PRELIMINARY
GEOTECHNICAL INVESTIGATION

5650 AND 5660 KEARNY MESA ROAD
SAN DIEGO, CALIFORNIA

GEOCON
INCORPORATED

GEOTECHNICAL
ENVIRONMENTAL
MATERIALS

PREPARED FOR

LINCOLN PROPERTY COMPANY
SAN DIEGO, CALIFORNIA

APRIL 23, 2019
REVISED MAY 1, 2019
PROJECT NO. G2389-42-01

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CH Realty/Acquisitions VIII, LLC
c/o LPC West, Inc.
600 B Street, Suite 1540
San Diego, California 90017

Attention: Mr. Scott Moffatt

Subject: PRELIMINARY GEOTECHNICAL INVESTIGATION
5650 AND 5660 KEARNY MESA ROAD
SAN DIEGO, CALIFORNIA

Dear Mr. Moffatt:

In accordance with your request and authorization of our proposal (LG-19102, dated March 27, 2019), we have performed a geotechnical investigation for the subject project. The accompanying report presents the findings of our study with conclusions and recommendations pertinent to the geotechnical aspects of developing the property as proposed. We have also provided geotechnical recommendations for storm water management.

It is our opinion that the site can be developed as currently proposed, provided the recommendations of this report are followed.

Should you have 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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PRELIMINARY GEOTECHNICAL INVESTIGATION

1. PURPOSE AND SCOPE

In accordance with your authorization, we have performed a geotechnical investigation for the proposed improvements to the property located at 5650 and 5660 Kearny Mesa Road in San Diego, California. (see Vicinity Map, Figure 1). The purpose of this study was to evaluate surface and subsurface soil conditions, general site geology, and to identify geotechnical constraints that might impact development of the property.

The field investigation was conducted on April 4 and 5, 2019 and consisted of drilling eight, small-diameter borings to depths ranging from 7 feet to 16 feet and excavating 11 exploratory trenches to depths ranging from 2 feet to 6 feet. The approximate locations of the exploratory borings and trenches are shown on the Geologic Map, Figure 2. Details of our field investigation and copies of the boring and trench logs are presented in Appendix A.

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

We performed six, borehole infiltration tests using a constant-head permeameter. The tests were conducted in 8-inch-diameter, drilled borings. The results of the infiltration testing and information relating to geotechnical aspects of storm water management are provided in Appendix C.

The conclusions and recommendations presented herein are based on our analysis of the data obtained from the field investigation, laboratory test results, and our experience with similar soil and geologic conditions on this and adjacent properties.

2. SITE AND PROJECT DESCRIPTION

The site is located at 5650 and 5660 Kearny Mesa Road in San Diego, California. The site is bordered to the west and southwest by one-story commercial buildings, to the southeast by Kearny Mesa Road and California State Route (SR) 163, and to the north and northeast by undeveloped land and SR 52. The site slopes gently to the northwest with site elevations ranging from approximately 427 feet Mean Sea Level (MSL) at the southeast corner of the site to approximately 402 feet MSL at the northwestern corner. Two one-story office structures and an abandoned two-story structure occupy the property. Asphalt concrete parking lots and driveways surround the buildings.

We understand planned development will consist of demolishing the existing buildings, paved parking lots, and driveways, and constructing a one-story 269,000 square foot warehouse building with 15,500 square feet of mezzanine space and loading locks, parking lots and access driveways. Storm water quality basins are planned at the northwest corner and along the southeastern side of the property adjacent to Kearny Mesa Road. We expect that grading will consist of cuts and fills of approximately 5 feet or less to achieve building pad grade.

The descriptions above are based on a review of the referenced site plan and discussions with you. If development plans differ significantly from those described herein, we should be contacted for review and possible revisions to this report.

3. SOIL AND GEOLOGIC CONDITIONS

Based on observations during our subsurface investigation, the site is underlain by undocumented fill, very old paralic deposits, and Tertiary age Stadium Conglomerate. The geologic units are described below. Their approximate lateral extent is shown on the Geologic Map, Figure 2.

3.1 Undocumented Fill (Qudf)

We encountered undocumented fill in the borings and trenches ranging from depths of approximately 1 to 6 feet. The deeper fill was in the southeast portion of the property near Kearny Mesa Road. The undocumented fill consists of loose to medium dense, damp to wet, silty to clayey, fine to coarse sand, and soft to very stiff, moist to wet, sandy to silty clay, with some gravel and cobble. The undocumented fill is not suitable for support of additional fill or structural loads in its present condition and will require remedial grading consisting of complete removal and replacement as compacted fill.

3.2 Very Old Paralic Deposits (Qvop)

We encountered very old paralic deposits underlying the undocumented fill and exposed at existing grade. The very old paralic deposits generally consist of medium dense to very dense, dry to moist, silty to clayey, fine to coarse sand, with some gravel and cobble. In some areas, the upper portion of this deposits consists of firm to hard, damp to wet, silty to sandy clay, which may require remedial grading. With the exception of the upper clay layer, the very old paralic deposits are suitable for the support of the proposed improvements.

3.3 Stadium Conglomerate (Tst)

We encountered Tertiary-age Stadium Conglomerate within Borings B-1 and B-3 below the very old paralic deposits. Stadium Conglomerate generally consists of very dense, locally cemented, silty to clayey, fine to medium sandstone to sandy conglomerate. The Stadium Conglomerate generally has a "very low" to "low" expansion potential (expansion index of 50 or less). The Stadium Conglomerate is suitable to support additional fill or structural loads.

4. GROUNDWATER

We did not encounter groundwater during our investigation; however, it is not uncommon for groundwater or seepage conditions to develop where none previously existed. Groundwater elevation is dependent on seasonal precipitation, irrigation, and land use, among other factors, and vary as a result. Proper surface drainage will be important to the future performance of the project.

5. GEOLOGIC HAZARDS

5.1 Geologic Hazard Category

City of San Diego (2008) shows the site within Geologic Hazard Categories 51 and 52. Geologic Hazard Category 51 is defined as *Level Mesas - underlain by terrace deposits and bedrock, nominal risk.* Geologic Hazard Category 52 is defined as *Other level areas, gently sloping to steep terrain, favorable geologic structure, Low risk.*

5.2 Ground Rupture

No evidence of faulting was observed during our investigation. The USGS (2016) shows that there are no mapped Quaternary faults crossing or trending toward the property. The site is not located within a currently established Alquist-Priolo Earthquake Fault Zone. The nearest active fault is the Newport-Inglewood/Rose Canyon Fault Zone, located approximately 5 miles west of the site. The risk associated with ground rupture hazard is low.

5.3 Seismicity

We performed a deterministic seismic hazard analysis using Risk Engineering (2015). Six known active faults were located within a search radius of 50 miles from the property. We used the 2008 USGS fault database, which provides several models and combinations of fault data, to evaluate the fault information. Based on this database, Newport-Inglewood/Rose Canyon Fault Zone, located approximately 5 miles west of the site, is the nearest known active fault and is 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 are 7.5 and 0.39 g, respectively. Table 5.3.1 lists the estimated maximum earthquake magnitude and peak ground acceleration for the most dominant faults in relationship to the site location. We calculated peak ground acceleration (PGA) using Boore and Atkinson (2008), Campbell and Bozorgnia (2008), and Chiou and Youngs (2007) acceleration-attenuation relationships.

TABLE 5.3.1
DETERMINISTIC SPECTRA SITE PARAMETERS

Fault Name	Distance from Site (miles)	Maximum Earthquake Magnitude (Mw)	Peak Ground Acceleration		
			Boore-Atkinson (2008) NGA USGS 2008 (g)	Campbell-Bozorgnia (2008) NGA USGS 2008 (g)	Chiou-Youngs (2007) NGA USGS 2008 (g)
Newport-Inglewood	5	7.5	0.32	0.32	0.39
Rose Canyon	5	6.9	0.27	0.30	0.33
Coronado Bank	19	7.4	0.16	0.12	0.14
Palos Verdes Connected	19	7.7	0.18	0.13	0.17
Elsinore	35	7.85	0.13	0.09	0.11
Earthquake Valley	40	6.8	0.07	0.05	0.04

We performed a probabilistic seismic hazard analysis for the site using Risk Engineering (2015). The computer program assumes that the occurrence rate of earthquakes on each mapped Quaternary fault is proportional to the fault 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 used acceleration-attenuation relationships suggested by Boore-Atkinson (2008), Campbell-Bozorgnia (2008), and Chiou-Youngs (2008) in the analysis. Table 5.3.2 presents the site-specific probabilistic seismic hazard parameters including acceleration-attenuation relationships and the probability of exceedence.

TABLE 5.3.2
PROBABILISTIC SEISMIC HAZARD PARAMETERS

Probability of Exceedence	Peak Ground Acceleration		
	Boore-Atkinson NGA USGS 2008 (g)	Campbell-Bozorgnia NGA USGS 2008 (g)	Chiou-Youngs (2007) NGA USGS 2008 (g)
2% in a 50 Year Period	0.40	0.42	0.47
5% in a 50 Year Period	0.28	0.29	0.31
10% in a 50 Year Period	0.20	0.20	0.21

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 frequency and duration of motion and soil conditions underlying the site. Seismic design of the structures should be evaluated in accordance with the California Building Code (CBC).

5.4 Liquefaction and Seismically Induced Settlement

The risk associated with liquefaction and seismically induced settlement hazard is low due to the lack of permanent, near surface groundwater and the dense nature of the underlying very old paralic deposits and Stadium Conglomerate.

5.5 Landslides

Our site reconnaissance and review of available geologic literature and geotechnical reports for the site vicinity indicate that no landslides are present on the property or at a location that could impact the site. The risk associated with landslide hazard at the site is low.

5.6 Tsunamis and Seiches

The site is approximately 7 miles from the Pacific Ocean at an elevation over 400 feet above MSL. The risk associated with inundation hazard due to tsunamis is low.

There are no upstream lakes or reservoirs. The risk associated with inundation hazard associated with seiche is low.

5.7 Subsidence

Based on the subsurface soil conditions encountered during our field investigation, the risk associated with ground subsidence hazard is low.

5.8 Flooding

The site is not located within a drainage or floodplain; therefore, the risk associated with flooding hazard is low.

6. CONCLUSIONS AND RECOMMENDATIONS

6.1 General

6.1.1 From a geotechnical engineering standpoint, it is our opinion that the site is suitable for the proposed improvements provided the recommendations presented herein are implemented in design and construction of the project.

6.1.2 The site is underlain by undocumented fill, very old paralic deposits, and Stadium Conglomerate. Remedial grading of the undocumented fill and the upper clay layer of the very old paralic deposits will be necessary in areas to receive structures or settlement-sensitive improvements.

6.1.3 Remedial grading should consist of the complete removal of unsuitable soil and its replacement with properly compacted fill. Soils derived from on-site excavations are suitable for use as compacted fill provided they are free of trash, debris, organics, and other detrimental materials.

6.1.4 The proposed structures can be supported on conventional shallow footings founded on compacted fill or on very old paralic deposits. Undercutting of the building pad or deepened footings will be required if a cut to fill transition is created within the building pad during grading.

6.1.5 Project grading and foundation plans have not been provided for our review. Geocon Incorporated should review the plans prior to the submittal to regulatory agencies for approval. Additional analysis and modifications to this report may be required.

6.1.6 Groundwater and/or seepage-related problems are not anticipated, provided that surface drainage is directed into properly designed drainage structures and away from pavement edges, buildings and other moisture-sensitive developments.

6.1.7 Based on our research no active, potentially active, or activity unknown faults are mapped crossing the site or are trending toward the site.

6.1.8 The risk associated with ground rupture, liquefaction, and landslide hazards are low.

6.1.9 With the exception of the possibility of strong seismic shaking, no significant geologic hazards were observed or are known to exist at the site or other locations that could adversely affect the proposed project.

6.1.10 It is our opinion that the proposed development will not destabilize or result in settlement of adjacent properties.

6.1.11 Groundwater was not encountered during our field investigation and is not expected to be encountered during grading operations.

6.1.12 Subsurface conditions observed may be extrapolated to reflect general soil/geologic conditions; however, some variations in subsurface conditions between trench and boring locations should be anticipated.

6.2 Excavation and Soil Characteristics

6.2.1 Excavation of the site soil should be possible with moderate to heavy effort using conventional heavy-duty equipment. The very old paralic deposits are cemented and will require a very heavy effort to excavate. Refusal was encountered in the backhoe trenches performed for this study.

6.2.2 Based on the soil types encountered during our recent field investigation, the onsite soils are expected to be both “non-expansive” (expansion index [EI] of 20 or less) and “expansive” (EI greater than 20) as defined by 2016 California Building Code (CBC) Section 1803.5.3. Table 6.2.1 presents soil classifications based on the expansion index. The soil encountered during the geotechnical investigation possess a *very low* to *medium* expansion potential.

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	

6.2.3 We performed laboratory tests on samples of the site soils to check the percentage of water-soluble sulfate content. Results of the laboratory water-soluble sulfate content tests are presented in Appendix B and indicate that the on-site materials tested possess “S0” sulfate exposure to concrete structures as defined by 2016 CBC Section 1904 and ACI 318-14 Chapter 19. Table 6.2.2 presents a summary of concrete requirements set forth by 2016

CBC Section 1904 and ACI 318. We recommend ACI guidelines be followed in determining the type of concrete to be utilized on the project. 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

6.2.4 Geocon Incorporated does not practice in the field of corrosion engineering. Therefore, further evaluation by a corrosion engineer may be needed if improvements susceptible to corrosion are planned.

6.3 Grading

6.3.1 All grading should be performed in accordance with the *Recommended Grading Specifications* contained in Appendix D. Where the recommendations of Appendix D conflict with this section of the report, the recommendations of this section take precedence.

6.3.2 Prior to commencing grading, a preconstruction conference should be held at the site with the owner or developer, grading contractor, civil engineer, and geotechnical engineer in attendance. Special soil handling and/or the grading plans can be discussed at that time.

6.3.3 Grading should be performed in conjunction with the observation and compaction testing services of Geocon Incorporated. Fill soil should be observed on a full-time basis during placement and tested to check in-place dry density and moisture content.

6.3.4 Grading of the site should commence with the removal of existing improvements, foundations, utilities, vegetation, and deleterious debris. Deleterious debris should be exported from the site and should not be mixed with the fill. Existing underground improvements that will be abandoned and concrete footings should be removed and the resulting excavations properly backfilled in accordance with the procedures described herein.

6.3.5 To provide support for the new structure and improvements, we recommend surficial soil (undocumented fill and the upper clay layer of the very old paralic deposits) be removed and replaced with properly compacted fill. Removal depths between 1 and 6 feet are expected across the site. The approximate removal depth at each boring and trench location is shown on Figure 2.

6.3.6 Grading may result in a cut to fill transition across the building pad. To reduce the potential for differential settlement, the cut portion of the transition should be over-excavated (undercut) at least 3 feet below proposed finish grade or at least one foot below the lowest foundation element, whichever is deeper, and replaced with properly compacted "very low" to "medium" expansive fill soils. The undercut should extend into the fill side of the pad such that at least 3 feet fill below pad grade and 1-foot of fill below footing bottom exists throughout the building pad. The undercut should extend to a horizontal distance of at least 5 feet outside the building pad limits. Overexcavations should be cut at a gradient of one percent toward the deeper fill area to promote drainage along the contact between the native soil and compacted fill.

6.3.7 Portions of the on-site soils are highly expansive. We recommend highly expansive soils be placed at a depth of at least 3 feet below finish grade, or in landscape areas outside of structural improvement areas. Alternatively, the expansive soils can be mixed with on-site very low and low expansive soils such that the combined mix has an EI less than 90. Significant mixing and processing may be required to properly mix the clay soils with the sandy soils to produce an acceptable mixed soil for use within structural improvement areas.

6.3.8 Prior to placing fill, the upper 12 inches of soil at the base of fill and undercut areas should be scarified, moisture conditioned as necessary, and compacted. Soils derived from onsite excavations are suitable for reuse as fill if free from vegetation, debris and other deleterious material. Fill lifts should be no thicker than will allow for adequate bonding and compaction. Fill, backfill, and scarified ground surfaces, should be compacted to a dry density of at least 90 percent of maximum dry density slightly above optimum moisture content, as determined in accordance with ASTM Test Procedure D 1557. Fill or backfill with in-place density test results indicating moisture contents less than optimum will require additional moisture conditioning prior to placing fill.

6.3.9 Imported fill, if needed, should consist of granular soil with a “very low” to “low” expansion potential (EI of 50 or less) that is free of deleterious material or stones larger than 3 inches and should be compacted as recommended above. Geocon Incorporated should perform laboratory testing on the soil prior to its arrival at the site to evaluate its suitability as fill material. The imported soil should be certified as being free of hazardous contaminants as well as chemical properties that could adversely impact proposed improvements.

6.4 Seismic Design Criteria

6.4.1 We used the computer program *OSHPD Seismic Design Maps* (SEAOC, 2019). Table 6.4.1 summarizes site-specific design criteria obtained from the 2016 California Building Code (CBC; Based on the 2015 International Building Code [IBC] and ASCE 7-10), Chapter 16 Structural Design, Section 1613 Earthquake Loads. The short spectral response uses a period of 0.2 second. The building structure and improvements should be designed using a Site Class C. We evaluated the site class based on the discussion in Section 1613.3.2 of the 2016 CBC and Table 20.3-1 of ASCE 7-10. The values presented in Table 6.4.1 are for the risk-targeted maximum considered earthquake (MCE_R).

