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GEOCON WEST, INC.

GEOTECHNICAL
ENVIRONMENTAL
MATERIALS

GEOTECHNICAL INVESTIGATION

PROPOSED MIXED-USE MULTI-FAMILY RESIDENTIAL DEVELOPMENT 5.6 ACRE AREA BOUNDED BY DOVE STREET, SCOTT DRIVE, CORINTHIAN WAY, AND MARTINGALE WAY NEWPORT BEACH, CALIFORNIA

PREPARED FOR

MACARTHUR SQUARE
A CALIFORNIA GENERAL PARTNERSHIP
IRVINE, CALIFORNIA

PROJECT NO. A9138-06-01

JUNE 12, 2014

GEOCON
WEST, INC.
GEOTECHNICAL ■ ENVIRONMENTAL ■ MATERIALS

Project No. A9138-06-01
June 12, 2014

MacArthur Square, a California General Partnership
17631 Fitch
Irvine, CA 92614

Attention: Mr. Lester C. Smull

Subject: GEOTECHNICAL INVESTIGATION
PROPOSED MIXED-USE MULTI-FAMILY RESIDENTIAL DEVELOPMENT
5.6 ACRE AREA BOUNDED BY DOVE STREET, SCOTT DRIVE,
CORINTHIAN WAY AND MARTINGALE WAY
NEWPORT BEACH, CALIFORNIA

Dear Mr. Smull:

In accordance with your authorization of our proposal dated February 24, 2014, we have prepared this geotechnical investigation report for the proposed mixed-use, multi-family residential development to be located at the 5.6 acre area bounded by Dove Street, Scott Drive, Corinthian Way, and Martingale Way in Newport 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 in this report are followed and implemented during design and construction. If you have any questions regarding this report, or if we may be of further service, please contact the undersigned.

Very truly yours,

GEOCON WEST, INC.

Petrina Zen	Susan Kirkgard	Jelisa M. Thomas
Staff Engineer	CEG 1754	PE 74946

(4+EMAIL) Addressee

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GEOTECHNICAL INVESTIGATION

1. PURPOSE AND SCOPE

This report presents the results of a geotechnical investigation for the proposed mixed-use, multi-family residential development located at the 5.6 acre area bounded by Dove Street, Scott Drive, Corinthian Way, and Martingale Way in Newport Beach, California (Vicinity Map, Figure 1). The purpose of this investigation was to evaluate the 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 explored on May 15, 2014 by drilling four 8-inch diameter borings utilizing a truck-mounted hollow-stem auger drilling machine and advancing two cone penetrometer tests (CPTs). The borings were drilled to depths between 10½ and 30½ feet below the existing ground surface. The CPTs were advanced to depths of 50½ feet below existing ground surface. The approximate locations of the exploratory borings and CPTs are depicted on the Site Plan (Figure 2). A detailed discussion of the field investigation, including boring logs and CPT 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 property is an approximately 5.6 acre parcel located in Newport Beach, California (see Vicinity Map, Figure 1). The property is bounded by Corinthian Way to the north, by Martingale Way to the east, by Scott Drive and Dove Street to the west, and by a multi-story commercial structure and paved parking to the south. The property is currently occupied by on-grade single-story commercial structures and paved parking. The site slopes gently to the west with approximately 3 feet of vertical relief and no pronounced highs or lows. Surface water drainage at the site appears to be by sheet flow along the ground surface to area drains and the city streets. Vegetation on site consists of bushes, plants, grass, and trees located in isolated planter areas.

Information concerning the proposed project was furnished by the client. It is our understanding that the proposed development will consist of four three- to four-story mixed-use, multi-family residential structures underlain by one- to two-levels of parking. The parking levels will extend horizontally throughout the entire footprint of these proposed structures. The finished floor elevation of the subterranean parking is anticipated to be approximately 5 feet the existing ground surface at the west side of the site, near the intersection of Dove Street and Scott Road; and is anticipated to be approximately 8½ feet below the existing ground surface at the east side of the site, along Martingale Way. In addition, a two-story on-grade structure is proposed at the west corner of the site. The proposed site development is depicted on the Site Plan (see Figure 2).

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

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

4. SOIL AND GEOLOGIC CONDITIONS

Based on our field investigation and published geologic maps, the site is underlain by artificial fill over Pleistocene age marine terrace deposits approximately 100 feet thick (Sprotte et al., 1980). These marine terrace deposits are composed mainly of silt, with some sand and clay (Sprotte et al., 1980; California Division of Mines and Geology, 1981). Detailed stratigraphic profiles are provided on the boring logs in Appendix A.

4.1 Artificial Fill

Artificial fill were encountered to a maximum depth of 4 feet in boring B1. The artificial fill generally consists of olive brown to dark reddish brown clayey sand, silty sand, and sandy silt. The artificial fill is characterized as slightly moist and medium dense or firm, with varying amounts of trace fine gravel. Deeper fill may occur between borings and on other parts of the site that were not directly explored.

4.2 Marine Terrace Deposits

The artificial fill is underlain by Pleistocene age marine terrace deposits which generally consist of yellowish brown to olive brown silty sand to sandy silt, silt and clay. The terrace deposits are predominantly slightly moist to moist and firm to stiff or loose to medium dense.

5. GROUNDWATER

The historically highest groundwater level in the area is reported to be at a depth of approximately 10 feet beneath the existing ground surface (California Division of Mines and Geology, 2001). Groundwater information presented in this document is generated from data collected in the early 1900's to the late 1990s.

Groundwater was encountered in boring B1 at a depth of 30 feet below the existing ground surface. Based on the depth of groundwater encountered in our borings, groundwater is not expected to be encountered during construction.

Based on current groundwater basin management practices, it is unlikely that groundwater levels will ever exceed the historic high levels. However, it is not uncommon for groundwater levels to vary seasonally or for groundwater seepage conditions to develop where none previously existed, especially in impermeable fine-grained soils which are heavily irrigated or after seasonal rainfall. In addition, recent requirements for stormwater infiltration could result in shallower seepage conditions in the immediate site vicinity. Proper surface drainage of irrigation and precipitation will be critical for future performance of the project. Recommendations for drainage are provided in the S*urface Drainage* section of this report (see Section 7.24).

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 (formerly known as California Division of Mines and Geology [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 currently established Alquist-Priolo Earthquake Fault Zone for surface fault rupture hazards. No active or potentially active faults with the potential for surface fault rupture are known to pass directly beneath the site. Therefore, the potential for surface rupture due to faulting occurring beneath the site during the design life of the proposed development is considered low. However, the site is located in the seismically active Southern California region, and could be subjected to moderate to strong ground shaking in the event of an earthquake on one of the many active Southern California faults. The faults in the vicinity of the site are shown in Figure 3 (Regional Fault Map).

The closest surface trace of an active fault to the site is the Newport-Inglewood Fault Zone located approximately 6.5 miles to the south-southwest (Ziony and Jones, 1989). Other nearby active faults are the Palos Verdes Fault Zone (offshore segment), the Whittier Fault, and the Elsinore Fault located approximately 16 miles southwest, 16½ miles north-northeast, and 17 miles northeast of the site, respectively (Ziony and Jones, 1989). The active San Andreas Fault Zone is located approximately 46 miles northeast of the site (Ziony and Jones, 1989).

The closest potentially active fault to the site is the Pelican Hill Fault located approximately 2.3 miles to the south-southwest (Ziony and Jones, 1989). Other nearby potentially active faults are the El Modeno Fault, Peralta Hills Fault, and the Los Alamitos Fault located approximately 11 miles north, 11 miles north-northeast, and 14½ miles northwest, respectively (Ziony and Jones, 1989).

The site is located within the vertical projection of the San Joaquin Hills Blind Thrust Fault. The San Joaquin Hills Blind Thrust Fault is a deep thrust fault underlying the San Joaquin Hills at the southern portion of the Orange County coastal plain. The San Joaquin Hills Blind Thrust Fault extends to within 2 km of the surface east of the San Joaquin Hills, dips between 20° and 30° to the west underneath the San Joaquin Hills, and extends to the base of the seismogenic crust at 17 km along the coast (Grant, et. al., 1999). Deformation related to an earthquake event originating along this blind thrust fault is limited to compressional folding at depth and do not present a potential surface fault rupture hazard. However, these active features are capable of generating future earthquakes.

6.2 Seismicity

As with all of Southern California, the site has experienced historic earthquakes from various regional faults. The seismicity of the region surrounding the site was formulated based on research of an electronic database of earthquake data. The epicenters of recorded earthquakes with magnitudes equal to or greater than 4.0 within a radius of 60 miles of the site are depicted on Figure 4 (Regional Seismicity Map). A partial list of earthquakes of moderate to major magnitude 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	50	E
Near Redlands	July 23, 1923	6.3	42	ENE
Long Beach	March 10, 1933	6.4	7	SSW
Tehachapi	July 21, 1952	7.5	113	NW
San Fernando	February 9, 1971	6.6	60	NW
Whittier Narrows	October 1, 1987	5.9	30	NNW
Sierra Madre	June 28, 1991	5.8	42	N
Landers	June 28, 1992	7.3	90	NE
Big Bear	June 28, 1992	6.4	70	NE
Northridge	January 17, 1994	6.7	54	NW

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

6.3 Estimation of Peak Ground Accelerations

The seismic exposure of the site may be investigated in two ways. The deterministic approach recognizes the Maximum Earthquake, which is the theoretical maximum event that could occur along a fault. The deterministic method assigns a maximum earthquake to a fault derived from formulas that correlate the length and other characteristics of the fault trace to the theoretical maximum magnitude earthquake. The probabilistic method considers the probability of exceedance of various levels of ground motion and is calculated by consideration of risk contributions from regional faults.

6.3.1 Deterministic Analysis

Table 1 provides a list of known faults within a 60 mile radius of the site. The maximum earthquake magnitude is indicated for each fault. In order to measure the distance of known faults to the site, the computer program *EQFAULT*, (Blake, 2000), was utilized.

Principal references used within *EQFAULT* in selecting faults to be included are Jennings (1994), Anderson (1984) and Wesnousky (1986). For this investigation, the ground motion generated by maximum earthquakes on each of the faults is assumed to attenuate to the site per the attenuation relation by Sadigh et al. (1997) modeling the soil underlying the site as Site Class “D”. The Site Class determination is based on the discussion in Section 1613.3.2 of the 2013 CBC and Table 20.3-1 of ASCE 7-10. The resulting calculated peak horizontal accelerations at the site are indicated on Table 1. These values are one standard deviation above the mean.

Using this methodology, the maximum earthquake resulting in the highest peak horizontal accelerations at the site would be a magnitude 6.6 event on the San Joaquin Hills Blind Thrust. Such an event would be expected to generate peak horizontal accelerations at the site of 0.934g.

While listing of peak accelerations is useful for comparison of potential effects of fault activity in a region, other considerations are important in seismic design, including the frequency and duration of motion and the soil conditions underlying the site.

The site could be subjected to moderate to severe ground shaking in the event of a major earthquake on any of the faults referenced above or other faults in Southern California. With respect to seismic shaking, the site is considered comparable to the surrounding developed area.

6.3.2 Probabilistic Analysis

The computer program *FRISKSP* (Blake, 2000) was used to perform a site-specific probabilistic seismic hazard analysis. The program is a modified version of FRISK (McGuire, 1978) that models faults as lines to evaluate site-specific probabilities of exceedance for given horizontal accelerations for each line source. Geologic parameters not included in the deterministic analysis are included in this analysis. The program operates under the assumption that the occurrence rate of earthquakes on each mapped Quaternary fault is proportional to the faults' slip rate. The program accounts for fault rupture length as a function of earthquake magnitude, and site acceleration estimates are made using the earthquake magnitude and closest distance from the site to the rupture zone.

Uncertainty in each of following are accounted for: (1) earthquake magnitude, (2) rupture length for a given magnitude, (3) location of the rupture zone, (4) maximum magnitude of a given earthquake, and (5) acceleration at the site from a given earthquake along each fault. After calculating the expected accelerations from all earthquake sources, the program then calculates the total average annual expected number of occurrences of the site acceleration greater than a specified value. Attenuation relationships suggested by Sadigh et al. (1997) were utilized in the analysis.

The Maximum Considered Earthquake (MCE) Ground Motion is the level of ground motion that has a 2 percent chance of exceedance in 50 years, with a statistical return period of 2,500 years. According to the 2013 California Building Code and ASCE 7-10, the MCE is to be utilized for the design of critical structures such as schools and hospitals. The Design Earthquake (DE) Ground Motion 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. The DE is typically used for the design of non-critical structures.

Based on the computer program *FRISKSP* (Blake, 2000), the MCE and DE is expected to generate ground motions at the site of approximately 0.65g and 0.34g, respectively. Graphical representation of the analysis is presented on Figure 5 (Probability of Exceedance).

6.4 Seismic Design Criteria

The following table summarizes summarizes site-specific design criteria obtained from the 2013 California Building Code (CBC; Based on the 2012 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. The values presented below are for the risk-targeted maximum considered earthquake (MCE_R).

2013 CBC SEISMIC DESIGN PARAMETERS

Parameter	Value	2013 CBC Reference
Site Class	D	Table 1613.3.2
MCE_R Ground Motion Spectral Response Acceleration – Class B (short), S_S	1.582g	Figure 1613.3.1(1)
MCE_R Ground Motion Spectral Response Acceleration – Class B (1 sec), S_1	0.580g	Figure 1613.3.1(2)
Site Coefficient, F_A	1.0	Table 1613.3.3(1)
Site Coefficient, F_V	1.5	Table 1613.3.3(2)
Site Class Modified MCE_R Spectral Response Acceleration (short), S_{MS}	1.582g	Section 1613.3.3 (Eqn 16-37)
Site Class Modified MCE_R Spectral Response Acceleration – (1 sec), S_{M1}	0.870g	Section 1613.3.3 (Eqn 16-38)
5% Damped Design Spectral Response Acceleration (short), S_{DS}	1.055g	Section 1613.3.4 (Eqn 16-39)
5% Damped Design Spectral Response Acceleration (1 sec), S_{D1}	0.580g	Section 1613.3.4 (Eqn 16-40)

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

ASCE 7-10 PEAK GROUND ACCELERATION

Parameter	Value	ASCE 7-10 Reference
Mapped MCE_G Peak Ground Acceleration, PGA	0.617g	Figure 22-7
Site Coefficient, F_{PGA}	1.0	Table 11.8-1
Site Class Modified MCE_G Peak Ground Acceleration, PGA_M	0.617g	Section 11.8.3 (Eqn 11.8-1)

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.

6.5 Liquefaction Potential

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

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

The State of California Seismic Hazard Zone Map for the Tustin Quadrangle (CDMG, 2001) indicates that the site is not located in an area designated as "liquefiable". The Orange County General Plan (2004) and the Newport Beach General Plan (2006) also indicate that site is not located within an area identified as having a potential for liquefaction. As stated previously, the soils encountered during exploration are generally composed of well consolidated Pleistocene age fine-grained soils. Based on these considerations it is our opinion that the site is not susceptible to liquefaction.

7.6 Slope Stability

The topography at the site is relatively level and the site is not located within an area identified as having a potential for slope instability (CDMG, 2001; City of Newport Beach, 2006). There are no known landslides near the site, nor is the site in the path of any known or potential landslides. Therefore, the potential for slope stability hazards to adversely affect the proposed development is considered low.

7.7 Earthquake-Induced Flooding

Earthquake-induced flooding is inundation caused by failure of dams or other water-retaining structures due to earthquakes. Review of the Orange County Safety Element (2004) indicates that the site is located within the inundation boundary of the Prado Dam. However, this dam, as well as others in California, are continually monitored by various governmental agencies (such as the State of California Division of Safety of Dams and the U.S. Army Corps of Engineers) to guard against the threat of dam failure. Current design, construction practices, and ongoing programs of review, modification, or total reconstruction of existing dams are intended to ensure that all dams are capable of withstanding the maximum considered earthquake (MCE) for the site. Therefore, the potential for inundation at the site as a result of an earthquake-induced dam failure is considered low.

7.8 Tsunamis, Seiches, and Flooding

The site is located approximately 5 miles from the Pacific Ocean. According to the City of Newport Beach General Plan (2006), the site is not within a tsunami inundation hazard zone. Therefore, tsunamis are not anticipated to adversely impact 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 within an area of minimal flooding (Zone X) as defined by the Federal Emergency Management Agency (FEMA, 2009, City of Newport Beach, 2006).

7.9 Oil Fields & Methane Potential

Based on a review of the California Division of Oil, Gas and Geothermal Resources (DOGGR) Oil and Gas Well Location Map W1-6, the site is not located within the limits of an oilfield. No oil wells are located within 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. Undocumented wells could be encountered during construction. Any wells encountered will need to be properly abandoned in accordance with the current requirements of the DOGGR.

As previously indicated, the site is not located within an oilfield. Therefore, the potential for methane at the site is considered very low. 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.

7.10 Subsidence

Subsidence occurs when a large portion of land is displaced vertically, usually due to the withdrawal of groundwater, oil, or natural gas. Soils that are particularly subject to subsidence include those with high silt or clay content. The site is not located within an area of known ground subsidence (Orange County, 2004). No large-scale extraction of groundwater, gas, oil, or geothermal energy is occurring or planned at the site or in the general site vicinity. There appears to be little or no potential for ground subsidence due to withdrawal of fluids or gases at the site.

