GEOTECHNICAL INVESTIGATION

1118 NORTH ANZA STREET
TOWNHOMES
PDS2018-TM-5628/REZ-18-003
EL CAJON, CALIFORNIA

GEOCON
INCORPORATED

GEOTECHNICAL
ENVIRONMENTAL
MATERIALS

PREPARED FOR

HALL LAND COMPANY, INC.
SOLANA BEACH, CALIFORNIA

MAY 10, 2018
PROJECT NO. G2259-32-01

Project No. G2259-32-01
May 10, 2018

Hall Land Company, Inc.
740 Lomas Santa Fe Drive, Suite 204
Solana Beach, California, 92057

Attention: Mr. Sean Santa Cruz

Subject: GEOTECHNICAL INVESTIGATION
1118 NORTH ANZA STREET TOWNHOMES
PDS2018-TM-5628/REZ-18-003
EL CAJON, CALIFORNIA

Dear Mr. Santa Cruz:

In accordance with your request, and our Proposal No. LG-18099 dated March 6, 2018, we have performed a geotechnical investigation on the subject property in El Cajon, California. The accompanying report presents our conclusions and recommendations pertaining to the geotechnical aspects of project development. The results of our study indicate that the site can be developed as planned, provided the recommendations of this report are followed.

If there are any questions regarding this report, or if we may be of further service, please contact the undersigned at your convenience.

Very truly yours,

GEOCON INCORPORATED

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GEOTECHNICAL INVESTIGATION

1. PURPOSE AND SCOPE

This report presents the results of a geotechnical investigation for the proposed *1118 North Anza Street* residential development located in El Cajon, California (see *Vicinity Map,* Figure 1). The purpose of this study was to evaluate the soil and geologic conditions on the site and provide specific geotechnical recommendations pertaining to the development of the property as proposed based on the conditions encountered.

The scope of our study consisted of the following:

- Reviewing satellite imagery, previous geotechnical reports and readily available published and unpublished geologic literature.
- Excavating nine exploratory trenches to evaluate the general extent and condition of surficial deposits across the site (see Appendix A for trench logs).
- Drilling 5 hollow-stem auger borings to evaluate the general extent and condition of surficial deposits and granitic rock across the site (see Appendix A for boring logs).
- Coring the surrounding roadways to evaluate the existing pavement sections on North Anza Street and Denver Lane. A total of 3 cores were drilled and backfilled with asphalt cold patch.
- Performing laboratory tests on selected soil samples collected to evaluate their physical properties (see Appendix B).
- Providing a storm water infiltration investigation to assist in evaluating feasibility of infiltrating storm water on-site in accordance with the 2016 storm water standards (see Appendix C). Three Aardvark constant head permeameter tests were performed to evaluate the permeability characteristics of the underlying soils.
- Preparing this report presenting our exploratory information and our conclusions and recommendations regarding the geotechnical aspects of developing the site as presently proposed. The approximate locations of the subsurface excavations are shown on the *Geologic Map*, Figure 2.

2. SITE AND PROJECT DESCRIPTION

The property consists of approximately 3 acres of land located north of Broadway, east of North Mollison Avenue, and south of East Bradley Avenue (see Vicinity Map, Figure 1). The site is currently occupied by four single family houses.

Topographically, the site is characterized as relatively flat with an elevation of approximately 446 to 451 feet above Mean Sea Level (MSL). The property is relatively void of vegetation. Several isolated trees and some sparse vegetation is present.

It is our understanding that the proposed development will consist of grading the property to accommodate a two-story, 39-unit development with associated underground utilities and landscaping improvements. Grading plans were not available, however, we understand that existing grades will be raised approximately 10 feet across the property. Maximum cut and fill depths are expected to be on the order of approximately 3 to 13 feet, respectively.

The descriptions contained herein are based upon the site reconnaissance and our understanding of the project. If project details vary significantly from those outlined herein, Geocon Incorporated should be notified for review and possible revisions to this report prior to final design submittal.

3. SOIL AND GEOLOGIC CONDITIONS

One surficial soil type and one geologic formation was encountered during the field investigation. The surficial deposit consists of colluvium and the formational unit consists of granitic rock. Each of the geologic units is described below in order of increasing age. The approximate extent of the deposits are shown on the *Geologic Map*.

3.1 Colluvium (Qcol)

Colluvium was observed in all of the exploratory excavations across the property. The colluvium overlies the granitic rock, varies in thickness from approximately 5 to 12 feet, and is generally comprised of loose to dense, dry to moist, silty/clayey sand and stiff, moist, sandy clay. The upper approximately 3 feet of these deposits will require remedial grading in areas of proposed improvements.

3.2 Granitic Rock (Kgr)

Cretaceous-age granitic rock of the Southern California Batholith underlies the site beneath the surficial deposits. Granitic rock was encountered between approximately 5 and 12 feet below the ground surface. The Granitic rock excavates as a brown to dark gray, silty/clayey, fine to coarse sand. Generally, the granitic rock is completely to highly weathered; however, hard core-stones may also be present. Fractures and/or joints within the rock may contribute to the propagation of groundwater or seepage. Excavations within this unit can typically be accomplished with conventional heavy-duty grading and trenching equipment with heavy to very heavy effort and possible refusal. The granitic rock is considered suitable for the support of the proposed development, however, is not expected to be encountered, except for underground excavations greater than approximately 5 feet below existing grades.

4. GROUNDWATER

Groundwater was not encountered during the field investigation, however, minor to moderate seepage was observed along the granitic rock contact. The seepage is not anticipated to significantly impact project development as presently proposed. Proper surface drainage of irrigation and rainwater will be important to future performance of the project.

5. GEOLOGIC HAZARDS

5.1 Faulting

Based on our reconnaissance and a review of published geologic maps and reports, the site is not located on any known “active,” “potentially active” or “inactive” fault traces as defined by the California Geological Survey (CGS).

The Newport-Inglewood and Rose Canyon Faults, located approximately 14 miles west of the site, are the closest known active faults. The CGS considers a fault seismically active when evidence suggests seismic activity within roughly the last 11,000 years. The CGS has included portions of the Rose Canyon Fault zone within an Alquist-Priolo Earthquake Fault Zone.

5.2 Seismicity-Deterministic Analysis

We used the computer program *EZ-FRISK* (Version 7.65) to determine the distance of known faults to the site and to estimate ground accelerations at the site for the maximum anticipated seismic event.

According to the results of the computer program *EZ-FRISK* (Version 7.65), 7 known active faults are located within a search radius of 50 miles from the property. We used acceleration attenuation relationships developed by Boore-Atkinson (2008) NGA USGS2008, Campbell-Bozorgnia (2008) NGA USGS, and Chiou-Youngs (2008) NGA in our analysis. The nearest known active faults are the Newport-Inglewood and Rose Canyon Fault Zones, located approximately 14 miles west of the site, respectively, and are the dominant sources of potential ground motion. Table 5.2 lists the estimated maximum earthquake magnitudes and PGA’s for the most dominant faults for the site location calculated for Site Class D as defined by Table 1613.3.2 of the 2016 California Building Code (CBC).

TABLE 5.2
DETERMINISTIC SPECTRA SITE PARAMETERS

Fault Name	Distance from Site (miles)	Maximum Earthquake Magnitude (Mw)	Peak Ground Acceleration		
			Boore-Atkinson 2008 (g)	Campbell-Bozorgnia 2008 (g)	Chiou-Youngs 2007 (g)
Newport-Inglewood	14	7.5	0.23	0.17	0.22
Rose Canyon	14	6.9	0.19	0.15	0.16
Coronado Bank	26	7.4	0.16	0.11	0.12
Palos Verdes Connected	26	7.7	0.18	0.12	0.15
Elsinore	29	7.85	0.18	0.12	0.15
Earthquake Valley	33	6.8	0.11	0.07	0.07
San Jacinto	49	7.88	0.12	0.08	0.10

5.3 Seismicity-Probabilistic Analysis

We used the computer program *EZ-FRISK* (version 7.65) to perform a probabilistic seismic hazard analysis. *EZ-FRISK* operates under the assumption 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 rupture length. Site acceleration estimates are made using the earthquake magnitude and distance from the site to the rupture zone. The program also accounts for uncertainty in each of following: (1) earthquake magnitude, (2) rupture length for a given magnitude, (3) location of the rupture zone, (4) maximum possible magnitude of a given earthquake, and (5) acceleration at the site from a given earthquake along each fault. By calculating the expected accelerations from considered earthquake sources, the program calculates the total average annual expected number of occurrences of site acceleration greater than a specified value. We utilized acceleration-attenuation relationships suggested by Boore-Atkinson (2008) NGA USGS 2008, Campbell-Bozorgnia (2008) NGA USGS 2008, and Chiou-Youngs (2008) NGA USGS 2008 in the analysis. Table 5.3 presents the site-specific probabilistic seismic hazard parameters including acceleration-attenuation relationships and the probability of exceedence for Site Class D.

TABLE 5.3
PROBABILISTIC SEISMIC HAZARD PARAMETERS

Probability of Exceedence	Peak Ground Acceleration		
	Boore-Atkinson, 2008 (g)	Campbell-Bozorgnia, 2008 (g)	Chiou-Youngs, 2008 (g)
2% in a 50 Year Period	0.43	0.36	0.41
5% in a 50 Year Period	0.33	0.27	0.30
10% in a 50 Year Period	0.26	0.22	0.23

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) or City of El Cajon guidelines.

5.4 Landslides

No evidence of ancient landslide deposits was encountered at the site during the geotechnical investigation.

5.5 Liquefaction and Seismically Induced Settlement

Liquefaction typically occurs when a site is located in a zone with seismic activity, onsite soils are cohesionless, groundwater is encountered within 50 feet of the surface, and soil relative densities are less than about 70 percent. If all four previous criteria are met, a seismic event could result in a rapid pore-water pressure increase from the earthquake-generated ground accelerations. Seismically induced settlement is settlement that may occur whether the potential for liquefaction exists or not. The potential for liquefaction and seismically induced settlement occurring within the site soils is considered to be “very low” due to the geologic conditions encountered, remedial grading recommended, and absence of groundwater.

6. CONCLUSIONS AND RECOMMENDATIONS

6.1 General

6.1.1 No soil or geologic conditions were encountered that, in the opinion of Geocon Incorporated, would preclude the development of the property as proposed, provided the recommendations of this report are followed.

6.1.2 The site is underlain by approximately 5 to 12 feet of colluvial deposits over granitic rock. Based on our observations and laboratory test results, the upper portions of the colluvial deposits are dry and potentially compressible when subjected to additional loading, and will require remedial grading.

6.1.3 With the exception of possible strong seismic shaking, no geologic hazards were observed or are known to exist based on our study that would adversely affect the proposed project. No special seismic design considerations, other than those recommended herein, are required.

6.1.4 The proposed structures can be supported by conventional continuous and isolated spread foundations supported entirely in compacted fill.

6.1.5 Any existing structures, foundation systems, pavement, utility lines, etc., should be removed and exported from the site prior to grading. Undocumented fills associated with these items and with past underground improvements should also be removed and properly compacted. Geocon Incorporated should observe the underlying geologic conditions and provide testing and observation services during the backfill of the resulting excavations where necessary.

6.2 Excavation and Soil Characteristics

6.2.1 Excavation of the surficial soils should be possible with light to moderate effort using conventional heavy-duty equipment. Excavations within the granitic rock, if any, should be possible with moderate to heavy effort using conventional heavy-duty equipment.

6.2.2 The soils encountered in the field investigation are considered to be “expansive” (expansion index [EI] greater than 20) as defined by 2016 California Building Code (CBC) Section 1803.5.3 based on laboratory testing. Table 6.2 presents soil classifications based on the expansion index.

TABLE 6.2
EXPANSION CLASSIFICATION BASED ON EXPANSION INDEX

Expansion Index (EI)	ASTM 4829 Expansion Classification	2016 CBC Expansion Classification
0 – 20	Very Low	Non-Expansive
21 – 50	Low	Expansive
51 – 90	Medium	
91 – 130	High	
Greater Than 130	Very High	

6.3 Corrosion

6.3.1 We performed laboratory tests on a sample of the site materials to evaluate the percentage of water-soluble sulfate content. Results from the laboratory water-soluble sulfate content tests are presented in Appendix B and indicate that the on-site materials at the locations tested possess "S0" to "S2" sulfate exposure and "Not Applicable" to "Severe" sulfate severity to concrete structures as defined by 2016 CBC Section 1904 and ACI 318-14 Chapter 19. 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 6.3 presents a summary of concrete requirements set forth by 2016 CBC Section 1904 and ACI 318.

TABLE 6.3
REQUIREMENTS FOR CONCRETE EXPOSED
TO SULFATE-CONTAINING SOLUTIONS

Sulfate Severity	Exposure Class	Water-Soluble Sulfate (SO_4) Percent by Weight	Cement Type (ASTM C 150)	Maximum Water to Cement Ratio by Weight[1]	Minimum Compressive Strength (psi)
Not Applicable	S0	$SO_4<0.10$	No Type Restriction	n/a	2,500
Moderate	S1	$0.10\leq SO_4<0.20$	II	0.50	4,000
Severe	S2	$0.20\leq SO_4\leq 2.00$	V	0.45	4,500
Very Severe	S3	$SO_4>2.00$	V+Pozzolan or Slag	0.45	4,500

[1] Maximum water to cement ratio limits do not apply to lightweight concrete.

6.3.2 Geocon Incorporated does not practice in the field of corrosion engineering. Therefore, if improvements that could be susceptible to corrosion are planned, it is recommended that further evaluation by a corrosion engineer be performed.

6.4 Grading Recommendations

6.4.1 All grading should be performed in accordance with the attached *Recommended Grading Specifications* (Appendix D). Where the recommendations of this section conflict with Appendix D, the recommendations of this section take precedence. All earthwork should be observed and all fills tested for proper compaction by Geocon Incorporated.

6.4.2 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.

6.4.3 Site preparation should begin with the removal of all deleterious material, asphalt concrete, concrete, and vegetation. The depth of removal should be such that material exposed in cut areas or soils to be used as fill are relatively free of organic matter and construction debris. Material generated during stripping and/or site demolition should be exported from the site.

6.4.4 The upper 3 feet of the surficial soils across the entire site should be removed, moisture conditioned, and properly compacted to provide uniform foundation conditions. The actual extent of unsuitable soil removal will be determined in the field by the geotechnical engineer and/or engineering geologist.

6.4.5 After removal of unsuitable materials is performed, the site should then be brought to final subgrade elevations with structural fill compacted in layers. In general, soils native to the site are suitable for re-use as fill if free from vegetation, debris, and other deleterious material. Layers of fill should be no thicker than will allow for adequate bonding and compaction. All fill, including backfill and scarified ground surfaces, should be compacted to at least 90 percent of maximum dry density at or above optimum moisture content, as determined in accordance with ASTM Test Procedure D1557. Fill materials below optimum moisture content will require additional moisture conditioning prior to placing additional fill.

6.4.6 Where practical, the upper 3 feet of the building pads should be comprised of soil with a "very low" to "low" expansion potential. The more highly expansive fill soils should be placed in the deeper fill areas, if present. "Very low" to "low" expansive soils are defined by the 2016 California Building Code (CBC) Section 1803.5.3 as those soils that have an Expansion Index of 50 or less.

6.4.7 Prior to placing fill, the ground surface should be scarified to a depth of 12 inches, moisture conditioned, and compacted to a dry density of at least 90 percent of the laboratory maximum dry density near to slightly above optimum moisture content, as determined by ASTM Test Method D 1557. Deeper processing and/or removal may be necessary in areas where loose, wet or dry soils are encountered.

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

6.4.9 Import fill should consist of granular materials with a “very low” to “low” expansion potential (EI of 50 or less) free of deleterious material or stones larger than 3 inches, and should be compacted as recommended above. Geocon Incorporated should be notified of the import soil source and should perform laboratory testing of import soil prior to its arrival at the site to determine its suitability as fill material.

6.5 Seismic Design Criteria

6.5.1 We used the computer program *U.S. Seismic Design Maps*, provided by the USGS. Table 6.5.1 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 seconds. The values presented in Table 6.5.1 are for the risk-targeted maximum considered earthquake (MCE_R). Based on soil conditions and planned grading, the building should be designed using a Site Class D. 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.

TABLE 6.5.1
2016 CBC SEISMIC DESIGN PARAMETERS

Parameter	Value	2016 CBC Reference
Site Class	D	Section 1613.3.2
MCE_R Ground Motion Spectral Response Acceleration – Class B (short), S_S	0.867g	Figure 1613.3.1(1)
MCE_R Ground Motion Spectral Response Acceleration – Class B (1 sec), S_1	0.336g	Figure 1613.3.1(2)
Site Coefficient, F_A	1.153	Table 1613.3.3(1)
Site Coefficient, F_V	1.728	Table 1613.3.3(2)
Site Class Modified MCE_R Spectral Response Acceleration (short), S_{MS}	1.000g	Section 1613.3.3 (Eqn 16-37)
Site Class Modified MCE_R Spectral Response Acceleration (1 sec), S_{M1}	0.580g	Section 1613.3.3 (Eqn 16-38)
5% Damped Design Spectral Response Acceleration (short), S_{DS}	0.666g	Section 1613.3.4 (Eqn 16-39)
5% Damped Design Spectral Response Acceleration (1 sec), S_{D1}	0.387g	Section 1613.3.4 (Eqn 16-40)

6.5.2 Table 6.5.2 presents additional seismic design parameters for projects located in Seismic Design Categories of D through F in accordance with ASCE 7-10 for the mapped maximum considered geometric mean (MCE_G).

TABLE 6.5.2
2016 CBC SITE ACCELERATION PARAMETERS

Parameter	Value, Site Class D	ASCE 7-10 Reference
Mapped MCE_G Peak Ground Acceleration, PGA	0.326g	Figure 22-7
Site Coefficient, F_{PGA}	1.174	Table 11.8-1
Site Class Modified MCE_G Peak Ground Acceleration, PGA_M	0.383g	Section 11.8.3 (Eqn 11.8-1)

6.5.3 Conformance to the criteria for seismic design does not constitute any guarantee or assurance that significant structural damage or ground failure will not occur in the event of a maximum level earthquake. The primary goal of seismic design is to protect life and not to avoid all damage, since such design may be economically prohibitive.

