GEOTECHNICAL INVESTIGATION

SAN DIEGO RIVER DISCOVERY CENTER
SAN DIEGO, CALIFORNIA

GEOCON
INCORPORATED

GEOTECHNICAL
ENVIRONMENTAL
MATERIALS

PREPARED FOR

SAN DIEGO RIVER PARK FOUNDATION
SAN DIEGO, CALIFORNIA

JANUARY 31, 2014
PROJECT NO. G1656-42-01

GEOCON
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GEOTECHNICAL ■ ENVIRONMENTAL ■ MATERIALS

Project No. G1656-42-01
January 31, 2014

San Diego River Park Foundation
4891 Pacific Highway, Suite 114
San Diego, California 92110

Attention: Mr. Rob Hutsel

Subject: GEOTECHNICAL INVESTIGATION
SAN DIEGO RIVER DISCOVERY CENTER
SAN DIEGO, CALIFORNIA

Dear Mr. Rob Hutsel:

In accordance with your authorization and our proposal (LG-13261, dated August 23, 2013), we are submitting the results of our geotechnical investigation for the proposed improvements on the subject site. The accompanying report presents the findings and conclusions from our study. Based on the results of our study, it is our opinion that the improvements can be constructed as proposed, provided the recommendations of this report are followed.

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

Very truly yours,

GEOCON INCORPORATED

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GEOTECHNICAL INVESTIGATION

1. PURPOSE AND SCOPE

This report presents the results of our geotechnical investigation for the planned improvements for proposed San Diego River Discovery Center located between Camino Del Rio North and the San Diego River, east of Texas Street in the Mission Valley area of San Diego, California. (see Vicinity Map, Figure 1). The purpose of the investigation was to evaluate soil and geologic conditions and geotechnical constraints that may impact areas of proposed development. This report provides recommendations relative to the geotechnical engineering aspects of developing the project as presently proposed based on the conditions encountered during this investigation.

The scope of our investigation included a site reconnaissance, field investigation, engineering analyses and preparation of this report. The field investigation included drilling 3 exploratory borings to examine and characterize the existing soils within the area of planned development. One boring (MW-1) was finished as a groundwater-monitoring well. Logs of the exploratory boring and a discussion of the field investigation are presented in Appendix A.

We performed laboratory tests on selected soil samples obtained during the field investigation to evaluate pertinent physical properties for engineering analyses and to assist in providing recommendations for foundation and pavement design criteria. Details of the laboratory testing and a summary of the test results are presented in Appendix B.

The base map used to generate Figure 2 is an untitled conceptual plan prepared by Roesling Nakamura Terada Architects, Inc. Figure 2 presents the approximate locations of the exploratory borings and the layout of the planned improvements. Included on Figure 2 are the approximate locations of boring from a previous study prepared by Geocon Incorporated titled *Geotechnical Investigation [for] Russel V. Grant Estate Property*, *San Diego*, *California*, dated April 13, 1998 (Proj. No.06060-22-01). The boring logs and laboratory testing from the previous study are provided in Appendix C.

The conclusions and recommendations presented herein are based on analyses of the data obtained from the field investigation, laboratory tests, and our experience with similar soil and geologic conditions.

2. SITE AND PROJECT DESCRIPTION

The site is located between Camino Del Rio North and the San Diego River east of Texas Street in the Mission Valley area of San Diego, California (see Figure 1). The site is generally undeveloped except for a unpaved access road and berm extending in a generally east-west direction. The topography of the project site slopes gently to the north with elevations of approximately 55 feet above Mean Sea Level (MSL) at the southeast corner to approximately 30 feet MSL in the northwest corner.

Based on discussions with you we understand that the proposed improvements consist of a 1-story building and 2-story building in the south central portion of the site; an amphitheater; river play area, trails; viewing piers; and parking/driveway areas.

The site description and proposed development are based on discussions with you, a site reconnaissance, and review of the referenced site plan. If development plans differ from those described herein, Geocon Incorporated should be contacted for review of the plans and possible revisions to this report.

3. SOIL AND GEOLOGIC CONDITIONS

The subject site is located within the Peninsular Ranges Geomorphic Province of Southern California, which is characterized by a series of northwest trending mountain ranges generally composed of Cretaceous putonic rock dissected by left-lateral, strike-slip faults. This style of faulting extends throughout the province and continues offshore. Specifically, the site is located in an alluviated valley eroded into Eocene, coastal-plane, sedimentary deposits by the San Diego River.

During our field investigation, we encountered undocumented fill overlying alluvial deposits. The undocumented fill and alluvial deposits are described below. A geologic cross section is provided on Figure 3.

3.1 Undocumented Fill

Undocumented fill soil was encountered in all borings to depths ranging from approximately 15 to 30 feet below existing grade. The undocumented fill is assumed to be settlement-pond material generated from a previous aggregate mine and is compressible and expansive, as well as fill to construct portions of the northern slope. The undocumented fill generally consisted of loose, moist, silty sand and soft, moist, sandy silt. The undocumented fill is not suitable for the support of settlement-sensitive structures and improvements and will require remedial grading to support structural improvements.

3.2 Alluvial Deposits (Qal)

Alluvial deposits were observed in all exploratory borings beneath the undocumented fill. The alluvium consisted of loose, saturated, sand, silty sand, and clayey sand and soft to stiff lean to fat clay. Portions of the alluvium are compressible and potentially liquefiable and will require foundation considerations for potential settlement.

4. GROUNDWATER

Groundwater was encountered in all borings at a depth of approximately 20 below ground surface. Groundwater elevation is dependent on seasonal precipitation; irrigation; land use; and other factors. Proper surface drainage will be important to future performance of the project.

5. GEOLOGIC HAZARDS

5.1 Geologic Hazard Category

The *City of San Diego Seismic Safety Study, Geologic Hazards and Faults, 2008 Edition, Grid Tile 21* defines the site as *Hazard Category* 31, Liquefaction: High Potential – shallow groundwater, major drainages, hydraulic fills.

5.2 Faulting and Seismicity

The site is not traversed by any active, potentially active, or inactive faults. However, the Texas Street Fault and Florida Canyon Faults are mapped on the southern hillside of Mission Valley south of the project site. Geocon Incorporated performed a recent fault study for the Quarry Falls project and Vulcan Materials batch plant, located approximately ½-mile north of the site and generally in line with the projections of the Texas Street and Florida Canyon Faults (see Geocon 2012). Geocon concluded that faulting on the Quarry Falls property and Vulcan Materials batch plant site was likely the extension of the Texas Street and Florida Canyon faults. A fault trench excavated on the Quarry Falls property found unbroken Late Pleistocene soils. Geocon concluded that the faults were not active during the last approximately 11,000 years; therefore, the faults were classified as Potentially Active and no structural setback was recommended. The location of the mapped Florida Canyon and Texas Street Faults (as shown on the San Diego Seismic Safety Study, 208) and the surveyed fault locations encountered on the Quarry Falls property and at the Vulcan Materials batch plant in relation to the site covered by this study is shown on Figure 3.

The computer program *EZ-FRISK* (Version 7.52) located 6 known active faults within a of 50 miles search radius centered on the property. The nearest known active fault is the Newport-Inglewood/Rose Canyon Fault Zone, located approximately 3 miles west of the site and is considered the dominant source of potential ground motion. Earthquakes that might occur on the Newport-Inglewood/Rose Canyon Fault Zone or other faults within the southern California and northern Baja California area are potential generators of significant ground motion at the site. The estimated deterministic maximum earthquake magnitude and peak ground acceleration for the Newport-Inglewood/Rose Canyon Fault Zone are 7.5 and 0.34g, respectively. Table 5.1.1 lists the estimated maximum earthquake magnitude and peak ground acceleration for the ten most dominant faults in relationship to the site location. We calculated peak ground acceleration (PGA) using Boore-Atkinson (2008) NGA USGS2008, Campbell-Bozorgnia (2008) NGA USGS, and Chiou-Youngs (2008) NGA acceleration-attenuation relationships.

TABLE 5.1.1
DETERMINISTIC SPECTRA SITE PARAMETERS

Fault Name	**Distance from Site (miles)**	**Maximum Earthquake Magnitude (Mw)**	**Peak Ground Acceleration**		
			Boore-Atkinson 2008 (g)	**Campbell-Bozorgnia 2008 (g)**	**Chiou-Youngs 2008 (g)**
Newport-Inglewood/Rose Canyon	3.3	7.5	0.34	0.21	0.26
Rose Canyon	3.3	6.9	0.32	0.21	0.25
Coronado Bank	15.7	7.4	0.23	0.14	0.18
Coronado Bank/Palos Verdes	15.7	7.7	0.27	0.14	0.19
Elsinore	38.1	7.85	0.22	0.10	0.14
Earthquake Valley	42.7	6.8	0.15	0.07	0.07

We used the computer program *EZ-FRISK* to perform a probabilistic seismic hazard analysis. The computer program *EZ-FRISK* (version 7.52) 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 earthquake magnitude as a function of fault rupture length, and site acceleration estimates are made using the earthquake magnitude and distance from the site to the rupture zone. The program also accounts for uncertainty in each of following: (1) earthquake magnitude, (2) rupture length for a given magnitude, (3) location of the rupture zone, (4) maximum possible magnitude of a given earthquake, and (5) acceleration at the site from a given earthquake along each fault. By calculating the expected accelerations from considered earthquake sources, the program calculates the total average annual expected number of occurrences of site acceleration greater than a specified value. We utilized acceleration-attenuation relationships suggested by Boore-Atkinson (2008) NGA USGS, Campbell-Bozorgnia (2008) NGA USGS, and Chiou-Youngs (2008) in the analysis. Table 5.1.2 presents the site-specific probabilistic seismic hazard parameters including acceleration-attenuation relationships and the probability of exceedence.

TABLE 5.1.2
PROBABILISTIC SEISMIC HAZARD PARAMETERS

Probability of Exceedence	Peak Ground Acceleration		
	Boore-Atkinson, 2008 (g)	Campbell-Bozorgnia, 2008 (g)	Chiou-Youngs, 2008 (g)
2% in a 50 Year Period	0.63	0.31	0.36
5% in a 50 Year Period	0.48	0.24	0.29
10% in a 50 Year Period	0.38	0.20	0.23

The California Geologic Survey (CGS) has a program that calculates the ground motion for a 10 percent of probability of exceedence in 50 years based on an average of several attenuation relationships. Table 5.1.3 presents the calculated results from the *Probabilistic Seismic Hazards Mapping Ground Motion* Page from the CGS website.

TABLE 5.1.3
PROBABILISTIC SITE PARAMETERS FOR SELECTED FAULTS
CALIFORNIA GEOLOGIC SURVEY

Calculated Acceleration (g) Firm Rock	Calculated Acceleration (g) Soft Rock	Calculated Acceleration (g) Alluvium
0.26	0.28	0.32

While listing 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. Seismic design of the structures should be evaluated in accordance with the California Building Code (CBC) guidelines.

5.3 Ground Rupture

The risk associated with ground rupture hazard is low due to the absence of active faults at the subject site.

5.4 Liquefaction Potential

We used the methods presented in the Youd, T. L *et al.* (2001) and SP117 (2008) to perform the liquefaction evaluation of the site using collected during our investigation. We used a ground surface acceleration of 0.416g, which is the Site Class Modified MCE_G Peak Ground Acceleration, PGA_M (see Table 6.6.2 of this report). We also used a mean weighted magnitude of 6.63 M_W obtained from the USGS website’s deaggregation program.

Based on our analyses, there is a high potential for liquefaction occurring within sandy layers of the alluvium. The risk associated with ground surface manifestations such as sand boils and loss of bearing strata as a result of liquefaction is low; however, we expect settlement could occur during a liquefaction event. Estimated settlements resulting from both static and liquefaction settlement are provided herein.

Provided building foundations are designed to accommodate estimated settlement from both static loading and liquefaction (as recommended herein), we do not expect that building collapse, as a result of ground failure, will occur. It is our opinion the recommendations contained in the referenced geotechnical investigation are in accordance with guidelines presented in Special Publication 117. Output of the liquefaction analysis are shown in Appendix D.

5.5 Effects of Liquefaction

Seismically-induced settlement could occur within the liquefied soil layers and non-liquefied layers after seismic shaking stops due to rearrangement of the sand particles. We estimated seismically-induced settlement due to liquefaction using procedures suggested by SP117 (2008). We calculated liquefaction settlement using relationships between cyclic stress ratios and volumetric strain. We estimate a seismic induced liquefaction settlement to be approximately 5 inches. We estimate differential settlement to be approximately ½ of the total settlement. We recommend foundations for the planned structures incorporate the estimated liquefaction settlement in the foundation design.

With respect to lateral spread, there is a free face within 50 feet of the building edge. In our opinion, the risk associated with lateral spread and/or flow slides impacting the property is moderate.

5.6 Cyclic Softening Evaluation

Liquefaction can cause aerial and differential settlement, lateral spreading, loss of bearing capacity, and sudden loss in soil strength. Soils prone to liquefaction are typically loose, saturated sands and, to a lesser degree, silt. Cyclic softening can result in similar hazards to those of liquefaction, but is a phenomenon related to fine-grained silt and clay soils. The term cyclic softening is used to describe the potential for fine-grain clayey silts and silty clays to experience a decrease in shear strength due to earthquake loading. The borings indicate layers of fine-grain clay and silt exist on the site.

The cyclic softening/failure potential of the fine-grained soils beneath the site was evaluated using procedures outlined by Idriss and Boulanger, 2008. The method developed by Idriss and Boulanger compares the soil undrained shear strength (Su), corrected for earthquake magnitude and seismic loading effects, to the earthquake-induced shear stress. The undrained strength of the soil, normalized by the effective soil overburden pressure, is termed cyclic resistance ratio (CRR) while the normalized earthquake-induced shear stress is termed cyclic stress ratio (CSR). The undrained shear strength of the underlying soils was evaluated using methods based on the in-situ cone penetration test soundings.

The cyclic resistance ratio (CRR) of the underlying soil was evaluated from the undrained shear strength and the procedure presented in Idriss and Boulanger, 2008. A groundwater depth of 15 feet was used in the analysis to determine effective soil overburden stresses.

The seismically induced shear stresses at the site were assessed using correlations between penetration resistance values and undrained shear strength.

The cyclic shear strength of the soil (CRR) was then compared to the earthquake-induced shear stress (CSR) to evaluate the potential for cyclic softening. A figure presenting the analysis is included in Appendix D. The CRR values are generally lower than the CSR values at depths below 15 feet, indicating a factor of safety less than 1 for cyclic softening under the design earthquake loading.

The estimated total dynamic settlement associated with cyclic softening within the underlying soil is anticipated to be approximately 1 to 2 inches. Differential dynamic settlement is estimated at approximately one-half the total dynamic settlement.

5.7 Lateral Spread and Flow Slide Potential

Lateral spreading and flow slide can occur on liquefiable sites that are adjacent to slopes such as river channels or large bodies of water. The observed horizontal ground displacement typically decreases with increased distances from the open face. The slope along the north side of the site constitutes an open face.

There are several different methods to calculate the magnitude of lateral spread as a function of distance from the free face. Using the method suggested by Bartlett and Youd, the estimated horizontal ground displacement near the proposed building edge is estimated to be approximately 3 feet (which we think is a conservative estimate). The results of the lateral spread analysis are provided in Appendix D. It is our opinion that the potential for lateral spread affecting the building is moderate. Lateral spread could impact surface and underground utilities along the northern side of the site.

We analyzed flow slide potential as a result of liquefaction using slope stability analyses to estimate slope displacement magnitudes. We performed the slope stability analysis using residual shear strength parameters for the potentially liquefiable soils. Residual shear strengths were determined using information provided in *Recommended Procedures for Implementation of DMG Special Publication 117 Guidelines for Analyzing and Mitigating Liquefaction in California*.

The stability analysis was performed using *SLOPEW* (2007) computer software distributed by Geo-Slope International. This program uses conventional slope stability equations and a two-dimensional limit-equilibrium method to calculate the factor of safety against slope instability. For our analysis, Spencer's Method with a circular failure mechanism was used. The results of the stability analysis indicate that under the static loading the existing slope has a factor of safety in excess of 2.0 (see figure in Appendix D). Under seismic loading where we expect residual shear strength values in the liquefiable layers, the slope has a factor of safety of 1.3 (see figure in Appendix D). Typically, factors of safety greater than 1.1 or 1.2 are satisfactory when evaluating stability under temporary conditions, such as seismic loading. Based on the results of our analysis, there is a low potential for flow side occurring on the property.

5.8 Landslides

Based on our review of published geologic maps for the site vicinity, it is our opinion landslides are not present at the property or at a location that could impact the site.

6. CONCLUSIONS AND RECOMMENDATIONS

6.1 General

6.1.1 No soil or geologic conditions were encountered during our investigation that would preclude development of the property as planned, provided the recommendations of this report are followed. Geocon Incorporated should be contacted to review grading and foundation plans prior to finalization to evaluate potential impacts and determine the necessity of providing revised recommendations.

6.1.2 The site is underlain by undocumented, hydraulically placed fill and alluvium. The undocumented fill was deposited in settlement ponds and is highly compressible and expansive. The undocumented fill is unsuitable for support of buildings and improvements and will require remedial grading. The alluvium is susceptible to seismically induced settlement.

6.1.3 Planned grading is not known at this time, hover, if fill is placed to raise pad grades, settlement monitoring should be performed prior to constructing buildings, underground improvements, and surface improvements.

6.1.4 With did not observe or know of significant geologic hazards on the site that would adversely affect proposed development with the exception of liquefaction and associated settlement and lateral spreading. Building foundations will need to be designed to accommodate settlement as a result of potential soil liquefaction and lateral spreading.

6.1.5 We encountered groundwater at a depth of approximately 21 to 22 feet below existing grade.

6.1.6 Subsurface conditions observed may be extrapolated to reflect general soil/geologic conditions at the site; however, some variations in subsurface conditions between boring locations should be expected.

6.2 Excavation and Soil Characteristics

6.2.1 The soil encountered in the field investigation is considered to be “expansive/non-expansive” (expansion index [EI] of greater than 20 / 20 or less) as defined by 2013 California Building Code (CBC) Section 1803.5.3. Table 6.2.1 presents soil classifications based on the expansion index. Based on laboratory testing, the on-site soils possess a “low” to “very high” expansion potential (expansion index of 50 or less).

TABLE 6.2.1
EXPANSION CLASSIFICATION BASED ON EXPANSION INDEX

Expansion Index (EI)	Expansion Classification	2013 CBC Expansion Classification
0 – 20	Very Low	Non-Expansive
21 – 50	Low	Expansive
51 – 90	Medium	
91 – 130	High	
Greater Than 130	Very High	

6.2.2 We performed laboratory tests on samples of the site materials during previous grading to evaluate the percentage of water-soluble sulfate content. Results from the laboratory water-soluble sulfate content tests are presented in Appendix B. The test results indicate that the on-site materials at the locations tested possess “Not Applicable” (“S0”) sulfate exposure to concrete structures as defined by 2013 CBC Section 1904 and ACI 318-08 Sections 4.2 and 4.3. Table 6.2.2 presents a summary of concrete requirements set forth by 2013 CBC Section 1904 and ACI 318. The presence of water-soluble sulfates is not a visually discernible characteristic; therefore, other soil samples from the site could yield different concentrations. Additionally, over time landscaping activities (i.e., addition of fertilizers and other soil nutrients) may affect the concentration.

TABLE 6.2.2
REQUIREMENTS FOR CONCRETE EXPOSED TO SULFATE-CONTAINING SOLUTIONS

Sulfate Exposure	Exposure Class	Water-Soluble Sulfate Percent by Weight	Cement Type	Maximum Water to Cement Ratio by Weight	Minimum Compressive Strength (psi)
Not Applicable	S0	0.00-0.10	--	--	2,500
Moderate	S1	0.10-0.20	II	0.50	4,000
Severe	S2	0.20-2.00	V	0.45	4,500
Very Severe	S3	> 2.00	V+Pozzolan or Slag	0.45	4,500

6.2.3 We performed laboratory tests on selected soil samples to check the corrosion potential to subsurface metal structures. A site is considered corrosive if the chloride ion concentration is 500 part per million (ppm) or greater, water-soluble sulfate concentration is 2,000 ppm

(0.2 percent) or greater, or the pH is 5.5 or less according to Caltrans *Corrosion Guidelines,* dated September 2003. The laboratory test results are presented in Appendix B.

6.2.4 Laboratory testing to evaluate corrosion potential to metal in contact with the soil was performed. The test results are included in Appendix B. With respect to metal, we recommend the site be considered very corrosive.

