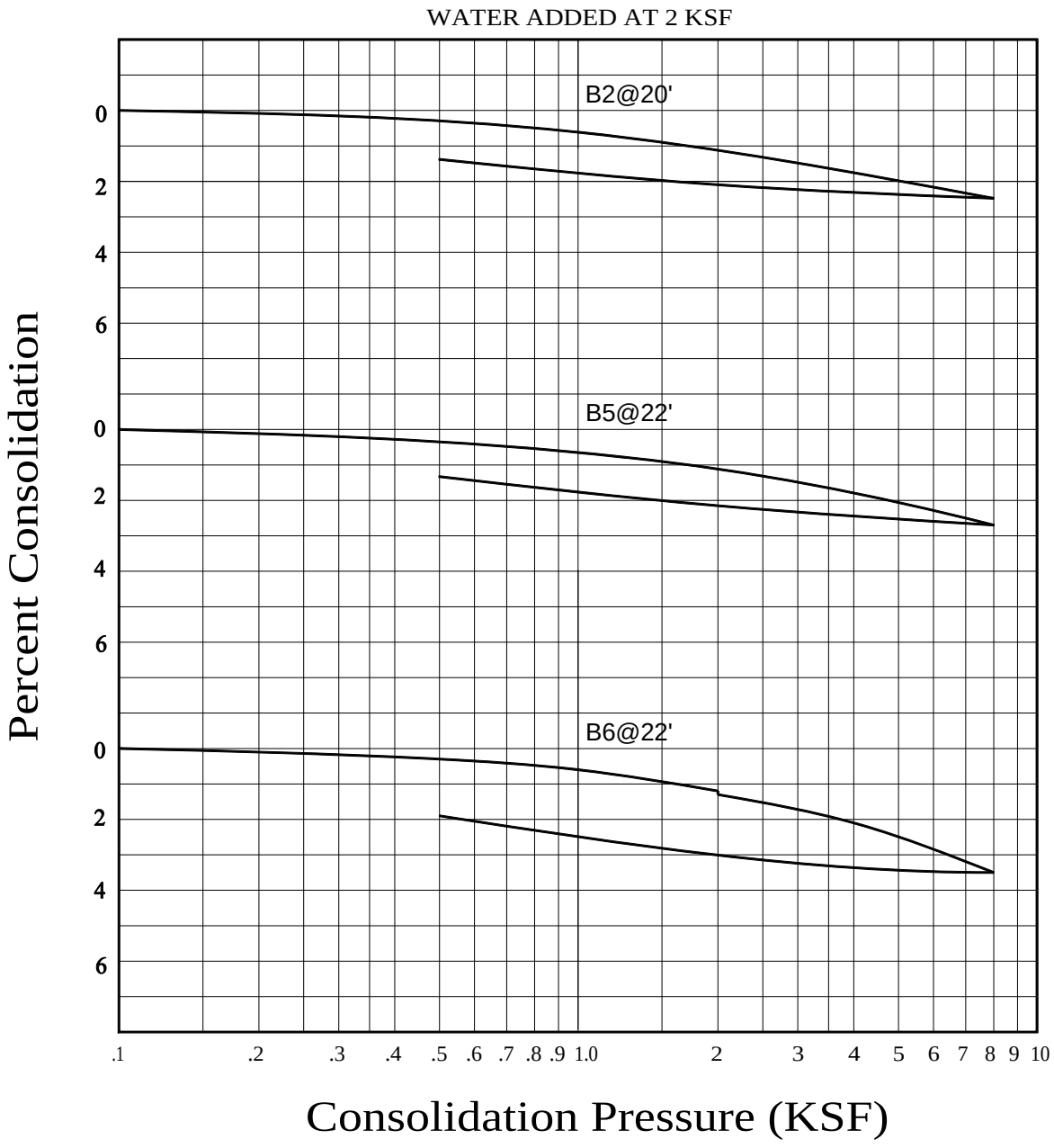
DRAFTED BY: PZ	CHECKED BY: JTA	JAN. 2017	PROJECT NO. A9542-06-01	FIG. B6



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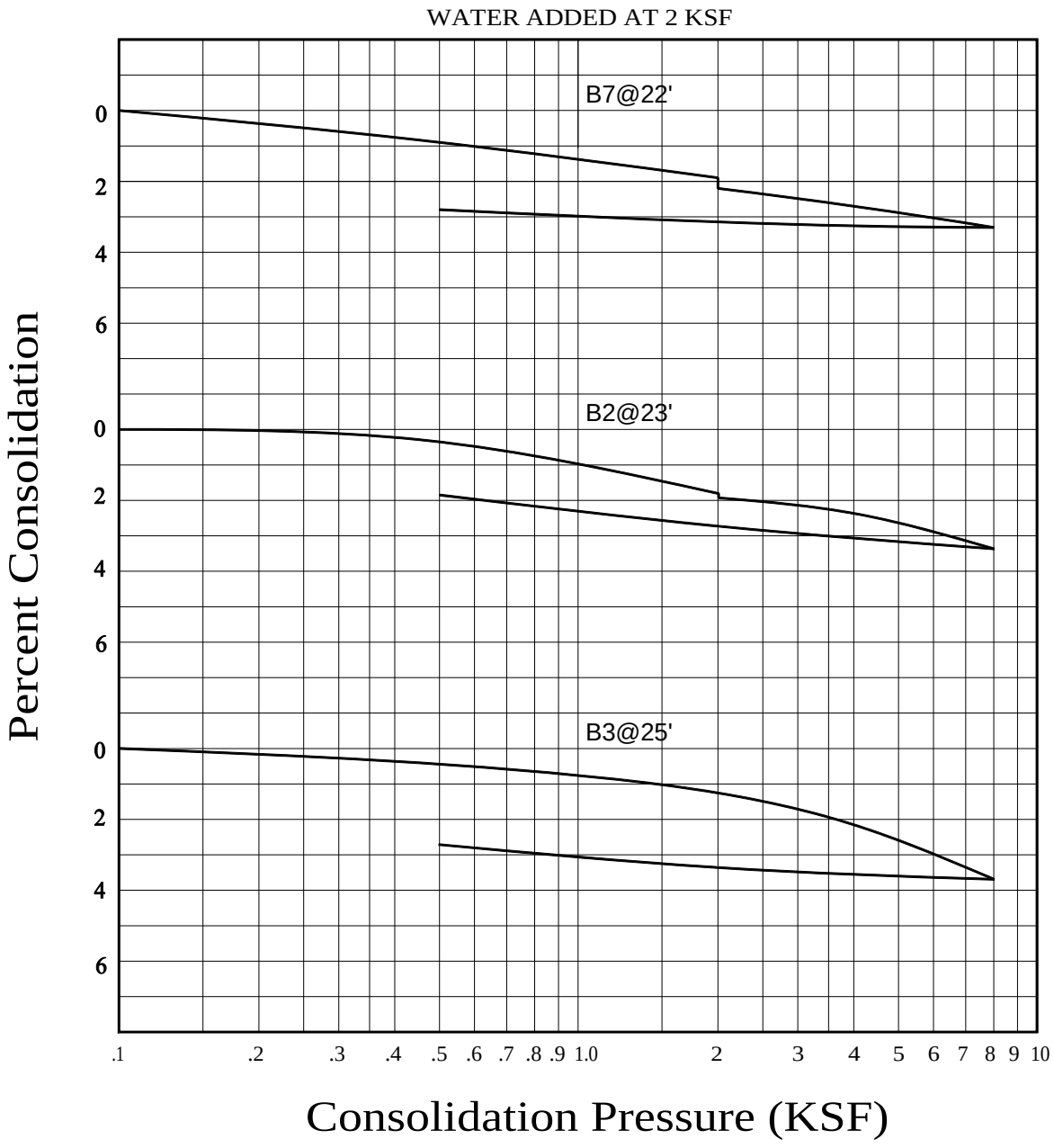
DRAFTED BY: PZ	CHECKED BY: JTA	JAN. 2017	PROJECT NO. A9542-06-01	FIG. B7