6.6 Foundation and Concrete Slab-On-Grade Recommendations

6.6.1 The following foundation recommendations are for proposed one- to three-story residential structures. The foundation recommendations have been separated into three categories based on either the maximum and differential fill thickness or Expansion Index. The foundation category criteria are presented in Table 6.6.1.

TABLE 6.6.1
FOUNDATION CATEGORY CRITERIA

Foundation Category	Maximum Fill Thickness, T (feet)	Differential Fill Thickness, D (feet)	Expansion Index (EI)
I	T<20	--	EI≤50
II	20≤T<50	10≤D<20	50<EI≤90
III	T≥50	D≥20	90<EI≤130

6.6.2 We will provide the final foundation category for the buildings after finish pad grades have been achieved and laboratory testing of the finish grade soil has been completed. However, we expect Category I foundations due to the planned import of low expansive soil.

6.6.3 Table 6.6.2 presents minimum foundation and interior concrete slab design criteria for conventional foundation systems.

TABLE 6.6.2
CONVENTIONAL FOUNDATION RECOMMENDATIONS BY CATEGORY

Foundation Category	Minimum Footing Embedment Depth (inches)	Continuous Footing Reinforcement	Interior Slab Reinforcement
I	12	Two No. 4 bars, one top and one bottom	6 x 6 - 10/10 welded wire mesh at slab mid-point
II	18	Four No. 4 bars, two top and two bottom	No. 3 bars at 24 inches on center, both directions
III	24	Four No. 5 bars, two top and two bottom	No. 3 bars at 18 inches on center, both directions

6.6.4 The embedment depths presented in Table 6.6.2 should be measured from the lowest adjacent pad grade for both interior and exterior footings. The conventional foundations should have a minimum width of 12 inches and 24 inches for continuous and isolated footings, respectively. A typical wall/column footing detail is presented on Figure 3.

6.6.5 The concrete slabs-on-grade should be a minimum of 4 inches thick for Foundation Categories I and II and 5 inches thick for Foundation Category III. The concrete slabs-on-grade should be underlain by 4 inches and 3 inches of clean sand for 4-inch thick and 5-inch-thick slabs, respectively. Slabs expected to receive moisture sensitive floor coverings or used to store moisture sensitive materials should be underlain by a vapor inhibitor covered with at least 2 inches of clean sand or crushed rock. If crushed rock will be used, the thickness of the vapor inhibitor should be at least 10 mil to prevent possible puncturing.

6.6.6 As a substitute, the layer of clean sand (or crushed rock) beneath the vapor inhibitor recommended in the previous section can be omitted if a vapor inhibitor that meets or exceeds the requirements of ASTM E 1745-97 (Class A), and that exhibits permeance not greater than 0.012 perm (measured in accordance with ASTM E 96-95) is used. This vapor inhibitor may be placed directly on properly compacted fill or formational materials. The vapor inhibitor should be installed in general conformance with ASTM E 1643-98 and the manufacturer's recommendations. Two inches of clean sand should then be placed on top of the vapor inhibitor to reduce the potential for differential curing, slab curl, and cracking. Floor coverings should be installed in accordance with the manufacturer's recommendations.

6.6.7 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 structures. 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 6.6.3 for the particular Foundation Category designated. The parameters presented in Table 6.6.3 are based on the guidelines presented in the PTI DC 10.5 design manual.

TABLE 6.6.3
POST-TENSIONED FOUNDATION SYSTEM DESIGN PARAMETERS

Post-Tensioning Institute (PTI), Third Edition Design Parameters	Foundation Category		
	I	II	III
Thornthwaite Index	-20	-20	-20
Equilibrium Suction	3.9	3.9	3.9
Edge Lift Moisture Variation Distance, e_M (feet)	5.3	5.1	4.9
Edge Lift, y_M (inches)	0.61	1.10	1.58
Center Lift Moisture Variation Distance, e_M (feet)	9.0	9.0	9.0
Center Lift, y_M (inches)	0.30	0.47	0.66

6.6.8 Foundation systems for the lots that possess a foundation Category I and a "very low" expansion potential (expansion index of 20 or less) can be designed using the method described in Section 1808 of the 2016 CBC. If post-tensioned foundations are planned, an alternative, commonly accepted design method (other than PTI DC 10.5) can be used. However, the post-tensioned foundation system should be designed with a total and differential deflection of 1 inch. Geocon Incorporated should be contacted to review the plans and provide additional information, if necessary. This foundation category alternative is commonly referred to as CAT 1A.

6.6.9 If an alternate design method is contemplated, Geocon Incorporated should be contacted to evaluate if additional expansion index testing should be performed to identify the lots that possess a "very low" expansion potential (expansion index of 20 or less).

6.6.10 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.

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

- The deflection criteria presented in Table 6.6.3 are still applicable.
- Interior stiffener beams should be used for Foundation Categories II and III.
- The width of the perimeter foundations should be at least 12 inches.
- The perimeter footing embedment depths should be at least 12 inches, 18 inches and 24 inches for foundation categories I, II, and III, respectively. The embedment depths should be measured from the lowest adjacent pad grade.

6.6.12 Our experience indicates post-tensioned slabs may be 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 structural engineer should design the foundation system to reduce the potential of edge lift occurring for the proposed structures.

6.6.13 During the construction of the post-tension foundation system, the concrete should be placed monolithically. Under no circumstances should cold joints be allowed to form between the footings/grade beams and the slab during the construction of the post-tension foundation system unless designed by the structural engineer.

6.6.14 Category I, II, or III foundations may be designed for an allowable soil bearing pressure of 2,000 pounds per square foot (psf) (dead plus live load). This bearing pressure may be increased by one-third for transient

6.6.15 Isolated footings, if present, should have the minimum embedment depth and width recommended for conventional foundations for a particular Foundation Category. 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 for Category III. Where this condition cannot be avoided, the isolated footings should be connected to the building foundation system with grade beams.

6.6.16 For Foundation Category III, consideration should be given to using interior stiffening beams and connecting isolated footings and/or increasing the slab thickness. In addition, consideration should be given to connecting patio slabs, which exceed 5 feet in width, to the building foundation to reduce the potential for future separation to occur.

6.6.17 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.

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

- For fill slopes less than 20 feet high, 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 reduce the potential for distress in the structures associated with strain softening and lateral fill extension. Specific design parameters or recommendations for either of these alternatives can be provided once the building location and fill slope geometry have been determined.

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

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

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

6.6.19 Exterior concrete flatwork not subject to vehicular traffic should be constructed in accordance with the recommendations herein. Slab panels should be a minimum of 4 inches thick and, when in excess of 8 feet square, should be reinforced with 6 x 6 - W2.9/W2.9 (6 x 6 - 6/6) welded wire mesh or No. 3 reinforcing bars at 18 inches on center in both directions to reduce the potential for cracking. In addition, concrete flatwork should be provided with crack control joints to reduce and/or control shrinkage cracking. Crack control spacing should be determined by the project structural engineer based upon the slab thickness and intended usage. Criteria of the American Concrete Institute (ACI) should be taken into consideration when establishing crack control spacing. A 4-inch-thick slab should have a maximum joint spacing of 10 feet. Subgrade soil for exterior slabs not subjected to vehicle loads should be compacted in accordance with criteria presented in the grading section prior to concrete placement. Subgrade soil should be properly compacted and the moisture content of subgrade soil should be checked prior to placing concrete.

6.6.20 The recommendations of this report are intended to reduce the potential for cracking of slabs and foundations due to expansive soil (if present), differential settlement of fill 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 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.

6.6.21 Concrete slabs should be provided with adequate crack-control joints, construction joints and/or expansion joints to reduce unsightly shrinkage cracking. The design of joints should consider criteria of the American Concrete Institute (ACI) when establishing crack-control spacing. Additional steel reinforcing, concrete admixtures and/or closer crack control joint spacing should be considered where concrete-exposed finished floors are planned.

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

6.7 Retaining Walls and Lateral Loads Recommendations

6.7.1 Retaining walls not restrained at the top and having a level backfill surface should be designed for an active soil pressure equivalent to the pressure exerted by a fluid with a density of 35 pounds per cubic foot (pcf). Where the backfill will be inclined at 2:1 (horizontal:vertical), an active soil pressure of 50 pcf is recommended. These soil pressures assume that the backfill materials within an area bounded by the wall and a 1:1 plane extending upward from the base of the wall possess an Expansion Index ≤50. Geocon Incorporated should be consulted for additional recommendations if backfill materials have an EI >50.

6.7.2 Retaining walls shall be designed to ensure stability against overturning sliding, excessive foundation pressure and water uplift. Where a keyway is extended below the wall base with the intent to engage passive pressure and enhance sliding stability, it is not necessary to consider active pressure on the keyway.

6.7.3 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 (total unit weight of soil should be taken as 130 pcf).

6.7.4 Soil contemplated for use as retaining wall backfill, including import materials, 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 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 for use as wall backfill if standard wall designs will be used.

6.7.5 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.

6.7.6 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.

6.7.7 In general, wall foundations having a minimum depth and width of one foot may be designed for an allowable soil bearing pressure of 2,000 psf, provided the soil within three feet below the base of the wall has an Expansion Index $\leq$ 90. The recommended allowable soil bearing pressure 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.

6.7.8 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.

6.7.9 The structural engineer should determine the Seismic Design Category for the project in accordance with Section 1613.3.5 of the 2016 CBC or Section 11.6 of ASCE 7-10. For structures assigned to 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 20H should be used for design. We used the peak ground acceleration adjusted for Site Class effects, PGA_M, of 0.383g calculated from ASCE 7-10 Section 11.8.3 and applied a pseudo-static coefficient of 0.33.

6.7.10 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 formational 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.

6.7.11 An ultimate friction coefficient of 0.35 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.

6.7.12 The recommendations presented above are generally applicable to the design of rigid concrete or masonry retaining walls having a maximum height of 12 feet. In the event that walls higher than 12 feet are planned, Geocon Incorporated should be consulted for additional recommendations.

6.8 Mechanically-Stabilized Earth (MSE) Retaining Walls

6.8.1 The geologic conditions in the vicinity of proposed retaining walls are anticipated to consist of compacted fill over formational materials. The formational materials consist of granitic rock. The compacted fill soils in the foundation and retained zones are expected to consist of imported silty sands. Based our experience with similar soil and geologic conditions, we recommend the following geotechnical parameters be used for design of the MSE retaining walls.

TABLE 6.8.1
MSE RETAINING WALL PARAMETERS

Parameter	Reinforced Zone	Retained Zone	Foundation Zone
Angle of Internal Friction	30 degrees	30 degrees	30 degrees
Cohesion	0 psf	0 psf	0 psf
Wet Unit Weight	125 pcf	125 pcf	125 pcf

6.8.2 The shear strength values used for the reinforced zone assume that predominately granular materials will be stockpiled for use as backfill. Geocon has no way of knowing whether these materials will actually be used as backfill behind the wall during construction. As such, once backfill materials have been selected and/or stockpiled, sufficient shear tests should be conducted on samples of the proposed backfill materials to verify they conform to actual design values. Results should be provided to the designer to re-evaluate stability of the walls. Dependent upon test results, the designer may require modifications to the original wall design (e.g., longer geogrid embedment lengths).

6.8.3 Backfill materials within the reinforced zone should be compacted to a dry density of at least 90 percent of the laboratory maximum dry density at or slightly above optimum moisture content in accordance with ASTM D 1557. This is applicable to the entire embedment length of the geogrid reinforcement. In addition, the wall designer has maximum particle size (typically 3-inches in size or less) and shape (angular/rounded) requirements for soil-rock fill within the reinforced zone. Typically, wall designers specify that heavy compaction equipment be excluded from within 3 feet of the face of the wall; however, smaller equipment (e.g., walk-behind, self-driven compactors or hand whackers) should be used to compact the materials without causing deformation of the wall. If the designer specifies no compactive effort for this zone, the materials are essentially not properly compacted and the geogrid within the uncompacted zone should not be relied upon for reinforcement and overall embedment lengths should be increased to account for the difference.

6.8.4 The wall designer should provide a drainage system sufficient to dissipate hydrostatic pressure behind the wall and to mitigate seepage through and beneath the wall. As such, a subdrain system consisting of a minimum 4-inch diameter, Schedule 40, perforated pvc pipe surrounded by at least 1 cubic foot of ¾-inch open-graded gravel and wrapped in filter fabric (Mirafi 140N or equivalent) should be incorporated into the wall design. In order to prevent soil piping into the open-graded gravel layer behind the wall, we recommend the filter fabric be extended to cover the entire gravel layer. The final segment of the subdrain should outlet into an approved drainage facility, such as storm drain or headwall structure.

The final segment of the drain should consist of solid pvc pipe. At the transition between the solid and perforated pipe, a concrete cut-off wall should be added to direct the subsurface water into the solid pipe.

6.8.5 A peak ground acceleration adjusted for Site Class effects, PGA_M, of 0.383g was calculated from ASCE 7-10 Section 11.8.3. The 2016 CBC seismic design parameters are provided herein.

6.8.6 Geosynthetic reinforcement must elongate to develop full tensile resistance. This elongation generally results in movement at the top of the wall. The amount of movement is dependent upon the height of the wall (e.g., higher walls rotate more), construction, and the type of geosynthetic used. In addition, over time reinforced-earth retaining walls have been known to exhibit creep and can undergo additional movement. Given this condition, the owner should be aware that structures and pavement placed within the reinforced and retained zones of the wall may undergo movement and should be designed to accommodate this movement.

6.9 Existing Pavements

6.9.1 Table 6.9 presents the existing pavement sections based on a total of 3 cores drilled on the surrounding roadways. The approximate locations of the 6-inch diameter cores are shown on Figure 2.

TABLE 6.9
EXISTING PCC PAVEMENT SECTION

Roadway (sample location)	Core No.	Existing AC Pavement Thickness (inch)	Existing Aggregate Base Thickness (inches)
Denver Street	C-1	2.75	6
North Anza Street	C-2	2.75	5
North Anza Street	C-3	5.25	7

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 waterproofing 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.

6.11 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.

