GEOTECHNICAL INVESTIGATION

TORREYANA LIFE SCIENCE PROJECT
11011 TORREYANA ROAD
SAN DIEGO, CALIFORNIA

GEOCON
INCORPORATED

GEOTECHNICAL
ENVIRONMENTAL
MATERIALS

PREPARED FOR

ALLIANCE DIVERSIFIED HOLDINGS, LLC
MIAMI, FLORIDA

JULY 28, 2022
PROJECT NO. G2972-42-01

GEOCON
INCORPORATED
GEOTECHNICAL ■ ENVIRONMENTAL ■ MATERIALS

Project No. G2972-42-01
July 28, 2022

Alliance Diversified Holdings, LLC
382 NE 191st Street
PMB 55807
Miami, Florida 33179-3899

Attention: Mr. Evan Wilson

Subject: GEOTECHNICAL INVESTIGATION
TORREYANA LIFE SCIENCE PROJECT
11011 TORREYANA ROAD
SAN DIEGO, CALIFORNIA

Dear Mr. Wilson:

In accordance with your request and authorization of our Proposal No. LG-22238 dated May 25, 2022, we herein submit the results of our geotechnical investigation for the subject project. We performed our investigation to evaluate the underlying soil and geologic conditions and potential geologic hazards, and to assist in the design of the proposed building and associated improvements.

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

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

Very truly yours,

GEOCON INCORPORATED

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TABLE OF CONTENTS

TABLE OF CONTENTS (Concluded)

GEOTECHNICAL INVESTIGATION

1. PURPOSE AND SCOPE

This report contains the results of our geotechnical investigation for the proposed life science structure located at 11011 Torreyana Road in the Torrey Pines area of San Diego, California (see Vicinity Map).

Vicinity Map

The purpose of the geotechnical investigation is to evaluate the surface and subsurface soil conditions and general site geology, and to identify geotechnical constraints that may affect development of the property including faulting, liquefaction, and seismic shaking based on the 2019 CBC seismic design criteria. In addition, we are providing recommendations for remedial grading, temporary shoring, shallow foundations, concrete slab-on-grade, concrete flatwork, pavement, permeable pavers and retaining walls.

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

1. *Grading Plan, Torreyana Life Science, 11011 Torreyana, San Diego, CA*, prepared by Ware Malcomb, undated.
2. *Limited Geotechnical Investigation for IDEC Pharmaceuticals Corporation, 11011 Torreyana Road, San Diego, California*, prepared by Geocon Incorporated, dated December 17, 1992 (Project No. 01549-02-04A).

3. *In-Place Density Tests; Report of Services from February 28 through May 1, 1980, Jaycor Office Building, Torrey Pines Science Park, Unit 2, Lot 7, San Diego, California*, prepared by Geocon Incorporated, dated May 28, 1980 (Project No. D-1549-T02).

4. *Testing and Observation Services During Grading Operations; Final Report of Work from September 27 through October 8, 1979, Jaycor Office Building, Torrey Pines Science Park, Unit 2, Lot 7, San Diego, California*, prepared by Geocon Incorporated, dated November 16, 1979 (Project No. D-1549-T01).

5. *Soil Investigation for Jaycor Office Building, Torrey Pines Science Park, Unit 2, Lot 7, San Diego, California*, prepared by Geocon Incorporated, dated September 6, 1978 (Project No. D-1549-T01).

The scope of this investigation included reviewing readily available published and unpublished geologic literature (see List of References), performing engineering analyses, and preparing this report. We also advanced three small-diameter borings to a maximum depth of 71 feet, performed percolation/infiltration testing, collected soil samples, and performed laboratory testing. We also included geologic data from eight exploratory borings drilled to a maximum depth of 25 feet in 1992 for a previously proposed development at the site. Approximate exploratory boring locations for the current and previous investigations are shown on the Geologic Map, Figure 2. The base map used for Figure 2 was created using the grading plan provided by Ware Malcomb. Logs of our exploratory borings and a discussion of the field investigations are presented in Appendices A and C.

We performed laboratory tests on selected soil samples obtained during the field investigations to evaluate pertinent physical and chemical properties for engineering analyses and to assist in providing recommendations. Details of the laboratory testing and a summary of the test results are presented in Appendices B and C. Appendix D contains a summary of our storm water management investigation.

2. SITE AND PROJECT DESCRIPTION

The site is located northeast of the intersection of Callan Road and Torreyana Road in the Torrey Pines mesa area of San Diego, California. The site sits atop an east facing hillside slope. Compacted fill associated with previous site grading and Very Old Paralic Deposits underly the site.

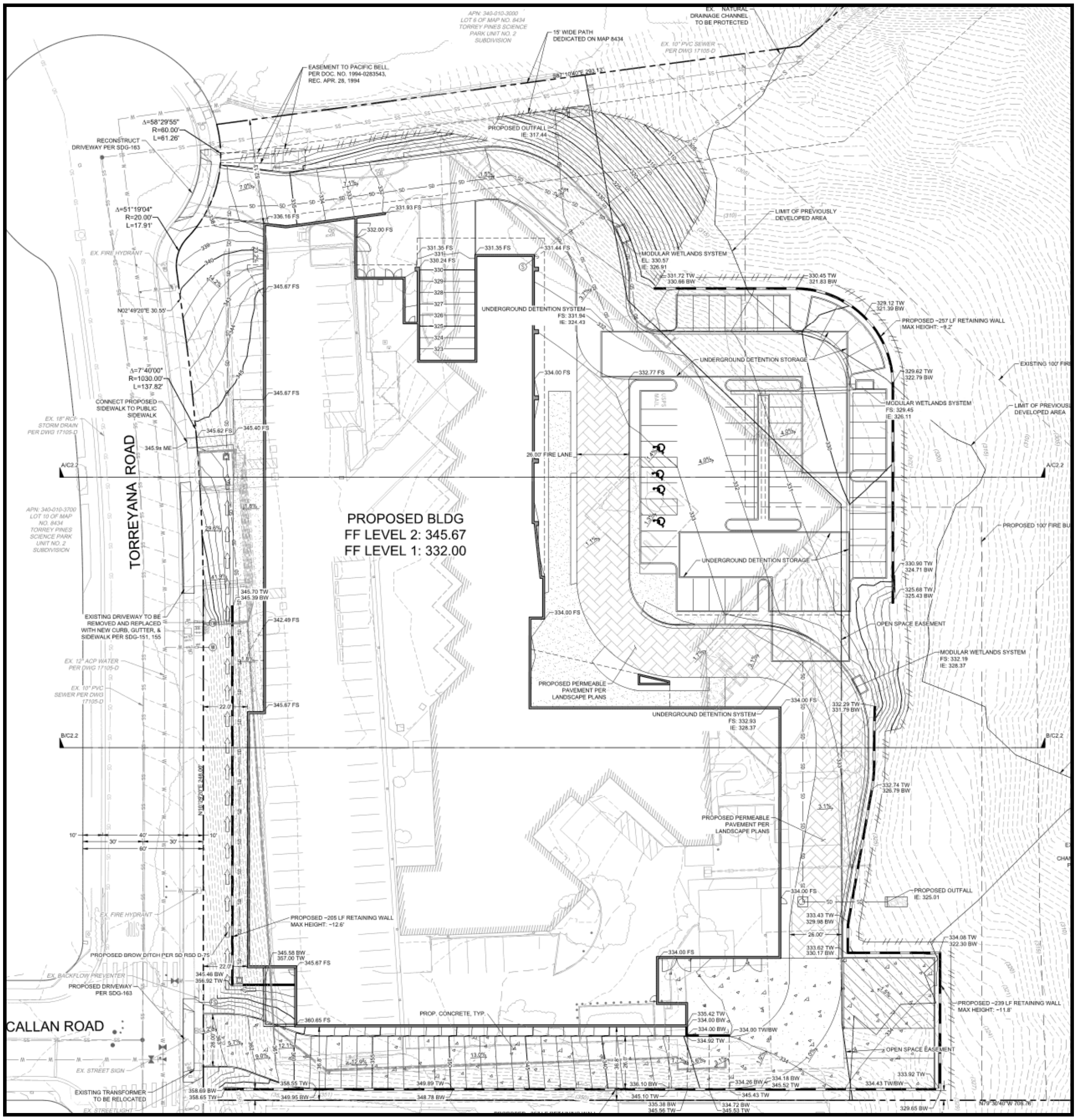
The site is presently occupied by a triangularly shaped, two-story building and a two-level concrete parking structure. A paved access driveway is located at the northwest corner of the property extending from Torreyana Road. A paved driveway is also present on the south side of the property extending from the lower level of the parking garage to mechanical equipment and enclosures.

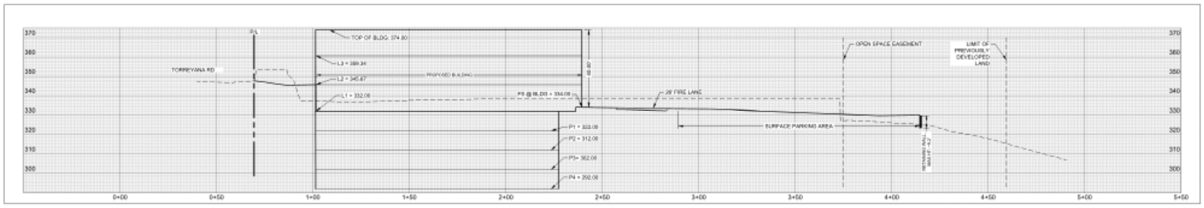
The site slopes from west to east with site elevations ranging from approximately 363 feet Mean Sea Level (MSL) at the southwest corner of the property adjacent to Torreyana Road to 321 feet on the slope face along the east side of the property at the edge of the proposed improvements. Manufactured slopes are present along the west and north sides of the property. A natural, descending slope approximately 250 feet in height extends from the east property boundary to properties adjacent to Flintkote Avenue. The Existing Site Plan below shows the current site conditions.

Existing Site Plan

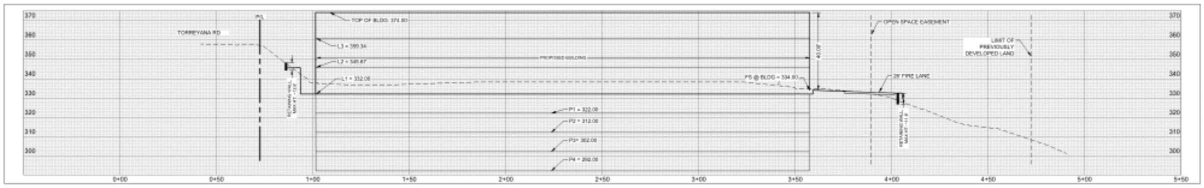
The referenced grading plan shows that site redevelopment consists of constructing a new three-story life science structure (two above grade and one basement level) over four levels of below grade parking. Exterior improvements include a surface parking lot, permeable pavers, and landscaping. Access to the property will be from a looping driveway that connects to Torreyana Road at the north and south sides of the property. We expect the driveway will be both Portland cement concrete, asphalt concrete, and permeable pavers. A portion of the existing slope (previously graded and native hillside area) at the northeast corner of the property will be graded to create room for the access driveway. Retaining walls up to approximately 13 feet tall are planned along the west, south, and east sides of the property. An underground storm water detention vault, modular wetland system, and an outfall are planned in the northeastern corner of the site for stormwater management.

Cuts up to approximately 50 feet are expected to reach the bottom parking level grade. Fills up to approximately 10 to 15 feet are also planned. A portion of the grading plan and proposed building sections taken from Ware Malcomb's referenced plans are shown below.


PROPOSED BLDG
FF LEVEL 2: 345.67
FF LEVEL 1: 332.00
TORREYANA ROAD
CALLAN ROAD
EASEMENT TO PACIFIC BELL, PER DOC. NO. 1994-0283543, REC. APR. 28, 1994
15' WIDE PATH DEDICATED ON MAP 8434
EX. NATURAL DRAINAGE CHANNEL TO BE PROTECTED
EX. 10" PVC SEWER PER DWG 17105-D
RECONSTRUCT DRIVEWAY PER SDG-163
PROPOSED OUTFALL IE: 317.44
LIMIT OF PREVIOUSLY DEVELOPED AREA
MODULAR WETLANDS SYSTEM
UNDERGROUND DETENTION SYSTEM
UNDERGROUND DETENTION STORAGE
PROPOSED ~257 LF RETAINING WALL MAX HEIGHT: ~9.2'
EXISTING 100' FIRE
PROPOSED 100' FIRE BU
26.00' FIRE LANE
CONNECT PROPOSED SIDEWALK TO PUBLIC SIDEWALK
EX. FIRE HYDRANT
EXISTING DRIVEWAY TO BE REMOVED AND REPLACED WITH NEW CURB, GUTTER, & SIDEWALK PER SDG-151, 155
EX. 12" ACP WATER PER DWG 17105-D
EX. 10" PVC SEWER PER DWG 17105-D
EX. 18" RCP STORM DRAIN PER DWG 17105-D
APN: 340-010-3700 LOT 10 OF MAP NO. 8434 TORREY PINES SCIENCE PARK UNIT NO. 2 SUBDIVISION
APN: 340-010-3000 LOT 6 OF MAP NO. 8434 TORREY PINES SCIENCE PARK UNIT NO. 2 SUBDIVISION
PROPOSED PERMEABLE PAVEMENT PER LANDSCAPE PLANS
OPEN SPACE EASEMENT
PROPOSED OUTFALL IE: 325.01
PROPOSED ~205 LF RETAINING WALL MAX HEIGHT: ~12.6'
PROPOSED ~239 LF RETAINING WALL MAX HEIGHT: ~11.8'
PROPOSED BROW DITCH PER SD RSD D-75
EX. BACKFLOW PREVENTER
PROPOSED DRIVEWAY PER SDG-163
EX. STREET SIGN
EXISTING TRANSFORMER TO BE RELOCATED
EX. STREETLIGHT
PROP. CONCRETE, TYP.
A/C2.2
B/C2.2



SECTION A-A
SCALE: 1" = 20'



SECTION B-B

Grading Site Plan and Building Sections

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

3. PREVIOUS GRADING AND GEOTECHNICAL REPORTS

Grading for Torreyana Road (performed circa 1976) resulted in the placement of fill along the west side of the site. Site grading operations for the existing structures and improvements were performed between 1979 and 1980. Site grading resulted in compacted fill under the eastern and northern portions of the site and Very Old Paralic Deposits exposed in the central portion of the site.

