APPENDIX C.
PRELIMINARY GEOTECHNICAL STUDY

UPDATE
GEOTECHNICAL INVESTIGATION

ALEXANDRIA SCIENCE VILLAGE
9393 TOWNE CENTRE DRIVE
SAN DIEGO, CALIFORNIA

GEOCON
INCORPORATED

GEOTECHNICAL
ENVIRONMENTAL
MATERIALS

PREPARED FOR

ALEXANDRIA®

OCTOBER 15, 2021
REVISED JUNE 22, 2022
PROJECT NO. G2101-52-02

Project No. G2101-52-02
October 15, 2021
Revised June 22, 2022

Alexandria Real Estate Equities, Inc.
10996 Torreyana Road, Suite 250
San Diego, California 92121

Attention: Mr. Christopher Clement

Subject: UPDATE GEOTECHNICAL INVESTIGATION
ALEXANDRIA SCIENCE VILLAGE
9393 TOWNE CENTRE DRIVE
SAN DIEGO, CALIFORNIA

Dear Mr. Clement:

In accordance with your request and our Proposal No. LG-19340, dated December 10, 2019, and Change Order dated October 11, 2021, we prepared this update geotechnical investigation report for the subject project. The accompanying report presents the results of our study and conclusions and recommendations pertaining to the geotechnical aspects of proposed project. We opine the proposed project can be constructed from a geotechnical engineering standpoint if the recommendation presented herein are incorporated into the design and construction operations.

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

Kelli A. James
RCE 79438

Shawn Foy Weedon
GE 2714

John Hoobs
CEG 1524

KAJ:SFW:JH:am

(e-mail) Addressee

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UPDATE GEOTECHNICAL INVESTIGATION

1. PURPOSE AND SCOPE

This report presents the results of our update geotechnical investigation for the proposed Alexandria Science Village project. The property is located at 9393 Towne Centre Drive, northeast of Towne Centre Drive and Executive Drive in the City of San Diego, California (see Vicinity Map).

Vicinity Map

The purpose of this study is to evaluate the surface and subsurface soil conditions and general site geology, and to identify geotechnical constraints that may impact development of the property. In addition, the purpose of this report is to provide foundation design criteria, preliminary pavement recommendations, 2019 CBC seismic design criteria, retaining wall recommendations, concrete flatwork design criteria, and excavation considerations. We used an electronic version of the site plan, provided by Michael Baker International, as the base for our Geologic Map (Figure 1). We also reviewed readily available published and unpublished geologic literature (see *List of References*).

The scope of the study also included a review of:

1. *Geotechnical Investigation, Podium 93, 9363, 9737, and 9393 Towne Centre Drive, San Diego, California*, prepared by Geocon Incorporated, dated July 11, 2017 (Project No. G2101-52-01).

2. *100 Percent Design Development Package, Alexandria Science Village, 9363, 9373 & 9393 Towne Centre Drive, San Diego, California*, architectural plans prepared by Miller Hull, civil

plans prepared by Michael Baker International, structural plans by DCI Engineers, dated May 24, 2022 (Project No. A19.0087.00).

3. *Update Shoring Recommendations, Alexandria Science Village, 9393 Towne Centre Drive, San Diego, California,* prepared by Geocon Incorporated, dated February 25, 2022 (G2101-52-02).

4. *Preliminary Soil and Geologic Investigation, Nexus Technology Center, San Diego, California,* prepared by Geocon Incorporated, dated December 9, 1985 (Project No. D-3592-M01).

5. *Final Soil and Geologic Investigation, Nexus Technology Center, San Diego, California,* prepared by Geocon Incorporated, dated January 20, 1986 (Project No. D-3592-M01).

6. *Geotechnical Report Update, Nexus Technology Center, Northeast Lot, San Diego, California*, prepared by Geocon Incorporated, dated June 2, 1999 (Project No. 06322-22-01).

We performed a field investigation that included excavating 2 small-diameter exploratory borings (in 2017) to a maximum depth of approximately 67 feet. The Geologic Map (Figure 1) presents the approximate locations of the borings. Appendix A presents the boring logs and other details of the field investigation. We tested selected soil samples obtained during the field investigation to evaluate pertinent physical and chemical soil properties for engineering analyses and to assist in providing recommendations for site grading and development. Details of the laboratory tests and a summary of the test results are presented in Appendix B and on the boring logs in Appendix A. We previously performed exploratory borings and trenches on the subject site in 1985, for the above referenced report. Logs of these previous borings and trenches are presented in Appendix C.

The Geologic Map, Figure 2, depicts the existing soil and geologic conditions. The plan depicts the proposed building location and mapped geologic contacts based on our site reconnaissance and field excavations. The conclusions and recommendations presented herein are based on analyses of the data reviewed as part of this study and our experience with similar soil and geologic conditions.

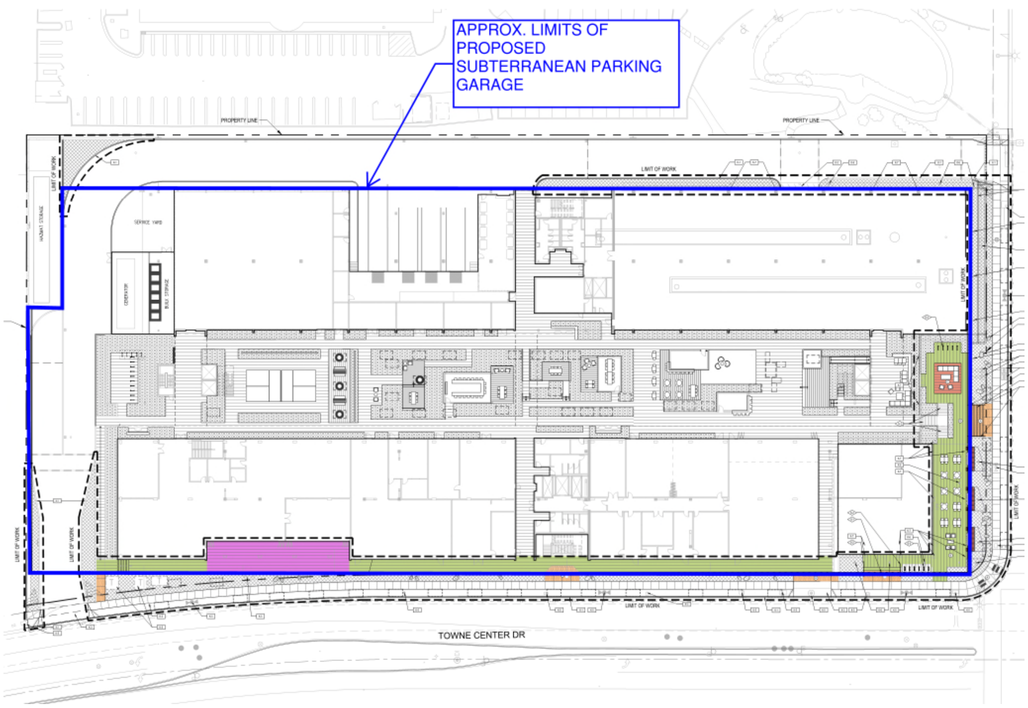
2. SITE AND PROJECT DESCRIPTION

The subject site is located at the northeast corner of Towne Centre Drive and Executive Drive in the University Towne Center area of the City of San Diego, California. The site consists of Lots 4 and 5 of Parcel 011876. The total area of both parcels is approximately 3.97 acres and currently consists of three, two-story office buildings which are connected below grade by one continuous level of subterranean parking. The buildings were constructed around 1989. Building 1 is located at the north end of the site, Building 2 is located along the western side, and Building 3 is located at the southern end. Buildings 2 and 3 are connected above grade by a two-story lobby. Surface parking exists east of Building 2 on the roof of the subterranean parking level. Access to the parking garage is provided from a driveway at the southeast corner of Building 3 on Executive Drive and from a ramp from the surface parking at the southeast corner of Building 1. The finish floor elevations of the existing buildings are unknown. The subterranean level finish floor is approximately 11 feet below the main level finish floor. The existing grade adjacent to the buildings range from approximate elevation 400 feet above Mean Sea Level (MSL) at the northwest corner of Building 1 to about 385 feet MSL at the southeast corner of Building 3. Grades at the south and west sides of Building 3 descend from the building edge to the streets below about 15 feet, and the grade at the western edge of Building 2 descends to the street up to 5 feet. The grades at the western and northern edges of Building 3 ascend toward the street and property to the north up to 3 and 8 feet, respectively. The eastern edge of the site descends toward the adjacent parking lot approximately 6 feet. An existing pond is located on the neighboring property about 30 feet east of Building 3. The site is located east of Town Centre Drive, north of Executive Drive, and west and south of existing commercial office buildings. The existing building located the north of the subject site was recently constructed in 2019 through 2020, since we issued our previous geotechnical investigation report in 2017. Based on available aerial photos, the existing building appears to be as close as approximately 35 feet from the northern edge of the site, and a drive lane runs along the northern edge of the project site. The Existing Site Plan shows the current site conditions.

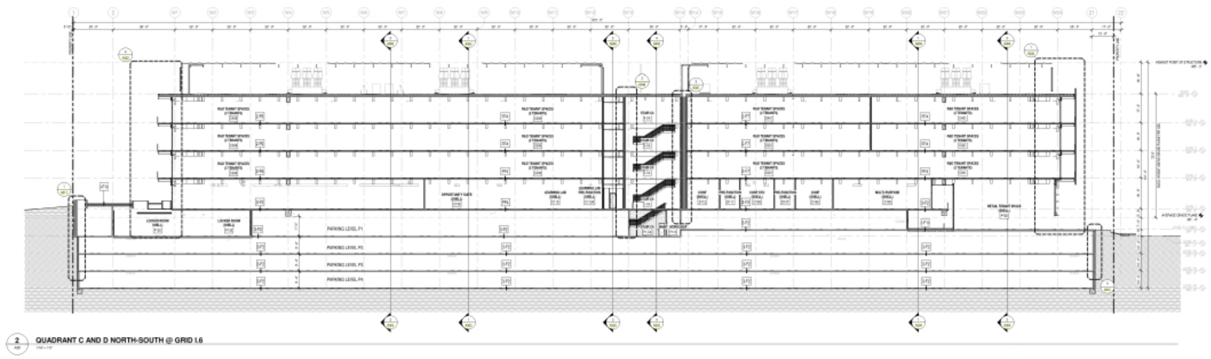
Existing Site Plan

Based on our review of readily available historical topographic surveys, it appears that the majority of the existing buildings on the project site are underlain by Very Old Paralic Deposits. The area below the southeastern portion of the property was previously underlain by a finger of a canyon that sloped from approximate elevation 385 feet MSL near the center of existing Building 3 down to about elevation 350 feet MSL at the southeast corner of the property. We assume that the area below the southeast corner of existing Building 3 was removed of surficial soil and alluvium prior to filling in the canyon, but we do not have documentation of the previous grading operations at this time. We estimate that the southeast corner of the properly may be underlain by as much as 35 feet of previously placed fill.

We understand, based on the referenced plans dated May 24, 2022, that the proposed project consists of demolishing the existing office buildings and subterranean parking garage and constructing a new science complex consisting of 4 stories above grade overlying 4 levels of parking. A courtyard corridor will separate the eastern and western buildings. We understand 4 levels of parking will be subterranean at the northern end of the site, and 3 levels will be subterranean at the south end where the first level of parking will daylight above grade. We understand the office buildings will be supported entirely on the parking structure, but the eastern drive lane will be supported on soil outside of the limits of the subterranean parking. Access to the property will be from Executive Drive and Towne Centre Drive. The finish floor elevation for the lowest level of subterranean parking is proposed at 350.75 feet MSL. The level 1 building finish floor elevations are planned to be 396.75 feet MSL. We assume cuts on the order of 55 to 60 feet will be required for the parking garage and foundation excavations and fills of up to about 10 feet will be required to construct the eastern drive lane and a retaining wall along the drive lane at the eastern edge of the site. The Proposed Site Plan and Proposed Building Cross-section show the planned building and improvements.



Proposed Site Plan



Proposed Building Cross-section (Looking East)

The locations and descriptions of the site and proposed development are based on the referenced grading plans and our understanding of project development. If project details vary significantly from those described herein, Geocon Incorporated should be contacted to evaluate the necessity for review and revision of this report.

3. GEOLOGIC SETTING

The site is located in a coastal plain environment within the southern portion of the Peninsular Ranges Geomorphic Province of southern California. The Peninsular Ranges is a geologic and geomorphic province that extends from the Imperial Valley to the Pacific Ocean and from the Transverse Ranges to the north and into Baja California to the south. The coastal plain of San Diego County is underlain by a thick sequence of relatively undisturbed and non-conformable sedimentary rocks that thicken to the west and range in age from Upper Cretaceous through the Pleistocene with intermittent deposition. The sedimentary units are deposited on bedrock, Cretaceous to Jurassic age igneous and metavolcanic rocks. Geomorphically, the coastal plain is characterized by a series of 21 stair-stepped, marine terraces which are younger to the west and have been dissected by west flowing rivers that drain the Peninsular Ranges to the east. 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 Peninsular Ranges Province is also dissected by the Elsinore Fault Zone that is associated with and sub-parallel to the San Andreas Fault Zone, which is the plate boundary between the Pacific and North American Plates.

The site is located within the western portion of the coastal plain geologic province on the western slope of a former south flowing canyon drainage that has dissected a terrace. The drainage flows to Rose Canyon drainage channel and enters the Pacific Ocean at Pacific Beach and Mission Bay. Shallow to deep fill soils are present across the site underlain by Very Old Paralic Deposits Unit 9 and Eocene-age Scripps Formation. The Very Old Paralic Deposits are middle to late Pleistocene age, roughly 855,000 years old, and is a shallow marine unit that has been designated as the Linda Vista Terrace. The Scripps Formation was deposited in a marine environment where sandstones and siltstones were formed and uncomformably underlies the terrace. The Regional Geologic Map shows the geologic units in the area of the site.

Regional Geologic Map

4. SOIL AND GEOLOGIC CONDITIONS

We encountered one surficial material consisting of previously placed fill and two geologic units consisting of Very Old Paralic Deposits (formerly called the Lindavista Formation) and the Scripps Formation during our field investigation. The occurrence and distribution of the units encountered, including descriptions of the units, are shown on the exploratory boring logs in Appendix A. The approximate lateral extent of the geologic conditions is presented on the Geologic Map, Figure 1. The subsurface relationship between the geologic units is presented on the Geologic Cross-Sections, Figure 2. We prepared the geologic cross-sections using interpolation between exploratory borings; therefore, actual geologic conditions between the borings may vary from those illustrated and should be considered approximate.

4.1 Previously Placed Fill (Qpf)

We encountered fill materials to a depth of 32 feet below existing grade in Boring B-1 and to a depth of 15½ feet in Boring B-2. We expect fill in the area of Boring B-1 was placed during the filling of a previous existing canyon. The fill we encountered in Boring B-2 is likely the retaining wall backfill for the subterranean garage. The area below the southeastern portion of the property was previously underlain by a finger of a canyon that sloped from approximate elevation 385 feet MSL near the center of existing Building 3 down to about elevation 350 feet MSL at the southeast corner of the property. We expect the area below the southeast corner of existing Building 3 was removed of surficial soil and alluvium prior to filling in the canyon (we do not have documentation of the previous grading operations). We estimate that the southeast corner of the property may be underlain by as much as 35 feet of fill. The previously placed fill consists of medium dense, damp to moist, brown to reddish brown, silty to clayey, fine to coarse sand. The fill materials located outside the limits of the proposed structure are adequate to support surface improvements and additional fill; however, the upper 1 to 2 feet would require remedial grading.

4.2 Very Old Paralic Deposits, Unit 9 (Qvop)

We encountered Quaternary-age Very Old Paralic Deposits (Unit 9) below the fill in Boring B-2. The Very Old Paralic Deposits were formerly called the Lindavista Formation. The Very Old Paralic Deposits in our Boring B-2 extended from below the previously placed fill to a depth of approximately 25 feet below existing grade at an elevation of 371 feet MSL. The Very Old Paralic Deposits generally consists of medium dense to very dense, damp, reddish brown, silty and clayey, fine to coarse sandstone. Some cemented areas exist within the deposits which may require very heavy effort to excavate and occasional refusal. The Very Old Paralic Deposits are considered suitable to support the planned improvements.

4.3 Scripps Formation (Tsc)

The Eocene-age Scripps Formation is mapped by Kennedy and Tan (2008) and exists below the Very Old Paralic Deposits and previously placed fill. The Scripps Formation generally consists of medium dense to very dense, damp to dry, olive brown to light yellowish brown, siltstone and claystone. The Scripps Formation can contain gypsum crystals that elevate the water-soluble sulfate content of the soil and may require special concrete requirements. In addition, cemented zones exist within the Scripps Formation that can cause very difficult excavations and rock breakers may be necessary. The Scripps Formation possesses adequate soil support characteristics for support of properly compacted fill and structural loading.

