GEOTECHNICAL INVESTIGATION

CAMPUS POINTE COMPLEX
4110 AND 4161 CAMPUS POINT COURT
10260 AND 10290 CAMPUS POINT DRIVE
SAN DIEGO, CALIFORNIA

GEOCON
INCORPORATED

GEOTECHNICAL
ENVIRONMENTAL
MATERIALS

PREPARED FOR

ALEXANDRIA REAL ESTATE EQUITIES, INC.
SAN DIEGO, CALIFORNIA

SEPTEMBER 19, 2019
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PROJECT NO. G2415-52-01

GEOCON
INCORPORATED
GEOTECHNICAL ■ ENVIRONMENTAL ■ MATERIALS

Project No. G2415-52-01
September 19, 2019
Revised October 31, 2019

Alexandria Real Estate Equities, Inc.
10996 Torreyana Road, Suite 250
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Attention: Mr. Christopher Clement

Subject: GEOTECHNICAL INVESTIGATION
CAMPUS POINTE COMPLEX
4110 AND 4161 CAMPUS POINT COURT
10260 AND 1290 CAMPUS POINT DRIVE
SAN DIEGO, CALIFORNIA

Dear Mr. Clement:

In accordance with your request and authorization of our Proposal No. LG-19212 dated June 5, 2019, we herein submit the results of our geotechnical investigation for the subject project. We performed our investigation to evaluate the underlying soil and geologic conditions and potential geologic hazards, and to assist in the design of the proposed building and associated improvements.

The accompanying report presents the results of our study and conclusions and recommendations pertaining to geotechnical aspects of the proposed project. The site is suitable for the proposed buildings and improvements provided the recommendations of this report are incorporated into the design and construction of the planned project.

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

Very truly yours,

GEOCON INCORPORATED

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GEOTECHNICAL INVESTIGATION

1. PURPOSE AND SCOPE

This report presents the results of our geotechnical investigation for a new commercial development located within the Campus Point business park in the City of San Diego, California (see Vicinity Map, Figure 1). The purpose of the geotechnical investigation is to evaluate the surface and subsurface soil conditions and general site geology, and to identify geotechnical constraints that may affect development of the property including faulting, liquefaction and seismic shaking based on the 2016 CBC seismic design criteria. In addition, we provided recommendations for remedial grading, shallow and deep foundations, concrete slabs-on-grade, concrete flatwork, pavement and retaining walls. We reviewed the following plans and geotechnical documents in preparation of this report:

1. *Preliminary Grading Plan, Campus Pointe, San Diego, California*, prepared by Michael Baker International, undated.
2. *Geotechnical Investigation, 10260 Campus Point Drive, San Diego, California*, prepared by Geocon Incorporated, dated February 15, 2019 (Project No. G2345-52-02).
3. *Preliminary Geotechnical Investigation, 10290 Campus Pointe Drive, San Diego, California*, prepared by Geocon Incorporated, dated June 11, 2015 (Project No. 07850-42-15).
4. *2nd Addendum to Geotechnical Investigation, 10290 Campus Pointe Drive, San Diego, California*, prepared by Geocon Incorporated, dated March 15, 2016 (Project No. 07850-42-15).
5. *Preliminary Fault Study, 10290 Campus Pointe Drive, San Diego, California*, prepared by Geocon Incorporated, dated May 27, 2015 (Project No. 7850-42-15).
6. *Report of Preliminary Geotechnical Investigation, Qualcomm Office Building, Eli Lillie Property, Campus Point Drive, San Diego, California*, prepared by Southern California Soil & Testing, Inc., dated October 13, 1995 (Project No. 9511205).
7. *Report of Fault Investigation, Qualcomm Office Building, Eli Lillie Property, Campus Point Drive, San Diego, California*, prepared by Southern California Soil & Testing, Inc., dated December 1, 1995 (Project No. 9511205).
8. *Final Report of Engineering Observation of Grading and Testing of Compacted Fill, Campus Point Lots 2 and 3, San Diego, California, TM 78-337. W.O. No. 70918*, prepared by Woodward-Clyde Consultants, dated March 7, 1980.

The scope of this investigation included reviewing readily available published and unpublished geologic literature (see List of References), performing engineering analyses, and preparing this report. We also advanced 16 exploratory borings to a maximum depth of about 85 feet, performed percolation/infiltration testing, obtained soil samples and performed laboratory testing. Appendix A presents the exploratory boring logs and details of the field investigation. The details of the laboratory tests and a summary of the test results are shown in Appendix B and on the boring logs in Appendix A. Appendix C presents previous exploratory excavation and laboratory data from Geocon and others. Appendix D presents a summary of our storm water management investigation.

2. SITE AND PROJECT DESCRIPTION

The subject property is located west of Campus Point Drive, north of the Campus Point Court terminus and east of Interstate 5 in San Diego, California (see Vicinity Map, Figure 1). The subject property is part of the existing Campus Point business park and includes the buildings addressed 4110 Campus Point Court, 4161 Campus Point Court, 10260 Campus Point Drive and 10290 Campus Point Drive (APN 343-230-3800, -4300, -4200, and -1400, respectively). The subject property currently possesses the two commercial buildings of 2- and 7-stories along with a central plant, soccer field, paved surface parking and drive areas and other associated improvements as shown on the Existing Site Plan.

Existing Site Plan

The majority of the site is generally flat to slightly sloping with elevations ranging from 265 feet Mean Sea Level (MSL) in the southwestern portion of the site to 305 feet MSL in the northeastern portion of the site. The western portion of the property includes a descending 2:1 (horizontal to vertical) slope with maximum height of approximately 150 feet. Additionally, a soil nail wall with maximum height of 40 feet was recently constructed by Caltrans to the west of 4161 Campus Point Court. Access to the site is from Campus Point Court to the south or Campus Point Drive to the east.

Based on a preliminary site plan prepared by LPA Design Studio, we understand the Campus Pointe complex will be improved to include the structures presented in Table 2.1 and shown on the Geologic Map, Figure 2. The existing structures addressed as 4110 and 4161 Campus Point Court will be demolished to develop the proposed structures. The structures addresses as 10260 and 10290 Campus Point Drive will remain in-use.

TABLE 2.1
PROPOSED BUILDING SUMMARY

Building Designation	Location on Property	Building Summary
CP4	North	6 Story Office Building over 1 Level Subterranean Parking
CP5 (Leidos)	North	3 Story Office Building over 2 Levels Subterranean Parking
CP6	West	5 Story Office Building over 2 Levels Subterranean Parking
CP7	West-Central	8 Story Office Building over 2 Levels Subterranean Parking
A1 – A3	Northeast	1 to 2 Story Amenity Buildings At-Grade
A4 – A5	Southeast	1 Story Retail Buildings At-Grade
Parking Structure	East	5-Level Parking Garage over 2 Levels Subterranean

The locations, site descriptions and proposed development are based on our site reconnaissance, review of published geologic literature, field investigations, and discussions with project personnel. If development plans differ from those described herein, Geocon Incorporated should be contacted for review of the plans and possible revisions to this report.

3. PREVIOUS GRADING

Woodward-Clyde Consultants (WCC) performed a geotechnical investigation for the Campus Point development site in 1978. The development originally consisted of steep hillside topography with a prominent north trending ridge line sloping away to canyon drainages to the east and west. Elevations ranged from a high of about 350 feet above Mean Sea Level (MSL) on the southern portion of the development near Genesee Avenue to a low of 130 feet MSL in the bottom of canyon on the west side of the ridge. The general geologic conditions consisted of surficial soil composed of undocumented fill, topsoil, landslide debris and alluvium overlying formational materials of the Very Old Paralic Deposits (previously called the Lindavista Formation), Scripps Formation and Ardath Shale. The 1978 report identified that faulting was present within the development. The faulting was not considered active and would not impact site development. A landslide was identified within the limits of grading to the southeast with relatively shallow features. The landslide was likely removed and replaced with properly compacted fill. The existing slopes were determined to be stable in their current and graded configuration. Groundwater and seepage conditions were not observed during their field investigation.

Grading of the development occurred in 1979 which created large, sheet-graded pads with maximum cuts from natural grade of approximately 50 feet and fill of about 120 feet deep on the western portion of the overall development adjacent to Interstate 5. The scope of the grading also included the undercutting of highly expansive soil, removal of landslide debris, removal of undocumented fill along Genesee Avenue and the proper burial and compaction of oversize rock at least 20 feet below finish grades. WCC provided the testing and observation services during grading operations consisting of performing laboratory and compaction testing. The field density test results indicated that the fill soil was placed at a dry density of at least 90 percent of the laboratory maximum dry density.

Subsequent to the mass grading observed by WCC, Geocon Incorporated performed a supplemental geotechnical investigation in November 1980 to evaluate if landslide debris was present on the development after completion of mass grading operations. The scope of work included the excavation of 14 exploratory trenches and one large-diameter boring. The report indicated that landsliding was likely present to the east of Campus Point Drive, but it would likely not affect development of original Lots 2 and 3 (10260 Campus Point Drive). Geocon’s Boring B-1 (1980), just south of the existing building, encountered approximately 20 feet of fill that was likely placed during removal and replacement of previous surficial materials or a shallow landslide on the site.

4. GEOLOGIC SETTING

The project site is located within the Peninsular Ranges Geomorphic Province. The region is characterized by northwest-trending structural blocks and intervening fault zones. The rock types in the Peninsular Ranges include igneous intrusive rocks associated with the Cretaceous-age Southern California Batholith, intruded into older metavolcanic and/or metasedimentary units in western and central San Diego County. In the western part of the county and along the coastal areas, the basement rocks are overlain by a thick sequence of Cretaceous to Tertiary-age marine and non-marine sedimentary formations, which are the result of transgressive and regressive cycles of the sea. These deposits in turn are partially covered by several Quaternary-age terrace deposits that are geologically younger to the west.

5. SOIL AND GEOLOGIC CONDITIONS

We encountered two surficial soil units (consisting of previously placed fill and topsoil) and two formational units (consisting of Scripps Formation and Ardath Shale). The occurrence, distribution, and description of each unit encountered is shown on the Geologic Map, Figure 2 and on the boring logs in Appendix A. Figures 3 and 4 show geology and the approximate fill thickness from the existing and proposed grades, respectively. The Geologic Cross-Sections, Figure 5, show the approximate subsurface relationship between the geologic units. The surficial soil and geologic units are described herein in order of increasing age.

5.1 Previously Placed Fill (Qpf)

Previously placed fill is located across a majority of the property and we encountered the fill in our current geotechnical borings B-1 through B-9, B-11, B-13 and B-14. We expect the fill was placed during mass grading in 1979 to 1980 under the observation and compaction testing of Woodward-Clyde Consultants (WCC). We encountered the fill with a thickness of 90 feet; however, we expect a maximum thickness of about 110 feet. The Geologic Map, Figure 2, provides the approximate fill thickness contours for the site. We expect most of the long-term fill settlement has likely occurred since the fill was placed roughly 40 years ago. The fill was placed over the Scripps Formation and Ardath Shale, which has provided suitable support for the existing fill soil.

The fill consists of medium dense to dense, damp to moist, silty to clayey, fine to medium sand and sandy silt. Based on our laboratory tests the fill has a “very low” to “medium” expansion potential (expansion index [EI] of 90 or less). The upper portion of the previously placed fill is not considered suitable for the proposed improvements and remedial grading will be required.

5.2 Topsoil (Qt – Unmapped)

We encountered topsoil in Boring B-8 below the fill and above the Ardath Shale. The topsoil is about 5 feet thick and consists of dark gray to black, sandy to silty clay. The topsoil was likely left in place during the original grading operations and is very limited in area. We do not expect we will encounter topsoil during the construction operations.

5.3 Scripps Formation (Tsc)

Tertiary-age Scripps Formation exists below the fill in Borings B-1, B-2, B-5 through B-7 and B-11 through B-16. The Scripps Formation is generally brown, yellowish brown to light gray, silty to clayey sandstone and sandy siltstone/claystone with layers of strongly cemented material. Our laboratory tests and experience indicate the Scripps Formation possesses a “very low” to “medium” expansion potential (expansion index of 90 or less). The Scripps Formation is generally considered suitable for support of properly compacted structural fill and improvements.

5.4 Ardath Shale (Ta)

We encountered the Tertiary-age Ardath Formation below the fill in Borings B-8 through B-10 and below the Scripps Formation in Boring B-11. The Tertiary-age formation typically consists of olive-gray and yellowish brown, sandy to clayey siltstone. The upper portion may contain thin beds of medium-grained sandstone similar to the overlying Scripps Formation (Kennedy and Tan, 2008). The Ardath Shale possesses areas of highly cemented concretionary beds. The Ardath Shale is generally considered suitable for support of properly compacted structural fill and improvements.

6. GROUNDWATER

We did not encounter groundwater during our site investigation. However, we did encounter minor seepage within the fill materials in Boring B-15. It is not uncommon for shallow seepage conditions to develop where none previously existed when sites are irrigated or infiltration is implemented. Seepage is dependent on seasonal precipitation, irrigation, land use, among other factors, and varies as a result. During the rainy season, seepage conditions may develop that would require special consideration. Proper surface drainage will be important to future performance of the project. We expect groundwater is deeper than about 200 feet below existing grade. We do not expect groundwater to be encountered during construction of the proposed development or adversely impact future construction and performance of the existing building.

7. GEOLOGIC HAZARDS

7.1 Geologic Hazard Category

The City of San Diego Seismic Safety Study, Geologic Hazards and Faults, Map Sheet 34 defines the site as a Hazard Category 52: *Other Level Areas, gently sloping to steep terrain, favorable geologic structure, Low Risk* and Hazard Category 25*: Ardath - Neutral or favorable geologic structure.* Two east-west trending faults are mapped to cross the southern and central portion of the subject site and are mapped within an area defined as Hazard Category 12: *Fault Zones – Potentially Active, Inactive, Presumed Inactive, or Activity Unknown.* Figure 6 presents the San Diego Seismic Safety Study map for the site.

7.2 Faulting

The site is not located within a State of California Earthquake Special Study Zone; however, based on published geologic literature (Kennedy and Tan, 2008) and the City of San Diego Seismic Safety Study (City of San Diego, 2008), the east-west trending, Salk Fault crosses the property. The Salk Fault is described as a down-to-the-south, normal fault juxtaposing the Tertiary-age Scripps Formation against the older Ardath Formation leaving the overlying Pleistocene-age Very Old Paralic Deposits un-deformed and is categorized as potentially active, inactive, presumed inactive, or activity unknown

(City of San Diego, 2008). The Regional Geologic Map, Figure 7, shows the mapped limits of the geologic units at the site.

The Pleistocene-age Very Old Paralic Deposits Unit 10, which correlates to the Tecolote Geologic Terrace, deposited roughly 800,000 years ago. Therefore, these faults are not considered active (indicating fault movement in the last 11,000 years) but rather classified as Potentially Active (movement of at least 11,000 years old but younger than 2 million years) and have not shown movement for at least 800,000 years.

Based on our review of previous fault studies performed on the property and the project plans, potentially active faults may traverse the proposed eastern ARE Central Building and Building CP5.

We performed the referenced Preliminary Fault Study (Geocon, 2015) for a site to the north (10290 Campus Point Drive) of the subject site that included review of previous fault studies and additional fault trenching. Our investigation concluded that previous grading at the site had removed the Quaternary deposits from the site making a direct determination of fault activity difficult; however, the east-west orientation of the observed faults indicates they are not part of the current tectonic setting. The minor displacements and poorly developed to non-existent fault gouge observed are indicative of low-risk fault rupture hazard.

Therefore, we opine, from a geotechnical standpoint, active faults do not cross the subject property and that the faulting identified at the site is at most potentially active and does not pose a risk of fault rupture hazard to the project. We opine setback zones are not required to mitigate fault rupture hazard.

7.3 Seismicity

According to the computer program *EZ-FRISK* (Version 7.65), 10 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. Based on this database, the nearest known active fault is the Newport-Inglewood Fault system, located approximately 3 miles southwest of the site, and is the dominant source of potential ground motion. Earthquakes that might occur on the Newport-Inglewood Fault or other faults within the southern California and northern Baja California area are potential generators of significant ground motion at the site. The estimated deterministic maximum earthquake magnitude and peak ground acceleration for the Newport-Inglewood Fault are 7.5 and 0.48g, respectively. Table 7.3.1 lists the estimated maximum earthquake magnitude and peak ground acceleration for the most dominant faults in relationship to the site location. We calculated peak ground acceleration (PGA) using Boore-Atkinson (2008) NGA USGS2008, Campbell-Bozorgnia (2008) NGA USGS 2008 and Chiou-Youngs (2007) NGA USGS2008 acceleration-attenuation relationships.

TABLE 7.3.1
DETERMINISTIC SPECTRA SITE PARAMETERS

Fault Name	Distance from Site (miles)	Maximum Earthquake Magnitude (Mw)	Peak Ground Acceleration		
			Boore-Atkinson 2008 (g)	Campbell-Bozorgnia 2008 (g)	Chiou-Youngs 2007 (g)
Newport-Inglewood	3	7.50	0.38	0.39	0.48
Rose Canyon	3	6.90	0.33	0.38	0.42
Coronado Bank	17	7.40	0.17	0.13	0.16
Palos Verdes Connected	17	7.70	0.20	0.15	0.19
Elsinore	34	7.85	0.13	0.09	0.11
Earthquake Valley	42	6.80	0.06	0.05	0.04
Palos Verdes	48	7.30	0.07	0.05	0.05

We used the computer program *EZ-FRISK* to perform a probabilistic seismic hazard analysis. The computer program *EZ-FRISK* 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 (2007) NGA USGS2008 in the analysis. Table 7.3.2 presents the site-specific probabilistic seismic hazard parameters including acceleration-attenuation relationships and the probability of exceedence.

TABLE 7.3.2
PROBABILISTIC SEISMIC HAZARD PARAMETERS

Probability of Exceedence	Peak Ground Acceleration		
	Boore-Atkinson, 2008 (g)	Campbell-Bozorgnia, 2008 (g)	Chiou-Youngs, 2007 (g)
2% in a 50 Year Period	0.47	0.50	0.56
5% in a 50 Year Period	0.31	0.32	0.35
10% in a 50 Year Period	0.22	0.22	0.23

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

7.4 Ground Rupture

Ground surface rupture occurs when movement along a fault is sufficient to cause a gap or rupture where the upper edge of the fault zone intersects the ground surface. The potential for ground rupture is considered to be very low due to the absence of active faults at the subject site.

7.5 Liquefaction

Liquefaction typically occurs when a site is located in a zone with seismic activity, onsite soils are cohesionless or silt/clay with low plasticity, groundwater is encountered within 50 feet of the surface and soil densities are less than about 70 percent of the maximum dry densities. If the four previous criteria are met, a seismic event could result in a rapid pore water pressure increase from the earthquake-generated ground accelerations. Due to the lack of a permanent, near-surface groundwater table and the very dense nature of the underlying fill and formational materials, liquefaction potential for the site is considered very low.

7.6 Storm Surge, Tsunamis, and Seiches

Storm surges are large ocean waves that sweep across coastal areas when storms make landfall. Storm surges can cause inundation, severe erosion and backwater flooding along the water front. The site is located approximately 1½ miles from the Pacific Ocean and is at an elevation of about 265 feet or greater above Mean Sea Level (MSL). Therefore, the potential of storm surges affecting the site is considered low.

A tsunami is a series of long period waves generated in the ocean by a sudden displacement of large volumes of water. Causes of tsunamis include underwater earthquakes, volcanic eruptions, or offshore slope failures. The County of San Diego Hazard Mitigation Plan (2010) maps zones of possible tsunami inundation for coastal areas throughout the county. The site is not included within one of these high-risk hazard areas, and the site is at a minimum elevation of 190 above feet MSL and is about 1½ miles from the Pacific Ocean. Therefore, the potential for the site to be affected by a tsunami is negligible.

A seiche is a run-up of water within a lake or embayment triggered by fault- or landslide-induced ground displacement. The site is not located in the vicinity of or downstream from such bodies of water. Therefore, the risk of seiches affecting the site is negligible.

7.7 Landslides

We did not observe evidence of previous or incipient slope instability on the hillside during our reconnaissance. The City of San Diego Seismic Safety Study, Geologic Hazards and Faults, Map Sheet 34 have mapped a landslide area approximately 300 feet southeast of the property on the descending slope on the east side of Campus Point Drive. Map Sheet 34 defines the area as Hazard Category 21: *Landslides, confirmed, known, or highly suspected*. We do not consider the potential for a landslide to be a significant hazard to this project. Lateral movement associated with slope creep could occur to structures and improvements located adjacent to slopes.

7.8 Slope Stability

Slope stability analyses for the existing fill slopes with inclinations as steep as 2:1 (horizontal to vertical) indicate a calculated factor of safety of at least 1.5 under static conditions for both deep-seated and surficial failure. Appendix E presents the results of the slope stability analyses.

We performed the slope stability analyses based on the interpretation of geologic conditions encountered during our field investigation. Additional analyses may be required during the grading operations if the geologic conditions vary significantly. We performed the slope stability analyses using the two-dimensional computer program *GeoStudio2014* created by Geo-Slope International Ltd. The existing and proposed slopes should be stable from shallow sloughing conditions provided the recommendations for grading and drainage are incorporated into the design and construction of the proposed slopes.

Slopes should be landscaped with drought-tolerant vegetation having variable root depths and requiring minimal landscape irrigation. In addition, slopes should be drained and properly maintained to reduce erosion.

8. CONCLUSIONS AND RECOMMENDATIONS

8.1 General

8.1.1 We did not encounter soil or geologic conditions during our exploration that would preclude the proposed development, provided the preliminary recommendations presented herein are followed and implemented during design and construction. We will provide supplemental recommendations if we observe variable or undesirable conditions during construction, or if the proposed construction will differ from that anticipated herein.

8.1.2 With the exception of possible moderate to strong seismic shaking, we did not observe or know of significant geologic hazards to exist on the site that would adversely affect the proposed project.

8.1.3 Based on our review of previous fault studies performed on the property, faults are present at the subject site and cross adjacent the proposed eastern ARE Central Building and Buildings CP5 and CP6. We opine the faults crossing the property are potentially active and do not pose a risk of fault rupture hazard to the project. Structural setback zones are not required to mitigate fault rupture hazard.

8.1.4 Our field investigation indicates the site is underlain by previously placed fill, Tertiary-age Scripps Formation and Tertiary-age Ardath Formation. The previously placed fill ranges up to 100 feet below existing grades, where present, and possesses a potential for future settlement on the range of about ½ inch to 2 inches. The design team will need to evaluate the tolerances of the proposed buildings to the settlement estimates provided herein and determine if a deep foundation extending through the fill is needed.

8.1.5 We did not encounter groundwater during our subsurface exploration and we do not expect it to be a constraint to project development. However, we did encounter seepage within the fill materials in Boring B-15 at a depth of about 59 feet, Seepage within surficial soils and rock materials may be encountered during the grading operations, especially during the rainy seasons.

8.1.6 Excavation of the existing fill, Scripps Formation and Ardath Shale should generally be possible with moderate to heavy effort using conventional, heavy-duty equipment during grading and trenching operations. We expect the Scripps Formation and Ardath Shale may be difficult to excavate and could generate oversize material that may require special handling.

8.1.7 Proper drainage should be maintained in order to preserve the engineering properties of the fill in both the building pads and slope areas. Recommendations for site drainage are provided herein.

8.1.8 We performed a storm water management investigation to help evaluate the potential for infiltration on the property. Based on the results of our field infiltration testing and laboratory testing, we opine full or partial infiltration on the property should be considered infeasible as discussed in Appendix D.

8.1.9 Based on our review of the project plans, we opine the planned development can be constructed in accordance with our recommendations provided herein. We do not expect the planned development will destabilize or result in settlement of adjacent properties.

8.1.10 Surface settlement monuments and canyon subdrains will not be required on this project.

8.2 Excavation and Soil Characteristics

8.2.1 Excavation of the in-situ soil should be possible with moderate to heavy effort using conventional heavy-duty equipment. Excavation of the formational materials will require very heavy effort and may generate oversized material using conventional heavy-duty equipment during the grading operations. Oversized rock (rocks greater than 12-inches in dimension) may be generated with the formational materials that can be incorporated into landscape use or deep compacted fill areas, if available.

8.2.2 The soil encountered in the field investigation is considered to be “expansive” (expansion index [EI] of greater than 20) as defined by 2016 California Building Code (CBC) Section 1803.5.3. Table 8.2.1 presents soil classifications based on the expansion index. We expect a majority of the soil encountered possess a “very low” to “medium” expansion potential (EI of 90 or less) in accordance with ASTM D 4829.

TABLE 8.2.1
EXPANSION CLASSIFICATION BASED ON EXPANSION INDEX

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

8.2.3 We performed laboratory tests on samples of the site materials to evaluate the percentage of water-soluble sulfate content. Appendix B presents results of the laboratory water-soluble sulfate content tests. The test results indicate the on-site materials at the locations tested possess "S0" sulfate exposure to concrete structures as defined by 2016 CBC Section 1904 and ACI 318-14 Chapter 19. However, some areas of the Scripps Formation possess "S1" to "S3" water-soluble sulfate contents and additional concrete design recommendations may be encountered during construction. Table 8.2.2 presents a summary of concrete requirements set forth by 2016 CBC Section 1904 and ACI 318. The presence of water-soluble sulfates is not a visually discernible characteristic; therefore, other soil samples from the site could yield different concentrations. Additionally, over time landscaping activities (i.e., addition of fertilizers and other soil nutrients) may affect the concentration. We should perform additional laboratory water-soluble sulfate testing during grading operations to evaluate the sulfate exposure at finish grade elevations of the proposed structure.

TABLE 8.2.2
REQUIREMENTS FOR CONCRETE EXPOSED TO SULFATE-CONTAINING SOLUTIONS

Exposure Class	Water-Soluble Sulfate (SO_4) Percent by Weight	Cement Type (ASTM C 150)	Maximum Water to Cement Ratio by Weight[1]	Minimum Compressive Strength (psi)
S0	$SO_4<0.10$	No Type Restriction	n/a	2,500
S1	$0.10\leq SO_4<0.20$	II	0.50	4,000
S2	$0.20\leq SO_4\leq 2.00$	V	0.45	4,500
S3	$SO_4>2.00$	V+Pozzolan or Slag	0.45	4,500

* Maximum water to cement ratio limits do not apply to lightweight concrete.

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

8.3 Grading

8.3.1 Grading should be performed in accordance with the recommendations provided in this report, the Recommended Grading Specifications contained in Appendix F and the City of San Diego Land Development Manual. Geocon Incorporated should observe the grading operations on a full-time basis and provide testing during the fill placement.

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

8.3.3 Site preparation should begin with the removal of deleterious material, debris, and vegetation. The depth of vegetation removal should be such that material exposed in cut areas or soil to be used as fill is relatively free of organic matter. Material generated during stripping and/or site demolition should be exported from the site. Asphalt and concrete should not be mixed with the fill soil unless approved by the Geotechnical Engineer.

8.3.4 Abandoned foundations and buried utilities (if encountered) should be removed and the resultant depressions and/or trenches should be backfilled with properly compacted material as part of the remedial grading.

8.3.5 The upper 3 feet of materials within the building pad areas should be removed and replaced with properly compacted fill. Additionally, the removals should be extended for buildings where formational materials are near or at grade such that at least 2 feet of fill will be below the bottom of the footings. These deepened removals (i.e. 2-foot below footing) could be required within the fill areas based on the conditions observed during grading. The bottom of the excavations should be sloped 1 percent to the adjacent street or deepest fill. The removals should extend at least 5 feet outside the perimeter of the proposed building and/or footings, where possible. The upper 1 to 2 feet of the existing materials outside the building pad and within the parking lot and driveways should be removed and replaced with properly compacted fill. Prior to any fill soil being placed, the existing ground surface should be scarified, moisture conditioned as necessary, and compacted to a depth of at least 12 inches. Deeper removals may be required if saturated or loose fill soil is encountered. A representative of Geocon should be on-site during removals to evaluate the limits of the remedial grading. Table 8.3.1 provides a summary of the grading recommendations.

8.3.6 We understand that storm water management basins are being considered for the northeastern portion of the property. These basins should not be undercut and the formational materials should be exposed at the base of the basins if infiltration is planned. The surrounding slopes for the basins should be included in the remedial grading to expose competent materials and replaced with compacted fill.

TABLE 8.3.1
SUMMARY OF GRADING RECOMMENDATIONS

Area	Removal Requirements
Building Pads	Removal of Upper 3 Feet of Existing Materials
Building Pads (Formation Near Grade)*	Undercut 2 Feet Below Bottom of Footing
Building Pads – Lateral Removal Limits	5 Feet Outside of Building Pad/Footing Area
Site Development	Removal of Upper 1 to 2 Feet of Existing Materials
Storm Water Basins (Unlined)	Remove to Formational Materials
Exposed Bottoms of Remedial Grading	Scarify Upper 12 Inches

* Removal below footings could be required for fill areas based on conditions observed during grading.

8.3.7 Some areas of overly wet and saturated soil could be encountered due to the existing landscape and pavement areas. The saturated soil would require additional effort prior to placement of compacted fill or additional improvements. Stabilization of the soil would include scarifying and air-drying, removing and replacement with drier soil, use of stabilization fabric (e.g. Tensar TX7 or other approved fabric), or chemical treating (i.e. cement or lime treatment).

8.3.8 The contractor should be careful during the remedial grading operations to avoid a "pumping" condition at the base of the removals. Where recompaction of the excavated bottom will result in a "pumping" condition, the bottom of the excavation should be tracked with low ground pressure earthmoving equipment prior to placing fill. If needed to improve the stability of the excavation bottoms, reinforcing fabric or 2- to 3-inch crushed rock can be placed prior to placement of compacted fill.

8.3.9 The site should then be brought to final subgrade elevations with fill compacted in layers. In general, soil native to the site is suitable for use from a geotechnical engineering standpoint as fill if relatively free from vegetation, debris and other deleterious material. Layers of fill should be about 6 to 8 inches in loose thickness and no thicker than will allow for adequate bonding and compaction. Fill, including backfill and scarified ground surfaces, should be compacted to a dry density of at least 90 percent of the laboratory maximum dry density near to slightly above optimum moisture content in accordance with ASTM Test Procedure D 1557. Fill materials placed below optimum moisture content may require additional moisture conditioning prior to placing additional fill. The upper 12 inches of subgrade soil underlying pavement should be compacted to a dry density of at least 95 percent of the laboratory maximum dry density near to slightly above optimum moisture content shortly before paving operations.

8.3.10 Import fill (if necessary) should consist of the characteristics presented in Table 8.3.2. Geocon Incorporated should be notified of the import soil source and should perform laboratory testing of import soil prior to its arrival at the site to determine its suitability as fill material.

TABLE 8.3.2
SUMMARY OF IMPORT FILL RECOMMENDATIONS

Soil Characteristic	Values
Expansion Potential	"Very Low" to Medium (Expansion Index of 90 or less)
Particle Size	Maximum Dimension Less Than 3 Inches
	Generally Free of Debris

8.4 Subdrains

8.4.1 With the exception of retaining wall drains, we do not expect the installation of other subdrains.

8.5 Excavation Slopes, Shoring and Tiebacks

8.5.1 The recommendations included herein are provided for stable excavations. It is the responsibility of the contractor to provide a safe excavation during the construction of the proposed project.

8.5.2 Temporary excavations should be made in conformance with OSHA requirements and as directed by the assigned competent person in the field (contractor). In general, special shoring requirements may not be necessary if temporary excavations will be less than 4 feet in height. Temporary excavations greater than 4 feet in height, however, should be sloped back at an appropriate inclination. These excavations should not be allowed to become saturated or to dry out. Surcharge loads should not be permitted to a distance equal to the height of the excavation from the top of the excavation. The top of the excavation should be a minimum of 15 feet from the edge of existing improvements. Excavations steeper than those recommended or closer than 15 feet from an existing surface improvement should be shored in accordance with applicable OSHA codes and regulations.

8.5.3 The design of temporary shoring is governed by soil and groundwater conditions, and by the depth and width of the excavated area. Continuous support of the excavation face can be provided by a system of soldier piles and wood lagging or sheet piles. Excavations exceeding 15 feet may require soil nails, tieback anchors or internal bracing to provide additional wall restraint.

8.5.4 The condition of existing buildings, streets, sidewalks, and other structures/improvements around the perimeter of the planned excavation should be documented prior to the start of shoring and excavation work. Special attention should be given to documenting existing cracks or other indications of differential settlement within these adjacent structures, pavements and other improvements. Underground utilities sensitive to settlement should be videotaped prior to construction to check the integrity of pipes. In addition, monitoring points should be established indicating location and elevation around the excavation and upon existing buildings. These points should be monitored on a weekly basis during excavation work and on a monthly basis thereafter. Inclinometers should be installed and monitored behind any shoring sections that will be advanced deeper than 30 feet below the existing ground surface.

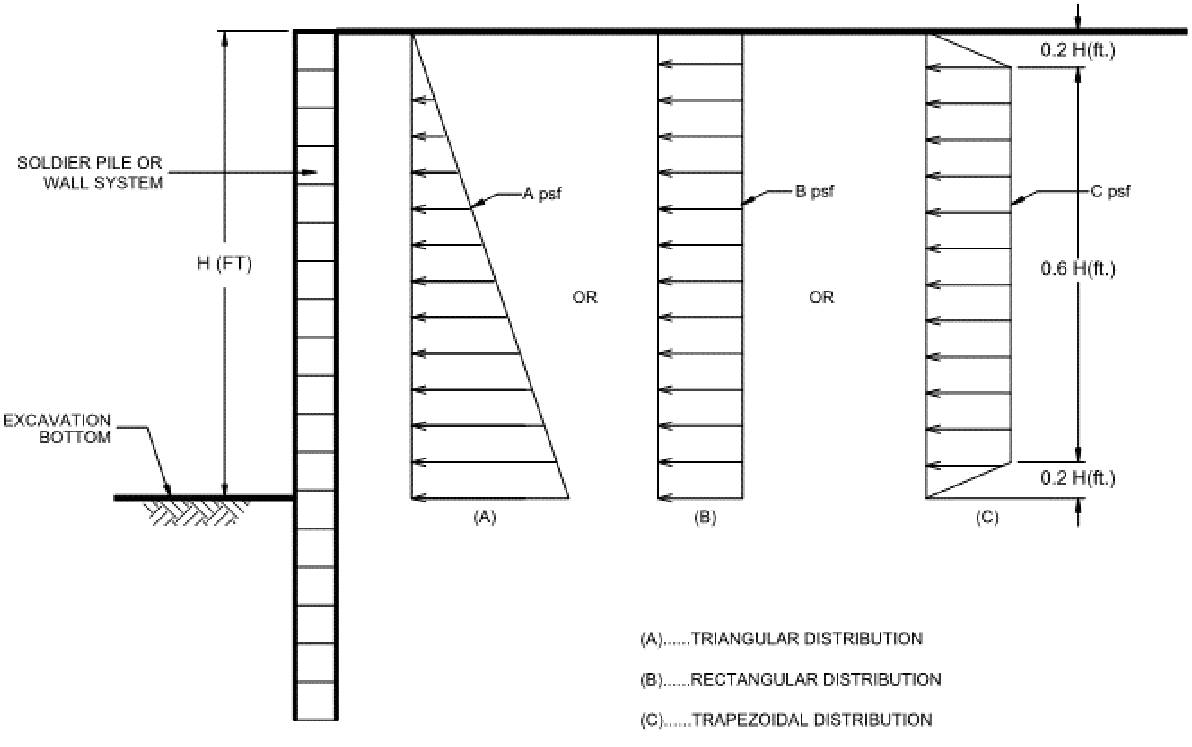
8.5.5 In general, ground conditions are moderately suited for soldier pile and tieback anchor wall construction techniques. However, gravel, cobble, and oversized material may be encountered in the existing materials that could be difficult to drill. Additionally, if cohesionless sands are encountered, some raveling may result along the unsupported portions of excavations. Cemented zones may be encountered within the formational units and could cause difficult excavations.

8.5.6 Temporary shoring with a level backfill should be designed using a lateral pressure envelope acting on the back of the shoring as presented in Table 8.5.1 assuming a level backfill. The distributions are shown on the Active Pressures for Temporary Shoring. Triangular distribution should be used for cantilevered shoring and, the trapezoidal and rectangular distribution should be used for multi-braced systems such as tieback anchors and rakers. The project shoring engineer should determine the applicable soil distribution for the design of the temporary shoring system. Additional lateral earth pressure due to the surcharging effects from construction equipment, sloping backfill, planned stockpiles, adjacent structures and/or traffic loads should be considered, where appropriate, during design of the shoring system.

TABLE 8.5.1
SUMMARY OF TEMPORARY SHORING WALL RECOMMENDATIONS

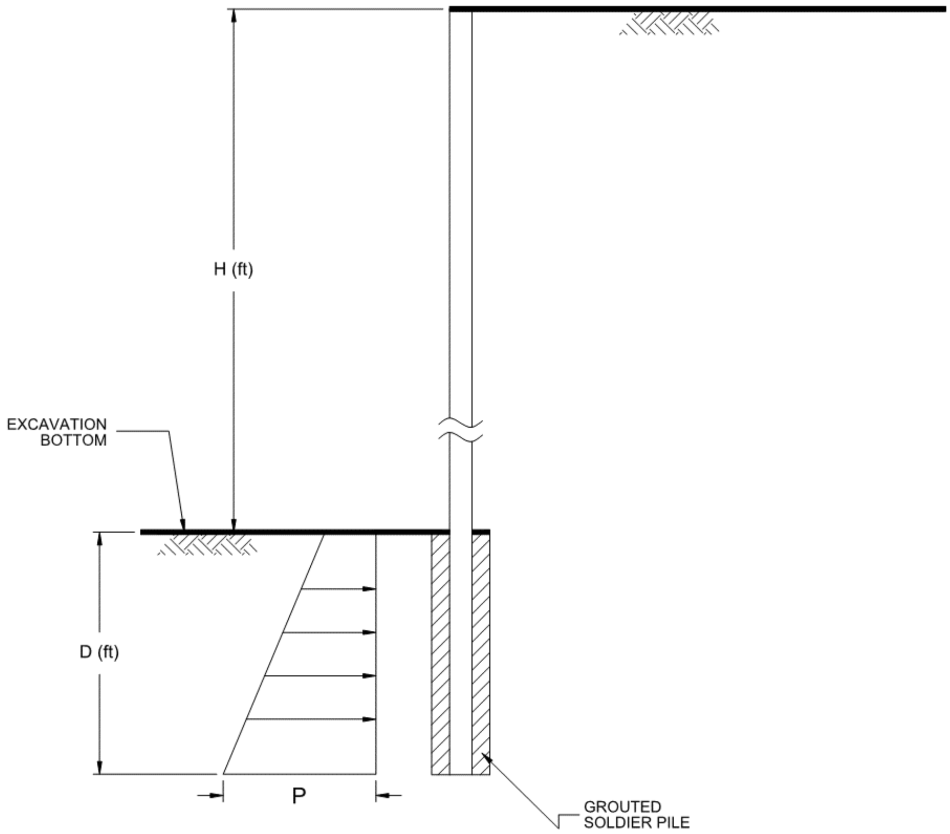
Parameter	Value
Triangular Distribution, A	34H psf
Rectangular Distribution, B	22H psf
Trapezoidal Distribution, C	27H psf
Passive Pressure, P	350D + 500 psf
Effective Zone Angle, E	28 degrees
Maximum Design Lateral Movement	1 Inch
Maximum Design Vertical Movement	½ Inch
Maximum Design Retained Height, H	40 Feet

* H equals the height of the retaining portion of the wall in feet.
* D equals the embedment depth of the retaining wall in feet.



Active Pressures on Temporary Shoring

8.5.7 The passive resistance can be assumed to act over a width of three pile diameters. Typically, soldier piles are embedded a minimum of 0.5 times the maximum height of the excavation (this depth is to include footing excavations) if tieback anchors are not employed. The project structural engineer should determine the actual embedment depth.



Passive Pressures on Temporary Shoring

8.5.8 Lateral movement of shoring is associated with vertical ground settlement outside of the excavation. Therefore, it is essential that the soldier pile and tieback system allow very limited amounts of lateral displacement. Earth pressures acting on a lagging wall can cause movement of the shoring toward the excavation and result in ground subsidence outside of the excavation. Consequently, horizontal movements of the shoring wall should be accurately monitored and recorded during excavation and anchor construction.

8.5.9 Survey points should be established at the top of the pile on at least 20 percent of the soldier piles. An additional point located at an intermediate point between the top of the pile and the base of the excavation should be monitored on at least 20 percent of the piles if tieback anchors will be used. These points should be monitored on a weekly basis during excavation work and on a monthly basis thereafter until the permanent support system is constructed.

8.5.10 The project civil engineer should provide the approximate location, depth, and pipe type of the underground utilities to the shoring engineer to help select the shoring type and shoring design. The shoring system should be designed to limit horizontal soldier pile movement to a maximum of 1 inch. The amount of horizontal deflection can be assumed to be essentially zero along the Active Zone and Effective Zone boundary, as shown in the Active Zone Detail herein. The magnitude of movement for intermediate depths and distances from the shoring wall can be linearly interpolated. We understand the City of San Diego may require the developer to prepare a hold harmless agreement for the planned construction operations and development regarding the existing utilities and improvements.

8.5.11 We should observe the drilled shafts for the soldier piles prior to the placement of steel reinforcement to check that the exposed soil conditions are similar to those expected and that footing excavations have been extended to the appropriate bearing strata and design depths. If unexpected soil conditions are encountered, foundation modifications may be required.

8.5.12 Experience has shown that the use of pressure grouting during formation of the bonded portion of the anchor will increase the soil-grout bond stress. A pressure grouting tube should be installed during the construction of the tieback. Post grouting should be performed if adequate capacity cannot be obtained by other construction methods.

8.5.13 Anchor capacity is a function of construction method, depth of anchor, batter, diameter of the bonded section and the length of the bonded section. Anchor capacity should be evaluated using the strength parameters shown in Table 8.5.2.

