UPDATE
GEOTECHNICAL INVESTIGATION

AVENTINE
AT SWEETWATER SPRINGS
SAN DIEGO COUNTY, CALIFORNIA

GEOCON
INCORPORATED

GEOTECHNICAL
ENVIRONMENTAL
MATERIALS

PREPARED FOR

CALATLANTIC HOMES
SAN DIEGO, CALIFORNIA

DECEMBER 22, 2017
REVISED APRIL 26, 2018
PROJECT NO. G2074-32-01

GEOCON
INCORPORATED
GEOTECHNICAL ■ ENVIRONMENTAL ■ MATERIALS

Project No. G2074-32-01
December 22, 2017
Revised April 26, 2018

CalAtantic Homes
16465 Via Esprillo, Suite 150
San Diego, California 92127

Attention: Mr. Arnie White

Subject: UPDATE GEOTECHNICAL INVESTIGATION
AVENTINE AT SWEETWATER SPRINGS
SAN DIEGO COUNTY, CALIFORNIA

Dear Mr. White:

In accordance with your request, and our Proposal No. LG-16469 dated December 6, 2016, and change order dated November 27, 2017, we have performed an update geotechnical investigation on the subject property. The accompanying report presents our findings, conclusions, and recommendations relative to the geotechnical aspects of developing the property as presently proposed. Based on the results of this study, the site can be developed as planned, provided the recommendations of this report are followed. This revised report was prepared to address updated grading plans and a new project name.

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

Troy K. Reist

Troy K. Reist	Trevor E. Myers	David B. Evans
CEG 2408	RCE 63773	CEG 1860

TROY K. REIST No. 2408 CERTIFIED ENGINEERING GEOLOGIST

TREVOR E. MYERS No. RCE63773 CIVIL

DAVID B. EVANS NO. 1860 CERTIFIED ENGINEERING GEOLOGIST

TKR:TEM:DBE:dmc

(e-mail) Addressee
(4/del) Hunsaker & Associates San Diego
Attention: Mr. Ryan Martin

TABLE OF CONTENTS

UPDATE GEOTECHNICAL INVESTIGATION

1. PURPOSE AND SCOPE

The purpose of this update geotechnical investigation was to evaluate the proposed grading for a 92-unit, detached condominium project located within San Diego County, California (see *Vicinity Map*, Figure 1). This report provides recommendations relative to the geotechnical engineering aspects of developing the property as proposed based on the conditions encountered during this investigation and a previous study by Geocon Incorporated. In addition, this report is intended to update our previous report entitled *Preliminary Geotechnical Investigation, 2778-2782 Sweetwater Springs Boulevard, County of San Diego, California,* dated January 6, 2016 (Project No. G2074-32-01) and to address the updated plans prepared by Hunsaker & Associates San Diego, Inc. entitled *Preliminary Grading Plan, Aventine at Sweetwater Springs, County of San Diego, California*, print dated March 16, 2018.

The scope of our study consisted of the following:

- Reviewing aerial photographs and readily available published and unpublished geologic literature.
- Reviewing the preliminary grading plans prepared by Hunsaker & Associates San Diego, Inc.
- Advancing five small-diameter borings to evaluate the general extent and condition of surficial deposits underlying the project site (see Appendix A). The logs of the six borings performed during the previous study are also contained in Appendix A.
- Excavating and exposing the existing retaining wall foundation located along the western property boundary in three locations to allow surveyors to record the footing elevations (see Appendix A).
- Performing laboratory tests on selected soil samples to evaluate the physical characteristics for engineering analysis (see Appendix B).
- Performing two infiltration tests within the proposed water quality/hydromodification basin and providing storm water BMP design information (See Appendix C).
- Preparing this report presenting our exploratory information and our conclusions and recommendations regarding the geotechnical aspects of developing the property as presently proposed. The approximate locations of the previous and recent subsurface excavations are shown on the *Geologic Map,* Figure 2.

2. SITE AND PROJECT DESCRIPTION

The approximately 10.5-acre site consists of a commercial shopping center identified as the Sweetwater Village Shopping Center, located at 2778-2782 Sweetwater Springs Boulevard. The property currently contains eight commercial buildings most of which are vacant and in need of some sort of repair along with an asphalt parking lot and other associated improvements. In addition, two existing retaining walls; one approximate 520 feet long with a maximum height of 14½ feet and the other approximately 300 feet long with a maximum height of 11½ feet are located along the southern and northwestern portions of the site, respectively.

Research for historical documents pertaining to the site resulted in the procurement of the original mass grading and fine grading plans. The site (originally identified as Sweetwater Village Unit 4, Lot 971) appears to have been graded in the mid to late 1970's and it is believed that compaction testing and observation services were provided by Southern California Soils and Testing, Incorporated (SCST). Cuts and fills of approximately 25 and 30 feet, respectively were required to achieve the existing grades. Geotechnical reports specifically relating to the grading operations could not be obtained from the County of San Diego or SCST directly. The original grading plan for the project was digitized and overlain on top of a Google Earth image as shown on the *Historical Grading Plan,* Figure 3 for reference. The plan depicts a satellite image of the site, our subsurface excavations and original ground topography prior to development.

Topographically, the site is characterized as relatively flat to gently sloping with a gradual rise from the southeast to the northwest. Elevations range from approximately 497 feet Mean Sea Level (MSL) in the northwestern portion of the site to 467 feet MSL within the southeastern portion of the project.

It is our understanding that site development consists of grading the site to accommodate ninety-two detached, 2,380- to 2,698-square-foot condominiums, a 14,880-square-foot active recreational area and tot lot, 24 guest parking stalls, 16 recreational area parking stalls, and a water quality/ hydromodification basin.

Based on a review of the preliminary earthwork exhibit, grading will consist of excavating approximately 23,200 cubic yards of soil and filling approximately 23,200 cubic yards, however, these estimates do not account for remedial grading or bulking and shrinking of the materials. The proposed pads will require maximum cut and fill depths on the order of approximately 5 feet or less.

The descriptions contained herein are based upon the site reconnaissance and, a review of the project plans and referenced report. 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

The site is underlain by previously placed fill, colluvium/older alluvium, and granitic rock. The soil and geologic units are shown on Figure 2 described below in order of increasing age. For purposes of this report, the colluvial and older alluvial deposits have been undifferentiated due to their similar characteristics. In addition, the surface contact between previously placed fill and colluvium/older alluvium/bedrock in the vicinity of the western retaining wall was not ampped.

3.1 Previously Placed Fill (Qpf)

Fill deposits associated with the previous grading operations were encountered in each of the exploratory borings advanced across the site. The fill varied in thickness from 5 to 27 feet, however, based on the original grading plans fills up to 30-feet-thick are present. The fill is characterized as medium dense to very dense, clayey to gravely sands to clayey to sandy gravels and stiff to very stiff, silty to gravelly clays with varying amounts of gravel.

Although, not observed directly within the excavations, it is anticipated that oversize material (rocks greater than 12 inches in dimension) may be present within portions of the fill deposits based on difficult drilling and sampling conditions encountered during our subsurface investigations.

3.2 Colluvium/Older Alluvium (Qc/Qoal)

Colluvium and/or older alluvial deposits were encountered in Boring Nos. B-2, B-4, B-7, B-10, and Trench Nos. T-2 and T-3 overlying granitic rock with a maximum thickness of 25 feet (Boring No. B-10). These deposits consist of very stiff to hard, moist, silty clays.

3.3 Granitic Rock (Kgr)

Cretaceous-age granitic rock was encountered within nine of the eleven borings advanced across the site underlying the surficial deposits. The rock material consisted of completely to highly weathered, weak granitic rock. Based on our subsurface information (Boring No. 5 and Trench No. T-1) and review of the *Historical Grading Plan*, it is anticipated that granitic rock may be encountered within the southwestern portion of the site during development.

4. COMPRESSION TESTING

The surficial deposits encountered during our study (previously placed fill and colluvium/older alluvium) consisted of medium dense to very dense, clayey to gravely sands to clayey to sandy gravels and stiff to hard, silty to gravelly clays. Based on information on the original grading plan, it appears that the fill may have been placed under geotechnical observation by SCST, although documentation in this regard could not be obtained at this time. In some areas the fills were placed over thick natural surficial deposits consisting of colluvium/older alluvium.

To evaluate the potential for settlement of these deposits, and to determine the extent of remedial grading that may be required during grading, a compression study was performed consisting of consolidation testing on fifteen undisturbed samples obtained at various intervals within the surficial deposits during our previous and recent studies. The laboratory test results are presented as Figures B-1 through B-15 in Appendix B.

5. GROUNDWATER

Groundwater was not encountered during our field investigation. Groundwater is not anticipated to impact the proposed development, however, perched water conditions may develop following periods of heavy precipitation or prolonged irrigation. In the event that surface seeps develop, shallow subdrains may be necessary to collect and convey the seepage to a suitable outlet facility.

6. GEOLOGIC HAZARDS

6.1 Faulting and Seismicity

Based on our previous observations during mass grading in adjacent areas, recent exploratory borings, 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 Rose Canyon Fault zone and the Newport-Inglewood Fault, located approximately 11 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.

According to the computer program *EZ-FRISK (Version 7.65),* 6 known active faults are located within a search radius of 50 miles from the property. We used the 2008 USGS fault database that provides several models and combinations of fault data to evaluate the fault information. The nearest active faults are the Newport-Inglewood and Rose Canyon Fault Zones, located approximately 11 miles west of the site and is the dominant source of potential ground motion. Earthquakes that might occur on the Newport-Inglewood and Rose Canyon Fault Zone or other faults within the southern California and northern Baja California area are potential generators of significant ground motion at the site. The estimated maximum earthquake magnitude and peak ground acceleration for the Newport-Inglewood Fault are 7.5 and 0.26g, respectively. Table 6.1.1 lists the estimated maximum earthquake magnitude and peak ground acceleration for the most dominant faults in relationship to the site location. We calculated peak ground acceleration (PGA) using Boore-Atkinson (2008) NGA USGS 2008, Campbell-Bozorgnia (2008) NGA USGS 2008, and Chiou-Youngs (2008) NGA acceleration-attenuation relationships.

TABLE 6.1.1
DETERMINISTIC SEISMIC SITE PARAMETERS

Fault Name	Distance from Site (miles)	Maximum Earthquake Magnitude (Mw)	Peak Ground Acceleration		
			Boore-Atkinson 2008 (g)	Campbell-Bozorgnia 2008 (g)	Chiou-Youngs 2008 (g)
Newport-Inglewood	11	7.5	0.25	0.20	0.26
Rose Canyon	11	6.9	0.22	0.18	0.20
Coronado Bank	22	7.4	0.18	0.12	0.15
Palos Verdes Connected	22	7.7	0.20	0.13	0.17
Elsinore	34	7.85	0.16	0.11	0.14
Earthquake Valley	38	6.8	0.10	0.07	0.06

We performed a site-specific probabilistic seismic hazard analysis using the computer program *EZ-FRISK*. Geologic parameters not addressed in the deterministic analysis are included in this analysis. The program operates under the assumption that the occurrence rate of earthquakes on each mappable Quaternary fault is proportional to the faults slip rate. The program accounts for fault rupture length as a function of earthquake magnitude, and site acceleration estimates are made using the earthquake magnitude and 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 in the analysis. Table 6.1.2 presents the site-specific probabilistic seismic hazard parameters including acceleration-attenuation relationships and the probability of exceedence for Site Class D.

TABLE 6.1.2
PROBABILISTIC SEISMIC HAZARD PARAMETERS

Probability of Exceedence	**Peak Ground Acceleration**		
	Boore-Atkinson, 2008 (g)	**Campbell-Bozorgnia, 2008 (g)**	**Chiou-Youngs, 2008 (g)**
2% in a 50 Year Period	0.42	0.35	0.40
5% in a 50 Year Period	0.32	0.26	0.29
10% in a 50 Year Period	0.24	0.21	0.22

While listing peak accelerations is useful for comparison of potential effects of fault activity in a region, other considerations are important in seismic design, including the frequency and duration of motion and the soil conditions underlying the site. Seismic design of the structures should be evaluated in accordance with the California Building Code (CBC) guidelines currently adopted by the County of San Diego.

6.2 Liquefaction

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 densities are less than about 70 percent of the maximum dry densities. If all four criteria are met, a seismic event could result in a rapid increase in pore water pressure from the earthquake-generated ground accelerations. The potential for liquefaction at the site is considered to be negligible due to the dense formational material encountered, remedial grading recommended, and lack of a shallow groundwater condition.

6.3 Landslides

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

7. CONCLUSIONS AND RECOMMENDATIONS

7.1 General

7.1.1 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.

7.1.2 The site is underlain by surficial units that include previously placed fill, colluvium, and/or older alluvium. Based on our observations and laboratory compression testing, the majority of these materials are suitable in their present condition to support the proposed improvements, however, <u>the upper 5 feet of the existing surface</u> will require remedial grading in the form of removal and compaction where improvements are planned. If excavations are planned into the existing surface, additional over excavation should be performed to provide a 5-foot mat of new fill below the improvements. The actual extent of unsuitable soil removal will be determined in the field by the geotechnical engineer and/or engineering geologist during grading.

7.1.3 The existing structures, foundation systems, utility lines and other improvements should be removed and exported from the site prior to grading. Geocon Incorporated should provide testing and observation services during the backfill of the resulting excavations that are deeper than 5 feet.

7.1.4 It is our understanding that the existing retaining walls located along the southern and northwestern property boundary were evaluated by a structural engineer. In addition, the wall footing adjacent to Unit Nos. 25 through 30 was exposed in three locations to allow Hunsaker & Associates to survey the limits and depth of the footing. This information should be shared with the structural engineer for proper evaluation of the current foundation placement and drainage modifications in regards to the proposed development.

7.2 Excavation and Soil Characteristics

7.2.1 Excavation of the surficial deposits should be possible with moderate to heavy effort using conventional heavy-duty equipment. Excavations advanced into the granitic rock will generally vary in difficulty with the depth of excavation depending on the degree of weathering, although blasting is not anticipated. Oversize material (defined as material greater than 12 inches in nominal dimension) may be generated during the grading operations within the previously placed fill and where granitic rock is encountered.

7.2.2 The soils encountered in the field investigation are considered to be "expansive" (expansion index [EI] of 20 or more) as defined by 2016 California Building Code (CBC) Section 1803.5.3 based on laboratory testing. Table 7.2 presents soil classifications based on the expansion index. The soil materials collected and tested for expansion index indicate a "very low" to "high" expansion potential (expansion index of 130 or less).

TABLE 7.2
EXPANSION CLASSIFICATION BASED ON EXPANSION INDEX

Expansion Index (EI)	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	

7.3 Corrosion

7.3.1 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.

7.4 Grading

7.4.1 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.

7.4.2 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.

7.4.3 Site preparation should begin with the removal of all deleterious material 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. Material generated during stripping and/or site demolition should be exported from the site.

7.4.4 The upper 5 feet of surficial deposits present below proposed finish grade in cut and fill areas will require remedial grading in the form of removal and compaction where improvements are planned. The actual extent of unsuitable soil removal will be determined in the field by the geotechnical engineer and/or engineering geologist.

7.4.5 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.

7.4.6 Where practical, the upper 3 feet of all building pads should be comprised of soil with a “very low” to “medium” expansion potential. The more highly expansive fill soils should be placed in the deeper fill areas and properly compacted. “Very low” to “medium” expansive soils are defined by the 2016 California Building Code (CBC) Section 1803.5.3 as those soils that have an Expansion Index of 90 or less. Rock fragments greater than 6 inches in maximum dimension should not be placed within 3 feet of finish grade in building pad areas or within 2 feet of the deepest utility.

7.4.7 If encountered, building pads exposing granitic rock within 3 feet of finish grade should be undercut at least 3 feet and replaced with properly compacted “very low” to “medium” expansive soil. In addition, undercutting of street areas/utility corridors should be considered to facilitate the excavation of underground utilities located in cut areas composed of marginally to non-rippable granitic rock. If subsurface improvements or landscape zones are planned outside these areas, consideration should be given to undercutting these areas as well. This can be evaluated during grading operations.

7.4.8 Oversize material (defined as material greater than 12 inches in nominal dimension) may be generated during the grading operations. Placement of oversize material within fills should be conducted in accordance with the recommendations in Appendix D.

7.5 Seismic Design Criteria

7.5.1 We used the computer program *U.S. Seismic Design Maps*, provided by the USGS. Table 7.5.1 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 7.5.1 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 7.5.1
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	0.849g	Figure 1613.3.1(1)
MCE_R Ground Motion Spectral Response Acceleration – Class B (1 sec), S_1	0.329g	Figure 1613.3.1(2)
Site Coefficient, F_A	1.160	Table 1613.3.3(1)
Site Coefficient, F_V	1.742	Table 1613.3.3(2)
Site Class Modified MCE_R Spectral Response Acceleration (short), S_{MS}	0.985g	Section 1613.3.3 (Eqn 16-37)
Site Class Modified MCE_R Spectral Response Acceleration (1 sec), S_{M1}	0.573g	Section 1613.3.3 (Eqn 16-38)
5% Damped Design Spectral Response Acceleration (short), S_{DS}	0.657g	Section 1613.3.4 (Eqn 16-39)
5% Damped Design Spectral Response Acceleration (1 sec), S_{D1}	0.382g	Section 1613.3.4 (Eqn 16-40)

7.5.2 Table 7.5.2 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 7.5.2
2016 CBC SITE ACCELERATION PARAMETERS

Parameter	Value, Site Class D	ASCE 7-10 Reference
Mapped MCE_G Peak Ground Acceleration, PGA	0.325g	Figure 22-7
Site Coefficient, F_{PGA}	1.175	Table 11.8-1
Site Class Modified MCE_G Peak Ground Acceleration, PGA_M	0.382g	Section 11.8.3 (Eqn 11.8-1)

7.5.3 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.

7.6 Foundation and Concrete Slabs-On-Grade Recommendations

7.6.1 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 7.6.1.

TABLE 7.6.1
FOUNDATION CATEGORY CRITERIA

Foundation Category	Maximum Fill Thickness, T (Feet)	Differential Fill Thickness, D (Feet)	Expansion Index (EI)
I	$T<20$	--	$EI \leq 50$
II	$20 \leq T<50$	$10 \leq D<20$	$50<EI \leq 90$
III	$T \geq 50$	$D \geq 20$	$90<EI \leq 130$

7.6.2 Due to surficial deposits and fill soils to remain in-place after grading, Category III foundations are recommended.

7.6.3 Table 7.6.2 presents minimum foundation and interior concrete slab design criteria for conventional foundation systems.

TABLE 7.6.2
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

7.6.4 The embedment depths presented in Table 7.6.2 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 4.

7.6.5 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.

7.6.6 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.

7.6.7 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 7.6.3 for the particular Foundation Category designated. The parameters presented in Table 7.6.3 are based on the guidelines presented in the PTI DC 10.5 design manual.

TABLE 7.6.3
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

7.6.8 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.

7.6.9 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).

7.6.10 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.

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

- The deflection criteria presented in Table 7.6.3 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.

7.6.12 Our experience indicates post-tensioned slabs are susceptible to excessive edge lift, regardless of the underlying soil conditions. Placing reinforcing steel at the bottom of the perimeter footings and the interior stiffener beams may mitigate this potential. Current PTI design procedures primarily address the potential center lift of slabs but, because of the placement of the reinforcing tendons in the top of the slab, the resulting eccentricity after tensioning reduces the ability of the system to mitigate edge lift. The structural engineer should design the foundation system to reduce the potential of edge lift occurring for the proposed structures.

7.6.13 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.

7.6.14 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 loads due to wind or seismic forces.

7.6.15 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.

7.6.16 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.

7.6.17 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.

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

- For fill slopes less than 20 feet high, building footings should be deepened such that the bottom outside edge of the footing is at least 7 feet horizontally from the face of the slope.
- When located next to a descending 3:1 (horizontal:vertical) fill slope or steeper, the foundations should be extended to a depth where the minimum horizontal distance is equal to H/3 (where H equals the vertical distance from the top of the fill slope to the base of the fill soil) with a minimum of 7 feet but need not exceed 40 feet. The horizontal distance is measured from the outer, deepest edge of the footing to the face of the slope. An acceptable alternative to deepening the footings would be the use of a post-tensioned slab and foundation system or increased footing and slab reinforcement. Specific design parameters or recommendations for either of these alternatives can be provided once the building location and fill slope geometry have been determined.
- If swimming pools are planned, Geocon Incorporated should be contacted for a review of specific site conditions.
- Swimming pools located within 7 feet of the top of cut or fill slopes are not recommended. Where such a condition cannot be avoided, the portion of the swimming pool wall within 7 feet of the slope face be designed assuming that the adjacent soil provides no lateral support. This recommendation applies to fill slopes up to 30 feet in height, and cut slopes regardless of height. For swimming pools located near the top of fill slopes greater than 30 feet in height, additional recommendations may be required and Geocon Incorporated should be contacted for a review of specific site conditions.
- Although other improvements, which are relatively rigid or brittle, such as concrete flatwork or masonry walls, may experience some distress if located near the top of a slope, it is generally not economical to mitigate this potential. It may be possible, however, to incorporate design measures which would permit some lateral soil movement without causing extensive distress. Geocon Incorporated should be consulted for specific recommendations.