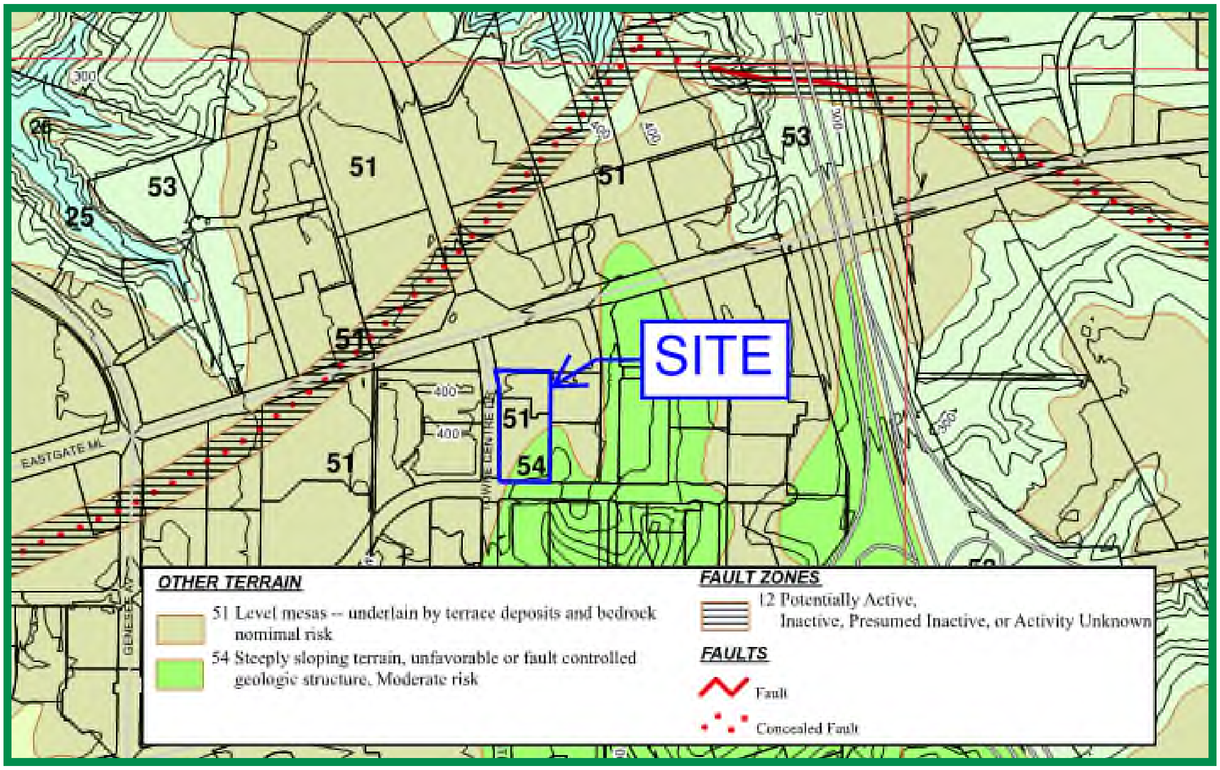
5. GROUNDWATER

We did not encounter groundwater or seepage in our exploratory excavations. We do not expect groundwater would significantly affect project development. We expect that groundwater would be at least 100 feet below existing grades. It is not uncommon for groundwater or seepage conditions to develop where none previously existed due to the permeability characteristics of the geologic units encountered on site. During the rainy season, seepage conditions may develop that would require special consideration during grading and shoring operations. Groundwater elevations are dependent on seasonal precipitation, irrigation and land use, among other factors, and vary as a result. Proper surface drainage will be critical to future performance of the project.

6. GEOLOGIC HAZARDS

6.1 Geologic Hazard Category

The City of San Diego Seismic Safety Study, Geologic Hazards and Faults, Map Sheet 34 defines the northwestern portion of the site with a Hazard Category 51*: Level mesas – Underlain by terrace deposits and bedrock – Nominal risk* and the southeastern portion of the site as Hazard Category 54: *Steeply sloping terrain, unfavorable or fault controlled geologic structure – Moderate Risk.* (as shown on the Hazard Category Map).



Hazard Category Map

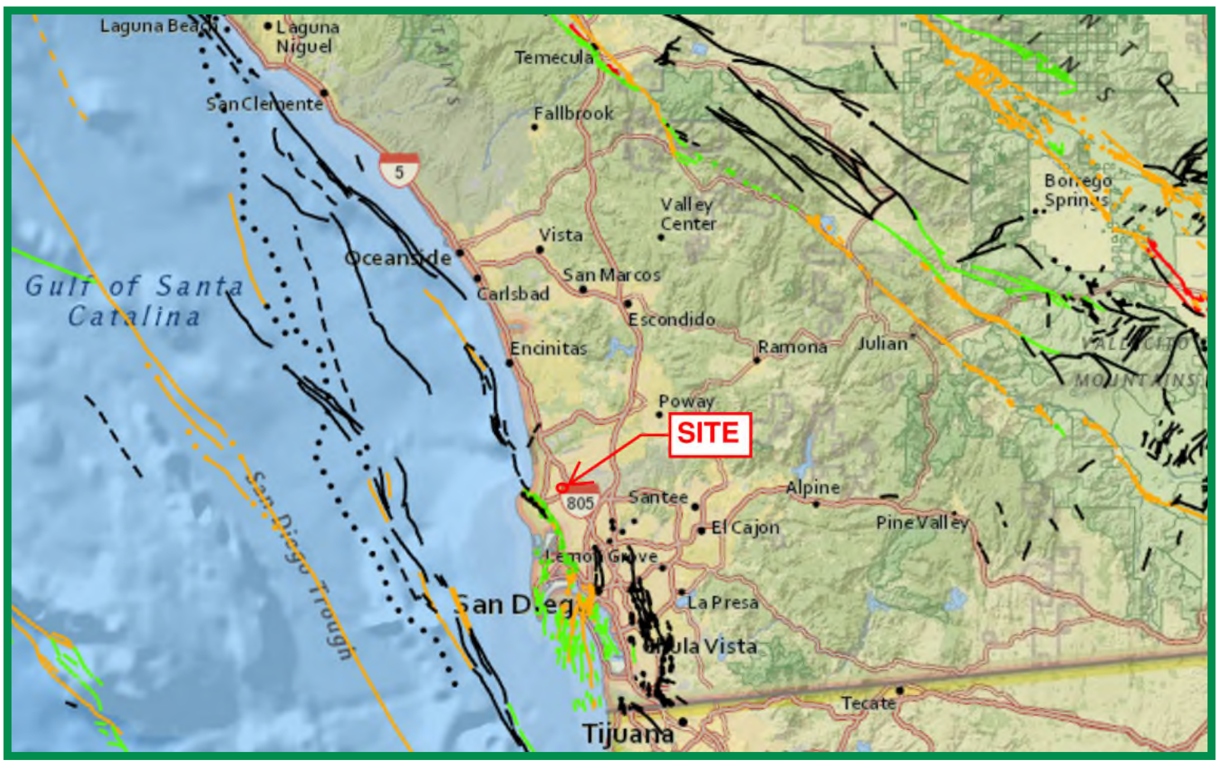
6.2 Faulting and Seismicity

A review of the referenced geologic materials and our knowledge of the general area indicate that the site is not underlain by 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 11,700 years. The site is not located within a State of California Earthquake Fault Zone.

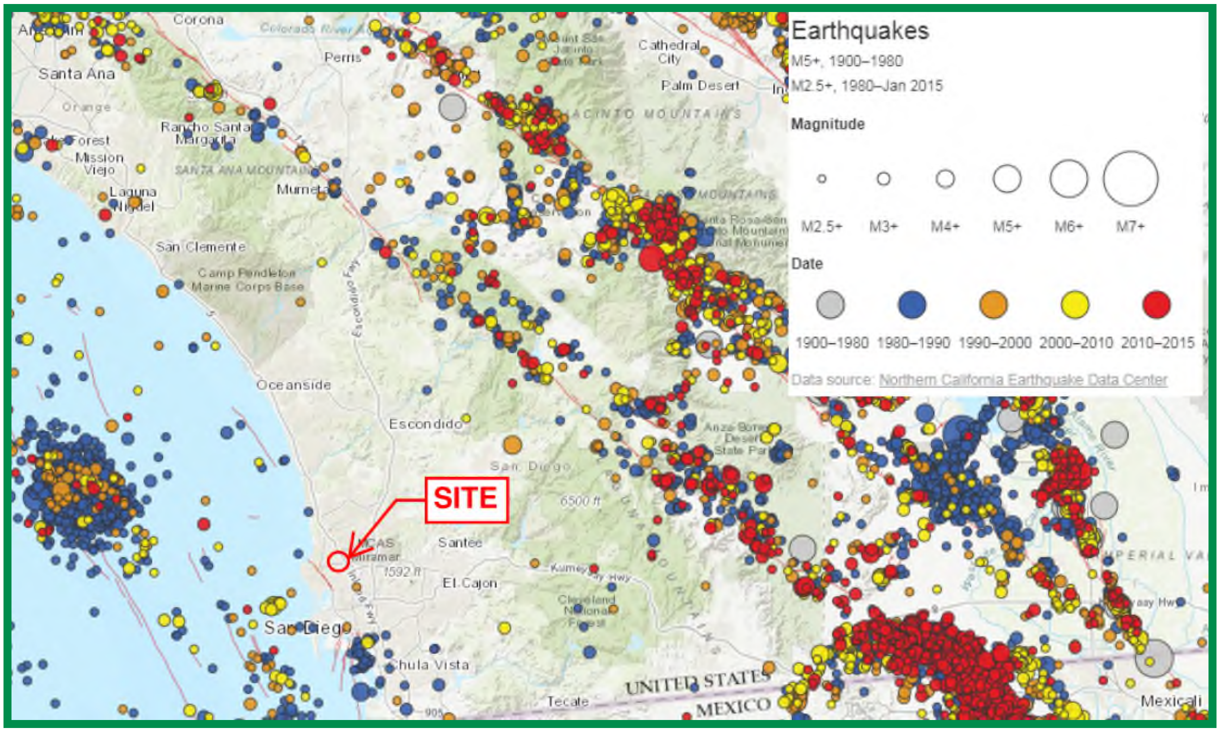
Based on the City of San Diego Seismic Safety Study, Geologic Hazards and Faults, Map Sheet 34, A concealed fault defined as *Fault Zone 12: Potentially Active, Inactive, Presumed Inactive, or Activity Unknown* is mapped approximately 525 feet northwest of the project site, trending in a northeast to southwest direction, and a fault/concealed fault defined as *Fault Zone 12: Potentially Active, Inactive, Presumed Inactive, or Activity Unknown* is mapped approximately 1,850 feet northeast of the site trending in a northwest to southeast direction. These faults/concealed faults will not impact the proposed development of the site.

The USGS has developed a program to evaluate the approximate location of faulting in the area of properties. The following figure shows the location of the existing faulting in the San Diego County and Southern California region. The fault traces are shown as solid, dashed and dotted that represent well-constrained, moderately constrained and inferred, respectively. The fault line colors represent faults with ages less than 150 years (red), 15,000 years (orange), 130,000 years (green), 750,000 years (blue) and 1.6 million years (black).



Faults in Southern California

The San Diego County and Southern California region is seismically active. The following figure presents the occurrence of earthquakes with a magnitude greater than 2.5 from the period of 1900 through 2015 according to the Bay Area Earthquake Alliance website.



Earthquakes in Southern California

Considerations important in seismic design include the frequency and duration of motion and the soil conditions underlying the site. Seismic design of structures should be evaluated in accordance with the California Building Code (CBC) guidelines currently adopted by the local agency.

6.3 Liquefaction

Liquefaction typically occurs when a site is located in a zone with seismic activity, onsite soil is cohesionless/silt or clay with low plasticity, groundwater is encountered within 50 feet of the surface, and soil relative densities are less than about 70 percent. If the four of the previous criteria are met, a seismic event could result in a rapid pore-water pressure increase from the earthquake-generated ground accelerations. Seismically induced settlement may occur whether the potential for liquefaction exists or not. The potential for liquefaction and seismically induced settlement occurring within the site soil is considered to be very low due to the dense nature of the fill, Very Old Paralic Deposits and Scripps Formation and lack of groundwater within 50 feet of the ground surface.

6.4 Storm Surge, Tsunamis, and Seiches

Storm surges are large ocean waves that sweep across coastal areas when storms make landfall. Storm surges can cause inundation, severe erosion and backwater flooding along the water front. The site is located approximately 2½ miles from the Pacific Ocean and is at an elevation of about 380 to 400 feet or greater above Mean Sea Level (MSL). Therefore, the potential of storm surges affecting the site is considered low.

A tsunami is a series of long-period waves generated in the ocean by a sudden displacement of large volumes of water. Causes of tsunamis include underwater earthquakes, volcanic eruptions, or offshore slope failures. The site is located approximately 2½ miles from the Pacific Ocean at an elevation of approximately 380 to 400 feet above Mean Sea Level. Therefore, the risk of tsunamis affecting the site is negligible.

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.

6.5 Erosion

The site is relatively flat and is not located adjacent to the Pacific Ocean coast or a free-flowing drainage where active erosion is occurring. Provided the engineering recommendations herein are followed and the project civil engineer prepares the grading plans in accordance with generally-accepted regional standards, we do not expect erosion to be a major impact to site development. In addition, we expect the proposed development would not increase the potential for erosion if properly designed.

7. CONCLUSIONS AND RECOMMENDATIONS

7.1 General

7.1.1 From a geotechnical engineering standpoint, we opine the site is suitable for development provided the recommendations presented herein are implemented in design and construction of the project.

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

7.1.3 Our fieldwork indicates the site is underlain by previously placed fill overlying Very Old Paralic Deposits and the Scripps Formation. The fill materials are considered suitable for the support of additional fill and/or settlement-sensitive structures. However, we expect the fill will be removed for a majority of the area of the planned structure during the excavation of the subterranean garage levels. Existing fill materials will likely be left in-place on the southeastern portion of the site. The Very Old Paralic Deposits and Scripps Formation are considered suitable for the support of compacted fill and settlement-sensitive structures.

7.1.4 During our investigation, we did not observe significant signs of distress on the exterior of the existing buildings and the parking areas. We have not been informed of distress occurring within the property.

7.1.5 We do not expect groundwater to be encountered during construction of the proposed development. During the rainy season, seepage conditions may develop that would require special consideration during grading and shoring operations.

7.1.6 The proposed structures can likely be supported on conventional shallow footings founded in Very Old Paralic Deposits or the Scripps Formation. Deepened shallow footings and/or deep foundations will be required for the proposed building where fill is exposed at finish grade where the existing fill soil extends deeper than the proposed finish grade elevation in the southeastern portion of the proposed building. The foundations should be extended into the formational materials or drilled piers should be installed if fill soils are present in excess of 10 feet at the bottom of the subterranean level. The drilled piers will help prevent differential settlement within the planned structures. Drilled pier recommendations are provided, herein.

7.1.7 Excavation of the fill materials, the Very Old Paralic Deposits and the Scripps Formation should generally be possible with moderate to heavy effort using conventional, heavy-duty equipment during grading and trenching operations. We expect very heavy effort with possible refusal in localized areas for excavations into strongly cemented portions of the Very Old Paralic Deposits and Scripps Formation and rock breakers may be required. Oversize material may be generated which would require special handling or exportation from the site.

7.1.8 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.

7.1.9 Surface settlement monuments and canyon subdrains will not be required on the project.

7.2 Excavation and Soil Characteristics

7.2.1 Excavation of the in-situ fill soils should be possible with moderate to heavy effort using conventional heavy-duty equipment. Excavation of the Very Old Paralic and Scripps Formation will require very heavy effort with possible refusal. The existing materials may generate oversized material using conventional heavy-duty equipment during the grading operations. Oversized rock (rocks greater than 12-inches in dimension) may be generated with these geologic units and may require export.

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 2019 California Building Code (CBC) Section 1803.5.3. Table 7.2.1 presents soil classifications based on the expansion index. We expect a majority of the soil encountered possess a "very low" to "medium" expansion potential (EI of 90 or less).

TABLE 7.2.1
EXPANSION CLASSIFICATION BASED ON EXPANSION INDEX

Expansion Index (EI)	ASTM D 4829 Expansion Classification	2019 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 2019 CBC Section 1904 and ACI 318-14 Chapter 19. However, soil associated with the Scripps Formation has been known to possess water-soluble sulfate exposures of "S0" to "S2". We will provide additional testing of the exposed soil at the finish grade elevation to evaluate the water-soluble sulfate exposure which may require higher strength concrete. Table 7.2.2 presents a summary of concrete requirements set forth by 2016 CBC Section 1904 and ACI 318. The presence of water-soluble sulfates is not a visually discernible characteristic; therefore, other soil samples from the site could yield different concentrations. Additionally, over time landscaping activities (i.e., addition of fertilizers and other soil nutrients) may affect the concentration.