TABLE 8.5.2
SOIL STRENGTH PARAMETERS FOR TEMPORARY SHORING

Description	Cohesion (psf)	Friction Angle (Degrees)
Compacted Fill (Qpf & Qcf)	300	28
Scripps Formation/Ardath Shale	300	32

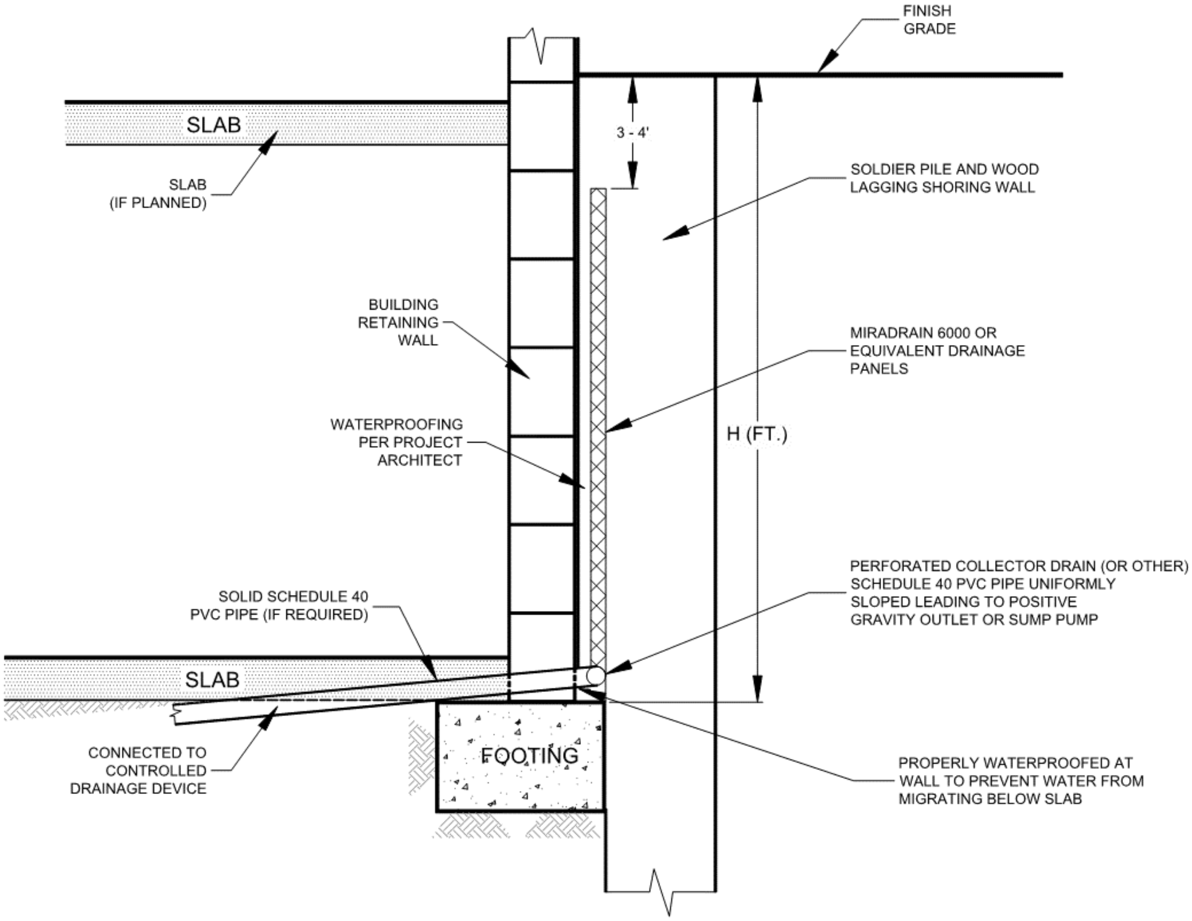
8.5.14 Grout should only be placed in the tieback anchor’s bonded section prior to testing. Tieback anchors should be proof-tested to at least 130 percent of the anchor’s design working load. Following a successful proof test, the tieback anchors should be locked off at 80 percent of the allowable working load. Tieback anchor test failure criteria should be established in project plans and specifications. The tieback anchor test failure criteria should be based upon a maximum allowable displacement at 130 percent of the anchor’s working load (anchor creep) and a maximum residual displacement within the anchor following stressing. Tieback anchor stressing should only be conducted after sufficient hydration has occurred within the grout. Tieback anchors that fail to meet project specified test criteria should be replaced or additional anchors should be constructed.

8.5.15 Lagging should keep pace with excavation. The excavation should not be advanced deeper than three feet below the bottom of lagging at any time. These unlagged gaps of up to three feet should only be allowed to stand for short periods of time in order to decrease the probability of soil instability and should never be unsupported overnight. Backfilling should be conducted when necessary between the back of lagging and excavation sidewalls to reduce sloughing in this zone and all voids should be filled by the end of each day. Further, the excavation should not be advanced further than four feet below a row of tiebacks prior to those tiebacks being proof tested and locked off unless otherwise specific by the shoring engineer.

8.5.16 If tieback anchors are employed, an accurate survey of existing utilities and other underground structures adjacent to the shoring wall should be conducted. The survey should include both locations and depths of existing utilities. Locations of anchors should be adjusted as necessary during the design and construction process to accommodate the existing and proposed utilities.

8.5.17 Tieback anchors within the City of San Diego right-of-way should be properly detentioned and removed where steel does not exist within the upper 20 feet from the existing grade. The Notice – Land Development Review/Shoring in City Right-Of-Way, prepared by the City of San Diego, dated July 1, 2003 should be reviewed and incorporated into the design of the tieback anchors. Procedures for removal of tieback anchors include unscrewing tendons using special couplings, use of explosives, or heat induction. Geocon Incorporated should be consulted if other methods of removal are planned.

8.5.18 The shoring system should incorporate a drainage system for the proposed retaining wall as shown herein.



Soldier Pile Wall Drainage Detail

8.6 Soil Nail Wall

8.6.1 As an alternative to temporary shoring followed by construction of a permanent basement wall, a soil nail wall can be used. Soil nail walls consist of installing closely spaced steel bars (nails) into a slope or excavation in a top-down construction sequence. Following installation of a horizontal row of nails, drains, waterproofing and wall reinforcing steel are placed and shotcrete applied to create a final wall. The wall should be designed by an engineer familiar with the design of soil nail walls.

8.6.2 Temporary soil nail walls should not be considered a permanent design to support the seismic lateral loads and soil pressures on a building wall. Therefore, the proposed building should be designed to support the expected lateral loads.

8.6.3 In general, ground conditions are moderately suited to soil nail wall construction techniques. However, localized gravel, cobble and oversized material could be encountered in the existing materials that could be difficult to drill. Additionally, relatively clean sands may be encountered within the existing soil that may result in some raveling of the unsupported excavation. Casing or specialized drilling techniques should be planned where raveling exists.

8.6.4 Testing of the soil nails should be performed in accordance with the guidelines of the Federal Highway Administration or similar guidelines. At least two verification tests should be performed to confirm design assumptions for each soil/rock type encountered. Verification tests nails should be sacrificial and should not be used to support the proposed wall. The bond length should be adjusted to allow for pullout testing of the verification nails to evaluate the ultimate bond stress. A minimum of 5 percent of the production nails should also be proof tested and a minimum of 4 sacrificial nails should be tested at the discretion of Geocon Incorporated. Consideration should be given to testing sacrificial nails with an adjusted bond length rather than testing production nails. Geocon Incorporated should observe the nail installation and perform the nail testing.

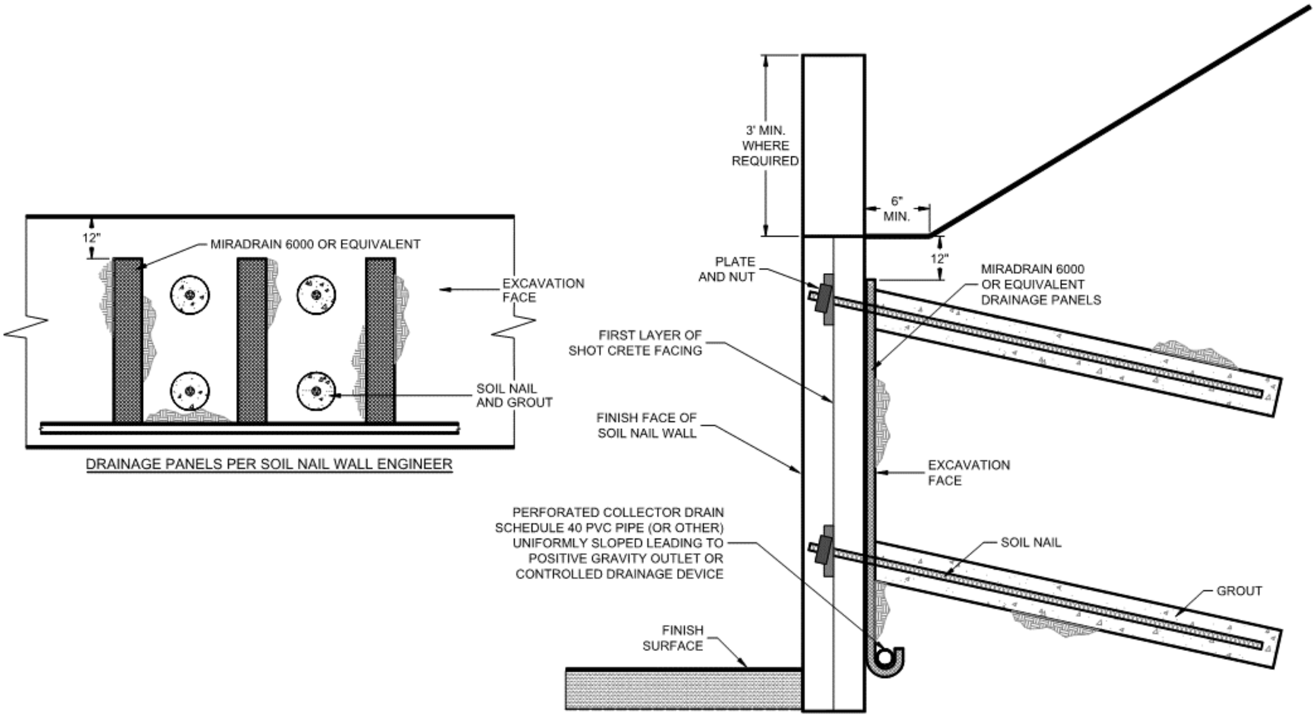
8.6.5 The soil strength parameters listed in Table 8.6 can be used in design of the soil nails. The bond stress is dependent on drilling method, diameter, and construction method. Therefore, the designer should evaluate the bond stress based on the existing soil conditions and the construction method.

TABLE 8.6
SOIL STRENGTH PARAMETERS FOR SOIL NAIL WALLS

Description	Cohesion (psf)	Friction Angle (degrees)	Estimated Ultimate Bond Stress (psi)*
Previously Placed Fill	300	28	10
Scripps Formation/Ardath Shale	300	32	20

* Assuming gravity fed, open hole drilling techniques.

8.6.6 A wall drain system should be incorporated into the design of the soil nail wall as shown herein. Corrosion protection should be provided for the nails if the wall will be a permanent structure.



Soil Nail Wall Drainage Detail

8.7 Seismic Design Criteria

8.7.1 Table 8.7.1 summarizes summarizes site-specific design criteria obtained from the 2019 California Building Code (CBC; Based on the 2018 International Building Code [IBC] and ASCE 7-16), Chapter 16 Structural Design, Section 1613 Earthquake Loads. We used the computer program *U.S. Seismic Design Maps*, provided by the Structural Engineers Association (SEA) to calculate the seismic design parameters. The short spectral response uses a period of 0.2 second. The buildings and improvements should be designed using a Site Class C where the fill thickness is 20 feet or less and/or when deep foundations are used, or a Site Class D where the fill is thicker than 20 feet. We evaluated the Site Class based on the discussion in Section 1613.2.2 of the 2019 CBC and Table 20.3-1 of ASCE 7-16. The values presented herein are for the risk-targeted maximum considered earthquake (MCE_R). Sites designated as Site Class D, E and F may require additional analyses if requested by the project structural engineer and client.

TABLE 8.7.1
2019 CBC SEISMIC DESIGN PARAMETERS

Parameter	Value		2019 CBC Reference
Site Class	C	D	Section 1613.2.2
Fill Thickness, T (Feet)	T≤20	T>20	--
MCE_R Ground Motion Spectral Response Acceleration – Class B (short), S_S	1.184g	1.184g	Figure 1613.2.1(1)
MCE_R Ground Motion Spectral Response Acceleration – Class B (1 sec), S_1	0.418g	0.418g	Figure 1613.2.1(2)
Site Coefficient, F_A	1.200	1.027	Table 1613.2.3(1)
Site Coefficient, F_V	1.500	1.882*	Table 1613.2.3(2)
Site Class Modified MCE_R Spectral Response Acceleration (short), S_{MS}	1.420g	1.215g	Section 1613.2.3 (Eqn 16-36)
Site Class Modified MCE_R Spectral Response Acceleration – (1 sec), S_{M1}	0.627g	0.786g*	Section 1613.2.3 (Eqn 16-37)
5% Damped Design Spectral Response Acceleration (short), S_{DS}	0.947g	0.810g	Section 1613.2.4 (Eqn 16-38)
5% Damped Design Spectral Response Acceleration (1 sec), S_{D1}	0.418g	0.524g*	Section 1613.2.4 (Eqn 16-39)

* Using the code-based values presented in this table, in lieu of a performing a ground motion hazard analysis, requires the exceptions outlined in ASCE 7-16 Section 11.4.8 be followed by the project structural engineer. Per Section 11.4.8 of ASCE/SEI 7-16, a ground motion hazard analysis should be performed for projects for Site Class "E" sites with Ss greater than or equal to 1.0g and for Site Class "D" and "E" sites with S1 greater than 0.2g. Section 11.4.8 also provides exceptions which indicates that the ground motion hazard analysis may be waived provided the exceptions are followed.

8.7.2 Table 8.7.2 presents the mapped maximum considered geometric mean (MCE_G) seismic design parameters for projects located in Seismic Design Categories of D through F in accordance with ASCE 7-16.

TABLE 8.7.2
ASCE 7-16 PEAK GROUND ACCELERATION

Parameter	Value		ASCE 7-16 Reference
Site Class	C	D	---
Fill Thickness, T (Feet)	T≤20	T>20	---
Mapped MCE_G Peak Ground Acceleration, PGA	0.532g	0.532g	Figure 22-7
Site Coefficient, F_{PGA}	1.200	1.100	Table 11.8-1
Site Class Modified MCE_G Peak Ground Acceleration, PGA_M	0.639g	0.585g	Section 11.8.3 (Eqn 11.8-1)

8.7.3 Conformance to the criteria in Tables 8.7.1 and 8.7.2 for seismic design does not constitute any kind of guarantee or assurance that significant structural damage or ground failure will not occur if a large earthquake occurs. The primary goal of seismic design is to protect life, not to avoid all damage, since such design may be economically prohibitive.

8.7.4 The project structural engineer and architect should evaluate the appropriate Risk Category and Seismic Design Category for the planned structures. The values presented herein assume a Risk Category of I / II / III and resulting in a Seismic Design Category D. Table 8.7.3 presents a summary of the risk categories.

TABLE 8.7.3
ASCE 7-16 RISK CATEGORIES

Risk Category	Building Use	Examples
I	Low risk to Human Life at Failure	Barn, Storage Shelter
II	Nominal Risk to Human Life at Failure (Buildings Not Designated as I, III or IV)	Residential, Commercial and Industrial Buildings
III	Substantial Risk to Human Life at Failure	Theaters, Lecture Halls, Dining Halls, Schools, Prisons, Small Healthcare Facilities, Infrastructure Plants, Storage for Explosives/Toxins
IV	Essential Facilities	Hazardous Material Facilities, Hospitals, Fire and Rescue, Emergency Shelters, Police Stations, Power Stations, Aviation Control Facilities, National Defense, Water Storage

8.8 Settlement Due to Fill Loads

8.8.1 Fill soil, even if properly compacted, will experience settlement over the lifetime of the improvements that it supports. The ultimate settlement potential of the fill is a function of the soil classification, placement relative compaction, and subsequent increases in the soil moisture content.

8.8.2 The proposed buildings will be underlain by a maximum thickness of compacted fill on the order of 100 feet. The settlement of compacted fill is expected to continue over a relatively extended time period resulting from both gravity loading and hydrocompression upon wetting from rainfall and/or landscape irrigation. The previously placed fill has existed for approximately 20 years; therefore, a majority of the expected settlement has likely occurred.

8.8.3 Due to the variable fill thickness, a potential for differential settlement across the proposed buildings exist and special foundation design consideration as discussed herein will be necessary. Based on measured settlement of similar fill depths on other sites and the time period since the fill was placed, we estimate that maximum settlement of the compacted fill will be approximately 0.15 percent for the compacted fills based on the existing fill thickness. Figure 2 provides the approximate thickness of fill and estimated maximum fill settlement in the area of the proposed buildings and improvements.

8.8.4 Table 8.8 presents the estimated total and differential fill thickness and settlements of the building pads using an estimated settlement of 0.15 percent for the existing fill soils. We understand some of the proposed buildings will include subterranean garages and/or offices 1- to 2-levels below grade and we reduced the fill thicknesses and settlements for these buildings accordingly as presented in Table 8.8. These settlement magnitudes should be considered in design of the foundation system and adjacent flatwork that connects to the proposed buildings. The Fill Thickness Maps, Figures 3 and 4, present the fill thickness contours for the existing and proposed site elevations, respectively.

TABLE 8.8
EXPECTED DIFFERENTIAL SETTLEMENT OF EXISTING FILL SOIL

Building No.	Maximum Depth of Fill Beneath Structure (Feet)	Maximum Fill Differential (Feet)	Estimated Maximum Settlement (Inches)	Estimated Differential Settlement (Inches)	Estimated Maximum Angular Distortion
CP4 [A]	0	0	---	---	---
CP5 [A]	15	15	0.3	0.3	1/2400
CP6 [A]	75	75	1.6	1.6	1/350
CP7 [A,B]	25	25	0.5	0.5	1/960
A1 – A3	0	0	---	---	---
A4 – A5	80	20	1.4	0.4	1/1200
Parking Structure [A]	70	70	1.5	1.5	1/370

[A] Assuming 12 foot subterranean excavation for CP4, 30 foot excavation for CP5, 25 foot excavation for CP6 and CP7, and 20 foot excavation for the parking structure.

[B] Existing ~20 foot tall retaining wall present within footprint of CP7.

8.8.5 Deep foundations such as driven piles or drilled piers are the most effective means of reducing the ultimate settlement potential of the proposed structures to a negligible amount. Alternatively, highly reinforced shallow foundation systems and slabs-on-grade may be used for support of the buildings; however, the shallow foundation systems would not eliminate the potential for cosmetic distress related to differential settlement of the underlying fill. Some cosmetic distress should be expected over the life of the structure as a result of long-term differential settlement. The owner, tenants, and future owners should be made aware that cosmetic distress, including separation of caulking at wall joints, small non-structural wall panel cracks, and separation of concrete flatwork is likely to occur. Recommendations for deep foundations can be provided to evaluate the comparative risks and costs upon request.

8.9 Shallow Foundations

8.9.1 The proposed structures can be supported on a shallow foundation system founded in the compacted fill and/or formational materials. Foundations for the structure should consist of continuous strip footings and/or isolated spread footings. 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. Table 8.9.1 provides a summary of the foundation design recommendations.

TABLE 8.9.1
SUMMARY OF FOUNDATION RECOMMENDATIONS (AT-GRADE)

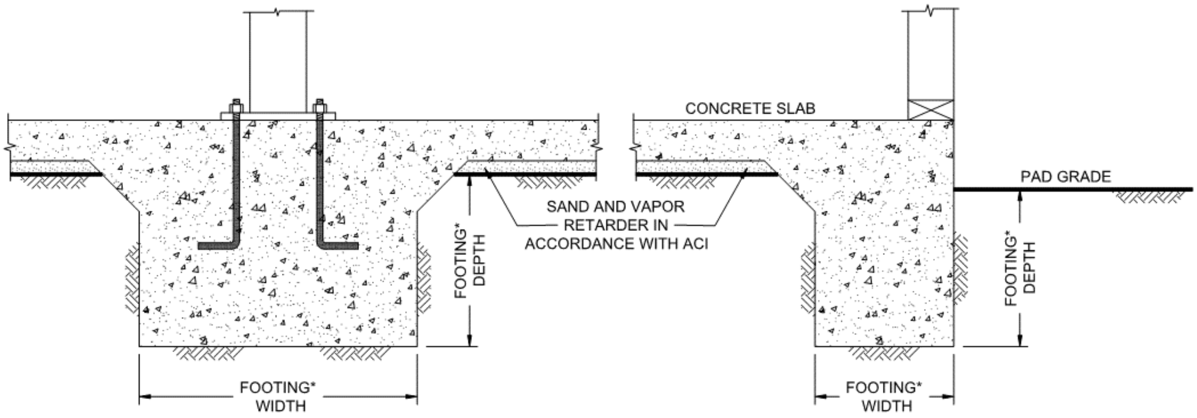
Parameter	Value
Bearing Material	Formation
Minimum Continuous Foundation Width	12 inches
Minimum Isolated Foundation Width	24 inches
Minimum Foundation Depth	24 Inches Below Lowest Adjacent Grade
Minimum Steel Reinforcement	4 No. 5 Bars, 2 at the Top and 2 at the Bottom
Allowable Bearing Capacity – Fill	2,500 psf
Allowable Bearing Capacity – Formation	6,000 psf
Bearing Capacity Increase	500 psf per Foot of Depth
	500 psf per Foot of Width
Maximum Allowable Bearing Capacity – Fill	4,000 psf
Maximum Allowable Bearing Capacity – Formation	8,000 psf
Estimated Total Settlement	1 Inch
Estimated Differential Settlement	½ Inch in 40 Feet
Footing Size Used for Settlement	9-Foot Square
Design Expansion Index	90 or less

8.9.2 We understand that several of the buildings are proposed to be supported at 1- to 2-levels below grade. Table 8.9.2 provides a summary of the foundation design recommendations for subterranean levels.

TABLE 8.9.2
SUMMARY OF FOUNDATION RECOMMENDATIONS WITH SUBTERRANEAN LEVELS

Parameter	Value
Minimum Continuous Foundation Width	12 inches
Minimum Isolated Foundation Width	24 inches
Minimum Foundation Depth	24 Inches Below Lowest Adjacent Grade
Minimum Steel Reinforcement	4 No. 5 Bars, 2 at the Top and 2 at the Bottom
Allowable Bearing Capacity – Fill	4,000 psf
Bearing Capacity – Formation	9,000 psf
Bearing Capacity Increase	500 psf per Foot of Depth
	500 psf per Foot of Width
Maximum Allowable Bearing Capacity - Fill	6,000 psf
Maximum Bearing Capacity - Formation	11,000 psf
Estimated Total Settlement	1 Inch
Estimated Differential Settlement	½ Inch in 40 Feet
Footing Size Used for Settlement	9-Foot Square
Design Expansion Index	90 or less

8.9.3 The foundations should be embedded in accordance with the recommendations herein and the Wall/Column Footing Dimension Detail. The embedment depths should be measured from the lowest adjacent pad grade for both interior and exterior footings. 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 (unless designed with a post-tensioned foundation system as discussed herein).



Wall/Column Footing Dimension Detail

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

8.9.5 Isolated footings outside of the slab area, if present, should have the minimum embedment depth and width recommended for conventional foundations. The isolated footings should be connected to the building foundation system with grade beams when located beyond the perimeter of the building and supporting structural elements connected to the building.

8.9.6 Overexcavation of the footings and replacement with slurry can be performed in areas where formational materials are not encountered at the bottom of the footing where the foundations are planned in the formational materials. Minimum two-sack slurry can be placed in the excavations for the conventional foundations to the bottom of proposed footing elevation.

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

- For fill slopes less than 20 feet high, 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.

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

8.9.8 We should observe the foundation excavations prior to the placement of reinforcing steel and concrete to check that the exposed soil conditions are similar to those expected and that they have been extended to the appropriate bearing strata. Foundation modifications may be required if unexpected soil conditions are encountered.

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

8.10 Mat Foundation

8.10.1 We understand the proposed retail buildings may be supported on a mat foundation. A mat foundation consists of a thick, rigid concrete mat that allows the entire footprint of the structure to carry building loads. In addition, the mat can tolerate significantly greater differential movements such as those associated with expansive soils or differential settlement. In this case, the mat foundation may be used to accommodate the relatively large differential settlements and associated angular distortion due to the potential fill settlement. Table 8.10 provides a summary of the foundation design recommendations.

TABLE 8.10
SUMMARY OF MAT FOUNDATION RECOMMENDATIONS

Parameter	Value
Minimum Foundation Depth	24 Inches Below Lowest Adjacent Grade
Minimum Steel Reinforcement	Per Structural Engineer
Bearing Capacity	800 psf
Estimated Total Settlement	1 Inch
Estimated Differential Settlement	½ Inch in 40 Feet
Foundation Size Used for Settlement Estimate	60-Foot-Square Mat Foundation
Modulus of Subgrade Reaction	100 to 150 pci
Design Expansion Index	90 or less

8.10.2 The modulus of subgrade reaction values should be modified as necessary using standard equations for mat size as required by the structural engineer. This value is a unit value for use with a 1-foot square footing. The modulus should be reduced in accordance with the following equation when used with larger foundations:

$$K_R = K\left[\frac{B+1}{2B}\right]^2$$

Where: K_R = reduced subgrade modulus
K = unit subgrade modulus
B = foundation width (in feet)

8.10.3 A mat foundation system will allow the structure to settle with the ground and should have sufficient rigidity to allow the structure to move as a single unit. Re-leveling of the mat foundation could be necessary through the use of mud jacking, compaction grouting or other similar techniques if differential settlement occurs

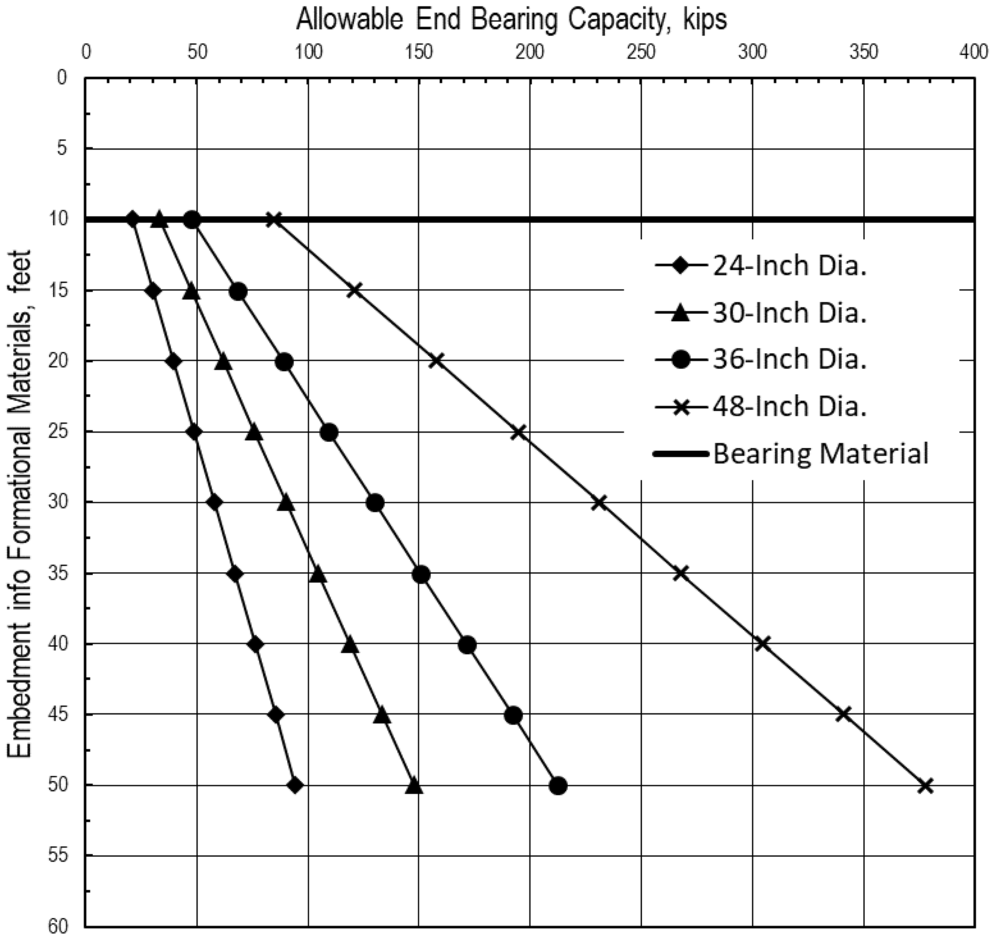
8.10.4 Slabs that may receive moisture-sensitive floor coverings or may be used to store moisture-sensitive materials should be underlain by a vapor retarder. The vapor retarder design should be consistent with the guidelines presented in the American Concrete Institute's (ACI) *Guide for Concrete Slabs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

8.10.5 The bedding sand thickness should be determined by the project foundation engineer, architect, and/or developer. However, we should be contacted to provide recommendations if the bedding sand is thicker than 6 inches. The foundation design engineer should provide appropriate concrete mix design criteria and curing measures to assure proper curing of the slab by reducing the potential for rapid moisture loss and subsequent cracking and/or slab curl. We suggest that the foundation design engineer present the concrete mix design and proper curing methods on the foundation plans. It is critical that the foundation contractor understands and follows the recommendations presented on the foundation plans.

8.11 Drilled Pier Recommendations

8.11.1 We understand that drilled piers may be used for foundation support. The foundation recommendations herein assume that the piers will extend through fill into the Scripps Formation or Ardath Shale materials. The piers should be embedded at least 10 feet within the formational materials.

8.11.2 Piers can be designed to develop support by end bearing within the formational materials and skin friction within the formational materials and portions of the fill soil. An allowable skin friction resistance of 200 psf and 500 psf can be used for that portion of the drilled pier embedded in fill soil and formational materials, respectively. The end bearing capacity can be determined by the End Bearing Capacity Chart. These allowable values possess a factor of safety of at least 2 and 3 for skin friction and end bearing, respectively.



End Bearing Capacity Chart

8.11.3 The diameter of the piers should be a minimum of 24 inches. The design length of the drilled piers should be determined by the designer based on the elevation of the pile cap or grade beam and the elevation of the top of the formational materials obtained from the Geologic Map and Geologic Cross-Sections presented herein. It is difficult to evaluate the exact length of the proposed drilled piers due to the variable thickness of the existing fill; therefore, some variation should be expected during drilling operations.

8.11.4 If pier spacing is at least three times the maximum dimension of the pier, no reduction in axial capacity for group effects is considered necessary. If piles are spaced between 2 and 3 pile diameters (center to center), the single pile axial capacity should be reduced by 25 percent. Geocon Incorporated should be contacted to provide single-pile capacity if piers are spaced closer than 2 diameters.

8.11.5 The allowable downward capacity may be increased by one-third when considering transient wind or seismic loads.

8.11.6 The formational materials may contain gravel and cobble and may possess very dense zones; therefore, the drilling contractor should expect difficult drilling conditions during excavations for the piers. Because a significant portion of the piers capacity will be developed by end bearing, the bottom of the borehole should be cleaned of loose cuttings prior to the placement of steel and concrete. Experience indicates that backspinning the auger does not remove loose material and a flat cleanout plate is necessary. Concrete should be placed within the excavation as soon as possible after the auger/cleanout plate is withdrawn to reduce the potential for discontinuities or caving

8.11.7 Pile settlement of production piers is expected to be on the order of ½ to 1 inch if the piers are loaded to their allowable capacities. Geocon should provide updated settlement estimates once the foundation plans are available. Settlements should be essentially complete shortly after completion of the building superstructure.

8.11.8 We can provide a lateral pile capacity analysis using the *LPILE* computer program once the pile type, size, and approximate length has been provided. The total capacity of pile groups should be considered less than the sum of the induvial pile capacities for pile spacing of less than 8D (where D is pile diameter) for lateral loads parallel to the pile group and 3D for loads perpendicular to the pile group. The reduction in capacity is based on pile spacing and positioning and can result in group efficiency on the order of 50 percent of the sum of single-pile capacities. We can evaluate the lateral capacity of pile groups using the *GROUP* computer program, if requested.

8.12 Concrete Slabs-On-Grade

8.12.1 Concrete slabs-on-grade for the structures should be constructed in accordance with Table 8.12.

TABLE 8.12
MINIMUM CONCRETE SLAB-ON-GRADE RECOMMENDATIONS

Parameter	Value
Minimum Concrete Slab Thickness	5 inches
Minimum Steel Reinforcement	No. 4 Bars 18 Inches on Center, Both Directions
Typical Slab Underlayment	3 to 4 Inches of Sand/Gravel/Base
Design Expansion Index	90 or less

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

8.12.3 The bedding sand thickness should be determined by the project foundation engineer, architect, and/or developer. It is common to have 3 to 4 inches of sand for 5-inch and 4-inch thick slabs, respectively, in the southern California region. However, we should be contacted to provide recommendations if the bedding sand is thicker than 6 inches. The foundation design engineer should provide appropriate concrete mix design criteria and curing measures to assure proper curing of the slab by reducing the potential for rapid moisture loss and subsequent cracking and/or slab curl. We suggest that the foundation design engineer present the concrete mix design and proper curing methods on the foundation plans. It is critical that the foundation contractor understands and follows the recommendations presented on the foundation plans.

8.12.4 Concrete slabs should be provided with adequate crack-control joints, construction joints and/or expansion joints to reduce unsightly shrinkage cracking. The design of joints should consider criteria of the American Concrete Institute (ACI) when establishing crack-control spacing. Crack-control joints should be spaced at intervals no greater than 12 feet.

Additional steel reinforcing, concrete admixtures and/or closer crack control joint spacing should be considered where concrete-exposed finished floors are planned.

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

8.12.6 The concrete slab-on-grade recommendations are based on soil support characteristics only. The project structural engineer should evaluate the structural requirements of the concrete slabs for supporting expected loads.

8.12.7 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.

8.13 Exterior Concrete Flatwork

8.13.1 Exterior concrete flatwork not subject to vehicular traffic should be constructed in accordance with the recommendations presented in Table 8.13. The recommended steel reinforcement would help reduce the potential for cracking.

TABLE 8.13
MINIMUM CONCRETE FLATWORK RECOMMENDATIONS

Expansion Index, EI	Minimum Steel Reinforcement* Options	Minimum Thickness
EI ≤ 90	6x6-W2.9/W2.9 (6x6-6/6) welded wire mesh	4 Inches
	No. 3 Bars 18 inches on center, Both Directions	
EI ≤ 130	4x4-W4.0/W4.0 (4x4-4/4) welded wire mesh	
	No. 4 Bars 12 inches on center, Both Directions	

* In excess of 8 feet square.

8.13.2 Even with the incorporation of the recommendations of this report, the exterior concrete flatwork has a potential to experience some uplift due to expansive soil beneath grade. The steel reinforcement should overlap continuously in flatwork to reduce the potential for vertical offsets within flatwork. Additionally, flatwork should be structurally connected to the curbs, where possible, to reduce the potential for offsets between the curbs and the flatwork.

8.13.3 Concrete flatwork should be provided with crack control joints to reduce and/or control shrinkage cracking. Crack control spacing should be determined by the project structural engineer based upon the slab thickness and intended usage. Criteria of the American Concrete Institute (ACI) should be taken into consideration when establishing crack control spacing. Subgrade soil for exterior slabs not subjected to vehicle loads should be compacted in accordance with criteria presented in the grading section prior to concrete placement. Subgrade soil should be properly compacted and the moisture content of subgrade soil should be verified prior to placing concrete. Base materials will not be required below concrete improvements.

8.13.4 Where exterior flatwork abuts the structure at entrant or exit points, the exterior slab should be dowelled into the structure’s foundation stemwall. This recommendation is intended to reduce the potential for differential elevations that could result from differential settlement or minor heave of the flatwork. Dowelling details should be designed by the project structural engineer.

8.13.5 The recommendations presented herein are intended to reduce the potential for cracking of exterior slabs as a result of differential movement. However, even with the incorporation of the recommendations presented herein, slabs-on-grade will still crack. The occurrence of concrete shrinkage cracks is independent of the soil supporting characteristics. Their occurrence may be reduced and/or controlled by limiting the slump of the concrete, the use of crack control joints and proper concrete placement and curing. Crack control joints should be spaced at intervals no greater than 12 feet. Literature provided by the Portland Concrete Association (PCA) and American Concrete Institute (ACI) present recommendations for proper concrete mix, construction, and curing practices, and should be incorporated into project construction.

8.14 Retaining Walls

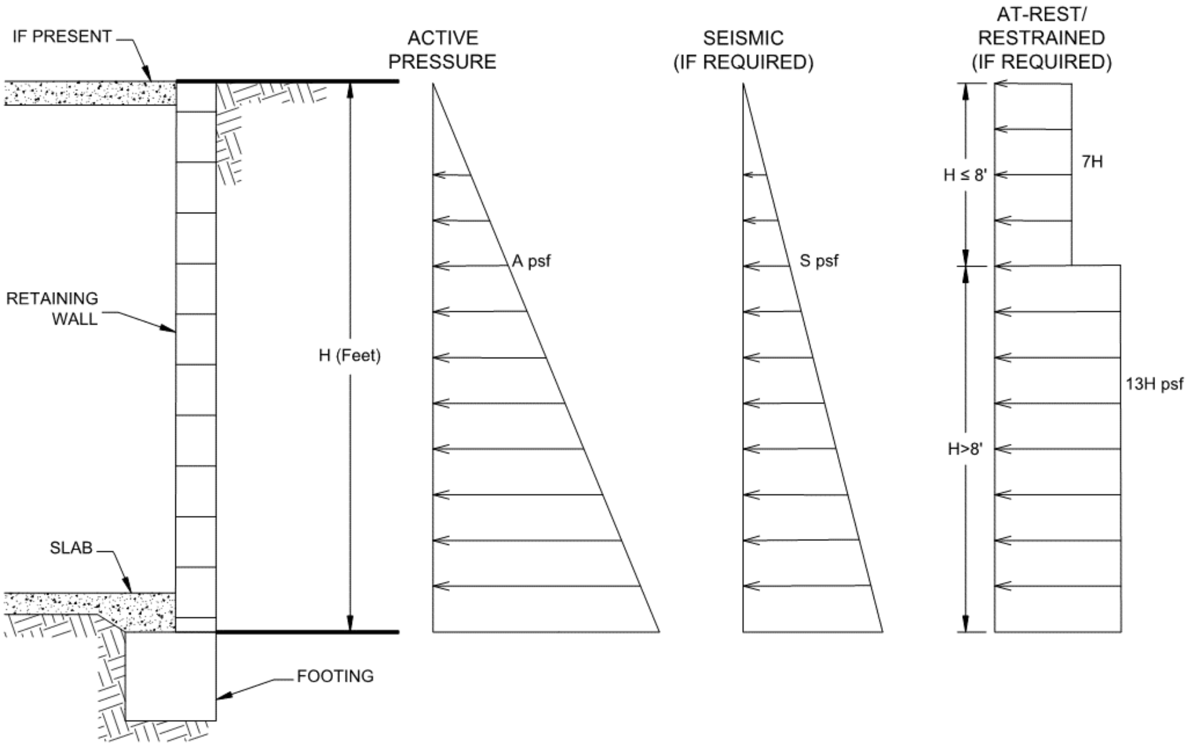
8.14.1 Retaining walls should be designed using the values presented in Table 8.14.1. Soil with an expansion index (EI) of greater than 90 should not be used as backfill material behind retaining walls.

TABLE 8.14.1
RETAINING WALL DESIGN RECOMMENDATIONS

Parameter	Value
Active Soil Pressure, A (Fluid Density, Level Backfill)	40 pcf
Active Soil Pressure, A (Fluid Density, 2:1 Sloping Backfill)	55 pcf
Seismic Pressure, S	15H psf
At-Rest/Restrained Walls Additional Uniform Pressure (0 to 8 Feet High)	7H psf
At-Rest/Restrained Walls Additional Uniform Pressure (8+ Feet High)	13H psf
Expected Expansion Index for the Subject Property	EI≤90

* H equals the height of the retaining portion of the wall.

8.14.2 The project retaining walls should be designed as shown in the Retaining Wall Loading Diagram.



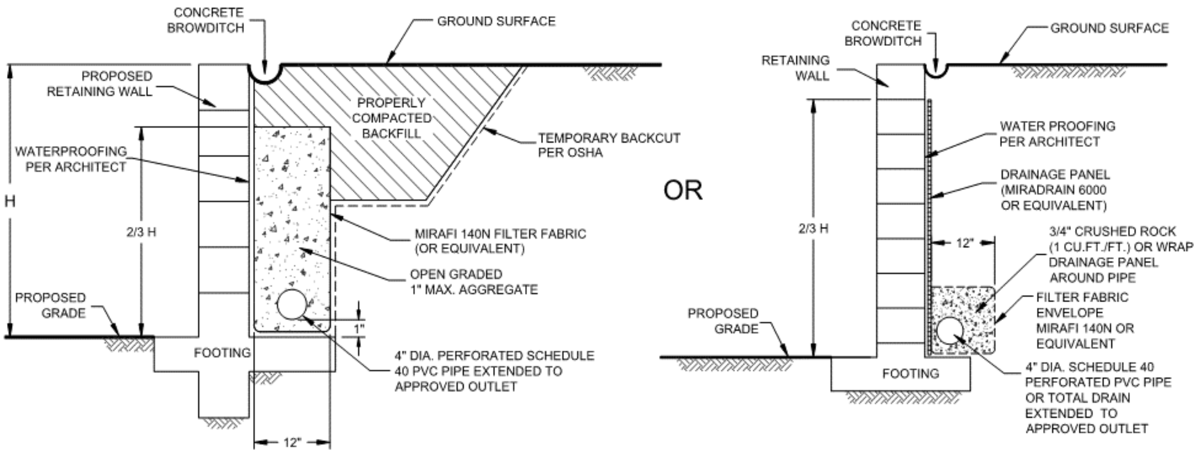
Retaining Wall Loading Diagram

8.14.3 Unrestrained walls are those that are allowed to rotate more than 0.001H (where H equals the height of the retaining portion of the wall) at the top of the wall. Where walls are restrained from movement at the top (at-rest condition), an additional uniform pressure should be added to the active soil pressure. For retaining walls subject to vehicular loads within a horizontal distance equal to two-thirds the wall height, a surcharge equivalent to 2 feet of fill soil should be added.

8.14.4 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.

8.14.5 Retaining walls should be designed to ensure stability against overturning sliding, and excessive foundation pressure. 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.

8.14.6 Drainage openings through the base of the wall (weep holes) should not be used where the seepage could be a nuisance or otherwise adversely affect the property adjacent to the base of the wall. The recommendations herein assume a properly compacted granular (EI of 90 or less) free-draining backfill material with no hydrostatic forces or imposed surcharge load. The retaining wall should be properly drained as shown in the Typical Retaining Wall Drainage Detail. If conditions different than those described are expected, or if specific drainage details are desired, Geocon Incorporated should be contacted for additional recommendations.



Typical Retaining Wall Drainage Detail

8.14.7 The retaining walls may be designed using either the active and restrained (at-rest) loading condition or the active and seismic loading condition as suggested by the structural engineer. Typically, it appears the design of the restrained condition for retaining wall loading may be adequate for the seismic design of the retaining walls. However, the active earth pressure combined with the seismic design load should be reviewed and also considered in the design of the retaining walls.

8.14.8 In general, wall foundations having should be designed in accordance with Table 8.14.2. The proximity of the foundation to the top of a slope steeper than 3:1 could impact the allowable soil bearing pressure. Therefore, retaining wall foundations should be deepened such that the bottom outside edge of the footing is at least 7 feet horizontally from the face of the slope.

TABLE 8.14.2
SUMMARY OF RETAINING WALL FOUNDATION RECOMMENDATIONS

Parameter	Value
Minimum Retaining Wall Foundation Width	12 inches
Minimum Retaining Wall Foundation Depth	12 Inches
Minimum Steel Reinforcement	Per Structural Engineer
Allowable Bearing Capacity	2,500 psf
Bearing Capacity Increase	300 psf per Foot of Depth
	300 psf per Foot of Width
Maximum Allowable Bearing Capacity	3,500 psf
Estimated Total Settlement	1 Inch
Estimated Differential Settlement	½ Inch in 40 Feet

8.14.9 The recommendations presented herein are generally applicable to the design of rigid concrete or masonry retaining walls. In the event that other types of walls (such as mechanically stabilized earth [MSE] walls, soil nail walls, or soldier pile walls) are planned, Geocon Incorporated should be consulted for additional recommendations.

8.14.10 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 retaining walls and improvements above the retaining walls should be designed to incorporate an appropriate amount of lateral deflection as determined by the structural engineer.

8.14.11 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.

8.15 Lateral Loading

8.15.1 Table 8.15 should be used to help design the proposed structures and improvements to resist lateral loads for the design of footings or shear keys. The allowable passive pressure assumes a horizontal surface extending at least 5 feet, or three times the surface generating the passive pressure, whichever is greater. The upper 12 inches of material in areas not protected by floor slabs or pavement should not be included in design for passive resistance.

TABLE 8.15
SUMMARY OF LATERAL LOAD DESIGN RECOMMENDATIONS

Parameter	Value
Passive Pressure Fluid Density	350 pcf
Coefficient of Friction (Concrete and Soil)	0.40
Coefficient of Friction (Along Vapor Barrier)	0.2 to 0.25*

* Per manufacturer’s recommendations.