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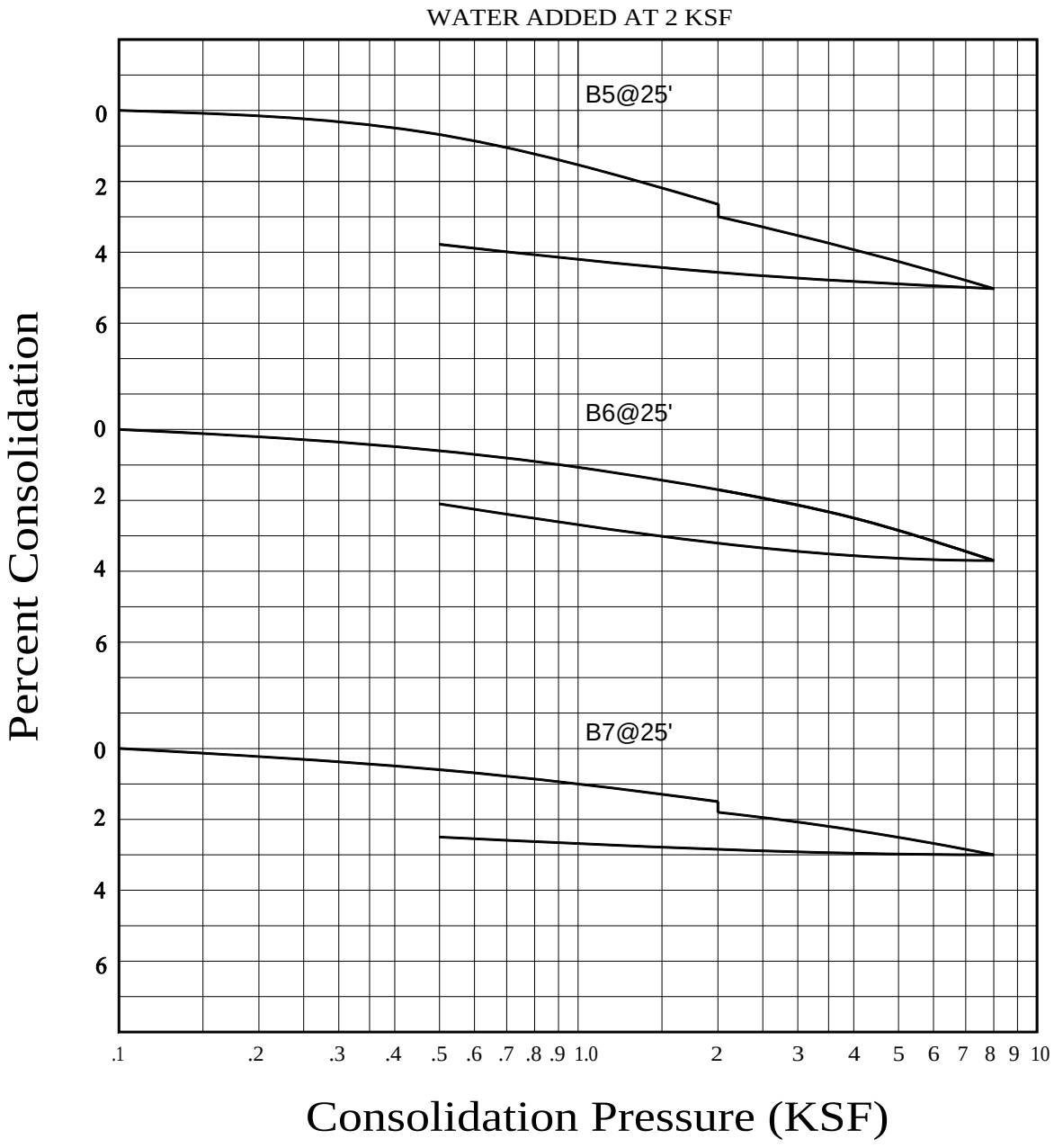
DRAFTED BY: PZ	CHECKED BY: JTA	JAN. 2017	PROJECT NO. A9542-06-01	FIG. B8