TABLE 7.2.2
REQUIREMENTS FOR CONCRETE EXPOSED TO SULFATE-CONTAINING SOLUTIONS

Exposure Class		Water-Soluble Sulfate (SO_4) Percent by Weight	Cement Type (ASTM C 150)	Maximum Water to Cement Ratio by Weight[1]	Minimum Compressive Strength (psi)
S0		$SO_4<0.10$	No Type Restriction	n/a	2,500
S1		$0.10\leq SO_4<0.20$	II	0.50	4,000
S2		$0.20\leq SO_4\leq 2.00$	V	0.45	4,500
S3	Option 1	$SO_4>2.00$	V+Pozzolan or Slag	0.45	4,500
	Option 2		V	0.40	5,000

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

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 Seismic Design Criteria – 2019 California Building Code

7.3.1 Table 7.3.1 summarizes site-specific design criteria obtained from the 2019 California Building Code (CBC; Based on the 2018 International Building Code [IBC] and ASCE 7-16), Chapter 16 Structural Design, Section 1613 Earthquake Loads. We used the computer program *U.S. Seismic Design Maps*, provided by the Structural Engineers Association (SEA) to calculate the seismic design parameters. The short spectral response uses a period of 0.2 second. Structures founded on a fill thickness of 20 feet and less can be designed using a Site Class C. We evaluated the Site Class based on the discussion in Section 1613.2.2 of the 2019 CBC and Table 20.3-1 of ASCE 7-16. The values presented herein are for the risk-targeted maximum considered earthquake (MCE_R). Sites designated as Site Class D, E and F may require additional analyses if requested by the project structural engineer and client.

TABLE 7.3.1
2019 CBC SEISMIC DESIGN PARAMETERS

Parameter	Value	2019 CBC Reference
Site Class	C	Section 1613.2.2
MCE_R Ground Motion Spectral Response Acceleration – Class B (short), S_S	1.144g	Figure 1613.2.1(1)
MCE_R Ground Motion Spectral Response Acceleration – Class B (1 sec), S_1	0.403g	Figure 1613.2.1(2)
Site Coefficient, F_A	1.200	Table 1613.2.3(1)
Site Coefficient, F_V	1.500*	Table 1613.2.3(2)
Site Class Modified MCE_R Spectral Response Acceleration (short), S_{MS}	1.373g	Section 1613.2.3 (Eqn 16-36)
Site Class Modified MCE_R Spectral Response Acceleration – (1 sec), S_{M1}	0.605g*	Section 1613.2.3 (Eqn 16-37)
5% Damped Design Spectral Response Acceleration (short), S_{DS}	0.915g	Section 1613.2.4 (Eqn 16-38)
5% Damped Design Spectral Response Acceleration (1 sec), S_{D1}	0.403g*	Section 1613.2.4 (Eqn 16-39)

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

7.3.2 Table 7.3.2 presents the mapped maximum considered geometric mean (MCE_G) seismic design parameters for projects located in Seismic Design Categories of D through F in accordance with ASCE 7-16.

TABLE 7.3.2
ASCE 7-16 PEAK GROUND ACCELERATION

Parameter	Value	ASCE 7-16 Reference
Mapped MCE_G Peak Ground Acceleration, PGA	0.513g	Figure 22-9
Site Coefficient, F_{PGA}	1.200	Table 11.8-1
Site Class Modified MCE_G Peak Ground Acceleration, PGA_M	0.616g	Section 11.8.3 (Eqn 11.8-1)

7.3.3 Conformance to the criteria in Tables 7.3.1 and 7.3.2 for seismic design does not constitute any kind of guarantee or assurance that significant structural damage or ground failure will not occur in the event of a large earthquake. The primary goal of seismic design is to protect life, not to avoid all damage, since such design may be economically prohibitive.

7.3.4 The project structural engineer and architect should evaluate the appropriate Risk Category and Seismic Design Category for the planned structures. The values presented herein assume a Risk Category of II and resulting in a Seismic Design Category D. Table 7.3.3 presents a summary of the risk categories in accordance with ASCE 7-16.

TABLE 7.3.3
ASCE 7-16 RISK CATEGORIES

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

7.4 Grading

7.4.1 Grading should be performed in accordance with the attached *Recommended Grading Specifications* contained in Appendix D. Where the recommendations of this section conflict with those of Appendix D, the recommendations of this section shall take precedence. Earthwork should be observed and fill tested for dry density and moisture content by Geocon Incorporated.

7.4.2 A pre-construction conference with the owner, city inspector, general contractor, civil engineer, and geotechnical engineer in attendance should be held at the site prior to the beginning remedial grading. Special soil handling requirements can be discussed at that time. Earthwork should be observed and compacted fill tested by representatives of Geocon Incorporated.

7.4.3 Grading of the site should commence with the removal of existing improvements, vegetation, and deleterious debris. Deleterious debris should be exported from the site and should not be mixed with the fill. Existing underground improvements within and below the proposed building areas should be removed and the resulting depressions properly backfilled in accordance with the procedures described herein. Deeper removals and/or moisture conditioning should be expected within areas existing improvements and landscape areas. Asphalt and concrete should not be mixed with the fill soil unless approved by the Geotechnical Engineer.

7.4.4 The existing soil within the building pad and parking structure areas should be removed to the planned finish grade elevation. If fill materials or surficial soil are encountered within the pad area, the upper 3 feet of soil should be removed and replaced with properly compacted fill.

7.4.5 For ancillary structures, such as site retaining walls, material should be removed to a depth of 2 feet below bottom of footing and replaced with properly compacted fill. The removals should extend at least 3 feet outside of ancillary structures, where possible. The upper 2 feet of the existing soil or 2 feet below proposed grade, whichever results in a deeper removal, within the planned improvement areas outside of the building areas (e.g. pavement and landscape areas) should be removed and replaced with properly compacted fill. The removals can be limited to the Very Old Paralic Deposits.

7.4.6 Deeper removals may be required in areas where loose or saturated materials are encountered. The removals should extend at least 2 feet outside of the surface improvement area, where possible. Table 7.4.1 provides a summary of the recommended grading operations.

TABLE 7.4.1
SUMMARY OF GRADING RECOMMENDATIONS

Area	Foundation Type	Grading Recommendations
Office Building / Subterranean Parking	Founded in Formational Materials (Shallow Foundations/Deepened Shallow Foundations)	Remove to Planned Pad Elevation. Process Upper 3 Feet of Existing Fill Below Pad Grade (Where Encountered)
Ancillary Structures/Retaining Walls	Shallow Foundations	Removal to 2 Feet Below Bottom of Footings
Site Development	--	Process Upper 1 to 2 Feet of Existing Materials
Lateral Grading Limits		3 Feet Outside of Ancillary Structures
		2 Feet Outside of Improvement Areas

7.4.7 Excavated soil that is generally free of deleterious debris and contamination can be placed as fill and compacted in layers to the design finish-grade elevations. Fill and backfill materials should be compacted to a dry density of at least 90 percent of the laboratory maximum dry density near to slightly above optimum moisture content as determined by ASTM Test Method D 1557. The upper 12 inches of fill beneath pavement areas should be compacted to a dry density of at least 95 percent of the laboratory maximum dry density near to slightly above optimum moisture content.

7.4.8 Import fill (if necessary) should consist of the characteristics presented in Table 7.4.2. Geocon Incorporated should be notified of the import soil source and should perform laboratory testing of import soil prior to its arrival at the site to determine its suitability as fill material.

TABLE 7.4.2
SUMMARY OF IMPORT FILL RECOMMENDATIONS

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

7.4.9 We should be onsite to provide testing and observation services during the grading and improvement operations for the planned development. We should observe the base of the removals prior to placement of the planned compacted fill to evaluate if the geologic conditions are in accordance with the recommendations presented herein.

7.5 Subdrains

7.5.1 With the exception of retaining wall drains, we do not expect the installation of other subdrain for the proposed building.

7.6 Excavation Slopes, Shoring, and Tiebacks

7.6.1 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.

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

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

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

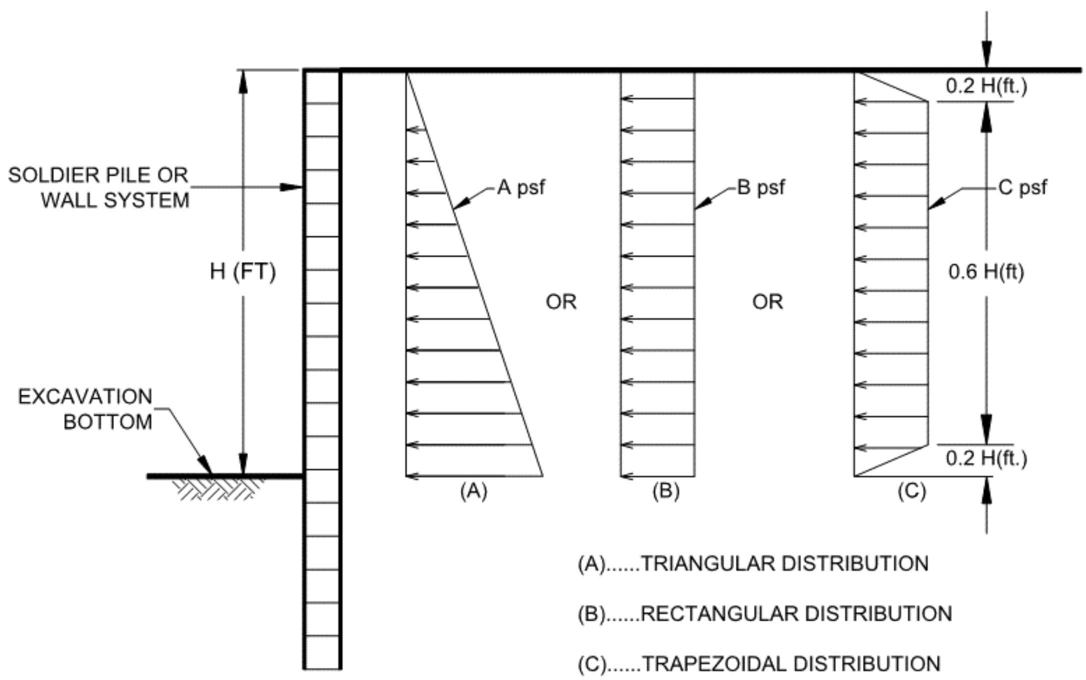
7.6.5 In general, ground conditions are moderately suited for soldier pile and tieback anchor wall construction techniques. However, gravel, cobble, and oversized material may be encountered in the Very Old Paralic Deposits and the formational materials that could be difficult to drill. Additionally, if cohesionless sands or gravels are encountered, some raveling may result along the unsupported portions of excavations.

7.6.6 Temporary shoring should be designed using a lateral pressure envelope acting on the back of the shoring as presented in Table 7.6.1 assuming a level backfill. The distributions are shown on the Active Pressures for Temporary Shoring. Triangular distribution should be used for cantilevered shoring and, the trapezoidal and rectangular distribution should be used for multi-braced systems such as tieback anchors and rakers. The project shoring engineer should determine the applicable soil distribution for the design of the temporary shoring system. Additional lateral earth pressure due to the surcharging effects from construction equipment, sloping backfill, planned stockpiles, adjacent structures and/or traffic loads should be considered, where appropriate, during design of the shoring system.

TABLE 7.6.1
SUMMARY OF TEMPORARY SHORING WALL RECOMMENDATIONS

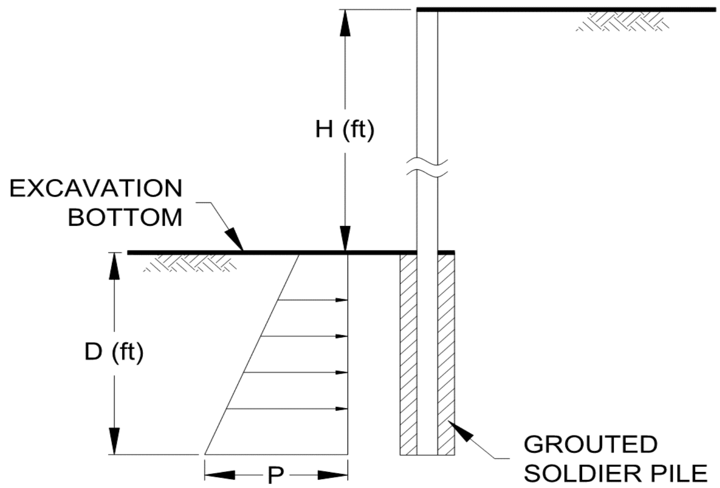
Parameter	Value
Triangular Distribution, A	31H psf
Rectangular Distribution, B	20H psf
Trapezoidal Distribution, C	25H psf
Passive Pressure, P	400D + 500 psf
Effective Zone Angle, E	30 degrees
Maximum Design Lateral Movement	1 Inch
Maximum Design Vertical Movement	½ Inch
Maximum Design Retained Height, H	60 Feet

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



Active Pressures on Temporary Shoring

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



Passive Pressures on Temporary Shoring

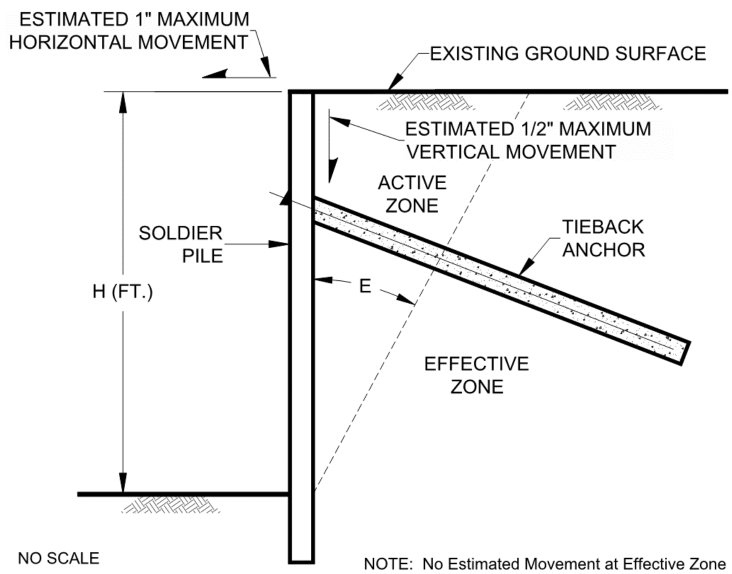
7.6.8 We should observe the drilled shafts for the soldier piles prior to the placement of steel reinforcement to check that the exposed soil conditions are similar to those expected and that footing excavations have been extended to the appropriate bearing strata and design depths. If unexpected soil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

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

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

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

7.6.12 Tieback anchors employed in shoring should be designed such that anchors fully penetrate the Active Zone behind the shoring. The Active Zone can be considered the wedge of soil from the face of the shoring to a plane extending upward from the base of the excavation as shown on the Active Zone Detail. Normally, tieback anchors are contractor-designed and installed, and there are numerous anchor construction methods available. Non-shrinkage grout should be used for the construction of the tieback anchors.



Active Zone Detail

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

7.6.14 Anchor capacity is a function of construction method, depth of anchor, batter, diameter of the bonded section and the length of the bonded section. Anchor capacity should be evaluated using the strength parameters shown in Table 7.6.2.

TABLE 7.6.2
SOIL STRENGTH PARAMETERS FOR TEMPORARY SHORING

Description	Cohesion (psf)	Friction Angle (Degrees)
Previously Placed Fill	250	24
Very Old Paralic Deposits	500	31
Scripps Formation	500	31

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

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

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

7.6.18 Tieback anchors within the City of San Diego right-of-way should be properly detentioned and removed where steel does not exist within the upper 20 feet from the existing grade. The Notice – Land Development Review/Shoring in City Right-Of-Way, prepared by the

City of San Diego, dated July 1, 2003 should be reviewed and incorporated into the design of the tieback anchors. Procedures for removal of tieback anchors include unscrewing tendons using special couplings, use of explosives, or heat induction. Geocon Incorporated should be consulted if other methods of removal are planned.

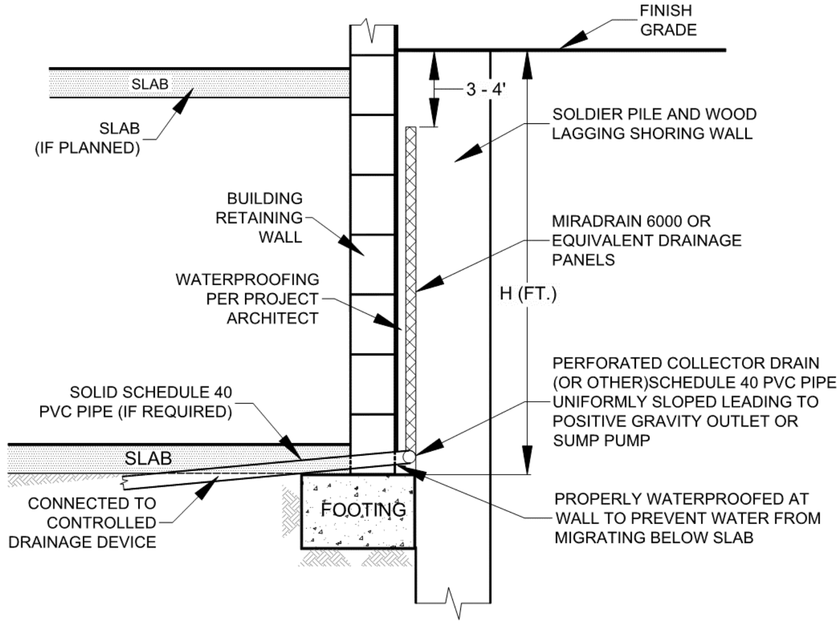
7.6.19 If a raker system is employed, the rakers should not be inclined steeper than 1:1 (horizontal:vertical) to provide an excavation to the raker foundation system with an inclination less than 1:1. A shallow or deep foundation system can be used for the raker system.