7. CONCLUSIONS AND RECOMMENDATIONS

7.1 General

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

7.1.2 Up to 4 feet of existing artificial fill was encountered during site exploration. The existing fill encountered is believed to be the result of past grading and construction activities at the site. Deeper fill may exist between borings and in other areas of the site that were not directly explored. Future demolition of the existing structures and improvements which occupy the site will likely disturb the upper few feet of existing site soils. It is our opinion that the existing fill, in its present condition, is not suitable for direct support of proposed foundations or slabs. The existing fill and site soils are suitable for re-use as engineered fill provided the recommendations in the *Grading* section of this report are followed (see Section 7.4).

7.1.3 Excavations for subterranean level are anticipated to penetrate through the existing artificial fill and expose competent alluvial soils throughout the excavation bottom.

7.1.4 Based on these considerations, the proposed structures may be supported on conventional foundation systems. At the subterranean levels, the conventional foundation system may derive support in the undisturbed alluvial soils at or below a depth of 5 feet. For the on-grade portion of the development, the conventional foundation system may derive support in newly placed engineered fill. All foundation excavations must be observed and approved in writing by the Geotechnical Engineer prior to placement of steel or concrete. Recommendations for the design of a conventional foundation system are provided in Section 7.6.

7.1.5 As a minimum, the upper 4 feet of existing site soils within the proposed on-grade footprint areas should be excavated and properly compacted for foundation and slab support. Excavation should be conducted as necessary to completely remove all artificial fill and any soft, unsuitable alluvium at the direction of the Geotechnical Engineer (a representative of Geocon). The excavation should extend laterally a minimum distance of three feet beyond the building footprint area or a distance equal to the depth of fill below the foundation, whichever is greater. Recommendations for earthwork are provided in the *Grading* section of this report (see Section 7.4).

7.1.6 The concrete slabs-on-grade and ramps for subterranean levels may derive support directly on the undisturbed alluvial soils at or below a depth of 5 feet. Any soils that are disturbed should be properly compacted for slab and ramp support. Where necessary, the existing artificial fill and alluvial soils are suitable for re-use as an engineered fill provided the procedures outlined in the Grading section of this report are followed (see Section 7.4).

7.1.7 Excavations of up to 12 feet in vertical height are anticipated for construction of subterranean level. Due to the depth of the excavation and the proximity to the property lines, city streets and adjacent offsite structures, excavation of the proposed subterranean levels will require sloping and/or shoring measures in order to provide a stable excavation. Where shoring is required it is recommended that a soldier pile shoring system be utilized. 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.

7.1.8 Due to the nature of the proposed design for subterranean levels, 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.

7.1.9 Foundations for small outlying structures, such as block walls up to 6 feet high, planter walls or trash enclosures, which will not be tied to the proposed 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, such as adjacent to property lines, foundations may bear in the undisturbed alluvial soils found at or below a depth of 2 feet. 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 Where new paving is to be placed, it is recommended that all existing fill and soft alluvial soils be excavated and properly recompacted for paving support. The client should be aware that removal and recompaction of all existing fill and soft alluvial soils in the area of new paving is not required, however, paving constructed over existing uncertified fill or unsuitable soils may experience increased settlement and/or cracking, and may therefore have a shorter design life and increased maintenance costs. As a minimum, the upper twelve inches of soil should be scarified and properly compacted for paving support. Paving recommendations are provided in the *Preliminary Pavement Recommendations* section of this report (see Section 7.11).

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

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

7.2 Soil and Excavation Characteristics

7.2.1 The in-situ soils can be excavated with moderate effort using conventional excavation equipment. Minor caving should be anticipated in vertical 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 sloped, 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 few feet of soils encountered during the investigation are considered to have a "moderate" (EI=51) expansive potential and are classified as "expansive" based on the 2013 California Building Code (CBC) Section 1803.5.3. Recommendations presented herein assume that the building foundations and slabs will derive support in these materials.

7.3 Minimum Resistivity, pH and Water-Soluble Sulfate

7.3.1 Potential of Hydrogen (pH) and resistivity testing, as well as chloride content testing, were performed on representative samples of on-site soil to generally evaluate the corrosion potential to surface utilities. The tests were performed in accordance with California Test Method Nos. 643 and 422 and indicate that the soils are considered "corrosive" with respect to corrosion of buried ferrous metals on site. The results are presented in Appendix B (Figure B10) and should be considered for design of underground structures.

7.3.2 Laboratory tests were performed on representative samples of the on-site soil to measure the percentage of water-soluble sulfate content. Results from the laboratory water-soluble sulfate tests are presented in Appendix B (Figure B10) and indicate that the on-site soil possess a "negligible" sulfate exposure to concrete structures as defined by 2013 CBC Section 1904 and ACI 318-11 Section 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 Grading

7.4.1 Grading is anticipated to include excavation and compaction of the upper site soils, excavation of site soils for the proposed subterranean level, foundations, and utility trenches, as well as placement of backfill for walls, ramps, and trenches.

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

7.4.3 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.

7.4.4 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 in writing by the Geotechnical Engineer. All existing underground improvements planned for removal should be completely excavated and the resulting depressions properly backfilled in accordance with the procedures described herein. Once a clean excavation bottom has been established it must be observed and approved in writing by the Geotechnical Engineer (a representative of Geocon West, Inc.).

7.4.5 As a minimum, it is recommended that the upper four feet of existing site soils within the proposed on-grade footprint areas be excavated and properly compacted for foundation and slab support. Deeper excavation should be conducted as necessary to completely remove all existing artificial fill or soft soil at the direction of the Geotechnical Engineer (a representative of Geocon). The limits of existing fill and/or soft soil removal will be verified by the Geocon representative during site grading activities. The engineered fill blanket should extend at least three feet beyond the edge of foundations or for a distance equal to the depth of fill below the foundations, whichever is greater.

7.4.6 All excavations must be observed and approved in writing by the Geotechnical Engineer (a representative of Geocon), prior to placing fill. If determined to be excessively soft, additional removals or stabilization of the excavation bottom may be required in order to provide a firm working surface upon which engineered fill can be placed and heavy equipment can operate. If required, recommendations for stabilization measures can be provided under separate cover.

7.4.7 The concrete slab-on-grade at the subterranean level may derive support directly on the undisturbed alluvial soils found at the excavation bottom. Any disturbed soils should be properly compacted for slab support. The concrete slab-on-grade for the ramp should derive support on a minimum of 12 inches of properly compacted fill.

7.4.8 All fill and backfill soils should be placed in horizontal loose layers approximately 6 to 8 inches thick, moisture conditioned to 2 percent above optimum moisture content, and properly compacted to a minimum 90 percent of the maximum dry density in accordance with ASTM D 1557 (latest edition).

7.4.9. Where new paving is to be placed, it is recommended that all existing fill and soft alluvium be excavated and properly compacted for paving support. As a minimum, the upper twelve inches of soil should be scarified, moisture conditioned to 2 percent above optimum moisture content, and compacted to at least 92 percent relative compaction, as determined by ASTM Test Method D 1557 (latest edition). Paving recommendations are provided in *Preliminary Pavement Recommendations* section of this report (see Section 7.11).

7.4.10 Foundations for small outlying structures, such as block walls up to 6 feet high, planter walls or trash enclosures, which will not be tied to the proposed structure, may be supported on conventional foundations bearing on a minimum of 12 inches of newly placed engineered fill which extends laterally at least 12 inches beyond the foundation area. Where excavation and proper compaction cannot be performed or is undesirable, foundations may derive support directly in the undisturbed alluvial soils found at or below a depth of 2 feet below the existing ground surface, and should be deepened as necessary to maintain a minimum 12 inch embedment into undisturbed alluvium. If the alluvial soils exposed in the excavation bottom are loose or disturbed, compaction of the soils will be required prior to placing steel or concrete. Compaction of the foundation excavation bottom is typically accomplished with a compaction wheel or mechanical whacker and must be observed and approved by a Geocon representative.

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

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

7.5 Shrinkage

7.5.1 Shrinkage results when a volume of material removed at one density is compacted to a higher density. A shrinkage factor of between 5 and 10 percent should be anticipated when excavating and compacting the existing fill and alluvium on site to an average relative compaction of 92 percent.

7.6 Conventional Foundation Design

7.6.1 It is recommended that a conventional foundation system be utilized for support of the proposed structures. At the subterranean levels, the conventional foundation system may derive support in the undisturbed alluvial soils found at or below a depth of 5 feet. Foundations for the proposed on-grade structure may derive support in newly placed engineered fill. Any exposed soft soils should be compacted to a dense state or penetrated by proposed foundations at the direction of the Geotechnical Engineer (a representative of Geocon).

7.6.2 Continuous footings deriving support in the recommended bearing materials may be designed for an allowable bearing capacity of 2,500 pounds per square foot, 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 materials.

7.6.3 Isolated spread foundations deriving support in the recommended bearing materials may be designed for an allowable bearing capacity of 2,700 pounds per square foot, and should be a minimum of 24 inches in width, 24 inches in depth below the lowest adjacent grade, and 12 inches into the recommended bearing materials.

7.6.4 The soil bearing pressure above may be increased by 100 psf and 400 psf for each additional foot of foundation width and depth, respectively, up to a maximum allowable soil bearing value of 3,500 pounds per square foot.

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

7.6.6 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. Reinforcement for spread footings should be designed by the project structural engineer.

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

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

7.6.9 Due to the expansion potential of the site soils, the moisture content in the slab and foundation subgrade should be maintained subsequent to grading and as necessary until concrete placement.

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

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

7.7 Miscellaneous Foundations

7.7.1 Foundations for small outlying structures, such as block walls, planter walls or trash enclosures, which will not be tied to the proposed structures, may be supported on conventional foundations bearing on a minimum of 12 inches of newly placed engineered fill which extends laterally at least 12 inches beyond the foundation area. Where excavation and compaction cannot be performed, such as adjacent to property lines, foundations may derive support in the undisturbed alluvium found at or below a depth of 24 inches, and should be deepened as necessary to maintain a 12 inch embedment in to the recommended bearing materials.

7.7.2 If the soils exposed in the excavation bottom are soft, compaction of the soft soils will be required prior to placing steel or concrete. Compaction of the foundation excavation bottom is typically accomplished with a compaction wheel or mechanical whacker and must be observed and approved in writing by a Geocon representative. Miscellaneous foundations may be designed for a bearing value of 1,500 pounds per square foot, and should be a minimum of 12 inches in width, 24 inches in depth below the lowest adjacent grade and 12 inches into the recommended bearing material. The allowable bearing pressure may be increased by up to one-third for transient loads due to wind or seismic forces.

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

7.8 Foundation Settlement

7.8.1 The maximum expected total settlement for a structure supported on a conventional foundation system designed with the maximum allowable bearing value of 3,500 psf and deriving support in the recommended bearing materials is estimated to be approximately 1½ inch and occur below the heaviest loaded structural element. A majority of the settlement of the foundation system is expected to occur on initial application of loading; however, minor additional settlements are expected within the first 12 months. Differential settlement is expected to be less than ¾ inch over a distance of twenty feet.

7.8.2 If structural connections are planned between the on-grade structure and the main structure, differential settlements across the connections will need to be carefully designed for. As the project progresses, Geocon should work closely with the project structural engineer to confirm the foundation loading configuration and anticipated total and differential settlement.

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

7.9 Lateral Design

7.9.1 Resistance to lateral loading may be provided by friction acting at the base of foundations and by passive earth pressure. An allowable coefficient of friction of 0.30 may be used with the dead load forces in the newly placed engineered fill and competent, undisturbed alluvium.

7.9.2 Passive earth pressure for the sides of foundations and slabs poured against the newly placed engineered fill and competent undisturbed alluvium may be computed as an equivalent fluid having a density of 200 pcf with a maximum earth pressure of 2,000 pcf. When combining passive and friction for lateral resistance, the passive component should be reduced by one-third.

7.10 Concrete Slabs-on-Grade

7.10.1 Exterior concrete slabs-on-grade at the ground surface subject to vehicle loading should be designed in accordance with the recommendations in the *Preliminary Pavement Recommendations* section of this report (Section 7.11).

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

7.10.3 Unless specifically designed and evaluated by the project structural engineer, the slab-on-grade at the subterranean level subject to vehicle loading should be a minimum of 5 inches thick and reinforced with No. 4 steel reinforcing bars placed 16 inches on center in both horizontal directions. The recommended slab thickness and reinforcement is in conformance with the *WRI/CRSI Design of Slab-on-Ground Foundations,* based on an effective plasticity index of 26 and assuming either Grade 40 or 60 steel. The concrete slab-on-grade may bear directly on the undisturbed alluvial soils found at the excavation bottom. Any disturbed soils should be properly compacted for slab support. The ramp may derive support in the undisturbed alluvial soils and/or engineered fill and the upper 12 inches of subgrade soils directly beneath the ramp should be moisture conditioned to two percent above optimum moisture and properly compacted to at least 92 percent relative compaction, as determined by ASTM Test Method D 1557 (latest edition) for ramp support.

7.10.4 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 1643-11 and the manufacturer's recommendations. A minimum thickness of 15 mils and a permeance of less than 0.01 perms is recommended. The vapor retarder should be installed in direct contact with the concrete slab with proper perimeter seal. If the California Green Building Code requirements apply to this project, the vapor retarder should be underlain by 4 inches of clean aggregate. It is important that the vapor retarder be puncture resistant since it will be in direct contact with angular gravel. As an alternative to the clean aggregate suggested in the California Green Building Code, it is our opinion that the concrete slab-on-grade may be underlain by a vapor retarder over 4-inches of clean sand (sand equivalent greater than 30), since the sand will serve a capillary break and will minimize the potential for punctures and damage to the vapor barrier.

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

7.10.6 For seismic design purposes, a coefficient of friction of 0.30 may be utilized between concrete slabs and subgrade soils without a moisture barrier, and 0.15 for slabs underlain by a moisture barrier.

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

7.10.8 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. Construction joints should be designed by the project structural engineer.

7.10.9 Due to the expansive potential of the anticipated subgrade soils, the moisture content of the slab subgrade should be maintained and sprinkled as necessary to maintain a moist condition as would be expected in any concrete placement. In addition, consideration should be given to doweling slabs into adjacent curbs and foundations to minimize movements and offsets which could lead to a potential tripping hazard.

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

7.11 Preliminary Pavement Recommendations

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

7.11.2 The following pavement sections are based on an assumed R-Value of 20. Once site grading activities are complete, it is recommended that laboratory testing confirm the properties of the soils serving as paving subgrade prior to placing pavement.

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

PRELIMINARY PAVEMENT DESIGN SECTIONS

Location	Estimated Traffic Index (TI)	Asphalt Concrete (inches)	Class 2 Aggregate Base (inches)
Automobile Traffic & Driveway	5	3	7
Trash Truck & Fire Lanes	7	4	12½

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

7.11.5 Unless specifically designed and evaluated by the project structural engineer, where exterior concrete paving will be utilized for support of vehicles, it is recommended that the concrete be a minimum of 6 inches of concrete reinforced with No. 3 steel reinforcing bars placed 18 inches on center in both horizontal directions. Concrete paving supporting vehicular traffic should be underlain by a minimum of 4 inches of aggregate base and a properly compacted subgrade.

7.11.6 The paving subgrade material should be moisture conditioned to 2 percent above optimum moisture content and compacted to at least 92 percent relative compaction, as determined by ASTM Test Method D 1557 (latest edition). Base material should be compacted to at least 95 percent relative compaction, as determined by ASTM Test Method D 1557 (latest edition).

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

7.12 Retaining Walls

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 12 feet. In the event that walls higher than 12 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* sections of this report (see Section 7.6).

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 30 pcf. Calculation of the recommended retaining wall pressures is provided in Figure 6.

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, walls may be designed utilizing a triangular distribution of pressure (at-rest pressure) of 50 pcf. Calculation of the recommended retaining wall pressures is provided in Figure 6.

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 alluvial soils. 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 to account for the expansive potential of the soil placed as engineered fill. 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, Q_L 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 shoring adjacent to the street or driveway areas should be designed to resist a uniform lateral pressure of 100 psf, acting as a result of an assumed 300 psf surcharge behind the shoring due to normal street traffic. If the traffic is kept back at least ten feet from the shoring, 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 2013 CBC).

7.13.2 A seismic load of 23 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 2013 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. We used the peak site acceleration, PGA_M, of 0.617g calculated from ASCE 7-10 Section 11.8.3 and applied a pseudo-static coefficient of 0.3.

7.14 Retaining Wall Drainage

7.14.1 Retaining walls should be provided with a drainage system extended at least two-thirds the height of the wall. At the base of the drain system, a subdrain covered with a minimum of 12 inches of gravel should be installed, and a compacted fill blanket or other seal placed at the surface (see Figure 7). 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 8). These vertical columns of drainage material would then be connected at the bottom of the wall to a collection panel or a one-cubic-foot rock pocket drained by a 4-inch subdrain pipe.

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

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

7.15 Elevator Pit Design

7.15.1 The elevator pit slab and retaining wall should be designed by the project structural engineer. As a minimum the slab-on-grade for the elevator pit bottom should be at least 4 inches thick and reinforced with No. 3 steel reinforcing bars placed 18 inches on center in both horizontal directions, positioned near the slab midpoint. Elevator pit walls may be designed in accordance with the recommendations in the *Foundation Design and Retaining Wall Design* section of this report (see Sections 7.6 and 7.12).

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

7.16 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 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. 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 on the order of 12 feet in height may be required for excavation and construction of the proposed subterranean level and foundations. The excavations are expected to expose artificial fill and alluvial soils, which are suitable for vertical excavations up to 5 feet in height where loose soils or caving sands are not present, and where not surcharged by adjacent traffic or structures.