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

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

7.7 Retaining Walls and Lateral Loads Recommendations

7.7.1 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 less than 50. Imported low expansion granular soil may be required. Alternatively, the granular low expansive onsite soil may be selectively stockpiled during grading.

7.7.2 If moderately expansive soils (EI greater than 50) are used for backfill, the active earth pressure would increase to 80 pcf for level backfill and 95 pcf for backfill inclined at 2:1 (horizontal:vertical). 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 less than 130. Backfill material exhibiting an Expansion Index greater than 130 should not be used.

7.7.3 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.

7.7.4 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).

7.7.5 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.

7.7.6 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.

7.7.7 Retaining walls should be provided with a drainage system adequate to prevent the buildup of hydrostatic forces and should be waterproofed as required by the project architect. The use of drainage openings through the base of the wall (weep holes) is not recommended where the seepage could be a nuisance or otherwise adversely affect the property adjacent to the base of the wall. A typical retaining wall drainage detail is presented on Figure 5. If conditions different than those described are expected, or if specific drainage details are desired, Geocon Incorporated should be contacted for additional recommendations.

7.7.8 In general, wall foundations having a minimum depth of 24 inches and width of 12 inches may be designed for an allowable soil bearing pressure of 2,000 psf. 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.

7.7.9 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.

7.7.10 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 19H should be used for design. We used the peak ground acceleration adjusted for Site Class effects, PGA_M, of 0.382g calculated from ASCE 7-10 Section 11.8.3 and applied a pseudo-static coefficient of 0.33.

7.7.11 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.

7.7.12 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.

7.7.13 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.

7.8 Slope Maintenance

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

7.9 Site Drainage and Moisture Protection

7.9.1 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.

7.9.2 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.

7.10 Grading and Foundation Plan Review

7.10.1 Geocon Incorporated should review the grading plans and foundation plans for the project prior to final design submittal to evaluate whether additional analyses and/or recommendations are required.

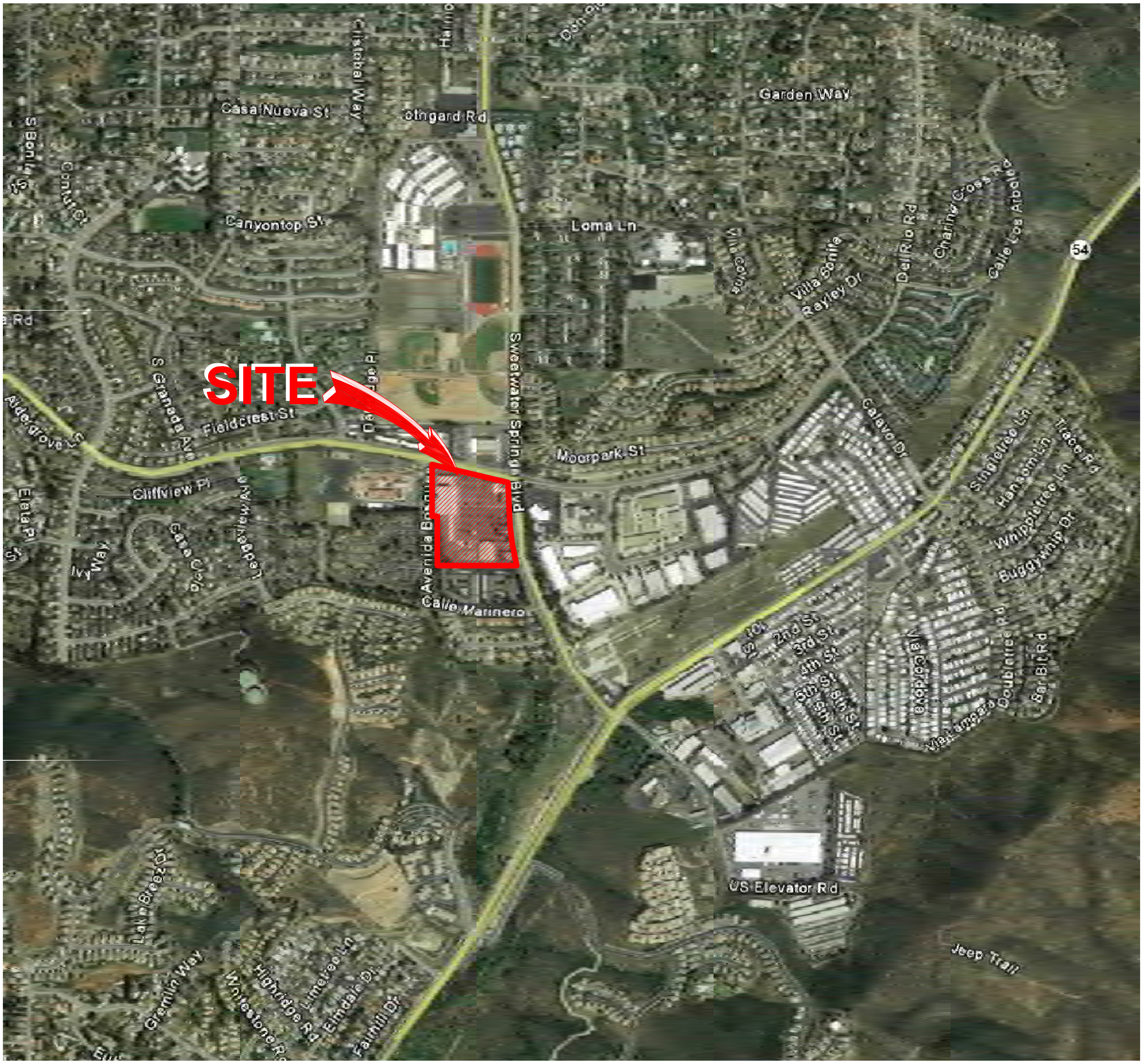
LIMITATIONS AND UNIFORMITY OF CONDITIONS

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 his representative to ensure that the information and recommendations contained herein are brought to the attention of the architect and engineer for the project and incorporated into the plans, and the necessary steps are taken to see that the contractor and subcontractors carry out such recommendations in the field.

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



THE GEOGRAPHICAL INFORMATION MADE AVAILABLE FOR DISPLAY WAS PROVIDED BY GOOGLE EARTH, SUBJECT TO A LICENSING AGREEMENT. THE INFORMATION IS FOR ILLUSTRATIVE PURPOSES ONLY; IT IS NOT INTENDED FOR CLIENT'S USE OR RELIANCE AND SHALL NOT BE REPRODUCED BY CLIENT. CLIENT SHALL INDEMNIFY, DEFEND AND HOLD HARMLESS GEOCON FROM ANY LIABILITY INCURRED AS A RESULT OF SUCH USE OR RELIANCE BY CLIENT.

N

NO SCALE

VICINITY MAP

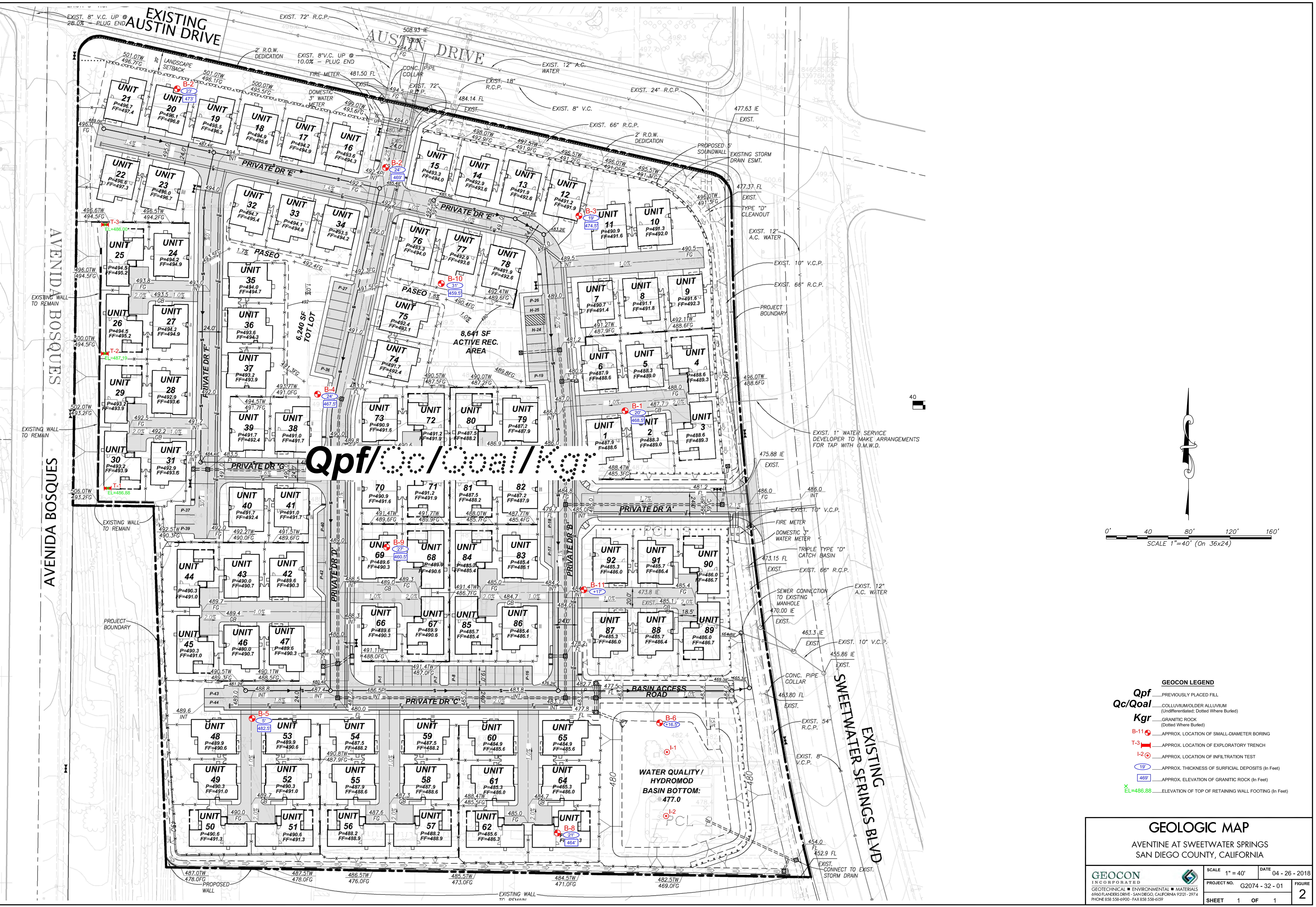
GEOCON
INCORPORATED

GEOTECHNICAL ■ ENVIRONMENTAL ■ MATERIALS
6960 FLANDERS DRIVE - SAN DIEGO, CALIFORNIA 92121 - 2974
PHONE 858 558-6900 - FAX 858 558-6159

AVENTINE AT SWEETWATER SPRINGS
SAN DIEGO COUNTY, CALIFORNIA

TR / RA		DSK/GTYPD	DATE 04 - 26 - 2018	PROJECT NO. G2074 - 32 - 01	FIG. 1

Plotted:04/25/2018 1:08PM | By:ALVIN LADRILLONO | File Location:Y:\PROJECTS\G2074-32-01 Sweetwater Village\DETAILS\G2074-32-01 Vicinity Map.dwg



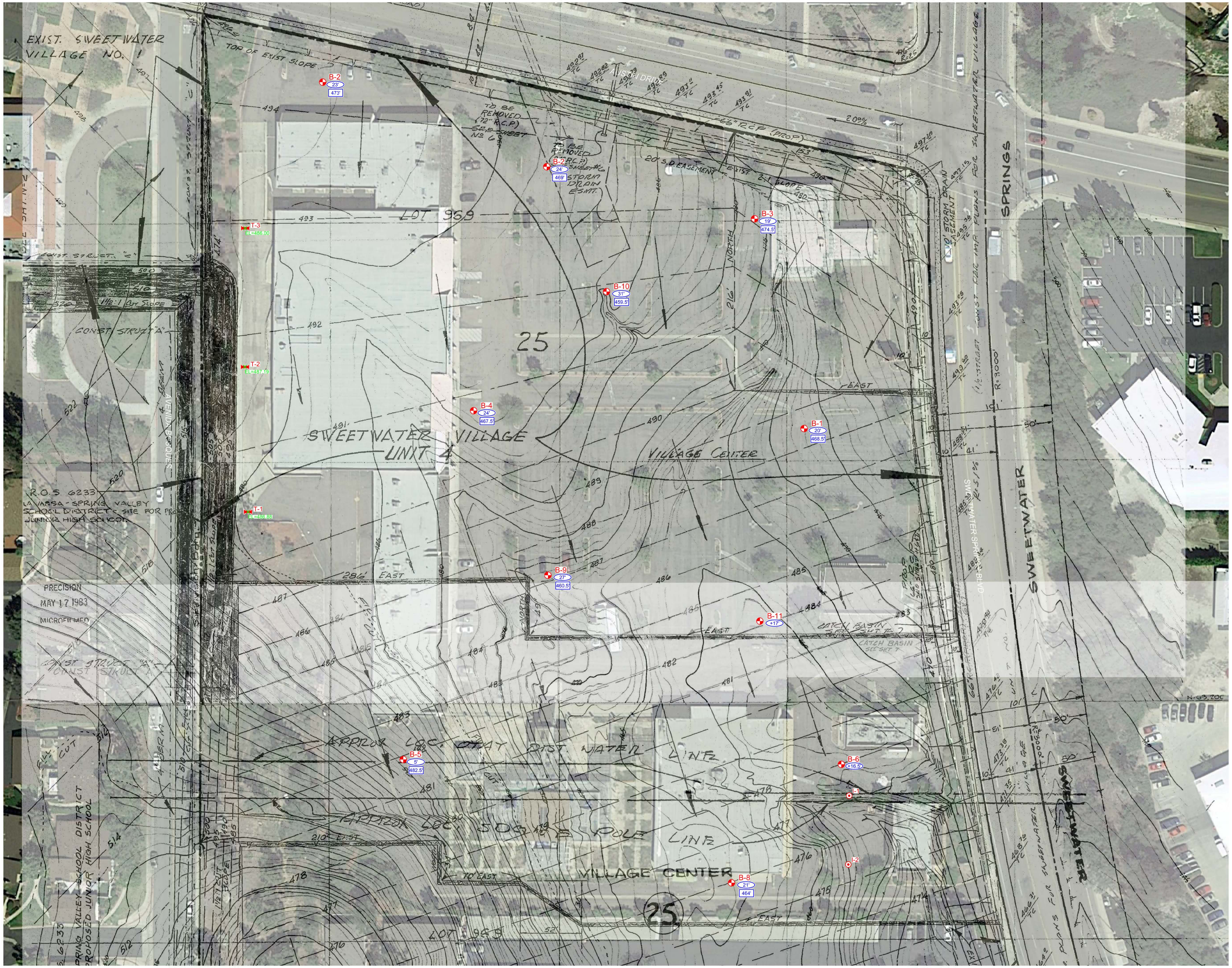
GEOLOGIC MAP

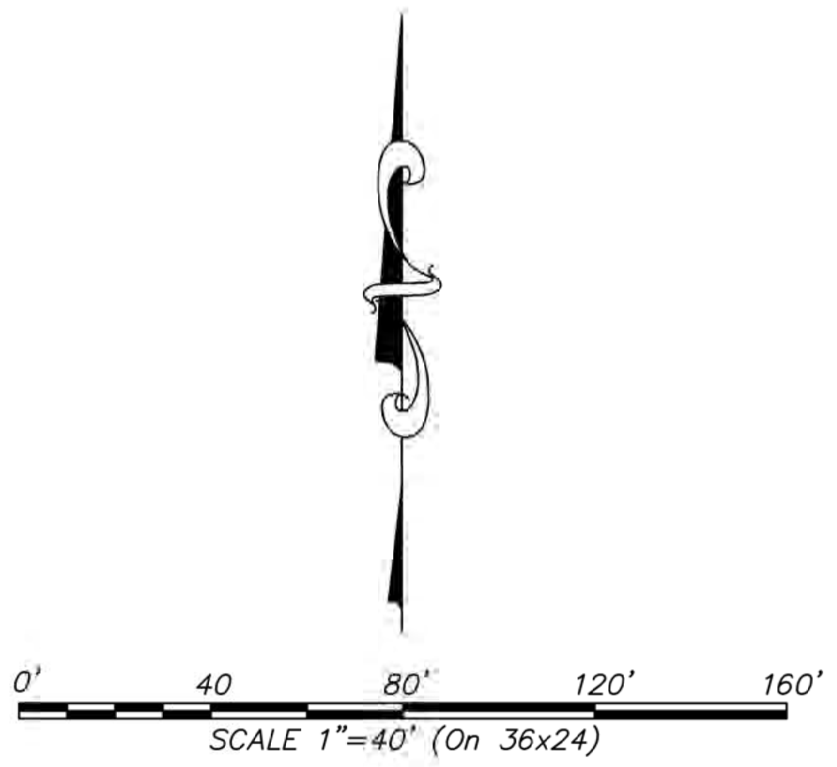
AVENTINE AT SWEETWATER SPRINGS
SAN DIEGO COUNTY, CALIFORNIA

GEOCON
INCORPORATED
GEOTECHNICAL ■ ENVIRONMENTAL ■ MATERIALS
6960 FLANDERS DRIVE - SAN DIEGO, CALIFORNIA 92121 - 2974
PHONE 858 558-6900 - FAX 858 558-6159

SCALE 1" = 40'
DATE 04 - 26 - 2018
PROJECT NO. G2074 - 32 - 01
FIGURE 2
SHEET 1 OF 1

Plotted:04/25/2018 12:55PM | By:ALVIN LADRILLONO | File Location:Y:\PROJECTS\G2074-32-01 Sweetwater Village\SHEETS\G2074-32-01 GeoMap(04-2018).dwg





GEOCON LEGEND

B-11 ⊕	........APPROX. LOCATION OF SMALL DIAMETER BORING
T-3	........APPROX. LOCATION OF EXPLORATORY TRENCH
I-2 ⊙	........APPROX. LOCATION OF INFILTRATION TEST
19'	........APPROX. THICKNESS OF SURFICIAL DEPOSITS (In Feet)
469'	........APPROX. ELEVATION OF GRANITIC ROCK (In Feet)
× EL=486.88	........ELEVATION OF TOP OF RETAINING WALL FOOTING (In Feet)

