APPENDIX C

GEOTECHNICAL ENGINEERING INVESTIGATION

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GEOTECHNICAL INVESTIGATION

PROPOSED SORRENTO ALAMITOS BAY SHORELINE TRAIL PROJECT PUBLIC RIGHT-OF-WAY ALONG THE LOS CERRITOS CHANNEL BETWEEN EAST 2ND STREET AND EAST APPIAN WAY LONG BEACH, CALIFORNIA

GEOCON
WEST, INC.

GEOTECHNICAL
ENVIRONMENTAL
MATERIALS

PREPARED FOR

MLA GREEN, INC
D.B.A. MIA LEHRER + ASSOCIATES
LOS ANGELES, CALIFORNIA

PROJECT NO. A9218-06-01

JUNE 11, 2015

GEOCON
WEST, INC.
GEOTECHNICAL ■ ENVIRONMENTAL ■ MATERIALS

Project No. A9218-06-01
June 11, 2015

MLA Green, Inc.
d.b.a. Mia Lehrer + Associates
3780 Wilshire Blvd, Suite 250
Los Angeles, CA 90010

Attention: Mr. Jeff Hutchins

Subject: GEOTECHNICAL INVESTIGATION
PROPOSED SORRENTO ALAMITOS BAY SHORELINE TRAIL
PUBLIC RIGHT-OF-WAY ALONG THE LOS CERRITOS CHANNEL
BETWEEN EAST 2ND STREET AND EAST APPIAN WAY
LONG BEACH, CALIFORNIA

Dear Mr. Hutchins:

In accordance with your authorization of our proposal dated July 17, 2014, we have prepared this geotechnical investigation report for the proposed Sorrento Alamitos Bay Shoreline Trail Project located in the public right-of-way along the Los Cerritos Channel between East 2nd Street and East Appian Way in the City of Long Beach, California. The accompanying report presents the findings of our study, and our conclusions and recommendations pertaining to the geotechnical aspects of proposed design and construction. Based on the results of our investigation, it is our opinion that the site can be developed as proposed, provided the recommendations of this report are followed and implemented during 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.

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GEOTECHNICAL INVESTIGATION

1. PURPOSE AND SCOPE

This report presents the results of a geotechnical investigation for the proposed Sorrento Alamitos Bay Shoreline Trail Project located in the public right-of-way along the Los Cerritos Channel between East 2nd Street and East Appian Way in the City of Long Beach, California (see Vicinity Map, Figure 1). The purpose of the investigation was to evaluate subsurface soil and geologic conditions underlying the site and, based on conditions encountered, to provide conclusions and recommendations pertaining to the geotechnical aspects of design and construction.

The scope of this investigation included a site reconnaissance, field exploration, laboratory testing, engineering analysis, and the preparation of this report. The site was explored on April 17 and May 4, 2015 by excavating thirteen 4-inch diameter borings to depths of approximately 2½ and 8½ feet below the existing ground surface utilizing by manual augers and digging equipment. The approximate locations of the exploratory borings are depicted on the Site Plan (see Figure 2). A detailed discussion of the field investigation, including boring logs, is presented in Appendix A.

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

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

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

2. SITE AND PROJECT DESCRIPTION

The subject site is located in the Naples area of Long Beach, along the waterfront on the south side of the Los Cerritos Channel between East 2nd Street and East Appian Way. The site is bounded by the channel on the west and north, and by multi-family and single-family residential developments on the south and east. Several private piers traverse the public right-of-way connecting the residential developments to private boat docks within Los Cerritos Channel. Topography in the area is relatively level and surface water drainage at the site appears to be by sheet flow along the existing ground contours to the city streets, area drains, and the Los Cerritos Channel.

Based on the information provided by the architect, it is our understanding that the proposed improvements include ADA-compliant walkways, ramps, stairs, site walls, and retaining walls (sea walls) to be constructed in the public right-of-way along the south side of the Los Cerritos Channel between East 2nd Street and East Appian Way. The proposed project will allow the public to easily access the waterfront location. Construction of the walkways, supporting walls, and other improvements will be restricted to existing filled areas (above high tide line) within the 15-foot-wide public right-of-way.

Based on the preliminary nature of the design at this time, wall and column loads were not available. It is anticipated that column loads for the proposed improvements will be up to 1½ kips, and wall loads will be up to 1 kips per linear foot.

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

3. GEOLOGIC SETTING

The site is located in the southern portion of the Los Angeles Basin, a coastal plain between the Santa Monica Mountains to the north, the Puente Hills and Whittier Fault to the east, the Palos Verdes Peninsula and Pacific Ocean to the west and south, and the Santa Ana Mountains and San Joaquin Hills to the southeast. The basin is underlain by a deep structural depression which has been filled by both marine and continental sediments which overlie a basement complex of igneous and metamorphic composition (Yerkes, et al., 1965). The sediments within the central portion of the basin extend to a maximum depth of 32,000 feet below sea level. Regionally, the site is located within the northern portion of the Peninsular Ranges geomorphic province characterized by northwest-trending physiographic and geologic features such as the nearby Newport-Inglewood Fault Zone

4. SOIL AND GEOLOGIC CONDITIONS

Based on our field investigation and published geologic maps of the area, the site is underlain by artificial fill and Older Paralic Deposits (interfingering near-shore marine and continental deposits) consisting of varying amounts of clay, silt, and sand (California Geological Survey, 2010). Detailed stratigraphic profiles are provided on the boring logs in Appendix A.

4.1 Artificial Fill

Artificial fill was encountered in two of our field explorations (borings B-6 and B-9) to a maximum depth of 3½ feet below existing ground surface. The artificial fill generally consists of fine- to medium-grained, dark gray to dark brown sand, silty sand, and clay. The artificial fill is characterized as slightly moist to moist and medium dense to firm. The fill is likely the result of past grading or dredging of the adjacent channel. Deeper fill may exist between excavations and in other portions of the site that were not directly explored.

4.2 Older Paralic Deposits

Pleistocene age Older Paralic Deposits were encountered at the ground surface and beneath the artificial fill and consists primarily of light brown to dark brown and gray poorly graded sand and silty sand with trace silt, rootlets, and seashells. The soils are primarily slightly moist to wet, medium dense and become denser with increased depth.

5. GROUNDWATER

Based on a review of the Seismic Hazard Zone Report for the Long Beach 7.5 Minute Quadrangle, Los Angeles County, California (California Division of Mines and Geology, 1998), the historically highest groundwater level in the site vicinity is less than 20 feet below the existing ground surface. However, there is no specific groundwater level information presented in this report for the site and immediate site vicinity. Groundwater information presented in this document is generated from data collected in the early 1900's to the late 1990s. Based on current groundwater basin management practices, it is unlikely that groundwater levels will ever exceed the historic high levels.

Groundwater was encountered in borings B1, B4, B5, B7, B8, B10 and B12 at depths between 1 and 2 feet below the existing ground surface. Based on the reported historic high groundwater level in the site vicinity and the depth to groundwater encountered during site exploration, groundwater is anticipated to impact the planned construction.

Also, it is common 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. 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 7.14).

6. GEOLOGIC HAZARDS

6.1 Surface Fault Rupture

The numerous faults in Southern California include active, potentially active, and inactive faults. The criteria for these major groups are based on criteria developed by the California Geological Survey (CGS, formerly known as 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 inthe 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 Reservoir Hill-Seal Beach segment of the Newport-Inglewood Fault Zone located approximately 0.6 mile to the northeast (Ziony and Jones, 1989). Other nearby active faults are the Palos Verdes Fault Zone, the Redondo Canyon Fault, and the Whittier Fault located approximately 7.1 miles southwest, 15.3 miles northwest, and 16.8 miles northeast of the site, respectively (Ziony and Jones, 1989). The active San Andreas Fault Zone is located approximately 47 miles northeast of the site.

The closest potentially active fault to the site is the Los Alamitos Fault located approximately 3.7 miles to the northeast (Ziony and Jones, 1989). Other nearby potentially active faults are the Norwalk Fault, the El Modeno Fault, the Pelican Hill Fault, and the Macarthur Park located approximately 10.4 miles northeast, 13.0 miles northeast, 16.0 miles southeast, and 19 miles northwest of the site, respectively (Ziony and Jones, 1989).

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

6.2 Seismicity

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

LIST OF HISTORIC EARTHQUAKES

Earthquake (Oldest to Youngest)	Date of Earthquake	Magnitude	Distance to Epicenter (Miles)	Direction to Epicenter
San Jacinto-Hemet area	April 21, 1918	6.8	66	E
Near Redlands	July 23, 1923	6.3	53	ENE
Long Beach	March 10, 1933	6.4	13	SE
Tehachapi	July 21, 1952	7.5	99	NW
San Fernando	February 9, 1971	6.6	59	NNW
Whittier Narrows	October 1, 1987	5.9	21	N
Sierra Madre	June 28, 1991	5.8	35	NNE
Landers	June 28, 1992	7.3	101	ENE
Big Bear	June 28, 1992	6.4	80	ENE
Northridge	January 17, 1994	6.7	39	NW
Hector Mine	October 16, 1999	7.1	121	ENE

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

6.3 Seismic Design Criteria

The following table summarizes summarizes site-specific design criteria obtained from the 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	1.563g	Figure 1613.3.1(1)
MCE_R Ground Motion Spectral Response Acceleration – Class B (1 sec), S_1	0.583g	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}	1.563g	Section 1613.3.3 (Eqn 16-37)
Site Class Modified MCE_R Spectral Response Acceleration – (1 sec), S_{M1}	0.875g	Section 1613.3.3 (Eqn 16-38)
5% Damped Design Spectral Response Acceleration (short), S_{DS}	1.042g	Section 1613.3.4 (Eqn 16-39)
5% Damped Design Spectral Response Acceleration (1 sec), S_{D1}	0.583g	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	0.601g	Figure 22-7
Site Coefficient, F_{PGA}	1.0	Table 11.8-1
Site Class Modified MCE_G Peak Ground Acceleration, PGA_M	0.601g	Section 11.8.3 (Eqn 11.8-1)

The Maximum Considered Earthquake Ground Motion (MCE) is the level of ground motion that has a 2 percent chance of exceedance in 50 years, with a statistical return period of 2,500 years. According to the 2013 California Building Code and ASCE 7-10, the MCE is to be utilized for the evaluation of liquefaction, lateral spreading, seismic settlements, and it is our understanding that the intent of the Building code is to maintain "Life Safety" during a MCE event. The Design Earthquake Ground Motion (DE) is the level of ground motion that has a 10 percent chance of exceedance in 50 years, with a statistical return period of 475 years.