7.17.2 Vertical excavations greater than five feet will require sloping and/or shoring measures in order to provide a stable excavation. Where sufficient space is available, temporary unsurcharged embankments up to 12 feet high could be sloped back at a uniform 1:1 slope gradient or flatter. 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. Geocon personnel should inspect the soils exposed in the cut slopes during excavation so that modifications of the slopes can be made if variations in the soil conditions occur. All excavations should be stabilized within 30 days of initial excavation.

7.18 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. 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 grading activities, foundations, and/or adjacent drainage systems.

7.18.4 The proposed soldier piles may also be designed as permanent piles. The required pile depth, dimension, spacing should be determined and designed by the project structural and shoring engineers. All piles utilized for shoring can also be incorporated into a permanent retaining wall system (shotcrete wall) and should be designed in accordance with the earth pressure provided in the *Retaining Walls* section of this report (see Section 7.12).

7.18.5 Drilled cast-in-place soldier piles should be placed no closer than 2 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 240 pounds per square foot per foot where in contact with alluvial soils. The allowable passive value may be doubled for isolated piles, spaced a minimum of three times the pile diameter. To develop the full lateral value, provisions should be implemented to assure firm contact between the soldier piles and the undisturbed alluvium.

7.18.6 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.

7.18.7 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.

7.18.8 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. Based on Table 19 of the *Transportation and Construction Induced Vibration Guidance Manual* (Caltrans 2004), 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.

7.18.9 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. 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.

7.18.10 Casing may be required since caving may occur in 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. Continuous observation of the drilling and pouring of the piles by the Geotechnical Engineer (a representative of Geocon West, Inc.), is required.

7.18.11 Groundwater was encountered at 30 feet below ground surface during site exploration and the contractor should be prepared for groundwater during pile installation should the need arise. 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.

7.18.12 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.

7.18.13 The frictional resistance between the soldier piles and retained soil may be used to resist the vertical component of the load. The coefficient of friction may be taken as 0.30 based on uniform contact between the steel beam and lean-mix concrete and alluvium. 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 500 pounds per square foot.

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

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

7.18.16 For design of shoring, it is recommended that an equivalent fluid pressure based on the following table, be utilized for design.

HEIGHT OF SHORING (FEET)	EQUIVALENT FLUID PRESSURE (Pounds Per Cubic Foot) (ACTIVE PRESSURE)	EQUIVALENT FLUID PRESSURE (Pounds Per Cubic Foot) (AT-REST PRESSURE)
Up to 12	25	45

7.18.17 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, the at-rest pressure should be considered for design purposes.

7.18.18 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.

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

For $x/H \leq 0.4$

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

and

For $x/H > 0.4$

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

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, Q_L is the vertical line-load and σH is the horizontal pressure at depth z.

7.18.20 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.

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

7.18.22 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.

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

7.19 Tie-Back Anchors

7.19.1 Tie-back anchors may be used to resist lateral loads. 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:

- Up to 5 feet below the top of the excavation – 825 pounds per square foot

7.19.3 An allowable friction capacity of 2 kips per linear foot may be utilized for anchors constructed with post-grouting techniques and a 20 foot length beyond the active wedge. Additional tieback length will yield higher capacity. The maximum allowable friction capacity is 3 kips per linear foot. 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 1,500 pounds per square foot in competent alluvial soil, provided the shallowest point of the footing is at least one foot below the lowest adjacent grade. The client should be aware that the utilization of rakers could significantly impact the construction schedule do to their intrusion into the construction site and potential interference with equipment.

7.23 Stormwater Infiltration

7.23.1 During the May 15, 2014 site exploration, boring B2 was utilized to perform percolation testing. The boring was advanced to a depth of 10 feet below the existing ground surface. Slotted casing was placed in the boring, and the annular space between the casing and excavation were filled with filter pack. The boring was then filled with water to pre-saturate the soils to a depth of approximately 3 feet below ground surface.

7.23.2 On May 16, 2014, upon returning to the site after the 24 hour pre-soak period, water was still present in the boring. The water depth was measured as 5 feet below the ground surface. Geocon remained onsite for an additional hour, and no further dissipation of the water was observed. Based on these considerations, these soils are considered impermeable and are not conducive for infiltration of stormwater. It is recommended that stormwater be retained, filtered, and discharged in accordance with the requirements of the local governing agency.

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 soils can adversely affect the performance of the planned improvements. Saturation of a soil can cause it to lose internal shear strength and increase its compressibility, resulting in a change in the original designed engineering properties. Proper drainage should be maintained at all times.

7.24.2 All site drainage should be collected and controlled in non-erosive drainage devices. Drainage should not be allowed to pond anywhere on the site, and especially not against any foundation or retaining wall. The site should be graded and maintained such that surface drainage is directed away from structures in accordance with 2013 CBC 1804.3 or other applicable standards. In addition, drainage should not be allowed to flow uncontrolled over any descending slope. Discharge from downspouts, roof drains and scuppers are not recommended onto unprotected soils within five feet of the building perimeter. Planters which are located adjacent to foundations should be sealed to prevent moisture intrusion into the soils providing foundation support. Landscape irrigation is not recommended within five 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. The building pad and pavement areas should be fine graded such that water is not allowed to pond.

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

7.25 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 present date. 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.

LIST OF REFERENCES

Anderson, J.G., 1984, *Synthesis of Seismicity and Geologic Data in California,* U.S. Geological Survey Open File Report 84-424.

Applied Technology Council, 1978, *Tentative Provisions for Development of Seismic Regulations for Buildings*, ATC Publication ATC 3-06, NBS Special Publication 510, NSF Publication 78-8.

Blake, T.F., 2000, *EQFAULT, A Computer Program for the Deterministic Prediction of Peak Horizontal Acceleration from Digitized California Faults*, Version 2.20.

Blake, T.F., 2000, *EQSEARCH, A Computer Program for the Estimation of Peak Horizontal Acceleration from California Historical Earthquake Catalogs,* Version 2.20.

Blake, T.F., 2000, *FRISKSP, A Computer Program for the Probabilistic Estimation of Uniform-Hazard Spectra Using 3-D Faults as Earthquake Sources.*

Boore, D.M., Joyner, W.B., and Fumal, T.E., 1997, *Equations for Estimating Horizontal Response Spectra and Peak Acceleration from Western North American Earthquakes,* A *Summary of Recent Work*, Seismological Research Letters, Vol. 68, No. 1, pp. 128-153.

Bryant, W.A. and Hart, E.W., 2007, *Fault-Rupture Hazard Zones in California, Alquist-Priolo Earthquake Fault Zoning Act with Index to Earthquake Fault Zone Maps*, California Division of Mines and Geology Special Publication 42, interim revision.

Bryant, W. A., 1988, *Recently Active Traces of the Newport-Inglewood Fault Zone, Los Angeles and Orange Counties, California,* California Division of Mines and Geology Open File Report 88-14.

Bryant, W. A., 1986, *Newport-Inglewood Fault Zone Across Southwest Newport Mesa, Orange County,* California, California Division of Mines and Geology Fault Evaluation Report FER 172.

California Department of Water Resources, 1967, *Progress Report on Groundwater Geology of the Coastal Plain of Orange County.*

California Division of Mines and Geology, 2001, *Seismic Hazard Evaluation of the Tustin 7.5-Minute Quadrangle, Orange County, California*, Open File Report 97-20.

California Division of Mines and Geology, 2001, *State of California*, *Seismic Hazard Zones, Tustin Quadrangle, Official Map Released: January 17, 2001.*

California Division of Mines and Geology, 1981, *Geologic Map of Orange County, California, Showing Mines and Mineral Deposits*, Bulletin 204, Plate 1, Scale: 1:48000.

California Division of Oil, Gas and Geothermal Resources, 2004; *Regional Wildcat Map, Orange County*, Map Number W1-6.

California Geological Survey, 2008, *Guidelines for Evaluating and Mitigating Seismic Hazards in California,* Special Publication 117A.

Chang, S.W., et al., 1994, *Ground Motions and Local Site Effects,* University of California at Berkeley Earthquake Engineering Research Center, Report No. UCB/EERC-94/08, p.28.

LIST OF REFERENCES (continued)

FEMA, 2009, *Flood Insurance Rate Map, Orange County, California and Incorporated Areas, Panel 286 of 539, Map Number 06059C0286J;* Online Flood Hazard Maps, http://www.esri.com/hazards/index.html.

Grant, L. B., Ballenger, L. J., and Runnerstrom, E. E., 2002, *Coastal Uplift of the San Joaquin Hills, Southern Los Angeles Basin, California, by a Large Earthquake Since A. D. 1635*, Bulletin of the Seismological Society of America, Vol. 92, No. 2, pp. 590-599.

Grant, L. B., Mueller, K. J., Gath, E. M., and Munro, R., 2000, *Late Quaternary Uplift and Earthquake Potential of the San Joaquin Hills, Southern Los Angeles Basin, California,* Geology*,* Vol. 28, No. 4, p. 384.

Grant, L. B., et. al., 1999, *Late Quaternary Uplift and Earthquake potential of the San Joaquin Hills, Southern Los Angeles Basin, California.* Geology; Vol. 27; No. 11; pg. 1013-1034

Hauksson, E., 1987, *Seismotectonics of the Newport-Inglewood Fault Zone in the Los Angeles Basin, Southern California*, Bulletin of the Seismological Society of America, Vol. 77, pp. 539–561.

Ishihara, K., 1985, *Stability of Natural Deposits During Earthquakes*, Proceedings of the Eleventh International Conference on Soil Mechanics and Foundation Engineering, A. A. Balkema Publishers, Rotterdam, Netherlands, vol. 1, pp. 321-376.

Jennings, C. W., 1994, *Fault Activity Map of California and Adjacent Areas with Locations and Ages of Recent Volcanic Eruptions*, California Division of Mines and Geology Map No. 6.

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

Martin, G.R., and Lew, M., 1999, Co-chairs and Editors of the Implementation Committee, *Recommended Procedures for Implementation of DMG Special Publication 117, Guidelines for Analyzing and Mitigating Liquefaction Hazards in California*, Organized through the Southern California Earthquake Center, University of Southern California.

McGuire, R. K., 1978, *FRISK-A computer based program for seismic risk analysis,* U. S. Geological Survey Open-File Report 78-1007.

Mendenhall, W. C., 1905, *Development of Underground Waters in the Western Coastal Plain Region, Southern California*, U. S. Geological Survey Water Supply Paper 139.

Orange County General Plan, 2004, *Safety Element,* Advance Planning Program, Environmental Management Agency.

Robertson, P.K., 2009, *Performance Based Earthquake Design Using the CPT,* Keynote Lecture, International Conference on Performance-based Design in Earthquake Geotechnical Engineering – from case history to practice, IS-Tokyo, June 2009.

Sadigh, K., Chang, C.Y., Egan, J.A., Makdisi, F., and Youngs, R.R., 1997, *Attenuation Relationships for Shallow Crustal Earthquakes Based on California Strong Motion Data*, Seismological Research Letters, Vol. 68, No. 1.

LIST OF REFERENCES (continued)

Newport Beach, City of, 2006, *Safety Element of the General Plan,* Figures S1 through S3.

Sprotte, E. C., Fuller, D. R., Greenwood, R. B., Mumm, H. A. Real, C. R., and Sherburne, R. W., 1980, *Classification and Mapping of Quaternary Sedimentary Deposits for Purposed of Seismic Zonation, South Coastal Los Angeles Basin, Orange County, California*, California Division of Mines and Geology Open File Report 80-19.

Toppozada, T. R., Bennett, J. H., Borchardt, G. A., Saul, R., and Davis, J .F., 1988, *Planning Scenario for a Major Earthquake on the Newport–Inglewood Fault Zone*, California Division of Mines *and Geology Special Publication 99.*

Wesnousky, S.G., 1986, *Earthquakes, Quaternary Faults and Seismic Hazard in California*, Journal of Geophysical Research, Vol. 91, No. B12, pp. 12,587–12,631.

Zhang, G., Robertson, P.K., Brachman, R., 2002*, Estimating Liquefaction Induced Ground Settlements from the CPT,* Canadian Geotechnical Journal, 39: pp 1168-1180.

Ziony, J.I., and Jones, L.M., 1989, *Map Showing Late Quaternary Faults and 1978–1984 Seismicity of the Los Angeles Region, California*, U.S. Geological Survey Miscellaneous Field Studies Map MF-1964.

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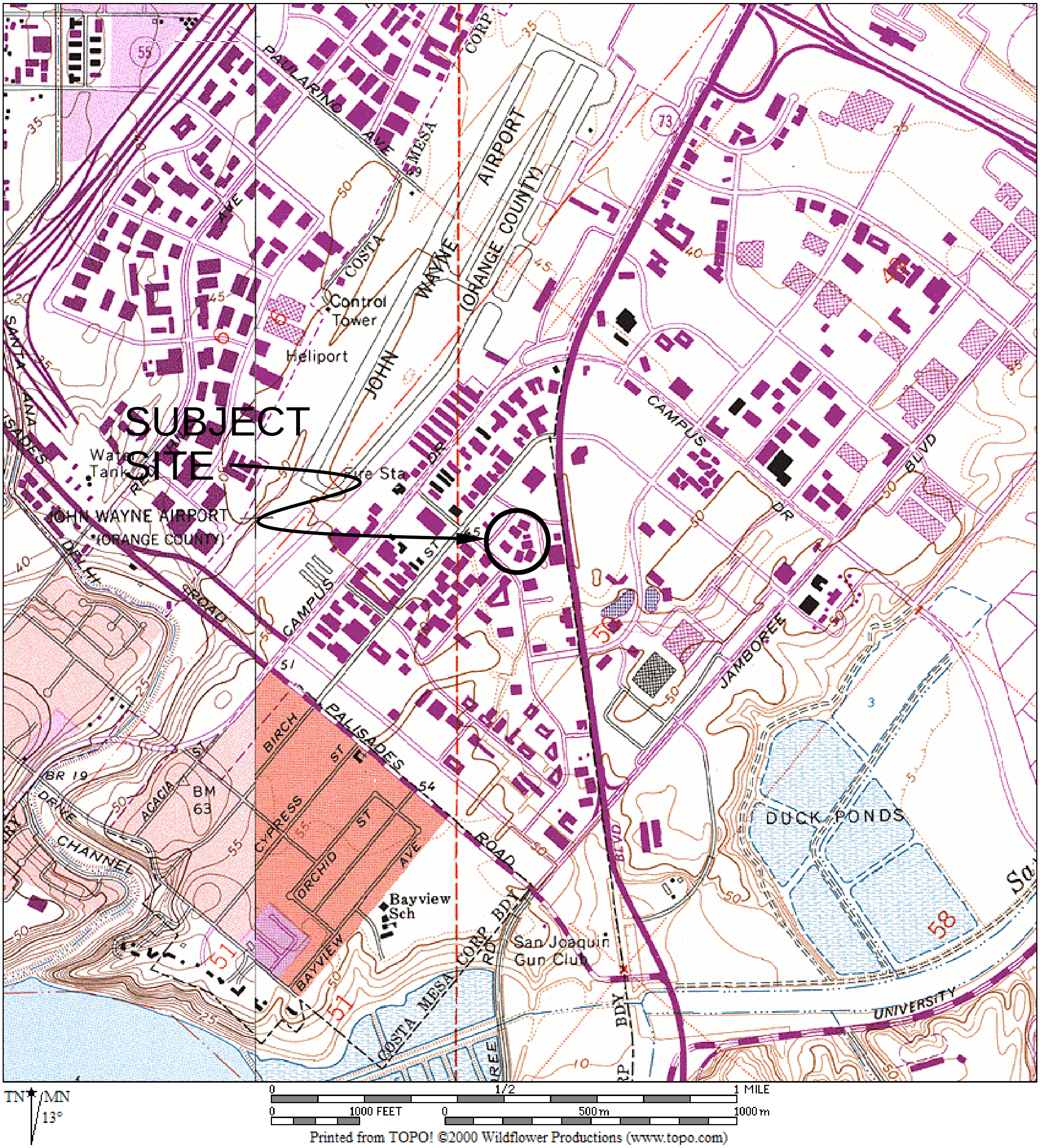