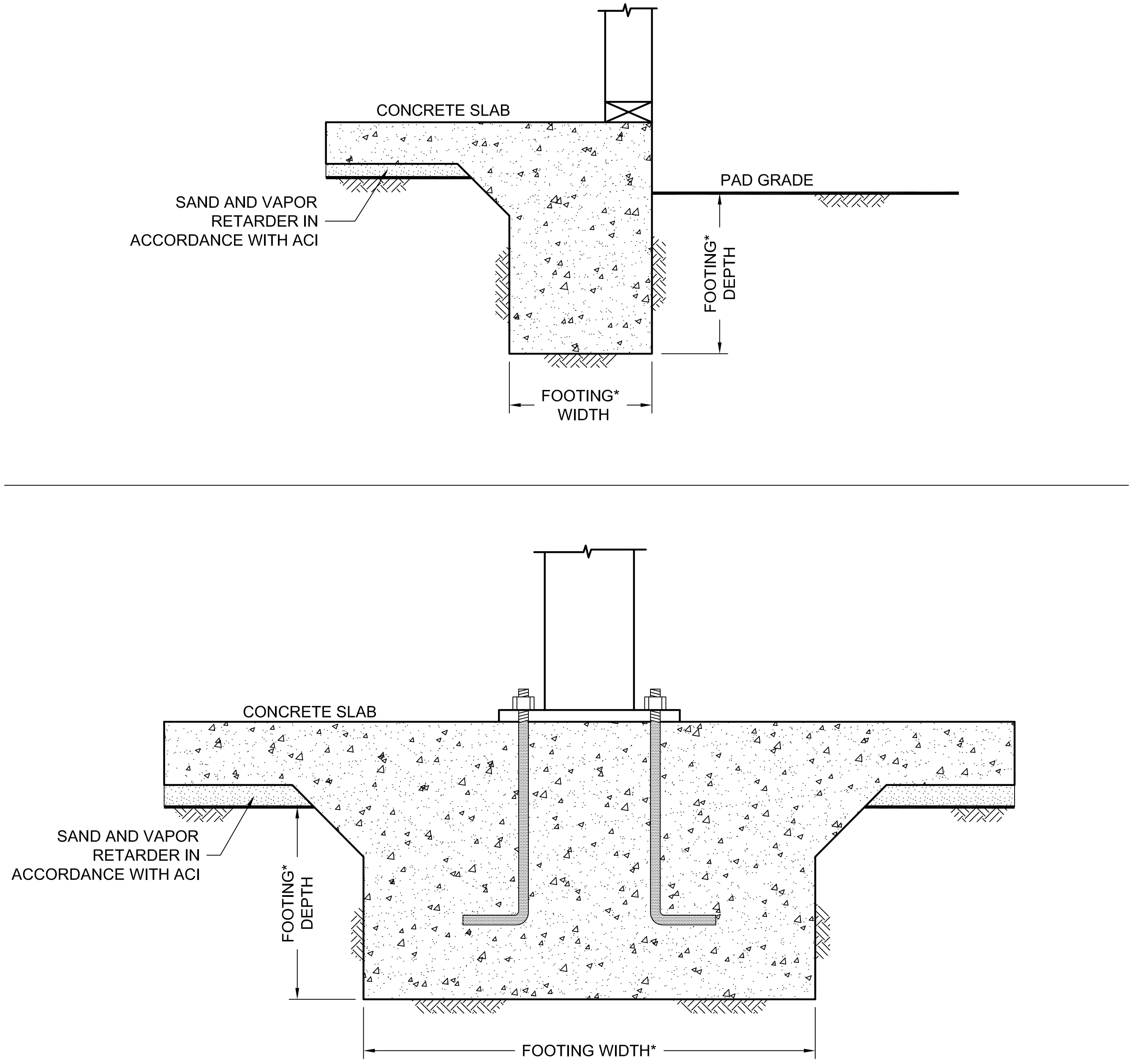
HISTORIC GRADING PLAN

AVENTINE AT SWEETWATER SPRINGS
SAN DIEGO COUNTY, CALIFORNIA

GEOCON
INCORPORATED
GEOTECHNICAL ■ ENVIRONMENTAL ■ MATERIALS
6960 FLANDERS DRIVE - SAN DIEGO, CALIFORNIA 92121 - 2974
PHONE 858 558-6900 - FAX 858 558-6159

SCALE 1" = 40'
DATE 04 - 26 - 2018
PROJECT NO. G2074 - 32 - 01
FIGURE 3
SHEET 1 OF 1

Plotted:04/25/2018 12:59PM | By:ALVIN LADRILLONO | File Location:Y:\PROJECTS\G2074-32-01 Sweetwater Village\SHEETS\G2074-32-01 HistoricGradingPlan.dwg



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

NO SCALE

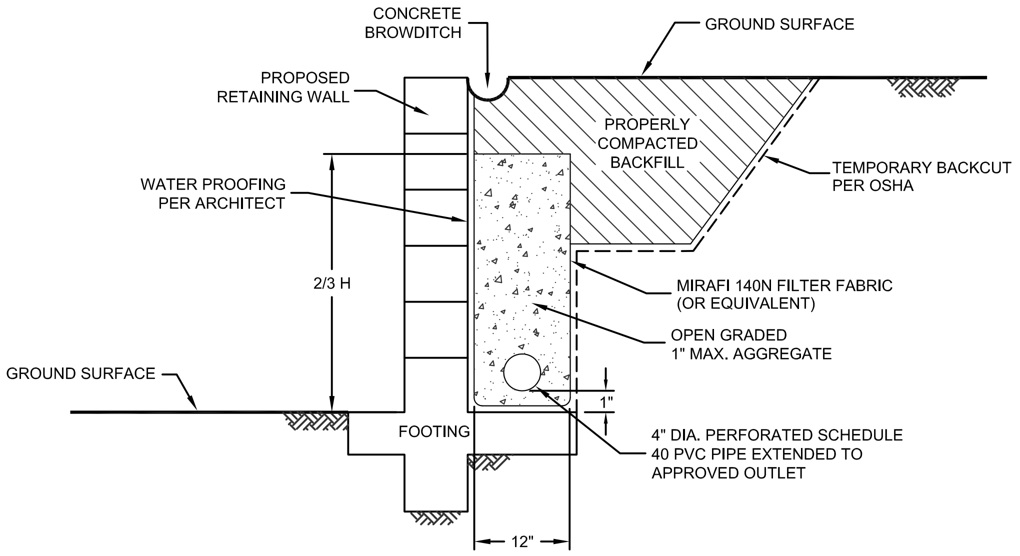
WALL / COLUMN FOOTING DIMENSION DETAIL

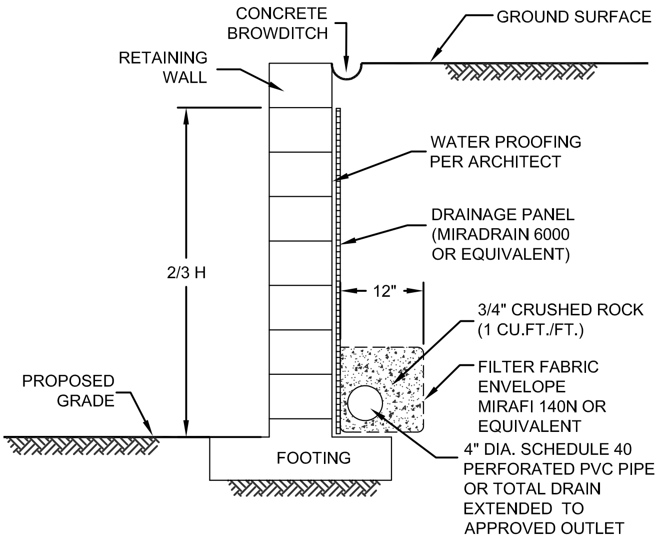
GEOCON
INCORPORATED
GEOTECHNICAL ■ ENVIRONMENTAL ■ MATERIALS
6960 FLANDERS DRIVE - SAN DIEGO, CALIFORNIA 92121 - 2974
PHONE 858 558-6900 - FAX 858 558-6159

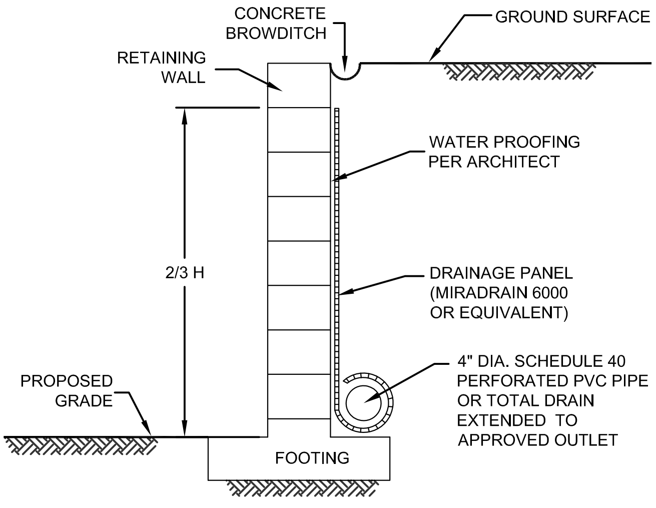
AVENTINE AT SWEETWATER SPRINGS
SAN DIEGO COUNTY, CALIFORNIA

TR / RA		DSK/GTYPD	DATE 04 - 26 - 2018	PROJECT NO. G2074 - 32 - 01	FIG. 4

Plotted:04/25/2018 1:08PM | By:ALVIN LADRILLONO | File Location:Y:\PROJECTS\G2074-32-01 Sweetwater Village\DETAILS\Wall-Column Footing Dimension Detail (COLFOOT2).dwg







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

GEOCON
INCORPORATED

GEOTECHNICAL ■ ENVIRONMENTAL ■ MATERIALS
6960 FLANDERS DRIVE - SAN DIEGO, CALIFORNIA 92121 - 2974
PHONE 858 558-6900 - FAX 858 558-6159

AVENTINE AT SWEETWATER SPRINGS
SAN DIEGO COUNTY, CALIFORNIA

TR / RA		DSK/GTYPD	DATE 04 - 26 - 2018	PROJECT NO. G2074 - 32 - 01	FIG. 5

Plotted:04/25/2018 1:07PM | By:ALVIN LADRILLONO | File Location:Y:\PROJECTS\G2074-32-01 Sweetwater Village\DETAILS\Typical Retaining Wall Drainage Detail (RWDD7A).dwg

APPENDIX A

APPENDIX A

FIELD INVESTIGATION

The previous and recent field investigations were performed on December 21, 2016, and December 6 and 7, 2017, respectively, and consisted of a site reconnaissance and the excavation of eleven small-diameter borings, three exploratory trenches and two infiltration tests. The approximate locations of the subsurface excavations are shown on the *Site Plan*, Figure 2.

The small-diameter borings (Boring Nos. B-1 through B-11) were advanced to a maximum depth of 31½ feet below existing grade using either a CME-75 rig equipped with 6-inch hollow-stem augers or a Mobile B-59 rig equipped with 8-inch hollow stem augers. Relatively undisturbed samples were obtained by driving either a California split-spoon (CAL) sampler or a Standard Penetration Test (SPT), split-tube sampler into the "undisturbed" soil mass. The CAL sampler was equipped with 1-inch by 2⅜-inch, brass sampler rings to facilitate removal and testing. Logs of the borings depicting the soil and geologic conditions encountered and the depth at which samples were obtained are presented on Figures A-1 through A-11.

The three backhoe trenches (T-1 through T-3) were advanced to maximum depth of 3½ feet using a John Deere 410 rubber-tire backhoe equipped with a 24-inch-wide bucket in order to expose the retaining wall foundation located along the western property. The concrete overlying the footings was saw cut and removed prior to exposing the foundation. The top of the foundation was surveyed and the information is presented on Figures 2 and 3. Logs of the backhoe trenches depicting the soil and geologic conditions encountered and approximate depth of the exposed footing are presented on Figures A-12 through A-14.

The results and discussion of the infiltration testing is discussed in *Appendix C* of this report.

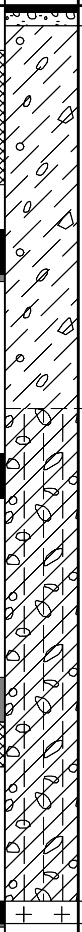
The soils encountered in the excavations were visually classified and logged in general accordance with American Society for Testing and Materials (ASTM) practice for Description and Identification of Soils (Visual Manual Procedure D 2488).

PROJECT NO. G2074-32-01

BORING B 1

ELEV. (MSL.) **488.5'** DATE COMPLETED **12-21-2016**

EQUIPMENT **CME 75** BY: **T. REIST**



DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0					2-inches of **ASPHALT CONCRETE** over 4-inches of **AGGREGATE BASE**			
2	B1-1 ▩			GC&CL	**FILL** Medium dense/very stiff, damp to moist, reddish to orange brown, Clayey GRAVEL to Gravelly CLAY; slow difficult drilling			
	B1-2 ■					25		
6	░				-No recovery of SPT sample at 6 feet	50/2"		
10	B1-3 ■			CL	Very stiff, moist, dark gray-green, Silty CLAY with gravel	37	121.1	15.1
	░				-No recovery of CAL sample at 15 feet	30		
16	B1-4 ▩							
20	B1-5 ■				**GRANITIC ROCK** Completely to highly weathered, dark green and white, weak GRANITIC ROCK -Grinding on rock at 20.5 feet for 20 mins	50/6" 50/1"		
					REFUSAL AT 20.5 FEET			

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

G2074-32-01_UPD_2018-04-26.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.

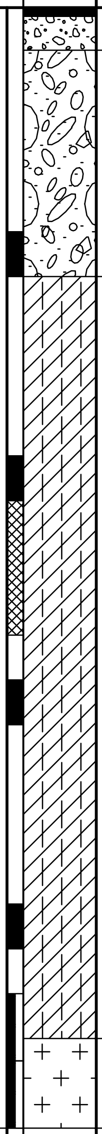
GEOCON

PROJECT NO. G2074-32-01

BORING B 2

ELEV. (MSL.) 496' DATE COMPLETED 12-21-2016

EQUIPMENT CME 75 BY: T. REIST



DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0					2.5-inches of **ASPHALT CONCRETE** over 9-inches of **AGGREGATE BASE**			
2				SM&GM	**FILL** Medium dense, damp, reddish brown, Gravelly, fine to coarse SAND to fine to coarse, Sandy GRAVEL with clay			
4								
	B2-1					50	109.6	21.1
6				CL/CH	**COLLUVIUM/OLDER ALLUVIUM** Very stiff, moist, brown with white caliche, Silty CLAY			
8								
10	B2-2					42	108.3	21.0
	B2-3							
12								
14								
	B2-4					48	99.8	23.5
16								
18								
20	B2-5					48	107.8	21.3
22					-Poor recovery of SPT sample at 22 feet	49		
					GRANITIC ROCK Highly weathered, dark green and white, weak GRANITIC ROCK			
24	B2-6					57		
					BORING TERMINATED AT 25 FEET			

G2074-32-01_UPD_2018-04-26.GPJ

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

SAMPLE SYMBOLS

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

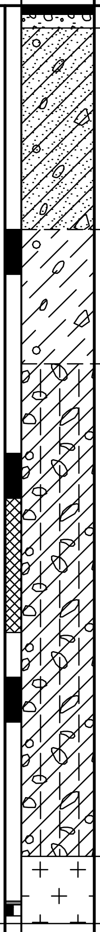
NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

PROJECT NO. G2074-32-01

BORING B 3

ELEV. (MSL.) **493.5'** DATE COMPLETED **12-21-2016**

EQUIPMENT **CME 75** BY: **T. REIST**



DEPTH IN FEET	SAMPLE NO.	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0				2.5-inches of **ASPHALT CONCRETE** over 3.5-inches of **AGGREGATE BASE**			
			CL	**FILL** Very stiff, damp to moist, orange brown to reddish brown, fine to coarse, Sandy CLAY with gravel			
2							
4							
	B3-1		GC/CL	Medium dense/very stiff, damp to moist, Clayey GRAVEL/Gravelly CLAY; slow difficult drilling	46	120.2	12.5
6							
8			CL	Stiff, moist, dark gray-green and brown, Silty CLAY with gravel			
10	B3-2				24	95.1	25.7
	B3-3						
12							
14							
	B3-4			-Becomes very stiff	28	121.1	13.0
16							
18							
				GRANITIC ROCK Highly weathered, dark green, weak GRANITIC ROCK			
20				-No recovery on CAL sample at 20 feet	50/1" 50/3"		

BORING TERMINATED AT 20.5 FEET

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

G2074-32-01_UPD_2018-04-26.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. G2074-32-01

BORING B 4

ELEV. (MSL.) **491.5'** DATE COMPLETED **12-21-2016**

EQUIPMENT **CME 75** BY: **T. REIST**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0					3-inches of **ASPHALT CONCRETE** over 4-inches of **AGGREGATE BASE**			
2				SM	**FILL** Medium dense, damp, reddish brown, Gravelly, fine to coarse SAND with clay			
4								
6	B4-1				-Blow counts likely not accurate due to gravel	73		
	B4-2							
8								
10					-Contact observed in upper portion of sample			
	B4-3			CL/CH	**COLLUVIUM/OLDER ALLUVIUM** Hard, moist, brown with white caliche, Silty CLAY	50	115.6	16.2
12								
14								
16	B4-4				-Becomes very stiff	42	108.5	20.4
18								
20	B4-5				-Becomes hard with some gravel present	62	128.8	13.0
22								
24					-Contact based on cuttings and drill rig efficiency			
	B4-6				**GRANITIC ROCK** Completely to highly weathered, dark green and white, weak GRANITIC ROCK; very slow drilling below 26 feet	54		
26	B4-7					50/6"		
					BORING TERMINATED AT 26.5 FEET			

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

G2074-32-01_UPD_2018-04-26.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. G2074-32-01

BORING B 5

ELEV. (MSL.) **487.5'** DATE COMPLETED **12-21-2016**

EQUIPMENT **CME 75** BY: **T. REIST**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0					2-inches of **ASPHALT CONCRETE** over 6-inches of **AGGREGATE BASE**			
				SM	**FILL** Medium dense, damp, reddish brown, Gravelly, fine to coarse SAND with clay			
2								
4								
	B5-1				**GRANITIC ROCK** Completely weathered, dark green and white, weak GRANITIC ROCK	64/11"		
6								
8								
10								
	B5-2				-Becomes highly weathered	80/8"		

BORING TERMINATED AT 11 FEET

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

G2074-32-01_UPD_2018-04-26.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. G2074-32-01

BORING B 6

ELEV. (MSL.) **482.5'** DATE COMPLETED **12-21-2016**

EQUIPMENT **CME 75** BY: **T. REIST**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0					3.5-inches of **ASPHALT CONCRETE** over 6-inches of **AGGREGATE BASE**			
				SC&GM	**FILL** Dense, damp, reddish brown, Clayey, fine to coarse SAND to Gravelly, fine to coarse SAND			
2								
4								
	B6-1				-Blow counts likely not accurate due to gravels	79/11"		
6								
	B6-2				-Very slow drilling and gravel content appears to be increasing with depth			
8								
10					-Becomes orange brown			
	B6-3			GM	Very dense, orange brown, fine to coarse, Sandy GRAVEL with clay	55		
12								
14								
	B6-4					81/10"		
16					-Very slow drilling; grinding on rock for 30 mins at 16.5 feet			
					REFUSAL AT 16.5 FEET			

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

G2074-32-01_UPD_2018-04-26.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. G2074-32-01

BORING B 7

ELEV. (MSL.) 493'   DATE COMPLETED 12-07-2017

EQUIPMENT MOBILE B-59   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	**FILL** Medium dense, damp to moist, reddish brown, Gravelly, fine to coarse SAND			
2								
4								
	B7-1 ■				-Becomes dense below 5 feet -Contact observed in shoe	50	116.6	10.8
6				CL/CH	**COLLUVIUM/OLDER ALLUVIUM** Very stiff, moist, brown with white caliche, Silty CLAY with gravel			
	B7-2 ▨							
8								
10	B7-3 ■				-Blow counts not accurate due to gravel	75	109.2	19.5
12	B7-4 ▨							
14								
	B7-5 ■				-Becomes stiff	21	106.4	20.3
16								
18								
20	B7-6 ■				-Becomes very stiff	27	108.4	20.4
22								
24					-Driller notes hard drilling at 24 feet; contact based on drill rig efficiency			
					GRANITIC ROCK Highly weathered, dark green and white, weak GRANITIC ROCK			
	B7-7 ■					32	124.1	14.3
26					BORING TERMINATED AT 26 FEET			

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

G2074-32-01_UPD_2018-04-26.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. G2074-32-01

BORING B 8

ELEV. (MSL.) 485' DATE COMPLETED 12-07-2017

EQUIPMENT MOBILE B-59 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	**FILL** Medium dense, damp, reddish brown, Gravelly, fine to coarse SAND with clay			
2								
4								
	B8-1	■				29	117.8	12.9
6								
8								
10								
	B8-2	■				38	---	7.1
12								
	B8-3	⊠						
14								
	B8-4	■				35	122.6	8.8
16								
18								
20								
	B8-5	■			-Contact observed in shoe	70/9"	---	7.0
					GRANITIC ROCK Highly weathered, dark green and white, weak GRANITIC ROCK			
					BORING TERMINATED AT 21 FEET			

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

G2074-32-01_UPD_2018-04-26.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 9

ELEV. (MSL.) **487.5** DATE COMPLETED **12-07-2017**

EQUIPMENT **MOBILE B-59** BY: **D. GITHENS**

DEPTH IN FEET	SAMPLE NO.	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0		SM	**FILL** Medium dense, damp to moist, reddish brown, Gravelly, fine to coarse SAND with clay			
5	B9-1			35	119.4	12.1
7	B9-2					
10	B9-3			47	---	9.2
15	B9-4			20	---	8.2
20	B9-5			29	113.0	15.6
22	B9-6	CL	Very stiff, moist, dark gray-green, Silty CLAY with gravel			
25	B9-7			49/10"	100.0	27.2
27			-Driller notes hard drilling at 27 feet; contact based on drill rig efficiency			
27			**GRANITIC ROCK** Highly weathered, dark green and white, weak GRANITIC ROCK			

Figure A-9,
Log of Boring B 9, Page 1 of 2

G2074-32-01_UPD_2018-04-26.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.