Deaggregation of the MCE peak ground acceleration was performed using the USGS 2008 Probabilistic Seismic Hazard Analysis (PSHA) Interactive Deaggregation online tool. The result of the deaggregation analysis indicates that the predominant earthquake contributing to the MCE peak ground acceleration is characterized as a 6.74 magnitude event occurring at a hypocentral distance of 10.8 kilometers from the site.

Deaggregation was also performed for the Design Earthquake (DE) peak ground acceleration, and the result of the analysis indicates that the predominant earthquake contributing to the DE peak ground acceleration is characterized as a 6.64 magnitude occurring at a hypocentral distance of 20.4 kilometers from the site.

Conformance to the criteria in the above tables for seismic design does not constitute any kind of guarantee or assurance that significant structural damage or ground failure will not occur if a large earthquake occurs. The primary goal of seismic design is to protect life, not to avoid all damage, since such design may be economically prohibitive.

6.4 Liquefaction Potential

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

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

The State of California Seismic Hazard Zone Map for the Long Beach Quadrangle (1999) indicates that the site is located in an area designated as “liquefiable”. In addition, the City of Long Beach (1988, 2004) and the County of Los Angeles (Leighton, 1990) indicate the site is located within an area identified as having a potential for liquefaction. Based on these considerations, it is our opinion that there is potential for liquefaction and associated ground deformations that could damage the proposed improvements and surrounding strucures.

It is our understanding that the intent of the Building Code is to maintain "Life Safety", and is intended to mitigate the effects of liquefaction induced settlement on proposed habitable structures. The proposed walkway and related improvements are not considered habitable structures; therefore, liquefaction analyses were neither required nor performed.

6.5 Slope Stability

The topography at the site and surrounding area is relatively level. According to the City of Long Beach (1988, 2004) and the County of Los Angeles Safety Element (Leighton, 1990), the site is not within an area identified as having a potential for slope instability. Additionally, the site is not within an area identified as having a potential for seismic slope instability (CDMG, 1999). There are no known landslides near the site, nor is the site in the path of any known or potential landslides. Therefore, the potential for slope stability hazards to adversely affect the proposed development is considered low.

6.6 Earthquake-Induced Flooding

Earthquake-induced flooding is inundation caused by failure of dams or other water-retaining structures due to earthquakes. Based on a review of the Los Angeles County Safety Element (Leighton, 1990), the site is located within the Prado Dam basin flood boundary. However, this reservoir, as well as others in California, is 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 and 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.

6.7 Tsunamis, Seiches, and Flooding

According to the California Geological Survey (2009) and the City of Long Beach (2004), the site is located within a tsunami inundation area. Due to the presence of the Palos Verdes Peninsula, Channel Islands, and the harbor breakwater, the Long Beach coastline and harbor are somewhat protected from tsunami inundation (Woodward-Clyde Consultants, 1988). However, the harbor and coastline are vulnerable to tsunamis generated in the South Seas and offshore Southern California (Woodward-Clyde Consultants, 1988). Published estimates of recurrence intervals indicate maximum wave heights of up to 7.0 feet and 9.7 feet for 100 and 500 year recurrence intervals, respectively (Houston and Garcia, 1974) and 3.0 feet for 50 year recurrence interval (City of Long Beach, 2004). Such events are not expected to cause major damage to on-shore features. However, there is considerable potential for damage to boats, harbor facilities, and light, seafront structures during such events. Warning times of approximately 6 to 12 hours would be expected for distant events. The potential for death or injury from this source is not considered great, although shoreline property damage could be substantial (City of Long Beach, 2004). Seiches are large waves generated in enclosed bodies of water in response to ground shaking. No major water-retaining structures are located immediately up gradient from the project site. Flooding from a seismically-induced seiche is considered unlikely.

The site is in a FEMA designated flood zone AE with a base flood elevation of 9 feet (FEMA, 2015; LACDPW, 2015b). Therefore there is a potential for flooding at the site.

6.8 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 (2004), the site is not located within the boundaries of an oil field. However, the Seal Beach Oil Field is immediately to the north and the Long Beach Oil Field to the southwest of the site. The nearest well to the site is the B&M Oil Co. Well Number 1 (API: 03706959), a plugged oil and gas production well located approximately 0.21 miles to the northwest (DOGGR, 2015). 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.

Since the site is not within the boundaries of a known oil field, the potential for methane and other volatile gases is considered 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.

6.9 Subsidence

Subsidence occurs when a large portion of land is displaced vertically, usually due to the withdrawal of groundwater, oil, or natural gas. Subsidence commonly occurs in such small magnitudes and over such large areas that is it generally imperceptible at an individual locality. Accordingly, it affects only regionally extensive structures sensitive to slight elevation changes, such as canals and pipelines. The rate of elevation change is usually uniform over a large enough area that it does not result in differential settlements that would cause damage to individual buildings. Soils that are particularly subject to subsidence include those with high silt or clay content.

Within the Long Beach area, a substantial level of subsidence has occurred between 1926 through 1967 due to petroleum production from the Wilmington Oil Field. As much as 30 feet of subsidence has been recorded near the Navy dry dock on Terminal Island between 1926 through 1967 (City of Long Beach, 2004).

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

7. CONCLUSIONS AND RECOMMENDATIONS

7.1 General

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

7.1.2 Up to 3½ feet of existing artificial fill was encountered during the site investigation. The existing fill encountered is believed to be the result of past grading and construction activities at the site. Deeper fill may exist in other areas of the site that were not directly explored. Demolition of existing lawn, garden, and improvements which occupy the area of proposed improvements is anticipated to disturb the upper few feet of existing site soils. The existing site soils are suitable for re-use as engineered fill provided the recommendations in the *Grading* section of this report are followed (see Section 7.4).

7.1.3 Based on these considerations, the proposed improvements, consisting of miscellaneous structures such as ADA-compliant walkways, ramps, stairs, and site walls, may be supported on a conventional foundation system bearing on a minimum of 12 inches of newly placed engineered fill. Where engineered fill is to be utilized for foundation support, the excavation should extend laterally a minimum distance of 12 inches beyond the structure footprint area.

7.1.4 As an alternative and where excavation and compaction cannot be performed, such as adjacent to property lines, proposed miscellaneous structures may be supported on a conventional foundation system deriving support in the competent undisturbed Older Paralic deposits found at or below a depth of 2 feet below the existing grade. Where competent Older Paralic deposits are to be utilized for foundation support, foundations should be deepened as necessary to penetrate through any encountered artificial fill or unsuitable soils to derive support in the competent undisturbed Older Paralic deposits. Recommendations for conventional foundation design are provided in the *Conventional Foundation Design* section of this report (see Section 7.5).

7.1.5 It is the intent of the Geotechnical Engineer to allow proposed foundations to derive support in both Older Paralic deposits and engineered fill if project conditions warrant such an occurrence. All foundation excavations must be observed and approved in writing by the Geotechnical Engineer (a representative of Geocon), prior to placing steel or concrete.

7.1.6 Where new foundations are to be constructed immediately adjacent to existing foundations, in order to prevent surcharging the existing foundation, the proposed foundation should be deepened as necessary to match or exceed the depth of the existing foundation. This condition may occur where proposed foundations are to be constructed adjacent to existing foundations.

7.1.7 Where proposed foundations will be deeper than an existing foundation, the new foundation must be designed to resist the surcharge imposed by the existing foundation. The surcharge area may be defined by a 1:1 projection down and away from the bottom of the existing foundation.

7.1.8 A deepened foundation system consisting of drilled cast-in-drilled hole (CIDH) concrete friction piles deriving support in undisturbed Older Paralic deposits may be utilized for support of the proposed retaining walls (sea walls). Recommendations for design and installation of deepened foundations are provided in the *Deepened Foundations* and *Deepened Foundation Installation* sections of this report (see Sections 7.7 and 7.8).

7.1.9 As a minimum, it is recommended that the upper 12 inches of existing site soils (fill and Older Paralic deposits) be scarified and property compacted for concrete slab-on-grade support. Recommendations for earthwork are provided in the *Grading* section of this report (see Section 7.4).

7.1.10 Performing open excavations adjacent to or deeper than the existing foundation system could potentially remove lateral support and/or undermine the existing foundations. Excavation for construction of new foundations immediately adjacent to existing foundations may require special excavation measures, such as trench shoring, in order to maintain lateral support of the existing adjacent foundation. This condition may occur where the proposed miscellaneous structures are to be constructed near existing structure foundations within the existing residential property lines. Excavation recommendations are provided in the *Temporary Excavations* section of this report (Section 7.13).

7.1.11 Due to the presence of shallow groundwater, a stormwater infiltration system is not considered feasible for this project. However, the suitability of storm water infiltration system should be determined by the project civil engineer in accordance with the requirements of the local governing agency. While it is our geotechnical opinion that water will percolate through the sandy site soils, it is our understanding that collected storm water is supposed to percolate through 10 feet of soil before reaching the water table since the soil is used as the filter medium.

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

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

7.2 Soil and Excavation Characteristics

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

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

7.2.3 All onsite excavations must be conducted in such a manner that potential surcharges from existing structures, construction equipment, and vehicle loads are resisted. The surcharge area may be defined by a 1:1 projection down and away from the bottom of an existing foundation or vehicle load. Penetrations below this 1:1 projection will require special excavation measures such as sloping and possibly shoring. Excavation recommendations are provided in the *Temporary Excavations* section of this report (see Section 7.13).