REFERENCE:U.S.G.S. TOPOGRAPHIC MAPS, 7.5 MINUTE SERIES, NEWPORT BEACH AND TUSTIN, CA QUADRANGLES

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AL		9000

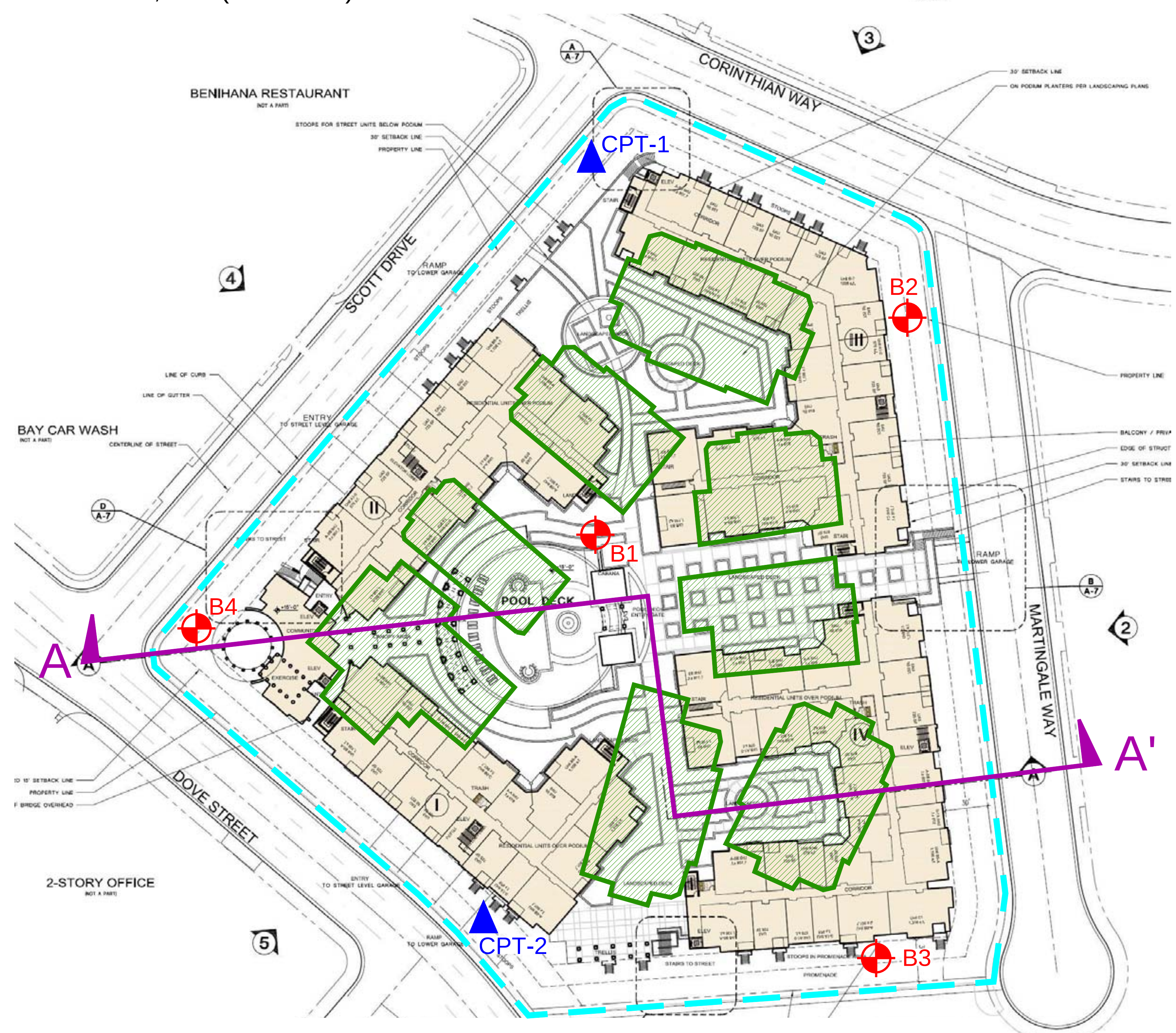
VICINITY MAP

MAC ARTHUR SQUARE

5.6 ACRE AREA BOUNDED BY DOVE STREET,
SCOTT DRIVE, CORINTHIAN WAY, AND MARTINGALE WAY
NEWPORT BEACH, CALIFORNIA

JUNE 12, 2014	PROJECT NO. A9138-06-01	FIG. 1

PLAN BY: ARK ARCHITECTS, INC (3/31/2014)



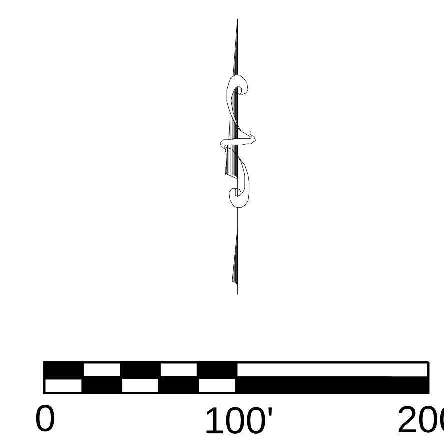
LEGEND

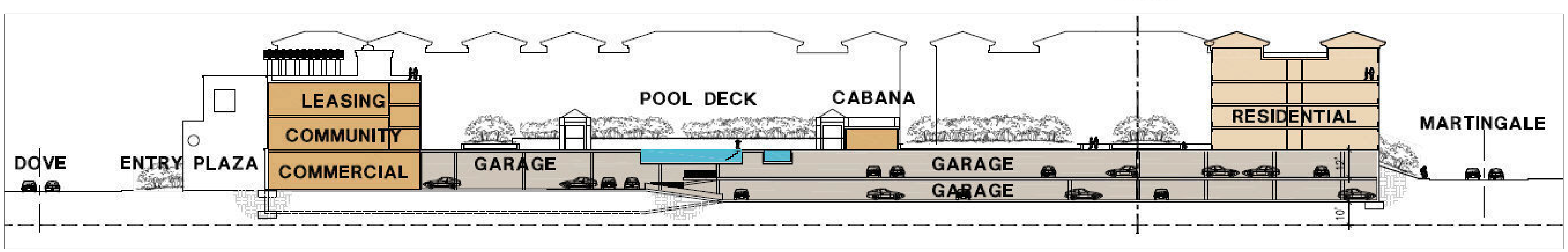
B4 — Approximate Location of Boring

CPT-2 — Approximate Location of CPT

Approximate Limits of Proposed Development

Approximate Location of Existing Structures





SECTION A-A'

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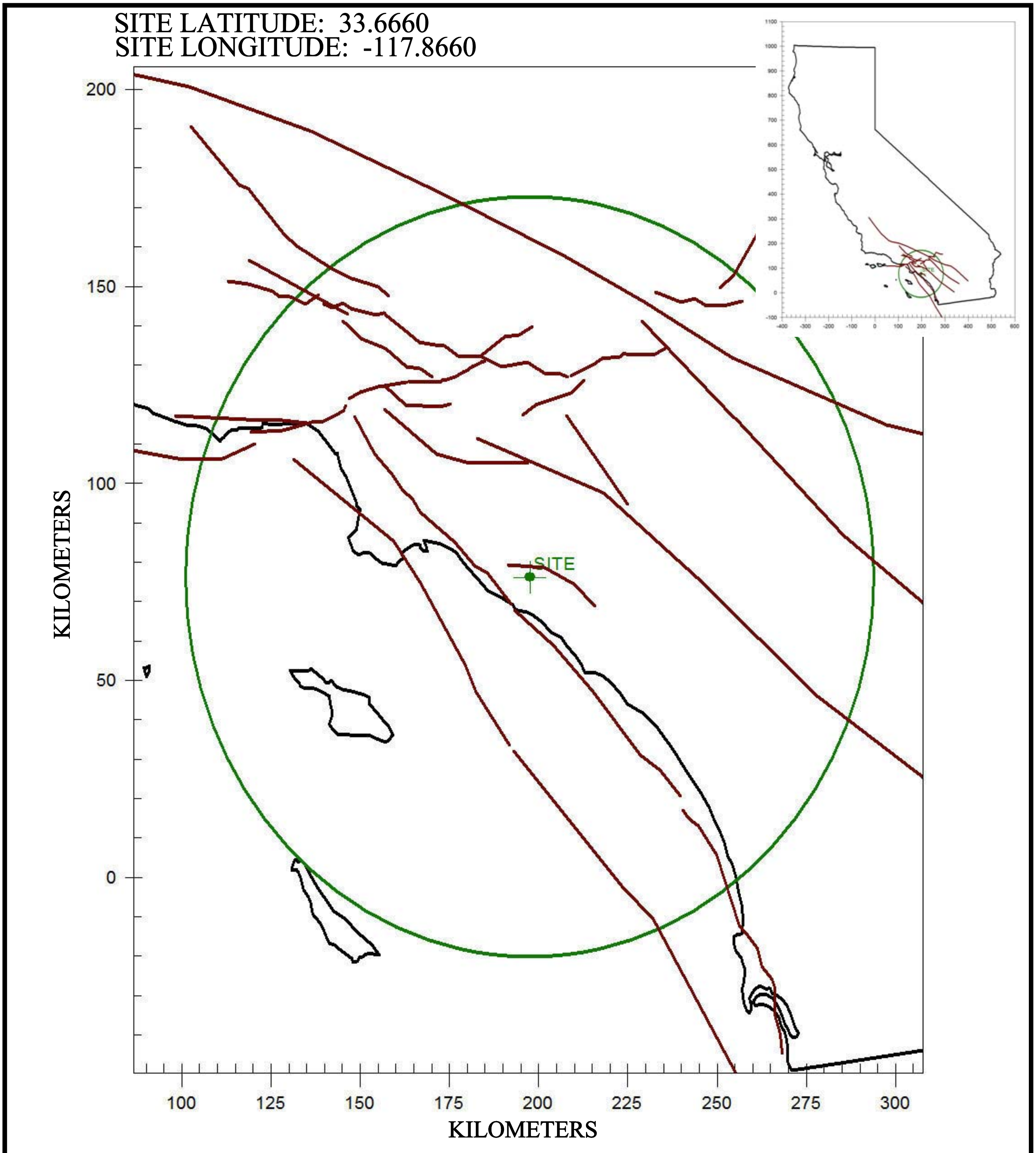
SITE PLAN

MAC ARTHUR SQUARE
5.6 ACRE AREA BOUNDED BY DOVE STREET,
SCOTT DRIVE, CORINTHIAN WAY, AND MARTINGALE WAY
NEWPORT BEACH, CALIFORNIA

JUNE 12, 2014	PROJECT NO. A9138-06-01	FIG. 2



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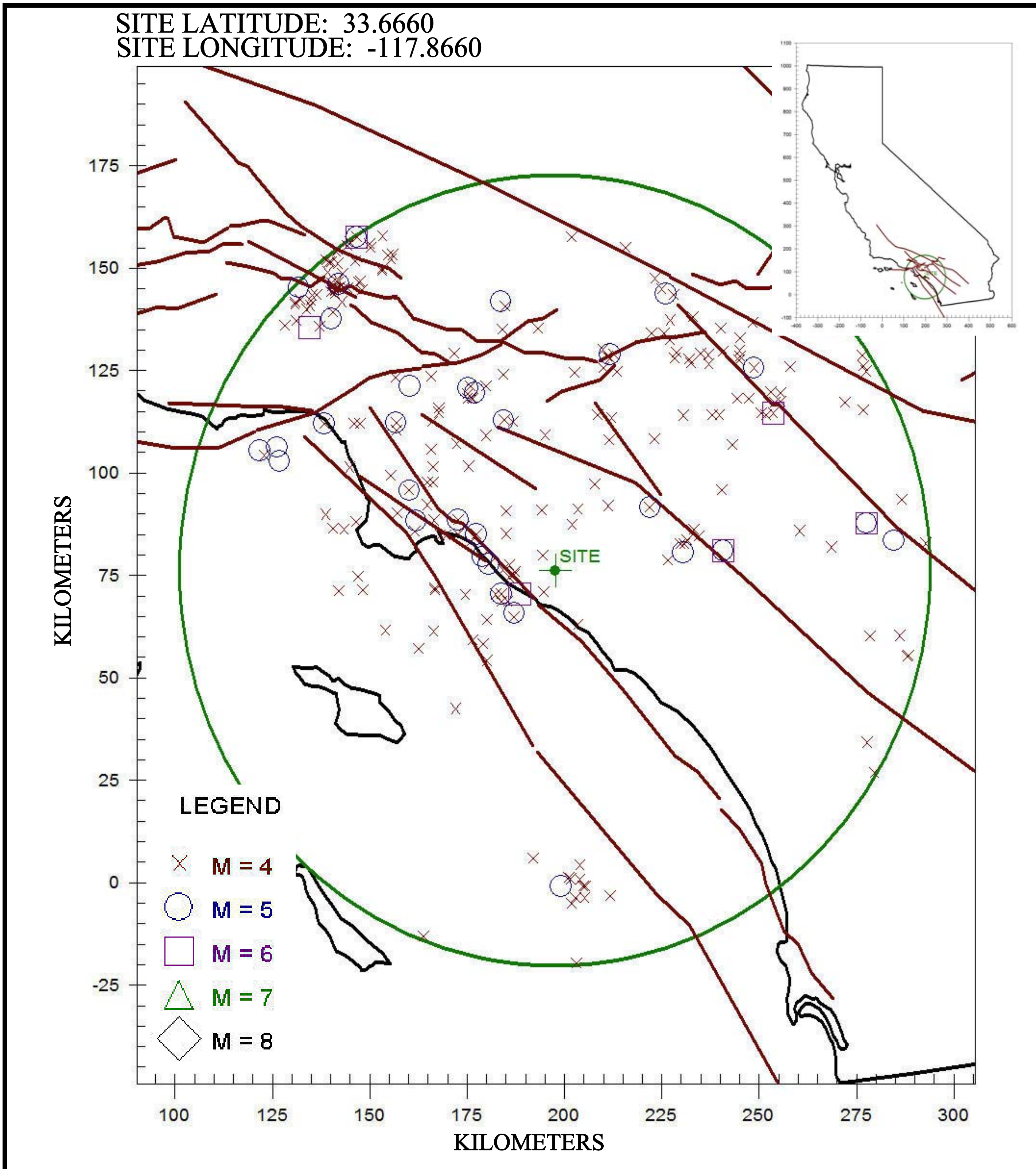
REGIONAL FAULT MAP

MAC ARTHUR SQUARE

5.6 ACRE AREA BOUNDED BY DOVE STREET,
SCOTT DRIVE, CORINTHIAN WAY, AND MARTINGALE WAY
NEWPORT BEACH, CALIFORNIA

AL		9000	JUNE 12, 2014	PROJECT NO. A9138-06-01	FIG. 3



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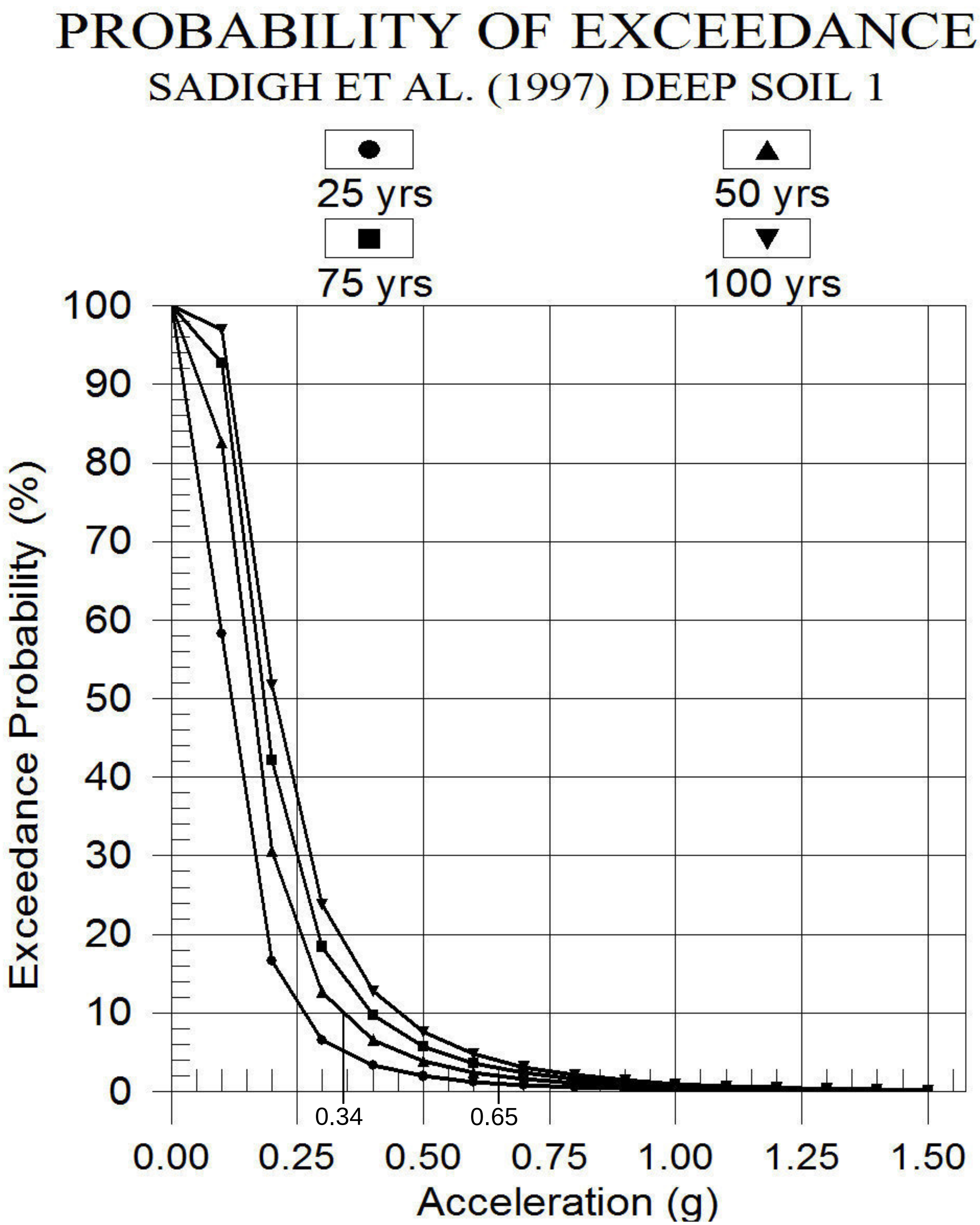
REGIONAL SEISMICITY MAP

MAC ARTHUR SQUARE

5.6 ACRE AREA BOUNDED BY DOVE STREET, SCOTT DRIVE, CORINTHIAN WAY, AND MARTINGALE WAY
NEWPORT BEACH, CALIFORNIA

JUNE 12, 2014	PROJECT NO. A9138-06-01	FIG. 4



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AL		9000

PROBABILITY OF EXCEEDANCE

MAC ARTHUR SQUARE

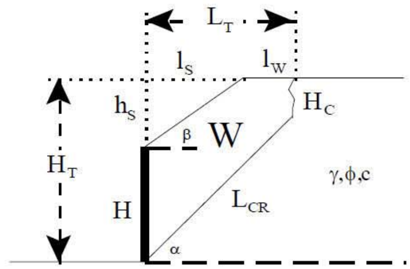
5.6 ACRE AREA BOUNDED BY DOVE STREET,
SCOTT DRIVE, CORINTHIAN WAY, AND MARTINGALE WAY
NEWPORT BEACH, CALIFORNIA