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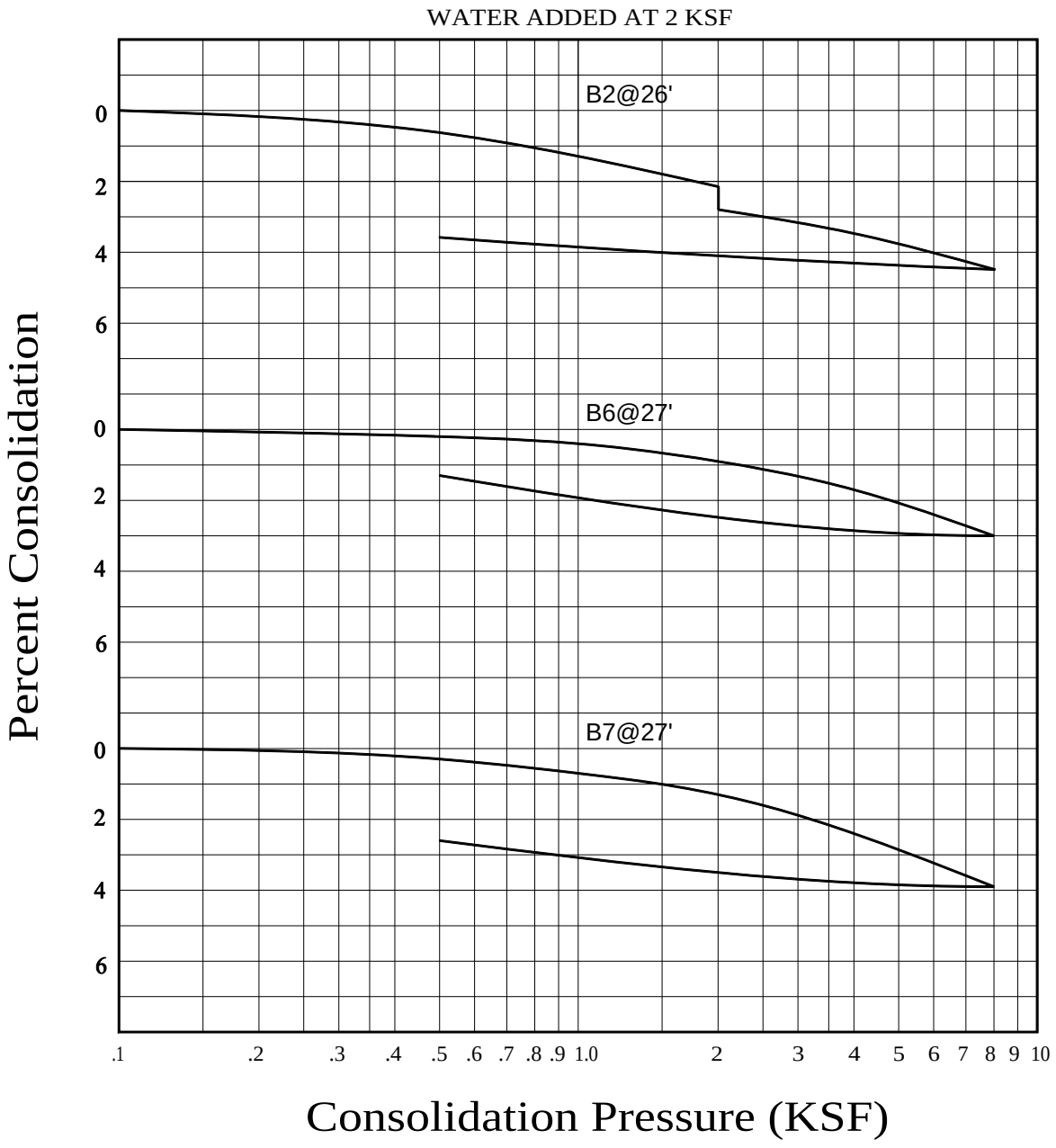
DRAFTED BY: PZ	CHECKED BY: JTA	JAN. 2017	PROJECT NO. A9542-06-01	FIG. B9



