APPENDIX F-1

Preliminary Geotechnical Investigation

PRELIMINARY GEOTECHNICAL INVESTIGATION

10290 CAMPUS POINTE DRIVE
SAN DIEGO, CALIFORNIA

GEOCON
INCORPORATED

GEOTECHNICAL
ENVIRONMENTAL
MATERIALS

PREPARED FOR

ALEXANDRIA REAL ESTATE EQUITIES, INC.
SAN DIEGO, CALIFORNIA

JUNE 11, 2015
PROJECT NO. 07850-42-15

Project No. 07850-42-15
June 11, 2015

Alexandria Real Estate Equities, Inc.
10996 Torreyana Road, Suite 250
San Diego, California 92122

Attention: Mr. Michael Barbera

Subject: PRELIMINARY GEOTECHNICAL INVESTIGATION
10290 CAMPUS POINTE DRIVE
SAN DIEGO, CALIFORNIA

Dear Mr. Barbera:

In accordance with your request, we have prepared this preliminary geotechnical investigation for the subject property. This study was prepared for the purpose of identifying site soil and geologic conditions and potential geotechnical constraints that could impact development, as well as to provide preliminary geotechnical recommendations for design and construction.

The accompanying report presents findings from our study relative to geotechnical engineering aspects of developing the site. No soil or geologic conditions were encountered which would preclude development.

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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(4/del) Addressee

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PRELIMINARY GEOTECHNICAL INVESTIGATION

1. PURPOSE AND SCOPE

This report presents the findings of our preliminary geotechnical investigation for the proposed development located at 10290 Campus Point Drive, in San Diego, California (see Vicinity Map, Figure 1). The purpose of this investigation was to observe site soil and geologic conditions, identify potential geotechnical constraints, to provide recommendations pertaining to geotechnical aspects of developing the property.

The scope of our study consisted of reviewing previous geotechnical reports that have been prepared for the site and adjacent projects, analyses of the data obtained from the previous investigations and fault studies, exploratory borings, and preparation of this report. Previous reports and maps reviewed for this study include the following:

1. Geocon Incorporated, (2015), *Preliminary Fault Study, 10290 Campus Point Drive, San Diego, California*, (Project No. 07850-42-15);
2. Geocon Incorporated, (2014), *Geotechnical and Geologic Fault Investigation, Campus Pointe Master Plan, 10300 Campus Point Drive, San Diego, California*, (Project No. 07850-42-11);
3. Geocon Incorporated, (2011), *Due Diligence Review of Geotechnical Reports, Qualcomm Building A, 10290 Campus Point Drive, San Diego, California*, (Project No. 07850-42-05);
4. Southern California Soil & Testing, Inc, (1995a), *Report of Preliminary Geotechnical Investigation, Qualcomm Office Building, Eli Lillie Property, Campus Point Drive, San Diego, California*, (SCS&T 9511205).
5. Southern California Soil & Testing, Inc, (1995b), *Report of Fault Investigation, Qualcomm Office Building, Eli Lillie Property, Campus Point Drive, San Diego, California*, (SCS&T 9511205).
6. California Geological Survey, (2008), *Geologic Map of the San Diego 30' x 60' Quadrangle, California*, Regional Geologic Map No. 3;
7. City of San Diego Development Services Department, (2008), *City of San Diego, Seismic Safety Study, Geologic Hazards and Faults*, Grid Tile: 34.

Other reports reviewed as part of this study are summarized on the *List of References* at the end of this report.

Details of the field investigation performed by Geocon Incorporated and boring logs are presented in Appendix A. A summary of laboratory tests performed on selected soil samples obtained during the field investigation are presented in Appendix B. Fault trench logs performed under References 1 and

4 are provided in Appendix C. Boring logs and laboratory test results performed previously by Geocon and others on the property are provided in Appendix D. The approximate locations of the borings and fault trenches are provided on Figure 2. The base map used to depict site conditions, boring and fault trenches, and site geology was taken from an AutoCAD file of the proposed site plan.

2. SITE AND PROJECT DESCRIPTION

The subject site occupies approximately 16.5 acres located at 10290 Campus Point Drive in San Diego, California. The property has been developed into a four-story office building and ancillary parking lots. Nearby development consists of office buildings and parking lots. The property is generally flat with drainage to the southwest.

Based on information contained in SCS&T (1995a), we expect the existing building is supported on shallow, conventional foundations for the portion of the structure founded on formational soils and drilled piers for the portions overlying previously placed fill

We understand that the proposed project consists of the construction of a new multi-story (1,200 car) parking structure with one to two stories of subterranean parking and a multi-story office building in the existing parking lot areas west of the existing office building. Additional improvements will include a soccer field with bleachers, ball courts, new parking areas, and improvements to existing surface improvements. A new 5-story entry addition is also planned for the existing building.

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

3. SOIL AND GEOLOGIC CONDITIONS

We encountered previously placed fill and the Scripps and Ardath formations during our field investigation. The occurrence and distribution of the units are presented on the boring logs in Appendix A and the approximate lateral extent of the units is shown on the Geologic Map, Figure 2 and Geologic Cross Sections, Figures 3 and 4. The previously placed fill and Scripps and Ardath formations are described below.

3.1 Previously Placed Fill (Qpf)

Based on our field investigation and previous fault trenches performed on the property, we expect previously placed fill ranging from less than 5 feet to greater than 20 feet exists within portions of the property. The deepest fills are located at the north and southeast ends of the site. The fills daylight within the south and central portions of the property. Based on our review of previous reports, the fill was placed during mass grading in 1979 to 1980 under the observation and compaction testing of Woodward-Clyde Consultants (WCC). Compaction reports documenting the fill could not be obtained.

Based on information obtained during our field investigation, the previously placed fill consists of medium dense silty sand and stiff sandy silt and clay. Laboratory consolidation tests indicate the fill has a low to moderate potential for loading induced compression. The fill is also expected to have a low to medium expansion potential.

We expect fill within the parking structure building pad will be removed to achieve below grade parking levels. With respect to the office building, because of the cut to fill transition within the building pad, we recommend the portion of the building pad underlain by fill be supported on deepened conventional foundations and drilled piers. The portion of the building pad underlain by formational soils can be founded on conventional shallow foundations.

3.2 Scripps Formation (Tsc)

The Scripps Formation was encountered within the eastern portion of the site during our study and previous field studies. This unit consists predominantly medium-grained, yellowish brown sandstone containing cobble-conglomerate beds (Kennedy and Tan, 2008). The Scripps Formation also typically contains localized areas of highly cemented concretionary beds. The Scripps Formation is expected to have a low to medium expansion potential. The Scripps Formation is suitable for support of the planned improvements. The basal contact of the Scripps Formation is conformable with the Tertiary-age Ardath Formation.

3.3 Ardath Formation (Ta)

The Tertiary-age Ardath Formation underlies the western portion of the site. The Ardath Formation consists an olive-gray and yellowish brown silty shale. The upper portion may contain thin beds of medium-grained sandstone similar to the overlying Scripps Formation (Kennedy and Tan, 2008). The Ardath Formation may contain localized areas of highly cemented concretionary beds. The Ardath Formation is expected to have a low to medium expansion potential and is suitable for support of structural loading in its existing condition.

4. GROUNDWATER

We did not observe groundwater during our field investigation. We do not anticipate that groundwater will be an issue during development of the property given the nature of the site geology, topography and our experience on the property. It is not uncommon for saturated conditions to develop where none existed previously, especially perched groundwater at the contact between fill and formational units.

5. GENERAL GEOLOGY AND GEOLOGIC SETTING

The San Diego area is located in the Coastal Plain sub-province of the Peninsular Ranges Physiographic Provence. In San Diego County the coastal plain runs parallel to the coast flanking the Peninsular Range and is characterized by a broad wedge of Tertiary sedimentary deposits that thicken from east to west capped by Quaternary marine terrace deposits.

The site is underlain by Tertiary-age Ardath and Scripps formations representing sedimentation in a transgressive/regressive, shallow-marine environment. The Ardath Formation grades conformably and alternately into the Scripps Formation, as such, the mapped contact between the two formations may be broad and diffuse. As shown in our boring logs and in reports by others, the stratigraphic position of the Scripps and Ardath formations can be inverted or juxtaposed while exhibiting conformable depositional contacts.

Bedding attitudes observed during previous geotechnical investigations for the surrounding property are generally horizontal or subhorizontal, exceptions being localized undulations and cross-laminations within a horizontally bedded unit.

Faulting along the present trend of the Rose Canyon fault zone began during the Pliocene, approximately 7 million years before present, and resulted in the formation of structural depressions occupied by San Diego Bay and Mission Bay. North of Mission Bay, compression and uplift occurring south of the fault resulted in the uplift of Mount Soledad. The Rose Canyon fault is considered a southerly extension of the Newport-Inglewood fault zone that may include the Descanso segment of the Agua Blanca fault zone in northern Baja California (Treiman, 1993). The onshore portion of the fault system extends from La Jolla on the north to San Diego Bay on the south.

6. GEOLOGIC HAZARDS

6.1 Geologic Hazard Category

City of San Diego Seismic Safety Study (2008) shows the site within Geologic Hazard Category 25, 52, and 12. Geologic Hazard Category 25 is defined as *Slide-Prone Formations – Ardath: neutral or favorable geologic structure*. Geologic Hazard Category 52 is defined as *Other level areas, gently sloping to steep terrain, favorable geologic structure, Low risk.* Geologic Hazard Category 12 is defined under *Fault Zones* as *Potentially Active, Inactive, Presumed Inactive, or Activity Unknown*.

6.2 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 very old terrace deposits (formerly Lindavista Formation) un-deformed and is categorized as potentially active, inactive, presumed inactive, or activity unknown (City of San Diego, 2008).

Southern California Soil & Testing, Inc, (SCS&T, 1995b) performed three fault trenches on the property. The locations of the fault trenches are shown on Figure 2. SCS&T's fault trench logs are provided in Appendix C. SCS&T reported observing and mapping the "Salk Fault". SCS&T did not differentiate between Scripps and Ardath formations in their logs, but do show the geologic contact between these formations at the fault line in their preliminary geotechnical investigation for the existing building (SCS&T, 1995a).

SCS&T found three fault traces with attitudes ranging from N72°E/70°W to N80°E/76°W. The fault traces were clay filled and/or jumbled ruptures. SCS&T concluded that these features were surface traces of the "Salk Fault"; however, the down-to-the-north orientation is not consistent with the Kennedy and Tan (2008) description. Based on our findings (Geocon Incorporated, 2014) the fault described by SCS&T is likely not the Salk Fault described by Kennedy and Tan, but is a minor, ancillary structure possibly related to the Salk Fault.

SCS&T also found several minor faults/features striking in a northeasterly direction (N20°E to N55°E) that are similar to the attitudes of a small unnamed fault noted in an earlier Woodward Clyde Consultants (WCC) report, dated April 6, 1979, referenced by SCS&T (1995b). A copy of WCC (1979) could not be obtained for review. The fault observed by WCC was purported to have displaced very old terrace deposits (formerly Lindavista Formation), but not Holocene soils. SCS&T (1995b) concluded that these splays are secondary faults associated with the easterly trending Salk Fault; however, because WCC had found the very old terrace deposits displaced, SCS&T considered the splays to be potentially active.

Based on a 3-foot vertical offset, SCS&T (1995b) provided an estimated strain rate ranging from approximately 0.001 to 0.0009 millimeters per year and concluded that this "…represents a very low strain rate and potential future movement along this fault is considered to be very low."

Geocon Incorporated (2014) excavated and logged a trench in the existing parking lot northeast of the subject site to evaluate the north eastward extension of the fault described by SCS&T (see Figure 2).

The trench was approximately 50 feet long and was excavated at least 5 feet into the underlying formational soil. Horizontally bedded sediments associated with the Scripps Formation were observed along with several minor shears and filled fractures. One fault, bearing N60°E, dipping 70°W, and showing approximately 3 inches of down-to-the-west movement was encountered in our fault trench. This fault appears to be the fault observed by SCS&T (1995b). A copy of the Geocon Incorporated (2014) fault trench log is provided in Appendix C.

Based on our review of previous fault studies performed on the property, faults likely cross the proposed parking structure building pad. It does not appear the faults cross the proposed office building pad.

Other minor faults, which strike in a northeasterly direction were found by SCS&T and are considered to be secondary faults associated with the fault identified by SCS&T as the Salk Fault.

Previous grading at the site has removed all Quaternary deposits from the site making a direct determination of fault activity impossible; however, the east-west orientation of the observed faults indicates they are not part of the current tectonic setting. The down-to-the-north sense of movement indicates that the faulting observed is likely not the Salk Fault described by Kennedy and Tan (2008). The minor displacements and poorly developed to non-existent fault gouge observed are indicative of low-risk fault rupture hazard.

It is our explicit opinion that the faulting described herein is at most potentially active and does not pose a risk of fault rupture hazard to the project. It is our express opinion that no setback zone is required to mitigate fault rupture hazard.

6.3 Seismicity

Six known active faults are located within a search radius of 50 miles from the property using the computer program *EZ-FRISK (Version 7.62)*. We used the 2008 USGS fault database, which provides several models and combinations of fault data to evaluate the fault information. Based on this database, the Newport-Inglewood/Rose Canyon Fault Zone, located approximately 3 miles west of the site, are the nearest known active faults and is the dominant source of potential ground motion. Earthquakes that might occur on the Newport-Inglewood/Rose Canyon Fault Zone or other faults within the southern California and northern Baja California area are potential generators of significant ground motion at the site. The estimated maximum earthquake magnitude and peak ground acceleration for the Newport-Inglewood/Rose Canyon Fault Zone are 7.5 and 0.47g, respectively. Table 6.3.1 lists the estimated maximum earthquake magnitude and peak ground acceleration for the most dominant faults in relation to the site location. We calculated peak ground acceleration (PGA) using Boore-Atkinson (2008) NGA USGS 2008, Campbell-Bozorgnia (2008)

NGA USGS 2008, and Chiou-Youngs (2007) NGA USGS 2008 acceleration-attenuation relationships.

TABLE 6.3.1
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/Rose Canyon	3	7.5	0.38	0.36	0.46
Rose Canyon	3	6.9	0.35	0.35	0.41
Coronado Bank	17	7.4	0.21	0.15	0.18
Palos Verdes/Coronado Bank	17	7.7	0.23	0.16	0.21
Elsinore	33	7.8	0.16	0.11	0.14
Earthquake Valley	42	6.8	0.09	0.06	0.05
Palos Verdes	48	7.3	0.10	0.07	0.07

In the event of a major earthquake on the referenced faults or other significant faults in the southern California and northern Baja California area, the site could be subjected to moderate to severe ground shaking. With respect to this hazard, the site is considered comparable to others in the general vicinity.

We performed a site-specific probabilistic seismic hazard analysis using the computer program *EZ-FRISK*. Geologic parameters not addressed in the deterministic analysis are included in this analysis. The program operates under the assumption that the occurrence rate of earthquakes on each mapped Quaternary fault is proportional to the faults slip rate. The program accounts for earthquake magnitude as a function of fault rupture length. Site acceleration estimates are made using the earthquake magnitude and distance from the site to the rupture zone. The program 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 USGS 2008 in the analysis. Table 6.3.2 presents the site-specific probabilistic seismic hazard parameters including acceleration-attenuation relationships and the probability of exceedence.

TABLE 6.3.2
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	0.52	0.47	0.55
5% in a 50 Year Period	0.37	0.33	0.37
10% in a 50 Year Period	0.27	0.24	0.26

The California Geologic Survey (CGS) provides a program for calculating the ground motion for a 10 percent of probability of exceedence in a 50-year period based on an average of several attenuation relationships. Table 6.3.3 presents the calculated results from the Probabilistic Seismic Hazards Mapping Ground Motion Page from the CGS website.

TABLE 6.3.3
PROBABILISTIC SITE PARAMETERS FOR SELECTED FAULTS
CALIFORNIA GEOLOGIC SURVEY

Calculated Acceleration (g) Firm Rock	Calculated Acceleration (g) Soft Rock	Calculated Acceleration (g) Alluvium
0.27	0.29	0.33

While listing peak accelerations is useful for comparison of potential effects of fault activity in a region, other considerations are important in seismic design, including the frequency and duration of motion and the soil conditions underlying the site. Seismic design of the structures should be performed in accordance with the 2030 California Building Code (CBC) guidelines currently adopted by the City of San Diego.

6.4 Ground Rupture

The risk associated with ground rupture hazard is low due to the absence of active faults on the property.

6.5 Liquefaction

The risk associated with liquefaction hazard is low for the site due to the dense nature of the underlying sediments and the lack of permanent, near-surface groundwater.

6.6 Landslides

Landslides were not observed or mapped in a location that could impact the proposed development. It is our opinion that the risk associated with landsliding hazard on the property is low.

6.7 Tsunamis and Seiches

The site is approximately 1.5 miles from the Pacific Ocean at an elevation over 300 feet above MSL. The risk associated with inundation hazard due to tsunamis is low.

There site is not located downstream lake or reservoir. The risk associated with inundation hazard associated with seiche is low.

7. CONCLUSIONS AND RECOMMENDATIONS

7.1 General

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

7.1.2 Our field investigation indicates the site is underlain by previously placed fill, Tertiary age Ardath Formation, and Tertiary age Scripps Formation. It is anticipated that all of the previously placed fill will be removed to achieve pad grade for the proposed parking structure. Within the proposed office building, fill is expected to underlie the northeastern half of the building pad. Where previously placed fill exists at grade, we recommend deepened footings that extend through the fill and/or drilled piers be constructed such that the office building is founded entirely on formational soils. Additionally, the proposed 5-story entry addition to the existing building should be supported on drilled piers to match the foundation for the existing building.

7.1.3 The Ardath and Scripps formation may be difficult to excavate and could generate oversize material that may require special handling.

7.1.4 Groundwater was not observed in the exploratory borings to the depths explored and is not expected to be encountered during construction of proposed improvements.

7.1.5 Based on our review of previous fault studies performed on the property, faults likely cross the proposed parking structure building pad. It does not appear the faults cross the proposed office building pad. It is our explicit opinion that the faults crossing the building pad are at most potentially active and do not pose a risk of fault rupture hazard to the project. It is our express opinion that no setback zone is required to mitigate fault rupture hazard.

7.1.6 We did not observe or know of significant geologic hazards on the site that would adversely impact the proposed development.

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

7.2 Excavation and Soil Characteristics

7.2.1 The soil encountered in the field investigation is considered to be "expansive" (expansion index [EI] of greater than 20) as defined by 2013 California Building Code (CBC) Section 1803.5.3. Table 7.2.1 presents soil classifications based on the expansion index. Based on laboratory testing, the on-site soils possess a "low" to "medium" expansion potential (expansion index of 90 or less).

TABLE 7.2.1
EXPANSION CLASSIFICATION BASED ON EXPANSION INDEX

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

7.2.2 Excavation of the *in situ* soil should be possible with moderate to heavy effort using conventional heavy-duty equipment. Strongly cemented formational materials could be encountered in excavations requiring a very heavy effort to excavate. The Ardath and Scripps Formations are known to contain isolated cemented zones that require very heavy effort to excavate. Excavation within the cemented zone will generate oversize material that will require special handling.

7.2.3 We performed laboratory tests on samples of the site soils to evaluate the percentage of water-soluble sulfate content. Results from the laboratory water-soluble sulfate content tests are presented in Appendix B. The test results indicate that the on-site materials at the locations tested possess "Not Applicable" (S0) sulfate exposure to concrete structures as defined by 2013 CBC Section 1904 and ACI 318-08 Sections 4.2 and 4.3. However, samples of soils tested for the adjacent Campus Point property to the northeast have exhibited "Moderate" (S1) characteristics. Table 7.2.2 presents a summary of concrete requirements set forth by 2013 CBC Section 1904 and ACI 318. The presence of water-soluble sulfates is not a visually discernible characteristic; therefore, other soil samples from the site could yield different concentrations. Additionally, over time landscaping activities (i.e., addition of fertilizers and other soil nutrients) may affect the concentration.

TABLE 7.2.2
REQUIREMENTS FOR CONCRETE EXPOSED TO SULFATE-CONTAINING SOLUTIONS

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

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

7.3 Subdrains

7.3.1 With the exception of wall drains, other subdrains are not required.

7.4 Grading

7.4.1 Grading should be performed in accordance with the *Recommended Grading Specifications* in Appendix E. Where the recommendations of this report conflict with Appendix E, the recommendations of this section take precedence.

7.4.2 Earthwork should be observed and compacted fill tested by representatives of Geocon Incorporated.

7.4.3 A pre-construction conference with the owner, contractor, civil engineer, and soil engineer in attendance should be held at the site prior to construction operations. Special soil handling requirements can be discussed at that time.

7.4.4 Grading of the site should commence with the removal of existing improvements, vegetation, and deleterious debris. Deleterious debris, if encountered, should be exported from the site and should not be mixed with the fill. Existing underground improvements within the proposed improvement areas that will be abandoned should be removed and the resulting excavations properly backfilled in accordance with the procedures described herein.

7.4.5 We expect the majority of grading will consist of excavations to achieve basement grade and minor cuts and fills from existing grade. In areas to receive fill, we recommend the upper 12 inches of existing fill or formational soil be scarified, moisture conditioned to at least optimum moisture content and recompacted to 90 percent relative compaction. Soil that is free of deleterious debris and contamination can then be placed as fill and compacted in layers to design finish-grade elevations. Fill and backfill materials should be placed and compacted to a dry density of at least 90 percent of the laboratory maximum dry density near to slightly above optimum moisture content as determined by the current version of ASTM Test Method D 1557. Rocks larger than 12 inches should not be placed in fill material or in utility trenches. The upper 12 inches of fill beneath pavement areas outside the building footprint 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.

7.5 Slope Stability

7.5.1 Slope stability analyses were performed for existing perimeter slope adjacent to the proposed office building. The deep-seated analysis was performed using the computer program Geoslope 2007 (see Figure 5). Surficial analysis for cut and fill slopes are shown on Figures 6 and 7. Our analyses utilized average drained direct shear strength parameters based on laboratory tests performed on the property and adjacent projects (Geocon 2014). The analyses indicate existing perimeter slope has calculated factors of safety in excess of 1.5 under static conditions for both deep-seated failure and shallow sloughing conditions.

7.6 Slopes

7.6.1 It is recommended that all slope excavations be observed during grading by an engineering geologist to verify that soil and geologic conditions do not differ significantly from those anticipated.

7.6.2 The outer 15 feet (or a distance equal to the height of the slope, whichever is less) of fill slopes should be composed of properly compacted granular *soil* fill to reduce the potential for surficial sloughing. All slopes should be compacted by backrolling with a loaded sheepsfoot roller at vertical intervals not to exceed 4 feet and should be track-walked at the completion of each slope such that the fill soils are uniformly compacted to at least 90 percent relative compaction to the face of the finished sloped.

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

7.7 Temporary Excavations

7.7.1 Temporary slopes should be constructed in conformance with OSHA requirements. Previously placed fill should be considered a Type B soil (Type C soil if seepage is encountered) and the Ardath and Scripps Formation can be considered Type A soil (Type B soil if seepage is encountered) in accordance with OSHA requirements. In general, no special shoring requirements will be necessary if temporary excavations will be less than 4 feet in height. Temporary excavations greater than 4 feet in height, however, should be laid back at an appropriate inclination. Surcharge loads should not be permitted within a distance equal to the depth of the excavation. 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. If vertical shoring will be required, Geocon Incorporated should be contacted to provide geotechnical parameters for design.

7.8 Seismic Design Criteria

7.8.1 We used the computer program *U.S. Seismic Design Maps*, provided by the USGS. Table 7.8.1 summarizes site-specific design criteria obtained from the 2013 California Building Code (CBC; Based on the 2012 International Building Code [IBC] and ASCE 7-10), Chapter 16 Structural Design, Section 1613 Earthquake Loads. The short spectral response uses a period of 0.2 second. The parking structure should be designed using a Site Class C. The office building and 5-story entry addition should be designed using a Site Class D. We evaluated the Site Class based on the discussion in Section 1613.3.2 of the 2013 CBC and Table 20.3-1 of ASCE 7-10. The values presented in Table 7.8.1 are for the risk-targeted maximum considered earthquake (MCE_R).

TABLE 7.8.1
2013 CBC SEISMIC DESIGN PARAMETERS

Parameter	Value		2010 CBC Reference
Site Class	D	C	Section 1613.3.2
MCE_R Ground Motion Spectral Response Acceleration – Class B (short), S_S	1.140 g	1.140 g	Figure 1613.3.1(1)
MCE_R Ground Motion Spectral Response Acceleration – Class B (1 sec), S_1	0.441 g	0.441 g	Figure 1613.3.1(2)
Site Coefficient, F_A	1.044	1.000	Table 1613.3.3(1)
Site Coefficient, F_V	1.559	1.359	Table 1613.3.3(2)
Site Class Modified MCE_R Spectral Response Acceleration (short), S_{MS}	1.190 g	1.140 g	Section 1613.3.3 (Eqn 16-37)
Site Class Modified MCE_R Spectral Response Acceleration (1 sec), S_{M1}	0.687 g	0.599 g	Section 1613.3.3 (Eqn 16-38)
5% Damped Design Spectral Response Acceleration (short), S_{DS}	0.794 g	0.760 g	Section 1613.3.4 (Eqn 16-39)
5% Damped Design Spectral Response Acceleration (1 sec), S_{D1}	0.458 g	0.399 g	Section 1613.3.4 (Eqn 16-40)

7.8.2 Table 7.8.2 presents additional seismic design parameters for projects located in Seismic Design Categories of D through F in accordance with ASCE 7-10 for the mapped maximum considered geometric mean (MCE_G).

TABLE 7.8.2
2013 CBC SITE ACCELERATION PARAMETERS

Parameter	Value		ASCE 7-10 Reference
Site Class	D	C	Section 1613.3.2
Mapped MCE_G Peak Ground Acceleration, PGA	0.488 g	0.488	Figure 22-7
Site Coefficient, F_{PGA}	1.012	1.000	Table 11.8-1
Site Class Modified MCE_G Peak Ground Acceleration, PGA_M	0.494 g	0.488 g	Section 11.8.3 (Eqn 11.8-1)

7.8.3 Conformance to the criteria in Tables 7.8.1 and 7.8.2 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.

7.9 Foundations

7.9.1 We recommend each of the proposed structures be founded on formational soil. We expect all of the previously placed fill will be removed to achieve pad grade within the parking structure building pad; however, within the proposed office building and the 5-story entry addition to the existing building, previously placed fill will be present below pad grade. Where fill is present, we recommend the footings be deepened to extend through the fill to bear entirely on native formational soil. Deepening the footing can be accomplished by drilled piers or conventional deepened footings that extend through the fill. Recommendations for both shallow and deep foundations are provided hereinafter.

7.10 Shallow Foundations

7.10.1 The following shallow foundation recommendations assume all new structural footings for the proposed structures will be founded directly on formational soils. Foundations can consist of continuous strip footings and/or isolated spread footings. Continuous footings should be at least 18 inches wide and extend at least 24 inches below lowest adjacent pad grade. Isolated spread footings should have a minimum width and depth of 2 feet. Concrete reinforcement for continuous footings should consist of at least four, No. 5 steel, reinforcing bars placed horizontally in the footings; two near the top and two near the bottom. The project structural engineer should design the concrete reinforcement for the spread footings. A typical wall/column footing dimension detail is presented on Figure 8.

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

7.10.3 The recommended allowable bearing capacity for foundations with minimum dimensions described above and bearing on native formational soil is 4,000 psf. The allowable soil bearing pressure may be increased by an additional 500 psf for each additional foot of depth and 300 psf for each additional foot of width, to a maximum allowable bearing capacity of 8,000 psf.

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

7.10.5 Total and differential settlements under the imposed allowable loads are estimated to be 1 inch and ½ inch, respectively in 40 feet.

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

7.10.7 No special subgrade presaturation is deemed necessary prior to placement of concrete. However, the slab and foundation subgrade should be moistened as necessary, to maintain a moist condition as would be expected in any such concrete placement.

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

7.11 Drilled Piers Foundations

7.11.1 Drilled pier foundations can be utilized where structures are underlain by previously placed fill.

7.11.2 Figure 9 presents the theoretical single pier allowable axial capacity versus pier embedment depth into formational materials (not total pier length) for 24-inch, 30-inch, 36-inch, and 48-inch-diameter drilled piers. We recommend drilled piers have a minimum pier diameter of 2 feet, a minimum length of 10 feet, and a minimum embedment into formational materials of 5 feet.