JUNE 12, 2014	PROJECT NO. A9138-06-01	FIG. 5

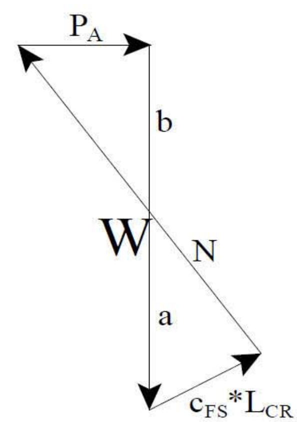
Retaining Wall Design with Transitioned Backfill
(Vector Analysis)

Input:

Retaining Wall Height	(H)	12.00	feet
Slope Angle of Backfill	(β)	0.0	degrees
Height of Slope above Wall	(h_s)	0.0	feet
Horizontal Length of Slope	(l_s)	0.0	feet
Total Height (Wall + Slope)	(H_T)	12.0	feet
Unit Weight of Retained Soils	(γ)	125.0	pcf
Friction Angle of Retained Soils	(ϕ)	27.0	degrees
Cohesion of Retained Soils	(c)	830.0	psf
Factor of Safety	(FS)	1.50	
Factored Parameters:	(ϕ_{FS})	18.8	degrees
	(c_{FS})	553.3	psf



Failure Angle (α) degrees	Height of Tension Crack (H_C) feet	Area of Wedge (A) feet^2	Weight of Wedge (W) lbs/lineal foot	Length of Failure Plane (L_{CR}) feet	a lbs/lineal foot	b lbs/lineal foot	Active Pressure (P_A) lbs/lineal foot
45	13.4	-18	-2220.4	-2.0	-2342.5	122.1	60.2
46	13.2	-14	-1783.9	-1.6	-1867.0	83.1	42.8
47	13.0	-11	-1427.2	-1.3	-1483.1	55.9	30.0
48	12.8	-9	-1138.3	-1.1	-1175.7	37.3	20.9
49	12.7	-7	-907.6	-0.9	-932.5	24.9	14.5
50	12.6	-6	-726.9	-0.7	-743.6	16.7	10.1
51	12.5	-5	-589.5	-0.6	-601.0	11.4	7.2
52	12.4	-4	-489.9	-0.5	-498.2	8.2	5.4
53	12.4	-3	-423.5	-0.5	-429.9	6.4	4.4
54	12.4	-3	-386.3	-0.4	-391.8	5.5	3.9
55	12.4	-3	-375.2	-0.4	-380.6	5.4	4.0
56	12.4	-3	-387.4	-0.5	-393.4	6.0	4.5
57	12.4	-3	-420.7	-0.5	-428.0	7.3	5.8
58	12.5	-4	-473.3	-0.6	-482.9	9.5	7.8
59	12.6	-4	-543.7	-0.7	-556.7	13.0	11.0
60	12.7	-5	-630.8	-0.8	-648.8	18.0	15.8
61	12.9	-6	-733.5	-1.0	-758.6	25.1	22.8
62	13.0	-7	-851.3	-1.2	-886.1	34.8	32.7
63	13.2	-8	-983.8	-1.4	-1031.5	47.7	46.4
64	13.5	-9	-1130.7	-1.6	-1195.4	64.6	65.2
65	13.7	-10	-1292.2	-1.9	-1378.7	86.5	90.3
66	14.0	-12	-1468.5	-2.2	-1582.7	114.2	123.5
67	14.4	-13	-1660.2	-2.6	-1809.3	149.2	167.1
68	14.8	-15	-1868.1	-3.0	-2060.8	192.7	223.5
69	15.2	-17	-2093.4	-3.4	-2339.9	246.4	296.2
70	15.7	-19	-2337.7	-3.9	-2650.3	312.5	389.2



Design Equations (Vector Analysis):

$a = c_{FS}*L_{CR}*\sin(90+\phi_{FS})/\sin(\alpha-\phi_{FS})$

$b = W-a$

$P_A = b*\tan(\alpha-\phi_{FS})$

$EFP = 2*P_A/H^2$

Maximum Active Pressure Resultant

$P_{A,\,max}$ 389.21 lbs/lineal foot

Equivalent Fluid Pressure (per lineal foot of wall)

$EFP = 2*P_A/H^2$

EFP	5.4 pcf	20.7 pcf
Design Wall for an Equivalent Fluid Pressure:	**30 pcf**	**50 pcf**
	Active	At-Rest

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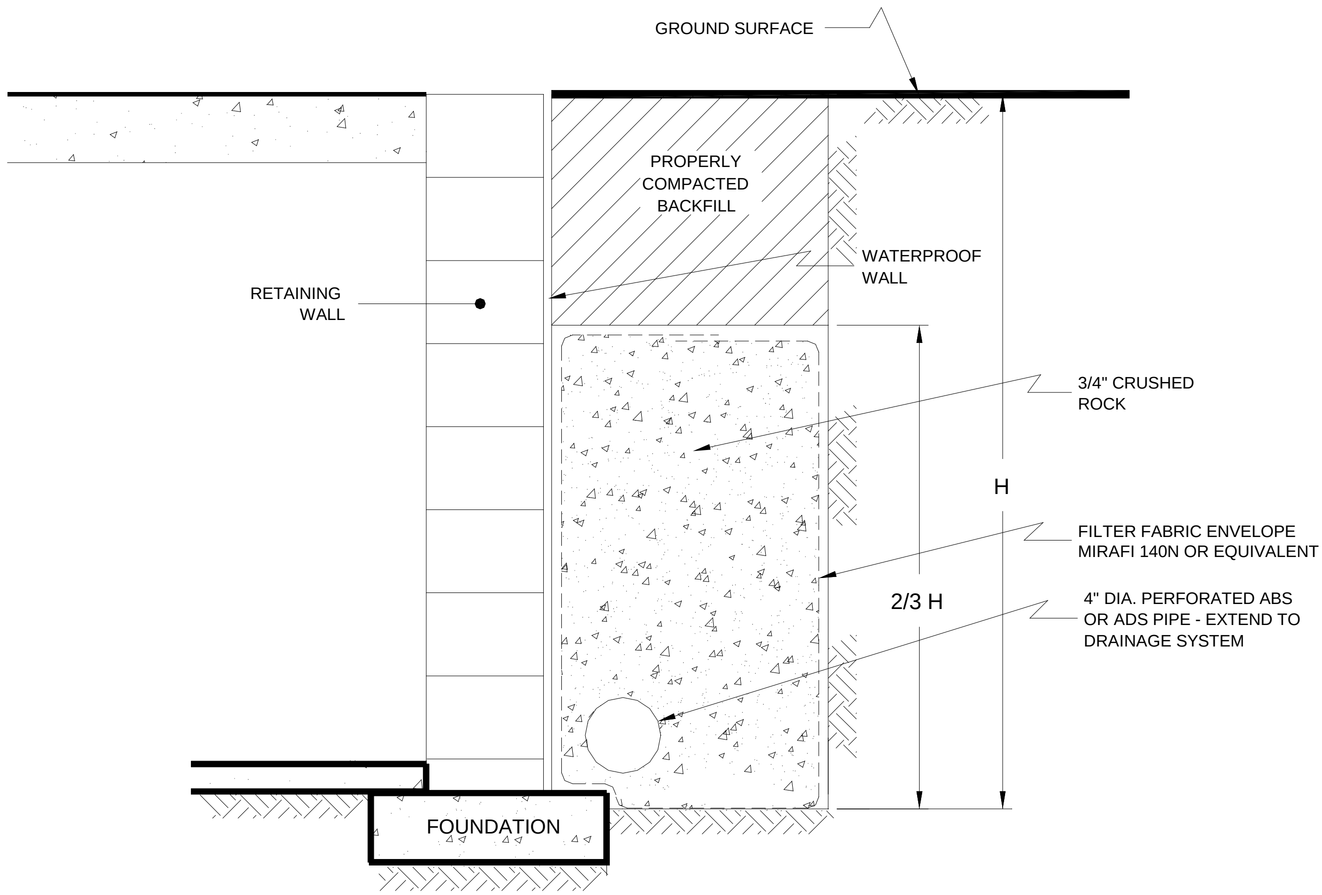
RETAINING WALL PRESSURE CALCULATION

MAC ARTHUR SQUARE

5.6 ACRE AREA BOUNDED BY DOVE STREET,
SCOTT DRIVE, CORINTHIAN WAY, AND MARTINGALE WAY
NEWPORT BEACH, CALIFORNIA

JUNE 12, 2014	PROJECT NO. A9138-06-01	FIG. 6

PA2014-150



NO SCALE

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

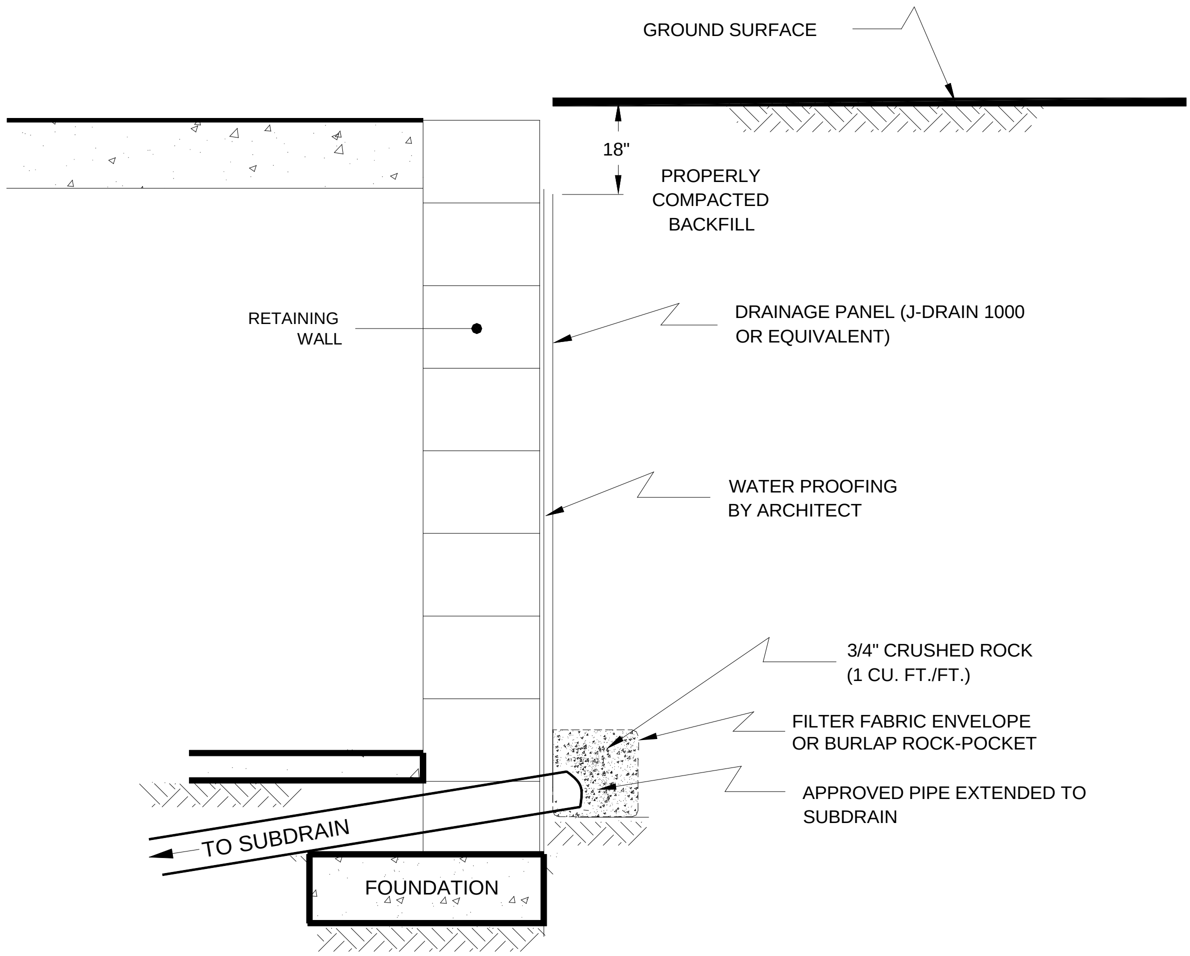
PZ		9000

RETAINING WALL DRAIN DETAIL

MAC ARTHUR SQUARE

5.6 ACRE AREA BOUNDED BY DOVE STREET,
SCOTT DRIVE, CORINTHIAN WAY, AND MARTINGALE WAY
NEWPORT BEACH, CALIFORNIA

JUNE 12, 2014	PROJECT NO. A9138-06-01	FIG. 7



NO SCALE

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WEST, INC.

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

PZ		9000

RETAINING WALL DRAIN DETAIL

MAC ARTHUR SQUARE

5.6 ACRE AREA BOUNDED BY DOVE STREET,
SCOTT DRIVE, CORINTHIAN WAY, AND MARTINGALE WAY
NEWPORT BEACH, CALIFORNIA

JUNE 12, 2014	PROJECT NO. A9138-06-01	FIG. 8

GEOCON

TABLE 1

FAULTS WITHIN 60 MILES OF THE SITE

DETERMINISTIC SITE PARAMETERS

ABBREVIATED FAULT NAME	APPROXIMATE DISTANCE mi (km)	ESTIMATED MAX. EARTHQUAKE EVENT MAXIMUM EARTHQUAKE MAG.(Mw)	PEAK SITE ACCEL. g	EST. SITE INTENSITY MOD.MERC.
SAN JOAQUIN HILLS	1.8(2.9)	6.6	0.934	XI
NEWPORT-INGLEWOOD (L.A.Basin)	5.2(8.4)	7.1	0.533	X
NEWPORT-INGLEWOOD (Offshore)	5.9(9.5)	7.1	0.504	X
PALOS VERDES	16.8(27.1)	7.3	0.276	IX
WHITTIER	17.0(27.4)	6.8	0.217	VIII
PUENTE HILLS BLIND THRUST	18.3(29.4)	7.1	0.298	IX
ELSINORE (GLEN IVY)	18.5(29.8)	6.8	0.200	VIII
CHINO-CENTRAL AVE. (Elsinore)	18.5(29.8)	6.7	0.245	IX
SAN JOSE	25.8(41.5)	6.4	0.144	VIII
CORONADO BANK	27.7(44.6)	7.6	0.207	VIII
ELSINORE (TEMECULA)	29.8(48.0)	6.8	0.118	VII
UPPER ELYSIAN PARK BLIND THRUST	30.9(49.8)	6.4	0.115	VII
SIERRA MADRE	32.2(51.9)	7.2	0.177	VIII
CUCAMONGA	32.7(52.6)	6.9	0.144	VIII
RAYMOND	34.3(55.2)	6.5	0.108	VII
VERDUGO	36.2(58.2)	6.9	0.127	VIII
CLAMSHELL-SAWPIT	36.2(58.2)	6.5	0.101	VII
HOLLYWOOD	37.7(60.6)	6.4	0.089	VII
SANTA MONICA	41.9(67.5)	6.6	0.088	VII
SAN JACINTO-SAN BERNARDINO	42.8(68.8)	6.7	0.071	VI
SAN JACINTO-SAN JACINTO VALLEY	43.7(70.4)	6.9	0.078	VII
ROSE CANYON	45.4(73.0)	7.2	0.092	VII
MALIBU COAST	45.9(73.8)	6.7	0.083	VII
SAN ANDREAS - SB-Coach. M-2b	47.7(76.8)	7.7	0.123	VII
SAN ANDREAS - SB-Coach. M-1b-2	47.7(76.8)	7.7	0.123	VII
SAN ANDREAS - San Bernardino M-1	47.7(76.8)	7.5	0.107	VII
SAN ANDREAS - Whole M-1a	47.7(76.8)	8.0	0.150	VIII
SAN ANDREAS - Mojave M-1c-3	48.0(77.2)	7.4	0.099	VII
SAN ANDREAS - Cho-Moj M-1b-1	48.0(77.2)	7.8	0.131	VIII
SAN ANDREAS - 1857 Rupture M-2a	48.0(77.2)	7.8	0.131	VIII
SIERRA MADRE (San Fernando)	49.0(78.8)	6.7	0.076	VII
CLEGHORN	50.2(80.8)	6.5	0.050	VI
NORTHRIDGE (E. Oak Ridge)	50.4(81.1)	7.0	0.088	VII
SAN GABRIEL	51.0(82.0)	7.2	0.079	VII
ANACAPA-DUME	52.5(84.5)	7.5	0.122	VII
ELSINORE (JULIAN)	52.9(85.2)	7.1	0.070	VI
SAN JACINTO-ANZA	54.8(88.2)	7.2	0.072	VI
NORTH FRONTAL FAULT ZONE (West)	55.7(89.7)	7.2	0.090	VII
SANTA SUSANA	58.1(93.5)	6.7	0.060	VI

39 FAULTS FOUND WITHIN THE SPECIFIED SEARCH RADIUS.

THE SAN JOAQUIN HILLS FAULT IS CLOSEST TO THE SITE.
IT IS ABOUT 1.8 MILES (2.9 km) AWAY.