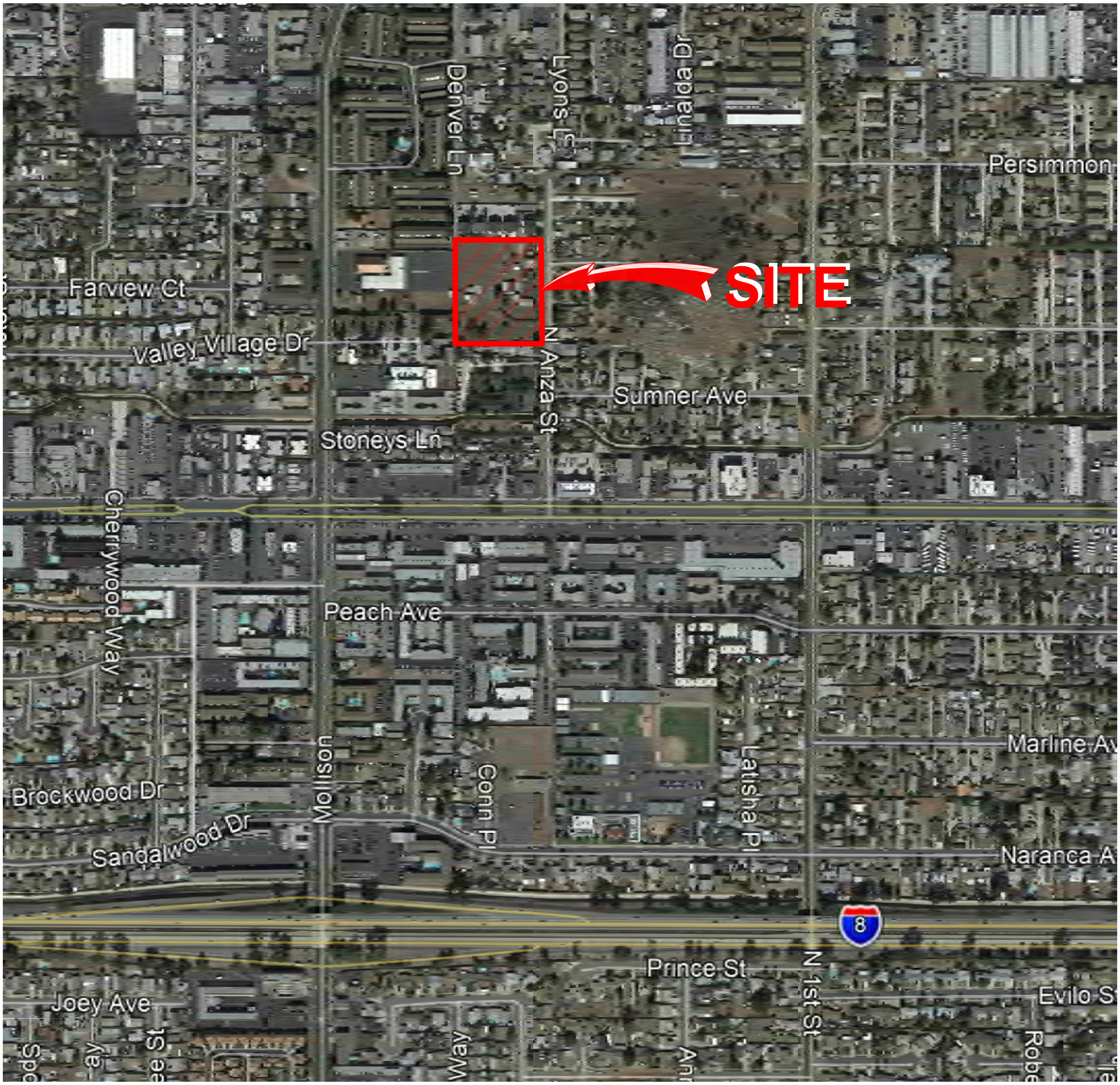
LIMITATIONS AND UNIFORMITY OF CONDITIONS

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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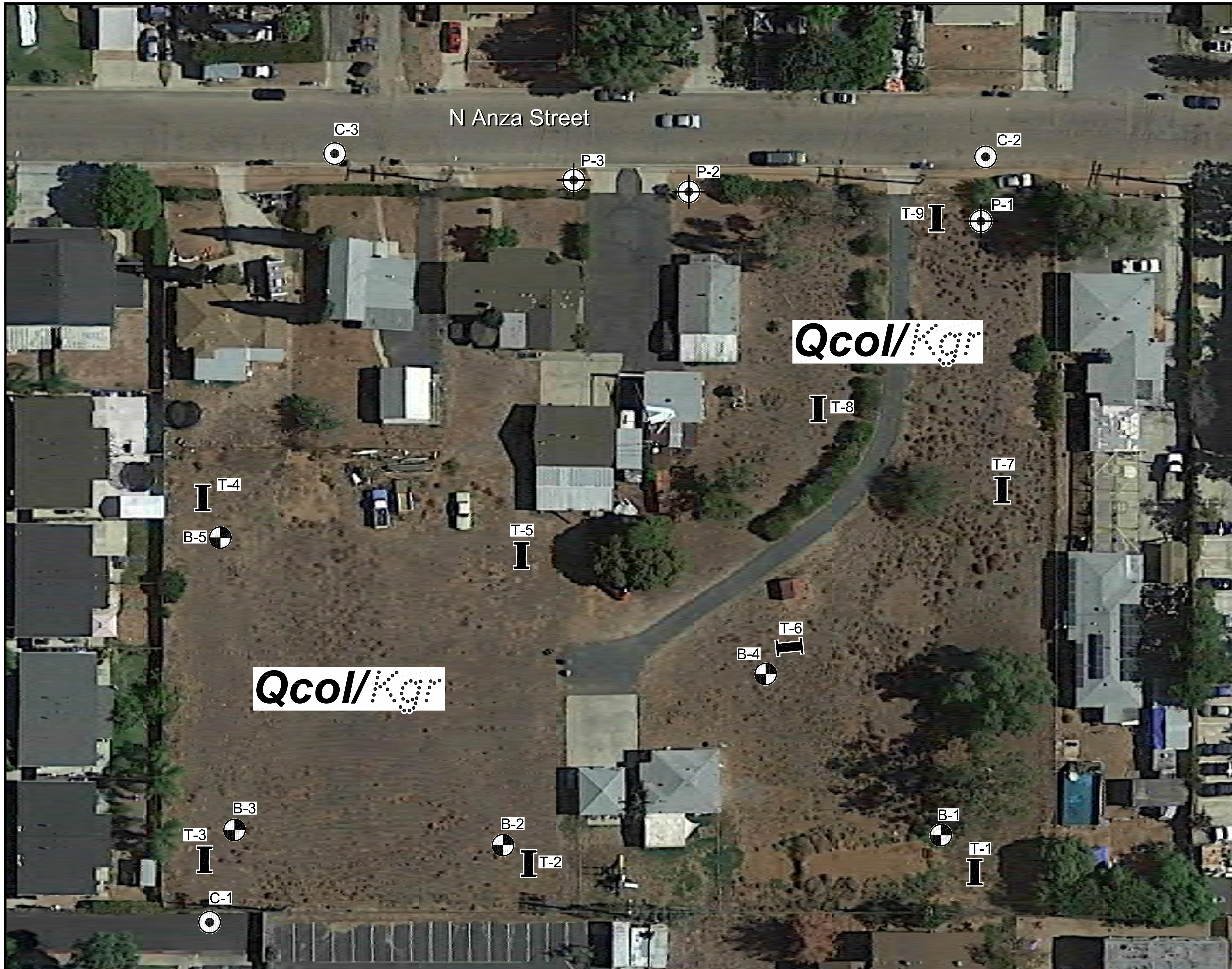
1118 N ANZA STREET TOWNHOMES
EL CAJON, CALIFORNIA

TM / CW		DSK/GTYPD	DATE 05 - 10 - 2018	PROJECT NO. G2259 - 32 - 01	FIG. 1

1118 N ANZA STREET TOWNHOMES
EL CAJON, CALIFORNIA



0' 50' 100'

SCALE 1"= 50' (On 11x17)

GEOCON LEGEND

Qcol ....COLLUVIUM

Kgr ....GRANITIC ROCK (Dotted Where Buried)

B-5 ....APPROX. LOCATION OF GEOTECHNICAL BORING

P-3 ....APPROX. LOCATION OF PERMEABILITY TEST

C-3 ....APPROX. LOCATION OF PAVEMENT CORE

T-8 ....APPROX. LOCATION OF EXPLORATORY TRENCH

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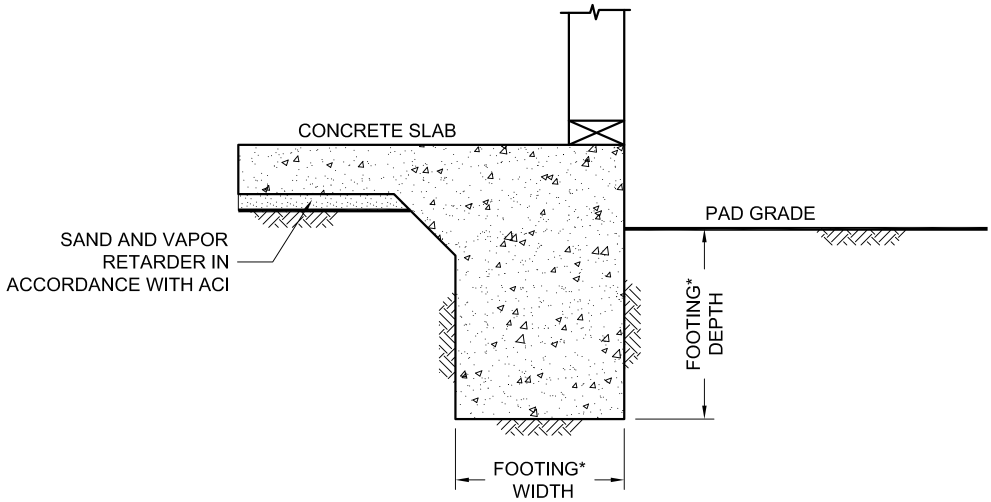
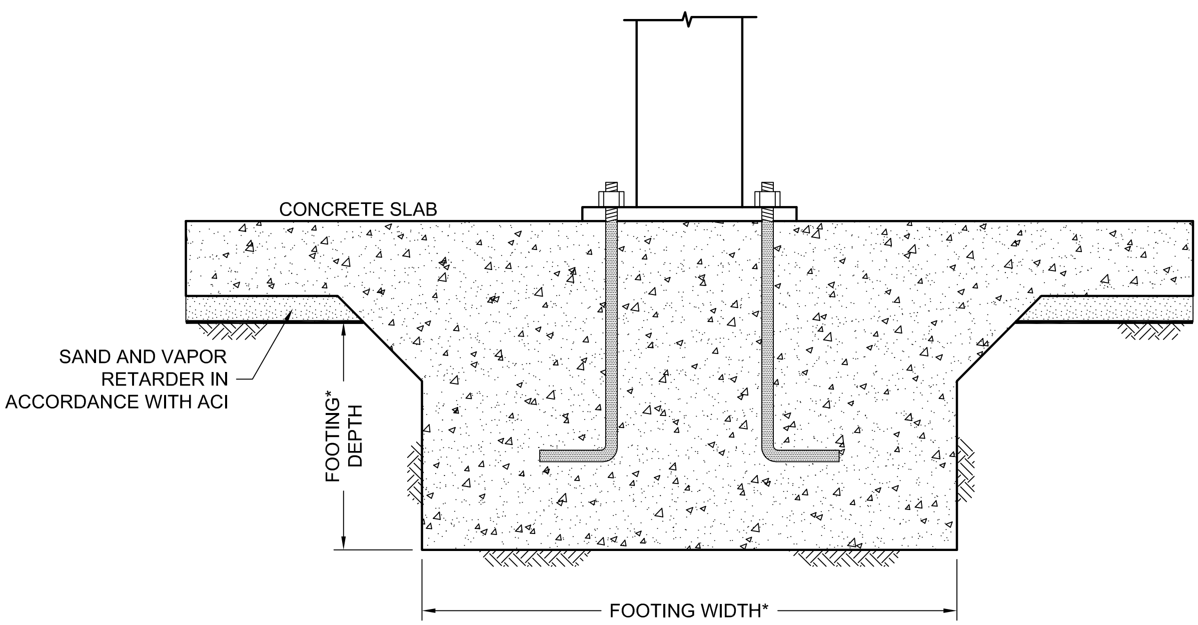
GEOLOGIC MAP

PROJECT NO. G2259 - 32 - 01
FIGURE 2
DATE 05 - 10 - 2018





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

NO SCALE

WALL / COLUMN FOOTING DIMENSION DETAIL

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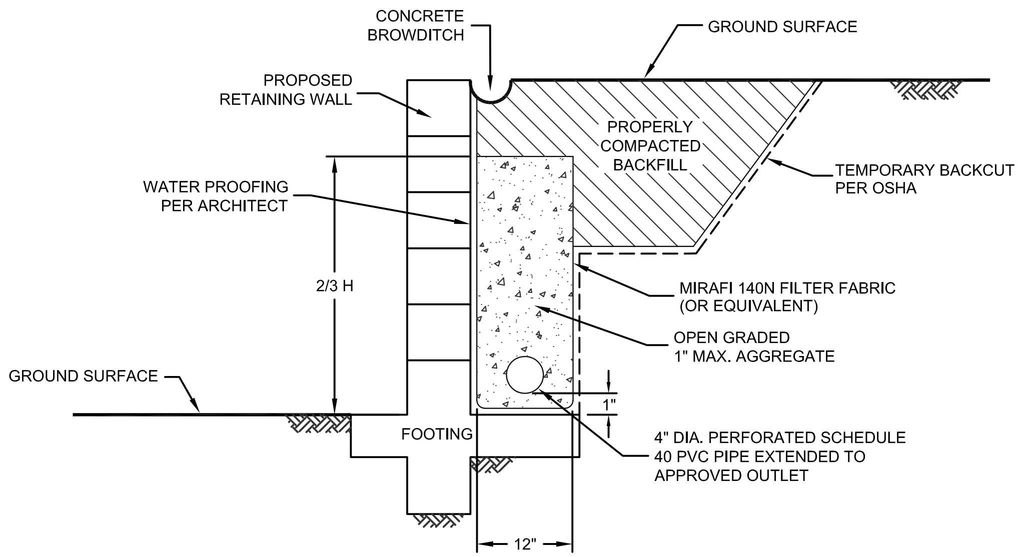
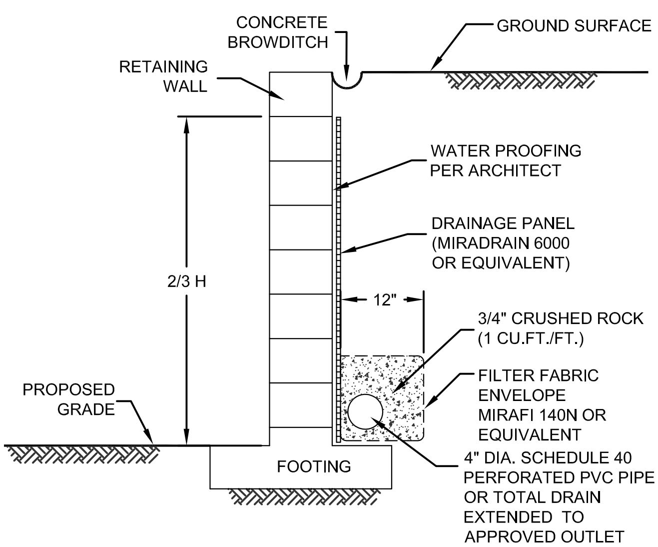
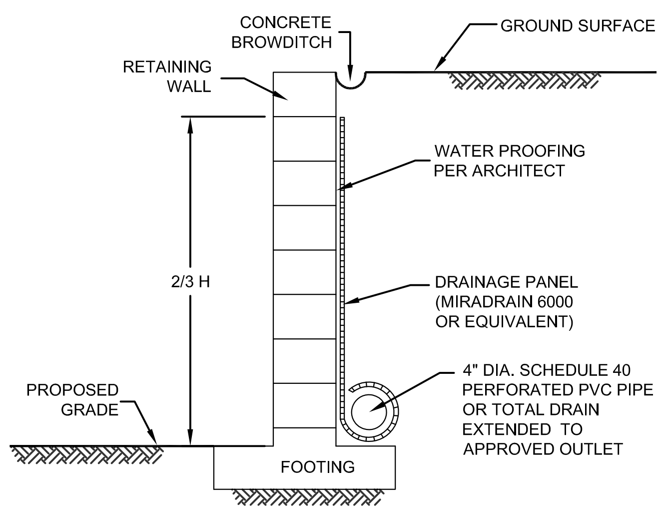
TM / CW		DSK/GTYPD	DATE 05 - 10 - 2018	PROJECT NO. G2259 - 32 - 01	FIG. 3







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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TM / CW		DSK/GTYPD	DATE 05 - 10 - 2018	PROJECT NO. G2259 - 32 - 01	FIG. 4

Plotted:05/10/2018 8:06AM | By:JONATHAN WILKINS | File Location:Y:\PROJECTS\G2259-32-01 1118 North Anza Street\DETAILS\Typical Retaining Wall Drainage Detail (RWDD7A).dwg

APPENDIX A

APPENDIX A

FIELD INVESTIGATION

The field investigation was performed on March 19 and March 20, 2018. The field investigation consisted of a visual site reconnaissance and excavating 9 exploratory trenches (Trench Nos. T-1 through T-9) and 5 hollow-stem auger borings (Boring Nos. B-1 through B-5) at various locations across the site. Three permeability tests were conducted on March 19, 2018, to evaluate storm water infiltration feasibility. In addition, 3 pavement cores were drilled to evaluate the existing pavement sections on Denver Lane and North Anza Street. The approximate locations of the trenches, borings, pavement cores, and permeability tests are shown on the *Geologic Map*, Figure 2. The results and discussion of the infiltration testing is discussed in *Appendix C* of this report.

The exploratory trenches performed by Hillside Excavating were advanced to depths of 7 to 17 feet using a JD 410G backhoe equipped with a 24-inch-wide bucket. Bulk samples were obtained for laboratory testing.

The borings were excavated by Scott's Drilling to depths of approximately 9 feet below existing grade using an Ingersoll Rand A-300 truck mounted drill rig. Relatively undisturbed and disturbed bulk samples were obtained from the borings for laboratory testing.

The soils encountered in the excavations were visually 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).

PROJECT NO. G2259-32-01

BORING B 1

ELEV. (MSL.) 446' DATE COMPLETED 03-20-2018

EQUIPMENT INGERSOL RAND 8-300 BY: D. GITHENS

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	**COLLUVIUM (Qcol)** Loose, dry, reddish brown, Silty, fine to medium SAND			
2								
	B1-1			CL	Stiff, moist, dark brown, Sandy CLAY	44	121.5	12.5
4								
	B1-2				-Becomes hard	56	110.7	17.8
6								
	B1-3					85	114.2	16.2
8					**GRANITIC ROCK (Kgr)** Completely weathered, brownish gray, weak GRANITIC ROCK; excavates as Clayey SAND/Sandy CLAY			
					BORING TERMINATED AT 9 FEET No groundwater encountered			

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

G2259-32-01.GPJ

SAMPLE SYMBOLS

... SAMPLING UNSUCCESSFUL ... STANDARD PENETRATION TEST ... DRIVE SAMPLE (UNDISTURBED)

... DISTURBED OR BAG SAMPLE ... CHUNK SAMPLE ... WATER TABLE OR SEEPAGE

GEOCON

PROJECT NO. G2259-32-01

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	**BORING B 2** ELEV. (MSL.) **447'** DATE COMPLETED **03-20-2018** EQUIPMENT **INGERSOL RAND 8-300** BY: **D. GITHENS**	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
					MATERIAL DESCRIPTION			
0				SM	**COLLUVIUM (Qcol)** Loose, dry, reddish brown, Silty, fine to medium SAND			
2				CL	Hard, moist, brown to dark brown, fine to medium Sandy CLAY			
	B2-1					50/3"	117.7	8.0
4								
	B2-2					50/5"	111.5	18.6
6								
	B2-3			SC	Dense, moist, brown to dark brown, Clayey fine to medium SAND	79	121.4	13.9
8					BORING TERMINATED AT 8 FEET No groundwater encountered			

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

G2259-32-01.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. G2259-32-01

BORING B 3

ELEV. (MSL.) **447'** DATE COMPLETED **03-20-2018**

EQUIPMENT **INGERSOL RAND 8-300** BY: **D. GITHENS**

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	**COLLUVIUM (Qcol)** Loose, dry, reddish brown, Silty, fine to medium SAND			
				SC	Medium dense, moist, Clayey fine to medium SAND			
2								
	B3-1				-Becomes dense	50/3"	109.8	10.4
4								
	B3-2			CL	Hard, moist, brown to dark brown, Sandy CLAY	78	107.3	21.3
6								
	B3-3					93	105.9	20.0
8					**GRANITIC ROCK (Kgr)** Highly weathered, brownish gray, moderately weak GRANITIC ROCK; excavates as Silty, fine to coarse SAND			

BORING TERMINATED AT 9 FEET
No groundwater encountered

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

G2259-32-01.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 4

ELEV. (MSL.) **448'** DATE COMPLETED **03-20-2018**

EQUIPMENT **INGERSOL RAND 8-300** BY: **D. GITHENS**

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	**COLLUVIUM (Qcol)** Loose, dry, reddish brown, Silty, fine to medium SAND			
2					-Becomes medium dense, moist			
	B4-1			SM-SC	Medium dense, moist, dark brown, Silty/Clayey SAND	41	112.1	3.1
4								
	B4-2					86	128.0	10.8
					-Becomes dense			
6								
	B4-3				**GRANITIC ROCK (Kgr)** Highly weathered, brownish gray, moderately weak GRANITIC ROCK; excavates as a fine to coarse sand with rock fragments	50/2"	126.2	10.0
8					BORING TERMINATED AT 8 FEET No groundwater encountered			

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

G2259-32-01.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. G2259-32-01

BORING B 5

ELEV. (MSL.) **450'** DATE COMPLETED **03-20-2018**

EQUIPMENT **INGERSOL RAND 8-300** BY: **D. GITHENS**

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	**COLLUVIUM (Qcol)** Loose, dry, reddish brown, Silty, fine to medium SAND			
				SC	Medium dense, moist, brown Clayey, fine to medium SAND			
2								
	B5-1				-Becomes medium dense, moist	50/1"	112.6	7.0
4				CL	Hard, moist, brown to dark brown, Sandy CLAY			
	B5-2					67	102.8	21.8
6								
	B5-3					82	117.5	16.8
8					BORING TERMINATED AT 8 FEET No groundwater encountered			