7.2.4 The soils encountered during field investigation are primarily granular in nature and are considered to be "non-expansive". The recommendations presented in this report assume that foundations and slabs will derive support in these materials.

7.3 Minimum Resistivity, pH and Water-Soluble Sulfate

7.3.1 Potential of Hydrogen (pH) and resistivity testing as well as chloride content testing were performed on representative samples of soil to generally evaluate the corrosion potential to surface utilities. The tests were performed in accordance with California Test Method Nos. 643 and 422 and indicate that the soils are considered "severely corrosive" with respect to corrosion of buried ferrous metals on site. The results are presented in Appendix B (Figure B4) and should be considered for design of underground structures.

7.3.2 Laboratory tests were performed on representative samples of the site materials to measure the percentage of water-soluble sulfate content. Results from the laboratory water-soluble sulfate tests are presented in Appendix B (Figure B4) and indicate that the on-site materials possess "negligible" sulfate exposure to concrete structures as defined by 2013 CBC Section 1904 and ACI 318-11, Sections 4.2 and 4.3. However, it is recommended to utilize Type V cement for concrete piles and concrete improvements that will be in contact with seawater.

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

7.4 Grading

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

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

7.4.3 Grading should commence with the removal of all existing vegetation and existing improvements from the area to be graded. Deleterious debris such as wood and root structure 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 improvement planned for removal should be completely excavated and the resulting depressions properly backfilled in accordance with the procedures described herein. Once a clean excavation bottom has been established it must be observed and approved in writing by the Geotechnical Engineer (a representative of Geocon West, Inc.).

7.4.4 Proposed improvements such as ADA-compliant walkways, ramps, stairs, and site walls, may be supported on a conventional foundation system bearing on a minimum of 12 inches of newly placed engineered fill. Where engineered fill is to be utilized for foundation support, the excavation should extend laterally a minimum distance of 12 inches beyond the structure footprint area.

7.4.5 As an alternative and where excavation and compaction cannot be performed, such as adjacent to property lines, proposed foundations may be supported on a conventional foundation system deriving support in the competent Older Paralic deposits found at or below a depth of 2 feet below the existing grade. Where competent Older Paralic deposits are to be utilized for foundation support, foundations should be deepened as necessary to penetrate through any encountered artificial fill or unsuitable soils to derive support in the competent undisturbed Older Paralic deposits.

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 by a Geocon representative.

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

7.4.7 Prior to construction of slabs, the upper 12 inches of subgrade should be moisture conditioned to near optimum moisture content and properly compacted to at least 95 percent relative compaction, as determined by ASTM Test Method D 1557 (latest edition).

7.4.8 It is anticipated that stable excavations for the recommended grading can be achieved with sloping measures. However, if excavations in close proximity to an adjacent property line and/or structure are required, special excavation measures may be necessary in order to maintain lateral support of the existing offsite improvements. Excavation recommendations are provided in the *Temporary Excavations* section of this report (Section 7.13).

7.4.9 Utility trenches should be properly backfilled in accordance with the requirements of the Green Book (latest edition). The pipe should be bedded with clean sands (Sand Equivalent greater than 30) to a depth of at least 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 trench excavation bottom must be observed and approved in writing by the Geotechnical Engineer (a representative of Geocon).

7.4.10 Although not anticipated for this project, all imported fill shall be observed, tested, and approved by Geocon West, Inc. prior to bringing soil to the site. Rocks larger than 6 inches in diameter shall not be used in the fill. If necessary, import soils used as structural fill should have an expansion index less than 20 and corrosivity properties that are equally or less detrimental to that of the existing onsite soils (see Figure B4). Import soils placed and compacted should be placed uniformly across the improvement area or in a manner that is approved by the Geotechnical Engineer (a representative of Geocon).

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

7.5 Conventional Foundation Design

7.5.1 Proposed miscellaneous structures, as such ADA-compliant walkways, ramps, stairs, and site walls may be supported on conventional shallow foundations deriving support in newly placement engineered fill and/or the Older Paralic deposits found at or below a depth of 2 feet.

7.5.2 It is the intent of the Geotechnical Engineer to allow building foundations to derive support in both Older Paralic deposits and engineered fill if project conditions warrant such an occurrence.

7.5.3 Continuous footings may be designed for an allowable bearing capacity of 1,800 pounds per square foot (psf), and should be a minimum of 12 inches in width and 18 inches in depth below the lowest adjacent grade, and 12 inches into the recommended bearing material.

7.5.4 Isolated spread foundations may be designed for an allowable bearing capacity of 2,000 psf, and should be a minimum of 12 inches in width, 18 inches in depth below the lowest adjacent grade, and 12 inches into the recommended bearing material.

7.5.5 The soil bearing pressure above may be increased by 200 psf and 400 psf for each additional foot of foundation width and depth, respectively, up to a maximum allowable soil bearing pressure of 2,500 psf.

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

7.5.7 Continuous footings should be reinforced with four No. 4 steel reinforcing bars, two placed near the top of the footing and two near the bottom. Reinforcement for spread footings should be designed by the project structural engineer.

7.5.8 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.

7.5.9 The moisture content in the engineered fill should be maintained prior to placement of concrete and the slab and foundation subgrade should be sprinkled as necessary to maintain a moist condition and prevent open excavation from caving in.

7.5.10 Where new foundations are to be constructed immediately adjacent to existing foundations, in order to prevent surcharging the existing foundation, the proposed foundation should be deepened as necessary to match or exceed the depth of the existing foundation. This condition may occur where the proposed miscellaneous structures are to be constructed near existing structures within the existing residential property lines.

7.5.11 Where proposed foundations will be deeper than an existing foundation, the new foundation must be designed to resist the surcharge imposed by the existing foundation. The surcharge area may be defined by a 1:1 projection down and away from the bottom of the existing foundation.

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

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

7.6 Lateral Design

7.6.1 Resistance to lateral loading may be provided by friction acting at the base of foundations, slabs and by passive earth pressure. An allowable coefficient of friction of 0.33 may be used with the dead load forces in the undisturbed Older Paralic deposits, and an allowable coefficient of friction of 0.40 may be used in properly compacted engineered fill.

7.6.2 Passive earth pressure for the sides of foundations poured against newly engineered fill and or undisturbed alluvium above the groundwater table may be computed as an equivalent fluid having a density of 210 pounds per cubic foot (pcf) with a maximum earth pressure of 2,100 psf. Passive earth pressure for the sides of foundations poured against newly engineered fill and/or undisturbed Older Paralic deposits below the groundwater table may be computed as an equivalent fluid having a density of 100 pcf with a maximum earth pressure of 1,000 psf (these values have been adjusted for buoyant forces). When combining passive and friction for lateral resistance, the passive component should be reduced by one-third. A one-third increase in the passive value may be used for wind or seismic loads.

7.7 Deepened Foundations

7.7.1 Deepened foundations consisting of drilled cast-in-place concrete friction piles deriving support in undisturbed Older Paralic deposits found at or below a depth of 2 feet below existing ground surface at the base of the sea wall may be utilized for support of new retaining walls (sea walls).

7.7.2 Drilled cast-in-place friction piles should be a minimum of 18 inches in diameter, and should be embedded a minimum of 8 feet in depth below the ground surface at the base of the sea wall and 6 feet into undisturbed Older Paralic deposits.

7.7.3 Friction piles may be designed based on a skin friction capacity of 180 psf (this value has been adjusted for buoyant forces). The allowable capacity may be doubled for isolated piles spaced more than twice the diameter. Caissons may be assumed fixed at an embedment depth of 7 feet below the ground surface

7.7.4 For design purposes, an allowable passive value for the soils may be assumed to be 130 psf per foot with a maximum allowable passive earth pressure is 1,300 pcf (these values have been adjusted for buoyant forces). The allowable passive value may be doubled for isolated piles placed more than twice the diameter. To develop the full lateral value, provisions should be implemented to assure firm contact between the piles and the undisturbed soils.

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

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

7.8 Deepened Foundation Installation

7.8.1 Casing will likely be required since caving is expected in the granular soils during excavation. The contractor should have casing available and should be prepared to use it. 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.

7.8.2 Groundwater was encountered and is anticipated during construction; piles placed below the water level require the use of a tremie to place the concrete into the bottom of the hole. A tremie shall consist of a water-tight tube, with a hopper at the top. The tube shall be equipped with a device that will close the discharge end and prevent water from entering the tube while it is being charged with concrete. The tremie shall be supported so as to permit free movement of the discharge end over the entire top surface of the work and to permit rapid lowering when necessary to retard or stop the flow of concrete. The discharge end shall be closed at the start of the work to prevent water entering the tube and shall be entirely sealed at all times, except when the concrete is being placed. The tremie tube shall be kept full of concrete. The flow shall be continuous until the work is completed and the resulting concrete seal shall be monolithic and homogeneous. The tip of the tremie tube shall always be kept about five 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.

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

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

7.9 Foundation Settlement

7.9.1 The maximum expected static settlement for a structure supported on a conventional foundation system deriving support in newly placed engineered fill or competent Older Paralic deposits is estimated to be less than ½ inch and occur below the heaviest loaded structural element. Settlement of the foundation system is expected to occur on initial application of loading. Differential settlement is not expected to exceed ¼ inch over a distance of 20 feet.

7.9.2 The maximum expected static settlement for a deepened foundation supported structure deriving support in undisturbed Older Paralic deposits is estimated to be less than ½ inch. Differential settlement between adjacent piles is not expected to exceed ¼ inch.

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

7.10 Exterior Concrete Slabs-on-Grade

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

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

7.11 Retaining Walls Design

7.11.1 The recommendations presented below are generally applicable to the design of rigid concrete or masonry retaining walls having a maximum height of 6 feet. In the event that walls significantly higher than 6 feet are planned, Geocon should be contacted for additional recommendations.

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

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

7.11.4 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.

7.11.5 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.