7.6.20 Shallow foundations for the raker system should consist of continuous strip footings and/or isolated spread footings as presented in Table 7.6.3.

TABLE 7.6.3
SUMMARY OF RAKER FOUNDATION RECOMMENDATIONS

Parameter	Value
Minimum Continuous Foundation Width	12 inches
Minimum Isolated Foundation Width	24 inches
Minimum Foundation Depth	12 Inches Below Lowest Adjacent Grade
Minimum Steel Reinforcement	Per Structural Engineer
Bearing Capacity	8,000 psf (bearing in Formation)
Bearing Capacity Increase	500 psf per Foot of Depth
	300 psf per Foot of Width
Maximum Bearing Capacity	11,000 psf (bearing in Formation)

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



Shoring Retaining Wall Drainage Detail

7.7 Soil Nail Wall

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

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

7.7.3 The wall should be designed by an engineer familiar with the design of soil nail walls.

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

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

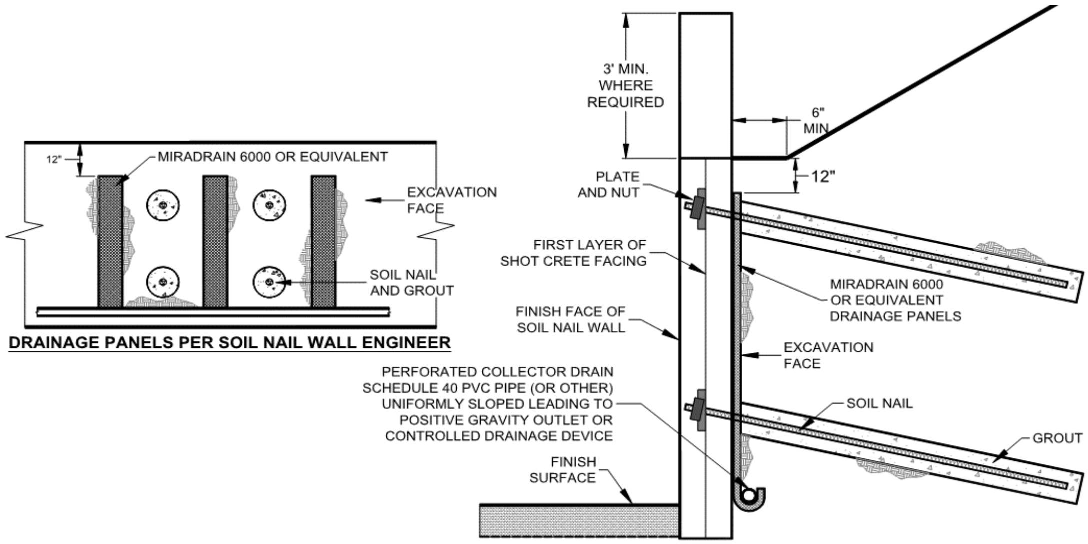
7.7.6 The soil strength parameters listed in Table 7.7 can be used in design of the soil nails. The bond stress is dependent on drilling method, diameter, and construction method (i.e. pressure grouting can increase the bond stress). Therefore, the designer should evaluate the bond stress based on the existing soil conditions and the construction method.

TABLE 7.7
SOIL STRENGTH PARAMETERS FOR SOIL NAIL WALLS

Description	Soil Unit Weight (pcf)	Cohesion (psf)	Friction Angle (degrees)	Estimated Ultimate Bond Stress (psi)
Previously Placed Fill	120	200	26	5
Very Old Paralic Deposits	125	500	31	20
Scripps Formation/Ardath Shale	125	500	31	20

*Assuming gravity fed, open hole drilling techniques.

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



Soil Nail Wall Drainage Detail

7.8 Shallow Foundation Recommendations – Building/Parking Structure

7.8.1 The proposed building with subterranean parking can be supported on a shallow foundation system founded in formational materials. Foundations for the structure should consist of continuous strip footings and/or isolated spread footings. In addition, the foundations should extend at least 6 inches into the Very Old Paralic Deposits or Scripps Formation. Table 7.8.1 provides a summary of the foundation design recommendations.

TABLE 7.8.1
SUMMARY OF FOUNDATION RECOMMENDATIONS – BUILDING WITH SUBTERRANEAN PARKING

Parameter	Value
Minimum Continuous Foundation Width, W_C	12 inches
Minimum Isolated Foundation Width, W_I	24 inches
Minimum Foundation Depth, D	24 Inches Below Lowest Adjacent Grade
Minimum Steel Reinforcement	4 No. 5 Bars, 2 at the Top and 2 at the Bottom
Allowable Bearing Capacity	8,000 psf (in Formation)
Bearing Capacity Increase	500 psf per Foot of Depth
	500 psf per Foot of Width
Maximum Allowable Bearing Capacity	11,000 psf
Estimated Total Settlement	1 Inch
Estimated Differential Settlement	½ Inch in 40 Feet
Footing Size Used for Settlement	14-Foot Square
Design Expansion Index	90 or less

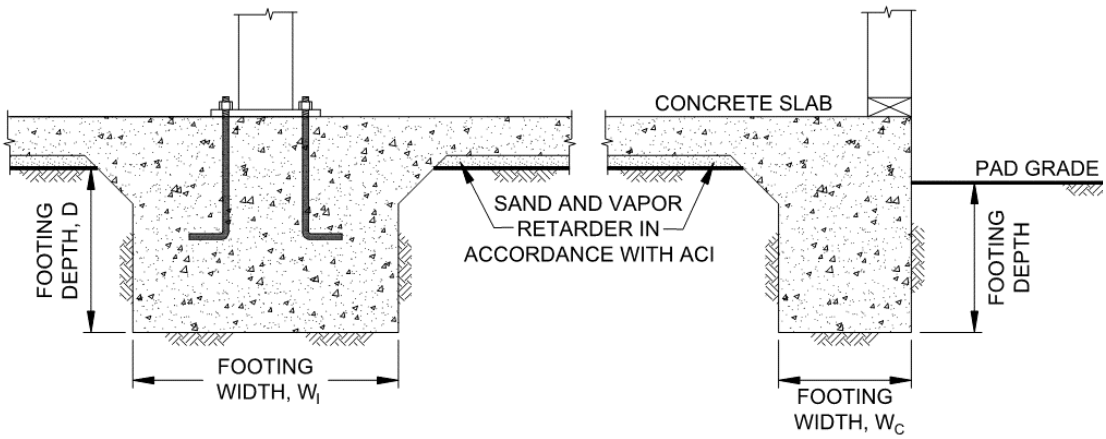
7.8.2 Overexcavation of the footings and replacement with slurry should be performed in areas where the Very Old Paralic Deposits or Scripps Formation is not encountered at the bottom of the footing. Minimum two-sack slurry can be placed in the excavations for the conventional foundations to the bottom of proposed footing elevation. Drilled piers may be required where the excavation depth exceeds about 10 feet. The depth of the overexcavation will likely exceed 10 feet in the southeastern portion of the site.

7.8.3 We understand ancillary structures and retaining walls may be planned for the property. The proposed ancillary structures can be supported on a shallow foundation system founded in the compacted fill/formational materials. Foundations for the ancillary structures should consist of continuous strip footings and/or isolated spread footings. Footings should be deepened such that the bottom outside edge of the footing is at least 7 feet horizontally from the face of the slope. Table 7.8.2 provides a summary of the foundation design recommendations for footings embedded in compacted fill.

TABLE 7.8.2
SUMMARY OF FOUNDATION RECOMMENDATIONS – ANCILLARY STRUCTURES

Parameter	Value
Minimum Continuous Foundation Width	12 inches
Minimum Isolated Foundation Width	12 inches
Minimum Foundation Depth	12 Inches Below Lowest Adjacent Grade
Minimum Steel Reinforcement	Per Structural
Allowable Bearing Capacity	2,000 psf
Bearing Capacity Increase	500 psf per Foot of Depth
	300 psf per Foot of Width
Maximum Allowable Bearing Capacity	3,000 psf
Estimated Total Settlement	1 Inch
Estimated Differential Settlement	½ Inch in 40 Feet
Footing Size Used for Settlement	5-Foot Square
Design Expansion Index	90 or less

7.8.4 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

7.8.5 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.

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

- 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.
- Swimming pools located within 7 feet of the top of cut or fill slopes are not recommended. Where such a condition cannot be avoided, the portion of the swimming pool wall within 7 feet of the slope face be designed assuming that the adjacent soil provides no lateral support.
- Although other improvements, which are relatively rigid or brittle, such as concrete flatwork or masonry walls, may experience some distress if located near the top of a slope, it is generally not economical to mitigate this potential. It may be possible, however, to incorporate design measures that would permit some lateral soil movement without causing extensive distress. Geocon Incorporated should be consulted for specific recommendations.

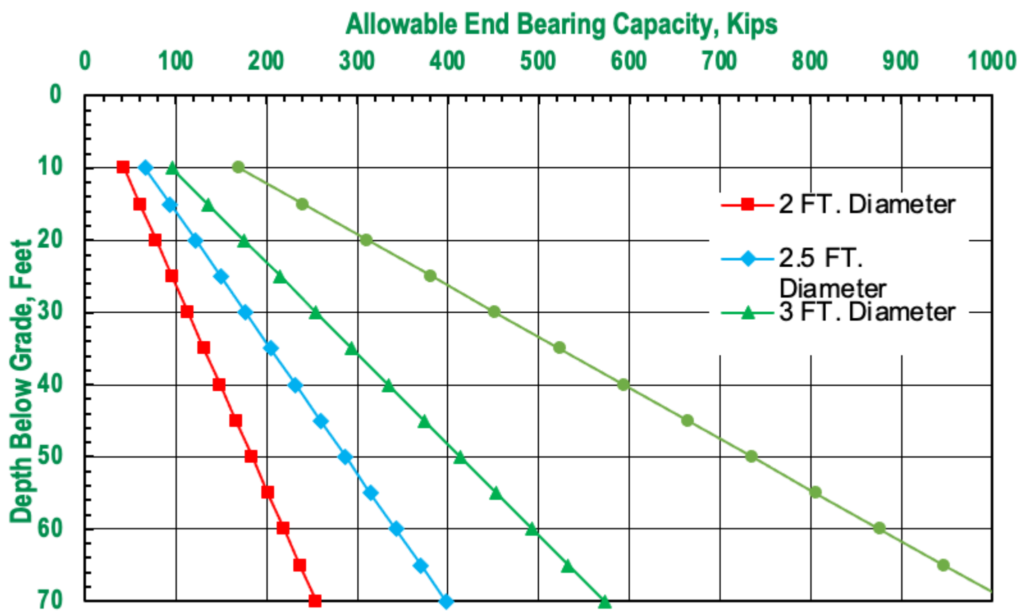
7.8.7 We should observe the foundation excavations prior to the placement of reinforcing steel and concrete to check that the exposed soil conditions are similar to those expected and that they have been extended to the appropriate bearing strata. Foundation modifications may be required if unexpected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

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

7.9 Drilled Pier Recommendations

7.9.1 Deep foundations can be utilized in areas if previously placed fill is present at the base of the building excavation, likely where the fill extends deeper than about 5 to 10 feet in the southeastern portion of the proposed building.

7.9.2 Piers can be designed to develop support by end bearing within the formational materials and skin friction within the formational materials and portions of the fill soil. 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 for skin friction and end bearing, respectively. The chart assumes the piles will be embedded at least 5 feet into the formational materials.



End Bearing Capacity Chart

7.9.3 Piers can be designed to develop support by end bearing within the formational materials and skin friction within the formational materials and portions of the fill soil using the design parameters presented in Table 7.9.

TABLE 7.9
SUMMARY OF DRILLED PIER RECOMMENDATIONS

Parameter	Value
Minimum Pile Diameter	2 Feet
Minimum Pile Spacing	3 Times Pile Diameter
Minimum Foundation Embedment Depth	5 Feet in Formational Materials
Allowable End Bearing Capacity	Per Chart
Allowable Skin Friction Capacity	500 psf (Formational Materials)
Estimated Total Settlement	½ Inch
Estimated Differential Settlement	½ Inch in 40 Feet

7.9.4 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.

7.9.5 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.

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

7.9.7 The existing materials may contain gravel and cobble and may possess very dense zones; therefore, the drilling contractor should expect difficult drilling conditions during excavations for the piers. Because a significant portion of the piers capacity will be developed by end bearing, the bottom of the borehole should be cleaned of loose cuttings prior to the placement of steel and concrete. Experience indicates that backspinning the auger does not remove loose material and a flat cleanout plate is necessary. 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.

7.9.8 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.

7.10 Concrete Slabs-On-Grade Recommendations

7.10.1 Concrete slabs-on-grade for the structures should be constructed in accordance with Table 7.10.1.

TABLE 7.10
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	90 or less

7.10.2 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.

7.10.3 The bedding sand thickness should be determined by the project foundation engineer, architect, and/or developer. It is common to see 3 inches and 4 inches of sand below the concrete slab-on-grade in the southern California area. 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.

7.10.4 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.

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

7.10.6 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.

7.10.7 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.

7.11 Concrete Flatwork Recommendations

7.11.1 Exterior concrete flatwork not subject to vehicular traffic should be constructed in accordance with the recommendations presented in Table 7.11. The recommended steel reinforcement would help reduce the potential for cracking.

TABLE 7.11
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.

7.11.2 The subgrade soil should be properly moisturized and compacted prior to the placement of steel and concrete. The subgrade soil should be compacted to a dry density of at least 90 percent of the laboratory maximum dry density near to slightly above optimum moisture content in accordance with ASTM D 1557.

7.11.3 Even with the incorporation of the recommendations within this report, the exterior concrete flatwork has a likelihood of experiencing some uplift due to expansive soil beneath grade; therefore, 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.

7.11.4 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.

7.11.5 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.

7.11.6 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.

7.12 Retaining Walls

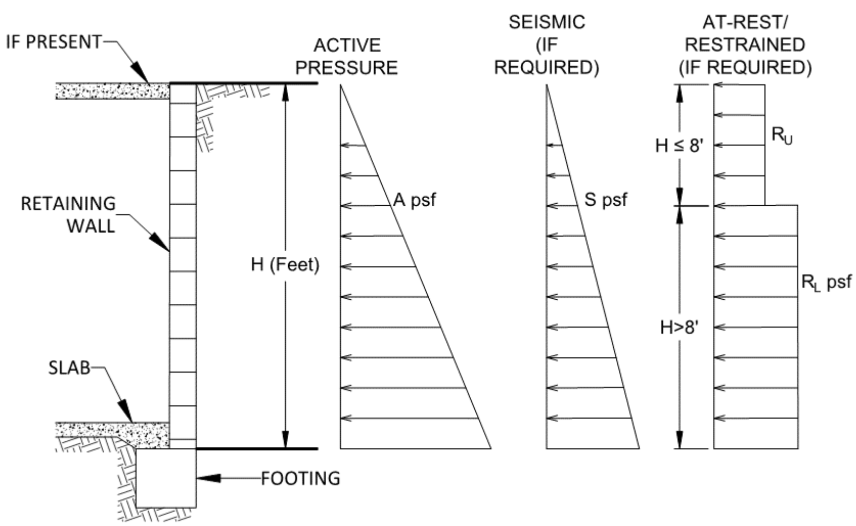
7.12.1 Retaining walls should be designed using the values presented in Table 7.12.1. Soil with an expansion index (EI) of greater than 90 should not be used as backfill material behind retaining walls.

TABLE 7.12.1
RETAINING WALL DESIGN RECOMMENDATIONS

Parameter	Value
Active Soil Pressure, A (Fluid Density, Level Backfill)	40 pcf
Active Soil Pressure, A (Fluid Density, 2:1 Sloping Backfill)	55 pcf
Seismic Pressure, S	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
Expected Expansion Index for the Subject Property	EI$\leq$90

H equals the height of the retaining portion of the wall

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



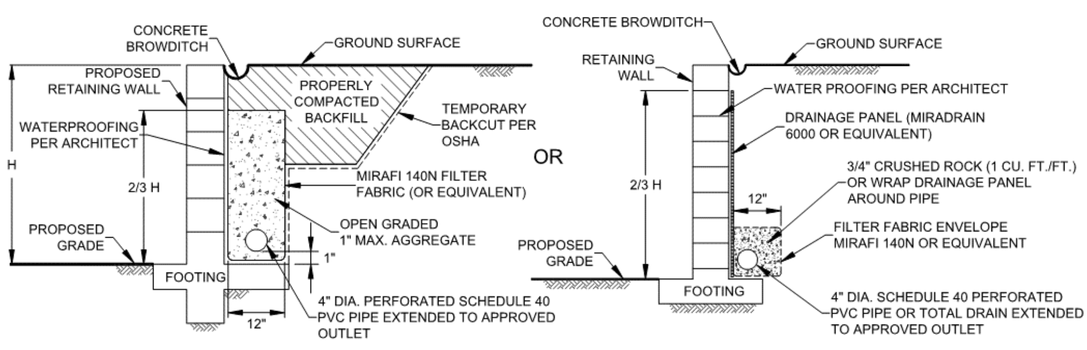
Retaining Wall Loading Diagram

7.12.3 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 applied to the wall. 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.