TABLE 6.4.1
2016 CBC SEISMIC DESIGN PARAMETERS

Parameter	Value	2016 CBC Reference
Site Class	C	Section 1613.3.2
MCE_R Ground Motion Spectral Response Acceleration – Class B (short), S_S	0.988 g	Figure 1613.3.1(1)
MCE_R Ground Motion Spectral Response Acceleration – Class B (1 sec), S_1	0.379 g	Figure 1613.3.1(2)
Site Coefficient, F_A	1.005	Table 1613.3.3(1)
Site Coefficient, F_V	1.421	Table 1613.3.3(2)
Site Class Modified MCE_R Spectral Response Acceleration (short), S_{MS}	0.993 g	Section 1613.3.3 (Eqn 16-37)
Site Class Modified MCE_R Spectral Response Acceleration (1 sec), S_{M1}	0.539 g	Section 1613.3.3 (Eqn 16-38)
5% Damped Design Spectral Response Acceleration (short), S_{DS}	0.662 g	Section 1613.3.4 (Eqn 16-39)
5% Damped Design Spectral Response Acceleration (1 sec), S_{D1}	0.359 g	Section 1613.3.4 (Eqn 16-40)

6.4.2 Table 6.4.2 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 (MCEG).

TABLE 6.4.2
2016 CBC SITE ACCELERATION DESIGN PARAMETERS

Parameter	Value	ASCE 7-10 Reference
Mapped MCE_G Peak Ground Acceleration, PGA	0.404 g	Figure 22-7
Site Coefficient, F_{PGA}	1.000	Table 11.8-1
Site Class Modified MCE_G Peak Ground Acceleration, PGA_M	0.404 g	Section 11.8.3 (Eqn 11.8-1)

6.4.3 Conformance to the criteria in Tables 6.4.1 and 6.4.2 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.

6.5 Foundation and Concrete Slabs-On-Grade Recommendations

6.5.1 The site is suitable for the use of shallow foundations. Foundations can consist of continuous strip footings and/or isolated spread footings. Continuous footings should be at least 12 inches wide and extend at least 24 inches below lowest adjacent pad grade. Isolated spread footings should have a minimum width and depth of 2 feet.

6.5.2 Minimum concrete-reinforcement for continuous footings should consist of at least four, No. 5, steel, bars placed horizontally in the footings; two near the top and two near the bottom. Concrete-reinforcement for the spread footings should be designed by the project structural engineer. A typical wall/column footing dimension detail is presented on Figure 3.

6.5.3 The minimum concrete-reinforcement recommended herein is based on soil characteristics only (EI of 90 or less) and is not intended to replace reinforcement required for structural considerations.

6.5.4 Foundations as proportioned above may be designed for an allowable soil bearing pressure of 3,000 pounds per square foot (psf). The recommended allowable soil bearing pressures may be increased by 300 psf and 500 psf for each additional foot of foundation width and depth, respectively, up to a maximum allowable soil bearing pressure of 4,000 psf.

6.5.5 The values presented herein are for dead plus live loads and may be increased by one-third when considering transient loads due to wind or seismic forces.

6.5.6 Settlement due to footing loads conforming to the above recommended allowable soil bearing pressures are expected to be less than 1-inch total and ½-inch differential over a span of 40 feet.

6.5.7 A modulus of subgrade reaction of 150 pounds per cubic inch (pci) to 200 pci can be used for design. This modulus value is for a foundation measuring 1 foot by 1 foot and should be modified for design using standard equations.

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

6.5.9 Footings should not be located within 7 feet of the tops of slopes. Footings that must be located within this zone should be extended in depth such that the outer bottom edge of the footing is at least 7 feet horizontally from the face of the finished slope.

6.5.10 No special subgrade presaturation (i.e., flooding to saturate soils to foundation depths to mitigate highly expansive soils) is deemed necessary prior to placement of concrete. However, the slab and foundation subgrade should be sprinkled as necessary to maintain a moist condition as would be expected in any concrete placement.

6.5.11 Foundation excavations should be observed by the geotechnical engineer prior to the placement of reinforcing steel and concrete to verify that the exposed soil conditions are consistent with those expected and have been extended to appropriate bearing strata.

6.5.12 New concrete slabs-on-grade should be at least 5 inches thick and be reinforced with No. 3 steel bars placed 18 inches on center in both directions. The concrete slab-on-grade recommendations are based on soil support characteristics only. The project structural engineer should evaluate the structural requirements of the concrete slabs for supporting planned loading. Thicker concrete slabs may be required for heavier loads.

6.5.13 A vapor retarder should be placed beneath slabs having moisture-sensitive floor coverings or that may be used to store moisture-sensitive materials. The project architect should specify the type of vapor retarder used based on the type of floor covering that will be installed. The vapor retarder design should be consistent with the guidelines presented in Section 9.3 of the

American Concrete Institute's (ACI) *Guide for Concrete Slabs that Receive Moisture-Sensitive Flooring Materials* (ACI 302.2R-06). As indicated in the ACI guide, reduced joint spacing, a low shrinkage mix design, or other measures to minimize slab curl will be required where the concrete is placed directly on the vapor barrier.

6.5.14 The project foundation engineer or architect should determine the thickness of bedding sand below the slab. In general, 3 to 4 inches of sand bedding is typically used. Geocon should be contacted to provide recommendations if the bedding sand is thicker than 6 inches.

6.5.15 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 plan. It is critical that the foundation contractor understands and follows the specifications presented on the foundation plan.

6.5.16 As an alternative to the conventional foundation recommendations, consideration should be given to the use of post-tensioned concrete slab and foundation systems for the support of the proposed structure. The post-tensioned systems should be designed by a structural engineer experienced in post-tensioned slab design and design criteria of the Post-Tensioning Institute (PTI) DC 10.5-12 *Standard Requirements for Design and Analysis of Shallow Post-Tensioned Concrete Foundations on Expansive Soils* or *WRI/CRSI Design of Slab-on-Ground Foundations*, as required by the 2016 California Building Code (CBC Section 1808.6.2). Although this procedure was developed for expansive soil conditions, it can also be used to reduce the potential for foundation distress due to differential fill settlement. The post-tensioned design should incorporate the geotechnical parameters presented in Table 6.5.1. The parameters presented in Table 6.5.1 are based on the guidelines presented in the PTI DC 10.5 design manual.

TABLE 6.5.1
POST-TENSIONED FOUNDATION SYSTEM DESIGN PARAMETERS

Post-Tensioning Institute (PTI) Third Edition Design Parameters	
Thornthwaite Index	-20
Equilibrium Suction	3.9
Edge Lift Moisture Variation Distance, e_M (feet)	4.9
Edge Lift, y_M (inches)	1.58
Center Lift Moisture Variation Distance, e_M (feet)	9.0
Center Lift, y_M (inches)	0.66

6.5.17 The foundations for the post-tensioned slabs should be embedded in accordance with the recommendations of the structural engineer. If a post-tensioned mat foundation system is planned, the slab should possess a thickened edge with a minimum width of 12 inches and extend below the clean sand or crushed rock layer.

6.5.18 If the structural engineer proposes a post-tensioned foundation design method other than PTI DC 10.5:

- The deflection criteria presented in Table 6.5.1 are still applicable.
- Interior stiffener beams should be used.
- The width of the perimeter foundations should be at least 12 inches.
- The perimeter footing embedment depths should be at least 24 inches. The embedment depths should be measured from the lowest adjacent pad grade.

6.5.19 Post-tensioned slabs are susceptible to excessive edge lift, regardless of the underlying soil conditions. Placing reinforcing steel at the bottom of the perimeter footings and the interior stiffener beams may mitigate this potential. The placement of the reinforcing tendons in the top of the slab and the resulting eccentricity after tensioning could reduce the ability of the system to mitigate edge lift. The structural engineer should design the foundation system to reduce the potential of edge lift occurring for the proposed structures.

6.5.20 During the construction of the post-tension foundation system, the concrete should be placed monolithically. Under no circumstances should cold joints form between the footings/grade beams and the slab during the construction of the post-tension foundation system.

6.5.21 Isolated footings outside of the post-tensioned slab area, if present, should have the minimum embedment depth and width recommended for conventional foundations. The use of isolated footings, which are located beyond the perimeter of the building and support structural elements connected to the building, are not recommended. Where this condition cannot be avoided, the isolated footings should be connected to the building foundation system with grade beams.

6.5.22 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.

6.5.23 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. A post-tensioned slab and foundation system or mat foundation system can be used to help reduce potential foundation distress associated with slope creep and lateral fill extension. Specific design parameters or recommendations for either of these alternatives can be provided if desired.

- 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 which would permit some lateral soil movement without causing extensive distress. Geocon Incorporated should be consulted for specific recommendations.

6.5.24 Exterior slabs and hardscape not subject to vehicle loads should be at least 4 inches thick and reinforced with No. 3 steel, bars, placed 24 inches on center in both directions at the slab midpoint. 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 as determined by the current version of ASTM D1557. Where expansive clay soils are present at finish grade, the subgrade soil should be moisture conditioned to 3 to 5 percent above optimum moisture content and compacted.

6.5.25 To control the location and spread of concrete shrinkage and/or expansion cracks, it is recommended that crack-control joints be included in the design of concrete slabs. Crack-control joint spacing should not exceed, in feet, twice the recommended slab thickness in inches (e.g., 10 feet by 10 feet for a 5-inch-thick slab). Crack-control joints should be created while the concrete is still fresh using a grooving tool or shortly thereafter using saw cuts. The structural engineer should take criteria of the American Concrete Institute into consideration when establishing crack-control spacing patterns.

6.5.26 The recommendations presented in 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. Their occurrence may be reduced and/or controlled by limiting the slump of the concrete, proper concrete placement and curing, and by the placement of crack control joints at periodic intervals, in particular, where re-entrant slab corners occur.

6.5.27 Geocon Incorporated should be consulted to provide additional design parameters as required by the structural engineer.

6.6 Retaining Walls and Lateral Loads

6.6.1 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. Expansive soil should not be used as backfill material behind retaining walls. Soil placed for retaining wall backfill should have an Expansion Index less than 50.

6.6.2 Where walls are restrained from movement at the top, an additional uniform pressure of 8H psf (where H equals the height of the retaining wall portion of the wall in feet) should be added to the active soil pressure where the wall possesses a height of 8 feet or less and 12H where the wall is greater than 8 feet. For retaining walls subject to vehicular loads within a horizontal distance equal to two-thirds the wall height, a surcharge equivalent to two feet of fill soil should be added.

6.6.3 Soil contemplated for use as retaining wall backfill, including imported soils, should be identified in the field prior to backfill. At that time Geocon Incorporated should obtain samples for laboratory testing to evaluate its suitability. Modified lateral earth pressures may be necessary if the backfill soil does not meet the required expansion index or shear strength. City or regional standard wall designs, if used, are based on a specific active lateral earth pressure and/or soil friction angle. In this regard, on-site soil or import soil to be used as backfill may or may not meet the values for standard wall designs. Geocon Incorporated should be consulted to assess the suitability of the on-site soil or import soil for use as wall backfill if standard wall designs will be used.

6.6.4 Unrestrained walls will move laterally when backfilled and loading is applied. The amount of lateral deflection is dependent on the wall height, the type of soil used for backfill, and loads acting on the wall. The wall designer should provide appropriate lateral deflection quantities for planned retaining walls structures, if applicable. These lateral values should be considered when planning types of improvements above retaining wall structures.

6.6.5 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 $\leq$50) free-draining backfill material with no hydrostatic forces or imposed surcharge load. A typical retaining wall drainage detail is presented on Figure 4. If conditions different than those described are expected, or if specific drainage details are desired, Geocon Incorporated should be contacted for additional recommendations.

6.6.6 In general, wall foundations having a minimum embedment depth and width of 1 foot may be designed for an allowable soil bearing pressure of 2,500 psf. The allowable soil bearing pressure may be increased by an additional 300 psf and 500 psf for each additional foot of foundation width and depth, respectively, to a maximum allowable bearing capacity of 4,000 psf. These values are for dead plus live loads and may be increased by one-third when considering transient loads due to wind or seismic forces.

6.6.7 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 anticipated. As a minimum, wall footings should be deepened such that the bottom outside edge of the footing is at least seven feet from the face of slope when located adjacent and/or at the top of descending slopes.

6.6.8 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 1803.5.12 of the 2016 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 13H should be used for design. We used the peak ground acceleration adjusted for Site Class effects, PGA_M, of 0.404 g calculated from ASCE 7-10 Section 11.8.3 and applied a pseudo-static coefficient of 0.33.

6.6.9 For resistance to lateral loads, a passive earth pressure equivalent to a fluid density of 300 pcf is recommended for footings or shear keys poured neat against properly compacted granular fill soils or undisturbed formation materials. The passive pressure assumes a horizontal surface extending away from the base of the wall at least five feet or three times the surface generating the passive pressure, whichever is greater. The upper 12 inches of material not protected by floor slabs or pavement should not be included in the design for lateral resistance. Where walls are planned adjacent to and/or on descending slopes, a passive pressure of 150 pcf should be used in design.

6.6.10 An allowable friction coefficient of 0.4 may be used for resistance to sliding between soil and concrete. This friction coefficient may be combined with the passive earth pressure when determining resistance to lateral loads.

6.7 Preliminary Flexible and Rigid Pavement Recommendations

6.7.1 Preliminary pavement recommendations for the driveways and parking lots are provided below. The final pavement sections should be based on the R-Value of the subgrade soil encountered at final subgrade elevation. For preliminary design, we used a laboratory R-Value of 5 (value from laboratory testing performed for this study). We calculated the preliminary flexible pavement sections for asphalt concrete using varying traffic indices (TIs) in general conformance with the *Caltrans Method of Flexible Pavement Design* (Highway Design Manual, Section 608.4). The project civil engineer or traffic engineer should determine the appropriate Traffic Index (TI) or traffic loading expected on the project for the various pavement areas that will be constructed. Recommended preliminary asphalt concrete pavement sections are provided on Table 6.7.1.

TABLE 6.7.1
PRELIMINARY ASPHALT CONCRETE PAVEMENT SECTIONS

Traffic Index	Asphalt Concrete (inches)	Class 2 Base (inches)
4.5	3	8
5	3	10
5.5	3	11.5
6	3.5	12.5
6.5	3.5	14.5
7	4	15.5
7.5	4	17.5
8	5	17.5

6.7.2 The use of geotextile reinforcing grid can be utilized to reduce the base section. Table 6.7.2 provides alternative design sections utilizing Tensar TX8 geogrid installed at the bottom of the base section.

TABLE 6.7.1
PRELIMINARY ASPHALT CONCRETE PAVEMENT SECTIONS
UTILIZING TENSAR TX8 GEOGRID

Traffic Index	Asphalt Concrete (inches)	Class 2 Base (inches)
4.5	3	4
5	3	5.5
5.5	3	7
6	3.5	7.5
6.5	3.5	9
7	4	9.5
7.5	4	11
8	5	10.5

6.7.3 The geogrid should be installed in accordance with manufacture's recommendations. Adjacent sections of geogrid should be overlapped at least 3 feet on the sides and ends. Care needs to be taken during base placement so damage to the grid does not occur.

6.7.4 Asphalt concrete should conform to Section 203-6 of the *Standard Specifications for Public Works Construction* (Green Book). Class 2 aggregate base materials should conform to

Section 26-1.02B of the *Standard Specifications of the State of California, Department of Transportation* (Caltrans).

6.7.5 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 a density of at least 95 percent of the laboratory Hveem density in accordance with ASTM D 2726.

6.7.6 A rigid Portland Cement concrete (PCC) pavement section should be placed in loading docks, 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 6.7.2.

TABLE 6.7.2
PRELIMINARY RIGID PAVEMENT DESIGN PARAMETERS

Design Parameter	Design Value
Modulus of subgrade reaction, k	50 pci
Modulus of rupture for concrete, M_R	500 psi
Traffic Category, TC	A and C
Average daily truck traffic, ADTT	1 and 300

6.7.7 Based on the criteria presented herein, the PCC pavement sections should have a minimum thickness as presented in Table 6.7.3.

TABLE 6.7.3
PRELIMINARY RIGID PAVEMENT RECOMMENDATIONS

Location	Portland Cement Concrete (inches)	Class 2 Base (inches)
Automobile Areas (TC=A, ADDT = 1)	5.5	4
Heavy Truck and Fire Lane Areas (TC=C, ADDT = 300)	8.0	4

6.7.8 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. This pavement section is based on a minimum concrete compressive strength of approximately 3,200 psi (pounds per square inch).

6.7.9 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, at the slab edge and taper back to the recommended slab thickness 3 feet behind the face of the slab (e.g., a 8-inch-thick slab would have a 9.6-inch-thick edge). Reinforcing steel will not be necessary within the concrete for geotechnical purposes with the exception of loading docks, trash bin enclosures, and dowels at construction joints as discussed below.

6.7.10 Loading aprons, such as those used for trash bin enclosures and loading docks, should be constructed using Portland cement concrete as recommended above for heavy truck traffic areas. The pavement should be reinforced with a minimum of No. 3 steel, bars, spaced 24 inches on center in both directions placed at the slab midpoint. The concrete should extend out from the loading dock or trash bin such that both the front and rear wheels of the truck will be located on reinforced concrete pavement when loading.

6.7.11 To control the location and spread of concrete shrinkage cracks, crack-control joints (weakened plane joints) should be included in the design of the concrete pavement slab. Crack-control joints should not exceed 30 times the slab thickness with a maximum spacing of 15 feet (e.g., a 7-inch-thick slab would have a 15-foot spacing pattern) and should be sealed with an appropriate sealant to prevent the migration of water through the control joint to the subgrade materials. The depth of the crack-control joints should be determined by the referenced ACI report.

