GEOTECHNICAL INVESTIGATION

COLLEGE VIEW
5420-22 55TH STREET
SAN DIEGO, CALIFORNIA

PREPARED FOR

PIERCE EDUCATION PROPERTIES, L.P.
SAN DIEGO, CALIFORNIA

OCTOBER 7, 2019
PROJECT NO. G2432-52-01

GEOCON

Project No. G2432-52-01
October 7, 2019

Pierce Education Properties, L.P.
8880 Rio San Diego Drive, Suite 750
San Diego, California 92108

Attention: Mr. Neal L. Singer

Subject: GEOTECHNICAL INVESTIGATION
COLLEGE VIEW
5420-22 55TH STREET
SAN DIEGO, CALIFORNIA

Dear Mr. Singer:

In accordance with your request and authorization of our Proposal No. LG-19288 dated August 1, 2019, we herein submit the results of our geotechnical investigation for the subject project. We performed our investigation to evaluate the underlying soil and geologic conditions and potential geologic hazards, and to assist in the design of the proposed building and associated improvements.

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

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

Very truly yours,

GEOCON INCORPORATED

Ken W. Haase
Senior Staff Geologist

Shawn Foy Weedon
GE 2714

REGISTERED PROFESSIONAL ENGINEER
SHAWN WEEDON
No. 2714
GEOTECHNICAL
STATE OF CALIFORNIA

Michael. C Ertwine
CEG 2659

PROFESSIONAL GEOLOGIST
MICHAEL C. ERTWINE
No. 2659
CERTIFIED ENGINEERING GEOLOGIST
STATE OF CALIFORNIA

KWH:SFW:MCE:kcd

(e-mail) Addressee

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GEOTECHNICAL INVESTIGATION

1. PURPOSE AND SCOPE

This report presents the results of our geotechnical investigation for the proposed College View development located at 5420 through 5422 55th Street, west of the San Diego State University campus in the City of San Diego, California (see Vicinity Map, Figure 1). The purpose of this geotechnical investigation is to evaluate the surface and subsurface soil conditions and general site geology, and to identify geotechnical constraints that may affect development of the property. In addition, we provided recommendations for remedial grading, shallow foundations, concrete slab-on-grade, concrete flatwork, pavement, retaining walls and storm water guidelines.

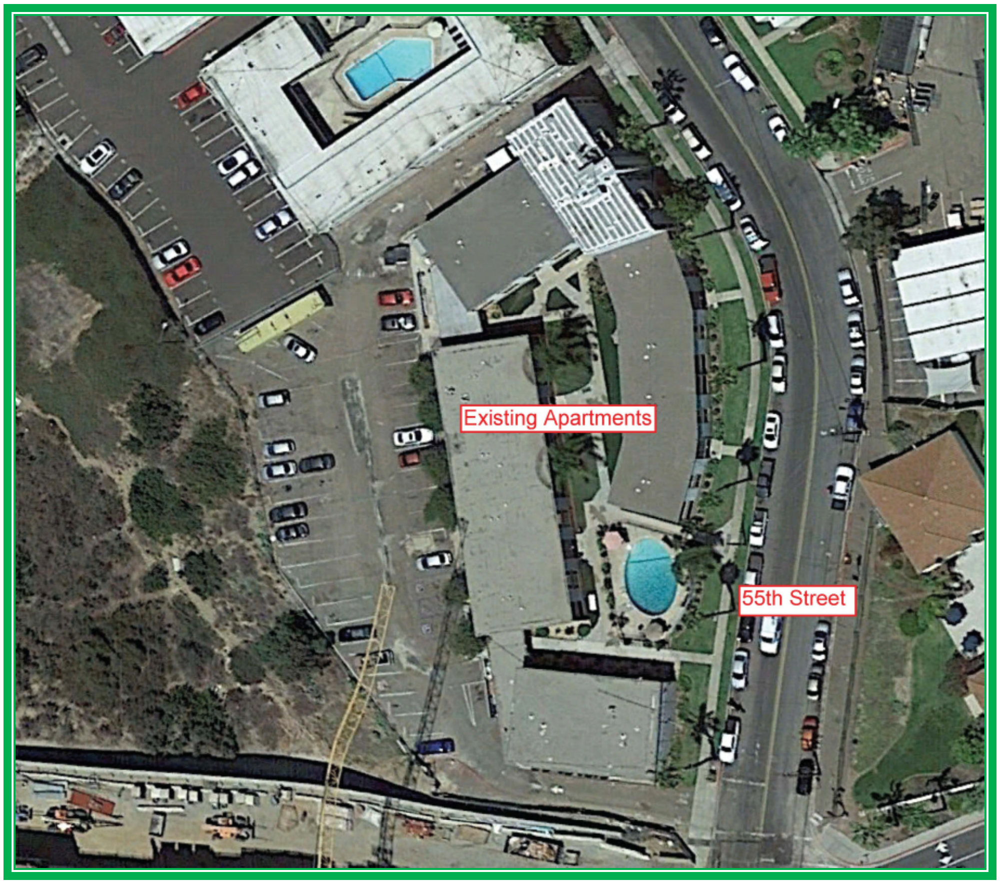
We reviewed the following plans and reports in preparation of this report:

1. *College View Concept Design,* prepared by ktgy Architecture + Planning, dated May 24, 2019 (Project #2018-0195).
2. *Storm Water Management Investigation, College View, 5420-22 55th Street, San Diego, California,* prepared by Geocon Incorporated, dated August 23, 2019 (Project No. G2432-52-01).
3. *Factual Geotechnical Report, West Campus Housing, San Diego State University, Remington Road and 55th Street, San Diego, California 92182* prepared by URS, dated December 17, 2013 (Project No. 27661317.10000).

The scope of this investigation included reviewing readily available published and unpublished geologic literature (see List of References); performing engineering analyses; and preparing this report. We also advanced 4 exploratory borings to a maximum depth of approximately 46½ feet, sampled soil and performed laboratory testing. We performed infiltration testing during our field exploration for the referenced Storm Water Management Investigation for the project. Appendix A presents the exploratory boring logs and details of the field investigation. The details of the laboratory tests and a summary of the test results are shown in Appendix B and on the boring logs in Appendix A.

2. SITE AND PROJECT DESCRIPTION

The site is located at 5420 and 5422 55th Street and situated north of Remington Road and west of the San Diego State University campus. The property currently consists of an apartment complex with 3 buildings with 2- to 4-levels situated at-grade. A parking lot occupies the western portion of the property. Associated hardscape and landscape improvements exist across the property along with a pool. A driveway is present along the north and south sides of the property to provide access to 55th Street. The Existing Site Plan shows the current property conditions. An existing apartment complex occupies the property to the north and a newly constructed student housing building exists to the south. Existing grades gently slope towards the northwest with elevations ranging from approximately 408 to 417 feet Mean Sea Level (MSL) across the site.



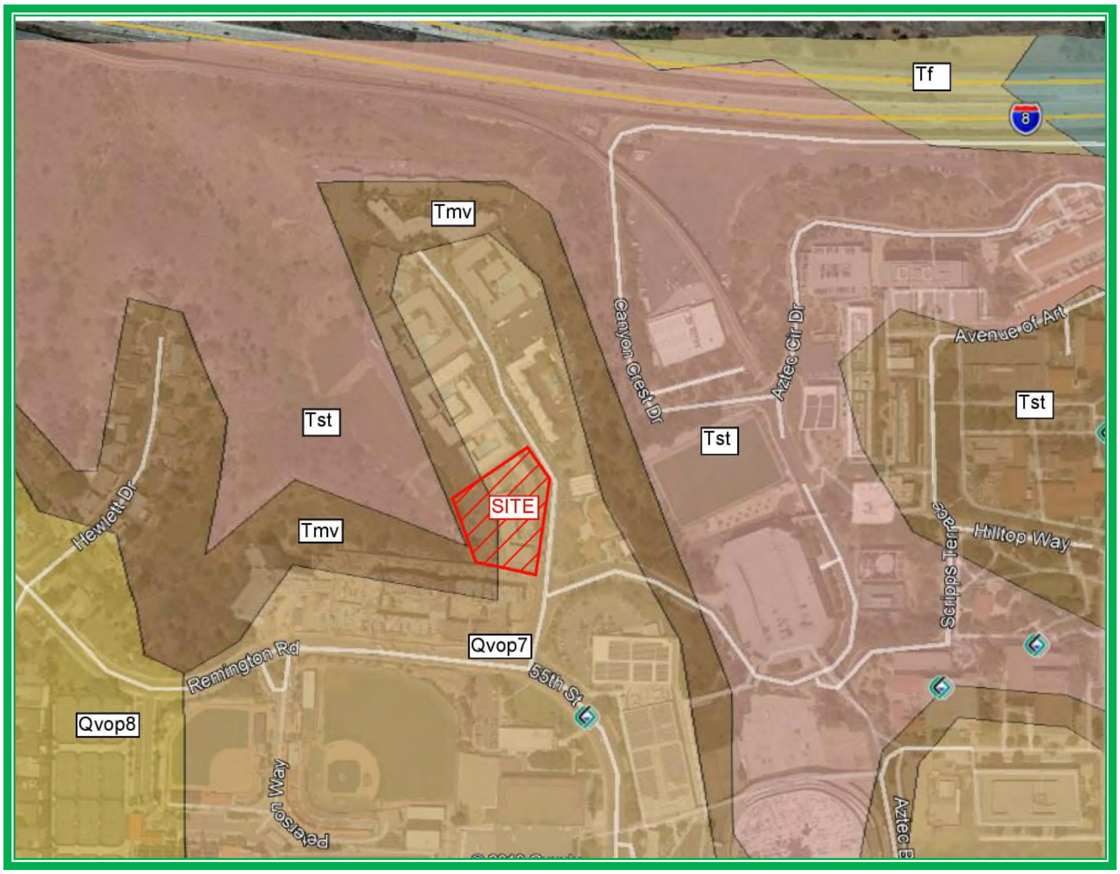
Existing Site Plan

We understand the project will consist of demolishing the existing apartment complex, associated amenities and parking lots then constructing a 6-story, 90 unit residential complex. The complex will consist of associated parking, fitness center, pool deck and amenities on the first and second levels and residential units for the remaining floors. We understand the pool deck will overhang the adjacent canyon and will be supported by piers. Modular wetlands proposed for the first level will be utilized for storm water requirements.

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

3. GEOLOGIC SETTING

Regionally, the site is located in the Peninsular Ranges geomorphic province. The province is bounded by the Transverse Ranges to the north, the San Jacinto Fault Zone on the east, the Pacific Ocean coastline on the west, and the Baja California on the south. The province is characterized by elongated northwest-trending mountain ridges separated by straight-sided sediment-filled valleys. The northwest trend is further reflected in the direction of the dominant geologic structural features of the province that are northwest to west-northwest trending folds and faults, such as the nearby Rose Canyon fault zone. The Regional Geologic Map shows the site in relation to mapped geology in the area (Kennedy Tan, 2008).



Regional Geologic Map

Locally, the site is within the coastal plain of San Diego County. The coastal plain is underlain by a thick sequence of relatively undisturbed and non-conformable sedimentary bedrock units that thicken to the west and range in age from Upper Cretaceous age through the Pleistocene age which have been deposited on Cretaceous to Jurassic age igneous and volcanic bedrock. Geomorphically, the coastal plain is characterized by a series of twenty-one, stair-stepped marine terraces (younger to the west) that have been dissected by west flowing rivers. The coastal plain is a relatively stable block that is dissected by relatively few faults consisting of the potentially active La Nacion Fault Zone and the active Rose Canyon Fault Zone.

The site is located on the western portion of the coastal plain. Marine and non-marine sedimentary units make up the geologic sequence encountered on the site and consist of undivided Pleistocene-age Very Old Paralic Deposits (Qvop) and Eocene-age Mission Valley Formation (Tmv). The Very Old Paralic Deposits are shallow marine deposits generally consisting of sand and silty sand units interfingered with layers of silt and clay. The Stadium Conglomerate (Tst) underlies the Very Old Paralic Deposits and the Mission Valley Formation and consists of marine and non-marine sandstone to silty sandstone.

4. SOIL AND GEOLOGIC CONDITIONS

We encountered a surficial soil unit (consisting of undocumented fill) and three formational units (consisting of Very Old Paralic Deposits, Mission Valley Formation and Stadium Conglomerate). The occurrence, distribution, and description of each unit encountered is shown on the Geologic Map, Figure 2 and on the boring logs in Appendix A. The Geologic Cross-Section, Figure 3, shows the approximate subsurface relationship between the geologic units. The geologic units are described herein in order of increasing age.