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

7.12 Retaining Wall Drainage

7.12.1 Retaining walls should be provided with 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 5). 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. 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.

7.12.2 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, 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 retaining walls and foundations.

7.13 Temporary Excavations

7.13.1 Excavations on the order of 6 feet in vertical height are anticipated for the proposed construction. The excavations are expected to expose artificial fill and Older Paralic deposits which are subject to excessive caving. Vertical excavations up to five feet in height may be attempted where loose soils or caving sands are not present, and where excavation are not surcharged by adjacent traffic or structures.

7.13.2 Vertical excavations greater than five feet or where surcharged by existing structures will require sloping or shoring measures in order to provide a stable excavation.

7.13.3 It is anticipated that sufficient space is available to complete the required earthwork for this project using sloping measures. Where sufficient space is available, temporary unsurcharged embankments may be sloped back at a uniform 1:1 slope gradient or flatter. A uniform slope does not have a vertical portion.

7.13.4 If excavations in close proximity to an adjacent property line and/or structure are required, special excavation measures such as slot-cutting or shoring may be necessary in order to maintain lateral support of offsite improvements. Recommendations for special excavation measures can be provided under separate cover, if necessary.

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

7.14 Surface Drainage

7.14.1 Proper surface drainage is critical to the future performance of the project. Infiltration of irrigation excess and storm runoff into the supporting soils can adversely affect the performance of the planned improvements. Saturation of a soil can cause it to lose internal shear strength and increase its compressibility, resulting in a change in the original designed engineering properties. Proper drainage should be maintained at all times.

7.14.2 All site drainage should be collected and transferred to the street in non-erosive drainage devices. Drainage should not be allowed to pond anywhere on the site, and especially not against any foundation or retaining wall. Drainage should not be allowed to flow uncontrolled over any descending slope. Positive site drainage should be provided away from structures, pavement, and the tops of slopes to swales or other controlled drainage structures.

7.15 Plan Review

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

LIMITATIONS AND UNIFORMITY OF CONDITIONS

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

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

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

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

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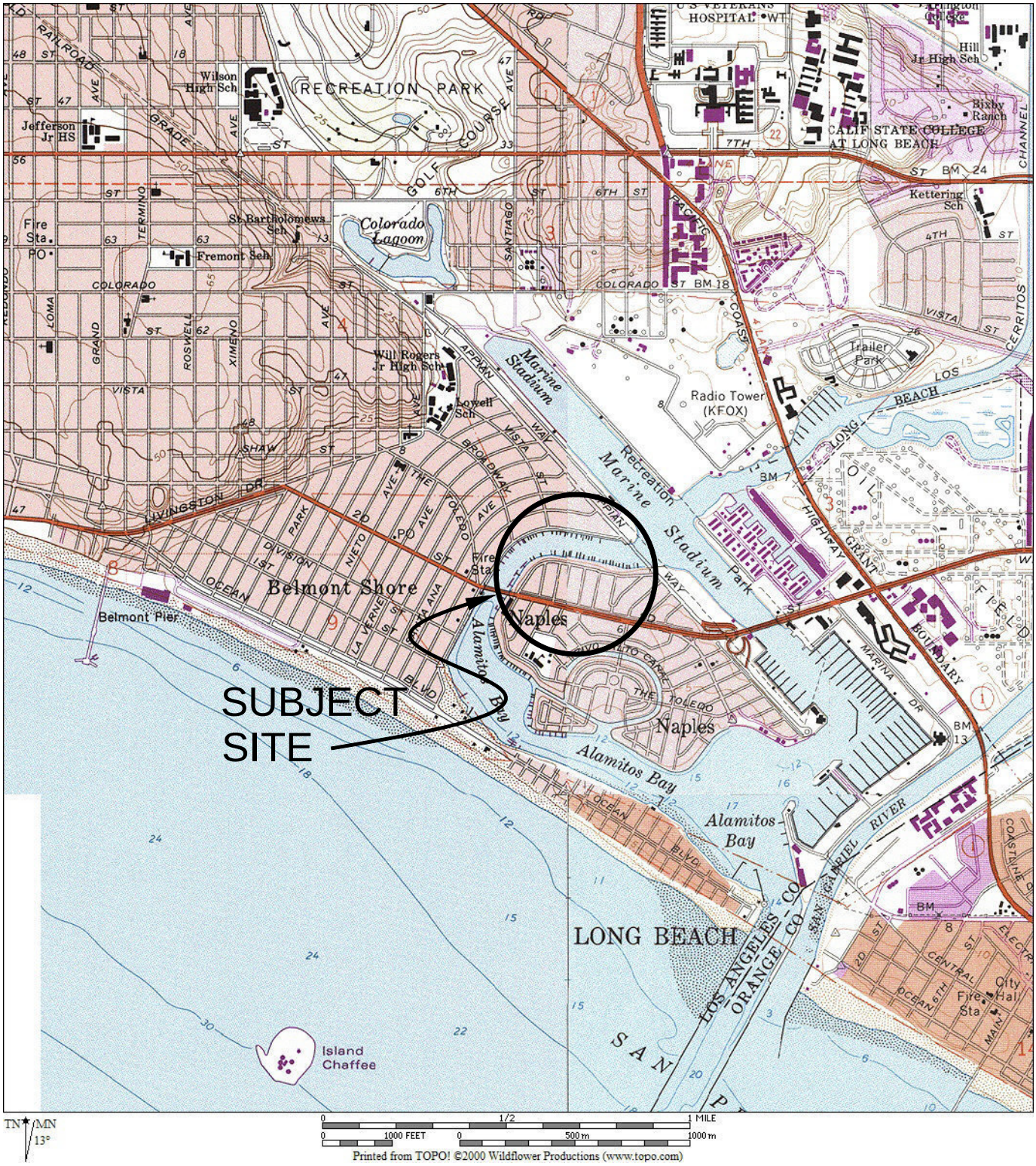

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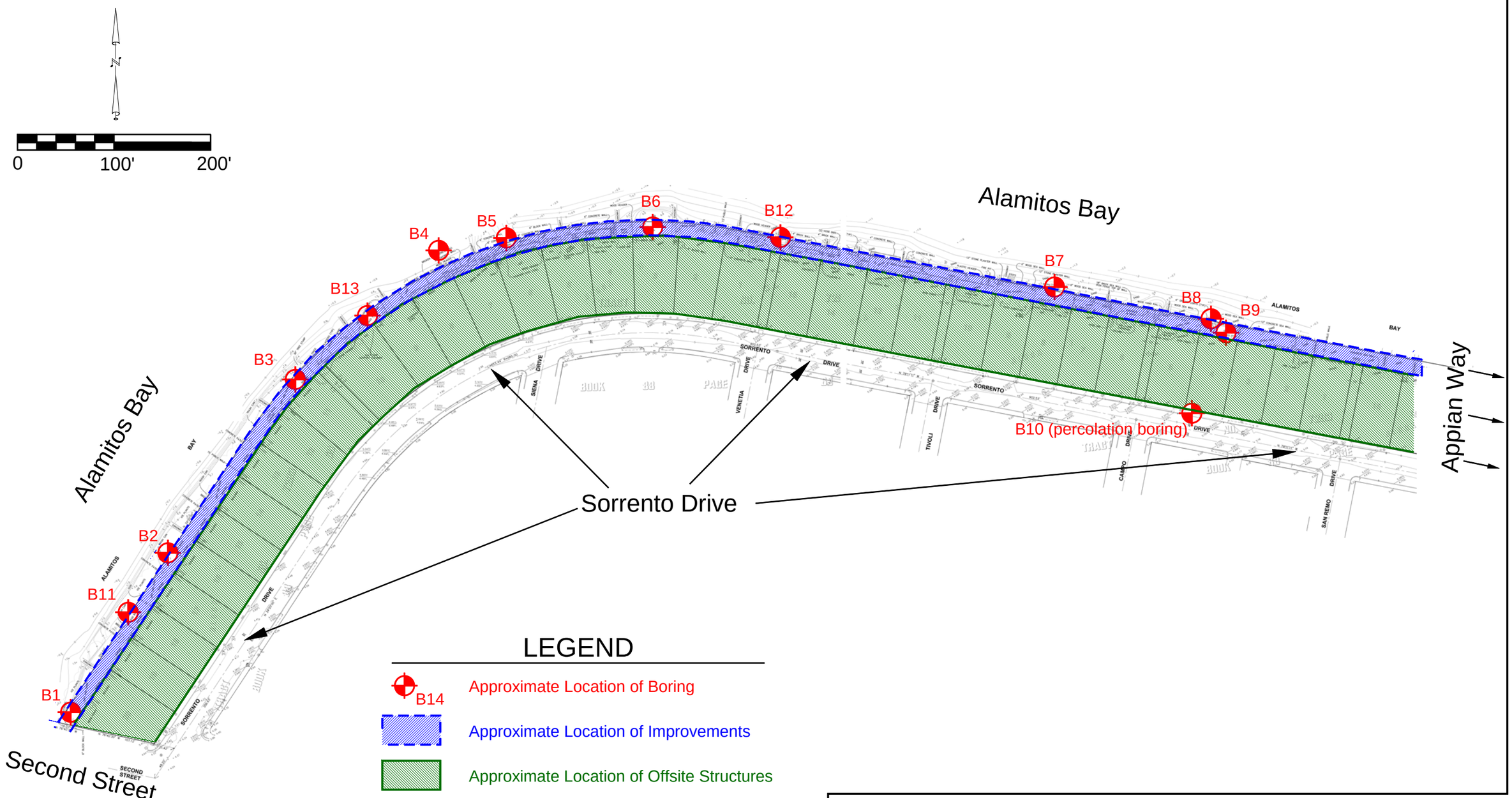
VICINITY MAP