6.7.12 To provide load transfer between adjacent pavement slab sections, a trapezoidal-keyed construction joint should be installed. As an alternative to the keyed joint, dowelling is recommended between construction joints. As discussed in the referenced ACI guide, dowels should consist of smooth, ⅞-inch-diameter reinforcing steel 14 inches long embedded a minimum of 6 inches into the slab on either side of the construction joint. Dowels should be located at the midpoint of the slab, spaced at 12 inches on center and lubricated to allow joint movement while still transferring loads. The project structural engineer may provide alternative recommendations for load transfer.

6.7.13 The performance of pavement is highly dependent on providing positive surface drainage away from the edge of the pavement. Ponding of water on or adjacent to the pavement will likely result in pavement distress and subgrade failure. Drainage from landscaped areas should be directed to controlled drainage structures. Landscape areas adjacent to the edge of asphalt pavements are not recommended due to the potential for surface or irrigation water to infiltrate the underlying permeable aggregate base and cause distress. Where such a condition cannot be avoided, consideration should be given to incorporating measures that will significantly reduce the potential for subsurface water migration into the aggregate base. If planter islands are planned, the perimeter curb should extend at least 6 inches below the level of the base materials.

6.8 Slope Maintenance

6.8.1 Slopes that are steeper than 3:1 (horizontal:vertical) may, under conditions which are both difficult to prevent and predict, be susceptible to near surface (surficial) slope instability. The instability is typically limited to the outer three feet of a portion of the slope and usually does not directly impact the improvements on the pad areas above or below the slope. The occurrence of surficial instability is more prevalent on fill slopes and is generally preceded by a period of heavy rainfall, excessive irrigation, or the migration of subsurface seepage. The disturbance and/or loosening of the surficial soils, as might result from root growth, soil expansion, or excavation for irrigation lines and slope planting, may also be a significant contributing factor to surficial instability. It is, therefore, recommended that, to the maximum extent practical: (a) disturbed/loosened surficial soils be either removed or properly recompacted, (b) irrigation systems be periodically inspected and maintained to eliminate leaks and excessive irrigation, and (c) surface drains on and adjacent to slopes be periodically maintained to preclude ponding or erosion. It should be noted that although the incorporation of the above recommendations should reduce the potential for surficial slope instability, it will not eliminate the possibility, and, therefore, it may be necessary to rebuild or repair a portion of the project's slopes in the future.

6.9 Storm Water Management

6.9.1 If storm water management devices are not properly designed and constructed, there is a risk for distress to improvements and property located hydrologically down gradient or adjacent to these devices. Factors such as the amount of water being detained, its residence time, and soil permeability have an important effect on seepage transmission and the potential adverse impacts that may occur if the storm water management features are not properly designed and constructed. We have not performed a hydrogeological study at the site. If infiltration of storm water runoff into the subsurface occurs, downstream improvements may be subjected to seeps, springs, slope instability, raised groundwater, movement of foundations and slabs, or other undesirable impacts as a result of water infiltration.

6.9.2 We performed an infiltration study on the property. A summary of our study and storm water management recommendations are provided in Appendix C. Based on the results of our study, full and partial infiltration is considered infeasible.

6.10 Site Drainage and Moisture Protection

6.10.1 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 2016 CBC 1804.4 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.

6.10.2 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.

6.10.3 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.

6.10.4 Landscaping planters adjacent to paved areas are not recommended due to the potential for surface or irrigation water to infiltrate the pavement's subgrade and base course. We recommend that subdrains to collect excess irrigation water and transmit it to drainage structures or impervious above-grade planter boxes be used. In addition, where landscaping is planned adjacent to the pavement, we recommend construction of a cutoff wall along the edge of the pavement that extends at least 6 inches below the bottom of the base material.

6.11 Grading and Foundation Plan Review

6.11.1 Geocon Incorporated should review the grading and foundation plans for the project prior to final design submittal to determine if additional analysis and/or recommendations are required.

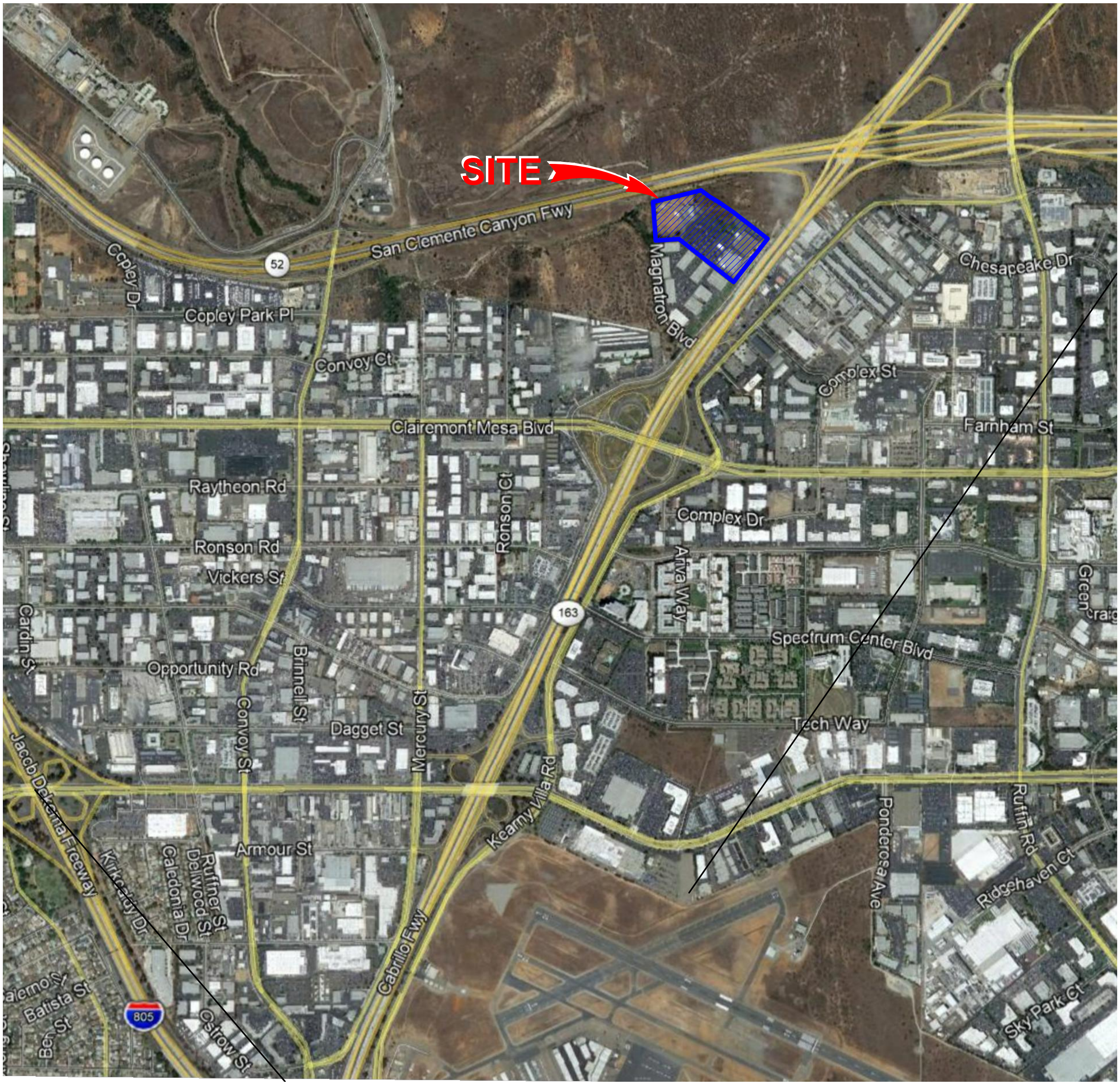
LIMITATIONS AND UNIFORMITY OF CONDITIONS

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 of his representative, to ensure that the information and recommendations contained herein are brought to the attention of the architect and engineer for the project and incorporated into the plans, and that 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 are due to natural processes or the works of man on this or adjacent properties. In addition, changes in applicable or appropriate standards may occur, whether they result from legislation or the broadening of knowledge. Accordingly, the findings of this report may be invalidated wholly or partially by changes outside our control. Therefore, this report is subject to review and should not be relied upon after a period of three years.



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N

NO SCALE

VICINITY MAP

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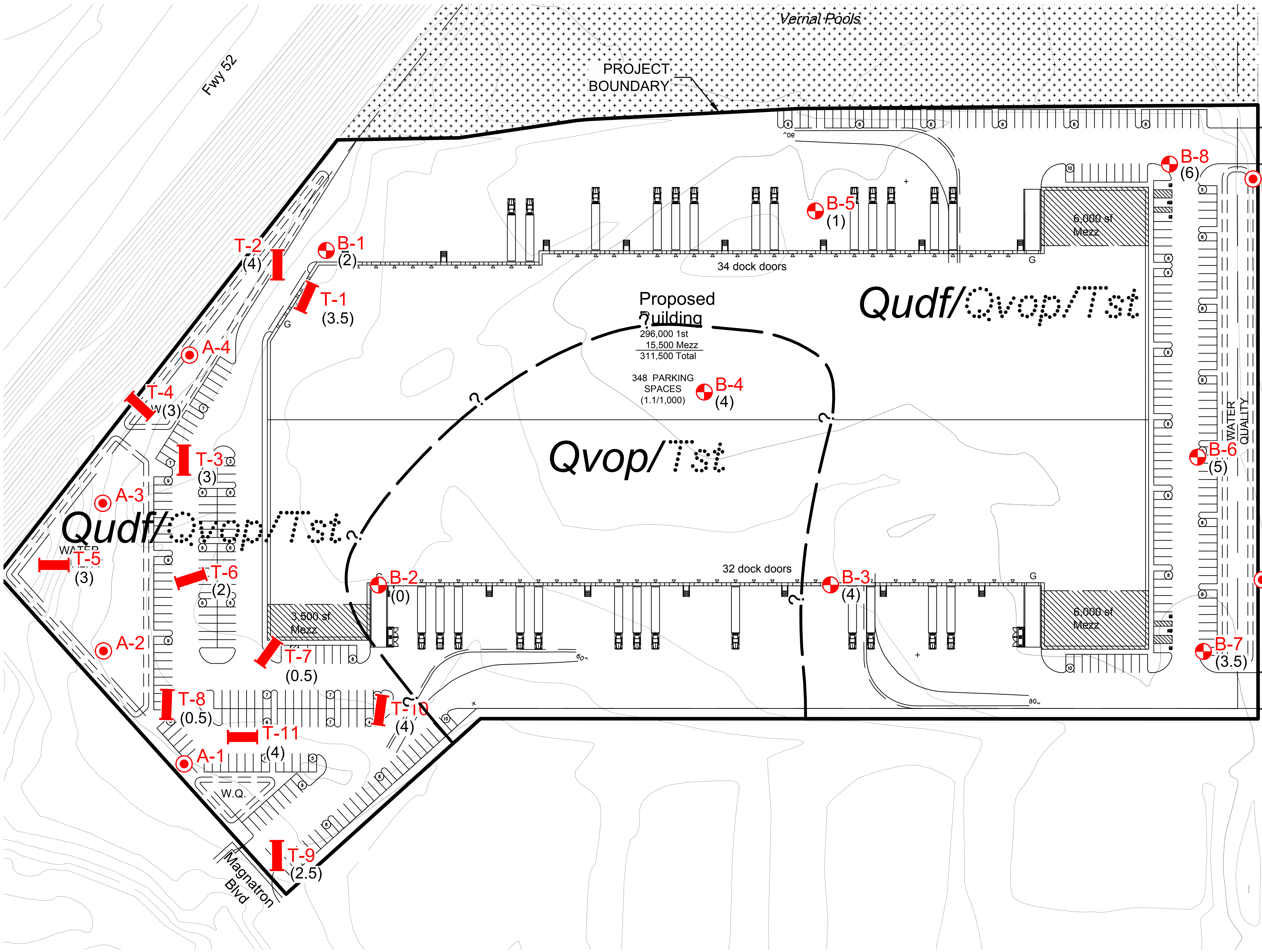
5650 AND 5660 KEARNY MESA
SAN DIEGO, CALIFORNIA

RM / AML		DSK/GTYPD	DATE 04 - 23 - 2019	PROJECT NO. G2389 - 42 - 01	FIG. 1

Plotted:04/22/2019 3:17PM | By:ALVIN LADRILLONO | File Location:Y:\PROJECTS\G2389-42-01 (5650 and 5660 Kearny Mesa Rd)\DETAILS\G2389-42-01 Vic Map.dwg



5650 AND 5660 KEARNY MESA
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GEOCON LEGEND

- Qudf ........UNDOCUMENTED FILL
- Qvop ....VERY OLD PARALIC DEPOSITS (Dotted Where Buried)
- Tst ....STADIUM CONGLOMERATE (Dotted Where Buried)
- ....APPROX. LOCATION OF GEOLOGIC CONTACT (Queried Where Uncertain)
- B-8 ....APPROX. LOCATION OF BORING
- T-11 ....APPROX. LOCATION OF TRENCH
- A-6 ....APPROX. LOCATION OF INFILTRATION TEST
- (4) ....APPROX. DEPTH OF SURFICIAL SOIL

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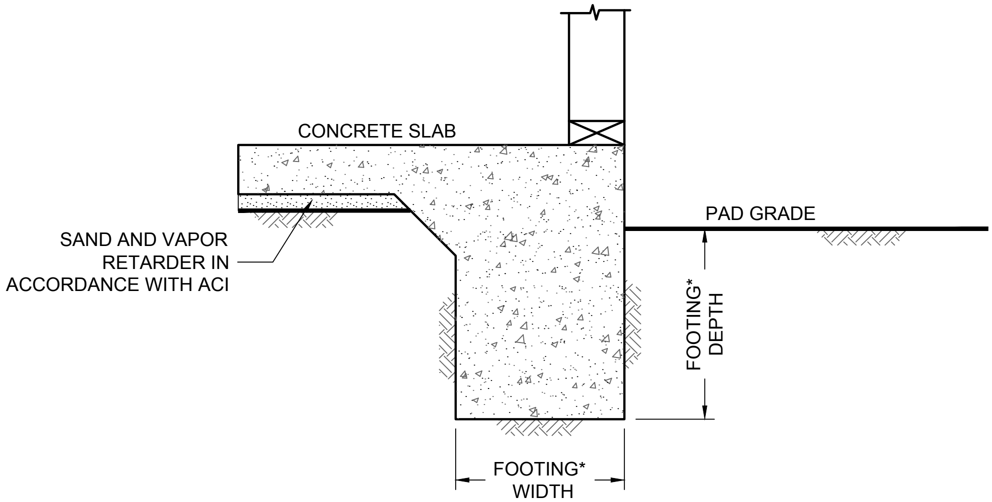
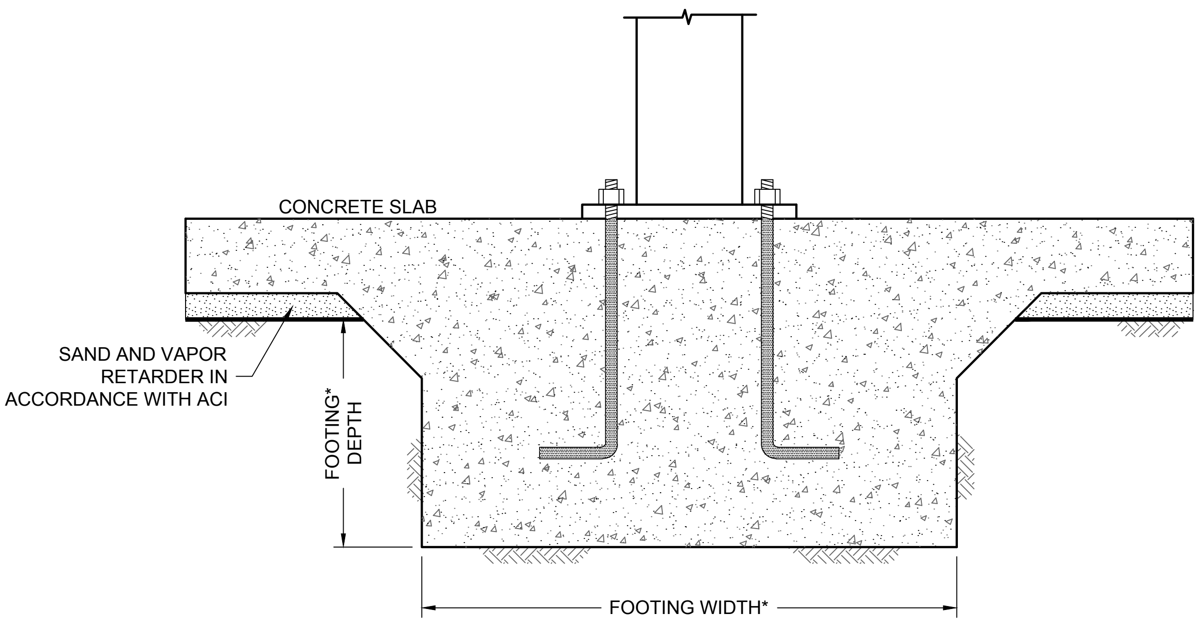
GEOLOGIC MAP