6.2.5 Geocon Incorporated does not practice in the field of corrosion engineering; therefore, further evaluation by a corrosion engineer may be needed to incorporate the necessary precautions to avoid premature corrosion of underground pipes and buried metal in direct contact with soil.

6.2.6 On-site soil can be excavated with moderate to heavy effort using conventional heavy-duty grading equipment.

6.3 Grading Recommendations

6.3.1 The grading recommendations provided herein assume that the structures will be founded on shallow mat slab foundation that can accommodate the expected total and differential settlement as a result of liquefaction, cyclic softening, and lateral spread.

6.3.2 All grading should be performed in accordance with the *Recommended Grading Specifications* contained in Appendix E. Where the recommendations of this section conflict with those of Appendix E, the recommendations of this section take precedence. Earthwork should be observed and fills tested for compaction by Geocon Incorporated.

6.3.3 Site preparation should begin with removal of all deleterious matter, vegetation, concrete, asphalt concrete and debris. The depth of removal should be such that materials to be used in fills are generally free of organic matter. Material generated during stripping operations and/or site demolition should be exported from the site.

6.3.4 Building pads should be graded in such a way to allow at least 5 feet of properly compacted fill beneath the base of the foundation. The remedial grading should extend at least 5 feet horizontally beyond the structure, where practical.

6.3.5 The on-site soils are highly expansive. We recommend fill placed within the upper 5 feet of building pad grade be “low” to “medium” expansive soil (Expansion Index less than 90). This will require selective grading and may require import soil.

6.3.6 As an option to reduce potential settlement as a result of building loading, the building pad could be surcharged with at least 3 feet of soil. The top of the surcharge fill should extend out to a horizontal distance of at least 5 feet beyond the limits of the building pad and structural footing areas. We recommend the bottom 1 foot of the surcharge fill be placed and compacted as structural fill. This will result in compacted fill at finish grade once settlement of the underlying alluvium occurs. We recommend surcharge fill and settlement monitoring occur before construction of improvements commences. If surcharging of the pad is not performed, the foundation will need to be designed to accommodate potential settlement as a result of building loads.

6.3.7 Surface improvement areas (concrete hardscape, pavement) should be placed on at least 36 inches of properly compacted "low" to "medium" expansive fill (EI less than 90). Sidewalks for pedestrian traffic only should be placed on 24 inches of "low" to "medium" expansive fill that is properly compacted. Where practical, the remedial grading should extend to a horizontal distance of at least 2 feet beyond the edge of the structural improvement. Additional remedial grading may be required if loose or otherwise unsuitable material is encountered at the base of the removals. Although a compacted fill mat will provide sufficient support for proposed parking lot improvements and vehicular traffic, settlement of undocumented fill left in-place could still occur. Future settlement may require periodic pavement repair and maintenance.

6.3.8 Wet, saturated and or yielding soils may be encountered at the base of remedial excavations within buildings pads and possibly within pavement and hardscape areas. Where yielding soils are encountered, the excavation bottom should be stabilized by placing a layer of geotextile reinforcing grid (Tensar TX7) across the base of the excavation. A minimum 12-inch layer of ¾-inch to 1.5-inch gravel should be placed on the geotextile grid and filter fabric (Mirafi 140N) should be placed across the gravel. One foot of soil should then be placed across the excavation and compacted. If yielding occurs, a second layer of Tensar TX7 geogrid should be placed across the excavation. Additional recommendations may be required if significant yielding still occurs after the second layer of reinforcing grid is placed.

6.3.9 Fill soil should be placed and compacted in layers to design finish-grade elevations. The layers should be no thicker than will allow for adequate bonding and compaction. All fill (including scarified ground surfaces and wall and utility trench backfill) should be compacted to at least 90 percent of maximum dry density as determined by ASTM D1557. For low to medium expansive soils, the soils should be compacted at a moisture content that is at or slightly above optimum moisture content. The placement of fill soil should be observed and tested by a representative of Geocon Incorporated.

6.3.10 Imported fill, if required, should consist of granular materials with an Expansion Index less than 50. Import soils should be tested by Geocon Incorporated prior to being imported to verify conformance with the recommended expansion criteria.

6.4 Mitigation of Compressible and Liquefiable Alluvium

6.4.1 Ground modification of potentially liquefiable soils and compressible alluvium can be performed for settlement-sensitive structures to reduce potential liquefaction and loading induced settlement and impacts associated with lateral spread. Some alternatives include stone columns, soil mixing, vibro-piers, or Geopier and Geopier Impact piers. Geocon Incorporated should be contacted to provide recommendations if ground improvement alternatives are desired.

6.5 Settlement Monitoring

6.5.1 Alluvial and undocumented fill deposits are moderately to highly compressible when subjected to increased vertical stress. The placement of additional fill to raise grade will cause settlement in the underlying soils. Therefore, settlement monitoring is recommended.

6.5.2 Once rough pad grade is attained (or surcharge fill placed if used), we recommend surface monuments be installed to measure settlement. The locations of monuments should be determined once development plans are prepared. A typical surface settlement monument detail is presented as Figure 4.

6.5.3 Surface settlement monuments should be read every approximately two weeks by the project surveyor until measured settlement is within tolerable limits such that additional settlement as a result of fill loading will not impact site improvements. Based on our experience with similar soil conditions, we estimate 3 to 6 months of monitoring would be necessary to demonstrate that primary consolidation is essentially complete.

6.6 Seismic Design Criteria

6.6.1 We used the computer program *U.S. Seismic Design Maps*, provided by the USGS. Table 6.6.1 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 short spectral response uses a period of 0.2 second. The site is classified as a Site Class F in accordance with the 2013 CBC Section 1613; however, if the period of the structure is less than 0.5 seconds and the structure does not fall under CBC Chapter 16A, the exception under ASCE 7-10 Section 20.3.1 can be used to determine Site Class. Using the exception allowed by

ASCE 7-10, a Site Class E should be used to design the planned structure. If the period of the structure is greater than 0.5 second or falls under CBC Chapter 16A, a site specific response spectrum will be required. We evaluated the Site Class based on the discussion in Section 1613.3.2 of the 2013 CBC and Table 20.3-1 of ASCE 7-10. The values presented in Table 6.5.1 are for the risk-targeted maximum considered earthquake (MCE_R)

TABLE 6.6.1
2013 CBC SEISMIC DESIGN PARAMETERS

Parameter	Value	2010 CBC Reference
MCE_R Ground Motion Spectral Response Acceleration – Class B (short), S_S	E	Section 1613.3.2
MCE_R Ground Motion Spectral Response Acceleration – Class B (1 sec), S_1	1.082 g	Figure 1613.3.1(1)
Site Coefficient, F_A	0.415 g	Figure 1613.3.1(2)
Site Coefficient, F_V	0.90	Table 1613.3.3(1)
Site Class Modified MCE_R Spectral Response Acceleration (short), S_{MS}	2.40	Table 1613.3.3(2)
Site Class Modified MCE_R Spectral Response Acceleration (1 sec), S_{M1}	0.973 g	Section 1613.3.3 (Eqn 16-37)
5% Damped Design Spectral Response Acceleration (short), S_{DS}	0.995 g	Section 1613.3.3 (Eqn 16-38)
5% Damped Design Spectral Response Acceleration (1 sec), S_{D1}	0.649 g	Section 1613.3.4 (Eqn 16-39)
MCE_R Ground Motion Spectral Response Acceleration – Class B (short), S_S	0.663 g	Section 1613.3.4 (Eqn 16-40)

6.6.2 Table 6.6.2 presents additional seismic design parameters for projects located in Seismic Design Categories of D through F in accordance with ASCE 7-10 for the mapped maximum considered geometric mean (MCE_G).

TABLE 6.6.2
2013 CBC SITE ACCELERATION PARAMETERS

Parameter	Value	ASCE 7-10 Reference
Mapped MCE_G Peak Ground Acceleration, PGA	0.462g	Figure 22-7
Site Coefficient, F_{PGA}	0.90	Table 11.8-1
Site Class Modified MCE_G Peak Ground Acceleration, PGA_M	0.416g	Section 11.8.3 (Eqn 11.8-1)

6.6.3 Conformance to the criteria in Table 6.6.1 and 6.6.2 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.

6.7 Foundation Settlement

6.7.1 The building foundation will need to be designed to accommodate total and differential settlement from both static and dynamic loading. We estimate total static settlement from building loads to be 2 inches or less. We estimate total dynamic settlement to be between 5 to7 inches as a result of liquefaction and strain softening. Because of the potential for liquefaction and strain softening settlement, we recommend the building be founded on a rigid mat slab.

6.7.2 Differential settlement is expected to be approximately one-half of the total settlement, or approximately 3.5 to 4.5 inches across the building pad. Releveling of the building slab may be required after a significant seismic event. Utility connections to the structures should be flexible to accommodate anticipated settlements.

6.8 Lateral Spreading

6.8.1 Based on our analysis, there is a potential for approximately 3 feet of permanent horizontal ground displacement within the building pad as a result of lateral spread. Although we believe this value is conservative, the building mat slab should be designed such that catestropic building failure does not occur for this magnitude of lateral spread.

6.9 Mat Slab Foundation Recommendations

6.9.1 A mat foundation consists of a thick rigid concrete mat that allows the entire footprint of the structure to carry building loads. In addition, the mat can tolerate significantly greater differential movements such as those associated with very large loads and settlement caused by compressible soils and liquefaction. The mat foundation system will allow the supported area to settle with the ground and should have sufficient rigidity to allow the structure to move as a single unit.

6.9.2 The mat foundation may be designed for an allowable soil contact bearing pressure of 1,000 pounds per square foot (psf) for properly compacted fill (dead plus live load). A modulus of subgrade reaction of 150 pounds per cubic inch for compacted fill can be used to evaluate deflection of the mat. These modulus values are for a foundation measuring

1 foot by 1 foot and should be modified for design of the mat foundation using standard equations. The bottom of the mat slab foundation should be embedded at least 12 inches below lowest adjacent grade.

6.9.3 Foundations should be designed to accommodate the total and differential settlement indicated in Section 6.7.

6.9.4 The project structural engineer should design reinforcement for the mat foundation.

6.10 Foundation Recommendations – General

6.10.1 A vapor retarder should underlie slabs that may receive moisture-sensitive floor coverings or may be used to store moisture-sensitive materials. The vapor retarder design should be consistent with the guidelines presented in the American Concrete Institute’s (ACI) Guide for Concrete Slabs that Receive Moisture-Sensitive Flooring Materials (ACI 302.2R-06). In addition, the membrane should be installed in a manner that prevents puncture, in accordance with manufacturer’s recommendations and ASTM requirements. The project architect or developer should specify the type of vapor retarder used based on the type of floor covering that will be installed and if the structure will possess a humidity-controlled environment.

6.10.2 The project foundation engineer, architect, and/or developer should determine the thickness of bedding sand below the slab. Generally, a 3 to 4 inch sand cushion is used. However, Geocon should be contacted to provide recommendations if the bedding sand is thicker than 6 inches.

6.10.3 The foundation design engineer should provide appropriate concrete mix design criteria and curing measures to assure proper curing of the slab by reducing the potential for rapid moisture loss and subsequent cracking and/or slab curl. We suggest that the foundation design engineer present the concrete mix design and proper curing methods on the foundation plans. It is critical that the foundation contractor understands and follows the specifications presented on the foundation plans.

6.10.4 Foundation excavations should be observed by a representative of Geocon Incorporated prior to the placement of reinforcing steel to check that the exposed soil conditions are similar to those expected and that they have been extended to the appropriate bearing strata. If unexpected soil conditions are encountered, modifications to the foundation may be required.

6.10.5 Exterior slab recommendations assume soils within 3 feet of pad grade consists of low to medium expansive soils (EI less than 90). Modified recommendations will be required if highly expansive soils are encountered near pad grade. Exterior slabs not subject to vehicle loads should be at least 4 inches thick and reinforced with No. 3 steel reinforcing bars spaced 24 inches. The reinforcing bars should be positioned at mid-height of the slab. Prior to construction of slabs, the subgrade should be moisture conditioned to at least optimum moisture content and compacted to a dry density of at least 90 percent of the laboratory maximum dry density.

6.10.6 Special subgrade presaturation is not deemed necessary prior to placing concrete; however, the exposed foundation and slab subgrade soil should be moisture conditioned, as necessary, to maintain a moist condition as would be expected in any such concrete placement.

6.10.7 Where buildings or other improvements are planned near the top of a slope steeper than 3:1 (horizontal:vertical), special foundations and/or design considerations are recommended due to the tendency for lateral soil movement to occur.

- For fill slopes less than 20 feet high or cut slopes regardless of height, building footings should be deepened such that the bottom outside edge of the footing is at least 7 feet horizontally from the face of the slope.

- When located next to a descending 3:1 (horizontal:vertical) fill slope or steeper, the foundations should be extended to a depth where the minimum horizontal distance is equal to H/3 (where H equals the vertical distance from the top of the fill slope to the base of the fill soil) with a minimum of 7 feet but need not exceed 40 feet. The horizontal distance is measured from the outer, deepest edge of the footing to the face of the slope. An acceptable alternative to deepening the footings would be the use of a post-tensioned slab and foundation system or increased footing and slab reinforcement. Specific design parameters or recommendations for either of these alternatives can be provided once the building location and fill slope geometry have been determined.

- If swimming pools are planned, Geocon Incorporated should be contacted for a review of specific site conditions.

- Swimming pools located within 7 feet of the top of cut or fill slopes are not recommended. Where such a condition cannot be avoided, the portion of the swimming pool wall within 7 feet of the slope face be designed assuming that the adjacent soil provides no lateral support. This recommendation applies to fill slopes up to 30 feet in height, and cut slopes regardless of height. For swimming pools located near the top of fill slopes greater than 30 feet in height, additional recommendations may be required and Geocon Incorporated should be contacted for a review of specific site conditions.

- Although other improvements, which are relatively rigid or brittle, such as concrete flatwork or masonry walls, may experience some distress if located near the top of a slope, it is generally not economical to mitigate this potential. It may be possible, however, to incorporate design measures that would permit some lateral soil movement without causing extensive distress. Geocon Incorporated should be consulted for specific recommendations.

6.10.8 The recommendations of this report are intended to reduce the potential for cracking of slabs due to expansive soil (if present), differential settlement of existing soil or soil with varying thicknesses. However, even with the incorporation of the recommendations presented herein, foundations, stucco walls, and slabs-on-grade placed on such conditions may still exhibit some cracking due to soil movement and/or shrinkage. The occurrence of concrete shrinkage cracks is independent of the supporting soil characteristics. The 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.

6.11 Retaining Wall and Lateral Load Recommendations

6.11.1 Retaining walls that are allowed to rotate more than 0.001H (where H equals the height of the retaining portion of the wall) at the top of the wall and having a level backfill surface should be designed for an active soil pressure equivalent to the pressure exerted by a fluid density of 35 pcf. Where the backfill will be inclined at 2:1 (horizontal:vertical), an active soil pressure of 50 pcf is recommended. These active pressures assume low expansive soil (Expansion Index less than 50) will be used as retaining wall backfill. Low expansive soils will require importing. **On-site silty and clayey soils should not be used as retaining wall backfill due to their fine grained and expansive nature**.

6.11.2 Where walls are restrained from movement at the top, an additional uniform pressure of 8H psf should be added to the active soil pressure where the walls are less than 8 feet tall. Walls in excess of 8 feet should be designed to accommodate an additional uniform pressure of 12H for restrained conditions.

6.11.3 Soil to be used as backfill should be stockpiled and samples obtained for laboratory testing to evaluate its suitability for use as wall backfill. Modified lateral earth pressures will be required if backfill soils do not meet the required expansion index. City or regional standard wall designs, if used, are based on a specific active lateral earth pressure and/or soil friction angle. On-site soils might not meet the design values used for City or regional standard wall design. Geocon Incorporated should be consulted if City or regional standard wall designs will be used to assess the suitability of on-site soil for use as wall backfill.

6.11.4 Retaining walls subject to vehicular loads within a horizontal distance equal to two-thirds the wall height, a surcharge equivalent to 2 feet of fill soil should be added.

6.11.5 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, retaining walls that support more than 6 feet of backfill should be designed with seismic lateral pressure in accordance with Section 18.3.5.12 of the 2013 CBC. The seismic load is dependent on the retained height where H is the height of the wall, in feet, and the calculated loads result in pounds per square foot (psf) exerted at the base of the wall and zero at the top of the wall. A seismic load of 20H should be used for design. We used the peak ground acceleration adjusted for Site Class effects, PGA_M, of 0.416g calculated from ASCE 7-10 Section 11.8.3 and applied a pseudo-static coefficient of 0.33.

6.11.6 In general, wall foundations having a minimum depth and width of one foot may be designed for an allowable soil bearing pressure of 2,000 psf, provided the soil within 3 feet below the base of the wall consists of compacted fill with an Expansion Index of less than 90. The proximity of the foundation to the top of a slope steeper than 3:1 could impact the allowable soil bearing pressure. Therefore, Geocon Incorporated should be consulted where such a condition is expected.

6.11.7 Retaining walls should be provided with a drainage system adequate to prevent the buildup of hydrostatic forces and should be waterproofed as required by the project architect. The use of drainage openings through the base of the wall (weep holes) is not recommended where the seepage could be a nuisance or otherwise adversely affect the property adjacent to the base of the wall. The above recommendations assume a properly compacted granular (EI of less than 50) free-draining backfill material with no hydrostatic forces or imposed surcharge load. Figure 5 presents a typical retaining wall drainage detail. If conditions different than those described are expected, Geocon Incorporated should be contacted for additional recommendations.

6.11.8 To resist lateral loads, a passive pressure equivalent to the pressure exerted by a fluid density of 300 pcf should be used for design of footings or shear keys poured neat against properly compacted granular fill soils. The allowable passive pressure assumes a horizontal surface extending at least 5 feet, or three times the surface generating the passive pressure, whichever is greater. The upper 12 inches of material in areas not protected by floor slabs or pavement should not be included in design for passive resistance.

6.11.9 If friction is to be used to resist lateral loads, an allowable coefficient of friction between soil and concrete of 0.30 should be used for design.

6.12 Preliminary Pavement Recommendations

6.12.1 The following pavement sections are based on an assumed R-Value of 10. Final pavement sections should be calculated once subgrade elevations have been attained and R-Value testing on actual subgrade samples is performed. We calculated the preliminary pavement sections using procedures outlined in the *California Highway Design Manual* (Caltrans). Table 6.12.1 presents the preliminary flexible pavement recommendations for varying traffic indices. The civil engineer should determine the appropriate traffic index for the anticipated traffic volume and pavement area.

TABLE 6.12.1
PRELIMINARY FLEXIBLE PAVEMENT SECTIONS

Traffic Index	Asphalt Concrete Thickness (inches)	Class 2 Aggregate Base Thickness (inches)
4.5	3	7.0
5.0	3	9.0
5.5	3	11
6.0	4	10.5
6.5	4	12.5
7.0	4	14.0

6.12.2 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 26-1.02B of the *Standard Specifications of the State of California, Department of Transportation* (Caltrans). Crushed aggregate base, as specified in the Green Book, can be used in lieu of Class 2 Aggregate Base.

6.12.3 Prior to placing base material, the subgrade should be scarified, moisture conditioned and recompacted to a minimum of 95 percent relative compaction. The depth of compaction should be at least 12 inches. The base material should be compacted to at least 95 percent relative compaction. Asphalt concrete should be compacted to at least 95 percent Hveem density.

6.12.4 A rigid Portland Cement concrete (PCC) pavement section should be placed in driveway entrance aprons and trash bin loading/storage areas. We calculated the rigid pavement section in general conformance with the procedure recommended by the American Concrete Institute report *ACI 330R-08 Guide for Design and Construction of Concrete Parking Lots* using the parameters presented in Table 6.12.2.

TABLE 6.12.2
PRELIMINARY RIGID PAVEMENT DESIGN PARAMETERS

Design Parameter	Design Value
Modulus of subgrade reaction, k	100 pci
Modulus of rupture for concrete, M_R	500 psi
Compressive Strength	3,000 psi
Traffic Category, TC	A and C
Average daily truck traffic, ADTT	10 (A) and 100 (C)

6.12.5 Based on the criteria presented herein, the PCC pavement sections should have a minimum thickness as presented in Table 6.12.3.