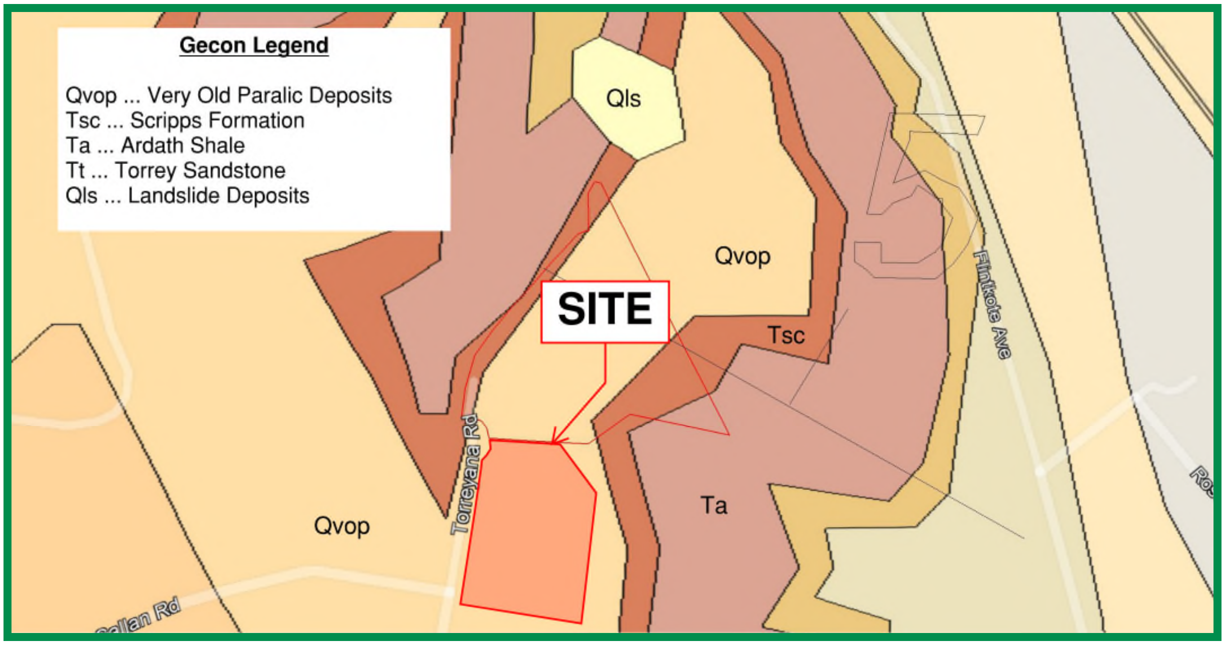
In 1992, after the construction of the structure, Geocon Incorporated performed a geotechnical investigation for building additions and improvements to the site. A report documenting the remedial grading for the additional improvements was not found during our record search; however, we expect grading for the improvements resulted in minor cuts and fills and the removal of undocumented landscape fill and replacement of compacted fill in improvement areas.

4. GEOLOGIC SETTING

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

Locally, the site is within the coastal plain of San Diego County. The coastal plain is underlain by a thick sequence of relatively undisturbed and non-conformable sedimentary bedrock units that thicken to the west and range in age from Upper Cretaceous age through the Pleistocene age which have been deposited on Cretaceous to Jurassic age igneous and volcanic bedrock. Geomorphically, the coastal plain is characterized by a series of 21, stair-stepped marine terraces (younger to the west) that have been dissected by west flowing rivers. The coastal plain is a relatively stable block that is dissected by relatively few faults consisting of the potentially active La Nacion Fault Zone and the active Rose Canyon Fault Zone.

Regional geologic maps show the site is underlain by Very Old Paralic Deposits, Scripps Formation and Ardath Shale. The Regional Geologic Map shows the geologic units in the area of the site.



Regional Geologic Map

5. SOIL AND GEOLOGIC CONDITIONS

We encountered previously placed fill, Very Old Paralic Deposits, Scripps Formation, and Ardath Shale in our exploratory borings. Approximate geologic contacts based on boring data and published geologic maps are shown on the Geologic Map, Figure 1. Boring logs are provided in Appendices A and C. The Geologic Cross-Sections, Figures 2 and 3, show the approximate subsurface relationship between the geologic units. We prepared the geologic cross-sections using interpolation between exploratory excavations and observations; therefore, actual geotechnical conditions may vary from those illustrated and should be considered approximate. The surficial soil and geologic units are described below.

5.1 Previously Placed Fill (Qpf_1)

We encountered previously placed fill (Qpf_1) to depths ranging from 1 to 16½ feet in current and previous borings. Geocon Incorporated provided compaction testing and observation services during the grading operations (see References 3 and 4). The fill was placed along the north and east portions of the site. In general, the previously placed fill consisted of medium dense, damp to moist, silty sand. Based on laboratory tests, the fill has a "very low" expansion index (expansion index [EI] of 20 or less). We expect the previously placed fill will be removed to reach planned basement grade within the proposed building pad area. Where the compacted fill present outside the new building footprint, the upper approximately 2 feet should be removed and recompacted at or above optimum moisture content to provide suitable support for new improvements.

5.2 Previously Placed Fill (Qpf_2)

The previously placed fill mapped along the west side of the site on Figure 1 (Qpf_2) was placed in 1976 under the observation of the City of San Diego during construction of Torreyana Road. We did not encounter the previously placed fill (Qpf_2) in our current or previous borings; however, based on a review of a geotechnical investigation performed prior to the mass grading of the site (Reference 5), we expect the fill extends to depths up to approximately 15 to 20 feet and consists of medium dense, clayey to silty sand, and sandy clay. We expect the previously placed fill (Qpf_2) in the planned building area will be removed to reach planned basement grade. Where the compacted fill present outside the new building footprint, the upper approximately 2 feet should be removed and recompacted at or above optimum moisture content to provide suitable support for new improvements.

5.3 Very Old Paralic Deposits (Qvop)

Quaternary-age Very Old Paralic Deposits (formerly called the Lindavista Formation) underlie the existing fill or are exposed at grade. Very Old Paralic Deposits were encountered to depths between 4 and 13 feet below existing grades in current Borings B-1 through B-3, and to the greatest depths explored ranging from 1½ to 25 feet in previous Borings B-1 through B-7. The Very Old Paralic Deposits generally consist of dense to very dense sandstone and generally possesses a "very low" to "low" expansive potential (expansion index of 50 or less). We expect the proposed building foundations will extend below the Very Old Paralic Deposits. Excavations within this unit may encounter difficult digging conditions in the cemented zones producing oversize material. In addition, coring and rock breaking equipment may be needed if very dense, cemented layers are encountered.

5.4 Scripps Formation (Tsc)

Eocene-age Scripps Formation was encountered below the previously placed fill or Very Old Paralic Deposits to depths ranging from 33 to 35½ feet below existing grades. The Scripps Formation generally consisted of very dense, yellowish brown to gray, silty sandstone and hard, sandy to clayey siltstone. The Scripps Formation can possess areas of highly cemented concretionary beds. The Scripps Formation typically possesses a “low to “medium” expansion potential (expansion index [EI] of 21 to 90) and can possess “S0” to “S2” water-soluble sulfate classifications. The Scripps Formation is considered suitable for support of structural loads.

5.5 Ardath Shale (Ta)

Tertiary-age Ardath Shale underlies the Scripps Formation and is exposed in most of the existing descending slopes to the east and northeast of the site below an elevation of approximately 300 to 305 feet MSL, or approximately 30 to 35 feet below the existing surface of the developed portion of the site. This unit is sometimes characterized by adverse bedding and slope instability. This geologic unit generally consists of hard, moist, gray, weakly indurated claystone. The Ardath Shale is considered suitable for support of the proposed new building. The Ardath Shale typically possesses a “low to “medium” expansion potential (expansion index [EI] of 21 to 90) and can possess “S0” to “S2” water-soluble sulfate classifications. We expect the building foundations at basement grade will extend into Ardath Shale.

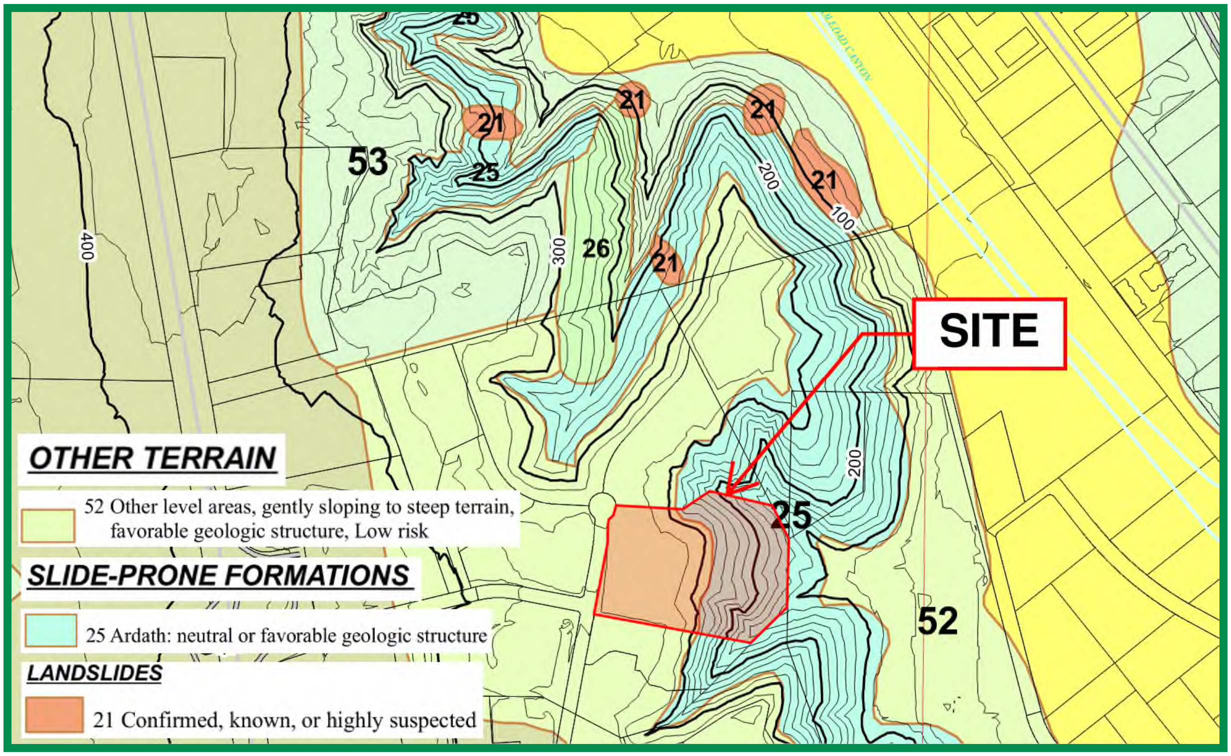
6. GROUNDWATER

We did not encounter groundwater or seepage during our site investigation and do not expect groundwater to be encountered during construction of the proposed development. We expect groundwater is deeper than 100 feet below existing grade. However, it is not uncommon for shallow seepage conditions to develop where none previously existed when sites are irrigated or infiltration is implemented. Seepage is dependent on seasonal precipitation, irrigation, land use, among other factors, and varies as a result. Proper surface drainage will be important to future performance of the project.

7. GEOLOGIC HAZARDS

7.1 Geologic Hazard Category

The City of San Diego Seismic Safety Study, Geologic Hazards and Faults, Sheet 34 defines the site with *Hazard Category 52: Other level areas, gently sloping to steep terrain, favorable geologic structure, Low Risk* and *Hazard Category 25: Ardath: Neutral or Favorable Geologic Structure* (as shown on the Hazard Category Map). Based on a review of the map, faults do not traverse the planned development area. We opine the existing geologic conditions are favorable for the planned development.



Hazard Category Map

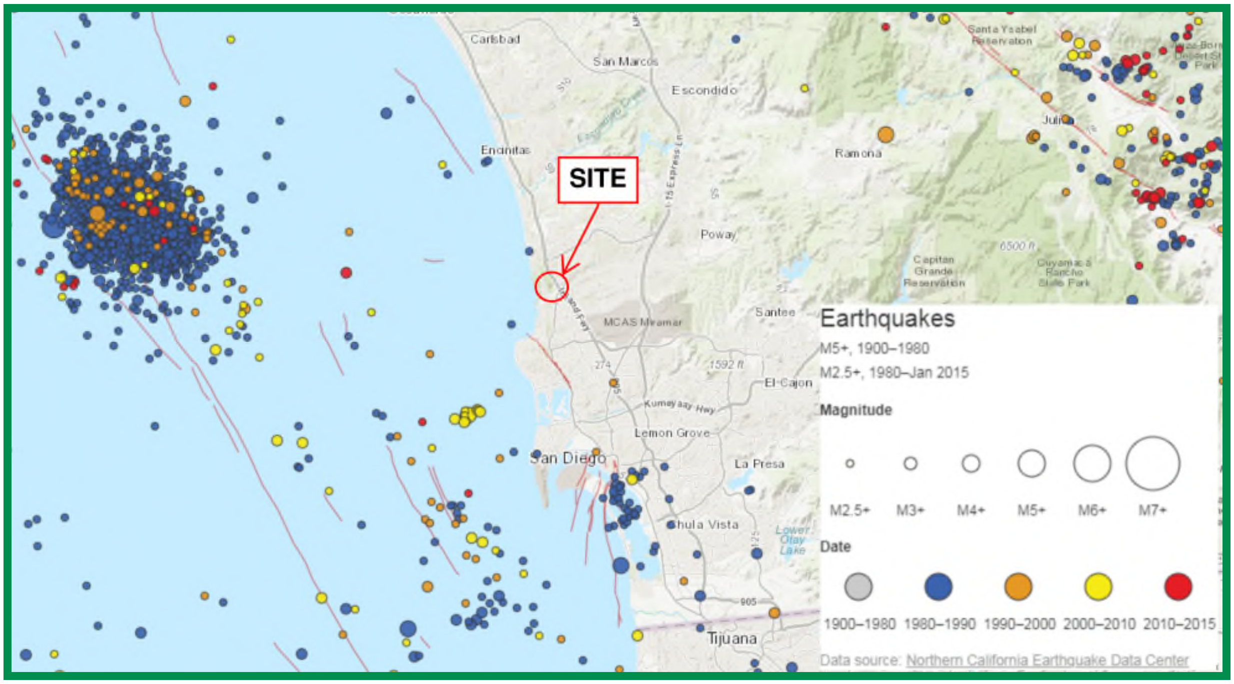
7.2 Faulting and Seismicity

Based on a review of the referenced geologic materials and our knowledge of the general area, 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.