8.15.2 The passive and frictional resistant loads can be combined for design purposes. The lateral passive pressures may be increased by one-third when considering transient loads due to wind or seismic forces.

8.16 Preliminary Pavement Recommendations

8.16.1 We calculated the flexible pavement sections in general conformance with the *Caltrans Method of Flexible Pavement Design* (Highway Design Manual, Section 608.4) using an estimated Traffic Index (TI) of 5.0, 5.5, 6.0, and 7.0 for parking stalls, driveways, medium truck traffic areas, and heavy truck traffic areas, respectively. The project civil engineer and owner should review the pavement designations to determine appropriate locations for pavement thickness. The final pavement sections for the parking lot should be based on the R-Value of the subgrade soil encountered at final subgrade elevation. We have assumed an

R-Value of 10 and 20 for subgrade soil. We assume the base materials will possess an R-Value of 78. Table 8.16.1 presents the preliminary flexible pavement sections.

TABLE 8.16.1
PRELIMINARY FLEXIBLE PAVEMENT SECTION

Location	Assumed Traffic Index	Assumed Subgrade R-Value	Asphalt Concrete (inches)	Class 2 Aggregate Base (inches)
Parking stalls for automobiles and light-duty vehicles	5.0	10	3	9
		20	3	7
Driveways for automobiles and light-duty vehicles	5.5	10	3	11
		20	3	9
Medium truck traffic areas	6.0	10	3.5	12
		20	3.5	10
Driveways for heavy truck traffic	7.0	10	4	14
		20	4	12

8.16.2 Prior to placing base materials, the upper 12 inches of the subgrade soil should be scarified, moisture conditioned as necessary, and recompacted to a dry density of at least 95 percent of the laboratory maximum dry density near to slightly above optimum moisture content as determined by ASTM D 1557. Similarly, the base material should be compacted to a dry density of at least 95 percent of the laboratory maximum dry density near to slightly above optimum moisture content. Asphalt concrete should be compacted to a density of at least 95 percent of the laboratory Hveem density in accordance with ASTM D 2726.

8.16.3 A rigid Portland cement concrete (PCC) pavement section should be placed in roadway aprons and cross gutters. We calculated the rigid pavement section in general conformance with the procedure recommended by the American Concrete Institute report ACI 330R-08 Guide for Design and Construction of Concrete Parking Lots using the parameters presented in Table 8.16.2.

TABLE 8.16.2
RIGID PAVEMENT DESIGN PARAMETERS

Design Parameter	Design Value
Modulus of subgrade reaction, k	50 pci
Modulus of rupture for concrete, M_R	500 psi
Traffic Category, TC	A and C
Average daily truck traffic, ADTT	10 and 100

8.16.4 Based on the criteria presented herein, the PCC pavement sections should have a minimum thickness as presented in Table 8.16.3.

TABLE 8.16.3
RIGID VEHICULAR PAVEMENT RECOMMENDATIONS

Location	Portland Cement Concrete (inches)
Automobile Parking Stalls (TC=A)	6.0
Driveways (TC=C)	7.5

8.16.5 The PCC vehicular pavement should be placed over subgrade soil that is compacted to a dry density of at least 95 percent of the laboratory maximum dry density near to slightly above optimum moisture content. This pavement section is based on a minimum concrete compressive strength of approximately 3,000 psi (pounds per square inch).

8.16.6 A thickened edge or integral curb should be constructed on the outside of concrete slabs subjected to wheel loads. The thickened edge should be 1.2 times the slab thickness or a minimum thickness of 2 inches, whichever results in a thicker edge, and taper back to the recommended slab thickness 4 feet behind the face of the slab (e.g., 6-inch and 7.5-inch-thick slabs would have an 8- and 9.5-inch-thick edge, respectively). Reinforcing steel will not be necessary within the concrete for geotechnical purposes with the possible exception of dowels at construction joints as discussed herein.

8.16.7 To control the location and spread of concrete shrinkage cracks, crack-control joints (weakened plane joints) should be included in the design of the concrete pavement slab. Crack-control joints should not exceed 30 times the slab thickness with a maximum spacing of 12 feet for 5.5-inch-thick and 15 feet for the 6.0-inch and thicker slabs and should be sealed with an appropriate sealant to prevent the migration of water through the control joint to the subgrade materials. The depth of the crack-control joints should be determined by the referenced ACI report. The depth of the crack-control joints should be at least ¼ of the slab thickness when using a conventional saw, or at least 1 inch when using early-entry saws on slabs 9 inches or less in thickness, as determined by the referenced ACI report discussed in the pavement section herein. Cuts at least ¼ inch wide are required for sealed joints, and a ⅜ inch wide cut is commonly recommended. A narrow joint width of $^{1}/_{10}$- to $^{1}/_{8}$-inch wide is common for unsealed joints.

8.16.8 To provide load transfer between adjacent pavement slab sections, a butt-type construction joint should be constructed. The butt-type joint should be thickened by at least 20 percent at the edge and taper back at least 4 feet from the face of the slab. As an alternative to the butt-type construction joint, dowelling can be used between construction joints for pavements of 7 inches or thicker. As discussed in the referenced ACI guide, dowels should consist of smooth, 1-inch-diameter reinforcing steel 14 inches long embedded a minimum of 6 inches into the slab on either side of the construction joint. Dowels should be located at the midpoint of the slab, spaced at 12 inches on center and lubricated to allow joint movement while still transferring loads. In addition, tie bars should be installed at the as recommended in Section 3.8.3 of the referenced ACI guide. The structural engineer should provide other alternative recommendations for load transfer.

8.16.9 Concrete curb/gutter should be placed on soil subgrade compacted to a dry density of at least 90 percent of the laboratory maximum dry density near to slightly above optimum moisture content. Cross-gutters that receives vehicular should be placed on subgrade soil compacted to a dry density of at least 95 percent of the laboratory maximum dry density near to slightly above optimum moisture content. Base materials should not be placed below the curb/gutter, or cross-gutters so water is not able to migrate from the adjacent parkways to the pavement sections. Where flatwork is located directly adjacent to the curb/gutter, the concrete flatwork should be structurally connected to the curbs to help reduce the potential for offsets between the curbs and the flatwork.

8.17 Site Drainage and Moisture Protection

8.17.1 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.

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

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

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

8.18 Grading and Foundation Plan Review

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

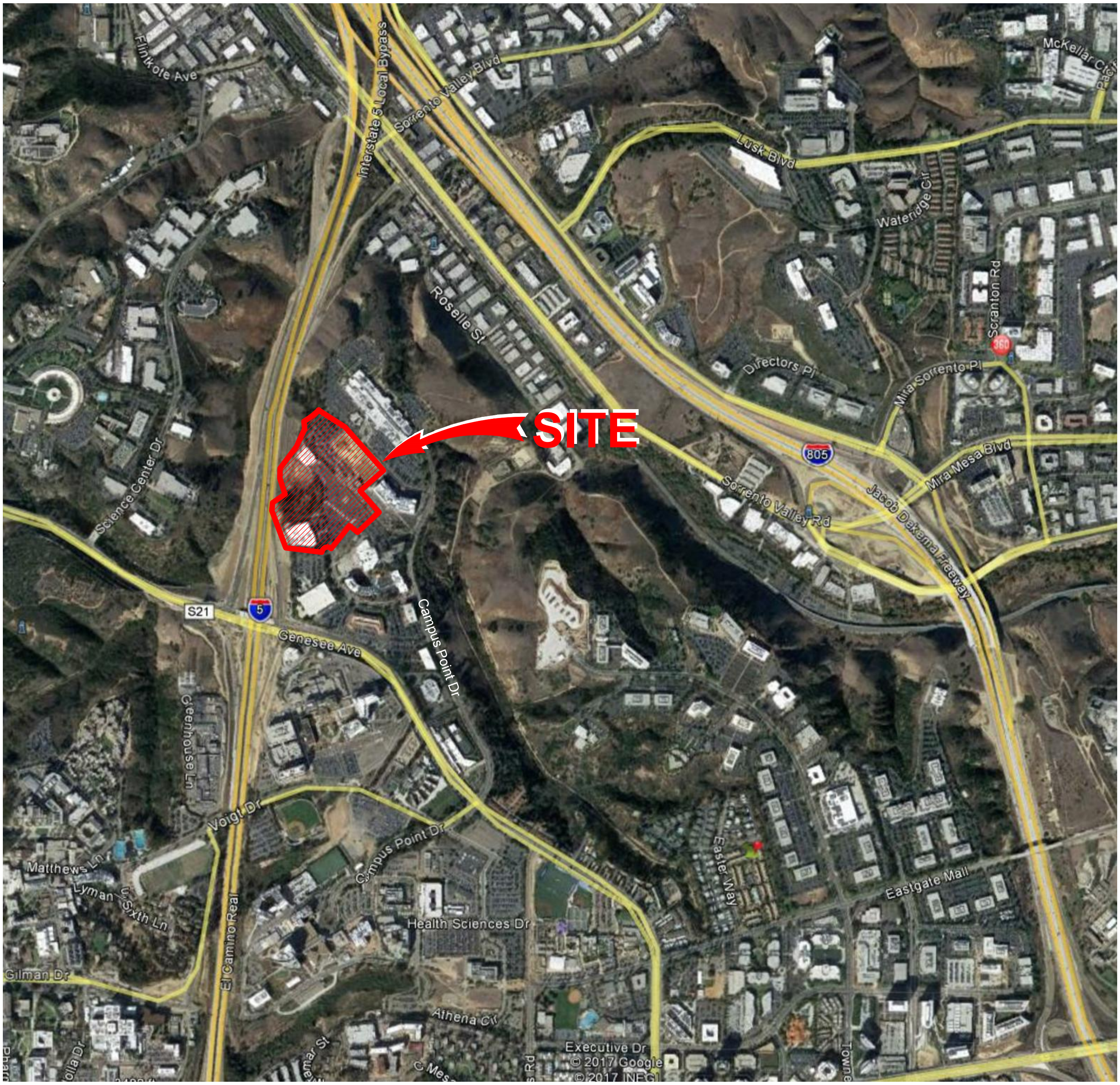
LIMITATIONS AND UNIFORMITY OF CONDITIONS

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.



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NO SCALE

VICINITY MAP

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

CAMPUS POINTE
SAN DIEGO, CALIFORNIA

ML / RA		DSK/GTYPD	DATE 10 - 31 - 2019	PROJECT NO. G2415 - 52 - 01	FIG. 1

Plotted:10/31/2019 8:41AM | By:ALVIN LADRILLONO | File Location:Y:\1_GEOTECH\G2000\G2415-52-01\2019-10-31\DETAILS\G2415-52-01 VicinityMap.dwg

GEOCON LEGEND

Qpf ....PREVIOUSLY PLACED FILL
Qudf ........UNDOCUMENTED FILL
Tsc ....SCRIPPS FORMATION (Dotted Where Buried)
Ta ....ARDATH SHALE (Dotted Where Buried)
B-12 ....APPROX. LOCATION OF BORING (This Study)
B-5 ....APPROX. LOCATION OF BORING (Geocon, January 2019)
B-16 ....APPROX. LOCATION OF BORING (Geocon, 2015/2016)
B-16 ....APPROX. LOCATION OF BORING (SCS&T, 1995)
FT-1 ....APPROX. LOCATION OF FAULT TRENCH (Geocon, 2015)
C C' ....APPROX. LOCATION OF FAULT TRENCH (SCS&T, 1995)
T-12 ....APPROX. LOCATION OF TRENCH (Geocon, 1980)
B-1 ....APPROX. LOCATION OF BORING (Geocon, 1980)
........APPROX. LOCATION OF GEOLOGIC CONTACT
(Dotted Where Buried; Queried Where Uncertain)
........APPROX. CITY OF SAN DIEGO FAULT STUDY ZONE
........CITY OF SAN DIEGO MAPPED FAULT LOCATION
(Dotted Where Buried)
3 3' ....APPROX. LOCATION OF GEOLOGIC CROSS-SECTION
........APPROX. LOCATION OF OBSERVED POTENTIALLY ACTIVE FAULT
(90)........APPROX. DEPTH TO FORMATION (In Feet)

GEOLOGIC MAP

CAMPUS POINTE
SAN DIEGO, CALIFORNIA

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

SCALE 1" = 60'
DATE 10 - 31 - 2019
PROJECT NO. G2415 - 52 - 01
FIGURE 2
SHEET 1 OF 3

Plotted:10/31/2019 7:42AM | By:ALVIN LADRILLONO | File Location:Y:\PROJECTS\G2415-52-01 Campus Point\SHEETS\G2415-52-01 GeoMap.dwg

GEOCON LEGEND

Qpf ....PREVIOUSLY PLACED FILL
Qudf ........UNDOCUMENTED FILL
Tsc ....SCRIPPS FORMATION (Dotted Where Buried)
Ta ....ARDATH SHALE (Dotted Where Buried)
........APPROX. LOCATION OF GEOLOGIC CONTACT (Dotted Where Buried; Queried Where Uncertain)
80 ....APPROX. THICKNESS OF FILL (From Existing Grade, In Feet)
1.2 ....APPROX. ESTIMATED SETTLEMENT (In Inches)

FILL THICKNESS MAP - EXISTING

CAMPUS POINTE
SAN DIEGO, CALIFORNIA

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

SCALE 1" = 60'
DATE 10 - 31 - 2019
PROJECT NO. G2415 - 52 - 01
SHEET 2 OF 3
FIGURE 3

Plotted:10/31/2019 8:11AM | By:ALVIN LADRILLONO | File Location:Y:\1_GEOTECH\G2000\G2415-52-01\2019-10-31\G2415-52-01 Fill Thickness Map (Existing).dwg

GEOCON LEGEND

- **Qpf** ....PREVIOUSLY PLACED FILL
- **Qudf** ........UNDOCUMENTED FILL
- **Tsc** ....SCRIPPS FORMATION (Dotted Where Buried)
- **Ta** ....ARDATH SHALE (Dotted Where Buried)
- ....APPROX. LOCATION OF GEOLOGIC CONTACT (Dotted Where Buried; Queried Where Uncertain)
- 80 ....APPROX. THICKNESS OF FILL (From Proposed Grade, In Feet)
- 1.2 ....APPROX. ESTIMATED SETTLEMENT (In Inches)
- 20 ....APPROX. THICKNESS OF FILL FOR SUBTERRANEAN STRUCTURES (From Proposed Grade, In Feet)
- 04 ....APPROX. ESTIMATED SETTLEMENT FOR SUBTERRANEAN STRUCTURES (In Inches)
- 3 3' ....APPROX. LOCATION OF GEOLOGIC CROSS-SECTION

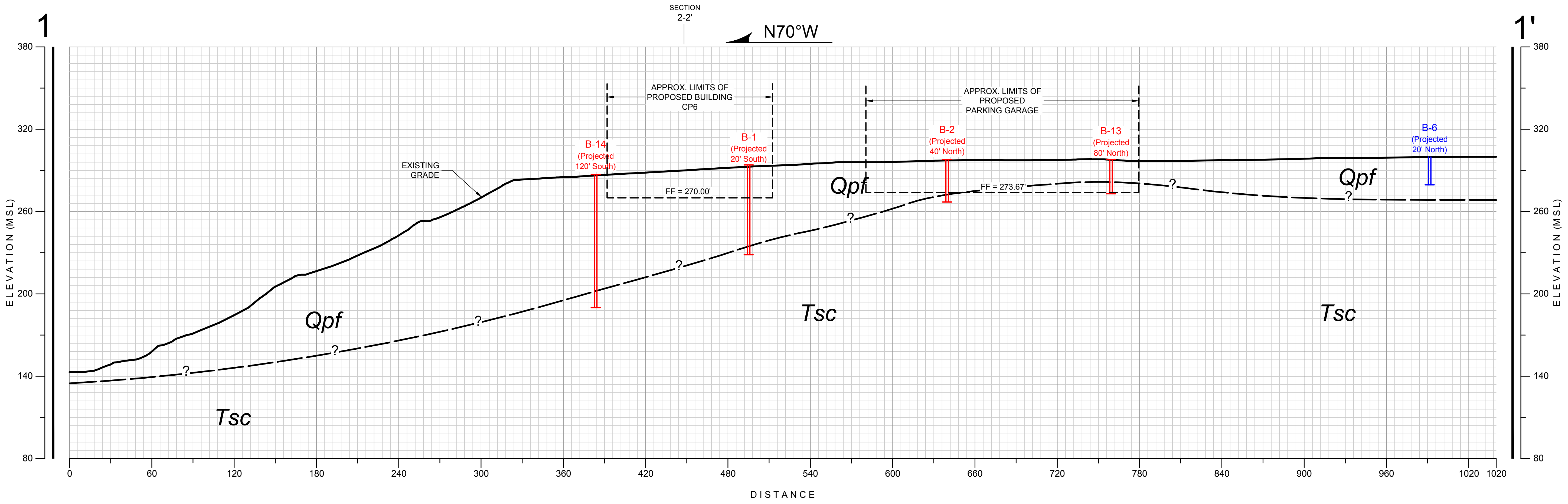
FILL THICKNESS MAP - PROPOSED

CAMPUS POINTE
SAN DIEGO, CALIFORNIA

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

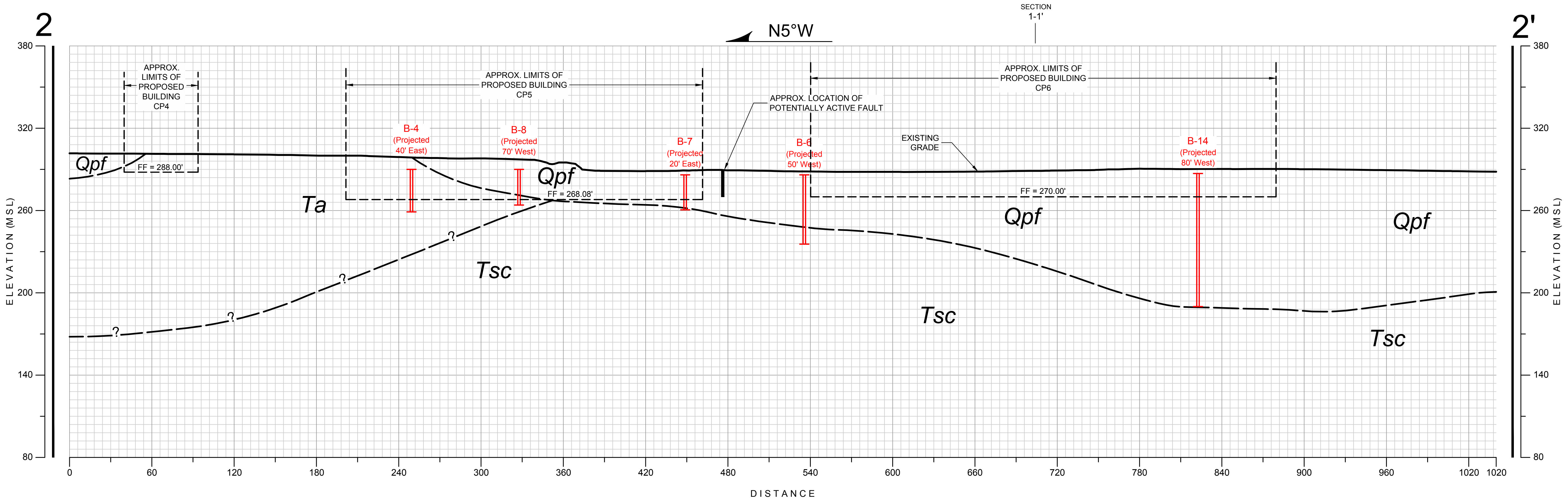
SCALE 1" = 60'	DATE 10 - 31 - 2019
PROJECT NO. G2415 - 52 - 01	FIGURE 4
SHEET 3 OF 3	

Plotted:10/31/2019 8:20AM | By:ALVIN LADRILLONO | File Location:Y:\1_GEOTECH\G2000\G2415-52-01\2019-10-31\G2415-52-01 Fill Thickness Map (Proposed).dwg



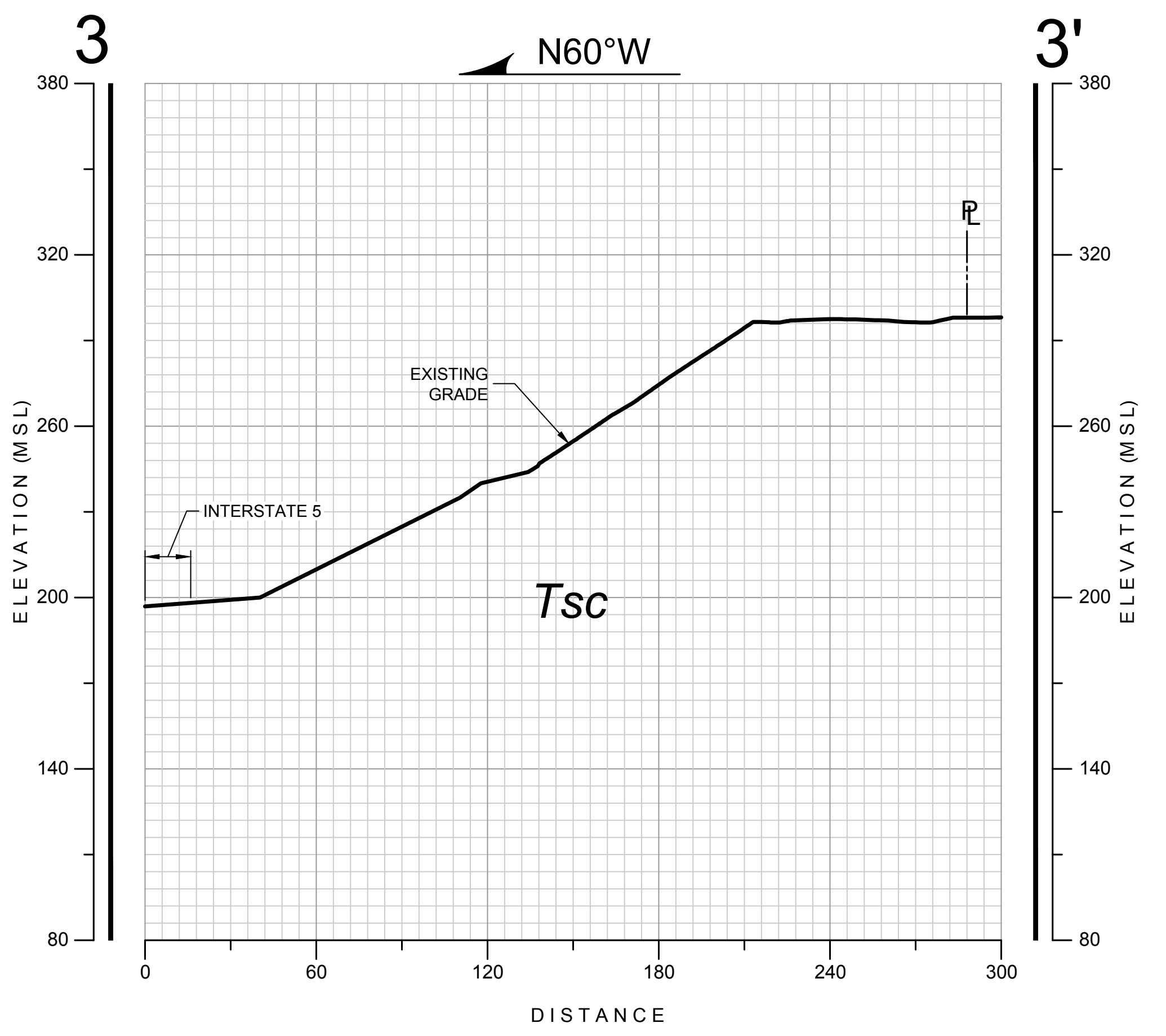
GEOLOGIC CROSS-SECTION 1-1'

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



GEOLOGIC CROSS-SECTION 2-2'

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



GEOLOGIC CROSS-SECTION 3-3'

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

Qpf ....PREVIOUSLY PLACED FILL

Tsc ....SCRIPPS FORMATION

Ta ....ARDATH SHALE

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

B-13 ....APPROX. LOCATION OF BORING (This Study)

B-6 ....APPROX. LOCATION OF BORING
(Geocon, 2016; See Appendix A)

GEOLOGIC CROSS SECTION

CAMPUS POINTE
SAN DIEGO, CALIFORNIA

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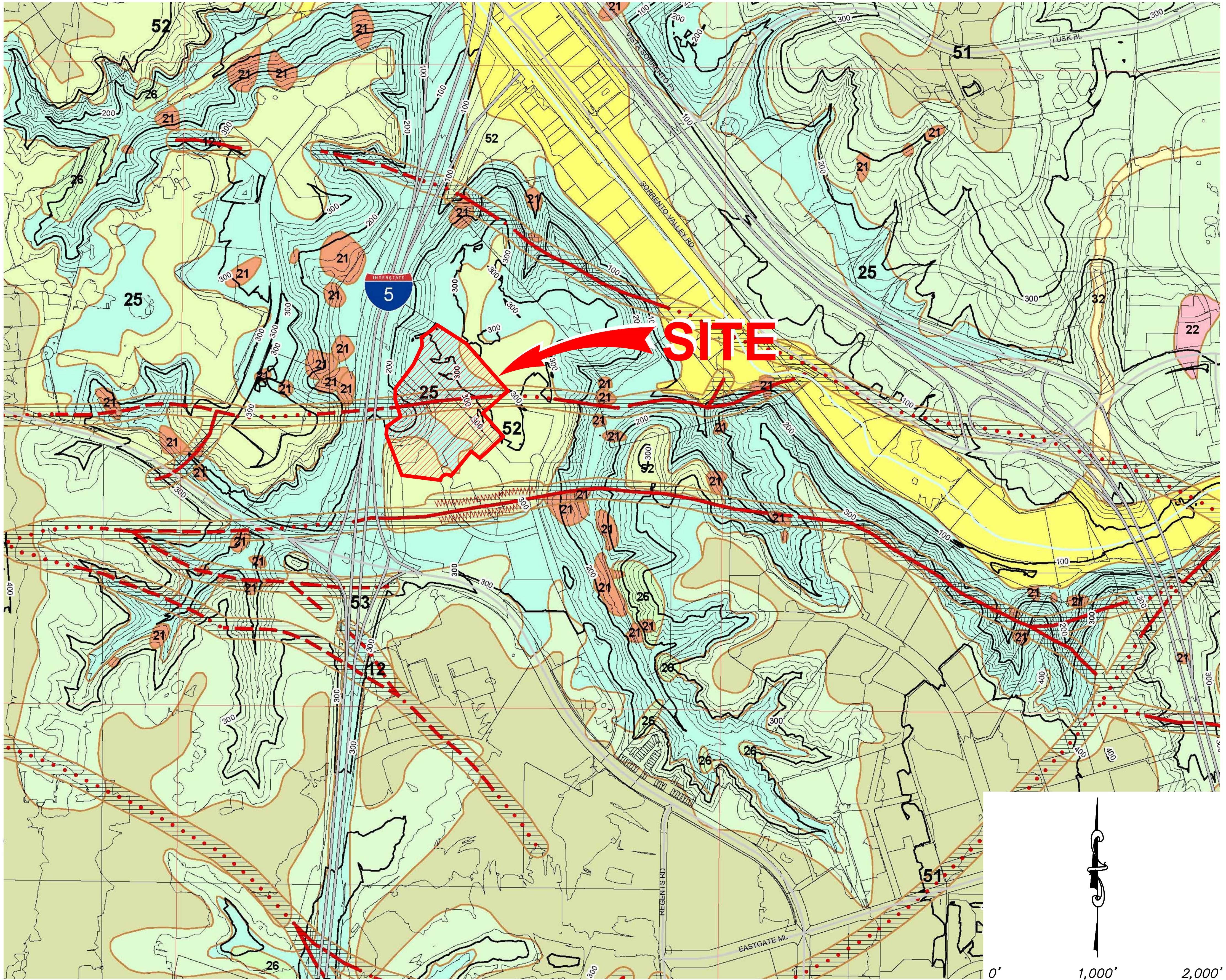
PHONE 858 558-6900 - FAX 858 558-6159

SCALE 1" = 60'
DATE 10 - 31 - 2019
PROJECT NO. G2415 - 52 - 01
FIGURE 5
SHEET 1 OF 1

Plotted:10/31/2019 8:36AM | By:ALVIN LADRILLONO | File Location:Y:\1_GEOTECH\G2000\G2415-52-01\2019-10-31\SHEETS\G2415-52-01 XSections.dwg



SOURCE: *City of San Diego Seismic Safety Study Map*

0' 1,000' 2,000'

SCALE 1"= 1,000' (On 11x17)

Geologic Hazard Categories

FAULT ZONES

- ▨ 11 Active, Alquist-Priolo Earthquake Fault Zone
- ▤ 12 Potentially Active, Inactive, Presumed Inactive, or Activity Unknown
- ■ 13 Downtown special fault zone

LANDSLIDES

- 🟧 21 Confirmed, known, or highly suspected
- ■ 22 Possible or conjectured

SLIDE-PRONE FORMATIONS

- 🟪 23 Friars: neutral or favorable geologic structure
- 🟦 24 Friars: unfavorable geologic structure
- ■ 25 Ardath: neutral or favorable geologic structure
- 🟩 26 Ardath: unfavorable geologic structure
- ■ 27 Otay, Sweetwater, and others

LIQUEFACTION

- 🟨 31 High Potential -- shallow groundwater major drainages, hydraulic fills
- ■ 32 Low Potential -- fluctuating groundwater minor drainages

COASTAL BLUFFS

- ■ 41 Generally unstable Numerous landslides, high steep bluffs, severe erosion, unfavorable geologic structure
- ■ 42 Generally unstable Unfavorable bedding plains, high erosion
- ■ 43 Generally unstable Unfavorable jointing, local high erosion
- ■ 44 Moderately stable Mostly stable formations, local high erosion
- ■ 45 Moderately stable Some minor landslides, minor erosion
- 🟫 46 Moderately stable Some unfavorable geologic structure, minor or no erosion
- 🟧 47 Generally stable Favorable geologic structure, minor or no erosion, no landslides
- ■ 48 Generally stable Broad beach areas, developed harbor

OTHER TERRAIN

- ■ 51 Level mesas -- underlain by terrace deposits and bedrock nominal risk
- ■ 52 Other level areas, gently sloping to steep terrain, favorable geologic structure, Low risk
- ■ 53 Level or sloping terrain, unfavorable geologic structure, Low to moderate risk
- 🟩 54 Steeply sloping terrain, unfavorable or fault controlled geologic structure, Moderate risk
- ■ 55 Modified terrain (graded sites) Nominal risk

Water (Bays and Lakes)

- ■

FAULTS

- ∧∨ Fault
- ⁄⁄ Inferred Fault
- ∴ Concealed Fault
- ⋀⋀⋀ Shear Zone

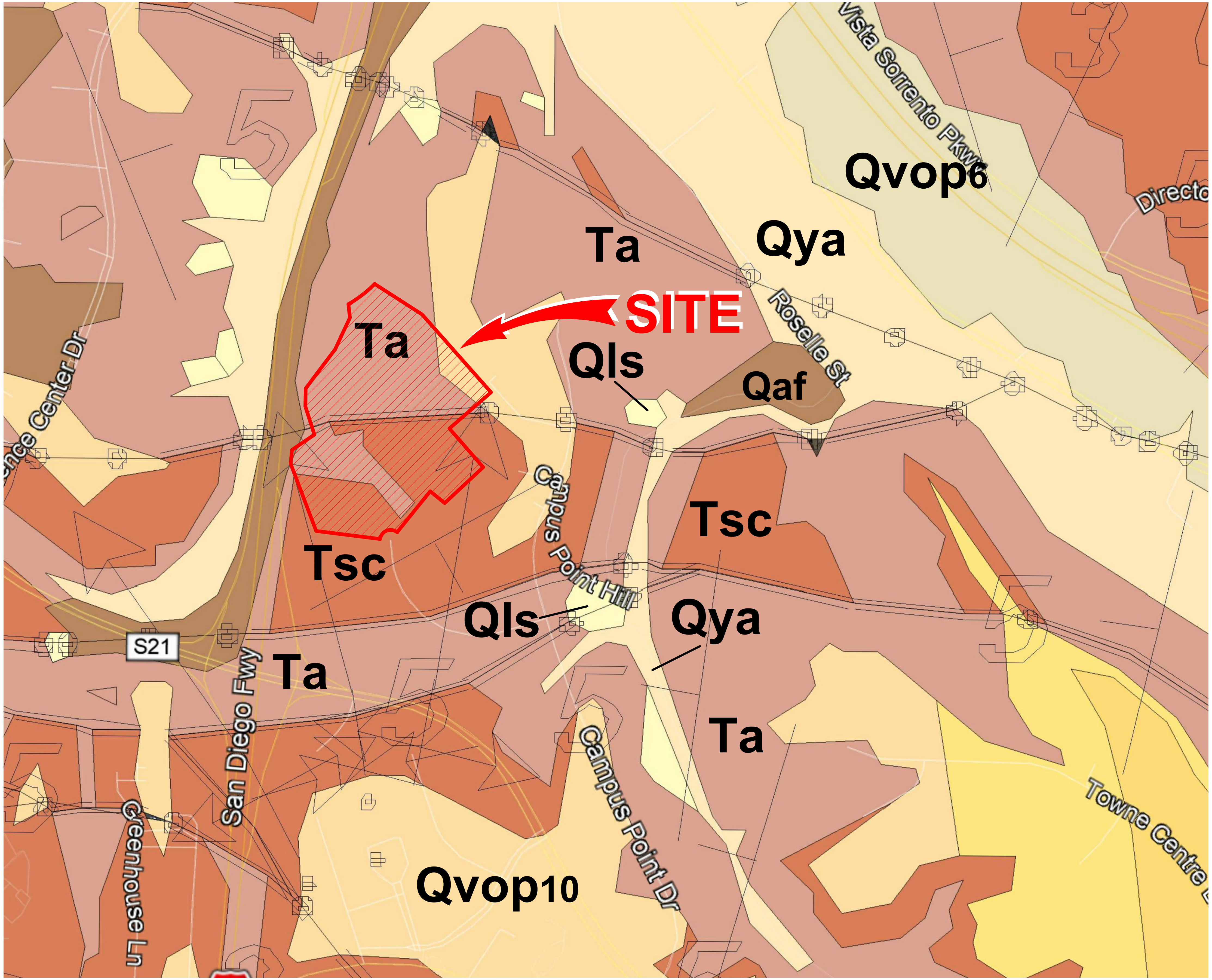
SAN DIEGO SEISMIC SAFETY MAP

CAMPUS POINTE
SAN DIEGO, CALIFORNIA

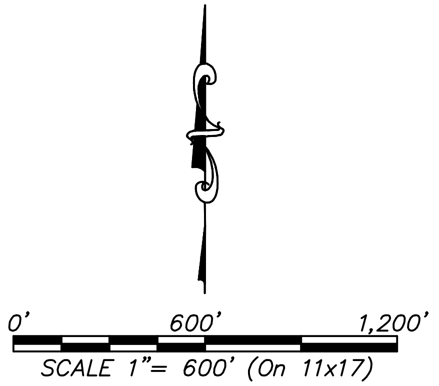
GEOCON
INCORPORATED
GEOTECHNICAL CONSULTANTS
6960 FLANDERS DRIVE - SAN DIEGO, CALIFORNIA 92121 - 2974
PHONE 858 558-6900 - FAX 858 558-6159

SCALE 1" = 1,000'
DATE 10 - 31 - 2019
PROJECT NO. G2415 - 52 - 01
FIGURE 6
SHEET 1 OF 1

Plotted:10/31/2019 8:39AM | By:ALVIN LADRILLONO | File Location:Y:\1_GEOTECH\G2000\G2415-52-01\2019-10-31\SHEETS\G2415-52-01 SeismicSafetyHazardMap.dwg



SOURCE: Kennedy P. Michael and Tan S. Siang, 2005, *Geologic Map of San Diego 30'x60' Quadrangle, California*
U.S. Geological Survey, Department of Earth Sciences, University of California, Riverside



GEOCON LEGEND

Qaf	ARTIFICIAL FILL
Qls	LANDSLIDE DEPOSITS UNDIVIDED
Qya	YOUNG ALLUVIAL FLOOD PLAIN DEPOSITS
Qvop10	VERY OLD PARALIC DEPOSITS, UNIT 10
Qvop6	VERY OLD PARALIC DEPOSITS, UNIT 6
Tsc	SCRIPPS FORMATION
Ta	ARDATH SHALE

REGIONAL GEOLOGIC MAP

CAMPUS POINTE
SAN DIEGO, CALIFORNIA

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

SCALE 1" = 600'
DATE 10 - 31 - 2019
PROJECT NO. G2415 - 52 - 01
FIGURE 7
SHEET 1 OF 1

Plotted:10/31/2019 8:38AM | By:ALVIN LADRILLONO | File Location:Y:\1_GEOTECH\G2000\G2415-52-01\2019-10-31\SHEETS\G2415-52-01 RegionalGeologicMap.dwg

APPENDIX A

APPENDIX A

FIELD INVESTIGATION

We performed the drilling operations on July 15 through 20, 2019. Borings extended to maximum depth of approximately 97 feet. The locations of the current exploratory borings are shown on the Geologic Map, Figure 2. The boring logs are presented in this Appendix. We located the borings in the field using a measuring tape and existing reference points; therefore, actual boring locations may deviate slightly.

The geotechnical borings were drilled by Baja Drilling to depths ranging from approximately 6 to 97 feet below existing grade using a CME 95 drill rig equipped with hollow-stem augers. The infiltration-test borings were drilled to depths of approximately 6 to 11 feet.

We obtained samples during our subsurface exploration in the borings using either a California sampler or a Standard Penetration Test (SPT) sampler. Both samplers are composed of steel and are driven to obtain ring samples. The California sampler has an inside diameter of 2.5 inches and an outside diameter of 3 inches. Up to 18 rings are placed inside the sampler that is 2.4 inches in diameter and 1 inch in height. The SPT sampler has an inside diameter of 1.5 inches and an outside diameter of 2 inches. We obtained ring samples at appropriate intervals, placed them in moisture-tight containers, and transported them to the laboratory for testing. The type of sample is noted on the exploratory boring logs.

The California sampler and SPT sampler were driven 12 and 18 inches, respectively. The sampler is connected to A rods and driven into the bottom of the excavation using a 140-pound hammer with a 30-inch drop. Blow counts are recorded for every 6 inches the sampler is driven. The penetration resistances shown on the boring logs are shown in terms of blows per foot. The values indicated on the boring logs are the sum of the last 12 inches of the sampler. If the sampler was not driven for 12 inches, an approximate value is calculated in term of blows per foot or the final 6-inch interval is reported. These values are not to be taken as N-values as adjustments have not been applied. We estimated elevations shown on the boring logs either from a topographic map or by using a benchmark. Each excavation was backfilled as noted on the boring logs.