BORING B 9

ELEV. (MSL.) **487.5** DATE COMPLETED **12-07-2017**

EQUIPMENT **MOBILE B-59** 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 (%)
30	B9-8 ■					50/6"	119.8	15.1
					BORING TERMINATED AT 31 FEET			

Figure A-9,
Log of Boring B 9, Page 2 of 2

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

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

PROJECT NO. G2074-32-01

BORING B 10

ELEV. (MSL.) **490.5'** DATE COMPLETED **12-07-2017**

EQUIPMENT **MOBILE B-59** 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				CL	**FILL** Stiff to very stiff, damp to moist, orange-brown to reddish brown, fine to coarse, Sandy CLAY with gravel			
2								
4								
	B10-1					48	128.9	6.3
6				CL/CH	**COLLUVIUM/OLDER ALLUVIUM** Stiff to very stiff, moist, brown with white caliche, Silty CLAY			
8								
10	B10-2					18	107.1	13.4
12								
14								
	B10-3					30	104.3	23.8
16								
18								
20	B10-4					29	113.9	19.3
22								
24	B10-5					18	105.3	24.0
26								
28								

Figure A-10,
Log of Boring B 10, Page 1 of 2

G2074-32-01_UPD_2018-04-26.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. G2074-32-01

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	**BORING B 10** ELEV. (MSL.) **490.5'** DATE COMPLETED **12-07-2017** EQUIPMENT **MOBILE B-59** BY: **D. GITHENS**	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
					MATERIAL DESCRIPTION			
30	B10-6 ■			CL		18	107.7	23.2
					-Contact observed in shoe			
		+ +			**GRANITIC ROCK** Highly weathered, dark green and white, weak GRANITIC ROCK			
					BORING TERMINATED AT 31.5 FEET			

Figure A-10,
Log of Boring B 10, Page 2 of 2

G2074-32-01_UPD_2018-04-26.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. G2074-32-01

BORING B 11

ELEV. (MSL.) **485'** DATE COMPLETED **12-07-2017**

EQUIPMENT **MOBILE B-59** 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				GM	**FILL** Medium dense, damp to moist, reddish brown, fine to coarse, Sandy GRAVEL; difficult drilling			
2								
4								
	B11-1					45	---	6.9
6								
8								
10								
	B11-2					23	127.9	9.3
				CL	Stiff, damp, gray brown with white caliche, Silty CLAY			
12								
14								
	B11-3					41	114.2	20.3
16								
					-Loud drill chatter; difficult drilling			
					REFUSAL AT 17 FEET			

Figure A-11,
Log of Boring B 11, Page 1 of 1

G2074-32-01_UPD_2018-04-26.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. G2074-32-01

TRENCH T 1

ELEV. (MSL.) **490'** DATE COMPLETED **12-06-2017**

EQUIPMENT **JD 410 Rubber Tire Backhoe** BY: **T. REIST**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0					6.5" thick **CONCRETE**			
					GRANITIC ROCK Highly weathered, dark green and white, weak GRANITIC ROCK			
2								
					*Edge of retaining wall encountered in western half of trench at 3 feet			
					TRENCH TERMINATED AT 3.5 FEET			

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

G2074-32-01_UPD_2018-04-26.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.

PROJECT NO. G2074-32-01

TRENCH T 2

ELEV. (MSL.) **488'** DATE COMPLETED **12-06-2017**

EQUIPMENT **JD 410 Rubber Tire Backhoe** BY: **T. REIST**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0					6.5" thick **CONCRETE**			
				CL/CH	**COLLUVIUM/OLDER ALLUVIUM** Very stiff, moist, brown with white caliche, Silty CLAY *Edge of retaining wall footing encountered in western half of trench at 8 inches			
2								
					TRENCH TERMINATED AT 3.5 FEET			

Figure A-13,
Log of Trench T 2, Page 1 of 1

G2074-32-01_UPD_2018-04-26.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. G2074-32-01

TRENCH T 3

ELEV. (MSL.) **487.5'** DATE COMPLETED **12-06-2017**

EQUIPMENT **JD 410 Rubber Tire Backhoe** BY: **T. REIST**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0					7" thick **CONCRETE**			
				CL/CH	**COLLUVIUM/OLDER ALLUVIUM** Very stiff, moist, brown with white caliche, Silty CLAY *Edge of retaining wall footing encountered in western half of trench at 1.5 feet			
2					TRENCH TERMINATED AT 2 FEET			

Figure A-14,
Log of Trench T 3, Page 1 of 1

G2074-32-01_UPD_2018-04-26.GPJ

SAMPLE SYMBOLS

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

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

GEOCON

APPENDIX B

APPENDIX B

LABORATORY TESTING

Laboratory tests were performed in accordance with generally accepted test methods of the American Society for Testing and Materials (ASTM) or other suggested procedures. Selected relatively undisturbed ring and bulk samples were tested for in-place dry density and moisture content, expansion index, shear strength and consolidation characteristics.

The results of our laboratory tests are summarized on Tables B-I and B-II and Figures B-1 through B-15. The results of the dry density and moisture content tests are presented on the boring logs in Appendix A.

TABLE B-I
SUMMARY OF LABORATORY DIRECT SHEAR TEST RESULTS

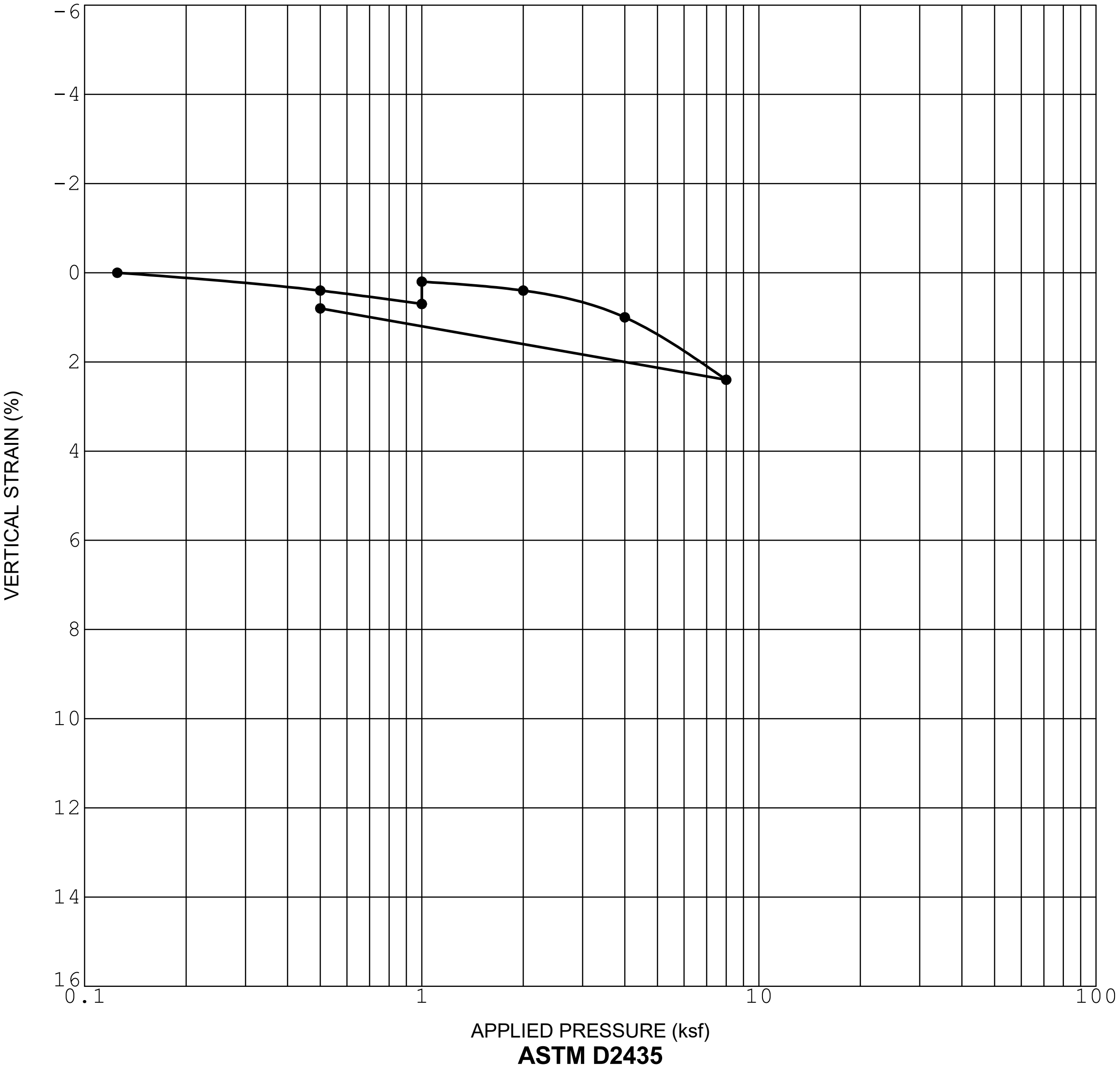
Sample No.	Geologic Unit Symbol (USCS Soil Type)	Dry Density (pcf)	Moisture Content (%)	Unit Cohesion (psf)	Angle of Shear Resistance (degrees)
B9-1	Qpf (SM)	119.4	12.1	720	30
B10-2	Qc/Qoal (CH)	107.1	13.4	125 [455]	37 [26]

[ ] Denotes Ultimate Shear Strength for silt and clay materials.

TABLE B-II
SUMMARY OF LABORATORY EXPANSION INDEX TEST RESULTS

Sample No.	Geologic Unit (USCS Soil Type)	Moisture Content (%)		Dry Density (pcf)	Expansion Index
		Before Test	After Test		
B7-2	Qc/Qoal (CH)	12.9	33.8	99.3	110
B9-2	Qpf (SM)	7.7	17.6	116.8	23

SAMPLE NO. B1-3



ASTM D2435

Initial Dry Density (pcf)	121.1
Initial Water Content (%)	15.1

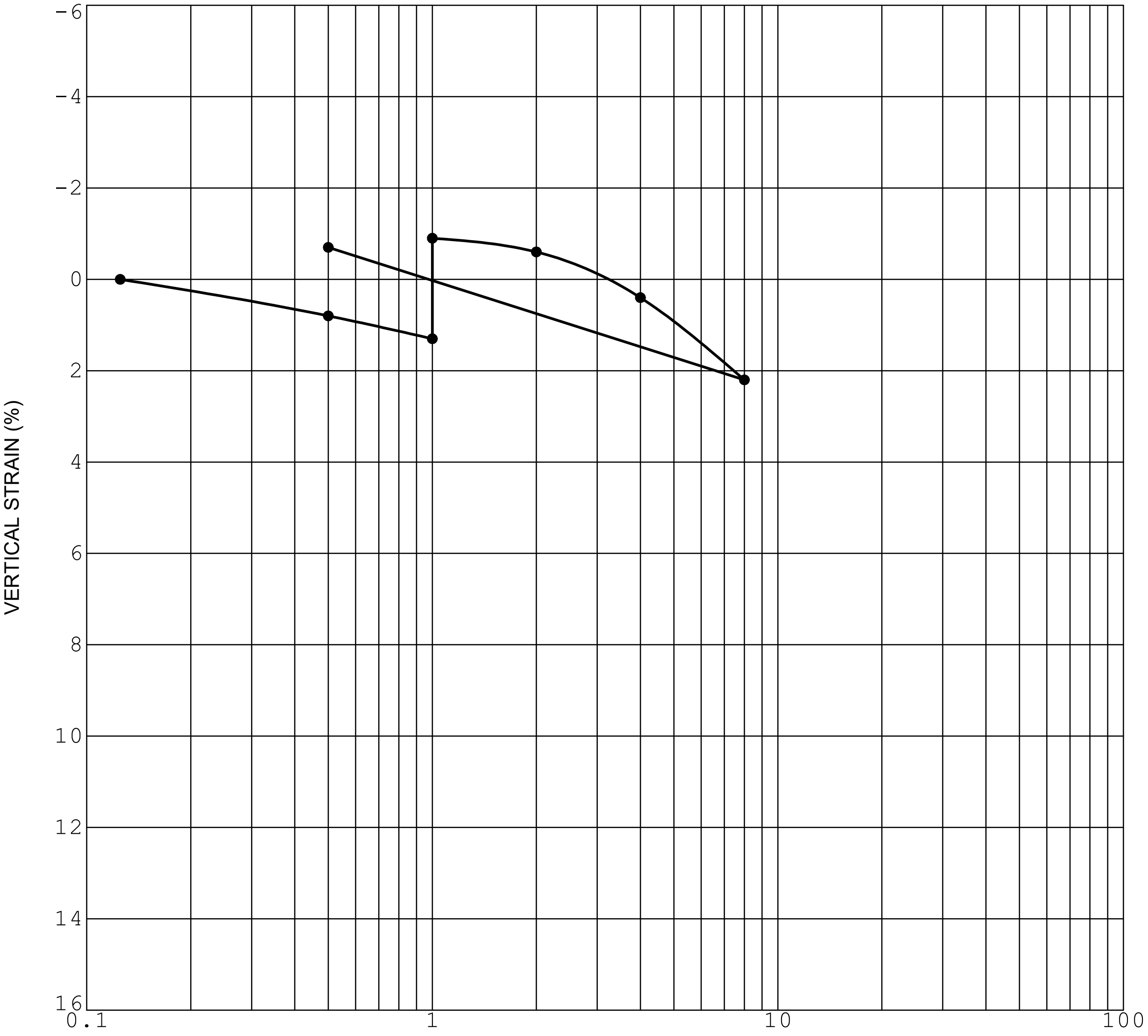
Initial Saturation (%)	100+
Sample Saturated at (ksf)	1.0

CONSOLIDATION CURVE

AVENTINE AT SWEETWATER SPRINGS

SAN DIEGO COUNTY, CALIFORNIA

SAMPLE NO. B2-1



ASTM D2435

Initial Dry Density (pcf)	109.6
Initial Water Content (%)	21.1

Initial Saturation (%)	100+
Sample Saturated at (ksf)	1.0

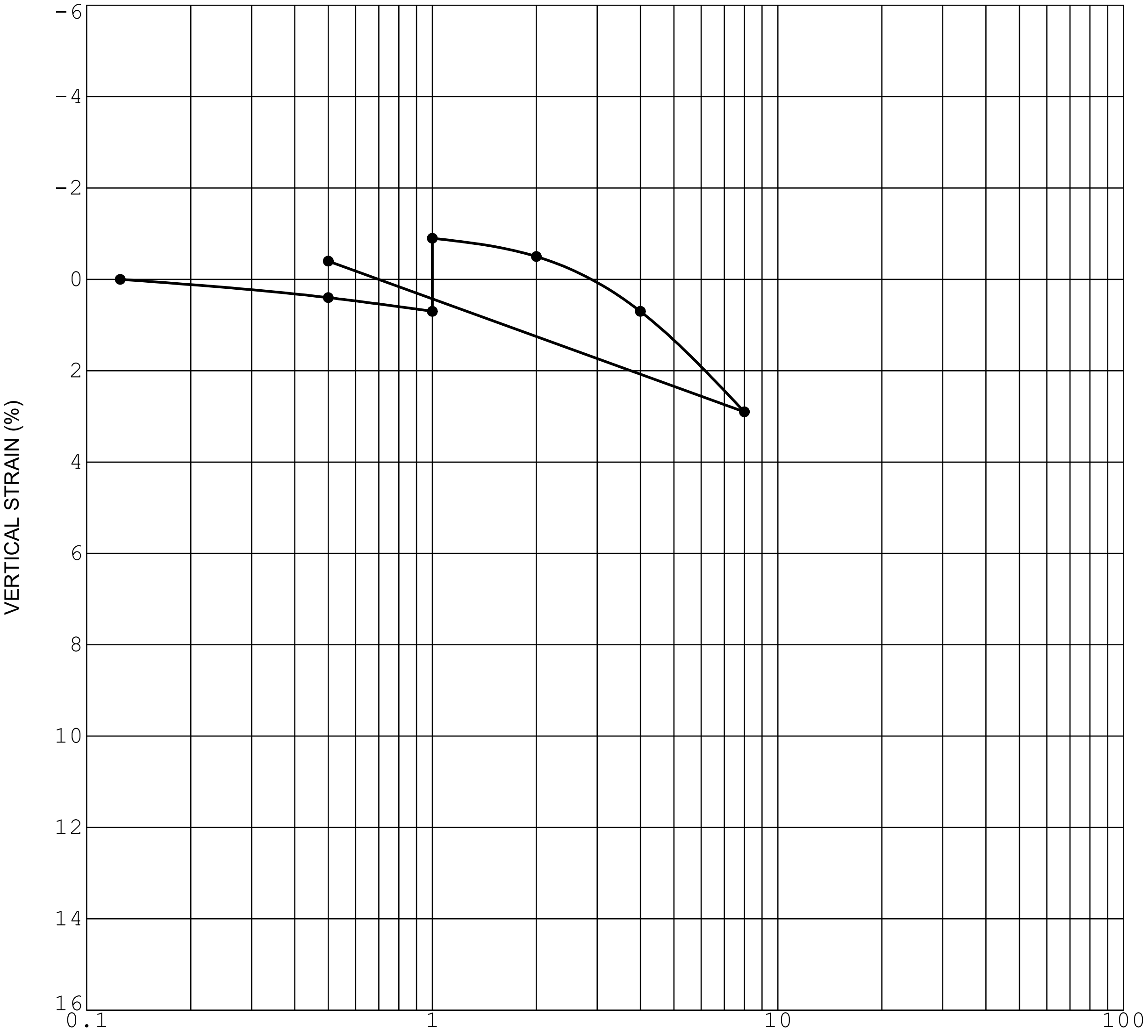
CONSOLIDATION CURVE

AVENTINE AT SWEETWATER SPRINGS

SAN DIEGO COUNTY, CALIFORNIA

PROJECT NO. G2074-32-01

SAMPLE NO. B2-2



APPLIED PRESSURE (ksf)