PROJECT NO. G2389 - 42 - 04
FIGURE 2
DATE 04 - 23 - 2019





*....SEE REPORT FOR FOUNDATION WIDTH AND DEPTH RECOMMENDATION

NO SCALE

WALL / COLUMN FOOTING DIMENSION DETAIL

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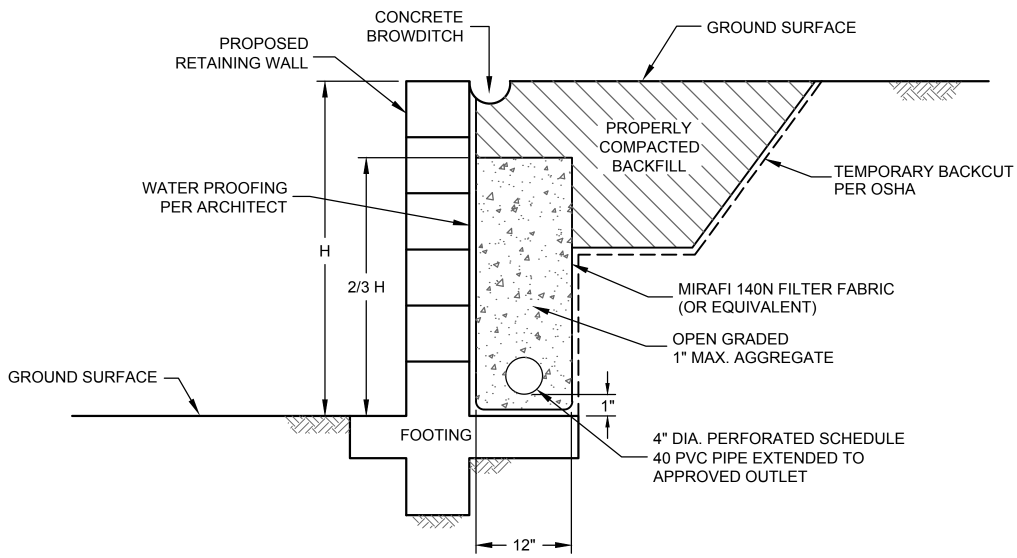
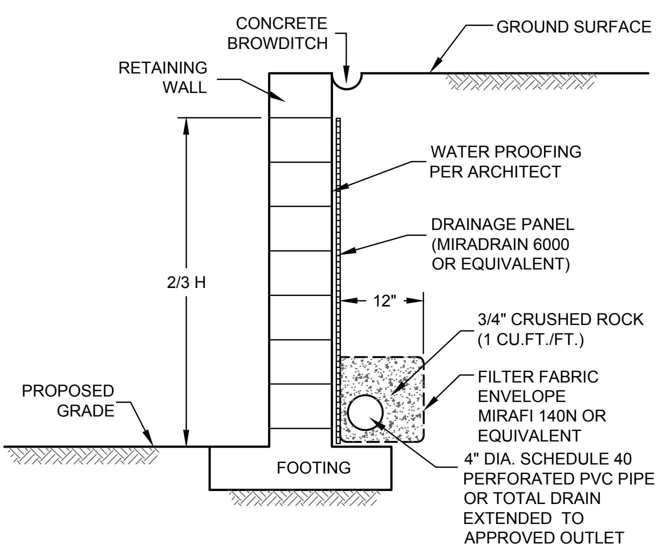
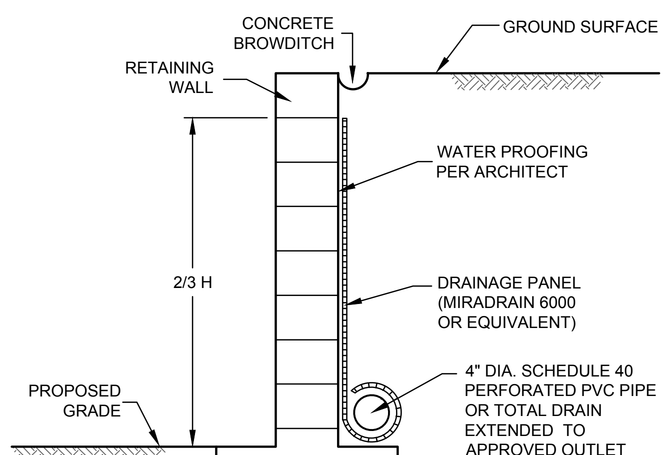
RM / AML		DSK/GTYPD	DATE 04 - 23 - 2019	PROJECT NO. G2389 - 42 - 01	FIG. 3







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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RM / AML		DSK/GTYPD	DATE 04 - 23 - 2019	PROJECT NO. G2389 - 42 - 01	FIG. 4

Plotted:04/22/2019 3:16PM | By:ALVIN LADRILLONO | File Location:Y:\PROJECTS\G2389-42-01 (5650 and 5660 Kearny Mesa Rd)\DETAILS\Typical Retaining Wall Drainage Detail (RWDD7A).dwg

APPENDIX A

APPENDIX A

FIELD INVESTIGATION

The field investigation was performed on April 4 and 5, 2019, and consisted of drilling eight, small-diameter borings and excavating 11 exploratory trenches. The approximate locations of the borings and trenches are shown on the Geologic Map, Figure 2. The boring and trench locations were determined in the field based on visual reference points; therefore, actual locations may deviate slightly. Logs of the borings and trenches are presented as Figures A-1 through A-19. The soil encountered 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). The logs depict the soil and geologic conditions observed and the depth at which samples were obtained.

We also performed six, in-place, hydraulic conductivity tests. The infiltration tests were conducted in 8-inch drilled borings ranging in depths from 5 to 10 feet below existing ground surface. Results from the infiltration testing are presented in Appendix C.

BORING B 1

ELEV. (MSL.) 416' DATE COMPLETED 04-04-2019

EQUIPMENT CME 75 BY: N. 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				SM/SC	**UNDOCUMENTED FILL (Qudf)** Loose, wet, dark brown, Silty to Clayey, fine to coarse SAND; few gravel and cobble			
	B1-1							
2				SM	**VERY OLD PARALIC DEPOSITS (Qvop)** Very dense, moist, brown to reddish brown, Silty, fine to coarse SAND; few gravel and cobble			
	B1-2					96/9"		
4								
	B1-3					75	106.3	13.3
				SC	Dense to very dense, moist, light gray and yellowish brown, Clayey, fine to medium SAND; trace gravel and cobble			
6								
8								
				SM	**STADIUM CONGLOMERATE (Tst)** Dense to very dense, damp, light yellowish brown, Silty, fine to medium SANDSTONE; trace gravel			
10	B1-4					71		
12								
14								
	B1-5					76		
16					BORING TERMINATED AT 16 FEET Groundwater not encountered Backfilled on 04-04-2019			

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

G2389-42-01.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. G2389-42-01

BORING B 2

ELEV. (MSL.) 417' DATE COMPLETED 04-04-2019

EQUIPMENT CME 75 BY: N. 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				SM	**VERY OLD PARALIC DEPOSITS (Qvop)** Very dense, damp, dark reddish brown, Silty, fine to coarse SAND; few clay; little gravel and cobble			
2								
	B2-1					50/6"		
					-Becomes dry to damp, yellowish brown and reddish brown			
4								
	B2-2					50/6"		
6	B2-3							
8								
				SC	Medium dense, moist, mottled reddish brown and gray, Clayey, fine SAND			
10	B2-4					26		
12								
14								
	B2-5					64		
16					BORING TERMINATED AT 16 FEET Groundwater not encountered Backfilled on 04-04-2019			

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

G2389-42-01.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

BORING B 3

ELEV. (MSL.) 419' DATE COMPLETED 04-04-2019

EQUIPMENT CME 75 BY: N. 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					5" **ASPHALT** over 5" coarse **SAND/GRAVEL**			
	B3-1			CL	**UNDOCUMENTED FILL (Qudf)** Stiff, moist, dark gray to black, Silty CLAY; trace gravel			
2				CL	**VERY OLD PARALIC DEPOSITS (Qvop)** Stiff, moist, mottled reddish brown and dark olive brown, Sandy CLAY			
	B3-2					35		
4				SM	Very dense, damp to moist, reddish brown, Silty, fine to coarse SAND; few gravel and cobble			
	B3-3					69	109.5	16.0
6								
8					-Gravel and cobble			
10	B3-4				-Cobble	50/6"		
	B3-5							
12								
14					**STADIUM CONGLOMERATE (Tst)** Very dense, damp, moist, light yellowish brown and light gray, Silty, fine to coarse SANDSTONE; trace gravel			
	B3-6					85/11"		
16					BORING TERMINATED AT 16 FEET Groundwater not encountered Backfilled on 04-04-2019			

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

G2389-42-01.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. G2389-42-01

BORING B 4

ELEV. (MSL.) 422' DATE COMPLETED 04-04-2019

EQUIPMENT CME 75 BY: N. 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					3" **ASPHALT** over **SUBGRADE**			
				CL	**VERY OLD PARALIC DEPOSITS (Qvop)**			
	B4-1				Very stiff, moist to wet, dark reddish brown, Sandy CLAY			
2								
	B4-2				-Becomes damp, reddish brown	60		
4				SM	Medium dense, damp to moist, dark reddish brown, Silty, fine to medium SAND; few gravel and cobble			
	B4-3					37	112.5	15.9
6								
				SC	Dense to very dense, moist, reddish brown to brown, Clayey, fine to coarse SAND; little gravel and cobble			
8								
10								
	B4-4					77/9"		

BORING TERMINATED AT 11 FEET
Groundwater not encountered
Backfilled on 04-04-2019

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

G2389-42-01.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

BORING B 5

ELEV. (MSL.) 422' DATE COMPLETED 04-04-2019

EQUIPMENT CME 75 BY: N. 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				SM/SC	**UNDOCUMENTED FILL (Qudf)** Loose, moist, olive brown, Silty to Clayey, fine to coarse SAND			
				SM	**VERY OLD PARALIC DEPOSITS (Qvop)** Very dense, damp to moist, reddish brown, Silty, fine to coarse SAND; little gravel and cobble; poor recovery			
2	B5-1					50/3"		
4	B5-2				-Cobble	50/3"		
6								
8								
10	B5-3				-Cobble	93/9"		
					BORING TERMINATED AT 11 FEET Groundwater not encountered Backfilled on 04-04-2019			

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

G2389-42-01.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

BORING B 6

ELEV. (MSL.) **422'** DATE COMPLETED **04-04-2019**

EQUIPMENT **CME 75** BY: **N. 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					7" **ASPHALT** over **SUBGRADE**			
	B6-1			SC	**UNDOCUMENTED FILL (Qudf)** Medium dense, moist, grayish brown, Clayey, fine to coarse SAND; some gravel and cobble			
2								
	B6-2				-Cobble	50/6"		
4								
	B6-3			SM	**VERY OLD PARALIC DEPOSITS (Qvop)** Very dense, damp, reddish brown, Silty, fine to coarse SAND; few clay; little gravel and cobble; hard drilling	93/10"		
6								
8								
10	B6-4				-Cobble	50/5"		
					BORING TERMINATED AT 10.5 FEET Groundwater not encountered Backfilled on 04-04-2019			

Figure A-6,
Log of Boring B 6, Page 1 of 1

G2389-42-01.GPJ

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

BORING B 7

ELEV. (MSL.) **421'** DATE COMPLETED **04-04-2019**

EQUIPMENT **CME 75** BY: **N. 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	B7-1				3" **ASPHALT** over 5" **AGGREGATE BASE**	80		
				CL	**UNDOCUMENTED FILL (Qudf)** Stiff, moist, dark gray to black, Sandy to Silty CLAY			
2				CL	**VERY OLD PARALIC DEPOSITS (Qvop)** Hard, moist, mottled gray and reddish brown, Sandy CLAY			
4				SM	Very dense, damp to moist, reddish brown, Silty, fine to medium SAND; little gravel and cobble			
6	B7-2					75	114.7	15.1
8								
10	B7-3				-Cobble	50/3"		
12								
14								
	B7-4				-Cobble	50/2"		
					BORING TERMINATED AT 15.5 FEET Groundwater not encountered Backfilled on 04-04-2019			

Figure A-7,
Log of Boring B 7, Page 1 of 1

G2389-42-01.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

BORING B 8

ELEV. (MSL.) **423'** DATE COMPLETED **04-04-2019**

EQUIPMENT **CME 75** BY: **N. 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					3" **ASPHALT** over 4" **AGGREGATE BASE**			
	B8-1			CL	**UNDOCUMENTED FILL (Qudf)** Stiff, moist, gray, Silty to Sandy CLAY; trace gravel			
2								
	B8-2					21		
4								
	B8-3				-Becomes firm to stiff	16		
6				SM	**VERY OLD PARALIC DEPOSITS (Qvop)** Very dense, damp, yellowish brown to reddish brown, Silty, fine to medium SAND; little gravel and cobble			

REFUSAL AT 7 FEET
Groundwater not encountered
Backfilled on 04-04-2019

Figure A-8,
Log of Boring B 8, Page 1 of 1

G2389-42-01.GPJ

SAMPLE SYMBOLS

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

TRENCH T 1

ELEV. (MSL.) **416'** DATE COMPLETED **04-05-2019**

EQUIPMENT **CME 75** BY: **N. 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				SM	**UNDOCUMENTED FILL (Qudf)** Loose, damp, brown, Silty, fine to medium SAND; gravel and cobble up to 6-inch in diameter			
2					-Becomes moist, dark, reddish brown			
4				SM	**VERY OLD PARALIC DEPOSITS (Qvop)** Very dense, damp, reddish brown, Silty, fine to coarse SAND; little gravel and cobble; cemented			

REFUSAL AT 4 FEET
Groundwater not encountered

Figure A-9,
Log of Trench T 1, Page 1 of 1

G2389-42-01.GPJ

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

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DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	TRENCH T 2 ELEV. (MSL.) 413' DATE COMPLETED 04-05-2019 EQUIPMENT CME 75 BY: N. BORJA	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
					MATERIAL DESCRIPTION			
0				SM	**UNDOCUMENTED FILL (Qudf)** Loose, moist, dark brown, Silty, fine to medium SAND; gravel			
2								
					-Becomes saturated, brown; roots			
4				SM	**VERY OLD PARALIC DEPOSITS (Qvop)** Very dense, damp, reddish brown, Silty, fine to coarse SAND; little gravel and cobble; cemented			
					REFUSAL AT 4.5 FEET Groundwater not encountered			

Figure A-10,
Log of Trench T 2, Page 1 of 1

G2389-42-01.GPJ

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

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PROJECT NO. G2389-42-01

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	TRENCH T 3 ELEV. (MSL.) 414' DATE COMPLETED 04-05-2019 EQUIPMENT CME 75 BY: N. BORJA	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
					MATERIAL DESCRIPTION			
0				SM	**UNDOCUMENTED FILL (Qudf)** Loose, damp, brown, Silty, fine to coarse SAND; gravel and cobble			
2				SC	Loose, moist, brown to reddish brown, Clayey, fine to medium SAND; gravel; trace cobble up to 8-inch in diameter			
				CL	**VERY OLD PARALIC DEPOSITS (Qvop)** Firm, moist, gray, Sandy to Silty CLAY			
4				SM	Dense to very dense, damp, reddish brown, Silty, fine to coarse SAND; gravel and cobble up to 8-inch in diameter; cemented			
					REFUSAL AT 4.5 FEET Groundwater not encountered			

Figure A-11,
Log of Trench T 3, Page 1 of 1

G2389-42-01.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. G2389-42-01

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	TRENCH T 4 ELEV. (MSL.) 410' DATE COMPLETED 04-05-2019 EQUIPMENT CME 75 BY: N. BORJA	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
					MATERIAL DESCRIPTION			
0				SM	**UNDOCUMENTED FILL (Qudf)** Loose, damp to moist, brown to dark brown, Silty, fine to coarse SAND; gravel and cobble			
2								
				SM	**VERY OLD PARALIC DEPOSITS (Qvop)** Very dense, damp, reddish brown, Silty, fine to coarse SAND; gravel and cobble up to 6-inch diameter			
4					REFUSAL AT 4 FEET Groundwater not encountered			

Figure A-12,
Log of Trench T 4, Page 1 of 1

G2389-42-01.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

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	TRENCH T 5 ELEV. (MSL.) 408' DATE COMPLETED 04-05-2019 EQUIPMENT CME 75 BY: N. BORJA	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
					MATERIAL DESCRIPTION			
0				SM/SC	**UNDOCUMENTED FILL (Qudf)** Loose, damp to moist, brown to dark brown, Silty to Clayey, fine to medium SAND; gravel and cobble up to 8-inch diameter			
2								
4				SM/SC	**VERY OLD PARALIC DEPOSITS (Qvop)** Dense, damp, reddish brown, Silty to Clayey, fine to medium SAND; gravel and cobble			
6					REFUSAL AT 6 FEET Groundwater not encountered			

Figure A-13,
Log of Trench T 5, Page 1 of 1

G2389-42-01.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.