TABLE 6.12.3
PRELIMINARY RIGID PAVEMENT RECOMMENDATIONS

Location	Portland Cement Concrete (inches)	Class 2 Aggregate Base (inches)
Automobile Areas (TC=A-10)	5.5	4
Entrance aprons and Heavy Truck/Fire Lane Areas (TC=C-100)	7	4

6.12.6 The PCC pavement should be placed over subgrade soil that is compacted to a dry density of at least 95 percent of the laboratory maximum dry density near to slightly above optimum moisture content.

6.12.7 Loading aprons such as trash bin enclosures should utilize Portland Cement concrete as recommended in Table 6.12.3 for heavy trucks. The pavement should be reinforced with No. 3 steel reinforcing bars spaced 24 inches on center in both directions placed at the slab midpoint. The concrete should extend out from the trash bin such that both the front and rear wheels of the trash truck will be located on reinforced concrete pavement when loading. Reinforcing steel, outside of trash bin loading areas, is not be necessary within the concrete for geotechnical purposes with the possible exception of dowels at construction joints as discussed herein.

6.12.8 A thickened edge or integral curb should be constructed on the outside of concrete slabs subjected to wheel loads. The thickened edge should be 1.2 times the slab thickness or a minimum thickness of 2 inches, whichever results in a thicker edge, and taper back to the recommended slab thickness 4 feet behind the face of the slab (e.g., a 7-inch-thick slab would have a 9-inch-thick edge).

6.12.9 To control the location and spread of concrete shrinkage cracks, it is recommended that crack control joints be included in the design of the concrete pavement slabs. Crack control joint spacing should not exceed 12 feet. The crack control joints should be created while the concrete is still fresh using a grooving tool or shortly thereafter using saw cuts. The depth of the crack-control joints should be determined following recommendations in Section 5.7 of the referenced ACI guide. The cracks should be sealed with an appropriate sealant to prevent the migration of water through the control joint to the subgrade materials. Appropriate fillers or sealers are discussed in the referenced ACI guide.

6.12.10 Construction joints should be provided at the interface between areas of concrete placed at different times during construction. Doweling is recommended between the joints in pavements designed to accommodate heavy truck traffic. Dowels should meet the recommendations in the referenced ACI guide.

6.12.11 The performance of pavements is highly dependent upon providing positive surface drainage away from the edge of pavements. Allowing water to pond on or adjacent to the pavement will likely result in saturation of the subgrade materials and subsequent pavement distress. Where landscape or planter islands are planned adjacent to pavement surfaces, the perimeter curb should extend at least 6 inches below the bottom of the Class 2 aggregate base and into the underlying subgrade. Drainage from landscaped areas should be directed to controlled drainage structures.

6.13 Hydraulic Conductivity Testing

6.13.1 We performed 4, field-saturated, borehole, hydraulic conductivity tests using the Aardvark Permeameter at the locations presented on the Boring Location Map, Figure 2. The 3-inch diameter test holes were hand augured to the testing depths. Table 6.13 presents the test results. The soil types encountered generally consisted of medium dense, silty, fine to medium sands.

TABLE 6.13
FIELD SATURATED HYDRAULIC CONDUCTIVITY TEST RESULTS

Location	Field Saturated Hydraulic Conductivity(in/hr)
P1	0.6
P2	6.0
P4	4.5
P4	3.0

6.13.2 Soil permeability values from in-situ tests can vary significantly from one location to another due to the non-homogeneous characteristics inherent to most soil. However, if a sufficient amount of field and laboratory test data is obtained, a general trend of soil permeability can usually be evaluated. For this project and for storm water purpose, the soil infiltration rates presented herein should be considered approximate values for preliminary design use only.

6.13.3 Soil density and grain-size distribution has a marked effect on soil permeability. Small increase in density and/or clay content can result in a dramatic decrease in soil permeability.

6.14 Bio-Retention Basin and Bio-Swale Recommendations

6.14.1 At the completion of grading the site will be underlain by compacted fill. Infiltrating into compacted fill generally results in settlement and distress to improvements placed over the compacted fill. It is our opinion the compacted fill is unsuitable for infiltration of storm water runoff due to the potential for adverse settlement.

6.14.2 Any detention basins, bioswales and bio-remediation areas should be designed by the project civil engineer and reviewed by Geocon Incorporated. Typically, bioswales consist of a surface layer of vegetation underlain by clean sand. A subdrain should be provided beneath the sand layer. Prior to discharging into the storm drainpipe, a seepage cutoff wall should be constructed at the interface between the subdrain and storm drainpipe. The concrete cut-off wall should extend at least 6-inches beyond the perimeter of the gravel-packed subdrain system.

6.14.3 Distress may be caused to planned improvements and properties located hydrologically downstream or adjacent to these devices. The distress depends on the amount of water to be detained, its residence time, soil permeability, and other factors. We have not performed a hydrogeology study at the site. Downstream and adjacent properties may be subjected to seeps, springs, slope instability, raised groundwater, movement of foundations and slabs, or other impacts as a result of water infiltration. Grading will result in compacted fill across the site underlain by undocumented fill. As such, where improvements are located near bioswales and bio-remediation areas, an impermeable barrier, such as a thick visqueen should be placed to prevent water infiltration in to the underlying fill soil.

6.14.4 The landscape architect should be consulted to provide the appropriate plant recommendations. If drought resistant plants are not used, irrigation may be required.

6.15 Site Drainage and Moisture Protection

6.15.1 Adequate site drainage is critical to reduce the potential for differential soil movement, erosion and subsurface seepage. Under no circumstances should water be allowed to pond adjacent to footings. The site should be graded and maintained such that surface drainage is directed away from structures in accordance with 2010 CBC 1804.3 or other applicable standards. In addition, surface drainage should be directed away from the top of slopes into swales or other controlled drainage devices. Roof and pavement drainage should be directed into conduits that carry runoff away from the proposed structure.

6.15.2 In the case of basement walls or building walls retaining landscaping areas, a water-proofing system should be used on the wall and joints, and a Miradrain drainage panel (or similar) should be placed over the waterproofing. The project architect or civil engineer should provide detailed specifications on the plans for all waterproofing and drainage.

6.15.3 Underground utilities should be leak free. Utility and irrigation lines should be checked periodically for leaks, and detected leaks should be repaired promptly. Detrimental soil movement could occur if water is allowed to infiltrate the soil for prolonged periods of time.

6.16 Limited Groundwater Quality Testing

6.16.1 We submitted groundwater samples collect from MW-1 to an analytical laboratory to test for gasoline range organics (GRO), diesel range organics (DRO), and volatile organic compounds (VOC). The results show that GRO and VOC values were in the non-detectable range; however, DRO concentration was slightly above the drinking water standard for San Francisco County, which is the only county in California to publish a standard. The complete report of testing is provided in Appendix B

6.17 Grading and Foundation Plan Review

6.17.1 Grading and foundation plans should be reviewed by Geocon Incorporated 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 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.

2. 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 Incorporated 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 Incorporated.

3. This report is issued with the understanding that it is the responsibility of the owner or 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.

4. 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 be 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.

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NO SCALE

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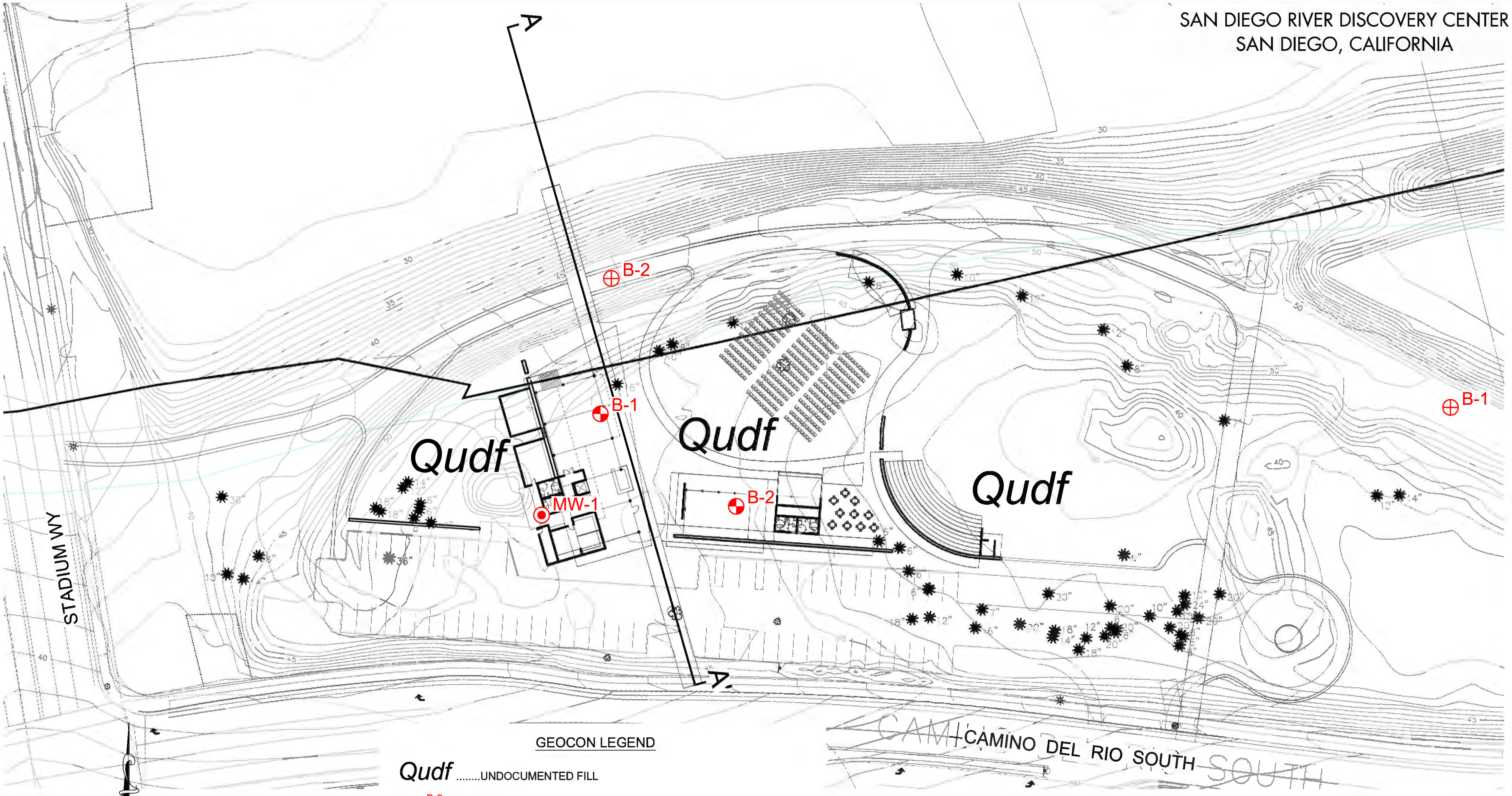
VICINITY MAP

SAN DIEGO RIVER DISCOVERY CENTER
SAN DIEGO, CALIFORNIA

DATE 01 - 31 - 2014	PROJECT NO. G1656 - 42 - 01	FIG. 1

Vicinity Map

SAN DIEGO RIVER DISCOVERY CENTER
SAN DIEGO, CALIFORNIA



GEOCON LEGEND

Qudf ........UNDOCUMENTED FILL

B-2 ....APPROX. LOCATION OF BORING
(Current Study)

B-2 ⊕........APPROX. LOCATION OF BORING FROM PREVIOUS STUDY
(Geocon, 1998)

MW-1 ....APPROX. LOCATION OF GROUNDWATER MONITORING WELL

A — A' ....APPROX. LOCATION OF CROSS SECTION

0' 30' 60' 90' 120'
SCALE 1"= 60'

GEOLOGIC MAP

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PROJECT NO. G1656 - 42 - 01
FIGURE 2
DATE 01 - 31 - 2014

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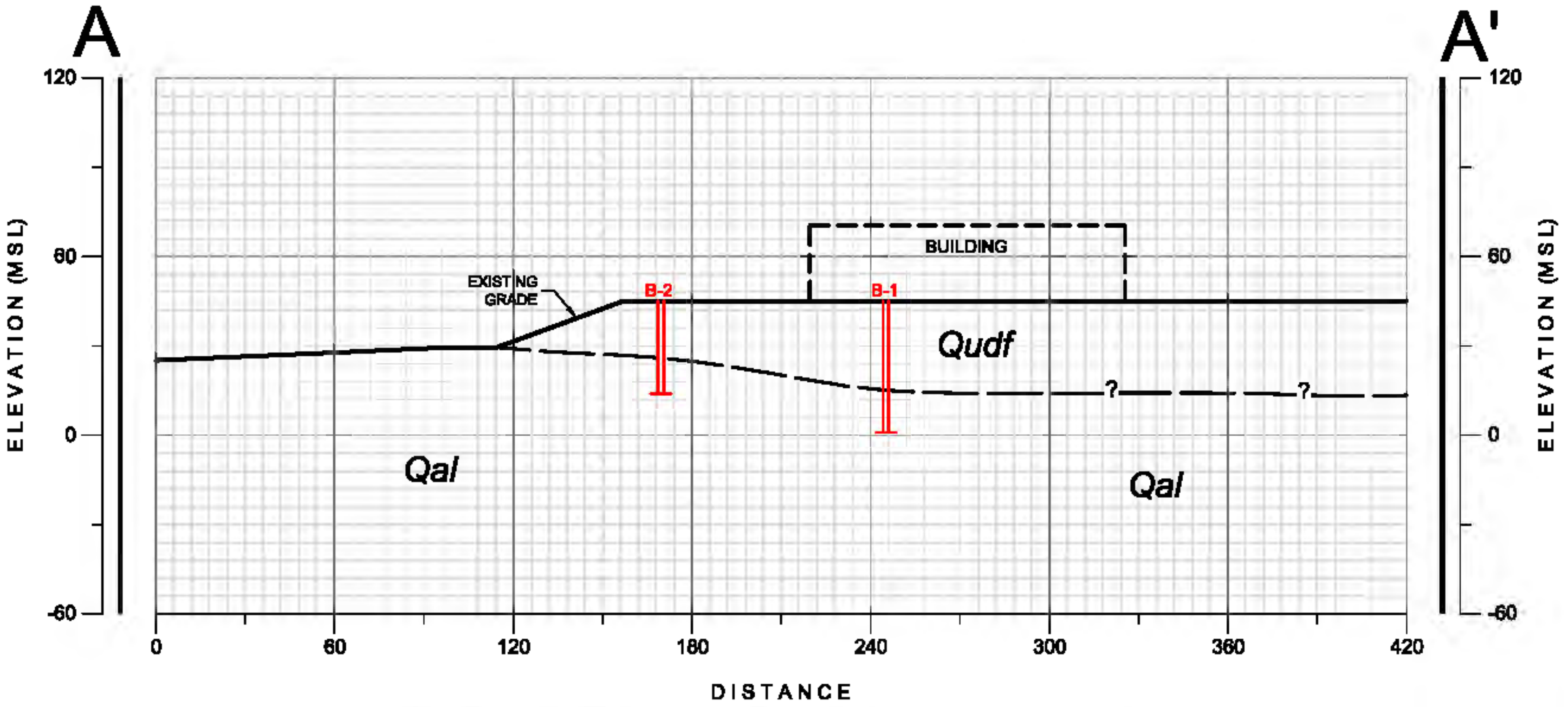
Y:\PROJECTS\G1656-42-01 (Discovery Center)\SHEETS\G1656-42-01 Map.dwg

SAN DIEGO RIVER DISCOVERY CENTER
SAN DIEGO, CALIFORNIA



GEOLOGIC CROSS-SECTION A-A'

SCALE: 1" = 60' (Vert. = Horiz.)

GEOCON LEGEND

Qudf ........UNDOCUMENTED FILL

Qal ....ALLUVIUM

~?~ ....APPROX. LOCATION OF GEOLOGIC CONTACT
(Queried Where Uncertain)

B-2 ....APPROX. LOCATION OF BORING

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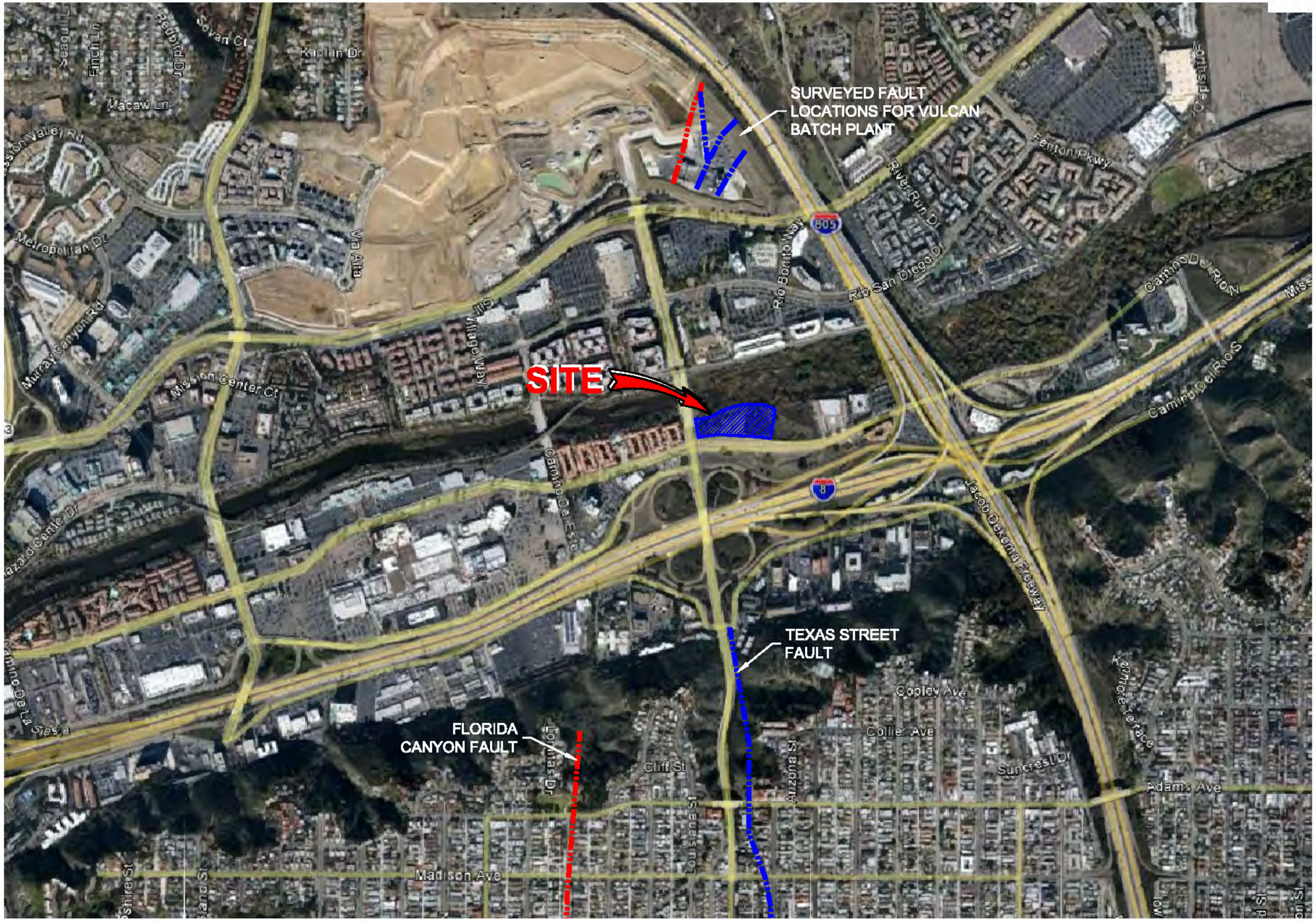
PROJECT NO. G1656 - 42 - 01
FIGURE 3
DATE 01 - 31 - 2014

LADRILLONO

Y:\PROJECTS\G1656-42-01 (Discovery Center)\SHEETS\G1656-42-01 XSection.dwg

SAN DIEGO RIVER DISCOVERY CENTER
SAN DIEGO, CALIFORNIA



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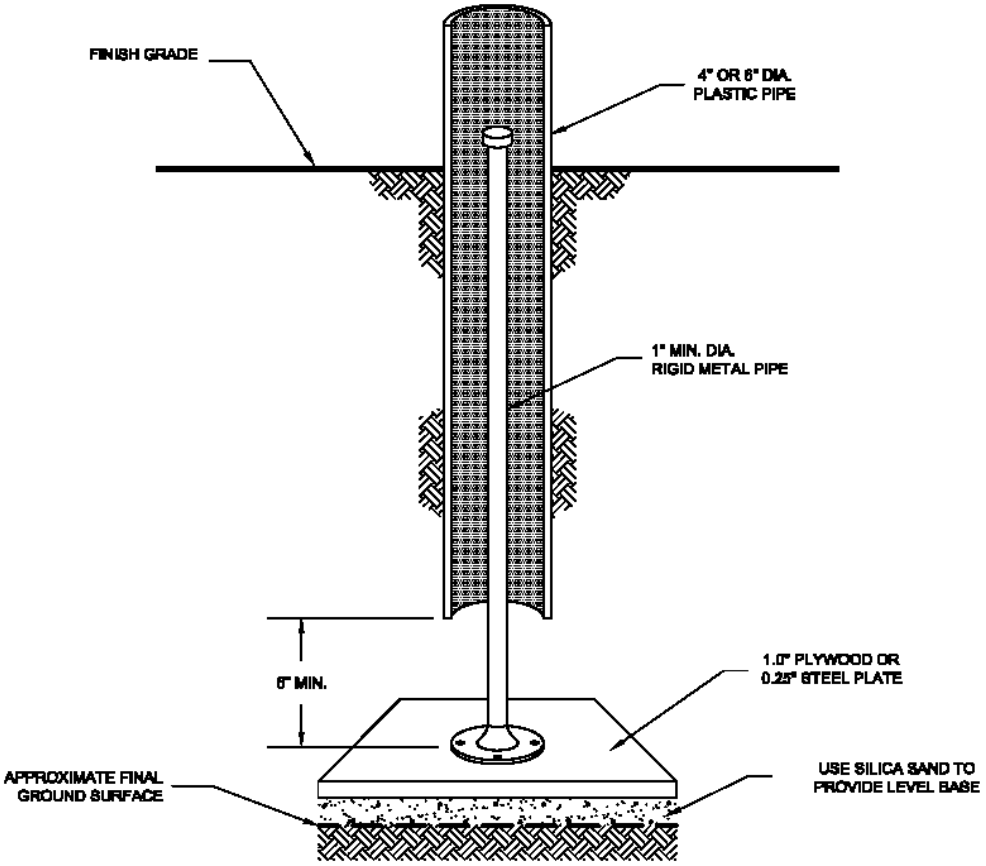
FAULT MAP

PROJECT NO. G1656 - 42 - 01
FIGURE 4
DATE 01 - 31 - 2014



NO SCALE

NOTES:

1......LOCATION OF SETTLEMENT PLATES SHALL BE CLEARLY MARKED AND READILY VISIBLE (RED FLAG) TO EQUIPMENT OPERATORS.