SORRENTO ALAMITOS BAY SHORELINE TRAIL
MLA GREEN, INC
BETWEEN SECOND STREET AND APPIAN WAY
LONG BEACH, CALIFORNIA

JUNE 2015	PROJECT NO. A9218-06-01	FIG. 1



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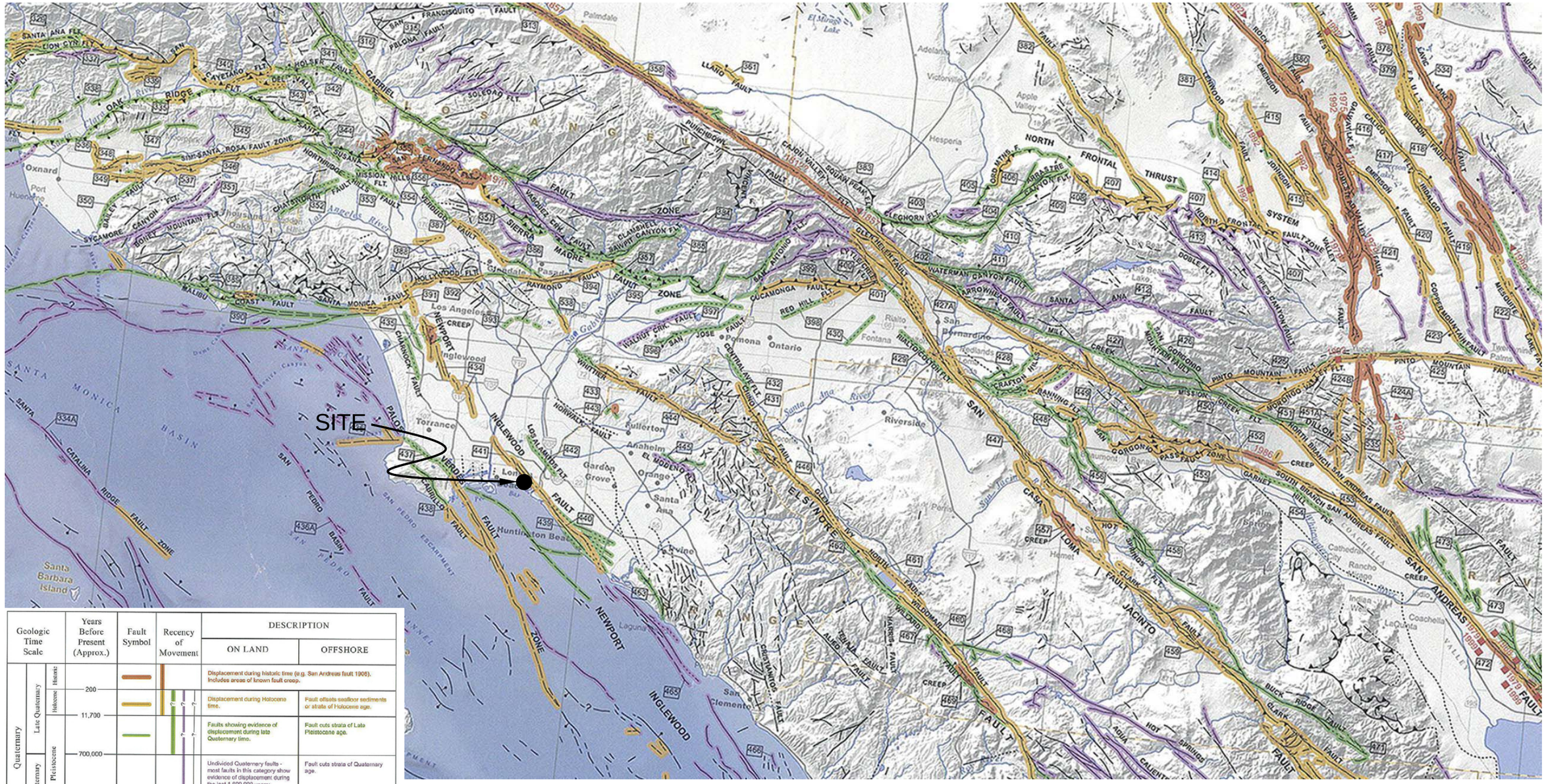
SITE PLAN

SORRENTO ALAMITOS BAY SHORELINE TRAIL
MLA GREEN, INC
BETWEEN SECOND STREET AND APPIAN WAY
LONG BEACH, CALIFORNIA

JUNE 2015 | PROJECT NO. A9218-06-01 | FIG. 2

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



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

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

0 12 24 Miles

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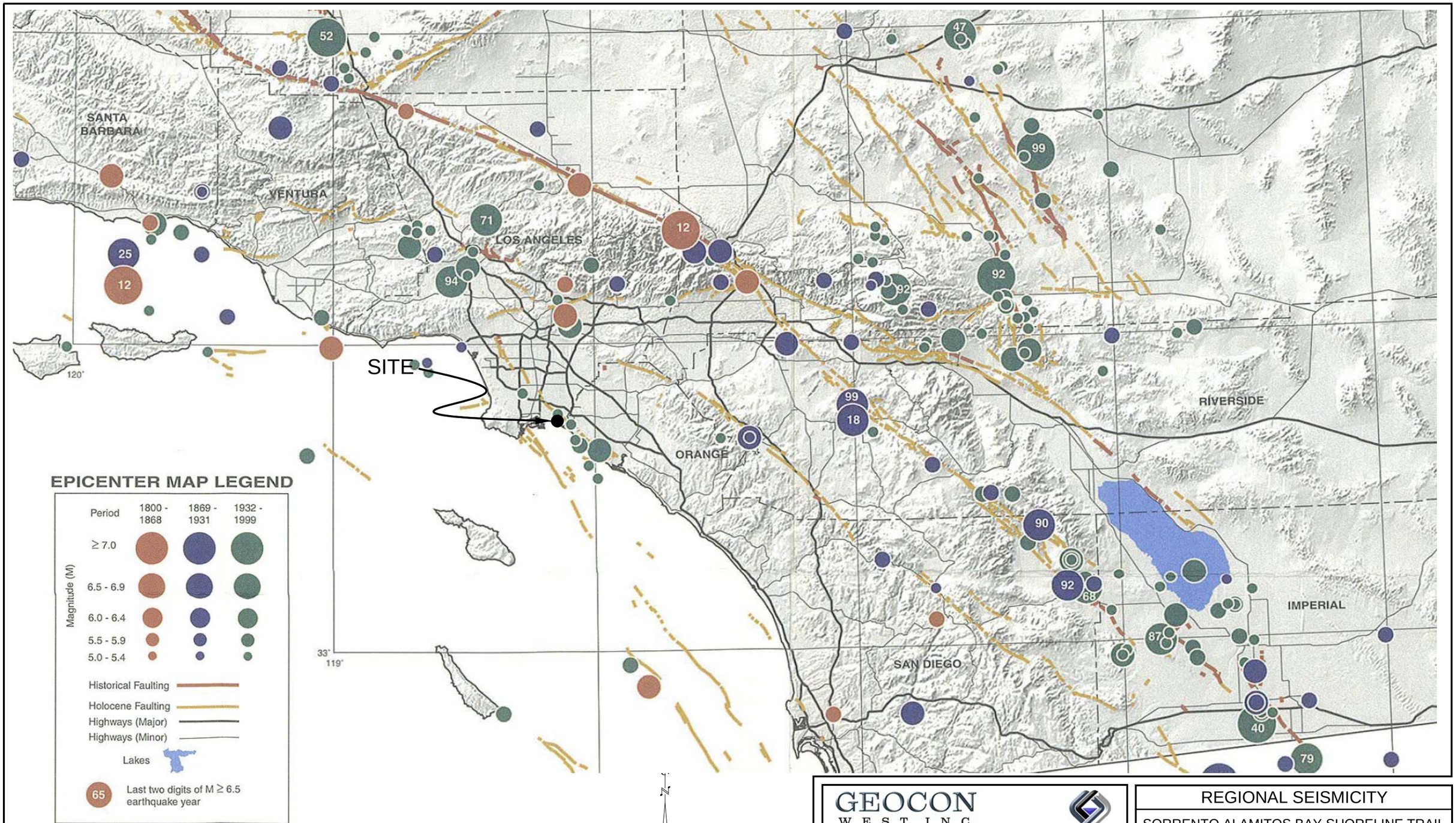
Drafted by: SB | Checked by: GAK/SFK