7.11.3 Allowable axial capacities given on Figure 8 are based on end bearing and skin friction for the portion of the pier embedded in formational materials. The capacities provided are based on a Factor of Safety of 3.0 applied to the ultimate end bearing capacity and 2.0 for skin friction. Skin friction has been neglected for the portion of the pier in previously placed fill.

7.11.4 Because a significant portion of the pier 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 or hand cleaning 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. Borehole sidewall instability may randomly occur if cohesionless soil is encountered.

7.11.5 For resistance to uplift, an allowable unit skin friction of 300 psf can be utilized for the portion of the pier in formational soils.

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

7.11.7 If pile spacing is at least three times the maximum dimension of the pile, no reduction in axial capacity or lateral load capacity is considered necessary for group effects. If pile spacing is closer than three pile diameters, an evaluation for group effects including appropriate reductions should be performed by Geocon Incorporated based on pile dimension and spacing.

7.11.8 It is anticipated that the on-site soils can be excavated with typical pier drilling equipment. However, concretions are common in the Ardath and Scripps Formation, which if encountered, will be difficult to drill. Pier drilling should be observed by a representative of the geotechnical engineer to evaluate proper embedment depth into formational soil and whether appropriate drilling procedures are being used.

7.11.9 Concrete should be placed the same day the shafts are excavated to reduce the potential for caving. If pier holes are left open overnight or for extended periods of time, cleaning and/or re-drilling of the hole will be necessary. Initial set of the concrete should be achieved before an adjacent pier boring is drilled.

7.11.10 The concrete should be placed in such a way as to minimize segregation of the aggregate. Tremies should be utilized for concrete placed below a depth of 20 feet.

7.11.11 Pier settlement is expected to be on the order of 1-inch or less for drilled piers. The majority of settlement should occur during construction.

7.12 Concrete Slabs-on-Grade

7.12.1 Building interior concrete slabs-on-grade should be at least 5 inches thick. Slab reinforcement should consist of No. 4 steel reinforcing bars spaced 18 inches on center in both horizontal directions placed at the middle of the slab.

7.12.2 A vapor retarder should underlie slabs that may receive moisture-sensitive floor coverings or may be used to store moisture-sensitive materials. The vapor retarder design should be consistent with the guidelines presented in the American Concrete Institute’s (ACI) *Guide for Concrete Slabs that Receive Moisture-Sensitive Flooring Materials* (ACI 302.2R-06).

In addition, the membrane should be installed in a manner that prevents puncture in accordance with manufacturer's recommendations and ASTM requirements. The project architect or developer should specify the type vapor retarder used based on the type of floor covering that will be installed and if the structure will possess a humidity-controlled environment.

7.12.3 The project foundation engineer, architect, and/or developer should determine the bedding sand thickness below concrete slabs. Typically, 3 to 4 inches of bedding sand is used. Geocon Incorporated should be contacted to provide recommendations if the bedding sand is thicker than 6 inches.

7.12.4 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.

7.12.5 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 vehicle, equipment and storage loads.

7.12.6 Exterior slabs not subject to vehicle loads should be at least 4 inches thick and reinforced with 6x6-W2.9/W2.9 (6x6-6/6) welded wire mesh. The mesh should be placed within the upper one-third of the slab. Proper mesh positioning is critical to future performance of the slabs. The contractor should take extra measures to provide proper mesh placement. Prior to construction of slabs, the subgrade should be moisture conditioned to at least optimum moisture content and compacted to a dry density of at least 90 percent of the laboratory maximum dry density.

7.12.7 In addition, concrete flatwork should be provided with crack control joints to reduce and/or control shrinkage cracking. The project structural engineer should determine crack control spacing based on slab thickness and intended usage. Criteria of the American Concrete Institute (ACI) should be taken into consideration when establishing crack control spacing.

7.12.8 To reduce the potential for heaving of exterior concrete flatwork underlain by expansive soils, flatwork should be structurally connected to the curbs, where possible, to reduce the potential for offsets between the curbs and the flatwork. 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. The project structural engineer should provide dowelling design and details.

7.12.9 The above slab-on-grade dimensions and minimum reinforcement recommendations are based upon soil conditions only and are not intended to be used in lieu of those required for structural purposes.

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

7.12.11 The recommendations of this report are intended to reduce the potential for cracking of slabs due to expansive soils (if present). However, even with the incorporation of the recommendations presented herein, foundations and slabs-on-grade placed on such conditions may still exhibit some cracking. 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, the use of crack control joints and proper concrete placement and curing. 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.

7.13 Retaining Walls

7.13.1 Retaining walls that are allowed to rotate more than 0.001H (where H equals the height of the retaining portion of the wall) at the top of the wall and having a level backfill surface should be designed for an active soil pressure equivalent to the pressure exerted by a fluid density of 35 pcf. Where the backfill will be inclined at 2:1 (horizontal:vertical), an active soil pressure of 52 pcf is recommended. These active pressures assume low expansive soil (Expansion Index less than 50) will be used as retaining wall backfill. Soils with a low expansion potential may require select grading or import.

7.13.2 Where walls are restrained from movement at the top, an additional uniform pressure of 8H psf should be added to the active soil pressure where the wall possesses a height of 8 feet or less and 12H where the wall is greater than 8 feet.

7.13.3 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.

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

7.13.5 Retaining walls should be provided with a drainage system adequate to prevent the buildup of hydrostatic forces and should be waterproofed as required by the project architect. The use of drainage openings through the base of the wall (weep holes) is not recommended where the seepage could be a nuisance or otherwise adversely affect the property adjacent to the base of the wall. The above recommendations assume a properly compacted granular (EI of less than 50) free-draining backfill material with no hydrostatic forces or imposed surcharge load. A typical retaining wall drainage detail is provided on Figure 10. If conditions different than those described are expected, Geocon Incorporated should be contacted for additional recommendations.

7.13.6 The structural engineer should determine the seismic design category for the project in accordance with Section 1613 of the CBC. If the project possesses a seismic design category of D, E, or F, retaining walls that support more than 6 feet of backfill should be designed with seismic lateral pressure in accordance with Section 18.3.5.12 of the 2013 CBC. The seismic load is dependent on the retained height where H is the height of the wall, in feet, and the calculated loads result in pounds per square foot (psf) exerted at the base of the wall and zero at the top of the wall. A seismic load of 22H should be used for design. We used the peak ground acceleration adjusted for Site Class effects, PGA_M, of 0.494g calculated from ASCE 7-10 Section 11.8.3 and applied a pseudo-static coefficient of 0.33.

7.13.7 In general, wall foundations having a minimum embedment depth of 24 inches and a width of 12 inches may be designed for an allowable soil bearing pressure of 2,500 psf for compacted fill and 4,000 psf for Ardath and Scripps Formations. The allowable soil bearing pressure may be increased by an additional 500 psf for each additional foot of depth and 300 psf for each additional foot of width to a maximum bearing capacity of

4,000 psf for fill and 8,000 psf for Ardath and Scripps Formation. The values presented above are for dead plus live loads and may be increased by one-third when considering transient loads due to wind or seismic forces. The proximity of the foundation to the top of a slope steeper than 3:1 could impact the allowable soil bearing pressure. Therefore, Geocon Incorporated should be consulted where such a condition is expected.

7.13.8 Foundation excavations should be observed by the geotechnical engineer (a representative of Geocon Incorporated) prior to the placement of reinforcing steel and concrete to observe that the exposed soil conditions are consistent with those anticipated and that they have been extended to the appropriate bearing strata. If unanticipated soil conditions are encountered, foundation modifications may be required.

7.14 Lateral Loading

7.14.1 To resist lateral loads, a passive pressure exerted by an equivalent fluid weight of 300 pounds per cubic foot (pcf) should be used for design of footings or shear keys poured neat against compacted fill. The allowable passive pressure assumes a horizontal surface extending at least 5 feet or three times the height of the surface generating the passive pressure, whichever is greater. The upper 12 inches of material not protected by floor slabs or pavement should not be included in the design for lateral resistance. Where walls are planned adjacent to and/or on descending slopes, a passive pressure of 150 pcf should be used in design.

7.14.2 If friction is to be used to resist lateral loads, an allowable coefficient of friction between soil and concrete of 0.35 should be used for design for footings founded in compacted fill or formational materials. The recommended passive pressure may be used concurrently with frictional resistance and may be increased by one-third for transient wind or seismic loading.

7.15 Preliminary Pavement Recommendations

7.15.1 The following preliminary pavement design sections are based on our experience with soil conditions within the surrounding area and laboratory R-value testing performed on adjacent projects. The preliminary sections presented herein are for budgetary estimating purposes only and are not for construction. Final pavement sections should be determined after the grading operations are completed, subgrade soils are exposed, and additional R-Value tests are performed on actual pavement subgrade samples. For preliminary design, we used a resistance value (R-Value) of 20 for subgrade soils and 78 for aggregate base.

7.15.2 Asphalt concrete pavement thicknesses were determined following procedures outlined in the *California Highway Design Manual* (Caltrans). Portland Cement concrete sections are based on methods suggested by the American Concrete Institute *Guide for Design and Construction of Concrete Parking Lots (ACI 330R-08)*.

7.15.3 The project civil engineer or traffic engineer should provide the actual TI that is appropriate for the project based on anticipated traffic loading and volumes. Tables 7.15.1 and 7.15.2 provide preliminary pavement design sections for varying Traffic Indices (TI).

TABLE 7.15.1
PRELIMINARY ASPHALT CONCRETE PAVEMENT SECTIONS

Traffic Index	Asphalt Concrete (inches)	Class 2 Aggregate Base (inches)
4.5	3	5.5
5	3	7
5.5	3	9
6	4	8.5
6.5	4	10
7	5	10
7.5	5	11.5

TABLE 7.15.2
PRELIMINARY PAVEMENT SECTIONS FOR PORTLAND CEMENT CONCRETE

Location	Traffic Category	Estimated Average Daily Truck Traffic (ADTT)	Concrete Thickness (inches)	Class 2 Aggregate Base Thickness (inches)
Automobile Parking	A	1 or less	5	4
Automobile Driveways	A	10 or less	6	4
Heavy Truck Traffic/Fire Lanes	B	25 or less	7	4

7.15.4 Class 2 aggregate base materials should conform to Section 26-1.02B of the *Standard Specifications of the State of California, Department of Transportation* (Caltrans) or Sections 400-2 and 203-6 of the *Standard Specifications for Public Works Construction (Greenbook)*. The aggregate base specifications are found in the *Regional Supplemental to Greenbook*.

7.15.5 Pavement subgrade soils should be scarified, moisture conditioned as necessary, and compacted to a dry density of at least 95 percent of the laboratory maximum dry density near to slightly above optimum moisture content in accordance with ASTM D 1557. The depth of compaction should be at least 12 inches. Base course material should be moisture conditioned near to slightly above optimum moisture content and compacted to a dry density of at least 95 percent of the laboratory maximum dry density. Asphalt concrete pavement should be compacted to at least 95 percent of the laboratory Hveem density in accordance with ASTM D 2726.

7.15.6 The following recommendations apply to the areas where Portland Cement Concrete pavement will be utilized to support vehicular traffic.

- Portland Cement concrete pavement should have a minimum concrete flexural strength (modulus of rupture, MR) of 500 pounds per square inch (psi) (compressive strength of 3,200 psi).
- To control the location and spread of concrete shrinkage cracks, it is recommended that crack control joints be included in the design of the concrete pavement slabs. Crack control joint spacing should not exceed 15 feet. The crack control joints should be created while the concrete is still fresh using a grooving tool or shortly thereafter using saw cuts. The joint should extend into the slab a minimum of one-fourth of the slab thickness.
- Construction joints should be provided at the interface between areas of concrete placed at different times during construction. Doweling is recommended between the joints to transfer anticipated truck traffic loading. Dowels should be located at the midpoint of the slab and be spaced at 12 inches on center.
- Joints should be filled with a joint filler or sealer to aid in preventing migration of water into subgrade and base materials. Appropriate fillers or sealers are discussed in the referenced ACI guide.

7.15.7 Where trash bin enclosures are planned, the pavement section should consist of 7 inches of Portland cement concrete reinforced with No. 3 bars spaced at 18 inches in each horizontal direction. The concrete loading area should extend out such that both the front and rear wheels of the truck will be located on reinforced concrete pavement when loading and unloading.

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

7.16 Bio-Retention Basin and Bio-Swale Recommendations

7.16.1 The site is underlain by previously placed fill and Ardath and Scripps Formations that is generally composed of silty to clayey sand, clayey to sandy silt and silty clay. The on-site soils generally have a fine content (minus 200) of 25 to 80 percent. Based on our experience with the on-site soils, the compacted fill and Ardath and Scripps Formations have very low permeability and typically very low infiltration characteristics. It is our opinion the compacted fill and Ardath and Scripps Formations area unsuitable for infiltration of storm-water runoff.

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

7.16.3 Distress may be caused to planned improvements and properties located hydrologically downstream or adjacent to these devices. The distress depends on the amount of water to be detained, its residence time, soil permeability, and other factors. We have not performed a hydrogeology study at the site. Downstream and adjacent properties may be subjected to seeps, springs, slope instability, raised groundwater, movement of foundations and slabs, or other impacts as a result of water infiltration. Due to site soil and geologic conditions (i.e., compacted fills and dense formational bedrock), permanent bio-retention basins should be lined with an impermeable barrier, such as a thick visqueen, to prevent water infiltration in to the underlying soils.

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

7.17 Site Drainage and Moisture Protection

7.17.1 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 2010 CBC 1804.3 or other applicable standards. In addition, surface drainage should be directed away from the top of slopes into swales or other controlled drainage devices. Roof and pavement drainage should be directed into conduits that carry runoff away from the proposed structure.

7.17.2 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.

7.17.3 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.

7.18 Grading and Foundation Plan Review

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

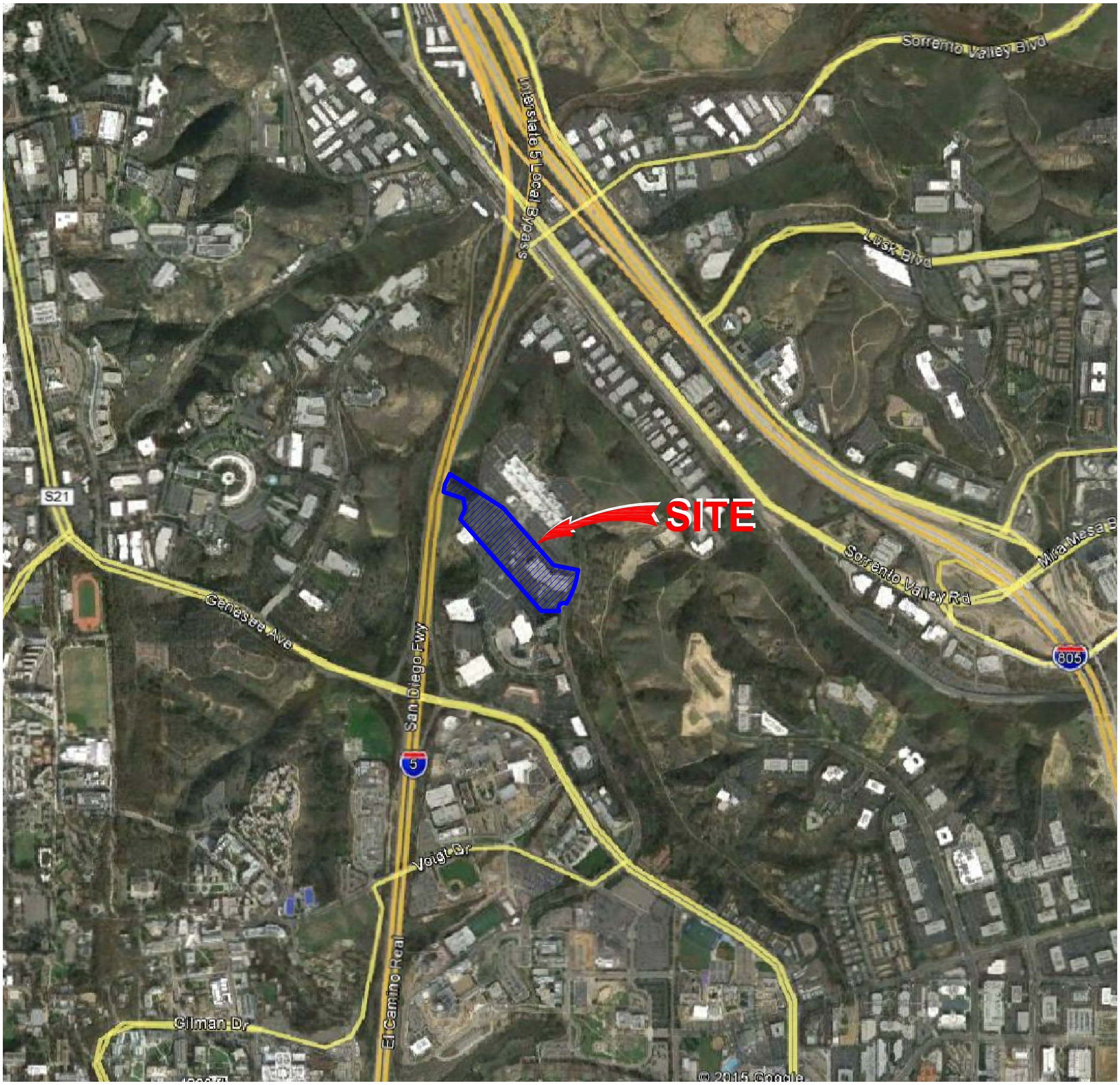
LIMITATIONS AND UNIFORMITY OF CONDITIONS

1. The firm that performed the geotechnical investigation for the project should be retained to provide testing and observation services during construction to provide continuity of geotechnical interpretation and to check that the recommendations presented for geotechnical aspects of site development are incorporated during site grading, construction of improvements, and excavation of foundations. If another geotechnical firm is selected to perform the testing and observation services during construction operations, that firm should prepare a letter indicating their intent to assume the responsibilities of project geotechnical engineer of record. A copy of the letter should be provided to the regulatory agency for their records. In addition, that firm should provide revised recommendations concerning the geotechnical aspects of the proposed development, or a written acknowledgement of their concurrence with the recommendations presented in our report. They should also perform additional analyses deemed necessary to assume the role of Geotechnical Engineer of Record.

2. The recommendations of this report pertain only to the site investigated and are based upon the assumption that the soil conditions do not deviate from those disclosed in the investigation. If any variations or undesirable conditions are encountered during construction, or if the proposed construction will differ from that anticipated herein, Geocon Incorporated should be notified so that supplemental recommendations can be given. The evaluation or identification of the potential presence of hazardous or corrosive materials was not part of the scope of services provided by Geocon Incorporated.

3. This report is issued with the understanding that it is the responsibility of the owner or his representative to ensure that the information and recommendations contained herein are brought to the attention of the architect and engineer for the project and incorporated into the plans, and the necessary steps are taken to see that the contractor and subcontractors carry out such recommendations in the field.

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



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N

NO SCALE

VICINITY MAP

GEOCON INCORPORATED

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PHONE 858 558-6900 - FAX 858 558-6159

10290 CAMPUS POINT DRIVE
SAN DIEGO, CALIFORNIA

RM / AML		DSK/GTYPD	DATE 06 - 11 - 2015	PROJECT NO. 07850 - 42 - 15	FIG. 1

Plotted:06/12/2015 7:50AM | By:ALVIN LADRILLONO | File Location:Y:\PROJECTS\07850-42-15 (10290 Campus Point)\DETAILS\07850-42-15 Vic Map.dwg

GEOCON LEGEND

Qpf ....PREVIOUSLY PLACED FILL

Tsc ....SCRIPPS FORMATION (Dotted Where Buried)

Ta ....ARDATH FORMATION (Dotted Where Buried)

........APPROX. LOCATION OF GEOLOGIC CONTACT (Dotted Where Buried, Queried Where Uncertain)

B-6 ....APPROX. LOCATION OF BORING (Geocon)

B-16 ....APPROX. LOCATION OF BORING (SCS&T, 1995))

B B' ....APPROX. LOCATION OF GEOLOGIC CROSS SECTIONS

FT-1 ....APPROX. LOCATION OF FAULT TRENCH (Geocon, See Appendix C)

C C' ....APPROX. LOCATION OF FAULT TRENCH (SCS&T, See Appendix C)

GEOLOGIC MAP

10290 CAMPUS POINT DRIVE
SAN DIEGO, CALIFORNIA

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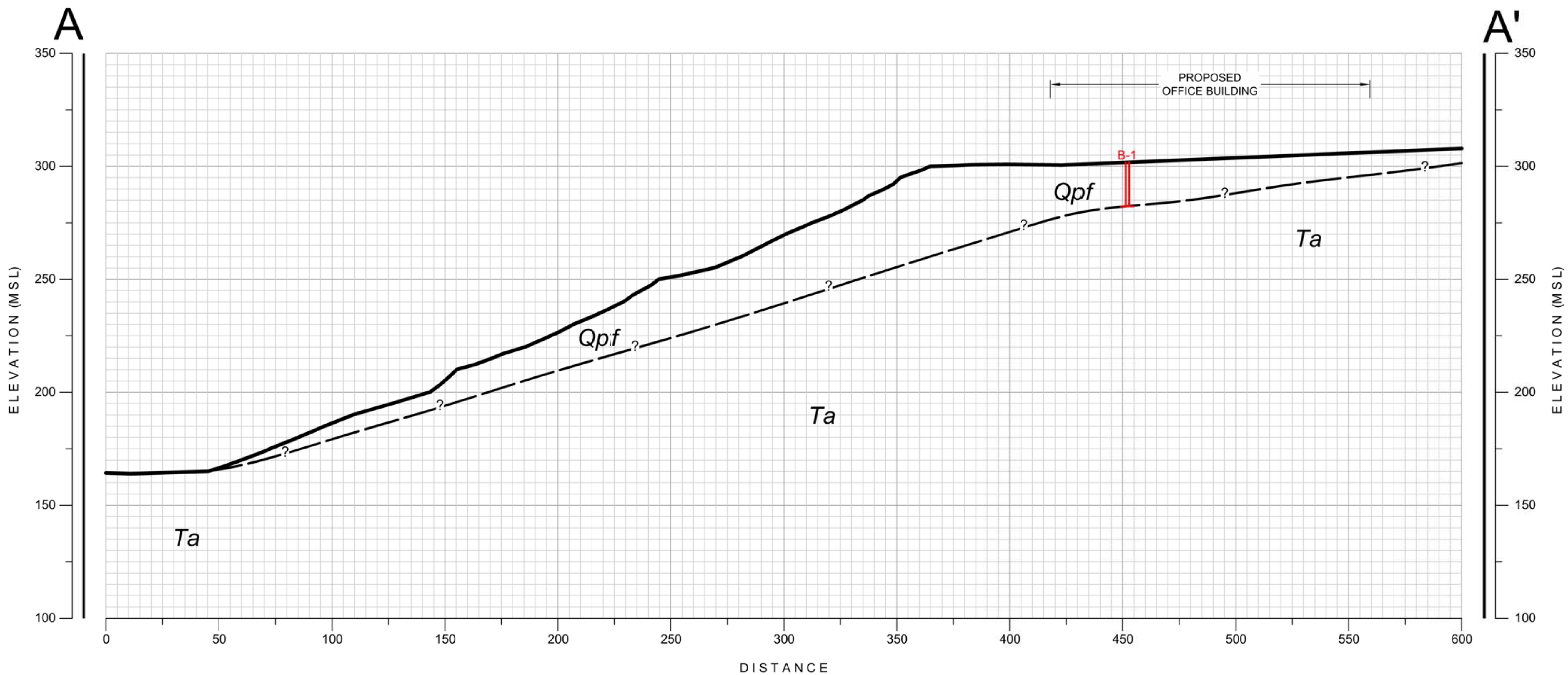
SCALE 1" = 80'	DATE 06 - 11 - 2015
PROJECT NO. 07850 - 42 - 15	FIGURE 2
SHEET 1 OF 1	

Plotted:06/12/2015 7:38AM | By:ALVIN LADRILLONO | File Location:Y:\PROJECTS\07850-42-15 (10290 Campus Point)\SHEETS\07850-42-15 Geo Map.dwg

10290 CAMPUS POINT DRIVE
SAN DIEGO, CALIFORNIA



GEOLOGIC CROSS-SECTION A-A'

SCALE: 1" = 50' (Vert. = Horiz.)

GEOCON LEGEND

Qpf........PREVIOUSLY PLACED FILL

Ta........ARDATH FORMATION

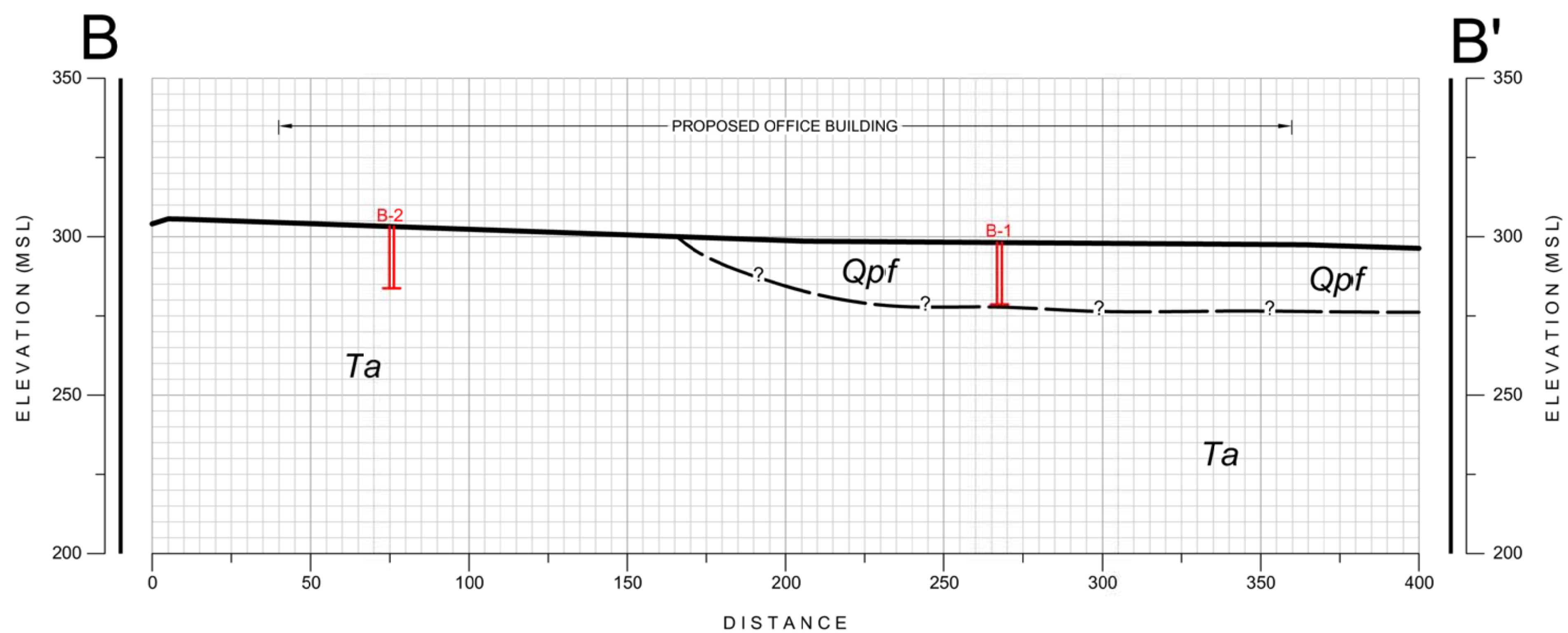
........APPROX. LOCATION OF GEOLOGIC CONTACT
(Queried Where Uncertain)

B-1 ....APPROX. LOCATION OF BORING

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

PROJECT NO. 07850 - 42 - 15
FIGURE 3
DATE 06 - 11 - 2015

10290 CAMPUS POINT DRIVE
SAN DIEGO, CALIFORNIA



GEOLOGIC CROSS-SECTION B-B'

SCALE: 1" = 50' (Vert. = Horiz.)