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

G2259-32-01.GPJ

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

GEOCON

PROJECT NO. G2259-32-01

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	TRENCH T 1 ELEV. (MSL.) 446' DATE COMPLETED 03-19-2018 EQUIPMENT JD 410G BACKHOE BY: J. PAGNILLO	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
					MATERIAL DESCRIPTION			
0				SM	**COLLUVIUM (Qcol)** Loose, moist, reddish brown, Silty, fine SAND			
2								
	T1-1			CL	Stiff, moist, dark brown, Sandy CLAY			
4								
6								
	T1-2							
8					-Becomes light grayish brown			
			▽		-Slight seepage at contact			
10					**GRANITIC ROCK (Kgr)** Completely weathered, greenish gray, weak, GRANITIC ROCK; excavates to Clayey SAND/Sandy CLAY			
	T1-3							
12								
14					-Becomes moderately weak			
					TRENCH TERMINATED AT 15 FEET Minor seepage at contact			

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

G2259-32-01.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 2

ELEV. (MSL.) 447' DATE COMPLETED 03-19-2018

EQUIPMENT JD 410G BACKHOE BY: J. PAGNILLO

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	**COLLUVIUM (Qcol)** Loose, moist, brown, Silty, fine SAND			
2				CL	Stiff, moist, brown to dark brown, fine to medium Sandy CLAY			
4								
6								
				SC	Medium dense to dense, moist, brown to dark brown, Clayey, fine to medium SAND			
8								
10								
			▽		-Slight seepage at contact			
12					**GRANITIC ROCK (Kgr)** Completely weathered, greenish gray, weak, GRANITIC ROCK; excavates to Clayey SAND/Sandy CLAY			
14								
					-Becomes moderately weathered and moderately weak			
16								
					TRENCH TERMINATED AT 17 FEET Heavy seepage at contact			

Figure A-7,
Log of Trench T 2, Page 1 of 1

G2259-32-01.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 3

ELEV. (MSL.) **447'** DATE COMPLETED **03-19-2018**

EQUIPMENT **JD 410G BACKHOE** BY: **J. PAGNILLO**

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	**COLLUVIUM (Qcol)** Loose, moist, brown, Silty, fine SAND			
1				SC	Medium dense, Clayey, fine to medium SAND			
4	T3-1			CL	Stiff, moist, brown to dark brown, Sandy CLAY			
6	T3-2							
8				SC	Medium dense to dense, light brown, Clayey SAND			
9	T3-3							
11					**GRANITIC ROCK (Kgr)** Completely weathered, greenish gray, weak, GRANITIC ROCK; excavates as Clayey SAND/Sandy CLAY			
14					TRENCH TERMINATED AT 14 FEET Moderate seepage at contact			

Figure A-8,
Log of Trench T 3, Page 1 of 1

G2259-32-01.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 4

ELEV. (MSL.) 450' DATE COMPLETED 03-19-2018

EQUIPMENT JD 410G BACKHOE BY: J. PAGNILLO

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	Loose, moist, brown, Silty, fine SAND			
				SC	Medium dense, moist, brown, Clayey, fine to medium SAND			
2								
4				CL	Stiff, moist, brown to dark brown, Sandy CLAY			
6								
8								
10					**GRANITIC ROCK (Kgr)** Moderately weak, brownish gray, moderately strong to strong, GRANITIC ROCK; excavates to Silty, fine to coarse SAND			
12								
					TRENCH TERMINATED AT 13 FEET No groundwater encountered			

Figure A-9,
Log of Trench T 4, Page 1 of 1

G2259-32-01.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 5

ELEV. (MSL.) **448'** DATE COMPLETED **03-19-2018**

EQUIPMENT **JD 410G BACKHOE** BY: **J. PAGNILLO**

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	**COLLUVIUM (Qcol)** Loose, dry, reddish brown, Silty, fine to medium SAND			
2					-Medium dense, with rootlets			
4					-Medium dense to dense			
6					**GRANITIC ROCK (Kgr)** Highly weathered, brownish gray, moderately strong, GRANITIC ROCK, excavates to fine to coarse SAND with rock fragments up to 8-inches size			
	T5-1							
8								
					TRENCH TERMINATED AT 9 FEET Groundwater not encountered			

Figure A-10,
Log of Trench T 5, Page 1 of 1

G2259-32-01.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. G2259-32-01

TRENCH T 6

ELEV. (MSL.) **448'** DATE COMPLETED **03-19-2018**

EQUIPMENT **JD 410G BACKHOE** BY: **J. PAGNILLO**

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	**COLLUVIUM (Qcol)** Loose, dry, reddish brown, Silty, fine to medium SAND			
2					-Becomes medium dense, moist, with rootlets			
					Becomes dense below 3 feet, Silty/Clayey SAND			
4				SM-SC				
6								
8			▽		-Slight seepage at contact			
					GRANITIC ROCK (Kgr) Highly weathered, brownish gray, moderately weak, GRANITIC ROCK; excavates to fine to coarse SAND with rock fragments up to 5-inches in size			
10								

TRENCH TERMINATED AT 11 FEET
Minor seepage at contact

Figure A-11,
Log of Trench T 6, Page 1 of 1

G2259-32-01.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. G2259-32-01

TRENCH T 7

ELEV. (MSL.) 447' DATE COMPLETED 03-19-2018

EQUIPMENT JD 410G BACKHOE BY: J. PAGNILLO

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	**COLLUVIUM (Qcol)** Loose, dry, reddish brown, Silty, fine to medium SAND with rootlets			
	T7-1							
2								
				CL	Stiff, moist, brown to dark brown, Sandy CLAY			
4								
6					**GRANITIC ROCK (Kgr)** Highly weathered, grayish brown, moderately weak GRANITIC ROCK			
8					-Becomes gray, moderately strong, excavates to fine to coarse SAND with rock fragments up to 4-inch size			
10					TRENCH TERMINATED AT 15 FEET No groundwater encountered			

Figure A-12,
Log of Trench T 7, Page 1 of 1

G2259-32-01.GPJ

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

PROJECT NO. G2259-32-01

TRENCH T 8

ELEV. (MSL.) **449'** DATE COMPLETED **03-19-2018**

EQUIPMENT **JD 410G BACKHOE** BY: **J. PAGNILLO**

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	**COLLUVIUM (Qcol)** Loose, dry, reddish brown, Silty, fine to medium SAND with rootlets			
2								
4				CL	Stiff, moist, brown to dark brown, Sandy CLAY			
6					**GRANITIC ROCK (Kgr)** Highly weathered, brownish gray, moderately weak, GRANITIC ROCK; excavates to fine to coarse SAND with rock fragments up to 6-inch size			
8					TRENCH TERMINATED AT 8 FEET No groundwater encountered			

Figure A-13,
Log of Trench T 8, Page 1 of 1

G2259-32-01.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. G2259-32-01

TRENCH T 9

ELEV. (MSL.)________ DATE COMPLETED **03-19-2018**

EQUIPMENT **JD 410G BACKHOE** BY: **J. PAGNILLO**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0				SC	**UNDOCUMENTED FILL (Qudf)** Loose, dry, reddish brown, Clayey SAND			
2				SM	**COLLUVIUM (Qcol)** Medium dense, dry, reddish brown, Silty, fine to medium SAND			
4				CL	Stiff, moist, brown to dark brown, Sandy CLAY			
6					**GRANITIC ROCK (Kgr)** Highly weathered, brownish gray, moderately weak, GRANITIC ROCK; excavates to fine to coarse SAND with rock fragments up to 4-inch size			
					TRENCH TERMINATED AT 7 FEET No groundwater encountered			

Figure A-14,
Log of Trench T 9, Page 1 of 1

G2259-32-01.GPJ

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

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 undisturbed and bulk samples were tested for shear strength, expansion potential, water-soluble sulfate content, and consolidation characteristics. The results of our laboratory tests are summarized on Tables B-I through B-III and Figures B-1 through B-6.

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

Sample No.	Geologic Unit	Dry Density (pcf)	Moisture Content (%)	Peak [Ultimate] Cohesion (psf)	Peak [Ultimate] Angle of Shear Resistance (degrees)
B4-1	Qcol	112.1	3.1	700 [660]	23 [23]

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

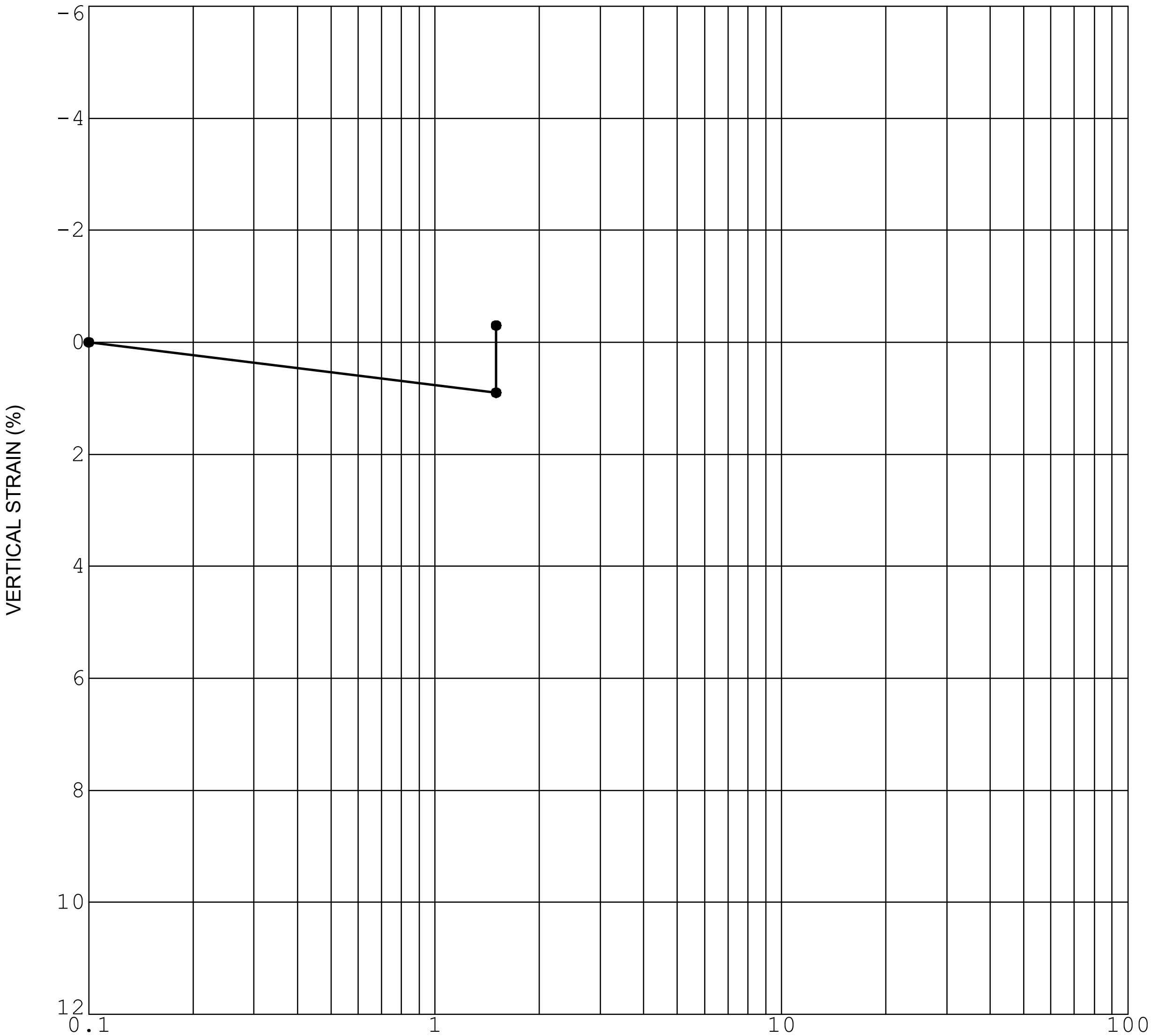
Sample No.	Moisture Content		Dry Density (pcf)	Expansion Index
	Before Test (%)	After Test (%)		
T1-1	12.3	27.4	102.1	63
T7-1	8.9	17.6	112.1	20

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

Sample No.	Water-Soluble Sulfate (%)	Sulfate Severity	Sulfate Class
T1-1	0.405	Severe	S2
T7-1	0.001	Not Applicable	S0