REGIONAL FAULT MAP

SORRENTO ALAMITOS BAY SHORELINE TRAIL
MLA GREEN, INC
BETWEEN SECOND STREET AND APPIAN WAY
LONG BEACH, CALIFORNIA

JUNE 2015 | PROJECT NO. A9218-06-01 | FIG. 3



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

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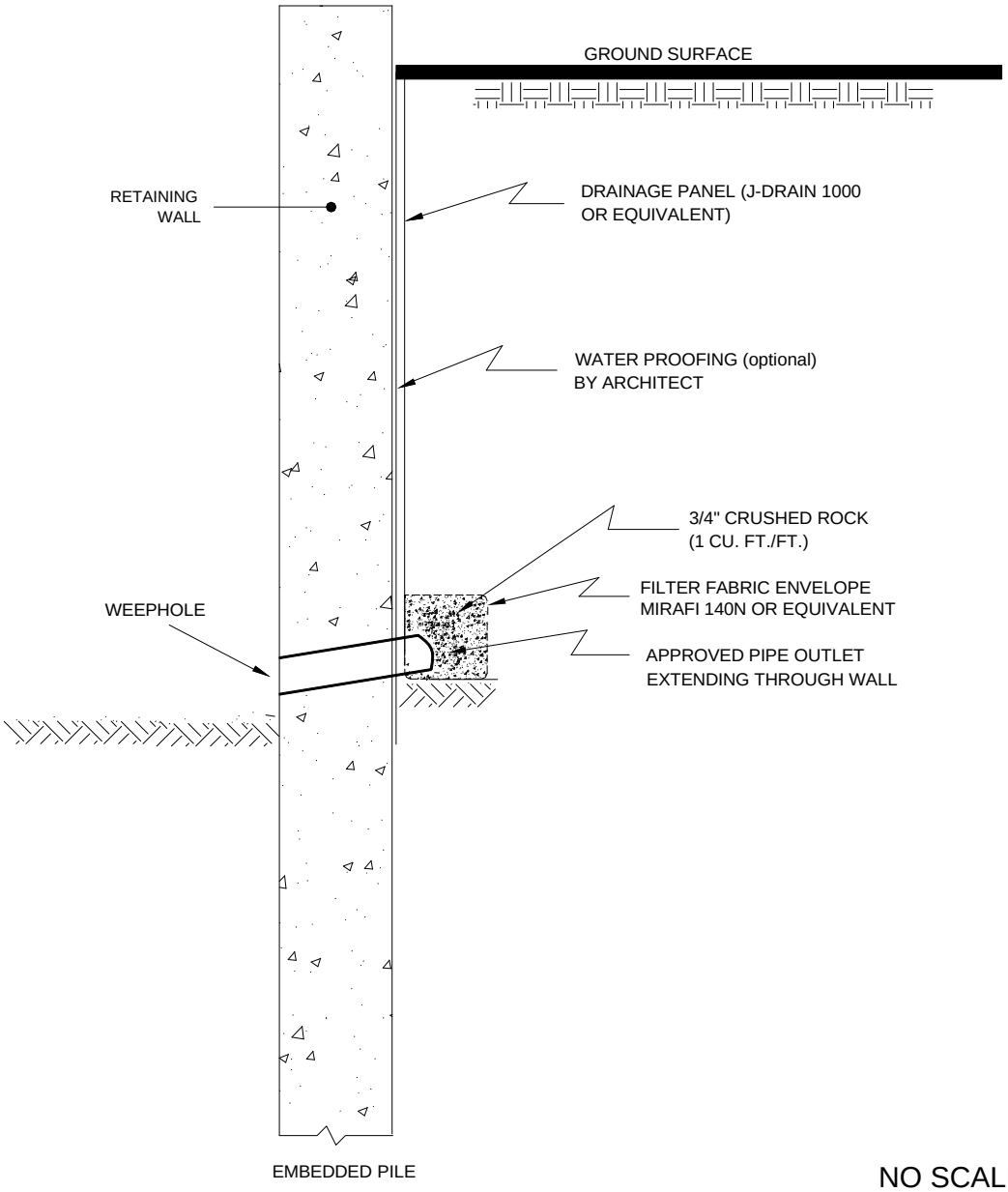
SORRENTO ALAMITOS BAY SHORELINE TRAIL
MLA GREEN, INC
BETWEEN SECOND STREET AND APPIAN WAY
LONG BEACH, CALIFORNIA

JUNE 2015 | PROJECT NO. A9218-06-01 | FIG. 4



NO SCALE

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RETAINING WALL DRAIN DETAIL

SORRENTO ALAMITOS BAY SHORELINE TRAIL
MLA GREEN, INC
BETWEEN SECOND STREET AND APPIAN WAY
LONG BEACH, CALIFORNIA

Drafted By: RDG	Checked By: HHD	JUNE 2015	PROJECT NO. A9218-06-01	FIG. 5

APPENDIX A

APPENDIX A

FIELD INVESTIGATION

The site was explored on April 17 and May 4, 2015 by excavating thirteen 4-inch-diameter borings utilizing by manual augers and digging equipment. The borings were excavated to depths of approximately 2½ to 8½ feet below the existing ground surface. Representative and relatively undisturbed samples were obtained by driving a 3 inch, O. D., California Modified Sampler into the "undisturbed" soil mass with blows from a slide hammer. The California Modified Sampler was equipped with 1-inch high by 2³/₈-inch diameter brass sampler rings to facilitate soil removal and testing. Bulk samples were also obtained.

The soil conditions encountered in the borings were visually examined, classified and logged in general accordance with the Unified Soil Classification System (USCS). Logs of the borings are presented on Figures A1 through A13. The logs depict the soil and geologic conditions encountered and the depth at which samples were obtained.

PROJECT NO. A9218-06-01

BORING 1

ELEV. (MSL.) _____ DATE COMPLETED **5/4/15**

EQUIPMENT **HAND AUGER** BY: **RDG**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT)*	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0			▼	SP	**OLDER PARALIC DEPOSITS** Sand, poorly graded, medium dense, wet, grayish brown, fine-grained, trace silt.			
2	B1@2.5'			SM	Silty Sand, medium dense, saturated, gray, fine-grained.		107.0	20.7
4								
6	B1@6.5'			SP	Sand, poorly graded, medium dense, saturated, gray, fine-grained.		106.0	21.3
					Total depth of boring: 7 feet. No fill encountered. Groundwater encountered at 1 foot. Backfilled with soil cuttings and tamped. Surface restored.			

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

A9218-06-01 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. A9218-06-01

BORING 2

ELEV. (MSL.) _____ DATE COMPLETED **5/4/15**

EQUIPMENT **HAND AUGER** BY: **RDG**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT)*	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0	BULK 0-2'			SP	**OLDER PARALIC DEPOSITS** Sand, poorly graded, slightly moist, light brown, fine-grained, trace rootlets.			
2	B2@2'				- light grayish brown		94.8	8.7
					Total depth of boring: 3 feet No fill encountered. No groundwater encountered. Backfilled with soil cuttings and tamped. Surface restored.			

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

A9218-06-01 BORING LOGS.GPJ

SAMPLE SYMBOLS

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

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

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PROJECT NO. A9218-06-01

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	**BORING 3** ELEV. (MSL.) _____ DATE COMPLETED **5/4/15** EQUIPMENT **HAND AUGER** BY: **RDG**	PENETRATION RESISTANCE (BLOWS/FT)*	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
					MATERIAL DESCRIPTION			
0	BULK 0-3.5'			SP	**OLDER PARALIC DEPOSITS** Sand, poorly graded, medium dense, slightly moist, light brown, fine- to medium-grained.			
2	B3@1.5'			SM	Silty Sand, medium dense, moist, mottled reddish gray and brown, fine-grained.		105.6	20.2
				SP	Sand, poorly graded, medium dense, wet, grayish brown, fine-grained.			
					Total depth of boring: 3.5 feet No fill encountered. No groundwater encountered. Backfilled with soil cuttings and tamped. Surface restored.			

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

A9218-06-01 BORING LOGS.GPJ

SAMPLE SYMBOLS

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

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

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PROJECT NO. A9218-06-01

BORING 4

ELEV. (MSL.) _____ DATE COMPLETED **5/4/15**

EQUIPMENT **HAND AUGER** BY: **RDG**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT)*	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0					**OLDER PARALIC DEPOSITS** Sand, poorly graded, medium dense, wet, light brown, fine-grained. - saturated, mottled reddish gray and brown			
2	B4@2'			SP			105.2	20.4
4	B4@4.5'				- grayish brown		104.6	20.7
					Total depth of boring: refusal at 5 feet No fill encountered. Groundwater encountered at 1.5 feet Backfilled with soil cuttings and tamped.			

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

A9218-06-01 BORING LOGS.GPJ

SAMPLE SYMBOLS

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

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

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PROJECT NO. A9218-06-01

BORING 5

ELEV. (MSL.) _____ DATE COMPLETED **4/17/15**

EQUIPMENT **HAND AUGER** BY: **RDG**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT)*	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0					**OLDER PARALIC DEPOSITS**			
2			▼	SP	Sand, poorly graded, medium dense, wet, light brown, fine- to medium-grained.			
4	B5@3.5'			SM	Silty Sand, medium dense, saturated, gray, fine-grained, trace clay.		75.4	45.4
6				SP	Sand, poorly graded, medium dense, saturated, gray, fine-grained.			
8	B5@7.5'						92.9	26.5

Total depth of boring: 8 feet
No fill encountered.
Groundwater encountered at 1 foot.
Backfilled with soil cuttings and tamped.

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

A9218-06-01 BORING LOGS.GPJ

SAMPLE SYMBOLS

☐ ... SAMPLING UNSUCCESSFUL ◧ ... STANDARD PENETRATION TEST ■ ... DRIVE SAMPLE (UNDISTURBED)

⊠ ... DISTURBED OR BAG SAMPLE ◪ ... CHUNK SAMPLE ▼ ... WATER TABLE OR SEEPAGE

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

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PROJECT NO. A9218-06-01

BORING 6

ELEV. (MSL.) _____ DATE COMPLETED **4/17/15**

EQUIPMENT **HAND AUGER** BY: **RDG**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT)*	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0	BULK 0-2'				**GRASS**			
					ARTIFICIAL FILL			
2				SP	Silty Sand, medium dense, slightly moist, dark brown, fine- to medium-grained.		106.6	18.2
					OLDER PARALIC DEPOSITS			
					Sand, poorly graded, medium dense, slightly moist, light brown, fine- to medium-grained.			
					Total depth of boring: 2.5 feet Fill to 1 foot. No groundwater encountered. Backfilled with soil cuttings and tamped.			

Figure A6,
Log of Boring 6, Page 1 of 1

A9218-06-01 BORING LOGS.GPJ

SAMPLE SYMBOLS

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

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

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PROJECT NO. A9218-06-01

BORING 7

ELEV. (MSL.) _____ DATE COMPLETED **4/17/15**

EQUIPMENT **HAND AUGER** BY: **RDG**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT)*	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0					**OLDER PARALIC DEPOSITS** Sand, poorly graded, medium dense, wet, light brown, fine- to medium-grained.			
2			▼	SP	- fine-grained			
4								
				SC	Clayey sand, poorly graded, medium dense, saturated, gray, fine-grained.			
6								
				SP	Sand, poorly graded, medium dense, saturated, gray, fine-grained, trace silt.			
8	B7@8'	■					93.9	26.6

Total depth of boring: 8.5 feet
No fill encountered.
Groundwater encountered at 2 feet.
Backfilled with soil cuttings and tamped.