4.1 Undocumented Fill (Qudf)

We encountered undocumented fill in our borings up to 4½ feet. The fill is likely associated with the existing development and improvements. Fill may also be located below the existing building. In general, the fill consists of loose to medium dense, moist, clayey sand with abundant gravel and cobble. The fill materials possess a “low” expansion index (expansion index of 21 to 50). The undocumented fill is not considered suitable in its current condition for the support of foundations or structural fill and remedial grading will required. The undocumented fill can be reused as compacted fill during grading operations provided it is free of roots and debris.

4.2 Very Old Paralic Deposits (Qvop)/Mission Valley Formation (Tmv) [Undivided]

Quaternary-age Very Old Paralic Deposits, Unit 7 (formerly called the Lindavista Formation) and Eocene-age Mission Valley Formation underlie the existing fill soil on the eastern portion of the site and is exposed at grade across the western area of the site. Due to difficult drilling conditions and similar geologic properties, the Very Old Paralic Deposits and Mission Valley Formation are described as undivided herein. These units extend to an approximate depth of 35 feet based on previous investigations and geologic mapping. They consist of dense to very dense, damp to moist, light brown to brown, clayey and sandy conglomerate. We expect these materials possess a “very low” to “low” expansive potential (expansion index of 50 or less). We estimate the Very Old Paralic Deposits extend to depths between 10 and 15 feet. The proposed building foundations will likely be embedded within these materials. Excavations within this unit will likely encounter difficult digging conditions in the cemented zones and oversize material with abundant gravel/cobbles will be generated.

4.3 Stadium Conglomerate (Tst)

We likely encountered Eocene-age Stadium Conglomerate within Boring B-1 below the Mission Valley Formation at approximate depth of 35 feet (375 Mean Sea Level) based on drilling conditions and previous geologic mapping. Stadium Conglomerate generally consists of very dense, locally cemented, silty to clayey, fine to medium sandstone to sandy conglomerate. The Stadium Conglomerate generally has a “very low” to “low” expansion potential (expansion index of 50 or less). The Stadium Conglomerate is considered suitable to support additional fill or structural loads. We expect the pier foundations for the western portion of the building may be embedded within the Stadium Conglomerate materials.

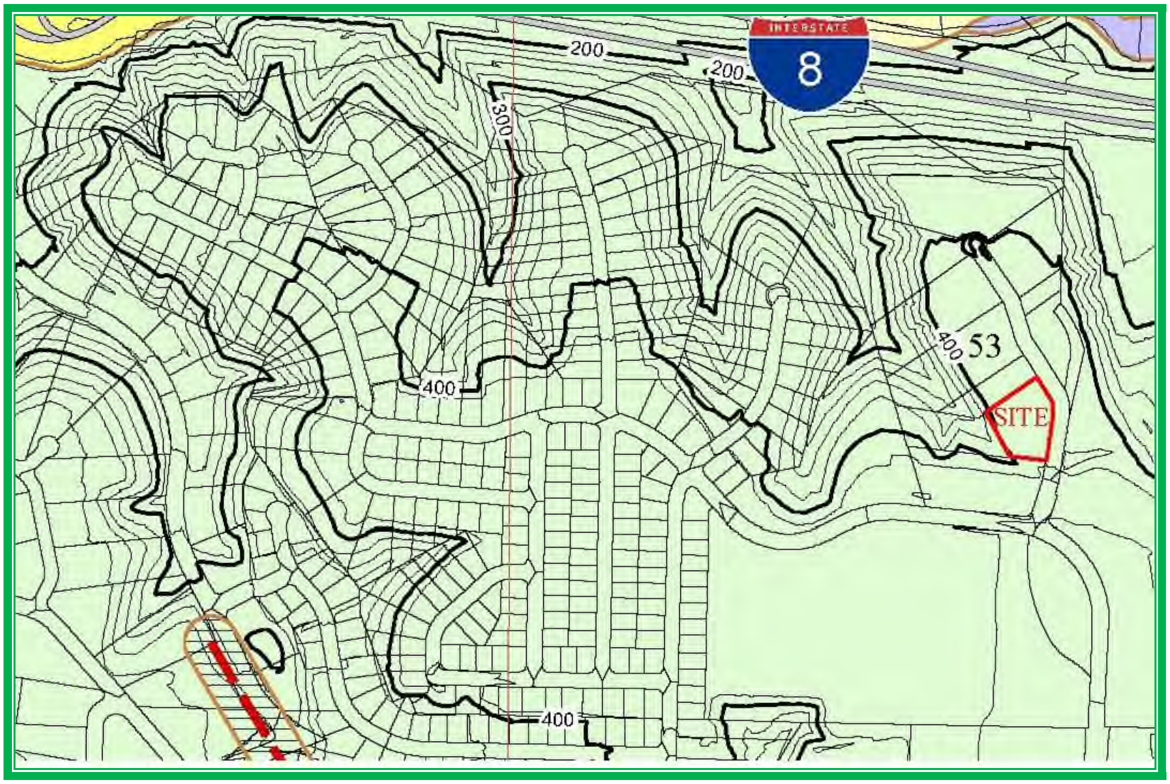
5. GROUNDWATER

We did not encounter groundwater or seepage during our site investigation to the maximum depth explored of 46½ feet. However, it is not uncommon for shallow seepage conditions to develop where none previously existed when sites are irrigated or infiltration is implemented. Seepage is dependent on seasonal precipitation, irrigation, land use, among other factors, and varies as a result. Proper surface drainage will be important to future performance of the project. We expect groundwater is deeper than about 80 feet below existing grade. We do not expect groundwater to be encountered during construction of the proposed development.

6. GEOLOGIC HAZARDS

6.1 Geologic Hazard Category

The City of San Diego Seismic Safety Study, Geologic Hazards and Faults, Map Sheet 22 defines the site with *Hazard Category 53: Level or sloping terrain, unfavorable geologic structure, Low to moderate risk.* Based on a review of the map, a fault does not traverse the planned development area. However, an unnamed fault is mapped about 3,100 feet southwest of the site.



City of San Diego Seismic Safety Study Geologic Hazard and Faults

6.2 Faulting and Seismicity

A review of the referenced geologic materials and our knowledge of the general area indicate that the site is not underlain by known active, potentially active, or inactive faults. An active fault is defined by the California Geological Survey (CGS) as a fault showing evidence for activity within the last 11,000 years. The site is not located within a State of California Earthquake Fault Zone.

According to the computer program *EZ-FRISK* (Version 7.65), 6 known active faults are located within a search radius of 50 miles from the property. We used the 2008 USGS fault database that provides several models and combinations of fault data to evaluate the fault information. Based on this database, the nearest known active fault is the Newport-Inglewood Fault system, located approximately 6 miles west of the site, and is the dominant source of potential ground motion. Earthquakes that might occur on the Newport-Inglewood Fault or other faults within the southern California and northern Baja California area are potential generators of significant ground motion at the site. The estimated deterministic maximum earthquake magnitude and peak ground acceleration for the Newport-Inglewood Fault are 7.5 and 0.34g, respectively. Table 6.2.1 lists the estimated maximum earthquake magnitude and peak ground acceleration for the most dominant faults in relationship to the site location. We calculated peak ground acceleration (PGA) using Boore-Atkinson (2008) NGA USGS2008, Campbell-Bozorgnia (2008) NGA USGS 2008 and Chiou-Youngs (2007) NGA USGS2008 acceleration-attenuation relationships.

TABLE 6.2.1
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	6	7.5	0.29	0.27	0.34
Rose Canyon	6	6.9	0.25	0.26	0.28
Coronado Bank	19	7.4	0.16	0.12	0.14
Palos Verdes Connected	19	7.7	0.18	0.13	0.17
Elsinore	35	7.8	0.12	0.09	0.11
Earthquake Valley	40	6.8	0.07	0.05	0.04

We used the computer program *EZ-FRISK* to perform a probabilistic seismic hazard analysis. The computer program *EZ-FRISK* operates under the assumption that the occurrence rate of earthquakes on each mappable Quaternary fault is proportional to the faults slip rate. The program accounts for fault rupture length as a function of earthquake magnitude, and site acceleration estimates are made using the earthquake magnitude and distance from the site to the rupture zone. The program also accounts for uncertainty in each of following: (1) earthquake magnitude, (2) rupture length for a given magnitude, (3) location of the rupture zone, (4) maximum possible magnitude of a given earthquake, and (5) acceleration at the site from a given earthquake along each fault. By calculating the expected accelerations from considered earthquake sources, the program calculates the total average annual expected number of occurrences of site acceleration greater than a specified value. We utilized acceleration-attenuation relationships suggested by Boore-Atkinson (2008) NGA USGS 2008, Campbell-Bozorgnia (2008) NGA USGS 2008 and Chiou-Youngs (2007) NGA USGS2008 in the analysis. Table 6.2.2 presents the site-specific probabilistic seismic hazard parameters including acceleration-attenuation relationships and the probability of exceedence.

TABLE 6.2.2
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	0.38	0.38	0.43
5% in a 50 Year Period	0.27	0.27	0.29
10% in a 50 Year Period	0.20	0.20	0.20

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

6.3 Ground Rupture

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

6.4 Liquefaction

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

6.5 Seiches and Tsunamis

Seiches are caused by the movement of an inland body of water due to the movement from seismic forces. The potential of seiches to occur is considered to be very low due to the absence of a nearby inland body of water.

A tsunami is a series of long-period waves generated in the ocean by a sudden displacement of large volumes of water. Causes of tsunamis include underwater earthquakes, volcanic eruptions, or offshore slope failures. Wave heights and run-up elevations from tsunamis along the San Diego Coast have historically fallen within the normal range of the tides. The site is located approximately 10 miles from the Pacific Ocean at an elevation of approximately 415 feet above Mean Sea Level. Therefore, the risk of tsunamis affecting the site is negligible.

6.6 Landslides

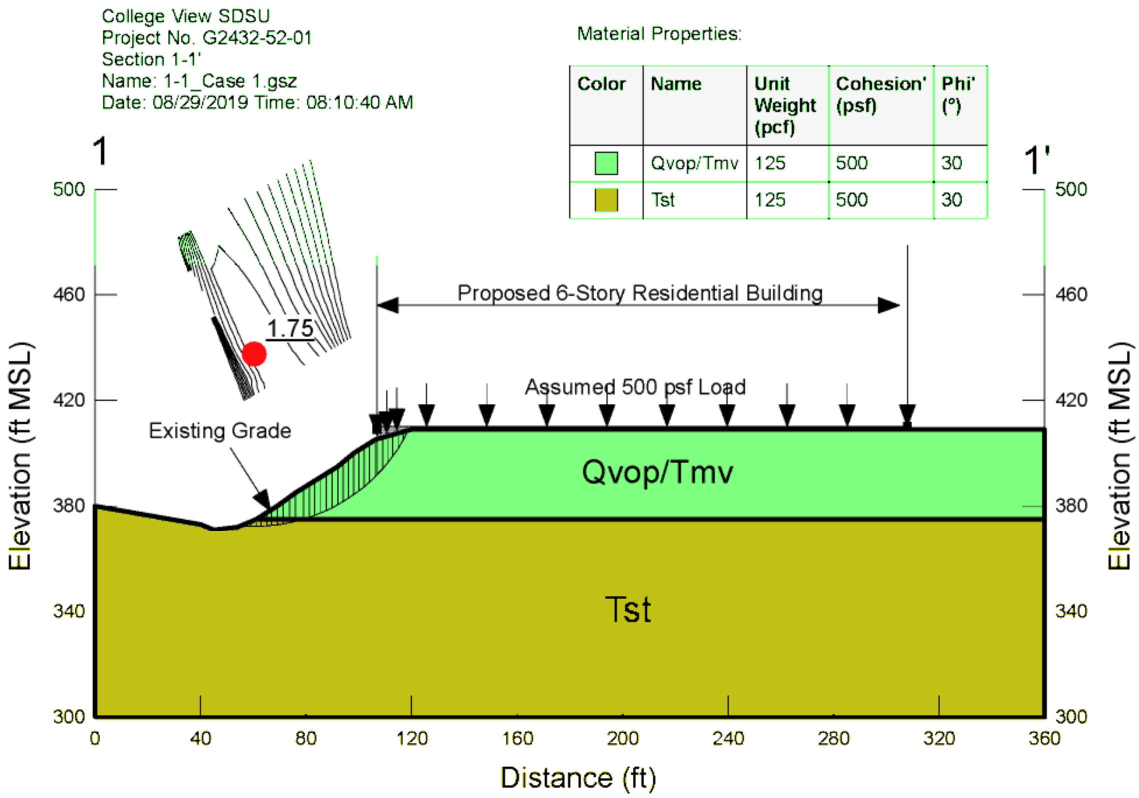
An existing 50-foot high descending slope exists on the western limits of the site. We did not observe evidence of previous or incipient slope instability at the site during our study and the property. Published geologic mapping indicates landslides are not present on or adjacent to the site. Although landslides are present on the north side of Interstate 8 in Mission Valley, this is primarily caused by the outcropped Friars Formation. The Friars Formation is not widely outcropped on the south side of Interstate 8 or nearby the project site. Therefore, in our professional opinion, the potential for a landslide is not a significant concern for this project. However, lateral movement associated with slope creep could occur to structures and improvements located adjacent to slopes.