2......CONTRACTOR SHALL MAINTAIN 10-FOOT HORIZONTAL CLEARANCE.

3......IN THE EVENT OF DAMAGE TO SETTLEMENT MONUMENT, GEOCON SHALL IMMEDIATELY BE NOTIFIED.

SETTLEMENT MONUMENT

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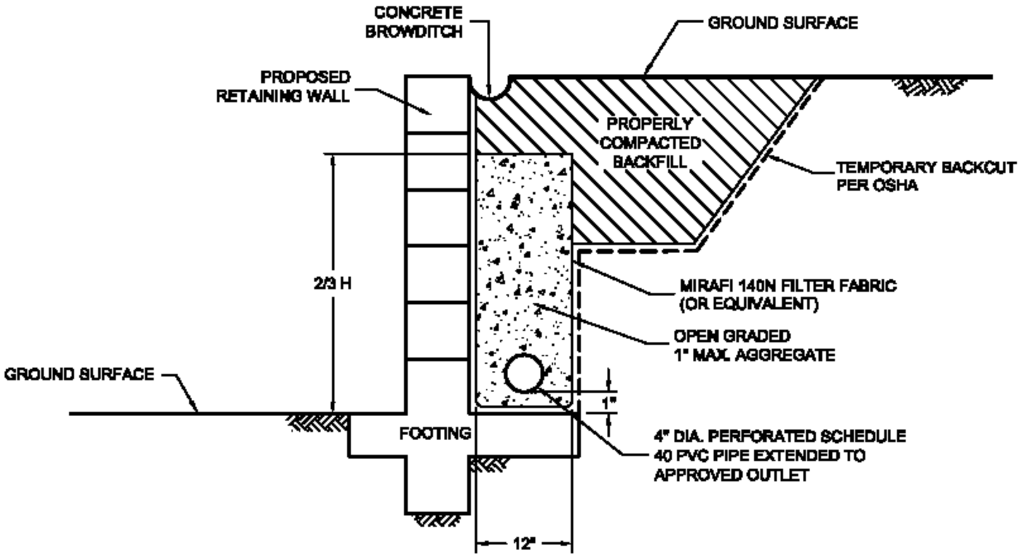
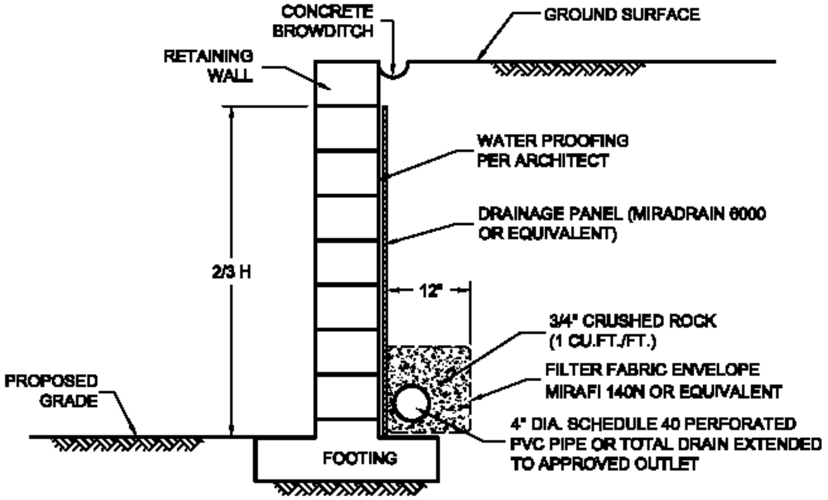
SAN DIEGO RIVER DISCOVERY CENTER
SAN DIEGO, CALIFORNIA

GC / AML		DSK/GTYPD	DATE 01 - 31 - 2014	PROJECT NO. G1656 - 42 - 01	FIG. 5





NOTE :

DRAIN SHOULD BE UNIFORMLY SLOPED TO GRAVITY OUTLET
OR TO A SUMP WHERE WATER CAN BE REMOVED BY PUMPING

NO SCALE

TYPICAL RETAINING WALL DRAIN DETAIL

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RWDD7

APPENDIX A

APPENDIX A

FIELD INVESTIGATION

We performed the field investigation on October 31, 2013, which consisted of drilling 3, small-diameter borings. The approximate locations of our exploratory borings are shown on the Site Plan, Figure 2. The borings were drilled to depths of approximately 31 to 44 feet below existing grade using truck-mounted drill rig equipped with 8-inch-diameter, hollow-stem augers. One boring (MW1) was finished as a groundwater monitoring well.

We obtained relatively undisturbed samples by driving a 3-inch-diameter, California Modified, split-tube sampler 12 inches into the undisturbed soil mass with blows from a hammer weighing 140 pounds, dropped from a height of 30 inches. The sampler was equipped with brass sampler rings to facilitate removal and testing of the soil. Bulk samples were also obtained.

The soil conditions encountered in the borings were visually examined, classified, and logged in general accordance with American Society for Testing and Materials (ASTM) practice for Description and Identification of Soils (Visual-Manual Procedure D 2488). Logs of borings are presented on Figures A-1 through A-4. The logs depict the soil and geologic conditions encountered and the depth at which samples were obtained.

PROJECT NO. G1656-42-01

BORING B 1

ELEV. (MSL.)_______ DATE COMPLETED **10-31-2013**

EQUIPMENT ____________________ BY: **N.G. BORJA**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0	B1-1	▨		SM/ML	**UNDOCUMENTED FILL** Loose, dry, light tan brown, Silty, fine SAND to Sandy SILT			
2				ML	Soft, damp, light tan, SILT; fine laminations			
	B1-2	■				12	88.9	15.5
4		▨						
	B1-3	■				15	84.8	19.0
6								
8	B1-4	■				5		
				CL	Soft, wet, mottled light olive and red brown, CLAY			
10	B1-5	■				10	76.5	34.5
	B1-6	▨						
12								
14				ML	Loose, moist, brown, Sandy SILT; trace clay			
	B1-7	■				7	77.5	34.0
16								
18								
20	B1-8	◧	▼			4		
22				SM	Loose, saturated, dark gray, fine to medium SAND; little silt			
24								
	B1-9	■				4	70.3	53.6
26								
28				CL	Soft, saturated, dark gray, Silty CLAY			

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

G1656-42-01.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. G1656-42-01

BORING B 1

ELEV. (MSL.)_______ DATE COMPLETED **10-31-2013**

EQUIPMENT ____________________ BY: **N.G. BORJA**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
30	B1-10					4		
				ML	**ALLUVIUM** Loose, saturated, dark gray, fine to coarse SAND; little silt			
32								
34								
	B1-11					6		
36								
38								
40	B1-12					PUSH 18"		
						50/5"		
42					-Hard drilling			
44					REFUSAL AT 44 FEET Groundwater encountered at 21 feet Backfilled with 14.3 ft^3 of bentonite slurry			

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

G1656-42-01.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. G1656-42-01

BORING B 2

ELEV. (MSL.)________ DATE COMPLETED 10-31-2013

EQUIPMENT ______________________ BY: N.G. BORJA

DEPTH IN FEET	SAMPLE NO.	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0			ML	**UNDOCUMENTED FILL** Soft, dry, light tan, Sandy SILT; fine laminations			
2							
4							
	■ B2-1				11		
6	▨ B2-2		CH	Soft, moist, light tan to tan, Silty CLAY			
8			ML	Soft, damp, light tan, Sandy SILT; fine laminations			
10	■ B2-3				10		
12							
			ML	Soft, damp, light brown, Sandy SILT; fine laminations			
14							
	■ B2-4				15		
16	▨ B2-5						
18							
			ML	Soft, wet, light olive, SILT; fine laminations			
20	■ B2-6				5	59.5	60.4
22		▼					
24							
	☐ B2-7				6		
26							
28							

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

G1656-42-01.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. G1656-42-01

BORING B 2

ELEV. (MSL.)_______ DATE COMPLETED **10-31-2013**

EQUIPMENT ____________________ BY: **N.G. BORJA**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
30	B2-8				-No recovery	7		
					BORING TERMINATED AT 31 FEET Groundwater encountered at 22 feet Backfilled with 9.8 ft^3 of bentonite slurry			

Figure A-2,
Log of Boring B 2, 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.

PROJECT NO. G1656-42-01

BORING MW 1

ELEV. (MSL.)_______ DATE COMPLETED 10-31-2013

EQUIPMENT **MARL M-5 (UNIMOG)** BY: **N.G. BORJA**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0	MW1-1			MH/CH	**UNDOCUMENTED FILL (Pond Tailings)** Firm, dry, light tan brown, Sandy to Clayey SILT			
				ML	Loose, moist, light tan brown, fine Sandy SILT; fine laminations			
2								
4								
	MW1-2						84.8	11.4
6								
8								
				CH	Soft, moist, mottled light tan brown and brown, Silty CLAY; highly plastic; (pp=1.5 tsf)			
10	MW1-3					9		
				ML	Soft, moist, light tan to yellowish brown, fine Sandy SILT; fine laminations			
12								
14								
	MW1-4					8		
16	MW1-5				-(pp=1.0 tsf)			
18								
20	MW1-6		▼			9		23.3
22								
24				SM	**ALLUVIUM** Loose, saturated, gray to yellowish brown, Silty, fine to medium SAND; gravel at 24 feet			
	MW1-7					7		
26								
28					-Becomes yellowish brown, fine- to coarse-grained; some silt			

G1656-42-01.GPJ

Figure A-3,
Log of Boring MW 1, 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.

PROJECT NO. G1656-42-01

BORING MW 1

ELEV. (MSL.)_______ DATE COMPLETED **10-31-2013**

EQUIPMENT **MARL M-5 (UNIMOG)** BY: **N.G. BORJA**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
30	MW1-8 ■					7	84.7	36.9
32								
				ML	Soft, saturated, olive brown to brown, Sandy SILT to Silty, fine SAND (pp=??			
34								
	MW1-9 ■					5		
36					BORING TERMINATED AT 36 FEET Groundwater encountered at 21 feet on 10-31-2013 Backfilled with 11.2 ft³ of bentonite slurry			

Figure A-3,
Log of Boring MW 1, Page 2 of 2

G1656-42-01.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 soil samples were tested for their: maximum dry density and optimum moisture content; expansion index (EI); shear strength; water-soluble sulfate; chloride ion; Atterberg Limits, swell/consolidation; and grain-size distribution. The results of our laboratory tests are presented on Tables B-I through B-VII and Figures B-1 through B-4. Laboratory testing from Geocon's 1998 investigation are included in Appendix C.

TABLE B-I
SUMMARY OF LABORATORY MAXIMUM DRY DENSITY AND OPTIMUM MOISTURE CONTENT TEST RESULTS
ASTM D 1557

Sample No.	Description	Maximum Dry Density (pcf)	Optimum Moisture Content (% dry wt.)
B1-1	Tan SILT, trace fine sand, trace clay	111.7	17.0
MW1-1	Tan SILT, trace fine sand, trace clay	111.9	15.0

TABLE B-II
SUMMARY OF LABORATORY EXPANSION INDEX TEST RESULTS
ASTM D 4829

Sample No.	Moisture Content (%)		Dry Density (pcf)	Expansion Index	Expansion Classification
	Before Test	After Test			
B1-1	14.0	31.5	96.5	101	High
B2-2	14.4	33.2	93.6	110	High

TABLE B-III
SUMMARY OF LABORATORY DIRECT SHEAR TEST RESULTS
ASTM D 3080

Sample No.	Dry Density (pcf)	Moisture Content (%)		Unit Cohesion (psf)	Angle of Shear Resistance (degrees)
		Initial	Final		
B2-4	72.2	38.0	48.8	144	25
MW1-4	83.0	16.8	32.6	185	26

TABLE B-IV
SUMMARY OF LABORATORY WATER-SOLUBLE SULFATE CALIFORNIA TEST NO. 417

Sample No.	Water-Soluble Sulfate (%)	Classification
B1-1	0.001	Negligible
MW1-1	0.001	Negligible

TABLE B-V
SUMMARY OF LABORATORY CHLORIDE ION CONTENT TEST RESULTS
AASHTO T 291

Sample No.	Chloride Ion Content (ppm)	Chloride Ion Content (%)
B1-1	111	0.011
MW1-1	226	0.023

TABLE B-VI
SUMMARY OF LABORATORY ATTERBERG LIMITS TEST RESULTS
ASTM D 4318

Sample No.	Liquid Limit	Plastic Limit	Plasticity Index
B1-10	61	29	32
MW1-9	50	27	23

TABLE B-VII
SUMMARY OF LABORATORY POTENTIAL OF HYDROGEN (PH) AND RESISTIVITY TEST RESULTS
CALIFORNIA TEST NO. 643

Sample No.	pH	Resistivity (ohm-centimeters)	Corrosive Rating
B1-1	8.3	890	Corrosive
MW1-1	8.4	1100	Corrosive

ADVANCED TECHNOLOGY
LABORATORIES

November 15, 2013

ACCREDITED IN ACCORDANCE WITH nelac
ELAP No.: 1838
NELAP No.: 02107CA
CSDLAC No.: 10196
ORELAP No.: CA300003
TCEQ No.: T104704502

Jon Layog
Geocon, Inc.
6960 Flanders Drive
San Diego, CA 92121
Tel: (858) 558-6100
Fax:(858) 558-8437

Re: ATL Work Order Number : 1303524
Client Reference : San Diego River Discovery Center, G1656-42-01

Enclosed are the results for sample(s) received on November 08, 2013 by Advanced Technology Laboratories. The sample(s) are tested for the parameters as indicated on the enclosed chain of custody in accordance with applicable laboratory certifications. The laboratory results contained in this report specifically pertains to the sample(s) submitted.

Thank you for the opportunity to serve the needs of your company. If you have any questions, please feel free to contact me or your Project Manager.

Sincerely,

Eddie Rodriguez
Laboratory Director

Certificate of Analysis

Geocon, Inc.	Project Number :	San Diego River Discovery Center, G165
6960 Flanders Drive	Report To :	Jon Layog
San Diego , CA 92121	Reported :	11/15/2013

SUMMARY OF SAMPLES

Sample ID	Laboratory ID	Matrix	Date Sampled	Date Received
MW1	1303524-01	Groundwater	11/08/13 0:00	11/08/13 14:00

Certificate of Analysis

Geocon, Inc.
6960 Flanders Drive
San Diego , CA 92121

Project Number : San Diego River Discovery Center, G165
Report To : Jon Layog
Reported : 11/15/2013

Client Sample ID MW1

Lab ID: 1303524-01

Gasoline Range Organics by EPA 8015B (Modified)

Analyst: SL

Analyte	Result (mg/L)	PQL (mg/L)	MDL (mg/L)	Dilution	Batch	Prepared	Date/Time Analyzed	Notes
Gasoline Range Organics	ND	0.05	NA	1	B3K0208	11/12/2013	11/12/13 14:04	
Surrogate: 4-Bromofluorobenzene	*105 %*	*70 - 130*			B3K0208	11/12/2013	*11/12/13 14:04*	

Diesel Range Organics by EPA 8015B

Analyst: CR

Analyte	Result (mg/L)	PQL (mg/L)	MDL (mg/L)	Dilution	Batch	Prepared	Date/Time Analyzed	Notes
DRO	**0.17**	0.05	NA	1	B3K0185	11/11/2013	11/11/13 18:47	
ORO	**0.20**	0.05	NA	1	B3K0185	11/11/2013	11/11/13 18:47	
Surrogate: p-Terphenyl	*80.7 %*	*38 - 151*			B3K0185	11/11/2013	*11/11/13 18:47*	

Volatile Organic Compounds by EPA 8260

Analyst: SL

Analyte	Result (ug/L)	PQL (ug/L)	MDL (ug/L)	Dilution	Batch	Prepared	Date/Time Analyzed	Notes
1,1,1,2-Tetrachloroethane	ND	0.50	NA	1	B3K0199	11/11/2013	11/11/13 21:54	
1,1,1-Trichloroethane	ND	0.50	NA	1	B3K0199	11/11/2013	11/11/13 21:54	
1,1,2,2-Tetrachloroethane	ND	0.50	NA	1	B3K0199	11/11/2013	11/11/13 21:54	
1,1,2-Trichloroethane	ND	0.50	NA	1	B3K0199	11/11/2013	11/11/13 21:54	
1,1-Dichloroethane	ND	0.50	NA	1	B3K0199	11/11/2013	11/11/13 21:54	
1,1-Dichloroethene	ND	0.50	NA	1	B3K0199	11/11/2013	11/11/13 21:54	
1,1-Dichloropropene	ND	0.50	NA	1	B3K0199	11/11/2013	11/11/13 21:54	
1,2,3-Trichloropropane	ND	0.50	NA	1	B3K0199	11/11/2013	11/11/13 21:54	
1,2,3-Trichlorobenzene	ND	0.50	NA	1	B3K0199	11/11/2013	11/11/13 21:54	
1,2,4-Trichlorobenzene	ND	0.50	NA	1	B3K0199	11/11/2013	11/11/13 21:54	
1,2,4-Trimethylbenzene	ND	0.50	NA	1	B3K0199	11/11/2013	11/11/13 21:54	
1,2-Dibromo-3-chloropropane	ND	0.50	NA	1	B3K0199	11/11/2013	11/11/13 21:54	
1,2-Dibromoethane	ND	0.50	NA	1	B3K0199	11/11/2013	11/11/13 21:54	
1,2-Dichlorobenzene	ND	0.50	NA	1	B3K0199	11/11/2013	11/11/13 21:54	
1,2-Dichloroethane	ND	0.50	NA	1	B3K0199	11/11/2013	11/11/13 21:54	
1,2-Dichloropropane	ND	0.50	NA	1	B3K0199	11/11/2013	11/11/13 21:54	
1,3,5-Trimethylbenzene	ND	0.50	NA	1	B3K0199	11/11/2013	11/11/13 21:54	
1,3-Dichlorobenzene	ND	0.50	NA	1	B3K0199	11/11/2013	11/11/13 21:54	
1,3-Dichloropropane	ND	0.50	NA	1	B3K0199	11/11/2013	11/11/13 21:54	
1,4-Dichlorobenzene	ND	0.50	NA	1	B3K0199	11/11/2013	11/11/13 21:54	
2,2-Dichloropropane	ND	0.50	NA	1	B3K0199	11/11/2013	11/11/13 21:54	
2-Chlorotoluene	ND	0.50	NA	1	B3K0199	11/11/2013	11/11/13 21:54	