ASTM D2435

Initial Dry Density (pcf)	108.3
Initial Water Content (%)	21.0

Initial Saturation (%)	100+
Sample Saturated at (ksf)	1.0

CONSOLIDATION CURVE

AVENTINE AT SWEETWATER SPRINGS

SAN DIEGO COUNTY, CALIFORNIA

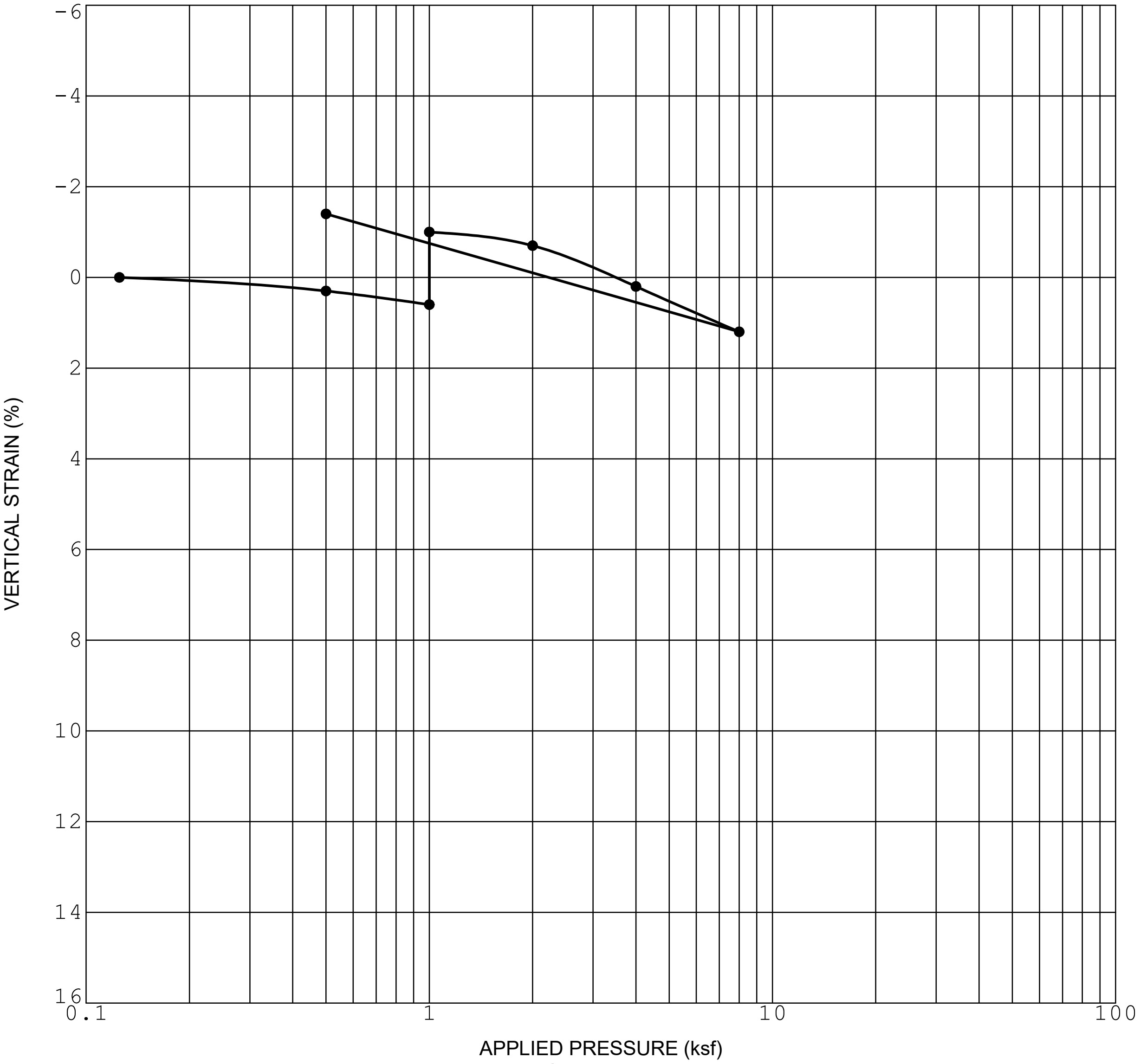
G2074-32-01_UPD_2018-04-26.GPJ

Figure B-3

GEOCON

PROJECT NO. G2074-32-01

SAMPLE NO. B2-4



ASTM D2435

Initial Dry Density (pcf)	99.8
Initial Water Content (%)	23.5

Initial Saturation (%)	94.2
Sample Saturated at (ksf)	1.0

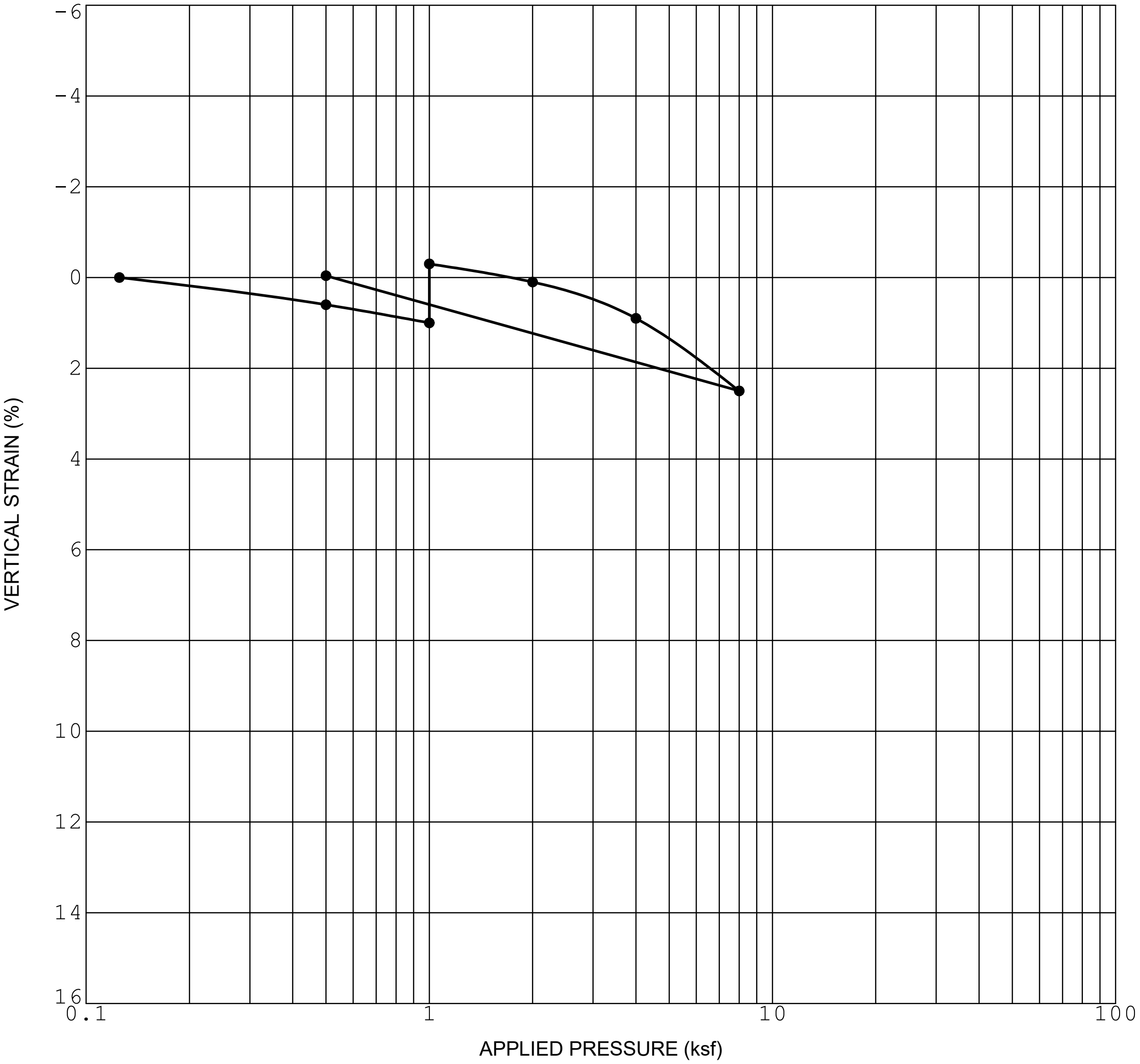
CONSOLIDATION CURVE

AVENTINE AT SWEETWATER SPRINGS

SAN DIEGO COUNTY, CALIFORNIA

PROJECT NO. G2074-32-01

SAMPLE NO. B2-5



ASTM D2435

Initial Dry Density (pcf)	107.8	**Initial Saturation (%)**	100+
Initial Water Content (%)	21.3	**Sample Saturated at (ksf)**	1.0

CONSOLIDATION CURVE

AVENTINE AT SWEETWATER SPRINGS

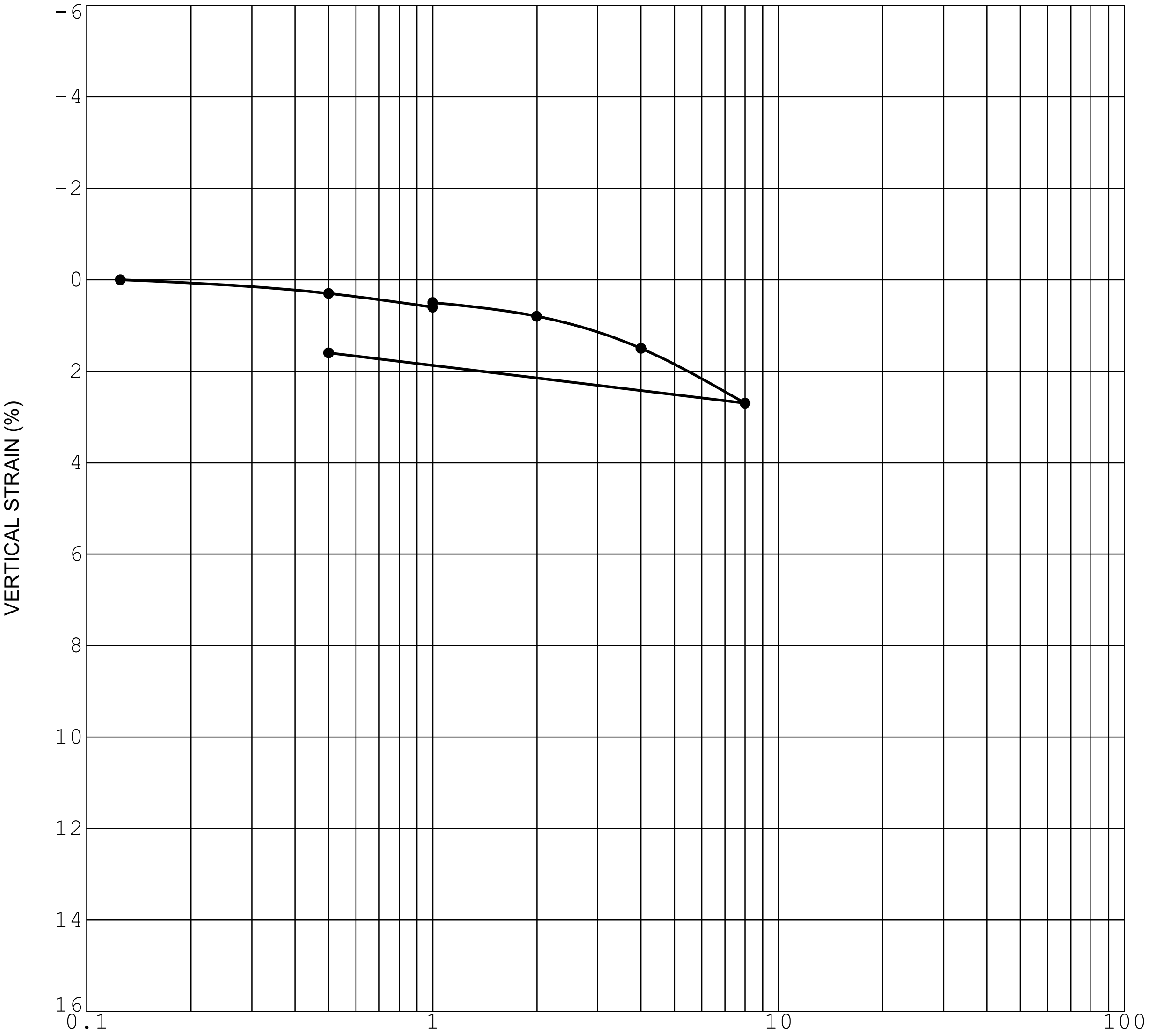
SAN DIEGO COUNTY, CALIFORNIA

G2074-32-01_UPD_2018-04-26.GPJ

Figure B-5

PROJECT NO. G2074-32-01

SAMPLE NO. B3-1



APPLIED PRESSURE (ksf)

ASTM D2435

Initial Dry Density (pcf)	120.2
Initial Water Content (%)	12.5

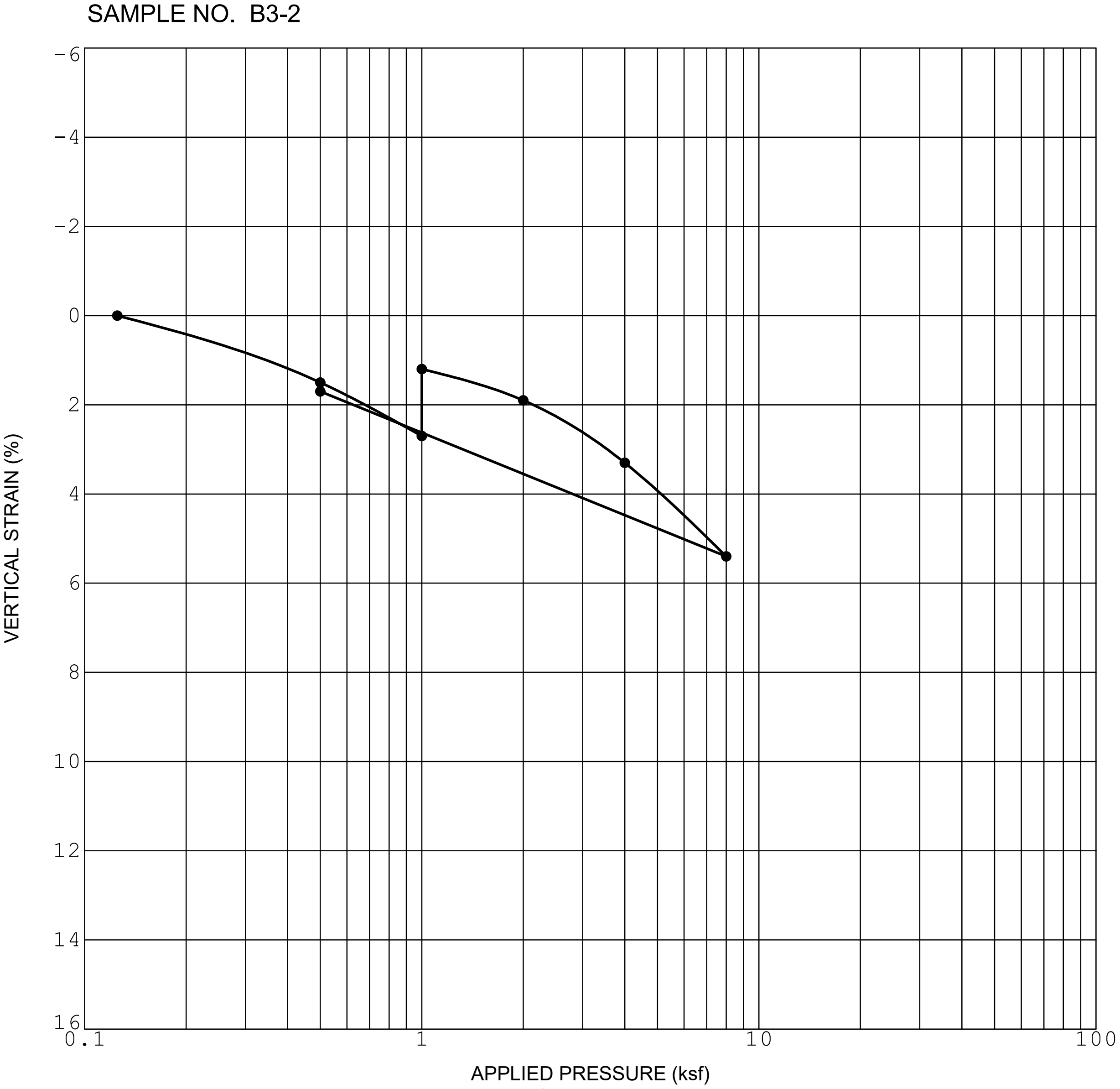
Initial Saturation (%)	87.2
Sample Saturated at (ksf)	1.0

CONSOLIDATION CURVE

AVENTINE AT SWEETWATER SPRINGS

SAN DIEGO COUNTY, CALIFORNIA

PROJECT NO. G2074-32-01



ASTM D2435

Initial Dry Density (pcf)	95.1
Initial Water Content (%)	25.7

Initial Saturation (%)	91.8
Sample Saturated at (ksf)	1.0

CONSOLIDATION CURVE

AVENTINE AT SWEETWATER SPRINGS

SAN DIEGO COUNTY, CALIFORNIA

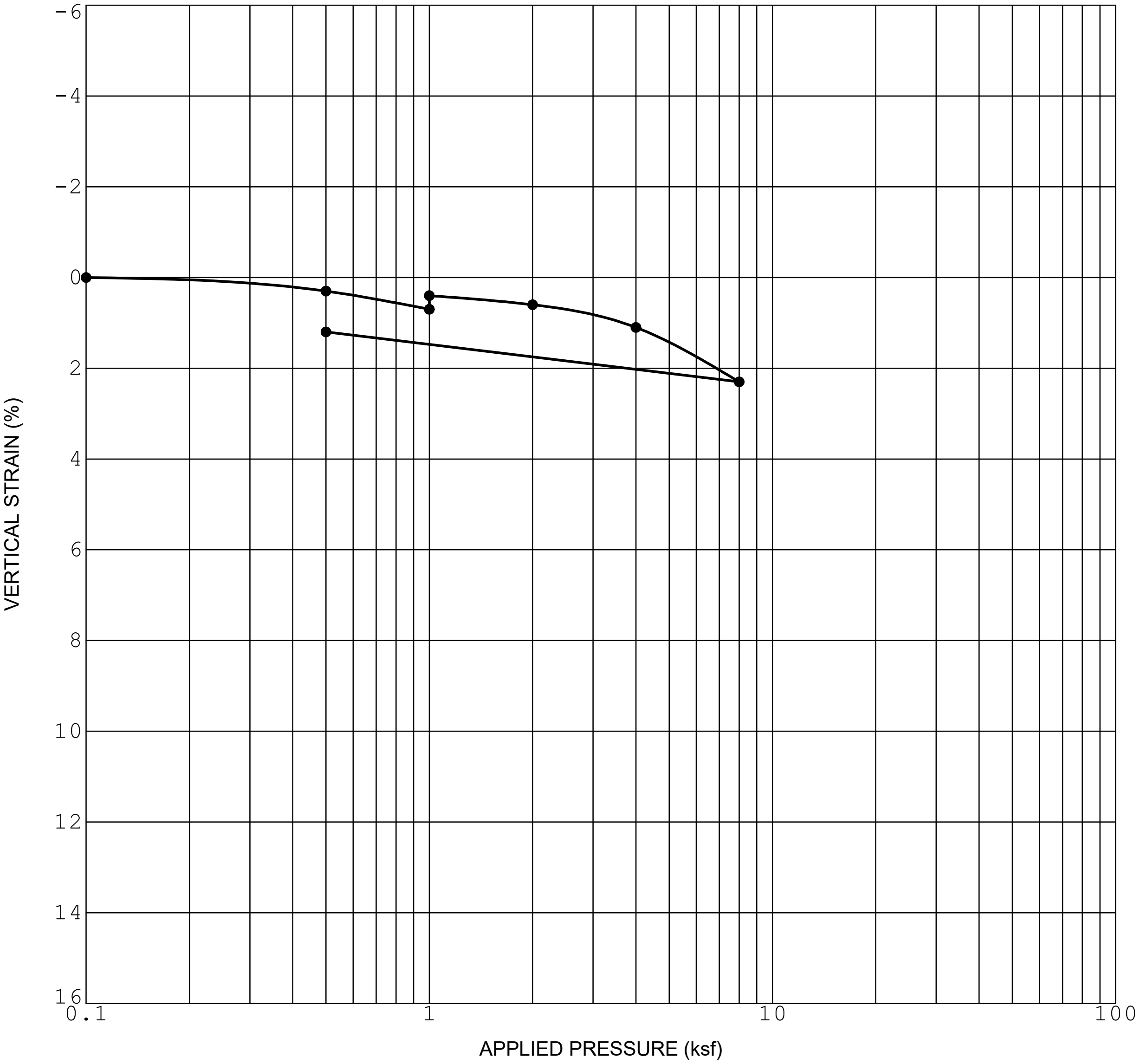
G2074-32-01_UPD_2018-04-26.GPJ

Figure B-7

GEOCON

PROJECT NO. G2074-32-01

SAMPLE NO. B3-4



ASTM D2435

Initial Dry Density (pcf)	121.1
Initial Water Content (%)	13.0

Initial Saturation (%)	93.1
Sample Saturated at (ksf)	1.0

CONSOLIDATION CURVE

AVENTINE AT SWEETWATER SPRINGS

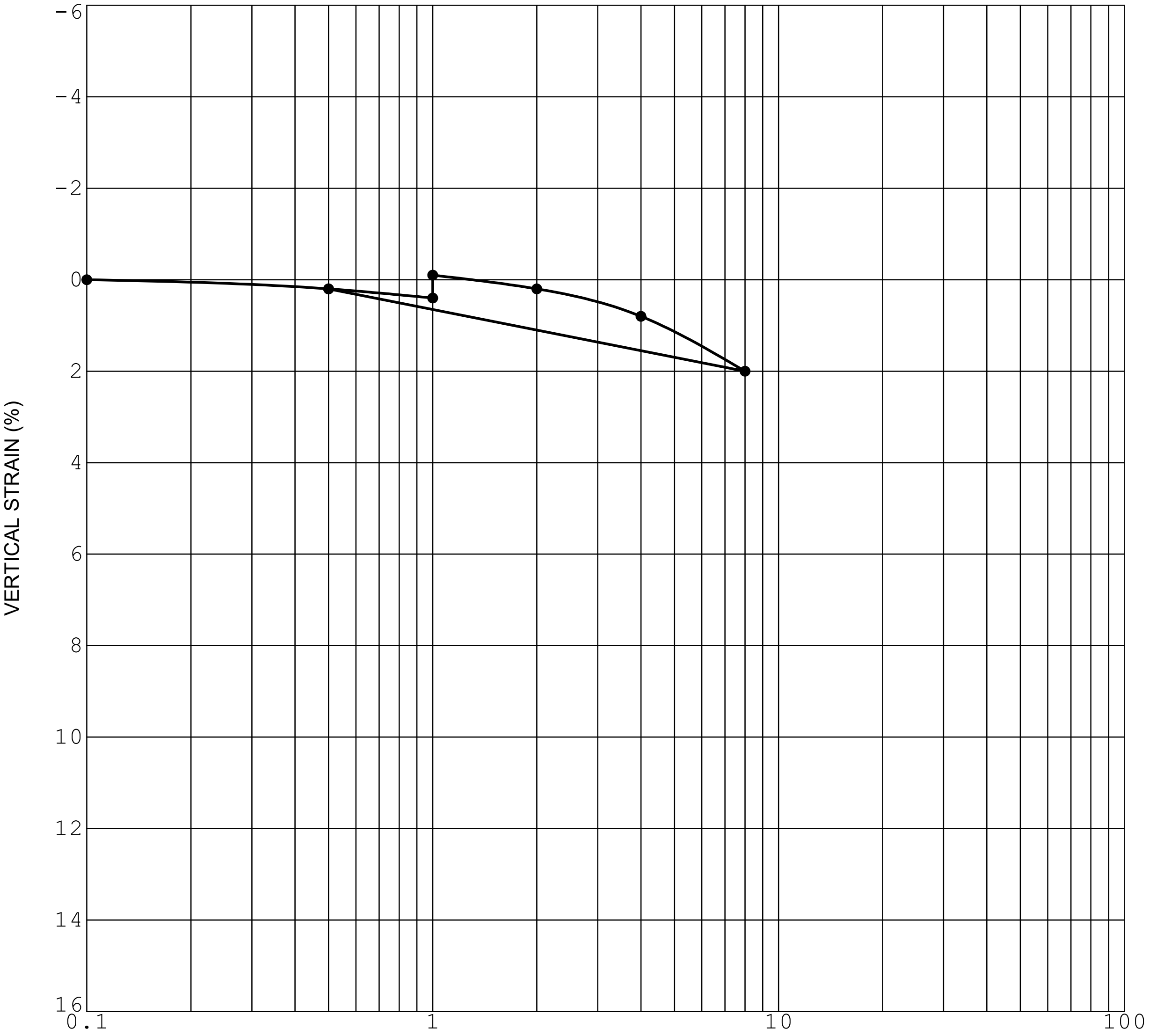
SAN DIEGO COUNTY, CALIFORNIA

G2074-32-01_UPD_2018-04-26.GPJ

Figure B-8

GEOCON

SAMPLE NO. B4-3



APPLIED PRESSURE (ksf)