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LONG BEACH APARTMENTS
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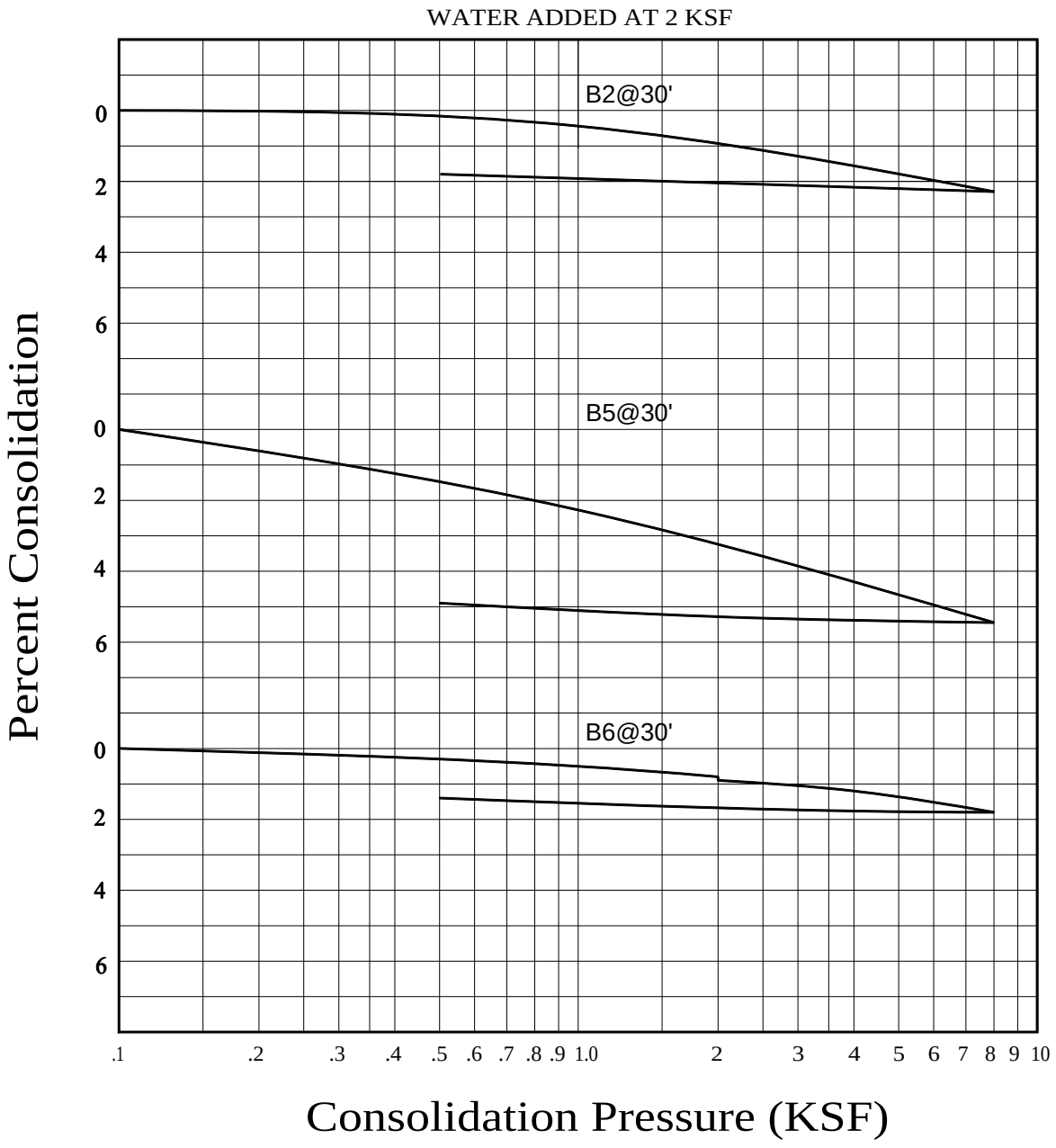
DRAFTED BY: PZ	CHECKED BY: JTA	JAN. 2017	PROJECT NO. A9542-06-01	FIG. B10