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

PROJECT NO. G2415-52-01

BORING B 1

ELEV. (MSL.) **294'** DATE COMPLETED **07-15-2019**

EQUIPMENT **CME 95** BY: **A. REKANI**

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" **ASPHALT CONCRETE** over 3" **BASE**			
				SM	**PREVIOUSLY PLACED FILL (Qpf)** Medium dense, moist, light reddish brown mottled with dark gray, Silty, fine to medium SAND			
2								
4								
	B1-1 ■					43	100.0	21.3
6	B1-2 ▨							
8								
10	B1-3 ■				-Becomes yellowish brown	23	100.7	23.9
12								
14								
	B1-4 ■					29	100.5	23.2
16								
18								
20	B1-5 ■				-Trace gravel	30	101.6	24.0
22								
24								
26								
28								

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

G2415-52-01.GPJ

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

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

GEOCON

PROJECT NO. G2415-52-01

BORING B 1

ELEV. (MSL.) **294'** DATE COMPLETED **07-15-2019**

EQUIPMENT **CME 95** BY: **A. REKANI**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
30	B1-6			SM		33	101.1	24.2
32								
34								
36								
38								
40	B1-7				-Becomes dense	53	109.8	16.0
42								
44								
46								
48								
50	B1-8				-Becomes medium dense, fine-grained	49	107.7	11.0
52								
54								
56								
58								

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

G2415-52-01.GPJ

SAMPLE SYMBOLS

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

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

BORING B 1

ELEV. (MSL.) **294'** DATE COMPLETED **07-15-2019**

EQUIPMENT **CME 95** BY: **A. REKANI**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
60	B1-9			SM	**SCRIPPS FORMATION (Tsc)** Very dense, moist, yellowish brown-gray, fine-grained SANDSTONE; oxide staining	50/2"	101.6	15.2
62								
64								
	B1-10					50/5"		9.3
					BORING TERMINATED AT 65.5 FEET No groundwater encountered			

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

G2415-52-01.GPJ

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

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

PROJECT NO. G2415-52-01

BORING B 2

ELEV. (MSL.) 298' DATE COMPLETED 07-15-2019

EQUIPMENT CME 95 BY: A. REKANI

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0					3" **ASPHALT CONCRETE** over 4" **BASE**			
				SM/SC	**PREVIOUSLY PLACED FILL (Qpf)** Medium dense to dense, moist, mottled light reddish brown and olive gray, Silty/Clayey SAND			
2								
4								
	■ B2-1					32	101.8	21.5
6	⊠ B2-2							
8								
10	■ B2-3					31	107.8	16.8
12								
14								
	■ B2-4					28	107.2	20.2
16								
18								
20	■ B2-5					51	108.6	16.2
					-Becomes dense; trace gravels			
22								
24								
	■ B2-6					51	116.2	15.4
26				SM	**SCRIPPS FORMATION (Tsc)** Dense, moist, gray to brown gray, Silty, fine- to medium-grained SANDSTONE			
28								

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

G2415-52-01.GPJ

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

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

GEOCON

BORING B 2

ELEV. (MSL.) 298' DATE COMPLETED 07-15-2019

EQUIPMENT CME 95 BY: A. REKANI

DEPTH IN FEET	SAMPLE NO.		LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
30	B2-7	■	⁙		SM		88/10"	104.3	6.3
						BORING TERMINATED AT 31 FEET No groundwater encountered			

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

G2415-52-01.GPJ

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

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

PROJECT NO. G2415-52-01

BORING B 3

ELEV. (MSL.) **294'** DATE COMPLETED **07-15-2019**

EQUIPMENT **CME 95** BY: **A. REKANI**

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" **ASPHALT CONCRETE** over 5" **BASE**			
				SM	**PREVIOUSLY PLACED FILL (Qpf)** Medium dense, moist, mottled light reddish brown and grayish brown, Silty, fine to medium SAND			
2								
4								
	■ B3-1					30	103.0	21.5
6	⊠ B3-2							
8								
10	■ B3-3					33	100.8	22.7
12								
14								
	■ B3-4					31	106.4	19.7
16								
18								
20	■ B3-5				-Becomes reddish to yellowish brown	36	111.3	16.1
22								
24								
	■ B3-6					36	108.0	16.2
26								
28								

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

G2415-52-01.GPJ

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

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

GEOCON

PROJECT NO. G2415-52-01

BORING B 3

ELEV. (MSL.) **294'** DATE COMPLETED **07-15-2019**

EQUIPMENT **CME 95** BY: **A. REKANI**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
30	B3-7 ■	░		SM		63	111.7	15.8
					BORING TERMINATED AT 31 FEET No groundwater encountered			

Figure A-3,
Log of Boring B 3, Page 2 of 2

G2415-52-01.GPJ

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

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

GEOCON

PROJECT NO. G2415-52-01

BORING B 4

ELEV. (MSL.) **290'** DATE COMPLETED **07-15-2019**

EQUIPMENT **CME 95** BY: **A. REKANI**

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" **ASPHALT CONCRETE** over 8" **BASE**			
				SM	**PREVIOUSLY PLACED FILL (Qpf)** Medium dense, moist, mottled light reddish brown and grayish brown, Silty, fine to medium SAND			
2								
4								
	■ B4-1					34	110.6	17.0
6	▩ B4-2							
8								
10	■ B4-3					38	108.1	18.8
12								
14								
	■ B4-4					23	106.3	22.7
16								
18								
20	■ B4-5				-Tough drilling, trace gravels	48	105.0	19.9
22								
24								
	☐				-No recovery at 25 feet due to rock in tip			
26	■ B4-6					22	103.4	17.4
28								

Figure A-4,
Log of Boring B 4, Page 1 of 2

G2415-52-01.GPJ

SAMPLE SYMBOLS

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

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

GEOCON

PROJECT NO. G2415-52-01

BORING B 4

ELEV. (MSL.) **290'** DATE COMPLETED **07-15-2019**

EQUIPMENT **CME 95** BY: **A. REKANI**

DEPTH IN FEET	SAMPLE NO.		LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
30	B4-7	■	▨		SM		61	111.1	16.4
						BORING TERMINATED AT 31 FEET No groundwater encountered			

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

G2415-52-01.GPJ

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

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

GEOCON

PROJECT NO. G2415-52-01

BORING B 5

ELEV. (MSL.) **295'** DATE COMPLETED **07-16-2019**

EQUIPMENT **CME 95** BY: **A. REKANI**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0					3" **ASPHALT CONCRETE** over 4" **BASE**			
				SM	**PREVIOUSLY PLACED FILL (Qpf)** Medium dense, moist, light reddish brown to yellowish brown, mottled with grayish brown, Silty, fine to medium SAND			
2								
4								
	B5-1					34	105.0	21.0
6	B5-2							
8								
10	B5-3					35	107.2	10.7
12								
14								
	B5-4					31	108.3	15.5
16								
18								
20	B5-5					48	107.5	17.8
22								
24								
	B5-6				-Becomes dense, yellowish brown	50	105.6	18.9
26								
28								

Figure A-5,
Log of Boring B 5, Page 1 of 3

G2415-52-01.GPJ

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

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

PROJECT NO. G2415-52-01

BORING B 5

ELEV. (MSL.) 295' DATE COMPLETED 07-16-2019

EQUIPMENT CME 95 BY: A. REKANI

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
30				SM				
	B5-7					49	109.4	18.6
32								
34								
36								
38								
40	B5-8				-Becomes very dense, trace gravels	80/11"	114.5	13.9
42								
44								
46								
48								
50	B5-9				-Becomes dense	60	109.0	20.5
52								
54								
56								
58								

Figure A-5,
Log of Boring B 5, Page 2 of 3

G2415-52-01.GPJ

SAMPLE SYMBOLS

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

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

GEOCON

PROJECT NO. G2415-52-01

BORING B 5

ELEV. (MSL.) **295'** DATE COMPLETED **07-16-2019**

EQUIPMENT **CME 95** BY: **A. REKANI**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
60	B5-10 ■			SM	**SCRIPPS FORMATION (Tsc)** Very dense, moist, yellowish brown, Silty, fine- to medium-grained SANDSTONE	50/5"	108.4	11.9
62								
64								
66								
68								
70	B5-11 ■				-Poor recovery	50/5"		7.9

BORING TERMINATED AT 70.5 FEET
No groundwater encountered

Figure A-5,
Log of Boring B 5, Page 3 of 3

G2415-52-01.GPJ

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

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

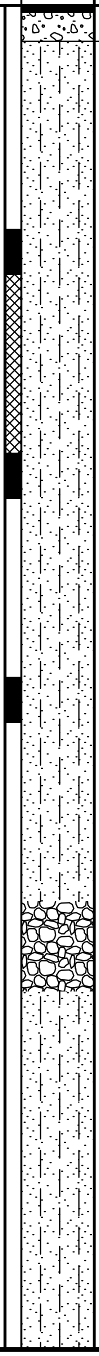
GEOCON

PROJECT NO. G2415-52-01

BORING B 6

ELEV. (MSL.) **286'** DATE COMPLETED **07-16-2019**

EQUIPMENT **CME 95** BY: **A. REKANI**



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" **ASPHALT CONCRETE** over 8" **BASE**			
				SM	**PREVIOUSLY PLACED FILL (Qpf)** Medium dense, moist, light reddish brown, yellowish brown mottled with gray, Silty, fine to medium SAND			
2								
4								
	B6-1					35	105.3	20.9
6	B6-2							
8								
10	B6-3					29	101.7	21.5
12								
14								
	B6-4					21	100.8	22.7
16								
18								
20					-No recovery at 20 feet; drilled to 21 feet, no recovery -Gravel and cobble sized rock fragments from 20'-22'			
22								
24								
26								
28								

Figure A-6,
Log of Boring B 6, Page 1 of 2

G2415-52-01.GPJ

SAMPLE SYMBOLS

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

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

GEOCON

BORING B 6

ELEV. (MSL.) 286' DATE COMPLETED 07-16-2019

EQUIPMENT CME 95 BY: A. REKANI

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
30				SM				
	B6-5 ■					36	104.1	21.3
32								
34								
36								
38								
40	B6-6 ■			SM	**SCRIPPS FORMATION (Tsc)** Very dense, moist, light yellowish brown to grayish brown, Silty, fine- to medium-grained SANDSTONE	50/6"	100.0	9.3
42								
44								
46								
48								
50	B6-7 ■					50/6"	101.9	7.8

BORING TERMINATED AT 50.5 FEET
No groundwater encountered

Figure A-6,
Log of Boring B 6, Page 2 of 2

G2415-52-01.GPJ

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

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

GEOCON

PROJECT NO. G2415-52-01

BORING B 7

ELEV. (MSL.) 286' DATE COMPLETED 07-16-2019

EQUIPMENT CME 95 BY: A. REKANI

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0					3" **ASPHALT CONCRETE** over 8" **BASE**			
				SM	**PREVIOUSLY PLACED FILL (Qpf)** Medium dense, moist, light yellowish to reddish brown mottled with gray, Silty, fine to medium SAND			
2								
4								
	B7-1					35	104.5	21.5
6	B7-2							
8								
10								
	B7-3					30	105.4	21.0
12								
14								
					-No recovery at 15 feet -Becomes very dense, gravel and cobble encountered	50/2"		
16								
					-No recovery	50/2"		
18								
20	B7-4				**SCRIPPS FORMATION (Tsc)** Very dense, moist, light yellowish brown to grayish brown, Silty, fine- to medium-grained SANDSTONE	92/9"		12.4
22								
24								
					-No recovery	50/5"		
					BORING TERMINATED AT 25.5 FEET No groundwater encountered			

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

G2415-52-01.GPJ

SAMPLE SYMBOLS

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

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

GEOCON

PROJECT NO. G2415-52-01

BORING B 8

ELEV. (MSL.) **290'** DATE COMPLETED **07-16-2019**

EQUIPMENT **CME 95** BY: **A. REKANI**

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" **ASPHALT CONCRETE** over 8" **BASE**			
				SM	**PREVIOUSLY PLACED FILL (Qpf)** Medium dense, reddish brown, Silty, fine SAND with trace gravel			
2								
4								
	B8-1					32	97.9	23.2
6	B8-2							
8								
10	B8-3					28	103.0	21.6
12								
14								
	B8-4			CL	**TOPSOIL (Qt)** Firm, moist, dark gray to black, Silty, Sandy CLAY; trace rootlets, wood; organic smell	13	91.3	23.2
16								
18								
20	B8-5			ML	**ARDATH SHALE (Ta)** Hard, damp, yellowish brown to reddish brown mottled with grayish brown, Sandy SILTSTONE	92/9"	122.3	17.4
22								
24								
	B8-6					97	109.7	20.1
26					BORING TERMINATED AT 26 FEET No groundwater encountered			

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

G2415-52-01.GPJ

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

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

GEOCON

BORING B 9

ELEV. (MSL.) **302'** DATE COMPLETED **07-17-2019**

EQUIPMENT **CME 95** BY: **A. REKANI**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0	B9-1			SM	**PREVIOUSLY PLACED FILL (Qpf)** Medium dense, moist, light yellowish to reddish brown, Silty, fine to medium SAND			
2								
4	B9-2			ML	**ARDATH SHALE (Ta)** Hard, damp, mottled reddish to grayish brown, fine Sandy SILTSTONE; trace gravel; laminated			
	B9-3					95/8"	110.1	18.4
6								
8								
10	B9-4					77/11"		
12								
14								
16								
18								
20	B9-5				-Driller reports difficult drilling	86/10"		
22								
24								
26								
28								

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

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

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

BORING B 9

ELEV. (MSL.) **302'** DATE COMPLETED **07-17-2019**

EQUIPMENT **CME 95** BY: **A. REKANI**

DEPTH IN FEET	SAMPLE NO.		LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
30	B9-6	■	▥		ML		50/6"		
						BORING TERMINATED AT 31 FEET No groundwater encountered			

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.

GEOCON

PROJECT NO. G2415-52-01

BORING B 10

ELEV. (MSL.) **301'** DATE COMPLETED **07-17-2019**

EQUIPMENT **CME 95** BY: **A. REKANI**

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	**PREVIOUSLY PLACED FILL (Qpf))** Loose, damp, yellowish brown, Silty, fine SAND with trace gravel			
	B10-1 ☒			ML	**ARDATH SHALE (Ta)** Very stiff, damp to moist, mottled yellowish to grayish brown, Clayey SILTSTONE			
2								
4								
	B10-2 ■			109.1	-Becomes hard	78/11"	18.4	
6					BORING TERMINATED AT 6 FEET No groundwater encountered			

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

G2415-52-01.GPJ

SAMPLE SYMBOLS

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

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

GEOCON

BORING B 11

ELEV. (MSL.) **300'** DATE COMPLETED **07-17-2019**

EQUIPMENT **CME 95** BY: **A. REKANI**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0	B11-1			SM	**PREVIOUSLY PLACED FILL (Qpf)** Medium dense, damp to moist, light yellowish brown, Silty, fine SAND; trace gravel			
2								
4				SM	**SCRIPPS FORMATION (Tsc)** Very dense, moist, light yellowish gray, Silty, fine- to medium-grained SANDSTONE; weakly cemented; friable			
	B11-2					82/11"	107.0	8.0
6								
8								
				ML	**ARDATH SHALE (Ta)** Hard, damp to moist, mottled yellowish to grayish brown, Sandy SILTSTONE			
10	B11-3					90/8"	112.1	15.9

BORING TERMINATED AT 11 FEET
No groundwater encountered

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

G2415-52-01.GPJ

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

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

PROJECT NO. G2415-52-01

BORING B 12

ELEV. (MSL.) 300'   DATE COMPLETED 07-17-2019

EQUIPMENT CME 95   BY: A. REKANI

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" **ASPHALT CONCRETE** over 6" **BASE**			
				SM	**SCRIPPS FORMATION (Tsc)** Very dense, moist, light reddish brown to yellowish gray, Silty, fine- to medium-grained SANDSTONE; weakly cemented; friable			
2	B12-1 ⊠							
4								
	B12-2 ■					98/9"	111.5	9.4
6								
8								
10								
	B12-3 ■					50/2"	112.7	8.7
					BORING TERMINATED AT 11 FEET No groundwater encountered			

Figure A-12,
Log of Boring B 12, Page 1 of 1

G2415-52-01.GPJ

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

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

GEOCON

PROJECT NO. G2415-52-01

BORING B 13

ELEV. (MSL.) **298'** DATE COMPLETED **07-17-2019**

EQUIPMENT **CME 95** BY: **A. REKANI**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0					3" **ASPHALT CONCRETE** over 12" **BASE**			
				SM	**PREVIOUSLY PLACED FILL (Qpf)** Medium dense, moist, yellowish to reddish brown, Silty, fine SAND with trace gravel			
2								
4								
	B13-1					39	105.6	19.8
6	B13-2							
					-Becomes very dense			
8								
10	B13-3					22	105.0	9.3
12								
14				SM	**SCRIPPS FORMATION (Tsc)** Very dense, moist, yellowish to grayish brown, Silty, fine-grained SANDSTONE; strongly cemented			
						50/2"		
16								
	B13-4					50/5"	102.9	21.6
18								
20								
22								
					-Driller reports difficult drilling			
24								
					-No recovery	50/1"		
					BORING TERMINATED AT 25 FEET No groundwater encountered			

Figure A-13,
Log of Boring B 13, Page 1 of 1

G2415-52-01.GPJ

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

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

GEOCON

PROJECT NO. G2415-52-01

BORING B 14

ELEV. (MSL.) **287'** DATE COMPLETED **07-18-2019**

EQUIPMENT **CME 95** BY: **A. REKANI**

DEPTH IN FEET: 0, 2, 4, 6, 8, 10, 12, 14, 16, 18, 20, 22, 24, 26, 28

SAMPLE NO.	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
		3" **ASPHALT CONCRETE** over 7" **BASE**			
	SM	**PREVIOUSLY PLACED FILL (Qpf)** Medium dense, moist, mottled yellowish brown and grayish brown, Silty, fine to medium SAND with trace gravel			
B14-1			46	116.0	11.9
B14-2					
B14-3	SC	Medium dense, moist, mottled yellowish to reddish brown and grayish brown, Clayey SAND with trace gravel sized rock fragments	24	102.6	23.1
B14-4	ML	Very stiff, moist, mottled yellowish brown to grayish brown, Sandy SILT	46	109.7	20.0
B14-5	SM	Medium dense, moist, mottled yellowish to reddish brown and grayish brown, Silty, fine SAND	42	110.2	19.6
B14-6			35	106.9	20.9

Figure A-14,
Log of Boring B 14, Page 1 of 4

G2415-52-01.GPJ

SAMPLE SYMBOLS

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

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

GEOCON

PROJECT NO. G2415-52-01

BORING B 14

ELEV. (MSL.) **287'** DATE COMPLETED **07-18-2019**

EQUIPMENT **CME 95** BY: **A. REKANI**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
30	▨			SM	-No recovery; rock in tip	72		
32								
34								
	■ B14-7				-Becomes dense, yellowish brown	63	108.3	17.9
36								
38								
40	■ B14-8					67	100.5	22.7
42								
44								
46								
48								
					-Auger chattering/bouncing			
50	■ B14-9			SC	Medium dense, moist, mottled yellowish-reddish brown gray brown, Clayey, fine SAND with few gravel	25	107.1	22.7
52								
54								
56								
58								

Figure A-14,
Log of Boring B 14, Page 2 of 4

G2415-52-01.GPJ

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

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

GEOCON

BORING B 14

ELEV. (MSL.) **287'** DATE COMPLETED **07-18-2019**

EQUIPMENT **CME 95** BY: **A. REKANI**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
60	B14-10			SC	Dense, moist, mottled dark brown, black and grayish brown, Clayey, fine to medium SAND with trace gravel; trace visible organics; trace charcoal	50	114.5	18.0
	B14-11							
62								
64								
66								
68								
70	B14-12				-Increase in silt and gravel	55	109.1	20.6
72								
74								
76								
78								
80	B14-13				-Becomes very dense	85/11"	115.1	11.6
	B14-15							
82								
84								
86								
88								

Figure A-14,
Log of Boring B 14, Page 3 of 4

G2415-52-01.GPJ

SAMPLE SYMBOLS

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

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

GEOCON

PROJECT NO. G2415-52-01

BORING B 14

ELEV. (MSL.) **287'** DATE COMPLETED **07-18-2019**

EQUIPMENT **CME 95** BY: **A. REKANI**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
90	B14-14			SM	**SCRIPPS FORMATION (Tsc)** Very dense, moist, yellowish to grayish brown, Silty, fine-grained SANDSTONE; weakly cemented; friable	50/3"		6.4
92								
					-Gravel from 93-97 feet			
94								
96						50/1"		
					-No recovery			

BORING TERMINATED AT 97 FEET
DUE TO REFUSAL ON ROCK
No groundwater encountered

Figure A-14,
Log of Boring B 14, Page 4 of 4

G2415-52-01.GPJ

SAMPLE SYMBOLS

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

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

PROJECT NO. G2415-52-01

BORING B 15

ELEV. (MSL.) **286'** DATE COMPLETED **07-20-2019**

EQUIPMENT **CME 95** BY: **A. REKANI**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0					3" **ASPHALT CONCRETE** over 10" **BASE**			
				SM	**PREVIOUSLY PLACED FILL (Qpf)** Medium dense, moist, yellowish to grayish brown, Silty, fine to medium SAND			
2								
4								
	B15-1					36	109.9	16.0
6	B15-2							
8								
10	B15-3				-Becomes dense, trace gravel	53	102.3	18.9
12								
14								
	B15-4			ML	Stiff, moist, mottled reddish brown and olive brown, Sandy SILT	24	105.6	20.3
16								
18								
20				SM	Medium dense, moist, yellowish to reddish brown, Silty, fine to medium SAND with trace gravel	43		
22								
24								
26								
28								

Figure A-15,
Log of Boring B 15, Page 1 of 3

G2415-52-01.GPJ

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

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GEOCON

PROJECT NO. G2415-52-01

BORING B 15

ELEV. (MSL.) **286'** DATE COMPLETED **07-20-2019**

EQUIPMENT **CME 95** BY: **A. REKANI**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
30	▒			SM	-No recovery; rock in tip	56		
32	B15-5 ■			ML	Stiff, moist, yellowish to reddish brown, Sandy SILT	33	108.3	20.3
34								
36								
38								
40	B15-6 ■					50	109.2	19.5
					-Becomes very stiff			
42								
44								
	B15-7 ⊠							
46								
48								
50	B15-8 ■					24	103.4	22.9
				CL	Firm, moist to wet, dark brown to black, highly plastic CLAY with trace organics (observed in tip)			
52								
54				ML	Stiff, moist, mottled yellowish-reddish brown, fine Sandy SILT			
56								
58								
			▽		-Seepage encountered at 59'			

Figure A-15,
Log of Boring B 15, Page 2 of 3

G2415-52-01.GPJ

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

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

GEOCON

PROJECT NO. G2415-52-01

BORING B 15

ELEV. (MSL.) **286'** DATE COMPLETED **07-20-2019**

EQUIPMENT **CME 95** BY: **A. REKANI**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
60	B15-9			SM		40	108.4	21.2
62								
64								
66								
68								
70	B15-10					66/2"	112.5	19.1
72				SM	**SCRIPPS FORMATION (Tsc)** Very dense, moist to wet, yellowish to grayish brown, Silty, fine-grained SANDSTONE; laminated; friable (contact in tip)			
74								
76								
78								
80						50/3"		
82					-No recovery; strongly cemented			
84					-No recovery; very dense, moist, reddish brown	50/2"		

PRACTICAL REFUSAL AT 85 FEET
Groundwater encountered at ~60'

Figure A-15,
Log of Boring B 15, Page 3 of 3

G2415-52-01.GPJ

SAMPLE SYMBOLS

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

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

GEOCON

PROJECT NO. G2415-52-01

BORING B 16

ELEV. (MSL.) 286' DATE COMPLETED 07-20-2019

EQUIPMENT CME 95 BY: A. REKANI

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0					4" **ASPHALT CONCRETE** over 10" **BASE**			
				ML	**PREVIOUSLY PLACED FILL (Qpf)** Medium dense, moist, mottled yellowish to reddish brown and grayish brown, Sandy SILT			
2								
4								
	B16-1					34	107.1	16.1
6								
8								
10	B16-2					30	103.4	21.7
	B16-3							
12								
14								
	B16-4					35	105.9	20.7
16								
18								
20	B16-5			CL	Firm, moist to wet, mottled dark gray and yellowish to reddish brown, Silty CLAY with trace gravel	13	98.2	26.8
22								
24								
	B16-6			ML	Medium dense, moist, mottled yellowish to reddish brown and grayish brown, Sandy SILT	37	107.7	21.4
26								
28								

Figure A-16,
Log of Boring B 16, Page 1 of 2

G2415-52-01.GPJ

SAMPLE SYMBOLS

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

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

GEOCON

BORING B 16

ELEV. (MSL.) **286'** DATE COMPLETED **07-20-2019**

EQUIPMENT **CME 95** BY: **A. REKANI**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
30	B16-7 ■	▨		SC	**SCRIPPS FORMATION (Tsc)** Very dense, damp to moist, yellowish to reddish brown, Silty, fine-grained SANDSTONE with gravel	50/2"		
32					BORING TERMINATED AT 32 FEET DUE TO REFUSAL ON GRAVEL No groundwater encountered			

Figure A-16,
Log of Boring B 16, 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.

APPENDIX B

APPENDIX B

LABORATORY TESTING

We performed laboratory tests in accordance with generally accepted test methods of the American Society for Testing and Materials (ASTM) or other suggested procedures. We tested selected soil samples for in-place dry density/moisture content, maximum density/optimum moisture content, shear strength, expansion index, water-soluble sulfate, R-Value, consolidation and gradation characteristics. The results of the laboratory tests are in Tables B-I through B-V and on Figures B-1 through B-6. The in-place dry density and moisture content of the samples tested are presented on the boring logs in Appendix A.

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

Sample No.	Description	Maximum Dry Density (pcf)	Optimum Moisture Content (% dry wt.)
B3-2	Reddish- to grayish-brown, Silty, fine to medium SAND (Qpf)	126.6	10.4
B11-1	Light yellowish-brown, Silty, fine SAND (Qpf)	127.8	10.4
B13-2	Yellowish- to reddish-brown, Silty, fine SAND (Qpf)	127.6	10.0
B14-15	Dark brown, Clayey, fine to medium SAND (Qpf)	125.8	11.2

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

Sample No.	Depth (feet)	Geologic Unit	Dry Density (pcf)	Moisture Content (%)		Unit Peak [Ultimate[1]] Cohesion (psf)	Angle of Peak [Ultimate[1]] Shear Resistance (degrees)
				Initial	Final		
B1-6	30	Qpf	101.1	24.2	25.9	1300 [1200]	27 [27]
B2-7	30	Tsc	100.9	6.6	21.7	550 [500]	30 [30]
B5-7	30	Qpf	109.4	18.6	19.4	500 [400]	37 [36]
B8-5	20	Ta	122.3	17.4	21.2	1000 [900]	31 [31]
B14-9	50	Qpf	100.4	22.0	25.6	1600 [1500]	32 [32]
B15-9	60	Qpf	108.4	21.2	22.7	1400 [1100]	29 [29]
B1-3[A]	10	Tsc	106.9	17.7	21.8	600 [600]	44 [36]
B5-2[A]	5	Qpf	102.2	9.3	21.4	300 [50]	40 [38]
B4-3[B]	10	Ta	109.7	16.6	18.8	1,300 [1,000]	32 [32]

[A] Results from previous investigation at 10260 Campus Point Drive (G2345-52-02).
[B] Results from previous investigation at 10290 Campus Point Drive (07850-42-15).

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

Sample No.	Moisture Content (%)		Dry Density (pcf)	Expansion Index	2016 CBC Expansion Classification	ASTM Soil Expansion Classification
	Before Test	After Test				
B2-2	10.8	21.9	106.9	65	Expansive	Medium
B8-2	10.3	21.6	107.7	67	Expansive	Medium
B14-15	10.2	19.9	109.6	52	Expansive	Medium
B15-7	10.2	21.5	108.3	77	Expansive	Medium

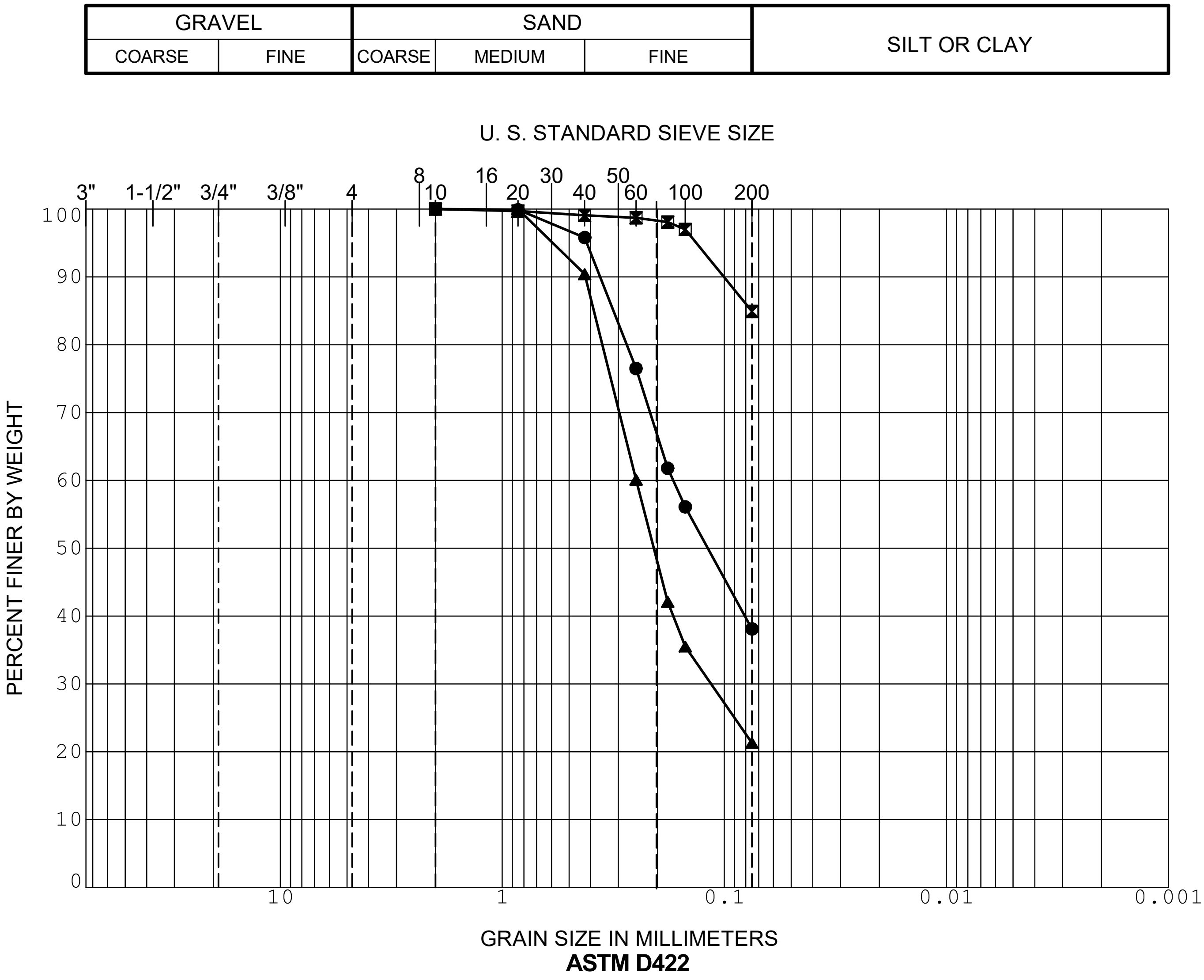
TABLE B-IV
SUMMARY OF LABORATORY WATER-SOLUBLE SULFATE TEST RESULTS
CALIFORNIA TEST NO. 417

Sample No.	Depth (feet)	Geologic Unit	Water-Soluble Sulfate (%)	ACI 318 Sulfate Exposure
B3-2	6	Qpf	0.012	S0
B11-1	0	Qpf	0.004	S0
B13-2	6	Qpf	0.031	S0
B14-15	80	Qpf	0.029	S0
B15-7	45	Qpf	0.057	S0

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

Sample No.	Depth (feet)	Description (Geologic Unit)	R-Value
B3-2	6	Reddish- to grayish-brown, Silty, fine to medium SAND (Qpf)	9
B11-1	0	Light yellowish-brown, Silty, fine SAND (Qpf)	26
B13-2	6	Yellowish- to reddish-brown, Silty, fine SAND (Qpf)	13

PROJECT NO. G2415-52-01



	SAMPLE	DEPTH (ft)	CLASSIFICATION	NAT WC	LL	PL	PI
●	B5-8	40.0	SM - Silty SAND				
⊠	B10-2	5.0	ML - Sandy SILT				
▲	B12-3	10.0	SM - Silty SAND				

GRADATION CURVE

CAMPUS POINTE

SAN DIEGO, CALIFORNIA

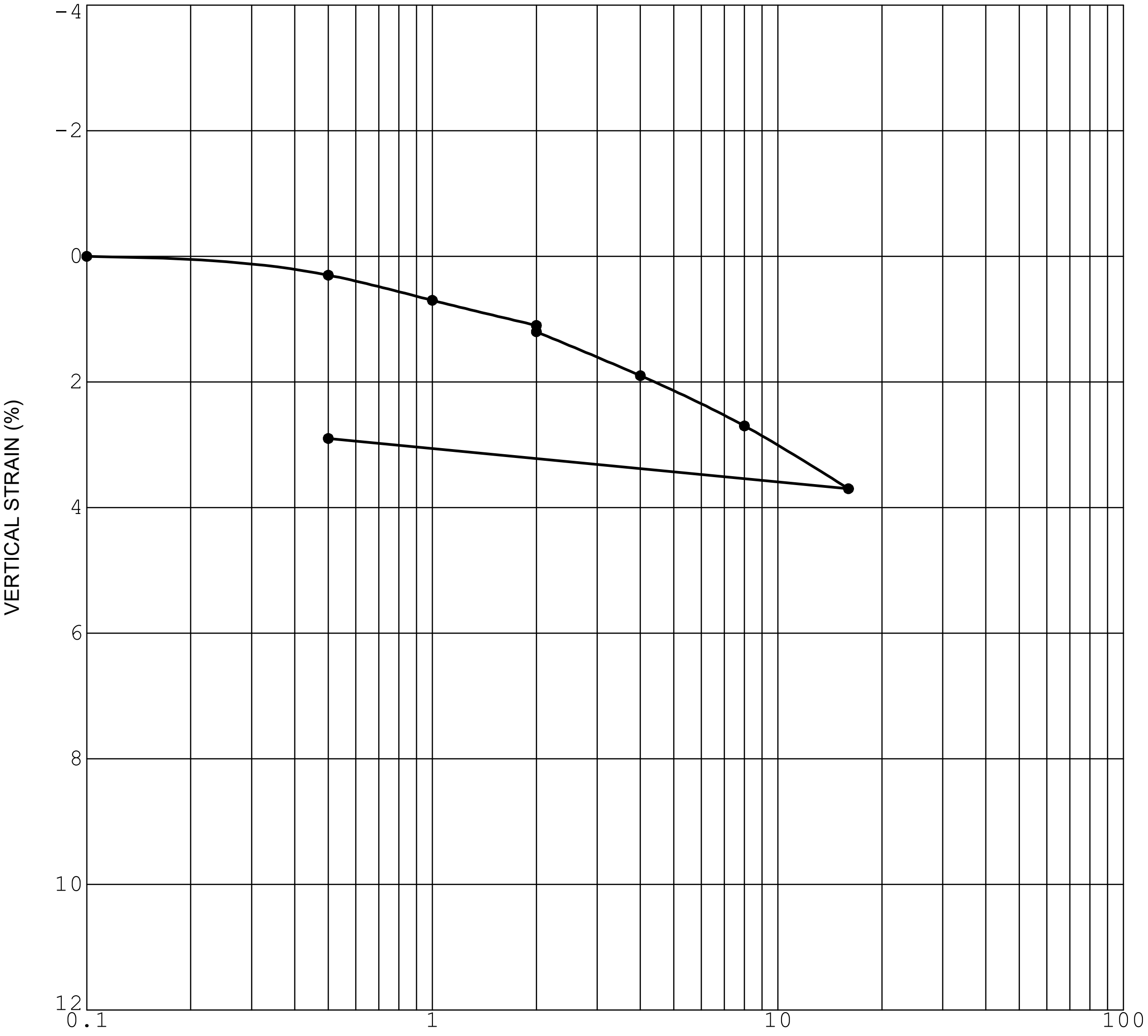
G2415-52-01.GPJ

Figure B-1

GEOCON

PROJECT NO. G2415-52-01

SAMPLE NO. B1-8



ASTM D2435

Initial Dry Density (pcf)	107.2
Initial Water Content (%)	11.0

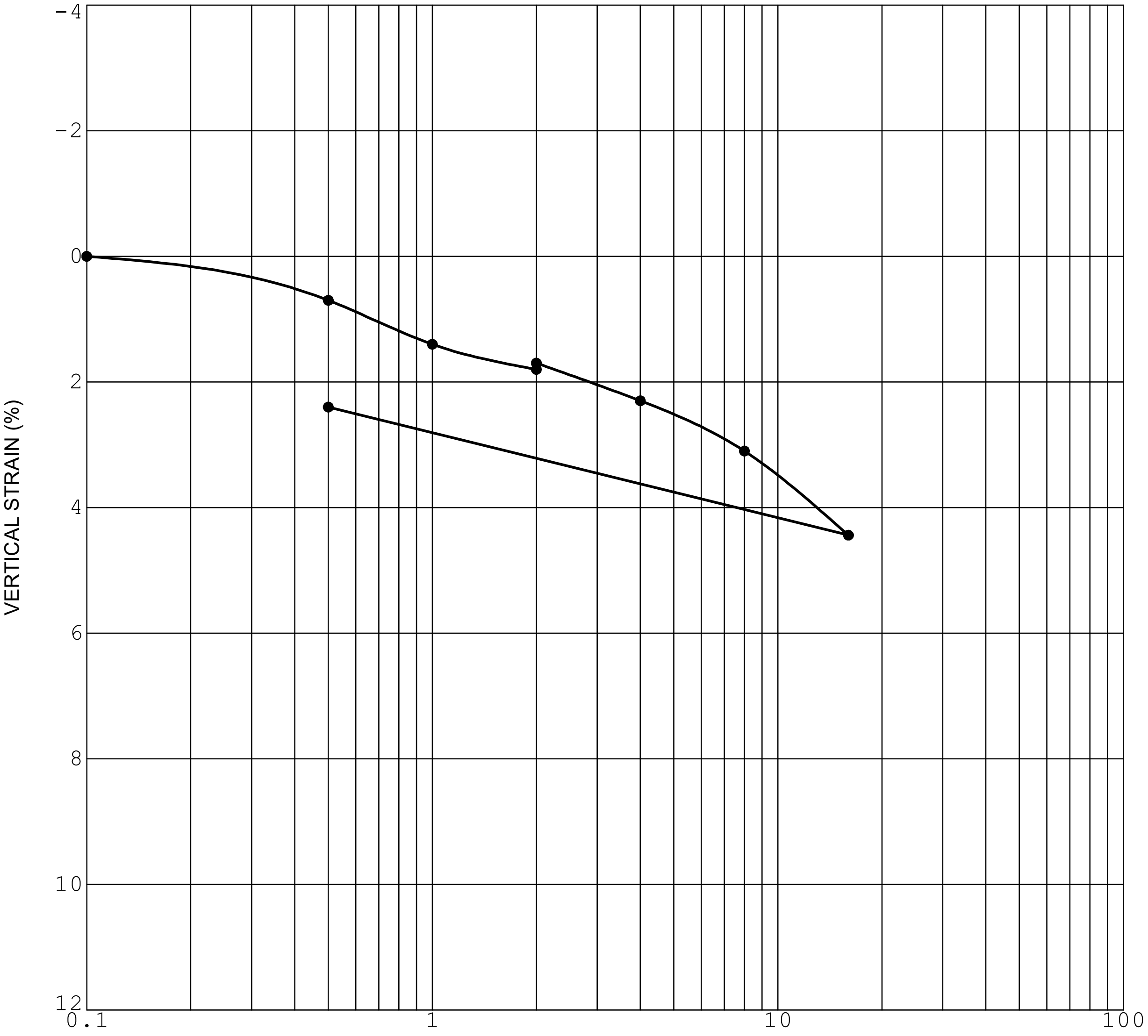
Initial Saturation (%)	53.5
Sample Saturated at (ksf)	2.0

CONSOLIDATION CURVE

CAMPUS POINTE

SAN DIEGO, CALIFORNIA

SAMPLE NO. B5-9



APPLIED PRESSURE (ksf)

ASTM D2435

Initial Dry Density (pcf)	109.0
Initial Water Content (%)	20.5

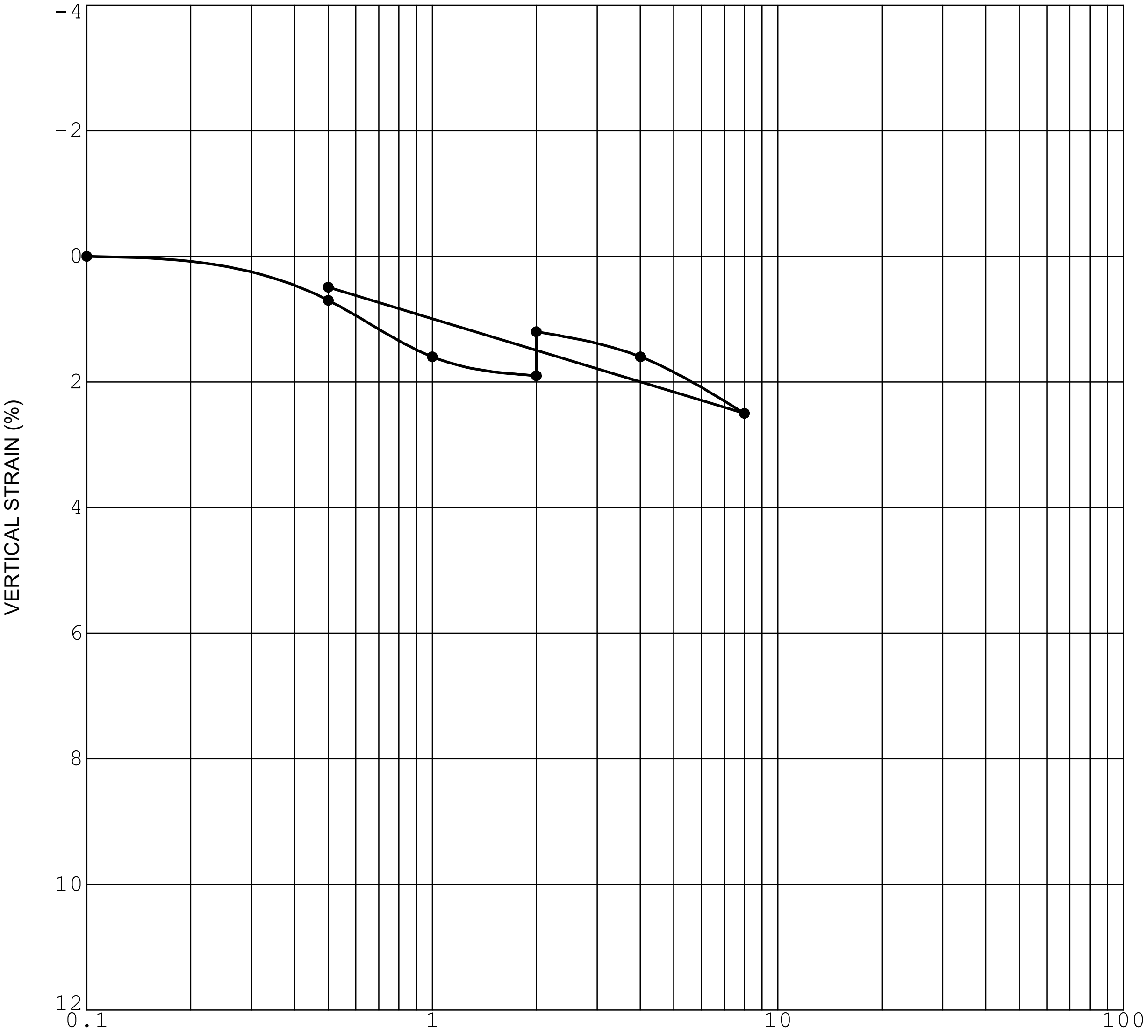
Initial Saturation (%)	100
Sample Saturated at (ksf)	2.0

CONSOLIDATION CURVE

CAMPUS POINTE

SAN DIEGO, CALIFORNIA

SAMPLE NO. B8-6



ASTM D2435

Initial Dry Density (pcf)	109.7
Initial Water Content (%)	20.1

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

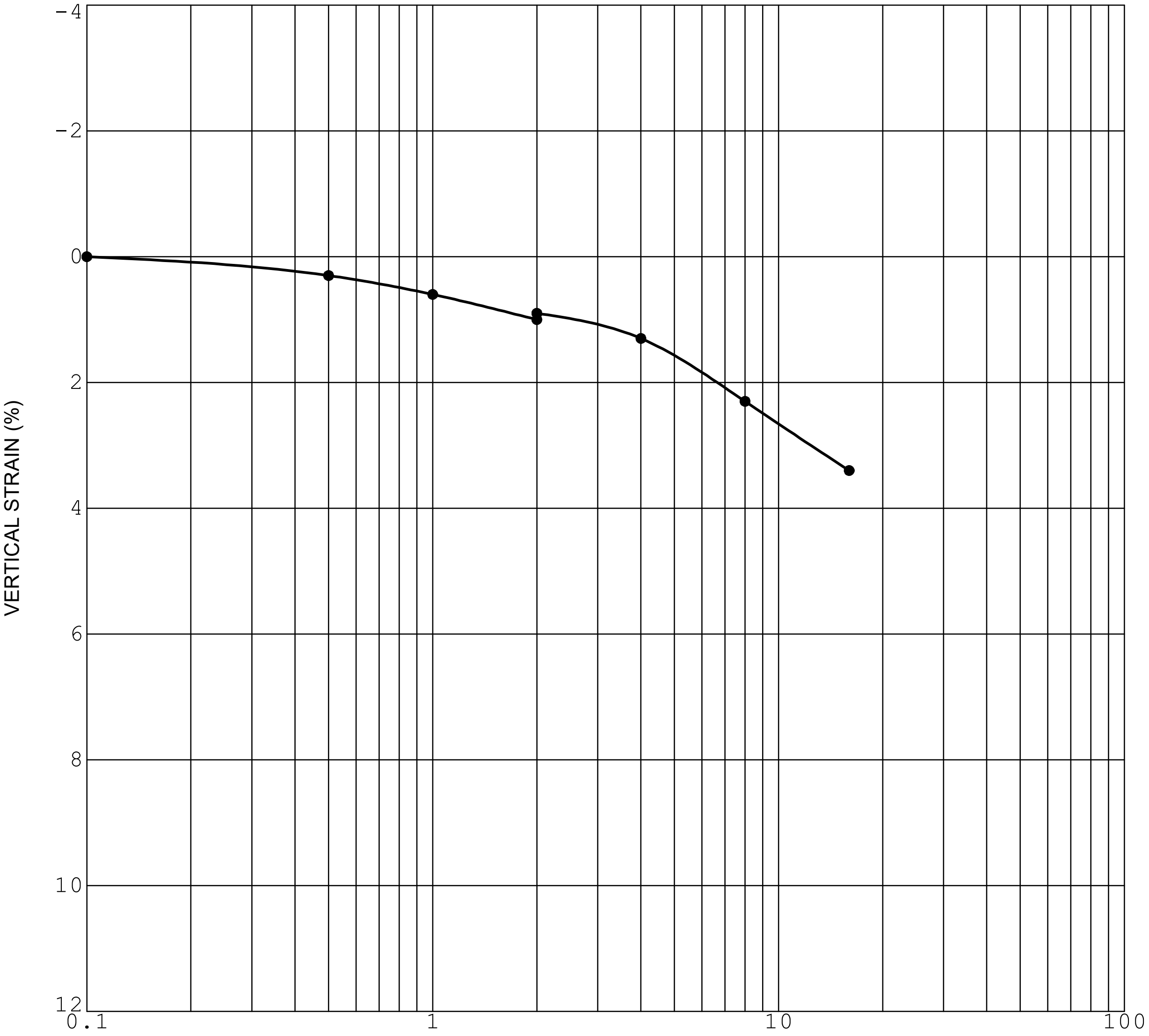
CONSOLIDATION CURVE

CAMPUS POINTE

SAN DIEGO, CALIFORNIA

GEOCON

SAMPLE NO. B14-9



APPLIED PRESSURE (ksf)