6.7 Slope Stability

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

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

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



Color	Name	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)
	Qvop/Tmv	125	500	30
	Tst	125	500	30

Slope Stability Analysis for 1-1'

7. CONCLUSIONS AND RECOMMENDATIONS

7.1 General

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

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

7.1.3 The undocumented fill is potentially compressible and unsuitable in its present condition for the support of compacted fill or settlement-sensitive improvements. Remedial grading of these materials should be performed as discussed herein. The Very Old Paralic Deposits and Mission Valley Formation materials are considered suitable for the support of proposed fill and structural loads.

7.1.4 We did not encounter groundwater during our subsurface exploration and we do not expect it to be a constraint to project development. However, seepage within surficial soils and rock materials may be encountered during the grading operations, especially during the rainy seasons.

7.1.5 Excavation of the undocumented fill, Very Old Paralic Deposits and Mission Valley Formation should generally be possible with moderate to heavy effort using conventional, heavy-duty equipment during grading and trenching operations. Very heavy effort should be expected if localized zones of moderately cemented material or gravel/cobble are encountered.

7.1.6 Proper drainage should be maintained in order to preserve the engineering properties of the fill in both the building pads and slope areas.

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

7.2 Excavation and Soil Characteristics

7.2.1 Excavation of the surficial soil should be possible with moderate effort using conventional heavy-duty equipment. Excavation of the formational materials will require moderate to heavy effort and may generate oversized material due to the presence of gravel and cobble in these units.

7.2.2 The soil encountered in the field investigation is considered to be "non-expansive" and "expansive" (expansion index [EI] of 20 or less and greater than 20, respectively) as defined by 2016 California Building Code (CBC) Section 1803.5.3. Table 7.2 presents soil classifications based on the expansion index. We expect a majority of the soil encountered possess a "very low" to "low" expansion potential (EI of 50 or less) in accordance with ASTM D 4829.

TABLE 7.2
EXPANSION CLASSIFICATION BASED ON EXPANSION INDEX

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

7.2.3 We performed laboratory tests on samples of the site materials to evaluate the percentage of water-soluble sulfate content. Appendix B presents results of the laboratory water-soluble sulfate content tests. The test results indicate the on-site materials at the locations tested possess "S0" sulfate exposure to concrete structures as defined by 2016 CBC Section 1904 and ACI 318-14 Chapter 19. 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.

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

7.3 Excavation Slopes

7.3.1 The recommendations included herein are provided for stable excavations. It is the responsibility of the contractor and their competent person to ensure all excavations, temporary slopes and trenches are properly constructed and maintained in accordance with applicable OSHA guidelines in order to maintain safety and the stability of the excavations and adjacent improvements. These excavations should not be allowed to become saturated or to dry out. Surcharge loads should not be permitted to a distance equal to the height of the excavation from the top of the excavation. The top of the excavation should be a minimum of 15 feet from the edge of existing improvements. Excavations steeper than those recommended or closer than 15 feet from an existing surface improvement should be shored in accordance with applicable OSHA codes and regulations.

7.3.2 The stability of the excavations is dependent on the design and construction of the shoring system and site condition. Therefore, Geocon Incorporated cannot be responsible for site safety and the stability of the proposed excavations.

7.4 Seismic Design Criteria – California Building Code

7.4.1 We used the Structural Engineers Association of California (SEAOC) and Office of Statewide Health Planning and Development (OSHPD) web application *Seismic Design Maps* to evaluate site-specific seismic design parameters in accordance with the 2016 CBC/ASCE 7-10, Chapter 16 Structural Design, Section 1613 Earthquake Loads. The short spectral response uses a period of 0.2 second. Building and improvements should be designed using a soil Site Class C. We evaluated the soil Site Class based on the discussion in Section 1613.3.2 of the 2016 CBC and Table 20.3-1 of ASCE 7-10. The values presented in Table 7.4.1 are for the risk-targeted maximum considered earthquake (MCE_R).

TABLE 7.4.1
2016 CBC SEISMIC DESIGN PARAMETERS

Parameter	Value	2016 CBC Reference
Site Class	C	Section 1613.3.2
Spectral Response – Class B (short), S_S	0.935g	Figure 1613.3.1(1)
Spectral Response – Class B (1 sec), S_1	0.358g	Figure 1613.3.1(2)
Site Coefficient, F_a	1.026	Table 1613.3.3(1)
Site Coefficient, F_v	1.442	Table 1613.3.3(2)
Maximum Considered Earthquake Spectral Response Acceleration (short), S_{MS}	0.959g	Section 1613.3.3 (Eqn 16-37)
Maximum Considered Earthquake Spectral Response Acceleration – (1 sec), S_{M1}	0.517g	Section 1613.3.3 (Eqn 16-38)
5% Damped Design Spectral Response Acceleration (short), S_{DS}	0.640g	Section 1613.3.4 (Eqn 16-39)
5% Damped Design Spectral Response Acceleration (1 sec), S_{D1}	0.344g	Section 1613.3.4 (Eqn 16-40)

7.4.2 Table 7.4.2 presents additional seismic design parameters for projects located in Seismic Design Categories of D through F in accordance with ASCE 7-10 for the mapped maximum considered geometric mean (MCE_G).

TABLE 7.4.2
2016 CBC SITE ACCELERATION DESIGN PARAMETERS

Parameter	Value	ASCE 7-10 Reference or 2016 CBC Reference
Site Class	C	Section 1613.3.2
Mapped MCE_G Peak Ground Acceleration, PGA	0.375g	Figure 22-7
Site Coefficient, F_{PGA}	1.025	Table 11.8-1
Site Class Modified MCE_G Peak Ground Acceleration, PGA_M	0.384g	Section 11.8.3 (Eqn 11.8-1)

7.4.3 The project structural engineer and architect should evaluate the appropriate Risk Category and Seismic Design Category for the planned structures. The values presented herein assume a Rick Category of I, II or III and resulting in a Seismic Design Category D.

7.4.4 Conformance to the criteria in Tables 7.4.1 and 7.4.2 for seismic design does not constitute any kind of guarantee or assurance that significant structural damage or ground failure will not occur if a large earthquake occurs. The primary goal of seismic design is to protect life, not to avoid all damage, since such design may be economically prohibitive.

7.5 Grading

7.5.1 Grading should be performed in accordance with the recommendations provided in this with the Recommended Grading Specifications contained in Appendix D. Where the recommendations of this section conflict with those of Appendix D, the recommendations of this section take precedence. Geocon Incorporated should observe the grading operations on a full-time basis and provide testing during the fill placement.

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

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

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

7.5.5 We expect that the majority of the planned structure will be founded on a shallow foundation system bearing in the Very Old Paralic Deposits. The remedial grading within the building area should extend through the undocumented fill to exposed the underlying Very Old Paralic Deposits and be replaced with properly compacted fill. The removals should extend at least 10 feet outside the perimeter of the proposed building and/or footings, where possible. We expect that the western portion of the building will be supported by a pier system over the existing slope and that grading within that area will likely be limited.

7.5.6 The upper 2 feet of materials within non-building improvement areas should be removed and replaced with properly compacted fill. The removals should extend at least 2 feet outside the improvement areas, where possible. Prior to fill soil being placed, the existing ground surface should be scarified, moisture conditioned as necessary, and compacted to a depth of at least 12 inches. Deeper removals may be required if saturated or loose fill soil is encountered. A representative of Geocon should be on-site during removals to evaluate the limits of the remedial grading. Table 7.5.1 provides a summary of the grading recommendations.

TABLE 7.5.1
SUMMARY OF GRADING RECOMMENDATIONS

Area	Removal Requirements
Site Development	Removal of Upper 2 Feet of Existing Materials
Building Pad (Shallow Foundations)	Removal to Very Old Paralic Deposits
Grading Limits	10 Feet Outside of Building/2 Feet Outside of Improvement Area
Exposed Bottoms of Remedial Grading	Scarify Upper 12 Inches

7.5.7 The site should then be brought to final subgrade elevations with fill compacted in layers. In general, soil native to the site is suitable for use from a geotechnical engineering standpoint as fill if relatively free from vegetation, debris and other deleterious material. The fill materials should be placed and compacted in accordance with Table 7.5.2.

TABLE 7.5.2
SUMMARY OF SOIL COMPACTION RECOMMENDATIONS

Parameter	Recommendations
Fill Lift Thickness	6 to 8 inches in Loose Condition
Compaction – Scarified Bottom Excavations, Fill and Wall/Utility Backfill	90 Percent of the Laboratory Maximum Dry Density
Compaction – Pavement Subgrade (Upper 12 Inches) and Base Materials	95 Percent of the Laboratory Maximum Dry Density
Moisture Content	Near to Slightly Above Optimum

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

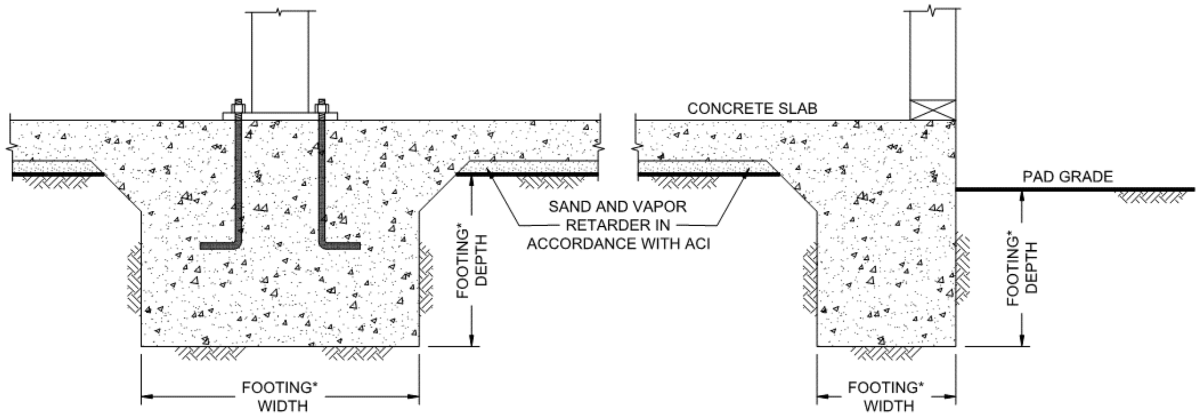
7.6 Shallow Foundations

7.6.1 The proposed structure can be supported on a shallow foundation system founded in the Very Old Paralic Deposits and/or Mission Valley Formation. Foundations for the structure should consist of continuous strip footings and/or isolated spread footings. Table 7.6 provides a summary of the foundation design recommendations.

TABLE 7.6
SUMMARY OF FOUNDATION RECOMMENDATIONS

Parameter	Value
Minimum Continuous Foundation Width	12 inches
Minimum Isolated Foundation Width	24 inches
Minimum Foundation Depth	24 Inches Below Lowest Adjacent Grade
	At Least 12 Inches into Very Old Paralic Deposits/Mission Valley Formation
Minimum Steel Reinforcement – Continuous Foundations	4 No. 5 Bars, 2 at the Top and 2 at the Bottom
Minimum Steel Reinforcement – Isolated Foundations	Per Structural Engineer
Allowable Bearing Capacity	4,000 psf
Bearing Capacity Increase	500 psf per Foot of Depth
	500 psf per Foot of Width
Maximum Allowable Bearing Capacity	8,000 psf
Estimated Total Settlement	½ Inch (6-Foot-Square Footing)
	1 Inch (12-Foot-Square Footing)
Estimated Differential Settlement	½ Inch in 40 Feet
Design Expansion Index	50 or less

7.6.2 The foundations should be embedded in accordance with the recommendations herein and the Wall/Column Footing Dimension Detail. The embedment depths should be measured from the lowest adjacent pad grade for both interior and exterior footings. Footings should be deepened such that the bottom outside edge of the footing is at least 7 feet horizontally from the face of the slope (unless designed with a post-tensioned foundation system as discussed herein).



Wall/Column Footing Dimension Detail

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

7.6.4 Overexcavation of the footings and replacement with slurry can be performed in areas where Very Old Paralic Deposits or Mission Valley Formation materials are not encountered at the bottom of the footing. Minimum two-sack slurry can be placed in the excavations for the conventional foundations to the bottom of proposed footing elevation.

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

- For fill slopes less than 20 feet high, building footings should be deepened such that the bottom outside edge of the footing is at least 7 feet horizontally from the face of the slope.
- When located next to a descending 3:1 (horizontal:vertical) fill slope or steeper, the foundations should be extended to a depth where the minimum horizontal distance is equal to H/3 (where H equals the vertical distance from the top of the fill slope to the base of the fill soil) with a minimum of 7 feet but need not exceed 40 feet. The horizontal distance is measured from the outer, deepest edge of the footing to the face of the slope.