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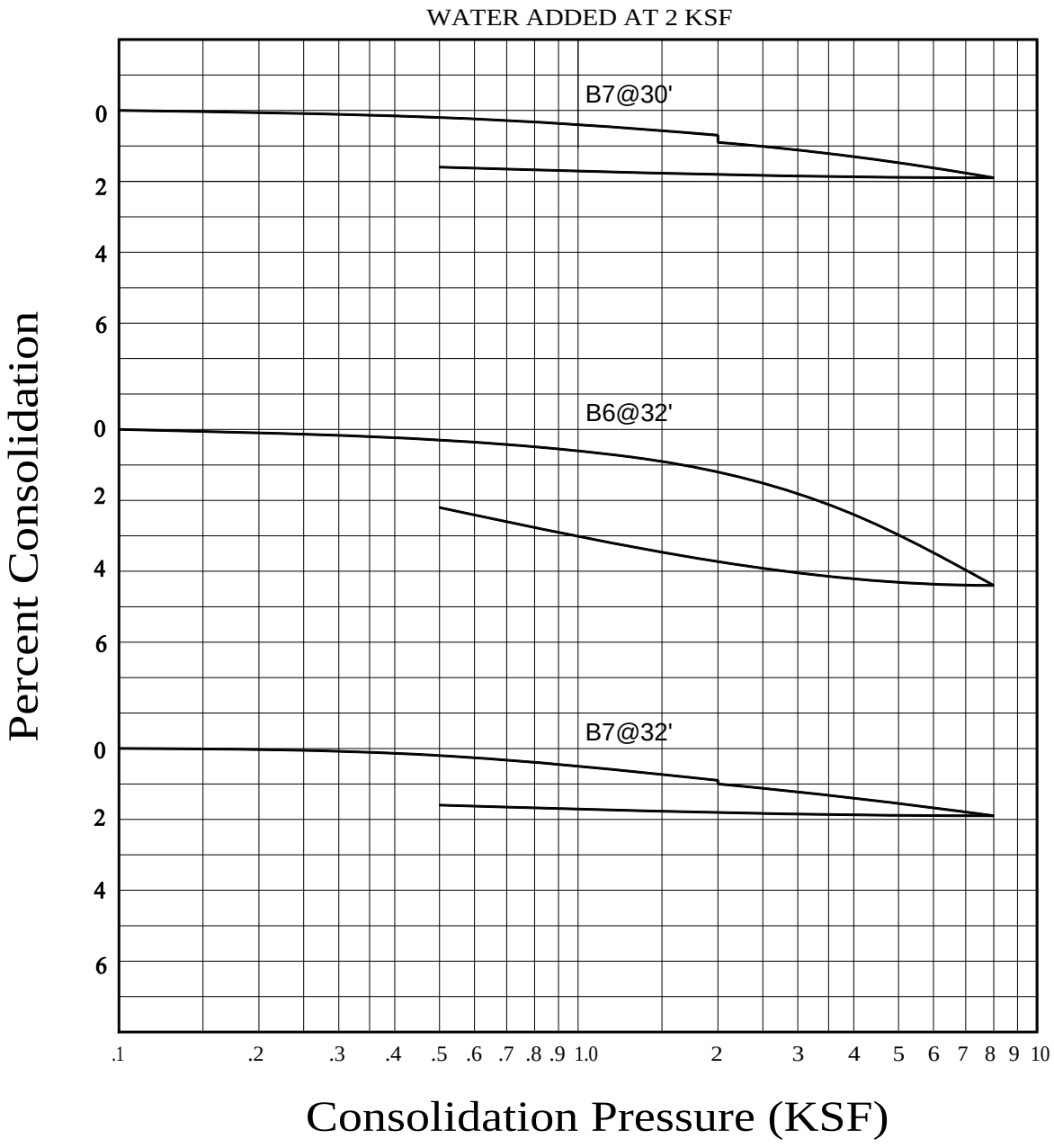
DRAFTED BY: PZ	CHECKED BY: JTA	JAN. 2017	PROJECT NO. A9542-06-01	FIG. B11