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 fault 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.

7.3 Ground Rupture

The USGS (2016) and Kennedy & Tan (2008) shows that there are no mapped Quaternary faults crossing or trending toward the property. The site is not located within a currently established Alquist-Priolo Earthquake Fault Zone (CEG, 2021a). No active faults are known to exist at the site. The nearest active fault, the Newport-Inglewood-Rose Canyon Fault Zone, lies approximately 2.8 miles west of the site. The risk associated with ground rupture hazard is low.

7.4 Liquefaction

Due to the lack of a permanent, near-surface groundwater table and the very dense nature of the underlying Very Old Paralic Deposits, Scripps Formation and Ardath Shale, seismically induced soil liquefaction hazard is low.

7.5 Tsunamis and Seiches

The site is not mapped within a State of California tsunami hazard zone (CGS, 2021b). The site is not located near a large body of water. The risk associated with flooding due to tsunami or seiche hazard is low.

7.6 Landslides

We did not observe evidence of previous or recent slope instability at the site, or in the descending slopes adjacent to the site during our study. The risk associated with ground movement hazard due to land sliding is low.

7.7 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.

7.8 Flooding

The site is mapped as an "Area of Minimal Flood Hazard", Zone X (FEMA, 2020). The risk of inundation hazard due to flooding is low.

8. CONCLUSIONS AND RECOMMENDATIONS

8.1 General

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

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

8.1.3 The upper portion of the previously placed fill should be removed and replaced as compacted fill to reestablish proper moisture content and provide suitable fill for support of planned improvements. Very Old Paralic Deposits, Scripps Formation and Ardath Shale are suitable for the support of proposed fill and structural loads.

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

8.1.5 Excavation of the previously placed fill should generally be possible with moderate to heavy effort using conventional, heavy-duty equipment during grading and trenching operations. We expect very heavy effort for excavations into strongly cemented portions of the Very Old Paralic Deposits, Scripps Formation and Ardath Shale.

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

8.1.7 Based on the results of our field infiltration testing and laboratory testing, we opine full or partial infiltration on the property is infeasible as discussed in Appendix D.

8.1.8 Subsurface conditions observed may be extrapolated to reflect general soil/geologic conditions; however, some variation in subsurface conditions between boring locations should be anticipated.

8.1.9 We do not expect the planned development will destabilize or result in settlement of adjacent properties or city right-of-way, if properly constructed.

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

8.2 Excavation and Soil Characteristics

8.2.1 Excavation of the in-situ soil should be possible with moderate to heavy effort using conventional heavy-duty equipment. Excavation of the formational materials will require very heavy effort and may generate oversized material. Due to the sizeable excavation depth for the building, oversize material will need to be exported. The grading and improvement contractors should review this report and evaluate the proper equipment to use for the planned excavations.

8.2.2 The on-site soils are considered to be both "non-expansive" (EI of 20 or less) or "expansive" (EI greater than 20) as defined by 2019 California Building Code (CBC) Section 1803.5.3. We expect a majority of the soil encountered possess a "very low" to "medium" expansion potential (EI of 90 or less) in accordance with ASTM D 4829. Table 8.2.1 presents soil classifications based on the expansion index.

TABLE 8.2.1
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	

8.2.3 We performed laboratory tests on samples of the site materials to evaluate the percentage of water-soluble sulfate content. Appendix B presents results of the laboratory water-soluble sulfate content tests. The test results indicate the on-site materials at the locations tested possess "S0" sulfate exposure to concrete structures as defined by 2019 CBC Section 1904 and ACI 318-19 Chapter 19. However, the Scripps Formation and Ardath Shale can possess sulfate exposure class "S1" and "S2". We recommend concrete foundations and slab-on-grade that will be in contact with the Scripps Formation or Ardath Shale be designed assuming an "S2" exposure class. Samples of finish grade soils should be obtained and tested during grading. Table 8.2.2 presents a summary of concrete requirements set forth by

2019 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 8.2.2
REQUIREMENTS FOR CONCRETE EXPOSED TO SULFATE-CONTAINING SOLUTIONS

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

8.2.4 We tested samples for chloride content tests to aid in evaluating the corrosion potential to subsurface metal structures. Appendix B presents the laboratory test results.

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

8.3 Slope Stability

8.3.1 Based on the conceptual grading plan, a 2:1 (horizontal to vertical) fill slope up to approximately 25 feet high is planned at the northeast end of the site. Slopes less than 10 feet high and flatter than 2:1 will be constructed in other portions of the site. Natural hillside slopes up to approximately 250 feet high exist along the east perimeter of the site. Slope stability analyses for the proposed fill slope and natural slopes indicate a calculated factor of safety of at least 1.5 under static conditions for both deep-seated and surficial failure. Table 8.3.1 presents the slope stability analysis for the proposed fill slope and existing hillside conditions. It is our opinion that the site will be adequately stable following project completion.

TABLE 8.3.1
SLOPE STABILITY EVALUATION – PROPOSED FILL SLOPE AND EXISTING NATURAL SLOPES

Parameter	Value	
	Proposed Fill Slope	Existing Natural Slope
Slope Height, H	25 Feet	250 Feet
Slope Inclination, I (Horizontal to Vertical)	2:1	2:1
Total Soil Unit Weight, γ	125 pcf	130 pcf
Friction Angle, f	28 Degrees	36 Degrees
Cohesion, C	250 psf	400 psf
Slope Factor $\gamma_{Cf}= (\gamma H \tan f)/C$	6.6	59
N_{Cf} (From Chart)	23	125
Factor of Safety = $(N_{Cf}C)/(\gamma H)$	1.84	1.54

8.3.2 Table 8.3.2 presents the surficial slope stability analysis for the proposed and existing natural slopes.

TABLE 8.3.2
SURFICIAL SLOPE STABILITY EVALUATION – PROPOSED FILL SLOPE AND EXISTING NATURAL SLOPES

Parameter	Value	
	Proposed Fill Slope	Existing Natural Slope
Slope Height, H	∞	∞
Vertical Depth of Saturation, Z	5 Feet	5 Feet
Slope Inclination, I (Horizontal to Vertical)	2:1 (26.6 Degrees)	2:1 (26.6 Degrees)
Total Soil Unit Weight, γ	125 pcf	130 pcf
Water Unit Weight, γ_W	62.4 pcf	62.4 pcf
Friction Angle, f	28 Degrees	36 Degrees
Cohesion, C	250 psf	400 psf
Factor of Safety = $(C+(\gamma+\gamma_W)Z\cos^2 I \tan f)/(\gamma Z \sin I \cos I)$	1.53	2.29

8.3.3 We recommend that cut slopes be observed during grading by an engineering geologist to check that the soil and geologic conditions do not differ significantly from those anticipated and to check if adverse bedding, sheared claystones, fractures or joints exist. Remedial grading procedures, if needed, will be provided if adverse geologic conditions are observed.

8.3.4 Slopes should be landscaped with drought-tolerant vegetation having variable root depths and requiring minimal landscape irrigation. Slopes should also be properly maintained to reduce erosion.

8.4 Grading

8.4.1 Grading should be performed in accordance with the recommendations provided in this report, the Recommended Grading Specifications contained in Appendix E and the local grading ordinance. Geocon Incorporated should observe the grading operations on a full-time basis and provide testing during the fill placement.

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

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

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

8.4.5 We expect the excavation to reach the level (Level P4) for the proposed building will expose formational Ardath Shale at pad grade. Where native formational soils are exposed at basement grade, no additional removal below pad elevation will be required.

8.4.6 In areas where improvements are planned outside of proposed new building pad, the upper 2 feet of existing fill should be processed, moisture conditioned as necessary and recompacted prior to placing new fill or structural improvements. Deeper excavations may be required in areas where loose or saturated materials are encountered. The excavations should extend at least 2 feet laterally outside of the improvement area, where practicable. Where native formational soils are encountered, removals are not required. Table 8.4.1 provides a summary of the remedial grading recommendations.

TABLE 8.4.1
SUMMARY OF REMEDIAL GRADING RECOMMENDATIONS

Area	Remedial Grading Excavation Requirements
Building Pad	Footings to be founded on native formational bedrock
Site Development	In areas of previously placed fills, process upper 2 feet of existing soils
Lateral Grading Limits	5 Feet Outside of Buildings
	2 Feet Outside of Improvement Areas
Exposed Bottoms of Excavations	Scarify Upper 12 Inches

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

8.4.8 Import fill (if necessary) should consist of the characteristics presented in Table 8.4.2. 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 evaluate its suitability as fill material.

TABLE 8.4.2
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

8.5 Subdrains

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

8.6 Excavation Slopes, Shoring and Tiebacks

8.6.1 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.

8.6.2 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.

8.6.3 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 or other applicable techniques. Excavations exceeding 15 feet may require soil nails, tieback anchors or internal bracing to provide additional wall restraint.

8.6.4 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.

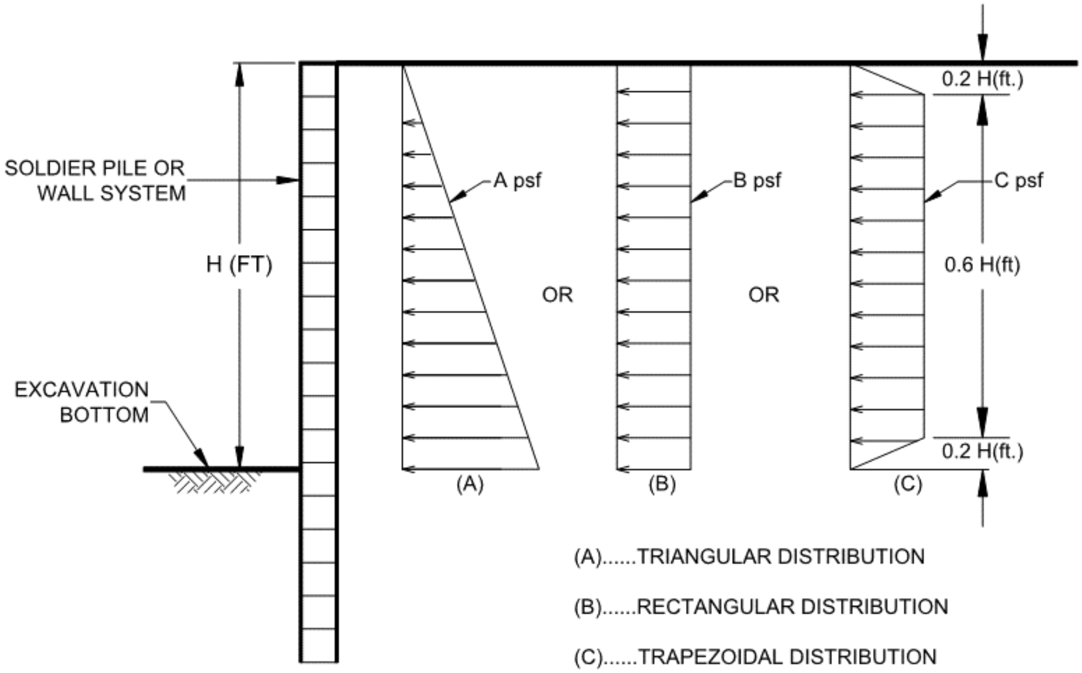
8.6.5 In general, ground conditions are moderately suited for soldier pile and tieback anchor wall construction techniques. However, gravel, cobble, and cemented material may be encountered in the existing bedrock soils that could be difficult to drill. Additionally, if cohesionless sands are encountered, some raveling may result along the unsupported portions of excavations.