Certificate of Analysis

Geocon, Inc.	Project Number :	San Diego River Discovery Center, G165
6960 Flanders Drive	Report To :	Jon Layog
San Diego , CA 92121	Reported :	11/15/2013

Client Sample ID MW1

Lab ID: 1303524-01

Volatile Organic Compounds by EPA 8260 **Analyst: SL**

Analyte	Result (ug/L)	PQL (ug/L)	MDL (ug/L)	Dilution	Batch	Prepared	Date/Time Analyzed	Notes
4-Chlorotoluene	ND	0.50	NA	1	B3K0199	11/11/2013	11/11/13 21:54	
4-Isopropyltoluene	ND	0.50	NA	1	B3K0199	11/11/2013	11/11/13 21:54	
Benzene	ND	0.50	NA	1	B3K0199	11/11/2013	11/11/13 21:54	
Bromobenzene	ND	0.50	NA	1	B3K0199	11/11/2013	11/11/13 21:54	
Bromodichloromethane	ND	0.50	NA	1	B3K0199	11/11/2013	11/11/13 21:54	
Bromoform	ND	0.50	NA	1	B3K0199	11/11/2013	11/11/13 21:54	
Bromomethane	ND	0.50	NA	1	B3K0199	11/11/2013	11/11/13 21:54	
Carbon tetrachloride	ND	0.50	NA	1	B3K0199	11/11/2013	11/11/13 21:54	
Chlorobenzene	ND	0.50	NA	1	B3K0199	11/11/2013	11/11/13 21:54	
Chloroethane	ND	0.50	NA	1	B3K0199	11/11/2013	11/11/13 21:54	
Chloroform	ND	0.50	NA	1	B3K0199	11/11/2013	11/11/13 21:54	
Chloromethane	ND	0.50	NA	1	B3K0199	11/11/2013	11/11/13 21:54	
cis-1,2-Dichloroethene	ND	0.50	NA	1	B3K0199	11/11/2013	11/11/13 21:54	
cis-1,3-Dichloropropene	ND	0.50	NA	1	B3K0199	11/11/2013	11/11/13 21:54	
Dibromochloromethane	ND	0.50	NA	1	B3K0199	11/11/2013	11/11/13 21:54	
Dibromomethane	ND	0.50	NA	1	B3K0199	11/11/2013	11/11/13 21:54	
Dichlorodifluoromethane	ND	0.50	NA	1	B3K0199	11/11/2013	11/11/13 21:54	
Ethylbenzene	ND	0.50	NA	1	B3K0199	11/11/2013	11/11/13 21:54	
Hexachlorobutadiene	ND	0.50	NA	1	B3K0199	11/11/2013	11/11/13 21:54	
Isopropylbenzene	ND	0.50	NA	1	B3K0199	11/11/2013	11/11/13 21:54	
m,p-Xylene	ND	1.0	NA	1	B3K0199	11/11/2013	11/11/13 21:54	
Methylene chloride	ND	1.0	NA	1	B3K0199	11/11/2013	11/11/13 21:54	
n-Butylbenzene	ND	0.50	NA	1	B3K0199	11/11/2013	11/11/13 21:54	
n-Propylbenzene	ND	0.50	NA	1	B3K0199	11/11/2013	11/11/13 21:54	
Naphthalene	ND	0.50	NA	1	B3K0199	11/11/2013	11/11/13 21:54	
o-Xylene	ND	0.50	NA	1	B3K0199	11/11/2013	11/11/13 21:54	
sec-Butylbenzene	ND	0.50	NA	1	B3K0199	11/11/2013	11/11/13 21:54	
Styrene	ND	0.50	NA	1	B3K0199	11/11/2013	11/11/13 21:54	
tert-Butylbenzene	ND	0.50	NA	1	B3K0199	11/11/2013	11/11/13 21:54	
Tetrachloroethene	ND	0.50	NA	1	B3K0199	11/11/2013	11/11/13 21:54	
Toluene	ND	0.50	NA	1	B3K0199	11/11/2013	11/11/13 21:54	
trans-1,2-Dichloroethene	ND	0.50	NA	1	B3K0199	11/11/2013	11/11/13 21:54	
Trichloroethene	ND	0.50	NA	1	B3K0199	11/11/2013	11/11/13 21:54	
Trichlorofluoromethane	ND	0.50	NA	1	B3K0199	11/11/2013	11/11/13 21:54	
Vinyl chloride	ND	0.50	NA	1	B3K0199	11/11/2013	11/11/13 21:54	
Surrogate: 1,2-Dichloroethane-d4	*115 %*		*70 - 130*		B3K0199	11/11/2013	*11/11/13 21:54*	
Surrogate: 4-Bromofluorobenzene	*75.9 %*		*70 - 130*		B3K0199	11/11/2013	*11/11/13 21:54*	

Certificate of Analysis

Geocon, Inc.	Project Number :	San Diego River Discovery Center, G165
6960 Flanders Drive	Report To :	Jon Layog
San Diego , CA 92121	Reported :	11/15/2013

Client Sample ID MW1

Lab ID: 1303524-01

Volatile Organic Compounds by EPA 8260 **Analyst: SL**

Analyte	Result (ug/L)	PQL (ug/L)	MDL (ug/L)	Dilution	Batch	Prepared	Date/Time Analyzed	Notes
Surrogate: Dibromofluoromethane	*106 %*	*70 - 130*			B3K0199	11/11/2013	*11/11/13 21:54*	
Surrogate: Toluene-d8	*97.2 %*	*70 - 130*			B3K0199	11/11/2013	*11/11/13 21:54*	

Certificate of Analysis

Geocon, Inc.
6960 Flanders Drive
San Diego , CA 92121

Project Number : San Diego River Discovery Center, G165
Report To : Jon Layog
Reported : 11/15/2013

QUALITY CONTROL SECTION

Gasoline Range Organics by EPA 8015B (Modified) - Quality Control

Analyte	Result (mg/L)	PQL (mg/L)	Spike Level	Source Result	% Rec	% Rec Limits	RPD	RPD Limit	Notes
Batch B3K0208 - GCVOAW									
Blank (B3K0208-BLK1)				Prepared: 11/12/2013 Analyzed: 11/12/2013					
Gasoline Range Organics	ND	0.05			NR				
Surrogate: 4-Bromofluorobenzene	*0.1053*		*0.100000*		*105*	*70 - 130*			
LCS (B3K0208-BS1)				Prepared: 11/12/2013 Analyzed: 11/12/2013					
Gasoline Range Organics	0.866000		1.00000		86.6	70 - 130			
Surrogate: 4-Bromofluorobenzene	*0.1054*		*0.100000*		*105*	*70 - 130*			
LCS Dup (B3K0208-BSD1)				Prepared: 11/12/2013 Analyzed: 11/12/2013					
Gasoline Range Organics	0.853000		1.00000		85.3	70 - 130	1.51	20	
Surrogate: 4-Bromofluorobenzene	*0.1101*		*0.100000*		*110*	*70 - 130*			
Duplicate (B3K0208-DUP1)		**Source: 1303532-01**		Prepared: 11/12/2013 Analyzed: 11/12/2013					
Gasoline Range Organics	ND	0.05		ND	NR			20	
Surrogate: 4-Bromofluorobenzene	*0.09674*		*0.100000*		*96.7*	*70 - 130*			

Certificate of Analysis

Geocon, Inc.		Project Number :	San Diego River Discovery Center, G165
6960 Flanders Drive		Report To :	Jon Layog
San Diego , CA 92121		Reported :	11/15/2013

Diesel Range Organics by EPA 8015B - Quality Control

Analyte	Result (mg/L)	PQL (mg/L)	Spike Level	Source Result	% Rec	% Rec Limits	RPD	RPD Limit	Notes
Batch B3K0185 - GCSEMI_DRO									
Blank (B3K0185-BLK1)				Prepared: 11/11/2013 Analyzed: 11/11/2013					
DRO	ND	0.05			NR				
ORO	ND	0.05			NR				
Surrogate: p-Terphenyl	*0.06562*		*8.00000E-2*		*82.0*	*38 - 151*			
LCS (B3K0185-BS1)				Prepared: 11/11/2013 Analyzed: 11/11/2013					
DRO	0.763540	0.05	1.00000		76.4	50 - 121			
Surrogate: p-Terphenyl	*0.06241*		*8.00000E-2*		*78.0*	*38 - 151*			
LCS Dup (B3K0185-BSD1)				Prepared: 11/11/2013 Analyzed: 11/11/2013					
DRO	0.760920	0.05	1.00000		76.1	50 - 121	0.344	20	
Surrogate: p-Terphenyl	*0.06345*		*8.00000E-2*		*79.3*	*38 - 151*			

Volatile Organic Compounds by EPA 8260 - Quality Control

Batch B3K0199 - MSVOAW_LL

Blank (B3K0199-BLK1) Prepared: 11/11/2013 Analyzed: 11/11/2013

Analyte	Result (ug/L)	PQL (ug/L)	Spike Level	Source Result	% Rec	% Rec Limits	RPD	RPD Limit	Notes
1,1,1,2-Tetrachloroethane	ND	0.50			NR				
1,1,1-Trichloroethane	ND	0.50			NR				
1,1,2,2-Tetrachloroethane	ND	0.50			NR				
1,1,2-Trichloroethane	ND	0.50			NR				
1,1-Dichloroethane	ND	0.50			NR				
1,1-Dichloroethene	ND	0.50			NR				
1,1-Dichloropropene	ND	0.50			NR				
1,2,3-Trichloropropane	ND	0.50			NR				
1,2,3-Trichlorobenzene	ND	0.50			NR				
1,2,4-Trichlorobenzene	ND	0.50			NR				
1,2,4-Trimethylbenzene	ND	0.50			NR				
1,2-Dibromo-3-chloropropane	ND	0.50			NR				
1,2-Dibromoethane	ND	0.50			NR				
1,2-Dichlorobenzene	ND	0.50			NR				
1,2-Dichloroethane	ND	0.50			NR				
1,2-Dichloropropane	ND	0.50			NR				
1,3,5-Trimethylbenzene	ND	0.50			NR				
1,3-Dichlorobenzene	ND	0.50			NR				
1,3-Dichloropropane	ND	0.50			NR				
1,4-Dichlorobenzene	ND	0.50			NR				
2,2-Dichloropropane	ND	0.50			NR				
2-Chlorotoluene	ND	0.50			NR				
4-Chlorotoluene	ND	0.50			NR				
4-Isopropyltoluene	ND	0.50			NR				
Benzene	ND	0.50			NR				
Bromobenzene	ND	0.50			NR				
Bromodichloromethane	ND	0.50			NR				
Bromoform	ND	0.50			NR				
Bromomethane	ND	0.50			NR				
Carbon tetrachloride	ND	0.50			NR				
Chlorobenzene	ND	0.50			NR				
Chloroethane	ND	0.50			NR				
Chloroform	ND	0.50			NR				
Chloromethane	ND	0.50			NR				
cis-1,2-Dichloroethene	ND	0.50			NR				
cis-1,3-Dichloropropene	ND	0.50			NR				
Dibromochloromethane	ND	0.50			NR				
Dibromomethane	ND	0.50			NR				
Dichlorodifluoromethane	ND	0.50			NR				
Ethylbenzene	ND	0.50			NR				
Hexachlorobutadiene	ND	0.50			NR				

Volatile Organic Compounds by EPA 8260 - Quality Control (cont'd)

Analyte	Result (ug/L)	PQL (ug/L)	Spike Level	Source Result	% Rec	% Rec Limits	RPD	RPD Limit	Notes
Batch B3K0199 - MSVOAW_LL (continued)									
Blank (B3K0199-BLK1) - Continued				Prepared: 11/11/2013 Analyzed: 11/11/2013					
Isopropylbenzene	ND	0.50			NR				
m,p-Xylene	ND	1.0			NR				
Methylene chloride	ND	1.0			NR				
n-Butylbenzene	ND	0.50			NR				
n-Propylbenzene	ND	0.50			NR				
Naphthalene	ND	0.50			NR				
o-Xylene	ND	0.50			NR				
sec-Butylbenzene	ND	0.50			NR				
Styrene	ND	0.50			NR				
tert-Butylbenzene	ND	0.50			NR				
Tetrachloroethene	ND	0.50			NR				
Toluene	ND	0.50			NR				
trans-1,2-Dichloroethene	ND	0.50			NR				
Trichloroethene	ND	0.50			NR				
Trichlorofluoromethane	ND	0.50			NR				
Vinyl chloride	ND	0.50			NR				
Surrogate: 1,2-Dichloroethane-d4	*29.47*		*25.0000*		*118*	*70 - 130*			
Surrogate: 4-Bromofluorobenzene	*20.23*		*25.0000*		*80.9*	*70 - 130*			
Surrogate: Dibromofluoromethane	*26.17*		*25.0000*		*105*	*70 - 130*			
Surrogate: Toluene-d8	*25.90*		*25.0000*		*104*	*70 - 130*			
LCS (B3K0199-BS1)				Prepared: 11/11/2013 Analyzed: 11/11/2013					
1,1-Dichloroethene	18.0900		20.0000		90.4	70 - 130			
Benzene	40.5300		40.0000		101	70 - 130			
Chlorobenzene	18.9400		20.0000		94.7	70 - 130			
MTBE	21.4800		20.0000		107	70 - 130			
Toluene	38.5000		40.0000		96.2	70 - 130			
Trichloroethene	18.0900		20.0000		90.4	70 - 130			
Surrogate: 1,2-Dichloroethane-d4	*29.11*		*25.0000*		*116*	*70 - 130*			
Surrogate: 4-Bromofluorobenzene	*18.55*		*25.0000*		*74.2*	*70 - 130*			
Surrogate: Dibromofluoromethane	*24.08*		*25.0000*		*96.3*	*70 - 130*			
Surrogate: Toluene-d8	*22.66*		*25.0000*		*90.6*	*70 - 130*			
LCS Dup (B3K0199-BSD1)				Prepared: 11/11/2013 Analyzed: 11/11/2013					
1,1-Dichloroethene	19.0100		20.0000		95.0	70 - 130	4.96	20	
Benzene	41.1600		40.0000		103	70 - 130	1.54	20	
Chlorobenzene	19.2000		20.0000		96.0	70 - 130	1.36	20	
MTBE	22.4600		20.0000		112	70 - 130	4.46	20	
Toluene	39.1600		40.0000		97.9	70 - 130	1.70	20	
Trichloroethene	18.2500		20.0000		91.2	70 - 130	0.881	20	
Surrogate: 1,2-Dichloroethane-d4	*28.97*		*25.0000*		*116*	*70 - 130*			

Volatile Organic Compounds by EPA 8260 - Quality Control (cont'd)

Analyte	Result (ug/L)	PQL (ug/L)	Spike Level	Source Result	% Rec	% Rec Limits	RPD	RPD Limit	Notes
Batch B3K0199 - MSVOAW_LL (continued)									
LCS Dup (B3K0199-BSD1) - Continued				Prepared: 11/11/2013 Analyzed: 11/11/2013					
Surrogate: 4-Bromofluorobenzene	*18.39*		*25.0000*		*73.6*	*70 - 130*			
Surrogate: Dibromofluoromethane	*24.40*		*25.0000*		*97.6*	*70 - 130*			
Surrogate: Toluene-d8	*22.56*		*25.0000*		*90.2*	*70 - 130*			

Certificate of Analysis

Geocon, Inc.	Project Number :	San Diego River Discovery Center, G165
6960 Flanders Drive	Report To :	Jon Layog
San Diego , CA 92121	Reported :	11/15/2013

Notes and Definitions

ND	Analyte is not detected at or above the Practical Quantitation Limit (PQL). When client requests quantitation against MDL, analyte is not detected at or above the Method Detection Limit (MDL)
PQL	Practical Quantitation Limit
MDL	Method Detection Limit
NR	Not Reported
RPD	Relative Percent Difference
CA1	CA-NELAP (CDPH)
CA2	CA-ELAP (CDPH)
OR1	OR-NELAP (OSPHL)
TX1	TX-NELAP (TCEQ)

Notes:
(1) The reported MDL and PQL are based on prep ratio variation and analytical dilution.
(2) The suffix [2C] of specific analytes signifies that the reported result is taken from the instrument's second column.

ADVANCED TECHNOLOGY
LABORATORIES
3275 Walnut Ave., Signal Hill, CA 90755
Tel: (562) 989-4045 • Fax: (562) 989-4040

CHAIN OF CUSTODY RECORD

Page 1 of 1

Instruction: Complete all shaded areas.

For Laboratory Use Only — ATLCOC Ver: 20130715

Method of Transport	Condition	Y	N	Condition	Y	N
☐ Client ☐ ATL	1. CHILLED	☐	☐	5. # OF SAMPLES MATCH COC	☐	☐
☐ FedEx ☐ OnTrac	2. HEADSPACE (VOA)	☐	☐	6. PRESERVED	☐	☐
☐ GSO	3. CONTAINER INTACT	☐	☐	7. COOLER TEMP, deg C:		
☐ Other: ______	4. SEALED	☐	☐			

CUSTOMER

Company: Geocon Consultants, Inc.
Address: 6960 Flanders Drive
City: San Diego State: CA Zip: 92121
Tel: (858) 558-6100
Fax: (858) 558-8437

SEND REPORT TO:
Attn: Email:
Company: Geocon Consultants, Inc.
Address: 6960 Flanders Drive
City: San Diego State: CA Zip: 92121

SEND INVOICE TO: ☒ same as SEND REPORT TO
Attn: Email:
Company:
Address:
City: State: Zip:

PROJECT SAMPLES

Project Name:
Quote No:
Special Instructions/Comments:
Project No.:
PO #:
Sampler:

QA/QC
☐ Routine
☐ Caltrans
☐ Legal
☐ RWQCB
☐ Level IV

ITEM	Lab No.	Sample ID / Location	Date	Time	8260 / 624 (Volatiles)	8015(GRO)	8015(DRO) & ORO	8270(Semi-volatiles)	8081(Organochlorine Pesticides)	8082(PCBs)	6010 / 7000(Title 22 Metals)	TO-15	SOIL / SEDIMENT / SLUDGE	SOLIDS / WIPE / FILTER	[✓ WATER] - DRINKING / [✓ GROUND]	WATER - STORM / WASTE	AQUEOUS / LAYERED - OIL	TAT	#	Type: 1=Tube; 2=VOA; 3=Liter; 4=Pint; 5=Jar, 6=Tedlar, 7=Canister	Material: 1=Glass; 2=Plastic; 3=Metal	Preservative: 1=HCl; 2=HNO3; 3=H2SO4; 4=4C; 5=Zn((Ac)2; 6=NaOH; 7=NA2S2O3	REMARKS
1	1303524-1	MW1			X	X												5	6	2	1	1/4	
2	1	MW1					X											5	1	3	1	4	
3																							
4																							
5																							
6																							
7																							
8																							
9																							
10																							

TERMS

1. Sample receiving hours: 7:30 AM to 7:30 PM Monday - Friday; Saturday 8:00 AM to 12:00 PM.
2. Samples Submitted AFTER 3:00 PM, are considered received the following Business day at 8:00 AM.
3. The following turnaround time conditions apply:
   - TAT = 0 : 300% Surcharge SAME BUSINESS DAY if received by 9:00 AM
   - TAT = 1 . 100% Surcharge NEXT BUSINESS DAY (COB 5:00 PM)
   - TAT = 2 : 50% Surcharge 2ND BUSINESS DAY (COB 5:00 PM)
   - TAT = 3 : 30% Surcharge 3RD BUSINESS DAY (COB 5:00 PM)
   - TAT = 4 : 20% Surcharge 4TH BUSINESS DAY (COB 5:00 PM)
   - TAT = 5 : NO SURCHARGE 5th BUSINESS DAY (COB 5:00 PM)
4. Weekend, holiday, after-hours work - ask for quote.
5. Subcontract TAT is 10 - 15 business days. Projects requiring shorter TATs will incur a surcharge respective to the subcontract lab --- ask for quote
6. Liquid and solid samples will be disposed of after 45 calendar days from receipt of samples; air samples will be disposed of after 14 calendar days after receipt of samples.
7. Electronic records maintained for five (5) years from report date.
8. Hard copy reports will be disposed of after 45 calendar days from report date.
9. Storage and Report Fees:
   - Liquid & solid samples: complimentary storage for forty five (45) calendar days from receipt of samples, $2/sample/month if extended storage or hold is requested.
   - Air samples: Complimentary storage for ten (10) calendar days from receipt of samples; $20/ sample/week if extended storage is requested.
   - Hard copy and regenerated reports/EDDs: $17 50 per hard copy report requested; $50.00 per regenerated/reformatted report; $35 per reprocessed EDD.
10. Rush TCLP/STLC samples: add 2 days to analysis TAT for extraction procedure.
11. Unanalyzed samples will incur a disposal fee of $7 per sample.