ASTM D2435

Initial Dry Density (pcf)	115.6		**Initial Saturation (%)**	98.8
Initial Water Content (%)	16.2		**Sample Saturated at (ksf)**	1.0

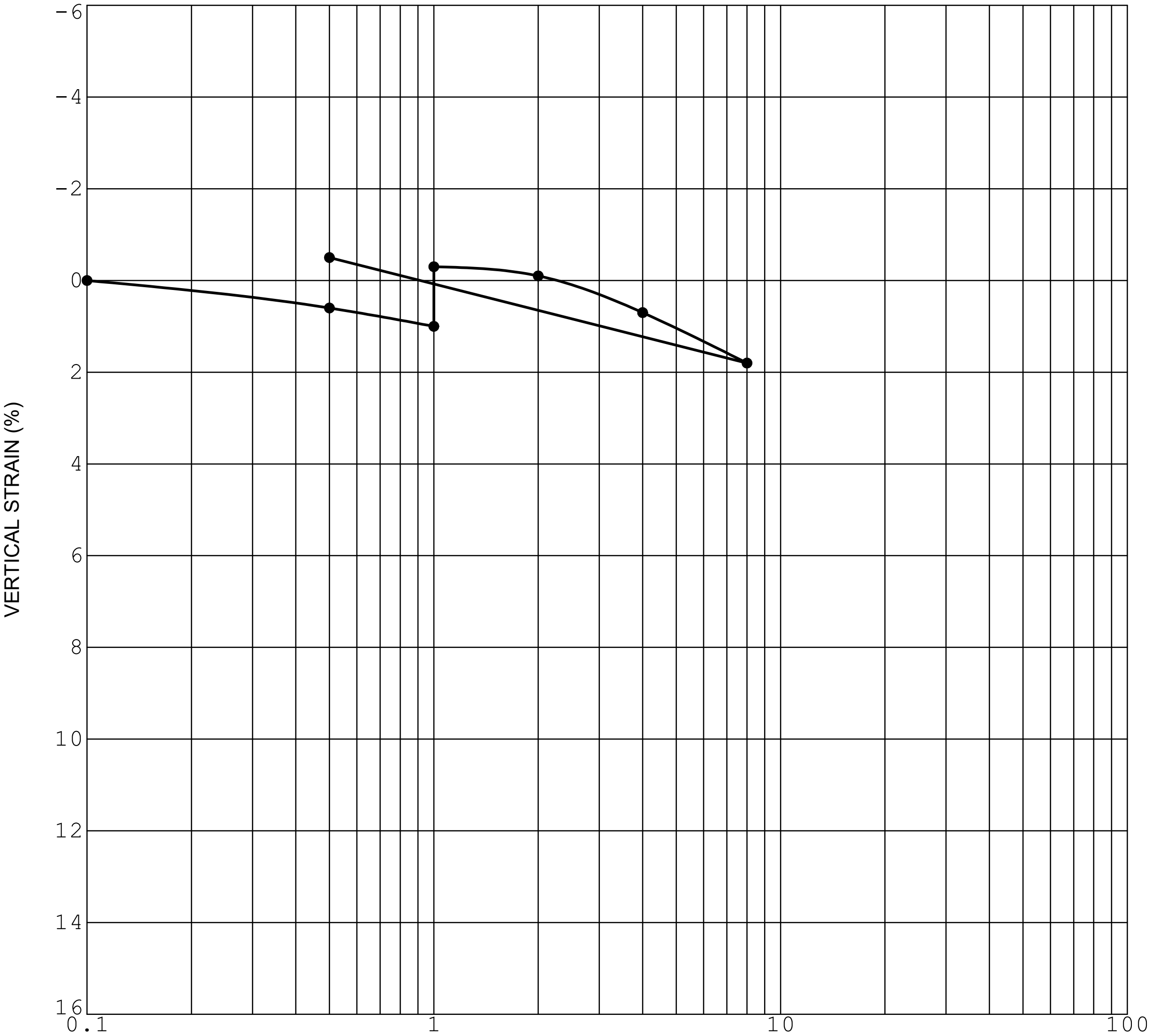
CONSOLIDATION CURVE

AVENTINE AT SWEETWATER SPRINGS

SAN DIEGO COUNTY, CALIFORNIA

PROJECT NO. G2074-32-01

SAMPLE NO. B4-4



ASTM D2435

Initial Dry Density (pcf)	108.5
Initial Water Content (%)	20.4

Initial Saturation (%)	100+
Sample Saturated at (ksf)	1.0

CONSOLIDATION CURVE

AVENTINE AT SWEETWATER SPRINGS

SAN DIEGO COUNTY, CALIFORNIA

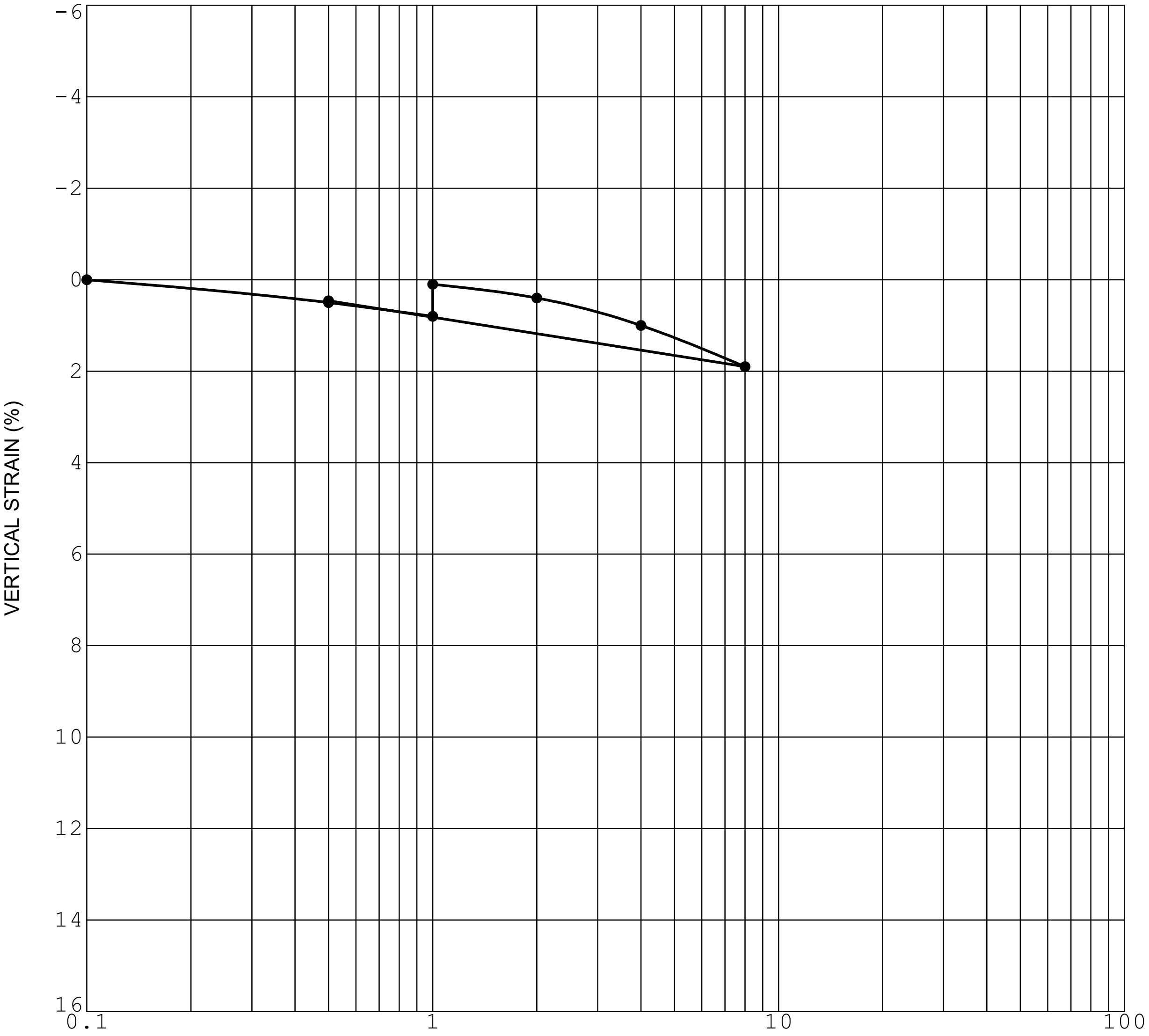
G2074-32-01_UPD_2018-04-26.GPJ

Figure B-10

GEOCON

PROJECT NO. G2074-32-01

SAMPLE NO. B4-5



APPLIED PRESSURE (ksf)

ASTM D2435

Initial Dry Density (pcf)	128.8
Initial Water Content (%)	13.0

Initial Saturation (%)	100+
Sample Saturated at (ksf)	1.0

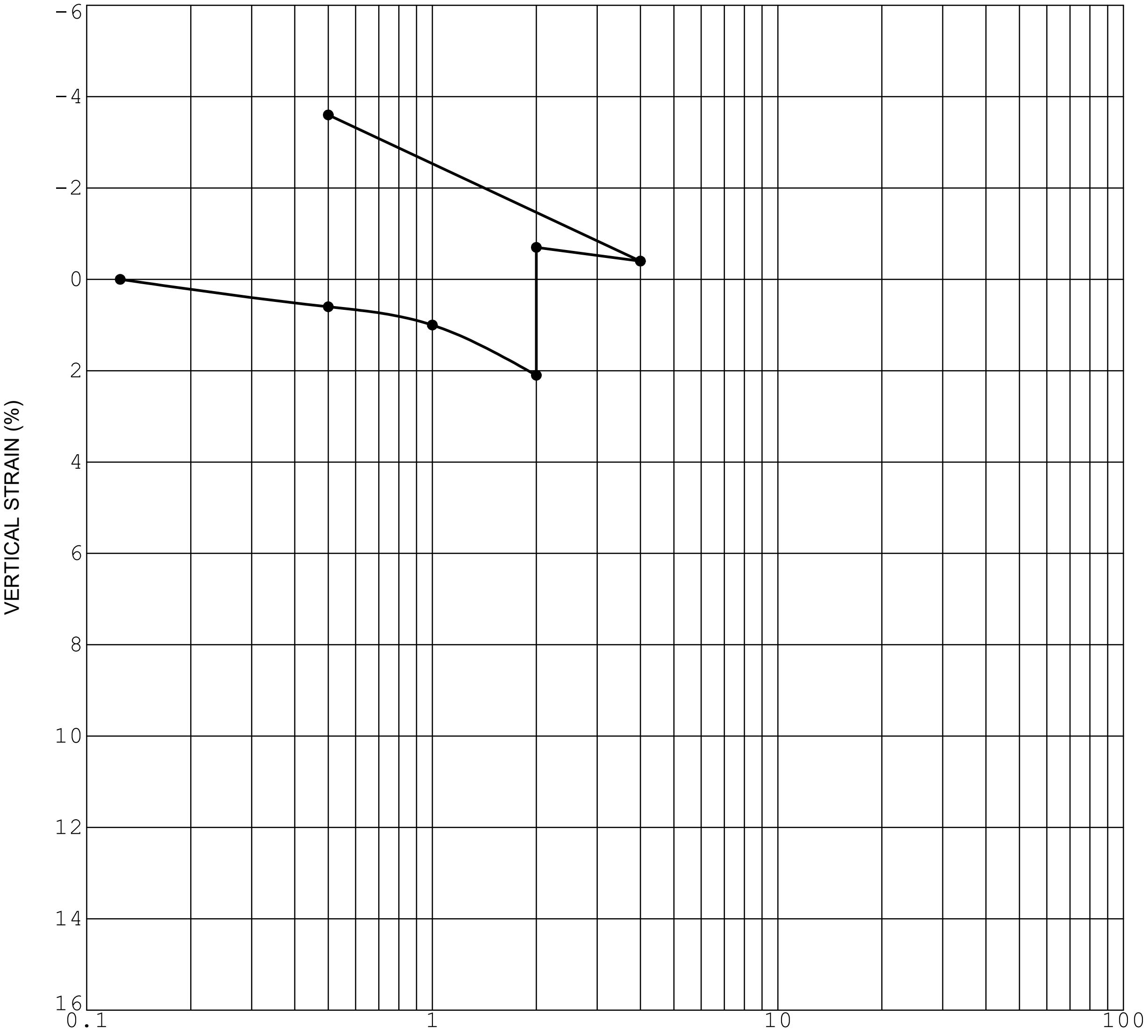
CONSOLIDATION CURVE

AVENTINE AT SWEETWATER SPRINGS

SAN DIEGO COUNTY, CALIFORNIA

PROJECT NO. G2074-32-01

SAMPLE NO. B7-5



ASTM D2435

Initial Dry Density (pcf)	106.4
Initial Water Content (%)	20.3

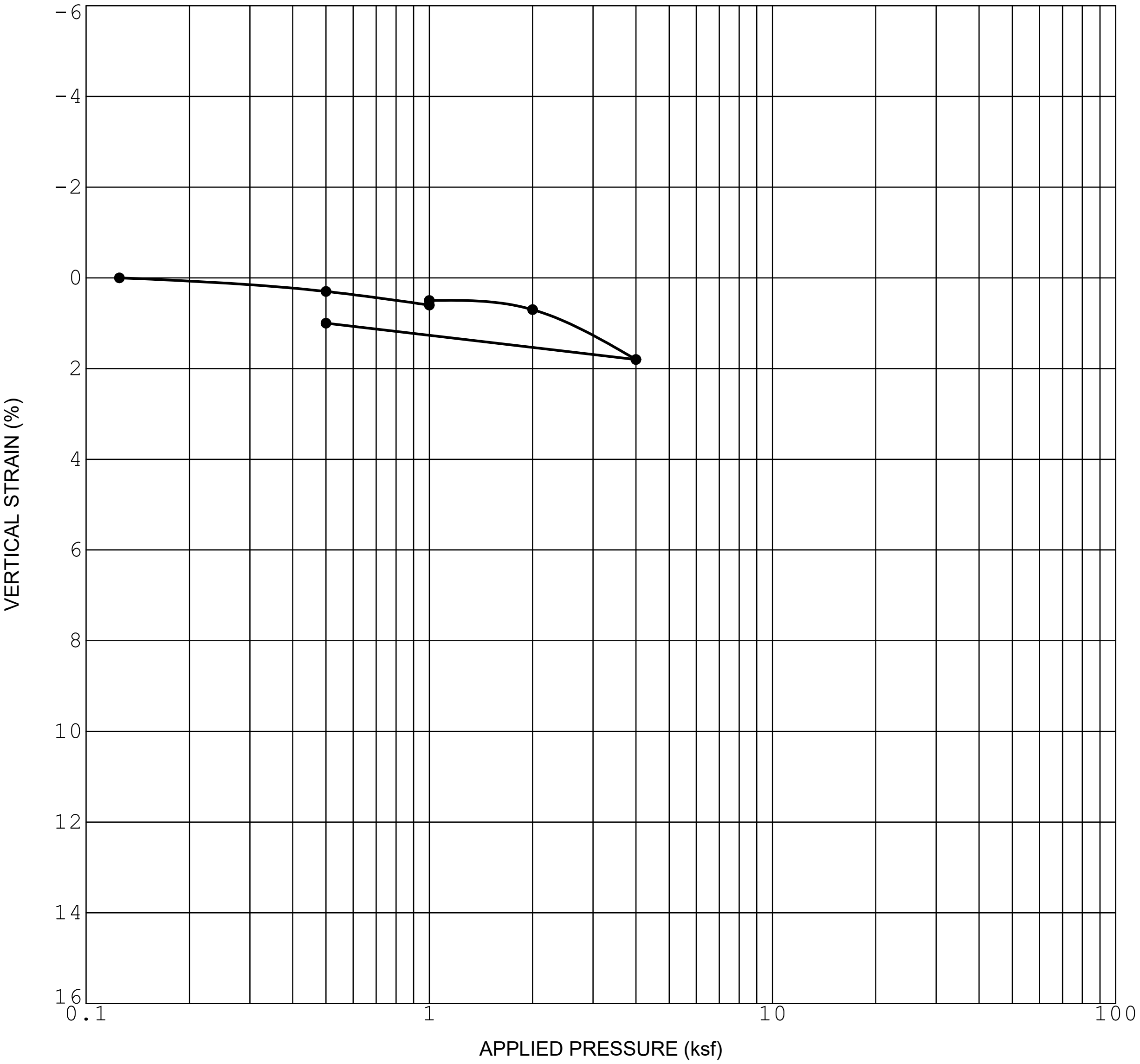
Initial Saturation (%)	96.5
Sample Saturated at (ksf)	2.0

CONSOLIDATION CURVE

AVENTINE AT SWEETWATER SPRINGS

SAN DIEGO COUNTY, CALIFORNIA

SAMPLE NO. B8-1



ASTM D2435

Initial Dry Density (pcf)	117.8
Initial Water Content (%)	12.9

Initial Saturation (%)	84.4
Sample Saturated at (ksf)	1.0

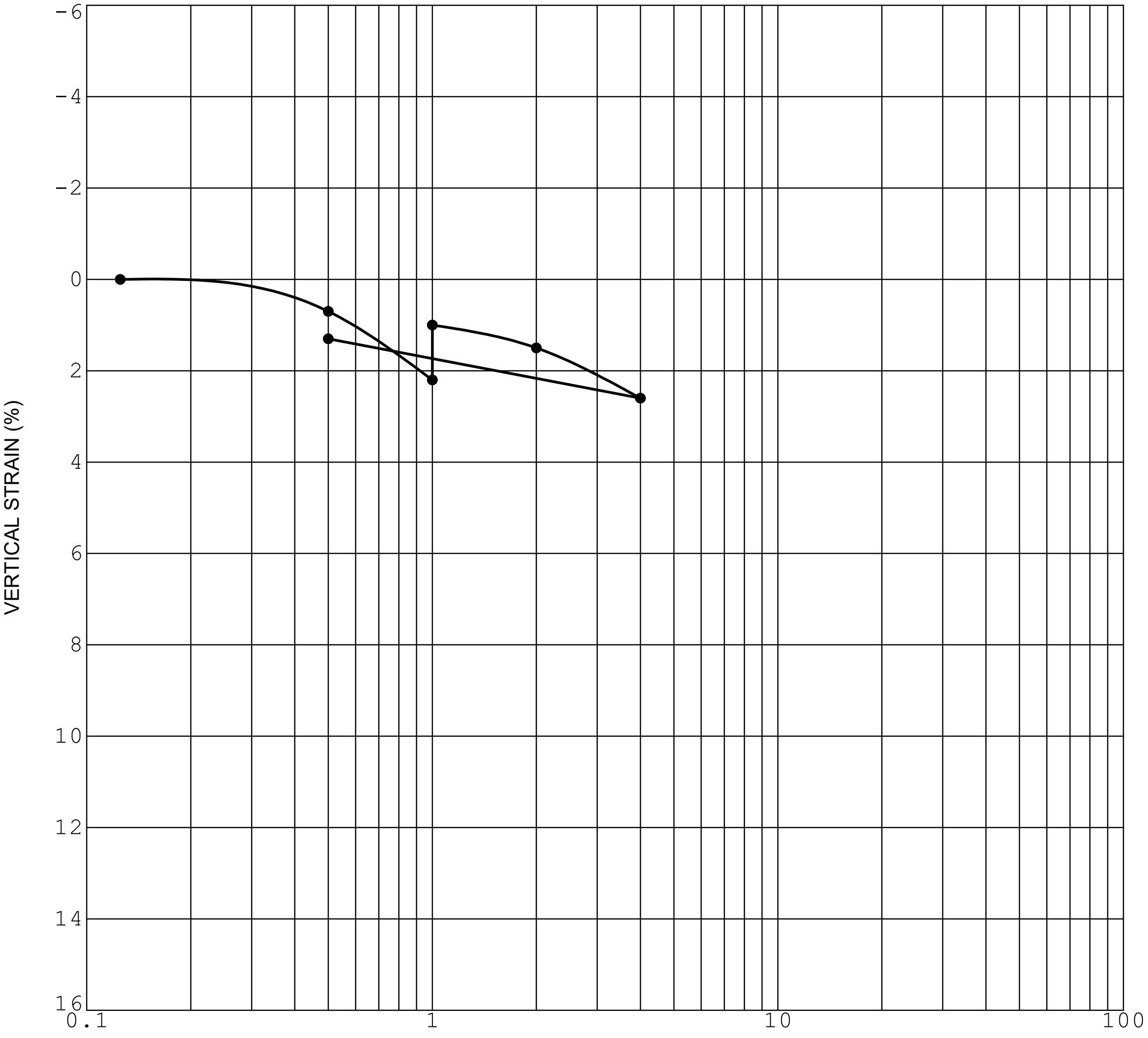
CONSOLIDATION CURVE

AVENTINE AT SWEETWATER SPRINGS

SAN DIEGO COUNTY, CALIFORNIA

PROJECT NO. G2074-32-01

SAMPLE NO. B9-5



APPLIED PRESSURE (ksf)