GEOCON LEGEND

Qpf........PREVIOUSLY PLACED FILL

Ta........ARDATH FORMATION

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

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

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PROJECT NO. 07850 - 42 - 15
FIGURE 4
DATE 06 - 11 - 2015

10290 Campus Point
Project No. 07850-42-15
Section A-A'
Name: A-A'.gsz
Date: 6/11/2015

MATERIAL PROPERTIES:
Name: Qpf Unit Weight: 125 pcf Cohesion: 450 psf Phi: 28 °
Name: Ta Unit Weight: 125 pcf Cohesion: 450 psf Phi: 38 °

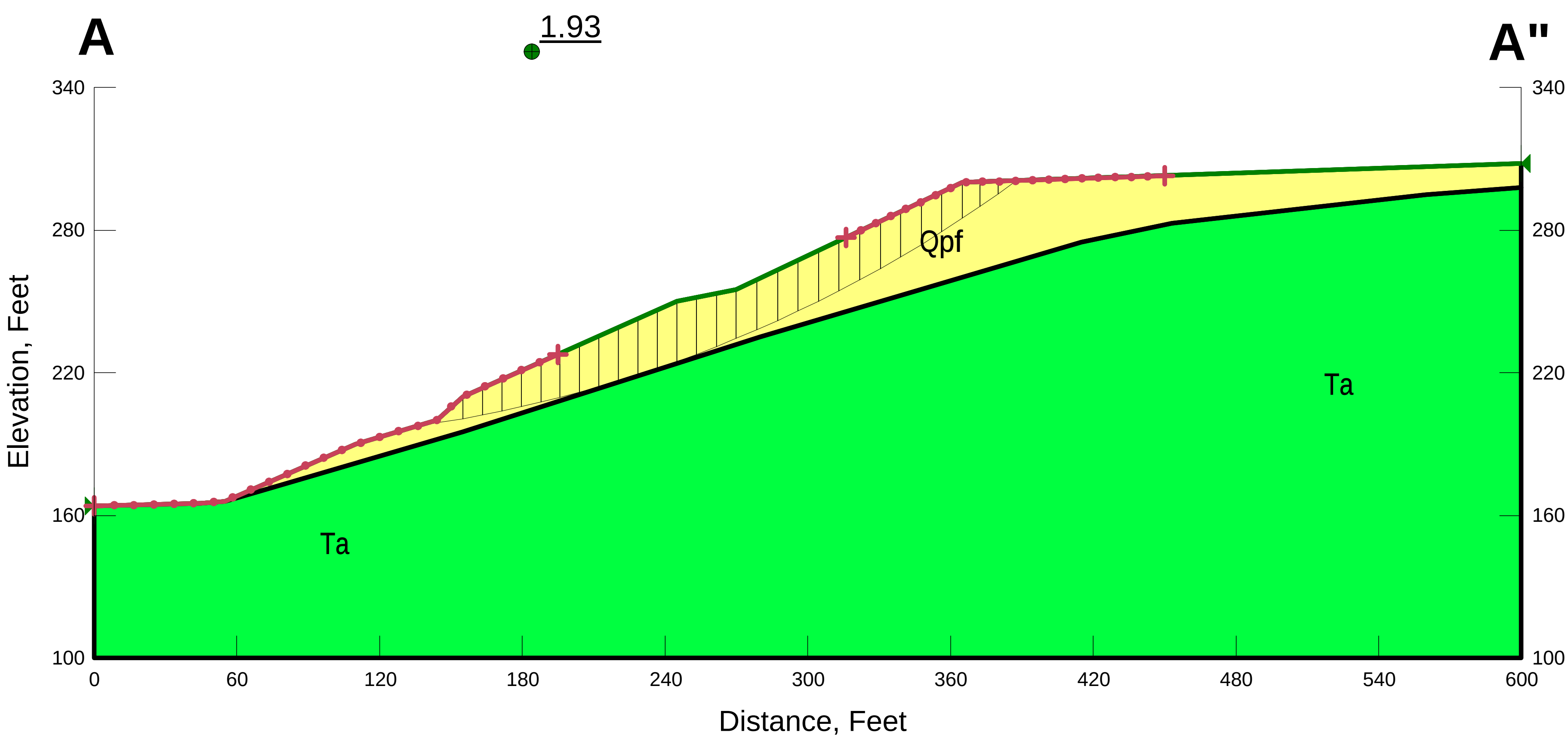

Figure 5

ASSUMED CONDITIONS :

SLOPE HEIGHT	H	= Infinite
DEPTH OF SATURATION	Z	= 3 feet
SLOPE INCLINATION	2 : 1	(Horizontal : Vertical)
SLOPE ANGLE	i	= 26.6 degrees
UNIT WEIGHT OF WATER	γ_w	= 62.4 pounds per cubic foot
TOTAL UNIT WEIGHT OF SOIL	γ_t	= 125 pounds per cubic foot
ANGLE OF INTERNAL FRICTION	ϕ	= 38 degrees
APPARENT COHESION	C	= 450 pounds per square foot

SLOPE SATURATED TO VERTICAL DEPTH Z BELOW SLOPE FACE

SEEPAGE FORCES PARALLEL TO SLOPE FACE

ANALYSIS :

$$FS = \frac{C + (\gamma_t - \gamma_w)\ Z \cos^2 i \tan \phi}{\gamma_t\ Z \sin i \cos i} = 3.8$$

REFERENCES :

1......Haefeli, R. *The Stability of Slopes Acted Upon by Parallel Seepage*, Proc. Second International Conference, SMFE, Rotterdam, 1948, 1, 57-62

2......Skempton, A. W., and F.A. Delory, *Stability of Natural Slopes in London Clay*, Proc. Fourth International Conference, SMFE, London, 1957, 2, 378-81

SURFICIAL SLOPE STABILITY ANALYSIS - CUT SLOPE

GEOCON INCORPORATED

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10290 CAMPUS POINT DRIVE
SAN DIEGO, CALIFORNIA

RM / AML		DSK/GTYPD	DATE 06 - 11 - 2015	PROJECT NO. 07850 - 42 - 15	FIG. 6

ASSUMED CONDITIONS :

SLOPE HEIGHT	H	= Infinite
DEPTH OF SATURATION	Z	= 3 feet
SLOPE INCLINATION	2 : 1	(Horizontal : Vertical)
SLOPE ANGLE	i	= 26.6 degrees
UNIT WEIGHT OF WATER	γ_w	= 62.4 pounds per cubic foot
TOTAL UNIT WEIGHT OF SOIL	γ_t	= 125 pounds per cubic foot
ANGLE OF INTERNAL FRICTION	ϕ	= 28 degrees
APPARENT COHESION	C	= 450 pounds per square foot

SLOPE SATURATED TO VERTICAL DEPTH Z BELOW SLOPE FACE

SEEPAGE FORCES PARALLEL TO SLOPE FACE

ANALYSIS :

$$FS = \frac{C + (\gamma_t - \gamma_w)\ Z \cos^2 i \tan \phi}{\gamma_t\ Z \sin i \cos i} = 3.5$$

REFERENCES :

1......Haefeli, R. *The Stability of Slopes Acted Upon by Parallel Seepage*, Proc. Second International Conference, SMFE, Rotterdam, 1948, 1, 57-62

2......Skempton, A. W., and F.A. Delory, *Stability of Natural Slopes in London Clay*, Proc. Fourth International Conference, SMFE, London, 1957, 2, 378-81

SURFICIAL SLOPE STABILITY ANALYSIS - FILL SLOPE

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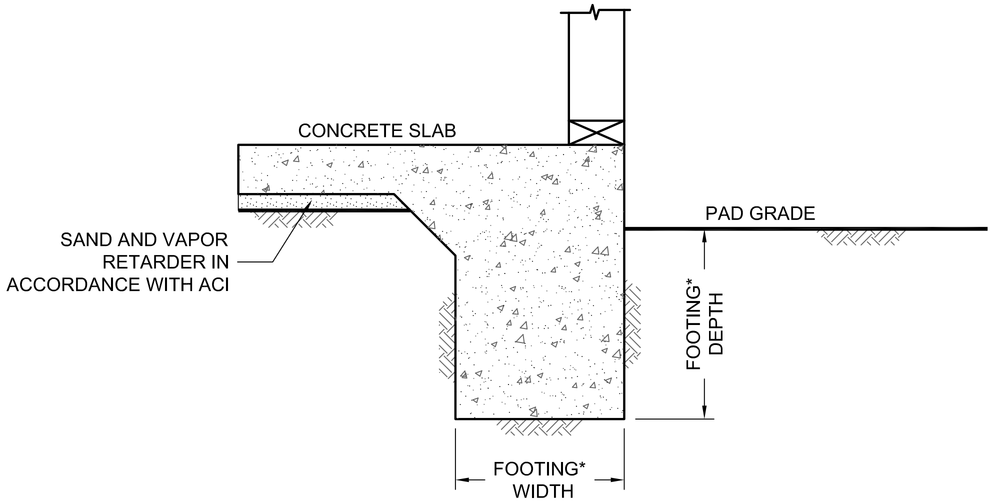
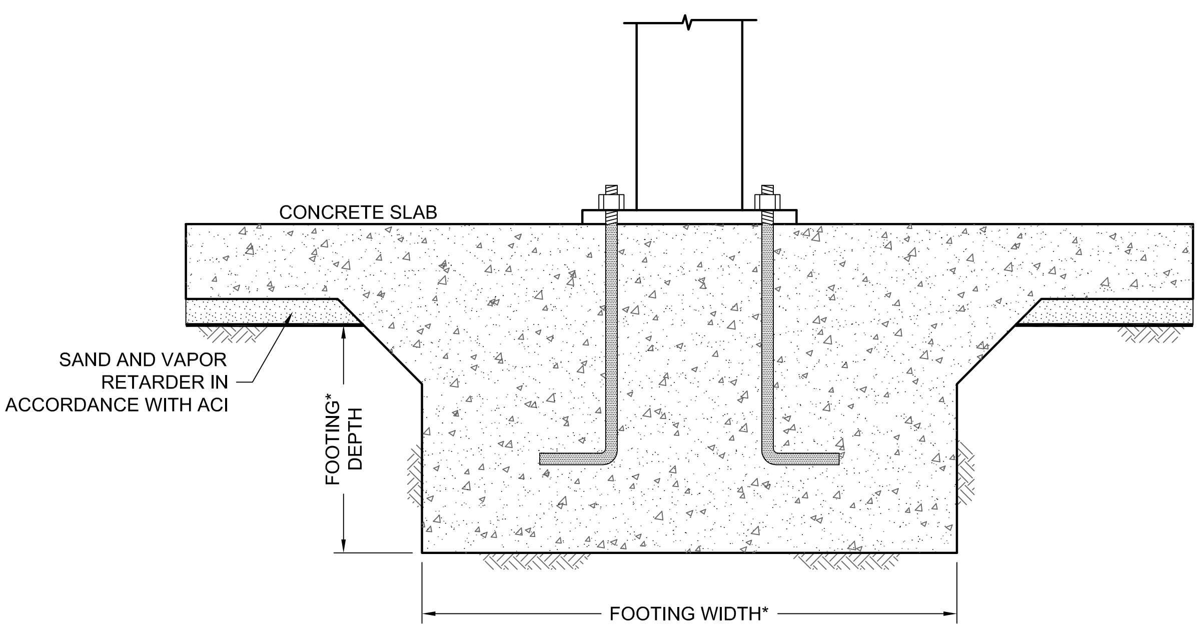
PHONE 858 558-6900 - FAX 858 558-6159

10290 CAMPUS POINT DRIVE
SAN DIEGO, CALIFORNIA

RM / AML		DSK/GTYPD	DATE 06 - 11 - 2015	PROJECT NO. 07850 - 42 - 15	FIG. 7





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

NO SCALE

WALL / COLUMN FOOTING DIMENSION DETAIL

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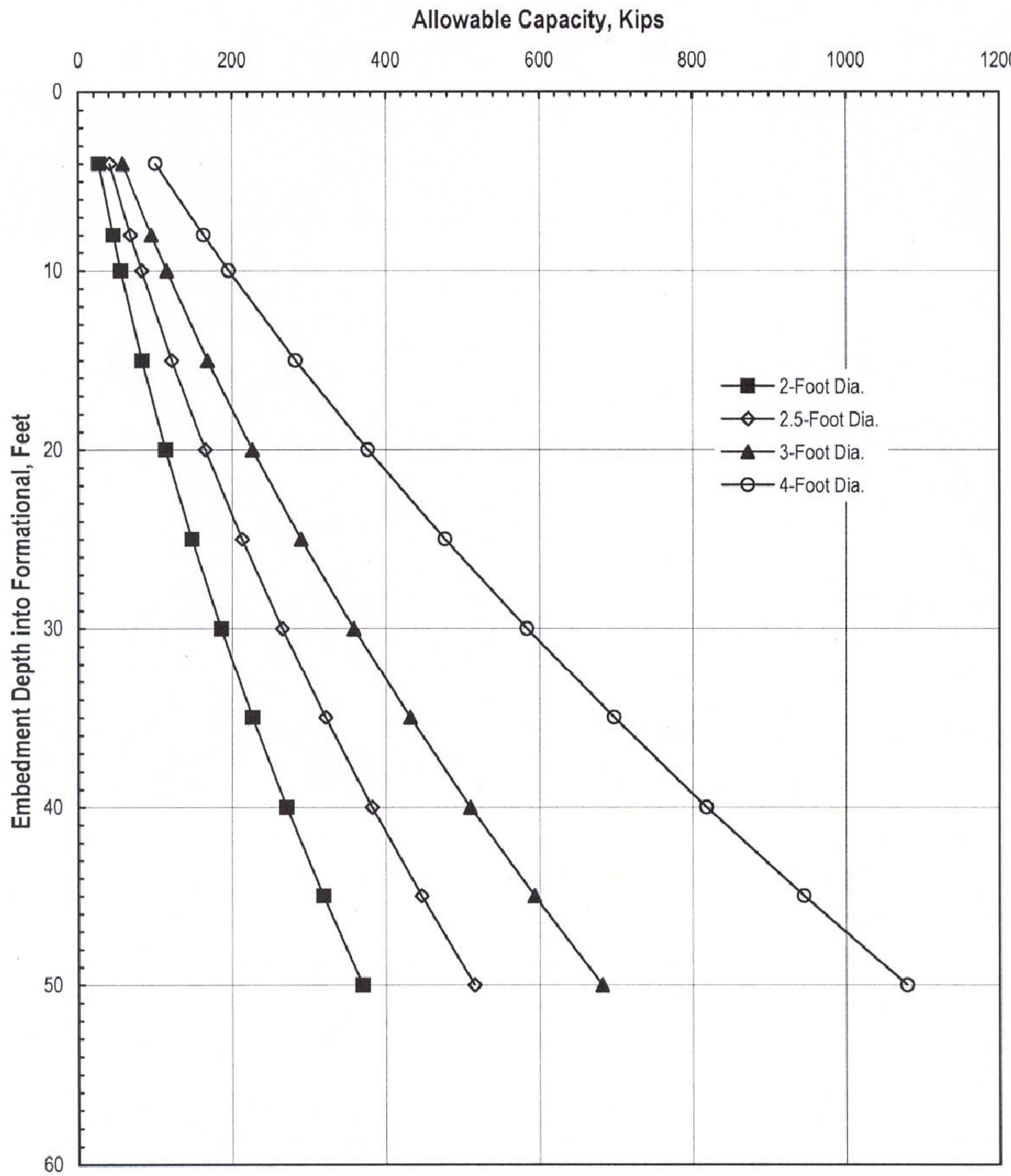
GEOTECHNICAL ■ ENVIRONMENTAL ■ MATERIALS
6960 FLANDERS DRIVE - SAN DIEGO, CALIFORNIA 92121 - 2974
PHONE 858 558-6900 - FAX 858 558-6159

10290 CAMPUS POINT DRIVE
SAN DIEGO, CALIFORNIA

RM / AML		DSK/GTYPD	DATE 06 - 11 - 2015	PROJECT NO. 07850 - 42 - 15	FIG. 8

Plotted:06/12/2015 7:57AM | By:ALVIN LADRILLONO | File Location:Y:\PROJECTS\07850-42-15 (10290 Campus Point)\DETAILS\Wall-Column Footing Dimension Detail (COLFOOT2).dwg



ALLOWABLE CAPACITY FOR DRILLED PIERS

GEOCON INCORPORATED

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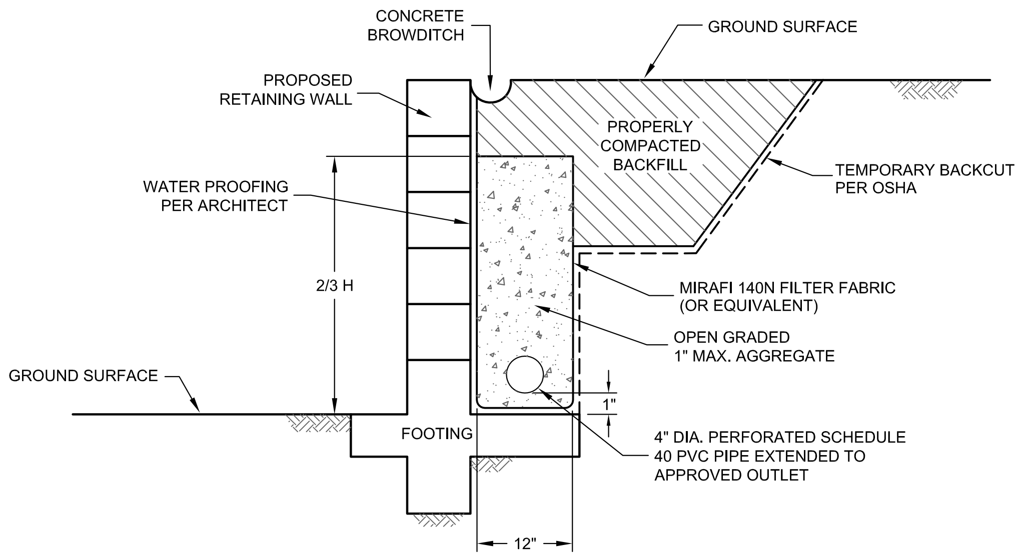
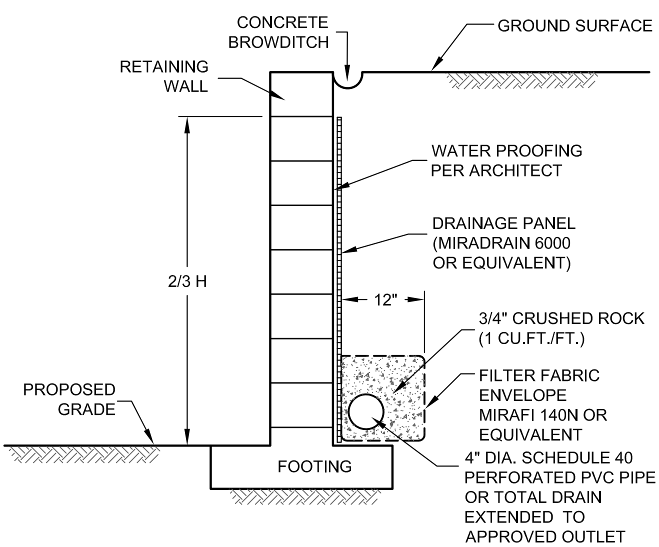
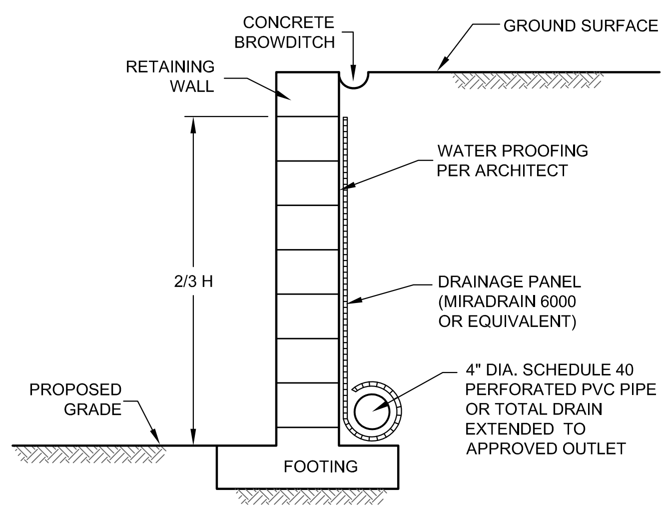
6960 FLANDERS DRIVE - SAN DIEGO, CALIFORNIA 92121 - 2974
PHONE 858 558-6900 - FAX 858 558-6159

10290 CAMPUS POINT DRIVE
SAN DIEGO, CALIFORNIA

RM / AML		DSK/GTYPD	DATE 06 - 11 - 2015	PROJECT NO. 07850 - 42 - 15	FIG. 9







NOTE :

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

NO SCALE

TYPICAL RETAINING WALL DRAIN DETAIL

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

10290 CAMPUS POINT DRIVE
SAN DIEGO, CALIFORNIA

RM / AML		DSK/GTYPD	DATE 06 - 11 - 2015	PROJECT NO. 07850 - 42 - 15	FIG. 10

APPENDIX A

APPENDIX A

FIELD INVESTIGATION

The field investigation was performed on May 26, 2015 and consisted of a site reconnaissance and drilling 6 small–diameter-auger borings. The approximate locations of the borings are shown on the Geologic Map (Figure 2).

The exploratory borings were drilled using a CME 75 drill rig with 8-inch diameter hollow-stem augers. The borings extended to a maximum depth of approximately 20 feet below existing grade. Logs of the borings depicting soil and geologic conditions encountered and the depth at which samples were obtained are presented on Figures A-1 through A-6.

Relatively undisturbed, ring samples as well as bulk samples were obtained from selected depths within the borings for laboratory analysis. The soils encountered were visually examined, classified, and logged in general accordance with ASTM Test Method D-2488 *Description and Identification of Soils (Visual-Manual Method)*.

PROJECT NO. 07850-42-15

BORING B 1

ELEV. (MSL.) 302' 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.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.) **302'** 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								
8					-Becomes tan brown; encountered hard cemented zone; different drilling between 7' to 9'			
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.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.) **305'** 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**			
2				SM	**ARDATH FORMATION** Dense to very dense damp light grayish brown, Silty, fine to medium SAND -Becomes damp to moist light yellowish brown			
4								
	B3-1					71/10"		
6	B3-2							
8								
10				SM	Medium dense, damp, light brown, Silty, fine to medium SAND			
	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.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.

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PROJECT NO. 07850-42-15

BORING B 4

ELEV. (MSL.) **302'** 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.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.) **301'** 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.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 6

ELEV. (MSL.) 302' 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 8.5" **BASE**			
	B6-1			SM	**PREVIOUSLY PLACED FILL** Medium dense, moist, yellowish brown to brown, Silty, fine to medium SAND, trace gravel; trace concrete			
2								
4								
	B6-2					24	105.2	21.0
6				ML	Stiff, moist, mottled yellowish brown to brown and gray, Sandy SILT			
					-Encountered cemented zone from 7' to 8'; hard drilling due to rock			
8								
10	B6-3				-Becomes very stiff	49	112.8	17.5
				SM	Medium dense to dense, moist, tan brown to yellowish brown, Silty, fine to medium SAND; few clay; trace gravel			
12								
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

07850-42-15.GPJ

Figure A-6,
Log of Boring B 6, 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

APPENDIX B

APPENDIX B

LABORATORY TESTING

Laboratory tests were performed in accordance with generally accepted test methods of the American Society for Testing and Materials (ASTM) or other suggested procedures. Selected soil samples were tested for their: in-place 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

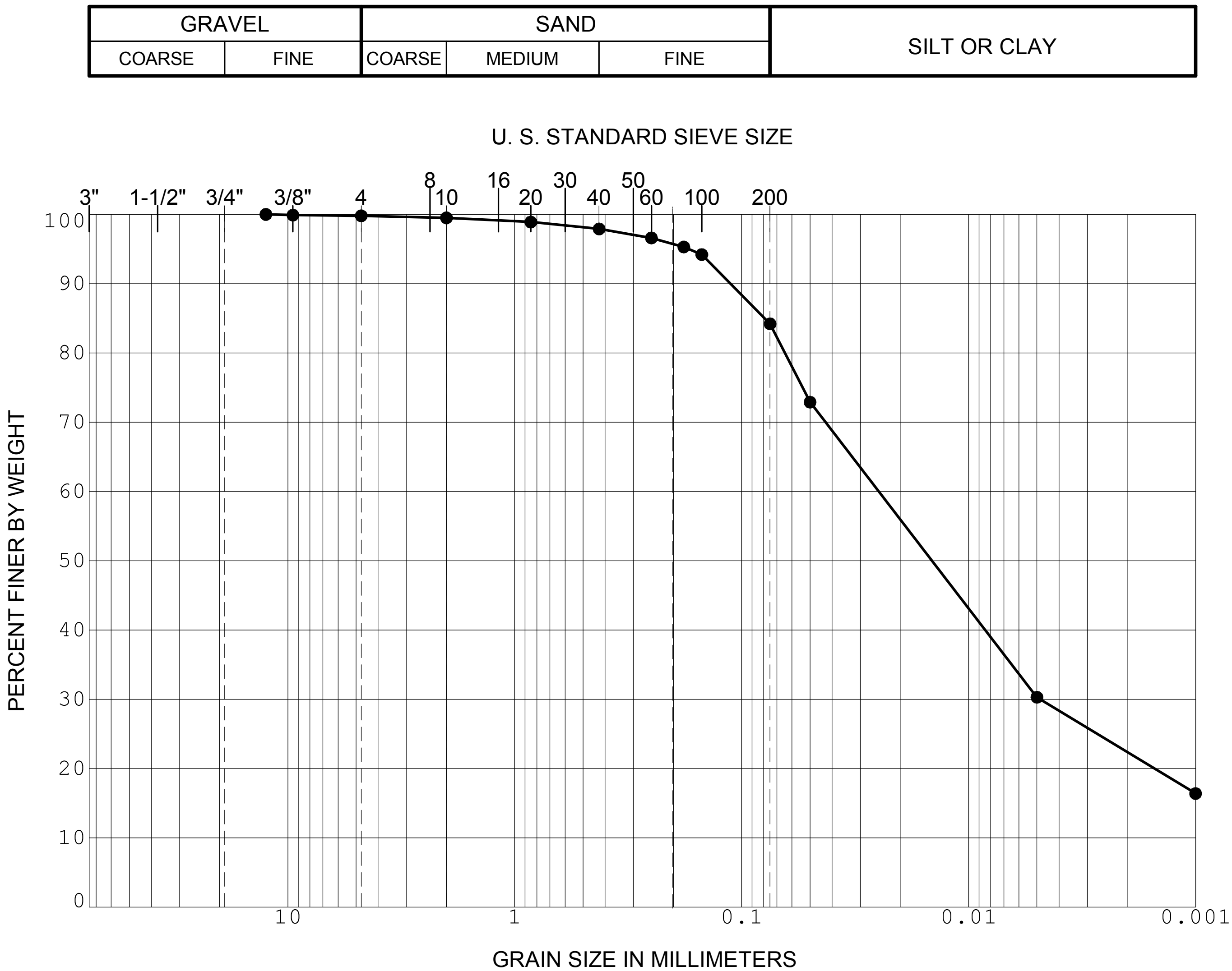
TABLE B-II
SUMMARY OF LABORATORY DIRECT SHEAR TEST RESULTS
ASTM D 3080

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

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

Sample No.	Water-Soluble Sulfate (%)	Classification
B1-1	0.015	Negligible (S0)
B4-2	0.025	Negligible (S0)