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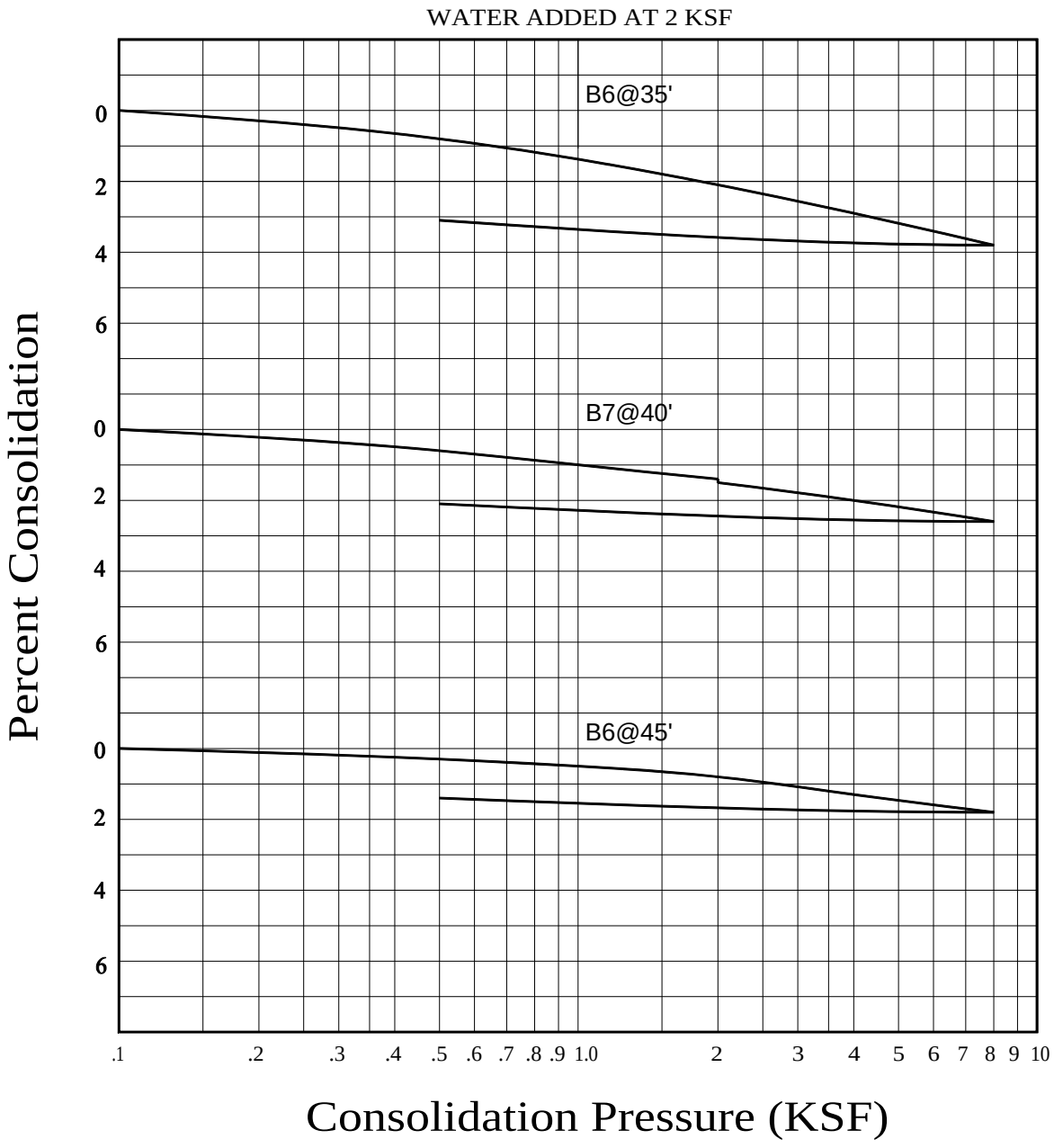
DRAFTED BY: PZ	CHECKED BY: JTA	JAN. 2017	PROJECT NO. A9542-06-01	FIG. B12