PROJECT NO. G2259-32-01

SAMPLE NO. B1-1



ASTM D2435

Initial Dry Density (pcf)	121.5
Initial Water Content (%)	12.5

Initial Saturation (%)	90.7
Sample Saturated at (ksf)	1.5

CONSOLIDATION CURVE

1118 N. ANZA STREET TOWNHOMES

EL CAJON, CALIFORNIA

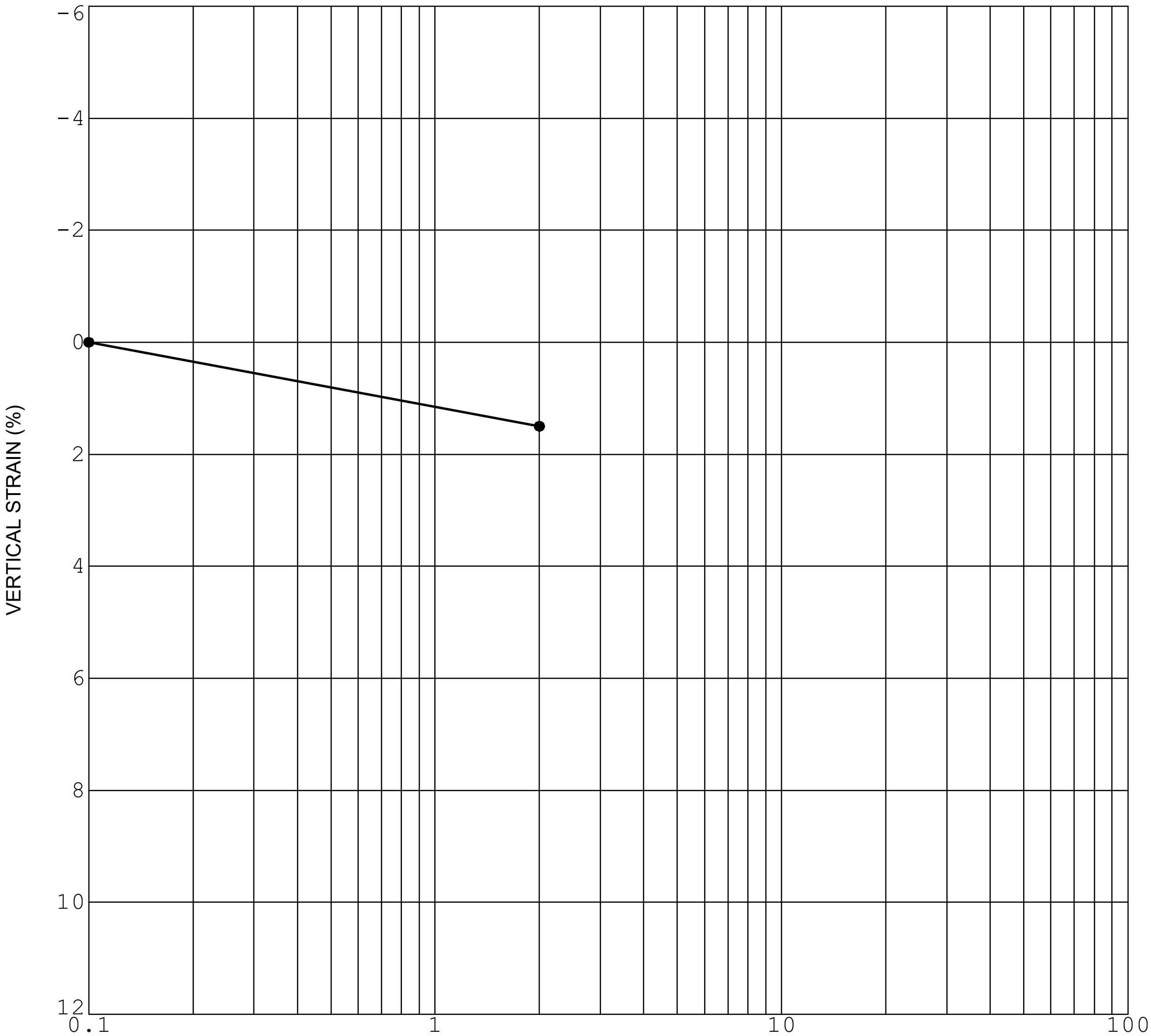
G2259-32-01.GPJ

Figure B-1

GEOCON

PROJECT NO. G2259-32-01

SAMPLE NO. B1-2



ASTM D2435

Initial Dry Density (pcf)	110.7
Initial Water Content (%)	17.8

Initial Saturation (%)	95.0
Sample Saturated at (ksf)	2.0

CONSOLIDATION CURVE

1118 N. ANZA STREET TOWNHOMES

EL CAJON, CALIFORNIA

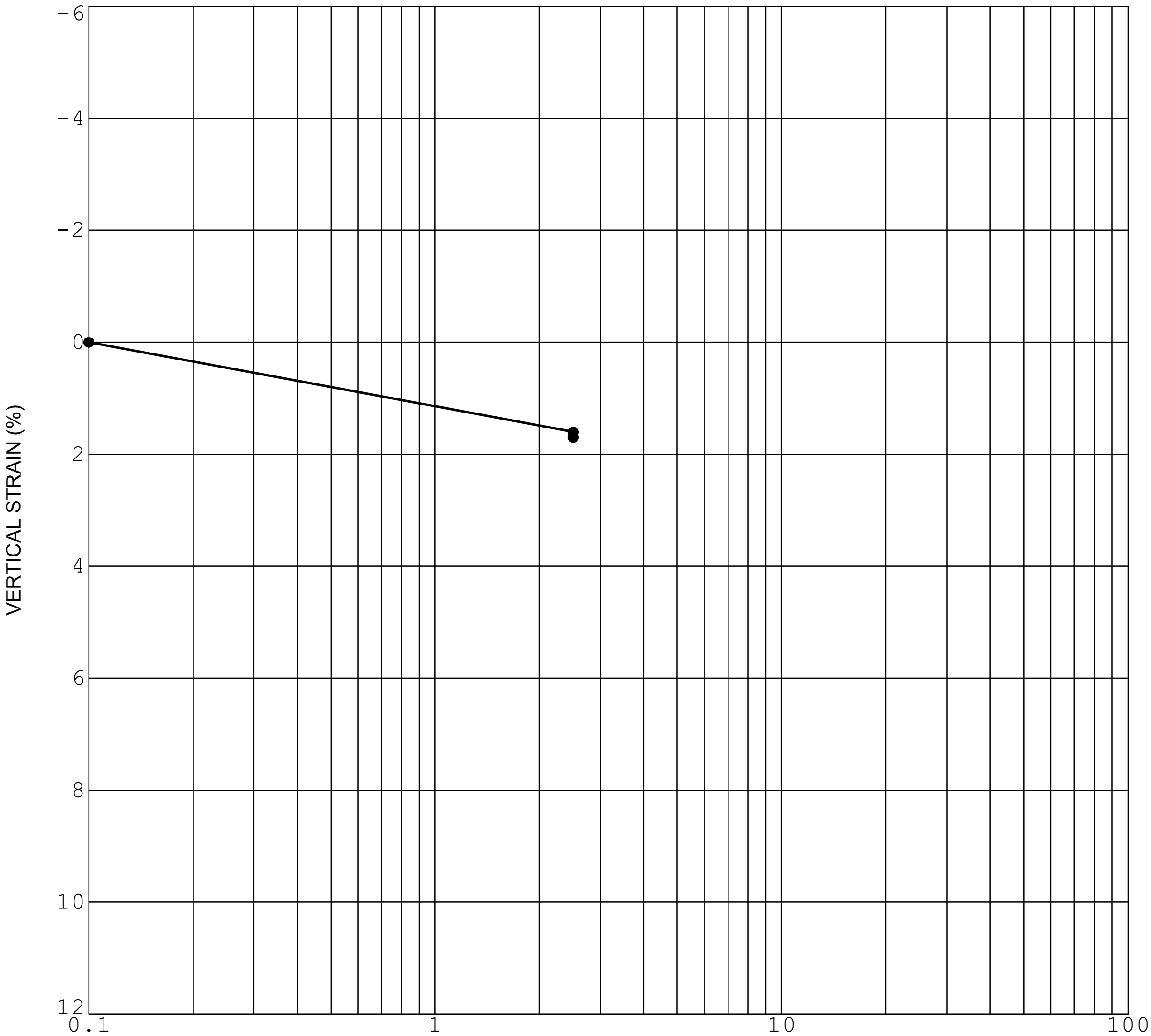
G2259-32-01.GPJ

Figure B-2

GEOCON

PROJECT NO. G2259-32-01

SAMPLE NO. B1-3



ASTM D2435

Initial Dry Density (pcf)	114.2
Initial Water Content (%)	16.2

Initial Saturation (%)	94.8
Sample Saturated at (ksf)	2.5

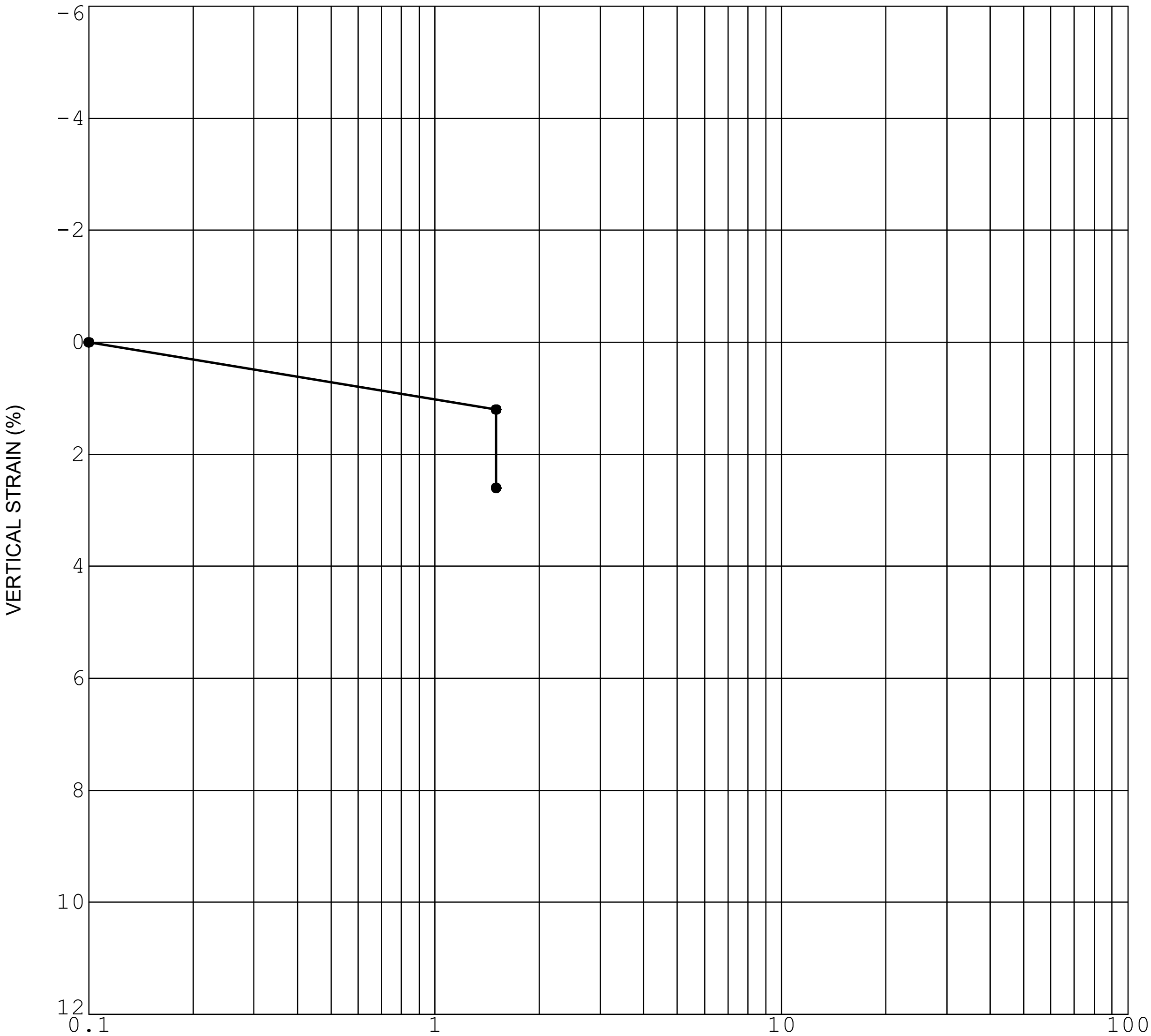
CONSOLIDATION CURVE

1118 N. ANZA STREET TOWNHOMES

EL CAJON, CALIFORNIA

PROJECT NO. G2259-32-01

SAMPLE NO. B2-1



ASTM D2435

Initial Dry Density (pcf)	117.7	**Initial Saturation (%)**	51.9
Initial Water Content (%)	8.0	**Sample Saturated at (ksf)**	1.5

CONSOLIDATION CURVE

1118 N. ANZA STREET TOWNHOMES

EL CAJON, CALIFORNIA

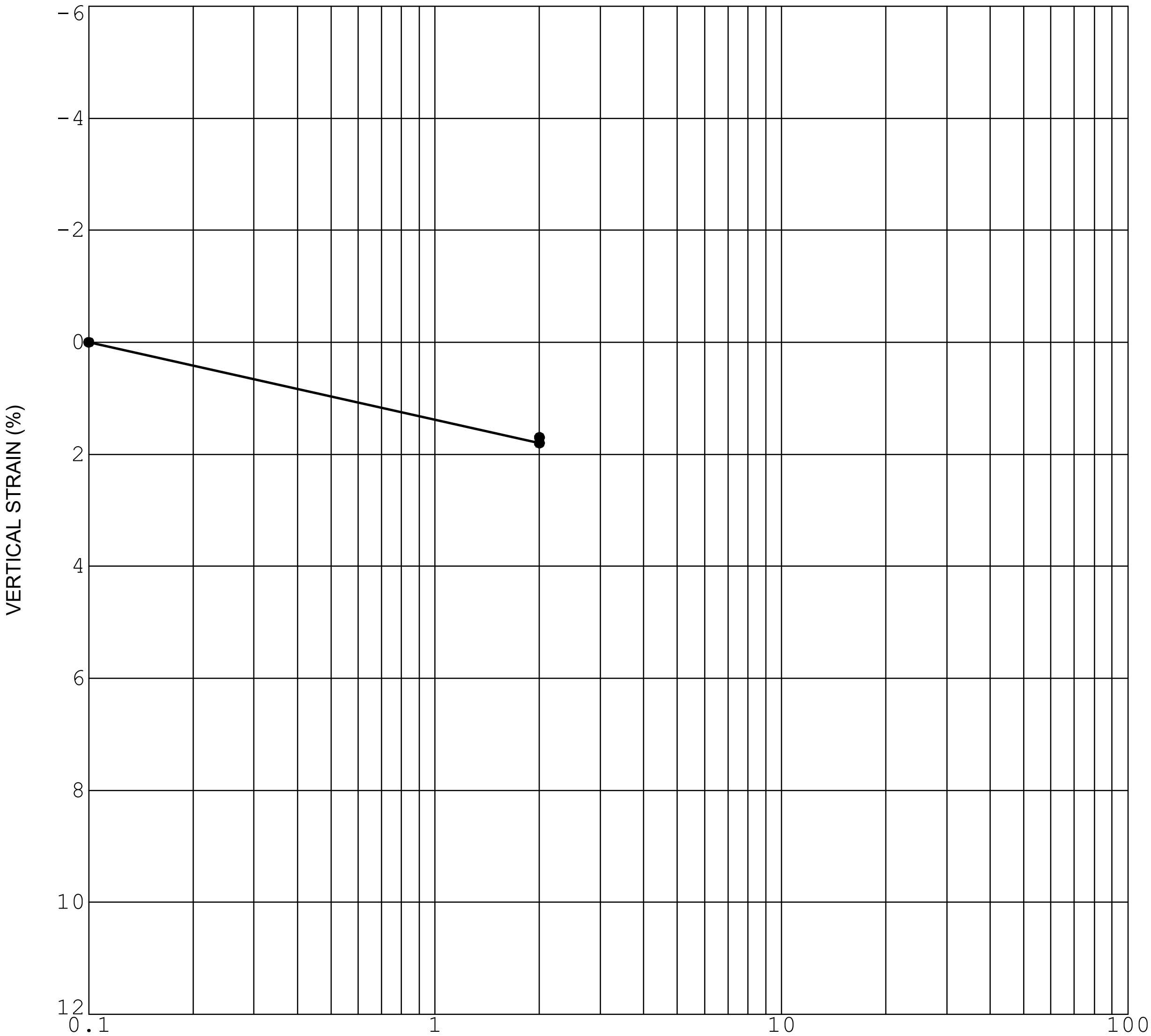
G2259-32-01.GPJ

Figure B-4

GEOCON

PROJECT NO. G2259-32-01

SAMPLE NO. B2-2



ASTM D2435

Initial Dry Density (pcf)	111.5
Initial Water Content (%)	18.6

Initial Saturation (%)	100+
Sample Saturated at (ksf)	2.0

CONSOLIDATION CURVE

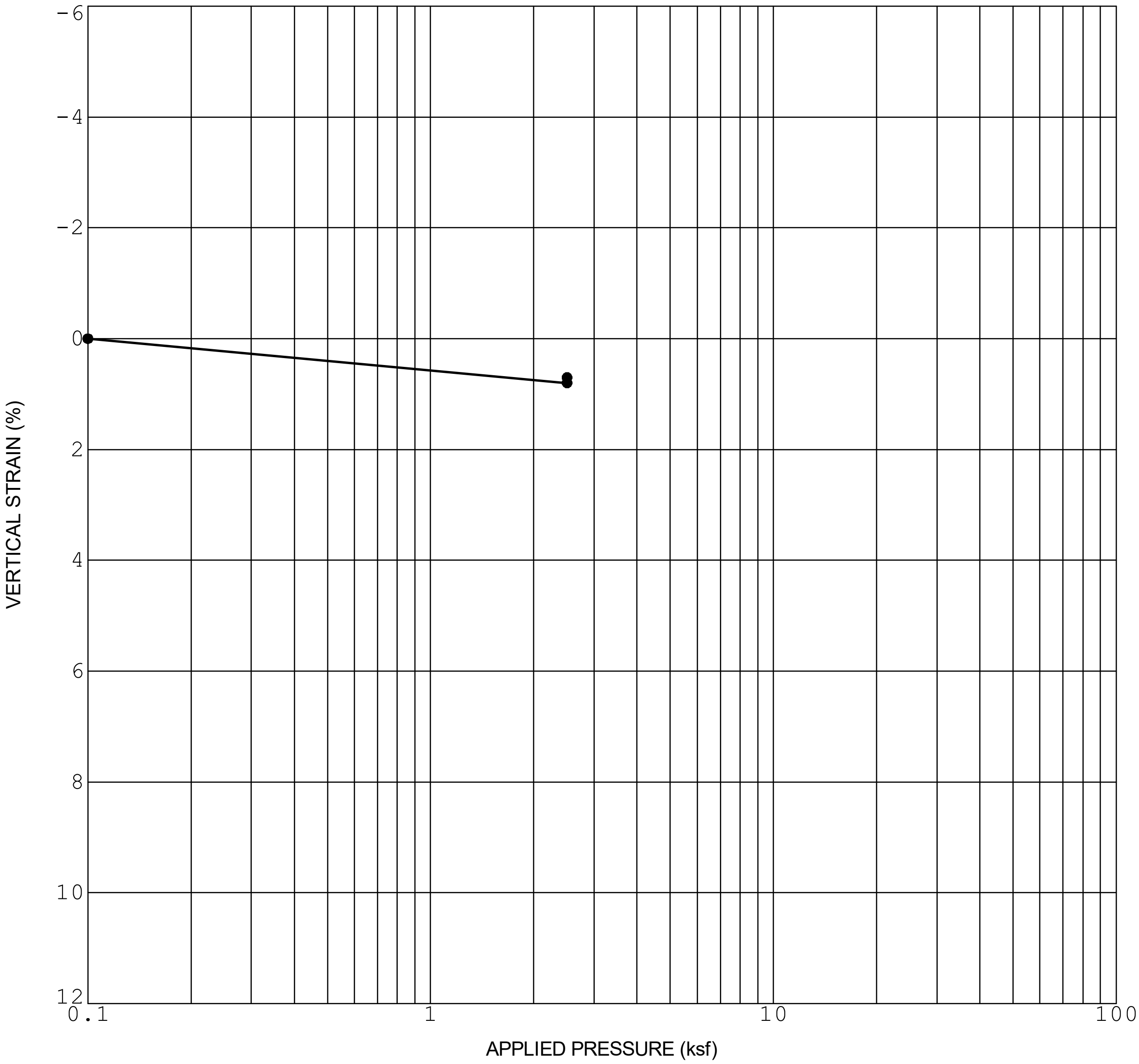
1118 N. ANZA STREET TOWNHOMES

EL CAJON, CALIFORNIA

GEOCON

PROJECT NO. G2259-32-01

SAMPLE NO. B2-3



ASTM D2435

Initial Dry Density (pcf)	121.4	**Initial Saturation (%)**	100+
Initial Water Content (%)	13.9	**Sample Saturated at (ksf)**	2.5

CONSOLIDATION CURVE

1118 N. ANZA STREET TOWNHOMES

EL CAJON, CALIFORNIA

G2259-32-01.GPJ

Figure B-6

GEOCON

APPENDIX C

APPENDIX C

STORM WATER MANAGEMENT INVESTIGATION

FOR

1118 NORTH ANZA STREET TOWNHOMES
EL CAJON, CALIFORNIA

PROJECT NO. G2259-32-01

APPENDIX C

STORM WATER MANAGEMENT INVESTIGATION

We understand storm water management devices are being proposed in accordance with the *2016 City of El Cajon BMP Design Manual,* commonly referred to as the *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 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. In addition, the USDA website also provides an estimated saturated hydraulic conductivity for the existing soil.

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 two units identified as Placentia sandy loam (PfC) and Vista coarse sandy loam (VsC). The Placentia sandy loam is shown extending across the vast majority of the property and is classified as Soil Group D. The Vista coarse sandy loam is identified as Soil Group B. 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	k_{SAT} of Most Limiting Layer (inches/hour)
Placentia sandy loam	PfC	98	D	0.00-0.06
Vista coarse sandy loam	VsC	2	B	1.98-5.95

In-Situ Testing

The infiltration rate, percolation rates and saturated hydraulic conductivity are different and have different meanings. Percolation rates tend to overestimate infiltration rates and saturated hydraulic conductivities by a factor of 10 or more. Table C-3 describes the differences in the definitions.

TABLE C-3
SOIL PERMEABILITY DEFINITIONS

Term	Definition
Infiltration Rate	The observation of the flow of water through a material into the ground downward into a given soil structure under long term conditions. This is a function of layering of soil, density, pore space, discontinuities and initial moisture content.
Percolation Rate	The observation of the flow of water through a material into the ground downward and laterally into a given soil structure under long term conditions. This is a function of layering of soil, density, pore space, discontinuities and initial moisture content.
Saturated Hydraulic Conductivity (k_{SAT}, Permeability)	The volume of water that will move in a porous medium under a hydraulic gradient through a unit area. This is a function of density, structure, stratification, fines content and discontinuities. It is also a function of the properties of the liquid as well as of the porous medium.

The degree of soil compaction or in-situ density has a significant impact on soil permeability and infiltration. Based on our experience and other studies we performed, an increase in compaction results in a decrease in soil permeability.