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

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

7.7 Concrete Slabs-On-Grade

7.7.1 Concrete slabs-on-grade for the structures should be constructed in accordance with Table 7.7.

TABLE 7.7
MINIMUM CONCRETE SLAB-ON-GRADE RECOMMENDATIONS

Parameter	Value
Minimum Concrete Slab Thickness	5 inches
Minimum Steel Reinforcement	No. 3 Bars 18 Inches on Center, Both Directions
Typical Slab Underlayment	3 to 4 Inches of Sand/Gravel/Base
Design Expansion Index	50 or less

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

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

7.7.4 Concrete slabs should be provided with adequate crack-control joints, construction joints and/or expansion joints to reduce unsightly shrinkage cracking. The design of joints should consider criteria of the American Concrete Institute (ACI) when establishing crack-control spacing. Crack-control joints should be spaced at intervals no greater than 12 feet. Additional steel reinforcing, concrete admixtures and/or closer crack control joint spacing should be considered where concrete-exposed finished floors are planned.

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

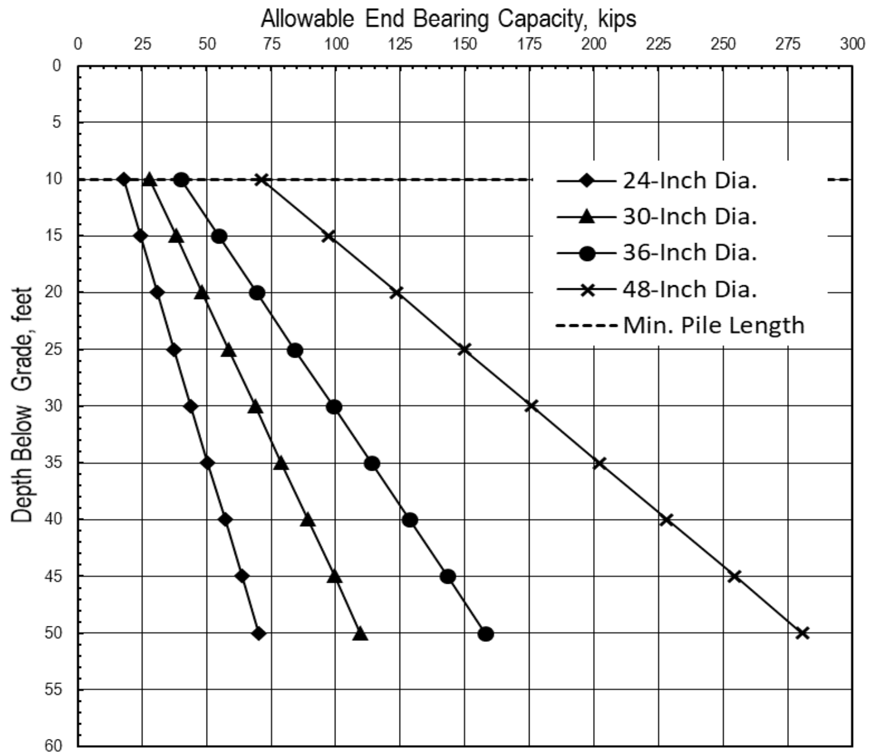
7.7.6 The concrete slab-on-grade recommendations are based on soil support characteristics only. The project structural engineer should evaluate the structural requirements of the concrete slabs for supporting expected loads.

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

7.8 Drilled Pier Recommendations

7.8.1 We understand that drilled piers may be used for foundation support for the portion of the building supported over the existing slope. The foundation recommendations herein assume that the piers will be at least 10 feet long and embedded at least 5 feet within the Very Old Paralic Deposits, Mission Valley or Stadium Conglomerate materials.

7.8.2 Piers can be designed to develop support by end bearing and skin friction within the formational materials. An allowable skin friction resistance of 400 psf can be used for the portion of the drilled pier embedded in formational materials. The end bearing capacity can be determined by the End Bearing Capacity Chart. These allowable values possess a factor of safety of at least 2 and 3 for skin friction and end bearing, respectively.



End Bearing Capacity Chart

7.8.3 The diameter of the piers should be a minimum of 24-inches. The piles should be embedded into the formational materials at least 5 feet and have a minimum length of 10 feet. The design length of the drilled piers should be determined by the designer based on the elevation of the pile cap or grade beam and the elevation of the top of the formational materials obtained from the Geologic Map and Geologic Cross-Sections presented herein. It is difficult to evaluate the exact length of the proposed drilled piers due to the variable thickness of the existing fill; therefore, some variation should be expected during drilling operations.

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

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

7.8.6 The formational materials contain gravel and cobble and may possess very dense/cemented zones; therefore, the drilling contractor should expect difficult drilling conditions during excavations for the piers. Because a significant portion of the piers capacity will be developed by end bearing, the bottom of the borehole should be cleaned of loose cuttings prior to the placement of steel and concrete. Experience indicates that backspinning the auger does not remove loose material and a flat cleanout plate is necessary. We expect localized seepage may be encountered during the drilling operations and casing may be required to maintain the integrity of the pier excavation, particularly if seepage or sidewall instability is encountered. 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.

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

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

7.9 Exterior Concrete Flatwork

7.9.1 Exterior concrete flatwork not subject to vehicular traffic should be constructed in accordance with the recommendations presented in Table 7.9. The recommended steel reinforcement would help reduce the potential for cracking.

TABLE 7.9
MINIMUM CONCRETE FLATWORK RECOMMENDATIONS

Expansion Index, EI	Minimum Steel Reinforcement* Options	Minimum Thickness
EI $\leq$ 90	6x6-W2.9/W2.9 (6x6-6/6) welded wire mesh	4 Inches
	No. 3 Bars 18 inches on center, Both Directions	

*In excess of 8 feet square.

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

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

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

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

7.10 Retaining Walls

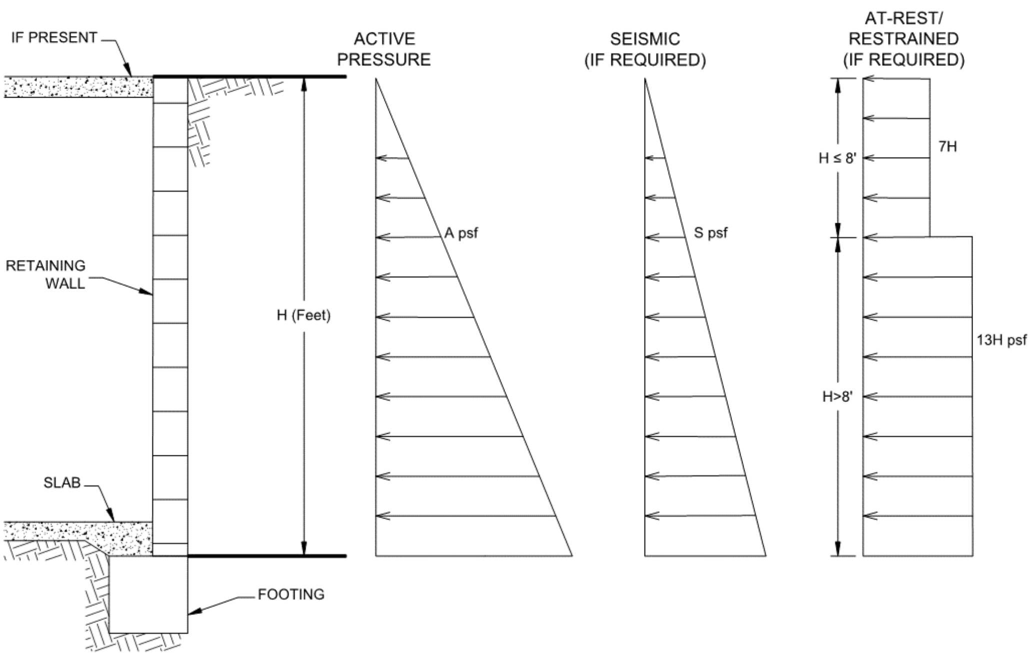
7.10.1 Retaining walls should be designed using the values presented in Table 7.10.1. Soil with an expansion index (EI) of greater than 50 should not be used as backfill material behind retaining walls.

TABLE 7.10.1
RETAINING WALL DESIGN RECOMMENDATIONS

Parameter	Value EI≤50
Active Soil Pressure, A (Fluid Density, Level Backfill)	35 pcf
Active Soil Pressure, A (Fluid Density, 2:1 Sloping Backfill)	45 psf
Seismic Pressure, S	17H psf
At-Rest/Restrained Walls Additional Uniform Pressure (0 to 8 Feet High)	7H psf
At-Rest/Restrained Walls Additional Uniform Pressure (8+ Feet High)	13H psf

H equals the height of the retaining portion of the wall

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



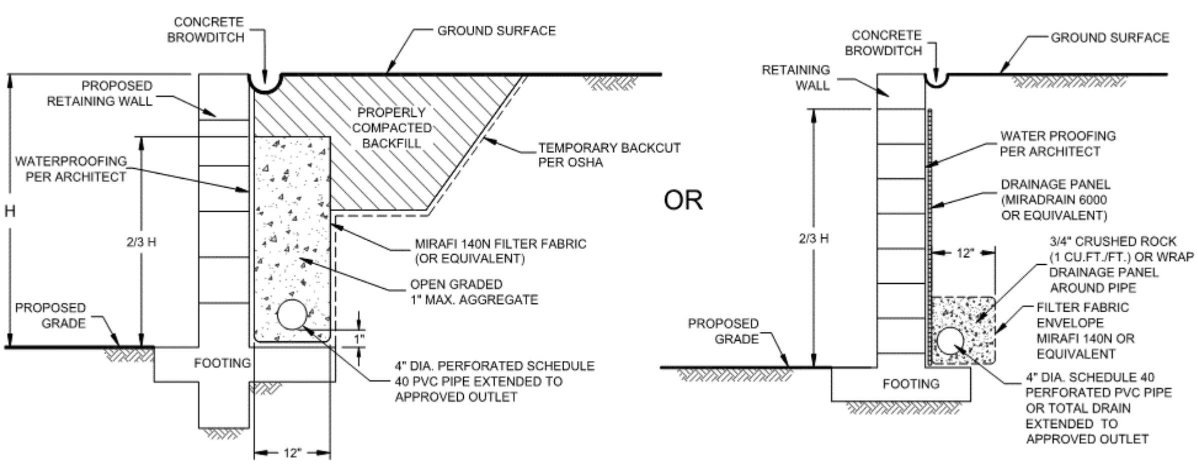
Retaining Wall Loading Diagram

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

7.10.4 The structural engineer should determine the Seismic Design Category for the project in accordance with Section 1613.3.5 of the 2016 CBC or Section 11.6 of ASCE 7-10. For structures assigned to Seismic Design Category of D, E, or F, retaining walls that support more than 6 feet of backfill should be designed with seismic lateral pressure in accordance with Section 1803.5.12 of the 2016 CBC. The seismic load is dependent on the retained height where H is the height of the wall, in feet, and the calculated loads (S) result in pounds per square foot (psf) exerted at the base of the wall and zero at the top of the wall.

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

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



Typical Retaining Wall Drainage Detail

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

7.10.8 In general, wall foundations having should be designed in accordance with Table 7.10.2. The proximity of the foundation to the top of a slope steeper than 3:1 could impact the allowable soil bearing pressure. Therefore, retaining wall foundations should be deepened such that the bottom outside edge of the footing is at least 7 feet horizontally from the face of the slope.

TABLE 7.10.2
SUMMARY OF RETAINING WALL FOUNDATION RECOMMENDATIONS

Parameter	Value
Minimum Retaining Wall Foundation Width	12 inches
Minimum Retaining Wall Foundation Depth	12 Inches
Minimum Steel Reinforcement	Per Structural Engineer
Bearing Capacity	2,000 psf
Estimated Total Settlement	1 Inch
Estimated Differential Settlement	½ Inch in 40 Feet

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

7.10.10 Unrestrained walls will move laterally when backfilled and loading is applied. The amount of lateral deflection is dependent on the wall height, the type of soil used for backfill, and loads acting on the wall. The retaining walls and improvements above the retaining walls should be designed to incorporate an appropriate amount of lateral deflection as determined by the structural engineer.

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

7.11 Lateral Loading

7.11.1 Table 7.11 should be used to help design the proposed structures and improvements to resist lateral loads for the design of footings or shear keys. The allowable passive pressure assumes a horizontal surface extending at least 5 feet, or three times the surface generating the passive pressure, whichever is greater. The upper 12 inches of material in areas not protected by floor slabs or pavement should not be included in design for passive resistance.

TABLE 7.11
SUMMARY OF LATERAL LOAD DESIGN RECOMMENDATIONS

Parameter	Value
Passive Pressure Fluid Density	350 pcf
Coefficient of Friction (Concrete and Soil)	0.35
Coefficient of Friction (Along Vapor Barrier)	0.2 to 0.25*

*Per manufacturer's recommendations.