PROJECT NO. 07850-42-15



	SAMPLE	DEPTH (ft)	CLASSIFICATION	NAT WC	LL	PL	PI
●	B2-1	1.0	ML - SILT with Sand				
⊠							
▲							

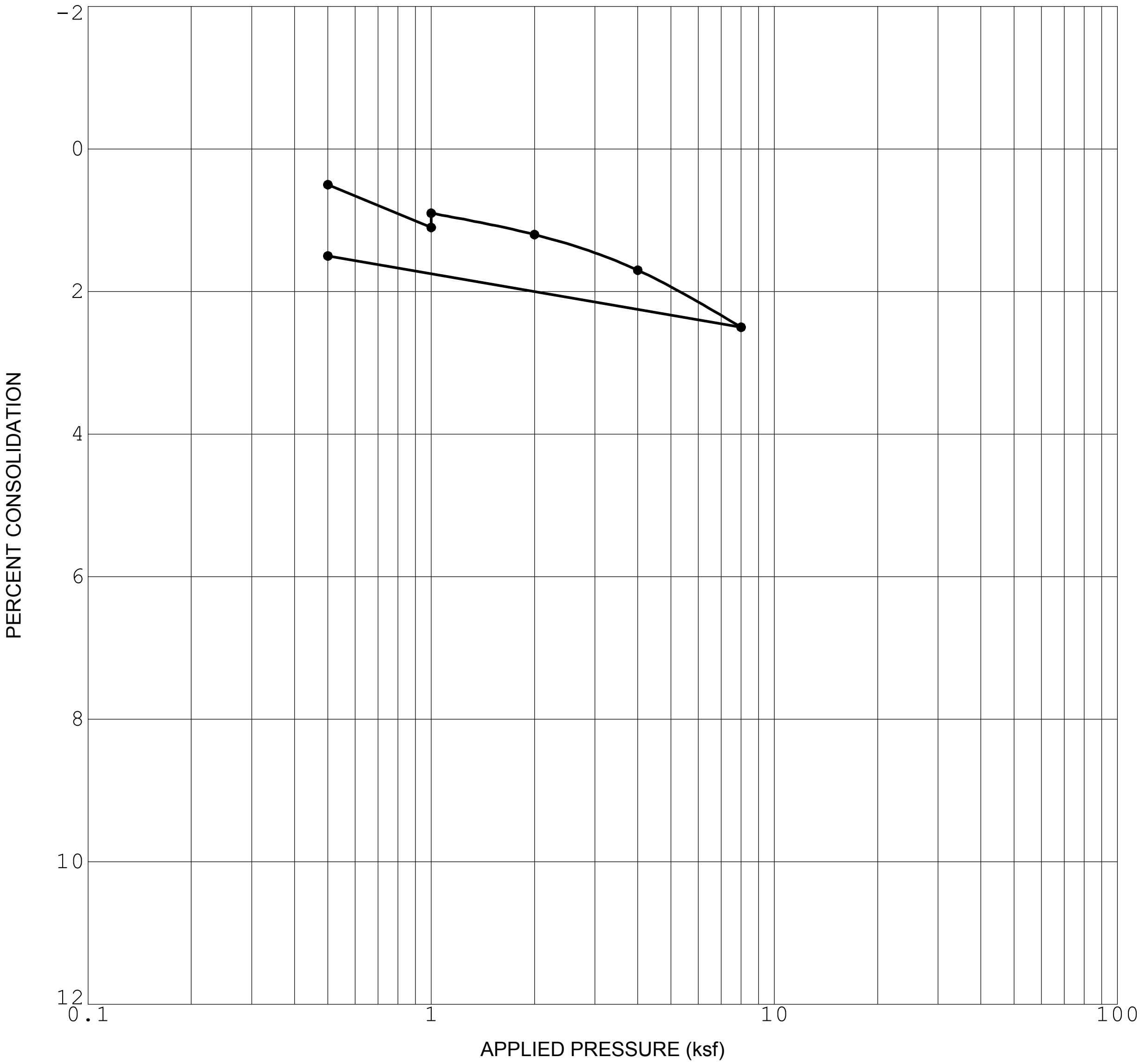
GRADATION CURVE

10290 CAMPUS POINT DRIVE

SAN DIEGO, CALIFORNIA

PROJECT NO. 07850-42-15

SAMPLE NO. B1-2



Initial Dry Density (pcf)	105.8
Initial Water Content (%)	20.5

Initial Saturation (%)	95.6
Sample Saturated at (ksf)	1.0

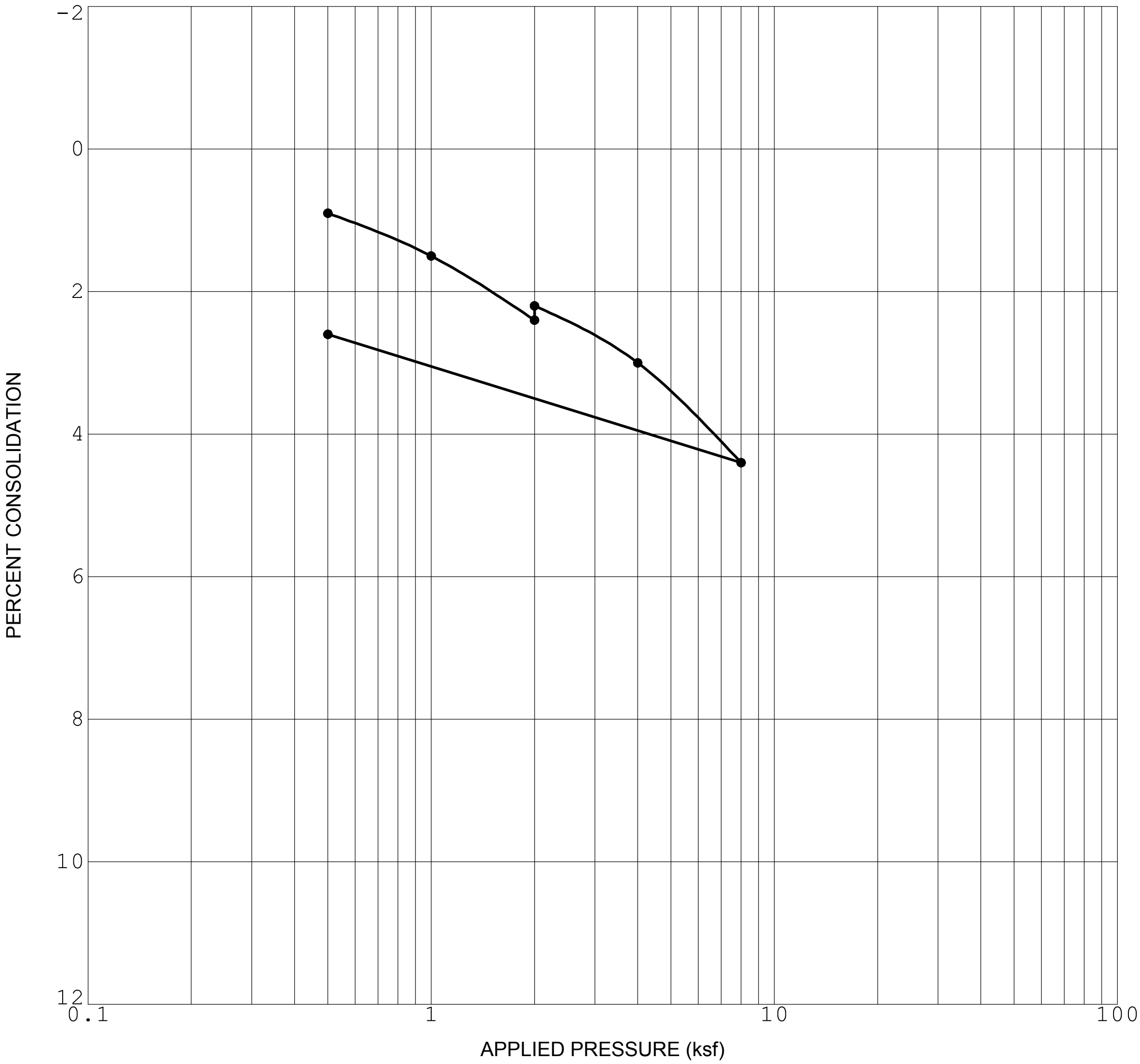
CONSOLIDATION CURVE

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

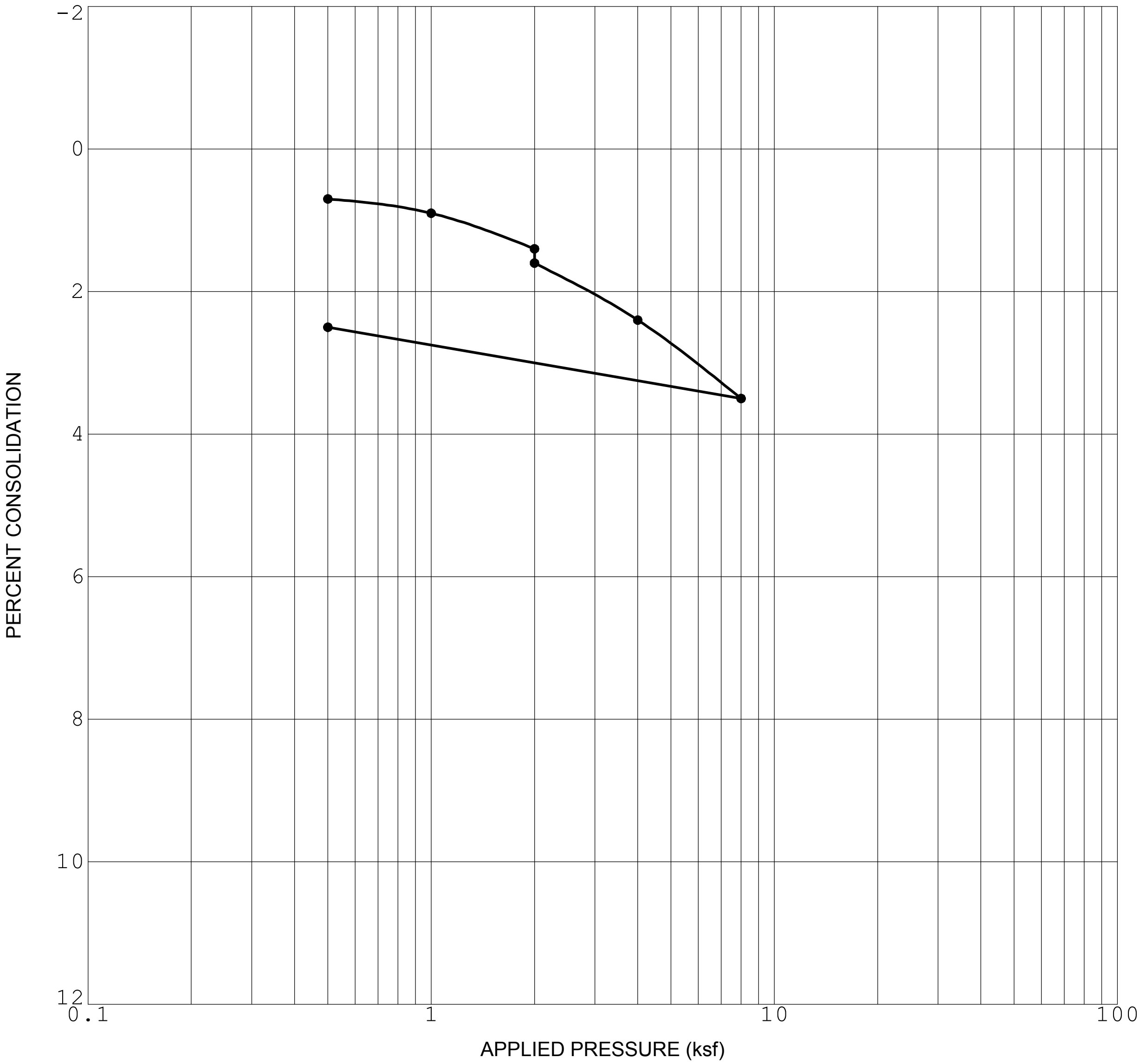
Initial Saturation (%)	99.5
Sample Saturated at (ksf)	2.0

CONSOLIDATION CURVE

10290 CAMPUS POINT DRIVE

SAN DIEGO, CALIFORNIA

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

APPENDIX C

APPENDIX C

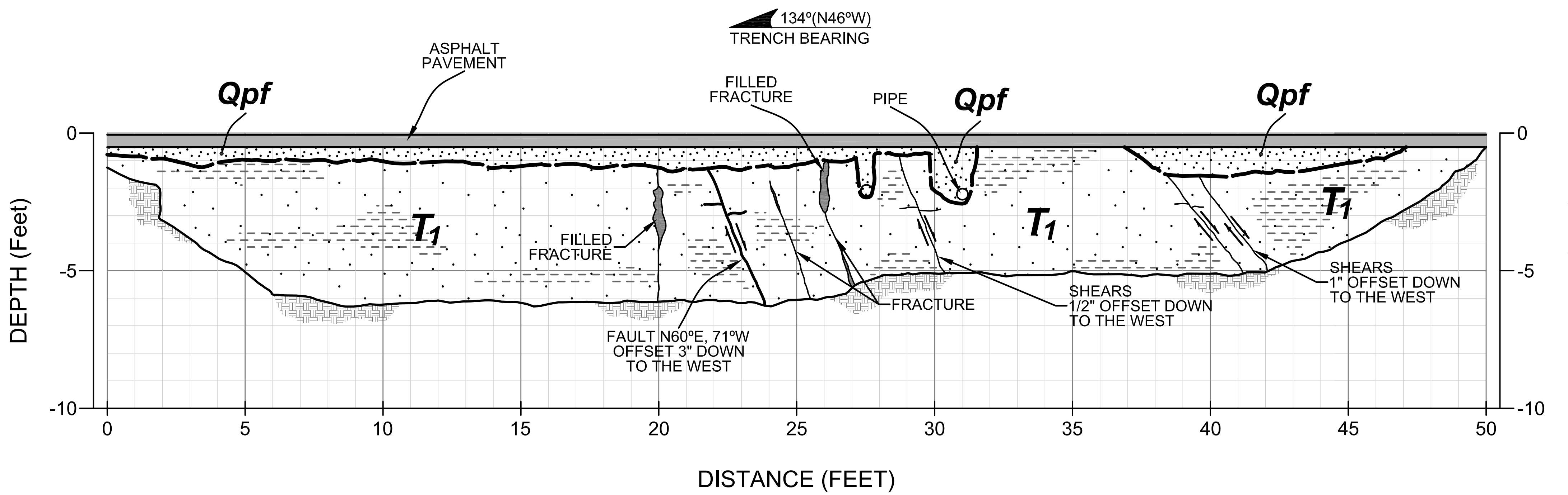
FAULT TRENCHES
PERFORMED BY GEOCON INCORPORATED AND SCS&T

FOR

10290 CAMPUS POINT DRIVE
SAN DIEGO, CALIFORNIA

PROJECT NO. 07850-42-15

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

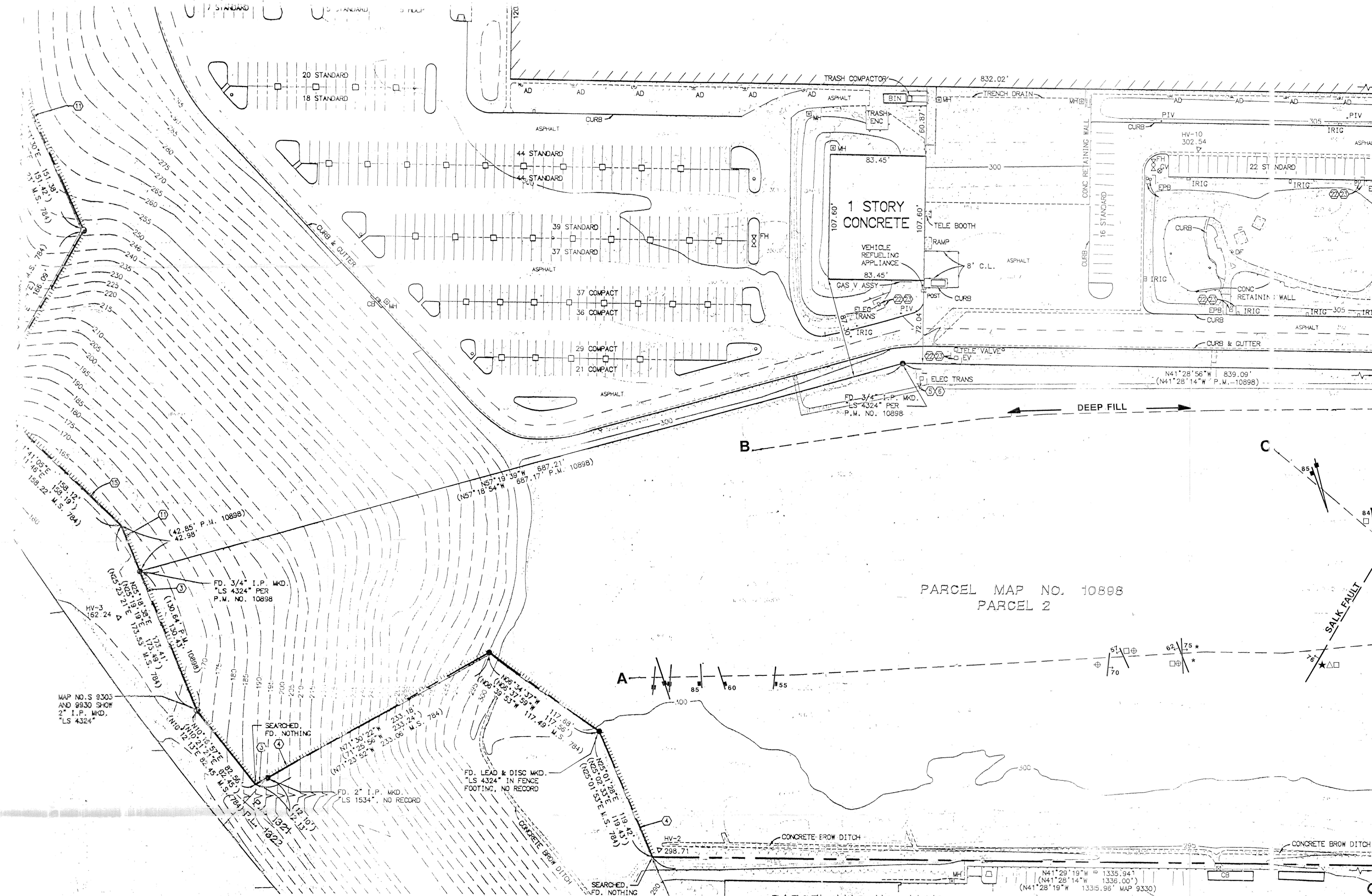
T_1 ....TERTIARY-AGE SCRIPPS FORMATION
Horizontally bedded, light gray and red brown banded, Clayey SANDSTONE

........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



JN: 501416
DRAWING: 1416ALT2

RBF/SHOLDERS & SANFORD
PROFESSIONAL ENGINEERS, PLANNERS & SURVEYORS
(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

SCALE 1" = 40'

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'

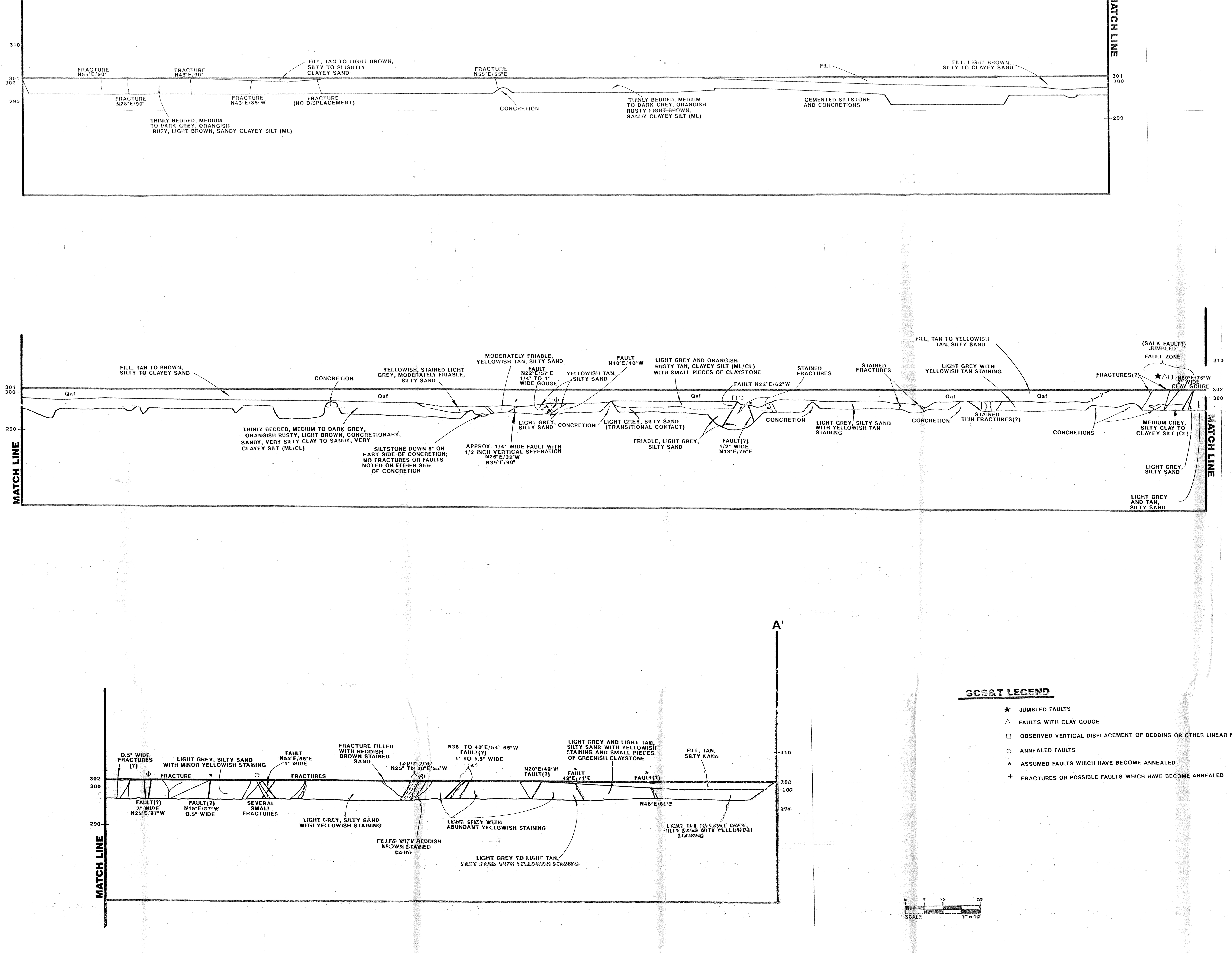
SOUTHERN CALIFORNIA SOIL & TESTING, INC.

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

SCALE 1" = 10'

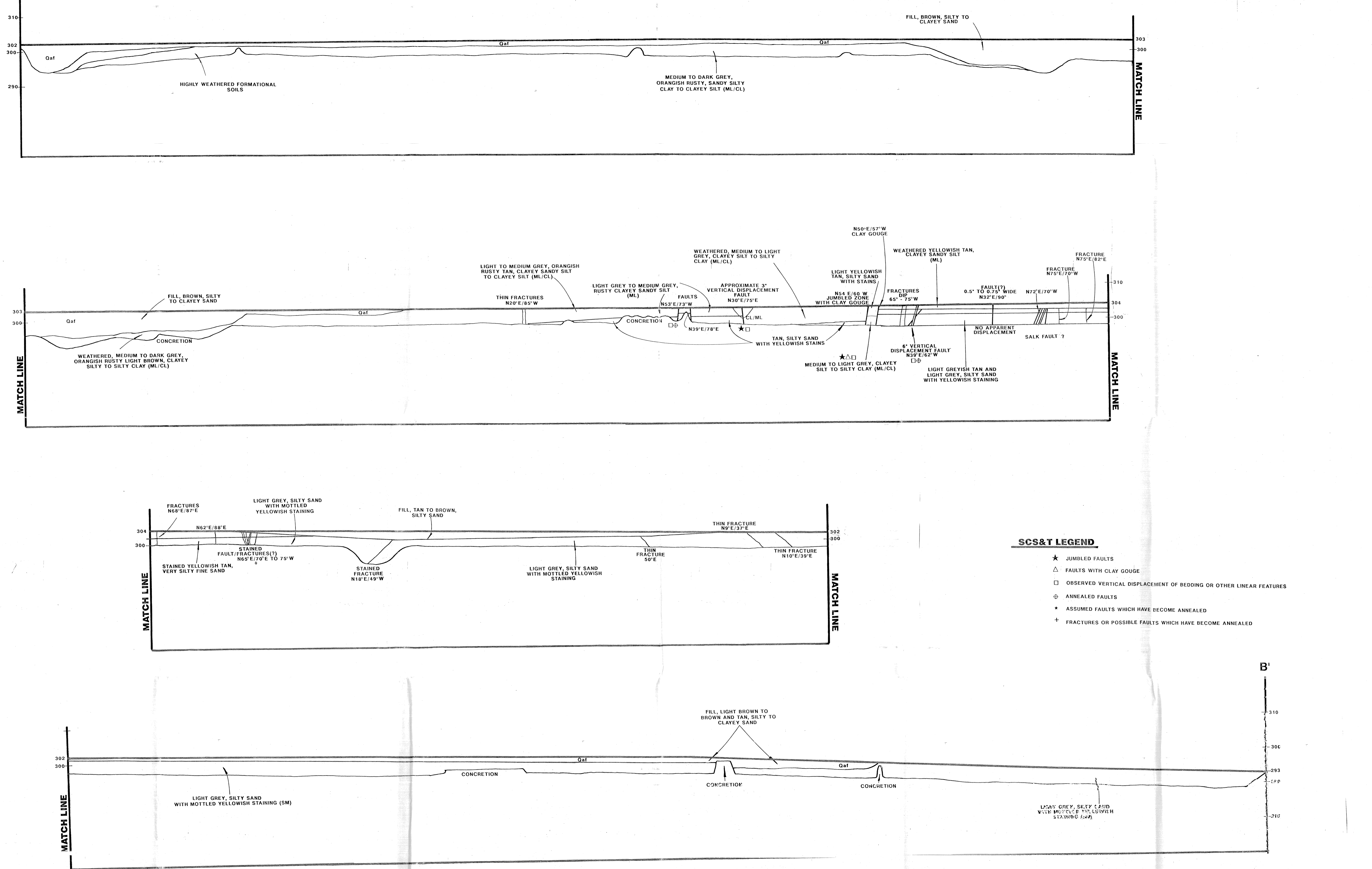
SOUTHERN CALIFORNIA SOIL & TESTING, INC.

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'

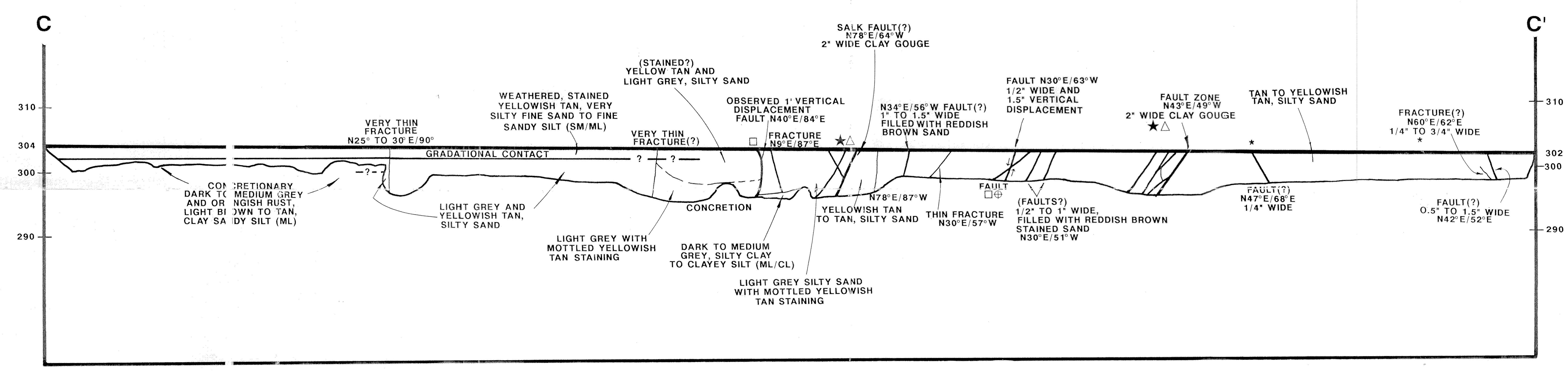
SOUTHERN CALIFORNIA SOIL & TESTING, INC.