As the authorized agent of the company above, I hereby purchase laboratory services from ATL as shown above and hereby guarantee payment as quoted.

JON LAYOG
Submitter Print Name
Signature

CUSTODY

Relinquished by: (Signature and Printed Name)	Date:	Time:	Received by: (Signature and Printed Name)	Date:	Time:
JON LAYOG	11/8/13	2:00	Edward Rodriguez	11-8-13	14:00
Edward Rodriguez	11-8-13	16:45		11/8/13	1645

APPENDIX C

APPENDIX C

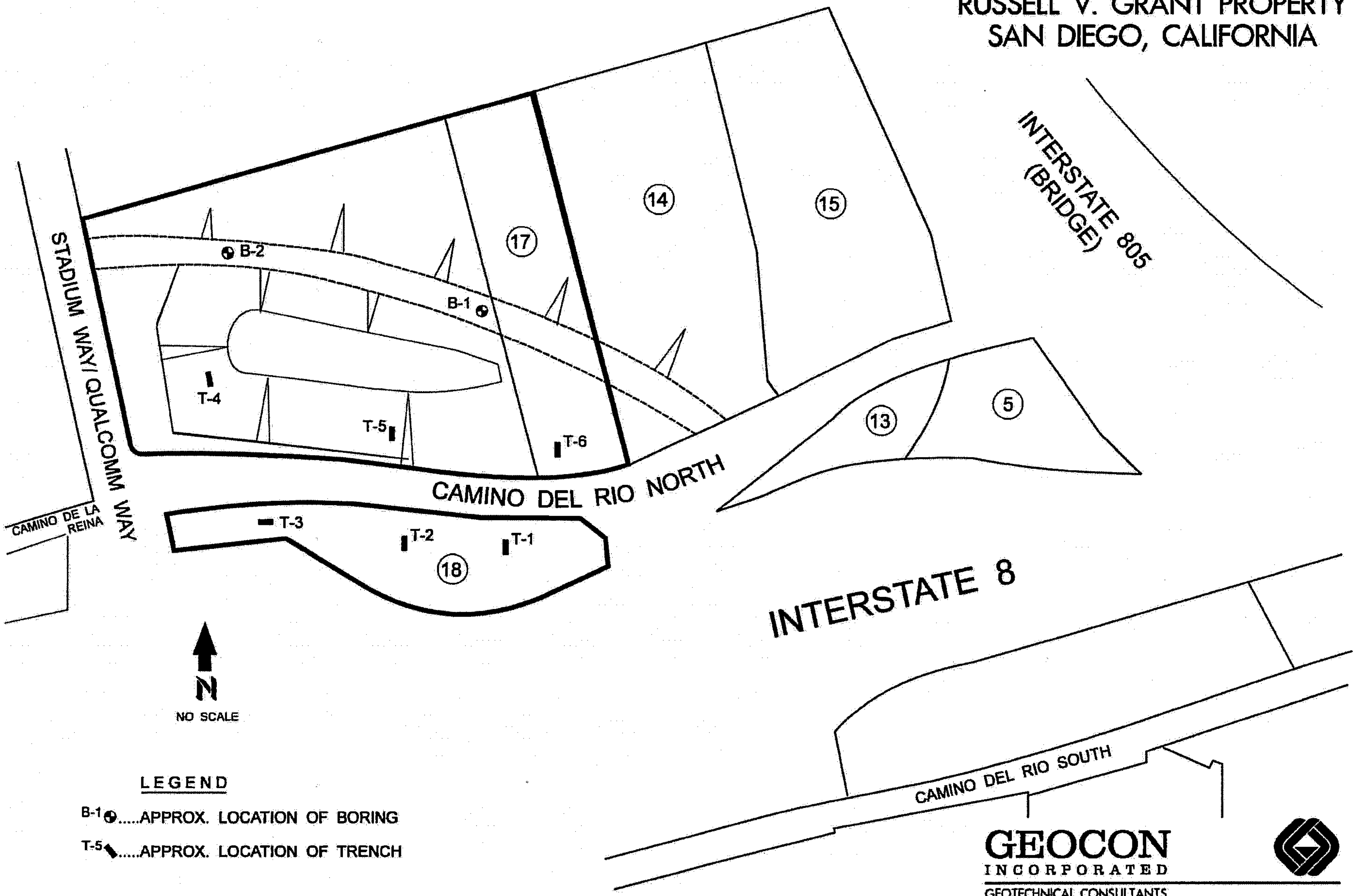
BORING AND TRENCH LOGS AND LABORATORY TESTING FROM GEOCON'S 1998 REPORT

FOR

SAN DIEGO RIVER DISCOVERY CENTER
SAN DIEGO, CALIFORNIA

PROJECT NO. G1656-42-01

RUSSELL V. GRANT PROPERTY
SAN DIEGO, CALIFORNIA



LEGEND

B-1 ⊕.....APPROX. LOCATION OF BORING

T-5 ▮.....APPROX. LOCATION OF TRENCH

GEOCON
INCORPORATED

GEOTECHNICAL CONSULTANTS
6960 FLANDERS DRIVE - SAN DIEGO, CALIFORNIA 92121-2974
PHONE 619 558-6900 - FAX 619 558-6159

PROJECT NO. 06060 - 22 - 01
SITE PLAN
FIGURE 2
DATE 04-13-1998

6060KB

PROJECT NO. 06060-22-01

BORING B 1

ELEV. (MSL.) 50 DATE COMPLETED 3/12/98

EQUIPMENT CME 85 mud-rotary

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0 2 4 6	B1-1			SP	**UNDOCUMENTED FILL** Moderately dense, moist, orange-brown, fine to coarse SAND, trace gravel and clay	14		
8 10 12	B1-2			SC	Dense, most, dark brown, Clayey, fine to coarse SAND, trace gravel -Cobbles at 12 to 13 feet	34	129.8	4.8
14 16	B1-3			SM	Dense, moist, dark brown to black, Silty, fine to very coarse SAND, trace gravel -Cobbles at 16.5 to 18 feet	33		
18 20 22	B1-4			SP	**ALLUVIUM** Moderately dense, moist, brown, fine to coarse SAND, trace gravel and silt	18	110.1	11.6
24 26	B1-5			SM	Loose, moist, gray to brown, Silty, very fine SAND, trace clay	10		
28				CL	Stiff, moist, black, very fine Sandy CLAY, trace cobbles and organics			

Figure A-1 Log of Boring B 1, page 1 of 2

RGEP

SAMPLE SYMBOLS

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

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

PROJECT NO. 06060-22-01

BORING B 1

ELEV. (MSL.) 50 DATE COMPLETED 3/12/98

EQUIPMENT CME 85 mud-rotary

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
30	B1-6			CL		10	116.7	17.8
32								
34					Soft, moist, yellow-brown to gray, Silty CLAY, trace gravel			
	B1-7			CH		4		
36								
38					Very dense, wet, gray-brown, Silty, medium to coarse SAND, trace gravel and cobbles			
40	B1-8			SM		70/7"		
42					-Cobbles from 40 to 45 feet			
44								
46	B1-9					85		
					BORING TERMINATED AT 46.5 FEET Water level undetermined (mud-rotary)			

Figure A-2 Log of Boring B 1, page 2 of 2

RGEP

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

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

PROJECT NO. 06060-22-01

BORING B 2

ELEV. (MSL.) 48 DATE COMPLETED 3/12/98

EQUIPMENT CME 85 mud-rotary

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0					**UNDOCUMENTED FILL** Moderately dense, moist, orange-brown, medium to coarse SAND interbedded with gray-brown, Silty CLAY, trace cobbles			
2	B2-1			SW/CL		22	110.9	13.4
4					Stiff, moist, brown, Sandy SILT and gray Clayey SILT interbedded			
	B2-2					13		
6								
8				CL/ML				
10	B2-3					39	74.0	44.4
12					Moderately dense, moist, brown, fine to coarse Silty SAND, trace clay			
14				SM				
	B2-4					16		
16								
18					**ALLUVIUM** Loose to moderately dense, moist, gray-brown, Clayey, fine to coarse SAND with gravel			
20	B2-5			SC		11		
22								
	B2-6					7		
24								
	B2-7					14		
26								
28				SM	Medium dense, moist, brown, Silty, very fine SAND			

Figure A-3 Log of Boring B 2, page 1 of 2

RGEP

SAMPLE SYMBOLS

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

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

PROJECT NO. 06060-22-01

BORING B 2

ELEV. (MSL.) 48 DATE COMPLETED 3/12/98

EQUIPMENT CME 85 mud-rotary

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
30	B2-8			CH	Stiff, moist, yellow-brown to brown, Silty CLAY	10	79.4	43.4
32								
34	B2-9			SM	Loose, wet, gray, fine to coarse Silty SAND	6		
36								
38	B2-10					7		
				CL	Stiff, wet, brown, Silty CLAY, trace very fine sand			
40	B2-11			SW-SM	Loose, wet, brown, Silty, medium to coarse SAND	8	107.8	20.3
42								
44	B2-12			SM	Dense, wet, gray-brown, Silty, very fine to fine SAND, trace cobbles -Cobbles from 43 to 54 feet	43		
46								
48								
50	B2-13			SP	Very dense, wet, brown, medium to coarse SAND and gravel, trace cobbles and clay	50/4"		
52								
54					BORING TERMINATED AT 54 FEET Water level undetermined (mud-rotary)			

Figure A-4 Log of Boring B 2, page 2 of 2

RGEP

SAMPLE SYMBOLS

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

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

PROJECT NO. 06060-22-01

TRENCH T 1

ELEV. (MSL.) 53 DATE COMPLETED 3/12/98

EQUIPMENT JD 710D BACKHOE

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0 2	T1-1 ▨			SM	**UNDOCUMENTED FILL** Medium loose, moist to very moist, orange-brown, slightly Clayey, Silty, fine to coarse SAND, with subrounded gravel, cobble and occasional rock up to 15 inches			
4 6	T1-2 ▨			CL	Medium soft, very moist, dark brown, ~~Silty~~ CLAY, medium plasticity -Becomes greenish, sandy at 3.5 feet -Becomes grayish brown at 4.5 feet -Cave in at 5 feet			
8 10 12 14			▽	SP	Medium dense, very moist, green-brown, Silty, fine to very coarse SAND, with abundant rounded to subrounded gravel and cobble -Becomes very moist to saturated orange at 13 feet -Seepage at 15.5 feet			
16					TRENCH TERMINATED AT 16 FEET REFUSAL ON CONCRETE PIPE OR CONCRETE SLAB			

Figure A-1, Log of Trench T 1

RGEP

SAMPLE SYMBOLS

☐ ... SAMPLING UNSUCCESSFUL

◧ ... STANDARD PENETRATION TEST

■ ... DRIVE SAMPLE (UNDISTURBED)

▨ ... DISTURBED OR BAG SAMPLE

◪ ... CHUNK SAMPLE

▼ ... WATER TABLE OR SEEPAGE

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

PROJECT NO. 06060-22-01

TRENCH T 2

ELEV. (MSL.) 54 DATE COMPLETED 3/12/98

EQUIPMENT JD 710D BACKHOE

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0 2 4 6 8 10				SM	**UNDOCUMENTED FILL** Medium loose, very moist, brown, Silty, fine to very coarse SAND with abundant subrounded gravel and cobble -Becomes dark brown at 3.5 feet -Becomes greenish brown at 4.0 feet			
12 14	T2-1 ▨			SM	**ALLUVIUM** Medium dense, moist, yellow-green, very Silty, very fine to fine SAND -Pieces of concrete with approximate dimension of 2 feet - very slow digging at 12.5 feet			
					TRENCH TERMINATED AT 15 FEET REFUSAL ON CONCRETE RUBBLE *CONCRETE PIECES WERE PROBABLY PLACED TO STABILIZED ALLUVIUM			

Figure A-2, Log of Trench T 2

RGEP

SAMPLE SYMBOLS

☐ ... SAMPLING UNSUCCESSFUL

▨ ... DISTURBED OR BAG SAMPLE

◧ ... STANDARD PENETRATION TEST

◪ ... CHUNK SAMPLE

■ ... DRIVE SAMPLE (UNDISTURBED)

▼ ... WATER TABLE OR SEEPAGE

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

PROJECT NO. 06060-22-01

TRENCH T 3

ELEV. (MSL.) 51 DATE COMPLETED 3/12/98

EQUIPMENT JD 710D BACKHOE

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0	T3-1 ▨				**UNDOCUMENTED FILL** Medium dense, moist, green-brown, Silty, fine to very coarse SAND with abundant subrounded gravel and cobble			
2								
				SM	-Become loose, dark brown at 3.0 feet			
4								
					-Becomes medium loose to medium dense, olive brown at 5.0 feet			
6								
8								
10					-Becomes dense at 10 feet -Becomes olive brown to orange brown at 12 feet			
12								
14	T3-2 ◣			SM	**ALLUVIUM** Medium dense, moist, yellow, very Silty, very fine to fine SAND with roots			
16								
18					TRENCH TERMINATED AT 18 FEET			

Figure A-3, Log of Trench T 3

RGEP

SAMPLE SYMBOLS

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

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

PROJECT NO. 06060-22-01

TRENCH T 4

ELEV. (MSL.) 42 DATE COMPLETED 3/12/98

EQUIPMENT JD 710D BACKHOE

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0 2 4 6 8 10	▨ T4-1			SM	**UNDOCUMENTED FILL** Loose, very moist, yellow-brown to olive-brown, Silty, fine to coarse SAND with few gravels and cobbles			
12 14 16	◣ T4-2			ML	**ALLUVIUM** Medium dense, very moist to saturated, yellow, Clayey SILT, medium plasticity			
					TRENCH TERMINATED AT 17 FEET			

Figure A-4, Log of Trench T 4

RGEP

SAMPLE SYMBOLS

☐ ... SAMPLING UNSUCCESSFUL

▨ ... DISTURBED OR BAG SAMPLE

◧ ... STANDARD PENETRATION TEST

◣ ... CHUNK SAMPLE

■ ... DRIVE SAMPLE (UNDISTURBED)

▼ ... WATER TABLE OR SEEPAGE

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

PROJECT NO. 06060-22-01

TRENCH T 5

ELEV. (MSL.) 42 DATE COMPLETED 3/12/98

EQUIPMENT JD 710D BACKHOE

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0	T5-1 ☒			MH	**ALLUVIUM** Loose, very moist, olive brown, Clayey SILT, medium to high plasticity abundant surficial roots -Becomes slightly sandy, less sandy at 2 feet -Becomes loose to medium loose at 5 feet			
2								
4								
6								
8								
10								
12				CH	Medium stiff, very moist, tan brown, Silty CLAY, high plasticity -Becomes yellow at 14 feet			
14								
16				SM	Medium dense, very moist, gray-yellow, Silty, very fine to fine SAND			

TRENCH TERMINATED AT 17 FEET

Figure A-5, Log of Trench T 5

RGEP

SAMPLE SYMBOLS

☐ ... SAMPLING UNSUCCESSFUL

☒ ... DISTURBED OR BAG SAMPLE

◧ ... STANDARD PENETRATION TEST

◩ ... CHUNK SAMPLE

■ ... DRIVE SAMPLE (UNDISTURBED)

▼ ... WATER TABLE OR SEEPAGE

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

PROJECT NO. 06060-22-01

TRENCH T 6

ELEV. (MSL.) 39 DATE COMPLETED 3/12/98

EQUIPMENT JD 710D BACKHOE

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0 2 4 6 8		▨		ML	**ALLUVIUM** Loose, very moist to saturated, brown, Clayey SILT medium plasticity -Becomes medium loose, very moist at 3 feet -Becomes damp, medium loose to medium dense at 7 feet			
10 12 14 16	T6-1 ◣	▨		CH	Stiff, moist to very moist, light brown, CLAY, medium to high plasticity			
					TRENCH TERMINATED AT 17 FEET			

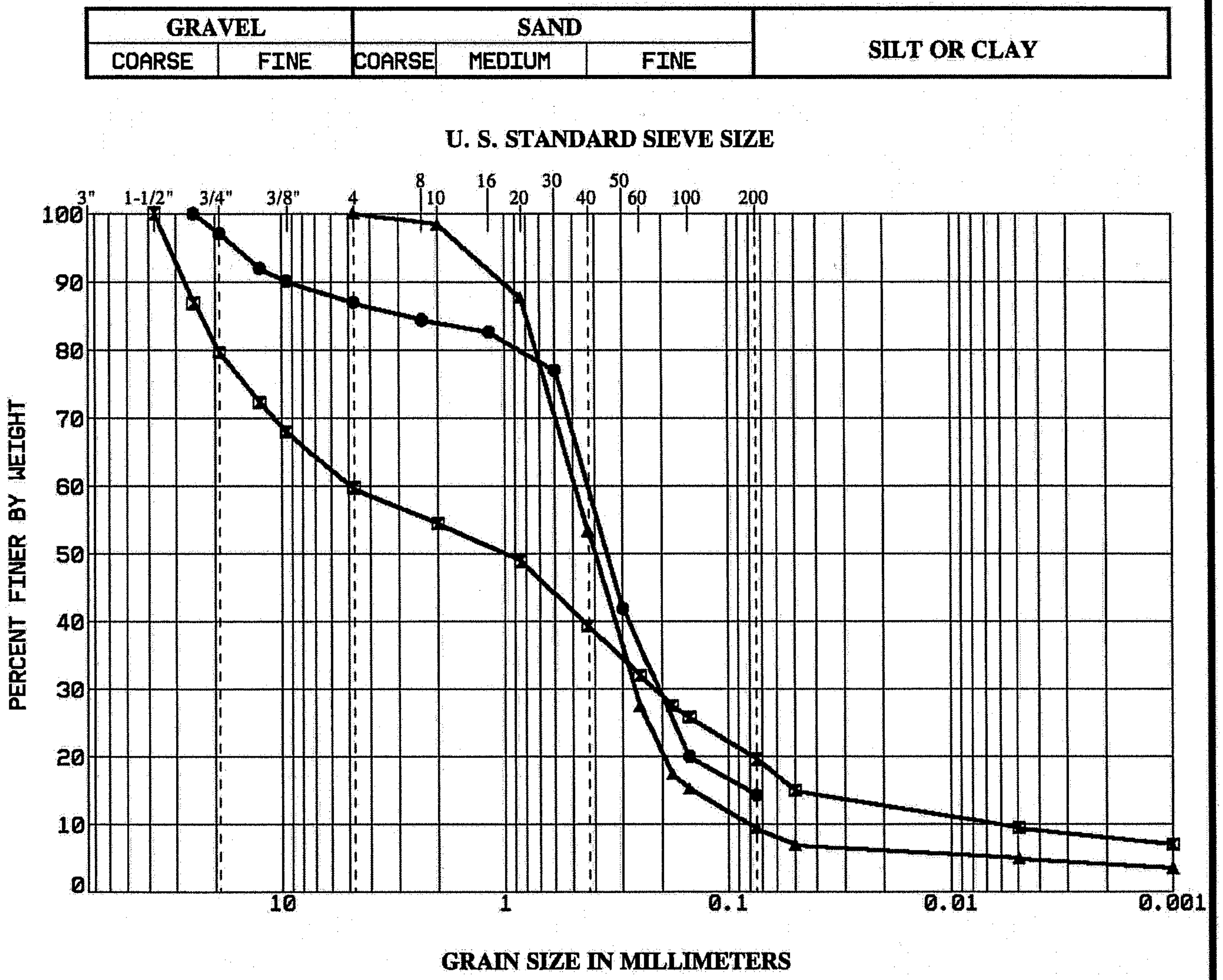
Figure A-6, Log of Trench T 6

RGEP

SAMPLE SYMBOLS			
	☐ ... SAMPLING UNSUCCESSFUL	◧ ... STANDARD PENETRATION TEST	■ ... DRIVE SAMPLE (UNDISTURBED)
	▩ ... DISTURBED OR BAG SAMPLE	◣ ... CHUNK SAMPLE	▼ ... WATER TABLE OR SEEPAGE

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

PROJECT NO. 06060-22-01



	SAMPLE	Depth (ft)	CLASSIFICATION	NAT WC	LL	PL	PI
●	B1-5	25.0	(SM) Silty SAND				
☒	B2-6	22.5	(SC) Clayey SAND with gravel				
▲	B2-11	40.0	(SW-SM) SAND with trace of silt	20.3			

GRADATION CURVE

RUSSELL V. GRANT ESTATE PROPERTY

SAN DIEGO, CALIFORNIA

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 soil samples were tested for their in-place dry density and moisture content, grain size distribution, maximum dry density and optimum moisture content, expansion potential and shear strength characteristics.