We performed three, constant head, Aardvark Permeameter Tests, P-1 through P-3, at locations shown on the attached Geologic Map, Figure 2. The test borings were 4 inches in diameter. The results of the tests provide parameters for the saturated hydraulic conductivity characteristics of on-site soil and geologic units. Table C-4 presents the results of the estimated field saturated hydraulic conductivity and estimated infiltration rates obtained from the Aardvark Permeameter tests. The field sheets are also attached herein. We applied a feasibility factor of safety of 2 to the field results for use in preparation of Worksheet C.4-1. The results of the testing indicate adjusted soil infiltration rates of 0.008, 0.019, and 0.115 inches per hour after applying a Factor of Safety of 2. Based on a discussion in the County of Riverside *Design Handbook for Low Impact Development Best Management Practices*, the infiltration rate should be considered equal to the saturated hydraulic conductivity rate.

TABLE C-4
FIELD PERMEAMETER INFILTRATION TEST RESULTS

Test No.	Geologic Unit	Test Depth (feet)	Field-Saturated Hydraulic Conductivity, k_{sat} (inch/hour)	Worksheet[1] Saturated Hydraulic Conductivity, k_{sat} (inch/hour)
P-1	Qcol	4.0	0.016	0.008
P-2	Qcol	3.2	0.038	0.019
P-3	Qcol	3.3	0.229	0.115

[1] Using a factor of safety of 2 for Worksheet C.4-1.

STORM WATER MANAGEMENT CONCLUSIONS

The Geologic Map, Figure 2, depicts the existing property, proposed development, the approximate lateral limits of the geologic units, the locations of the field excavations and the in-situ infiltration test locations.

Soil Types

Proposed Compacted Fill – Compacted fill will be placed across the entire property during site development. Proposed remedial grading will consist of removing the upper 3 feet of soil and replacement as compacted fill. The proposed storm water BMP’s will be founded in compacted fill placed above granitic rock. The compacted fill will be comprised of on-site silty/clay sand and sandy clay. The fill will be compacted to a dry density of at least 90 percent of the laboratory maximum dry density. In our experience, compacted fill does not possess infiltration rates appropriate for infiltration BMP’s. Hazards that occur as a result of fill soil saturation include a potential for hydro-consolidation of the granular fill soils, long term fill settlement, differential fill settlement, and lateral movement associated with saturated fill relaxation. The potential for lateral water migration to adversely impact existing or proposed structures, foundations, utilities, and roadways, is high. Therefore, full infiltration should be considered infeasible.

Section D.4.2 of the *2016 Storm Water Standards* (SWS) provides a discussion regarding fill materials used for infiltration. The SWS states:

- *For engineered fills, infiltration rates may still be quite uncertain due to layering and heterogeneities introduced as part of construction that cannot be precisely controlled.* Due to these uncertainties, full and partial infiltration should be considered geotechnically infeasible and liners and subdrains should be used in areas where infiltration BMP’s are founded in compacted fill.

- *Where possible, infiltration BMPs on fill material should be designed such that their infiltrating surface extends into native soils.* The underlying granitic rock below the compacted fill is expected between 5 to 12 feet below existing grade, or 15 to 22 feet below finish grade after remedial grading is performed and site raised approximately 10 feet. Full and partial infiltration should be considered geotechnically infeasible within the compacted fill and liners and subdrains should be used. If the infiltration BMP’s extended below the compacted fill, partial infiltration may be feasible if the infiltration BMP extends below the compacted fill.

- *Because of the uncertainty of fill parameters as well as potential compaction of the native soils, an infiltration BMP may not be feasible.* Therefore, full infiltration should be considered geotechnically infeasible. Partial infiltration may be feasible if the infiltration BMP extends below the compacted fill.

Infiltration Rates

The results of the three infiltration rates (including the feasibility factor of safety of 2) obtained within the colluvium were 0.008, 0.019, and 0.115 inches per hour (iph). Based on the results of the infiltration testing, none of the tests meet the minimum threshold for full infiltration; therefore, full infiltration is considered infeasible.

Groundwater Elevations

Groundwater was not encountered during the field investigation. Minor to moderate seepage was observed along the granitic rock contact during the field exploration. Groundwater is not anticipated to significantly impact project development as presently proposed. Proper surface drainage of irrigation and rainwater will be important to future performance of the project.

Soil or Groundwater Contamination

Based on our review of the Geotracker website, no soil or groundwater contamination is suspected.

New or Existing Utilities

Existing utilities are present within right of ways adjacent to the existing streets, generally beneath public sidewalks and roadways. We expect that all on-site utilities will be removed prior to site development. Full infiltration near existing or proposed utilities should be avoided to prevent lateral water migration into the permeable trench backfill materials.

Existing and Planned Structures

Residential developments surround the property. Existing structures are situated in close proximity to the north, west, and southern property boundaries. Anza Street is located immediately to the east. If water is allowed to infiltrate into the soil, the water could migrate laterally and into other properties in the vicinity of the subject site. The water migration may negatively affect other buildings and improvements in the area.

Slopes

The site is relatively flat and significant slopes do not exist adjacent to the site. Infiltration BMP's situated in close proximity to descending fill slopes are not recommended due to the potential for daylight water seepage and lateral water migration.

Recommendations

Due to the relatively low infiltration rates obtained, and close proximity to public and private improvements, foundations, and roadways, full infiltration of storm water is considered geotechnically infeasible. Partial infiltration of storm water may be feasible if the BMP is directed below any compacted fill. Otherwise, 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. Seams and penetrations of the liners 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. If designing any storm water infiltration BMP's for partial infiltration, side liners and a subdrain are recommended.

Storm Water Standard Worksheets

The SWS requests the geotechnical engineer complete the *Categorization of Infiltration Feasibility Condition* (Worksheet C.4-1 or I-8) worksheet information to help evaluate the potential for infiltration on the property. The attached Form 1-8 presents the completed information for the submittal process.

The regional storm water standards also have a worksheet (Worksheet D.5-1 or Form I-9) that helps the project civil engineer estimate the factor of safety based on several factors. Table C-5 describes the suitability assessment input parameters related to the geotechnical engineering aspects for the factor of safety determination.

TABLE C-5
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.
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 information in Table C-5, Table C-6 presents the estimated factor values for the evaluation of the factor of safety. This table only provides 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-6
FACTOR OF SAFETY WORKSHEET DESIGN VALUES – PART A[1]

Suitability Assessment Factor Category	Assigned Weight (w)	Factor Value (v)	Product (p = w x v)
Assessment Methods	0.25	3	0.75
Predominant Soil Texture	0.25	3	0.75
Site Soil Variability	0.25	2	0.50
Depth to Groundwater/ Impervious Layer	0.25	1	0.25
Suitability Assessment Safety Factor, $S_A = \sum p$			2.25

[1] The project civil engineer should complete Worksheet D.5-1 or Form I-9 using the data on this table. Additional information is required to evaluate the design factor of safety.

Categorization of Infiltration Feasibility Condition	Worksheet C.4-1

Part 1 - Full Infiltration Feasibility Screening Criteria

Would infiltration of the full design volume be feasible from a physical perspective without any undesirable consequences that cannot be reasonably mitigated?

Criteria	Screening Question	Yes	No
1	**Is the estimated reliable infiltration rate below proposed facility locations greater than 0.5 inches per hour?** The response to this Screening Question shall be based on a comprehensive evaluation of the factors presented in Appendix C.2 and Appendix D.		X

Provide basis: Based on results of permeability testing in 3 locations across the site, the unfactored infiltration rate was measured to be 0.016, 0.038, and 0.229 inches/hour using a constant head borehole permeameter. If applying a feasibility factor of safety of 2.0, the infiltration rates would be 0.008, 0.019 and 0.115 iph, which are less than the required threshold value of 0.5 iph. The USDA web soil survey website indicates the vast majority of the underlying soils belong to Placentia sandy loam (PfC) which is identified as Hydrologic Soil Group D, which is not conducive to infiltration BMP's. Information collected from the USDA website is attached. The Aardvark Permeameter test results are attached. In accordance with the Riverside County storm water procedures, which reference the United States Bureau of Reclamation Well Permeameter Method (USBR 7300), the saturated hydraulic conductivity is equal to the unfactored infiltration rate.

Criteria	Screening Question	Yes	No
2	**Can infiltration greater than 0.5 inches per hour be allowed without increasing risk of geotechnical hazards (slope stability, groundwater mounding, utilities, or other factors) that cannot be mitigated to an acceptable level?** The response to this Screening Question shall be based on a comprehensive evaluation of the factors presented in Appendix C.2.		X

Provide basis:

No slopes greater than 25% are proposed in the vicinity of the proposed basins, a liquefaction potential is very low to negligible, and the landslide potential is very low to negligible. However, the potential for lateral water migration to adversely impact existing and proposed utilities, adversely impact proposed foundations and improvements is high. Compacted fill will be placed across the property and result in fills of approximately 10 to 13 feet thick. Infiltration BMP's founded in compacted fill should be avoided to prevent adverse shrinking/swelling of the expansive soils, and adverse hydro-consolidation of the granular fill soils which causes differential settlement.

Worksheet C.4-1 Page 2 of 4			
Criteria	Screening Question	**Yes**	**No**
3	**Can infiltration greater than 0.5 inches per hour be allowed without increasing risk of groundwater contamination (shallow water table, storm water pollutants or other factors) that cannot be mitigated to an acceptable level?** The response to this Screening Question shall be based on a comprehensive evaluation of the factors presented in Appendix C.3.	X	
Provide basis: Groundwater is not located within 10 feet from any proposed infiltration BMP, therefore the risk of storm water infiltration BMP's adversely impacting groundwater is considered negligible.			
4	**Can infiltration greater than 0.5 inches per hour be allowed without causing potential water balance issues such as change of seasonality of ephemeral streams or increased discharge of contaminated groundwater to surface waters?** The response to this Screening Question shall be based on a comprehensive evaluation of the factors presented in Appendix C.3.	X	
Provide basis: It is our opinion there are no adverse impacts to groundwater, water balance impacts to stream flow, or impacts on any downstream water rights. It should be noted that researching downstream water rights or evaluating water balance issues to stream flows is beyond the scope of the geotechnical consultant.			
Part 1 Result*	If all answers to rows 1 - 4 are "**Yes**" a full infiltration design is potentially feasible. The feasibility screening category is **Full Infiltration** If any answer from row 1-4 is "**No**", infiltration may be possible to some extent but would not generally be feasible or desirable to achieve a "full infiltration" design. Proceed to Part 2		No Full Infiltration

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

Worksheet C.4-1 Page 3 of 4

Part 2 – Partial Infiltration vs. No Infiltration Feasibility Screening Criteria

Would infiltration of water in any appreciable amount be physically feasible without any negative consequences that cannot be reasonably mitigated?

Criteria	Screening Question	Yes	No
5	**Do soil and geologic conditions allow for infiltration in any appreciable rate or volume?** The response to this Screening Question shall be based on a comprehensive evaluation of the factors presented in Appendix C.2 and Appendix D.		X

Provide basis: Based on results of permeability testing in 3 locations across the site, the unfactored infiltration rate was measured to be 0.016, 0.038, and 0.229 inches/hour using a constant head borehole permeameter. If applying a feasibility factor of safety of 2.0, the infiltration rates would be 0.008, 0.019 and 0.115 iph, therefore, 2 of the 3 tests indicated rates below 0.05 iph, which should be considered the low bound threshold for partial infiltration (based on 2016 City of San Diego Storm Water Manual). After grading, the actual BMP's would be situated in compacted fill over granitic rock, and infiltration BMP's founded in compacted fill are not recommended (see discussion in Appendix C of the Geotechnical Investigation).

Criteria	Screening Question	Yes	No
6	**Can Infiltration in any appreciable quantity be allowed without increasing risk of geotechnical hazards (slope stability, groundwater mounding, utilities, or other factors) that cannot be mitigated to an acceptable level?** The response to this Screening Question shall be based on a comprehensive evaluation of the factors presented in Appendix C.2.		X

Provide basis: No slopes greater than 25% are proposed in the vicinity of the proposed basins, a liquefaction potential is very low to negligible, and the landslide potential is very low to negligible. However, the potential for lateral water migration to adversely impact existing and proposed utilities, adversely impact proposed foundations and improvements is high. Compacted fill will be placed across the property and result in fills of approximately 10 to 13 feet thick. Infiltration BMP's founded in compacted fill should be avoided to prevent adverse shrinking/swelling of the expansive soils, and adverse hydro-consolidation of the granular fill soils which causes differential settlement.

<table>
<tr><th colspan="4">Worksheet C.4-1 Page 4 of 4</th></tr>
<tr><td>Criteria</td><td>Screening Question</td><td>Yes</td><td>No</td></tr>
<tr><td>7</td><td>Can Infiltration in any appreciable quantity be allowed without posing significant risk for groundwater related concerns (shallow water table, storm water pollutants or other factors)? The response to this Screening Question shall be based on a comprehensive evaluation of the factors presented in Appendix C.3.</td><td>X</td><td></td></tr>
<tr><td colspan="4">Provide basis:
Groundwater is not located within 10 feet from any proposed infiltration BMP, therefore the risk of storm water infiltration BMP’s adversely impacting groundwater is considered negligible.</td></tr>
<tr><td>8</td><td>Can infiltration be allowed without violating downstream water rights? The response to this Screening Question shall be based on a comprehensive evaluation of the factors presented in Appendix C.3.</td><td>X</td><td></td></tr>
<tr><td colspan="4">Provide basis: It is our opinion there are no adverse impacts to groundwater, water balance impacts to stream flow, or impacts on any downstream water rights. It should be noted that researching downstream water rights or evaluating water balance issues to stream flows is beyond the scope of the geotechnical consultant.</td></tr>
<tr><td>Part 2 Result*</td><td colspan="2">If all answers from row 1-4 are yes then partial infiltration design is potentially feasible. The feasibility screening category is Partial Infiltration.

If any answer from row 5-8 is no, then infiltration of any volume is considered to be infeasible within the drainage area. The feasibility screening category is No Infiltration.</td><td>No Infiltration</td></tr>
</table>

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

GEOCON

Aardvark Permeameter Data Analysis

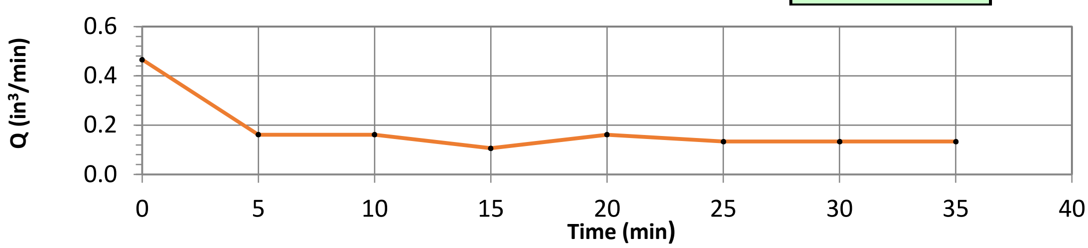
Project Name:	1118 North Anza Street	Date:	3/22/2018
Project Number:	G2259-32-01	By:	DEG
Test Number:	P-1		

Borehole Diameter, d (in.):	4.00
Borehole Depth, **H** (in):	48.00
Distance Between Reservoir & Top of Borehole (in.)	25.00
Estimated Depth to Water Table, **S** (feet):	20.00
Height APM Raised from Bottom (in.):	2.00
Pressure Reducer Used:	No

Ref. EL (feet, MSL):	0.0
Bottom EL (feet, MSL):	-4.0

Distance Between Reservoir and APM Float, **D** (in.):	63.75
Head Height Calculated, **h** (in.):	5.71
Head Height Measured, **h** (in.):	6.00
Distance Between Constant Head and Water Table, **L** (in.):	198.00

Reading	Time Elapsed (min)	Water Weight Consumed (lbs)	Water Volume Consumed (in^3)	Q (in^3/min)
1	0.00	0.000	0.00	0.00
2	5.00	0.085	2.35	0.471
3	5.00	0.030	0.83	0.166
4	5.00	0.030	0.83	0.166
5	5.00	0.020	0.55	0.111
6	5.00	0.030	0.83	0.166
7	5.00	0.025	0.69	0.138
8	5.00	0.025	0.69	0.138
9	5.00	0.025	0.69	0.138
			Steady Flow Rate, Q (in^3/min):	0.138



Soil Matric Flux Potential, ϕ_m

ϕ_m= 0.00269 in^2/min

Field-Saturated Hydraulic Conductivity (Infiltration Rate)

K_{sat} = 2.75E-04 in/min 0.016 in/hr

GEOCON

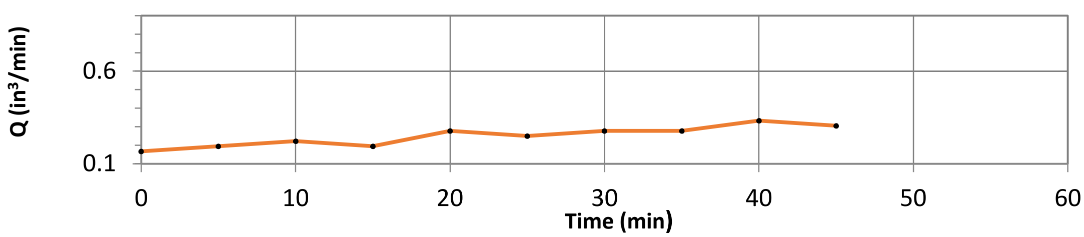
Aardvark Permeameter Data Analysis

Project Name:	1118 North Anza Street	Date:	3/22/2018
Project Number:	G2259-32-01	By:	DEG
Test Number:	P-2	Ref. EL (feet, MSL):	0.0
		Bottom EL (feet, MSL):	-3.3

Borehole Diameter, d (in.):	4.00
Borehole Depth, **H** (in):	39.00
Distance Between Reservoir & Top of Borehole (in.)	27.00
Estimated Depth to Water Table, **S** (feet):	20.00
Height APM Raised from Bottom (in.):	2.00
Pressure Reducer Used:	No

Distance Between Reservoir and APM Float, **D** (in.):	56.75
Head Height Calculated, **h** (in.):	5.69
Head Height Measured, **h** (in.):	6.00
Distance Between Constant Head and Water Table, **L** (in.):	207.00