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

7.12 Preliminary Pavement Recommendations

7.12.1 We calculated the flexible pavement sections in general conformance with the *Caltrans Method of Flexible Pavement Design* (Highway Design Manual, Section 608.4) using an estimated Traffic Index (TI) of 5.0, 5.5, 6.0, and 7.0 for parking stalls, driveways, medium truck traffic areas, and heavy truck traffic areas, respectively. The project civil engineer and owner should review the pavement designations to determine appropriate locations for pavement thickness. The final pavement sections for the parking lot should be based on the R-Value of the subgrade soil encountered at final subgrade elevation. We assumed a R-Value of 20 and 78 for the subgrade soil and base materials, respectively, for the purposes of this preliminary analysis. Table 7.12.1 presents the preliminary flexible pavement sections.

TABLE 7.12.1
PRELIMINARY FLEXIBLE PAVEMENT SECTION

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

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

7.12.3 A rigid Portland cement concrete (PCC) pavement section should be placed in roadway aprons and cross gutters. We calculated the rigid pavement section in general conformance with the procedure recommended by the American Concrete Institute report ACI 330R-08 Guide for Design and Construction of Concrete Parking Lots using the parameters presented in Table 7.12.2.

TABLE 7.12.2
RIGID PAVEMENT DESIGN PARAMETERS

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

7.12.4 Based on the criteria presented herein, the PCC pavement sections should have a minimum thickness as presented in Table 7.12.3.

TABLE 7.12.3
RIGID VEHICULAR PAVEMENT RECOMMENDATIONS

Location	Portland Cement Concrete (inches)
Automobile Parking Stalls (TC=A)	6.0
Driveways (TC=C)	7.0

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

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

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

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

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

7.12.10 We understand grasscrete may be utilized for the emergency vehicle access lanes on the north and south sides of the project. We calculated the grasscrete paver section in general conformance with the *Caltrans Method of Flexible Pavement Design* (Highway Design Manual, Section 608.4) using an estimated Traffic Index (TI) of 6.0. We understand the Grasscrete paver possesses an equivalent asphalt thickness of 3 inches. The Grasscrete should be installed in accordance with the manufacturer's recommendations. Table 7.12.4 presents two options for the paver underlayment using compacted Class 2 permeable base materials or aggregate rock. Table 7.12.4 presents the recommended permeable paver pavement section.

TABLE 7.12.4
GRASSCRETE PAVEMENT SECTION

Location	Traffic Index (TI)	Assumed Subgrade R-Value	Equivalent Paver Asphalt Concrete Thickness (inches)	Option 1		Option 2
				Estimated Sand Thickness (inches)	Base Materials (inches)	ASTM C 33 Aggregate
Emergency Vehicle Access	6.0	20	3	1 -1½	12	3" Sand / 3" #8 / 9" #57

7.12.11 The aggregate presented in Option 2 should be in conformance with ASTM C33 as shown in Table 7.12.5.

TABLE 7.12.5
AGGREGATE GRADATION LIMITS PER ASTM C33

Sieve Size	Percent Passing Sieves		
	Choker Sand	No. 8	No. 57
1.5 Inches	--	--	100
1 Inch	--	--	95-100
0.5 Inch	--	100	25-60
0.375 Inch	100	85-100	--
No. 4	95-100	10-30	0-10
No. 8	80-100	0-10	0-5
No. 16	50-85	0-5	--
No. 30	25-60	--	--
No. 50	5-30	--	--
No. 100	0-10	--	--
No. 200	0-3	--	--

7.12.12 The Class 2 permeable base/aggregate section can be thickened to increase the water capacity as required by the project civil engineer. Prior to placing base/aggregate materials, the subgrade soil should be scarified, moisture conditioned as necessary, and recompacted to a dry density of at least 95 percent of the laboratory maximum dry density near to slightly above optimum moisture content as determined by ASTM D 1557. The depth of compaction should be at least 12 inches. Similarly, the base materials 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. We are not able to perform compaction curves on aggregate; however, some compactive effort should be applies during the installation of the aggregate.

7.12.13 The grasscrete should be installed and maintained in accordance with the manufacturer's recommendations. The owners should be made aware and responsible for the maintenance program. In addition, the grasscrete pavement tends to shift vertically and horizontally during the life of the pavement and should be expected. The grasscrete normally requires a concrete border to prevent lateral movement from traffic. The concrete border surrounding the pavers should be embedded at least 6 inches into the subgrade to reduce the potential for water migration to the adjacent landscape areas and pavement areas.

7.12.14 The subgrade of the Grasscrete areas should be graded to allow water to flow to a subdrain at a minimum gradient of 2 percent. A subdrain should be installed within the base/aggregate materials at the low point of the subgrade to reduce the potential for water to build up within the paving section. The subdrain can be elevated above the subgrade a maximum of 3 inches within the base section. The subdrain should be connected to an approved drainage device. The subdrain should consist of a 3-inch diameter perforated Schedule 40, PVC pipe. A continuous impermeable liner or rigid impermeable barrier should be installed along the sides of the water quality paver section to prevent water migration. The liner or impermeable barrier should consist of a high-density polyethylene (HDPE) with a minimum thickness of 15 mil or equivalent and extend at least 12-inches below the subgrade elevation. The liner/barrier should be sealed at the connections in accordance with manufacturer recommendations and should be properly waterproofed at the drain connection.

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

7.13 Site Drainage and Moisture Protection

7.13.1 Adequate site drainage is critical to reduce the potential for differential soil movement, erosion and subsurface seepage. Under no circumstances should water be allowed to pond adjacent to footings. The site should be graded and maintained such that surface drainage is directed away from structures in accordance with 2016 CBC 1804.4 or other applicable standards. In addition, surface drainage should be directed away from the top of slopes into swales or other controlled drainage devices. Roof and pavement drainage should be directed into conduits that carry runoff away from the proposed structure.

7.13.2 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.

7.13.3 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.

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

7.13.5 Our report, *Storm Water Management Investigation, College View 5420-22 55th Street, San Diego, California* dated August 23, 2019 should be incorporated into the site drainage and storm water design for the proposed development. We should be contacted to update the recommendations, as necessary.

7.14 Grading and Foundation Plan Review

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

LIMITATIONS AND UNIFORMITY OF CONDITIONS

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

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

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

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

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N

NO SCALE

VICINITY MAP

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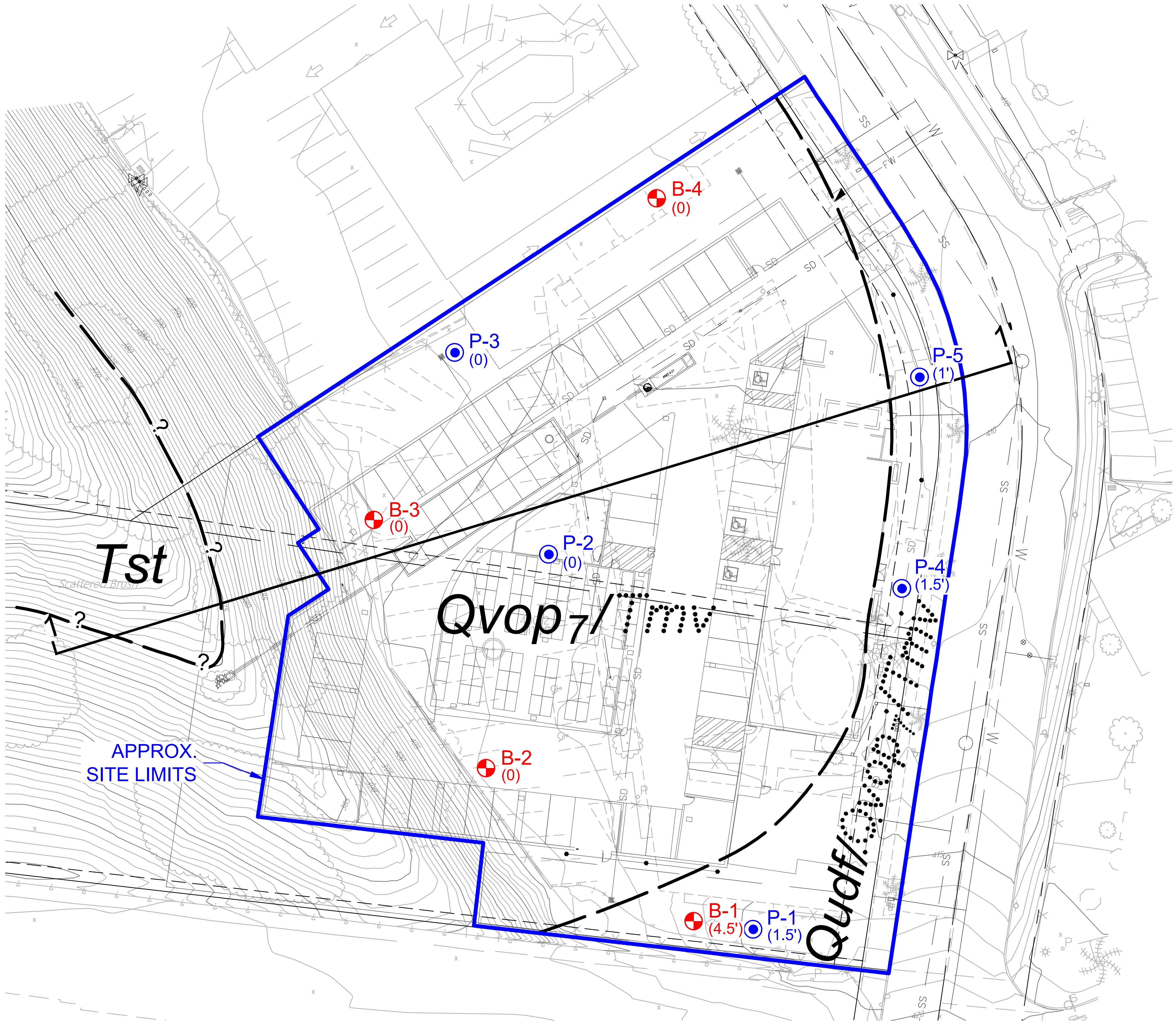
COLLEGE VIEW
5420-22 55TH STREET
SAN DIEGO, CALIFORNIA

RM / AML		DSK/GTYPD	DATE 10 - 07 - 2019	PROJECT NO. G2432 - 52 - 01	FIG. 1

Plotted:10/07/2019 8:09AM \| By:JONATHAN WILKINS \| File Location:Y:\PROJECTS\G2432-52-01 (College View)\DETAILS\G2432-52-01 Vic Map.dwg

COLLEGE VIEW
5420-22 55TH STREET
SAN DIEGO, CALIFORNIA



0' 20' 40' 60' 80'

SCALE 1"=40'(on 11x17)

GEOCON LEGEND

Qudf	........UNDOCUMENTED FILL
$Qvop_7$	........VERY OLD PARALIC DEPOSITS (Dotted Where Buried)
Tmv	........MISSION VALLEY FORMATION (Dotted Where Buried)
Tst	........STADIUM CONGLOMERATE
?	........APPROX. LOCATION OF GEOLOGIC CONTACT (Queried Where Uncertain)
B-4	........APPROX. LOCATION OF BORING
P-5	........APPROX. LOCATION OF AARDVARK PERMEAMETER TEST
(4')	........APPROX. DEPTH TO FORMATION (Feet)
1 1'	........APPROX. LOCATION OF GEOLOGIC CROSS SECTION