7.12.4 The structural engineer should determine the Seismic Design Category for the project in accordance with Section 1613.3.5 of the 2019 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 2019 CBC. The seismic load is dependent on the retained height where H is the height of the wall, in feet, and the calculated loads result in pounds per square foot (psf) exerted at the base of the wall and zero at the top of the wall.

7.12.5 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.

7.12.6 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

7.12.7 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.

7.12.8 In general, wall foundations should be designed in accordance with Table 7.11.2. 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 7.12.2
SUMMARY OF RETAINING WALL FOUNDATION RECOMMENDATIONS

Parameter	Value
Minimum Retaining Wall Foundation Width	12 inches
Minimum Retaining Wall Foundation Depth	12 Inches
Minimum Steel Reinforcement	Per Structural Engineer
Allowable Bearing Capacity	2,000 psf
Bearing Capacity Increase	500 psf per Foot of Depth
	300 psf per Foot of Width
Maximum Allowable Bearing Capacity	3,000 psf
Estimated Total Settlement	1 Inch
Estimated Differential Settlement	½ Inch in 40 Feet

7.12.9 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.

7.12.10 It is common to see retaining walls constructed in the areas of the elevator pits. The retaining walls should be properly drained and designed in accordance with the recommendations presented herein. If the elevator pit walls are not drained, the walls should be designed with an increased active pressure with an equivalent fluid density of 90 pcf. It is also common to see seepage and water collection within the elevator pit. The pit should be designed and properly waterproofed to prevent seepage and water migration into the elevator pit.

7.12.11 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.

7.12.12 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.

7.13 Lateral Loading

7.13.1 Table 7.13 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 7.13
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.

7.13.2 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.

7.14 Preliminary Pavement Recommendations

7.14.1 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 should be based on the R-Value of the subgrade soil encountered at final subgrade elevation. We assumed an R-Value of 6 and 78 for the subgrade soil and base materials, respectively, for the purposes of this preliminary analysis, based on our laboratory test results. Table 7.14.1 presents the preliminary flexible pavement sections.

TABLE 7.14.1
PRELIMINARY FLEXIBLE PAVEMENT SECTION

Location	Assumed Traffic Index	Assumed Subgrade R-Value	Asphalt Concrete (inches)	Class 2 Aggregate Base (inches)
Parking stalls for automobiles and light-duty vehicles	5.0	6	3	10
Driveways for automobiles and light-duty vehicles	5.5	6	3	12
Medium Truck Traffic Areas	6.0	6	3.5	13
Driveways for heavy truck traffic	7.0	6	4	16

7.14.2 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 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. Asphalt concrete should be compacted to a density of at least 95 percent of the laboratory Hveem density in accordance with ASTM D 2726.

7.14.3 Base materials should conform to Section 26-1.028 of the *Standard Specifications for the State of California Department of Transportation (Caltrans)* with a ¾-inch maximum size aggregate. The asphalt concrete should conform to Section 203-6 of the *Standard Specifications for Public Works Construction (Greenbook)*.

7.14.4 The base thickness can be reduced if a reinforcement geogrid or cement-treated base materials are used during the installation of the pavement. Geocon should be contact for additional recommendations, if required.

7.14.5 A rigid Portland Cement concrete (PCC) pavement section should be placed in driveway entrance aprons, trash bin loading/storage areas and loading dock areas. The concrete pad for trash truck areas should be large enough such that all the truck wheels will be positioned on the concrete during loading. 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 7.14.2.

TABLE 7.14.2
RIGID PAVEMENT DESIGN PARAMETERS

Design Parameter	Design Value
Modulus of subgrade reaction, k	50 pci
Modulus of rupture for concrete, M_R	500 psi
Concrete Compressive Strength	3,000 psi
Traffic Category, TC	A and C
Average daily truck traffic, ADTT	10 and 100

7.14.6 Based on the criteria presented herein, the PCC pavement sections should have a minimum thickness as presented in Table 7.14.3.

TABLE 7.14.3
RIGID PAVEMENT RECOMMENDATIONS

Location	Portland Cement Concrete (inches)
Automobile Parking Areas (TC=A-1)	6.0
Heavy Truck and Fire Lane Areas (TC=C)	7.5

7.14.7 The PCC 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. The use of base materials below concrete surface improvements will not be required.

7.14.8 The rigid pavement should also be designed and constructed incorporating the parameters presented in Table 7.14.4.

TABLE 7.14.4
ADDITIONAL RIGID PAVEMENT RECOMMENDATIONS

Subject	Value
Thickened Edge	1.2 Times Slab Thickness
	Minimum Increase of 2 Inches
	4 Feet Wide
Crack Control Joint Spacing	30 Times Slab Thickness
	Max. Spacing of 12 feet for 5.5-Inch-Thick
	Max. Spacing of 15 Feet for Slabs 6 Inches and Thicker
Crack Control Joint Depth	Per ACI 330R-08
	1 Inch Using Early-Entry Saws on Slabs Less Than 9 Inches Thick
Crack Control Joint Width	¼-Inch for Sealed Joints
	⅜-Inch is Common for Sealed Joints
	$^{1}/_{10}$- to $^{1}/_{8}$-Inch is Common for Unsealed Joints

7.14.9 Reinforcing steel will not be necessary within the concrete for geotechnical purposes with the possible exception of dowels at construction joints as discussed herein.

7.14.10 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 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.

7.14.11 To provide load transfer between adjacent pavement slab sections, a butt-type construction joint should be installed. 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.

7.14.12 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.

7.14.13 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.

7.15 Slope Maintenance

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

7.16 Site Drainage and Moisture Protection

7.16.1 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.

7.16.2 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.

7.16.3 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.

7.16.4 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.

7.16.5 We should prepare a storm water infiltration feasibility report of storm water management devices are planned.

7.17 Grading and Foundation Plan Review

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

7.18 Testing and Observation Services During Construction

7.18.1 Geocon Incorporated should provide geotechnical testing and observation services during the grading operations, foundation construction, utility installation, retaining wall backfill and pavement installation. Table 7.18 presents the typical geotechnical observations we would expect for the proposed improvements.

TABLE 7.18
EXPECTED GEOTECHNICAL TESTING AND OBSERVATION SERVICES

<table>
<tr><th>Construction Phase</th><th>Observations</th><th>Expected Time Frame</th></tr>
<tr><td rowspan="3">Grading</td><td>Base of Removal</td><td>Part Time During Removals</td></tr>
<tr><td>Geologic Logging</td><td>Part Time to Full Time</td></tr>
<tr><td>Fill Placement and Soil Compaction</td><td>Full Time</td></tr>
<tr><td>Soldier Piles</td><td>Solder Pile Drilling Depth</td><td>Part Time</td></tr>
<tr><td rowspan="2">Tieback Anchors</td><td>Tieback Drilling and Installation</td><td>Full Time</td></tr>
<tr><td>Tieback Testing</td><td>Full Time</td></tr>
<tr><td rowspan="2">Soil Nail Walls</td><td>Soil Nail Drilling and Installation</td><td>Full Time</td></tr>
<tr><td>Soil Nail Testing</td><td>Full Time</td></tr>
<tr><td rowspan="2">Foundations</td><td>Drilling Operations for Piles</td><td>Full Time</td></tr>
<tr><td>Foundation Excavation Observations</td><td>Part Time</td></tr>
<tr><td>Utility Backfill</td><td>Fill Placement and Soil Compaction</td><td>Part Time to Full Time</td></tr>
<tr><td>Retaining Wall Backfill</td><td>Fill Placement and Soil Compaction</td><td>Part Time to Full Time</td></tr>
<tr><td>Subgrade for Sidewalks, Curb/Gutter and Pavement</td><td>Soil Compaction</td><td>Part Time</td></tr>
<tr><td rowspan="2">Pavement Construction</td><td>Base Placement and Compaction</td><td>Part Time</td></tr>
<tr><td>Asphalt Concrete Placement and Compaction</td><td>Full Time</td></tr>
</table>

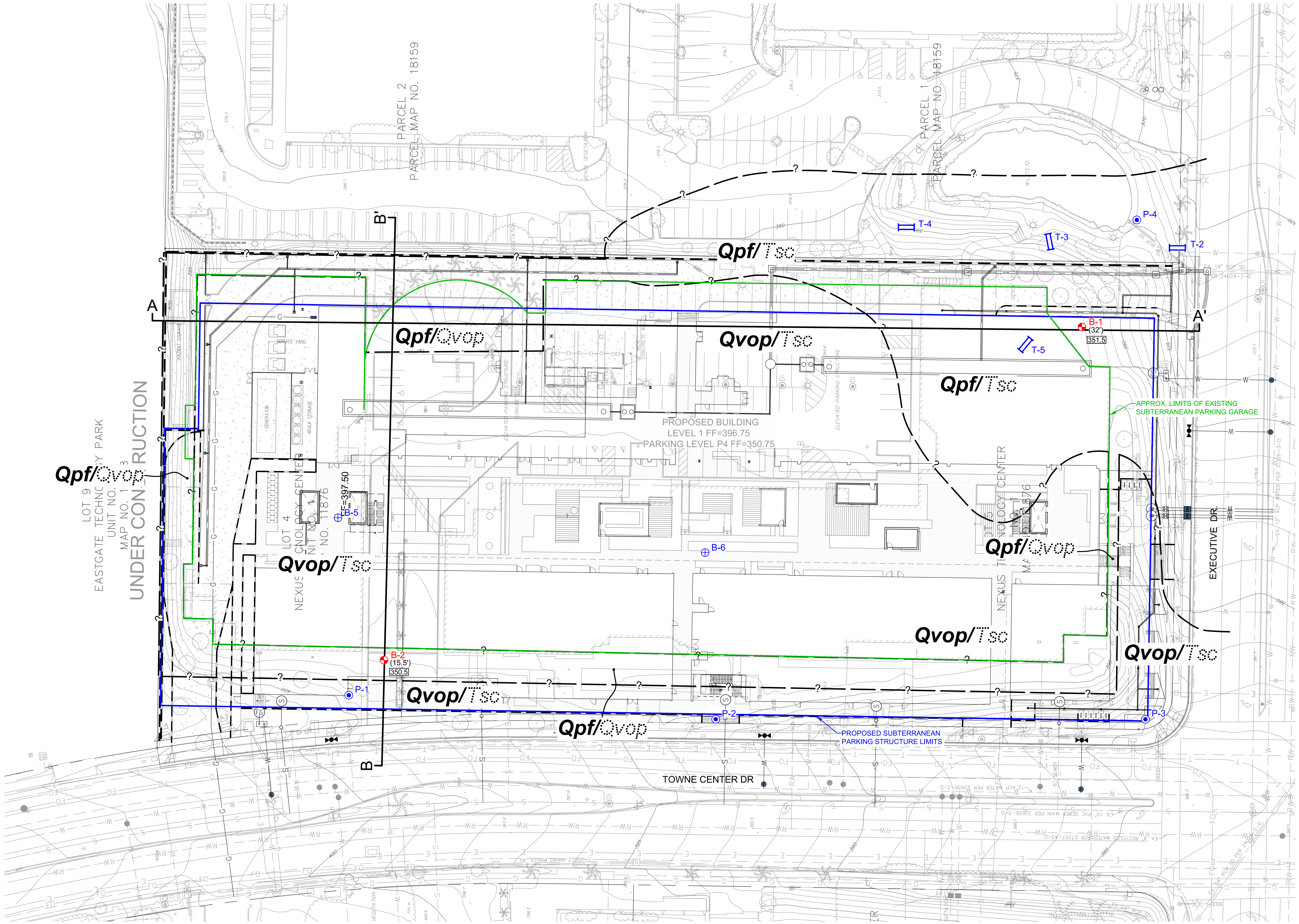
LIMITATIONS AND UNIFORMITY OF CONDITIONS

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.



GRAPHIC SCALE

0' 15' 30' 45' 60' 90' 120'

SCALE 1"=30' (on 36x24)

GEOCON LEGEND

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

Qvop........VERY OLD PARALIC DEPOSITS (Dotted Where Buried)

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

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

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

P-4 ....APPROX. LOCATION OF INFILTRATION TEST

B-5 & B-6 ....APPROX. LOCATION OF PREVIOUS BORING (Geocon, 1985)

T-2 - T-5 ....APPROX. LOCATION OF PREVIOUS TRENCH (Geocon, 1985)

(32') ....APPROX. DEPTH TO FORMATION BELOW EXISTING GRADE (In Feet, MSL)

351.5 ....APPROX. ELEVATION OF FORMATION (In Feet, MSL)

B B' ....APPROX. LOCATION OF GEOLOGIC CROSS-SECTION

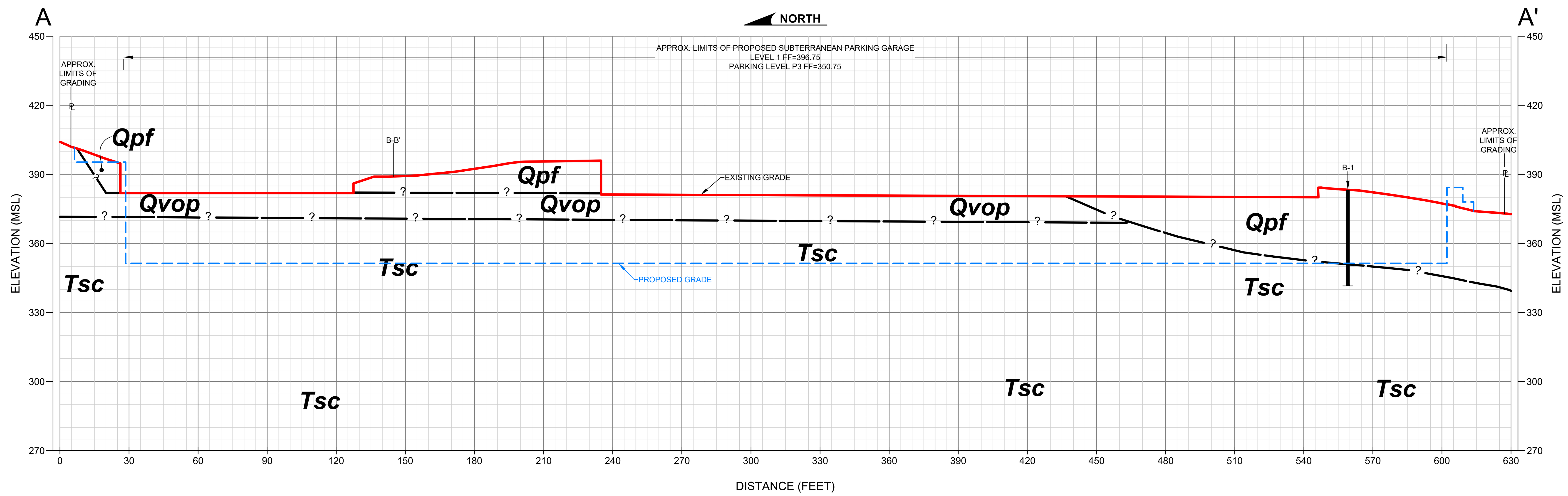
GEOLOGIC MAP

ALEXANDRIA SCIENCE VILLAGE
9393 TOWNE CENTRE DRIVE
SAN DIEGO, CALIFORNIA

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

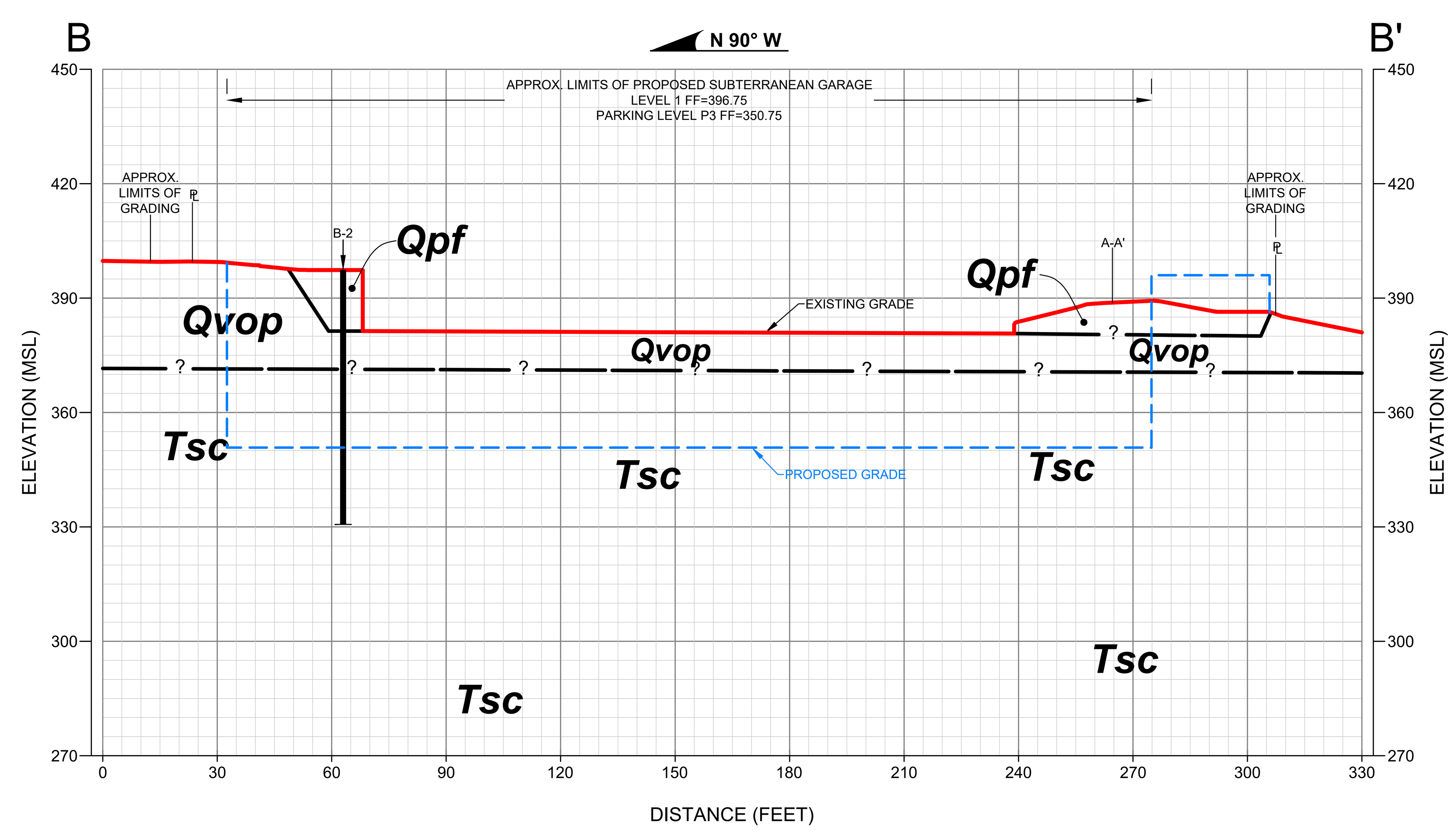
SCALE 1" = 30'	DATE 06 - 22 - 2022		
PROJECT NO. G2101 - 52 - 02		FIGURE 1	
SHEET 1 OF 1			

Plotted:06/22/2022 8:35AM | By:ALVIN LADRILLONO | File Location:Y:\PROJECTS\G2101-52-02 (Podium 93)\SHEETS\G2101-52-02 Geo Map 20.dwg



GEOLOGIC CROSS-SECTION A-A'

SCALE: 1" = 30' (Vert. = Horiz.)