TRENCH T 6

ELEV. (MSL.) **416'** DATE COMPLETED **04-05-2019**

EQUIPMENT **CME 75** BY: **N. 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	T6-1			SM/SC	**UNDOCUMENTED FILL (Qudf)** Loose, damp to moist, brown to dark brown, Silty to Clayey, fine to coarse SAND; gravel and cobble			
2				SM	**VERY OLD PARALIC DEPOSITS (Qvop)** Very dense, damp, reddish brown, Silty, fine to coarse SAND; gravel and cobble			
4					REFUSAL AT 4 FEET Groundwater not encountered			

Figure A-14,
Log of Trench T 6, Page 1 of 1

G2389-42-01.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

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	TRENCH T 7 ELEV. (MSL.) 417' DATE COMPLETED 04-05-2019 EQUIPMENT CME 75 BY: N. BORJA	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
					MATERIAL DESCRIPTION			
0				SM	**UNDOCUMENTED FILL (Qudf)**			
				SM	Loose, dry to damp, brown, Silty, fine to medium SAND; few gravel and cobble			
					VERY OLD PARALIC DEPOSITS (Qvop) Dense, damp, reddish brown, Silty, fine to medium SAND; few clay			
2					-Becomes very dense; some gravel and cobble; cemented			
					REFUSAL AT 2 FEET Groundwater not encountered			

Figure A-15,
Log of Trench T 7, Page 1 of 1

G2389-42-01.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

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	TRENCH T 8 ELEV. (MSL.) 413' DATE COMPLETED 04-05-2019 EQUIPMENT CME 75 BY: N. BORJA	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
					MATERIAL DESCRIPTION			
0				SM	**UNDOCUMENTED FILL (Qudf)** Loose, dry to damp, brown, Silty, fine to medium SAND; few gravel and cobble			
				SM	**VERY OLD PARALIC DEPOSITS (Qvop)** Dense, damp, reddish brown, Silty, fine to medium SAND; few clay			
2					-Becomes very dense, yellowish brown to reddish brown; gravel and cobble			
					REFUSAL AT 2.5 FEET Groundwater not encountered Backfilled on 04-04-2019			

Figure A-16,
Log of Trench T 8, Page 1 of 1

G2389-42-01.GPJ

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

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GEOCON

PROJECT NO. G2389-42-01

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	TRENCH T 9 ELEV. (MSL.) 411' DATE COMPLETED 04-05-2019 EQUIPMENT CME 75 BY: N. BORJA	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
					MATERIAL DESCRIPTION			
0				SM/SC	**UNDOCUMENTED FILL (Qudf)** Loose, damp to moist, brown, Silty to Clayey, fine to coarse SAND			
2				CL	**VERY OLD PARALIC DEPOSITS (Qvop)** Firm, moist, gray to olive brown, Silty to Sandy CLAY			
				SM	Very dense, damp, yellowish brown to medium brown, Silty, fine to coarse SAND			
					REFUSAL AT 3 FEET Groundwater not encountered			

Figure A-17,
Log of Trench T 9, Page 1 of 1

G2389-42-01.GPJ

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

GEOCON

PROJECT NO. G2389-42-01

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	TRENCH T 10 ELEV. (MSL.) 416' DATE COMPLETED 04-05-2019 EQUIPMENT CME 75 BY: N. BORJA	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
					MATERIAL DESCRIPTION			
0				CL	**UNDOCUMENTED FILL (Qudf)** Soft to firm, moist, dark brown and dark reddish brown, Sandy CLAY; trace gravel and cobble			
2				CL	**VERY OLD PARALIC DEPOSITS (Qvop)** Fine to stiff, moist, reddish brown and gray, Silty to Sandy CLAY			
4				SM	Very dense, damp, yellowish brown and reddish brown, Silty, fine to coarse SAND; trace gravel and cobble			
					REFUSAL AT 4.5 FEET Groundwater not encountered			

Figure A-18,
Log of Trench T 10, Page 1 of 1

G2389-42-01.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

TRENCH T 11

ELEV. (MSL.) **416'** DATE COMPLETED **04-05-2019**

EQUIPMENT **CME 75** BY: **N. 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				SM	**UNDOCUMENTED FILL (Qudf)** Loose, damp, grayish brown, Silty, fine to coarse SAND			
2								
				CL	**VERY OLD PARALIC DEPOSITS (Qvop)** Stiff, moist, reddish brown, Sandy CLAY; trace gravel			
4				SM	Very dense, damp, reddish brown to yellowish brown, Silty, fine to coarse SAND; gravel and cobble			

REFUSAL AT 5.5 FEET
Groundwater not encountered

Figure A-19,
Log of Trench T 11, Page 1 of 1

G2389-42-01.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 in-place dry density and moisture content, maximum dry density and optimum moisture content, shear strength, expansion index, water-soluble sulfate content, chloride content, pH and resistivity, resistance value (R-Value), and gradation characteristics. The results of our laboratory tests are presented on the following tables and graphs. The in-place dry density and moisture content results are presented on the exploratory boring logs in Appendix A.

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	Brown, Clayey, fine to coarse SAND; little gravel	128.1	9.1
B6-1	Grayish brown, Clayey, fine to course SAND; some gravel	124.0	10.6

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

Sample No.	Dry Density (pcf)	Moisture Content (%)	Peak Unit Cohesion (psf)	Peak Angle of Shear Resistance (degrees)
B3-3	109.5	16.0	800	32

TABLE B-III
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	9.0	19.2	112.0	17	Very Low
B3-1	11.6	23.6	100.7	40	Low
B6-1	10.4	17.0	107.9	28	Low
B8-1	12.4	28.4	100.9	83	Medium
T6-1	12.8	26.3	97.5	68	Medium

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

Sample No.	Water-Soluble Sulfate (%)	Classification
B1-1	0.011	S0
B3-1	0.026	S0
B6-1	0.026	S0
T6-1	0.016	S0

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

Sample No.	Chloride Content (%)	Chloride Content (ppm)
T6-1	0.020	195

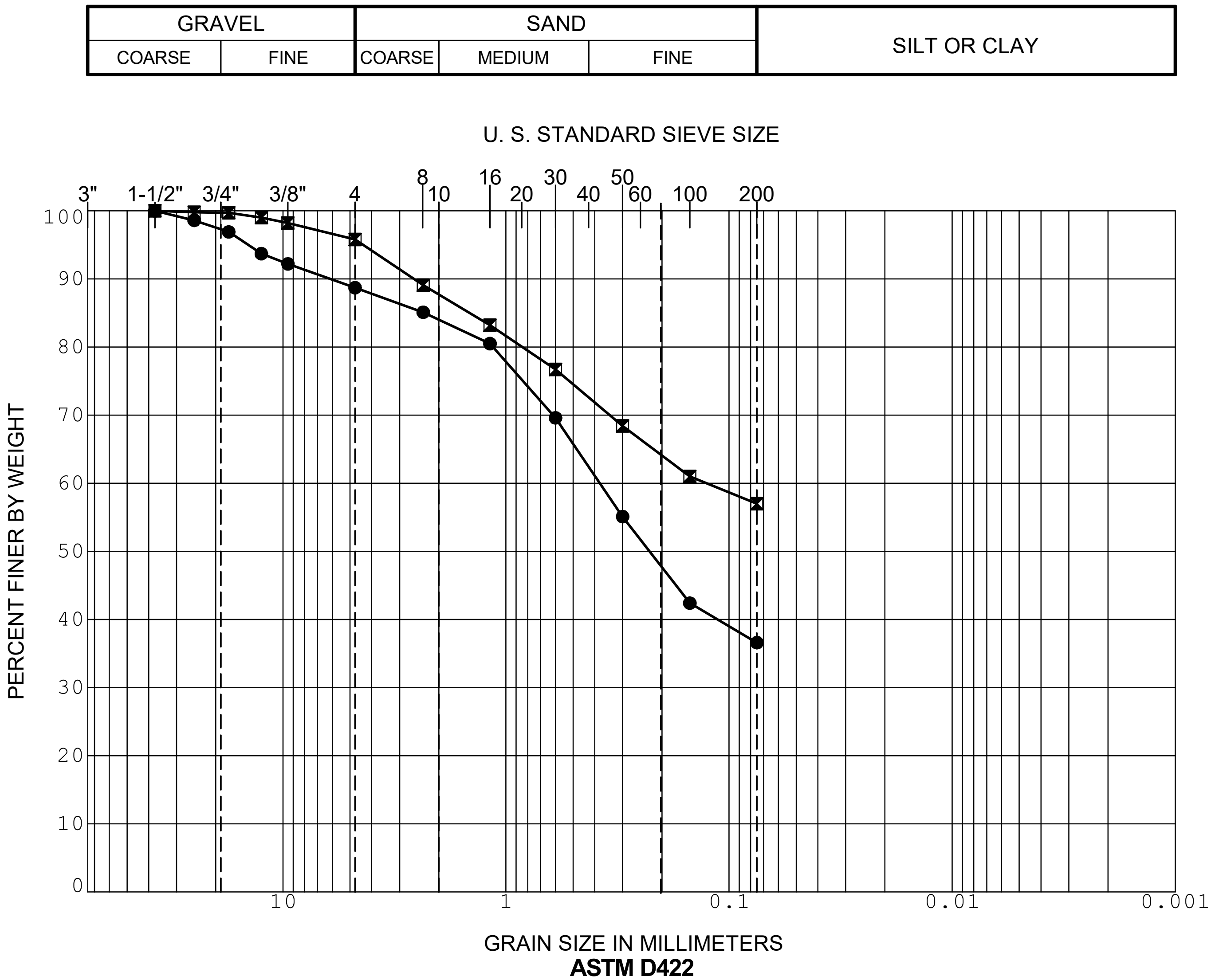
TABLE B-VI
SUMMARY OF LABORATORY POTENTIAL OF HYDROGEN (PH) AND RESISTIVITY TEST RESULTS
CALIFORNIA TEST NO. 643

Sample No.	pH	Minimum Resistivity (ohm-centimeters)
T6-1	5.0	1300

TABLE B-VII
SUMMARY OF LABORATORY RESISTANCE VALUE (R-VALUE) TEST RESULTS
ASTM D 2844

Sample No.	Description	R-Value
B1-1	Brown, Clayey, fine to coarse SAND; little gravel	5
B8-1	Brownish gray, Sandy CLAY; trace gravel	1

PROJECT NO. G2389-42-01



	SAMPLE	DEPTH (ft)	CLASSIFICATION	NAT WC	LL	PL	PI
●	B1-1	4.0	SC - Clayey SAND				
⊠	B8-1	1.0	CL - Sandy CLAY				
▲							

GRADATION CURVE

5650 AND 5660 KEARNY MESA

SAN DIEGO, CALIFORNIA

GEOCON

APPENDIX C

APPENDIX C

STORM WATER MANAGEMENT

We understand storm water management devices are being proposed in accordance with the current Storm Water Standards (SWS). If not properly constructed, there is a potential for distress to improvements and properties located hydrologically down gradient or adjacent to these devices. Factors such as the amount of water to be detained, its residence time, and soil permeability have an important effect on seepage transmission and the potential adverse impacts that may occur if the storm water management features are not properly designed and constructed. We have not performed a hydrogeological study at the site. If infiltration of storm water runoff occurs, downstream properties and improvements may be subjected to seeps, springs, slope instability, raised groundwater, movement of foundations and slabs, or other undesirable impacts as a result of water infiltration.

Hydrologic Soil Group

The United States Department of Agriculture (USDA), Natural Resources Conservation Services, possesses general information regarding the existing soil conditions for areas within the United States. The USDA website also provides the Hydrologic Soil Group. Table C-1 presents the descriptions of the hydrologic soil groups.

TABLE C-1
HYDROLOGIC SOIL GROUP DEFINITIONS

Soil Group	Soil Group Definition
A	Soils having a high infiltration rate (low runoff potential) when thoroughly wet. These consist mainly of deep, well drained to excessively drained sands or gravelly sands. These soils have a high rate of water transmission.
B	Soils having a moderate infiltration rate when thoroughly wet. These consist chiefly of moderately deep or deep, moderately well drained or well drained soils that have moderately fine texture to moderately coarse texture. These soils have a moderate rate of water transmission.
C	Soils having a slow infiltration rate when thoroughly wet. These consist chiefly of soils having a layer that impedes the downward movement of water or soils of moderately fine texture or fine texture. These soils have a slow rate of water transmission.
D	Soils having a very slow infiltration rate (high runoff potential) when thoroughly wet. These consist chiefly of clays that have a high shrink-swell potential, soils that have a high water table, soils that have a claypan or clay layer at or near the surface, and soils that are shallow over nearly impervious material. These soils have a very slow rate of water transmission.

The property is underlain by undocumented fill and Very Old Paralic Deposits. Table C-2 presents the information from the USDA website for the subject property.

TABLE C-2
USDA WEB SOIL SURVEY – HYDROLOGIC SOIL GROUP

Map Unit Name	Map Unit Symbol	Approximate Percentage of Property	Hydrologic Soil Group
Redding gravelly loam, 2 to 9 percent slopes	RdC	99	D
Redding cobbly loam, 9 to 30 percent slopes	RdE	1	D

Infiltration Testing

We performed borehole infiltration tests using a constant-head permeameter. The tests were performed in 8-inch-diameter, drilled borings. The Geologic Map, Figure 2 shows the approximate locations of the infiltration tests. Table C-3 presents the results of the testing. The calculation sheets are also provided herein.

We used the guidelines presented in the Riverside County Low Impact Development BMP Design Handbook, which references the United States Bureau of Reclamation Well Permeameter Test Method (USBR 7300-89). Based on this widely-accepted guideline, the saturated hydraulic conductivity (Ksat) is equal to the infiltration rate. The Ksat value determined from the Aardvark Permeameter test is the unfactored infiltration rate.

TABLE C-3
UNFACTORED HYDRAULIC CONDUCTIVITY TEST RESULTS

Test No.	Depth (inches)	Field Infiltration Rate, I (inches/hour)	Factored* Field Infiltration Rate, I (in/hr)
A-1	60	0.006	0.003
A-2	60	0.007	0.0035
A-3	96	0.041	0.021
A-4	120	0.001	0.0005
A-5	60	0.002	0.001
A-6	84	0.002	0.001

* Factor of Safety of 2.0 for feasibility determination.

STORM WATER MANAGEMENT CONCLUSIONS

Soil Types

Undocumented Fill – We encountered undocumented fill at existing grades throughout a majority of the site. The undocumented fill is loose and compressible. Recommendations have been provided to remove and replace the undocumented fill as compacted fill within the project limits. Infiltration should not occur within undocumented fill due to the potential for adverse settlement.

Compacted Fill – At the completion of grading, we expect portions of the site will be underlain by compacted fill. Water that is allowed to infiltrate into the compacted fill could cause saturation and settlement of the fill. Infiltration into compacted fill is considered infeasible and not recommended from a geotechnical engineering perspective.

Very Old Paralic Deposits (Qvop) and Stadium Conglomerate (Tst) – Very old paralic deposits and Stadium Conglomerate formation were encountered on the property. These two geologic units are sufficiently impermeable that infiltration is infeasible. The very old paralic deposits also have an expansive upper clay layer.

Groundwater Elevation

Groundwater was not encountered in exploratory trenches or borings.

Existing Utilities

Existing utilities service the existing buildings and landscape areas. We expect many of these utilities will be abandoned and removed during site development.

Soil or Groundwater Contamination

We are unaware of contaminated soil on the property. Therefore, full and partial infiltration associated with this risk is considered feasible.

Slopes

We have not been provided with a grading plan for the proposed new building. If new slopes are planned, we recommend a 50-foot setback from the top of slopes. Basins within 50 feet of the top of slopes should be lined to prevent lateral water migration to the face of the slope.

Infiltration Rates

The results of the infiltration rates show rates ranging from 0.001 to 0.041 inches per hour. The infiltration rates are not adequate to support full or partial infiltration.

Storm Water Management Devices

Liners and subdrains should be incorporated into the design and construction of the planned storm water devices. The liners should be impermeable (e.g. High-density polyethylene, HDPE, with a thickness of about 30 mil or equivalent Polyvinyl Chloride**,** PVC) to prevent water migration. The subdrains should be perforated within the liner area, installed at the base and above the liner, be at least 3 inches in diameter and consist of Schedule 40 PVC pipe. The subdrains outside of the liner should consist of solid pipe. The penetration of the liners at the subdrains should be properly waterproofed. The subdrains should be connected to a proper outlet. The devices should also be installed in accordance with the manufacturer's recommendations.

Storm Water Standard Worksheets

The SWS requests the geotechnical engineer complete the *Categorization of Infiltration Feasibility Condition* (Worksheet C.4-1) worksheet information to help evaluate the potential for infiltration on the property. The attached Worksheet C.4-1 presents the completed information for the submittal process.

The regional storm water standards also have a worksheet (Worksheet Form D.5-1) that helps the project civil engineer estimate the factor of safety based on several factors. Table C-4 describes the suitability assessment input parameters related to the geotechnical engineering aspects for the factor of safety determination.

TABLE C-4
SUITABILITY ASSESSMENT RELATED CONSIDERATIONS FOR INFILTRATION FACILITY SAFETY FACTORS

Consideration	High Concern – 3 Points	Medium Concern – 2 Points	Low Concern – 1 Point
Assessment Methods	Use of soil survey maps or simple texture analysis to estimate short-term infiltration rates. Use of well permeameter or borehole methods without accompanying continuous boring log. Relatively sparse testing with direct infiltration methods	Use of well permeameter or borehole methods with accompanying continuous boring log. Direct measurement of infiltration area with localized infiltration measurement methods (e.g., Infiltrometer). Moderate spatial resolution	Direct measurement with localized (i.e. small-scale) infiltration testing methods at relatively high resolution or use of extensive test pit infiltration measurement methods.

TABLE C-4 (Concluded)
SUITABILITY ASSESSMENT RELATED CONSIDERATIONS FOR INFILTRATION FACILITY SAFETY FACTORS

Consideration	High Concern – 3 Points	Medium Concern – 2 Points	Low Concern – 1 Point
Predominant Soil Texture	Silty and clayey soils with significant fines	Loamy soils	Granular to slightly loamy soils
Site Soil Variability	Highly variable soils indicated from site assessment or unknown variability	Soil boring/test pits indicate moderately homogenous soils	Soil boring/test pits indicate relatively homogenous soils
Depth to Groundwater/ Impervious Layer	<5 feet below facility bottom	5-15 feet below facility bottom	>15 feet below facility bottom

Based on our geotechnical investigation and the previous table, Table C-5 presents the estimated factor values for the evaluation of the factor of safety. This table only presents the suitability assessment safety factor (Part A) of the worksheet. The project civil engineer should evaluate the safety factor for design (Part B) and use the combined safety factor for the design infiltration rate.