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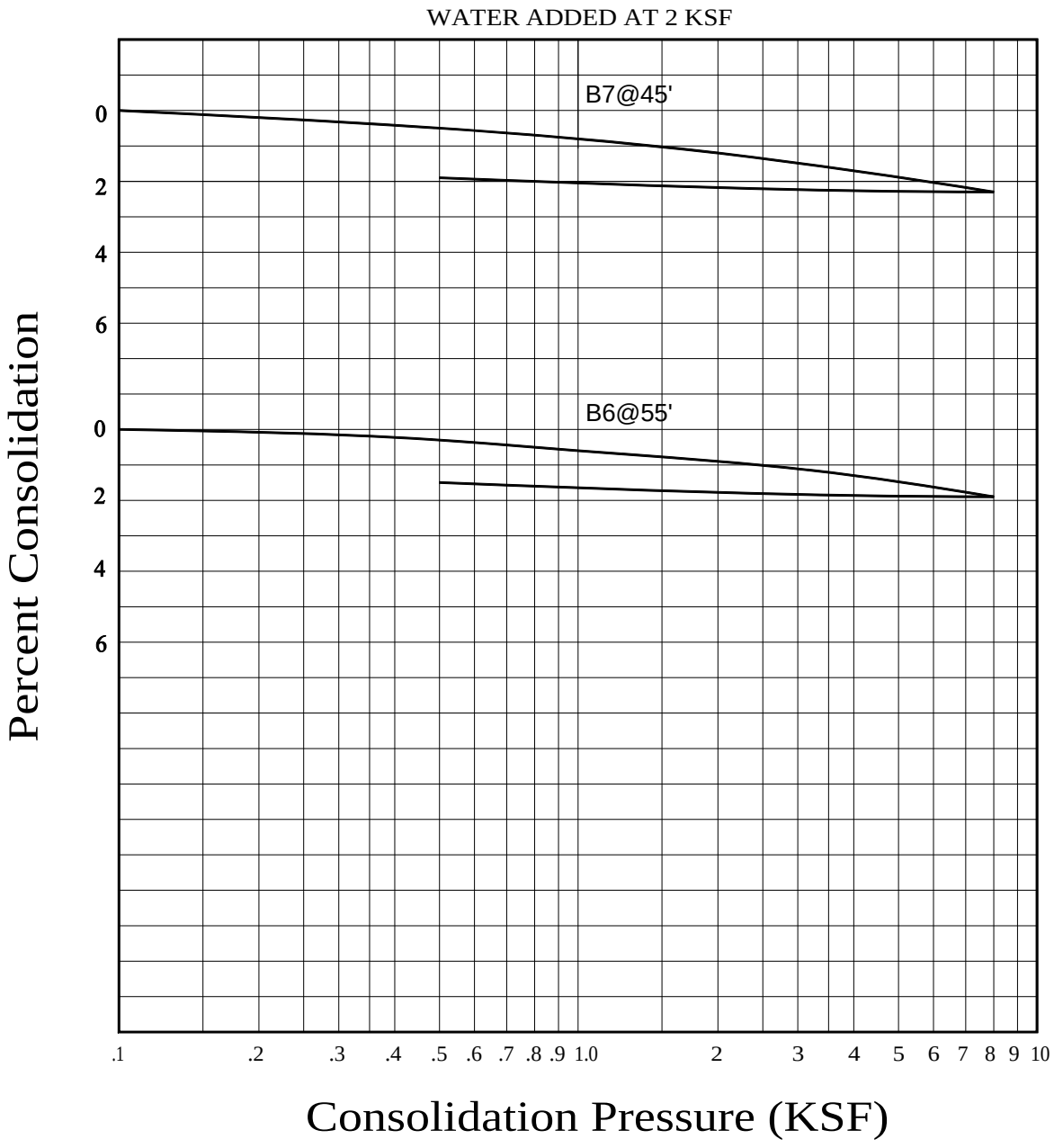
DRAFTED BY: PZ	CHECKED BY: JTA	JAN. 2017	PROJECT NO. A9542-06-01	FIG. B13



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CONSOLIDATION TEST RESULTS

LONG BEACH APARTMENTS
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DRAFTED BY: PZ	CHECKED BY: JTA	JAN. 2017	PROJECT NO. A9542-06-01	FIG. B14

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				
B2 @ 0-5'	11.0	22.3	112.6	43	Low	Expansive
B4 & B5 @ 0-5'	10.7	20.5	110.6	30	Low	Expansive
B5 @ 16'	14.7	37.8	96.0	144	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 (%)
B2 @ 0-5'	Dark Brown Silty Sand	129.0	12.0
B4 & B5 @ 0-5'	Dark Brown Silty Sand	133.0	10.0

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LABORATORY TEST RESULTS

LONG BEACH APARTMENTS
ENSEMBLE INVESTMENTS
3RD STREET & PACIFIC AVENUE
LONG BEACH, CALIFORNIA

DRAFTED BY: PZ	CHECKED BY: JTA	JAN. 2017	PROJECT NO. A9542-06-01	FIG. B15

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

Sample No.	pH	Resistivity (ohm centimeters)
B2 @ 0-5'	6.18	2000 (Highly Corrosive)
B4 & B5 @ 0-5'	6.79	1800 (Highly Corrosive)
B3 @ 20'	6.73	1300 (Highly Corrosive)
B6 @ 20-25'	8.90	3400 (Moderately Corrosive)