ASTM D2435

Initial Dry Density (pcf)	107.1
Initial Water Content (%)	22.7

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

CONSOLIDATION CURVE

CAMPUS POINTE

SAN DIEGO, CALIFORNIA

SAMPLE NO. B15-8

APPLIED PRESSURE (ksf)

ASTM D2435

Initial Dry Density (pcf)	103.4
Initial Water Content (%)	22.9

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

CONSOLIDATION CURVE

CAMPUS POINTE

SAN DIEGO, CALIFORNIA

APPENDIX C

APPENDIX C

BORING, TRENCH LOGS, AND LABORATORY TESTING FROM:

GEOCON INCORPORATED (2019) – 10260 CAMPUS POINT DRIVE,
GEOCON INCORPORATED (2015) – 10290 CAMPUS POINT DRIVE
SOUTHERN CALIFORNIA SOILS & TESTING (1995) – 10290 CAMPUS POINT DRIVE
GEOCON INCORPORATED (1980) – CAMPUS POINT, PHASE II

FOR

CAMPUS POINTE
SAN DIEGO, CALIFORNIA

PROJECT NO. G2415-52-01

PROJECT NO. G2345-52-02

BORING B 1

ELEV. (MSL.) **290'** DATE COMPLETED **01-30-2019**

EQUIPMENT **UNIMOG W/ 6" HSA** BY: **K. JAMES**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0					**4-INCH ASPHALT CONCRETE**			
	B1-1			SM	**UNDOCUMENTED FILL (Qudf)** Medium dense, damp, brown, Silty fine to medium SAND			
2				SM	**SCRIPPS FORMATION (Tsc)** Dense, damp, light gray to yellowish brown, Silty, fine SANDSTONE -Strongly cemented layer encountered from approximate 16 to 24 inches below ground surface			
4								
	B1-2					66	106.5	11.5
6								
8					-Becomes very dense			
10	B1-3					38/11"	106.8	20.1
					-Becomes light gray and yellowish brown mottled			
12								
14				SP	Very dense, damp, gray and red-brown mottled, fine to medium SANDSTONE			
	B1-4					81		
16								
18								
20	B1-5					78		
22								
24								
	B1-6					71/11"		
26								
28								
				SM	Very dense, damp, brown, Silty, fine to medium SANDSTONE			

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

G2345-52-02.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 1

ELEV. (MSL.) **290'** DATE COMPLETED **01-30-2019**

EQUIPMENT **UNIMOG W/ 6" HSA** BY: **K. JAMES**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
30	B1-7			SM		50/5.5"	99.0	15.8
	B1-8							
32								
34				SP	Very dense, damp, brown, Silty, fine to medium SANDSTONE			
	B1-9					50/5"	96.7	9.8
36								
38								
40	B1-10				-Becomes wet	81/11"		

BORING TERMINATED AT 40 FEET 11 INCHES
Boring backfilled with approximate 8 cu. ft. bentonite
No groundwater encountered

Figure A-1,
Log of Boring B 1, 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. G2345-52-02

BORING B 2

ELEV. (MSL.) **291'** DATE COMPLETED **01-30-2019**

EQUIPMENT **UNIMOG W/ 6" HSA** BY: **K. JAMES**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0					**4-INCH ASPHALT CONCRETE**			
	B2-1 ▨			SM	**UNDOCUMENTED FILL (Qudf)** Medium dense, damp, brown, Silty, fine to medium SAND			
2				SM	**SCRIPPS FORMATION (Tsc)** Very dense, damp, light yellowish brown, Silty fine to medium SANDSTONE			
4	B2-2 ■					74/10"	109.4	8.5

BORING TERMINATED AT 5 FEET
Backfilled with soil cuttings
No groundwater encountered

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

G2345-52-02.GPJ

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

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

GEOCON

BORING B 3

ELEV. (MSL.) **293'** DATE COMPLETED **01-30-2019**

EQUIPMENT **UNIMOG W/ 6" HSA** BY: **K. JAMES**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0					**5-INCH ASPHALT CONCRETE**			
				SM	**UNDOCUMENTED FILL (Qudf)** Loose, damp, brown to yellowish brown, Silty fine to medium SAND			
2								
4								
	B3-1					16	96.9	10.6
6				SM	**SCRIPPS FORMATION (Tsc)** Dense, damp, light brown and yellowish brown, Silty, fine to medium SAND			
8								
10	B3-2					46		

BORING TERMINATED AT 11.5 FEET
Backfilled with soil cuttings
No groundwater encountered

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

G2345-52-02.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. G2345-52-02

BORING B 4

ELEV. (MSL.) **294'** DATE COMPLETED **01-30-2019**

EQUIPMENT **UNIMOG W/ 6" HSA** BY: **K. JAMES**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0					**4-INCH ASPHALT CONCRETE**			
	☒ B4-1			SC	**PREVIOUSLY PLACED FILL** Medium dense, moist, brown to yellowish brown, Clayey, fine to medium SAND			
2								
4								
	■ B4-2					22	97.3	13.7
6					BORING TERMINATED AT 6 FEET Backfilled with soil cuttings No groundwater encountered			

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

G2345-52-02.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. G2345-52-02

BORING B 5

ELEV. (MSL.) 295' DATE COMPLETED 01-30-2019

EQUIPMENT UNIMOG W/ 6" HSA BY: K. JAMES

DEPTH IN FEET	SAMPLE NO.	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0			**4-INCH ASPHALT CONCRETE**			
	B5-1	SC	**PREVIOUSLY PLACED FILL (Qpf)** Medium dense, damp, brown to yellowish brown, Clayey, fine to medium SAND			
2						
4						
	B5-2					
6		SP	Dense, damp, light brown, fine to medium SAND	51		
8						
		SC	**SCRIPPS FORMATION (Tsc)** Very dense, brown to yellowish brown, Clayey, fine to medium SAND			
10	B5-3			57/6.5"	101.7	17.1
12						
14						
		SM	Very dense, strongly cemented, moist, light gray, Silty fine to medium SANDSTONE			
	B5-4			72/11"	98.3	9.6
16			-Difficult drilling			
18						
20	B5-5		-strongly cemented	50/0.2"		
22			-Very difficult drilling			
24			-Becomes weakly cemented; brown to yellowish brown			
	B5-6			76/10"	96.5	12.3
26						
28						

Figure A-5,
Log of Boring B 5, Page 1 of 2

G2345-52-02.GPJ

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

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

GEOCON

BORING B 5

ELEV. (MSL.) **295'** DATE COMPLETED **01-30-2019**

EQUIPMENT **UNIMOG W/ 6" HSA** BY: **K. JAMES**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
30	B5-7			SM		50/6"		
32								
34								
	B5-8				-Becomes light brown	50/5"		
36								
38				SP	Very dense, damp, light brown, fine to medium SANDSTONE			
					-Difficult drilling			
40	B5-9					60		
42								
				SM	Very dense, damp, light grayish brown, Silty, fine to medium SANDSTONE			
44								
	B5-10					85	108.7	13.8
46								

BORING TERMINATED AT 46 FEET
Backfilled with approximately 9 cu. ft. bentonite
No groundwater encountered

Figure A-5,
Log of Boring B 5, Page 2 of 2

G2345-52-02.GPJ

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

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PROJECT NO. G2345-52-02

TEST PIT P 1

ELEV. (MSL.) **290'** DATE COMPLETED **01-30-2019**

EQUIPMENT **UNIMOG W/ 6" HSA** BY: **K. JAMES**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0					**4-INCH ASPHALT CONCRETE**			
				SC	**UNDOCUMENTED FILL (Qudf)** Medium dense, moist, brown, Clayey, fine to medium SAND			
2				ML	**SCRIPPS FORMATION (Tsc)** Very dense, moist, yellowish brown, Sandy SILTSTONE			
4								
	P1-1					52		
6					BORING TERMINATED AT 6 FEET Backfilled with soil cuttings No groundwater encountered			

Figure A-6,
Log of Test Pit P 1, 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.

GEOCON

PROJECT NO. G2345-52-02

TEST PIT P 2

ELEV. (MSL.) **290'** DATE COMPLETED **01-30-2019**

EQUIPMENT **UNIMOG W/ 6" HSA** BY: **K. JAMES**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0					**4-INCH ASPHALT CONCRETE**			
				SC	**UNDOCUMENTED FILL (Qudf)** Medium dense, moist, brown, Clayey, fine to medium SAND			
2				ML	**SCRIPPS FORMATION (Tsc)** Very dense, moist, yellowish brown, Sandy SILTSTONE			
4								
6					BORING TERMINATED AT 6 FEET Backfilled with soil cuttings No groundwater encountered			

Figure A-7,
Log of Test Pit P 2, Page 1 of 1

G2345-52-02.GPJ

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

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APPENDIX B

LABORATORY TESTING

We performed laboratory tests in accordance with generally accepted test methods of the American Society for Testing and Materials (ASTM) or other suggested procedures. We tested selected soil samples for in-place dry density and moisture content, maximum density and optimum moisture content, direct shear strength, expansion index, water soluble sulfate, R-Value, unconfined compressive strength, gradation characteristics and consolidation characteristics. Tables B-I through B-VI and Figures B-1 and B-2 present the results of our laboratory tests. The in-place dry density and moisture content of the samples tested are presented on the boring logs in Appendix A.

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

Sample No.	Description (Geologic Unit)	Maximum Dry Density (pcf)	Optimum Moisture Content (% dry wt.)
B2-1	Light Yellowish Brown, Silty, fine to medium SAND (Tsc)	131.6	8.7
B5-1	Brown to yellowish brown, Clayey, fine to medium SAND (Qudf)	130.4	9.4

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

Sample No.	Dry Density (pcf)	Moisture Content (%)		Peak [Ultimate[1]] Cohesion (psf)	Peak [Ultimate[1]] Angle of Shear Resistance (degrees)
		Initial	Final		
B1-3	106.9	17.7	21.8	600 [600]	44 [36]
B5-2[2]	102.2	9.3	21.4	300 [50]	40 [38]

[1] Ultimate at end of test at 0.2-inch deflection.

[2] Remolded to a dry density of about 90 percent of the laboratory maximum dry density.

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

Sample No.	Geologic Unit	Moisture Content (%)		Dry Density (pcf)	Expansion Index	2016 CBC Expansion Classification	ASTM Soil Expansion Classification
		Before Test	After Test				
B1-8	Tsc	9.7	17.1	111.7	19	Non-Expansive	Very Low
B5-1	Qpf	9.8	19.4	110.7	55	Expansive	Medium

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

Sample No.	Depth (feet)	Geologic Unit	Water-Soluble Sulfate (%)	ACI 318 Sulfate Exposure
B1-8	30-35	Tsc	0.007	S0
B2-1	0.5-4	Tsc	0.006	S0
B5-1	0.5-5	Qpf	0.042	S0

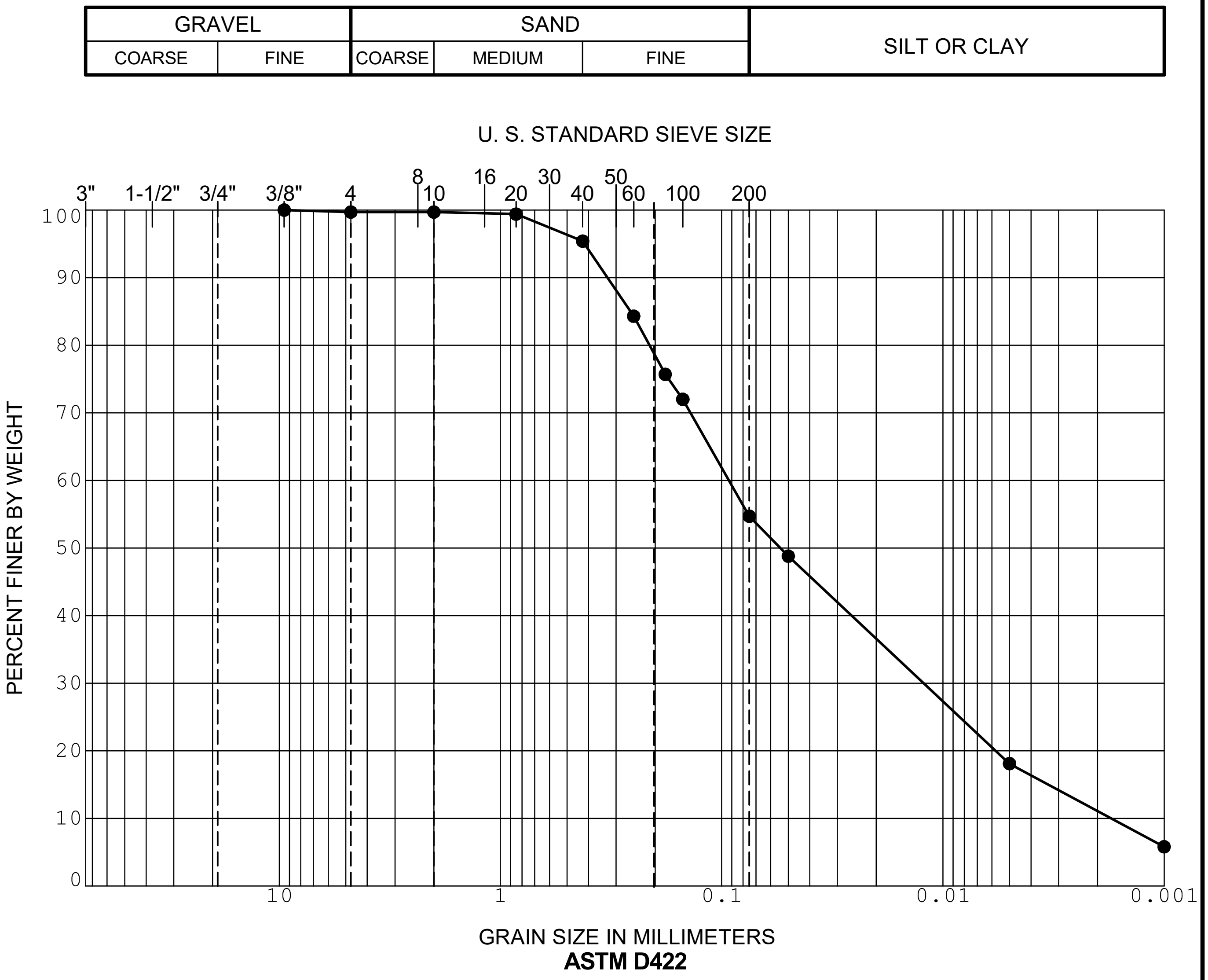
TABLE B-V
SUMMARY OF LABORATORY RESISTANCE VALUE (R-VALUE) TEST RESULTS
ASTM D 2844

Sample No.	R-Value
B4-1	20

TABLE B-VI
SUMMARY OF LABORATORY UNCONFINED COMPRESSIVE STRENGTH TEST RESULTS
ASTM D 1558

Sample No.	Depth (feet)	Geologic Unit	Hand Penetrometer Reading, Unconfined Compression Strength (tsf)	Undrained Shear Strength (ksf)
B1-2	5	Tsc	4.5	4.5
B1-7	30	Tsc	4.5	4.5
B2-2	4	Tsc	4.0	4.0
B3-1	5	Qudf	2.5	2.5
B4-2	5	Qpf	4.0	4.0
B5-3	10	Tsc	3.5	3.5
B5-10	45	Tsc	4.5	4.5

PROJECT NO. G2345-52-02



	SAMPLE	DEPTH (ft)	CLASSIFICATION	NAT WC	LL	PL	PI
●	P1-1	4.5	ML - Sandy SILT				
☒							
▲							

GRADATION CURVE

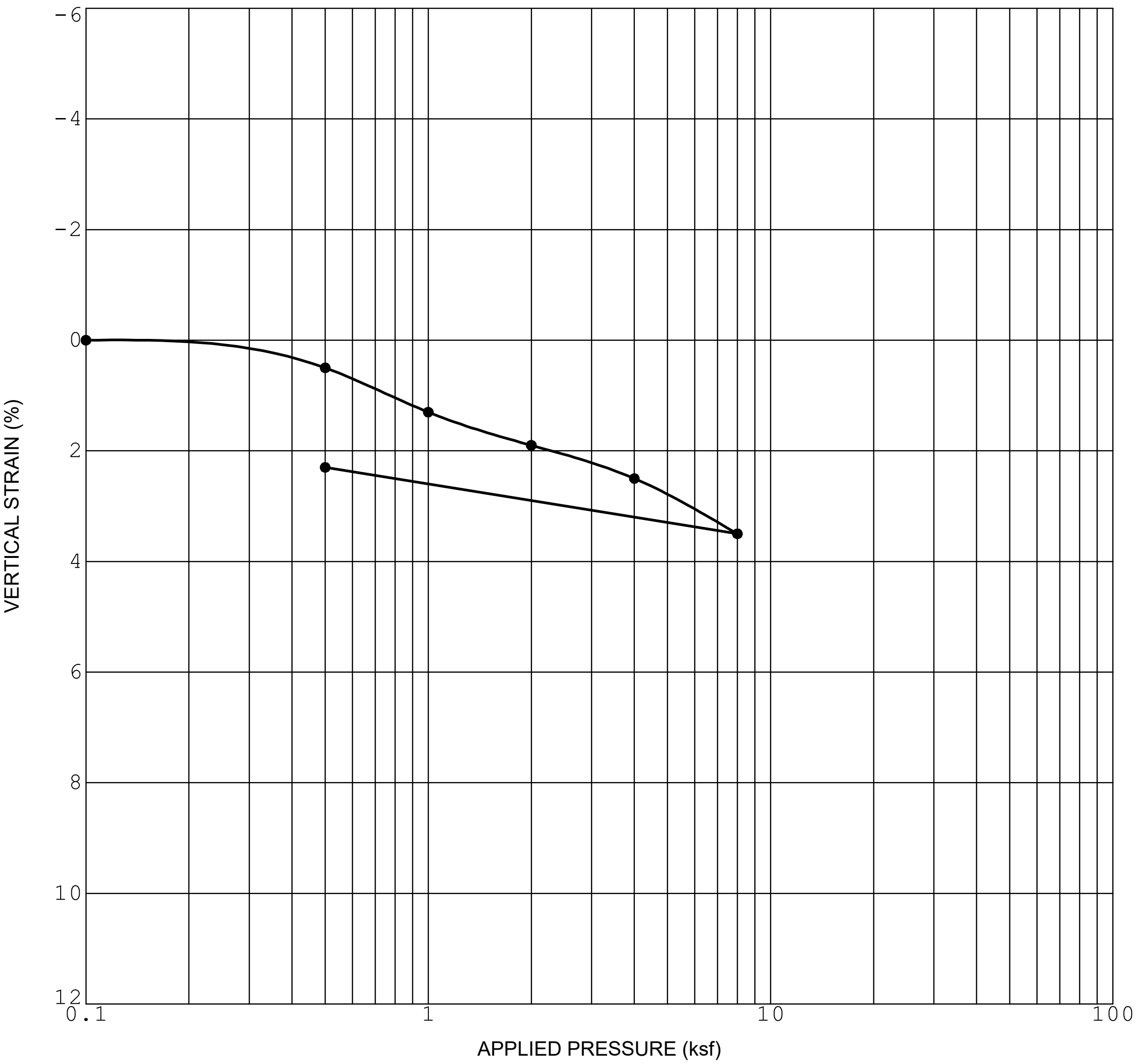
10260 CAMPUS POINT DRIVE

SAN DIEGO, CALIFORNIA

Figure B-1

PROJECT NO. G2345-52-02

SAMPLE NO. B4-2



ASTM D2435

Initial Dry Density (pcf)	106.8
Initial Water Content (%)	20.1

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

CONSOLIDATION CURVE

10260 CAMPUS POINT DRIVE

SAN DIEGO, CALIFORNIA

G2345-52-02.GPJ

Figure B-2

GEOCON

PROJECT NO. 07850-42-15

BORING B 1

ELEV. (MSL.) **300'** DATE COMPLETED **05-26-2015**

EQUIPMENT **CME 75** BY: **N. BORJA**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0					3/4" **ASPHALT CONCRETE** Over 6" **BASE**			
	B1-1			SM	**PREVIOUSLY PLACED FILL** Medium dense, moist, yellowish brown, Silty, fine to medium SAND; few clay			
2								
				ML	Stiff, moist, yellowish brown to brown, Sandy SILT; few clay			
4								
	B1-2					22	105.8	20.5
6								
8								
				SM	Medium dense, moist, mottled yellowish brown and gray, Silty, fine to medium SAND; trace clay			
10								
	B1-3					21	104.2	21.3
12								
14								
	B1-4					21	100.1	24.6
16								
18								
	B1-5					25	102.8	22.6
				SM/ML	**ARDATH SHALE** Dense, moist, mottled yellowish brown, gray, and reddish brown, Silty, fine to medium SAND and Sandy SILT			

BORING TERMINATED AT 19.5 FEET
No groundwater encountered
Boring finished on 05/26/2015

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

07850-42-15 (UPDATED).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. 07850-42-15

BORING B 2

ELEV. (MSL.) **300'** DATE COMPLETED **05-26-2015**

EQUIPMENT **CME 75** BY: **N. BORJA**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0					3" **ASPHALT CONCRETE** Over 5.5" **BASE**			
	B2-1			SM	**ARDATH FORMATION** Very dense, damp, mottled yellowish brown and gray, Silty, fine to medium SAND			
2								
4								
	B2-2					50/3"		
6								
					-Becomes tan brown; encountered hard cemented zone; different drilling between 7' to 9'			
8								
10	B2-3					69/11"		
	B2-4							
12								
				SM	Very dense, damp, mottled brown and yellowish brown to reddish brown, Silty, fine to medium SAND; moderately cemented			
14								
	B2-5					50/5"		
16								
18								
	B2-6				-Hard cemented zone or rock encountered; very difficult drilling below 18'; poor recovery at 18.5' sample	50/2"		

BORING TERMINATED AT 19.5 FEET
No groundwater encountered
Boring finished on 05/26/2015

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

07850-42-15 (UPDATED).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. 07850-42-15

BORING B 3

ELEV. (MSL.) **303'** DATE COMPLETED **05-26-2015**

EQUIPMENT **CME 75** BY: **N. BORJA**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0					4" **ASPHALT CONCRETE** Over 7" **BASE**			
				SM	**ARDATH FORMATION** Dense to very dense damp light grayish brown, Silty, fine to medium SAND			
2					-Becomes damp to moist light yellowish brown			
4								
	B3-1					71/10"		
6	B3-2							
8								
				SM	Medium dense, damp, light brown, Silty, fine to medium SAND			
10	B3-3					30		
				ML	Stiff, damp, light gray, Sandy SILT			
12								
14				SM	Very dense, damp, yellowish brown, Silty, fine to medium SAND			
	B3-4					82/10"		
16								
18					-Becomes dense			
	B3-5					71		

BORING TERMINATED T 19.5 FEET
No groundwater encountered
Boring finished on 05/26/2015

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

07850-42-15 (UPDATED).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. 07850-42-15

BORING B 4

ELEV. (MSL.) **300'** DATE COMPLETED **05-26-2015**

EQUIPMENT **CME 75** BY: **N. BORJA**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0					2.5" **ASPHALT CONCRETE** Over 4" **RECYCLED BASE**			
				ML/SM	**ARDATH FORMATION** Hard, damp, mottled, yellowish brown to tan and gray, Sandy SILT to Silty, fine-grained SAND			
2				SM/SP-SM	Dense, damp, light gray, fine to medium SAND; weakly cemented			
				SM	Dense to very dense, damp, mottled tan brown and gray, Silty, fine to medium grained SAND; weakly cemented; massive			
4								
	B4-1					71/11"		
6	B4-2							
8								
10	B4-3					77/10"	109.7	16.6
12								
14								
	B4-4				-Excavates with few gypsum	79/11"		
16								
18					-Poor recovery			
	B4-5					50/2"		

BORING TERMINATED AT 19.5 FEET
No groundwater encountered
Boring finished on 05/26/2015

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

07850-42-15 (UPDATED).GPJ

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

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PROJECT NO. 07850-42-15

BORING B 5

ELEV. (MSL.) **298'** DATE COMPLETED **05-26-2015**

EQUIPMENT **CME 75** BY: **N. BORJA**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0					4" **ASPHALT CONCRETE** Over 4" **RECYCLED BASE**			
2				SM/ML	**PREVIOUSLY PLACED FILL** Medium dense, damp to moist, mottled tan and gray, Silty, fine to medium SAND to Sandy SILT			
4				SM	**SCRIPPS FORMATION** Dense, moist, mottled light brown and brown, Silty, fine-grained SAND			
6	B5-1				-Excavates with reddish brown and yellowish brown staining	57/11"		
8								
10	B5-2					76/10"		
12								
14					-Becomes brown to light brown; excavates with black specs			
16	B5-3					77/9"		
18								
					-Becomes light grayish brown to light brown			
	B5-4				BORING TERMINATED AT 19.5 FEET No groundwater encountered Boring finished on 05/26/2015	77/8"		

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

07850-42-15 (UPDATED).GPJ

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

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GEOCON

PROJECT NO. 07850-42-15

BORING B 6

ELEV. (MSL.) **302'** DATE COMPLETED **05-26-2015**

EQUIPMENT **CME 75** BY: **N. BORJA**

DEPTH IN FEET	SAMPLE NO.	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0			4" **ASPHALT CONCRETE** Over 8.5" **BASE**			
2	B6-1	SM	**PREVIOUSLY PLACED FILL** Medium dense, moist, yellowish brown to brown, Silty, fine to medium SAND, trace gravel; trace concrete			
4						
	B6-2			24	105.2	21.0
6		ML	Stiff, moist, mottled yellowish brown to brown and gray, Sandy SILT			
8			-Encountered cemented zone from 7' to 8'; hard drilling due to rock			
10	B6-3		-Becomes very stiff	49	112.8	17.5
12		SM	Medium dense to dense, moist, tan brown to yellowish brown, Silty, fine to medium SAND; few clay; trace gravel			
14		CL	Stiff, moist, mottled dark brown, dark gray, and gray, Sandy CLAY; trace gravel, trace organics, slight organic odor; sample chunk of formation in shoe			
	B6-4			25	109.7	14.8
16		SM	Medium dense, damp, mottled brown and gray, Silty, fine to medium SAND; little chunks of siltstone			
18						
	B6-5			32	104.6	10.0

BORING TERMINATED AT 19.5 FEET
No groundwater encountered
Boring finished on 05/26/2015

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

07850-42-15 (UPDATED).GPJ

SAMPLE SYMBOLS

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

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GEOCON

PROJECT NO. 07850-42-15

BORING B 7

ELEV. (MSL.) **300'** DATE COMPLETED **02-29-2016**

EQUIPMENT **CME 95** BY: **G. CANNON**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0				SC	**SCRIPPS FORMATION** Very dense, moist, light olive, Clayey, fine SAND			
2								
4								
	B7-1					70/11"		
6								
8				CL	Hard, moist, light olive, fine Sandy CLAY			
10								
	B7-2					83/9"		
12								
				SP	Very dense, moist, light gray, fine SAND			
14								
	B7-3					75/10"		
16								
18								
20								
	B7-4					93/10"		
22								
24								
	B7-5					91/10"		
26								
28								

Figure A-7,
Log of Boring B 7, Page 1 of 2

07850-42-15 (UPDATED).GPJ

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

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GEOCON

PROJECT NO. 07850-42-15

BORING B 7

ELEV. (MSL.) 300' DATE COMPLETED 02-29-2016

EQUIPMENT CME 95 BY: G. CANNON

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
30					-Refusal of sampler on concretion			
32	B7-6				-No sample, rock in shoe	50/4"		
34				SM	Very dense, light reddish gray, Silty fine SAND			
36	B7-7					50/6"		

BORING TERMINATED AT 36 FEET
No groundwater encountered
Backfilled with cuttings

Figure A-7,
Log of Boring B 7, Page 2 of 2

07850-42-15 (UPDATED).GPJ

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

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GEOCON

PROJECT NO. 07850-42-15

BORING B 8

ELEV. (MSL.) **300'** DATE COMPLETED **02-29-2016**

EQUIPMENT **CME 95** BY: **G. CANNON**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0				ML	**ARDATH FORMATION** Hard, moist, olive, fine Sandy, Clayey SILT			
2								
4								
	B8-1 ■					75		
6	B8-2 ▨							
8								
10	B8-3 ■					82/11"		
12								
14								
	B8-4 ■				-Sample disturbed, rock in sampler	50/3"		
16								
18								
20	B8-5 ■					70/11"		
22								
				CL	Hard, moist, olive brown, fine Sandy, Silty CLAY			
24								
	B8-6 ■					50/4"		
26								
28								

Figure A-8,
Log of Boring B 8, Page 1 of 2

07850-42-15 (UPDATED).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. 07850-42-15

BORING B 8

ELEV. (MSL.) **300'** DATE COMPLETED **02-29-2016**

EQUIPMENT **CME 95** BY: **G. CANNON**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
30	B8-7 ■	▨				87/10"		
32		▨						
34		▨						
	B8-8 ■	▨				95/10"		
36					BORING TERMINATED AT 36 FEET No groundwater encountered Backfilled with cuttings			

Figure A-8,
Log of Boring B 8, Page 2 of 2

07850-42-15 (UPDATED).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. 07850-42-15

BORING B 9

ELEV. (MSL.) 303' DATE COMPLETED 02-29-2016

EQUIPMENT CME 95 BY: G. CANNON

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0	B9-1			SM	**PREVIOUSLY PLACED FILL** Medium dense, moist, yellowish brown, Silty SAND			
2								
4				SM	**ARDATH FORMATION** Medium dense, moist, light olive, Silty fine SAND			
6	B9-2					21	100.3	12.6
8				CL	Hard, moist, gray, Silty CLAY			
10	B9-3					75/10"	114.5	15.1
12				CL	Hard, moist, olive brown, Silty, fine Sandy CLAY			
14	B9-4					84/10"		
16								
18				ML	Hard, moist, olive brown, fine Sandy SILT			
20	B9-5					74/10"		

BORING TERMINATED AT 21 FEET
No groundwater encountered
Backfilled with cuttings

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

07850-42-15 (UPDATED).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. 07850-42-15

BORING B 10

ELEV. (MSL.) **303'** DATE COMPLETED **02-29-2016**

EQUIPMENT **CME 95** BY: **G. CANNON**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0	B10-1			SM	**PREVIOUSLY PLACED FILL** Medium dense, moist, yellowish brown, Silty SAND			
2				ML	**ARDATH FORMATION** Hard, moist, light olive, fine Sandy SILT			
4								
	B10-2					87/9"		
6								
8								
10	B10-3					77/9"	109.3	19.3
12								
				CL	Hard, moist, gray, Silty CLAY			
14								
	B10-4					50/6"	110.2	17.4
16								
18				ML	Hard, moist, red brown, Clayey SILT			
20	B10-5					100/10"		
					BORING TERMINATED AT 21 FEET No groundwater encountered Backfilled with cuttings			

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

07850-42-15 (UPDATED).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. 07850-42-15

BORING B 11

ELEV. (MSL.) 298' DATE COMPLETED 03-01-2016

EQUIPMENT CME 95 BY: G. CANNON

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0	▨ B11-1			CL-CL	**FILL** Stiff, moist, olive brown, fine Sandy SILT with clay			
2								
4								
				CL	**TOPSOIL** Hard, moist, dark brown, CLAY	24		
6	■ B11-2			ML	**ARDATH FORMATION** Hard, moist, olive brown, Clayey SILT	35		
8				CL	Very stiff, moist, olive brown, Silty CLAY			
10	■ B11-3					39		
12								
14								
	■ B11-4				-Becomes hard	60		
16								
18								
					Hard, moist, olive, Silty CLAY			
20	■ B11-5					44		

BORING TERMINATED AT 21 FEET
No groundwater encountered
Backfilled with cuttings

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

07850-42-15 (UPDATED).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 12

ELEV. (MSL.) 301' DATE COMPLETED 03-01-2016

EQUIPMENT CME 95 BY: G. CANNON

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	**ARDATH FORMATION** Hard, moist, olive brown, Silty CLAY			
	B12-1							
2								
4								
	B12-2					70	109.0	13.5
6								
8				ML	Hard, moist, olive brown, fine Sandy SILT			
10	B12-3					77/10"		
12								
				CL	Hard, moist, grayish brown, Silty CLAY			
14								
	B12-4					77/11"		
16								
18								
20	B12-5					67		
					BORING TERMINATED AT 21 FEET No groundwater encountered Backfilled with cuttings			

Figure A-12,
Log of Boring B 12, Page 1 of 1

07850-42-15 (UPDATED).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.

APPENDIX B

LABORATORY TESTING

Laboratory tests were performed in accordance with generally accepted test methods of the American Society for Testing and Materials (ASTM) or other suggested procedures. Selected soil samples were tested for their: in-place moisture density; expansion index (EI); shear strength; water-soluble sulfate; gradation; and consolidation characteristics. The results of our laboratory tests are presented on the following tables and figures.

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

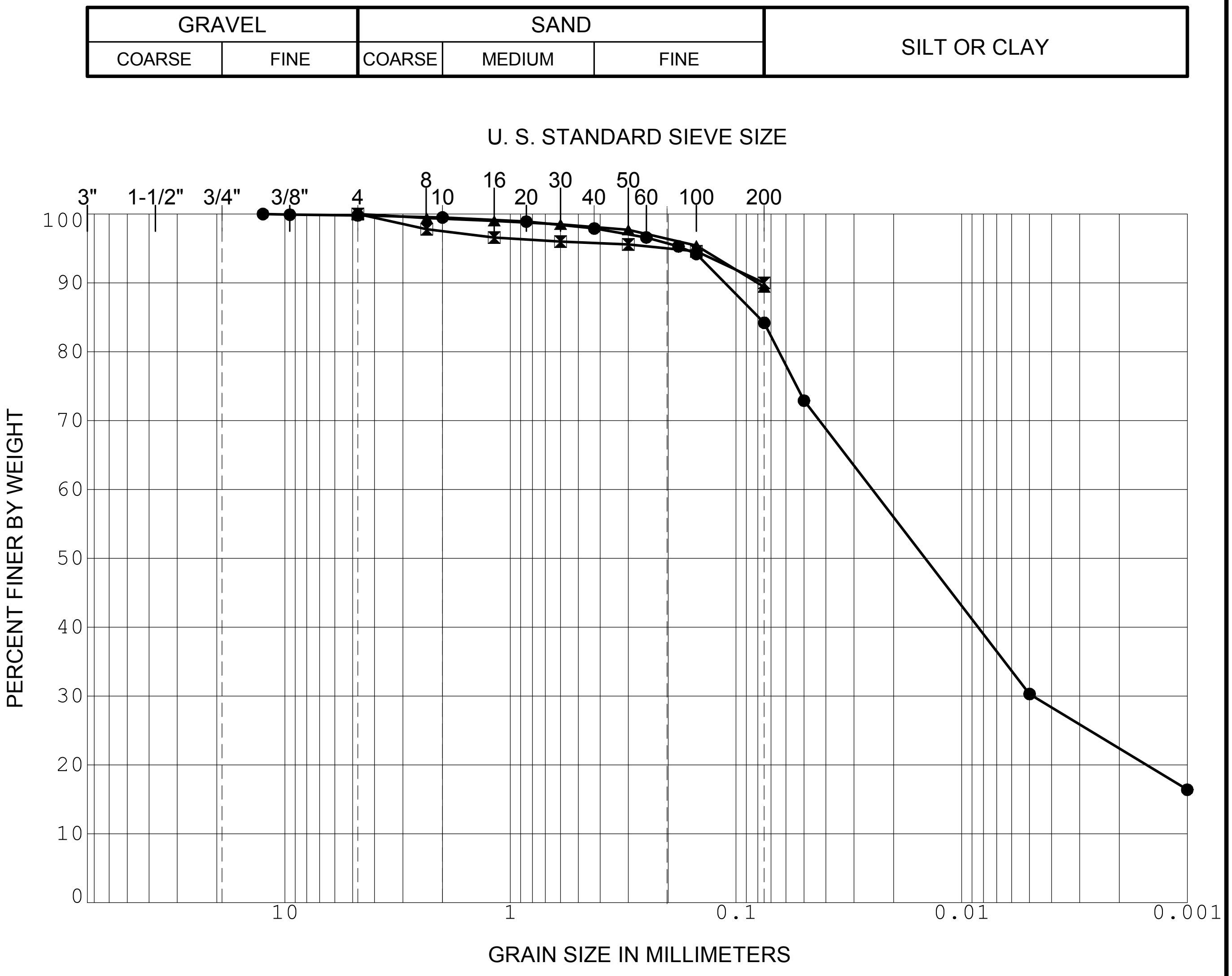
Sample No.	Moisture Content (%)		Dry Density (pcf)	Expansion Index	Expansion Classification
	Before Test	After Test			
B1-1	10.8	25.1	106.8	67	Medium
B4-2	11.1	20.3	106.7	28	Low
B8-7	12.0	26.2	102.6	68	Medium
B10-1	9.5	20.3	110.7	57	Medium
B11-3	14.6	29.0	95.1	67	Medium

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

Sample No.	Dry Density (pcf)	Moisture Content (%)		Unit Cohesion (psf)	Angle of Shear Resistance (degrees)
		Initial	Final		
B4-3	109.7	16.6	18.8	1330	32
B10-3	109.3	19.3	21.2	800	17
B12-2	109.0	13.5	18.8	1000	14

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

Sample No.	Water-Soluble Sulfate (%)	Classification
B1-1	0.015	Not Applicable (S0)
B4-2	0.025	Not Applicable (S0)
B8-7	1.010	Severe (S2)
B10-1	0.073	Not Applicable (S0)
B11-3	1.051	Severe (S2)



	SAMPLE	DEPTH (ft)	CLASSIFICATION	NAT WC	LL	PL	PI
●	B2-1	1.0	ML - SILT with Sand				
⧗	B10-3	10.0	(ML) Yellowish brown SILT				
▲	B11-3	10.0	(CL) Yellowish brown Lean CLAY				

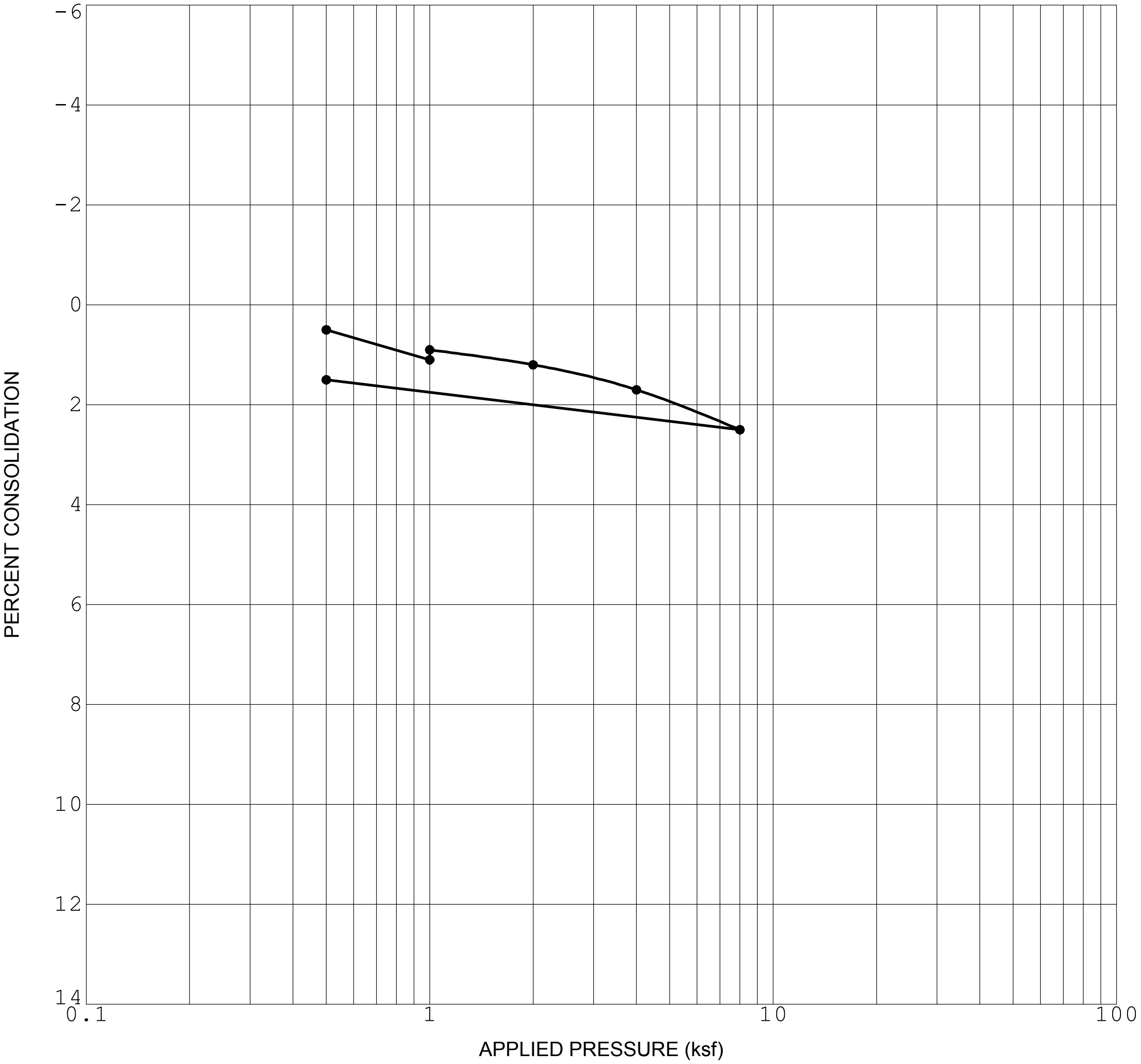
GRADATION CURVE

10290 CAMPUS POINT DRIVE

SAN DIEGO, CALIFORNIA

PROJECT NO. 07850-42-15

SAMPLE NO. B1-2



Initial Dry Density (pcf)	105.8		**Initial Saturation (%)**	95.6
Initial Water Content (%)	20.5		**Sample Saturated at (ksf)**	1.0