ASTM D2435

Initial Dry Density (pcf)	113.0
Initial Water Content (%)	15.6

Initial Saturation (%)	88.8
Sample Saturated at (ksf)	1.0

CONSOLIDATION CURVE

AVENTINE AT SWEETWATER SPRINGS

SAN DIEGO COUNTY, CALIFORNIA

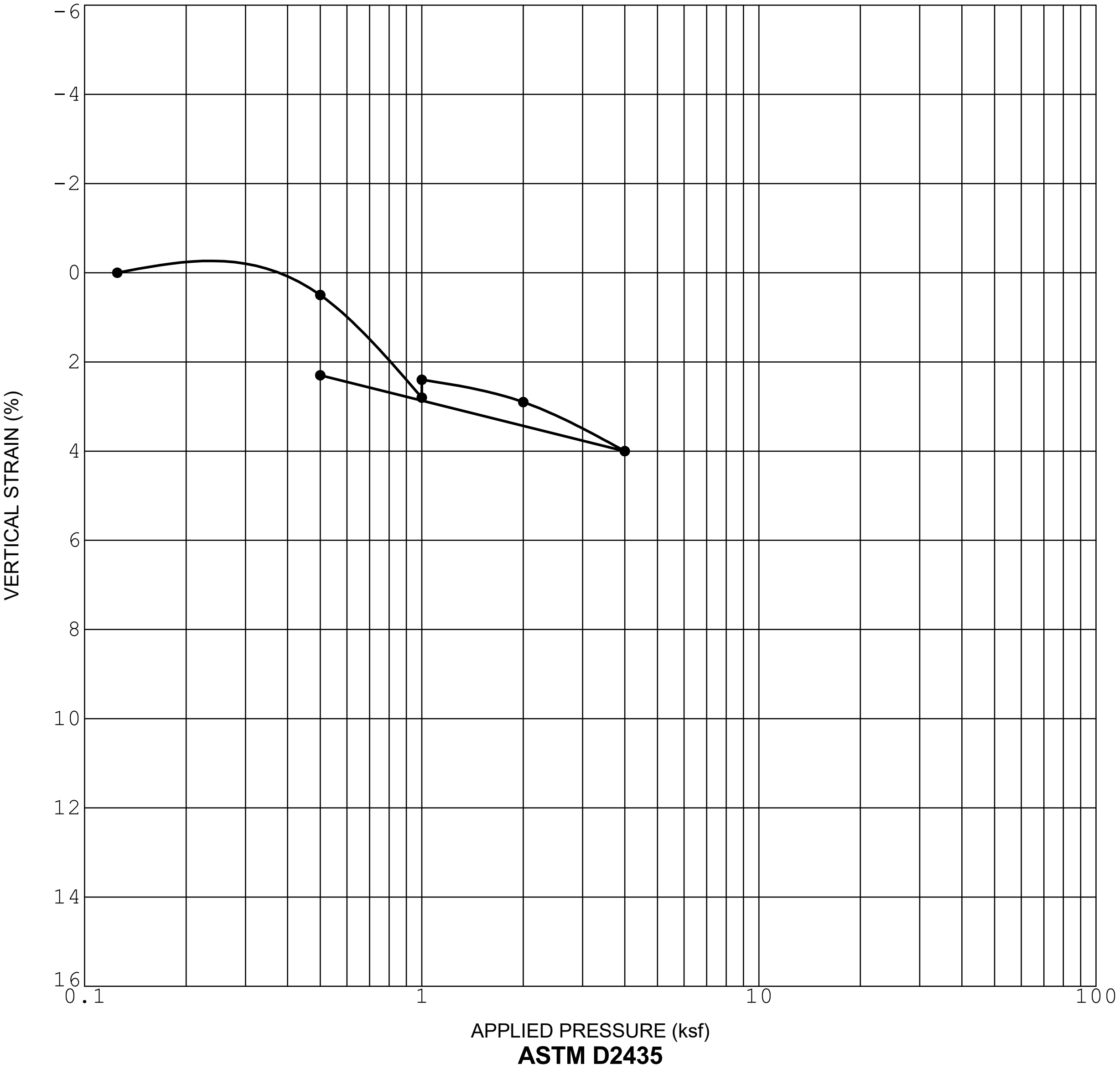
G2074-32-01_UPD_2018-04-26.GPJ

Figure B-14

GEOCON

PROJECT NO. G2074-32-01

SAMPLE NO. B10-2



ASTM D2435

Initial Dry Density (pcf)	96.4
Initial Water Content (%)	27.2

Initial Saturation (%)	100+
Sample Saturated at (ksf)	1.0

CONSOLIDATION CURVE

AVENTINE AT SWEETWATER SPRINGS

SAN DIEGO COUNTY, CALIFORNIA

APPENDIX C

APPENDIX C

STORM WATER MANAGEMENT INVESTIGATION

FOR

AVENTINE AT SWEETWATER SPRINGS
SAN DIEGO COUNTY, CALIFORNIA

PROJECT NO. G2074-32-01

APPENDIX C

STORM WATER MANAGEMENT INVESTIGATION

We understand storm water management devices are being proposed in accordance with the 2016 County of San Diego BMP Design Manual For Permanent Site Design, Storm Water Treatment and Hydromodification Management, 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 three units identified as Diablo Clay (DaC), Diablo Clay (DaE), and Diablo-Urban land complex (DcF). These units are classified as Soil Group D. 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)
Diablo Clay	DaC	51	D	0.06 – 0.20
Diablo Clay	DaE	7	D	0.06 – 0.20
Diablo-Urban Land Complex	DcF	42	D	0.06 – 0.20

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 2 Aardvark Permeameter Tests, I-1 and I-2, at locations shown on the attached Geologic Map, Figure 2. The test borings were 8 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 Form I-8. The results of the testing within the previously placed fill exposed in the proposed basin footprint indicate an adjusted soil infiltration rate of approximately 0.04 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)
I-1	Qpf	4.1	0.073	0.037
I-2	Qpf	2.4	0.088	0.044

[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, and the locations of the field excavations and the in-situ infiltration test locations.

Soil Types

Proposed Compacted Fill – Compacted fill has been placed across the entire property during previous site development. Proposed remedial grading will consist of removing the upper 5 feet of soil and replacement as compacted fill. The proposed storm water basin will be founded in previously placed fill over granitic rock. The fill soils beneath the basin are expected to be approximately 16 feet thick. The compacted fill is comprised of silty/clayey sand. The fill was 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 expected below the compacted fill is expected to be approximately 16 feet below proposed finish grades after remedial grading is performed. Full and partial infiltration should be considered geotechnically infeasible within the compacted fill and liners and subdrains should be used.

- *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 and partial infiltration should be considered geotechnically infeasible. Partial infiltration may be feasible if the infiltration BMP extends below the compacted fill, but that is considered unlikely due to the depth of fill and expected low permeability of the underlying granitic rock.

Infiltration Rates

The results of the two infiltration rates (including the feasibility factor of safety of 2) obtained within the proposed basin footprint were approximately 0.04 inches per hour (iph). Based on the results of the infiltration testing, these tests did not meet the minimum threshold for full infiltration; therefore, full infiltration is considered infeasible.

Groundwater Elevations

We did not encounter groundwater during our field exploration. Groundwater is not expected to be a geotechnical constraint. We expect to encounter groundwater at an elevation of approximately 110 feet above Mean Sea Level (MSL), or approximately 380 feet below the ground surface.

Soil or Groundwater Contamination

Soil or groundwater contamination is not expected.

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 and commercial developments surround the property. Public streets are located immediately adjacent to the eastern and southern property boundaries. 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 proposed basin is situated adjacent to an existing 2:1 fill slope. Infiltration of storm water may result in slope instability and daylight water seepage.

Recommendations

Due to the relatively low infiltration rates obtained within the footprint of the proposed basin, potential for slope instability and daylight water seepage, 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 considered feasible if the infiltration is extended below the compacted fill, but this is considered unlikely due to the depth to encounter a suitable bearing surface (i.e. approximately 16 feet below the bottom of proposed basin). If partial infiltration was desired, 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 into the compacted fill. 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. The side liner should extend to the granitic rock.

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 I-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	2	0.50
Site Soil Variability	0.25	2	0.50
Depth to Groundwater/ Impervious Layer	0.25	1	0.25
Suitability Assessment Safety Factor, $S_A = \sum p$			2.0

[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

Form I-8

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?

<table>
<tr><th>Criteria</th><th>Screening Question</th><th>Yes</th><th>No</th></tr>
<tr><td>1</td><td>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.</td><td></td><td>X</td></tr>
<tr><td colspan="4">Provide basis: Based on results of permeability testing in two locations within the proposed basin footprint, the unfactored infiltration rate was measured to be approximately 0.073 and 0.088 inches/hour using a constant head borehole permeameter. If applying a feasibility factor of safety of 2.0, the infiltration rates would be 0.0365 iph and 0.044 iph, which are less than the required threshold value of 0.5 iph. The USDA web soil survey website indicates the underlying soils belong to Diablo Clay. Diablo clay is identified as Hydrologic Soil Group D. 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.
.</td></tr>
<tr><td>2</td><td>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.</td><td></td><td>X</td></tr>
<tr><td colspan="4">Provide basis: A liquefaction potential is very low to negligible, and the landslide potential is very low to negligible. Existing utilities are present along the perimeter public roadways within the right of ways. The proposed basin is situated adjacent to an existing 2:1 fill slope. Infiltration of storm water may result in slope instability and daylight water seepage. Mitigation measures would be required to limit the adverse impacts of water infiltration, such as slope instability, daylight water seepage, and lateral water migration that may adversely impact on-site and adjacent foundations, roadways, and public and private improvements.</td></tr>
</table>

<table>
<tr><th colspan="4">Form I-8 Page 2 of 4</th></tr>
<tr><th>Criteria</th><th>Screening Question</th><th>Yes</th><th>No</th></tr>
<tr><td>3</td><td>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.</td><td>X</td><td></td></tr>
<tr><td colspan="4">Provide basis: Based on the USGS website, groundwater is expected to be encountered greater than 300 feet below the ground surface. 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>4</td><td>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.</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 1 Result *</td><td>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</td><td colspan="2">No Full 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 Agency/Jurisdictions to substantiate findings

Form I-8 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: The proposed basin will be founded on approximately 16 feet of compacted fill over granitic rock. The test results indicate poor infiltration rates. Saturating compacted fill should be avoided (see discussion in Appendix C). The adverse impacts of storm water infiltration could be reasonably mitigated to acceptable levels using side liners and subdrains, however it is considered infeasible in this case due to the depth to encounter a suitable infiltration surface (i.e. 16 feet below bottom of proposed basin). Saturation of the compacted fill should be avoided to prevent slope instability, daylight water seepage, settlement, and distress to adjacent structures and improvements.

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: The proposed basin is situated adjacent to an existing 2:1 fill slope. Infiltration of storm water may result in slope instability and daylight water seepage. Ground water mounding is not expected, no landslides are in the vicinity, and utility impacts could be reasonably mitigated using side liners to prevent lateral water migration. We do not recommend saturating the compacted fill. Any partial infiltration BMP should be extended below the compacted fill and into the underlying formational materials. However, partial infiltration is considered infeasible in this case due to the depth to encounter a suitable infiltration surface (i.e. 16 feet below the bottom of the proposed basin).

Form I-8 Page 4 of 4

<table>
<tr><th>Criteria</th><th>Screening Question</th><th>Yes</th><th>No</th></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 or contributing to the flow of contaminated surface waters into the groundwater table 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: Geocon is not aware of any downstream water rights that would be affected by incidental infiltration of storm water. Researching downstream water rights is beyond the scope of the geotechnical consultant.</td></tr>
<tr><td>Part 2 Result*</td><td>If all answers from row 5-8 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 colspan="2">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 Agency/Jurisdictions to substantiate findings

Aardvark Permeameter Data Analysis

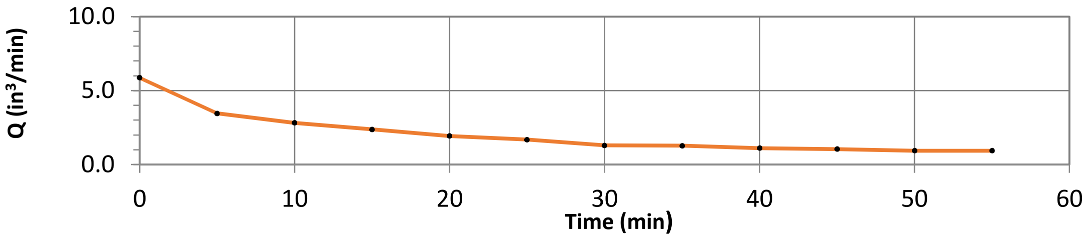
Project Name: Sweetwater Springs
Project Number: G2074-32-01
Test Number: P-1
Date: 12/7/2017
By: JML

Borehole Diameter, d (in.):	8.00
Borehole Depth, **H** (in):	49.00
Distance Between Reservoir & Top of Borehole (in.)	29.00
Estimated Depth to Water Table, **S** (feet):	100.00
Height APM Raised from Bottom (in.):	1.00
Pressure Reducer Used:	No

Ref. EL (feet, MSL): 482.0
Bottom EL (feet, MSL): 477.9

Distance Between Resevoir and APM Float, **D** (in.):	69.75
Head Height Calculated, **h** (in.):	4.73
Head Height Measured, **h** (in.):	11.25
Distance Between Constant Head and Water Table, **L** (in.):	1162.25

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	1.060	29.35	5.871
3	5.00	0.625	17.31	3.462
4	5.00	0.510	14.12	2.825
5	5.00	0.430	11.91	2.382
6	5.00	0.350	9.69	1.938
7	5.00	0.305	8.45	1.689
8	5.00	0.235	6.51	1.302
9	5.00	0.230	6.37	1.274
10	5.00	0.200	5.54	1.108
11	5.00	0.190	5.26	1.052
12	5.00	0.170	4.71	0.942
13	5.00	0.170	4.71	0.942
			Steady Flow Rate, Q (in^3/min):	0.978



Soil Matric Flux Potential, Φ_m

Φ_m= 0.00132 in^2/min

Field-Saturated Hydraulic Conductivity (Infiltration Rate)

K_{sat} = 1.21E-03 in/min 0.073 in/hr

GEOCON

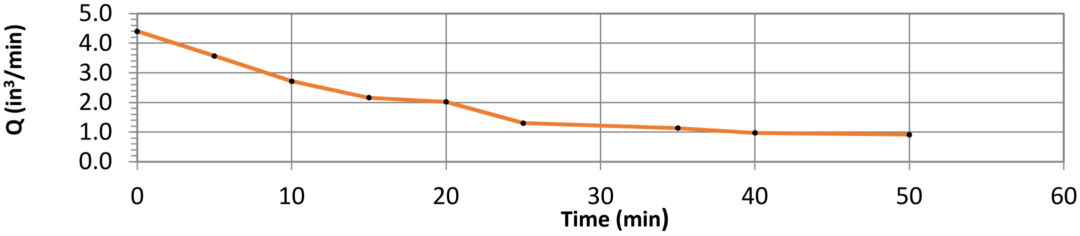
Aardvark Permeameter Data Analysis

Project Name: Sweetwater Springs
Project Number: G2074-32-01
Test Number: P-2
Date: 12/7/2017
By: JML
Ref. EL (feet, MSL): 480.0
Bottom EL (feet, MSL): 477.6

Borehole Diameter, d (in.): 8.00
Borehole Depth, **H** (in): 29.00
Distance Between Reservoir & Top of Borehole (in.) 29.50
Estimated Depth to Water Table, **S** (feet): 100.00
Height APM Raised from Bottom (in.): 1.00
Pressure Reducer Used: No

Distance Between Resevoir and APM Float, **D** (in.): 50.25
Head Height Calculated, **h** (in.): 4.67
Head Height Measured, **h** (in.): 9.00
Distance Between Constant Head and Water Table, **L** (in.): 1180.00

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	0.795	22.02	4.403
3	5.00	0.645	17.86	3.572
4	5.00	0.490	13.57	2.714
5	5.00	0.390	10.80	2.160
6	5.00	0.365	10.11	2.022
7	10.00	0.470	13.02	1.302
8	5.00	0.205	5.68	1.135
9	10.00	0.350	9.69	0.969
10	5.00	0.165	4.57	0.914
11	5.00	0.160	4.43	0.886
			Steady Flow Rate, Q (in^3/min):	0.923



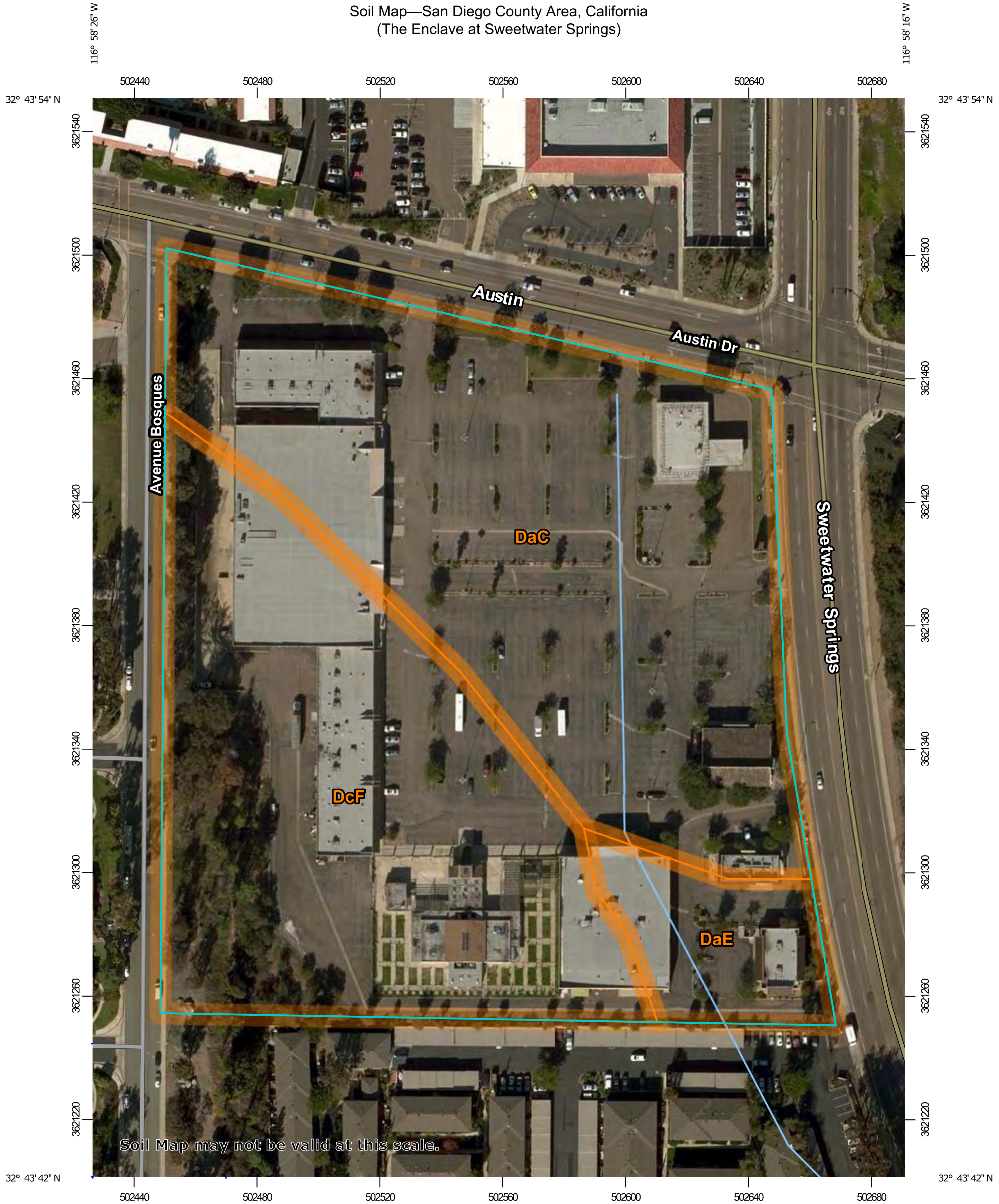
Soil Matric Flux Potential, ϕ_m

ϕ_m= 0.0016 in^2/min

Field-Saturated Hydraulic Conductivity (Infiltration Rate)