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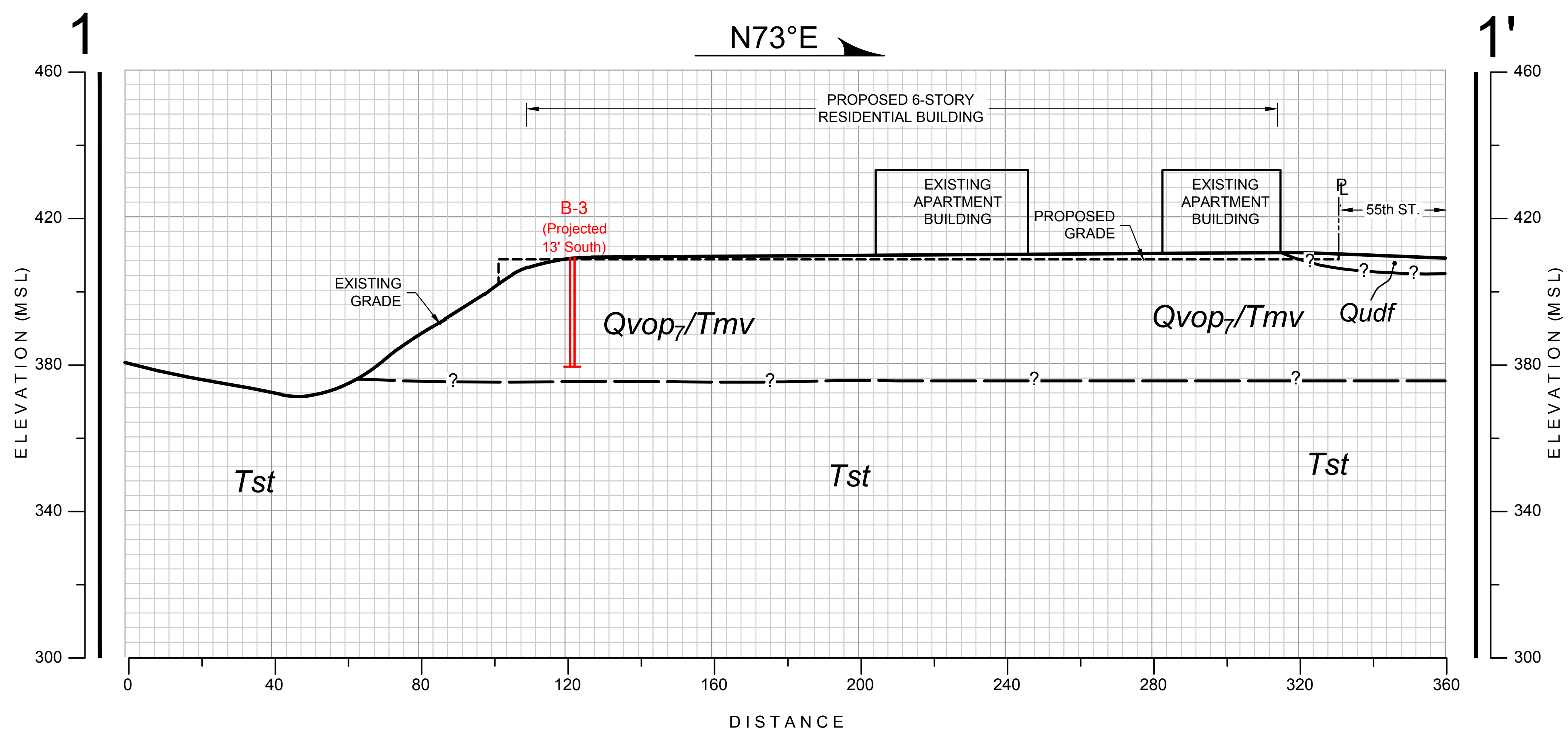
GEOLOGIC MAP

PROJECT NO. G2432 - 52 - 01
FIGURE 2
DATE 10 - 07 - 2019

Plotted:10/07/2019 8:09AM | By:JONATHAN WILKINS | File Location:Y:\PROJECTS\G2432-52-01 (College View)\SHEETS\G2432-52-01 Geo Map.dwg

COLLEGE VIEW
5420-22 55TH STREET
SAN DIEGO, CALIFORNIA



GEOLOGIC CROSS-SECTION 1-1'

SCALE: 1" = 40' (Vert. = Horiz.)

GEOCON LEGEND

Qudf ........UNDOCUMENTED FILL

Qvop 7 ....VERY OLD PARALIC DEPOSITS

Tmv ....MISSION VALLEY FORMATION

Tst ....STADIUM CONGLOMERATE

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

B-3 ....APPROX. LOCATION OF BORING

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PROJECT NO. G2432 - 52 - 01
FIGURE 3
DATE 10 - 07 - 2019

APPENDIX A

APPENDIX A

FIELD INVESTIGATION

We performed the drilling operations on August 8 and 9, 2019. The geotechnical borings were drilled to depths ranging from approximately 6½ to 46½ feet below existing grade using a CME 75 drill rig equipped with hollow-stem augers. The 5 infiltration-test borings were drilled to depths of approximately 2 to 8 feet. The locations of the exploratory borings are shown on the Geologic Map, Figure 2. The boring logs are presented in this appendix. We located the borings in the field using existing reference points; therefore, actual boring locations may deviate slightly.

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

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

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

BORING B 1

ELEV. (MSL.) **410'** DATE COMPLETED **08-08-2019**

EQUIPMENT **CME 75** BY: **K. HAASE**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0					**5" ASPHALT**			
	B1-1			SC	**UNDOCUMENTED FILL (Qudf)** Loose to medium dense, moist, brown to grayish brown, Clayey, fine to medium SAND; abundant gravel and cobble			
2								
4								
	B1-2					48		
6				SM	**VERY OLD PARALIC DEPOSITS/MISSION VALLEY FORMATION-Undivided ($Qvop_7$/Tmv)** Dense to very dense, damp to moist, light brown to brown, Silty, fine- to coarse-grained, Sandy CONGLOMERATE			
8								
	B1-3				-Becomes clayey	50/3"		
10								
12								
14								
	B1-4					50/5"		
16								
18								
	B1-5				-No recovery	50/3"		
					BORING TERMINATED AT 19.5 FEET Groundwater not encountered			

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

G2432-52-01.GPJ

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

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

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BORING B 2

ELEV. (MSL.) 411' DATE COMPLETED 08-08-2019

EQUIPMENT CME 75 BY: K. HAASE

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0					**5" ASPHALT**			
	B2-1			SM	**VERY OLD PARALIC DEPOSITS/MISSION VALLEY FORMATION-Undifferentiated (Qvop$_7$/Tmv)** Dense to very dense, light brown to brown, Silty, fine- to medium-grained, Sandy CONGLOMERATE			
2								
4								
	B2-2				-No recovery	55		
6								
8								
10	B2-3				-No recovery	33		
12								
14								
	B2-4					22		
16								
18								
20								
22								

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

G2432-52-01.GPJ

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

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

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BORING B 2

ELEV. (MSL.) 411' DATE COMPLETED 08-08-2019

EQUIPMENT CME 75 BY: K. HAASE

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
24								
	B2-5			SC	Dense to very dense, light brown to brown, Clayey, fine- to medium-grained, Sandy CONGLOMERATE	31		
26								
28								
30	B2-6				-No recovery	29		
32								
34								
	B2-7			SC	**STADIUM CONGLOMERATE (Tst)** Very dense, moist, reddish brown, Clayey, fine- to coarse-grained, Sandy CONGLOMERATE (Approximate Depth)	31		
36								
38								
40	B2-8				-No recovery	50/1"		
42								
44								
46								
					BORING REFUSAL AT 46.5 FEET Groundwater not encountered			

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

G2432-52-01.GPJ

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

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

BORING B 3

ELEV. (MSL.) **409'** DATE COMPLETED **08-08-2019**

EQUIPMENT **CME 75** BY: **K. HAASE**

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**			
	B3-1			SM	**VERY OLD PARALIC DEPOSITS/MISSION VALLEY FORMATION-Undivided (Qvop$_7$/Tmv)** Dense to very dense, damp to moist, brown, Silty, fine- to coarse-grained, Sandy CONGLOMERATE			
2								
4								
	B3-2				-No recovery	50/6"		
6								
8								
10	B3-3				-Becomes light reddish brown	40		
12								
14								
	B3-4			SC	Dense to very dense, damp to moist, brown, Clayey, fine- to coarse-grained, Sandy CONGLOMERATE	24		
16								
18								
20	B3-5					14		
22								

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

G2432-52-01.GPJ

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

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

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BORING B 3

ELEV. (MSL.) **409'** DATE COMPLETED **08-08-2019**

EQUIPMENT **CME 75** BY: **K. HAASE**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
24								
	B3-6				-No recovery	19		
26								
28								
					BORING TERMINATED AT 29 FEET Groundwater not encountered Backfilled with bentonite chips			

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

G2432-52-01.GPJ

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

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

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BORING B 4

ELEV. (MSL.) **410'** DATE COMPLETED **08-09-2019**

EQUIPMENT **CME 75** BY: **K. HAASE**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0					5" **ASPHALT**			
	B4-1				**VERY OLD PARALIC DEPOSITS/MISSION VALLEY FORMATION-Undifferentiated (Qvop$_7$/Tmv)** Very dense, moist, brown, Clayey, fine- to coarse-grained, Sandy CONGLOMERATE			
2								
4								
	B4-2				-No recovery	50/1"		
6								

BORING REFUSAL AT 6.5 FEET
Groundwater not encountered

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

G2432-52-01.GPJ

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

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

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BORING P 1

ELEV. (MSL.) 415' DATE COMPLETED 08-08-2019

EQUIPMENT CME 75 BY: K. HAASE

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0					5" **ASPHALT**			
				SM	**UNDOCUMENTED FILL (Qudf)** Medium dense, moist, brown, Silty, fine to corse SAND; abundant gravel and cobble			
2				SM	**VERY OLD PARALIC DEPOSITS/MISSION VALLEY FORMATION-Undivided ($Qvop_7$/Tmv)** Very dense, damp to moist, brown, Silty, fine- to coarse-grained, Sandy CONGLOMERATE			
4								
					BORING TERMINATED AT 5 FEET Groundwater not encountered			

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

G2432-52-01.GPJ

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

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

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BORING P 2

ELEV. (MSL.) **412'** DATE COMPLETED **08-08-2019**

EQUIPMENT **CME 75** BY: **K. HAASE**

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**			
				SC	**VERY OLD PARALIC DEPOSITS/MISSION VALLEY FORMATION-Undivided (Qvop$_7$/Tmv)** Dense to very dense, moist, brown, Clayey, fine- to coarse-grained, Sandy CONGLOMERATE			
2								
4								
6								
8					BORING TERMINATED AT 8 FEET Groundwater not encountered			

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

G2432-52-01.GPJ

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

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

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PROJECT NO. G2432-52-01

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	BORING P 3 ELEV. (MSL.) 409' DATE COMPLETED 08-08-2019 EQUIPMENT CME 75 BY: K. HAASE	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
					MATERIAL DESCRIPTION			
0					5" **ASPHALT**			
				SC	**VERY OLD PARALIC DEPOSITS/MISSION VALLEY FORMATION-Undivided ($Qvop_7$/Tmv)** Dense to very dense, moist, dark brown, Clayey, fine- to coarse-grained, Sandy CONGLOMERATE			
2								
4								
					BORING TERMINATED AT 5 FEET Groundwater not encountered			

Figure A-7,
Log of Boring P 3, Page 1 of 1

G2432-52-01.GPJ

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

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

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DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	BORING P 4 ELEV. (MSL.) 411' DATE COMPLETED 08-09-2019 EQUIPMENT HAND AUGER BY: K. HAASE	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
					MATERIAL DESCRIPTION			
0				SC	**UNDOCUMENTED FILL (Qudf)** Loose, moist, dark brown, Clayey, fine to coarse SAND; some gravel			
2				SC	**VERY OLD PARALIC DEPOSITS/MISSION VALLEY FORMATION-Undivided (Qvop$_7$/Tmv)** Very dense, damp to moist, brown, Clayey, fine- to coarse-grained Sandy CONGLOMERATE			
					BORING TERMINATED AT 2.5 FEET Groundwater not encountered			

Figure A-8,
Log of Boring P 4, Page 1 of 1

G2432-52-01.GPJ

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

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

PROJECT NO. G2432-52-01

BORING P 5

ELEV. (MSL.) 409'   DATE COMPLETED 08-09-2019

EQUIPMENT HAND AUGER   BY: K. HAASE

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0				SC	**UNDOCUMENTED FILL (Qudf)** Loose, moist, brown, Clayey, fine to medium SAND; some gravel			
				SC	**VERY OLD PARALIC DEPOSITS/STADIUM CONGLOMERATE-Undivided ($Qvop_7$/Tst)** Very dense, damp to moist, brown, Clayey, fine- to coarse-grained Sandy CONGLOMERATE			
2					BORING TERMINATED AT 2 FEET Groundwater not encountered			

Figure A-9,
Log of Boring P 5, Page 1 of 1

G2432-52-01.GPJ

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

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

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APPENDIX B

APPENDIX B

LABORATORY TESTING

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

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

Sample No.	Description	Maximum Dry Density (pcf)	Optimum Moisture Content (% dry wt.)
B1-1	Light brown, Clayey, fine to coarse SAND; little gravel	127.8	10.1
B3-1	Light brown, Silty, fine to coarse SAND; some gravel	137.4	6.9

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

Sample No.	Depth (feet)	Geologic Unit	Dry Density (pcf)	Moisture Content (%)		Unit Peak [Ultimate[1]] Cohesion (psf)	Angle of Peak [Ultimate[1]] Shear Resistance (degrees)
				Initial	Final		
B1-1	0 - 5	Qudf	115.1	10.3	16.2	1150 [1100]	26 [26]
B3-1	0 – 5	Qvop/Tmv	123.7	7.1	11.3	1100 [1050]	30 [30]