TABLE C-5
FACTOR OF SAFETY WORKSHEET D.5-1 DESIGN VALUES[1]

Suitability Assessment Factor Category	Assigned Weight (w)	Factor Value (v)	Product (p = w x v)
Assessment Methods	0.25	2	0.50
Predominant Soil Texture	0.25	2	0.50
Site Soil Variability	0.25	3	0.75
Depth to Groundwater/Impervious Layer	0.25	1	0.25
Suitability Assessment Safety Factor, $S_A = \Sigma p$			2.0

[1] The project civil engineer should complete Worksheet D.5-1 using the data on this table. Additional information is required to evaluate the design factor of safety.

CONCLUSIONS

Our results indicate the site has slow infiltration characteristics. Because of the site conditions, it is our opinion that there is a high potential for lateral water migration. It is our opinion that full or partial infiltration is infeasible on this site. Our evaluation included the soil and geologic conditions, estimated settlement and volume change of the underlying soil, slope stability, utility considerations, groundwater mounding, retaining walls, foundations and existing groundwater elevations.

GEOCON

Aardvark Permeameter Data Analysis

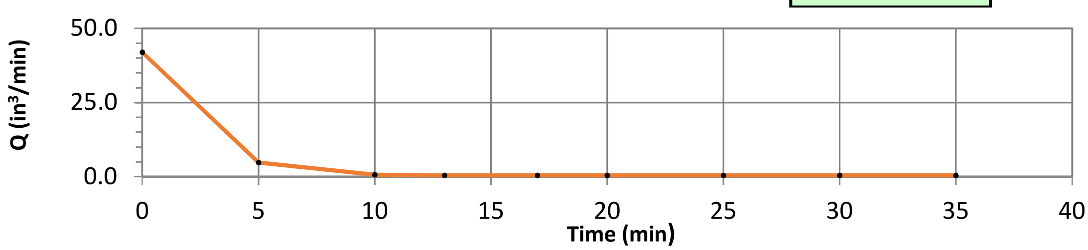
Project Name:	5650/5660 Kearny Mesa Rd	Date:	4/4/2019
Project Number:	G2389-42-01	By:	JML
Test Number:	A-1		

Borehole Diameter, d (in.):	8.00	Ref. EL (feet, MSL):	412.0
Borehole Depth, **H** (in):	60.00	Bottom EL (feet, MSL):	407.0
Distance Between Reservoir & Top of Borehole (in.)	31.00		
Estimated Depth to Water Table, **S** (feet):	50.00		
Height APM Raised from Bottom (in.):	16.00		
Pressure Reducer Used:	No		

Distance Between Resevoir and APM Float, **D** (in.): 67.75

Head Height Measured, **h** (in.): 19.73

Reading	Time Elapsed (min)	Water Weight Consummed (lbs)	Water Volume Consummed (in^3)	Q (in^3/min)
1	0.00	0.000	0.00	0.00
2	5.00	9.135	252.97	50.594
3	5.00	10.355	286.75	57.351
4	3.00	6.700	185.54	61.846
5	4.00	6.745	186.78	46.696
6	3.00	4.550	126.00	42.000
7	5.00	0.865	23.95	4.791
8	5.00	0.115	3.18	0.637
9	5.00	0.085	2.35	0.471
10	5.00	0.080	2.22	0.443
11	5.00	0.080	2.22	0.443
12	5.00	0.085	2.35	0.471
13	5.00	0.085	2.35	0.471
14	5.00	0.085	2.35	0.471
			Steady Flow Rate, Q (in^3/min):	0.471



Soil Matric Flux Potential, Φ_m

Φ_m= 0.00378 in^2/min

Field-Saturated Hydraulic Conductivity (Infiltration Rate)

K_{sat} = 9.59E-05 in/min 0.006 in/hr

GEOCON

Aardvark Permeameter Data Analysis

Project Name: 5650/5660 Kearny Mesa Rd

Project Number: G2389-42-01

Test Number: A-2

Date: 4/4/2019

By: JML

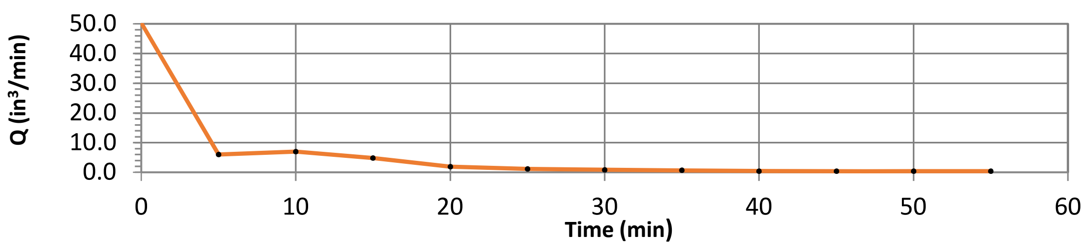
Ref. EL (feet, MSL): 413.0

Bottom EL (feet, MSL): 408.0

Borehole Diameter, d (in.):	8.00
Borehole Depth, **H** (in):	60.00
Distance Between Reservoir & Top of Borehole (in.)	30.50
Estimated Depth to Water Table, **S** (feet):	50.00
Height APM Raised from Bottom (in.):	6.00
Pressure Reducer Used:	No

Distance Between Resevoir and APM Float, **D** (in.):	77.25
Head Height Measured, **h** (in.):	9.76

Reading	Time Elapsed (min)	Water Weight Consummed (lbs)	Water Volume Consummed (in^3)	Q (in^3/min)
1	0.00	0.000	0.00	0.00
2	5.00	9.075	251.31	50.262
3	5.00	1.090	30.18	6.037
4	5.00	1.270	35.17	7.034
5	5.00	0.865	23.95	4.791
6	5.00	0.335	9.28	1.855
7	5.00	0.200	5.54	1.108
8	5.00	0.155	4.29	0.858
9	5.00	0.120	3.32	0.665
10	5.00	0.075	2.08	0.415
11	5.00	0.070	1.94	0.388
12	5.00	0.075	2.08	0.415
13	5.00	0.065	1.80	0.360
			Steady Flow Rate, Q (in^3/min):	0.360



Soil Matric Flux Potential, Φ_m

Φ_m= 0.0047 in^2/min

Field-Saturated Hydraulic Conductivity (Infiltration Rate)

K_{sat} = 1.20E-04 in/min 0.007 in/hr

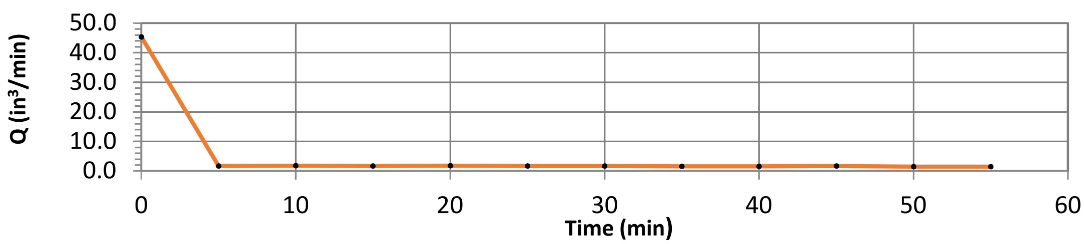
GEOCON

Aardvark Permeameter Data Analysis

Project Name:	5650/5660 Kearny Mesa Rd	Date:	4/4/2019
Project Number:	G2389-42-01	By:	JML
Test Number:	A-3	Ref. EL (feet, MSL):	410.0
		Bottom EL (feet, MSL):	402.0

Borehole Diameter, d (in.):	8.00
Borehole Depth, **H** (in):	96.00
Distance Between Reservoir & Top of Borehole (in.)	31.00
Estimated Depth to Water Table, **S** (feet):	50.00
Height APM Raised from Bottom (in.):	1.00
Pressure Reducer Used:	No
Distance Between Resevoir and APM Float, **D** (in.):	118.75
Head Height Measured, **h** (in.):	4.90

Reading	Time Elapsed (min)	Water Weight Consummed (lbs)	Water Volume Consummed (in^3)	Q (in^3/min)
1	0.00	0.000	0.00	0.00
2	5.00	8.185	226.66	45.332
3	5.00	0.310	8.58	1.717
4	5.00	0.315	8.72	1.745
5	5.00	0.300	8.31	1.662
6	5.00	0.325	9.00	1.800
7	5.00	0.300	8.31	1.662
8	5.00	0.300	8.31	1.662
9	5.00	0.280	7.75	1.551
10	5.00	0.285	7.89	1.578
11	5.00	0.305	8.45	1.689
12	5.00	0.270	7.48	1.495
13	5.00	0.260	7.20	1.440
			Steady Flow Rate, Q (in^3/min):	1.440



Soil Matric Flux Potential, Φ_m

Φ_m = 0.0272 in^2/min

Field-Saturated Hydraulic Conductivity (Infiltration Rate)

K_{sat} = 6.90E-04 in/min | 0.041 in/hr

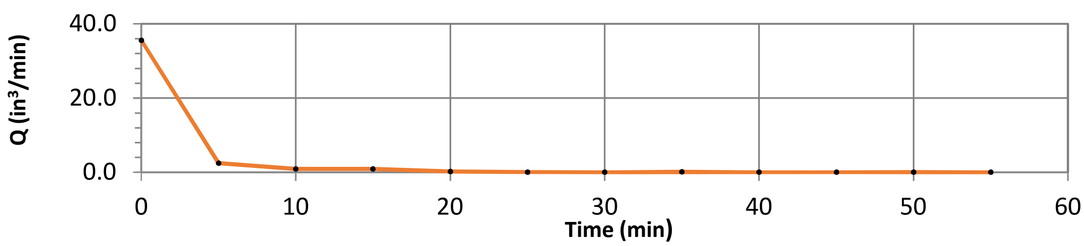
GEOCON

Aardvark Permeameter Data Analysis

Project Name:	5650/5660 Kearny Mesa Rd	Date:	4/4/2019
Project Number:	G2389-42-01	By:	JML
Test Number:	A-4	Ref. EL (feet, MSL):	411.0
		Bottom EL (feet, MSL):	401.0

Borehole Diameter, d (in.):	8.00
Borehole Depth, **H** (in):	120.00
Distance Between Reservoir & Top of Borehole (in.)	31.00
Estimated Depth to Water Table, **S** (feet):	50.00
Height APM Raised from Bottom (in.):	1.00
Pressure Reducer Used:	Yes
Distance Between Resevoir and APM Float, **D** (in.):	142.75
Head Height Measured, **h** (in.):	5.00

Reading	Time Elapsed (min)	Water Weight Consummed (lbs)	Water Volume Consummed (in^3)	Q (in^3/min)
1	0.00	0.000	0.00	0.00
2	5.00	6.430	178.06	35.612
3	5.00	0.450	12.46	2.492
4	5.00	0.170	4.71	0.942
5	5.00	0.165	4.57	0.914
6	5.00	0.040	1.11	0.222
7	5.00	0.010	0.28	0.055
8	5.00	0.005	0.14	0.028
9	5.00	0.020	0.55	0.111
10	5.00	0.005	0.14	0.028
11	5.00	0.005	0.14	0.028
12	5.00	0.010	0.28	0.055
13	5.00	0.005	0.14	0.028
			Steady Flow Rate, Q (in^3/min):	0.028



Soil Matric Flux Potential, Φ_m

Φ_m= 0.0005 in^2/min

Field-Saturated Hydraulic Conductivity (Infiltration Rate)

K_{sat} = 1.31E-05 in/min 0.001 in/hr

GEOCON

Aardvark Permeameter Data Analysis

Project Name: 5650/5660 Kearny Mesa Rd

Project Number: G2389-42-01

Test Number: A-5

Date: 4/4/2019

By: JML

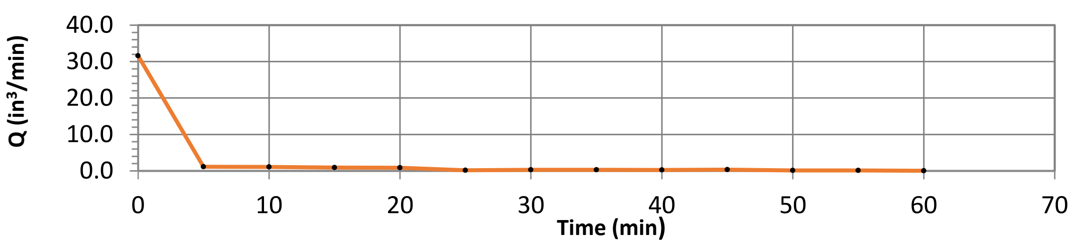
Ref. EL (feet, MSL): 423.0

Bottom EL (feet, MSL): 418.0

Borehole Diameter, d (in.):	8.00
Borehole Depth, **H** (in):	60.00
Distance Between Reservoir & Top of Borehole (in.):	31.00
Estimated Depth to Water Table, **S** (feet):	50.00
Height APM Raised from Bottom (in.):	1.00
Pressure Reducer Used:	No

Distance Between Resevoir and APM Float, **D** (in.):	82.75
Head Height Measured, **h** (in.):	4.78

Reading	Time Elapsed (min)	Water Weight Consummed (lbs)	Water Volume Consummed (in^3)	Q (in^3/min)
1	0.00	0.000	0.00	0.00
2	5.00	5.700	157.85	31.569
3	5.00	0.205	5.68	1.135
4	5.00	0.200	5.54	1.108
5	5.00	0.165	4.57	0.914
6	5.00	0.155	4.29	0.858
7	5.00	0.030	0.83	0.166
8	5.00	0.055	1.52	0.305
9	5.00	0.055	1.52	0.305
10	5.00	0.045	1.25	0.249
11	5.00	0.065	1.80	0.360
12	5.00	0.025	0.69	0.138
13	5.00	0.020	0.55	0.111
14	5.00	0.010	0.28	0.055
			Steady Flow Rate, Q (in^3/min):	0.055



Soil Matric Flux Potential, Φ_m

Φ_m= 0.0011 in^2/min

Field-Saturated Hydraulic Conductivity (Infiltration Rate)

K_{sat} = 2.68E-05 in/min   0.002 in/hr

GEOCON

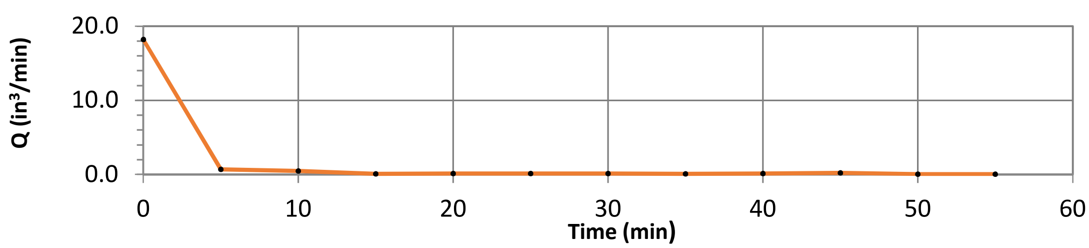
Aardvark Permeameter Data Analysis

Project Name:	5650/5660 Kearny Mesa Rd	Date:	4/4/2019
Project Number:	G2389-42-01	By:	JML
Test Number:	A-6	Ref. EL (feet, MSL):	426.0
		Bottom EL (feet, MSL):	419.0

Borehole Diameter, d (in.):	8.00
Borehole Depth, **H** (in):	84.00
Distance Between Reservoir & Top of Borehole (in.)	30.00
Estimated Depth to Water Table, **S** (feet):	50.00
Height APM Raised from Bottom (in.):	1.00
Pressure Reducer Used:	No

Distance Between Resevoir and APM Float, **D** (in.):	105.75
Head Height Measured, **h** (in.):	4.85

Reading	Time Elapsed (min)	Water Weight Consummed (lbs)	Water Volume Consummed (in^3)	Q (in^3/min)
1	0.00	0.00	0.00	0.00
2	5.00	3.290	91.11	18.222
3	5.00	0.125	3.46	0.692
4	5.00	0.085	2.35	0.471
5	5.00	0.015	0.42	0.083
6	5.00	0.025	0.69	0.138
7	5.00	0.025	0.69	0.138
8	5.00	0.020	0.55	0.111
9	5.00	0.015	0.42	0.083
10	5.00	0.020	0.55	0.111
11	5.00	0.035	0.97	0.194
12	5.00	0.010	0.28	0.055
13	5.00	0.010	0.28	0.055
			Steady Flow Rate, Q (in^3/min):	0.055



Soil Matric Flux Potential, Φ_m

Φ_m= 0.0010 in^2/min

Field-Saturated Hydraulic Conductivity (Infiltration Rate)

K_{sat} = 2.66E-05 in/min 0.002 in/hr

Categorization of Infiltration Feasibility Condition based on Geotechnical Conditions	Worksheet C.4-1: Form I-8A[10]
Part 1 - Full Infiltration Feasibility Screening Criteria	
DMA(s) Being Analyzed:	Project Phase:
Site: 5650 and 5660 Kearny Mesa Road	Preliminary
Criteria 1: Infiltration Rate Screening	
1A	Is the mapped hydrologic soil group according to the NRCS Web Soil Survey or UC Davis Soil Web Mapper Type A or B and corroborated by available site soil data[11]? ☐Yes; the DMA may feasibly support full infiltration. Answer "Yes" to Criteria 1 Result or continue to Step 1B if the applicant elects to perform infiltration testing. ☐No; the mapped soil types are A or B but is not corroborated by available site soil data (continue to Step 1B). ☑No; the mapped soil types are C, D, or "urban/unclassified" and is corroborated by available site soil data. Answer "No" to Criteria 1 Result. ☐No; the mapped soil types are C, D, or "urban/unclassified" but is not corroborated by available site soil data (continue to Step 1B).
1B	Is the reliable infiltration rate calculated using planning phase methods from Table D.3-1? ☐Yes; Continue to Step 1C. ☐No; Skip to Step 1D.
1C	Is the reliable infiltration rate calculated using planning phase methods from Table D.3-1 greater than 0.5 inches per hour? ☐Yes; the DMA may feasibly support full infiltration. Answer "Yes" to Criteria 1 Result. ☐No; full infiltration is not required. Answer "No" to Criteria 1 Result.
1D	**Infiltration Testing Method.** Is the selected infiltration testing method suitable during the design phase (see Appendix D.3)? Note: Alternative testing standards may be allowed with appropriate rationales and documentation. ☐Yes; continue to Step 1E. ☐No; select an appropriate infiltration testing method.