8.6.6 Temporary shoring should be designed using a lateral pressure envelope acting on the back of the shoring as presented in Table 8.6.1 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 8.6.1
SUMMARY OF TEMPORARY SHORING WALL RECOMMENDATIONS

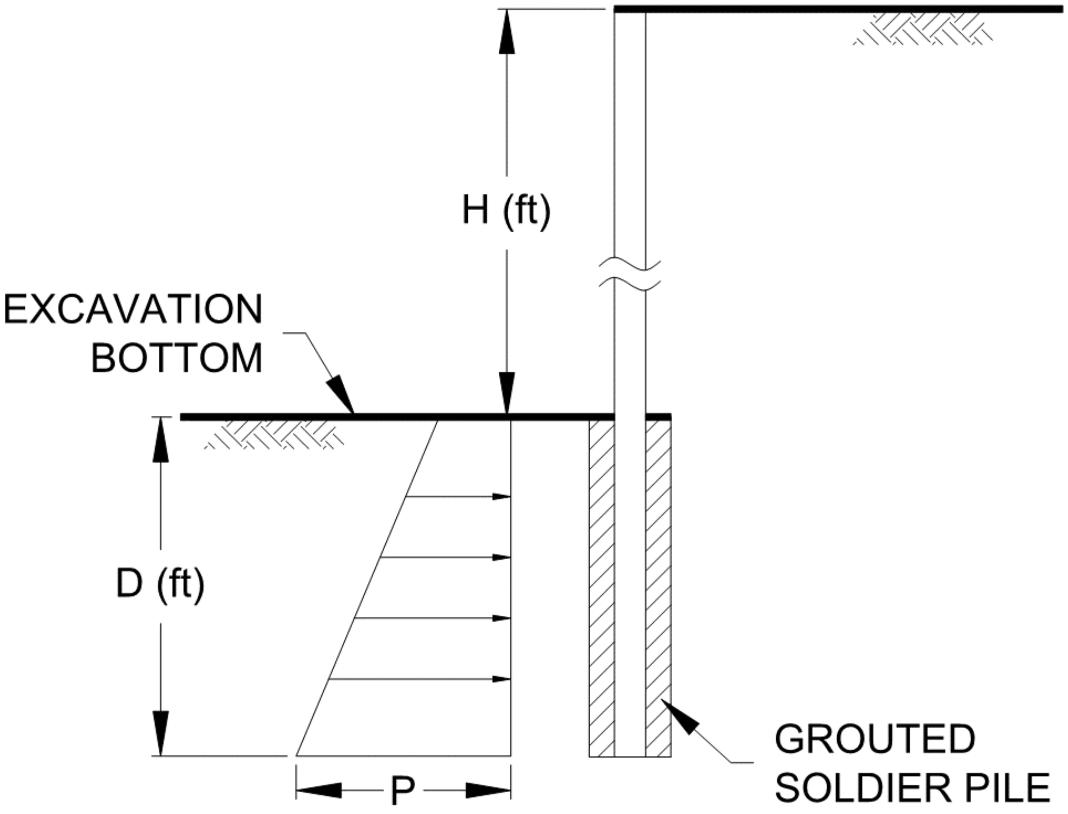
Parameter	Value
Triangular Distribution, A	29H psf
Rectangular Distribution, B	19H psf
Trapezoidal Distribution, C	24H psf
Passive Pressure, P	350D + 500 psf
Effective Zone Angle, E	28 degrees
Maximum Design Lateral Movement	1 Inch
Maximum Design Vertical Movement	½ Inch
Maximum Design Retained Height, H	50 Feet

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



Active Pressures on Temporary Shoring

8.6.7 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

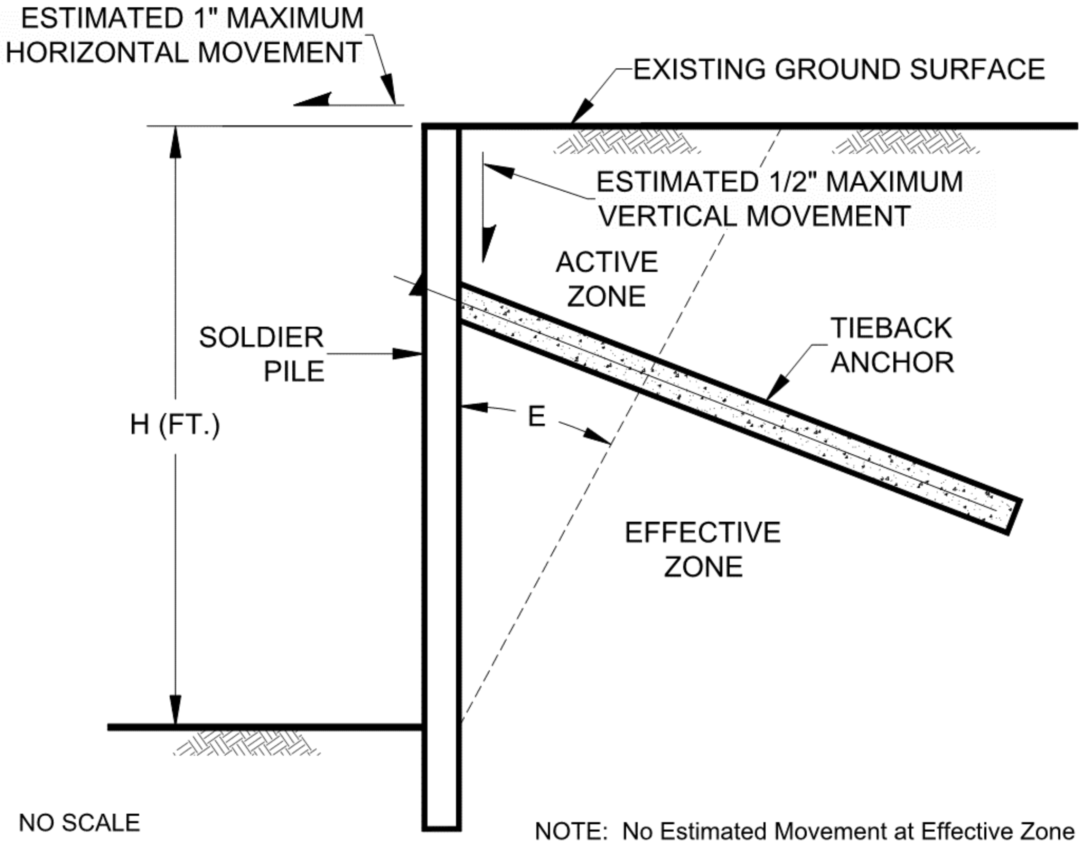
8.6.8 We should observe the drilled shafts for the soldier piles prior to the placement of steel reinforcement to check that the exposed soil conditions are similar to those expected and that footing excavations have been extended to the appropriate bearing strata and design depths. If unexpected soil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

8.6.9 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.

8.6.10 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.

8.6.11 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.

8.6.12 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

8.6.13 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.

8.6.14 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 8.6.2.

TABLE 8.6.2
SOIL STRENGTH PARAMETERS FOR TEMPORARY SHORING

Description	Cohesion (psf)	Friction Angle (Degrees)
Previously Placed Fill	250	28
Very Old Paralic Deposits/Scripps Formation/Ardath Shale	400	36

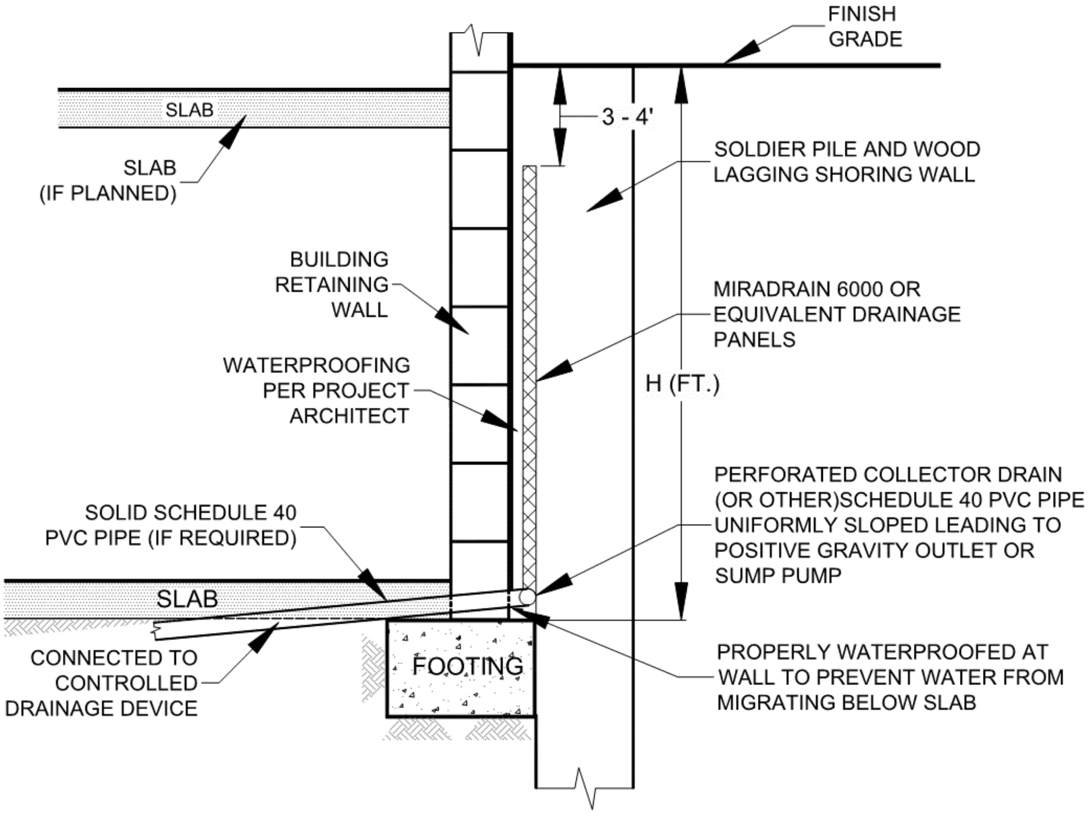
8.6.15 Grout should only be placed in the tieback anchor’s bonded section prior to testing. Tieback anchors should be proof-tested to at least 130 percent of the anchor’s design working load. Following a successful proof test, the tieback anchors should be locked off at 80 percent of the allowable working 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.

8.6.16 Lagging should keep pace with excavation. The excavation should not be advanced deeper than three feet below the bottom of lagging at any time or as determined by the shoring contractor. These unlagged gaps 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. Proper 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. It may be necessary to backfill with slurry to help prevent future lateral movement behind the supported excavation. 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. Surface sloughing may occur during the excavation process.

8.6.17 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.

8.6.18 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.

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



Shoring Retaining Wall Drainage Detail

8.7 Soil Nail Wall

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

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

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

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

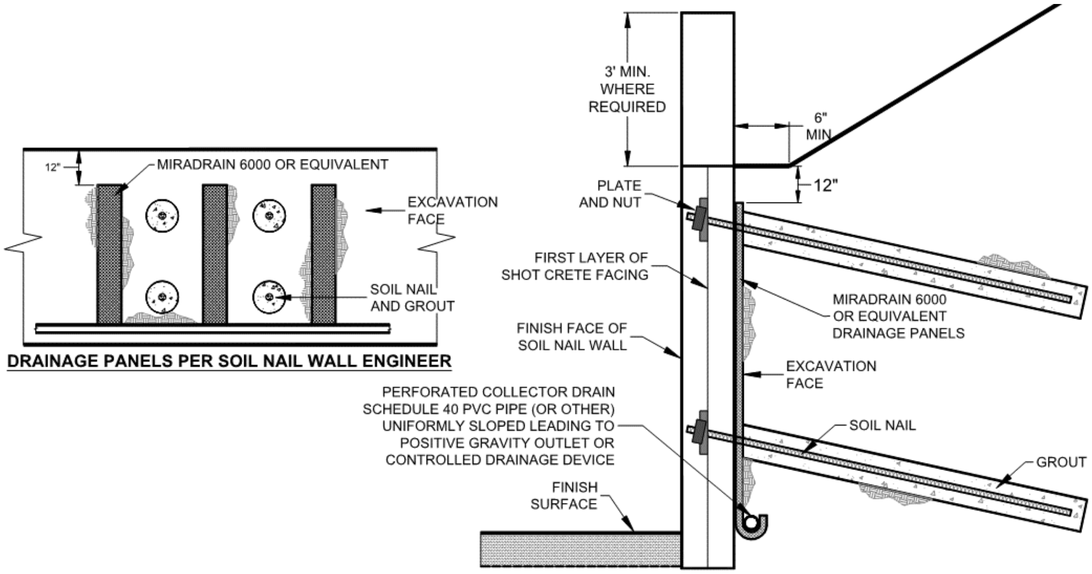
8.7.5 The soil strength parameters listed in Table 8.7.1 can be used in design of the soil nails. The bond stress is dependent on drilling method, diameter, and construction method. Therefore, the designer should evaluate the bond stress based on the existing soil conditions and the construction method.

TABLE 8.7.1
SOIL STRENGTH PARAMETERS FOR SOIL NAIL WALLS

Description	Soil Density (pcf)	Cohesion (psf)	Friction Angle (Degrees)	Estimated Ultimate Bond Stress (psi)*
Previously Placed Fill	125	250	28	10
Very Old Paralic Deposits/Scripps Formation/Ardath Shale	130	400	36	20

*Assuming gravity fed, open hole drilling techniques.

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



Soil Nail Wall Drainage Detail

8.8 Seismic Design Criteria – 2019 California Building Code

8.8.1 Table 8.8.1 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. We evaluated the Site Class based on the discussion in Section 1613.2.2 of the 2019 CBC and Table 20.3-1 of ASCE 7-16. The values presented herein are for the risk-targeted maximum considered earthquake (MCE_R). Sites designated as Site Class D, E and F may require additional analyses if requested by the project structural engineer and client.

TABLE 8.8.1
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	1.215g	Figure 1613.2.1(1)
MCE_R Ground Motion Spectral Response Acceleration – Class B (1 sec), S_1	0.429g	Figure 1613.2.1(2)
Site Coefficient, F_A	1.2	Table 1613.2.3(1)
Site Coefficient, F_V	1.5	Table 1613.2.3(2)
Site Class Modified MCE_R Spectral Response Acceleration (short), S_{MS}	1.457g	Section 1613.2.3 (Eqn 16-36)
Site Class Modified MCE_R Spectral Response Acceleration – (1 sec), S_{M1}	0.643g	Section 1613.2.3 (Eqn 16-37)
5% Damped Design Spectral Response Acceleration (short), S_{DS}	0.972g	Section 1613.2.4 (Eqn 16-38)
5% Damped Design Spectral Response Acceleration (1 sec), S_{D1}	0.429g	Section 1613.2.4 (Eqn 16-39)

8.8.2 Using the code-based values presented in this Table 8.8.1, in lieu of a performing a ground motion hazard analysis, requires the exceptions outlined in ASCE 7-16 Section 11.4.8 be followed by the project structural engineer. Per Section 11.4.8 of ASCE/SEI 7-16, a ground motion hazard analysis should be performed for projects for Site Class “E” sites with Ss greater than or equal to 1.0g and for Site Class “D” and “E” sites with S1 greater than 0.2g. Section 11.4.8 also provides exceptions which indicates that the ground motion hazard analysis may be waived provided the exceptions are followed.

8.8.3 Table 8.8.2 presents the mapped maximum considered geometric mean (MCE_G) seismic design parameters for projects located in Seismic Design Categories of D through F in accordance with ASCE 7-16.