Reading	Time Elapsed (min)	Water Weight Consumed (lbs)	Water Volume Consumed (in^3)	Q (in^3/min)
1	0.00	0.000	0.00	0.00
2	5.00	0.030	0.83	0.166
3	5.00	0.035	0.97	0.194
4	5.00	0.040	1.11	0.222
5	5.00	0.035	0.97	0.194
6	5.00	0.050	1.38	0.277
7	5.00	0.045	1.25	0.249
8	5.00	0.050	1.38	0.277
9	5.00	0.050	1.38	0.277
10	5.00	0.060	1.66	0.332
11	5.00	0.055	1.52	0.305
			Steady Flow Rate, Q (in^3/min):	0.318



Soil Matric Flux Potential, Φ_m

Φ_m= 0.0062 in^2/min

Field-Saturated Hydraulic Conductivity (Infiltration Rate)

K_{sat} = 6.31E-04 in/min 0.038 in/hr

GEOCON

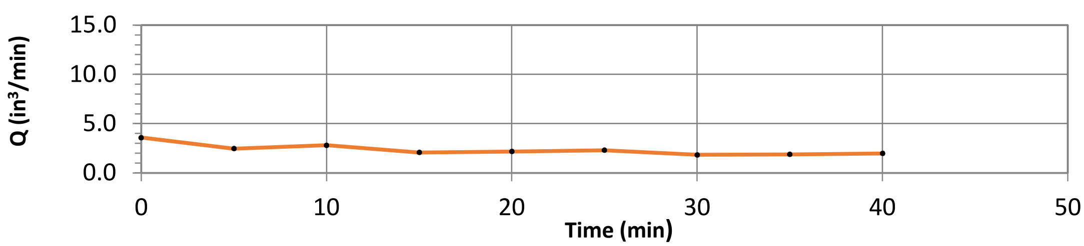
Aardvark Permeameter Data Analysis

Project Name:	1118 North Anza Street	Date:	3/22/2018
Project Number:	G2259-32-01	By:	DEG
Test Number:	P-3	Ref. EL (feet, MSL):	0.0
		Bottom EL (feet, MSL):	-2.9

Borehole Diameter, d (in.):	4.00
Borehole Depth, **H** (in):	35.00
Distance Between Reservoir & Top of Borehole (in.)	27.00
Estimated Depth to Water Table, **S** (feet):	20.00
Height APM Raised from Bottom (in.):	2.00
Pressure Reducer Used:	No

Distance Between Reservoir and APM Float, **D** (in.):	52.75
Head Height Calculated, **h** (in.):	5.68
Head Height Measured, **h** (in.):	6.00
Distance Between Constant Head and Water Table, **L** (in.):	211.00

Reading	Time Elapsed (min)	Water Weight Consumed (lbs)	Water Volume Consumed (in^3)	Q (in^3/min)
1	0.00	0.000	0.00	0.00
2	5.00	0.645	17.86	3.572
3	5.00	0.445	12.32	2.465
4	5.00	0.505	13.98	2.797
5	5.00	0.375	10.38	2.077
6	5.00	0.390	10.80	2.160
7	5.00	0.415	11.49	2.298
8	5.00	0.330	9.14	1.828
9	5.00	0.340	9.42	1.883
10	5.00	0.355	9.83	1.966
			Steady Flow Rate, Q (in^3/min):	1.925



Soil Matric Flux Potential, Φ_m

Φ_m= 0.0374 in^2/min

Field-Saturated Hydraulic Conductivity (Infiltration Rate)

K_{sat} = 3.82E-03 in/min 0.229 in/hr

United States Department of Agriculture

NRCS

Natural Resources Conservation Service

A product of the National Cooperative Soil Survey, a joint effort of the United States Department of Agriculture and other Federal agencies, State agencies including the Agricultural Experiment Stations, and local participants

Custom Soil Resource Report for

San Diego County Area, California

April 9, 2018

Preface

Soil surveys contain information that affects land use planning in survey areas. They highlight soil limitations that affect various land uses and provide information about the properties of the soils in the survey areas. Soil surveys are designed for many different users, including farmers, ranchers, foresters, agronomists, urban planners, community officials, engineers, developers, builders, and home buyers. Also, conservationists, teachers, students, and specialists in recreation, waste disposal, and pollution control can use the surveys to help them understand, protect, or enhance the environment.

Various land use regulations of Federal, State, and local governments may impose special restrictions on land use or land treatment. Soil surveys identify soil properties that are used in making various land use or land treatment decisions. The information is intended to help the land users identify and reduce the effects of soil limitations on various land uses. The landowner or user is responsible for identifying and complying with existing laws and regulations.

Although soil survey information can be used for general farm, local, and wider area planning, onsite investigation is needed to supplement this information in some cases. Examples include soil quality assessments (http://www.nrcs.usda.gov/wps/portal/nrcs/main/soils/health/) and certain conservation and engineering applications. For more detailed information, contact your local USDA Service Center (https://offices.sc.egov.usda.gov/locator/app?agency=nrcs) or your NRCS State Soil Scientist (http://www.nrcs.usda.gov/wps/portal/nrcs/detail/soils/contactus/?cid=nrcs142p2_053951).

Great differences in soil properties can occur within short distances. Some soils are seasonally wet or subject to flooding. Some are too unstable to be used as a foundation for buildings or roads. Clayey or wet soils are poorly suited to use as septic tank absorption fields. A high water table makes a soil poorly suited to basements or underground installations.

The National Cooperative Soil Survey is a joint effort of the United States Department of Agriculture and other Federal agencies, State agencies including the Agricultural Experiment Stations, and local agencies. The Natural Resources Conservation Service (NRCS) has leadership for the Federal part of the National Cooperative Soil Survey.

Information about soils is updated periodically. Updated information is available through the NRCS Web Soil Survey, the site for official soil survey information.

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Contents

How Soil Surveys Are Made

Soil surveys are made to provide information about the soils and miscellaneous areas in a specific area. They include a description of the soils and miscellaneous areas and their location on the landscape and tables that show soil properties and limitations affecting various uses. Soil scientists observed the steepness, length, and shape of the slopes; the general pattern of drainage; the kinds of crops and native plants; and the kinds of bedrock. They observed and described many soil profiles. A soil profile is the sequence of natural layers, or horizons, in a soil. The profile extends from the surface down into the unconsolidated material in which the soil formed or from the surface down to bedrock. The unconsolidated material is devoid of roots and other living organisms and has not been changed by other biological activity.

Currently, soils are mapped according to the boundaries of major land resource areas (MLRAs). MLRAs are geographically associated land resource units that share common characteristics related to physiography, geology, climate, water resources, soils, biological resources, and land uses (USDA, 2006). Soil survey areas typically consist of parts of one or more MLRA.

The soils and miscellaneous areas in a survey area occur in an orderly pattern that is related to the geology, landforms, relief, climate, and natural vegetation of the area. Each kind of soil and miscellaneous area is associated with a particular kind of landform or with a segment of the landform. By observing the soils and miscellaneous areas in the survey area and relating their position to specific segments of the landform, a soil scientist develops a concept, or model, of how they were formed. Thus, during mapping, this model enables the soil scientist to predict with a considerable degree of accuracy the kind of soil or miscellaneous area at a specific location on the landscape.

Commonly, individual soils on the landscape merge into one another as their characteristics gradually change. To construct an accurate soil map, however, soil scientists must determine the boundaries between the soils. They can observe only a limited number of soil profiles. Nevertheless, these observations, supplemented by an understanding of the soil-vegetation-landscape relationship, are sufficient to verify predictions of the kinds of soil in an area and to determine the boundaries.

Soil scientists recorded the characteristics of the soil profiles that they studied. They noted soil color, texture, size and shape of soil aggregates, kind and amount of rock fragments, distribution of plant roots, reaction, and other features that enable them to identify soils. After describing the soils in the survey area and determining their properties, the soil scientists assigned the soils to taxonomic classes (units). Taxonomic classes are concepts. Each taxonomic class has a set of soil characteristics with precisely defined limits. The classes are used as a basis for comparison to classify soils systematically. Soil taxonomy, the system of taxonomic classification used in the United States, is based mainly on the kind and character of soil properties and the arrangement of horizons within the profile. After the soil scientists classified and named the soils in the survey area, they compared the individual soils with similar soils in the same taxonomic class in other areas so that they could confirm data and assemble additional data based on experience and research.

The objective of soil mapping is not to delineate pure map unit components; the objective is to separate the landscape into landforms or landform segments that have similar use and management requirements. Each map unit is defined by a unique combination of soil components and/or miscellaneous areas in predictable proportions. Some components may be highly contrasting to the other components of the map unit. The presence of minor components in a map unit in no way diminishes the usefulness or accuracy of the data. The delineation of such landforms and landform segments on the map provides sufficient information for the development of resource plans. If intensive use of small areas is planned, onsite investigation is needed to define and locate the soils and miscellaneous areas.

Soil scientists make many field observations in the process of producing a soil map. The frequency of observation is dependent upon several factors, including scale of mapping, intensity of mapping, design of map units, complexity of the landscape, and experience of the soil scientist. Observations are made to test and refine the soil-landscape model and predictions and to verify the classification of the soils at specific locations. Once the soil-landscape model is refined, a significantly smaller number of measurements of individual soil properties are made and recorded. These measurements may include field measurements, such as those for color, depth to bedrock, and texture, and laboratory measurements, such as those for content of sand, silt, clay, salt, and other components. Properties of each soil typically vary from one point to another across the landscape.

Observations for map unit components are aggregated to develop ranges of characteristics for the components. The aggregated values are presented. Direct measurements do not exist for every property presented for every map unit component. Values for some properties are estimated from combinations of other properties.

While a soil survey is in progress, samples of some of the soils in the area generally are collected for laboratory analyses and for engineering tests. Soil scientists interpret the data from these analyses and tests as well as the field-observed characteristics and the soil properties to determine the expected behavior of the soils under different uses. Interpretations for all of the soils are field tested through observation of the soils in different uses and under different levels of management. Some interpretations are modified to fit local conditions, and some new interpretations are developed to meet local needs. Data are assembled from other sources, such as research information, production records, and field experience of specialists. For example, data on crop yields under defined levels of management are assembled from farm records and from field or plot experiments on the same kinds of soil.

Predictions about soil behavior are based not only on soil properties but also on such variables as climate and biological activity. Soil conditions are predictable over long periods of time, but they are not predictable from year to year. For example, soil scientists can predict with a fairly high degree of accuracy that a given soil will have a high water table within certain depths in most years, but they cannot predict that a high water table will always be at a specific level in the soil on a specific date.

After soil scientists located and identified the significant natural bodies of soil in the survey area, they drew the boundaries of these bodies on aerial photographs and identified each as a specific map unit. Aerial photographs show trees, buildings, fields, roads, and rivers, all of which help in locating boundaries accurately.

Soil Map

The soil map section includes the soil map for the defined area of interest, a list of soil map units on the map and extent of each map unit, and cartographic symbols displayed on the map. Also presented are various metadata about data used to produce the map, and a description of each soil map unit.

Soil Map

Map Scale: 1:829 if printed on A portrait (8.5" x 11") sheet.
Map projection: Web Mercator Corner coordinates: WGS84 Edge tics: UTM Zone 11N WGS84

MAP LEGEND

Area of Interest (AOI)

- Area of Interest (AOI)

Soils

- Soil Map Unit Polygons
- Soil Map Unit Lines
- Soil Map Unit Points

Special Point Features

- Blowout
- Borrow Pit
- Clay Spot
- Closed Depression
- Gravel Pit
- Gravelly Spot
- Landfill
- Lava Flow
- Marsh or swamp
- Mine or Quarry
- Miscellaneous Water
- Perennial Water
- Rock Outcrop
- Saline Spot
- Sandy Spot
- Severely Eroded Spot
- Sinkhole
- Slide or Slip
- Sodic Spot
- Spoil Area
- Stony Spot
- Very Stony Spot
- Wet Spot
- Other
- Special Line Features

Water Features

- Streams and Canals

Transportation

- Rails
- Interstate Highways
- US Routes
- Major Roads
- Local Roads

Background

- Aerial Photography

MAP INFORMATION

The soil surveys that comprise your AOI were mapped at 1:24,000.

Warning: Soil Map may not be valid at this scale.

Enlargement of maps beyond the scale of mapping can cause misunderstanding of the detail of mapping and accuracy of soil line placement. The maps do not show the small areas of contrasting soils that could have been shown at a more detailed scale.

Please rely on the bar scale on each map sheet for map measurements.

Source of Map: Natural Resources Conservation Service
Web Soil Survey URL:
Coordinate System: Web Mercator (EPSG:3857)

Maps from the Web Soil Survey are based on the Web Mercator projection, which preserves direction and shape but distorts distance and area. A projection that preserves area, such as the Albers equal-area conic projection, should be used if more accurate calculations of distance or area are required.

This product is generated from the USDA-NRCS certified data as of the version date(s) listed below.

Soil Survey Area: San Diego County Area, California
Survey Area Data: Version 12, Sep 13, 2017

Soil map units are labeled (as space allows) for map scales 1:50,000 or larger.

Date(s) aerial images were photographed: Dec 7, 2014—Jan 4, 2015

The orthophoto or other base map on which the soil lines were compiled and digitized probably differs from the background imagery displayed on these maps. As a result, some minor shifting of map unit boundaries may be evident.

Map Unit Legend

Map Unit Symbol	Map Unit Name	Acres in AOI	Percent of AOI
PfC	Placentia sandy loam, thick surface, 2 to 9 percent slo pes	3.3	97.9%
VsC	Vista coarse sandy loam, 5 to 9 percent slopes	0.1	2.1%
Totals for Area of Interest		**3.3**	**100.0%**

Map Unit Descriptions

The map units delineated on the detailed soil maps in a soil survey represent the soils or miscellaneous areas in the survey area. The map unit descriptions, along with the maps, can be used to determine the composition and properties of a unit.

A map unit delineation on a soil map represents an area dominated by one or more major kinds of soil or miscellaneous areas. A map unit is identified and named according to the taxonomic classification of the dominant soils. Within a taxonomic class there are precisely defined limits for the properties of the soils. On the landscape, however, the soils are natural phenomena, and they have the characteristic variability of all natural phenomena. Thus, the range of some observed properties may extend beyond the limits defined for a taxonomic class. Areas of soils of a single taxonomic class rarely, if ever, can be mapped without including areas of other taxonomic classes. Consequently, every map unit is made up of the soils or miscellaneous areas for which it is named and some minor components that belong to taxonomic classes other than those of the major soils.

Most minor soils have properties similar to those of the dominant soil or soils in the map unit, and thus they do not affect use and management. These are called noncontrasting, or similar, components. They may or may not be mentioned in a particular map unit description. Other minor components, however, have properties and behavioral characteristics divergent enough to affect use or to require different management. These are called contrasting, or dissimilar, components. They generally are in small areas and could not be mapped separately because of the scale used. Some small areas of strongly contrasting soils or miscellaneous areas are identified by a special symbol on the maps. If included in the database for a given area, the contrasting minor components are identified in the map unit descriptions along with some characteristics of each. A few areas of minor components may not have been observed, and consequently they are not mentioned in the descriptions, especially where the pattern was so complex that it was impractical to make enough observations to identify all the soils and miscellaneous areas on the landscape.

The presence of minor components in a map unit in no way diminishes the usefulness or accuracy of the data. The objective of mapping is not to delineate pure taxonomic classes but rather to separate the landscape into landforms or landform segments that have similar use and management requirements. The delineation of such segments on the map provides sufficient information for the development of resource plans. If intensive use of small areas is planned, however, onsite investigation is needed to define and locate the soils and miscellaneous areas.

An identifying symbol precedes the map unit name in the map unit descriptions. Each description includes general facts about the unit and gives important soil properties and qualities.

Soils that have profiles that are almost alike make up a *soil series*. Except for differences in texture of the surface layer, all the soils of a series have major horizons that are similar in composition, thickness, and arrangement.

Soils of one series can differ in texture of the surface layer, slope, stoniness, salinity, degree of erosion, and other characteristics that affect their use. On the basis of such differences, a soil series is divided into *soil phases*. Most of the areas shown on the detailed soil maps are phases of soil series. The name of a soil phase commonly indicates a feature that affects use or management. For example, Alpha silt loam, 0 to 2 percent slopes, is a phase of the Alpha series.

Some map units are made up of two or more major soils or miscellaneous areas. These map units are complexes, associations, or undifferentiated groups.

A *complex* consists of two or more soils or miscellaneous areas in such an intricate pattern or in such small areas that they cannot be shown separately on the maps. The pattern and proportion of the soils or miscellaneous areas are somewhat similar in all areas. Alpha-Beta complex, 0 to 6 percent slopes, is an example.

An *association* is made up of two or more geographically associated soils or miscellaneous areas that are shown as one unit on the maps. Because of present or anticipated uses of the map units in the survey area, it was not considered practical or necessary to map the soils or miscellaneous areas separately. The pattern and relative proportion of the soils or miscellaneous areas are somewhat similar. Alpha-Beta association, 0 to 2 percent slopes, is an example.

An *undifferentiated group* is made up of two or more soils or miscellaneous areas that could be mapped individually but are mapped as one unit because similar interpretations can be made for use and management. The pattern and proportion of the soils or miscellaneous areas in a mapped area are not uniform. An area can be made up of only one of the major soils or miscellaneous areas, or it can be made up of all of them. Alpha and Beta soils, 0 to 2 percent slopes, is an example.

Some surveys include *miscellaneous areas*. Such areas have little or no soil material and support little or no vegetation. Rock outcrop is an example.

San Diego County Area, California

PfC—Placentia sandy loam, thick surface, 2 to 9 percent slo pes

Map Unit Setting

National map unit symbol: hbfn
Elevation: 50 to 2,500 feet
Mean annual precipitation: 12 to 18 inches
Mean annual air temperature: 61 to 63 degrees F
Frost-free period: 200 to 300 days
Farmland classification: Farmland of statewide importance

Map Unit Composition

Placentia and similar soils: 85 percent
Minor components: 11 percent
Estimates are based on observations, descriptions, and transects of the mapunit.