Note that it is not required to investigate each and every criterion in the worksheet, a single "no" answer in Part 1, Part 2, Part 3, or Part 4 determines a full, partial, or no infiltration condition.

[10] This form must be completed each time there is a change to the site layout that would affect the infiltration feasibility condition. Previously completed forms shall be retained to document the evolution of the site storm water design.

[11] Available data includes site-specific sampling or observation of soil types or texture classes, such as obtained from borings or test pits necessary to support other design elements.

Categorization of Infiltration Feasibility Condition based on Geotechnical Conditions	Worksheet C.4-1: Form I-8A[10]
1E	**Number of Percolation/Infiltration Tests.** Does the infiltration testing method performed satisfy the minimum number of tests specified in Table D.3-2? ☐ Yes; continue to Step 1F. ☐ No; conduct appropriate number of tests.
IF	**Factor of Safety.** Is the suitable Factor of Safety selected for full infiltration design? See guidance in D.5; Tables D.5-1 and D.5-2; and Worksheet D.5-1 (Form I-9). ☐ Yes; continue to Step 1G. ☐ No; select appropriate factor of safety.
1G	**Full Infiltration Feasibility.** Is the average measured infiltration rate divided by the Factor of Safety greater than 0.5 inches per hour? ☐ Yes; answer "Yes" to Criteria 1 Result. ☐ No; answer "No" to Criteria 1 Result.
Criteria 1 Result	Is the estimated reliable infiltration rate greater than 0.5 inches per hour within the DMA where runoff can reasonably be routed to a BMP? ☐ Yes; the DMA may feasibly support full infiltration. Continue to Criteria 2. ☑ No; full infiltration is not required. Skip to Part 1 Result.

Summarize infiltration testing methods, testing locations, replicates, and results and summarize estimates of reliable infiltration rates according to procedures outlined in D.5. Documentation should be included in project geotechnical report.

We performed six field-saturated, hydraulic conductivity tests, A-1 through A-6, using a Soil Moisture Corp Aardvark Permeameter. The location of the infiltration tests are shown on the Geologic Map, Figure 2. The test holes were excavated using a drill rig equipped with an 8-inch diameter auger. The unfactored test results of the saturated hydraulic conductivity tests ranged from 0.001 to 0.0041 in/hr. The factored rate ranges from 0.0004 to 0.021 in/hr using a safety factor of 2.0.

Categorization of Infiltration Feasibility Condition based on Geotechnical Conditions		Worksheet C.4-1: Form I-8A[10]	
Criteria 2: Geologic/Geotechnical Screening			
2A	If all questions in Step 2A are answered "Yes," continue to Step 2B. For any "No" answer in Step 2A answer "No" to Criteria 2, and submit an "Infiltration Feasibility Condition Letter" that meets the requirements in Appendix C.1.1. The geologic/geotechnical analyses listed in Appendix C.2.1 do not apply to the DMA because one of the following setbacks cannot be avoided and therefore result in the DMA being in a no infiltration condition. The setbacks must be the closest horizontal radial distance from the surface edge (at the overflow elevation) of the BMP.		
2A-1	Can the proposed full infiltration BMP(s) avoid areas with existing fill materials greater than 5 feet thick below the infiltrating surface?	☐Yes	☐No
2A-2	Can the proposed full infiltration BMP(s) avoid placement within 10 feet of existing underground utilities, structures, or retaining walls?	☐Yes	☐No
2A-3	Can the proposed full infiltration BMP(s) avoid placement within 50 feet of a natural slope (>25%) or within a distance of 1.5H from fill slopes where H is the height of the fill slope?	☐Yes	☐No
2B	When full infiltration is determined to be feasible, a geotechnical investigation report must be prepared that considers the relevant factors identified in Appendix C.2.1. If all questions in Step 2B are answered "Yes," then answer "Yes" to Criteria 2 Result. If there are "No" answers continue to Step 2C.		
2B-1	**Hydroconsolidation.** Analyze hydroconsolidation potential per approved ASTM standard due to a proposed full infiltration BMP. Can full infiltration BMPs be proposed within the DMA without increasing hydroconsolidation risks?	☐Yes	☐No
2B-2	**Expansive Soils.** Identify expansive soils (soils with an expansion index greater than 20) and the extent of such soils due to proposed full infiltration BMPs. Can full infiltration BMPs be proposed within the DMA without increasing expansive soil risks?	☐Yes	☐No

Categorization of Infiltration Feasibility Condition based on Geotechnical Conditions		Worksheet C.4-1: Form I-8A[10]	
2B-3	**Liquefaction**. If applicable, identify mapped liquefaction areas. Evaluate liquefaction hazards in accordance with Section 6.4.2 of the City of San Diego's Guidelines for Geotechnical Reports (2011 or most recent edition). Liquefaction hazard assessment shall take into account any increase in groundwater elevation or groundwater mounding that could occur as a result of proposed infiltration or percolation facilities. Can full infiltration BMPs be proposed within the DMA without increasing liquefaction risks?	☐Yes	☐No
2B-4	**Slope Stability**. If applicable, perform a slope stability analysis in accordance with the ASCE and Southern California Earthquake Center (2002) Recommended Procedures for Implementation of DMG Special Publication 117, Guidelines for Analyzing and Mitigating Landslide Hazards in California to determine minimum slope setbacks for full infiltration BMPs. See the City of San Diego's Guidelines for Geotechnical Reports (2011) to determine which type of slope stability analysis is required. Can full infiltration BMPs be proposed within the DMA without increasing slope stability risks?	☐Yes	☐No
2B-5	**Other Geotechnical Hazards.** Identify site-specific geotechnical hazards not already mentioned (refer to Appendix C.2.1). Can full infiltration BMPs be proposed within the DMA without increasing risk of geologic or geotechnical hazards not already mentioned?	☐Yes	☐No
2B-6	**Setbacks.** Establish setbacks from underground utilities, structures, and/or retaining walls. Reference applicable ASTM or other recognized standard in the geotechnical report. Can full infiltration BMPs be proposed within the DMA using established setbacks from underground utilities, structures, and/or retaining walls?	☐ Yes	☐ No

Categorization of Infiltration Feasibility Condition based on Geotechnical Conditions		Worksheet C.4-1: Form I-8A[10]	
2C	**Mitigation Measures.** Propose mitigation measures for each geologic/geotechnical hazard identified in Step 2B. Provide a discussion of geologic/geotechnical hazards that would prevent full infiltration BMPs that cannot be reasonably mitigated in the geotechnical report. See Appendix C.2.1.8 for a list of typically reasonable and typically unreasonable mitigation measures. Can mitigation measures be proposed to allow for full infiltration BMPs? If the question in Step 2 is answered "Yes," then answer "Yes" to Criteria 2 Result. If the question in Step 2C is answered "No," then answer "No" to Criteria 2 Result.	☐Yes	☐No
Criteria 2 Result	Can infiltration greater than 0.5 inches per hour be allowed without increasing risk of geologic or geotechnical hazards that cannot be reasonably mitigated to an acceptable level?	☐Yes	☐No
Summarize findings and basis; provide references to related reports or exhibits.			

Part 1 Result – Full Infiltration Geotechnical Screening [12]	Result
If answers to both Criteria 1 and Criteria 2 are "Yes", a full infiltration design is potentially feasible based on Geotechnical conditions only. If either answer to Criteria 1 or Criteria 2 is "No", a full infiltration design is not required.	☐Full infiltration Condition ☑Complete Part 2

[12] To be completed using gathered site information and best professional judgement considering the definition of MEP in the MS4 Permit. Additional testing and/or studies may be required by City Engineer to substantiate findings.

Categorization of Infiltration Feasibility Condition based on Geotechnical Conditions	Worksheet C.4-1: Form I-8A[10]
Part 2 – Partial vs. No Infiltration Feasibility Screening Criteria	
DMA(s) Being Analyzed:	**Project Phase:**
Site: 5650 and 5660 Kearny Mesa Road	Preliminary

Criteria 3 : Infiltration Rate Screening

3A	**NRCS Type C, D, or "urban/unclassified":** Is the mapped hydrologic soil group according to the NRCS Web Soil Survey or UC Davis Soil Web Mapper is Type C, D, or "urban/unclassified" and corroborated by available site soil data? ☐ Yes; the site is mapped as C soils and a reliable infiltration rate of 0.15 in/hr. is used to size partial infiltration BMPS. Answer "Yes" to Criteria 3 Result. ☐ Yes; the site is mapped as D soils or "urban/unclassified" and a reliable infiltration rate of 0.05 in/hr. is used to size partial infiltration BMPS. Answer "Yes" to Criteria 3 Result. ☑ No; infiltration testing is conducted (refer to Table D.3-1), continue to Step 3B.
3B	**Infiltration Testing Result:** Is the reliable infiltration rate (i.e. average measured infiltration rate/2) greater than 0.05 in/hr. and less than or equal to 0.5 in/hr? ☐ Yes; the site may support partial infiltration. Answer "Yes" to Criteria 3 Result. ☑ No; the reliable infiltration rate (i.e. average measured rate/2) is less than 0.05 in/hr., partial infiltration is not required. Answer "No" to Criteria 3 Result.
Criteria 3 Result	Is the estimated reliable infiltration rate (i.e., average measured infiltration rate/2) greater than or equal to 0.05 inches/hour and less than or equal to 0.5 inches/hour at any location within each DMA where runoff can reasonably be routed to a BMP? ☐ Yes; Continue to Criteria 4. ☑ No: Skip to Part 2 Result.

Summarize infiltration testing and/or mapping results (i.e. soil maps and series description used for infiltration rate).

We performed six field-saturated, hydraulic conductivity tests, A-1 through A-6, using a Soil Moisture Corp Aardvark Permeameter. The location of the infiltration tests are shown on the Geologic Map, Figure 2. The test holes were excavated using a drill rig equipped with an 8-inch diameter auger. The unfactored test results of the saturated hydraulic conductivity tests ranged from 0.001 to 0.0041 in/hr. The factored rate ranges from 0.0004 to 0.021 in/hr using a safety factor of 2.0.

Categorization of Infiltration Feasibility Condition based on Geotechnical Conditions		Worksheet C.4-1: Form I-8A[10]	
Criteria 4: Geologic/Geotechnical Screening			
4A	If all questions in Step 4A are answered "Yes," continue to Step 2B. For any "No" answer in Step 4A answer "No" to Criteria 4 Result, and submit an "Infiltration Feasibility Condition Letter" that meets the requirements in Appendix C.1.1. The geologic/geotechnical analyses listed in Appendix C.2.1 do not apply to the DMA because one of the following setbacks cannot be avoided and therefore result in the DMA being in a no infiltration condition. The setbacks must be the closest horizontal radial distance from the surface edge (at the overflow elevation) of the BMP.		
4A-1	Can the proposed partial infiltration BMP(s) avoid areas with existing fill materials greater than 5 feet thick?	☐Yes	☐No
4A-2	Can the proposed partial infiltration BMP(s) avoid placement within 10 feet of existing underground utilities, structures, or retaining walls?	☐Yes	☐No
4A-3	Can the proposed partial infiltration BMP(s) avoid placement within 50 feet of a natural slope (>25%) or within a distance of 1.5H from fill slopes where H is the height of the fill slope?	☐Yes	☐No
4B	When full infiltration is determined to be feasible, a geotechnical investigation report must be prepared that considers the relevant factors identified in Appendix C.2.1 If all questions in Step 4B are answered "Yes," then answer "Yes" to Criteria 4 Result. If there are any "No" answers continue to Step 4C.		
4B-1	**Hydroconsolidation.** Analyze hydroconsolidation potential per approved ASTM standard due to a proposed full infiltration BMP. Can partial infiltration BMPs be proposed within the DMA without increasing hydroconsolidation risks?	☐Yes	☐No
4B-2	**Expansive Soils.** Identify expansive soils (soils with an expansion index greater than 20) and the extent of such soils due to proposed full infiltration BMPs. Can partial infiltration BMPs be proposed within the DMA without increasing expansive soil risks?	☐Yes	☐No

Categorization of Infiltration Feasibility Condition based on Geotechnical Conditions		Worksheet C.4-1: Form I-8A[10]	
4B-3	**Liquefaction**. If applicable, identify mapped liquefaction areas. Evaluate liquefaction hazards in accordance with Section 6.4.2 of the City of San Diego's Guidelines for Geotechnical Reports (2011). Liquefaction hazard assessment shall take into account any increase in groundwater elevation or groundwater mounding that could occur as a result of proposed infiltration or percolation facilities. Can partial infiltration BMPs be proposed within the DMA without increasing liquefaction risks?	☐Yes	☐No
4B-4	**Slope Stability**. If applicable, perform a slope stability analysis in accordance with the ASCE and Southern California Earthquake Center (2002) Recommended Procedures for Implementation of DMG Special Publication 117, Guidelines for Analyzing and Mitigating Landslide Hazards in California to determine minimum slope setbacks for full infiltration BMPs. See the City of San Diego's Guidelines for Geotechnical Reports (2011) to determine which type of slope stability analysis is required. Can partial infiltration BMPs be proposed within the DMA without increasing slope stability risks?	☐Yes	☐No
4B-5	**Other Geotechnical Hazards.** Identify site-specific geotechnical hazards not already mentioned (refer to Appendix C.2.1). Can partial infiltration BMPs be proposed within the DMA without increasing risk of geologic or geotechnical hazards not already mentioned?	☐Yes	☐No
4B-6	**Setbacks.** Establish setbacks from underground utilities, structures, and/or retaining walls. Reference applicable ASTM or other recognized standard in the geotechnical report. Can partial infiltration BMPs be proposed within the DMA using recommended setbacks from underground utilities, structures, and/or retaining walls?	☐Yes	☐No
4C	**Mitigation Measures.** Propose mitigation measures for each geologic/geotechnical hazard identified in Step 4B. Provide a discussion on geologic/geotechnical hazards that would prevent partial infiltration BMPs that cannot be reasonably mitigated in the geotechnical report. See Appendix C.2.1.8 for a list of typically reasonable and typically unreasonable mitigation measures. Can mitigation measures be proposed to allow for partial infiltration BMPs? If the question in Step 4C is answered "Yes," then answer "Yes" to Criteria 4 Result. If the question in Step 4C is answered "No," then answer "No" to Criteria 4 Result.	☐Yes	☐No

Categorization of Infiltration Feasibility Condition based on Geotechnical Conditions			Worksheet C.4-1: Form I-8A[10]
Criteria 4 Result	Can infiltration of greater than or equal to 0.05 inches/hour and less than or equal to 0.5 inches/hour be allowed without increasing the risk of geologic or geotechnical hazards that cannot be reasonably mitigated to an acceptable level?	☐ Yes	☐ No
Summarize findings and basis; provide references to related reports or exhibits.			

Part 2 – Partial Infiltration Geotechnical Screening Result[13]	Result
If answers to both Criteria 3 and Criteria 4 are "Yes", a partial infiltration design is potentially feasible based on geotechnical conditions only. If answers to either Criteria 3 or Criteria 4 is "No", then infiltration of any volume is considered to be infeasible within the site.	☐ Partial Infiltration Condition ☑ No Infiltration Condition

[13] To be completed using gathered site information and best professional judgement considering the definition of MEP in the MS4 Permit. Additional testing and/or studies may be required by City Engineer to substantiate findings.

APPENDIX D

APPENDIX D

RECOMMENDED GRADING SPECIFICATIONS

FOR

5650 AND 5660 KEARNY MESA ROAD
SAN DIEGO, CALIFORNIA

PROJECT NO. G2389-42-01

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. 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, and/or 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

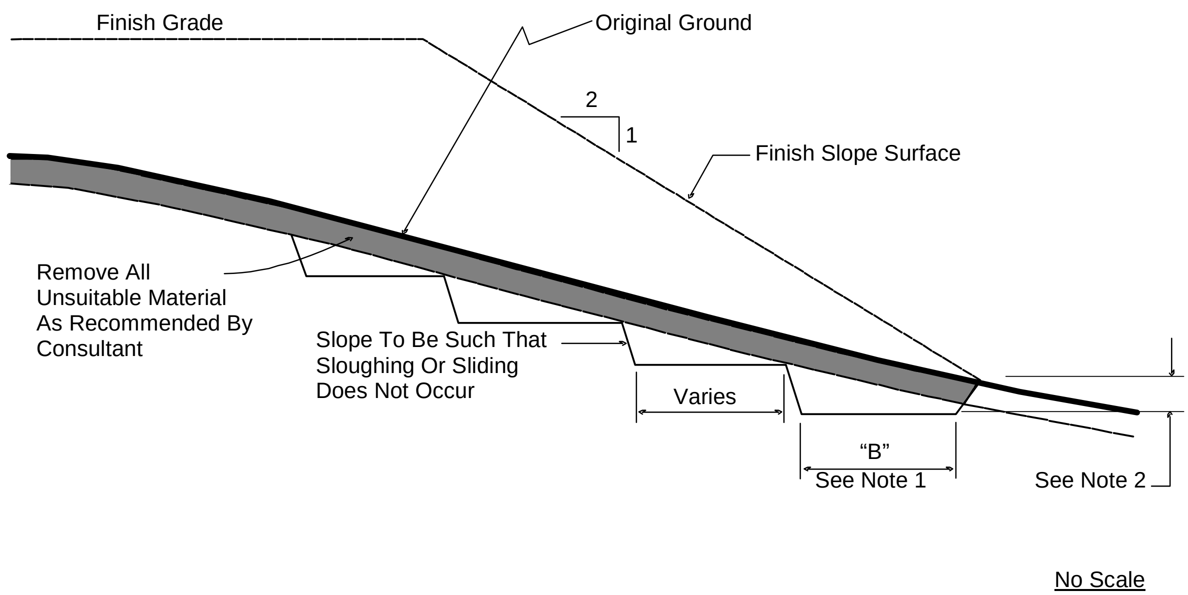
4.1 Areas to be excavated and filled shall be cleared and grubbed. Clearing shall consist of complete removal above the ground surface of trees, stumps, brush, vegetation, man-made structures, and similar debris. Grubbing shall consist of removal of stumps, roots, buried logs and other unsuitable material 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 Asphalt pavement material removed during clearing operations should be properly disposed at an approved off-site facility or in an acceptable area of the project evaluated by Geocon and the property owner. 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.