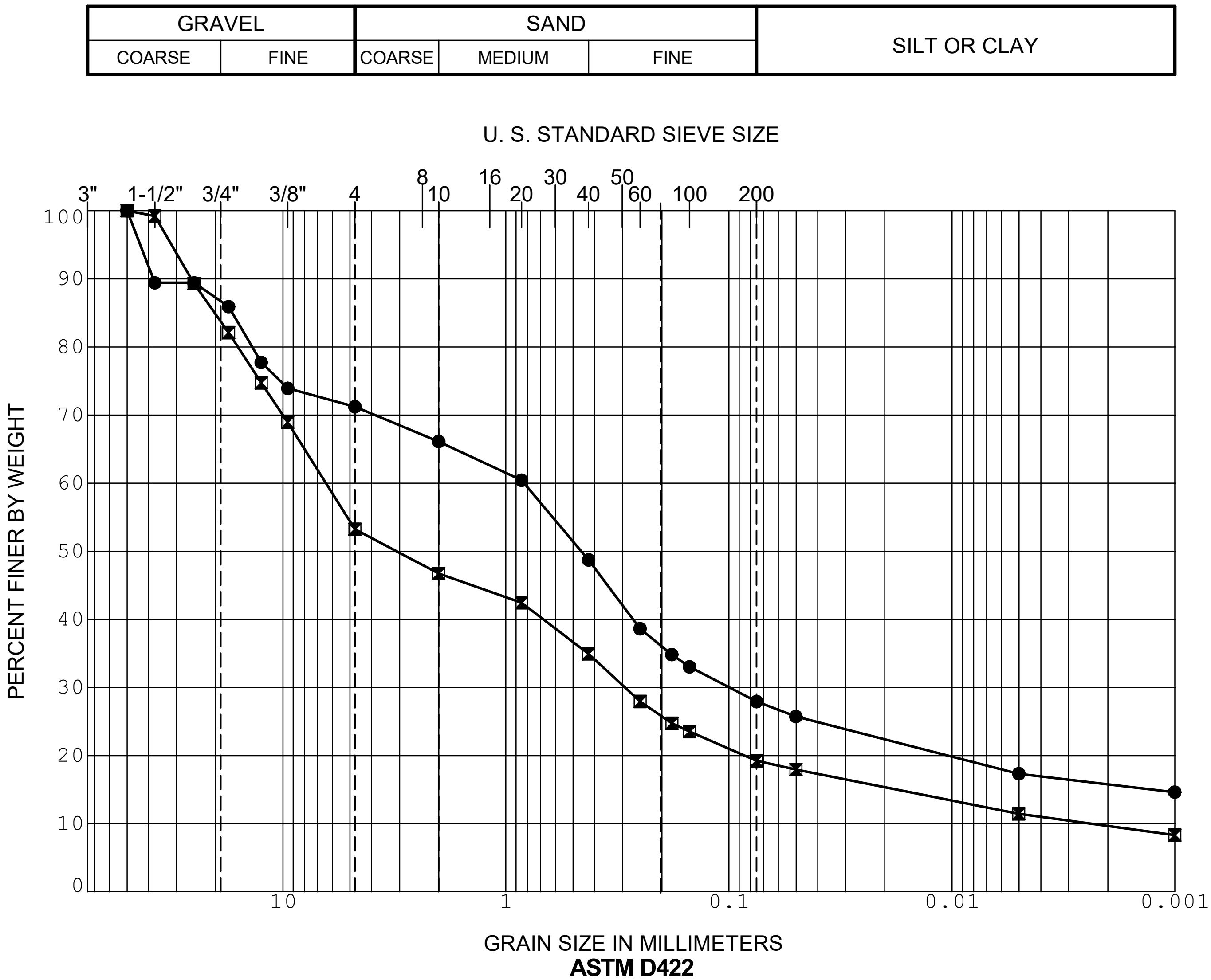
Remolded to a dry density of about 90 percent of the laboratory maximum dry density.

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

Sample No.	Moisture Content (%)		Dry Density (pcf)	Expansion Index	2016 CBC Expansion Classification	ASTM Soil Expansion Classification
	Before Test	After Test				
B2-1	8.8	76.2	114.8	32	Expansive	Low
B4-1	8.9	17.3	112.3	21	Expansive	Low

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

Sample No.	Depth (Feet)	Geologic Unit	Water-Soluble Sulfate (%)	ACI 318 Sulfate Exposure
B1-1	0 - 5	Qudf	0.050	S0
B3-1	0 - 5	Qvop/Tst	0.014	S0



	SAMPLE	DEPTH (ft)	CLASSIFICATION	NAT WC	LL	PL	PI
●	B1-2	5.0	SC - Clayey SAND				
⊠	B2-1	0.5	GC - Clayey GRAVEL				
▲							

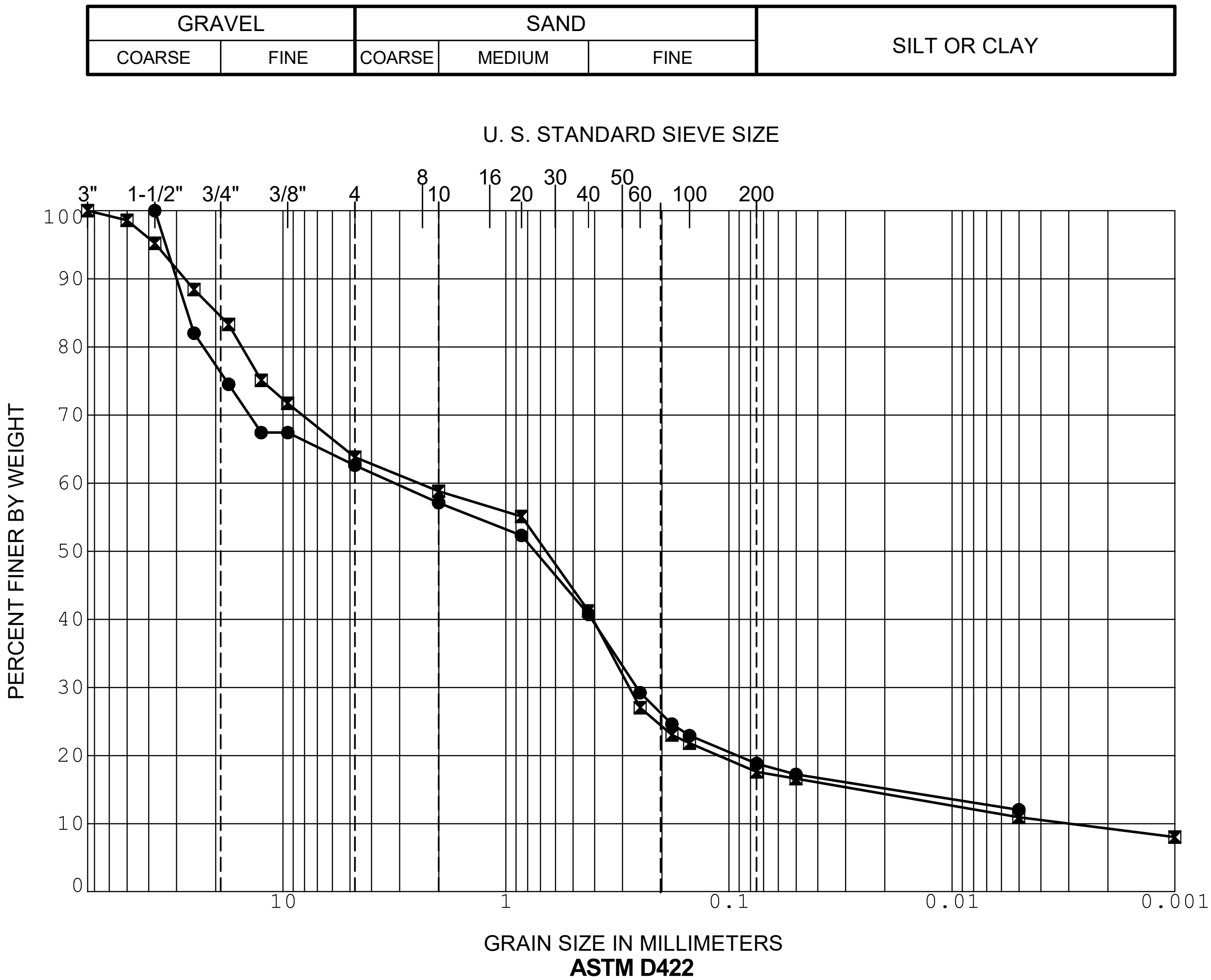
GRADATION CURVE

COLLEGE VIEW
5240 55TH STREET
SAN DIEGO, CALIFORNIA

G2432-52-01.GPJ

Figure B-1

PROJECT NO. G2432-52-01



	SAMPLE	DEPTH (ft)	CLASSIFICATION	NAT WC	LL	PL	PI
●	B2-4	15.0	SC - Clayey SAND with gravel				
⊠	B4-1	0.5	SC - Clayey SAND with gravel				
▲							

GRADATION CURVE

COLLEGE VIEW
5240 55TH STREET
SAN DIEGO, CALIFORNIA

G2432-52-01.GPJ

Figure B-2

GEOCON

APPENDIX C

APPENDIX C

RECOMMENDED GRADING SPECIFICATIONS

FOR

COLLEGE VIEW
5420-22 55TH STREET
SAN DIEGO, CALIFORNIA

PROJECT NO. G2432-52-01

RECOMMENDED GRADING SPECIFICATIONS

1. GENERAL

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

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

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

2. DEFINITIONS

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

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

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

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

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

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

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

3. MATERIALS

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

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

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

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

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

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

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

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

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

4. CLEARING AND PREPARING AREAS TO BE FILLED

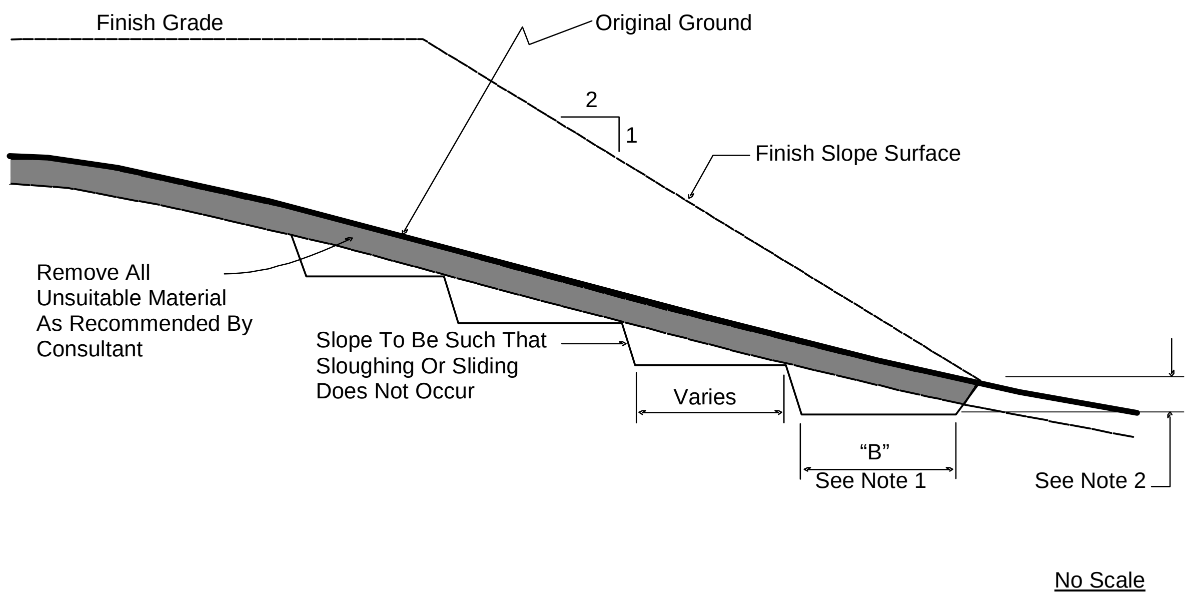
4.1 Areas to be excavated and filled shall be cleared and grubbed. Clearing shall consist of complete removal above the ground surface of trees, stumps, brush, vegetation, man-made structures, and similar debris. Grubbing shall consist of removal of stumps, roots, buried logs and other unsuitable material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

4.2 Asphalt pavement material removed during clearing operations should be properly disposed at an approved off-site facility or in an acceptable area of the project evaluated by Geocon and the property owner. Concrete fragments that are free of reinforcing steel may be placed in fills, provided they are placed in accordance with Section 6.2 or 6.3 of this document.