SUMMARY OF LABORATORY CHLORIDE CONTENT TEST RESULTS EPA NO. 325.3

Sample No.	Chloride Ion Content (%)
B2 @ 0-5'	0.012
B4 & B5 @ 0-5'	0.007
B3 @ 20'	0.012
B6 @ 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*
B2 @ 0-5'	0.019	Negligible
B4 & B5 @ 0-5'	0.020	Negligible
B3 @ 20'	0.021	Negligible
B6 @ 20-25'	0.001	Negligible

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

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

CORROSIVITY TEST RESULTS

LONG BEACH APARTMENTS
ENSEMBLE INVESTMENTS
3RD STREET & PACIFIC AVENUE
LONG BEACH, CALIFORNIA

DRAFTED BY: PZ	CHECKED BY: JTA	JAN. 2017	PROJECT NO. A9542-06-01	FIG. B16

Project No. A9542-06-01
April 10, 2019

Ensemble Investments
444 West Ocean Boulevard, Suite 1108
Long Beach, California 90802

Attn: Ms Jewelle Kennedy

Subject: RESPONSE TO SOILS REPORT REVIEW COMMENTS
PROPOSED LONG BEACH APARTMENTS
MULTI-FAMILY RESIDENTIAL DEVELOPMENT
3RD STREET AND PACIFIC AVENUE
LONG BEACH, CALIFORNIA

References: *Updated Geotechnical Investigation, Proposed Long Beach Apartments, Multi-Family Residential Development, 3rd Street and Pacific Avenue, Long Beach, California,* prepared by Geocon West, Inc., dated January 31, 2017;

Memorandum of 3rd and Pacific Project Addendum – Technical Reports Peer Review – Geotechnical Report, prepared by ESA Community Development, dated January 14, 2018.

Dear Ms. Kennedy:

This letter has been prepared in response to soils report review comments provided in the referenced Memorandum dated January 14, 2018. A copy of the memorandum letter is appended herein.

Comment: *Section 7.5.1: Is an elevator shaft proposed at or below this depth? Can it be designed for hydrostatic pressure?*

Response: Section 7.5.1 indicates that the historic high groundwater level is at a depth of 25 feet below the ground surface. Furthermore, we have confirmed with the project architect that the elevator pits will extend to depths of 26½ and 34 feet below the ground surface for the north and south towers, respectively. Therefore, the elevator shafts are anticipated to extend below the historic high groundwater level. We have also confirmed with the project design team that the proposed structure, including the elevator shafts, will be designed for hydrostatic pressures where extending below the historic high groundwater level of 25 feet.

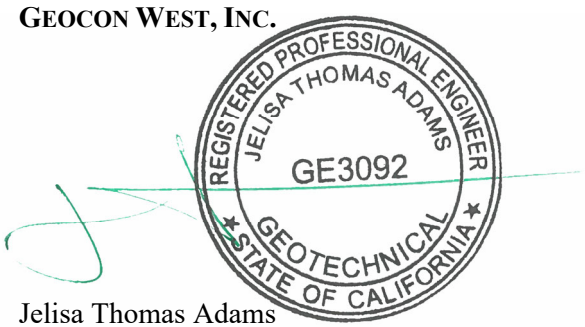
Comment: *Section 7.16: No discussion of dewatering here but there is mention of how to mitigate buoyant forces. Provide clarification on the need for dewatering.*

Response: Section 7.16 of the referenced Geotechnical Investigation report provides recommendations for the design and construction of hydraulic elevator piston. We understand that this type of elevator will not be used on the project, and that a mechanical elevator will be used. As indicated in the response above, where the structure extends below the historic high water table, a hydrostatic design will be used. Permanent dewatering is not proposed for this project.

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

Very truly yours,

GEOCON WEST, INC.



Jelisa Thomas Adams
GE 3092

Enclosure: Memorandum of 3rd and Pacific Project Addendum – Technical Reports Peer Review – Geotechnical Report

(Email) Addressee

ESA Community Development

626 Wilshire Boulevard
Suite 1100
Los Angeles, CA 90017
213.599.4300 phone
213.599.4301 fax

www.esassoc.com

memorandum

date January 14, 2018

to Alexis Oropeza and Amy Harbin, Long Beach Development Services Planning Bureau

from Kimberly Comacho, ESA

subject 3rd and Pacific Project Addendum - Technical Reports Peer Review –Geotechnical Report, Geocon West, Inc. January 31, 2017.

Study: **Geotechnical Report**

Section	Page	Paragraph	Line/ Sentence	Comment or Issue	Proposed Resolution
7.5.1	14	1st	1st	Is an elevator shaft proposed at or below this depth? Can it be designed for hydrostatic pressure?	Could be more of a question for the applicant but will need to know if a permanent dewatering system is being proposed for project.
7.16	25			No discussion of dewatering here but there is mention of how to mitigate buoyant forces.	Provide clarification on the need for dewatering.