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. 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, 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.

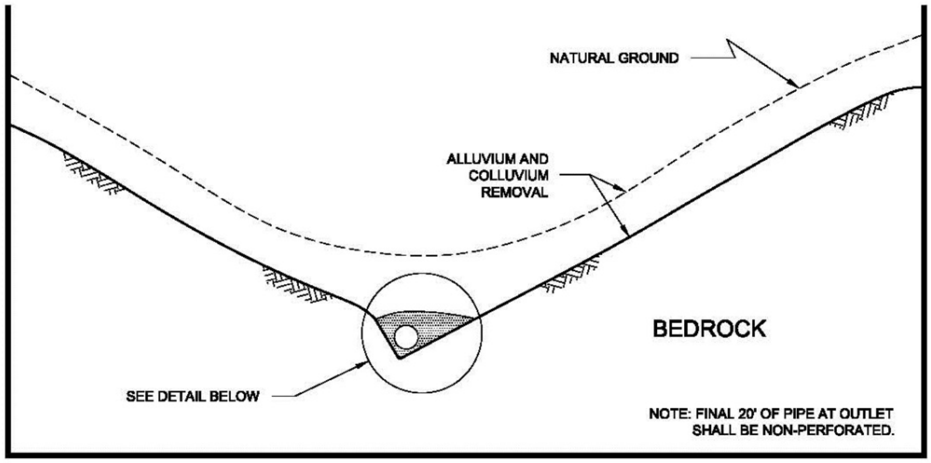
6.3.6 To reduce the potential for "piping" of fines into the *rock* fill from overlying *soil* fill material, a 2-foot layer of graded filter material shall be placed above the uppermost lift of *rock* fill. The need to place graded filter material below the *rock* should be determined by the Consultant 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.

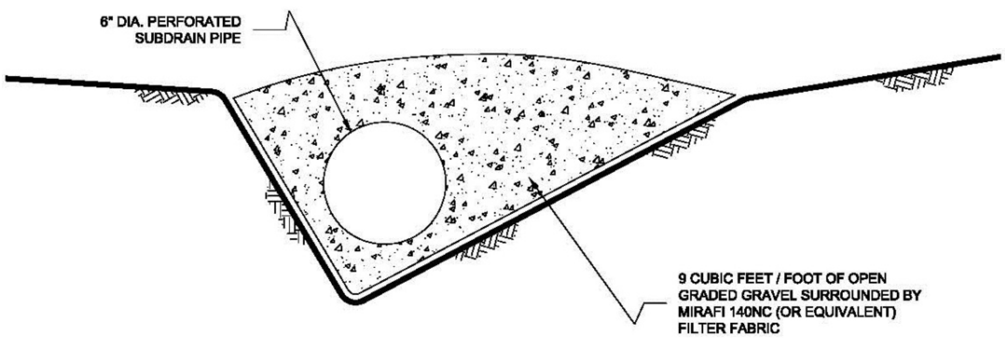
6.3.7 *Rock* fill placement should be continuously observed during placement by the Consultant.

7. SUBDRAINS

7.1 The geologic units on the site may have permeability characteristics and/or fracture systems that could be susceptible under certain conditions to seepage. The use of canyon subdrains may be necessary to mitigate the potential for adverse impacts associated with seepage conditions. Canyon subdrains with lengths in excess of 500 feet or extensions of existing offsite subdrains should use 8-inch-diameter pipes. Canyon subdrains less than 500 feet in length should use 6-inch-diameter pipes.

TYPICAL CANYON DRAIN DETAIL





NOTES:

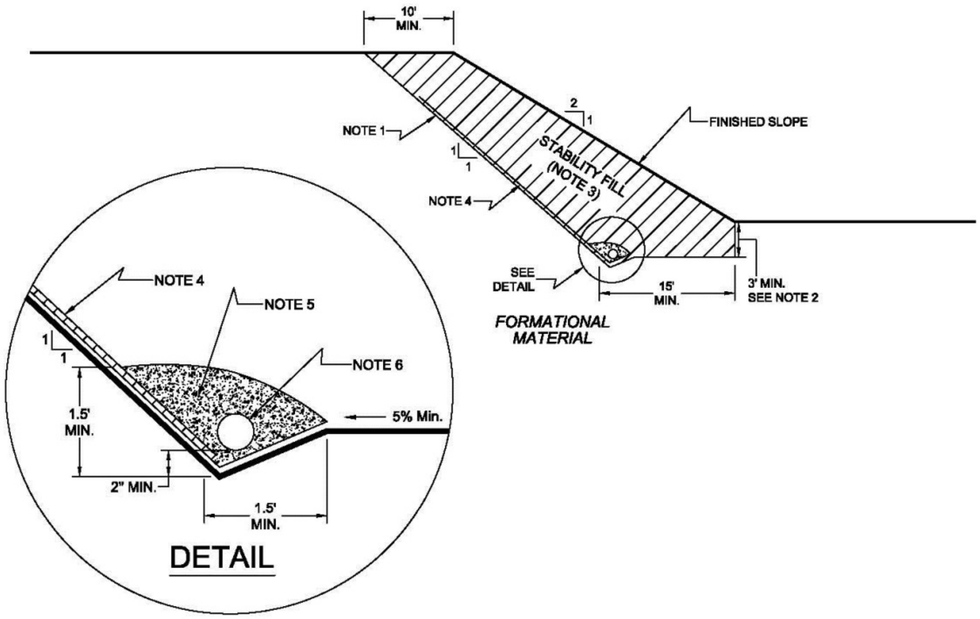
1......8-INCH DIAMETER, SCHEDULE 80 PVC PERFORATED PIPE FOR FILLS IN EXCESS OF 100-FEET IN DEPTH OR A PIPE LENGTH OF LONGER THAN 500 FEET.

2......6-INCH DIAMETER, SCHEDULE 40 PVC PERFORATED PIPE FOR FILLS LESS THAN 100-FEET IN DEPTH OR A PIPE LENGTH SHORTER THAN 500 FEET.

NO SCALE

7.2 Slope drains within stability fill keyways should use 4-inch-diameter (or lager) pipes.

TYPICAL STABILITY FILL DETAIL



NOTES:

1.....EXCAVATE BACKCUT AT 1:1 INCLINATION (UNLESS OTHERWISE NOTED).

2.....BASE OF STABILITY FILL TO BE 3 FEET INTO FORMATIONAL MATERIAL, SLOPING A MINIMUM 5% INTO SLOPE.

3.....STABILITY FILL TO BE COMPOSED OF PROPERLY COMPACTED GRANULAR SOIL.

4.....CHIMNEY DRAINS TO BE APPROVED PREFABRICATED CHIMNEY DRAIN PANELS (MIRADRAIN G200N OR EQUIVALENT) SPACED APPROXIMATELY 20 FEET CENTER TO CENTER AND 4 FEET WIDE. CLOSER SPACING MAY BE REQUIRED IF SEEPAGE IS ENCOUNTERED.

5.....FILTER MATERIAL TO BE 3/4-INCH, OPEN-GRADED CRUSHED ROCK ENCLOSED IN APPROVED FILTER FABRIC (MIRAFI 140NC).

6.....COLLECTOR PIPE TO BE 4-INCH MINIMUM DIAMETER, PERFORATED, THICK-WALLED PVC SCHEDULE 40 OR EQUIVALENT, AND SLOPED TO DRAIN AT 1 PERCENT MINIMUM TO APPROVED OUTLET.

NO SCALE

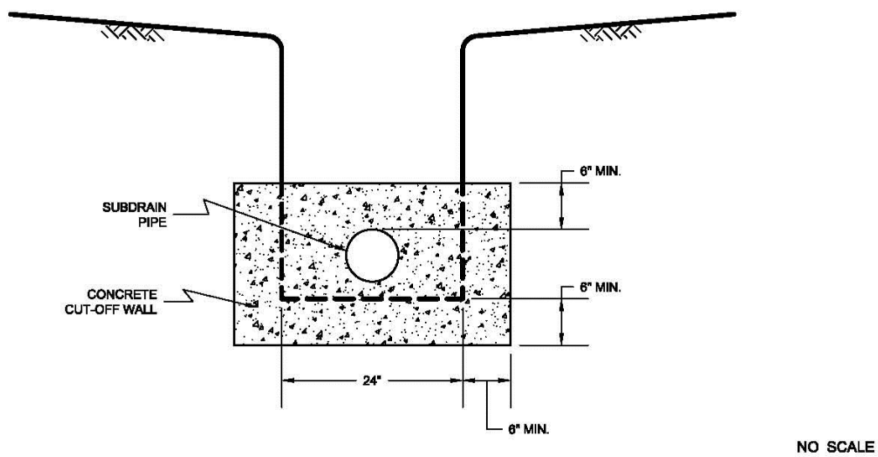
7.3 The actual subdrain locations will be evaluated in the field during the remedial grading operations. Additional drains may be necessary depending on the conditions observed and the requirements of the local regulatory agencies. Appropriate subdrain outlets should be evaluated prior to finalizing 40-scale grading plans.

7.4 *Rock* fill or *soil-rock* fill areas may require subdrains along their down-slope perimeters to mitigate the potential for buildup of water from construction or landscape irrigation. The subdrains should be at least 6-inch-diameter pipes encapsulated in gravel and filter fabric. *Rock* fill drains should be constructed using the same requirements as canyon subdrains.

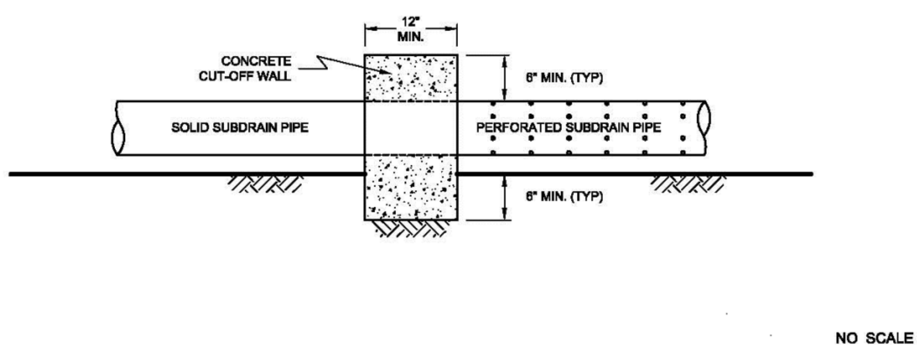
7.5 Prior to outletting, the final 20-foot segment of a subdrain that will not be extended during future development should consist of non-perforated drainpipe. At the non-perforated/ perforated interface, a seepage cutoff wall should be constructed on the downslope side of the pipe.

TYPICAL CUT OFF WALL DETAIL

FRONT VIEW

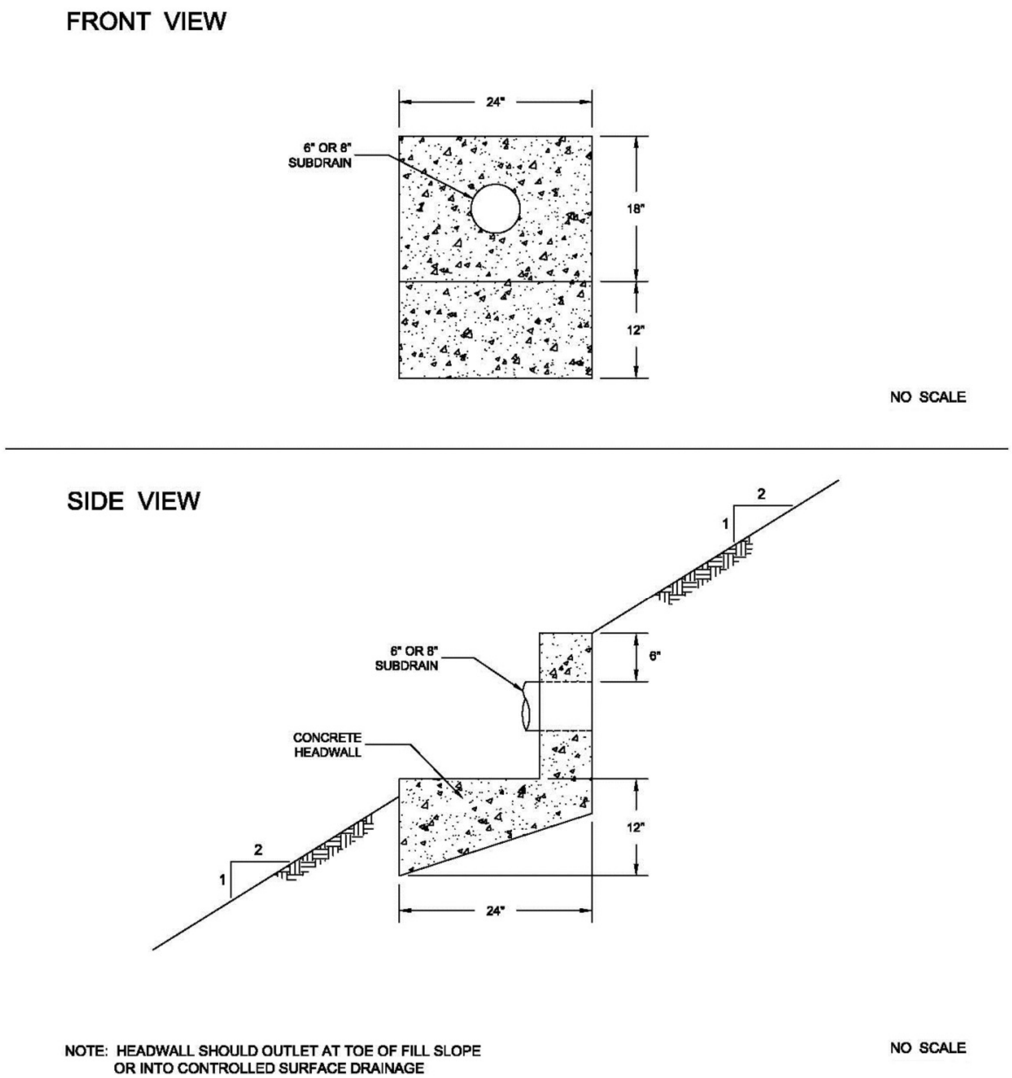


SIDE VIEW



7.6 Subdrains that discharge into a natural drainage course or open space area should be provided with a permanent headwall structure.

TYPICAL HEADWALL DETAIL



7.7 The final grading plans should show the location of the proposed subdrains. After completion of remedial excavations and subdrain installation, the project civil engineer should survey the drain locations and prepare an "as-built" map showing the drain locations. The final outlet and connection locations should be determined during grading operations. Subdrains that will be extended on adjacent projects after grading can be placed on formational material and a vertical riser should be placed at the end of the subdrain. The grading contractor should consider videoing the subdrains shortly after burial to check proper installation and functionality. The contractor is responsible for the performance of the drains.

8. OBSERVATION AND TESTING

8.1 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.

8.2 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.

8.3 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.

8.4 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.

8.5 We should observe the placement of subdrains, to check that the drainage devices have been placed and constructed in substantial conformance with project specifications.

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

8.6.1 Soil and Soil-Rock Fills:

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

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

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

8.6.1.4. Expansion Index Test, ASTM D 4829, *Expansion Index Test*.

9. PROTECTION OF WORK

9.1 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.

9.2 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.

10. CERTIFICATIONS AND FINAL REPORTS

10.1 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.

10.2 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.

LIST OF REFERENCES

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3. Kennedy, M. P., and S. S. Tan (2008), *Geologic Map of the San Diego 30' x 60' Quadrangle, California*, USGS Regional Geologic Map Series, 1:100,000 Scale, Map No. 3;

4. Risk Engineering (2019), *EZ-FRISK (Version 7.65)*, software package used to perform site-specific earthquake hazard analyses, accessed April 17, 2019;

5. SEAOCC (2018), *Seismic Design Maps*, website interface that queries the U.S. Geological Survey (USGS) web servers and retrieves the seismic design variables using ASCE 7-16, ASCE 7-10, ASCE 41-13, ASCE 41-17, IBC 2015, IBC 2012, NEHRP-2015, and NEHRP 2009 seismic design map data, http://seismicmaps.org, accessed April 17, 2019;

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8. USGS (2016), *Quaternary Fault and Fold Database of the United States:* U.S. Geological Survey website, http://earthquakes,usgs.gov/hazards/qfaults.