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

A9218-06-01 BORING LOGS.GPJ

SAMPLE SYMBOLS

☐ ... SAMPLING UNSUCCESSFUL ◧ ... STANDARD PENETRATION TEST ■ ... DRIVE SAMPLE (UNDISTURBED)

⊠ ... DISTURBED OR BAG SAMPLE ◪ ... CHUNK SAMPLE ▼ ... WATER TABLE OR SEEPAGE

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

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PROJECT NO. A9218-06-01

BORING 8

ELEV. (MSL.) _____ DATE COMPLETED **4/17/15**

EQUIPMENT **HAND AUGER** BY: **RDG**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT)*	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0	BULK 0-5'		▼		**OLDER PARALIC DEPOSITS** Sand, poorly graded, medium dense, saturated, light brown, fine-grained, trace seashells.			
2								
4	B8@3.5'			SP	- gray, trace clay - no clay, trace silt		97.0	30.6
6	B8@6.5'						101.1	21.5

Total depth of boring: refusal at 7 feet
No fill encountered.
Groundwater encountered at 1 foot.
Backfilled with soil cuttings and tamped.

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

A9218-06-01 BORING LOGS.GPJ

SAMPLE SYMBOLS

☐ ... SAMPLING UNSUCCESSFUL ◧ ... STANDARD PENETRATION TEST ■ ... DRIVE SAMPLE (UNDISTURBED)

⊠ ... DISTURBED OR BAG SAMPLE ◪ ... CHUNK SAMPLE ▼ ... WATER TABLE OR SEEPAGE

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

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PROJECT NO. A9218-06-01

BORING 9

ELEV. (MSL.) _____ DATE COMPLETED **4/17/15**

EQUIPMENT **HAND AUGER** BY: **RDG**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT)*	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0	BULK 0-2'				**ARTIFICIAL FILL** Silty Sand, medium dense, slightly moist, brown.			
					Clay, firm, moist, mottled brown and gray, trace fine-grained sand.			
2					Sand, poorly graded, medium dense, slightly moist, brown, fine- to medium-grained, trace silt.		127.4	22.2
4				SP	**OLDER PARALIC DEPOSITS** Sand, poorly graded, medium dense, moist, light brown to brown, fine-grained.			

Total depth of boring: 4.5 feet
Fill to 3.5 feet.
No groundwater encountered.
Backfilled with soil cuttings and tamped.

Figure A9,
Log of Boring 9, Page 1 of 1

A9218-06-01 BORING LOGS.GPJ

SAMPLE SYMBOLS

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

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

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PROJECT NO. A9218-06-01

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	**BORING 10** ELEV. (MSL.) _____ DATE COMPLETED **5/4/15** EQUIPMENT **HAND AUGER** BY: **RDG**	PENETRATION RESISTANCE (BLOWS/FT)*	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
					MATERIAL DESCRIPTION			
0					**GRASS**			
					ARTIFICIAL FILL Silty Sand, medium dense, slightly moist, dark brown, fine- to medium-grained.			
2			▼					
					OLDER PARALIC DEPOSITS Sand, poorly graded, medium dense, slightly moist, light brown, fine- to medium-grained.			
					Total depth of boring: 2.5 feet Fill to 1 foot. Groundwater encountered at 2 feet. Backfilled with soil cuttings and tamped.			

Figure A10,
Log of Boring 10, Page 1 of 1

A9218-06-01 BORING LOGS.GPJ

SAMPLE SYMBOLS

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

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

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PROJECT NO. A9218-06-01

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	**BORING 11** ELEV. (MSL.) _____ DATE COMPLETED **5/4/15** EQUIPMENT **HAND AUGER** BY: **RDG**	PENETRATION RESISTANCE (BLOWS/FT)*	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
					MATERIAL DESCRIPTION			
0	BULK 0-2'			SP	**OLDER PARALIC DEPOSITS** Sand, poorly graded, medium dense, moist, light brown, fine-grained.			
2							85.6	22.4
					Total depth of boring: 3 feet No fill encountered. No groundwater encountered. Backfilled with soil cuttings and tamped.			

Figure A11,
Log of Boring 11, Page 1 of 1

A9218-06-01 BORING LOGS.GPJ

SAMPLE SYMBOLS

☐ ... SAMPLING UNSUCCESSFUL ◧ ... STANDARD PENETRATION TEST ■ ... DRIVE SAMPLE (UNDISTURBED)

⊠ ... DISTURBED OR BAG SAMPLE ◪ ... CHUNK SAMPLE ▼ ... WATER TABLE OR SEEPAGE

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

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PROJECT NO. A9218-06-01

BORING 12

ELEV. (MSL.) _____ DATE COMPLETED **5/4/15**

EQUIPMENT **HAND AUGER** BY: **RDG**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT)*	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0					**OLDER PARALIC DEPOSITS** Sand, poorly graded, medium dense, wet, light brown.			
2			▼	SP	- mottled reddish gray and brown			
4	B12@4'				- grayish brown		106.4	21.1

Total depth of boring: 4.5 feet
No fill encountered.
Groundwater encountered at 1.5 feet.
Backfilled with soil cuttings and tamped.

Figure A12,
Log of Boring 12, Page 1 of 1

A9218-06-01 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. A9218-06-01

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	**BORING 13** ELEV. (MSL.) _____ DATE COMPLETED **5/4/15** EQUIPMENT **HAND AUGER** BY: **RDG**	PENETRATION RESISTANCE (BLOWS/FT)*	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
					MATERIAL DESCRIPTION			
0					**OLDER PARALIC DEPOSITS** Sand, poorly graded, medium dense, slightly moist, light brown, fine-grained, trace rootlets.			
2	B13@2'			SP	- light grayish brown, no rootlets		99.6	8.5
					Total depth of boring: 3 feet No fill encountered. No groundwater encountered. Backfilled with soil cuttings and tamped.			

Figure A13,
Log of Boring 13, Page 1 of 1

A9218-06-01 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.

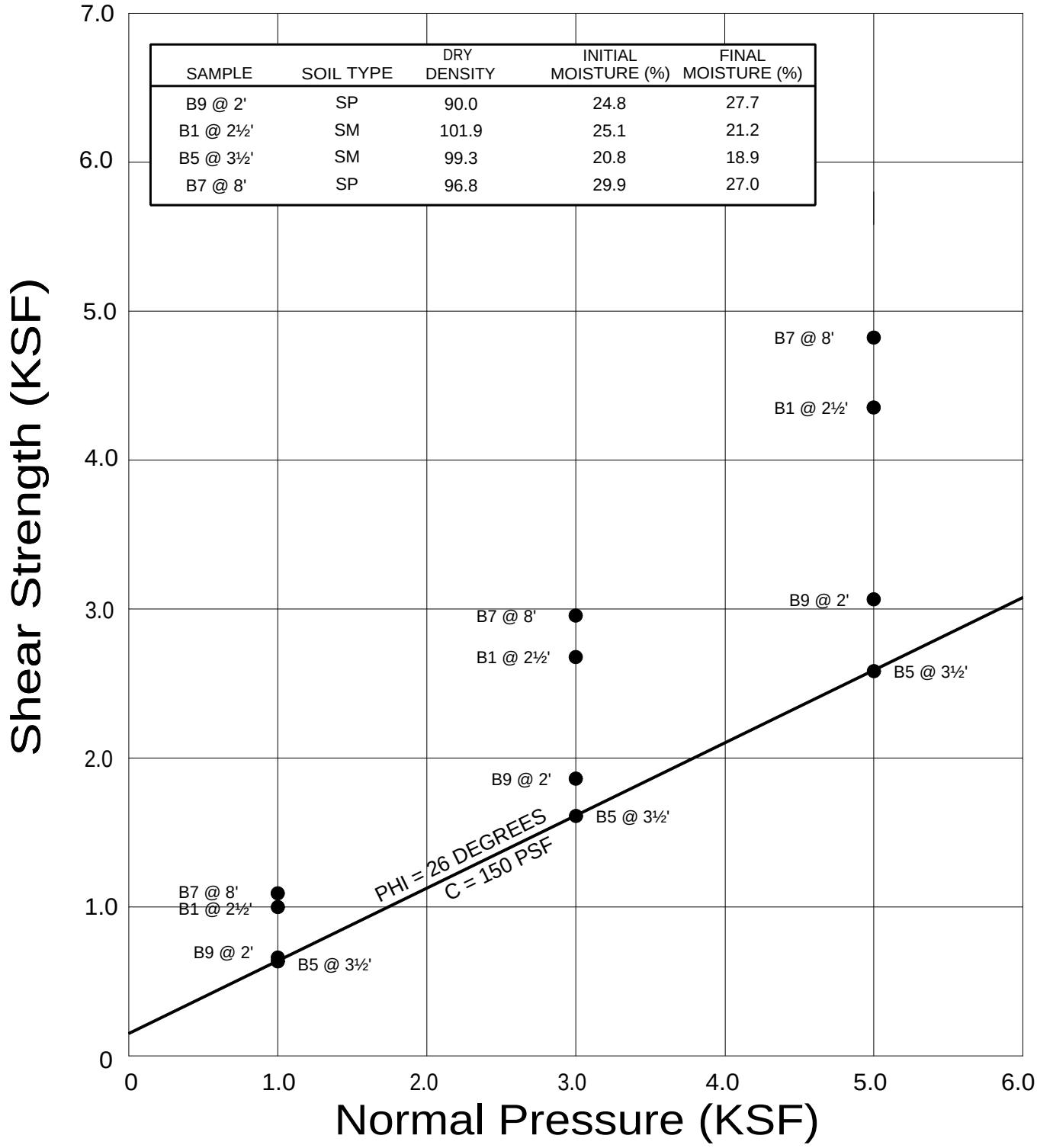
GEOCON

APPENDIX B

APPENDIX B

LABORATORY TESTING

Laboratory tests were performed in accordance with generally accepted test methods of the "American Society for Testing and Materials (ASTM)", or other suggested procedures. Selected samples were tested for direct shear strength, consolidation, corrosivity, moisture density relationships, in-place dry density, and moisture content. The results of the laboratory tests are summarized in Figures B1 through B5. 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 (%)
B9 @ 2'	SP	90.0	24.8	27.7
B1 @ 2½'	SM	101.9	25.1	21.2
B5 @ 3½'	SM	99.3	20.8	18.9
B7 @ 8'	SP	96.8	29.9	27.0