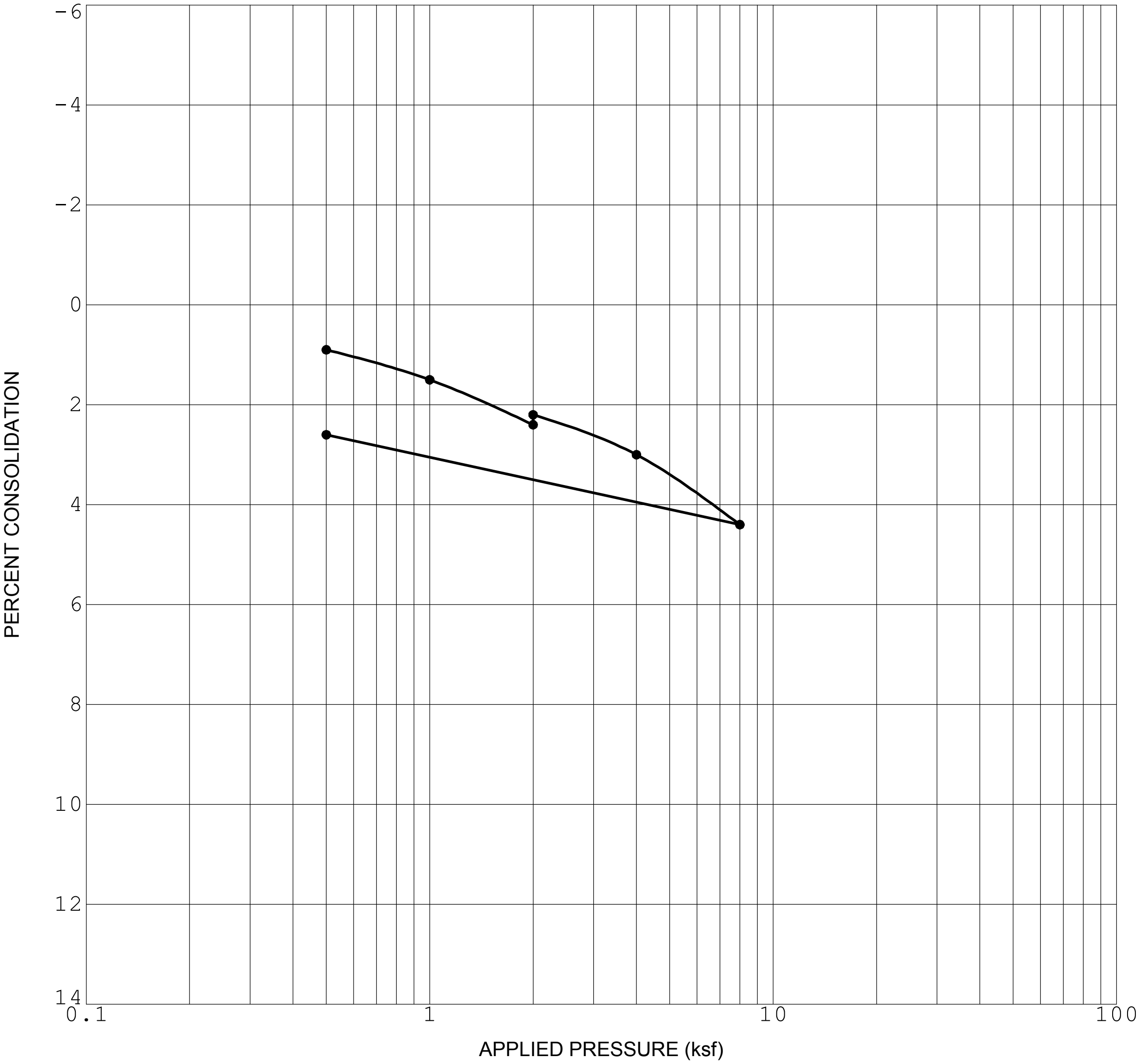
CONSOLIDATION CURVE

10290 CAMPUS POINT DRIVE

SAN DIEGO, CALIFORNIA

PROJECT NO. 07850-42-15

SAMPLE NO. B1-4



Initial Dry Density (pcf)	100.1
Initial Water Content (%)	24.6

Initial Saturation (%)	99.5
Sample Saturated at (ksf)	2.0

CONSOLIDATION CURVE

10290 CAMPUS POINT DRIVE

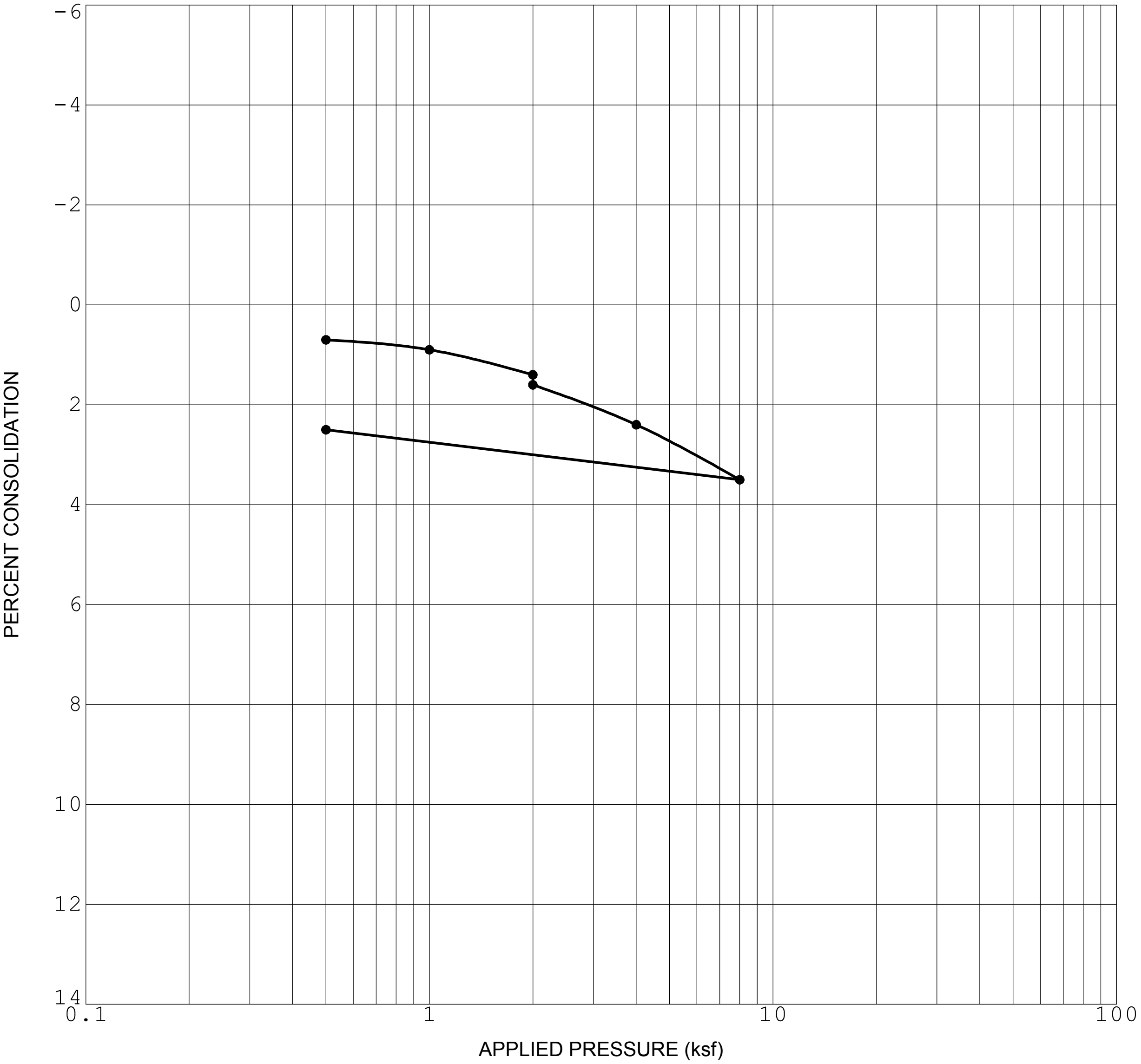
SAN DIEGO, CALIFORNIA

07850-42-15 (UPDATED).GPJ

Figure B-3

GEOCON

SAMPLE NO. B6-4



Initial Dry Density (pcf)	109.7
Initial Water Content (%)	14.8

Initial Saturation (%)	76.6
Sample Saturated at (ksf)	2.0

CONSOLIDATION CURVE

10290 CAMPUS POINT DRIVE

SAN DIEGO, CALIFORNIA

SUBSURFACE EXPLORATION LEGEND

UNIFIED SOIL CLASSIFICATION CHART

SOIL DESCRIPTION		GROUP SYMBOL	TYPICAL NAMES
I. COARSE GRAINED, more than half of material is larger than No. 200 sieve size.			
GRAVELS More than half of coarse fraction is larger than No. 4 sieve size but smaller than 3".	CLEAN GRAVELS	GW	Well graded gravels, gravel-sand mixtures, little or no fines.
		GP	Poorly graded gravels, gravel sand mixtures, little or no fines.
	GRAVELS WITH FINES (Appreciable amount of fines)	GM	Silty gravels, poorly graded gravel-sand-silt mixtures.
		GC	Clayey gravels, poorly graded gravel-sand, clay mixtures.
SANDS More than half of coarse fraction is smaller than No. 4 sieve size.	CLEAN SANDS	SW	Well graded sand, gravelly sands, little or no fines.
		SP	Poorly graded sands, gravelly sands, little or no fines.
	SANDS WITH FINES (Appreciable amount of fines)	SM	Silty sands, poorly graded sand and silty mixtures.
		SC	Clayey sands, poorly graded sand and clay mixtures.
II. FINE GRAINED, more than half of material is smaller than No. 200 sieve size.			
	SILTS AND CLAYS	ML	Inorganic silts and very fine sands, rock flour, sandy silt or clayey-silt-sand mixtures with slight plasticity.
	Liquid Limit less than 50	CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays.
		OL	Organic silts and organic silty clays or low plasticity.
	SILTS AND CLAYS	MH	Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts.
	Liquid Limit greater than 50	CH	Inorganic clays of high plasticity, fat clays.
		OH	Organic clays of medium to high plasticity.
	HIGHLY ORGANIC SOILS	PT	Peat and other highly organic soils.

▼ — Water level at time of excavation or as indicated

US — Undisturbed, driven ring sample or tube sample

CK — Undisturbed chunk sample

BG — Bulk sample

SP — Standard penetration sample

SOUTHERN CALIFORNIA SOIL & TESTING, INC.

QUALCOMM/IVAC	
BY: CHC	DATE: 10-10-95
JOB NUMBER: 9511205	Plate No. 2

DEPTH (ft.)	SAMPLE TYPE	SOIL CLASSIFICATION	**BORING NUMBER 1** ELEVATION DESCRIPTION	APPARENT MOISTURE	APPARENT CONSISTENCY OR DENSITY	PENETRATION RESISTANCE (blows/ft. of drive)	DRY DENSITY (pcf)	MOISTURE CONTENT (%)	RELATIVE COMPACTION (%)
0		SM	FILL, Tan to Light Brown, SILTY SAND	Humid	Loose				
2				Moist	Dense				
	US					47	108.8	10.1	
4	BAG								
	US	ML	Yellow-Green Tan and Medium Grey, SLIGHTLY CLAYEY, VERY SANDY SILT	Moist	Stiff	38	104.9	19.8	
6									
8	BAG								
10	US	ML		Moist	Stiff	30	100.3	23.1	
12									
14									
16	US	SM	Tan to Reddish Tan, SILTY SAND	Moist	Dense	35	106.1	17.9	
18		SM	REWORKED ALLUVIUM, Grey to Dark Brown, SLIGHTLY CLAYEY SILTY SAND with Roots and Organic Odor, Topsoil and Subsoil	Moist	Medium Dense				
20							105.7	12.3	
	US					46	118.5	12.0	
22		SM-SC	SCRIPPS FORMATION, Light Reddish Tan, CLAYEY SILTY SAND	Moist	Dense				
24		SM	Tan, SILTY SAND	Moist	Very Dense				
26	US					50/5"	96.8	9.0	
28			Light Grey						
30	US		Bottom at 30.5 Feet			50/5"			

SOUTHERN CALIFORNIA
SOIL & TESTING, INC.

SUBSURFACE EXPLORATION LOG

LOGGED BY: JRH

DATE LOGGED: 09-28-95

JOB NUMBER: 9511205

Plate No. 3

DEPTH (ft.)	SAMPLE TYPE	SOIL CLASSIFICATION	**BORING NUMBER 2** ELEVATION DESCRIPTION	APPARENT MOISTURE	APPARENT CONSISTENCY OR DENSITY	PENETRATION RESISTANCE (blows/ft. of drive)	DRY DENSITY (pcf)	MOISTURE CONTENT (%)	RELATIVE COMPACTION (%)
0		SM	SCRIPPS FORMATION, Light Tan to Yellow Tan, SILTY SAND	Humid	Loose				
2	BAG			Moist	Dense				
4	US	SM-ML	VERY SILTY SAND	Moist	Dense/ Hard	44	101.3	8.0	
6	US	SM	Tan to Light Brown, SILTY SAND	Moist	Dense	68	101.7	7.8	
8									
10	US	SM	Tan, SILTY SAND	Moist	Dense	50/5"	103.7	7.9	
12									
14									
16	US	ML	Yellow Tan, SANDY SILT	Moist	Hard	86	109.8	18.2	
			Bottom at 16 Feet						

SOUTHERN CALIFORNIA SOIL & TESTING, INC.

SUBSURFACE EXPLORATION LOG

LOGGED BY: JRH

DATE LOGGED: 09-28-95

JOB NUMBER: 9511205

Plate No. 4

BORING NUMBER 3

ELEVATION

DEPTH (ft.)	SAMPLE TYPE	SOIL CLASSIFICATION	DESCRIPTION	APPARENT MOISTURE	APPARENT CONSISTENCY OR DENSITY	PENETRATION RESISTANCE (blows/ft. of drive)	DRY DENSITY (pcf)	MOISTURE CONTENT (%)	RELATIVE COMPACTION (%)
0		ML-CL	SCRIPPS FORMATION, Medium Grey to Yellow Tan, SANDY SILT TO SILTY CLAY	Humid	Soft				
2	US			Moist	Hard	67	108.5	17.6	
4									
	US	ML-SM	Yellow Tan to Light Grey, VERY SANDY SILT	Moist	Very Dense	50/6"	102.9	8.8	
6									
8									
10	US	SM	Light Grey, SILTY SAND	Moist	Very Dense	50/5"	96.9	6.9	
12			Refusal at 12 Feet on Highly Cemented Concretion						

SOUTHERN CALIFORNIA SOIL & TESTING, INC.

SUBSURFACE EXPLORATION LOG

LOGGED BY: JRH

DATE LOGGED: 09-28-95

JOB NUMBER: 9511205

Plate No. 5

DEPTH (ft.)	SAMPLE TYPE	SOIL CLASSIFICATION	**BORING NUMBER 4** ELEVATION DESCRIPTION	APPARENT MOISTURE	APPARENT CONSISTENCY OR DENSITY	PENETRATION RESISTANCE (blows/ft. of drive)	DRY DENSITY (pcf)	MOISTURE CONTENT (%)	RELATIVE COMPACTION (%)
0		SM	FILL, Tan to Light Brown, SILTY SAND with Rock	Humid	Loose				
2	US BAG			Moist	Dense	50			
4									
6	US					30	103.1	9.9	
8	BAG	SM-ML	SCRIPPS FORMATION, Light Tan to Tan, SILTY SAND/SANDY SILT	Moist	Very				
10	US					50/5"	97.2	8.7	
12		SM	SILTY SAND						
14									
	US	SM	SILTY SAND	Moist	Very Dense	50/5"	93.2	8.1	
16									
18									
20			Bottom at 20 Feet						

SOUTHERN CALIFORNIA SOIL & TESTING, INC.

SUBSURFACE EXPLORATION LOG

LOGGED BY: JRH

DATE LOGGED: 09-28-95

JOB NUMBER: 9511205

Plate No. 6

BORING NUMBER 5

ELEVATION

DEPTH (ft.)	SAMPLE TYPE	SOIL CLASSIFICATION	DESCRIPTION	APPARENT MOISTURE	APPARENT CONSISTENCY OR DENSITY	PENETRATION RESISTANCE (blows/ft. of drive)	DRY DENSITY (pcf)	MOISTURE CONTENT (%)	RELATIVE COMPACTION (%)
0	BAG	SM	FILL OR WEATHERED FORMATIONAL, Yellow Tan, SILTY SAND	Humid Moist	Loose Dense				
2									
4		SM	SCRIPPS FORMATION, Light Grey with Yellow Tan, SILTY SAND	Moist	Very Dense				
	US					50/5"	98.9	6.4	
6									
8	BAG								
10	US					50/4"			
12									
14									
	US	SM	SILTY SAND	Moist	Very Dense	50/4"	96.3	6.3	
16			Bottom at 15.5 Feet						

SOUTHERN CALIFORNIA SOIL & TESTING, INC.

SUBSURFACE EXPLORATION LOG

LOGGED BY: JRH

DATE LOGGED: 09-28-95

JOB NUMBER: 9511205

Plate No. 7

BORING NUMBER 6

ELEVATION

DEPTH (ft.)	SAMPLE TYPE	SOIL CLASSIFICATION	DESCRIPTION	APPARENT MOISTURE	APPARENT CONSISTENCY OR DENSITY	PENETRATION RESISTANCE (blows/ft. of drive)	DRY DENSITY (pcf)	MOISTURE CONTENT (%)	RELATIVE COMPACTION (%)
0		SM	SCRIPPS FORMATION, Yellow Tan, SILTY SAND	Humid	Loose				
2	US			Moist	Dense				
						50/4"	105.0	7.3	
4									
	US	SM				50/6"	96.6	10.3	
6									
8									
10									
	US	SM	Light Grey and Yellow Tan, SILTY SAND	Moist	Very Dense	50/2"	97.8	8.4	
12	BAG								
14									
			Bottom at 15 Feet						

SOUTHERN CALIFORNIA SOIL & TESTING, INC.

SUBSURFACE EXPLORATION LOG

LOGGED BY: JRH

DATE LOGGED: 09-28-95

JOB NUMBER: 9511205

Plate No. 8

BORING NUMBER 7

ELEVATION

DEPTH (ft.)	SAMPLE TYPE	SOIL CLASSIFICATION	DESCRIPTION	APPARENT MOISTURE	APPARENT CONSISTENCY OR DENSITY	PENETRATION RESISTANCE (blows/ft. of drive)	DRY DENSITY (pcf)	MOISTURE CONTENT (%)	RELATIVE COMPACTION (%)
0		SM	WEATHERED SCRIPPS FORMATION, TAN, SILTY SAND	Humid	Loose				
2		SM	SCRIPPS FORMATION, Yellow Tan to Tan, SILTY SAND	Moist	Dense				
4									
6			Bottom at 5 Feet						

BORING NUMBER 8

DEPTH (ft.)	SAMPLE TYPE	SOIL CLASSIFICATION	DESCRIPTION	APPARENT MOISTURE	APPARENT CONSISTENCY OR DENSITY	PENETRATION RESISTANCE (blows/ft. of drive)	DRY DENSITY (pcf)	MOISTURE CONTENT (%)	RELATIVE COMPACTION (%)
0		SM	SCRIPPS FORMATION, Light Tan to Yellow Tan, SILTY SAND	Humid	Loose				
2				Moist	Dense				
4									
6			Bottom at 5 Feet						

BORING NUMBER 9

DEPTH (ft.)	SAMPLE TYPE	SOIL CLASSIFICATION	DESCRIPTION	APPARENT MOISTURE	APPARENT CONSISTENCY OR DENSITY	PENETRATION RESISTANCE (blows/ft. of drive)	DRY DENSITY (pcf)	MOISTURE CONTENT (%)	RELATIVE COMPACTION (%)
0		SM	FILL, Tan to Yellow Tan, SILTY SAND	Humid	Loose				
2				Moist	Dense				
4		SM	SCRIPPS FORMATION, Yellow Tan, SILTY SAND	Moist	Very Dense				
6			Bottom at 5 Feet						

SOUTHERN CALIFORNIA SOIL & TESTING, INC.

SUBSURFACE EXPLORATION LOG

LOGGED BY: JRH	DATE LOGGED: 09-28-95
JOB NUMBER: 9511205	Plate No. 9

DEPTH (ft.)	SAMPLE TYPE	SOIL CLASSIFICATION	**BORING NUMBER 10** ELEVATION DESCRIPTION	APPARENT MOISTURE	APPARENT CONSISTENCY OR DENSITY	PENETRATION RESISTANCE (blows/ft. of drive)	DRY DENSITY (pcf)	MOISTURE CONTENT (%)	RELATIVE COMPACTION (%)
0		SM	FILL, Tan to Yellow Tan, SILTY SAND	Humid	Loose				
2				Moist	Dense				
4									
6		ML	SCRIPPS FORMATION, Yellow Tan, SANDY SILT	Moist	Hard				
8			Bottom at 7 Feet						

SOUTHERN CALIFORNIA SOIL & TESTING, INC.

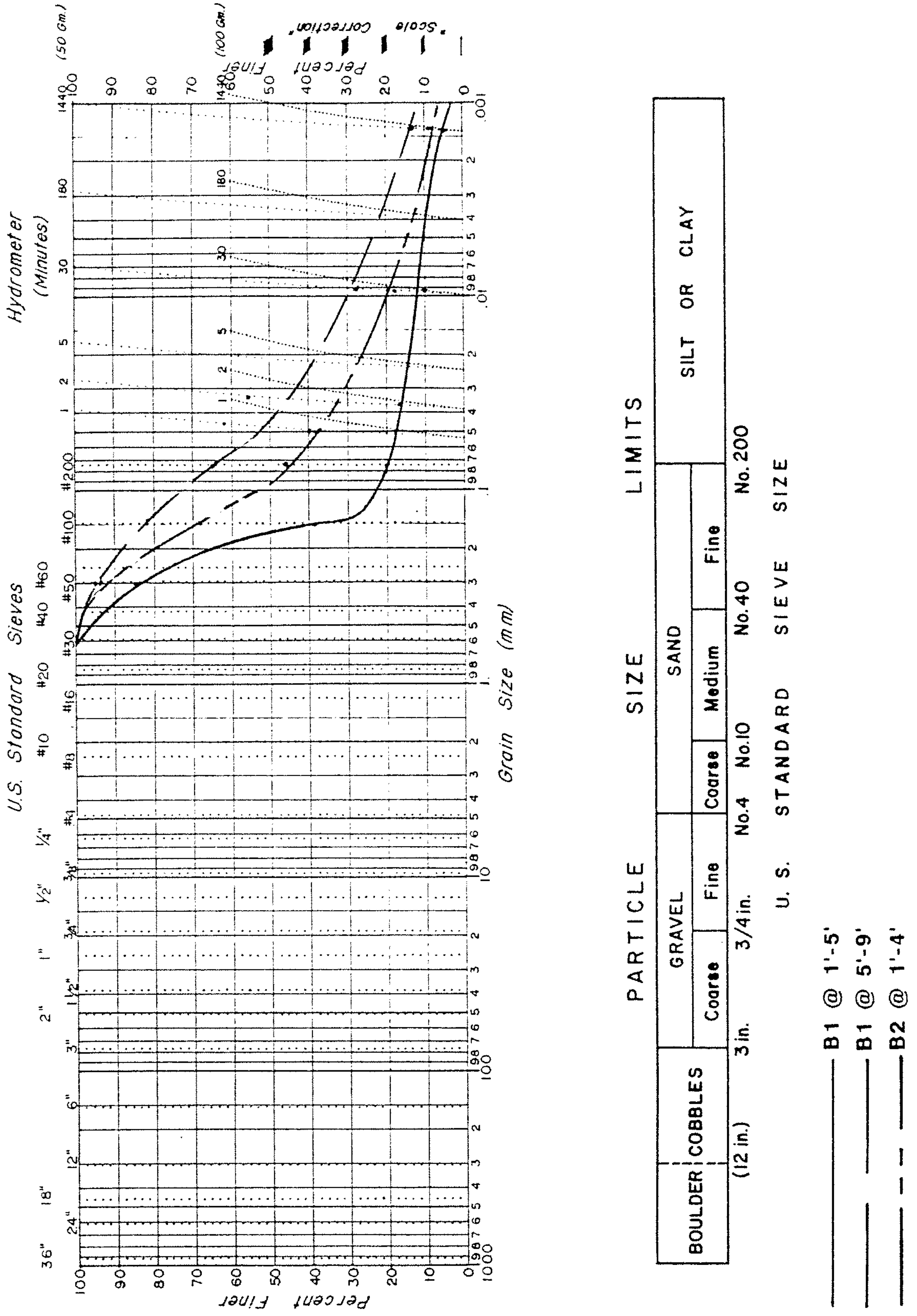
SUBSURFACE EXPLORATION LOG

LOGGED BY: JRH

DATE LOGGED: 09-28-95

JOB NUMBER: 9511205

Plate No. 10



SOUTHERN CALIFORNIA SOIL & TESTING, INC.

QUALCOMM/IVAC

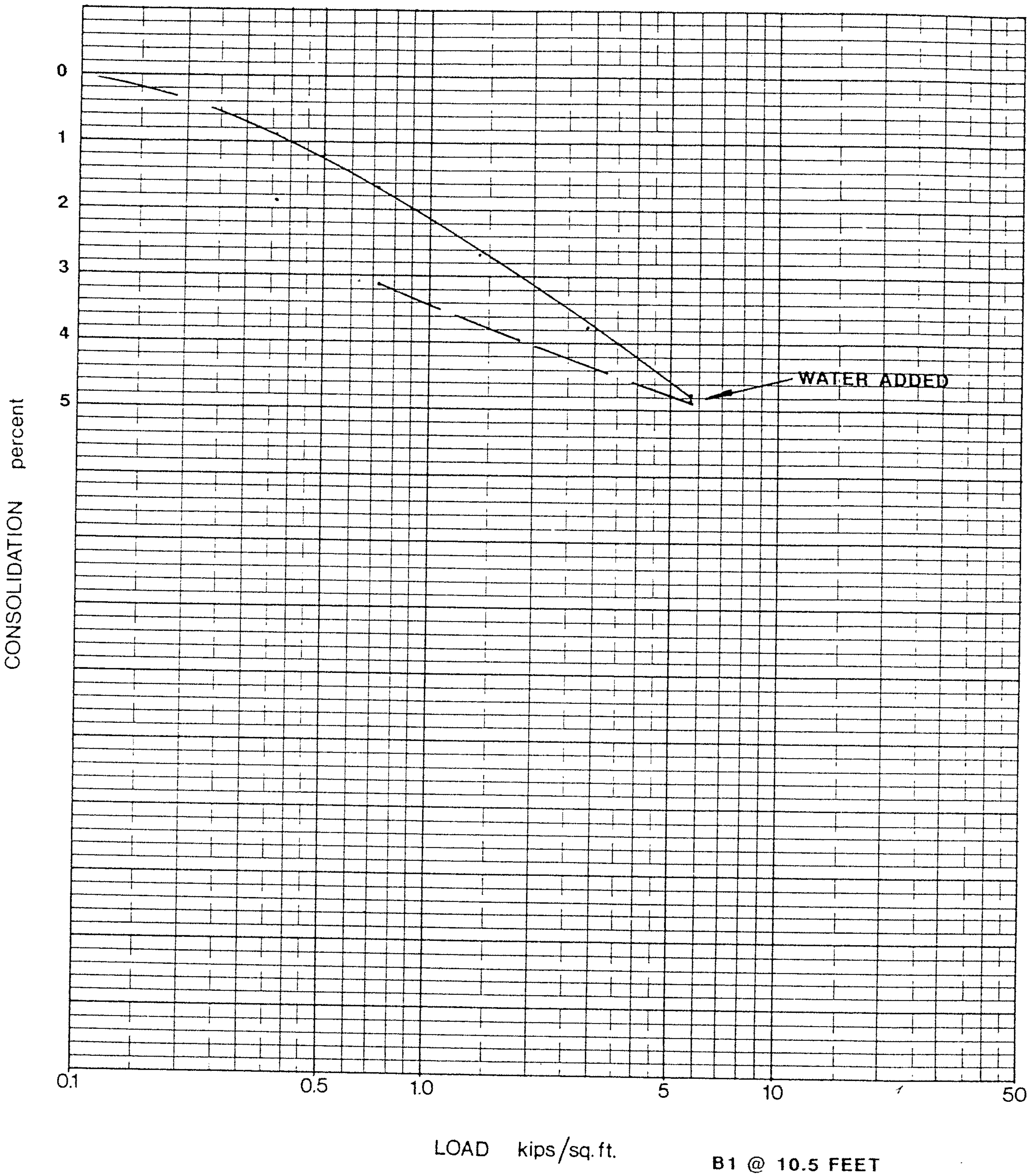
GRAIN SIZE DISTRIBUTION

BY
CHC

DATE
10-13-95

JOB NO.
9511205

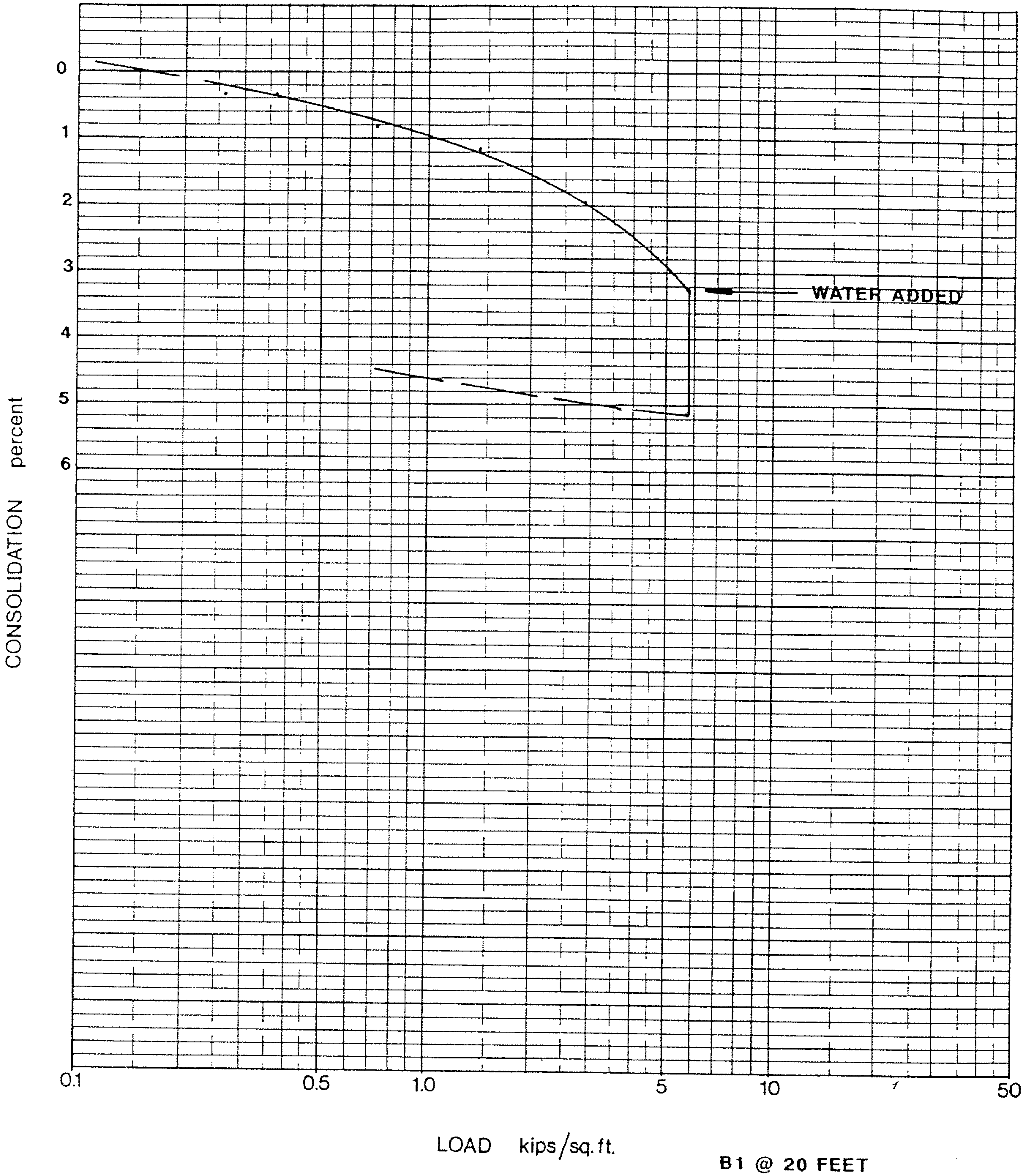
Plate No. 11



B1 @ 10.5 FEET

SOUTHERN CALIFORNIA
SOIL & TESTING LAB, INC.
6280 RIVERDALE STREET
SAN DIEGO, CALIFORNIA 92120

QUALCOMM/IVAC	
BY CHC	DATE 10-13-95
JOB NO. 9511205	Plate No. 12

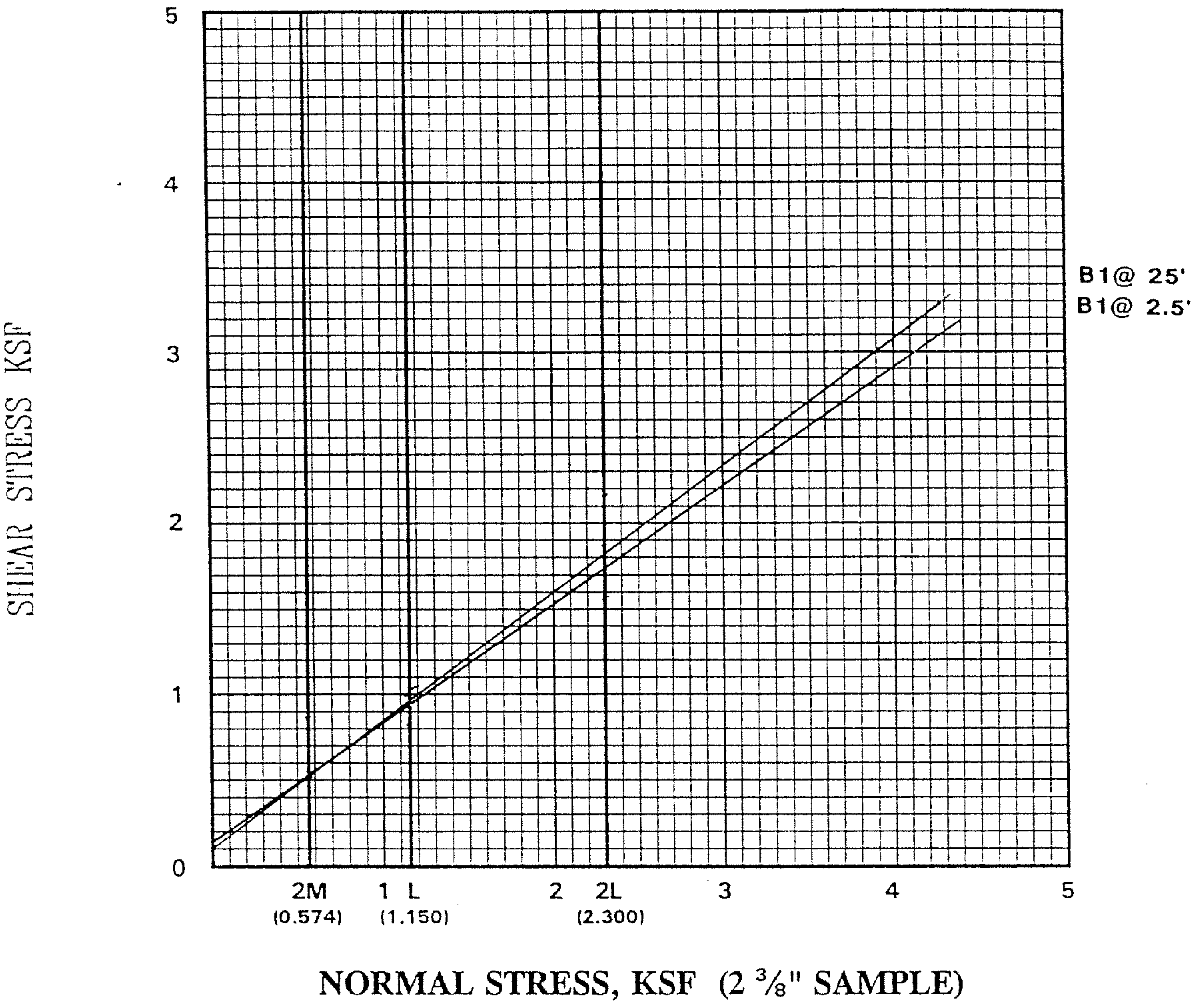


B1 @ 20 FEET

SOUTHERN CALIFORNIA
SOIL & TESTING LAB, INC.
6280 RIVERDALE STREET
SAN DIEGO, CALIFORNIA 92120

QUALCOMM/IVAC	
BY CHC	DATE 10-13-95
JOB NO. 9511205	Plate No. 13

DIRECT SHEAR SUMMARY



SAMPLE	DESCRIPTION	ANGLE OF INTERNAL FRICTION	COHESION INTERCEPT (PSF)
B1 @ 2.5'	Undisturbed	35 Degrees	150 psf
B1 @ 25'	Undisturbed	37 Degrees	100 psf

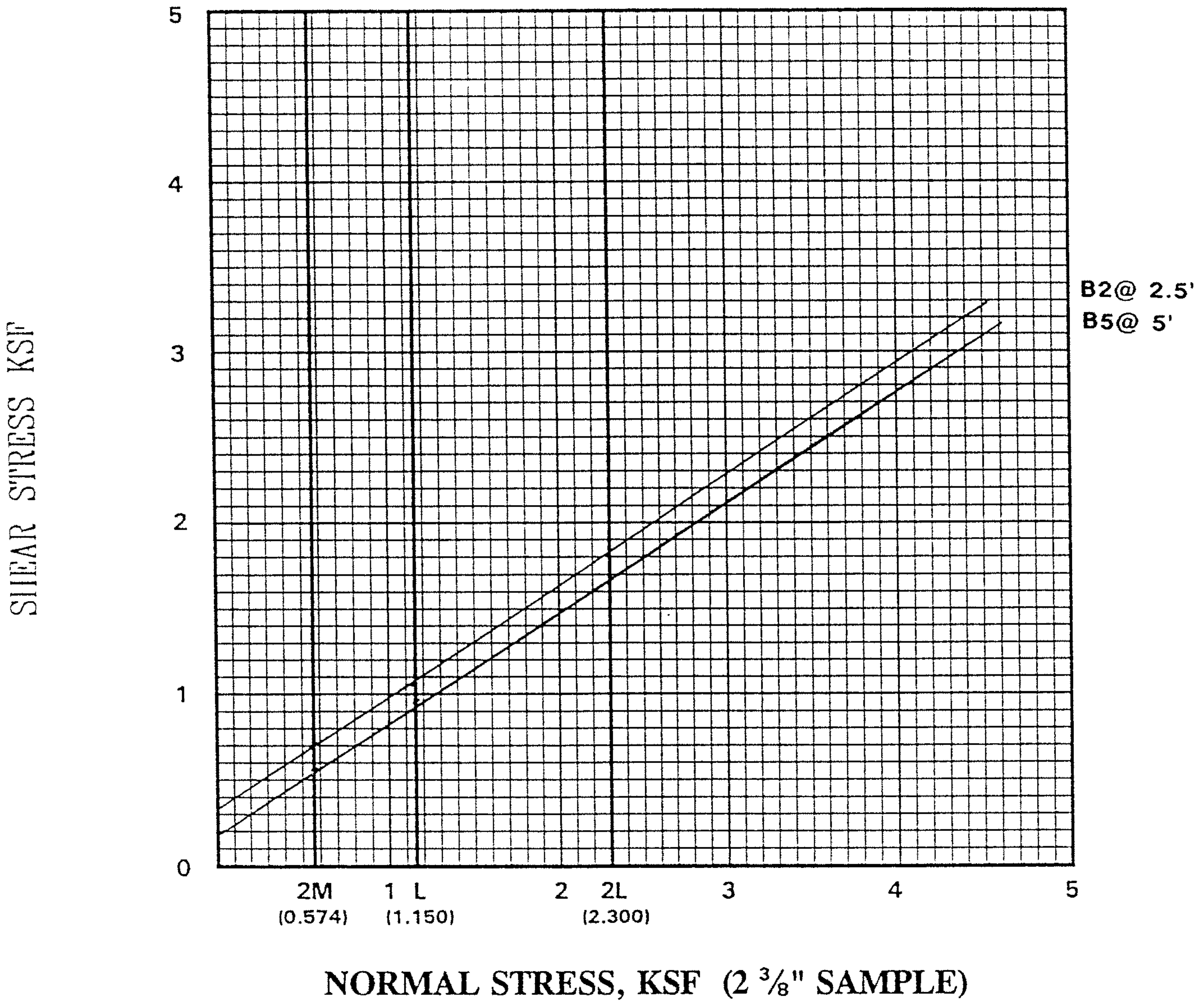
PROVING RING No. ____________

SOUTHERN CALIFORNIA
SOIL & TESTING, INC.

QUALCOMM/IVAC

BY:	CHC	**DATE:**	10-13-95
JOB NUMBER:	9511205	**PLATE No.:**	14

DIRECT SHEAR SUMMARY



SAMPLE	DESCRIPTION	ANGLE OF INTERNAL FRICTION	COHESION INTERCEPT (PSF)
B2 @ 2.5'	Undisturbed	33 Degrees	350 psf
B5 @ 5'	Undisturbed	33 Degrees	175 psf

PROVING RING No. ____________

SOUTHERN CALIFORNIA
SOIL & TESTING, INC.