LARGEST MAXIMUM-EARTHQUAKE SITE ACCELERATION: 0.9336 g

APPENDIX A

FIELD INVESTIGATION

The site was explored on May 15, 2014 by drilling four 8-inch diameter borings utilizing a truck-mounted hollow-stem auger drilling machine. The borings were drilled to depths between 10½ and 30½ feet below the existing ground surface. The approximate locations of the explorations are depicted on the Site Plans (see Figure 2).

The soil conditions encountered in the boring were visually examined, classified and logged in general accordance with the Unified Soil Classification System (USCS). The logs of the borings are presented on Figures A1 through A4. The logs depict the soil and geologic conditions encountered and the depth at which samples were obtained.

In addition, two Cone Penetrometer Tests (CPT) were advanced by Kehoe Testing and Engineering to depths of 50½ feet below the existing ground surface. Logs of the CPT soundings are presented as Figures A5 and A6.

PROJECT NO.

BORING 1

ELEV. (MSL.) - - DATE COMPLETED **5/15/14**

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: 3" BASE: 4"**			
					ARTIFICIAL FILL Silty Sand/Sandy Silt, medium dense to stiff, slightly moist, reddish brown, fine- to medium-grained, trace fine gravel, some clay			
2	B1@2'					35	125.3	6.5
4					**MARINE TERRACE DEPOSITS** Sandy Silt, stiff, slightly moist, yellowish brown, fine- to medium-grained, some clay			
	B1@5'					27	121.7	8.4
6				ML	-Loose, increase in medium-grained with some coarse-grained, decrease in silt content			
	B1@7'					11	103.7	2.4
8								
					Silt with Sand, stiff, slightly moist, yellowish brown, very fine- to fine-grained			
10	B1@10'					24	115.1	8.9
	BULK 10-15'							
12	B1@12'					25	115.7	9.4
				ML				
14					-Slightly porous			
	B1@15'					35	115.2	11.2
16								
					Clay with Sand, stiff, slightly moist, olive brown with oxidation mottles, very fine- to fine-grained, some silt, trace fine gravel, moderate plasticity			
18	B1@18'					25	111.3	10.5
20	B1@20'					39	111.4	12.2
				CL				
22	B1@22'					27	107.2	13.7
24					-Increase in silt content			
	B1@25'					23	108.0	13.6
26					Sandy Silt, stiff, moist, olive brown with oxidation mottles, very fine- to fine-grained			
	B1@27'			ML	-Increase in sand content	33	111.6	14.2
28								
			▼	SP-SM	Sand with Silt, poorly graded, medium dense, wet, olive brown to yellowish			

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

A9138-06-01 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.

GEOCON

PROJECT NO.

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	BORING 1 ELEV. (MSL.) - - DATE COMPLETED 5/15/14 EQUIPMENT HOLLOW STEM AUGER BY: PZ	PENETRATION RESISTANCE (BLOWS/FT)*	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
					MATERIAL DESCRIPTION			
30	B1@30'				brown, fine- to medium-grained	27	104.2	11.8
					Total depth of boring: 30.5 feet. Fill to 4 feet. Groundwater encountered at 30 feet. Backfilled with soil cuttings and tamped. Asphalt patched. *Penetration resistance for 140 pound hammer falling 30 inches by auto hammer.			

A9138-06-01 BORING LOGS.GPJ

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

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

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

GEOCON

PROJECT NO.

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	BORING 2 ELEV. (MSL.) - - DATE COMPLETED 5/15/14 EQUIPMENT HOLLOW STEM AUGER BY: PZ	PENETRATION RESISTANCE (BLOWS/FT)*	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
					MATERIAL DESCRIPTION			
0	BULK 0-5'				**ASPHALT: 4" BASE: 3"** **ARTIFICIAL FILL** Clayey Sand/Sandy Clay, firm, slightly moist, dark brown, fine-grained, moderate plasticity			
2	B2@2'				**MARINE TERRACE DEPOSITS** Sandy Silt, firm, slight moist, reddish brown, fine-grained with trace coarse-grained, slighty porous, trace clay	27	119.9	5.9
4	B2@4'			ML	-Increase in silt content, stiff -Yellowish brown, very fine- to fine-grained, trace clay	23	120.8	7.2
6	B2@6'					19	124.0	8.9
8	B2@8'			SM	Silty Sand, loose, slightly moist, yellowish brown, fine- to medium-grained with trace coarse-grained -Decrease in silt content	12	102.0	3.3
10	B2@10'				Total depth of boring: 10.5 feet. Fill to 1.5 feet. Fabric material encountered within asphalt. No groundwater encountered. Percolation testing performed. Backfilled with soil cuttings and tamped. Asphalt patched. *Penetration resistance for 140 pound hammer falling 30 inches by auto hammer.	17	103.0	2.2

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

A9138-06-01 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.

GEOCON

PROJECT NO.

BORING 3

ELEV. (MSL.) **- -** DATE COMPLETED **5/15/14**

EQUIPMENT **HOLLOW STEM AUGER** BY: **PZ**

DEPTH IN FEET	SAMPLE NO.	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT)*	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0			**ASPHALT: 4" BASE: 6"** **ARTIFICIAL FILL** Clayey Sand, firm, slightly moist, olive brown, fine- to medium-grained			
2		ML	**MARINE TERRACE DEPOSITS** Sandy Silt, firm, slightly moist, yellowish brown, fine-grained -Decrease in silt content			
4		SM	Silty Sand, loose, slightly moist, yellowish brown, very fine- to fine-grained			
	B3@5'			10	100.5	2.7
6		ML	Silt with Sand, firm, slightly moist, olive brown, very fine- to fine-grained			
8			-Increase in sand content, stiff			
10	B3@10' BULK 10-15'	ML		29	114.5	6.0
			-Decrease in sand content			
12	B3@12'		-Slightly porous, trace clay, oxidation mottles	26	111.2	12.8
14						
	B3@15'		-Trace fine gravel, stiff	32	106.6	15.4
16						
			Clay with Sand, firm, slightly moist, olive brown, very fine- to fine-grained, some silt, moderate plasticity			
18	B3@18'			19	115.4	9.3
20	B3@20'			22	108.8	13.9
			-Trace fine gravel -Decrease in sand content			
22	B3@22'			25	100.9	16.3
		CL				
24						
	B3@25'		-Moderate to high plasticity	21	103.7	15.0
26			-Increase in silt content, firm			
	B3@27'			16	107.8	15.4
28						

A9138-06-01 BORING LOGS.GPJ

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

SAMPLE SYMBOLS

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

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

GEOCON

PROJECT NO.

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	BORING 3 ELEV. (MSL.) - - DATE COMPLETED 5/15/14 EQUIPMENT HOLLOW STEM AUGER BY: PZ	PENETRATION RESISTANCE (BLOWS/FT)*	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
					MATERIAL DESCRIPTION			
30	B3@30'					20	103.6	16.3
					Total depth of boring: 30.5 feet. Fill to 1.5 feet. No groundwater encountered. Backfilled with soil cuttings and tamped. Asphalt patched. *Penetration resistance for 140 pound hammer falling 30 inches by auto hammer.			

A9138-06-01 BORING LOGS.GPJ

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

SAMPLE SYMBOLS

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

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

GEOCON

PROJECT NO.

BORING 4

ELEV. (MSL.) **- -** DATE COMPLETED **5/15/14**

EQUIPMENT **HOLLOW STEM AUGER** BY: **PZ**

DEPTH IN FEET	SAMPLE NO.	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT)*	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0	BULK 0-5'		**ASPHALT:3" BASE: 4"**			
			ARTIFICIAL FILL Clayey Sand, medium dense, slightly moist, dark brown, fine- to medium-grained with trace coarse-grained, trace fine gravel			
2	B4@2'	CL		19	120.8	11.1
			MARINE TERRACE DEPOSITS Sandy Clay, firm, slightly moist, yellowish brown, fine-grained			
4	B4@4'			18	112.3	10.6
			-Increase in sand content			
			Silty Sand, loose, slightly moist, yellowish brown, fine-grained			
6	B4@6'	SM		8	103.1	4.1
			-Increase in silt content			
			Sandy Silt, firm, slightly moist, olive brown, very fine- to fine-grained			
8	B4@8'			18	107.4	11.1
			Silt with Sand, firm, slightly moist, yellowish brown, very fine- to fine-grained			
10	B4@10'			14	104.4	11.6
12	B4@12'	ML	-Some clay	19	112.4	11.9
14						
	B4@15'			17	105.2	16.9
16						
			Clay with Sand, firm, slightly moist, olive brown, very fine- to fine-grained, moderate plasticity			
18	B4@18'	CL		16	101.5	22.3
20	B4@20'			20	107.6	20.1

Total depth of boring: 20.5 feet.
Fill to 1 foot.
No groundwater encountered.
Backfilled with soil cuttings and tamped.
Asphalt patched.

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

A9138-06-01 BORING LOGS.GPJ

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

SAMPLE SYMBOLS

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

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

GEOCON

Kehoe Testing and Engineering
714-901-7270
rich@kehoetesting.com
www.kehoetesting.com

CPT: CPT-1
Total depth: 50.10 ft, Date: 5/16/2014
Cone Type: Vertek

Project: Geocon West, Inc.
Location: 4200 Scott Dr. Newport Beach, CA

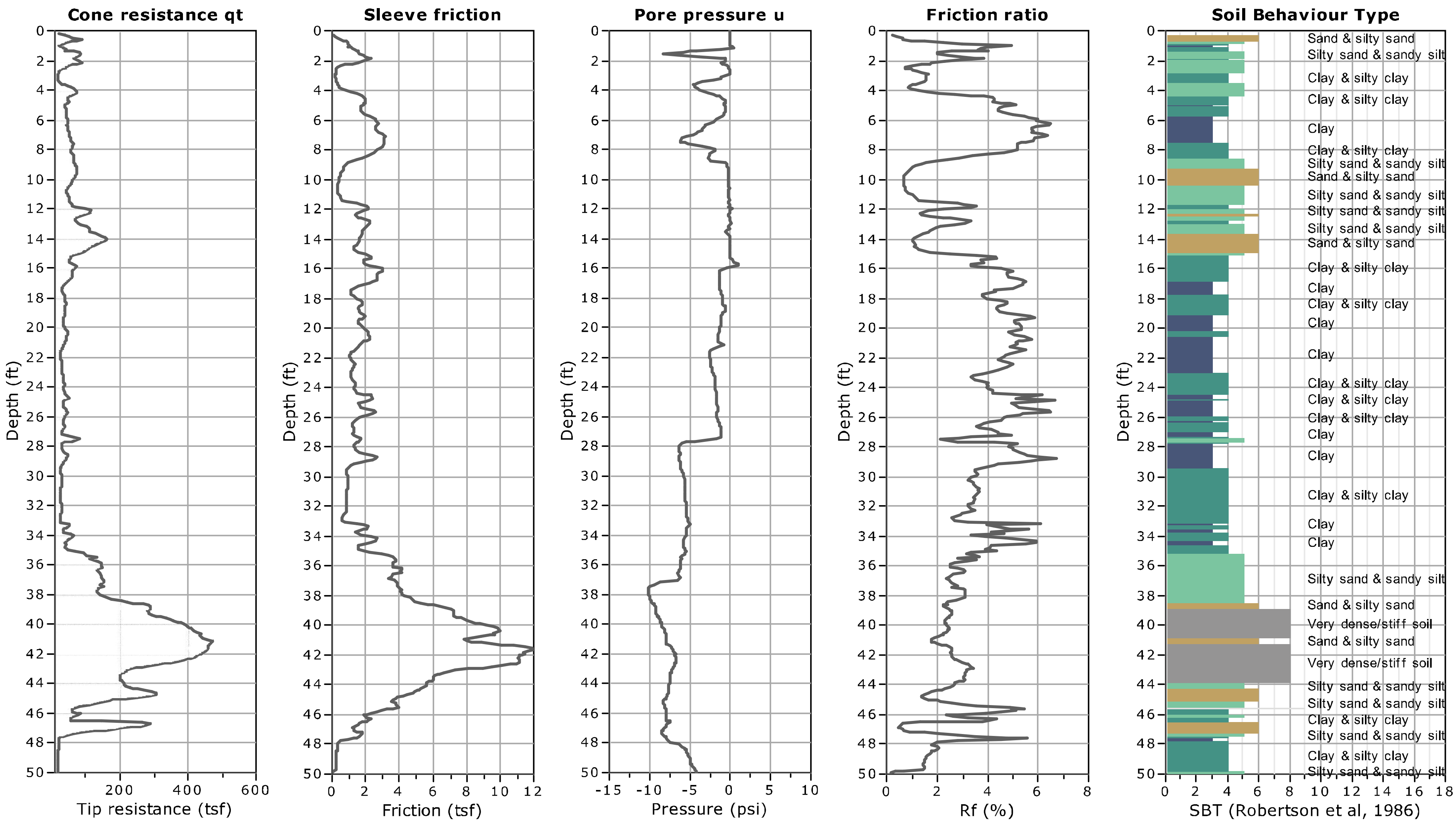


FIG. A5

Kehoe Testing and Engineering
714-901-7270
rich@kehoetesting.com
www.kehoetesting.com

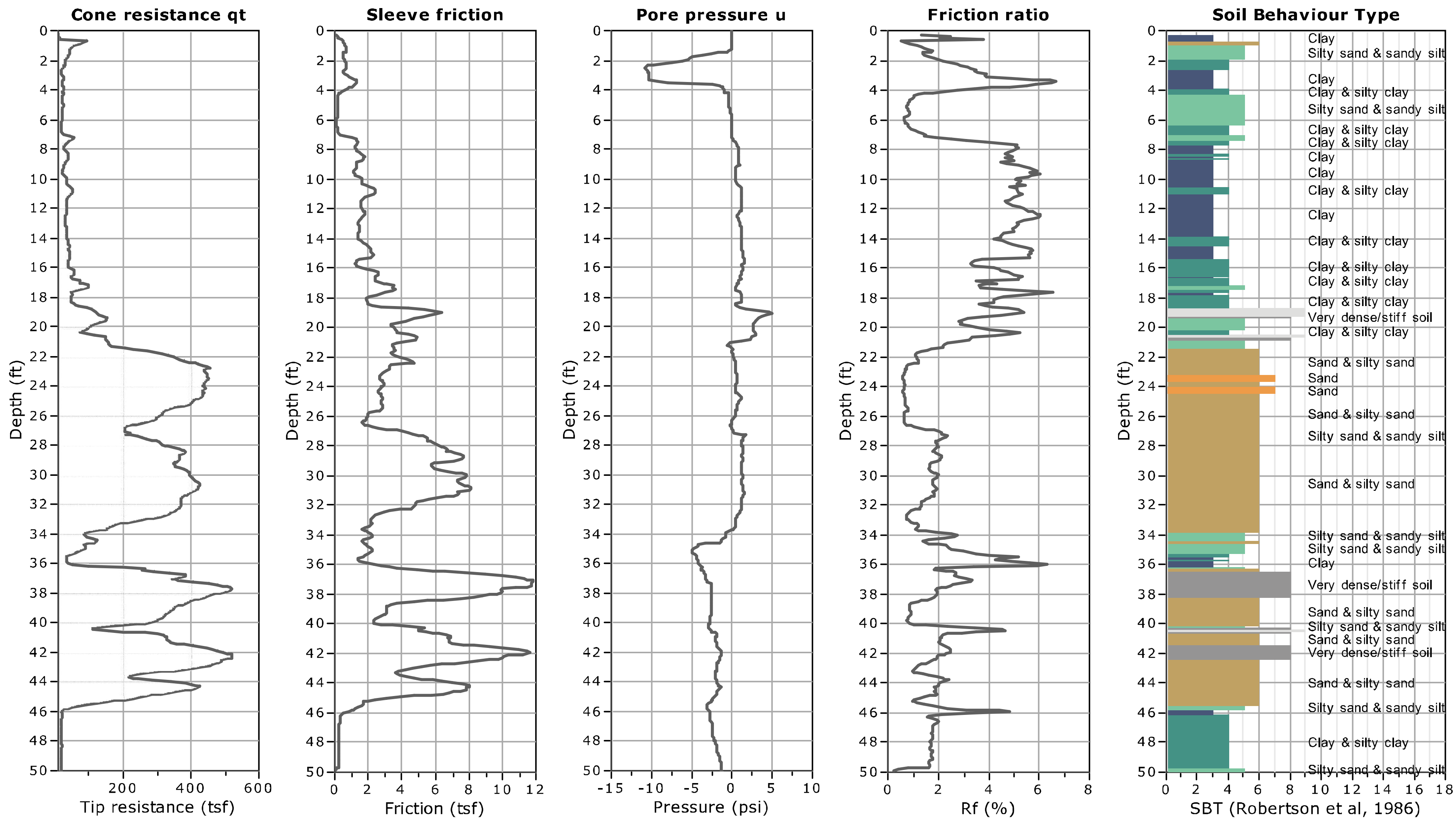
CPT: CPT-2

Total depth: 50.05 ft, Date: 5/16/2014

Cone Type: Vertek

Project: Geocon West, Inc.