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

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.: 4

APPENDIX D

APPENDIX D

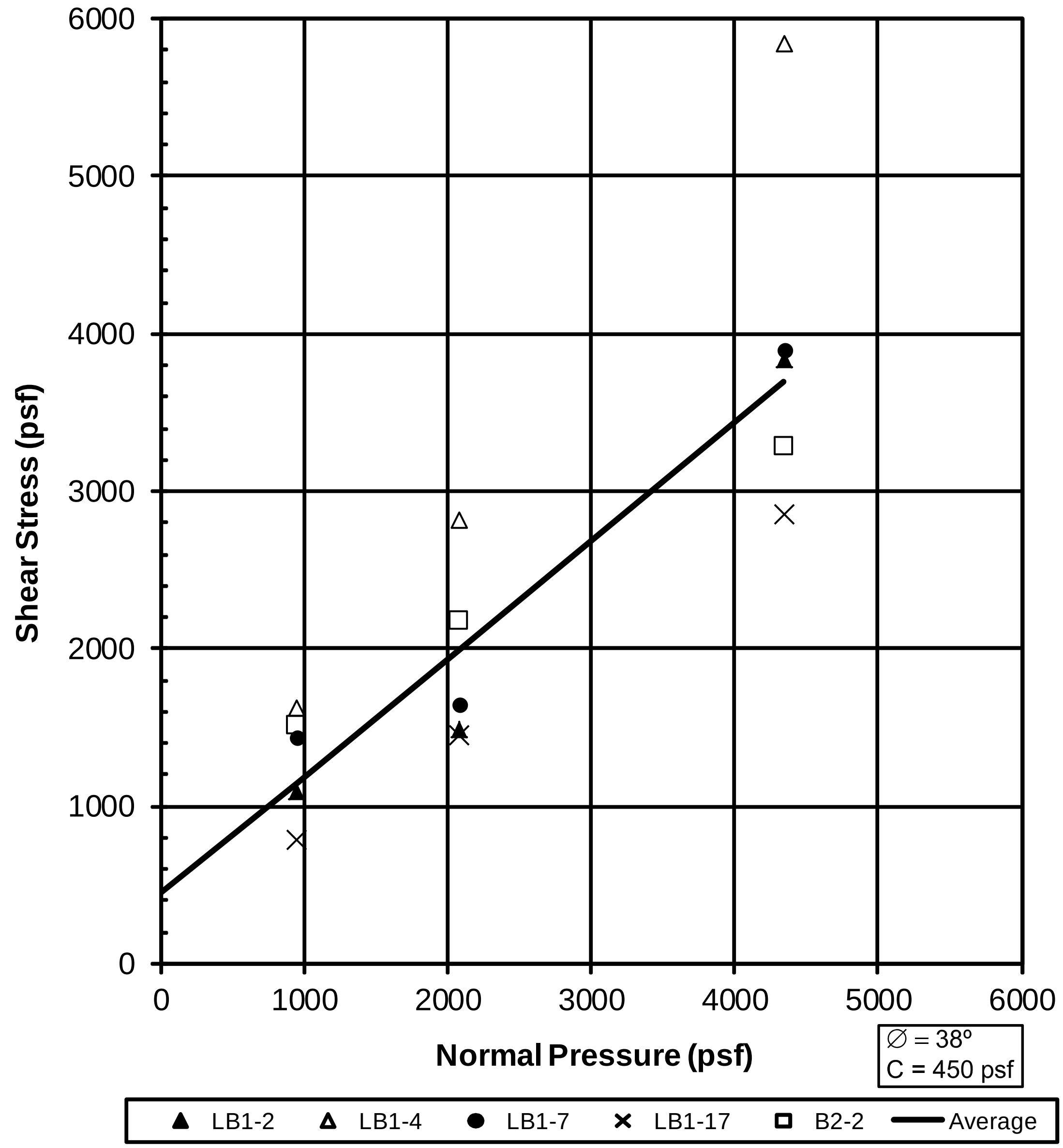
EXPLORATORY BORING AND LABORATORY TESTING PERFORMED PREVIOUSLY BY GEOCON AND OTHERS

FOR

10290 CAMPUS POINT DRIVE SAN DIEGO, CALIFORNIA

PROJECT NO. 07850-42-15

ARDATH (ULTIMATE)



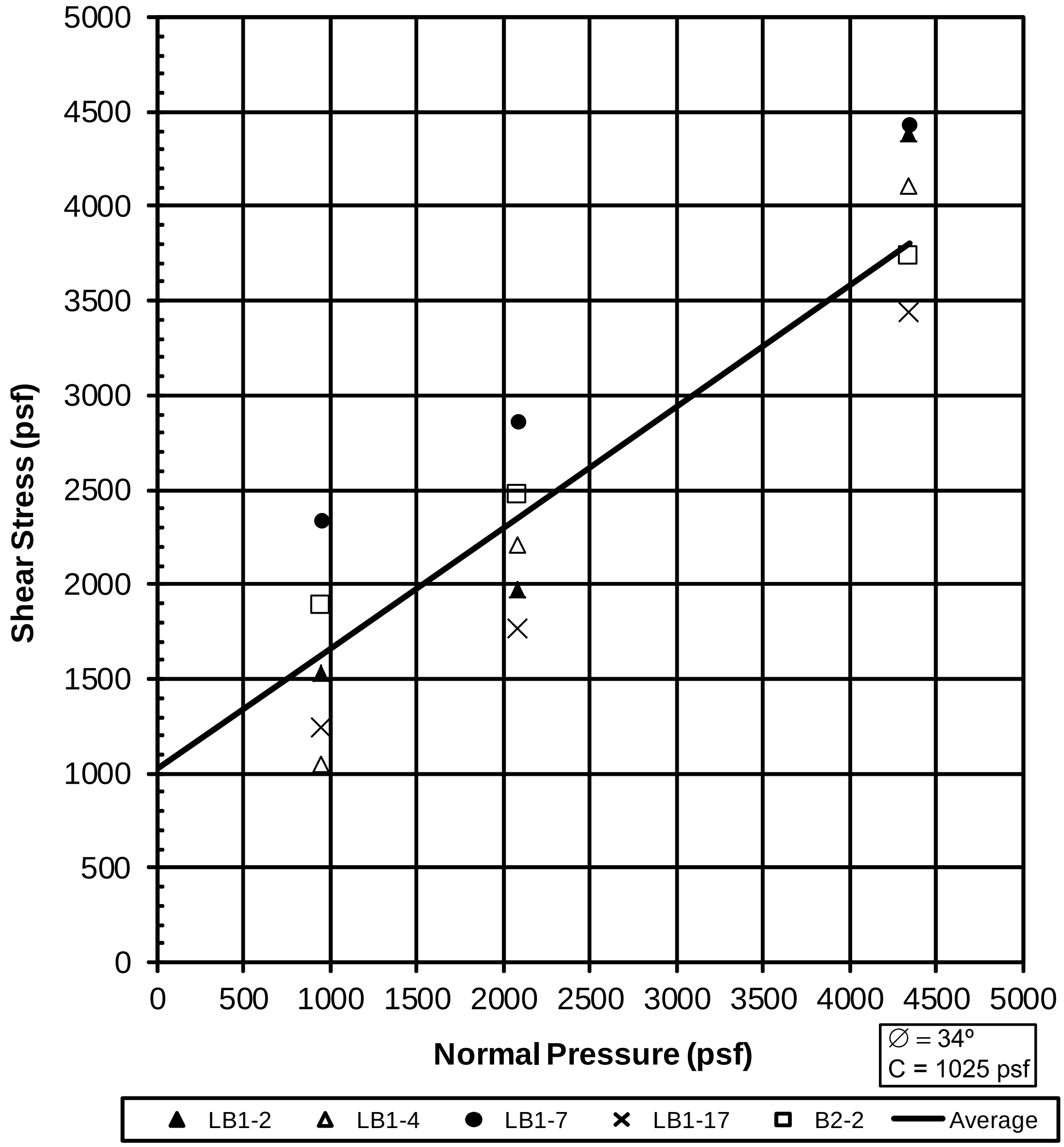
SHEAR STRENGTH TEST RESULTS (ULTIMATE VALUES)

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

CAMPUS POINT MULTI-LEVEL PARKING STRUCTURE
10300 CAMPUS POINT DRIVE
SAN DIEGO, CALIFORNIA

RA / RA		DSK/GTYPD	DATE 11 - 11 - 2014	PROJECT NO. 07850 - 42 - 14	FIG. C-4

ARDATH (PEAK)



SHEAR STRENGTH TEST RESULTS (PEAK VALUES)

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

CAMPUS POINT MULTI-LEVEL PARKING STRUCTURE
10300 CAMPUS POINT DRIVE
SAN DIEGO, CALIFORNIA

RA / RA		DSK/GTYPD	DATE 11 - 11 - 2014	PROJECT NO. 07850 - 42 - 14	FIG. C-5

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									
	US	SM	Tan to Reddish Tan, SILTY SAND	Moist	Dense	35	106.1	17.9	
16									
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				
	US					50/5"	96.8	9.0	
26									
28			Light Grey						
	US		Bottom at 30.5 Feet			50/5"			
30									

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,	Humid	Loose				
2	BAG		SILTY SAND	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

DEPTH (ft.)	SAMPLE TYPE	SOIL CLASSIFICATION	BORING NUMBER 3 ELEVATION 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									
6	US	ML-SM	Yellow Tan to Light Grey, VERY SANDY SILT	Moist	Very Dense	50/6"	102.9	8.8	
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									
16	US	SM	SILTY SAND	Moist	Very Dense	50/5"	93.2	8.1	
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

DEPTH (ft.)	SAMPLE TYPE	SOIL CLASSIFICATION	BORING NUMBER 5 ELEVATION 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

DEPTH (ft.)	SAMPLE TYPE	SOIL CLASSIFICATION	BORING NUMBER 6 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, Yellow Tan, SILTY SAND	Humid	Loose				
2	US			Moist	Dense	50/4"	105.0	7.3	
4									
6	US	SM				50/6"	96.6	10.3	
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

DEPTH (ft.)	SAMPLE TYPE	SOIL CLASSIFICATION	BORING NUMBER 7 ELEVATION 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						
0		SM	SCRIPPS FORMATION, Light Tan to Yellow Tan, SILTY SAND	Humid Moist	Loose Dense				
2									
4									
6			Bottom at 5 Feet						
			BORING NUMBER 9						
0		SM	FILL, Tan to Yellow Tan, SILTY SAND	Humid Moist	Loose Dense				
2									
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

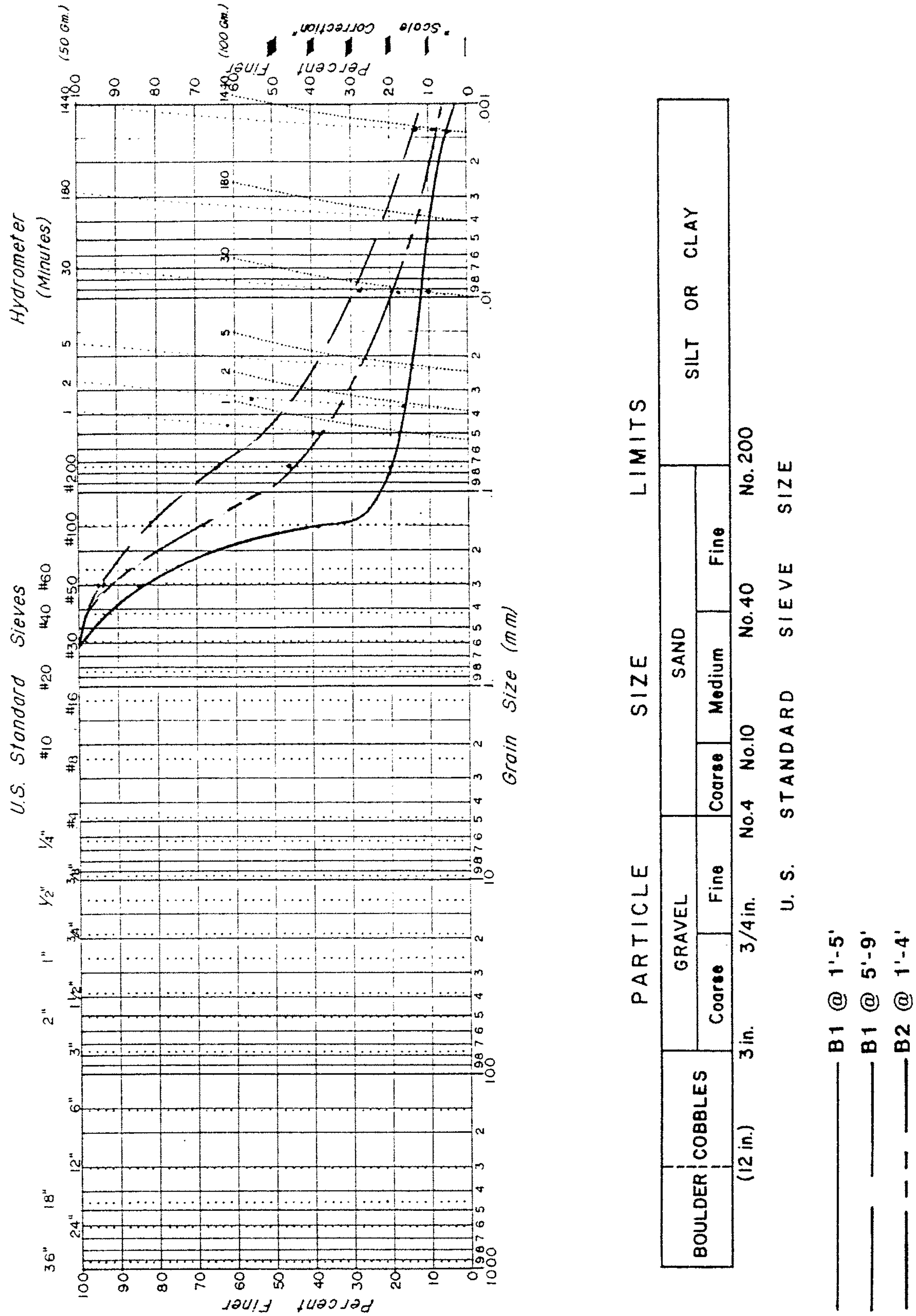
DEPTH (ft.)	SAMPLE TYPE	SOIL CLASSIFICATION	BORING NUMBER 10 ELEVATION DESCRIPTION	APPARENT MOISTURE	APPARENT CONSISTENCY OR DENSITY	PENETRATION RESISTANCE (blows/ft. of drive)	DRY DENSITY (pcf)	MOISTURE CONTENT (%)	RELATIVE COMPACTION (%)
0		SM	FILL, Tan to Yellow Tan, SILTY SAND	Humid	Loose				
2				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

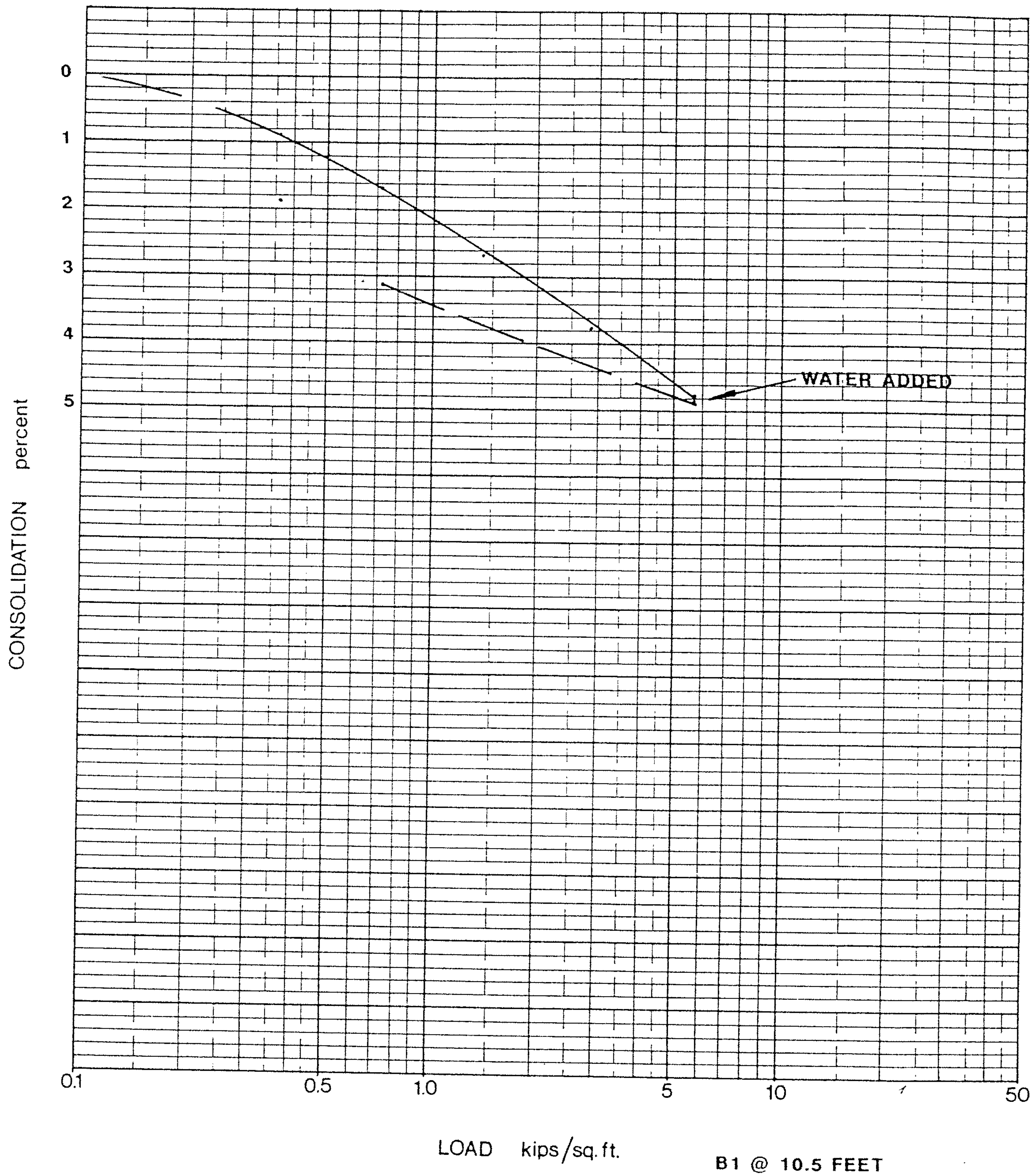
JOB NUMBER: 9511205 | Plate No. 10



SOUTHERN CALIFORNIA SOIL & TESTING, INC.

QUALCOMM/IVAC

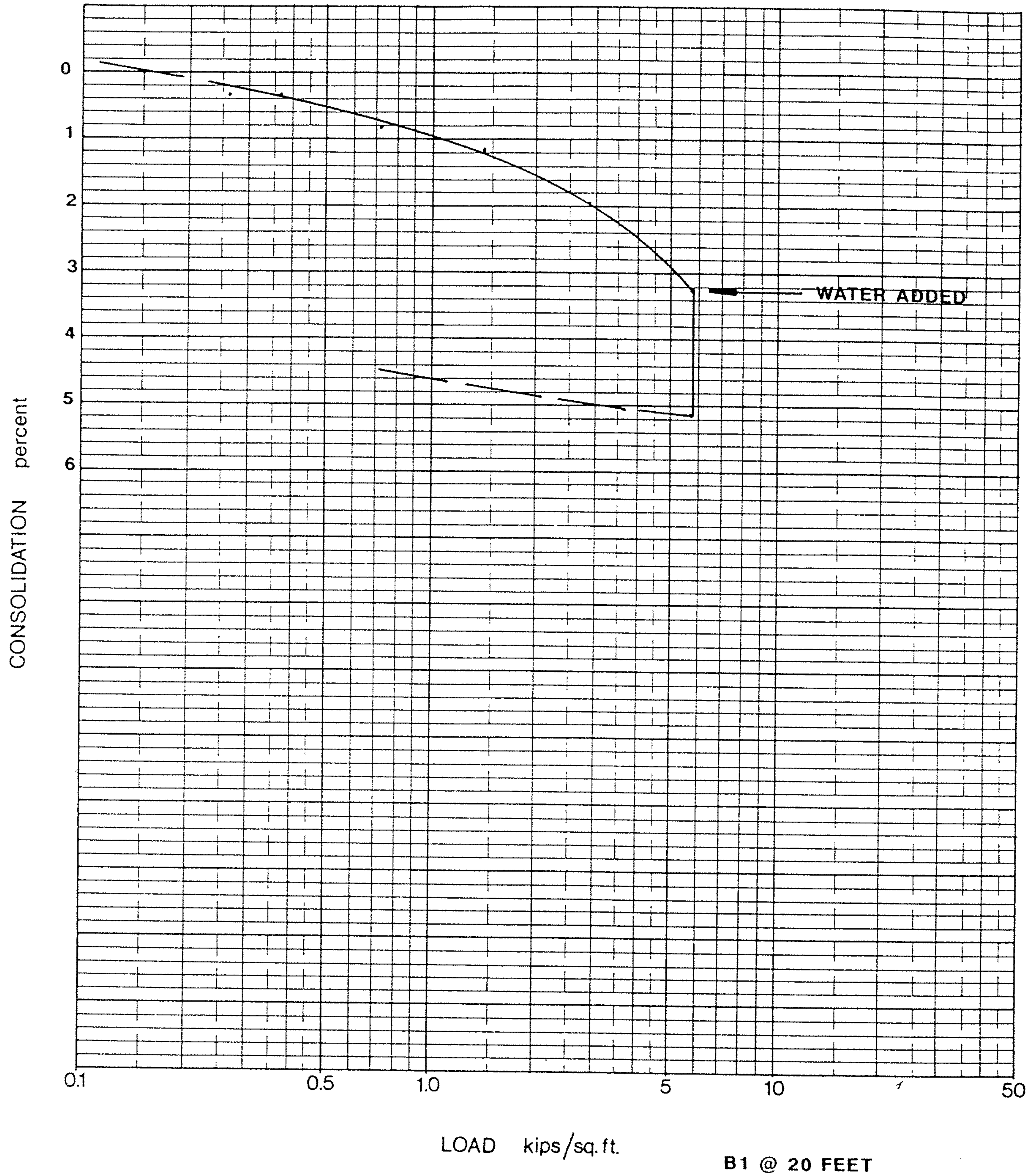
GRAIN SIZE DISTRIBUTION	BY CHC	DATE 10-13-95
	JOB NO. 9511205	Plate No. 11



B1 @ 10.5 FEET

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

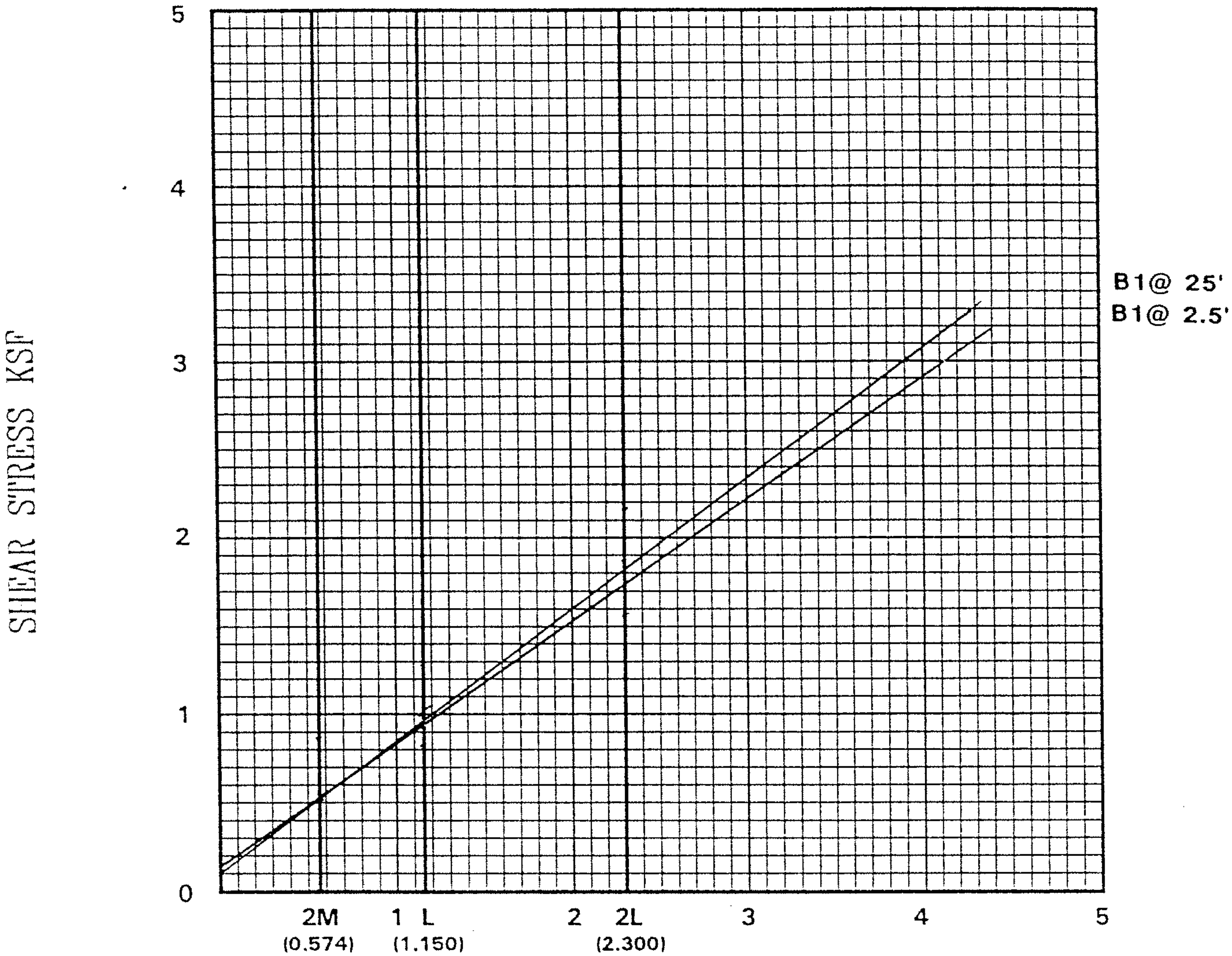
QUALCOMM/IVAC	
BY CHC	DATE 10-13-95
JOB NO. 9511205	Plate No. 12



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



NORMAL STRESS, KSF (2 ⅜" SAMPLE)

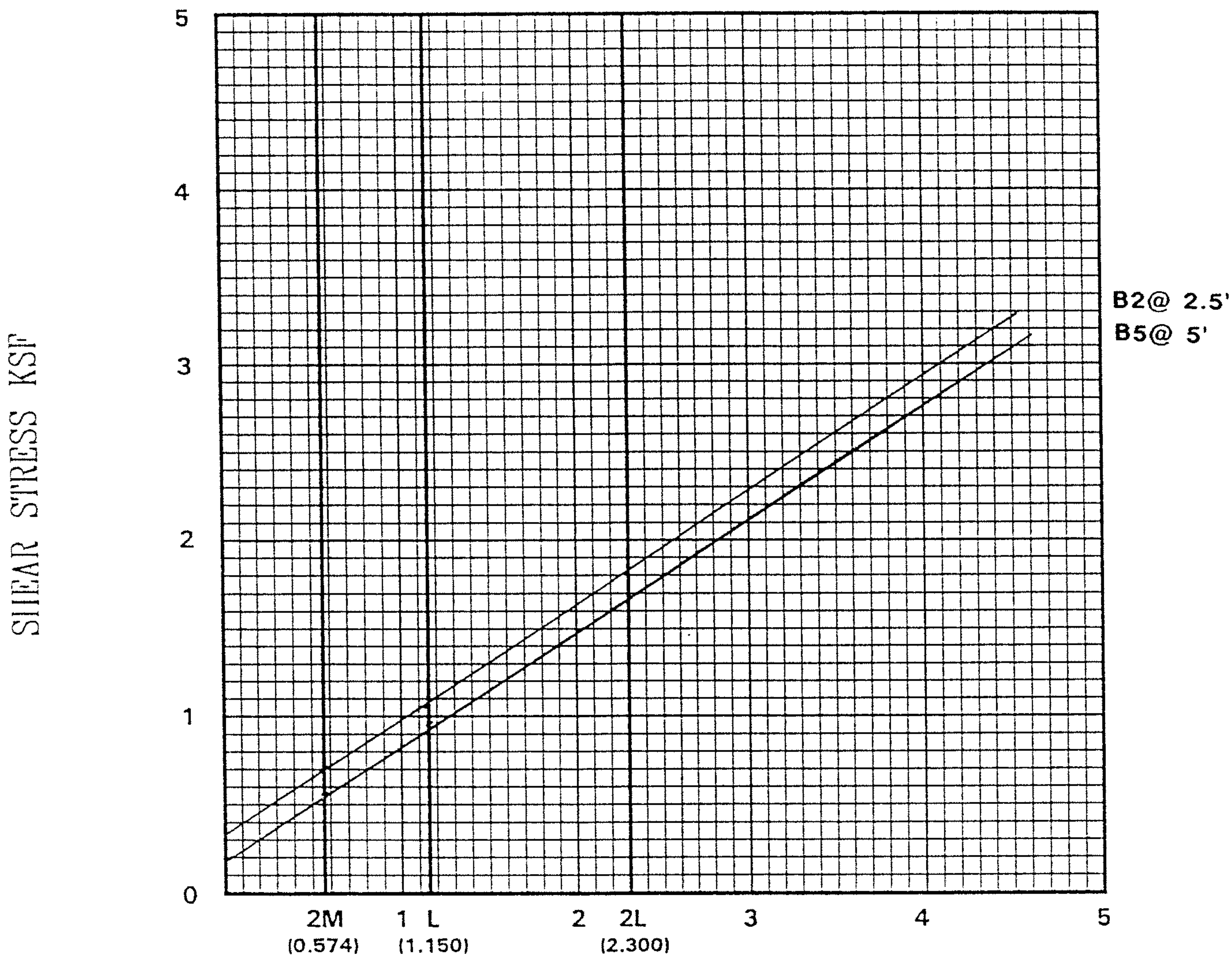
SAMPLE	DESCRIPTION	ANGLE OF INTERNAL FRICTION	COHESION INTERCEPT (PSF)
B1 @ 2.5'	Undisturbed	35 Degrees	150 psf
B1 @ 25'	Undisturbed	37 Degrees	100 psf