QUALCOMM/IVAC

BY: CHC
DATE: 10-13-95
JOB NUMBER: 9511205
PLATE No.: 15

File No. D-2379-M01
November 10, 1980

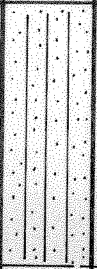
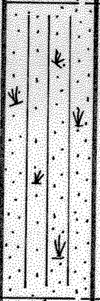
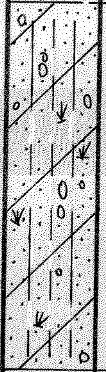
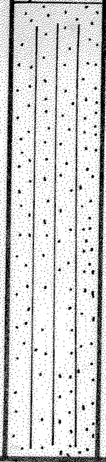
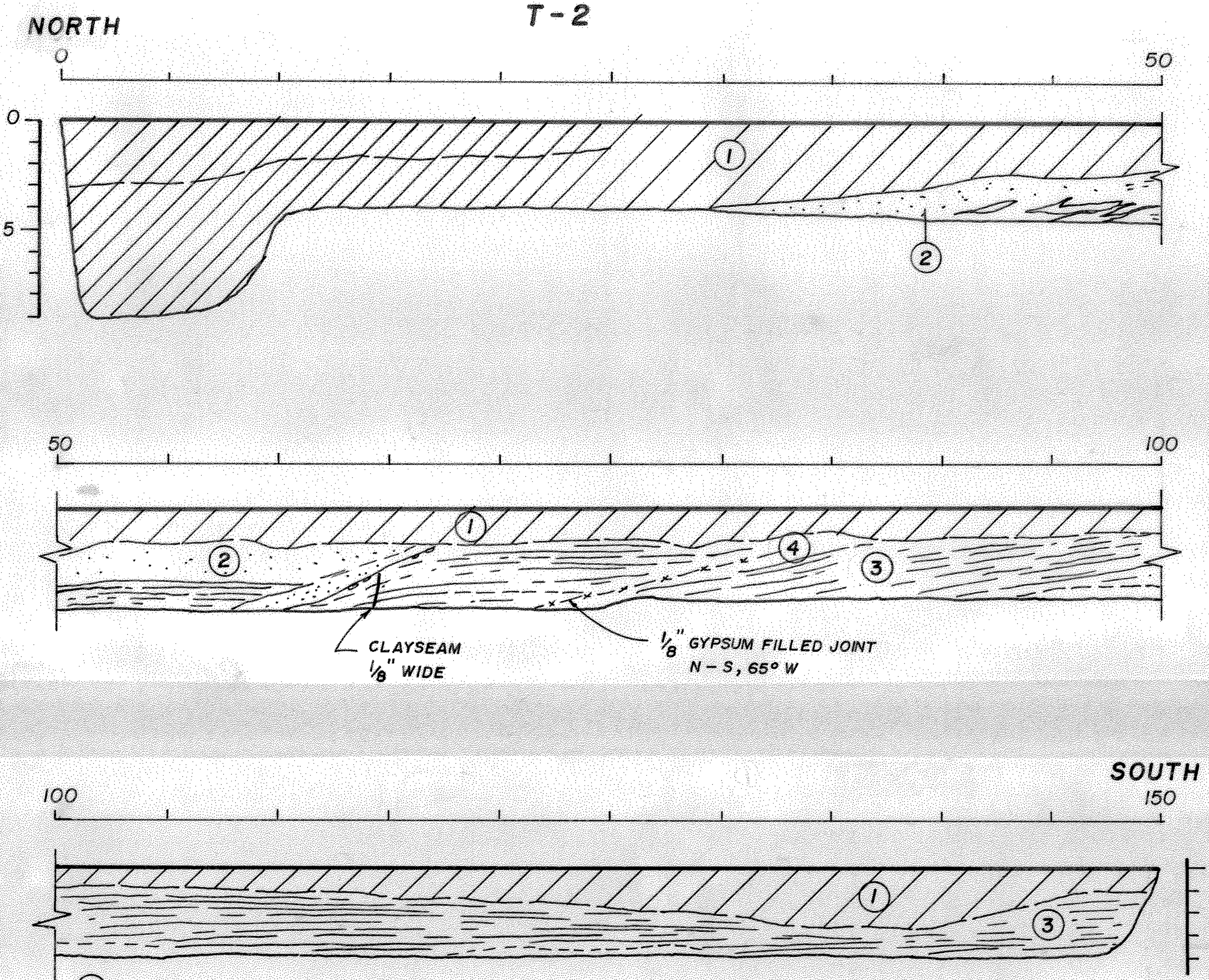
DEPTH IN FEET	SAMPLE NUMBER	LOG & LOCATION OF SAMPLE	Penetration Resistance Blows/ft	DESCRIPTION	IN-PLACE	
					DRY DENSITY p.c.f	MOISTURE CONTENT % dry wt
				BORING NO. 1		
0 2 4				FILL Medium dense, dry, angular SILTSTONE fragments in a Sandy matrix		
6 8 10				Loose to medium dense, damp, tan-brown, SAND with angular gray SILTSTONE, topsoil layers/fragments, organics		
12 14 16 18				Loose, dary, Sandy GRAVEL layer 1' thick, soft, moist, dark brown, Silty CLAY with angular SILTSTONE fragments and organics (grass, rocks) frequent cobbles		
20 22 24 26 28 30				SCRIPPS FORMATION Very dense, damp, interbedded (beds thickness within 1") reddish-brown, very fine SANDSTONE and light gray Sandy SILTSTONE, very well cemented Gravel attitude N10°W/9°W BORING TERMINATED AT 30.0 FEET		

Figure 18, Log of Test Boring 1



① FILL, dry to moist, medium dense to loose, Silty fine SAND (CLAY)

② Dense, massive bedded, reddish-brown, Silty SANDSTONE with shale rip-up clasts

③ Very dense, light gray SILTSTONE

④ Gypsum bed

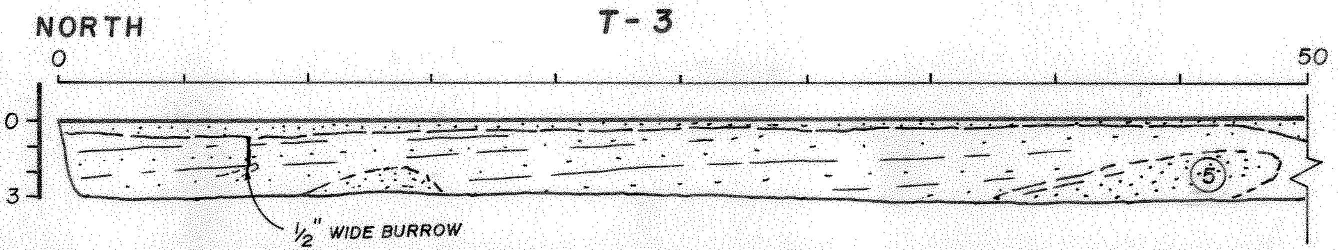
LOG OF TRENCH NO. 2

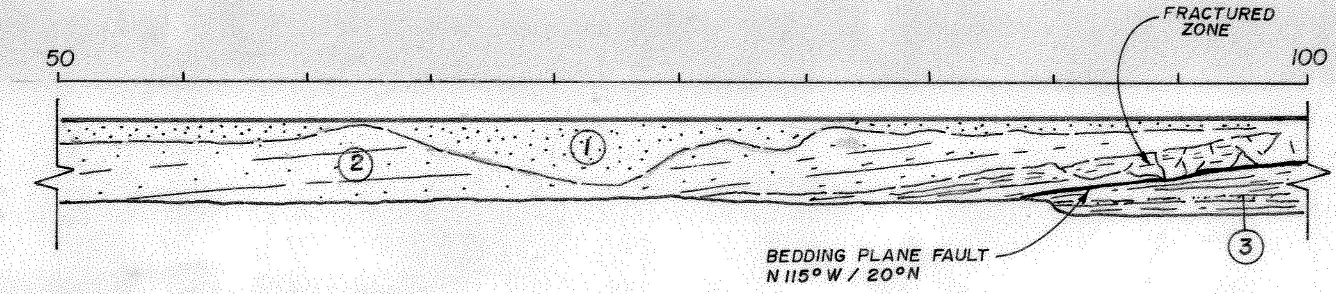
CAMPUS POINT, PHASE II
San Diego, California

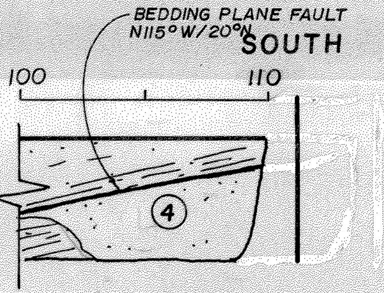
FIGURE 4

GEOCON, INCORPORATED

PAGE A-2







① Light brown, very dense, massive SANDSTONE with siltstone clasts

② Light gray, dense, damp, medium cemented SANDSTONE

③ Stiff, damp, dark gray, very well cemented, Silty CLAYSTONE

④ Dense, well cemented SANDSTONE

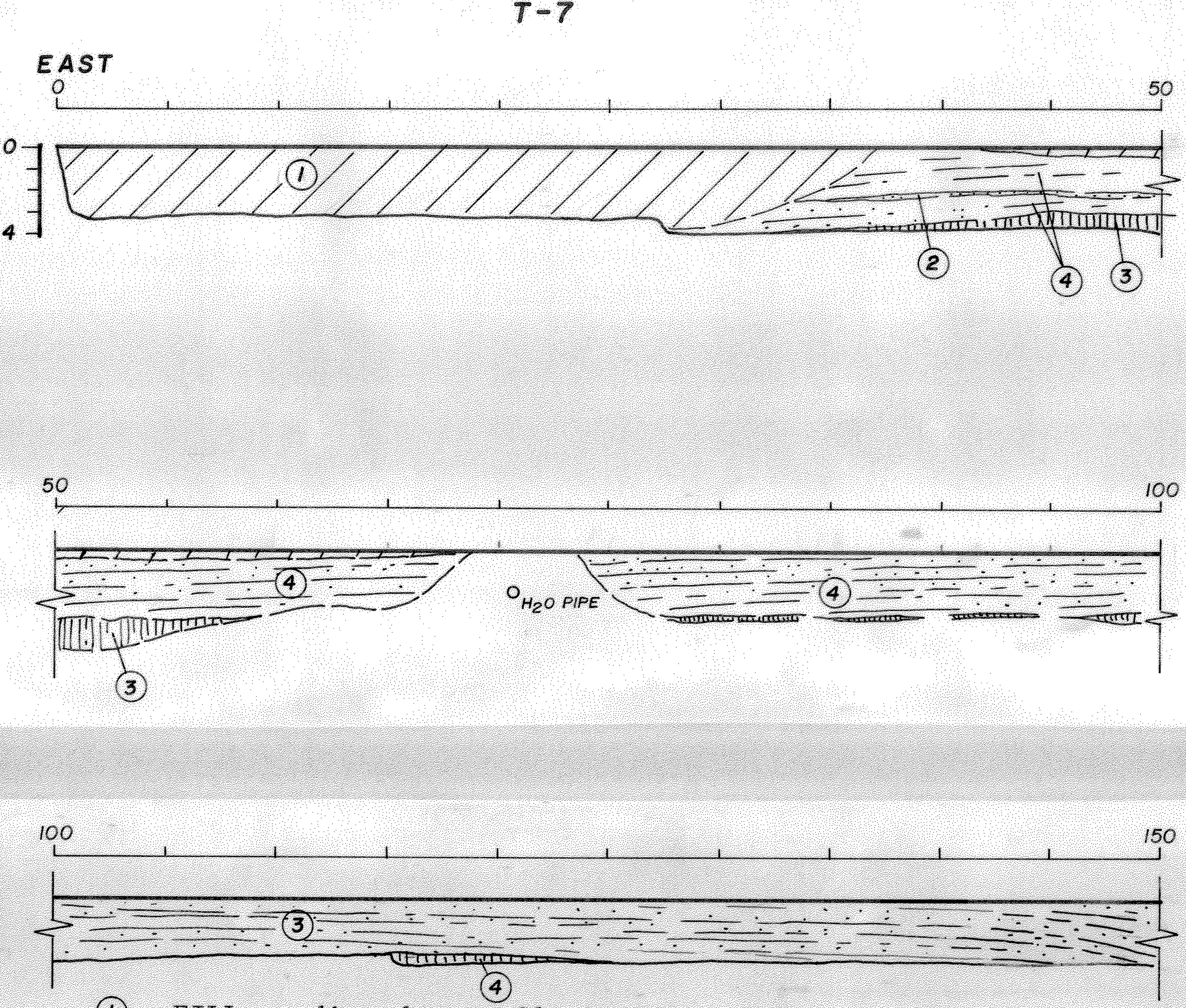
⑤ Hard, very well cemented, concretion with laminated Silty fine SAND, reddish-brown stain

LOG OF TRENCH NO. 3

CAMPUS POINT, PHASE II
San Diego, California

FIGURE 5

GEOCON, INCORPORATED



① FILL, medium dense, Clayey Silty SAND

② Loose Sandy layer

③ Very dense, massive, light brown-tan, well cemented Silty SANDSTONE

④ Very dense, damp, gray to tan, massive, very fine Sandy Clayey SILTSTONE/SANDSTONE

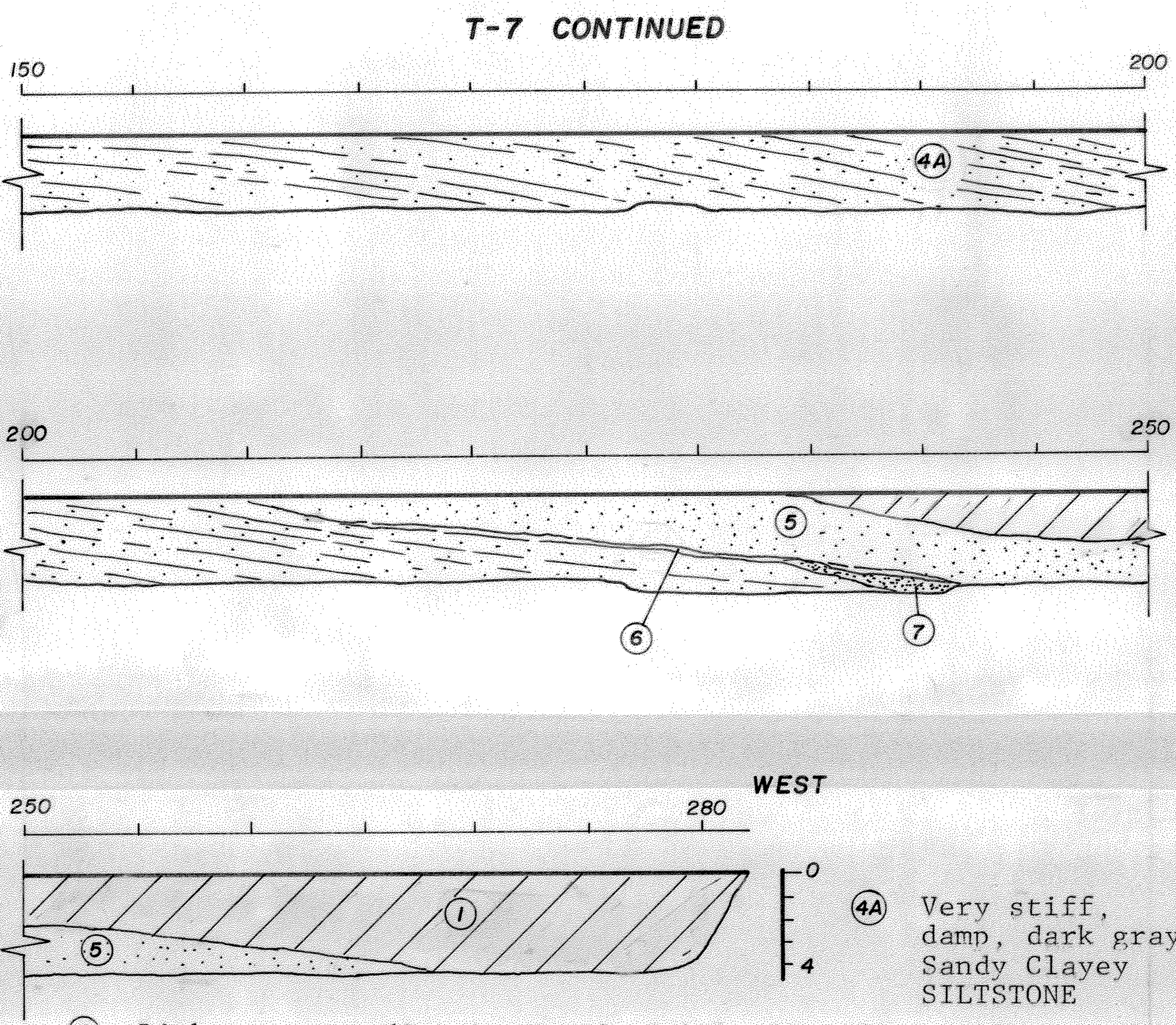
LOG OF TRENCH NO. 7

CAMPUS POINT, PHASE II
San Diego, California

Continued next page

FIGURE 9

GEOCON, INCORPORATED



④A Very stiff, damp, dark gray, Sandy Clayey SILTSTONE

⑤ Light gray, medium cemented, dense, dry, fine to medium SAND

⑥ Highly fractured & sheared zone, thickness 1/2"-1/2', dark brown, reddish-gray, Silty CLAYSTONE, dipping NW ≈ 15°, possible bedding plane fault

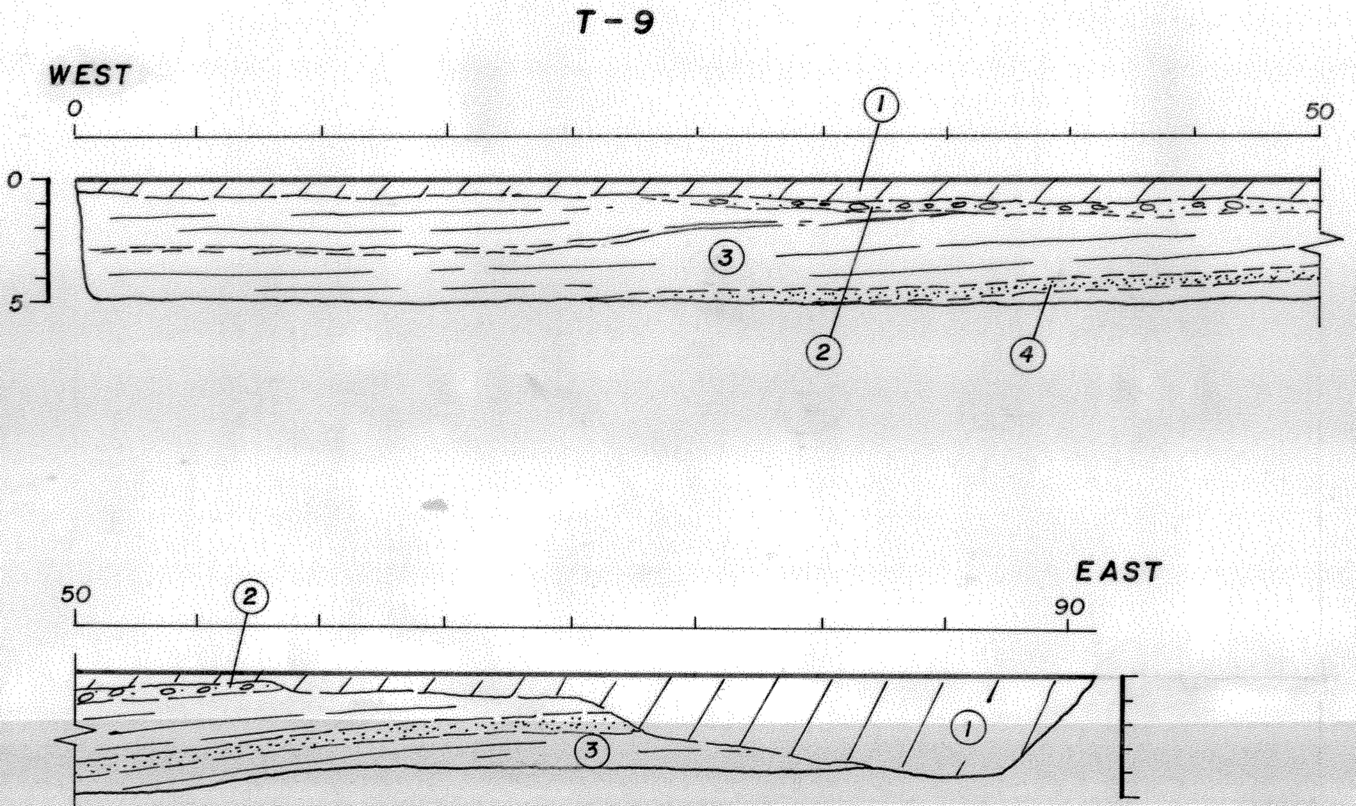
⑦ Hard, cemented concretion

LOG OF TRENCH NO. 7 CONTINUED

CAMPUS POINT, PHASE II
San Diego, California

FIGURE 10

GEOCON, INCORPORATED



① FILL

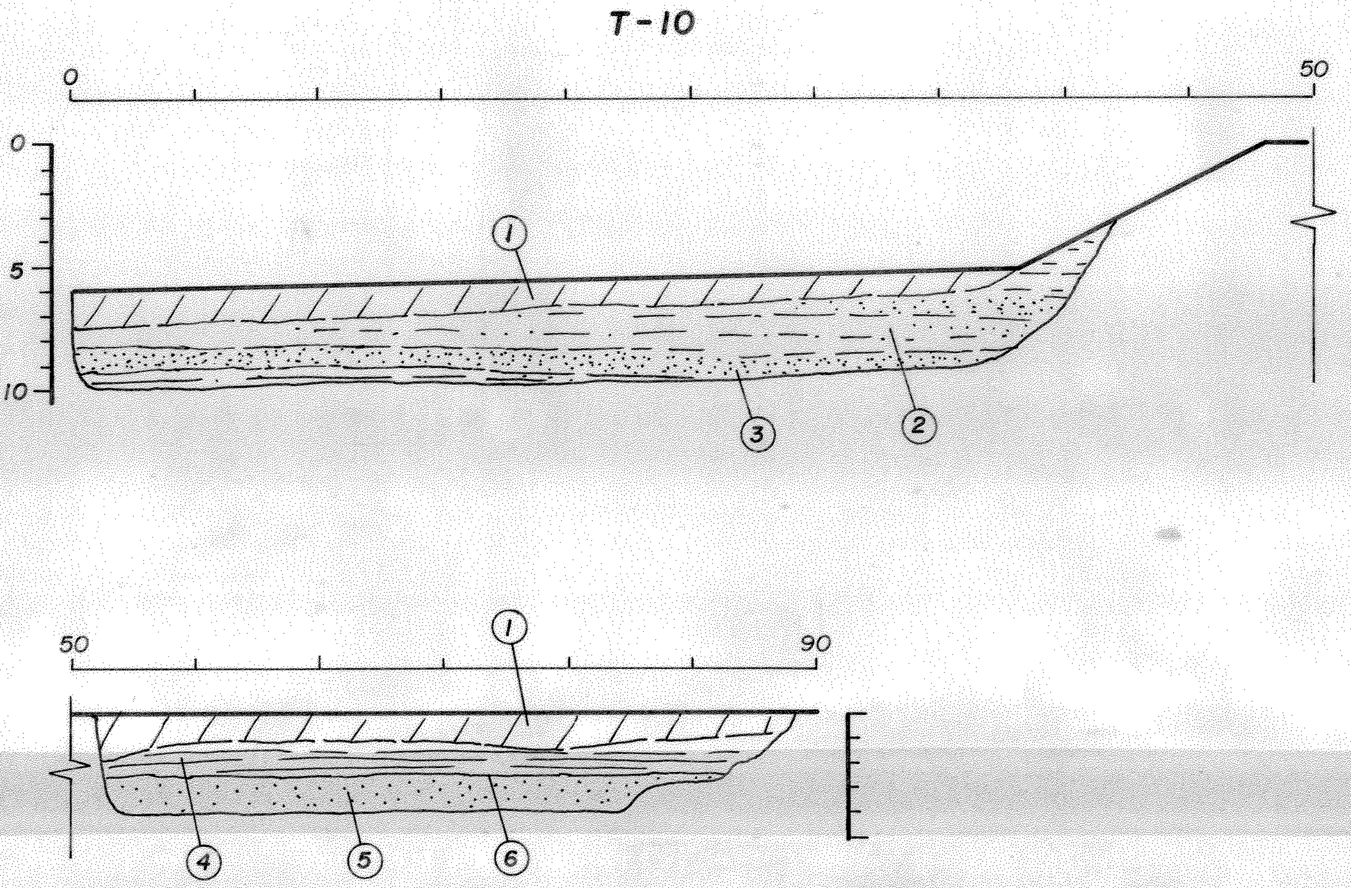
② Red-brown, Sandy pebble CONGLOMERATE

③ Very dense, moist, gray, reddish stained Clayey SILTSTONE, very well cemented, fissile, horizontally bedded

④ Red-brown, Silty SANDSTONE

LOG OF TRENCH NO. 9

CAMPUS POINT, PHASE II
San Diego, California



① FILL, loose Silty SAND

② Very dense, moist, gray, fine Sandy SILTSTONE

③ Red-brown, Silty STANDSTONE layer 8" thick

④ Very dense, gray SILTSTONE with red stains

⑤ Gray fine SAND

⑥ Red stain

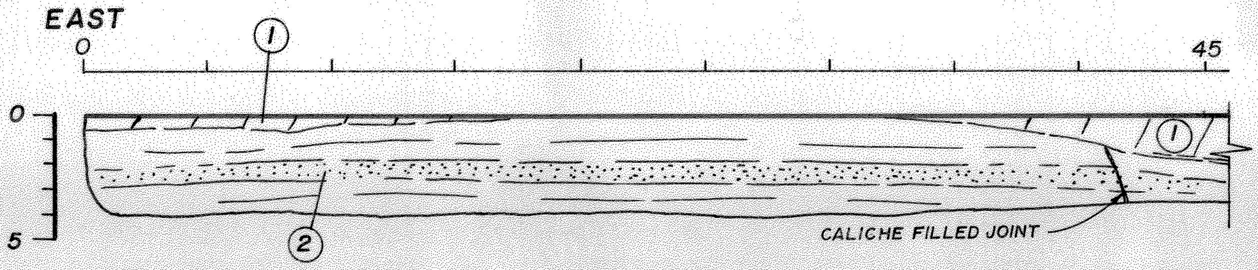
LOG OF TRENCH NO. 10

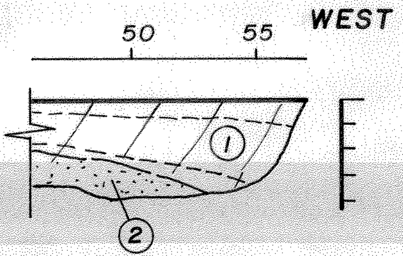
CAMPUS POINT, PHASE II
San Diego, California

FIGURE 13

GEOCON, INCORPORATED

T-12





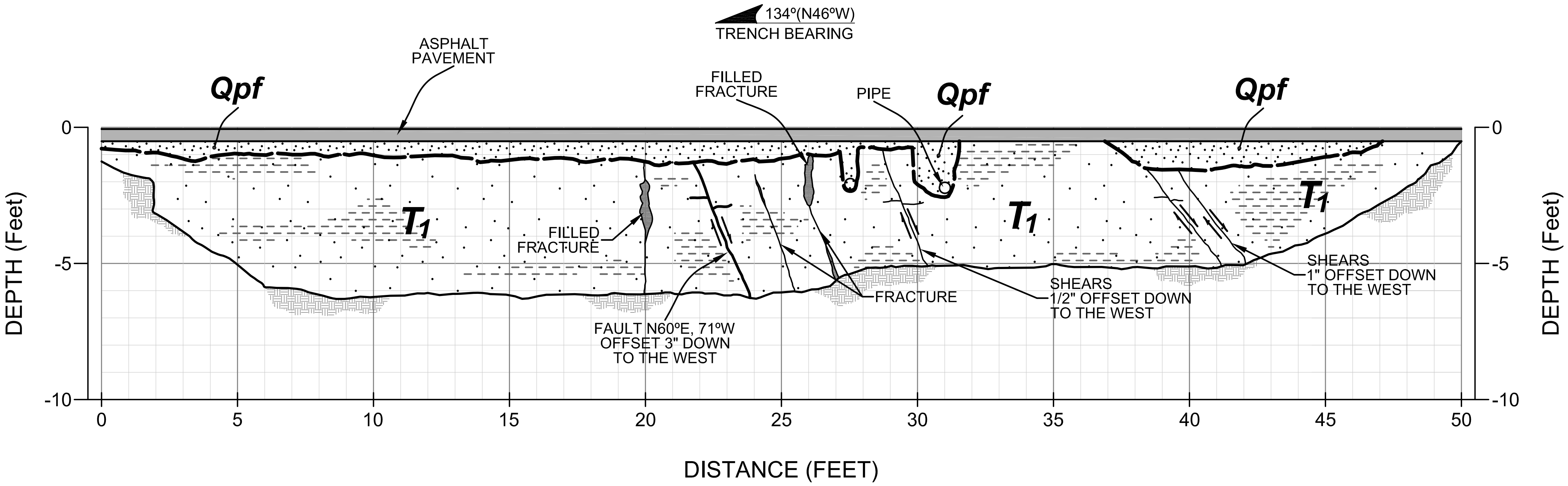
① FILL, loose to medium dense, dry to moist, Silty SAND/ Silty CLAY (TOPSOIL)

② Massive bedded, dense, gray, Silty medium SANDSTONE

LOG OF TRENCH NO. 12

CAMPUS POINT, PHASE II
San Diego, California

CAMPUS POINTE MASTER PLAN
10300 CAMPUS POINT DRIVE
SAN DIEGO, CALIFORNIA



FAULT TRENCH FT-1

SCALE: 1" = 5' (Vert. = Horiz.)

GEOCON LEGEND

Qpf ....PREVIOUSLY PLACED FILL

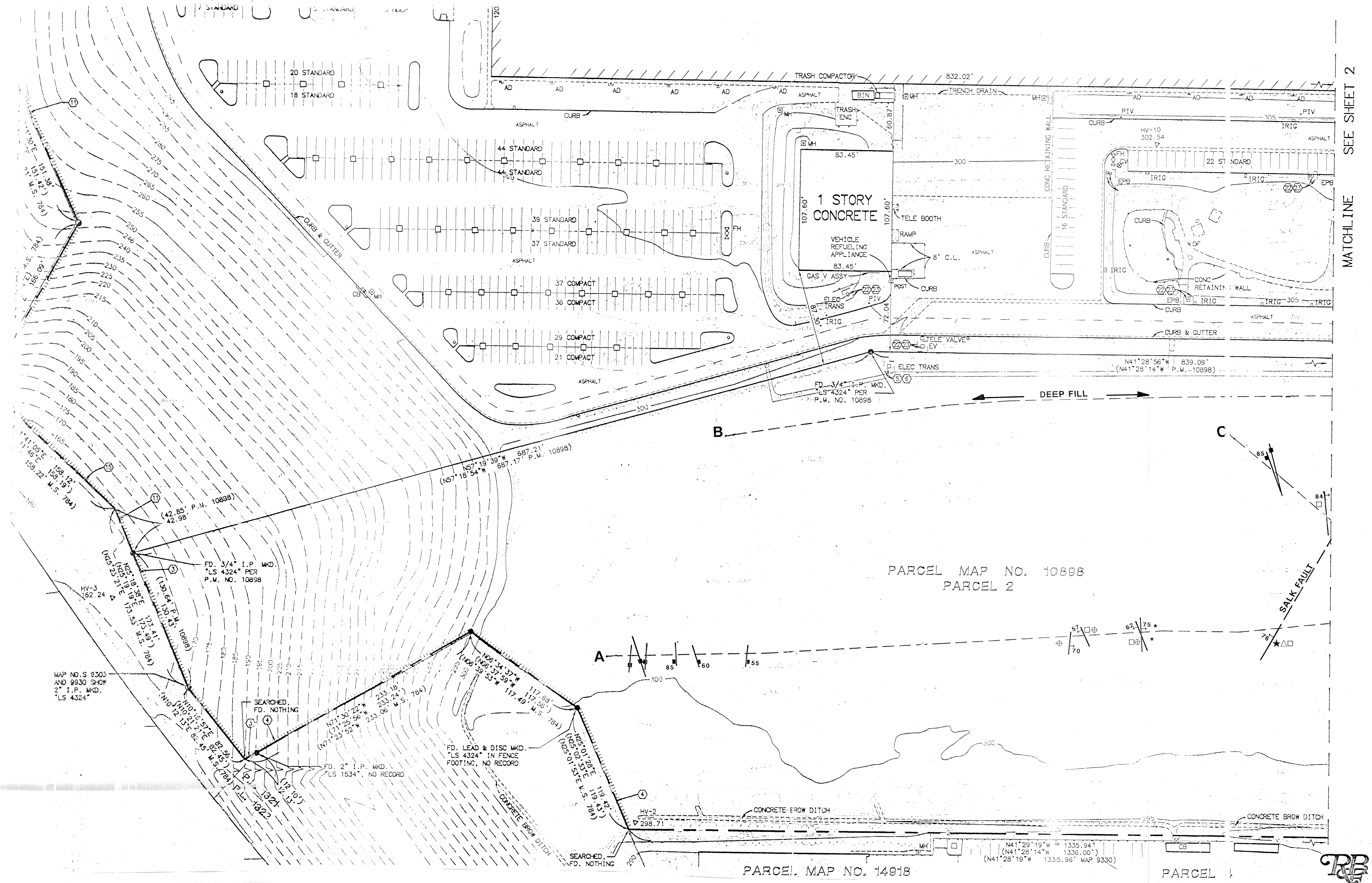
T_1 ....TERTIARY-AGE SCRIPPS FORMATION
Horizontally bedded, light gray and red brown banded,
Clayey SANDSTONE

〜 ....APPROX. LOCATION OF GEOLOGIC CONTACT

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

FAULT TRENCH FT-1
PROJECT NO. 07850 - 42 - 11
FIGURE 4
DATE 02 - 03 - 2014

Plotted:02/03/2014 3:00PM | By:ALVIN LADRILLONO | File Location:Y:\PROJECTS\07850-42-11 Campus Pointe Master Plan\SHEETS\07850-42-11 FaultTrench.dwg



JN: 501416
DRAWING: 1416ALT2

RBF/SHOLDERS & SANFORD
PROFESSIONAL ENGINEERS, PLANNERS & SURVEYORS
2669 FIFTH AVENUE • SAN DIEGO, CALIFORNIA 92103-5015
(619) 299-7272
PLOT: 10-19-95

SCS&T LEGEND

55	FRACTURES
55	FAULTS
★	JUMBLED FAULTS
△	FAULTS WITH CLAY GOUGE
☐	OBSERVED VERTICAL DISPLACEMENT OF BEDDING OR OTHER LINEAR FEATURES
⊕	ANNEALED FAULTS
*	ASSUMED FAULTS WHICH HAVE BECOME ANNEALED
+	FRACTURES OR POSSIBLE FAULTS WHICH HAVE BECOME ANNEALED

SOUTHERN CALIFORNIA
SOIL & TESTING, INC.

QUALCOMM

BY: CHC/JRH/SD	DATE: 11-16-95
JOB No.: 9511205	PLATE No.: 1A

DATA TABLE

NO.	BRNG/DELTA	RADIUS	LENGTH
C1	231°57'08" (231°58'55"	60.00' 60.00'	242.90' 242.93')
C2	140°35'42" (140°41'10"	60.00' 60.00'	147.23' 147.33' P.M. 10898)
C3	91°21'27" (91°17'45"	60.00' 60.00'	95.67' 95.60' P.M. 10898)
C4	00°40'10" (00°38'31"	682.00' 682.00'	7.97' 7.64' P.M. 10898)
C5	N50°09'46"E (N50°11'40"E	— —	28.44' 28.42')
C6	05°43'16" (05°41'17"	618.00' 618.00'	61.71' 61.35')

SCS&T LEGEND

- 55 FRACTURES
- 55 FAULTS
- ★ JUMBLED FAULTS
- △ FAULTS WITH CLAY GOUGE
- ☐ OBSERVED VERTICAL DISPLACEMENT OF BEDDING OR OTHER LINEAR FEATURES
- ⊕ ANNEALED FAULTS
- * ASSUMED FAULTS WHICH HAVE BECOME ANNEALED
- + FRACTURES OR POSSIBLE FAULTS WHICH HAVE BECOME ANNEALED
- ••••••• EASTERLY END OF FAULT ZONE

0 20 40 80
SCALE 1"=40'

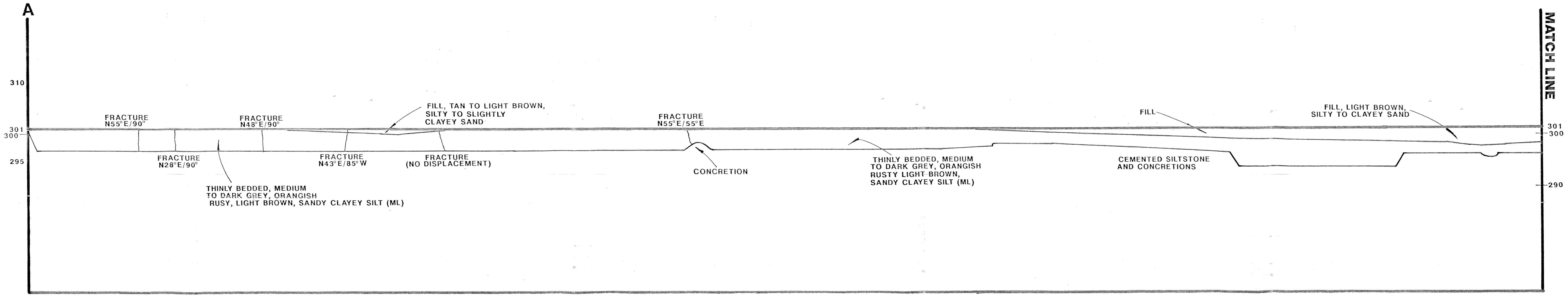
SOUTHERN CALIFORNIA
SOIL & TESTING, INC.

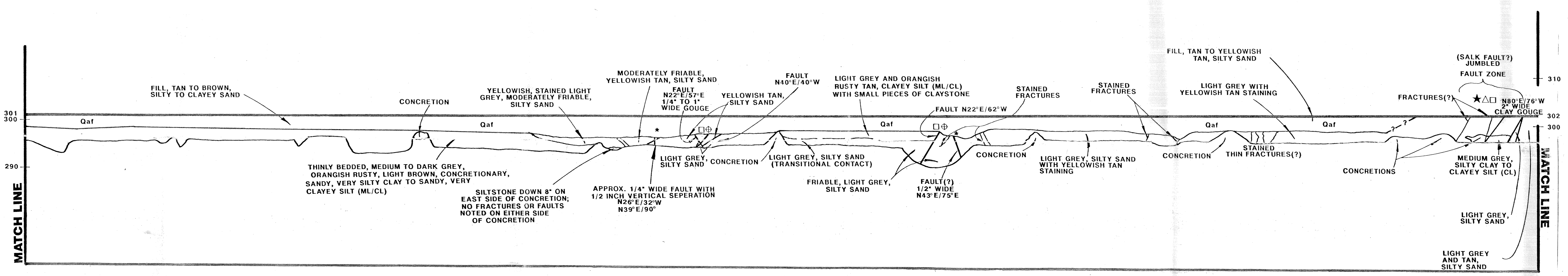
QUALCOMM

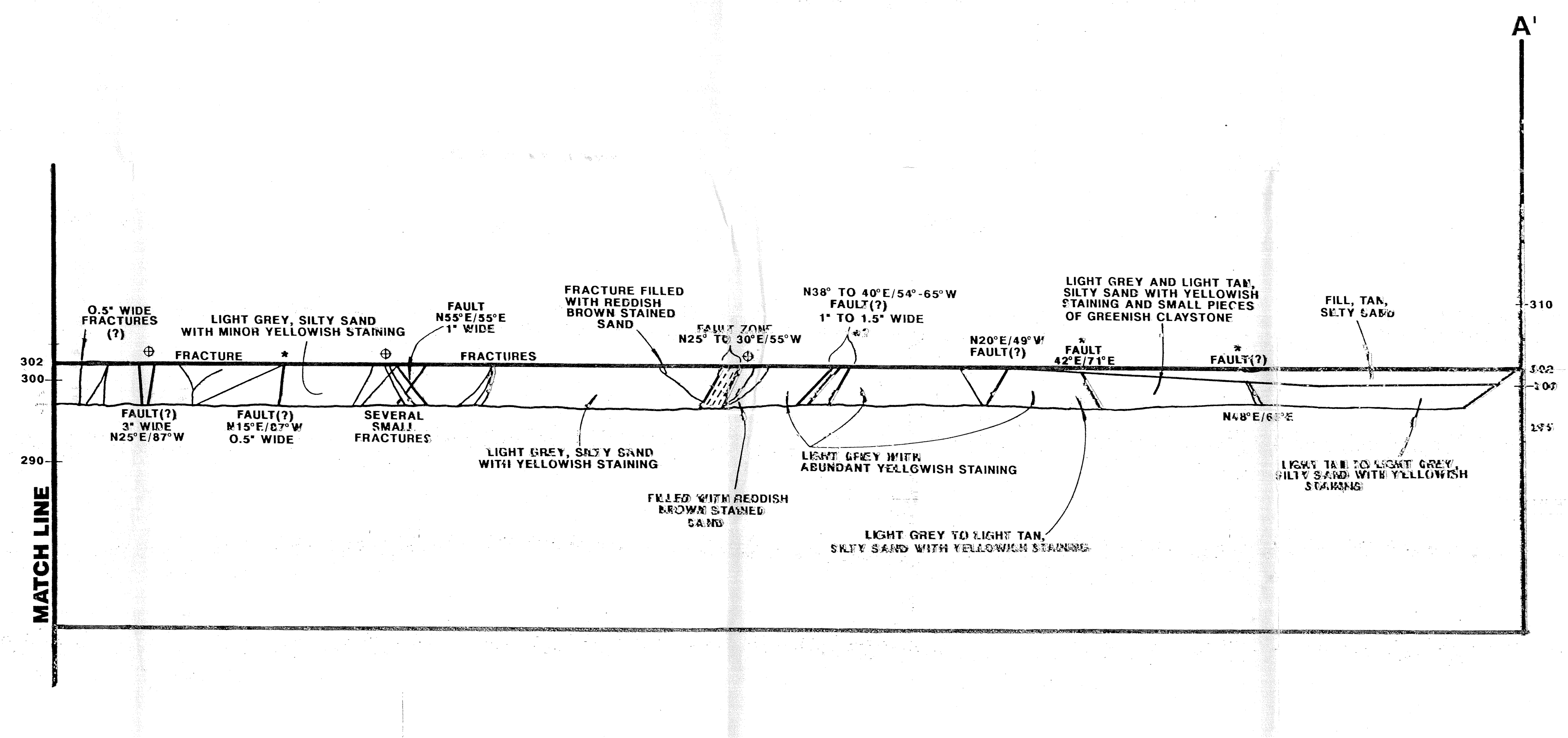
BY: CHC/JRH/SD DATE: 11-16-95
JOB No.: 9511205 PLATE No.: 1B

CROSS SECTION A - A'

SCALE: 1" = 10'

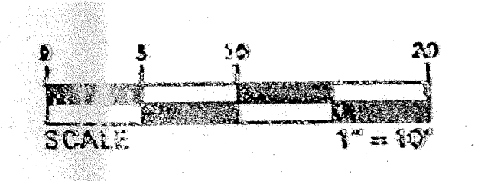






SCS&T LEGEND

- ★ JUMBLED FAULTS
- △ FAULTS WITH CLAY GOUGE
- □ OBSERVED VERTICAL DISPLACEMENT OF BEDDING OR OTHER LINEAR FEATURES
- ⊕ ANNEALED FAULTS
- * ASSUMED FAULTS WHICH HAVE BECOME ANNEALED
- \+ FRACTURES OR POSSIBLE FAULTS WHICH HAVE BECOME ANNEALED



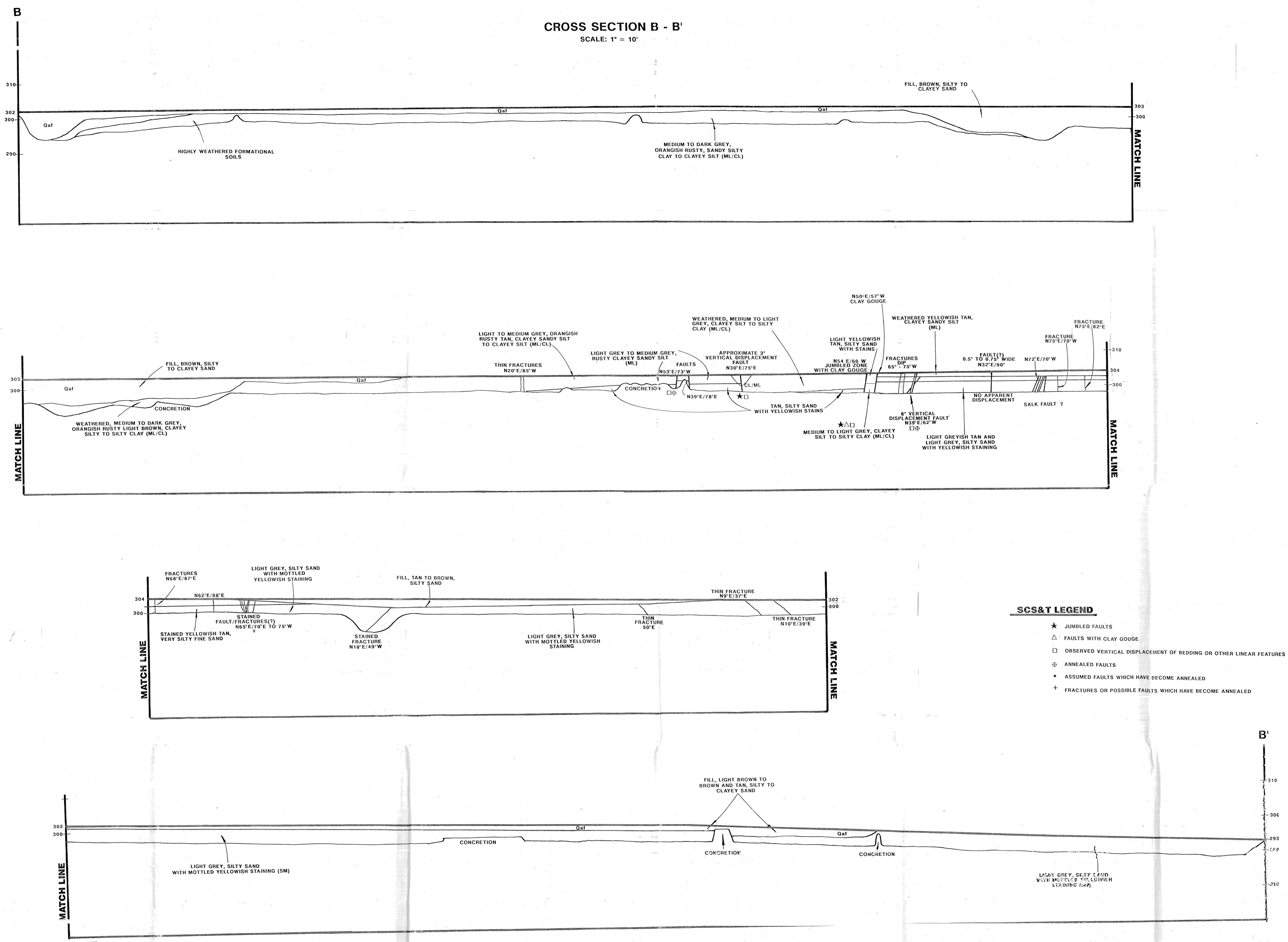
SOUTHERN CALIFORNIA
SOIL & TESTING, INC.