The results of our laboratory tests are presented on Tables B-I through B-III and Figure B-1. The in-place dry density and moisture content results are indicated on the exploratory boring logs.

TABLE B-I
SUMMARY OF IN-PLACE DENSITY AND DIRECT SHEAR TEST RESULTS

Sample No.	Dry Density (pcf)	Moisture Content (%)	Unit Cohesion (psf)	Angle of Shear Resistance (degrees)
B1-2	129.8	4.8		
B1-4	110.1	11.6		
B1-6	116.7	17.8		
B2-1	110.9	13.4		
B2-3	74.0	44.4	950	43
B2-8	79.4	43.4		
B2-11	107.8	20.3		

TABLE B-II
SUMMARY OF LABORATORY EXPANSION INDEX TEST RESULTS

Sample No.	Moisture Content		Dry Density (pcf)	Expansion Index
	Before Test (%)	After Test (%)		
T2-1	13.6	34.0	97.9	24
T5-1	15.9	52.2	91.1	239

TABLE B-III
SUMMARY OF LABORATORY MAXIMUM DRY DENSITY AND OPTIMUM MOISTURE CONTENT TEST RESULTS
ASTM D-1557-91

Sample No.	Description	Maximum Dry Density(pcf)	Optimum Moisture Content (% dry wt.)
T4-1	Olive brown, Silty, fine to coarse SAND	120.0	12.0

APPENDIX D

APPENDIX D

LIQUEFACTION AND CYCLIC STRAIN SOFTENING ANALYSIS

FOR

SAN DIEGO RIVER DISCOVERY CENTER
SAN DIEGO, CALIFORNIA

PROJECT NO. G1656-42-01

GEOCON

Client: San Diego Discovery Park
File No. G1656-42-01
Boring: B-2 (1998 Boring)

LIQUEFACTION SETTLEMENT ANALYSIS
AMERICAN SOCIETY OF CIVIL ENGINEERS

(SATURATED SAND AT INITIAL LIQUEFACTION CONDITION)

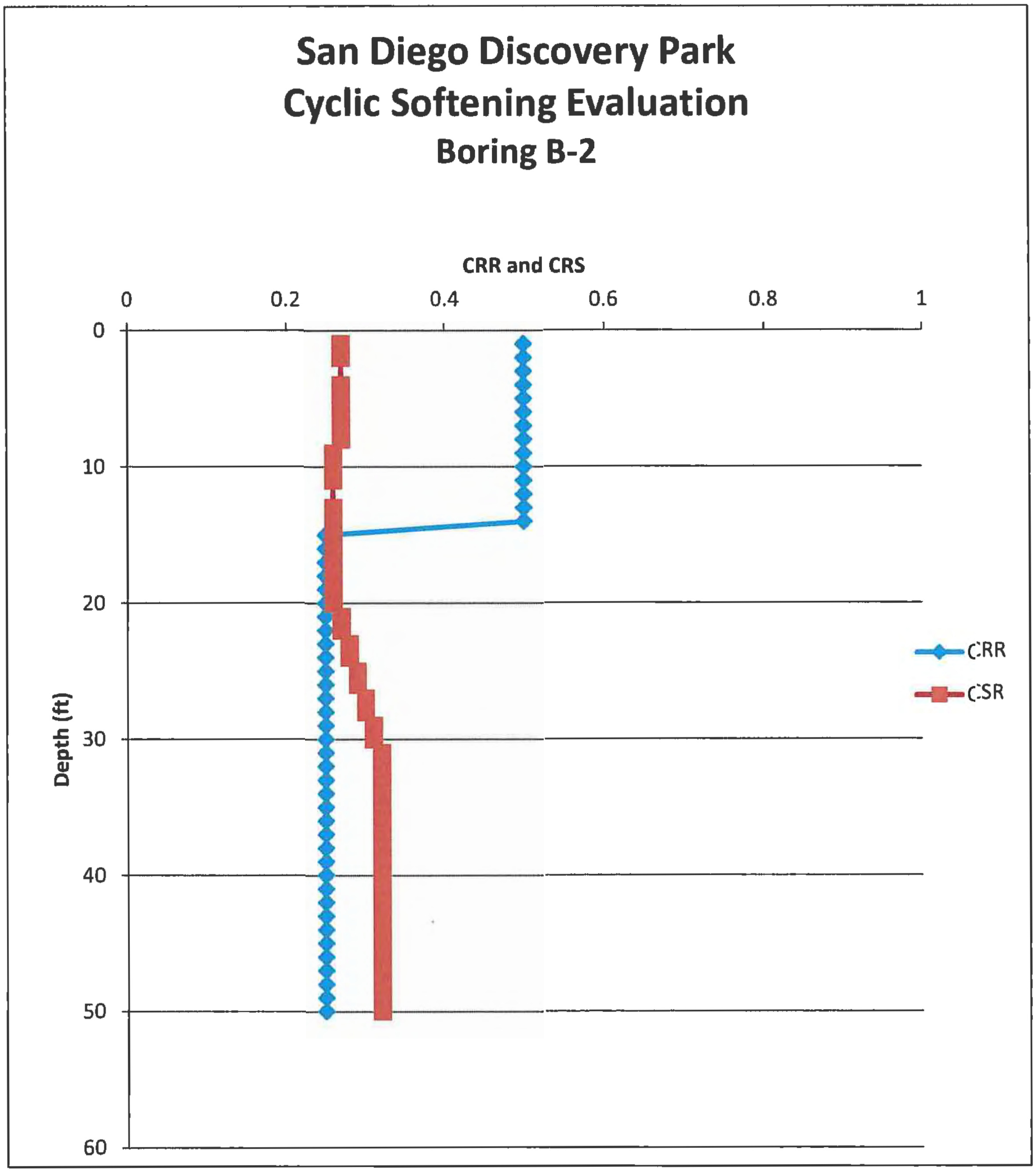
NCEER (1996) METHOD
EARTHQUAKE INFORMATION:

Earthquake Magnitude:	6.6
Peak Horiz. Acceleration (g)	0.42
Calculated Mag.Wtg.Factor:	0.733
Historic High Groundwater:	15.0

DEPTH TO BASE	BLOW COUNT N	WET DENSITY (PCF)	TOTAL STRESS O (TSF)	EFFECT STRESS O' (TSF)	REL. DEN. Dr (%)	ADJUST BLOWS (N1)60	Tav/O'$_o$	LIQUEFACTION SAFETY FACTOR	Volumetric Strain [e_{15}} (%)	EQ. SETTLE. Pe (in.)
1	17	126	0.032	0.032	86	35	0.270	--	0.00	0.00
2	17	126	0.095	0.095	86	35	0.270	--	0.00	0.00
3	17	126	0.158	0.158	86	35	0.270	--	0.00	0.00
4	13	126	0.221	0.221		28	0.270	~	0.00	0.00
5	13	106	0.279	0.279		28	0.270	~	0.00	0.00
6	13	106	0.332	0.332		28	0.270	~	0.00	0.00
7	13	106	0.385	0.385		28	0.270	~	0.00	0.00
8	13	106	0.438	0.438		28	0.270	~	0.00	0.00
9	31	106	0.491	0.491		43	0.270	~	0.00	0.00
10	31	106	0.544	0.544		43	0.270	~	0.00	0.00
11	31	106	0.597	0.597		43	0.270	~	0.00	0.00
12	16	106	0.650	0.650		23	0.270	~	0.00	0.00
13	16	106	0.703	0.703	67	19	0.270	--	0.00	0.00
14	16	106	0.756	0.756	67	19	0.270	--	0.00	0.00
15	16	106	0.809	0.793	67	19	0.276	1.08	1.30	0.16
16	16	106	0.862	0.815	67	19	0.286	1.05	1.30	0.16
17	16	106	0.915	0.837	67	19	0.296	1.02	1.30	0.16
18	11	115	0.970	0.861	54	15	0.305	0.81	1.70	0.20
19	11	115	1.027	0.887	54	15	0.313	0.79	1.70	0.20
20	11	115	1.085	0.913	54	15	0.321	0.77	1.70	0.20
21	11	115	1.142	0.939	54	15	0.329	0.76	1.70	0.20
22	5	115	1.200	0.966	36	9	0.336	0.44	2.70	0.32
23	5	115	1.257	0.992	36	9	0.343	0.43	2.70	0.32
24	5	115	1.315	1.018	36	9	0.349	0.43	2.70	0.32
25	14	115	1.372	1.045	59	18	0.355	0.86	1.70	0.20
26	14	115	1.430	1.071	59	18	0.361	0.85	1.70	0.20
27	14	115	1.487	1.097	59	18	0.367	0.84	1.70	0.20
28	8	115	1.545	1.124	43	12	0.372	0.54	2.30	0.28
29	8	115	1.602	1.150	43	12	0.377	0.53	2.30	0.28
30	8	115	1.660	1.176		12	0.382	~	0.00	0.00
31	8	115	1.717	1.202		15	0.386	~	0.00	0.00
32	8	115	1.775	1.229		15	0.391	~	0.00	0.00
33	8	115	1.832	1.255		15	0.395	~	0.00	0.00
34	8	115	1.890	1.281		15	0.399	~	0.00	0.00
35	6	115	1.947	1.308	36	8	0.403	0.36	2.70	0.32
36	6	115	2.005	1.334	36	8	0.406	0.36	2.70	0.32
37	6	115	2.062	1.360	36	8	0.410	0.36	2.70	0.32
38	7	115	2.120	1.387		14	0.413	~	0.00	0.00
39	7	115	2.177	1.413		14	0.417	~	0.00	0.00
40	7	115	2.235	1.439		14	0.420	~	0.00	0.00
41	7	130	2.296	1.469	38	8	0.423	0.32	2.70	0.32
42	7	130	2.361	1.503	38	8	0.425	0.32	2.70	0.32
43	43	130	2.426	1.537	92	39	0.427	Non-Liq.	0.00	0.00
44	43	130	2.491	1.571	92	42	0.429	Non-Liq.	0.00	0.00
45	43	130	2.556	1.604	92	42	0.431	Non-Liq.	0.00	0.00
46	43	130	2.621	1.638	92	42	0.433	Non-Liq.	0.00	0.00
47	43	130	2.686	1.672	92	42	0.434	Non-Liq.	0.00	0.00
48	43	130	2.751	1.706	92	38	0.436	Non-Liq.	0.00	0.00
49	43	130	2.816	1.740	92	38	0.438	Non-Liq.	0.00	0.00
50	43	130	2.881	1.773	92	38	0.439	Non-Liq.	0.00	0.00

TOTAL SETTLEMENT = 5.0 INCHES

Figure 8


San Diego Discovery Park
Cyclic Softening Evaluation
Boring B-2
CRR and CRS
0
0.2
0.4
0.6
0.8
1
Depth (ft)
0
10
20
30
40
50
60
CRR
CSR

Prediction of Lateral Spread using Bartlett and Youd

M =	6.63	earthquake magnitude
R =	5.30	horizontal distance to seismic source (km)
T(15) =	7.00	thickness of saturated layers (meters) with N1(60) < 15
F(15) =	20.00	average fines content in T(15) passing #200
D50(15) =	0.50	average D50 in T(15) (mm)
H =	15.00	height of free face (feet)

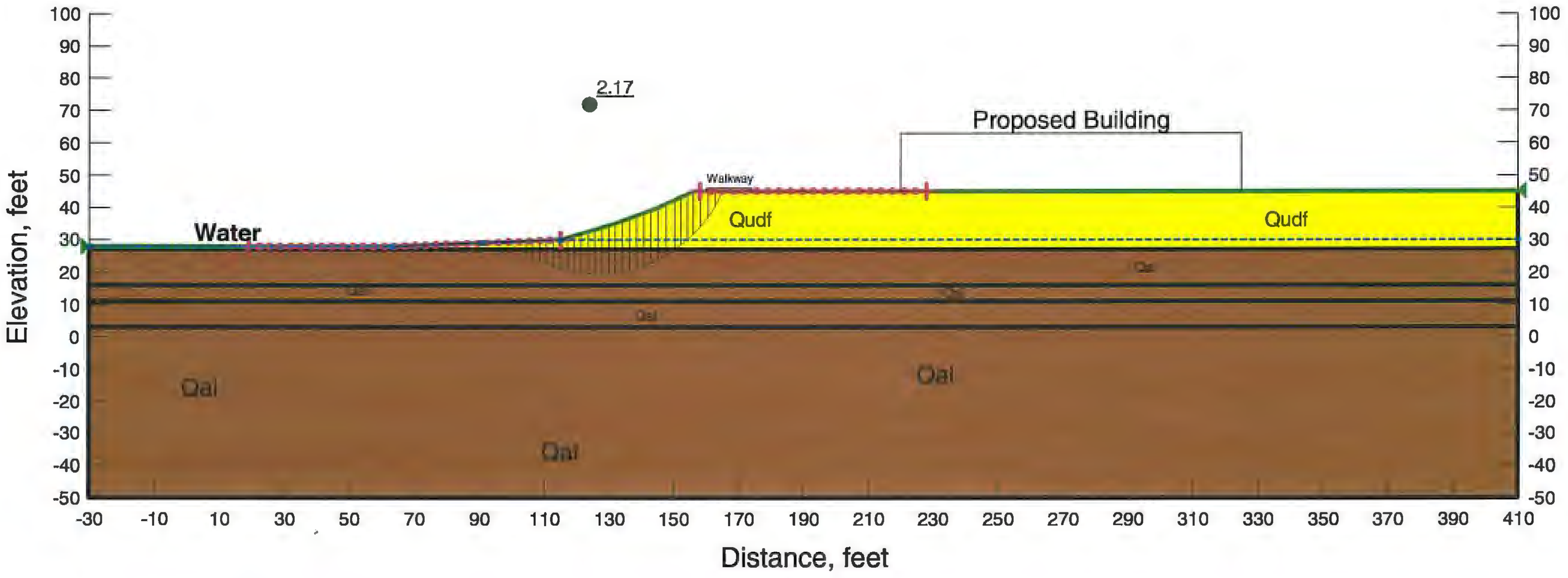
Distance From Slope Face, L (feet)	W	DH (meters)	DH (feet)
30	50.0	1.9	6
50	30.0	1.3	4
70	21.4	1.1	3
90	16.7	0.9	3
110	13.6	0.8	3
125	12.0	0.7	2

W = free face ratio (%), 100(H/L)
DH = predicted horizontal ground displacement

San Diego River Discovery Center
Project No. G1656-42-01
Section A-A'
Name: A-A Static.gsz
Date: 1/31/2014 Time: 12:44:55 PM

Qudf	Mohr-Coulomb	100 pcf	200 psf	25 °
Qal	Mohr-Coulomb	100 pcf	125 psf	25 °

Method: Spencer
Static Conditions
PWP Conditions Source: Piezometric Line
Slip Surface Option: Entry and Exit



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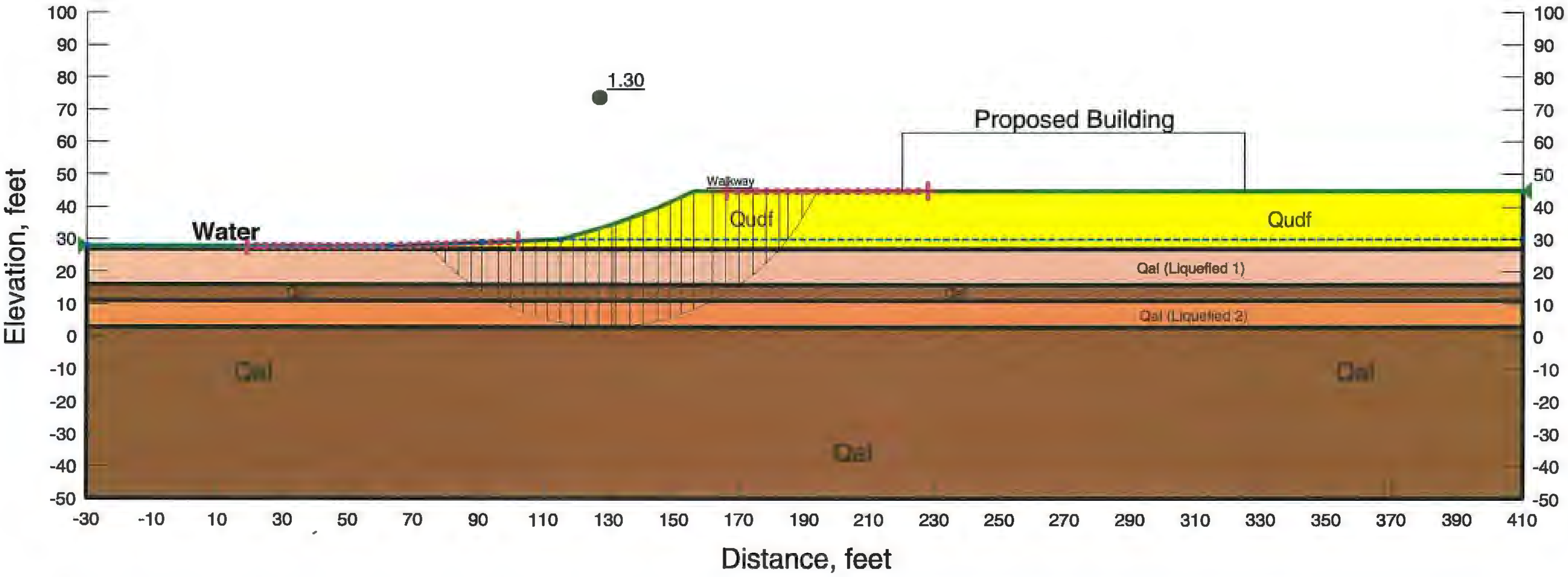
San Diego River Discovery Center
Project No. G1656-42-01
Section A-A'
Name: A-A (Seismic).gsz
Date: 1/31/2014 Time: 12:47:28 PM

Qudf	Mohr-Coulomb	100 pcf	200 psf	25 °
Qal	Mohr-Coulomb	100 pcf	125 psf	25 °
Qal (Liquefied 1)	Undrained (Phi=0)	100 pcf	400 psf	
Qal (Liquefied 2)	Undrained (Phi=0)	100 pcf	200 psf	

Method: Spencer
Seismic Conditions - Soil Residual Shear Strength
PWP Conditions Source: Piezometric Line
Slip Surface Option: Entry and Exit



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APPENDIX E

APPENDIX E

RECOMMENDED GRADING SPECIFICATIONS

FOR

SAN DIEGO RIVER DISCOVERY CENTER
SAN DIEGO, CALIFORNIA

PROJECT NO. G1656-42-01

RECOMMENDED GRADING SPECIFICATIONS

1. GENERAL

1.1 These Recommended Grading Specifications shall be used in conjunction with the Geotechnical Report for the project prepared by Geocon Incorporated. The recommendations contained in the text of the Geotechnical Report are a part of the earthwork and grading specifications and shall supersede the provisions contained hereinafter in the case of conflict.

1.2 Prior to the commencement of grading, a geotechnical consultant (Consultant) shall be employed for the purpose of observing earthwork procedures and testing the fills for substantial conformance with the recommendations of the Geotechnical Report and these specifications. The Consultant should provide adequate testing and observation services so that they may assess whether, in their opinion, the work was performed in substantial conformance with these specifications. It shall be the responsibility of the Contractor to assist the Consultant and keep them apprised of work schedules and changes so that personnel may be scheduled accordingly.

1.3 It shall be the sole responsibility of the Contractor to provide adequate equipment and methods to accomplish the work in accordance with applicable grading codes or agency ordinances, these specifications and the approved grading plans. If, in the opinion of the Consultant, unsatisfactory conditions such as questionable soil materials, poor moisture condition, inadequate compaction, adverse weather, result in a quality of work not in conformance with these specifications, the Consultant will be empowered to reject the work and recommend to the Owner that grading be stopped until the unacceptable conditions are corrected.

2. DEFINITIONS

2.1 **Owner** shall refer to the owner of the property or the entity on whose behalf the grading work is being performed and who has contracted with the Contractor to have grading performed.

2.2 **Contractor** shall refer to the Contractor performing the site grading work.

2.3 **Civil Engineer** or **Engineer of Work** shall refer to the California licensed Civil Engineer or consulting firm responsible for preparation of the grading plans, surveying and verifying as-graded topography.