GEOLOGIC CROSS-SECTION B-B'

SCALE: 1" = 30' (Vert. = Horiz.)

GEOCON LEGEND

Qpf ....PREVIOUSLY PLACED FILL

Qvop ....VERY OLD PARALIC DEPOSITS

Tsc ....SCRIPPS FORMATION

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

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

GEOLOGIC CROSS - SECTIONS

ALEXANDRIA SCIENCE VILLAGE
9393 TOWN CENTER DRIVE
SAN DIEGO, CALIFORNIA

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

SCALE 1" = 30'	DATE 06 - 22 - 2022		
PROJECT NO. G2101 - 52 - 02		FIGURE 2	
SHEET 1 OF 1			

APPENDIX A

APPENDIX A

FIELD INVESTIGATION

We performed the field investigation during period of March 14, 2017. Our subsurface exploration consisted of drilling 2 small-diameter exploratory borings to a maximum depth of approximately 67 feet using a truck-mounted drill rig and an 8-inch hollow stem auger. The approximate locations of the exploratory borings are shown on the Geologic Map, Figure 2. Boring logs, and an explanation of the geologic units encountered are presented on Figures A-1 through A-2. We located the borings in the field using existing reference points; therefore, actual locations may deviate slightly.

We obtained samples during our boring excavations using a California split-spoon sampler or a Standard Penetration Test (SPT) sampler. Both samplers are composed of steel and are driven to obtain the soil samples. The California sampler has an inside diameter of 2.5 inches and an outside diameter of 2.875 inches. Up to 18 rings that are 2.4 inches in diameter and 1 inch in height are placed inside the sampler. The SPT sampler has an inside diameter of 1.5 inches and an outside diameter of 2 inches. Ring samples at appropriate intervals were retained in moisture-tight containers and transported to the laboratory for testing. Bulk samples were also retained from the borings for laboratory testing. The type of sample is noted on the exploratory boring logs.

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

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

BORING B 1

ELEV. (MSL.) 383.5' DATE COMPLETED 03-14-2017

EQUIPMENT DIETRICH 120 BY: K. JAMES

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0					4" **REINFORCED CONCRETE PAVEMENT** Over 3" **BASE**			
	B1-1			SC	**PREVIOUSLY PLACED FILL (Qpf)** Medium dense, damp, reddish brown, Silty, fine to coarse SAND			
2								
4								
	B1-2				-Rock in sampler tip	48	120.4	11.4
6								
8								
10	B1-3				-Becomes mottled reddish brown and light brown	26	104.3	14.6
12								
14								
	B1-4				-2" cobble on top of sampler; some concretion; few gravel; trace carbon; trace mica	34	111.5	11.3
16								
18					-Difficult drilling			
20	B1-5				-Becomes dense, two 2-inch cobbles stuck in sampler rings at top	66	123.7	12.1
22								
24					-Difficult drilling			
					-No recovery; very dense; gravel in drilling cuttings	81/11"		
26								
28					-Difficult drilling			
				SM	Medium dense, damp, reddish brown, Silty, fine to medium SAND; moderately cemented; trace gravel; trace carbon; trace clay			

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

G2101-52-01.GPJ

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

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

GEOCON

BORING B 1

ELEV. (MSL.) 383.5' DATE COMPLETED 03-14-2017

EQUIPMENT DIETRICH 120 BY: K. JAMES

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
30	B1-6					27	116.7	9.7
32	B1-7			ML	-Significant color change **SCRIPPS FORMATION (Tsc)** Medium dense, damp, yellowish brown, Sandy SILTSTONE; strongly indurated; trace clay -Very difficult drilling			
34								
36	B1-8					18	106.4	12.1
38					-Becomes very dense, light brown, carbonize			
40	B1-9					88/8"	100.2	13.1
	B1-10				-Drilling terminated after 30 minutes and no increase in depth	50/5"		

REFUSAL AT 41.35 FEET
Groundwater not encountered
Backfilled with 14.1 ft³ bentonite grout
Patched with concrete

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

G2101-52-01.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 2

ELEV. (MSL.) **396'** DATE COMPLETED **03-15-2017**

EQUIPMENT **DIETRICH 120** BY: **K. JAMES**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0					3" **REINFORCED CEMENT PAVEMENT** Over 6" **BASE**			
	B2-1			SM	**PREVIOUSLY PLACED FILL (Qpf)** Loose, moist, reddish brown, Silty, fine to coarse SAND; few gravel -Hand auger 5 feet			
2								
4								
	B2-2					12	116.1	12.5
6								
8					-Becomes brown to reddish brown			
10	B2-3					6	109.4	13.1
12								
14								
	B2-4					50/5"	100.0	13.1
16				SM	**VERY OLD PARALIC DEPOSITS (Qvop)** Very dense, damp, reddish brown, Silty, fine to coarse SANDSTONE; moderately cemented -2" rock in sampler tip -difficult drilling			
18								
20	B2-5					74/11"		
	B2-6							
22								
24								
	92.2			ML	**SCRIPPS FORMATION (Tsc)** Very dense, damp, light yellowish brown, Clayey SILTSTONE with sand; moderately indurated	50/4"	92.2	13.0
26								
28								

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

G2101-52-01.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 2

ELEV. (MSL.) 396' DATE COMPLETED 03-15-2017

EQUIPMENT DIETRICH 120 BY: K. JAMES

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
30	B2-8				-Becomes light yellowish brown to light gray; strongly indurated	83/10"		
32					-Difficult drilling			
34				CL	Medium dense, dry, yellowish brown to olive brown,Silty CLAYSTONE; moderately cemented; some dark brown concretions			
	B2-9					22	107.4	18.7
36								
					-Very difficult drilling			
38								
40								
42				ML	Very dense, dry, yellowish brown, Clayey SILTSTONE; strong cemented -Very difficult drilling			
44								
	B2-10					50/5"	104.1	18.3
46								
48								
50								
				CL	Very dense, dry, light yellowish brown, Silty CLAYSTONE; strongly cemented -Very difficult drilling			
52								
54								
	B2-11					50/5"	104.2	18.6
56								
58								
					-Very difficult drilling; may be near refusal			

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

G2101-52-01.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 2

ELEV. (MSL.) 396' DATE COMPLETED 03-15-2017

EQUIPMENT DIETRICH 120 BY: K. JAMES

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
60	B2-12							
62								
64					-Very difficult drilling			
	B2-13					50/3"		
66					-Little to no drilling progress after 1 hour of drilling			

REFUSAL AT 67 FEET
Groundwater not encountered
Backfilled with 22.9 ft³ bentonite grout
Patched with concrete

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

G2101-52-01.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.

APPENDIX B

APPENDIX B

LABORATORY TESTING

We performed laboratory tests in accordance with generally accepted test methods of the American Society for Testing and Materials (ASTM) or other suggested procedures. We tested selected soil samples for in-place density and moisture content, maximum dry density and optimum moisture content, shear strength, expansion index, water-soluble sulfate characteristics, resistance value, gradation, consolidation characteristics, and unconfined compressive strength/undrained shear strength. The results of our laboratory tests are presented on the tables and figures, herein. In addition, the in-place dry density and moisture content results are presented on the boring logs in Appendix A.

SUMMARY OF LABORATORY MAXIMUM DRY DENSITY AND OPTIMUM MOISTURE CONTENT TEST RESULTS ASTM D 1557

Sample No.	Depth (feet)	Description (Geologic Unit)	Maximum Dry Density (pcf)	Optimum Moisture Content (% dry wt.)
B1-1	0-5	Reddish brown, Clayey, fine to coarse SAND (Qudf)	128.4	9.5
B2-1	0-5	Reddish brown, Silty, fine to coarse SAND; few gravel (Qudf)	132.6	8.2
B2-12	60-67	Light yellowish brown, Silty Claystone (Tsc)	127.7	9.8

SUMMARY OF LABORATORY DIRECT SHEAR TEST RESULTS ASTM D 3080

Sample No.	Depth (feet)	Geologic Unit	Dry Density (pcf)	Moisture Content (%)		Peak [Ultimate*] Cohesion [Ultimate*] (psf)	Peak [Ultimate*] Angle of Shear Resistance (degrees)
				Initial	Final		
B1-1**	0-5	Qudf	111.7	11.8	16.4	525 [425]	24 [24]
B1-6	30	Qvop	116.7	9.7	14.9	350 [350]	36 [36]
B2-9	35	Tsc	107.4	18.7	20.9	625 [475]	24 [24]
B2-11	55	Tsc	104.2	18.6	22.8	800 [725]	30 [28]

*Ultimate measured at 0.2-inch deflection.
**Sample Remolded

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	7.9	16.1	117.4	4	Non-Expansive	Very Low
B2-12	9.5	19.8	111.1	64	Expansive	Medium

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

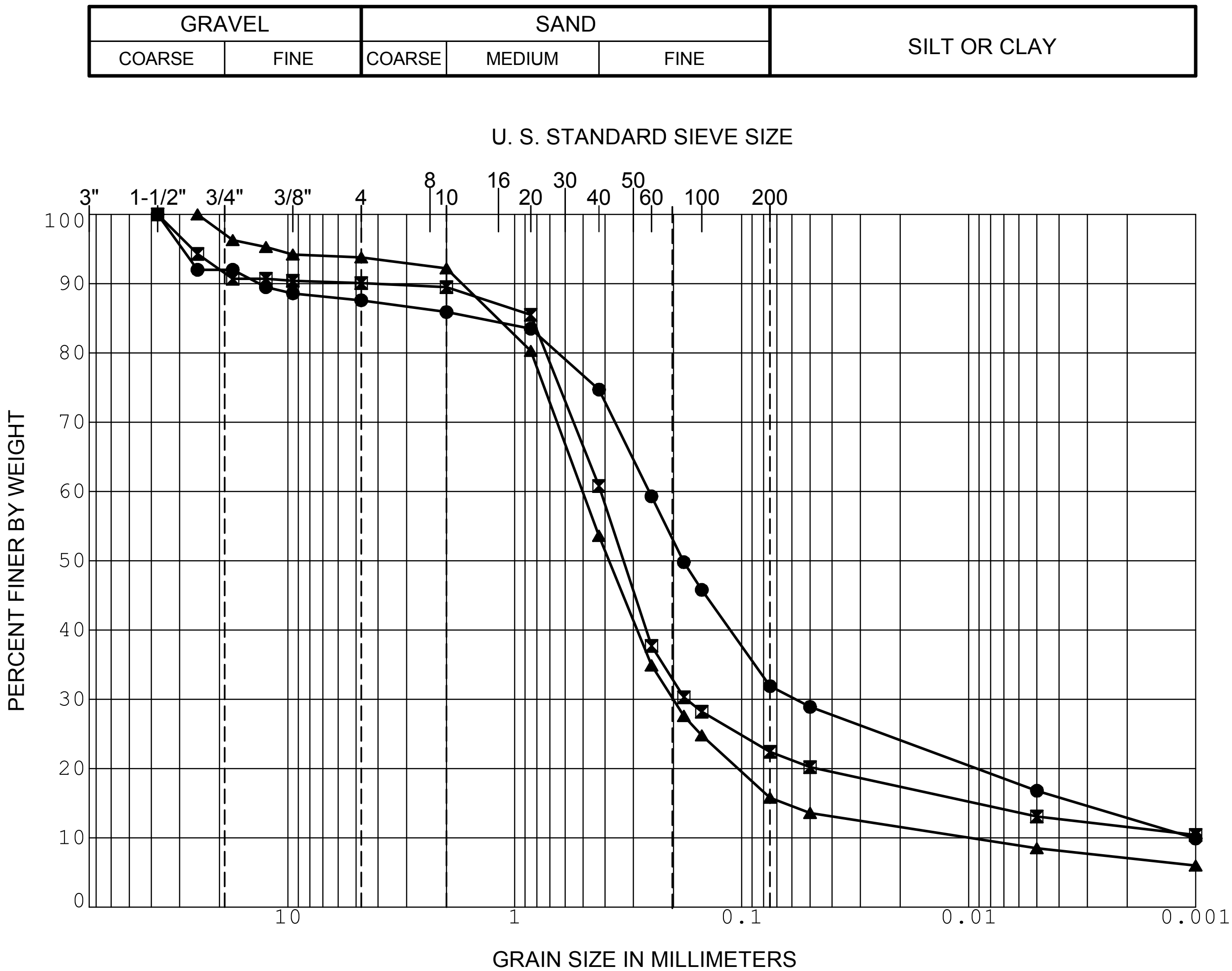
Sample No.	Depth (feet)	Geologic Unit	Water-Soluble Sulfate (%)	Sulfate Class
B2-1	0-5	Qudf	0.021	S0
B2-12	60-67	Tsc	0.011	S0

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

Sample No.	Depth (feet)	Description (Geologic Unit)	R-Value
B2-1	0-5	Reddish brown, Silty, fine to coarse SAND; few gravel (Qudf)	6

SUMMARY OF LABORATORY UNCONFINED COMPRESSIVE STRENGTH TEST RESULTS
ASTM D 1558

Sample No.	Depth (feet)	Geologic Unit	Hand Penetrometer Reading, Unconfined Compression Strength (tsf)	Undrained Shear Strength (ksf)
B1-2	5	Qudf	4.5	4.5
B1-4	15	Qudf	3.0	3.0
B1-5	20	Qudf	4.5	4.5
B1-8	35	Tsc	3.0	3.0
B1-9	40	Tsc	3.0	3.0
B2-4	15	Qudf	1.5	1.5
B2-7	25	Tsc	3.5	3.5
B2-10	45	Tsc	4.5	4.5
B2-11	55	Tsc	2.5	2.5