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



NORMAL STRESS, KSF (2 ⅜" SAMPLE)

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

APPENDIX E

APPENDIX E

RECOMMENDED GRADING SPECIFICATIONS

FOR

10290 CAMPUS POINT DRIVE
SAN DIEGO, CALIFORNIA

PROJECT NO. 07850-42-15

RECOMMENDED GRADING SPECIFICATIONS

1. GENERAL

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

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

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

2. DEFINITIONS

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

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

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

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

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

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

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

3. MATERIALS

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

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

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

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

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

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

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

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

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

4. CLEARING AND PREPARING AREAS TO BE FILLED

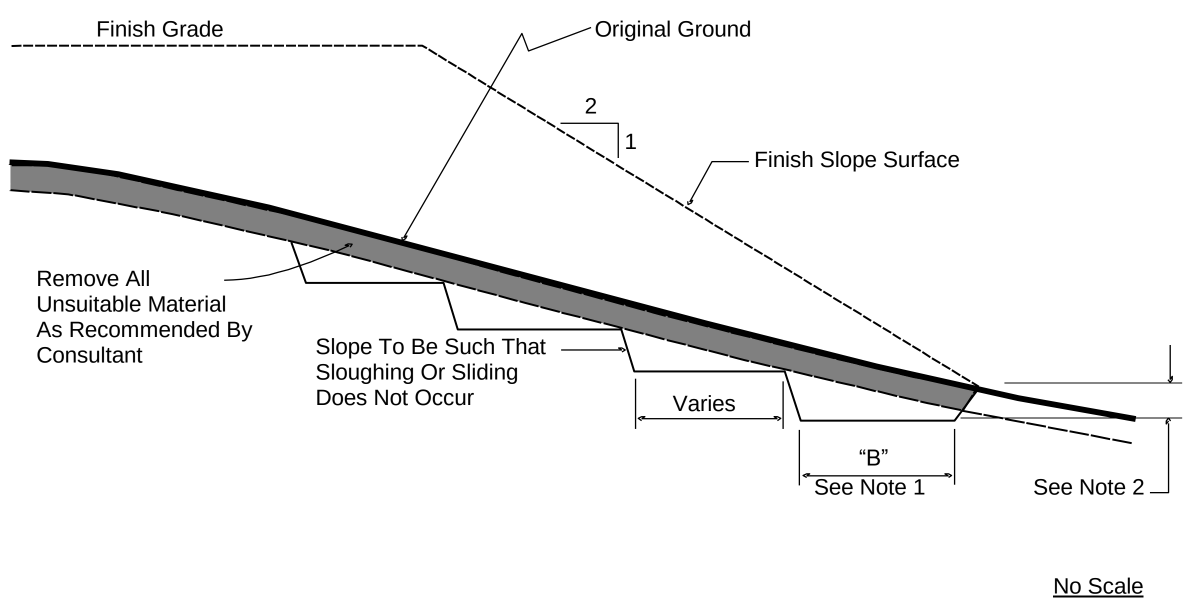
4.1 Areas to be excavated and filled shall be cleared and grubbed. Clearing shall consist of complete removal above the ground surface of trees, stumps, brush, vegetation, man-made structures, and similar debris. Grubbing shall consist of removal of stumps, roots, buried logs and other unsuitable material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

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

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

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

TYPICAL BENCHING DETAIL



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

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

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

5. COMPACTION EQUIPMENT

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

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

6. PLACING, SPREADING AND COMPACTION OF FILL MATERIAL

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

7. OBSERVATION AND TESTING

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

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

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

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

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

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

7.6.1 Soil and Soil-Rock Fills:

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

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

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

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

7.6.2 Rock Fills

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

8. PROTECTION OF WORK

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

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

9. CERTIFICATIONS AND FINAL REPORTS

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

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

LIST OF REFERENCES

Anderson, J. G., *Synthesis of Seismicity and Geological Data in California*, U.S. Geological Survey Open-file Report 84-424, 1984, pp. 1-186;

Boore, D. M., and G. M Atkinson (2006), *Boore-Atkinson NGA Ground Motion Relations for the Geometric Mean Horizontal Component of Peak and Spectral Ground Motion Parameters*, Report Number PEER 2007/01, May 2007;

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

California Geological Survey, (2008), *Geologic Map of the San Diego 30' x 60' Quadrangle, California*, Regional Geologic Map No. 3;

City of San Diego Development Services Department, (2008), *City of San Diego, Seismic Safety Study, Geologic Hazards and Faults*, Grid Tile: 34;

Geocon Incorporated, *Preliminary Fault Study, 10290 Campus Point Drive, San Diego, California*, dated May 27, 2015 (Project No. 07850-42-15);

Geocon Incorporated, *Update Geotechnical Investigation, Campus Point Multi-Level Parking Structure, 10300 Campus Point Drive, San Diego, California*, dated November 11, 2014 (Project No. 07850-42-14);

Geocon Incorporated, *Geotechnical and Geologic Fault Investigation, Campus Pointe Master Plan, 10300 Campus Point Drive, San Diego, California*, dated February 3, 2014 (Project No. 07850-42-11);

Geocon Incorporated, *Due Diligence Review of Geotechnical Reports, Qualcomm Building A, 10290 Campus Point Drive, San Diego, California*, dated February 15,2011 (Project No. 07850-42-05);

Jennings, C. W., *Fault Activity Map of California And Adjacent Areas with Locations and Ages of Recent Volcanic Eruptions*, California Geological Survey, formerly California Division of Mines and Geology, 1994;

Kennedy, M. P. and S. S Tan, *Geologic Map of the San Diego 30'x60' Quadrangle,, California*, California Geologic Survey, 2008;

Ploessel, M. R. and J. E. Slosson, *Repeatable High Ground Accelerations From Earthquakes*, California Geology, September 1974;

Risk Engineering, *EZ-FRISK*, 2008;

SCS&T, *Report of Preliminary Geotechnical Investigation for Qualcomm Office Building Eli Lillie Property, Campus Point Drive, San Diego, California,* dated October 13, 1995 (Report No. 1, Job No. 9511205);

LIST OF REFERENCES (Concluded)

SCS&T, *Report of Fault Investigation for Qualcomm Office Building Eli Lillie Property, Campus Point Drive, San Diego, California*, dated December 1, 1995 (Report No. 2, Job No. 9511205);

SCS&T, *Update Report for Qualcomm/Ivac Building Modifications, Campus Point Drive, San Diego, California*, dated January 30, 1996 (Report No. 2, Job No. 9611006);

SCS&T, *Report of Geotechnical Investigation for Proposed Qualcomm, Campus Point Facility Improvements, San Diego, California*, prepared by Southern California Soil and Testing Incorporated, dated December 29, 1997 (Job No. 9711277.1);

Unpublished reports and maps on file with Geocon Incorporated;

United States Department of Agriculture, *1953 Stereoscopic Aerial Photographs;*

USGS computer program, *2002 Interactive Deaggregation,* http://eqint.cr.usgs.gov/deaggint/2002/index.php;

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

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

APPENDIX F-2

Addendum to Storm Water Management Recommendations

Project No. 07850-42-15
September 20, 2016

Alexandria Real Estate Equities, Inc.
10996 Torreyana Road, Suite 250
San Diego, California 92122

Attention: Mr. Michael Barbera

Subject: ADDENDUM TO STORM WATER MANAGEMENT RECOMMENDATIONS
CAMPUS POINT BOULEVARD
10290 CAMPUS POINT DRIVE
SAN DIEGO, CALIFORNIA

References:

1. *Storm Water Management Recommendations, 10290 Campus Point Drive, San Diego, California*, prepared by Geocon Incorporated, dated June 7, 2016 (Project No. 07850-42-15).

2. *Response to Geotechnical Review Comments, 10290 Campus Pointe Drive, San Diego, California*, dated August 5, 2016, prepared by Geocon Incorporated (Project No. 07850-42-15).

3. *Response to Geotechnical Review Comments, 10290 Campus Pointe Drive, San Diego, California*, dated August 11, 2016, prepared by Geocon Incorporated (Project No. 07850-42-15).

4. *Response to Geotechnical Review Comments, 10290 Campus Pointe Drive, San Diego, California*, dated August 22, 2016, prepared by Geocon Incorporated (Project No. 07850-42-15).

5. *Preliminary Geotechnical Investigation, 10290 Campus Pointe Drive San Diego, California*, dated June 11, 2015, prepared by Geocon Incorporated (Project No. 07850-42-15).

6. *Second Addendum to Geotechnical Investigation, 10290 Campus Point Drive, San Diego, California*, prepared by Geocon Incorporated, dated March 15, 2016 (Project No. 07850-42-15).

Dear Mr. Barbera:

We have prepared this addendum letter with respect to storm water management recommendations for the subject site. Recommendations for storm water management are provided in Reference 1 and in the response letters to City review comments (References 2 through 4). As required by the City of San Diego, we have performed additional infiltration tests within the bottom of the basin excavation. Based on the test results, it is our opinion that the recommendations contained in the previous correspondence remain applicable. Full infiltration is considered infeasible; however, the site is considered feasible for partial infiltration provided design measures are taken to ensure seepage water from the basin does not impact the proposed adjacent below grade retaining walls and structures.

In-Situ Testing

We performed 2 field-saturated, hydraulic conductivity tests at depths of approximately 16 inches below the basin bottom using a Soil Moisture Corp Aardvark Permeameter. Table 1 presents the results of the infiltration test. The Aardvark Permeameter test data is attached.

TABLE 1
UNFACTORED, FIELD-SATURATED, HYDRAULIC CONDUCTIVITY TEST RESULTS USING THE SOILMOISTURE CORP AARDVARK PERMEAMETER

Location	Depth (inches)	Geologic Unit	Field Infiltration Rate, I (inches/hour)	Field Saturated Hydraulic Conductivity, K (inches/hour)
A-1	17	Ardath/Scripps Formation	0.08	0.05
A-2	16	Ardath/Scripps Formation	0.22	0.12

We also performed three excavation percolation tests at depths between 17 and 24 inches below the basin bottom. Table 2 presents the calculated infiltration rates.

TABLE 2
UNFACTORED INFILTRATION TEST RESULTS FROM EXCAVATION PERCOLATION TEST PITS

Test No.	Depth (inches)	Geologic Unit	Infiltration Rate, I (inches/hour)
P-1	17	Ardath/Scripps Formation	1.08
P-2	24	Ardath/Scripps Formation	0.09
P-3	19	Ardath/Scripps Formation	0.42

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

STORM WATER MANAGEMENT CONCLUSIONS

Infiltration Rates

The results of the testing show 4 of the 5 infiltration tests had rates less than 0.5 inches per hour. Boring logs and the geologic history of the bedrock units show the on-site soils are highly variable. It is our opinion that there is a high probability for lateral water migration because of variable soil conditions and interlayered siltstone and claystone beds within the formational bedrock units. Therefore, based on the results of the field infiltration tests, full infiltration is considered infeasible because of the varying infiltration rates and potential for lateral water migration and ground water mounding. However, partial infiltration is considered feasible provided precautions are taken to reduce impacts to adjacent below grade retaining walls and structures.

Storm Water Standard Worksheets

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

The regional storm water standards also have a worksheet (Worksheet D.5-1 or Form I-9) that helps the project civil engineer estimate the factor of safety based on several factors. Table 3 describes the suitability assessment input parameters related to the geotechnical engineering aspects for the factor of safety determination.

TABLE 3
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.

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

Table 4 presents the estimated factor values for the evaluation of the factor of safety. The factor of safety is determined using the information contained in Table 3 and the results of our geotechnical investigation. Table 4 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 of Worksheet D.5-1) and use the combined safety factor for the design infiltration rate.

TABLE 4
FACTOR OF SAFETY WORKSHEET D.5-1 DESIGN VALUES – PART A[1]

Suitability Assessment Factor Category	Assigned Weight (w)	Factor Value (v)	Product (p = w x v)
Assessment Methods	0.25	2	0.5
Predominant Soil Texture	0.25	2	0.5
Site Soil Variability	0.25	3	0.75
Depth to Groundwater/Impervious Layer	0.25	1	0.25
Suitability Assessment Safety Factor, $S_A = \Sigma p$			2

1 The project civil engineer should complete Worksheet D.5-1 or Form I-9 to determine the overall factor of safety.

CONCLUSIONS

Our results indicate the site has highly variable sub-surface permeability conditions and infiltration characteristics. Because of these site conditions, it is our opinion that there is a high probability for lateral water migration and in our opinion full infiltration is infeasible on this site. However, partial infiltration is considered feasible. Side liners should be installed to reduce the potential for lateral migration of seepage within the basin area.

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

Very truly yours,

GEOCON INCORPORATED

Rodney C. Mikesell
GE 2533

RCM:dmc

Attachments: Figure 1
Worksheet C.4-1
Aardvark Permeameter Data Analysis
Boring Logs

(e-mail) Addressee
(e-mail) Gensler
Attention: Mr. Steve Schrader
(e-mail) Michael Baker International
Attention: Mr. Brian Oliver

Categorization of Infiltration Feasibility Condition			Worksheet C.4-1
Part 1 - Full Infiltration Feasibility Screening Criteria **Would infiltration of the full design volume be feasible from a physical perspective without any undesirable consequences that cannot be reasonably mitigated?**			
Criteria	Screening Question	Yes	No
1	**Is the estimated reliable infiltration rate below proposed facility locations greater than 0.5 inches per hour?** The response to this Screening Question shall be based on a comprehensive evaluation of the factors presented in Appendix C.2 and Appendix D.		X
Provide basis: The infiltration test results were as follows: A-1: 0.08 in/hr A-2: 0.22 in/hr P-1: 1.08 in/hr P-2: 0.09 in/hr P-3: 0.42 in/hr Four of the five tests indicated test results less than 0.5 inches per hour. This shows the soil is variable and a reliable design infiltration rate below proposed facility locations is not greater than 0.5 inches/hour. Additionally, based on the USGS Soil Survey, 100 percent of the site consists of a unit that possess a Hydrologic Soil Group D classification with an estimated k_{SAT} of 0.10 to 1.3 inches per hour.			
2	**Can infiltration greater than 0.5 inches per hour be allowed without increasing risk of geotechnical hazards (slope stability, groundwater mounding, utilities, or other factors) that cannot be mitigated to an acceptable level?** The response to this Screening Question shall be based on a comprehensive evaluation of the factors presented in Appendix C.2.		X
Provide basis: The specific geologic or geotechnical hazard for this site is the potential for groundwater mounding and lateral migration of infiltration water. The area of the proposed basin is underlain by dense formational soils of the Scripps Formation and Ardath Formation (see Geocon report dated June 11, 2015 and March 15, 2016). Four of the five tests performed at the bottom of the basin have a factored infiltration rate less than 0.5 iph. The variability observed in these test results is a reflection of the heterogeneous, anisotropic nature of the site hydrological properties. Since the site geology is composed of interbedded sandstone and siltstone/claystone (as geotechnical borings performed show) we expect that infiltration of storm water will be carried by the more permeable sandstone layers and occluded by the siltstone/claystone layers; therefore, the site is highly prone to groundwater mounding beneath basins and lateral migration of infiltrated groundwater. Therefore, it is our opinion that the site is not feasible for full infiltration. Due to the layering of the soils as is evident on the boring longs in the referenced reports, we are not aware of any reasonable mitigation methods that could be performed to mitigate the geologic conditions to an acceptable level where groundwater mounding and lateral migration will not occur under full infiltration conditions.			

Worksheet C.4-1 Page 2 of 4			
Criteria	Screening Question	**Yes**	**No**
3	**Can infiltration greater than 0.5 inches per hour be allowed without increasing risk of groundwater contamination (shallow water table, storm water pollutants or other factors) that cannot be mitigated to an acceptable level?** The response to this Screening Question shall be based on a comprehensive evaluation of the factors presented in Appendix C.3.	X	
Provide basis: Groundwater is expected to be deeper than 100 feet. Summarize findings of studies; provide reference to studies, calculations, maps, data sources, etc. Provide narrative discussion of study/data source applicability.			
4	**Can infiltration greater than 0.5 inches per hour be allowed without causing potential water balance issues such as change of seasonality of ephemeral streams or increased discharge of contaminated groundwater to surface waters?** The response to this Screening Question shall be based on a comprehensive evaluation of the factors presented in Appendix C.3.	X	
Provide basis: There are no known contaminants at the site and groundwater is in excess of 20 feet below the bottom of the basin. Response provided by Michael Baker International, the project's civil engineer. Summarize findings of studies; provide reference to studies, calculations, maps, data sources, etc. Provide narrative discussion of study/data source applicability.			
Part 1 Result*	If all answers to rows 1 - 4 are "**Yes**" a full infiltration design is potentially feasible. The feasibility screening category is **Full Infiltration** If any answer from row 1-4 is "**No**", infiltration may be possible to some extent but would not generally be feasible or desirable to achieve a "full infiltration" design. Proceed to Part 2		

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

Worksheet C.4-1 Page 3 of 4

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

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

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

Provide basis:

Based on our study, appreciable infiltration rates were measured.

A-1: 0.08 in/hr
A-2: 0.22 in/hr
P-1: 1.08 in/hr
P-2: 0.09 in/hr
P-3: 0.42 in/hr

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

Provide basis:

The specific geologic or geotechnical hazard for this site is the potential for groundwater mounding and lateral migration of infiltration water. The area of the proposed basin is underlain by dense formational soils of the Scripps Formation and Ardath Formation (see Geocon report dated June 11, 2015 and March 15, 2016). The infiltration test results performed on the property very widely across the site. The variability observed in the test results is a reflection of the heterogeneous, anisotropic nature of the site hydrological properties. Since the site geology is composed of interbedded sandstone and siltstone/claystone (as geotechnical borings performed show) we expect that infiltration of storm water will be carried by the more permeable sandstone layers and occluded by the siltstone/claystone layers; therefore, the site is highly prone to groundwater mounding beneath basins and lateral migration of infiltrated groundwater.

Under partial infiltration, mitigation measures should be taken to reduce potential impacts as a result of groundwater mounding and lateral water migration. Proposed below grade retaining walls for the parking structure and other proposed adjacent structures should be constructed with wall drains to intercept seepage and outlet it from behind the walls. The existing building west of the infiltration basin is supported on drilled piers so we do not expect lateral migration of infiltration to impact the building structure. There are no slopes or known existing utilities within the proposed area of the basin that are expected to be impacted by partial infiltration.

<table>
<tr><th colspan="4">Worksheet C.4-1 Page 4 of 4</th></tr>
<tr><th>Criteria</th><th>Screening Question</th><th>Yes</th><th>No</th></tr>
<tr><td>7</td><td>Can Infiltration in any appreciable quantity be allowed without posing significant risk for groundwater related concerns (shallow water table, storm water pollutants or other factors)? The response to this Screening Question shall be based on a comprehensive evaluation of the factors presented in Appendix C.3.</td><td>X</td><td></td></tr>
<tr><td colspan="4">Provide basis:

Groundwater is expected to be at depths greater than 100 feet.

Summarize findings of studies; provide reference to studies, calculations, maps, data sources, etc. Provide narrative discussion of study/data source applicability and why it was not feasible to mitigate low infiltration rates.</td></tr>
<tr><td>8</td><td>Can infiltration be allowed without violating downstream water rights? The response to this Screening Question shall be based on a comprehensive evaluation of the factors presented in Appendix C.3.</td><td>X</td><td></td></tr>
<tr><td colspan="4">Provide basis:

There are no known downstream water rights. Response provided by Michael Baker International, the project's civil engineer.

Summarize findings of studies; provide reference to studies, calculations, maps, data sources, etc. Provide narrative discussion of study/data source applicability and why it was not feasible to mitigate low infiltration rates.</td></tr>
<tr><td>Part 2 Result*</td><td colspan="2">If all answers from row 1-4 are yes then partial infiltration design is potentially feasible. The feasibility screening category is Partial Infiltration.

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

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

GEOCON

Aardvark Permeameter Data Analysis

Project Name:	Campus Point	Date:	9/19/2016
Project Number:	07850-42-15	By:	JTL
Borehole Location:	A-1	Ref. EL (feet, MSL):	
		Bottom EL (feet, MSL):	

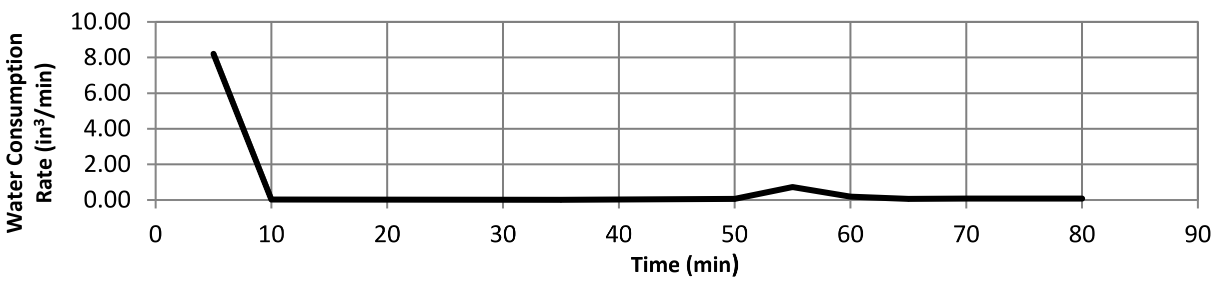
Borehole Diameter (inches):	4.00
Borehole Depth, **H** (feet):	1.42
Distance Between Reservoir & Top of Borehole (feet):	2.42
Depth to Water Table, **s** (feet):	1000
Height APM Raised from Bottom (inches):	0.00

Wetted Area, **A** (in^2): 58.18

Distance Between Resevoir and APM, **D** (feet):	3.24
Head Height, **h** (inches):	3.63
Distance Between Constant Head and Water Table, **L** (inches):	11987

Reading	Time (min)	Time Elapsed (min)	Reservoir Water Weight (g)	Resevoir Water Weight (lbs)	Interval Water Consumption (lbs)	Total Water Consumption (lbs)	*Water Consumption Rate (in^3/min)
1	0.00			22.045			
2	5.00	5.00		20.565	1.48	1.48	8.20
3	10.00	5.00		20.560	0.01	1.49	0.03
4	35.00	25.00		20.550	0.01	1.50	0.01
5	50.00	15.00		20.520	0.03	1.53	0.06
6	55.00	5.00		20.390	0.13	1.66	0.72
7	60.00	5.00		20.355	0.04	1.69	0.19
8	65.00	5.00		20.345	0.01	1.70	0.06
9	70.00	5.00		20.330	0.02	1.72	0.08
10	75.00	5.00		20.315	0.01	1.73	0.08
11	80.00	5.00		20.300	0.02	1.75	0.08

Steady Flow Rate, Q (in^3/min): 8.32E-02



Field-Saturated Hydraulic Conductivity (Infiltration Rate)

Case 1: L/h > 3 K_{sat} = 7.71E-04 in/min 0.05 in/hr

GEOCON

Aardvark Permeameter Data Analysis

Project Name: Campus Point
Date: 9/19/2016
Project Number: 07850-42-15
By: JTL
Borehole Location: A-2
Ref. EL (feet, MSL):
Bottom EL (feet, MSL):

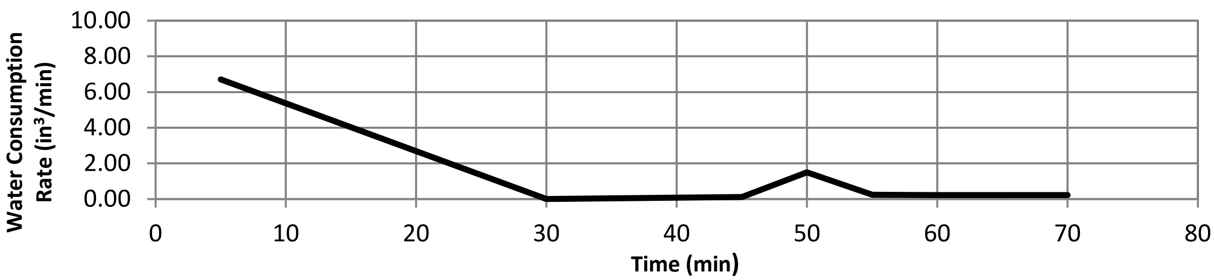
Borehole Diameter (inches): 4.00
Borehole Depth, **H** (feet): 1.29
Distance Between Reservoir & Top of Borehole (feet): 2.42
Depth to Water Table, **s** (feet): 1000
Height APM Raised from Bottom (inches): 0.00

Wetted Area, **A** (in^2): 58.11

Distance Between Resevoir and APM, **D** (feet): 3.10
Head Height, **h** (inches): 3.62
Distance Between Constant Head and Water Table, **L** (inches): 11988

Reading	Time (min)	Time Elapsed (min)	Reservoir Water Weight (g)	Resevoir Water Weight (lbs)	Interval Water Consumption (lbs)	Total Water Consumption (lbs)	*Water Consumption Rate (in^3/min)
1	0.00			19.155			
2	5.00	5.00		17.945	1.21	1.21	6.71
3	30.00	25.00		17.940	0.00	1.22	0.01
4	45.00	15.00		17.875	0.07	1.28	0.12
5	50.00	5.00		17.605	0.27	1.55	1.50
6	55.00	5.00		17.560	0.05	1.60	0.25
7	60.00	5.00		17.520	0.04	1.64	0.22
8	65.00	5.00		17.480	0.04	1.68	0.22
9	70.00	5.00		17.440	0.04	1.72	0.22

Steady Flow Rate, Q (in^3/min): 2.22E-01



Field-Saturated Hydraulic Conductivity (Infiltration Rate)

Case 1: L/h > 3 K_{sat} = 2.06E-03 in/min 0.12 in/hr

APPENDIX G

Hydrology and Hydraulic Study

Hydrology and Hydraulic Study

For

ARE Campus Point PDP

Prepared For:

Alexandria Real Estate Equities
10996 Torreyana Road, Suite 250
San Diego, CA 92121

City of San Diego

PTS: 336364

Michael Baker
INTERNATIONAL

Prepared By:

Michael Baker International
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San Diego, CA 92124
858.614.5000
Richard S. Tomlinson, Jr. PE,
QSP, QSD, CPSWQ

Job Number:

149488

Prepared:

January 8, 2016

REGISTERED PROFESSIONAL ENGINEER
RICHARD S. TOMLINSON, JR.
No. 59276
EXP. 6/30/17
CIVIL
STATE OF CALIFORNIA

Revised:

June 23, 2016

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Appendix D -- Proposed Condition Hydrologic Work Map & Calculations

Section 1 Project Description and Scope

1.1. Project Data

Project Owner: Alexandria Real Estate Equities
10996 Torreyanna Road, Suite 250
San Diego, CA 92121

Project Site Address: Campus Pointe Boulevard

Planning Area/
Community Area/
Development Name: University City

APN Number(s): 343-230-13-00

Project Location: Latitude: 32.892777°
Longitude:-117.22298°

Project Site Area: 4.12 Acres

Adjacent Streets:
North: Roselle Street
South: Genesee Avenue
East: Towne Center Drive
West: Genesee Avenue

Adjacent Land Uses:
North: Open Space
South: Commercial
East: Commercial
West: Commercial

1.2. Scope of Report

This report addresses the Hydrologic and Hydraulic aspects of the project. This report does not discuss required water quality measures to be implemented on a permanent basis, nor does it address construction storm water issues. Post construction storm water issue discussions can be found under separate cover in the project "Water Quality Technical Report."

In addition, because this project proposes to disturb over one acre, a Storm Water Pollution Protection Plan for construction activities has been prepared and an NOI will be filed with the State of California prior to the start of construction.

Because this project is discharging into the City of San Diego MS-4 system, and not into directly into the Waters of The United States or any other regulated natural system, the project is not required to obtain a 401 or 404 permit.

The 401 or 404 permit is only required for projects that extend into the waters of the US and wetlands. This project is entirely within built up areas, and is reducing the flows from the site by as much as 99%.