Description of Placentia

Setting

Landform: Alluvial fans
Landform position (two-dimensional): Toeslope
Landform position (three-dimensional): Base slope, rise
Down-slope shape: Linear
Across-slope shape: Convex
Parent material: Alluvium derived from granite

Typical profile

H1 - 0 to 13 inches: sandy loam
H2 - 13 to 34 inches: clay, sandy clay
H2 - 13 to 34 inches:

Properties and qualities

Slope: 2 to 9 percent
Depth to restrictive feature: More than 80 inches
Natural drainage class: Moderately well drained
Runoff class: Very high
Capacity of the most limiting layer to transmit water (Ksat): Very low to moderately low (0.00 to 0.06 in/hr)
Depth to water table: More than 80 inches
Frequency of flooding: None
Frequency of ponding: None
Salinity, maximum in profile: Very slightly saline to moderately saline (2.0 to 8.0 mmhos/cm)
Sodium adsorption ratio, maximum in profile: 25.0
Available water storage in profile: Low (about 3.8 inches)

Interpretive groups

Land capability classification (irrigated): 3e
Land capability classification (nonirrigated): 4e
Hydrologic Soil Group: D
Ecological site: CLAYPAN (1975) (R019XD061CA)
Hydric soil rating: No

Minor Components

Bonsall

Percent of map unit: 5 percent
Hydric soil rating: No

Ramona

Percent of map unit: 5 percent
Hydric soil rating: No

Unnamed, ponded

Percent of map unit: 1 percent
Landform: Depressions
Hydric soil rating: Yes

VsC—Vista coarse sandy loam, 5 to 9 percent slopes

Map Unit Setting

National map unit symbol: hbh8
Elevation: 400 to 3,900 feet
Mean annual precipitation: 10 to 18 inches
Mean annual air temperature: 59 to 64 degrees F
Frost-free period: 210 to 300 days
Farmland classification: Farmland of statewide importance

Map Unit Composition

Vista and similar soils: 85 percent
Minor components: 15 percent
Estimates are based on observations, descriptions, and transects of the mapunit.

Description of Vista

Setting

Landform: Hills
Landform position (two-dimensional): Backslope
Landform position (three-dimensional): Side slope
Down-slope shape: Convex
Across-slope shape: Convex
Parent material: Residuum weathered from granodiorite and quartz-diorite

Typical profile

H1 - 0 to 19 inches: coarse sandy loam
H2 - 19 to 35 inches: coarse sandy loam, sandy loam
H2 - 19 to 35 inches: weathered bedrock
H3 - 35 to 39 inches:

Properties and qualities

Slope: 5 to 9 percent
Depth to restrictive feature: 20 to 40 inches to paralithic bedrock
Natural drainage class: Well drained
Runoff class: Low

Capacity of the most limiting layer to transmit water (Ksat): High (1.98 to 5.95 in/hr)
Depth to water table: More than 80 inches
Frequency of flooding: None
Frequency of ponding: None
Available water storage in profile: Low (about 5.6 inches)

Interpretive groups

Land capability classification (irrigated): 3e
Land capability classification (nonirrigated): 4e
Hydrologic Soil Group: B
Ecological site: LOAMY (1975) (R019XD029CA)
Hydric soil rating: No

Minor Components

Fallbrook

Percent of map unit: 5 percent
Hydric soil rating: No

Bonsall

Percent of map unit: 5 percent
Hydric soil rating: No

Greenfield

Percent of map unit: 3 percent
Hydric soil rating: No

Ramona

Percent of map unit: 2 percent
Hydric soil rating: No

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APPENDIX D

APPENDIX D

RECOMMENDED GRADING SPECIFICATIONS

FOR

1118 NORTH ANZA STREET TOWNHOMES
EL CAJON, CALIFORNIA

PROJECT NO. G2259-32-01

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

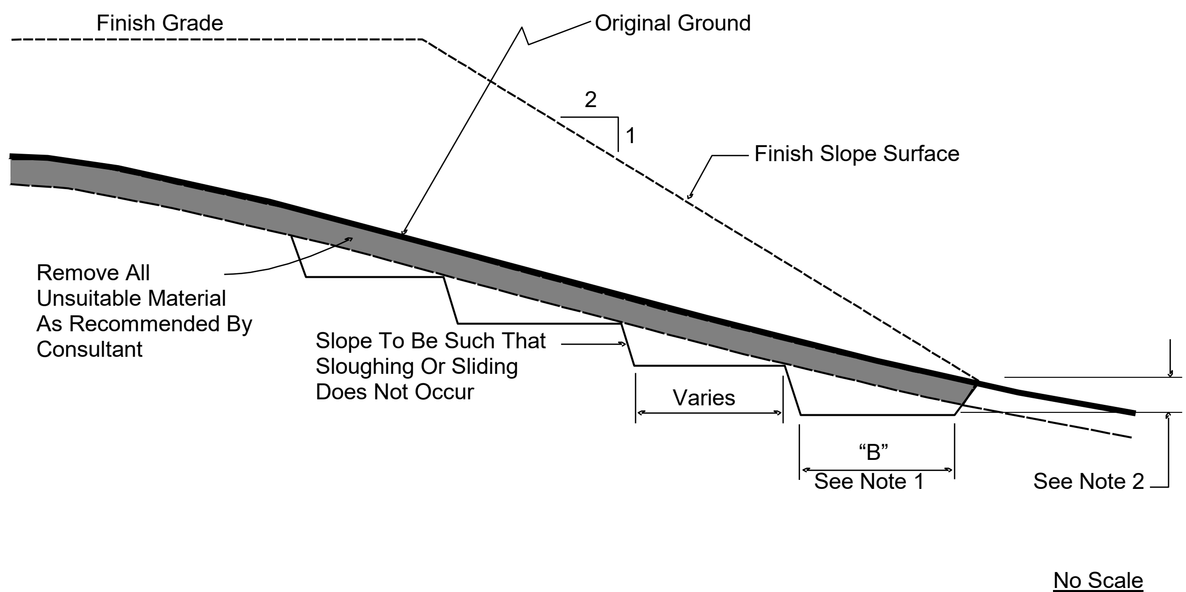
4.1 Areas to be excavated and filled shall be cleared and grubbed. Clearing shall consist of complete removal above the ground surface of trees, stumps, brush, vegetation, man-made structures, and similar debris. Grubbing shall consist of removal of stumps, roots, buried logs and other unsuitable material 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.

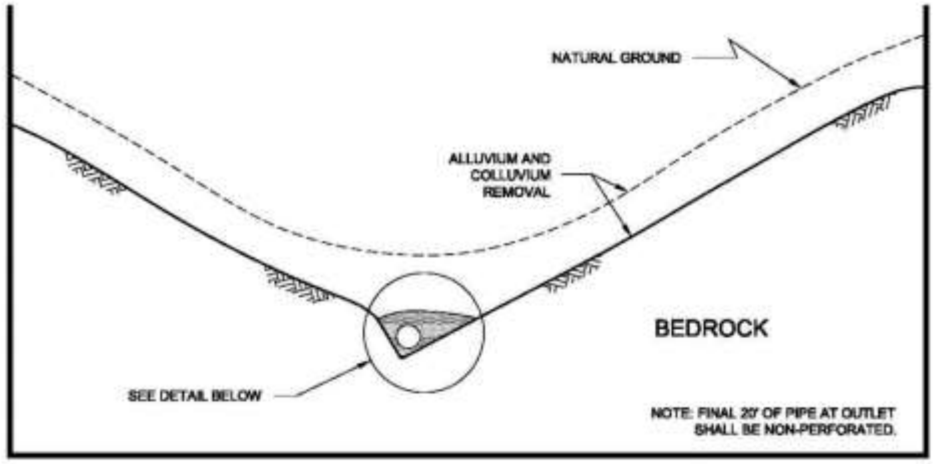
6.3.6 To reduce the potential for "piping" of fines into the *rock* fill from overlying *soil* fill material, a 2-foot layer of graded filter material shall be placed above the uppermost lift of *rock* fill. The need to place graded filter material below the *rock* should be determined by the Consultant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

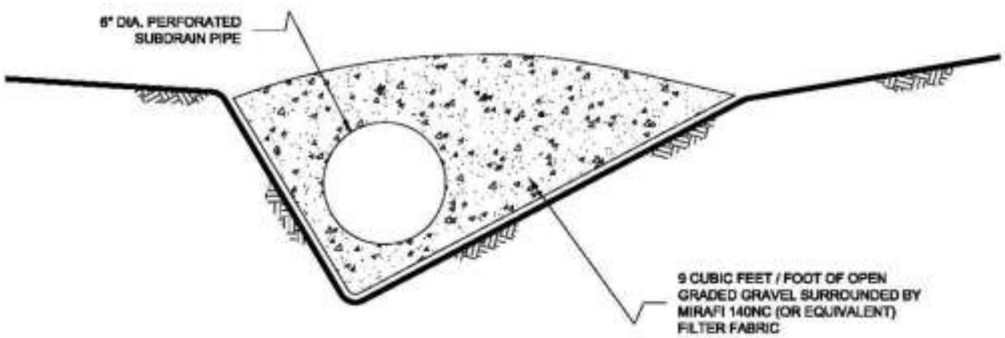
6.3.7 *Rock* fill placement should be continuously observed during placement by the Consultant.

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:

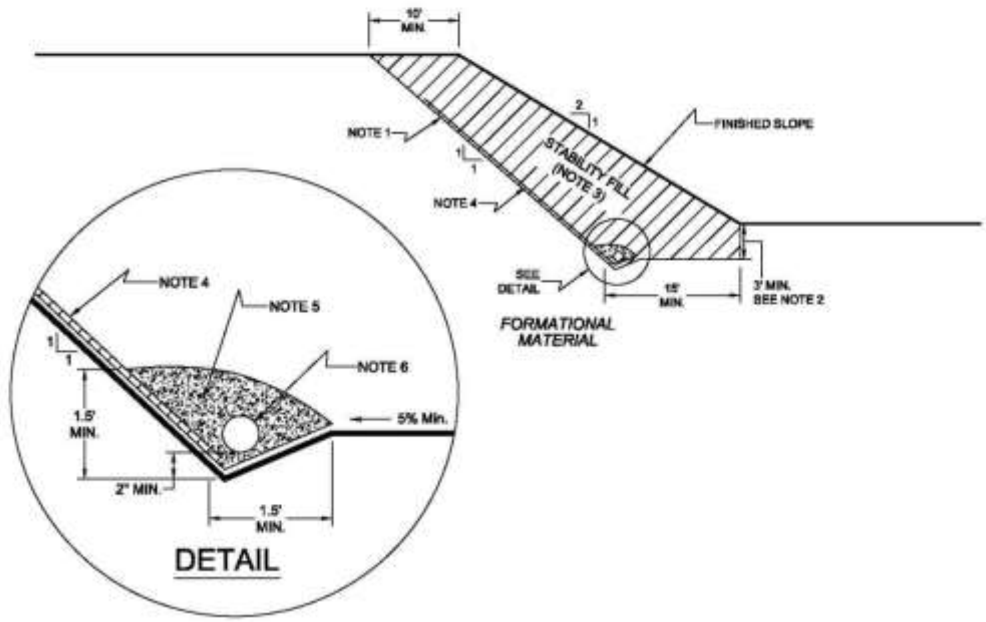
1......8-INCH DIAMETER, SCHEDULE 80 PVC PERFORATED PIPE FOR FILLS IN EXCESS OF 100-FEET IN DEPTH OR A PIPE LENGTH OF LONGER THAN 500 FEET.

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

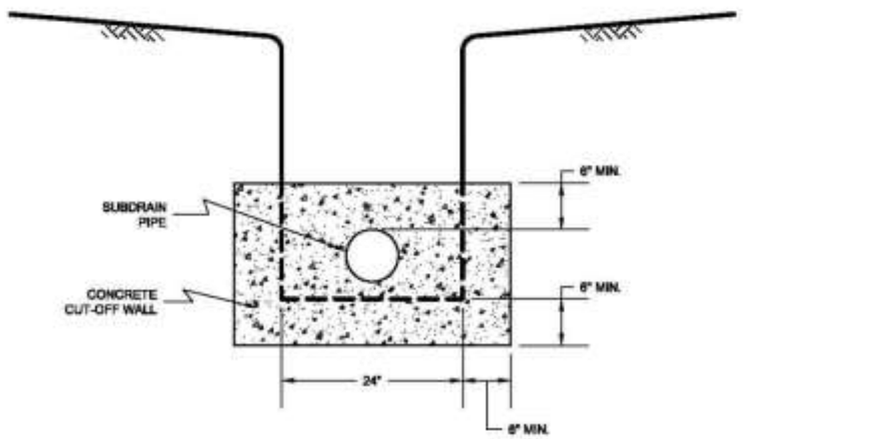
7.3 The actual subdrain locations will be evaluated in the field during the remedial grading operations. Additional drains may be necessary depending on the conditions observed and the requirements of the local regulatory agencies. Appropriate subdrain outlets should be evaluated prior to finalizing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

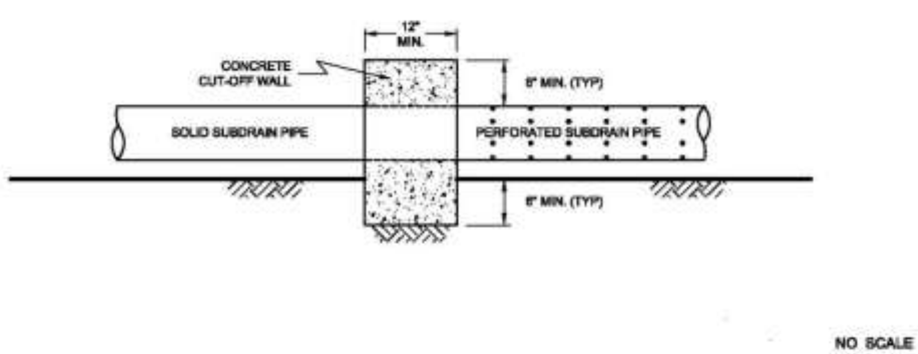
7.5 Prior to outletting, the final 20-foot segment of a subdrain that will not be extended during future development should consist of non-perforated drainpipe. At the non-perforated/ perforated interface, a seepage cutoff wall should be constructed on the downslope side of the pipe.

TYPICAL CUT OFF WALL DETAIL

FRONT VIEW

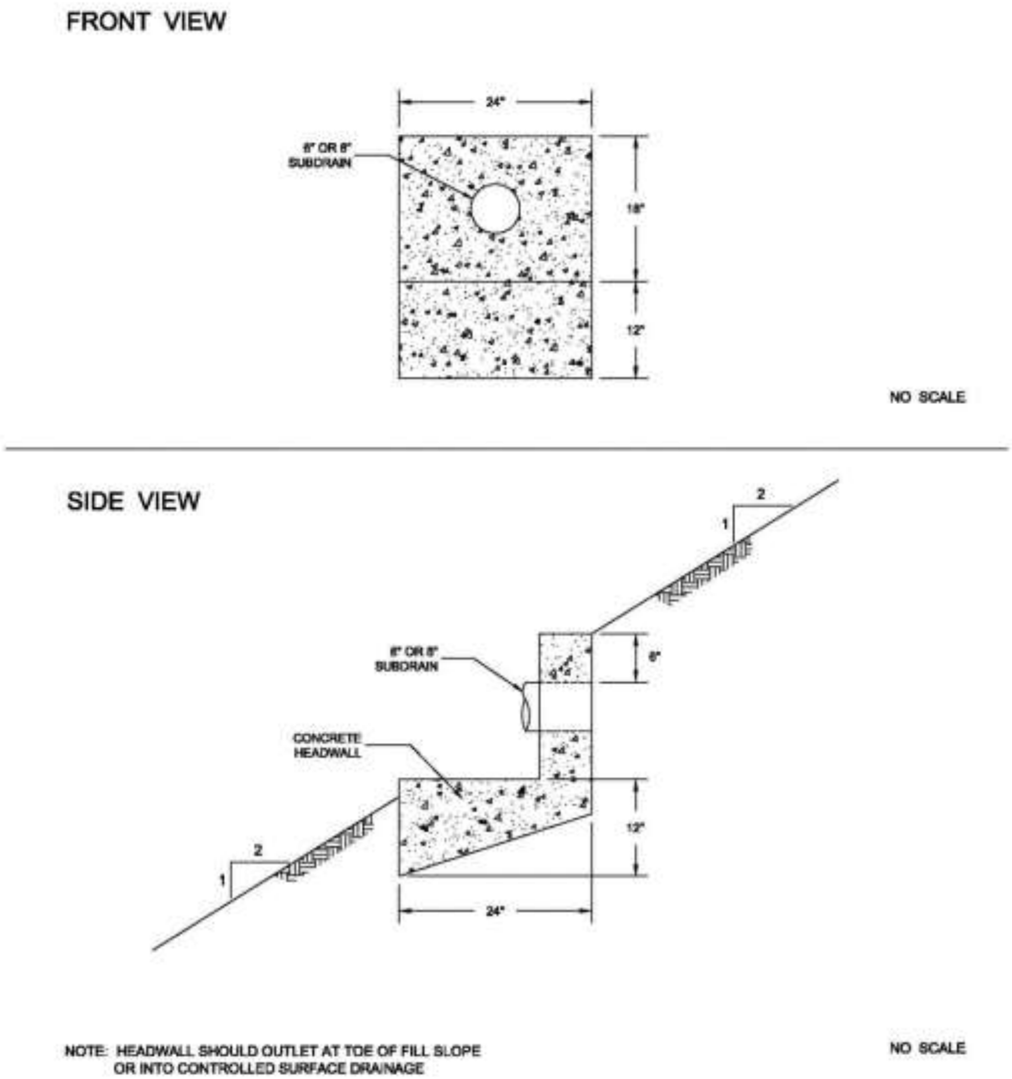


SIDE VIEW



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. California Geological Survey (2003), *Seismic Shaking Hazards in California, Based on the USGS/CGS Probabilistic Seismic Hazards Assessment (PSHA) Model, 2002 (revised April 2003). 10% probability of being exceeded in 50 years.* (http://redirect.conservation.ca.gov/cgs/rghm/pshamap/pshamain.html).

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5. *Fault Activity Map of California and Adjacent Areas,* California Division of Mines and Geology, compiled by C. W. Jennings, 1994.

6. http://www.historicaerials.com.

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

8. Risk Engineering (2015), *EZ-FRISK (version 7.65).*

9. Unpublished reports and maps on file with Geocon Incorporated.

10. USGS (2011), Seismic Hazard Curves and Uniform Hazard Response Spectra (version 5.1.0, dated February 2, 2011), http://earthquake.usgs.gov/research/hazmaps/design/.