QUALCOMM

BY: CHC/JRH/SD	DATE: 11-16-95
JOB No.: 9511205-	PLATE No.: 2

CROSS SECTION B - B'

SCALE: 1" = 10'



SCS&T LEGEND

★ JUMBLED FAULTS

△ FAULTS WITH CLAY GOUGE

□ OBSERVED VERTICAL DISPLACEMENT OF BEDDING OR OTHER LINEAR FEATURES

⊕ ANNEALED FAULTS

* ASSUMED FAULTS WHICH HAVE BECOME ANNEALED

\+ FRACTURES OR POSSIBLE FAULTS WHICH HAVE BECOME ANNEALED

0 5 10 20
SCALE 1" = 10'

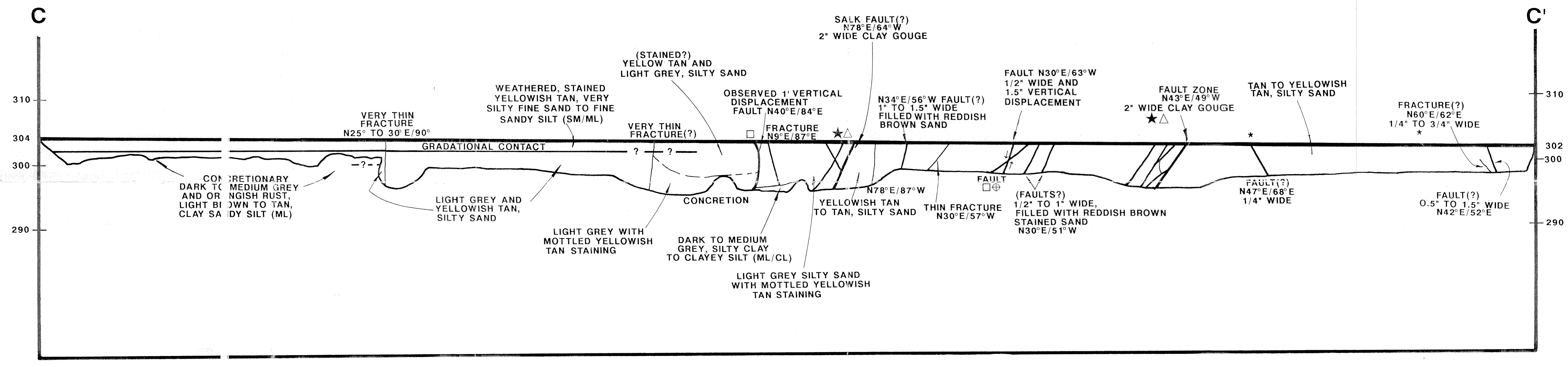
SOUTHERN CALIFORNIA
SOIL & TESTING, INC.

QUALCOMM

BY: CHC/JRH/SD	DATE: 11-16-95
JOB No.: 9511205-	PLATE No.: 3

CROSS SECTION C - C'

SCALE: 1" = 10'



SCS&T LEGEND

★ JUMBLED FAULTS

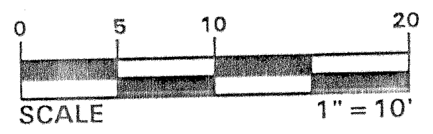
△ FAULTS WITH CLAY GOUGE

□ OBSERVED VERTICAL DISPLACEMENT OF BEDDING OR OTHER LINEAR FEATURES

⊕ ANNEALED FAULTS

* ASSUMED FAULTS WHICH HAVE BECOME ANNEALED

\+ FRACTURES OR POSSIBLE FAULTS WHICH HAVE BECOME ANNEALED



SOUTHERN CALIFORNIA
SOIL & TESTING, INC.

QUALCOMM

BY: CHC/JRH/SD	DATE: 11-16-95
JOB No.: 9511205-	PLATE No.: 4

APPENDIX D

APPENDIX D

STORM WATER MANAGEMENT INVESTIGATION

FOR

CAMPUS POINTE
SAN DIEGO, CALIFORNIA

PROJECT NO. G2415-52-01

APPENDIX D

STORM WATER MANAGEMENT INVESTIGATION

We understand storm water management devices are being proposed in accordance with the *2018 City of San Diego 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 D-I presents the descriptions of the hydrologic soil groups. If a soil is assigned to a dual hydrologic group (A/D, B/D, or C/D), the first letter is for drained areas and the second is for undrained areas. In addition, the USDA website also provides an estimated saturated hydraulic conductivity for the existing soil.

TABLE D-I
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 man-made previously placed fill and should be classified as Soil Group D. The Hydrologic Soil Group Map presents output from the USDA website showing the limits of the soil units.

Hydrologic Soil Group Map

Table D-II presents the information from the USDA website for the subject property.

TABLE D-II
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)
Altamont Clay, 30 to 50 percent Slopes, Warm MAAT, MLRA 20	AtF	73.4	C	0.06 – 0.57
Chesterton Fine Sandy Loam, 5 to 9 percent Slopes	CfC	26.6	D	0.00 – 0.06

* The property should be considered to possess a Hydrologic Soil Group D due to the existing fill materials.

In Situ Testing

We performed four constant-head infiltration tests at the locations shown on the Geologic Map, Figure 2. Table D-III presents the results of the infiltration tests. The field data sheets are attached herein. We applied a feasibility factor of safety of 2.0 to our estimated infiltration rates to provide input on Worksheet C.4-1. Soil infiltration rates from in-situ tests can vary significantly from one location to another due to the heterogeneous characteristics inherent to most soil.

TABLE D-III
INFILTRATION TEST RESULTS

Test No.	Geologic Unit	Test Elevation (feet, MSL)	Field-Saturated Hydraulic Conductivity/Infiltration Rate, k_{sat} (inch/hour)	Worksheet Infiltration Rate[1] (inch/hour)
P-1 (B-9)	Ta	297	0.003	0.002
P-2 (B-10)	Ta	298	0.015	0.008
P-3 (B-11)	Tsc	295	0.091	0.046
P-4 (B-12)	Tsc	298	0.071	0.036
Average			0.045	0.023

* Using a Factor of Safety of 2.

Infiltration categories include full infiltration, partial infiltration and no infiltration. Table D-IV presents the commonly accepted definitions of the potential infiltration categories based on the infiltration rates.

TABLE D-IV
INFILTRATION CATEGORIES

Infiltration Category	Field Infiltration Rate, I (Inches/Hour)	Factored Infiltration Rate*, I (Inches/Hour)
Full Infiltration	I > 1.0	I > 0.5
Partial Infiltration	0.10 < I ≤ 1.0	0.05 < I ≤ 0.5
No Infiltration (Infeasible)	I < 0.10	I < 0.05

* Using a Factor of Safety of 2.

Based on our observations and test results, the infiltration rates for the formational materials onsite (Scripps Formation and Ardath Shale) are less than 0.05 inches per hour. Therefore, full and partial infiltration on the property should be considered infeasible based on the calculated infiltrations rates. Vertical cutoff walls or liners should be installed on the sides and bottom of the infiltration basin and a drain should be installed at the base of the basin.

GEOTECHNICAL CONSIDERATIONS

Groundwater Elevations

We did not encounter groundwater or seepage during our site investigation. We expect groundwater is deeper than about 200 feet below existing grade.

New or Existing Utilities

Utilities are located on and adjacent to the property within the existing parking area and roadways. Therefore, full and partial infiltration within the areas near these utilities should be considered infeasible. Setbacks for infiltration should be incorporated. The setback for infiltration devices should be a minimum of 10 feet and a 1:1 plane of 1 foot below the closest edge of the deepest adjacent utility.

Existing or Planned Structures

Structures are present along the northern, eastern and southern boundaries of the property, and several structures are proposed on-site as described herein. Water should not be allowed to infiltrate in areas where it could affect the neighboring properties and adjacent structures. Mitigation for existing structures consists of not allowing water infiltration within 10 feet of the existing foundations.

Slopes

A descending slope with a height of approximately 150 feet exists on the western portion of the property. Infiltration should not be allowed within a distance of 50 feet or a distance of 1.5H from a slope where H is the height of the slope (about 225 feet from the top of the existing slope).

Soil or Groundwater Contamination

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

CONCLUSIONS AND RECOMMENDATIONS

Storm Water Evaluation Narrative

The majority of the site is underlain by varying depths of fill overlying the Scripps Formation and Ardath Shale (see Geologic Map, Figure 2). Infiltration is not allowed in areas with 5 feet and thicker of fill. Descending slopes exist west of the property along Campus Point Drive with a height up to approximately 150 feet. Infiltration should not be allowed within 50 feet or 1.5 times the height of existing slopes (225 feet).

We performed two infiltration tests within the Scripps Formation and two within the Ardath Shale in the northeastern portion of the site where formational materials are present near existing and proposed grade. We located our infiltration tests within the area of the site with adequate setbacks from slopes and fills of less than 5 feet. The results indicate an average rate of less than 0.05 inches per hour (with an applied factor of safety of 2).

Storm Water Evaluation Conclusion

Infiltration should be considered infeasible within the existing fill soils on the southern and western portions of the property. Full and partial infiltration should be considered infeasible at the site because the average infiltration rate is less than 0.05 inches per hour within formational materials. Mitigation measures do not exist that allow an increase to the infiltration rates.

Storm Water Management Devices

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

Storm Water Standard Worksheets

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

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 D-IV describes the suitability assessment input parameters related to the geotechnical engineering aspects for the factor of safety determination.

TABLE D-V
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 previous table, Table D-V presents the estimated factor values for the evaluation of the factor of safety. This table only presents the suitability assessment safety factor (Part A) of the worksheet. The project civil engineer should evaluate the safety factor for design (Part B) and use the combined safety factor for the design infiltration rate.

TABLE D-VI
FACTOR OF SAFETY WORKSHEET DESIGN VALUES – PART A1

Suitability Assessment Factor Category	Assigned Weight (w)	Factor Value (v)	Product (p = w x v)
Assessment Methods	0.25	2	0.50
Predominant Soil Texture	0.25	3	0.75
Site Soil Variability	0.25	2	0.50
Depth to Groundwater/ Impervious Layer	0.25	1	0.25
Suitability Assessment Safety Factor, $S_A = \sum p$			2.00

* 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 based on Geotechnical Conditions	**Worksheet C.4-1: Form I-8A[10]**

Part 1 - Full Infiltration Feasibility Screening Criteria

DMA(s) Being Analyzed:	**Project Phase:**
Campus Pointe	Design

Criteria 1: Infiltration Rate Screening

1A	Is the mapped hydrologic soil group according to the NRCS Web Soil Survey or UC Davis Soil Web Mapper Type A or B and corroborated by available site soil data[11]? ☐ Yes; the DMA may feasibly support full infiltration. Answer "Yes" to Criteria 1 Result or continue to Step 1B if the applicant elects to perform infiltration testing. ☐ No; the mapped soil types are A or B but is not corroborated by available site soil data (continue to Step 1B). ☐ No; the mapped soil types are C, D, or "urban/unclassified" and is corroborated by available site soil data. Answer "No" to Criteria 1 Result. ☒ No; the mapped soil types are C, D, or "urban/unclassified" but is not corroborated by available site soil data (continue to Step 1B).
1B	Is the reliable infiltration rate calculated using planning phase methods from Table D.3-1? ☒ Yes; Continue to Step 1C. ☐ No; Skip to Step 1D.
1C	Is the reliable infiltration rate calculated using planning phase methods from Table D.3-1 greater than 0.5 inches per hour? ☐ Yes; the DMA may feasibly support full infiltration. Answer "Yes" to Criteria 1 Result. ☒ No; full infiltration is not required. Answer "No" to Criteria 1 Result.
1D	**Infiltration Testing Method.** Is the selected infiltration testing method suitable during the design phase (see Appendix D.3)? Note: Alternative testing standards may be allowed with appropriate rationales and documentation. ☐ Yes; continue to Step 1E. ☐ No; select an appropriate infiltration testing method.

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

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

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

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

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

The majority of the site is underlain by varying depths of fill overlying the Scripps Formation and Ardath Shale (see Geologic Map, Figure 2). Infiltration is not allowed in areas with 5 feet and thicker of fill. Descending slopes exist west of the property along Campus Point Drive with a height up to approximately 150 feet. Infiltration should not be allowed within 50 feet or 1.5 times the height of existing slopes (225 feet).

We performed two infiltration tests within the Scripps Formation and two within the Ardath Shale in the northeastern portion of the site. The results indicate an average rate of less than 0.05 inches per hour (with an applied factor of safety of 2). Therefore, infiltration is considered infeasible within the formational Scripps Formation and infeasible at the site.

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

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

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

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

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

Categorization of Infiltration Feasibility Condition based on Geotechnical Conditions	Worksheet C.4-1: Form I- 8A[10]

Part 2 – Partial vs. No Infiltration Feasibility Screening Criteria

DMA(s) Being Analyzed:	**Project Phase:**
Campus Pointe	Design

Criteria 3: Infiltration Rate Screening

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

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

The majority of the site is underlain by varying depths of fill overlying the Scripps Formation and Ardath Shale (see Geologic Map, Figure 2). Infiltration is not allowed in areas with 5 feet and thicker of fill. Descending slopes exist west of the property along Campus Point Drive with a height up to approximately 150 feet. Infiltration should not be allowed within 50 feet or 1.5 times the height of existing slopes (225 feet).

We performed two infiltration tests within the Scripps Formation and two within the Ardath Shale in the northeastern portion of the site. The results indicate an average rate of less than 0.05 inches per hour (with an applied factor of safety of 2). Therefore, infiltration is considered infeasible within the formational Scripps Formation and infeasible at the site.

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

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

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

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

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

GEOCON

Aardvark Permeameter Data Analysis

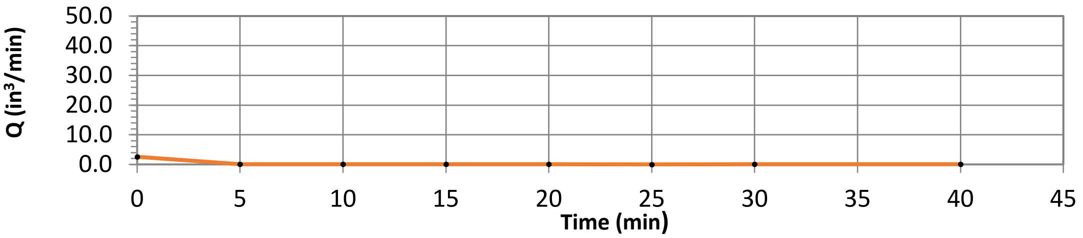
Project Name: Campus Pointe
Project Number: G2415-52-01
Test Number: P-1
Date: 7/17/2019
By: MRL
Ref. EL (feet, MSL): 302.0
Bottom EL (feet, MSL): 297.0

Borehole Diameter, d (in.): 8.00
Borehole Depth, **H** (in): 60.00
Distance Between Reservoir & Top of Borehole (in.): 29.00
Estimated Depth to Water Table, **S** (feet): 200.00
Height APM Raised from Bottom (in.): 2.00
Pressure Reducer Used: No

Distance Between Resevoir and APM Float, **D** (in.): 79.75
Head Height Calculated, **h** (in.): 5.77
Head Height Measured, **h** (in.): 5.00
Distance Between Constant Head and Water Table, **L** (in.): 2345.00

Reading	**Time Elapsed (min)**	**Water Weight Consummed (lbs)**	**Water Volume Consummed (in^3)**	**Q (in^3/min)**
1	0.00	0.000	0.00	0.00
2	5.00	0.465	12.88	2.575
3	5.00	0.005	0.14	0.028
4	5.00	0.010	0.28	0.055
5	5.00	0.010	0.28	0.055
6	5.00	0.005	0.14	0.028
7	5.00	0.000	0.00	0.000
8	10.00	0.010	0.28	0.028
9	5.00	0.005	0.14	0.028

Steady Flow Rate, Q (in^3/min): 0.018



Soil Matric Flux Potential, Φ_m

Φ_m= 0.0001 in^2/min

Field-Saturated Hydraulic Conductivity (Infiltration Rate)

***K_{sat}* =** 5.39E-05 in/min 0.003 in/hr

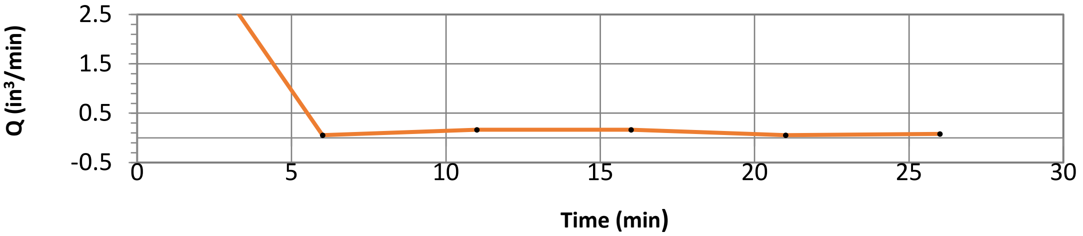
Aardvark Permeameter Data Analysis

Project Name: Campus Pointe
Date: 7/17/2019
Project Number: G2415-52-01
By: MRL
Test Number: P-2
Ref. EL (feet, MSL): 301.0
Bottom EL (feet, MSL): 296.7

Borehole Diameter, d (in.): 8.00
Borehole Depth, **H** (in): 52.00
Distance Between Reservoir & Top of Borehole (in.): 29.00
Estimated Depth to Water Table, **S** (feet): 200.00
Height APM Raised from Bottom (in.): 1.00
Pressure Reducer Used: No

Distance Between Resevoir and APM Float, **D** (in.): 72.75
Head Height Calculated, **h** (in.): 4.74
Head Height Measured, **h** (in.): 4.00
Distance Between Constant Head and Water Table, **L** (in.): 2352.00

Reading	**Time Elapsed (min)**	**Water Weight Consummed (lbs)**	**Water Volume Consummed (in^3)**	**Q (in^3/min)**
1	0.00	0.000	0.00	0.00
2	6.00	0.800	22.15	3.692
3	5.00	0.990	27.42	5.483
4	5.00	0.010	0.28	0.055
5	5.00	0.030	0.83	0.166
6	5.00	0.030	0.83	0.166
7	5.00	0.010	0.28	0.055
8	5.00	0.015	0.42	0.083
			Steady Flow Rate, Q (in^3/min):	0.069



Soil Matric Flux Potential, Φ_m

Φ_m= 0.0003 in^2/min

Field-Saturated Hydraulic Conductivity (Infiltration Rate)

K_{sat} = 2.48E-04 in/min 0.015 in/hr

GEOCON

Aardvark Permeameter Data Analysis

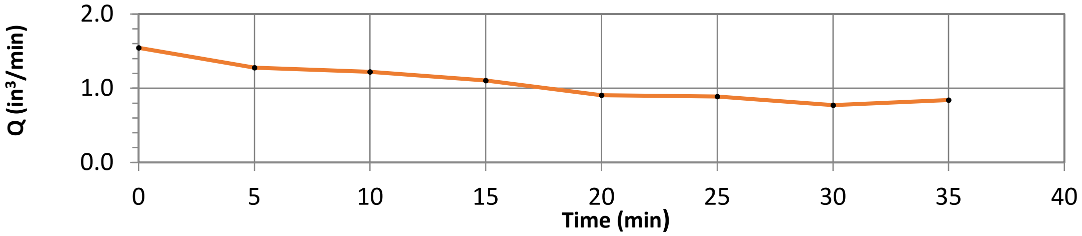
Project Name: Campus Pointe
Project Number: G2415-52-01
Test Number: P-3

Date: 7/17/2019
By: MRL
Ref. EL (feet, MSL): 300.0
Bottom EL (feet, MSL): 294.7

Borehole Diameter, d (in.): 8.00
Borehole Depth, **H** (in): 64.00
Distance Between Reservoir & Top of Borehole (in.): 27.00
Estimated Depth to Water Table, **S** (feet): 200.00
Height APM Raised from Bottom (in.): 4.00
Pressure Reducer Used: No

Distance Between Resevoir and APM Float, **D** (in.): 79.75
Head Height Calculated, **h** (in.): 7.77
Head Height Measured, **h** (in.): 8.00
Distance Between Constant Head and Water Table, **L** (in.): 2344.00

Reading	Time Elapsed (min)	Water Weight Consummed (lbs)	Water Volume Consummed (in^3)	Q (in^3/min)
1	0.00	0.000	0.00	0.00
2	5.00	0.108	2.99	0.598
3	5.00	0.279	7.73	1.545
4	5.00	0.231	6.40	1.279
5	5.00	0.220	6.11	1.221
6	5.00	0.200	5.52	1.105
7	5.00	0.164	4.53	0.906
8	5.00	0.161	4.44	0.889
9	5.00	0.140	3.86	0.773
10	5.00	0.152	4.20	0.839
			Steady Flow Rate, Q (in^3/min):	0.834



Soil Matric Flux Potential, Φ_m

Φ_m= 0.0017 in^2/min

Field-Saturated Hydraulic Conductivity (Infiltration Rate)

K_{sat} = 1.51E-03 in/min 0.091 in/hr

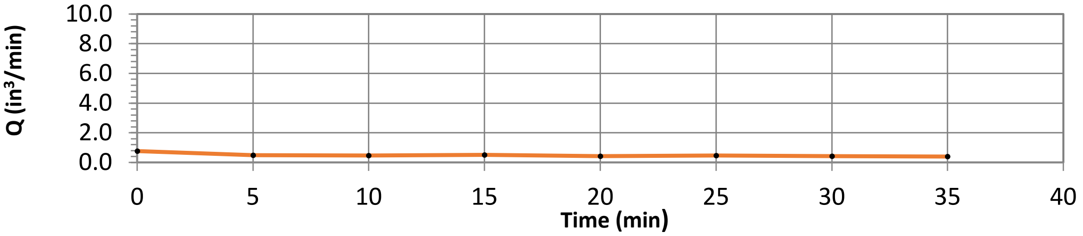
Aardvark Permeameter Data Analysis

Project Name: Campus Pointe
Project Number: G2415-52-01
Test Number: P-4
Date: 7/17/2019
By: MRL
Ref. EL (feet, MSL): 300.0
Bottom EL (feet, MSL): 298.0

Borehole Diameter, d (in.): 8.00
Borehole Depth, **H** (in): 24.00
Distance Between Reservoir & Top of Borehole (in.): 28.00
Estimated Depth to Water Table, **S** (feet): 200.00
Height APM Raised from Bottom (in.): 2.00
Pressure Reducer Used: No

Distance Between Resevoir and APM Float, **D** (in.): 42.75
Head Height Calculated, **h** (in.): 5.64
Head Height Measured, **h** (in.): 5.00
Distance Between Constant Head and Water Table, **L** (in.): 2381.00

Reading	Time Elapsed (min)	Water Weight Consummed (lbs)	Water Volume Consummed (in^3)	Q (in^3/min)
1	0.00	0.000	0.00	0.00
2	5.00	0.138	3.82	0.764
3	5.00	0.089	2.48	0.495
4	5.00	0.083	2.29	0.459
5	5.00	0.092	2.54	0.508
6	5.00	0.077	2.14	0.429
7	5.00	0.083	2.31	0.462
8	5.00	0.075	2.08	0.415
9	5.00	0.071	1.98	0.395
			Steady Flow Rate, Q (in^3/min):	0.405



Soil Matric Flux Potential, Φ_m

Φ_m= 0.0013 in^2/min

Field-Saturated Hydraulic Conductivity (Infiltration Rate)

K_{sat} = 1.18E-03 in/min 0.071 in/hr

APPENDIX E

APPENDIX E

SLOPE STABILITY ANALYSIS

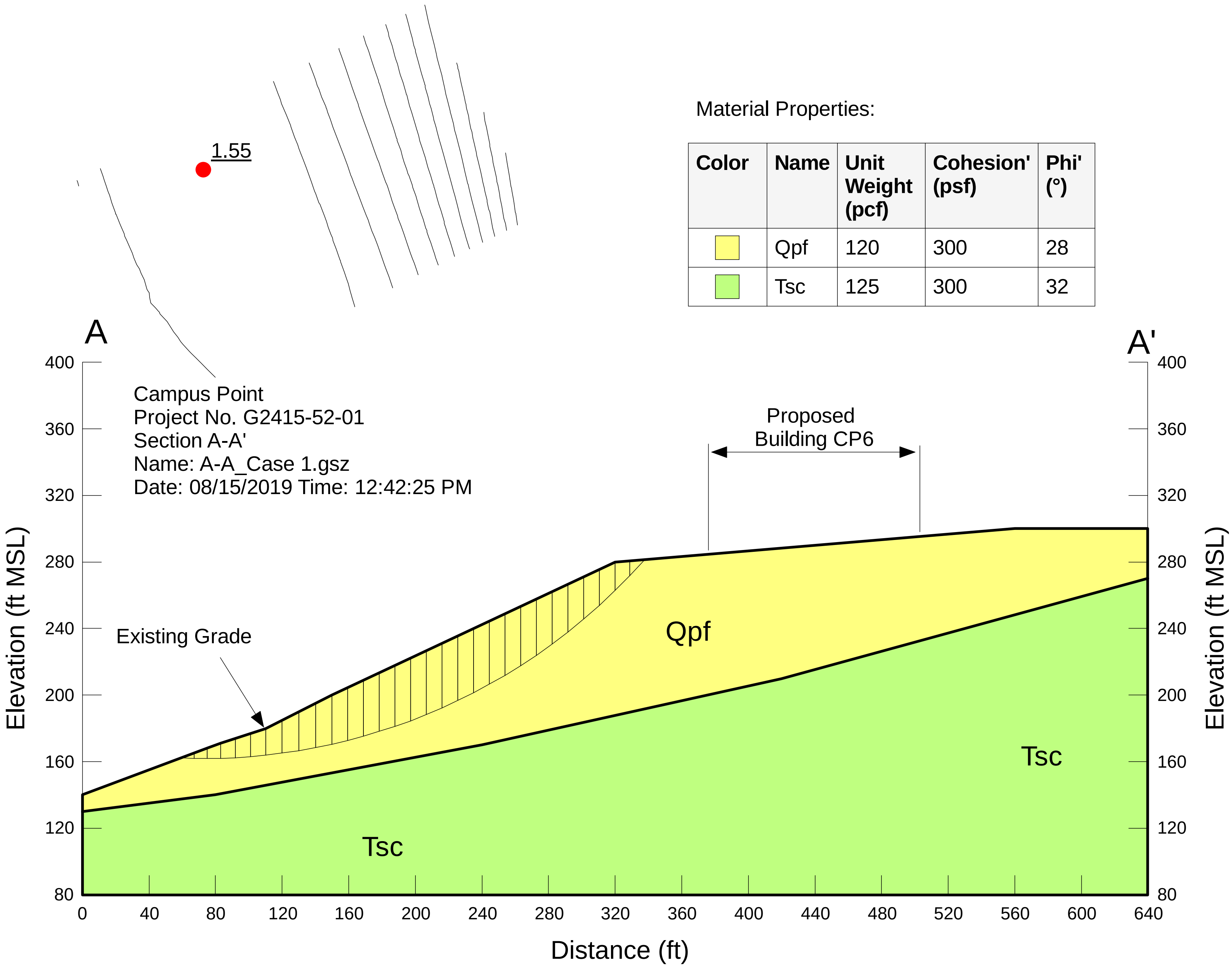
FOR

CAMPUS POINTE
SAN DIEGO, CALIFORNIA

PROJECT NO. G2415-52-01

Material Properties:

Color	Name	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)
🟨	Qpf	120	300	28
🟩	Tsc	125	300	32



Campus Point
Project No. G2415-52-01
Section A-A'
Name: A-A_Case 1.gsz
Date: 08/15/2019 Time: 12:42:25 PM

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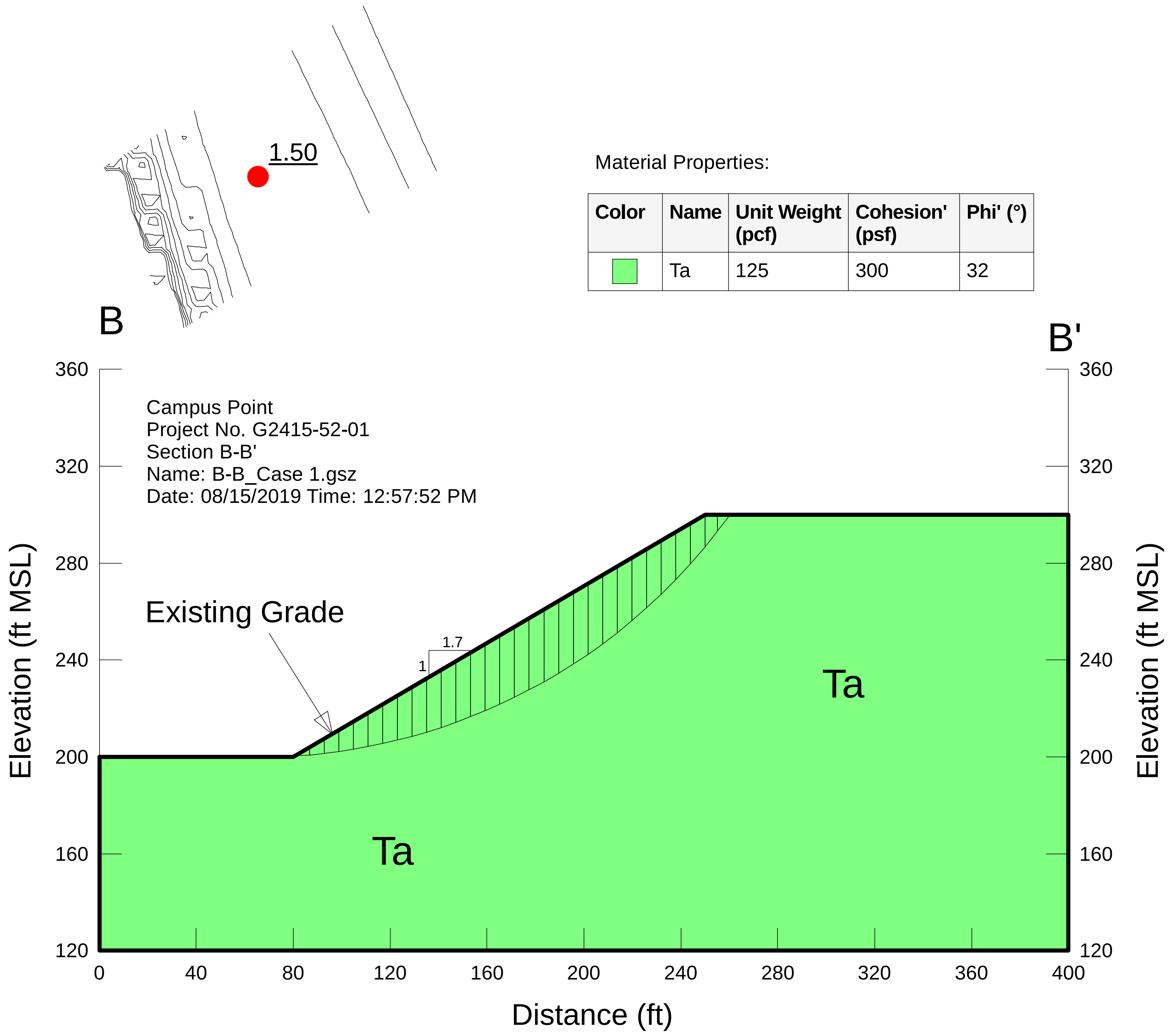

Material Properties:

Color	**Name**	**Unit Weight (pcf)**	**Cohesion' (psf)**	**Phi' (°)**
🟩	Ta	125	300	32

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APPENDIX F

APPENDIX F

RECOMMENDED GRADING SPECIFICATIONS

FOR

CAMPUS POINTE
SAN DIEGO, CALIFORNIA

PROJECT NO. G2415-52-01

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

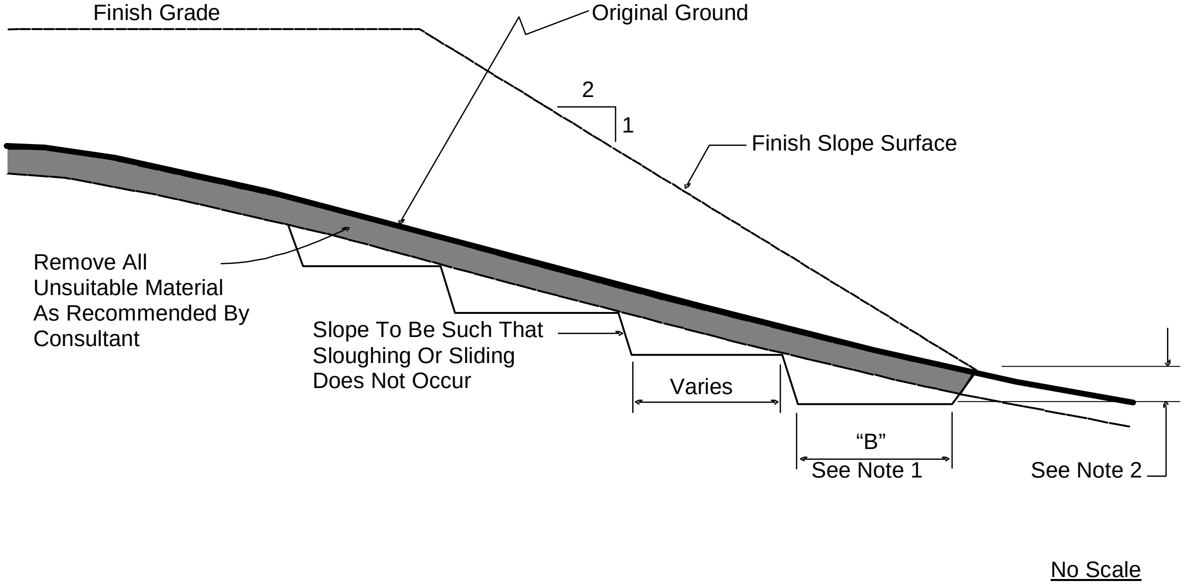
4.1 Areas to be excavated and filled shall be cleared and grubbed. Clearing shall consist of complete removal above the ground surface of trees, stumps, brush, vegetation, man-made structures, and similar debris. Grubbing shall consist of removal of stumps, roots, buried logs and other unsuitable material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

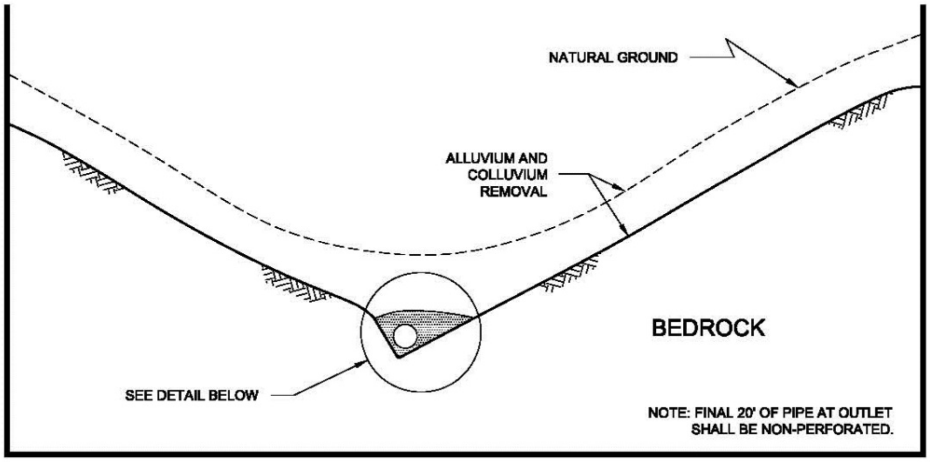
6.3.6 To reduce the potential for "piping" of fines into the *rock* fill from overlying *soil* fill material, a 2-foot layer of graded filter material shall be placed above the uppermost lift of *rock* fill. The need to place graded filter material below the *rock* should be determined by the Consultant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

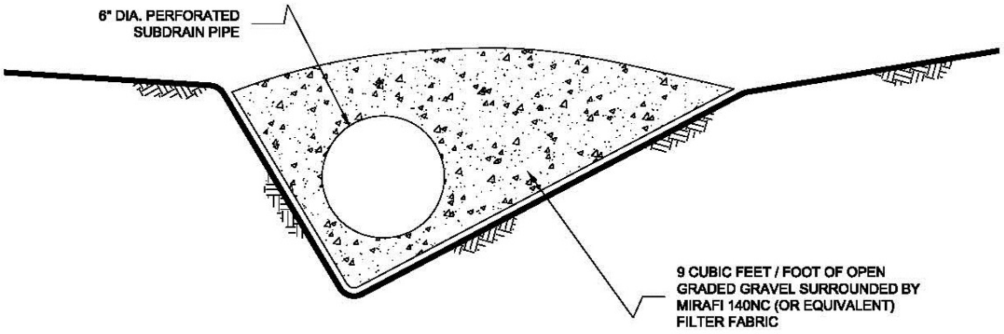
7. SUBDRAINS

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



NOTE: FINAL 20' OF PIPE AT OUTLET SHALL BE NON-PERFORATED.



NOTES:

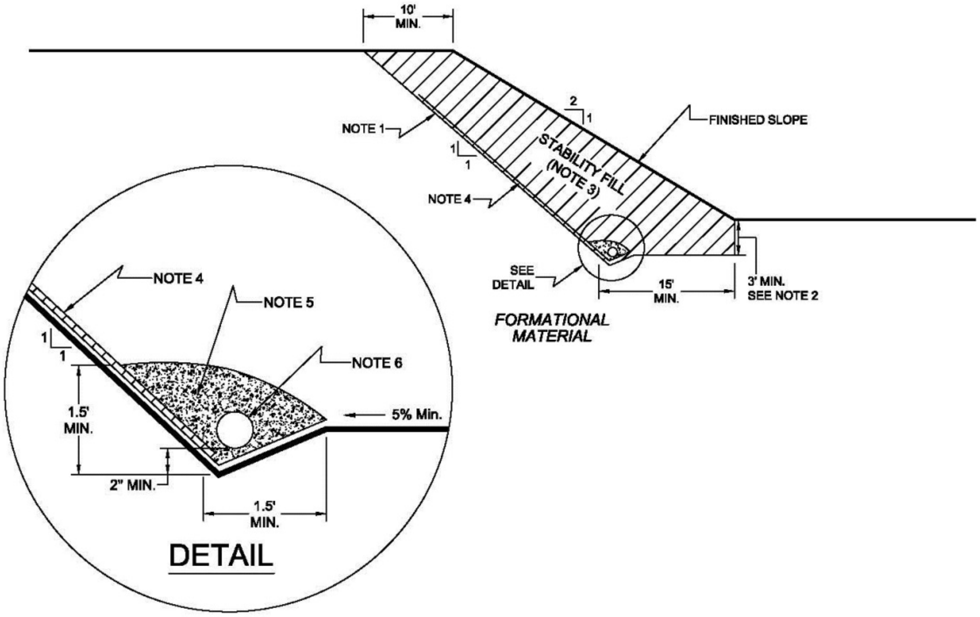
1......6-INCH DIAMETER, SCHEDULE 80 PVC PERFORATED PIPE FOR FILLS IN EXCESS OF 100-FEET IN DEPTH OR A PIPE LENGTH OF LONGER THAN 500 FEET.

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

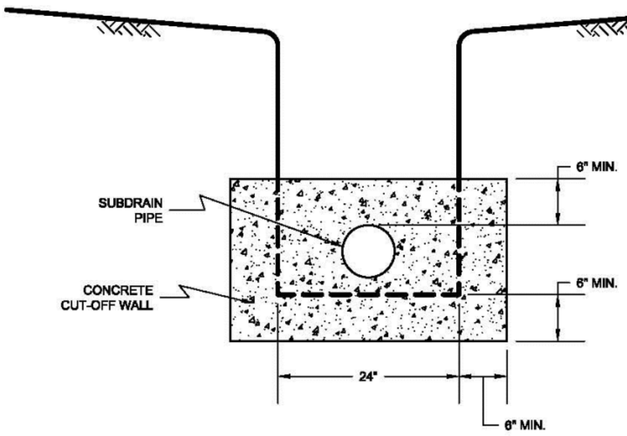
7.3 The actual subdrain locations will be evaluated in the field during the remedial grading operations. Additional drains may be necessary depending on the conditions observed and the requirements of the local regulatory agencies. Appropriate subdrain outlets should be evaluated prior to finalizing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

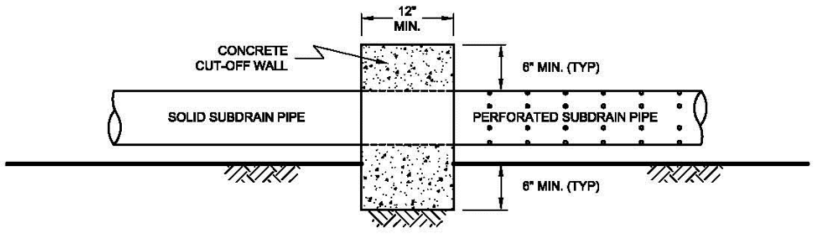
TYPICAL CUT OFF WALL DETAIL

FRONT VIEW



NO SCALE

SIDE VIEW

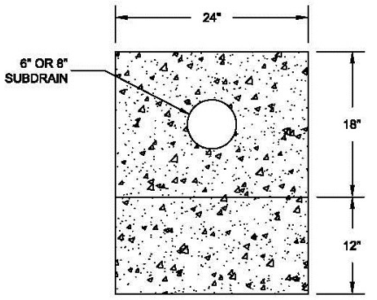


NO SCALE

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

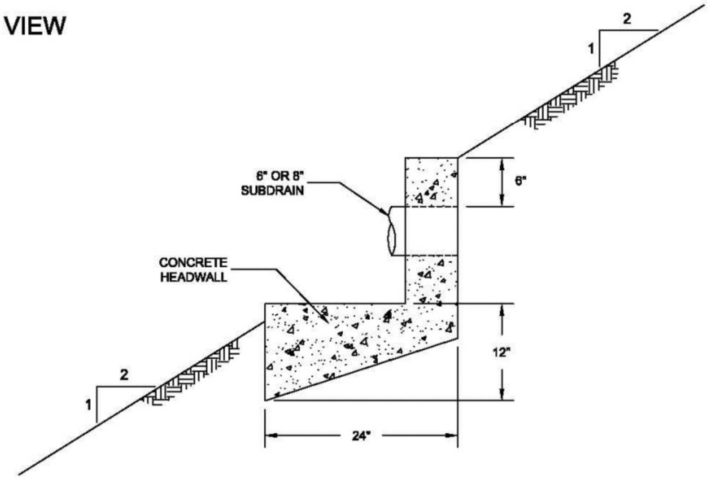
TYPICAL HEADWALL DETAIL

FRONT VIEW



NO SCALE

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. *2016 California Building Code, California Code of Regulations, Title 24, Part 2, based on the 2015 International Building Code,* prepared by California Building Standards Commission, dated July, 2016.

2. American Concrete Institute, *ACI 318-11, Building Code Requirements for Structural Concrete and Commentary,* dated August, 2011.

3. American Concrete Institute, *ACI 330-08, Guide for the Design and Construction of Concrete Parking Lots,* dated June, 2008.

4. American Society of Civil Engineers (ASCE), *ASCE 7-10, Minimum Design Loads for Buildings and Other Structures*, Second Printing, April 6, 2011.

5. Boore, D. M., and G. M Atkinson (2006), *Ground Motion Prediction Equations for the Average Horizontal Component of PGA, PVG, and 5%-Ramped PSA at Spectral Periods Between 0.01s and 10.0s,* Earthquake Spectra, Vol. 24, Issue I, February 2008.

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

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