TABLE 8.8.2
ASCE 7-16 PEAK GROUND ACCELERATION

Parameter	Value	ASCE 7-16 Reference
Mapped MCE_G Peak Ground Acceleration, PGA	0.548g	Figure 22-9
Site Coefficient, F_{PGA}	1.200	Table 11.8-1
Site Class Modified MCE_G Peak Ground Acceleration, PGA_M	0.657g	Section 11.8.3 (Eqn 11.8-1)

8.8.4 Conformance to the criteria in Tables 8.8.1 and 8.8.2 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.

8.8.5 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 8.8.3 presents a summary of the risk categories in accordance with ASCE 7-16.

TABLE 8.8.3
ASCE 7-16 RISK CATEGORIES

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

8.9 Shallow Foundations

8.9.1 The proposed structures can be supported on a shallow foundation system founded in the formational bedrock soils. Foundations for the structures should consist of continuous strip footings and/or isolated spread footings. Table 8.9.1 provides a summary of the foundation design recommendations. These values are based on at least 30 feet of soil being removed to achieve pad grade for the lowest basement level of the building. Table 8.9.2 provides foundation recommendations for footings supported in native Very Old Paralic Deposits or formational bedrock near existing site grades. Table 8.9.3 is for ancillary structures.

TABLE 8.9.1
SUMMARY OF FOUNDATION RECOMMENDATIONS FOR FOOTINGS AT DEPTHS GREATER THAN 30 FEET BELOW EXISTING GRADE

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	10,000 psf
Bearing Capacity Increase	500 psf per Foot of Depth
	300 psf per Foot of Width
Maximum Allowable Bearing Capacity	14,000 psf
Estimated Total Settlement	1 Inch
Estimated Differential Settlement	½ Inch in 40 Feet
Footing Size Used for Settlement	9-Foot Square
Design Expansion Index	90 or less

TABLE 8.9.2
SUMMARY OF FOUNDATION RECOMMENDATIONS FOR FOOTINGS AT DEPTHS NEAR EXISTING GRADE IN NATIVE FORMATIONAL SOILS

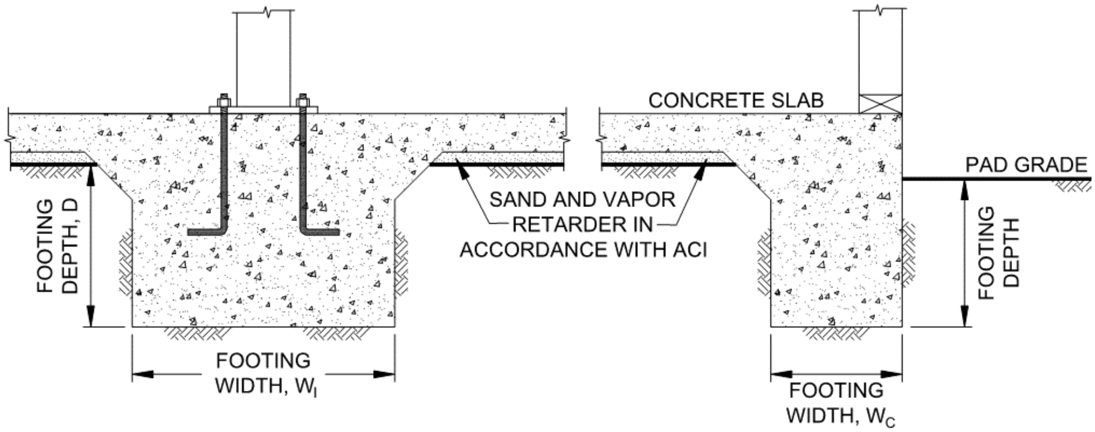
Parameter	Value
Minimum Continuous Foundation Width, W_C	12 inches
Minimum Isolated Foundation Width, W_I	24 inches
Minimum Foundation Depth, D	24 Inches Below Lowest Adjacent Grade
Minimum Steel Reinforcement	4 No. 5 Bars, 2 at the Top and 2 at the Bottom
Allowable Bearing Capacity	4,000 psf
Bearing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

8.9.2 Table 8.9.3 provides a summary of the foundation design recommendations for the ancillary structures embedded in properly compacted fill or formational materials. We expect mechanical equipment/enclosure yards and trash enclosures (if proposed) will be supported at or near existing grade.

TABLE 8.9.3
SUMMARY OF FOUNDATION RECOMMENDATIONS – ANCILLARY STRUCTURES

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, 2 at the Bottom
Allowable Bearing Capacity	2,500 psf
Bearing Capacity Increase	500 psf per Foot of Depth
	300 psf per Foot of Width
Maximum Allowable Bearing Capacity	4,000 psf
Estimated Total Settlement	1 Inch
Estimated Differential Settlement	½ Inch in 40 Feet
Footing Size Used for Settlement	7-Foot Square
Design Expansion Index	90 or less

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



Wall/Column Footing Dimension Detail

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

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

- For fill slopes less than 20 feet high, building footings should be deepened such that the bottom outside edge of the footing is at least 7 feet horizontally from the face of the slope.
- When located next to a descending 3:1 (horizontal:vertical) fill slope or steeper, the foundations should be extended to a depth where the minimum horizontal distance is equal to H/3 (where H equals the vertical distance from the top of the fill slope to the base of the fill soil) with a minimum of 7 feet but need not exceed 40 feet. The horizontal distance is measured from the outer, deepest edge of the footing to the face of the slope. A post-tensioned slab and foundation system or mat foundation system can be used to help reduce potential foundation distress associated with slope creep and lateral fill extension. Specific design parameters or recommendations for either of these alternatives can be provided, if needed.
- Although other improvements, which are relatively rigid or brittle, such as concrete flatwork or masonry walls, may experience some distress if located near the top of a slope, it is generally not economical to mitigate this potential. It may be possible, however, to incorporate design measures that would permit some lateral soil movement without causing extensive distress. Geocon Incorporated should be consulted for specific recommendations.

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

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

8.10 Concrete Slabs-On-Grade

8.10.1 Concrete slabs-on-grade for the structure should be constructed in accordance with Table 8.10.

TABLE 8.10
MINIMUM CONCRETE SLAB-ON-GRADE RECOMMENDATIONS

Parameter	Value
Minimum Concrete Slab Thickness	5 inches
Minimum Steel Reinforcement	No. 3 Bars 24 Inches on Center, Both Directions
Typical Slab Underlayment	3 to 4 Inches of Sand/Gravel/Base
Design Expansion Index	90 or less

8.10.2 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.

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

8.10.4 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.

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

8.10.6 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.

8.10.7 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.

8.11 Exterior Concrete Flatwork

8.11.1 Exterior concrete flatwork not subject to vehicular traffic should be constructed in accordance with the recommendations presented in Table 8.11. The recommended steel reinforcement would help reduce the potential for cracking.

TABLE 8.11
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.

8.11.2 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.

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

8.11.4 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.

8.11.5 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.

8.11.6 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.

8.12 Retaining Walls

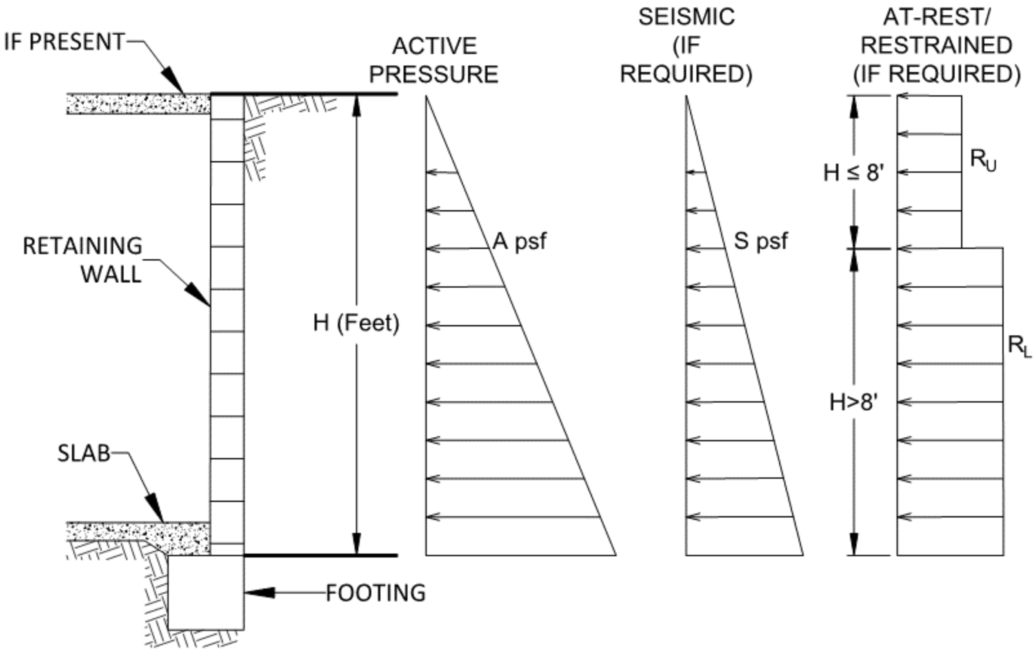
8.12.1 Retaining walls should be designed using the values presented in Table 8.12.1. Soil with an expansion index (EI) of greater than 90 should not be used as backfill material behind retaining walls.

TABLE 8.12.1
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	15H psf
At-Rest/Restrained Walls Additional Uniform Pressure (0 to 8 Feet High)	7H psf
At-Rest/Restrained Walls Additional Uniform Pressure (8+ Feet High)	13H psf
Expected Expansion Index for the Subject Property	EI≤ 90

H equals the height of the retaining portion of the wall

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



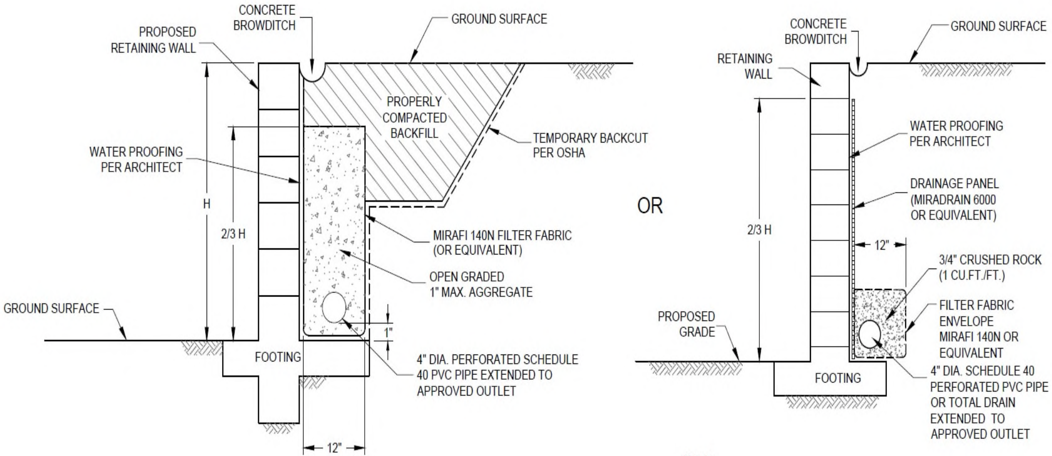
Retaining Wall Loading Diagram

8.12.3 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 to the upper 10 feet of the retaining wall.

8.12.4 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 7-16. 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.

8.12.5 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.

8.12.6 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.



NOTE :

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

Typical Retaining Wall Drainage Detail

8.12.7 In general, wall foundations should be designed in accordance with Table 8.12.2. The proximity of the foundation to the top of a slope steeper than 3:1 could impact the allowable soil bearing pressure. Therefore, retaining wall foundations should be deepened such that the bottom outside edge of the footing is at least 7 feet horizontally from the face of the slope.

TABLE 8.12.2
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	2,500 psf
Bearing Capacity Increase	500 psf per Foot of Depth
	300 psf per Foot of Width
Maximum Allowable Bearing Capacity	4,000 psf
Estimated Total Settlement	1 Inch
Estimated Differential Settlement	½ Inch in 40 Feet

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

8.12.9 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.

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

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

8.13 Lateral Loading

8.13.1 Table 8.13 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 8.13
SUMMARY OF LATERAL LOAD DESIGN RECOMMENDATIONS

Parameter	Value
Passive Pressure Fluid Density	350 pcf
Passive Pressure Fluid Density Adjacent to and/or on Descending Slopes	150 pcf
Coefficient of Friction (Concrete and Soil)	0.35
Coefficient of Friction (Along Vapor Barrier)	0.2 to 0.25*

*Per manufacturer's recommendations.

8.13.2 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.

8.14 Preliminary Pavement Recommendations

8.14.1 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 for the parking lot should be based on the R-Value of the subgrade soil encountered at final subgrade elevation. We have used an assumed R-Value of 15 for the subgrade soil and 78 for base materials. Table 8.14.1 presents the preliminary flexible pavement sections.

TABLE 8.14.1
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	15	3	8
Driveways for Automobiles and Light-Duty Vehicles	5.5	15	3	10
Medium Truck Traffic Areas	6.0	15	3.5	11
Driveways for Heavy Truck Traffic	7.0	15	4	13

8.14.2 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 material should be compacted to a dry density of at least 95 percent of the laboratory maximum dry density near to slightly above optimum moisture content. Asphalt concrete should be compacted to a density of at least 95 percent of the laboratory Hveem density in accordance with ASTM D 2726.

8.14.3 Base materials should conform to Section 26-1.02B of the *Standard Specifications for The State of California Department of Transportation (Caltrans)* with a ¾-inch maximum size aggregate. Asphalt concrete should conform to Section 203-6 of the *Standard Specifications for Public Works Construction (Greenbook).*

8.14.4 A rigid Portland cement concrete (PCC) pavement section should be placed in roadway aprons and cross gutters. We calculated the rigid pavement section in general conformance with the procedure recommended by the American Concrete Institute report ACI 330-21 *Commercial Concrete Parking Lots and Site Paving Design and Construction – Guide.* Table 8.14.2 provides the traffic categories and design parameters used for the calculations for 20-year design life.