Location: 4200 Scott Dr. Newport Beach, CA



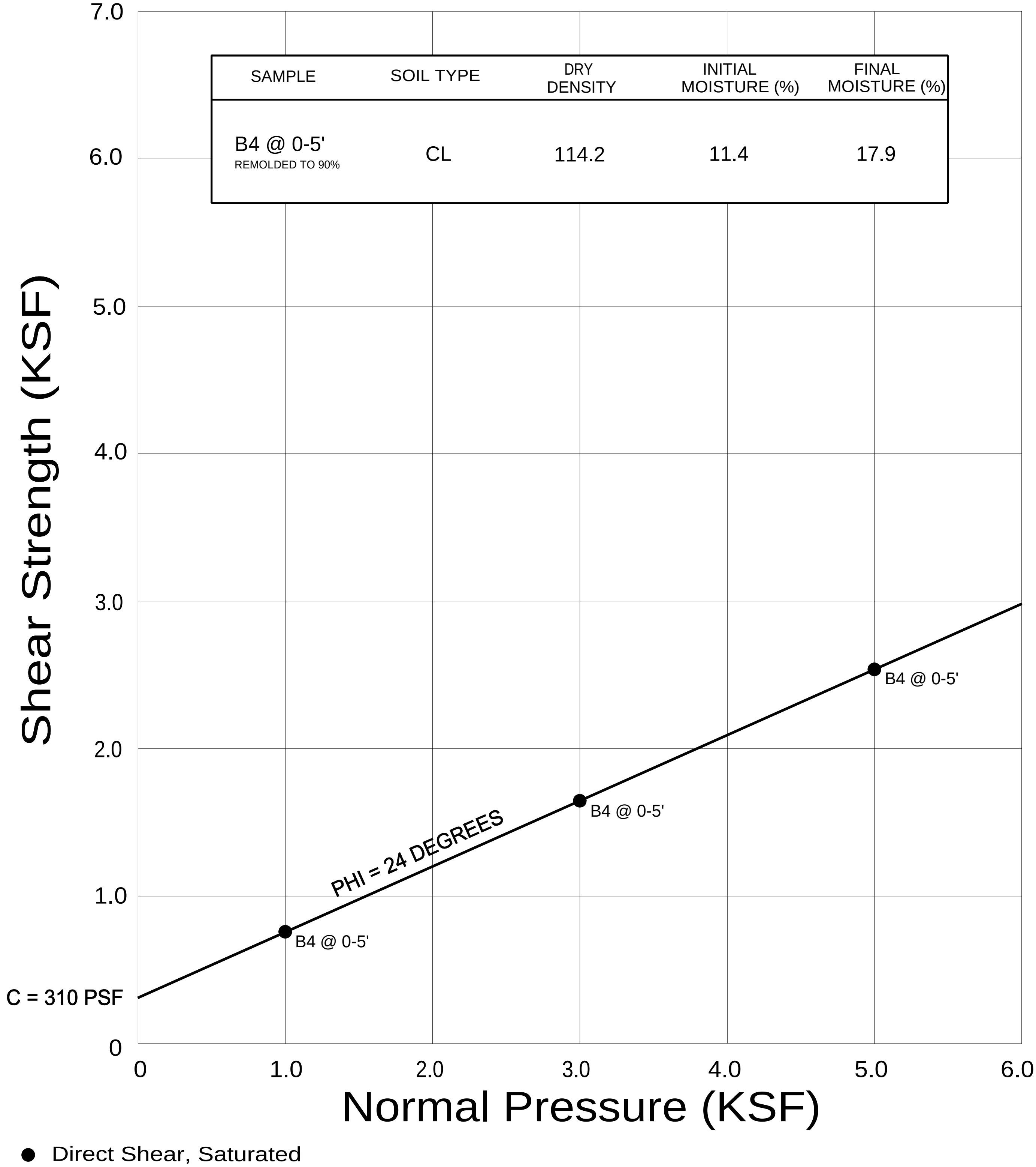
CPeT-IT v.1.7.6.33 - CPTU data presentation & interpretation software - Report created on: 5/20/2014, 11:43:52 AM
Project file: C:\GeoconNewportBch5-14\CPeT Data\Plot Data\Plots.cpt

FIG. A6

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, moisture density relationships, corrosivity, plasticity indices, in-place dry density and moisture content. The results of the laboratory tests are summarized in Figures B1 through B10. 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 (%)
B4 @ 0-5' REMOLDED TO 90%	CL	114.2	11.4	17.9

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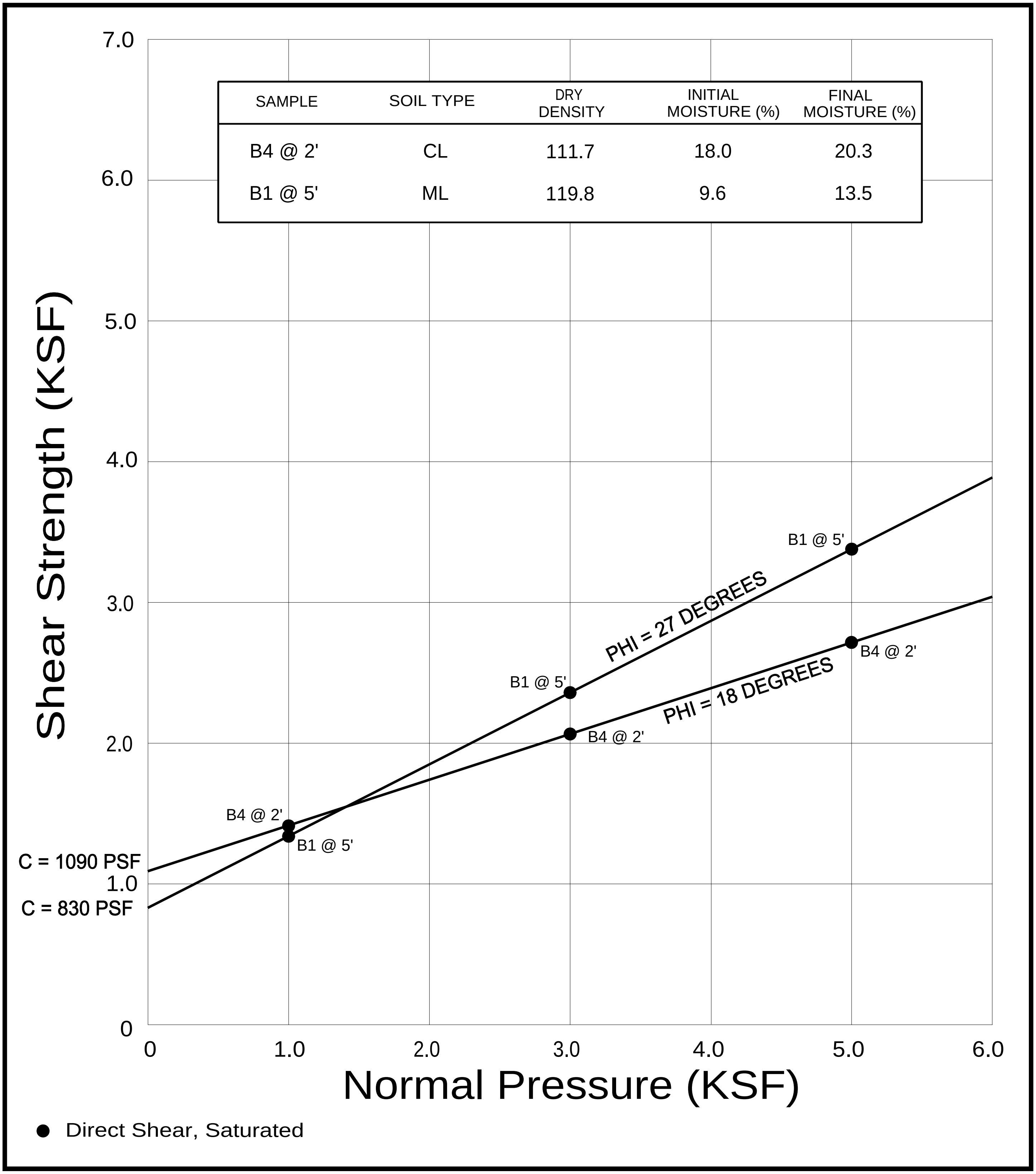
PZ		9000

DIRECT SHEAR TEST RESULTS

MAC ARTHUR SQUARE
5.6 ACRE AREA BOUNDED BY DOVE STREET,
SCOTT DRIVE, CORINTHIAN WAY, AND MARTINGALE WAY
NEWPORT BEACH, CALIFORNIA

JUNE 12, 2014	PROJECT NO. A9138-06-01	FIG. B1



SAMPLE	SOIL TYPE	DRY DENSITY	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B4 @ 2'	CL	111.7	18.0	20.3
B1 @ 5'	ML	119.8	9.6	13.5

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PZ		9000

DIRECT SHEAR TEST RESULTS

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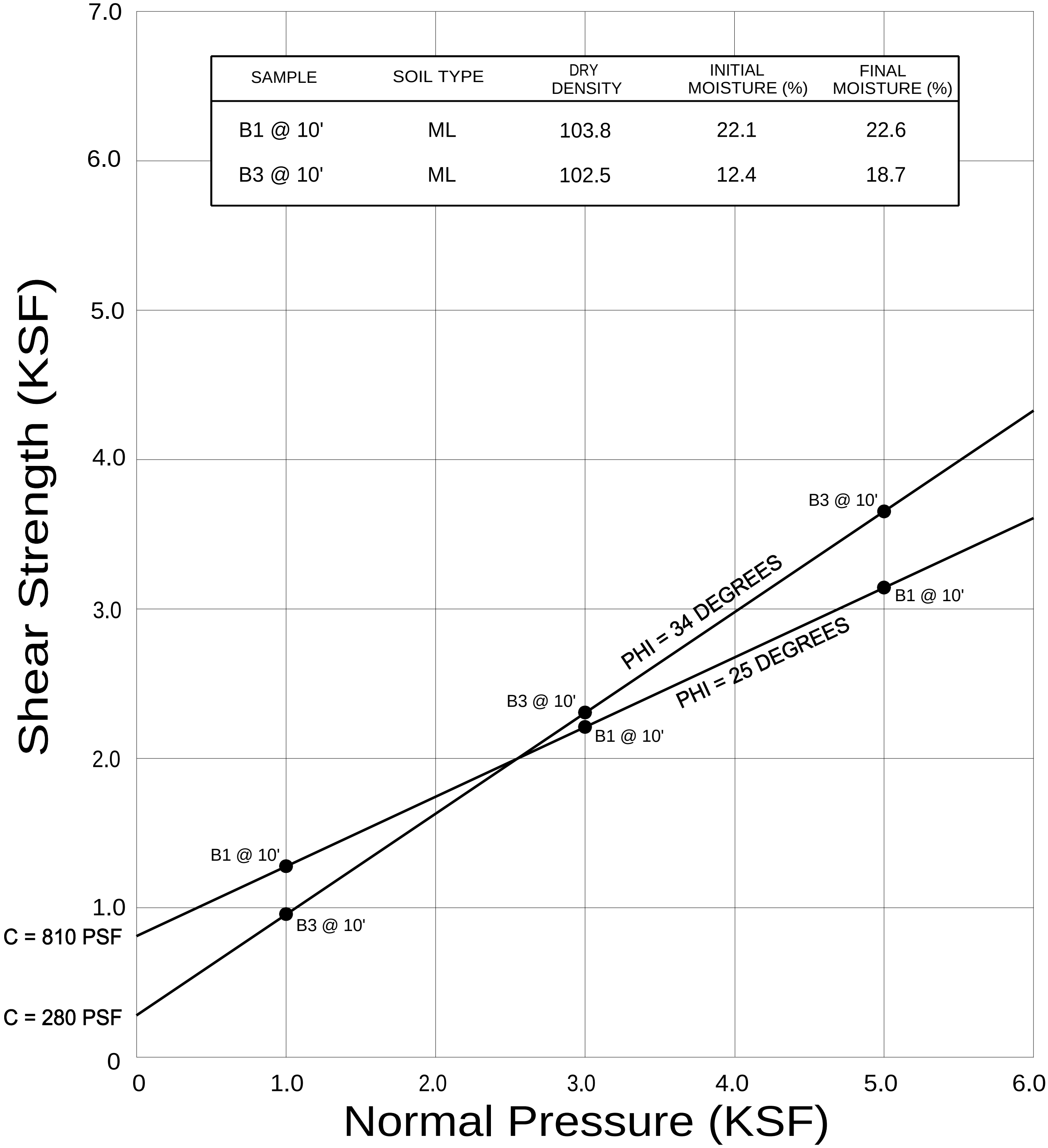
NEWPORT BEACH, CALIFORNIA

JUNE 12, 2014	PROJECT NO. A9138-06-01	FIG. B2



SAMPLE	SOIL TYPE	DRY DENSITY	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B1 @ 10'	ML	103.8	22.1	22.6
B3 @ 10'	ML	102.5	12.4	18.7

● Direct Shear, Saturated

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PZ		9000

DIRECT SHEAR TEST RESULTS

MAC ARTHUR SQUARE

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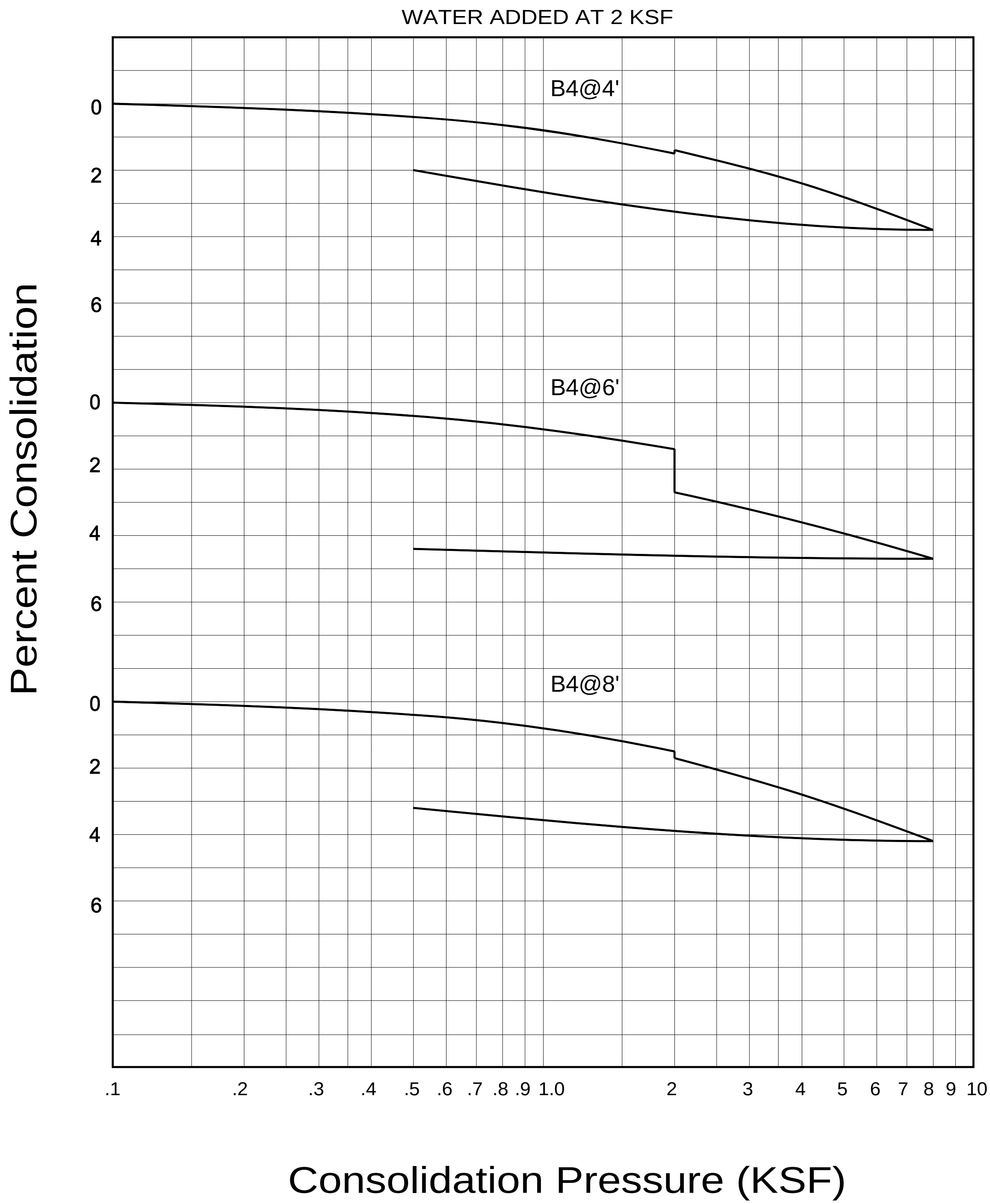
SCOTT DRIVE, CORINTHIAN WAY, AND MARTINGALE WAY
NEWPORT BEACH, CALIFORNIA