● Direct Shear, Saturated

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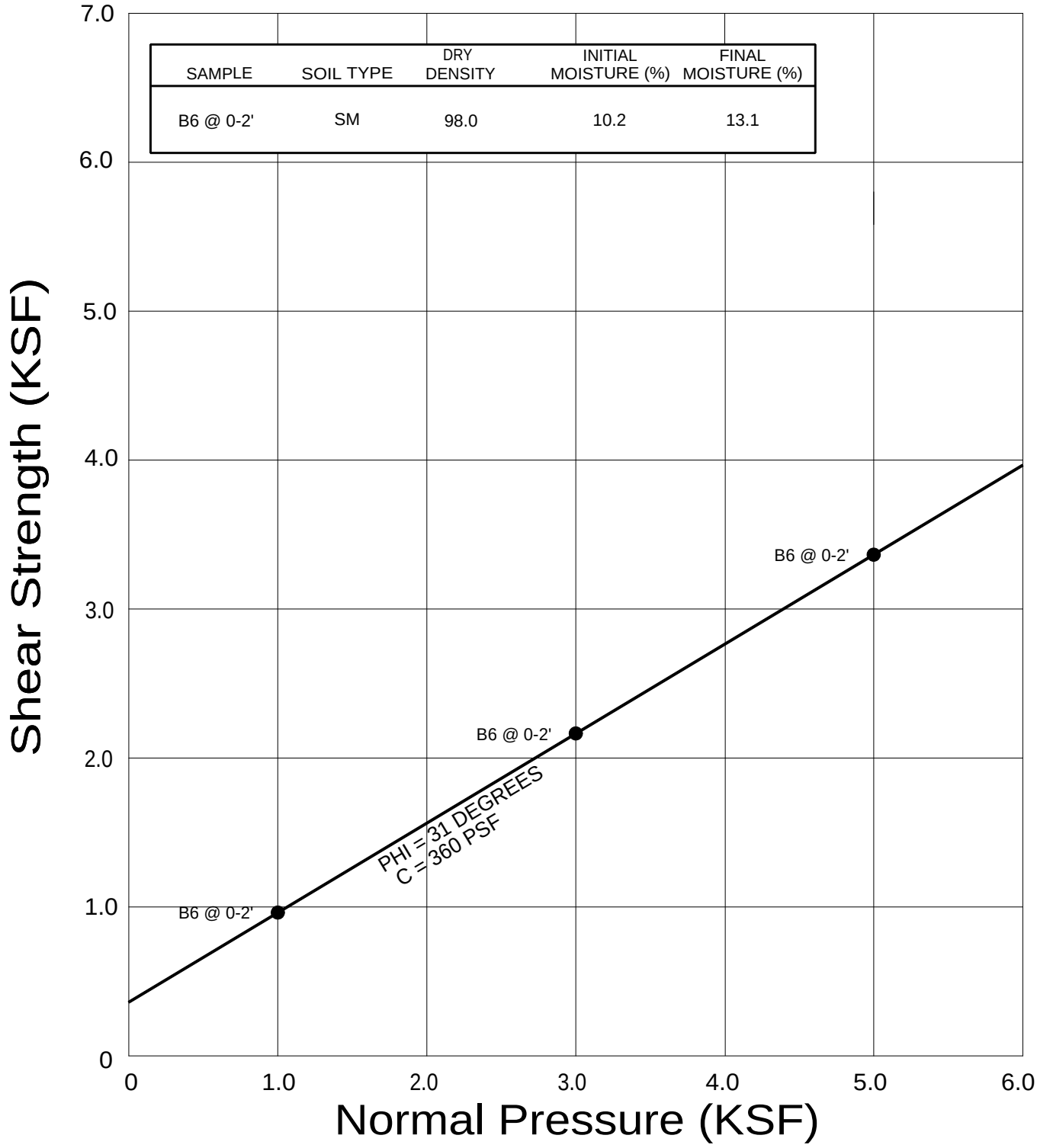
DIRECT SHEAR TEST RESULTS

SORRENTO ALAMITOS BAY SHORELINE TRAIL
MLA GREEN, INC
BETWEEN SECOND STREET AND APPIAN WAY
LONG BEACH, CALIFORNIA

Drafted by: RDG.	Checked by: HHD	JUNE 2015	PROJECT NO. A9218-06-01	FIG. B1



SAMPLE	SOIL TYPE	DRY DENSITY	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B6 @ 0-2'	SM	98.0	10.2	13.1

● Direct Shear, Saturated

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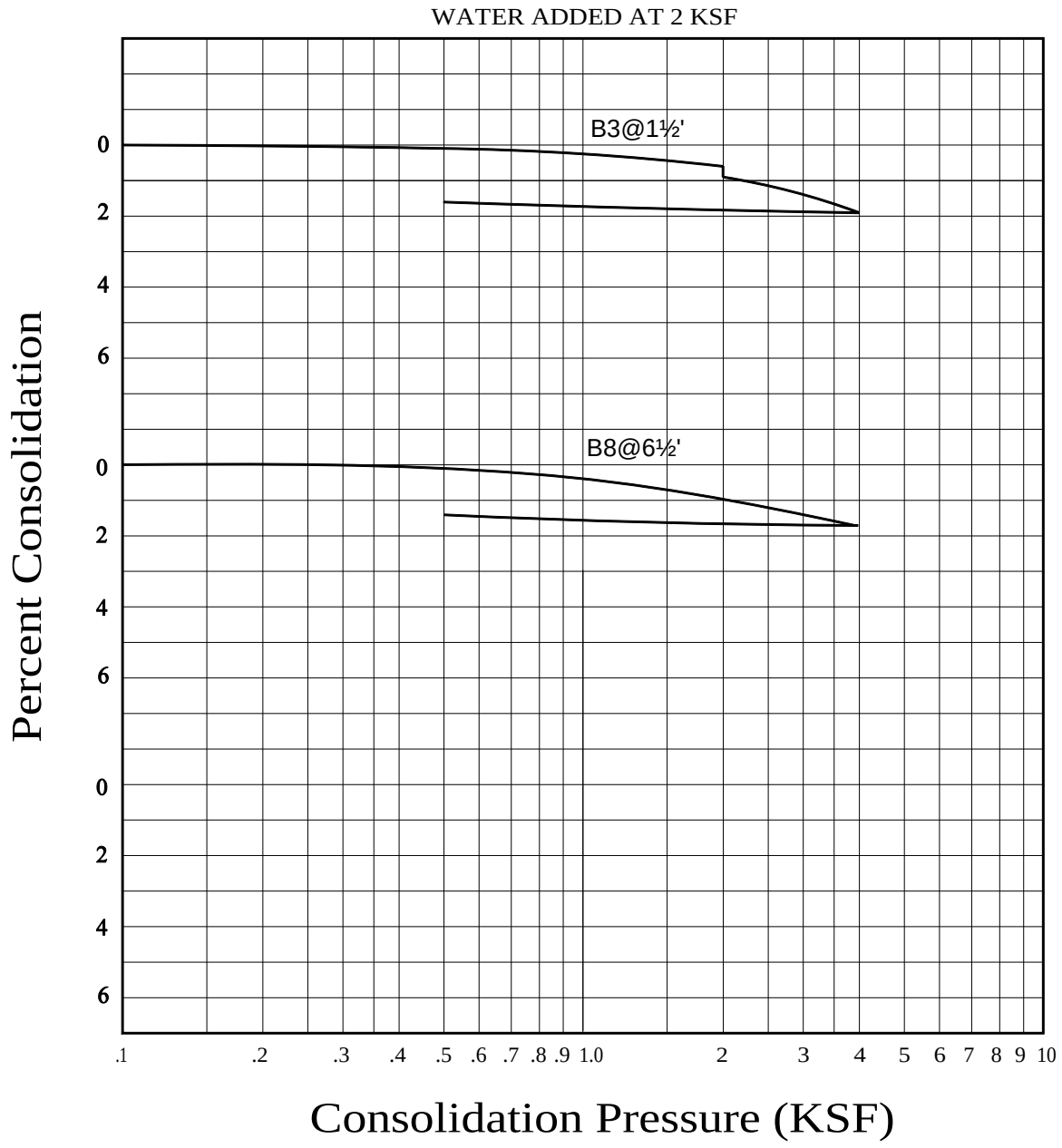
DIRECT SHEAR TEST RESULTS

SORRENTO ALAMITOS BAY SHORELINE TRAIL
MLA GREEN, INC
BETWEEN SECOND STREET AND APPIAN WAY
LONG BEACH, CALIFORNIA

Drafted by: RDG.	Checked by: HHD	JUNE 2015	PROJECT NO. A9218-06-01	FIG. B2



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

CONSOLIDATION TEST RESULTS

SORRENTO ALAMITOS BAY SHORELINE TRAIL
MLA GREEN, INC
BETWEEN SECOND STREET AND APPIAN WAY
LONG BEACH, CALIFORNIA

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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 (%)
B6 @ 0 -2	Brown Silty Sand	109.0	11.0

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LABORATORY TEST RESULTS

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

Sample No.	pH	Resistivity (Ohm Centimeters)
B3 @ 1½'	7.9	210 (Severely Corrosive)
B6 @ 0-2'	8.5	230 (Severely Corrosive)

SUMMARY OF LABORATORY CHLORIDE CONTENT TEST RESULTS EPA NO. 325.3

Sample No.	Chloride Ion Content (%)
B3 @ 1½'	0.405
B6 @ 0-2'	0.280

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

Sample No.	Water Soluble Sulfate (% SO_4)	Sulfate Exposure*
B3 @ 1½'	0.023	Negligible
B6 @ 0-2'	0.023	Negligible

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

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CORROSIVITY TEST RESULTS

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MLA GREEN, INC
BETWEEN SECOND STREET AND APPIAN WAY
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