TABLE 8.14.2
TRAFFIC CATEGORIES

Traffic Category	Description	Reliability (%)	Slabs Cracked at End of Design Life (%)
A	Car Parking Areas and Access Lanes	60	15
B	Entrance and Truck Service Lanes	60	15
C	School or City Buses (Excluding Large Articulated Buses)	75	15
D	Heavy Duty Trucks (Gross Weight of 80 Kips)	75	15
E	Garbage or Fire Truck Lane	75	15

8.14.5 We used the parameters presented in Table 8.14.3 to calculate the pavement design sections. We should be contacted to provide updated design sections, if necessary.

TABLE 8.14.3
RIGID PAVEMENT DESIGN PARAMETERS

Design Parameter	Design Value
Modulus of Subgrade Reaction, k	100 pci
Modulus of Rupture for Concrete, M_R	500 psi
Concrete Compressive Strength	3,000 psi
Concrete Modulus of Elasticity, E	3,150,000 psi

8.14.6 Based on the criteria presented herein, the PCC pavement sections should have a minimum thickness as presented in Table 8.14.4.

TABLE 8.14.4
RIGID VEHICULAR PAVEMENT RECOMMENDATIONS

Traffic Category	Trucks Per Day	Portland Cement Concrete, T (Inches)
A = Car Parking Areas and Access Lanes	10	5½
B = Entrance and Truck Service Lanes	10	6
	50	6½
	100	6½
C = School or City Buses	50	9½
	100	9½
D = Heavy Duty Trucks	50	6½
	100	7
E = Garbage or Fire Truck Lanes	5	6½
	10	7

8.14.7 The PCC vehicular pavement should be placed over subgrade soil that is compacted to a dry density of at least 95 percent of the laboratory maximum dry density near to slightly above optimum moisture content. The garbage truck pad should be large enough such that all wheels are on the concrete pad during the loading operations.

8.14.8 Adequate joint spacing should be incorporated into the design and construction of the rigid pavement in accordance with Table 8.14.5.

TABLE 8.14.5
MAXIMUM JOINT SPACING

Pavement Thickness, T (Inches)	Maximum Joint Spacing (Feet)
4<T<5	10
5≤T<6	12.5
6≤T	15

8.14.9 The rigid pavement should also be designed and constructed incorporating the parameters presented in Table 8.14.6.

TABLE 8.14.6
ADDITIONAL RIGID PAVEMENT RECOMMENDATIONS

Subject	Value
Thickened Edge	1.2 Times Slab Thickness Adjacent to Structures
	1.5 Times Slab Thickness Adjacent to Soil
	Minimum Increase of 2 Inches
	4 Feet Wide
Crack Control Joint Depth	Early Entry Sawn = T/6 to T/5, 1.25 Inch Minimum
	Conventional (Tooled or Conventional Sawing) = T/4 to T/3
Crack Control Joint Width	¼-Inch for Sealed Joints and Per Sealer Manufacturer's Recommendations
	$^{1}/_{16}$- to $^{1}/_{4}$-Inch is Common for Unsealed Joints

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

8.14.11 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 in accordance with the referenced ACI guide.

8.14.12 To provide load transfer between adjacent pavement slab sections, a butt-type construction joint should be constructed. The butt-type joint should be thickened by at least 20 percent at the edge and taper back at least 4 feet from the face of the slab.

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

8.15 Preliminary Concrete Paver Recommendations

8.15.1 We calculated the concrete paver section in general conformance with the *Caltrans Method of Flexible Pavement Design* (Highway Design Manual, Section 608.4) using an estimated Traffic Index (TI) of 5.0, 5.5, 6.0, and 7.0 for parking stalls, driveways, medium truck traffic areas, and heavy truck traffic areas, respectively. Based on the Interlocking Concrete Pavement Institute (ICPI), the pavers should possess a minimum thickness of 3⅛ inches overlying 1 to 1½ inch of sand. We used an equivalent asphalt concrete section equal to the thickness of the pavers of approximately 3 inches in accordance with Interlocking Concrete Pavement Institute (ICPI) *Tech Spec Number 4*. In addition, the pavers should be installed in a pattern appropriate for vehicular traffic. Tables 8.15.1 and 8.15.2 present two options for the paver underlayment: 1) compacted base materials or 2) aggregate materials.

TABLE 8.15.1
CONCRETE PAVER PAVEMENT SECTION USING AGGREGATE BASE

Location	Traffic Index (TI)	Assumed Subgrade R-Value	Minimum Concrete Paver Thickness (inches)	Paver Underlayment	
				Sand Thickness (inches)	Class 2 Aggregate Base (inches)
Parking stalls for automobiles and light-duty vehicles	5.0	15	3	1 -1½	8
Driveways for automobiles and light-duty vehicles	5.5	15	3	1 -1½	10
Medium truck traffic areas	6.0	15	3	1 -1½	12
Driveways for heavy truck traffic	7.0	15	3	1 -1½	15

TABLE 8.15.2
CONCRETE PAVER PAVEMENT SECTION USING AGGREGATE MATERIALS

Location	Traffic Index (TI)	Assumed Subgrade R-Value	Minimum Concrete Paver Thickness (inches)	Paver Underlayment
				ASTM C 33 Aggregate
Parking stalls for automobiles and light-duty vehicles	5.0	15	3	3-inch Sand/ 3-inch #8 stone/ 8-inch #57 stone
Driveways for automobiles and light-duty vehicles	5.5	15	3	3-inch Sand / 3-inch #8 stone/ 11-inch #57 stone
Medium truck traffic areas	6.0	15	3	3-inch Sand / 3-inch #8 stone/ 13-inch #57 stone
Driveways for heavy truck traffic	7.0	15	3	3-inch Sand / 3-inch #8 stone/ 17-inch #57 stone

8.15.2 The aggregate presented in Table 8.15.2 should be in conformance with ASTM C33 as shown in Table 8.15.3.

TABLE 8.15.3
AGGREGATE GRADATION LIMITS PER ASTM C33

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

8.15.3 Prior to placing base/aggregate materials, the subgrade soil should be scarified, moisture conditioned as necessary, and recompacted to a dry density of at least 95 percent of the laboratory maximum dry density near to slightly above optimum moisture content as determined by ASTM D 1557. The depth of compaction should be at least 12 inches. Similarly, the base materials should be compacted to a dry density of at least 95 percent of the laboratory maximum dry density near to slightly above optimum moisture content.

8.15.4 If permeable paver areas will be utilized, the subgrade should be graded to a low point to allow water to flow to a subdrain. The subdrain should be placed at the bottom of the base/aggregate section below the pavers and run the distance of the paver area to reduce the potential for water to build up within the paving section. The drain should be connected to an approved drainage device.

8.15.5 The pavers should be installed and maintained in accordance with the manufacturer's recommendations. Pavers tend to shift vertically and horizontally during the life of the pavement and periodic maintenance may be required to relevel paver areas. The owner should be made aware and responsible for the maintenance program. The pavers should be placed tightly adjacent to each other and the spacing between the paver units should be filled with appropriate filler. A polymer sand (Poly-Sand) can be used on the decorative paver area to help prevent water infiltration.

8.15.6 The pavers normally require a concrete border to prevent lateral movement from traffic. The concrete border surrounding the pavers should be embedded at least 6 inches into the subgrade to reduce the potential for water migration to the adjacent landscape areas and pavement areas.

8.15.7 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 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.

8.16 Grasscrete Pavement Recommendations

8.16.1 We recommend the grasscrete pavers be underlain by the same base section required for a flexible pavement system using an equivalent paver thickness (i.e., grasscrete equivalent to 3 inches of asphalt concrete). Table 8.16.1 summarizes the recommended Class 2 Base section assuming 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. We calculated the base thicknesses in general conformance with the *Caltrans Method of Flexible Pavement Design* (Highway Design Manual, Section 608.4).

TABLE 8.16.1
STRUCTURAL PAVEMENT SECTIONS FOR GRASSCRETE PAVERS

Estimated Traffic Index (TI)	Class 2 Base (inches)
5	8
5.5	10
6.0	11.5
7	15

8.16.2 Manufactures recommendations should be followed when constructing the grasscrete.

8.16.3 Prior to placing the paver cells, the subgrade should be scarified, moisture conditioned to optimum or slightly above optimum moisture content, and recompacted to a minimum of 95 percent relative compaction. The depth of compaction should be at least 12 inches.

8.16.4 Class 2 aggregate base materials should conform to Section 26-1.02B of the *Standard Specifications of the State of California, Department of Transportation* (Caltrans). Base should be compacted to a minimum of 95 percent relative compaction.

8.16.5 Edge restraints should be incorporated into the pavement section to maintain horizontal interlock while the pavement is subjected to vehicular loads. The edge restraints can consist of concrete edge bands or concrete curbs.

8.16.6 The performance of pavement, aggregate base, and subgrade materials are highly dependent on providing positive surface drainage. Ponding of water on or adjacent to the structural surface may result in saturation of the subgrade materials and subsequent distress. In addition, saturation of the bedding sand may result in pumping and sand loss beneath pavers causing differential settlement. To reduce the occurrence for such situations, a positive surface drainage gradient should be maintained across the pavement surface. Drainage from landscaped areas should be directed to controlled drainage structures.

8.17 Storm Water Management

8.17.1 If storm water management devices are not properly designed and constructed, there is a risk for distress to improvements and property located hydrologically down gradient or adjacent to these devices. Factors such as the amount of water being detained, its residence time, and soil permeability have an important effect on seepage transmission and the potential adverse impacts that may occur if the storm water management features are not properly designed and constructed. We have not performed a hydrogeological study at the site. If infiltration of storm water runoff into the subsurface occurs, downstream improvements may be subjected to seeps, springs, slope instability, raised groundwater, movement of foundations and slabs, or other undesirable impacts as a result of water infiltration.

8.17.2 Storm water management recommendations are provided in Appendix D. Based on infiltration testing, the site is considered to be infeasible to infiltration of storm water.

8.18 Site Drainage and Moisture Protection

8.18.1 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 2019 CBC 1804.4 or other applicable standards. In addition, surface drainage should be directed away from the top of slopes into swales or other controlled drainage devices. Roof and pavement drainage should be directed into conduits that carry runoff away from the proposed structure.

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

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

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

8.18.5 Appendix D provides storm water management recommendations.

8.19 Grading and Foundation Plan Review

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

8.20 Testing and Observation Services During Construction

8.20.1 Geocon Incorporated should provide geotechnical testing and observation services during the grading operations, foundation construction, utility installation, retaining wall backfill and pavement installation. Table 8.20 presents the typical geotechnical observations we would expect for the proposed improvements.

TABLE 8.20
EXPECTED GEOTECHNICAL TESTING AND OBSERVATION SERVICES

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

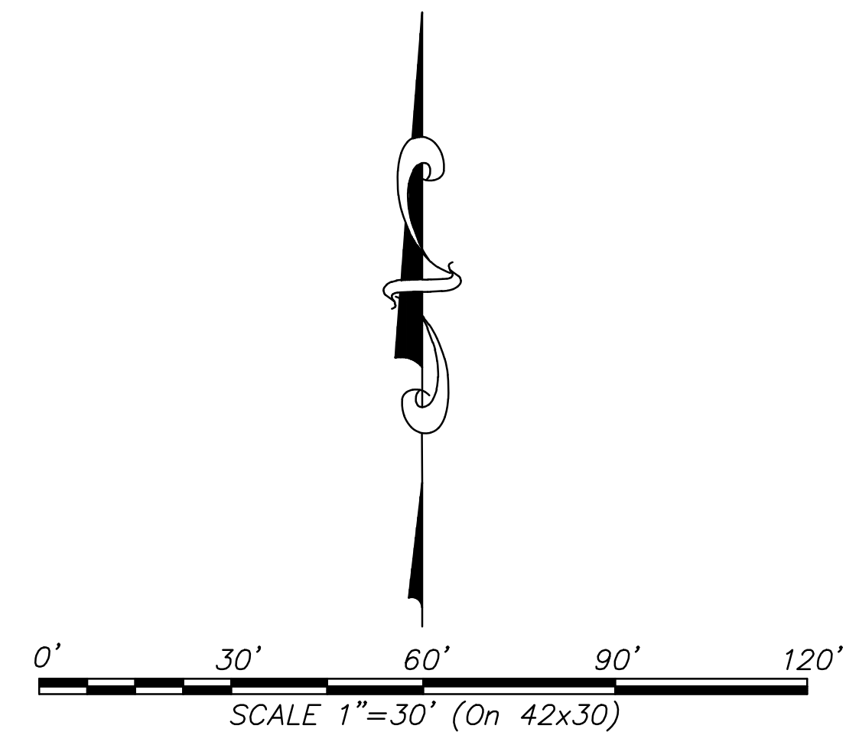
LIMITATIONS AND UNIFORMITY OF CONDITIONS

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.