1.3. Project Site Information

1.3.1 Project Location

The project is located on at 10300 Campus Pointe Drive in the City and County of San Diego, in the Sorrento Valley Community of the City of San Diego. The project is located just to the east of Interstate 5, west of Interstate 805, and just south of the 5/805 merge. The project is located northerly of Genesee Avenue. Please refer to Figure 1 below for a Vicinity Map.

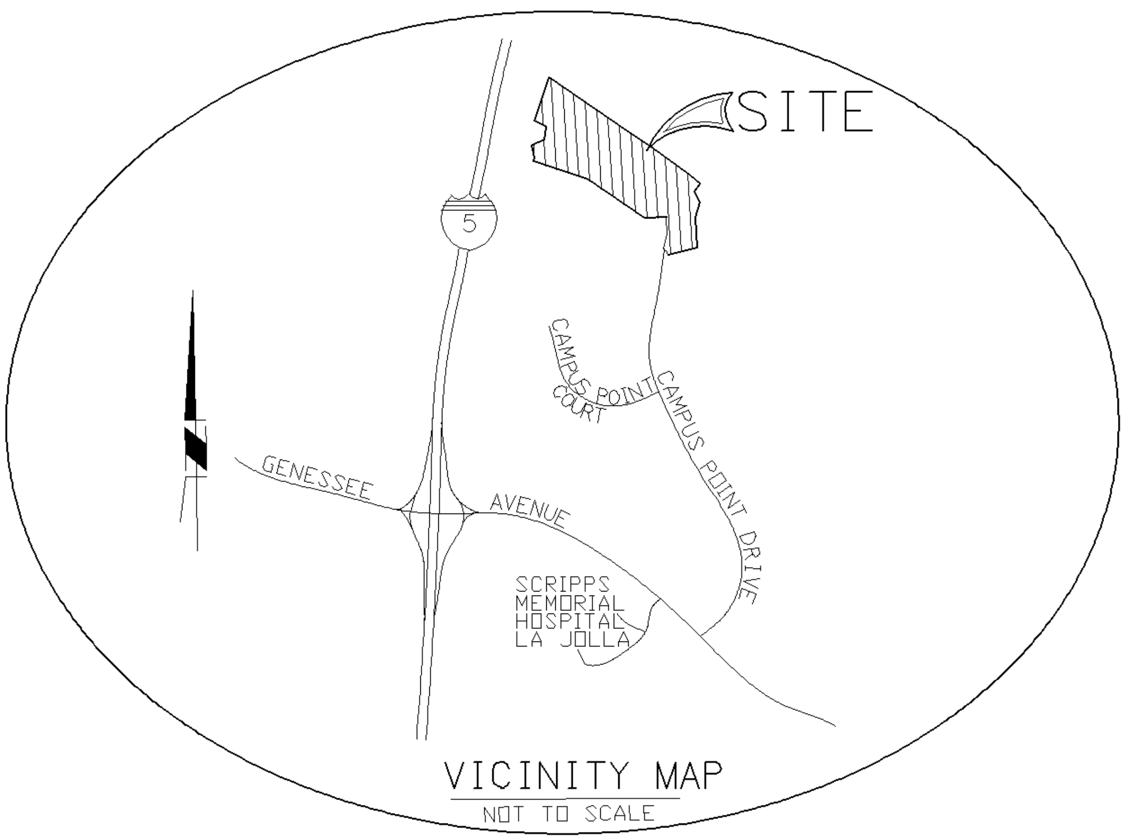


Figure 1: Vicinity Map

1.3.2 Project Description

The project proposes the completion of a new driveway and entry road called the Boulevard. The project also proposes the construction of new hardscape and landscape. In addition, underground storm drain, catch basins, curb inlets and biofiltration basins are proposed. In order to accomplish the construction, the project proposes the demolition of existing parking, hardscape and landscape.

1.3.3 Site Topography

Although the perimeter of the campus has slopes up to 130 feet tall, the core of the campus is relatively flat. The site has a maximum elevation of approximately 320 feet mean sea level (MSL). The lowest part of the graded area is at the southwest corner of the site at around elevation 295. Slopes surround the site on both the west and north sides of the site.

1.3.4 Land Use and Vegetation

The majority of the 22.8 acre site is currently project site is currently developed. The site is designated as commercial land use and is currently made up of a very large building along with associated hardscape, and landscape. The vegetation in the landscaped areas consists of primarily lawn and trees.

1.3.5 FEMA Information

The Federal Emergency Management Agency (FEMA) has mapped the floodplain of Soledad Canyon as a special flood hazard area, Zone AE (FIRM Panel 06073C-1338G). The project site does not lie within the mapped floodplain.

a) Flood Zone Definitions

Zone A -- Areas subject to inundation by the 1-percent-annual-chance flood event generally determined using approximate methodologies. Because detailed hydraulic analyses have not been performed, no Base Flood Elevations (BFEs) or flood depths are shown. Mandatory flood insurance purchase requirements and floodplain management standards apply.

Zone AE -- Areas subject to inundation by the 1-percent-annual-chance flood event determined by detailed methods. Base Flood Elevations

(BFEs) are shown. Mandatory flood insurance purchase requirements and floodplain management standards apply.

Zone X (Shaded) – Areas between the limits of the base flood and the 0.2-percent-annual-chance (or 500-year) flood.

Zone X (Unshaded) Areas of minimal flood hazard, which are the areas outside the SFHA and higher than the elevation of the 0.2-percent-annual-chance flood

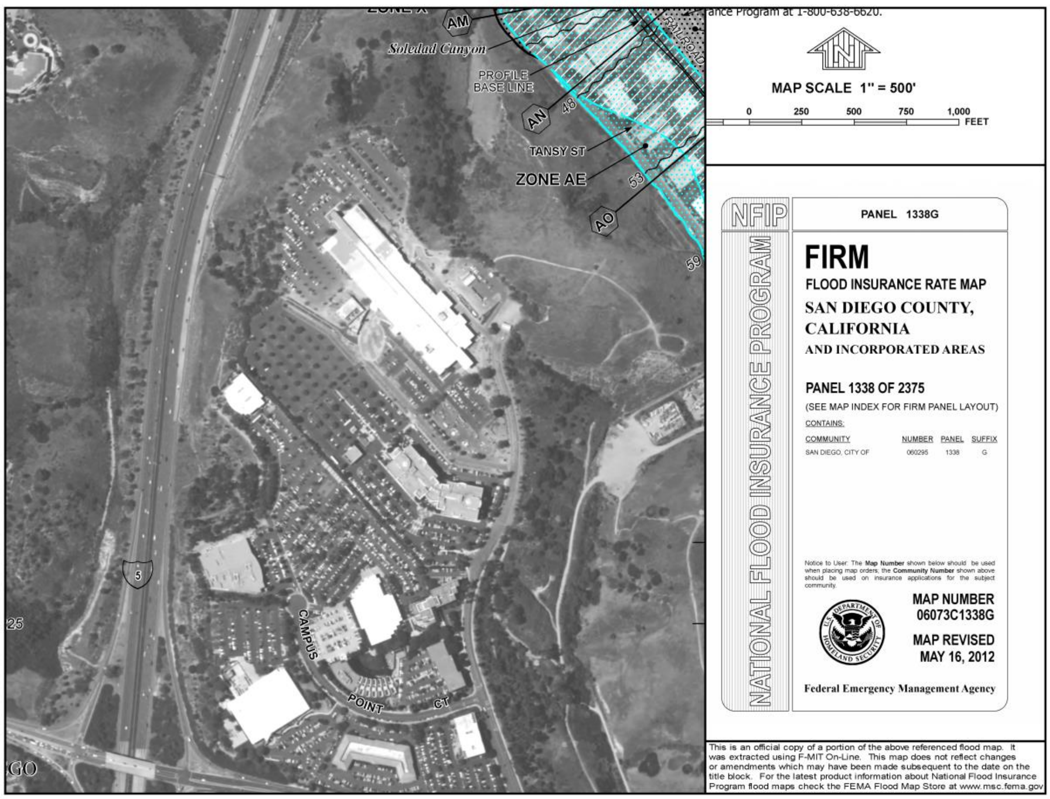


Figure 2: FEMA Firmette

1.3.6 Existing Drainage Improvements

The site currently drains to three directions, however, drainage from the project flows to only two of the three POC. The PDP project, in the existing and proposed condition flow to one of two points of connection, one to the west and the other two the southwest.

The first point of concentration is to the west. Drainage from the westerly side of the site flows into a 24" RCP storm drain. The storm drain flows to the west down the slope, before being discharge at the bottom of the canyon.

The second point of connection is to the southeast. Drainage from the southwest portion of the site, flows to the south, where it enters a storm drain that runs along southerly side of the property. This drainage then flows to the east where it flows into the canyon.

1.3.7 Proposed Improvements

The proposed drainage system includes a series of catch basins and PVC and HDPE pipe. The project also proposes two pump stations. The pump stations, one located in the northwest corner of the project and one located to the south west corner of the project pump the storm drainage to the proposed infiltration basin. The infiltration basin will infiltrate the flows from the majority of the PDP site, with the SDHM estimating that 98.77% of the runoff will be infiltrated.

Basin B, includes a portion of the road not being constructed under the Boulevard project, a ministerial project that is being processed under a separate permit. This roadway drains to a biofiltration basin which uses passive infiltration. The passive infiltration does not meet the 85th percentile requirement, hence it has been designed as an infiltration basin.

Because the use of the project does not change from commercial to commercial, there is no change in runoff co-efficient. With no change in runoff co-efficient and area, it is anticipated that the runoff will not change.

However, in the mitigated condition, the flows are drastically reduced. In fact, 69.3% of runoff is infiltrated in Basin B and 98.8% in Basin A.

Through careful design of the site, minimal off-site flows enter the site. Basin A has offsite flows that enter the site from the north. These flows are being captured and treated within the Infiltration Basin within Basin A.

Section 2 Study Objectives

The specific objectives of this study are as follows:

- To provide hydrologic analysis of the project site for the 100-year, 6-hour storm event under existing and proposed conditions,
- To provide a hydraulic analysis of the project to ensure that the correct sizes of pipes and inlets have been chosen,
- And to ensure that no additional runoff or downstream impacts occur due to this project.

Section 3 Methodology

3.1. Hydrology

Hydrologic analysis has been completed using the Rational Method (Q = CIA). Whereas,

Q = rate of flow in cubic feet per second

C = Coefficient of runoff,

I = intensity of rainfall based on the time of concentration and the 6-hour, 100-year precipitation

A=Area of the basin.

For this project, a composite coefficient of runoff was used. Data was entered into an Excel Spreadsheet which calculates the runoff based on the County of San Diego methodology electronically, therefore reducing errors.

The following software packages were used in the analysis of the project:

- Microsoft Excel (Rational Method Hydrology)
- AutoCAD Civil 3d Hydraflow Hydragraph Extension 2013 (Storm Routing)
- RatHydro (Rational Method Hydragraphs)
- Flowmaster (Hydraulic Analysis for Open Channels and Pipes for Storm Routing)

3.2. Hydraulics

Proposed improvements include new grated storm drain inlets in paved areas, and a new underground storm drain system. Private underground storm drain will consist of PVC or HDPE pipe with watertight joints. Public storm drain, if applicable, will consist of reinforced concrete pipe, with a minimum strength of 2000-D.

Capacity calculations for the inlets have been performed using the standard weir and orifice equations. Grate perimeter and open area values have been reduced to account for the bars, and an additional 50-percent to account for potential clogging.

Runoff will ultimately be discharged from the project site at the same location as the existing condition, to the existing cleanout at the southwest corner of the project site.

Proposed improvements will not increase the total peak flow runoff, as compared to existing conditions, through the removal of pavement and installation of vegetation.

Manning's equation was used to calculate the depth of flow being conveyed through proposed pipes and for existing pipes which experience additional flows as a result of the proposed improvements. Proposed pipes with diameters of less than 12 inches were not individually calculated for depth and velocity, however, the capacity was verified against tables showing the maximum flow in the smaller pipes.

The following software packages were used in the analysis of the project:

- Hydraflow Hydragraph Extension for AutoCAD Civil 3d 2013 (Storm Routing)
- Hydraflow Storm Sewer Extension for AutoCAD Civil 3d 2013 (Hydraulic and Energy Grade Lines)
- Hydraflow Express Extensions Extension for AutoCAD Civil 3d 2013 (Storm Routing)
- RatHydro (Rational Method Hydrographs)
- Bentley Flowmaster (Hydraulic Analysis for Open Channels and Pipes for Storm Routing)

3.3. Hydromodification

Flow control is considered a storm water management issue, and is therefore addressed in the Water Quality Technical Report.

However, the preconditions for the Hydromodification on all of the new surfaces is pervious condition. In those areas where there is run on, the run on surface used for Hydromodification is the surface in the existing condition.

Section 4 Results

4.1. Hydrologic Results

The following tables summarize the hydrologic analysis of the project.

- **Table 1 – Existing Condition**, summarizes the existing hydrologic properties of the project site.

Sub Basin No.	Runoff Coefficient	Basin Intensity	Basin Area (acres)	Runoff (cfs)
Basin A	0.93	5.18	11.09	53.44
Basin B	0.76	4.46	0.52	1.42
TOTALS			**11.61**	**55.86**

- **Table 2 – Proposed Condition (Unmitigated)**, summarizes the proposed condition hydrology of the site in the unmitigated condition.
-

Table 3 – Comparison of Existing and Proposed Flows (100-year) compares existing flows to the proposed flows.

Table 1 – Existing Condition (100-year)

Sub Basin No.	Runoff Coefficient	Basin Intensity	Basin Area (acres)	Runoff (cfs)
Basin A	0.93	5.18	11.09	53.44
Basin B	0.76	4.46	0.52	1.42
TOTALS			**11.61**	**55.86**

Table 2 – Proposed Condition (Unmitigated) (100-year)

Sub Basin No.	Runoff Coefficient	Basin Intensity	Basin Area (acres)	Runoff (cfs)
Basin A	0.73	3.24	11.09	26.29
Basin B	0.86	5.57	0.52	2.49
TOTALS			**11.61**	**28.78**

Table 3 – Comparison of Existing and Proposed Flows (100-year)

Sub Basin No.	Existing Condition (cfs)	Proposed Condition (cfs)	Difference
Basin A	53.44	26.29	-27.15
Basin B	1.42	2.49	+1.07
TOTALS	**55.86**	**28.78**	**-26.08**

Section 5. Conclusions

As indicated in the Table of Hydrologic Results, the proposed improvements will not increase the total 100-year, 6-hour peak flow rate.

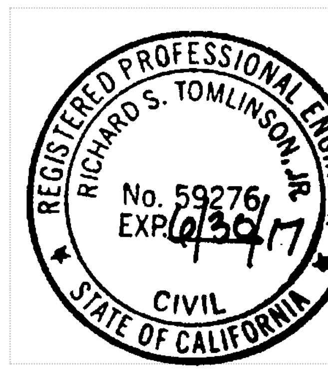
Proposed private grated inlets, all of which are in a sump condition, shall capture the generated flows without significant ponding. In the unlikely event that grated inlets become completely clogged, the proposed site grades shall provide overland release to adjacent drainage areas.

There is not a significant concern for erosion as the site is previously developed. Potential for erosion for the proposed condition shall be minimized by following items listed in the Erosion Control Plan (part of the Rough Grading Plans). Runoff shall flow over relatively flat areas where scour is not a concern. Runoff is not proposed over any sloped areas.

Because the flows in the 100-year event and all flows from the Q2 to Q25 have been reduced, some by as much as 99%, no downstream effects are anticipated. The reduction has been obtained by the addition of pervious areas, an infiltration basin and a biofiltration basin.

Section 5 Certification

This Hydrology and Hydraulics report has been prepared under the direction of the following Registered Civil Engineer. The Registered Civil Engineer attests to the technical information contained herein and the engineering data upon which recommendations, conclusions, and decisions are based. The plans and specifications in this Hydrology and Hydraulics report are not for construction purposes; the contractor shall refer to final approved construction documents for plans and specifications.



Richard S. Tomlinson, Jr. RCE 59276 June 23, 2016

Section 6 References

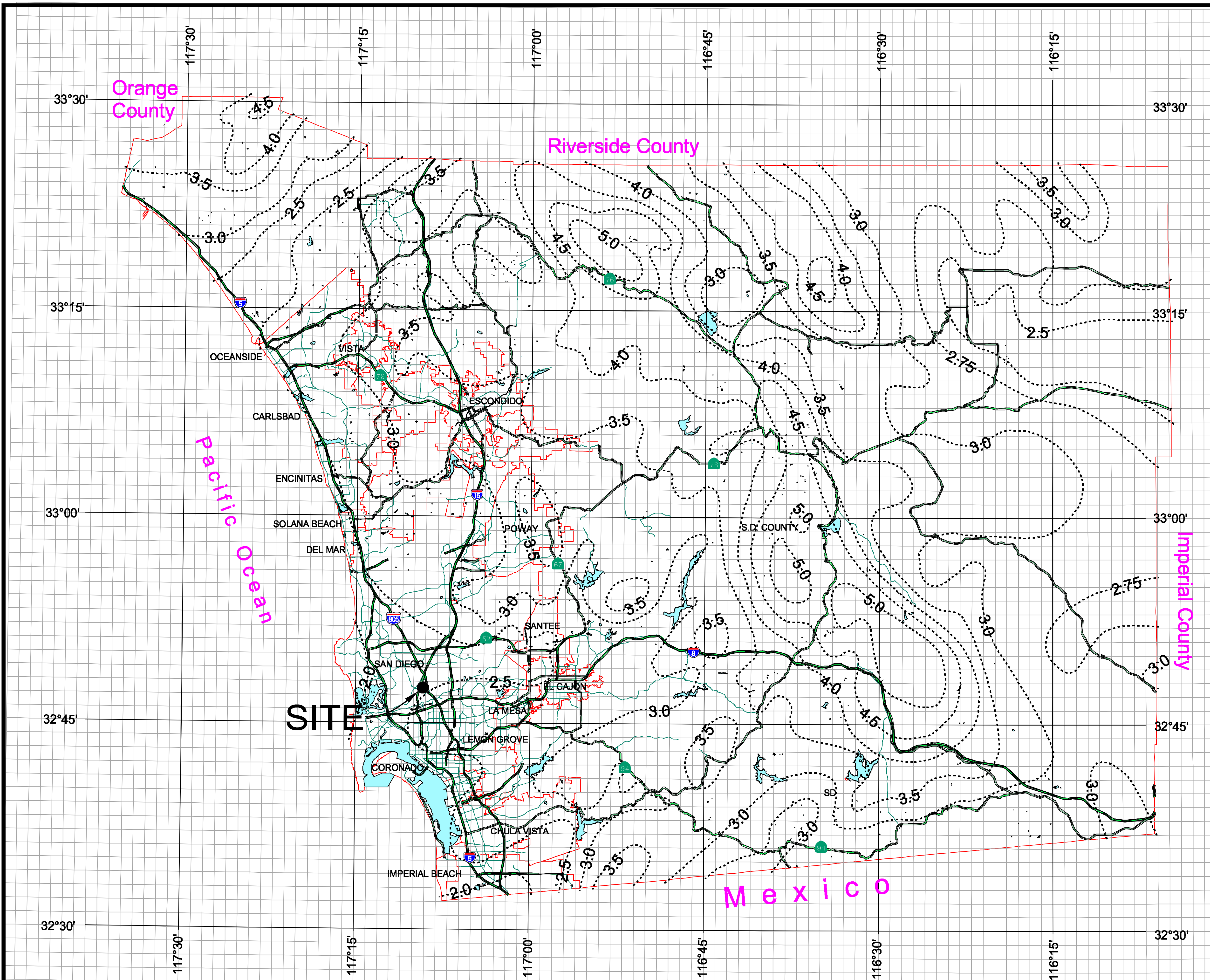
San Diego Municipal Code, 2008. Land Development Manual (March 2008). Storm Water Standards.

County of San Diego, 1984. Drainage Design Manual (April 1984)

FEMA, 1997. FEMA. (June 17, 1997). Flood Insurance Study, San Diego County.

Appendix A

Rainfall Isopluvials



County of San Diego Hydrology Manual

Rainfall Isopluvials

100 Year Rainfall Event - 6 Hours

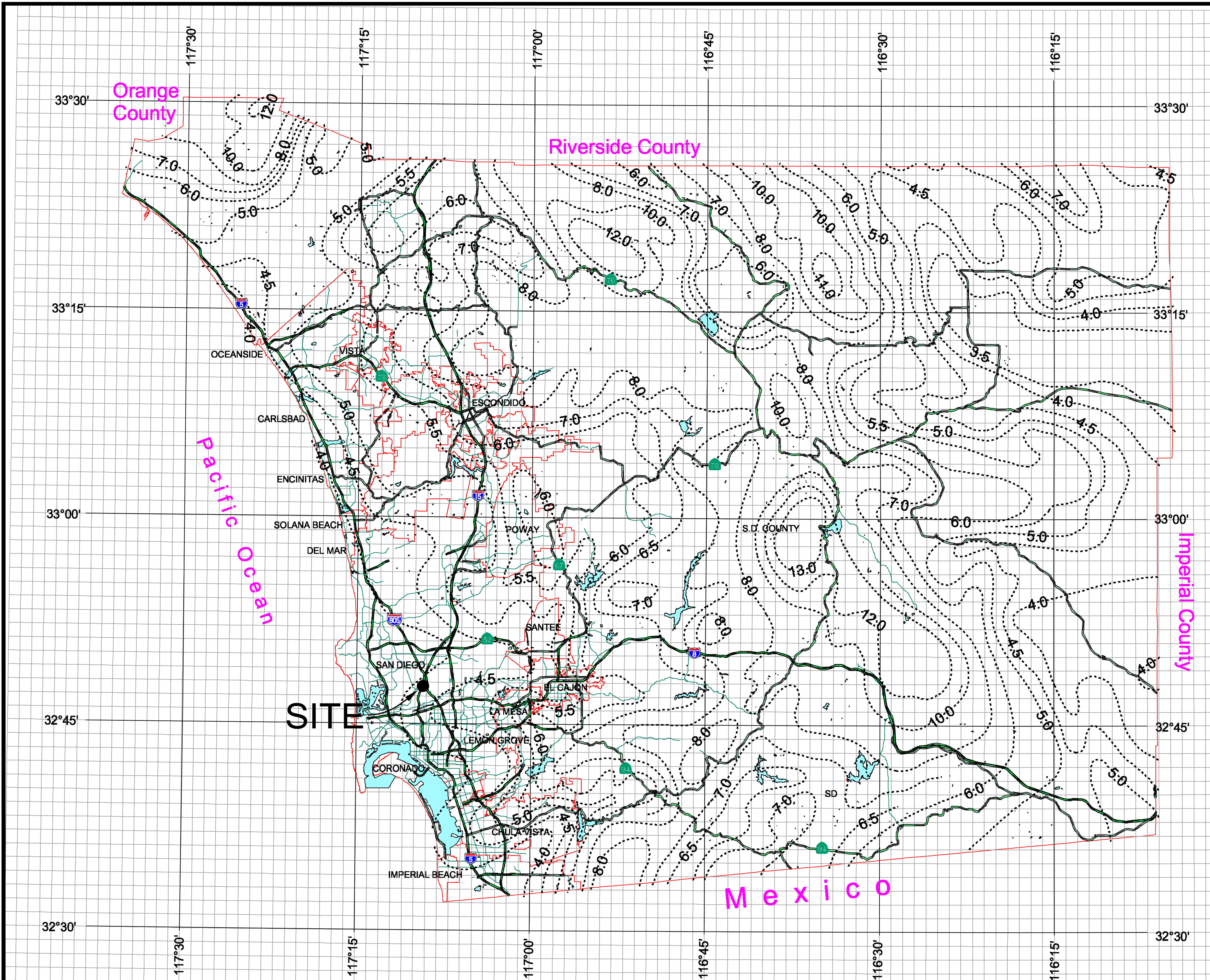
Isopluvial (inches)

DPW GIS
Department of Public Works
Geographic Information Services

SanGIS
We Have San Diego Covered!

N
W E
S

3 0 3 Miles



County of San Diego Hydrology Manual

Rainfall Isopluvials

100 Year Rainfall Event - 24 Hours

Isopluvial (inches)

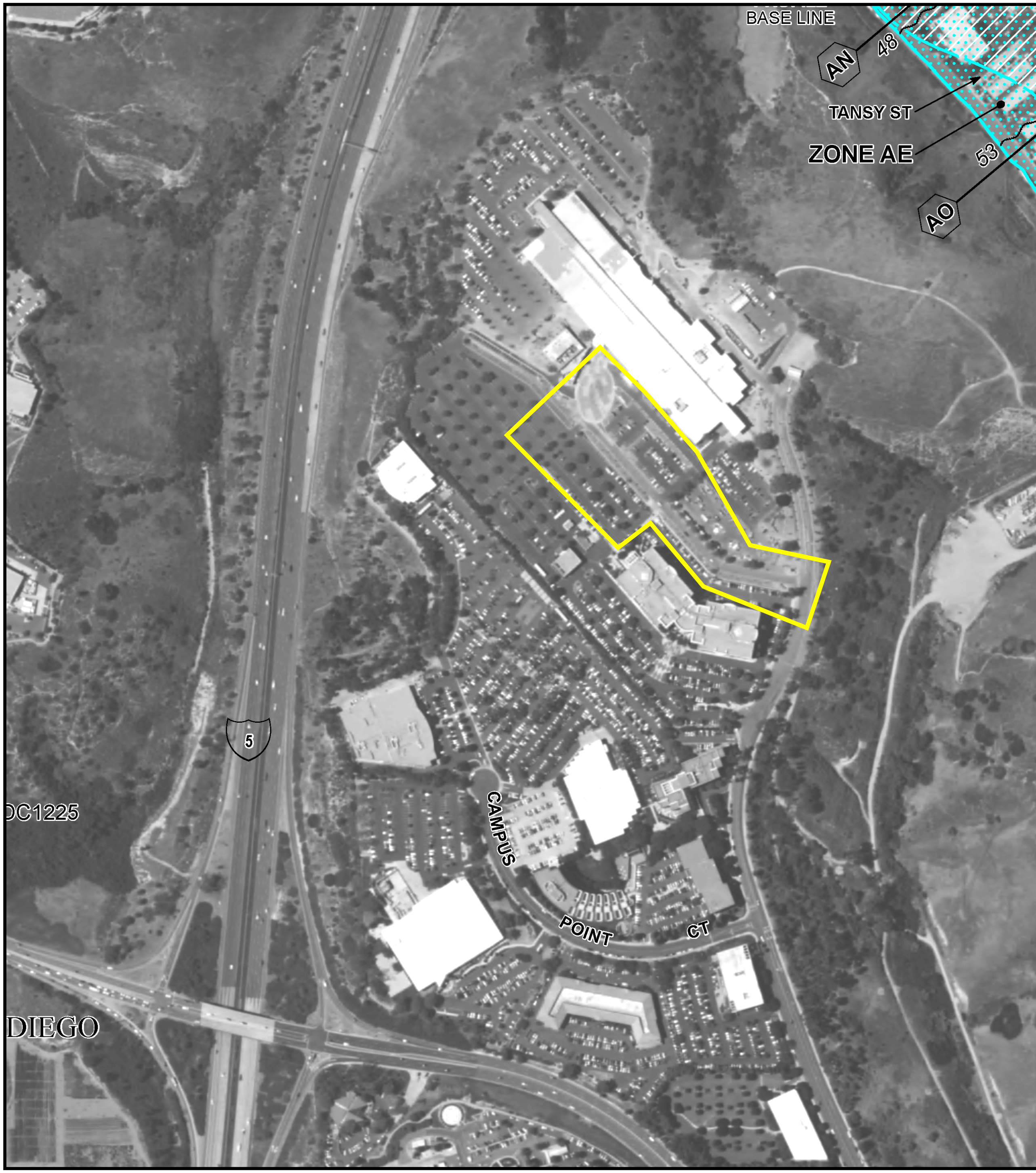
DPW GIS
Department of Public Works
Geographic Information Services

SanGIS
We Have San Diego Covered!

N S W E

3 0 3 Miles

Appendix B
FEMA Flood Plain Maps



ance Program at 1-800-638-6620.

MAP SCALE 1" = 500'

0 250 500 750 1,000
FEET

NFIP

NATIONAL FLOOD INSURANCE PROGRAM

PANEL 1338G

FIRM

FLOOD INSURANCE RATE MAP

SAN DIEGO COUNTY, CALIFORNIA

AND INCORPORATED AREAS

PANEL 1338 OF 2375

(SEE MAP INDEX FOR FIRM PANEL LAYOUT)

CONTAINS:

COMMUNITY	NUMBER	PANEL	SUFFIX
SAN DIEGO, CITY OF	060295	1338	G

Notice to User: The **Map Number** shown below should be used when placing map orders; the **Community Number** shown above should be used on insurance applications for the subject community.