2.4 **Consultant** shall refer to the soil engineering and engineering geology consulting firm retained to provide geotechnical services for the project.

2.5 **Soil Engineer** shall refer to a California licensed Civil Engineer retained by the Owner, who is experienced in the practice of geotechnical engineering. The Soil Engineer shall be responsible for having qualified representatives on-site to observe and test the Contractor's work for conformance with these specifications.

2.6 **Engineering Geologist** shall refer to a California licensed Engineering Geologist retained by the Owner to provide geologic observations and recommendations during the site grading.

2.7 **Geotechnical Report** shall refer to a soil report (including all addenda) which may include a geologic reconnaissance or geologic investigation that was prepared specifically for the development of the project for which these Recommended Grading Specifications are intended to apply.

3. MATERIALS

3.1 Materials for compacted fill shall consist of any soil excavated from the cut areas or imported to the site that, in the opinion of the Consultant, is suitable for use in construction of fills. In general, fill materials can be classified as *soil* fills, *soil-rock* fills or *rock* fills, as defined below.

3.1.1 **Soil fills** are defined as fills containing no rocks or hard lumps greater than 12 inches in maximum dimension and containing at least 40 percent by weight of material smaller than ¾ inch in size.

3.1.2 **Soil-rock fills** are defined as fills containing no rocks or hard lumps larger than 4 feet in maximum dimension and containing a sufficient matrix of soil fill to allow for proper compaction of soil fill around the rock fragments or hard lumps as specified in Paragraph 6.2. **Oversize rock** is defined as material greater than 12 inches.

3.1.3 **Rock fills** are defined as fills containing no rocks or hard lumps larger than 3 feet in maximum dimension and containing little or no fines. Fines are defined as material smaller than ¾ inch in maximum dimension. The quantity of fines shall be less than approximately 20 percent of the rock fill quantity.

3.2 Material of a perishable, spongy, or otherwise unsuitable nature as determined by the Consultant shall not be used in fills.

3.3 Materials used for fill, either imported or on-site, shall not contain hazardous materials as defined by the California Code of Regulations, Title 22, Division 4, Chapter 30, Articles 9 and 10; 40CFR; and any other applicable local, state or federal laws. The Consultant shall not be responsible for the identification or analysis of the potential presence of hazardous materials. However, if observations, odors or soil discoloration cause Consultant to suspect the presence of hazardous materials, the Consultant may request from the Owner the termination of grading operations within the affected area. Prior to resuming grading operations, the Owner shall provide a written report to the Consultant indicating that the suspected materials are not hazardous as defined by applicable laws and regulations.

3.4 The outer 15 feet of *soil-rock* fill slopes, measured horizontally, should be composed of properly compacted *soil* fill materials approved by the Consultant. *Rock* fill may extend to the slope face, provided that the slope is not steeper than 2:1 (horizontal:vertical) and a soil layer no thicker than 12 inches is track-walked onto the face for landscaping purposes. This procedure may be utilized provided it is acceptable to the governing agency, Owner and Consultant.

3.5 Samples of soil materials to be used for fill should be tested in the laboratory by the Consultant to determine the maximum density, optimum moisture content, and, where appropriate, shear strength, expansion, and gradation characteristics of the soil.

3.6 During grading, soil or groundwater conditions other than those identified in the Geotechnical Report may be encountered by the Contractor. The Consultant shall be notified immediately to evaluate the significance of the unanticipated condition

4. CLEARING AND PREPARING AREAS TO BE FILLED

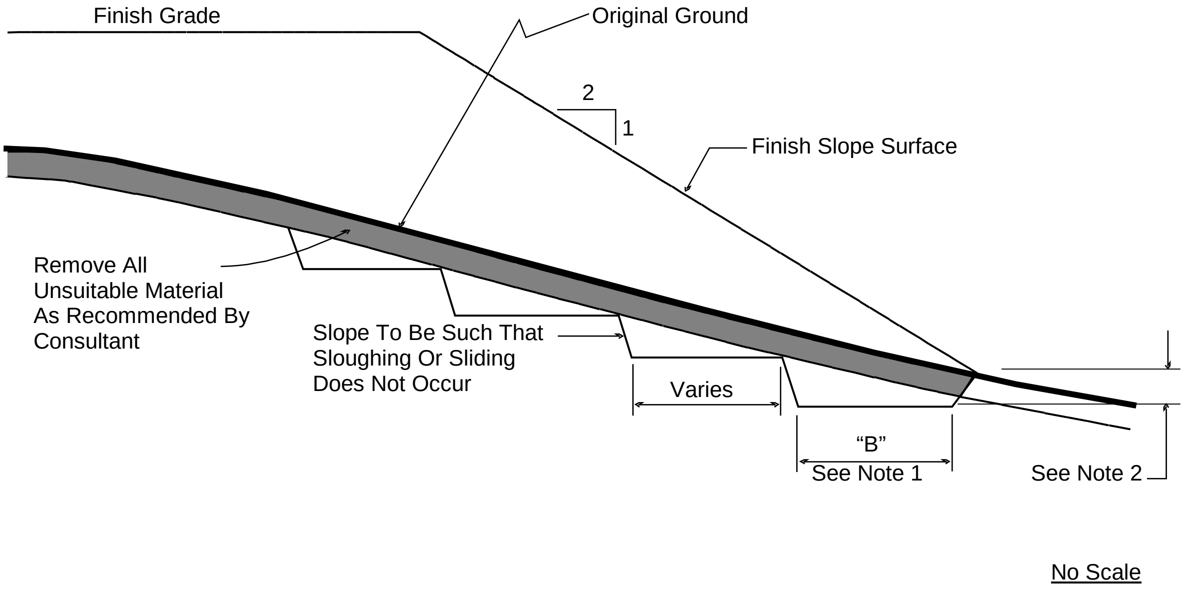
4.1 Areas to be excavated and filled shall be cleared and grubbed. Clearing shall consist of complete removal above the ground surface of trees, stumps, brush, vegetation, man-made structures, and similar debris. Grubbing shall consist of removal of stumps, roots, buried logs and other unsuitable material and shall be performed in areas to be graded. Roots and other projections exceeding 1½ inches in diameter shall be removed to a depth of 3 feet below the surface of the ground. Borrow areas shall be grubbed to the extent necessary to provide suitable fill materials.

4.2 Any asphalt pavement material removed during clearing operations should be properly disposed at an approved off-site facility. Concrete fragments that are free of reinforcing steel may be placed in fills, provided they are placed in accordance with Section 6.2 or 6.3 of this document.

4.3 After clearing and grubbing of organic matter and other unsuitable material, loose or porous soils shall be removed to the depth recommended in the Geotechnical Report. The depth of removal and compaction should be observed and approved by a representative of the Consultant. The exposed surface shall then be plowed or scarified to a minimum depth of 6 inches and until the surface is free from uneven features that would tend to prevent uniform compaction by the equipment to be used.

4.4 Where the slope ratio of the original ground is steeper than 5:1 (horizontal:vertical), or where recommended by the Consultant, the original ground should be benched in accordance with the following illustration.

TYPICAL BENCHING DETAIL



DETAIL NOTES: (1) Key width "B" should be a minimum of 10 feet, or sufficiently wide to permit complete coverage with the compaction equipment used. The base of the key should be graded horizontal, or inclined slightly into the natural slope.

(2) The outside of the key should be below the topsoil or unsuitable surficial material and at least 2 feet into dense formational material. Where hard rock is exposed in the bottom of the key, the depth and configuration of the key may be modified as approved by the Consultant.

4.5 After areas to receive fill have been cleared and scarified, the surface should be moisture conditioned to achieve the proper moisture content, and compacted as recommended in Section 6 of these specifications.

5. COMPACTION EQUIPMENT

5.1 Compaction of *soil* or *soil-rock* fill shall be accomplished by sheepsfoot or segmented-steel wheeled rollers, vibratory rollers, multiple-wheel pneumatic-tired rollers, or other types of acceptable compaction equipment. Equipment shall be of such a design that it will be capable of compacting the *soil* or *soil-rock* fill to the specified relative compaction at the specified moisture content.

5.2 Compaction of *rock* fills shall be performed in accordance with Section 6.3.

6. PLACING, SPREADING AND COMPACTION OF FILL MATERIAL

6.1 *Soil* fill, as defined in Paragraph 3.1.1, shall be placed by the Contractor in accordance with the following recommendations:

6.1.1 *Soil* fill shall be placed by the Contractor in layers that, when compacted, should generally not exceed 8 inches. Each layer shall be spread evenly and shall be thoroughly mixed during spreading to obtain uniformity of material and moisture in each layer. The entire fill shall be constructed as a unit in nearly level lifts. Rock materials greater than 12 inches in maximum dimension shall be placed in accordance with Section 6.2 or 6.3 of these specifications.

6.1.2 In general, the *soil* fill shall be compacted at a moisture content at or above the optimum moisture content as determined by ASTM D 1557-09.

6.1.3 When the moisture content of *soil* fill is below that specified by the Consultant, water shall be added by the Contractor until the moisture content is in the range specified.

6.1.4 When the moisture content of the *soil* fill is above the range specified by the Consultant or too wet to achieve proper compaction, the *soil* fill shall be aerated by the Contractor by blading/mixing, or other satisfactory methods until the moisture content is within the range specified.

6.1.5 After each layer has been placed, mixed, and spread evenly, it shall be thoroughly compacted by the Contractor to a relative compaction of at least 90 percent. Relative compaction is defined as the ratio (expressed in percent) of the in-place dry density of the compacted fill to the maximum laboratory dry density as determined in accordance with ASTM D 1557-09. Compaction shall be continuous over the entire area, and compaction equipment shall make sufficient passes so that the specified minimum relative compaction has been achieved throughout the entire fill.

6.1.6 Where practical, soils having an Expansion Index greater than 50 should be placed at least 3 feet below finish pad grade and should be compacted at a moisture content generally 2 to 4 percent greater than the optimum moisture content for the material.

6.1.7 Properly compacted *soil* fill shall extend to the design surface of fill slopes. To achieve proper compaction, it is recommended that fill slopes be over-built by at least 3 feet and then cut to the design grade. This procedure is considered preferable to track-walking of slopes, as described in the following paragraph.

6.1.8 As an alternative to over-building of slopes, slope faces may be back-rolled with a heavy-duty loaded sheepsfoot or vibratory roller at maximum 4-foot fill height intervals. Upon completion, slopes should then be track-walked with a D-8 dozer or similar equipment, such that a dozer track covers all slope surfaces at least twice.

6.2 *Soil-rock* fill, as defined in Paragraph 3.1.2, shall be placed by the Contractor in accordance with the following recommendations:

6.2.1 Rocks larger than 12 inches but less than 4 feet in maximum dimension may be incorporated into the compacted *soil* fill, but shall be limited to the area measured 15 feet minimum horizontally from the slope face and 5 feet below finish grade or 3 feet below the deepest utility, whichever is deeper.

6.2.2 Rocks or rock fragments up to 4 feet in maximum dimension may either be individually placed or placed in windrows. Under certain conditions, rocks or rock fragments up to 10 feet in maximum dimension may be placed using similar methods. The acceptability of placing rock materials greater than 4 feet in maximum dimension shall be evaluated during grading as specific cases arise and shall be approved by the Consultant prior to placement.

6.2.3 For individual placement, sufficient space shall be provided between rocks to allow for passage of compaction equipment.

6.2.4 For windrow placement, the rocks should be placed in trenches excavated in properly compacted *soil* fill. Trenches should be approximately 5 feet wide and 4 feet deep in maximum dimension. The voids around and beneath rocks should be filled with approved granular soil having a Sand Equivalent of 30 or greater and should be compacted by flooding. Windrows may also be placed utilizing an "open-face" method in lieu of the trench procedure, however, this method should first be approved by the Consultant.

6.2.5 Windrows should generally be parallel to each other and may be placed either parallel to or perpendicular to the face of the slope depending on the site geometry. The minimum horizontal spacing for windrows shall be 12 feet center-to-center with a 5-foot stagger or offset from lower courses to next overlying course. The minimum vertical spacing between windrow courses shall be 2 feet from the top of a lower windrow to the bottom of the next higher windrow.

6.2.6 Rock placement, fill placement and flooding of approved granular soil in the windrows should be continuously observed by the Consultant.

6.3 *Rock* fills, as defined in Section 3.1.3, shall be placed by the Contractor in accordance with the following recommendations:

6.3.1 The base of the *rock* fill shall be placed on a sloping surface (minimum slope of 2 percent). The surface shall slope toward suitable subdrainage outlet facilities. The *rock* fills shall be provided with subdrains during construction so that a hydrostatic pressure buildup does not develop. The subdrains shall be permanently connected to controlled drainage facilities to control post-construction infiltration of water.

6.3.2 *Rock* fills shall be placed in lifts not exceeding 3 feet. Placement shall be by rock trucks traversing previously placed lifts and dumping at the edge of the currently placed lift. Spreading of the *rock* fill shall be by dozer to facilitate *seating* of the rock. The *rock* fill shall be watered heavily during placement. Watering shall consist of water trucks traversing in front of the current rock lift face and spraying water continuously during rock placement. Compaction equipment with compactive energy comparable to or greater than that of a 20-ton steel vibratory roller or other compaction equipment providing suitable energy to achieve the required compaction or deflection as recommended in Paragraph 6.3.3 shall be utilized. The number of passes to be made should be determined as described in Paragraph 6.3.3. Once a *rock* fill lift has been covered with *soil* fill, no additional *rock* fill lifts will be permitted over the *soil* fill.

6.3.3 Plate bearing tests, in accordance with ASTM D 1196-09, may be performed in both the compacted *soil* fill and in the *rock* fill to aid in determining the required minimum number of passes of the compaction equipment. If performed, a minimum of three plate bearing tests should be performed in the properly compacted *soil* fill (minimum relative compaction of 90 percent). Plate bearing tests shall then be performed on areas of *rock* fill having two passes, four passes and six passes of the compaction equipment, respectively. The number of passes required for the *rock* fill shall be determined by comparing the results of the plate bearing tests for the *soil* fill and the *rock* fill and by evaluating the deflection variation with number of passes. The required number of passes of the compaction equipment will be performed as necessary until the plate bearing deflections are equal to or less than that determined for the properly compacted *soil* fill. In no case will the required number of passes be less than two.

6.3.4 A representative of the Consultant should be present during *rock* fill operations to observe that the minimum number of “passes” have been obtained, that water is being properly applied and that specified procedures are being followed. The actual number of plate bearing tests will be determined by the Consultant during grading.

6.3.5 Test pits shall be excavated by the Contractor so that the Consultant can state that, in their opinion, sufficient water is present and that voids between large rocks are properly filled with smaller rock material. In-place density testing will not be required in the *rock* fills.

6.3.6 To reduce the potential for “piping” of fines into the *rock* fill from overlying *soil* fill material, a 2-foot layer of graded filter material shall be placed above the uppermost lift of *rock* fill. The need to place graded filter material below the *rock* should be determined by the Consultant prior to commencing grading. The gradation of the graded filter material will be determined at the time the *rock* fill is being excavated. Materials typical of the *rock* fill should be submitted to the Consultant in a timely manner, to allow design of the graded filter prior to the commencement of *rock* fill placement.

6.3.7 *Rock* fill placement should be continuously observed during placement by the Consultant.

7. OBSERVATION AND TESTING

7.1 The Consultant shall be the Owner’s representative to observe and perform tests during clearing, grubbing, filling, and compaction operations. In general, no more than 2 feet in vertical elevation of *soil* or *soil-rock* fill should be placed without at least one field density test being performed within that interval. In addition, a minimum of one field density test should be performed for every 2,000 cubic yards of *soil* or *soil-rock* fill placed and compacted.

7.2 The Consultant should perform a sufficient distribution of field density tests of the compacted *soil* or *soil-rock* fill to provide a basis for expressing an opinion whether the fill material is compacted as specified. Density tests shall be performed in the compacted materials below any disturbed surface. When these tests indicate that the density of any layer of fill or portion thereof is below that specified, the particular layer or areas represented by the test shall be reworked until the specified density has been achieved.

7.3 During placement of *rock* fill, the Consultant should observe that the minimum number of passes have been obtained per the criteria discussed in Section 6.3.3. The Consultant should request the excavation of observation pits and may perform plate bearing tests on the placed *rock* fills. The observation pits will be excavated to provide a basis for expressing an opinion as to whether the *rock* fill is properly seated and sufficient moisture has been applied to the material. When observations indicate that a layer of *rock* fill or any portion thereof is below that specified, the affected layer or area shall be reworked until the *rock* fill has been adequately seated and sufficient moisture applied.

7.4 A settlement monitoring program designed by the Consultant may be conducted in areas of *rock* fill placement. The specific design of the monitoring program shall be as recommended in the Conclusions and Recommendations section of the project Geotechnical Report or in the final report of testing and observation services performed during grading.

7.5 The Consultant should observe the placement of subdrains, to verify that the drainage devices have been placed and constructed in substantial conformance with project specifications.

7.6 Testing procedures shall conform to the following Standards as appropriate:

7.6.1 Soil and Soil-Rock Fills:

7.6.1.1 Field Density Test, ASTM D 1556-07, *Density of Soil In-Place By the Sand-Cone Method.*

7.6.1.2 Field Density Test, Nuclear Method, ASTM D 6938-08A, *Density of Soil and Soil-Aggregate In-Place by Nuclear Methods (Shallow Depth).*

7.6.1.3 Laboratory Compaction Test, ASTM D 1557-09, *Moisture-Density Relations of Soils and Soil-Aggregate Mixtures Using 10-Pound Hammer and 18-Inch Drop.*

7.6.1.4. Expansion Index Test, ASTM D 4829-08A, *Expansion Index Test.*

7.6.2 Rock Fills

7.6.2.1 Field Plate Bearing Test, ASTM D 1196-09 (Reapproved 1997) *Standard Method for Nonreparative Static Plate Load Tests of Soils and Flexible Pavement Components, For Use in Evaluation and Design of Airport and Highway Pavements.*

8. PROTECTION OF WORK

8.1 During construction, the Contractor shall properly grade all excavated surfaces to provide positive drainage and prevent ponding of water. Drainage of surface water shall be controlled to avoid damage to adjoining properties or to finished work on the site. The Contractor shall take remedial measures to prevent erosion of freshly graded areas until such time as permanent drainage and erosion control features have been installed. Areas subjected to erosion or sedimentation shall be properly prepared in accordance with the Specifications prior to placing additional fill or structures.

8.2 After completion of grading as observed and tested by the Consultant, no further excavation or filling shall be conducted except in conjunction with the services of the Consultant.

9. CERTIFICATIONS AND FINAL REPORTS

9.1 Upon completion of the work, Contractor shall furnish Owner a certification by the Civil Engineer stating that the lots and/or building pads are graded to within 0.1 foot vertically of elevations shown on the grading plan and that all tops and toes of slopes are within 0.5 foot horizontally of the positions shown on the grading plans. After installation of a section of subdrain, the project Civil Engineer should survey its location and prepare an *as-built* plan of the subdrain location. The project Civil Engineer should verify the proper outlet for the subdrains and the Contractor should ensure that the drain system is free of obstructions.

9.2 The Owner is responsible for furnishing a final as-graded soil and geologic report satisfactory to the appropriate governing or accepting agencies. The as-graded report should be prepared and signed by a California licensed Civil Engineer experienced in geotechnical engineering and by a California Certified Engineering Geologist, indicating that the geotechnical aspects of the grading were performed in substantial conformance with the Specifications or approved changes to the Specifications.

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2. California Department of Conservation, Division of Mines and Geology, *Probabilistic Seismic Hazard Assessment for the State of California*, Open File Report 96-08, 1996.

3. California Geological Survey, *Seismic Shaking Hazards in California*, Based on the USGS/CGS Probabilistic Seismic Hazards Assessment (PSHA) Model, 2002 (revised April 2003). 10% probability of being exceeded in 50 years.
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4. Campbell, K. W. and Y. Bozorgnia, *NGA Ground Motion Model for the Geometric Mean Horizontal Component of PGA, PGV, PGD and 5% Damped Linear Elastic Response Spectra for Periods Ranging from 0.01 to 10 s,* Preprint of version submitted for publication in the NGA Special Volume of Earthquake Spectra, Volume 24, Issue 1, pages 139-171, February 2008.

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8. Risk Engineering, *EZ-FRISK (Version 7.52)*, 2011.

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10. USGS computer program, *Seismic Hazard Curves and Uniform Hazard Response Spectra (Version 5.1.0, February 10, 2010).*