JUNE 12, 2014	PROJECT NO. A9138-06-01	FIG. B3



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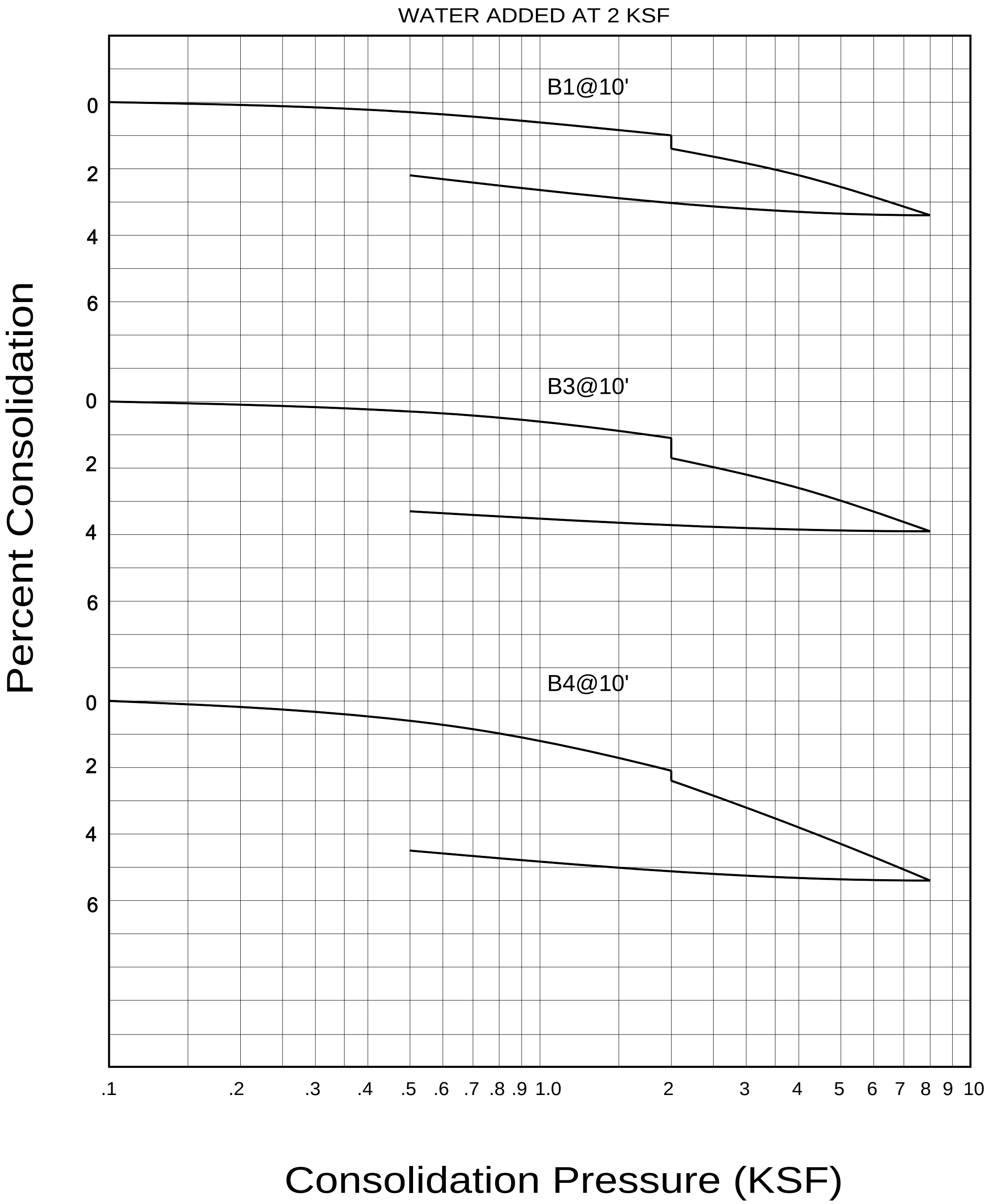
CONSOLIDATION TEST RESULTS

MAC ARTHUR SQUARE

5.6 ACRE AREA BOUNDED BY DOVE STREET,
SCOTT DRIVE, CORINTHIAN WAY, AND MARTINGALE WAY
NEWPORT BEACH, CALIFORNIA

PZ		9000	JUNE 12, 2014	PROJECT NO. A9138-06-01	FIG. B4



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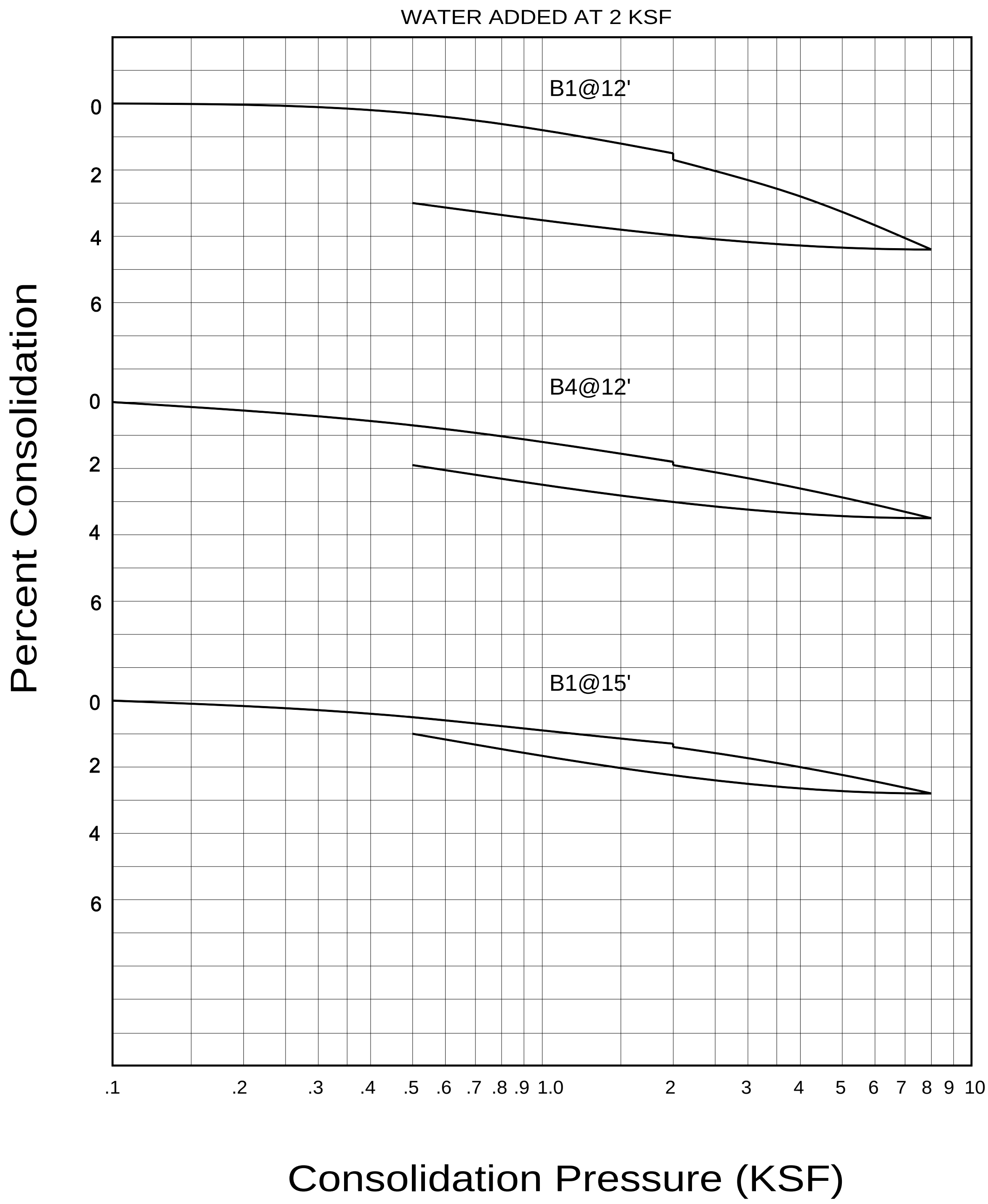
PZ		9000

CONSOLIDATION TEST RESULTS

MAC ARTHUR SQUARE

5.6 ACRE AREA BOUNDED BY DOVE STREET,
SCOTT DRIVE, CORINTHIAN WAY, AND MARTINGALE WAY
NEWPORT BEACH, CALIFORNIA

JUNE 12, 2014	PROJECT NO. A9138-06-01	FIG. B5



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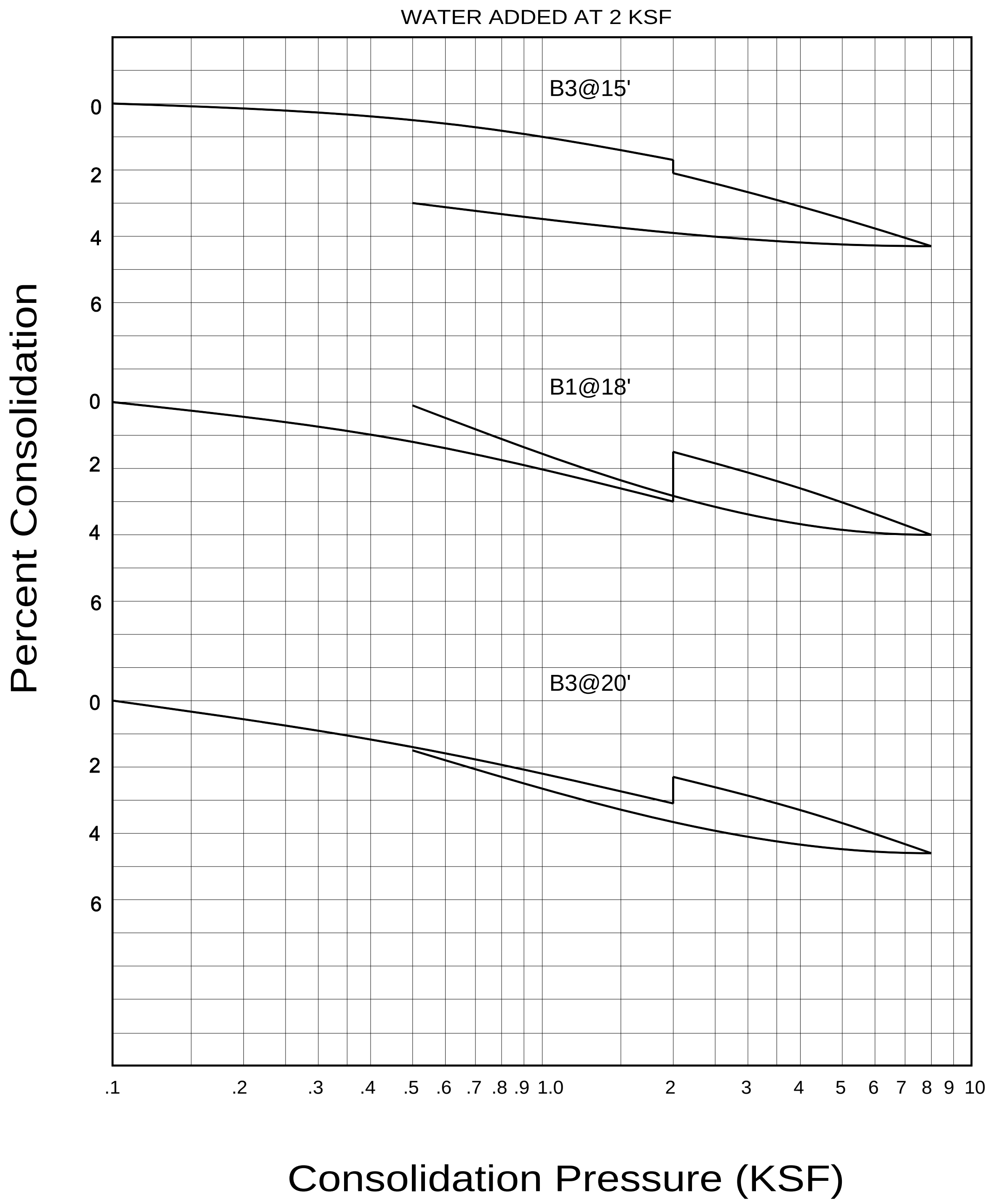
PZ		9000

CONSOLIDATION TEST RESULTS

MAC ARTHUR SQUARE

5.6 ACRE AREA BOUNDED BY DOVE STREET,
SCOTT DRIVE, CORINTHIAN WAY, AND MARTINGALE WAY
NEWPORT BEACH, CALIFORNIA

JUNE 12, 2014	PROJECT NO. A9138-06-01	FIG. B6



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PZ		9000

CONSOLIDATION TEST RESULTS

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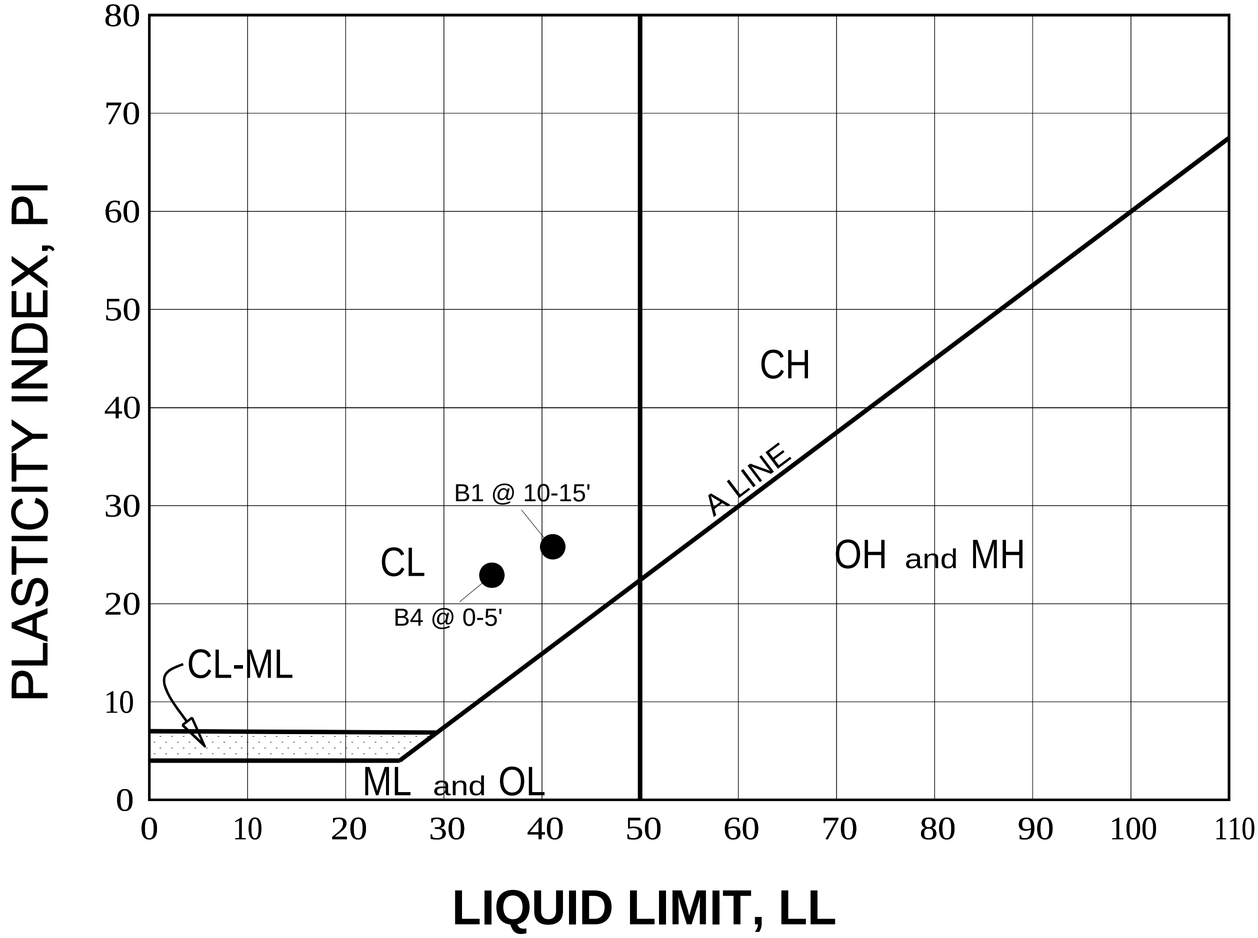
5.6 ACRE AREA BOUNDED BY DOVE STREET,
SCOTT DRIVE, CORINTHIAN WAY, AND MARTINGALE WAY
NEWPORT BEACH, CALIFORNIA

JUNE 12, 2014	PROJECT NO. A9138-06-01	FIG. B7



BORING NUMBER	DEPTH (FEET)	LL	PL	PI	SOIL BEHAVIOR
B4	0 - 5	34.9	11.9	22.9	CL
B1	10 - 15	41.1	15.3	25.8	CL
- -	- - -	- - -	- - -	- - -	- - -
- -	- - -	- - -	- - -	- - -	- - -
- -	- - -	- - -	- - -	- - -	- - -

*N/P indicates Non-Plastic

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PZ		9000

ATTERBERG LIMITS

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5.6 ACRE AREA BOUNDED BY DOVE STREET,
SCOTT DRIVE, CORINTHIAN WAY, AND MARTINGALE WAY
NEWPORT BEACH, CALIFORNIA

JUNE 12, 2014	PROJECT NO. A9138-06-01	FIG. B8

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				
B4 @ 0-5'	9.9	20.7	108.9	51	Moderate	Expansive

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

** Reference: 2013 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 (%)
B4 @ 0-5'	Brown Sandy Clay	128.0	10.5

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LABORATORY TEST RESULTS

MAC ARTHUR SQUARE

5.6 ACRE AREA BOUNDED BY DOVE STREET,
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NEWPORT BEACH, CALIFORNIA

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

Sample No.	pH	Resistivity (ohm centimeters)
B4 @ 0-3'	7.32	2200 (Moderately Corrosive)
B1 & B3 MIX @ 10-15'	7.51	710 (Severely Corrosive)

SUMMARY OF LABORATORY CHLORIDE CONTENT TEST RESULTS EPA NO. 325.3

Sample No.	Chloride Ion Content (%)
B4 @ 0-3'	0.003
B1 & B3 MIX @ 10-15'	0.016

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

Sample No.	Water Soluble Sulfate (% SO_4)	Sulfate Exposure*
B4 @ 0-3'	0.012	Negligible
B1 & B3 MIX @ 10-15'	0.089	Negligible

* Reference: 2013 California Building Code, Section 1904.3 and ACI 318-11 Section 4.3.

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PZ		9000

CORROSIVITY TEST RESULTS

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NEWPORT BEACH, CALIFORNIA

JUNE 12, 2014	PROJECT NO. A9138-06-01	FIG. B10