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

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

TYPICAL BENCHING DETAIL



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

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

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

5. COMPACTION EQUIPMENT

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

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

6. PLACING, SPREADING AND COMPACTION OF FILL MATERIAL

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

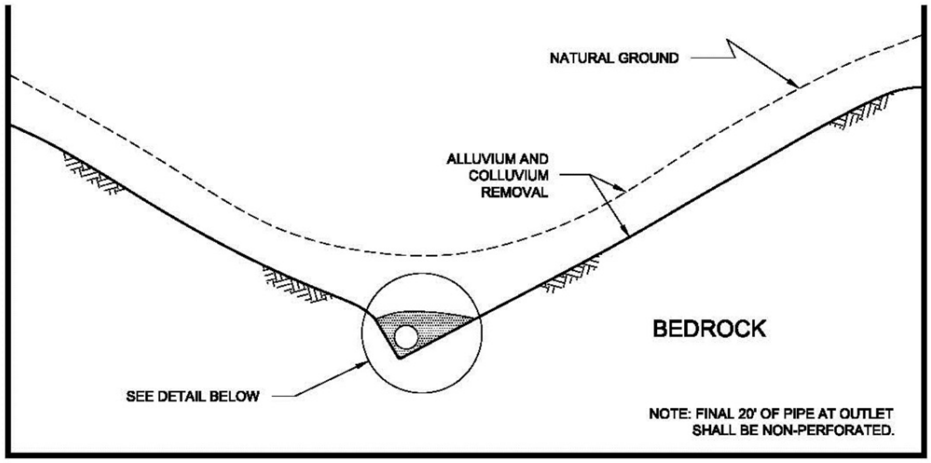
6.3.6 To reduce the potential for "piping" of fines into the *rock* fill from overlying *soil* fill material, a 2-foot layer of graded filter material shall be placed above the uppermost lift of *rock* fill. The need to place graded filter material below the *rock* should be determined by the Consultant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

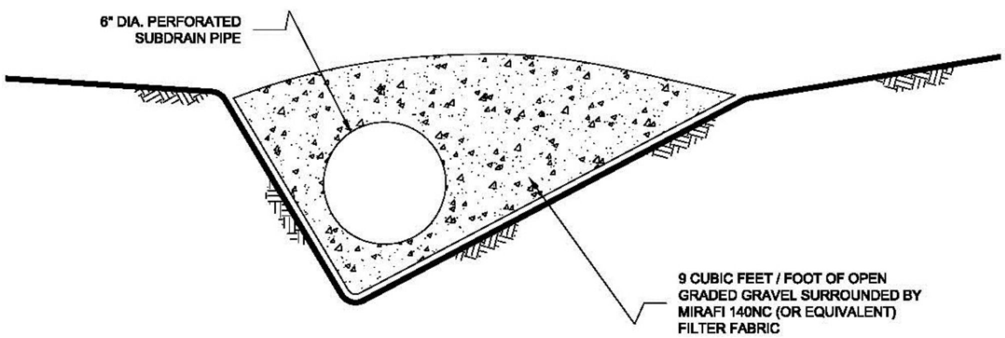
6.3.7 *Rock* fill placement should be continuously observed during placement by the Consultant.

7. SUBDRAINS

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

TYPICAL CANYON DRAIN DETAIL





NOTES:

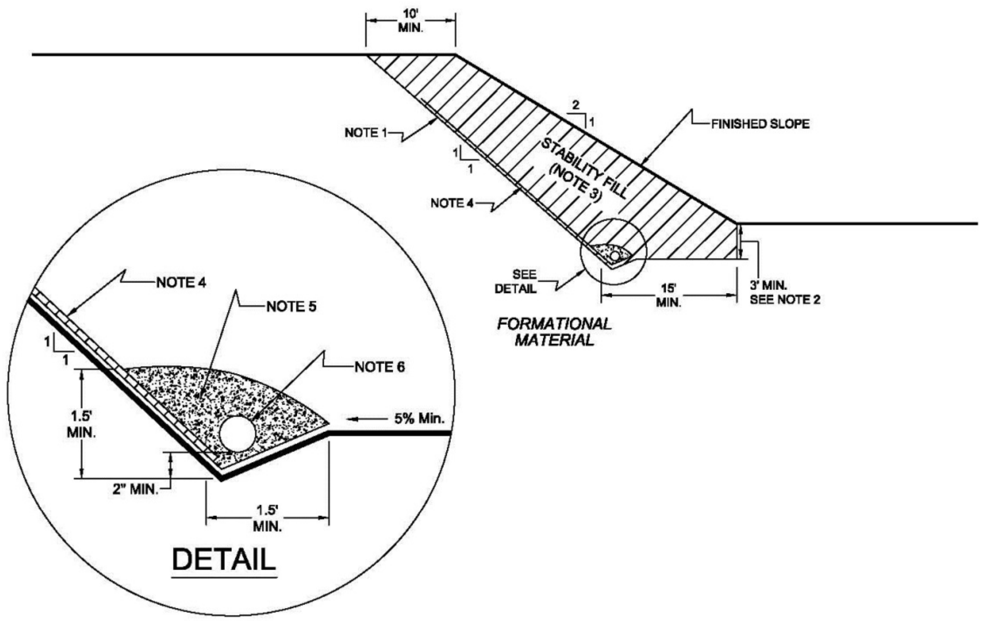
1......8-INCH DIAMETER, SCHEDULE 80 PVC PERFORATED PIPE FOR FILLS IN EXCESS OF 100-FEET IN DEPTH OR A PIPE LENGTH OF LONGER THAN 500 FEET.

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

NO SCALE

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

TYPICAL STABILITY FILL DETAIL



NOTES:

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

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

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

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

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

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

NO SCALE

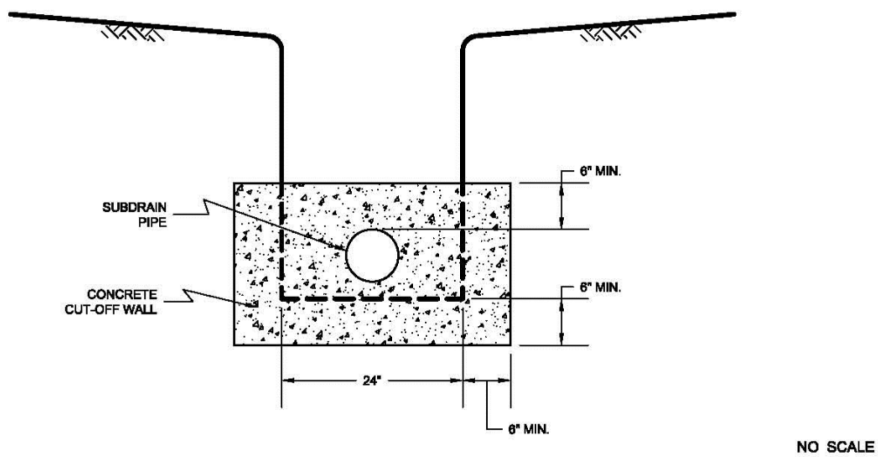
7.3 The actual subdrain locations will be evaluated in the field during the remedial grading operations. Additional drains may be necessary depending on the conditions observed and the requirements of the local regulatory agencies. Appropriate subdrain outlets should be evaluated prior to finalizing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

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

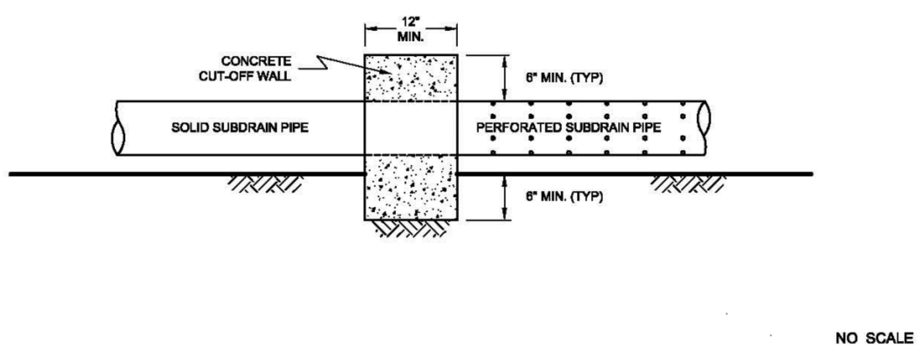
7.5 Prior to outletting, the final 20-foot segment of a subdrain that will not be extended during future development should consist of non-perforated drainpipe. At the non-perforated/ perforated interface, a seepage cutoff wall should be constructed on the downslope side of the pipe.

TYPICAL CUT OFF WALL DETAIL

FRONT VIEW

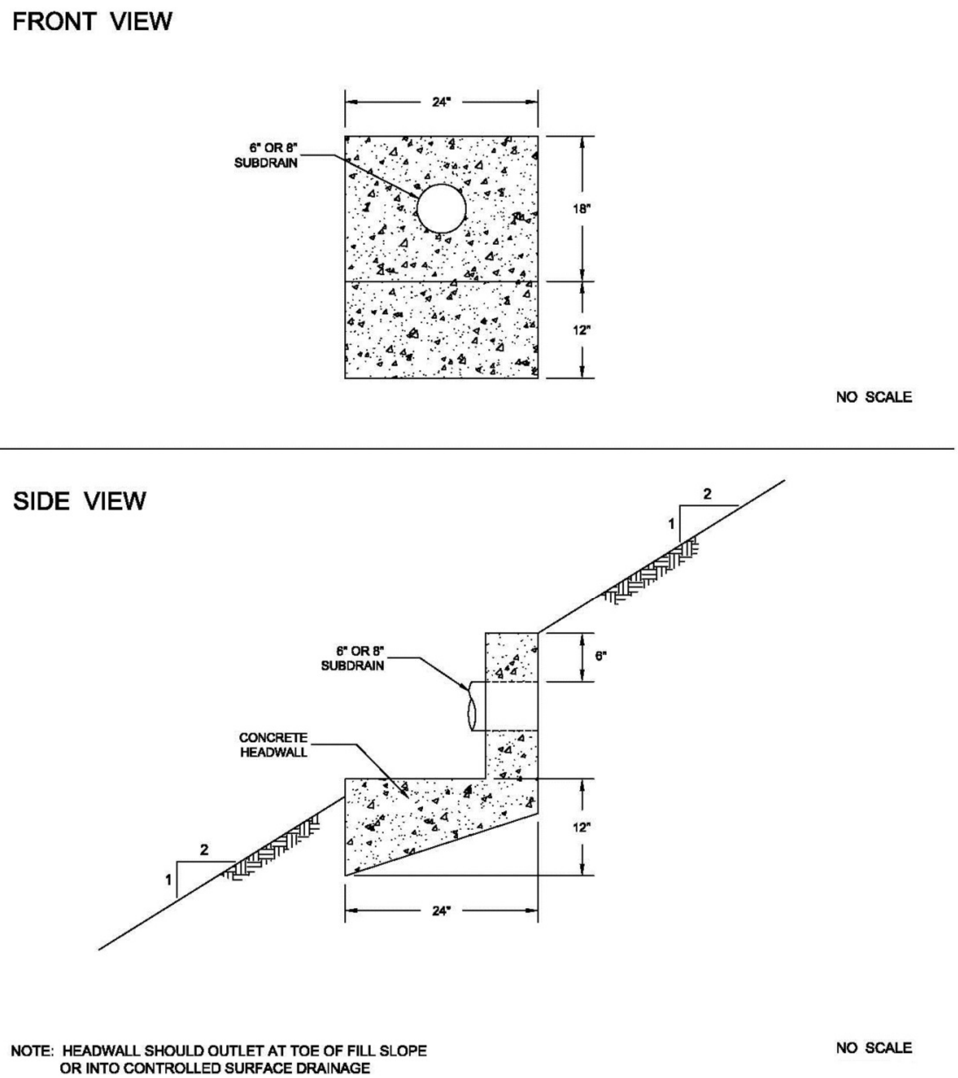


SIDE VIEW



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

TYPICAL HEADWALL DETAIL



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

8. OBSERVATION AND TESTING

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

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

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

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

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

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

8.6.1 Soil and Soil-Rock Fills:

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

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

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

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

9. PROTECTION OF WORK

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

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

10. CERTIFICATIONS AND FINAL REPORTS

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

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

LIST OF REFERENCES

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

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

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

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

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

7. Chiou, Brian 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.

8. Historical Aerial Photos. *http://www.historicaerials.com*

9. Jennings, C. W., 1994, California Division of Mines and Geology, *Fault Activity Map of California and Adjacent Areas*, California Geologic Data Map Series Map No. 6.

10. Kennedy, M. P. and S. S. Tan, 2008, *Geologic Map of the San Diego 30'x60' Quadrangle, California*, USGS Regional Map Series Map No. 3, Scale 1:100,000.

11. Risk Engineering, *EZ-FRISK*, 2016.

12. Structural Engineers Association of California (SEAOC) and Office of Statewide Health Planning and Development (OSHPD), *Seismic Design Maps*, https://seismicmaps.org/, accessed January 11, 2019.

13. Unpublished reports, aerial photographs, and maps on file with Geocon Incorporated.

14. URS, *Factual Geotechnical Report, West Campus Housing, San Diego State University, Remington Road and 55th Street, San Diego, California 92182*, dated December 17, 2013 (Project No. 27661317.10000)