GEOCON LEGEND

Qpf_1........PREVIOUSLY PLACED FILL (Tested by Geocon)

Qpf_2........PREVIOUSLY PLACED FILL (Tested by City of San Diego)

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

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

Ta........ARDATH SHALE

B-3 ⊕ ....APPROX. LOCATION OF BORING (Current Study)

B-8 ⦿ ....APPROX. LOCATION OF PREVIOUS BORING (Geocon, 1992)

A-2 ⌖ ....APPROX. LOCATION OF INFILTRATION TEST

(16.5') ....DEPTH TO FORMATION (In Feet)

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

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

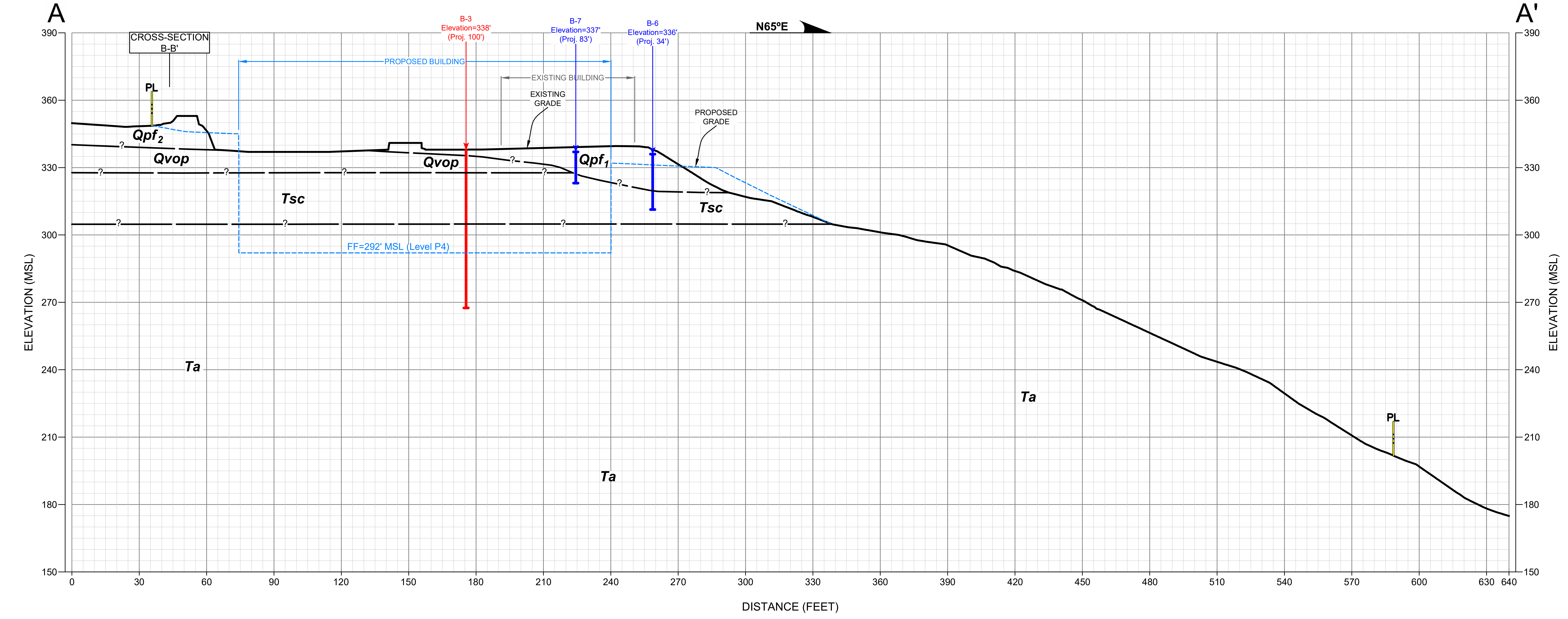
GEOLOGIC MAP

TORREYANA LIFE SCIENCE PROJECT
11011 TORREYANA ROAD
SAN DIEGO, CALIFORNIA

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

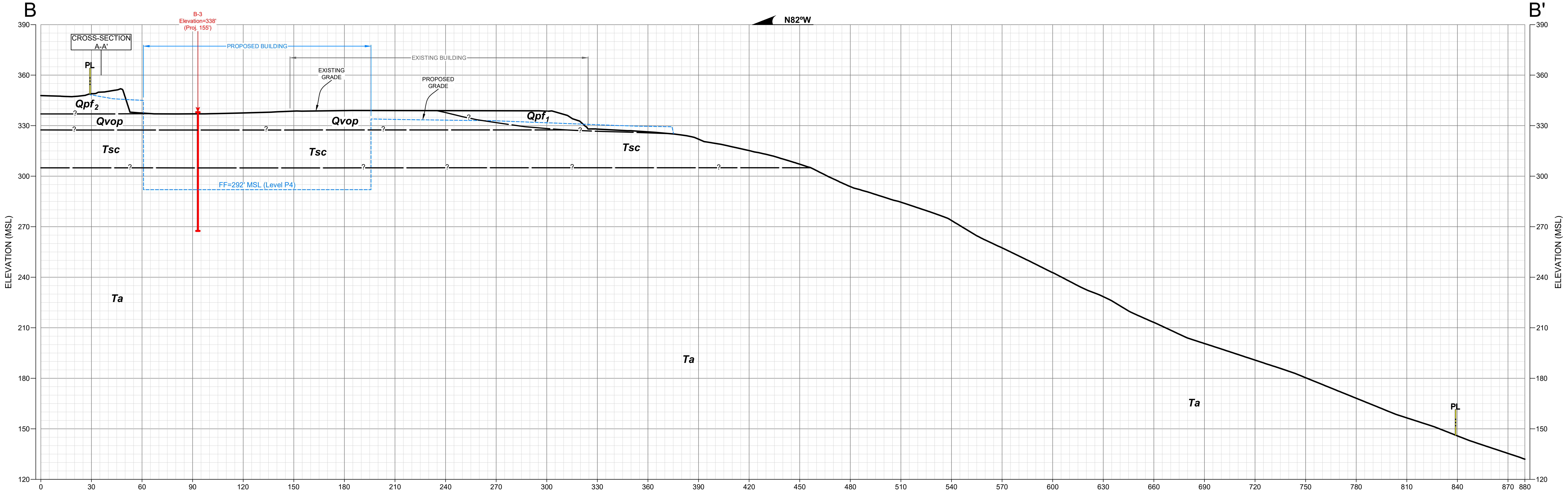
SCALE 1" = 30'	DATE 07 - 28 - 2022		
PROJECT NO. G2972 - 42 - 01		FIGURE 1	
SHEET 1 OF 1			

Plotted:07/28/2022 9:53AM \| By:RUBEN AGUILAR \| File Location:Y:\PROJECTS\G2972-42-01 (Torreyana Life Science)\SHEETS\G2972-42-01 GeoMap.dwg



GEOLOGIC CROSS-SECTION A-A'

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



GEOLOGIC CROSS-SECTION B-B'

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

GEOCON LEGEND

Qpf_1	........PREVIOUSLY PLACED FILL (Tested by Geocon)
Qpf_2	........PREVIOUSLY PLACED FILL (Tested by City of San Diego)
Qvop	........VERY OLD PARALIC DEPOSITS
Tsc	........SCRIPPS FORMATION
Ta	........ARDATH SHALE
B-3	........APPROX. LOCATION OF BORING (Current Study)
B-8	........APPROX. LOCATION OF PREVIOUS BORING (Geocon, 1992)
?—	........APPROX. LOCATION OF GEOLOGIC CONTACT (Queried Where Uncertain)

GEOLOGIC CROSS - SECTIONS

TORREYANA LIFE SCIENCE PROJECT
11011 TORREYANA ROAD
SAN DIEGO, CALIFORNIA

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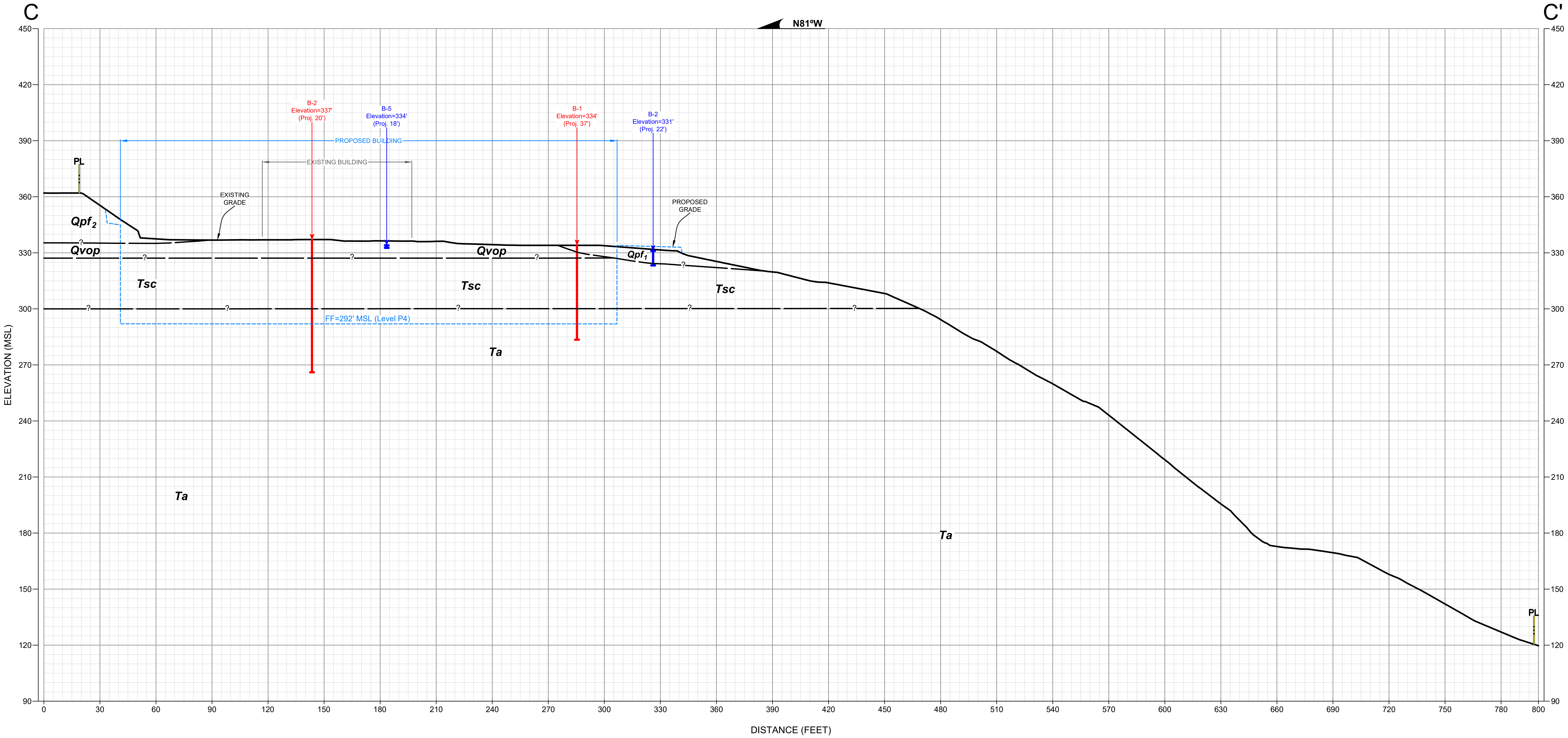
GEOTECHNICAL ■ ENVIRONMENTAL ■ MATERIALS
6960 FLANDERS DRIVE - SAN DIEGO, CALIFORNIA 92121 - 2974
PHONE 858 558-6900 - FAX 858 558-6159

SCALE 1" = 30'
DATE 07 - 28 - 2022
PROJECT NO. G2972 - 42 - 01
FIGURE 2
SHEET 1 OF 1

Plotted:07/28/2022 9:54AM | By:RUBEN AGUILAR | File Location:Y:\PROJECTS\G2972-42-01 (Torreyana Life Science)\SOURCE\SECTION\G2972-42-01 Profiles.dwg



GEOLOGIC CROSS-SECTION C-C'

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

GEOCON LEGEND

Qpf₁........PREVIOUSLY PLACED FILL (Tested by Geocon)
Qpf₂........PREVIOUSLY PLACED FILL (Tested by City of San Diego)
Qvop........VERY OLD PARALIC DEPOSITS
Tsc........SCRIPPS FORMATION
Ta........ARDATH SHALE
B-3 ....APPROX. LOCATION OF BORING (Current Study)
B-8 ....APPROX. LOCATION OF PREVIOUS BORING (Geocon, 1992)
?........APPROX. LOCATION OF GEOLOGIC CONTACT
(Queried Where Uncertain)

GEOLOGIC CROSS - SECTIONS

TORREYANA LIFE SCIENCE PROJECT
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PHONE 858 558-6900 - FAX 858 558-6159

SCALE 1" = 30'
DATE 07 - 28 - 2022
PROJECT NO. G2972 - 42 - 01
FIGURE 3
SHEET 1 OF 1

Plotted:07/28/2022 9:55AM | By:RUBEN AGUILAR | File Location:Y:\PROJECTS\G2972-42-01 (Torreyana Life Science)\SOURCE\SECTION\G2972-42-01 Profiles.dwg

APPENDIX A

APPENDIX A

FIELD INVESTIGATION

We performed our field investigation on June 24, 2022. Borings were extended to maximum depth of approximately 71 feet using a CME 95 drill rig equipped with hollow-stem augers. The locations of the exploratory borings are shown on the Geologic Map, Figure 1, and the boring logs are presented in this Appendix. We located the borings in the field using a measuring tape and existing reference points; therefore, actual boring locations may deviate slightly.

We obtained samples from the borings using a California Modified sampler. We obtained soil samples at appropriate intervals and transported them to the laboratory for testing. Bulk soil samples were also collected. The type of sample is noted on the exploratory boring logs.

The sampler was driven 12 inches using a 140-pound hammer with a 30-inch drop. The penetration resistances on the boring logs are shown in terms of blows per foot. These values are not to be taken as N-values as adjustments have not been applied. We estimated elevations shown on the boring logs based on grading plans prepared for the original store.