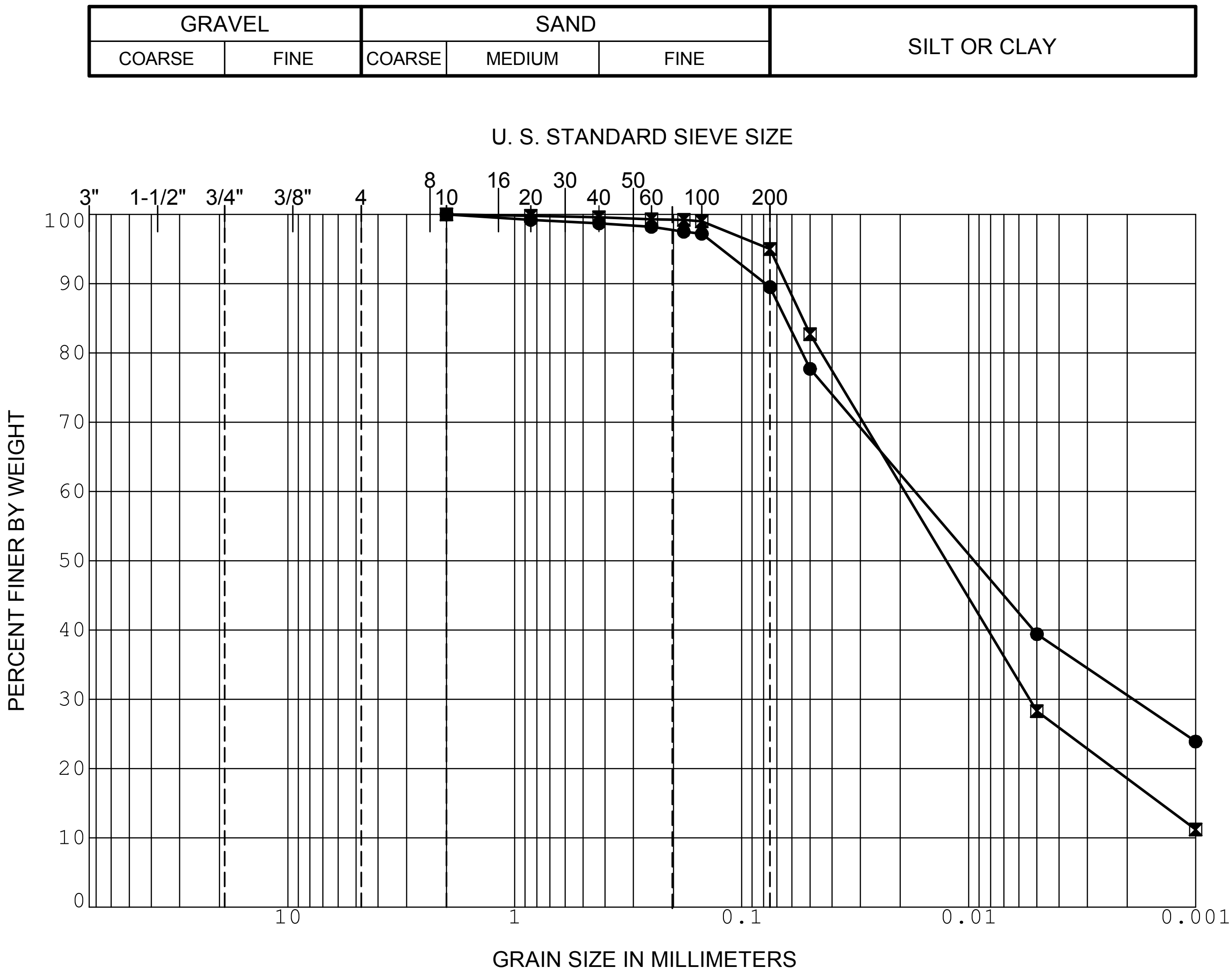


	SAMPLE	DEPTH (ft)	CLASSIFICATION	NAT WC	LL	PL	PI
●	B1-3	10.0	(SC) Clayey SAND	14.6			
⊠	B2-2	5.0	(SM) Silty SAND	12.5			
▲	B2-5	20.0	(SM) Silty SAND				

GRADATION CURVE

ALEXANDRIA SCIENCE VILLAGE
9363, 9373 AND 9393 TOWNE CENTRE DRIVE
SAN DIEGO, CALIFORNIA

PROJECT NO. G2101-52-02



	SAMPLE	DEPTH (ft)	CLASSIFICATION	NAT WC	LL	PL	PI
●	B2-9	35.0	(CL) Lean CLAY	18.7			
⊠	B2-11	55.0	(CL) Lean CLAY	18.6			
▲							

GRADATION CURVE

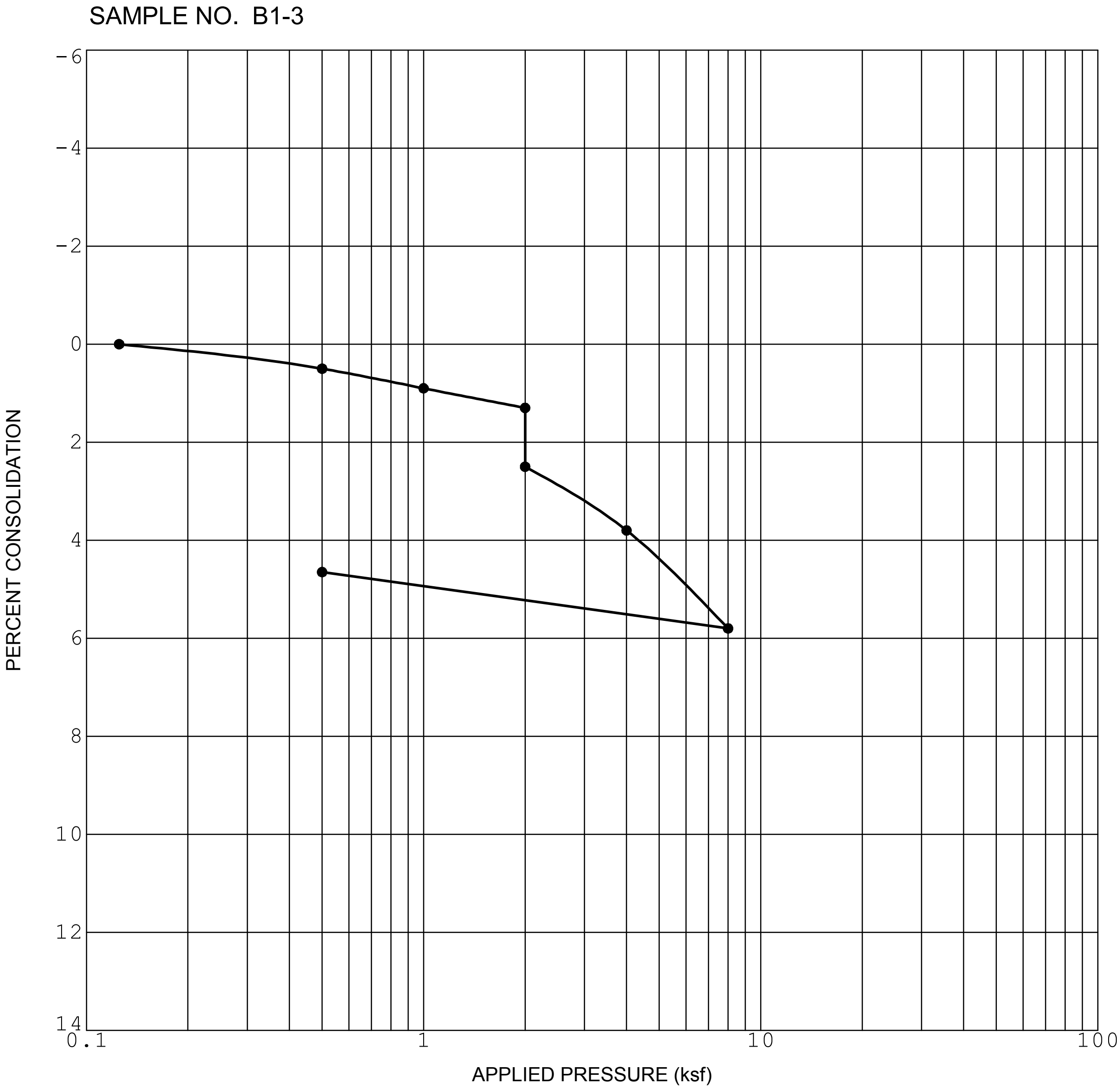
ALEXANDRIA SCIENCE VILLAGE
9363, 9373 AND 9393 TOWNE CENTRE DRIVE
SAN DIEGO, CALIFORNIA

G2101-52-01.GPJ

Figure B-2

GEOCON

PROJECT NO. G2101-52-02



Initial Dry Density (pcf)	104.3
Initial Water Content (%)	14.6

Initial Saturation (%)	65.8
Sample Saturated at (ksf)	2.0

CONSOLIDATION CURVE

ALEXANDRIA SCIENCE VILLAGE
9363, 9373 AND 9393 TOWNE CENTRE DRIVE
SAN DIEGO, CALIFORNIA

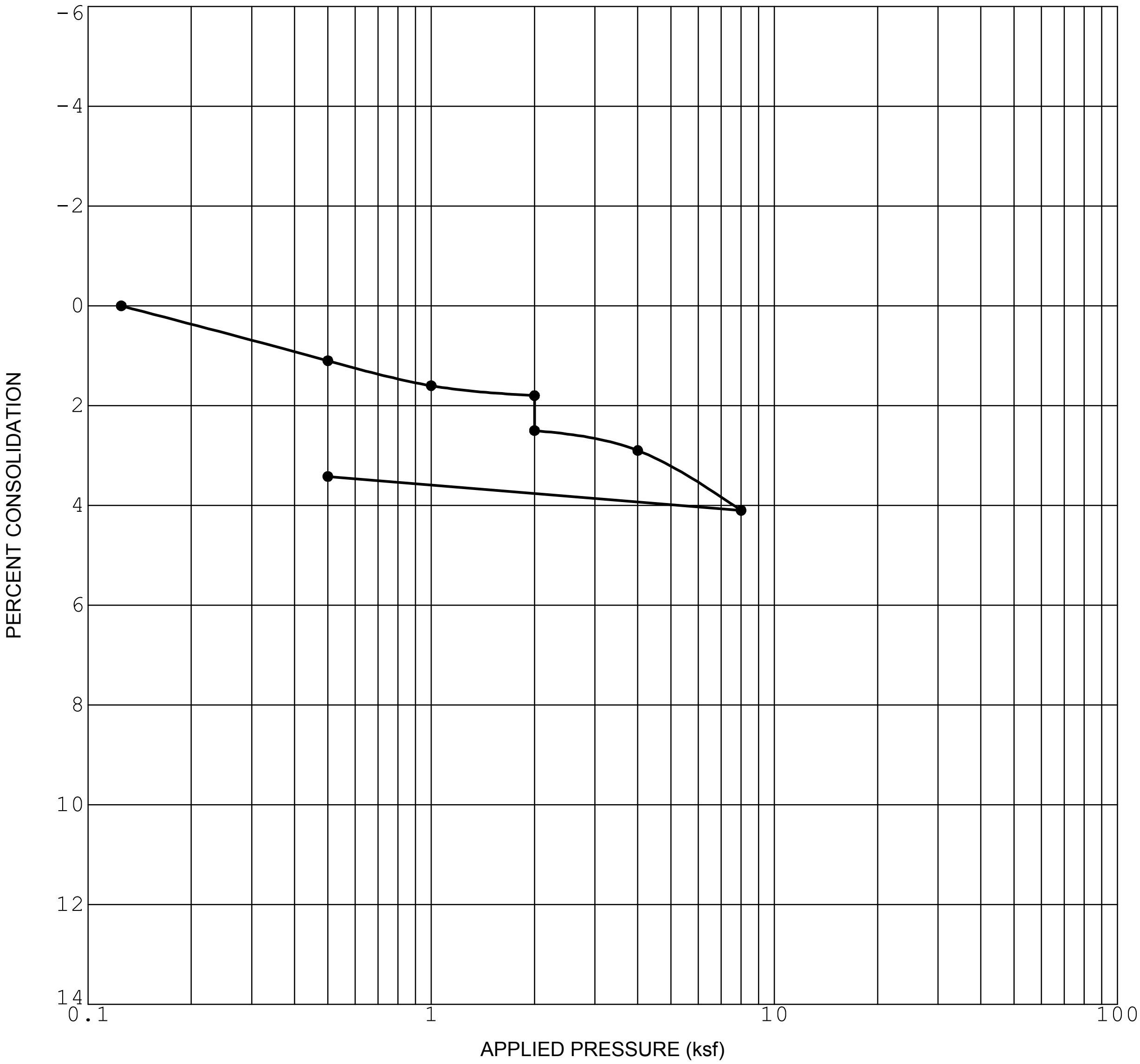
G2101-52-01.GPJ

Figure B-3

GEOCON

PROJECT NO. G2101-52-02

SAMPLE NO. B2-2



Initial Dry Density (pcf)	116.1		**Initial Saturation (%)**	77.8
Initial Water Content (%)	12.5		**Sample Saturated at (ksf)**	2.0

CONSOLIDATION CURVE

ALEXANDRIA SCIENCE VILLAGE
9363, 9373 AND 9393 TOWNE CENTRE DRIVE
SAN DIEGO, CALIFORNIA

G2101-52-01.GPJ

Figure B-4

GEOCON

APPENDIX C

APPENDIX C

LOG OF PREVIOUS EXPLORATORY BORINGS AND TRENCHES
PERFORMED BY GEOCON INCORPORATED, 1985

FOR

ALEXANDRIA SCIENCE VILLAGE
9363, 9373, AND 9393 TOWNE CENTRE DRIVE
SAN DIEGO, CALIFORNIA

PROJECT NO. G2101-52-02

File No. D-3592-M01
January 20, 1986

BORING NO. 5

ELEVATION 395MSL DATE DRILLED 11/15/85

EQUIPMENT Mobile B-50 Drill Rig

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (U.S.C.S)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE BLOWS/FT.	DRY DENSITY P.C.F.	MOISTURE CONTENT, %
0					TOPSOIL Loose, wet, brown, silty SAND			
2					LINDAVISTA FORMATION			
4								
6	5-1				Dense, damp, red brown, silty SAND	50/3"	133.8	6.8
8								
10	5-2						111.6	7.6
12								
14					gravels			
16								
					BORING TERMINATED AT 17 FEET ON GRAVELS			

Figure A-4, Log of Boring No. 5

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

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

File No. D-3592-M01
January 20, 1986

BORING NO. 6

ELEVATION 402MSL DATE DRILLED 11/15/85

EQUIPMENT Mobile B-50 Drill Rig

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (U.S.C.S.)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE BLOWS/FT.	DRY DENSITY P.C.F.	MOISTURE CONTENT, %
0 – 2					TOPSOIL Loose, wet, brown, silty SAND			
2 – 12					LINDAVISTA FORMATION Dense, damp, orange red, silty fine SAND			
10	6-1					50/3.5"		
12 – 17					Dense to very dense, damp, orange brown, fine to medium SAND			
15	6-2					50/2.5"	122.0	7.5
17 – 20					gravels Dense, damp, red brown silty SAND with gravels			
20					BORING TERMINATED AT 20 FEET			

Figure A-5, Log of Boring No. 6

SAMPLE SYMBOLS
☐ — SAMPLING UNSUCCESSFUL
☒ — DISTURBED OR BAG SAMPLE
◧ — STANDARD PENETRATION TEST
◩ — CHUNK SAMPLE
■ — DRIVE SAMPLE (UNDISTURBED)
$\underline{\nabla}$ — 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.

File No. D-3592-M01
January 20, 1986

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (U.S.C.S.)	TRENCH NO. 1 ELEVATION ____ DATE DRILLED 12/5/85 EQUIPMENT Trackhoe	PENETRATION RESISTANCE BLOWS/FT.	DRY DENSITY P.C.F.	MOISTURE CONTENT, %
					MATERIAL DESCRIPTION			
0					TOPSOIL Very loose, wet brown medium gray silty SAND			
2								
4	T1-1	☒			SCRIPPS FORMATION Very dense, moist light brown massive medium gray SAND			
6								
8								
10								
12					TRENCH TERMINATED AT 12 FEET			
					TRENCH NO. 2			
0					ALLUVIUM Very loose, wet black clayey SAND			
2								
4					ALLUVIUM/SLOPEWASH Loose, medium dense moist/wet brown clayey SAND			
6								
8								
10					SCRIPPS FORMATION Very dense, damp light brown cemented SAND			

Figure A-10, Log of Test Trenches 1 and 2

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

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

File No. D-3592-M01
January 20, 1986

TRENCH NO. 3

ELEVATION ______ DATE DRILLED 12/5/85

EQUIPMENT Trackhoe

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (U.S.C.S)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE BLOWS/FT.	DRY DENSITY P.C.F.	MOISTURE CONTENT, %
0	3-1				ALLUVIUM Very loose, wet dark brown silty SAND			
4					ALLUVIUM/SLOPEWASH Loose, medium dense, moist brown clayey SAND			
8					SCRIPPS FORMATION			
10					TRENCH TERMINATED AT 10 FEET			

TRENCH NO. 4

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (U.S.C.S)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE BLOWS/FT.	DRY DENSITY P.C.F.	MOISTURE CONTENT, %
0					ALLUVIUM Very loose, wet, blackish brown silty SAND			
					heavy seepage, caving			
4					ALLUVIUM/COLLUVIUM Loose to medium dense, yellowish brown clayey SAND			
8					SCRIPPS FORMATION Very dense, moist, massive yellowish tan, weakley cemented SANDSTONE			
12					TRENCH TERMINATED AT 12 FEET			

Figure A-11, Log of Test Trenches 3 and 4

SAMPLE SYMBOLS

- ☐ — SAMPLING UNSUCCESSFUL
- ☒ — DISTURBED OR BAG SAMPLE
- STANDARD PENETRATION TEST
- CHUNK SAMPLE
- DRIVE SAMPLE (UNDISTURBED)
- WATER TABLE OR SEEPAGE

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

File No. D-3592-M01
January 20, 1986

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (U.S.C.S.)	TRENCH NO. 5 ELEVATION ______ DATE DRILLED 12/5/85 EQUIPMENT Trackhoe	PENETRATION RESISTANCE BLOWS/FT.	DRY DENSITY P.C.F.	MOISTURE CONTENT, %
					MATERIAL DESCRIPTION			
0–2					ALLUVIUM Very loose, wet, blackish brown silty SAND			
2–5					ALLUVIUM/COLLUVIUM Loose to medium dense, wet, yellowish brown clayey SAND with cobbles			
5–7								
7–					SCRIPPS FORMATION Very dense, moist, yellowish gray interbedded SANDSTONE/SILTSTONE			
					TRENCH TERMINATED AT 7 FEET			