MAP NUMBER
06073C1338G

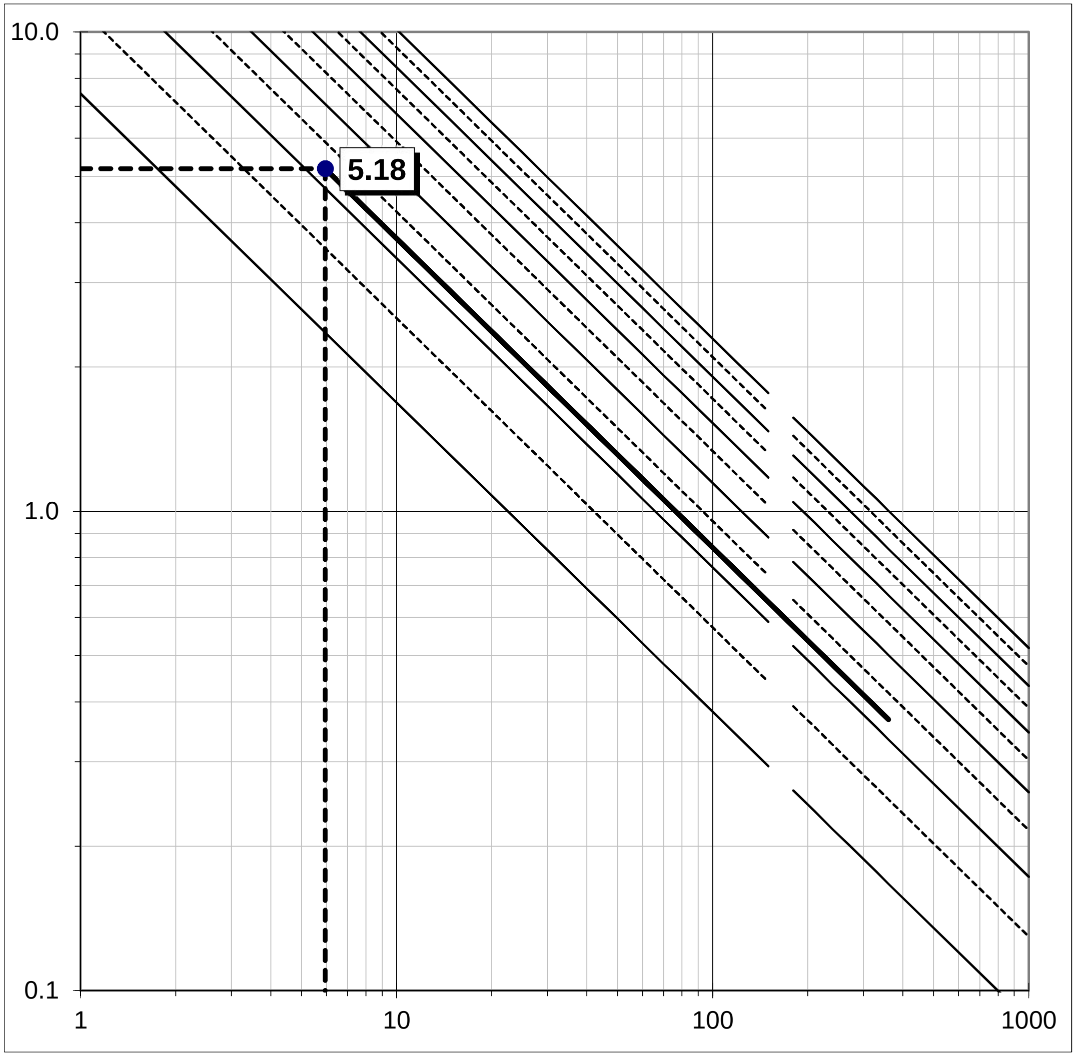
MAP REVISED
MAY 16, 2012

Federal Emergency Management Agency

This is an official copy of a portion of the above referenced flood map. It was extracted using F-MIT On-Line. This map does not reflect changes or amendments which may have been made subsequent to the date on the title block. For the latest product information about National Flood Insurance Program flood maps check the FEMA Flood Map Store at www.msc.fema.gov

Michael Baker
INTERNATIONAL

Appendix C
Existing Condition Hydrologic Work Map & Calculations



Campus Point SDP

Basin A Existing

Time of Concentration Calculations

Natural Areas

Land Use =	Commercial	
C =	0.93	
Dist. =	600.00	ft.
slope =	2.000	%
T_c =	5.94	min.

$$T_C = \frac{1.8(1.1-C)\sqrt{D}}{\sqrt[3]{s}}$$

* Minimum T_c = 5 Minutes

Weighted C Value Calculation

	Area	
Pervious	1.380	
Impervious	9.700	
Total	11.080	
Actual Impervious		0.88
Tabulated Impervious		0.80
Coeefecient		0.85
Revised 'C'		0.93
Use 'C'		0.93

Basin Intensity Calculations

Selected Frequency,	**100**	year
P_6 =	2.2	in.
P_{24} =	3.8	in.
P_6 / P_{24} =	58%	
Adjusted **P_6**=	2.20	in.
T_c (**D**) =	5.94	min.
I =	5.18	in/hr

P_6 must be within 45% to 65% of P_{24}. Adjust P_6 as needed.

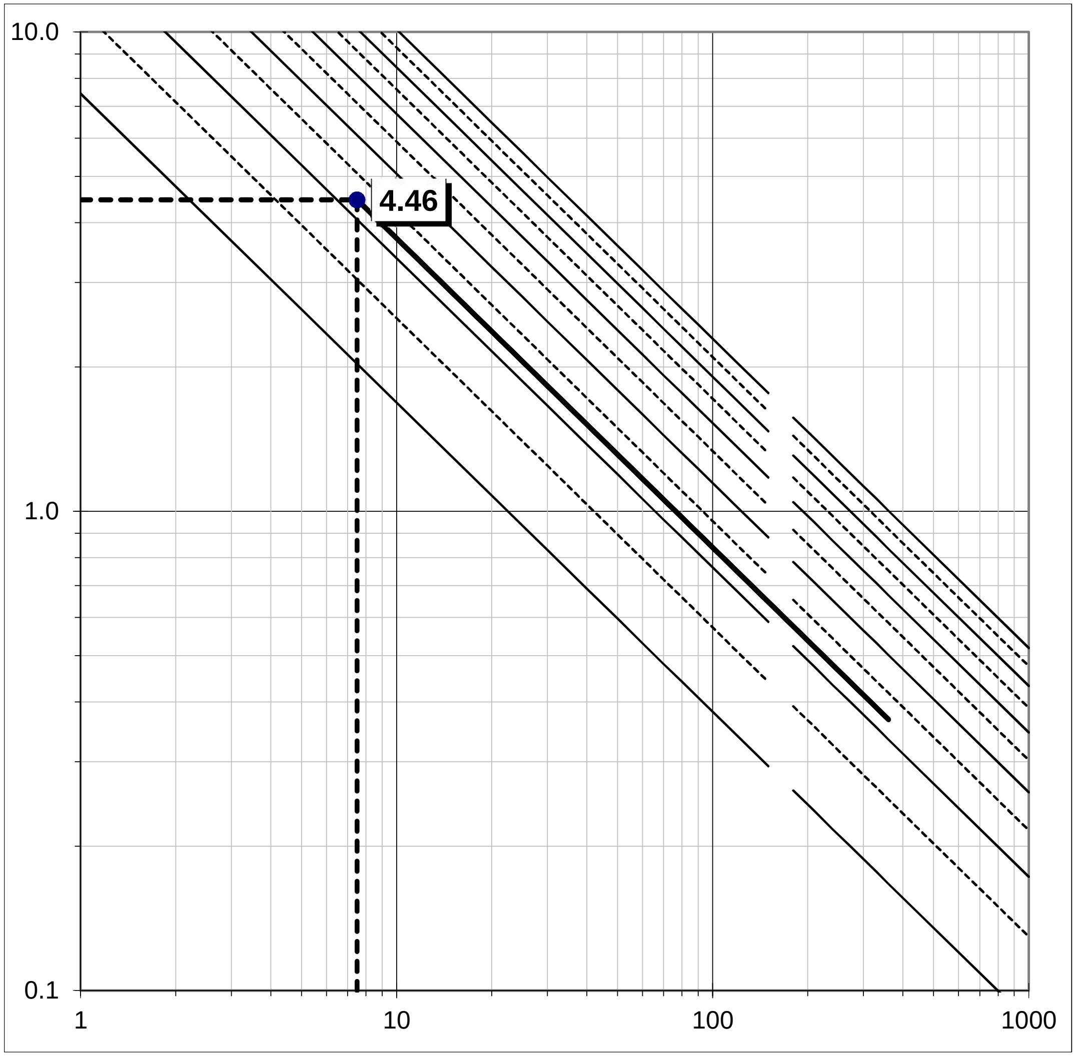
$$I = 7.44 P_6 D^{-0.645}$$

Basin Flow Calculations

Q =	**53.438**	**cfs**
C =	0.93	
I =	5.18	in/hr
A =	11.080	ac.

$$Q = C * I * A$$

RBF Job No. 139861



Campus Point SDP

Basin B Existing

Time of Concentration Calculations

Natural Areas

Land Use =	Commercial	
C =	0.76	
Dist. =	310.00	ft.
slope =	3.000	%
T_c =	7.49	min.

$$T_C = \frac{1.8(1.1-C)\sqrt{D}}{\sqrt[3]{s}}$$

* Minimum T_c = 5 Minutes

Weighted C Value Calculation

	Area
Pervious	0.120
Impervious	0.300
Total	0.420

Actual Impervious	0.71
Tabulated Impervious	0.80
Coeefecient	0.85
Revised 'C'	0.76
Use 'C'	0.76

Basin Intensity Calculations

Selected Frequency,	**100**	year	
P_6 =	2.2	in.	P_6 must be within
P_{24} =	3.8	in.	45% to 65% of P_{24}.
P_6 / P_{24} =	58%		Adjust P_6 as needed.
Adjusted $\mathbf{P_6}$=	2.20	in.	
T_c (**D**) =	7.49	min.	
I =	4.46	in/hr	

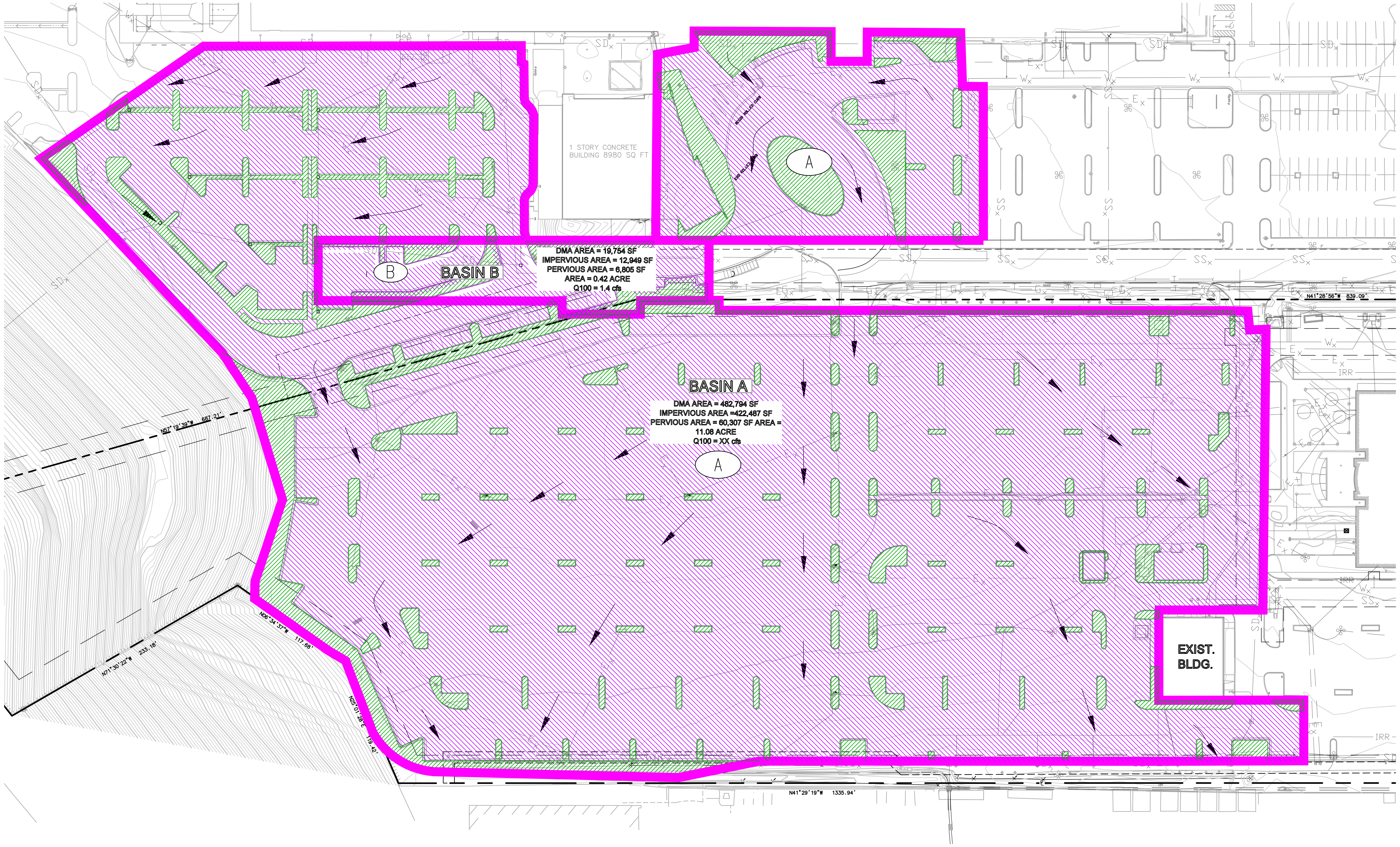
$$I = 7.44 P_6 D^{-0.645}$$

Basin Flow Calculations

Q =	**1.423**	**cfs**
C =	0.76	
I =	4.46	in/hr
A =	0.420	ac.

$$Q = C * I * A$$

RBF Job No. 139861



LEGEND

LIMITS OF OVERALL DRAINAGE BASIN	
BASIN	X-X
IMPERVIOUS AREA	
PERVIOUS AREA	
BIORETENTION BASIN	
DRAINAGE BASIN	
FLOW PATH	
POINT COMPLIANCE	
MODULAR WETLAND SYSTEM	
UNDERGROUND VAULT	UV

SCALE: 1"=50'

50 0 50 100 150

SCALE: 1"=50'

CAMPUS POINT

10290 CAMPUS POINT DR.
SAN DIEGO, CA

EXISTING CONDITIONS
IMPERVIOUS AREAS

06 / 22 / 16

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9755 Clairemont Mesa Boulevard
San Diego, CA 92124
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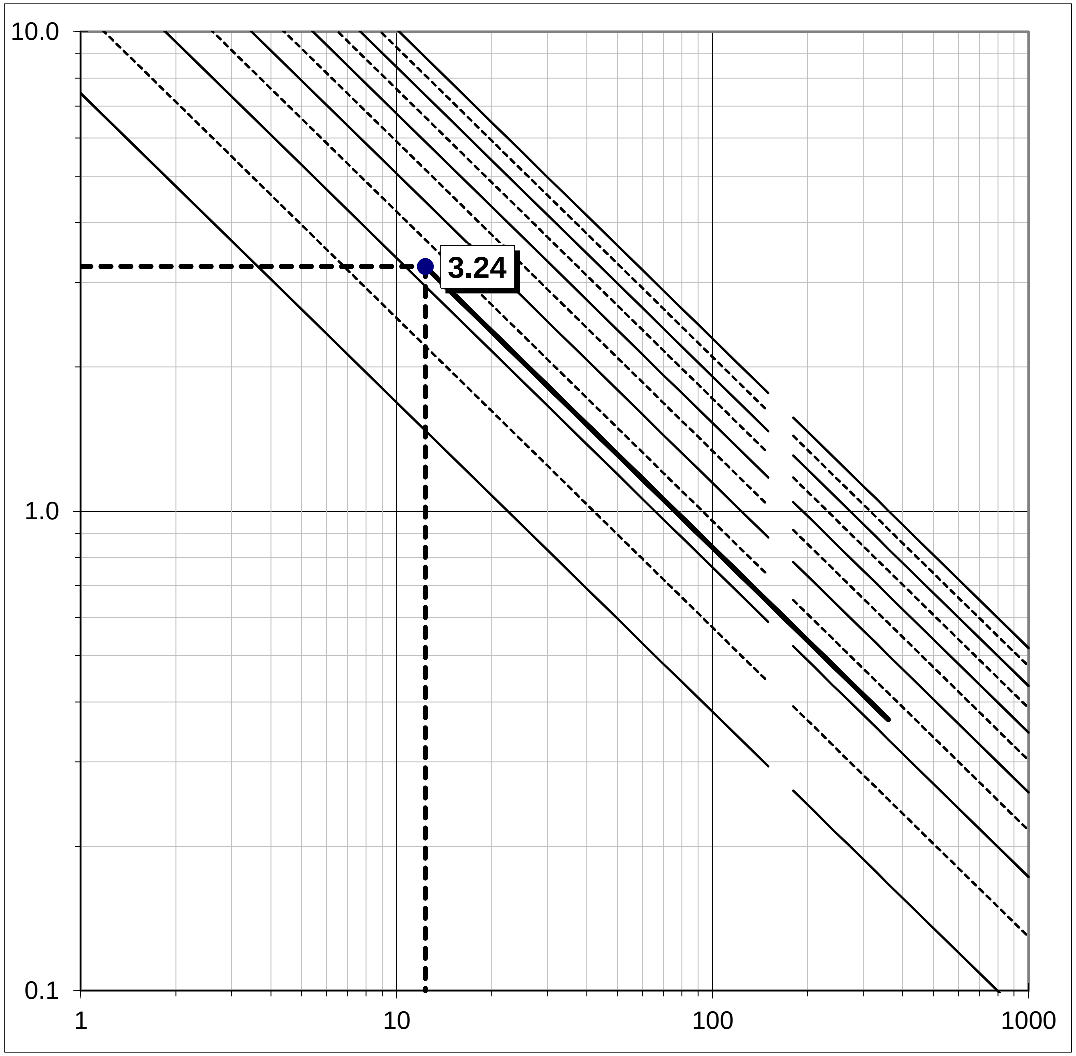
Appendix D
Proposed Condition Hydrologic Work Map & Calculations



Campus Point SDP

Basin A Proposed

Time of Concentration Calculations

Natural Areas

Land Use =	Commercial	
C =	0.73	
Dist. =	550.00	ft.
slope =	2.000	%
T_c =	12.33	min.

$$T_C = \frac{1.8(1.1 - C)\sqrt{D}}{\sqrt[3]{s}}$$

* Minimum T_c = 5 Minutes

Weighted C Value Calculation

	Area	
Pervious	3.448	
Impervious	7.640	
Total	11.088	
Actual Impervious		0.69
Tabulated Impervious		0.80
Coeefecient		0.85
Revised 'C'		0.73
Use 'C'		0.73

Basin Intensity Calculations

Selected Frequency,	**100**	year	
P_6 =	2.2	in.	P_6 must be within
P_{24} =	3.8	in.	45% to 65% of P_{24}.
P_6 / P_{24} =	58%		Adjust P_6 as needed.
Adjusted $\mathbf{P_6}$=	2.20	in.	
T_c (**D**) =	12.33	min.	
I =	3.24	in/hr	

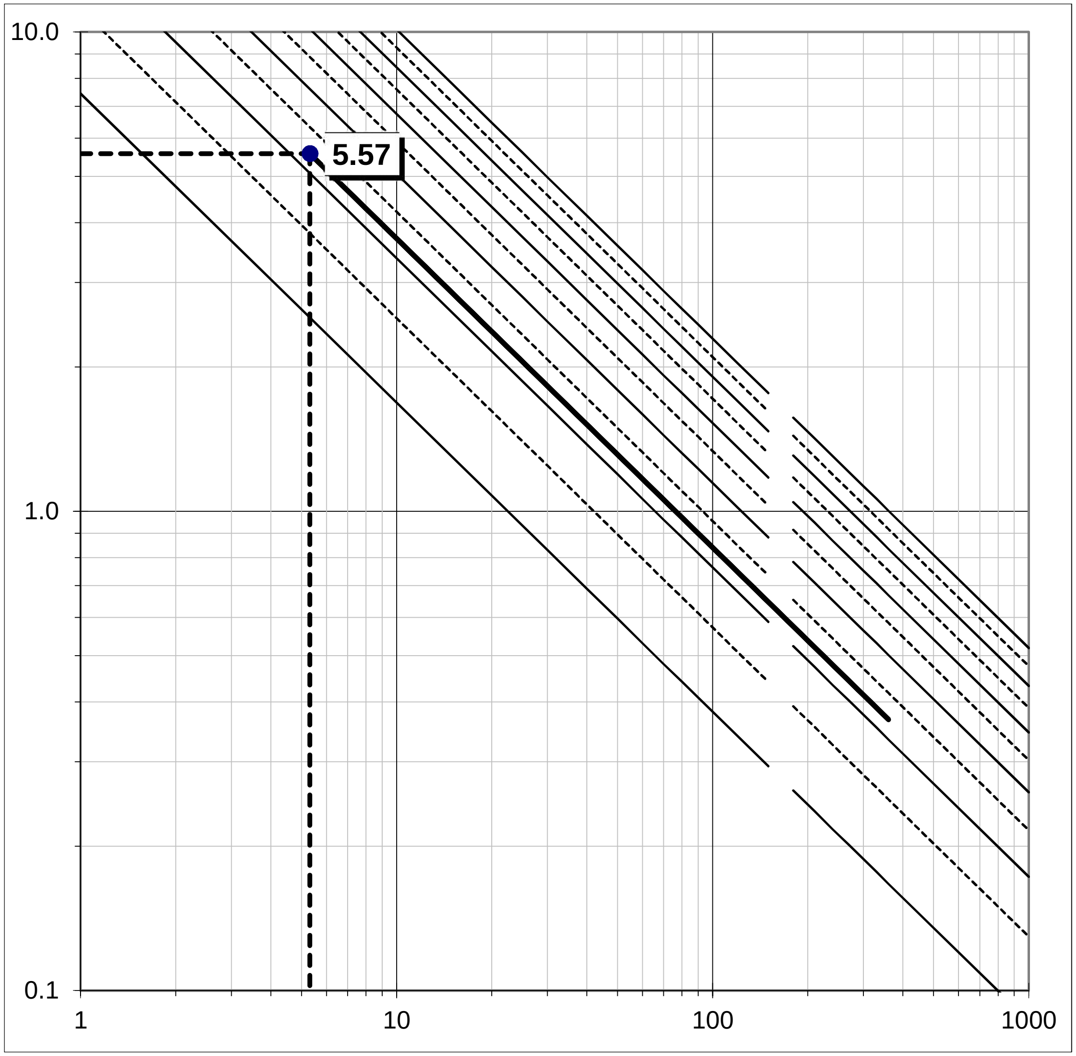
$$I = 7.44 P_6 D^{-0.645}$$

Basin Flow Calculations

Q =	**26.292**	**cfs**
C =	0.73	
I =	3.24	in/hr
A =	11.088	ac.

$$Q = C * I * A$$

RBF Job No. 139861



Campus Point SDP

Basin B Proposed

Time of Concentration Calculations

Natural Areas

Land Use =	Commercial	
C =	0.86	
Dist. =	310.00	ft.
slope =	3.000	%
T_c =	5.31	min.

$$T_C = \frac{1.8(1.1-C)\sqrt{D}}{\sqrt[3]{s}}$$

* Minimum T_c = 5 Minutes

Weighted C Value Calculation

	Area	
Pervious	0.100	
Impervious	0.420	
Total	0.520	
Actual Impervious		0.81
Tabulated Impervious		0.80
Coeefecient		0.85
Revised 'C'		0.86
Use 'C'		0.86

Basin Intensity Calculations

Selected Frequency,		**100**	year
P_6 =	2.2	in.	P_6 must be within
P_{24} =	3.8	in.	45% to 65% of P_{24}.
P_6 / P_{24} =	58%		Adjust P_6 as needed.
Adjusted **P_6**=	2.20	in.	
T_c (**D**) =	5.31	min.	
I =	5.57	in/hr	

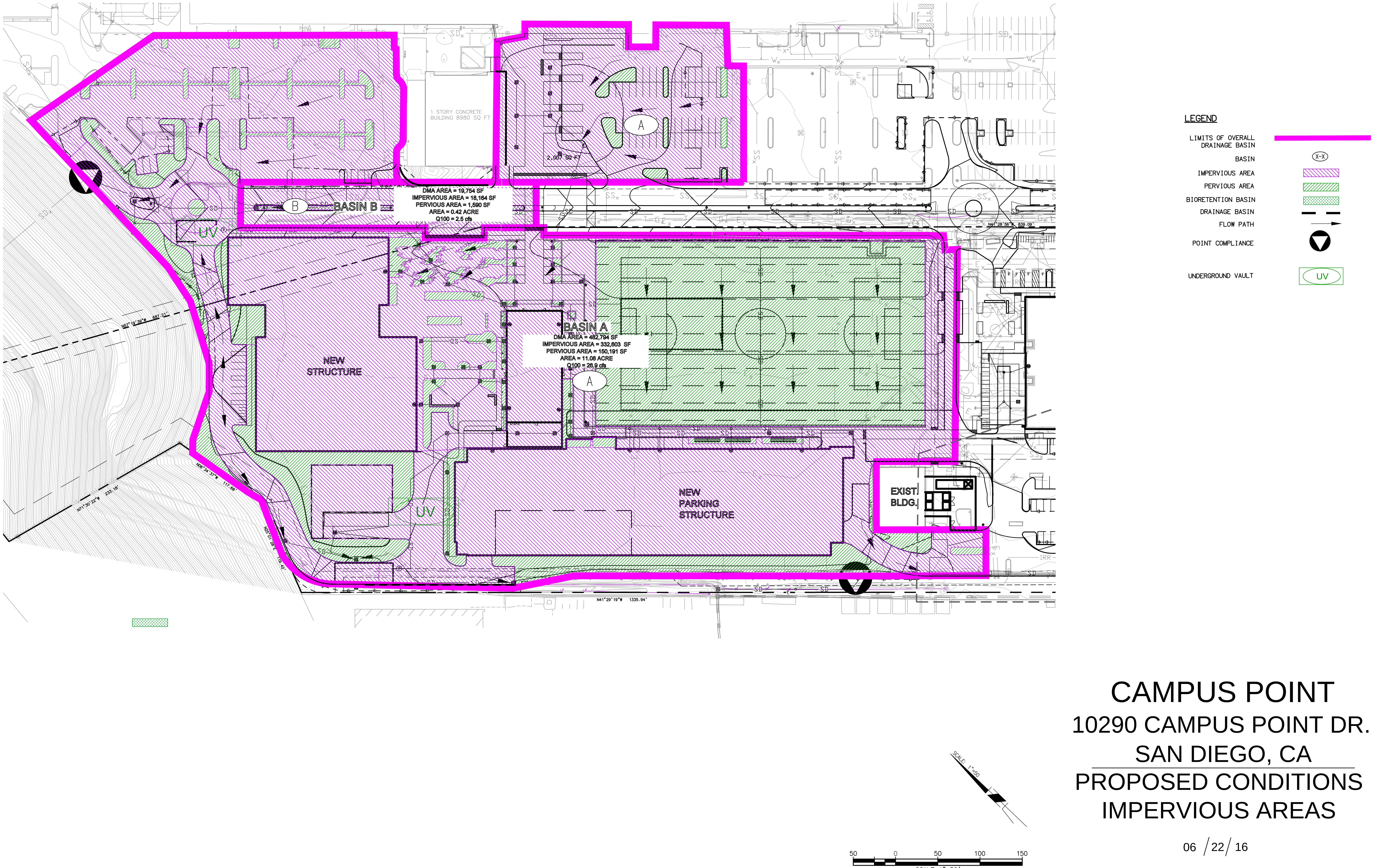
$$I = 7.44 P_6 D^{-0.645}$$

Basin Flow Calculations

Q =	**2.487**	**cfs**
C =	0.86	
I =	5.57	in/hr
A =	0.520	ac.

$$Q = C * I * A$$

RBF Job No. 139861



CAMPUS POINT

10290 CAMPUS POINT DR.

SAN DIEGO, CA

PROPOSED CONDITIONS IMPERVIOUS AREAS

06/22/16

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