K_{sat} = 1.47E-03 in/min 0.088 in/hr

Soil Map—San Diego County Area, California
(The Enclave at Sweetwater Springs)



Map Scale: 1:1,710 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
DaC	Diablo clay, 2 to 9 percent slopes	5.9	51.0%
DaE	Diablo clay, 15 to 30 percent slopes	0.8	7.2%
DcF	Diablo-Urban land complex, 15 to 50 percent slopes	4.8	41.8%
Totals for Area of Interest		**11.5**	**100.0%**

San Diego County Area, California

DaC—Diablo clay, 2 to 9 percent slopes

Map Unit Setting

National map unit symbol: hbb8
Elevation: 30 to 3,000 feet
Mean annual precipitation: 12 to 35 inches
Mean annual air temperature: 57 to 61 degrees F
Frost-free period: 200 to 320 days
Farmland classification: Farmland of statewide importance

Map Unit Composition

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

Description of Diablo

Setting

Landform: Hillslopes
Landform position (two-dimensional): Backslope
Landform position (three-dimensional): Side slope
Down-slope shape: Convex
Across-slope shape: Convex
Parent material: Calcareous sandstone and shale

Typical profile

H1 - 0 to 15 inches: clay
H2 - 15 to 32 inches: clay, silty clay loam
H2 - 15 to 32 inches: weathered bedrock
H3 - 32 to 36 inches:

Properties and qualities

Slope: 2 to 9 percent
Depth to restrictive feature: 24 to 40 inches to paralithic bedrock
Natural drainage class: Well drained
Runoff class: Very high
Capacity of the most limiting layer to transmit water (Ksat): Moderately low to moderately high (0.06 to 0.20 in/hr)
Depth to water table: More than 80 inches
Frequency of flooding: None
Frequency of ponding: None
Calcium carbonate, maximum in profile: 10 percent
Available water storage in profile: Moderate (about 7.7 inches)

Interpretive groups

Land capability classification (irrigated): 3e
Land capability classification (nonirrigated): 3e
Hydrologic Soil Group: D
Hydric soil rating: No

Minor Components

Altamont

Percent of map unit: 10 percent
Hydric soil rating: No

Linne

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

Olivenhain

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

Data Source Information

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

San Diego County Area, California

DaE—Diablo clay, 15 to 30 percent slopes

Map Unit Setting

National map unit symbol: hbbb
Elevation: 200 to 3,250 feet
Mean annual precipitation: 9 to 25 inches
Mean annual air temperature: 59 to 63 degrees F
Frost-free period: 200 to 310 days
Farmland classification: Not prime farmland

Map Unit Composition

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

Description of Diablo

Setting

Landform: Hillslopes
Landform position (two-dimensional): Backslope
Landform position (three-dimensional): Side slope
Down-slope shape: Convex
Across-slope shape: Convex
Parent material: Calcareous sandstone and shale

Typical profile

H1 - 0 to 15 inches: clay
H2 - 15 to 32 inches: clay, silty clay loam
H2 - 15 to 32 inches: weathered bedrock
H3 - 32 to 36 inches:

Properties and qualities

Slope: 15 to 30 percent
Depth to restrictive feature: 24 to 40 inches to paralithic bedrock
Natural drainage class: Well drained
Runoff class: Very high
Capacity of the most limiting layer to transmit water (Ksat): Moderately low to moderately high (0.06 to 0.20 in/hr)
Depth to water table: More than 80 inches
Frequency of flooding: None
Frequency of ponding: None
Calcium carbonate, maximum in profile: 10 percent
Available water storage in profile: Moderate (about 7.7 inches)

Interpretive groups

Land capability classification (irrigated): 4e
Land capability classification (nonirrigated): 4e
Hydrologic Soil Group: D
Ecological site: CLAYEY (1975) (R019XD001CA)

Hydric soil rating: No

Minor Components

Altamont
Percent of map unit: 10 percent
Hydric soil rating: No

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

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

Data Source Information

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

San Diego County Area, California

DcF—Diablo-Urban land complex, 15 to 50 percent slopes

Map Unit Setting

National map unit symbol: hbbg
Elevation: 200 to 3,250 feet
Mean annual precipitation: 9 to 25 inches
Mean annual air temperature: 59 to 63 degrees F
Frost-free period: 200 to 310 days
Farmland classification: Not prime farmland

Map Unit Composition

Diablo and similar soils: 50 percent
Urban land: 30 percent
Estimates are based on observations, descriptions, and transects of the mapunit.

Description of Diablo

Setting

Landform: Hillslopes
Landform position (two-dimensional): Backslope
Landform position (three-dimensional): Side slope
Down-slope shape: Convex
Across-slope shape: Convex
Parent material: Calcareous sandstone and shale

Typical profile

H1 - 0 to 15 inches: clay
H2 - 15 to 32 inches: clay, silty clay loam
H2 - 15 to 32 inches: weathered bedrock
H3 - 32 to 36 inches:

Properties and qualities

Slope: 15 to 45 percent
Depth to restrictive feature: 24 to 40 inches to paralithic bedrock
Natural drainage class: Well drained
Runoff class: Very high
Capacity of the most limiting layer to transmit water (Ksat): Moderately low to moderately high (0.06 to 0.20 in/hr)
Depth to water table: More than 80 inches
Frequency of flooding: None
Frequency of ponding: None
Calcium carbonate, maximum in profile: 10 percent
Available water storage in profile: Moderate (about 7.7 inches)

Interpretive groups

Land capability classification (irrigated): None specified
Land capability classification (nonirrigated): 6e
Hydrologic Soil Group: D
Hydric soil rating: No

Description of Urban Land

Typical profile

H1 - 0 to 6 inches: variable

Interpretive groups

Land capability classification (irrigated): None specified
Land capability classification (nonirrigated): 8
Hydric soil rating: No

Data Source Information

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

APPENDIX D

APPENDIX D

RECOMMENDED GRADING SPECIFICATIONS

FOR

AVENTINE AT SWEETWATER SPRINGS
SAN DIEGO COUNTY, CALIFORNIA

PROJECT NO. G2074-32-01

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

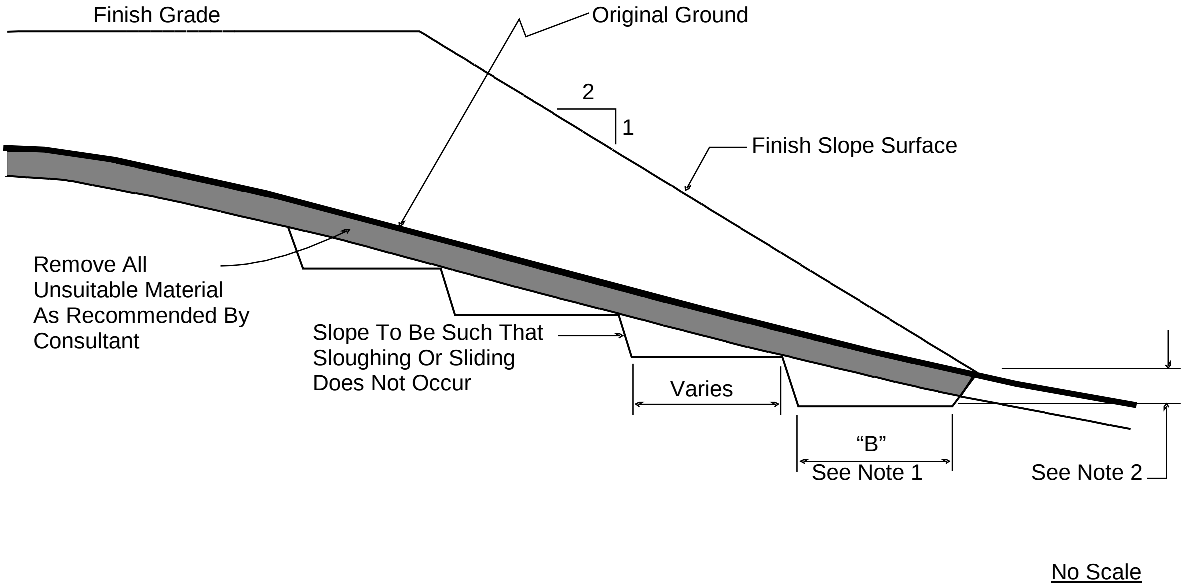
4.1 Areas to be excavated and filled shall be cleared and grubbed. Clearing shall consist of complete removal above the ground surface of trees, stumps, brush, vegetation, man-made structures, and similar debris. Grubbing shall consist of removal of stumps, roots, buried logs and other unsuitable material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

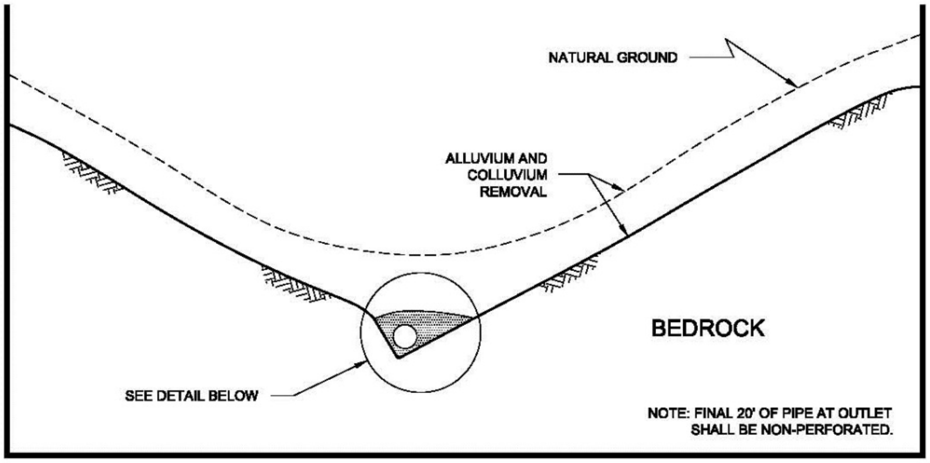
6.3.6 To reduce the potential for "piping" of fines into the *rock* fill from overlying *soil* fill material, a 2-foot layer of graded filter material shall be placed above the uppermost lift of *rock* fill. The need to place graded filter material below the *rock* should be determined by the Consultant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

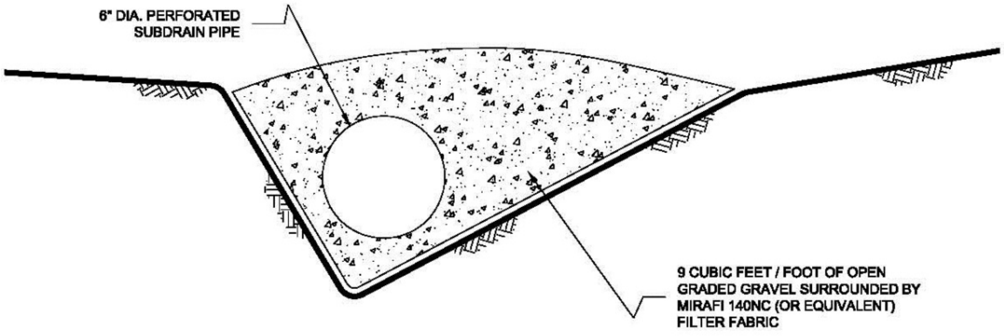
6.3.7 *Rock* fill placement should be continuously observed during placement by the Consultant.

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:

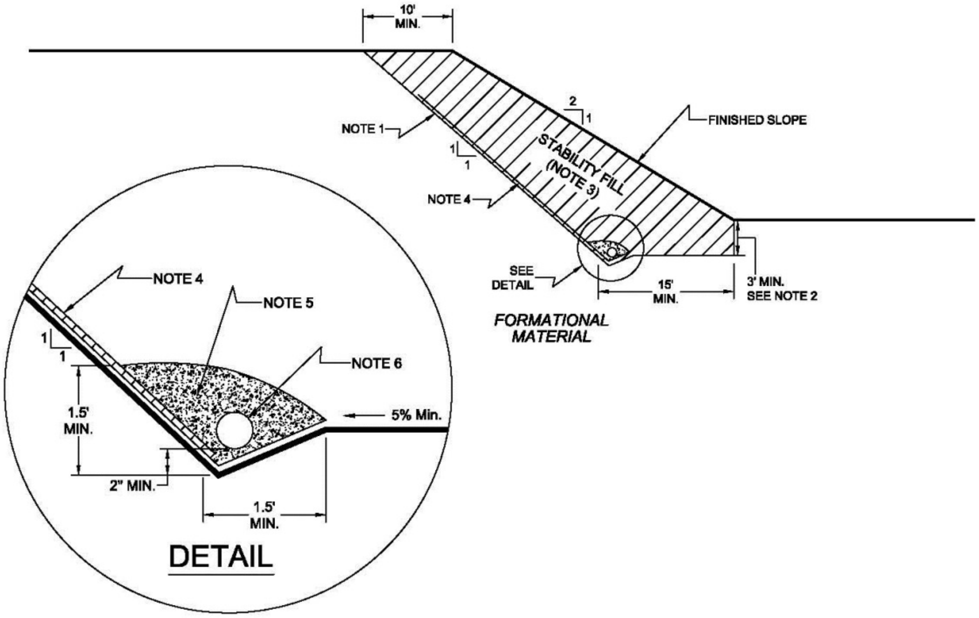
1......8-INCH DIAMETER, SCHEDULE 80 PVC PERFORATED PIPE FOR FILLS IN EXCESS OF 100-FEET IN DEPTH OR A PIPE LENGTH OF LONGER THAN 500 FEET.

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

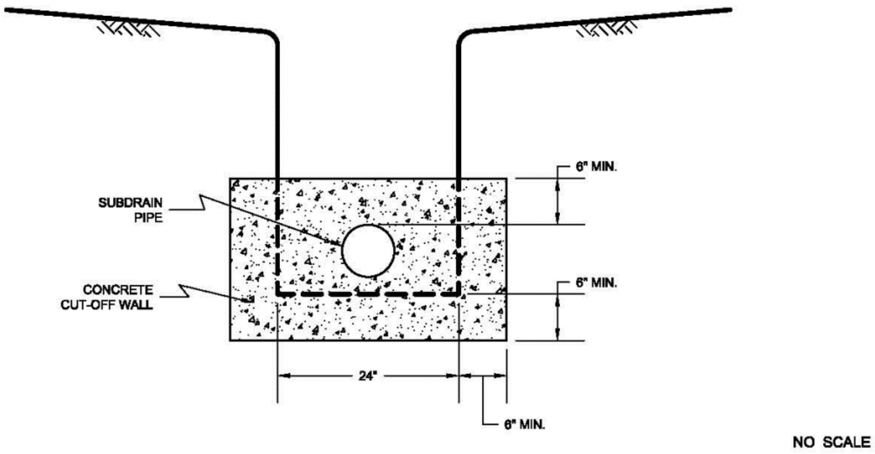
7.3 The actual subdrain locations will be evaluated in the field during the remedial grading operations. Additional drains may be necessary depending on the conditions observed and the requirements of the local regulatory agencies. Appropriate subdrain outlets should be evaluated prior to finalizing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

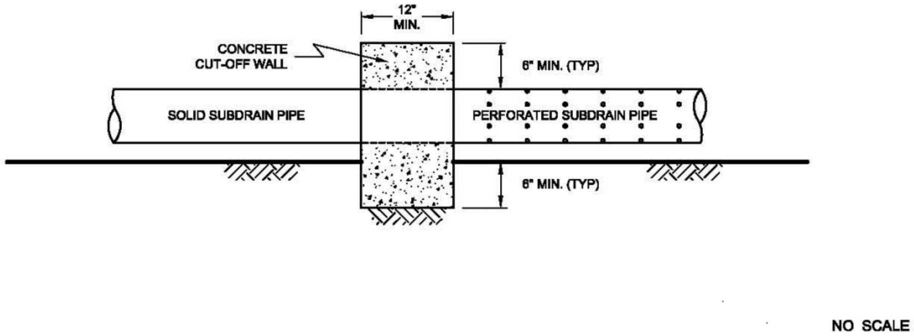
7.5 Prior to outletting, the final 20-foot segment of a subdrain that will not be extended during future development should consist of non-perforated drainpipe. At the non-perforated/ perforated interface, a seepage cutoff wall should be constructed on the downslope side of the pipe.

TYPICAL CUT OFF WALL DETAIL

FRONT VIEW



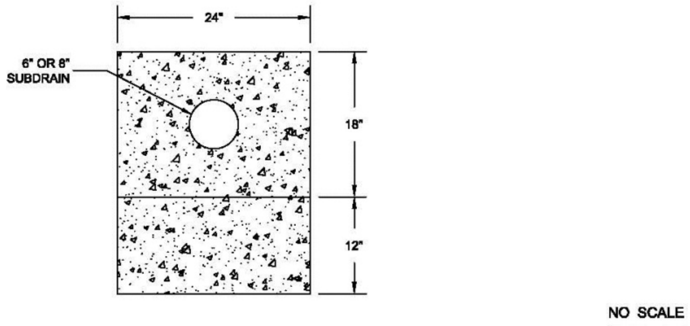
SIDE VIEW



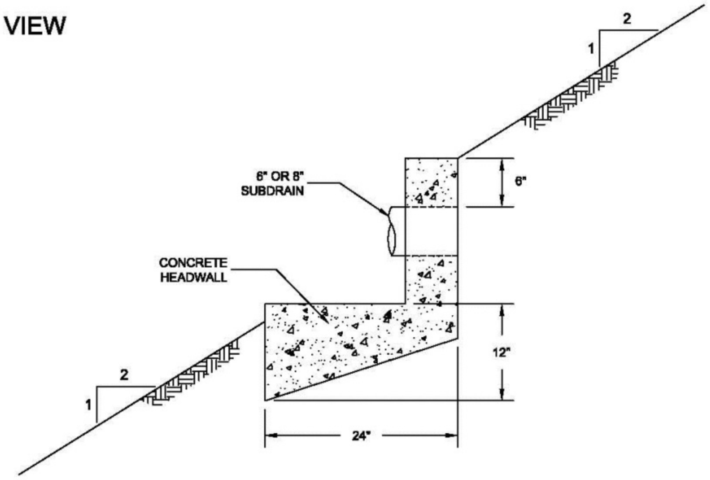
7.6 Subdrains that discharge into a natural drainage course or open space area should be provided with a permanent headwall structure.

TYPICAL HEADWALL DETAIL

FRONT VIEW



SIDE VIEW



NOTE: HEADWALL SHOULD OUTLET AT TOE OF FILL SLOPE
OR INTO CONTROLLED SURFACE DRAINAGE

NO SCALE

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

1. Boore, D. M., and G. M. Atkinson (2007), *Boore-Atkinson NGA Ground Motion Relations for the Geometric Mean Horizontal Component of Peak and Spectral Ground Motion Parameters*, Report Number PEER 2007/01.

2. Chiou, B. S. J., and R. R. Youngs (2008), *A NGA Model for the Average Horizontal Component of Peak Ground Motion and Response Spectra*, preprint for article to be published in NGA Special Edition for Earthquake Spectra.

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).

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

5. California Department of Water Resources, Water Data Library. http://www.water.ca.gov/waterdatalibrary.

6. Campbell, K. W., Y. Bozorgnia (2008), *NGA Ground Motion Model for the Geometric Mean Horizontal Component of PGA, PGV, PGD and 5% Damped Linear Elastic Response Spectra for Periods Ranging from 0.01 to 10 s*, Earthquake Spectra, Volume 24, Issue 1, pages 139-171, February 2008.

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

8. http://www.historicaerials.com.

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

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

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

12. 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/.

13. USGS computer program, *Seismic Hazard Curves and Uniform Hazard Response Spectra.*