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (U.S.C.S.)	TRENCH NO. 6	PENETRATION RESISTANCE BLOWS/FT.	DRY DENSITY P.C.F.	MOISTURE CONTENT, %
0–6					ALLUVIUM Very loose, wet, dark gray Silty SAND			
3			------		----seepage, caving			
6–7					SCRIPPS FORMATION Very dense, wet, yellow-gray, interbedded fine SANDSTONE/SILTSTONE			
					TRENCH TERMINATED AT 7 FEET			

Figure A-12, Log of Test Trenches 5 and 6

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

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

APPENDIX D

APPENDIX D

RECOMMENDED GRADING SPECIFICATIONS

FOR

ALEXANDRIA SCIENCE VILLAGE
9363, 9373, AND 9393 TOWNE CENTRE DRIVE
SAN DIEGO, CALIFORNIA

PROJECT NO. G2101-52-02

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

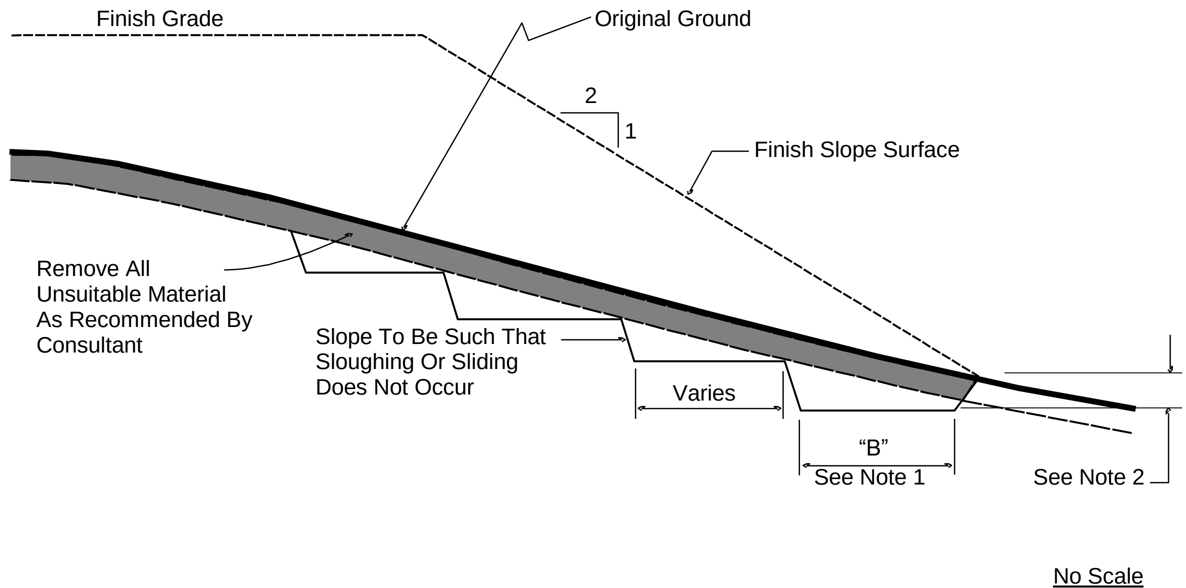
4.1 Areas to be excavated and filled shall be cleared and grubbed. Clearing shall consist of complete removal above the ground surface of trees, stumps, brush, vegetation, man-made structures, and similar debris. Grubbing shall consist of removal of stumps, roots, buried logs and other unsuitable material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

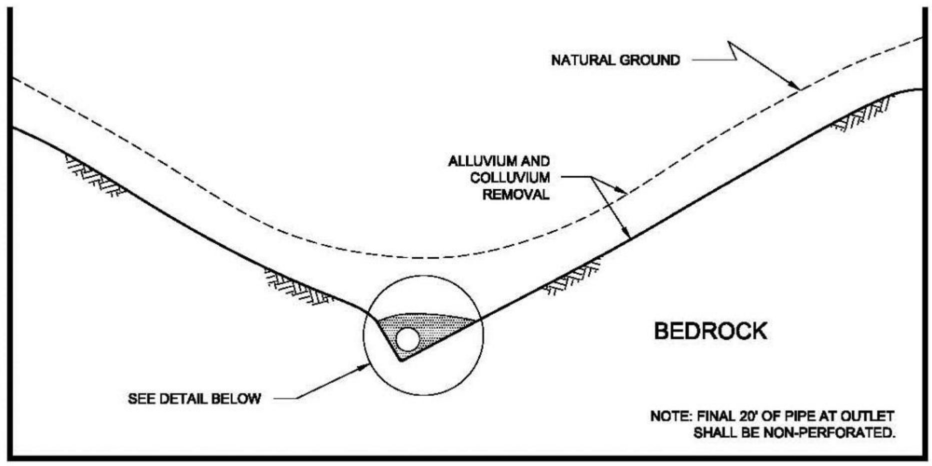
6.3.6 To reduce the potential for “piping” of fines into the *rock* fill from overlying *soil* fill material, a 2-foot layer of graded filter material shall be placed above the uppermost lift of *rock* fill. The need to place graded filter material below the *rock* should be determined by the Consultant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

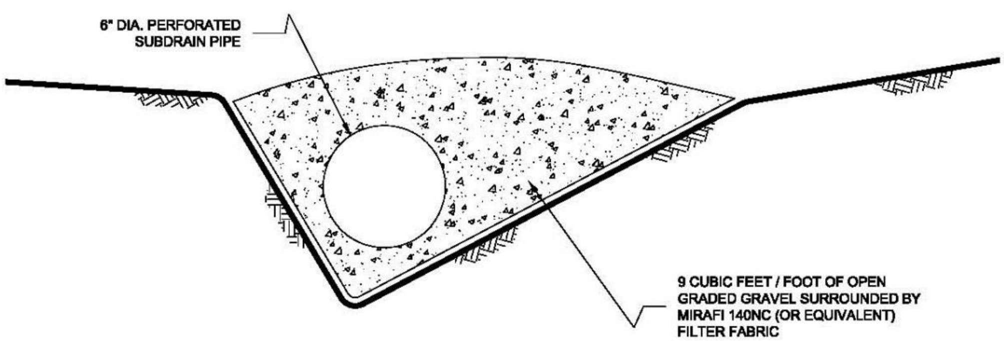
6.3.7 *Rock* fill placement should be continuously observed during placement by the Consultant.

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:

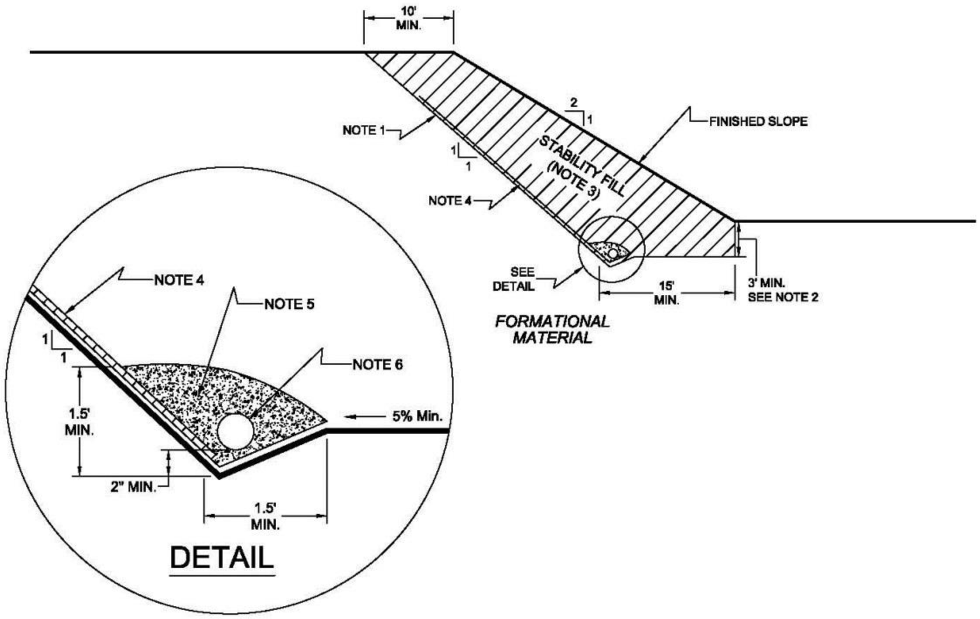
1......6-INCH DIAMETER, SCHEDULE 80 PVC PERFORATED PIPE FOR FILLS IN EXCESS OF 100-FEET IN DEPTH OR A PIPE LENGTH OF LONGER THAN 500 FEET.

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

NO SCALE

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

TYPICAL STABILITY FILL DETAIL



NOTES:

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

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

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

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

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

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

NO SCALE

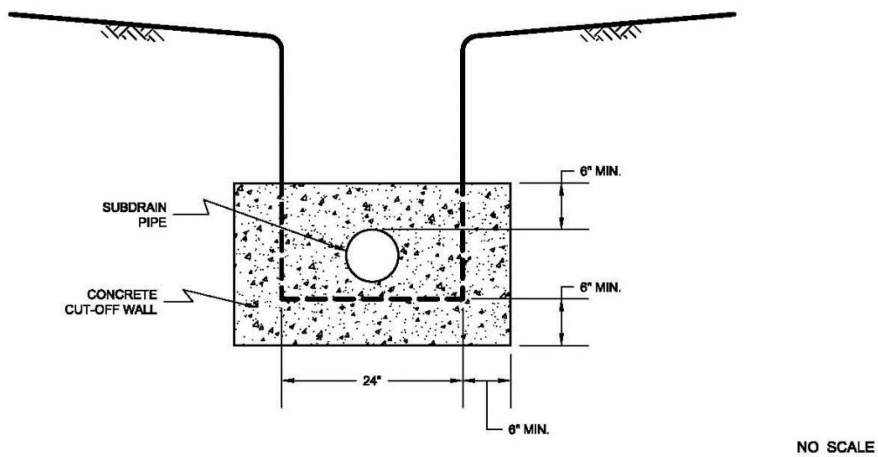
7.3 The actual subdrain locations will be evaluated in the field during the remedial grading operations. Additional drains may be necessary depending on the conditions observed and the requirements of the local regulatory agencies. Appropriate subdrain outlets should be evaluated prior to finalizing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

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

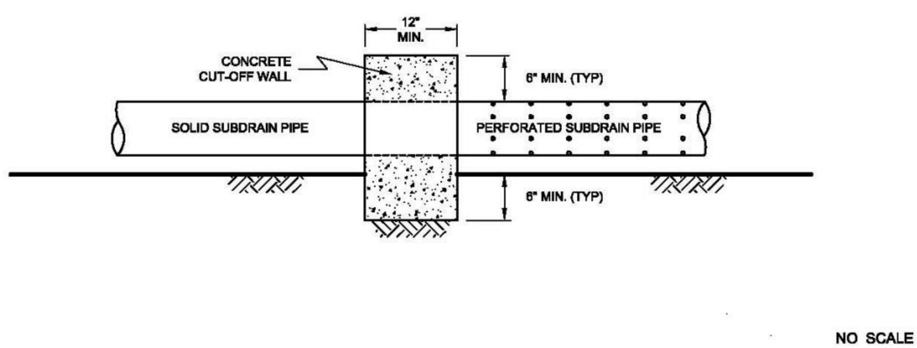
7.5 Prior to outletting, the final 20-foot segment of a subdrain that will not be extended during future development should consist of non-perforated drainpipe. At the non-perforated/ perforated interface, a seepage cutoff wall should be constructed on the downslope side of the pipe.

TYPICAL CUT OFF WALL DETAIL

FRONT VIEW

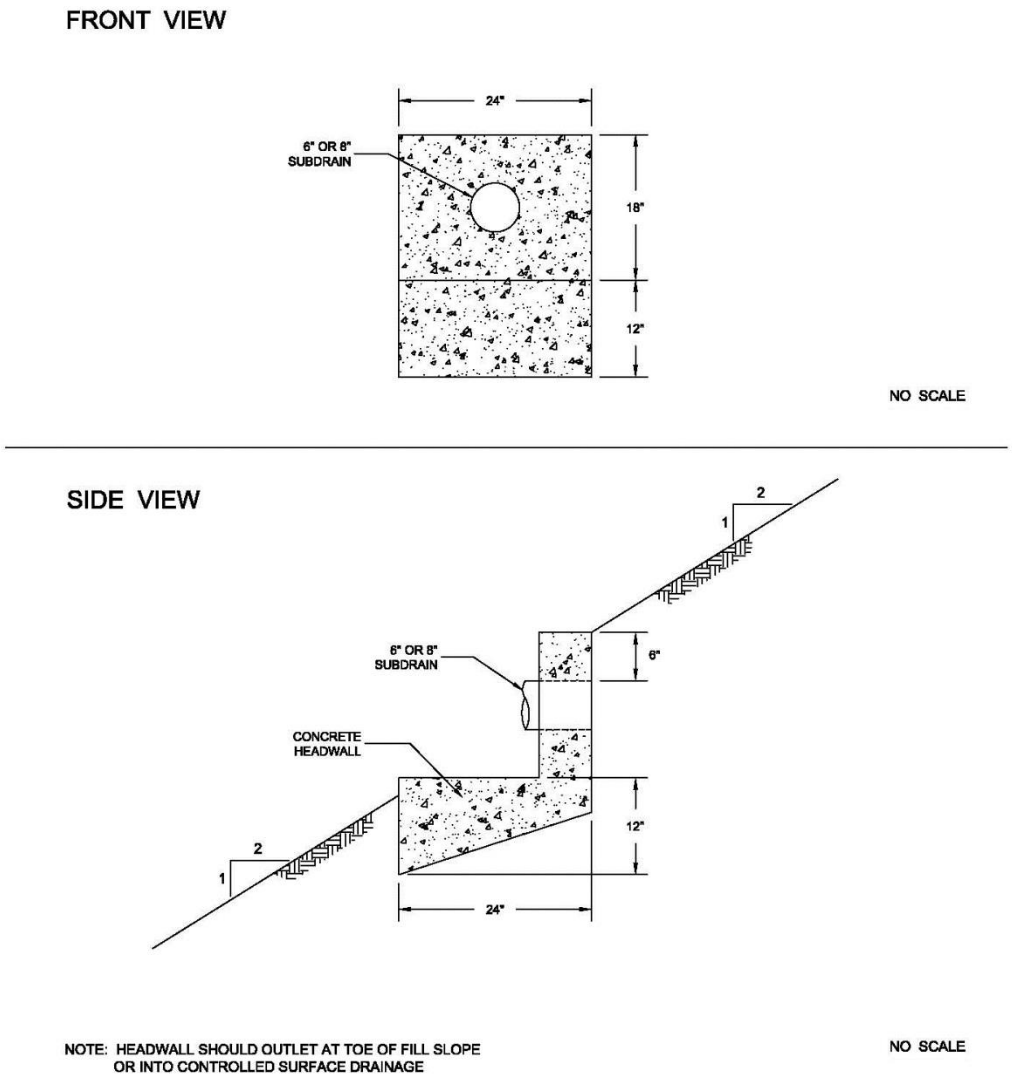


SIDE VIEW



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.

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

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4. American Society of Civil Engineers (ASCE), *ASCE 7-16, Minimum Design Loads and Associated Criteria for Buildings and Other Structures*, 2017.

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

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

7. *City of San Diego Seismic Safety Study, Geologic Hazards and Faults*, 2008 edition, Map Sheet 34.

8. County of San Diego, *San Diego County Multi Jurisdiction Hazard Mitigation Plan, San Diego, California – Final Draft*, dated 2017.

9. Geocon, 1985. Preliminary Soil and Geologic Investigation for Nexus Technology Center, San Diego, California, December 9 (Project No. D-3592-MO1).

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11. Geocon, 1999. *Geotechnical Report Update, Nexus Technology Center, Northeast Lot, San Diego, California*, June 2 (Project No. 06322-22-01).

12. Geocon, 2017. *Geotechnical Investigation, Podium 93, 9363, 9737, and 9393 Towne Centre Drive, San Diego, California,* prepared by Geocon Incorporated, dated July 11 (Project No. G2101-52-01).

13. 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.

14. Legg, M. R., J. C. Borrero, and C. E. Synolakis (2002), *Evaluation of Tsunami Risk to Southern California Coastal Cities*, 2002 NEHRP Professional Fellowship Report, dated January.

15. Miller Hull, 2021. International, 2021. *Schematic Design Package, Alexandria Science Village, 9363, 9373 & 9393 Towne Centre Drive, San Diego, California*, civil plans prepared by Michael Baker International, structural plans by DCI Engineers, dated September 7, (Miller Hull Project No. A19.0087.00).

16. SEAOC Web Application, OSHPD Seismic Design Maps, https://seismicmaps.org/

17. Special Publication 117A, Guidelines For Evaluating and Mitigating Seismic Hazards in *California 2008*, California Geological Survey, Revised and Re-adopted September 11, 2008.

18. United States Geological Survey, *7.5 Minute Quadrangle Series*, La Jolla, 1996.

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

20. USGS computer program, Seismic Hazard Curves and Uniform Hazard Response Spectra, http://earthquake.usgs.gov/research/hazmaps/design/.