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

PROJECT NO. G2972-42-01

BORING B 1

ELEV. (MSL.) 334' DATE COMPLETED 06-24-2022

EQUIPMENT CME 95 BY: N. BORJA

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0					**4¼" ASPHALT CONCRETE Over 7¾" BASE**			
	B1-1			SM	**PREVIOUSLY PLACED FILL (Qpf$_1$)** Medium dense, damp, brown, Silty, fine to medium SAND; trace gravel -Becomes pale yellowish brown			
2								
4				SM	**VERY OLD PARALIC DEPOSITS (Qvop)** Very dense, damp, mottled tan brown and yellowish brown, Silty, fine- to coarse-grained SANDSTONE; trace gravel			
	B1-2					50/5"		
6								
8								
10	B1-3					50/5"	102.1	9.2
12								
				SM	**SCRIPPS FORMATION (Tsc)** Very dense, damp, mottled pale yellowish brown, tan, and light gray, Silty, fine- to medium-grained SANDSTONE			
14								
	B1-4					76/10"		
16								
18								
20	B1-5				-No recovery	50/4"		
22								
24				SM/ SP-SM	Very dense, damp, very light gray and pale yellow, fine- to coarse-grained SANDSTONE; few silt			
	B1-6					50/6"		3.1
26								
28								

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

G2972-42-01.GPJ

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

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

GEOCON

PROJECT NO. G2972-42-01

BORING B 1

ELEV. (MSL.) **334'** DATE COMPLETED **06-24-2022**

EQUIPMENT **CME 95** BY: **N. BORJA**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
30	B1-7				-No recovery	50/4"		
32								
34								
	B1-8					80/8"		
36				SM	**ARDATH SHALE (Ta)** Very dense, damp, light brown, Silty, fine- to medium-grained SANDSTONE			
38								
40	B1-9					79/8"		
	B1-10				-Hard drilling below 41 feet			
42								
44								
	B1-11					50/3"		
46								
48					-Becomes mottled tan brown and olive brown			
50	B1-12					50/5"		
					BORING TERMINATED AT 50.5 FEET Groundwater not encountered Backfilled with 16.5 ft³ bentonite			

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

G2972-42-01.GPJ

SAMPLE SYMBOLS

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

PROJECT NO. G2972-42-01

BORING B 2

ELEV. (MSL.) **337'** DATE COMPLETED **06-24-2022**

EQUIPMENT **CME 95** BY: **N. BORJA**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0					**4½" ASPHALT CONCRETE Over 7" BASE**			
2				SM	**VERY OLD PARALIC DEPOSITS (Qvop)** Very dense, damp, brown, Silty, fine- to coarse-grained SANDSTONE; trace gravel			
4				SM	**SCRIPPS FORMATION (Tsc)** Very dense, damp, mottled light gray and yellowish brown, Silty, fine- to coarse-grained SANDSTONE			
6	B2-1 B2-2					50/4"		
8					-Becomes cemented; hard drilling			
10	B2-3				-Poor recovery; slough	50/2"		
12								
14				SM/ML	Very dense, damp, mottled olive brown and tan brown, Silty, fine- to medium-grained SANDSTONE and Sandy SILTSTONE			
	B2-4					50/5"	100.8	11.5
16								
18								
					-Becomes mottled olive brown and grayish brown			
20	B2-5 B2-6					50/5.5"		
22								
24				SM	Very dense, damp, mottled light gray and yellowish brown, Silty, fine-medium-grained SANDSTONE			
	B2-7					96/9"	98.0	12.3
26								
28								

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

SAMPLE SYMBOLS

☐ ... SAMPLING UNSUCCESSFUL ◧ ... STANDARD PENETRATION TEST ■ ... DRIVE SAMPLE (UNDISTURBED)

▨ ... DISTURBED OR BAG SAMPLE ◪ ... CHUNK SAMPLE ▼ ... WATER TABLE OR ▽ ... SEEPAGE

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

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BORING B 2

ELEV. (MSL.) **337'** DATE COMPLETED **06-24-2022**

EQUIPMENT **CME 95** BY: **N. BORJA**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
30	B2-8				-Hard drilling below 30 feet	50/4.5"		
32								
34				SM	**ARDATH SHALE (Ta)** Very dense, damp, olive brown, Silty, fine- to medium-grained SANDSTONE			
	B2-9					50/5.5"		
36								
38								
40	B2-10					50/5"	108.5	11.5
42								
					-Becomes mottled olive brown and tan brown			
44								
	B2-11					50/4"		
46								
48								
50	B2-12				-Hard cemented zone or concretion between 50 and 52 feet; no recovery at 50 feet -Penetration rate of approx. 2 min/in	50/1"		
52								
54					-Becomes mottled olive brown, gray, and reddish brown; some mica			
	B2-13					64/10"		
56								
58								

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

G2972-42-01.GPJ

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

PROJECT NO. G2972-42-01

BORING B 2

ELEV. (MSL.) 337' DATE COMPLETED 06-24-2022

EQUIPMENT CME 95 BY: N. BORJA

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
60	B2-14				-No recovery	50/1"		
62				ML	Hard, damp, gray to dark gray, Sandy SILTSTONE			
64								
	B2-15					50/3"		
66								
68								
70	B2-16					78/9"		

BORING TERMINATED AT 71 FEET
Groundwater not encountered
Backfilled with 23.4 ft³ bentonite

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

G2972-42-01.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 3

ELEV. (MSL.) **338'** DATE COMPLETED **06-24-2022**

EQUIPMENT **CME 95** BY: **N. BORJA**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0					**3" ASPHALT CONCRETE Over 8" BASE**			
2				SM	**PREVIOUSLY PLACED FILL (Qpf$_1$)** Medium dense, damp, brown to tan brown, Silty, fine to medium SAND; trace gravel			
4				SM	**VERY OLD PARALIC DEPOSITS (Qvop)** Very dense, damp, mottled reddish brown and yellowish brown, Silty, fine- to medium-grained SANDSTONE; trace gravel			
	B3-1					79/8"		
6								
8								
10	B3-2					50/3"		
	B3-3			SM	**SCRIPPS FORMATION (Tsc)** Very dense, damp, mottled gray and yellowish brown, Silty, fine- to medium-grained SANDSTONE			
12								
14				ML	Hard, damp, mottled tan brown, olive brown, and gray, Sandy SILTSTONE			
	B3-4					77/9"	109.6	17.0
16								
18								
				SM	Very dense, damp, mottled light gray and yellowish brown, Silty, fine- to medium-grained SANDSTONE -Poor recovery at 20 feet; disturbed sample			
20	B3-5					50/5"		
22								
24								
	B3-6					50/5.5"		
26								
28								
					-Becomes light brown and reddish brown			

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

SAMPLE SYMBOLS

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

PROJECT NO. G2972-42-01

BORING B 3

ELEV. (MSL.) **338'** DATE COMPLETED **06-24-2022**

EQUIPMENT **CME 95** BY: **N. BORJA**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
30	B3-7 B3-8				-Poor recovery	50/5"		
32								
				SM	**ARDATH SHALE (Ta)** Very dense, damp, light brown, Silty, fine- to medium-grained SANDSTONE			
34								
	B3-9					50/6"	99.0	13.6
36								
38								
40	B3-10					50/6"		
42								
44								
	B3-11				-No recovery	50/4"		
46								
48								
50	B3-12				-Poor recovery	50/3"		
52								
54					-Becomes mottled gray and tan brown			
	B3-13					50/6"	92.7	19.1
56					-Hard drilling below 56 feet			
58								

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

G2972-42-01.GPJ

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

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

GEOCON

PROJECT NO. G2972-42-01

BORING B 3

ELEV. (MSL.) **338'** DATE COMPLETED **06-24-2022**

EQUIPMENT **CME 95** BY: **N. BORJA**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
60	B3-14				-Poor recovery due to rock	50/3"		
62								
64								
	B3-15				-Poor recovery due to rock	50/6"		
66				ML	Hard, damp, gray to dark gray, Sandy SILTSTONE			
68								
					-Hard drilling at 69 feet; cemented zone			
70	B3-16				-Poor recovery due to rock; slough	50/1"		

BORING TERMINATED AT 70.5 FEET
Groundwater not encountered
Backfilled with 23.5 ft³ bentonite

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

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

APPENDIX B

APPENDIX B

LABORATORY TESTING

We performed laboratory tests in accordance with generally accepted test methods of the American Society for Testing and Materials (ASTM) or other suggested procedures. We tested selected soil samples for in-place dry density/moisture content, expansion index, water-soluble sulfate, water-soluble chloride ion content, R-Value, and direct shear strength. The results of our current laboratory tests are presented herein. The in-place dry density and moisture content of the samples tested are presented on the boring logs in Appendix A.

SUMMARY OF LABORATORY EXPANSION INDEX TEST RESULTS
ASTM D 4829

Sample No.	Moisture Content (%)		Dry Density (pcf)	Expansion Index	2019 CBC Expansion Classification	ASTM Soil Expansion Classification
	Before Test	After Test				
B1-1	9.4	15.8	112.0	0	Non-Expansive	Very Low
B2-2	9.4	16.2	112.8	15	Non-Expansive	Very Low

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

Sample No.	Depth (feet)	Geologic Unit	Water-Soluble Sulfate (%)	ACI 318 Sulfate Exposure
B1-1	1-5	Qpf/Qvop	0.008	S0
B2-4	15	Tsc	0.006	S0
B2-10	40	Ta	0.007	S0

SUMMARY OF LABORATORY CHLORIDE TEST RESULTS
AASHTO T 291

Sample No.	Depth (Feet)	Geologic Unit	Chloride Ion Content (ppm)	Chloride Ion Content (%)
B1-1	1-5	Qpf/Qvop	65	0.007
B2-4	15	Tsc	41	0.004
B2-10	40	Ta	25	0.003

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

Sample No.	Depth (Feet)	Description (Geologic Unit)	R-Value
B1-1	1-5	Brown, Silty fine to medium SAND; trace gravel (Qpf/Qvop)	70

SAMPLE NO.:	B1-3	GEOLOGIC UNIT:	Qvop
SAMPLE DEPTH (FT):	10'	NATURAL/REMOLDED:	N

INITIAL CONDITIONS

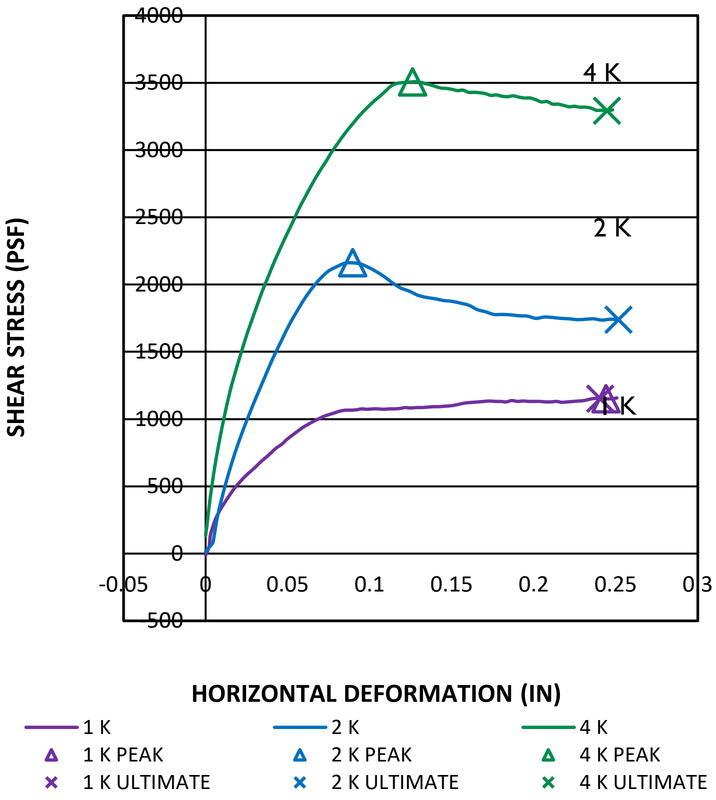
NORMAL STRESS TEST LOAD	1 K	2 K	4 K	AVERAGE
ACTUAL NORMAL STRESS (PSF):	1000	2000	4000	--
WATER CONTENT (%):	9.1	9.7	8.7	9.2
DRY DENSITY (PCF):	101.0	106.8	98.5	102.1

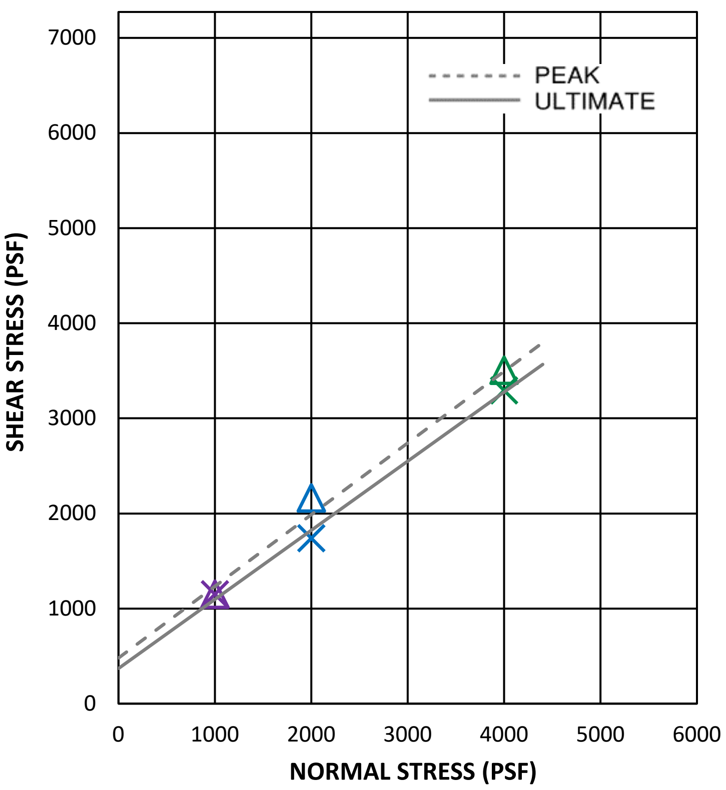
AFTER TEST CONDITIONS

NORMAL STRESS TEST LOAD	1 K	2 K	4 K	AVERAGE
WATER CONTENT (%):	20.9	18.7	21.7	20.4
PEAK SHEAR STRESS (PSF):	1151	2162	3507	--
ULT.-E.O.T. SHEAR STRESS (PSF):	1151	1739	3293	--

RESULTS

PEAK	COHESION, C (PSF)	480
	FRICTION ANGLE (DEGREES)	37
ULTIMATE	COHESION, C (PSF)	370
	FRICTION ANGLE (DEGREES)	36





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AASHTO T-236

TORREYANA LIFE SCIENCE PROJECT

PROJECT NO.: G2972-42-01

SAMPLE NO.:	**B2-7**	GEOLOGIC UNIT:	**Tsc**
SAMPLE DEPTH (FT):	**25'**	NATURAL/REMOLDED:	**N**

INITIAL CONDITIONS

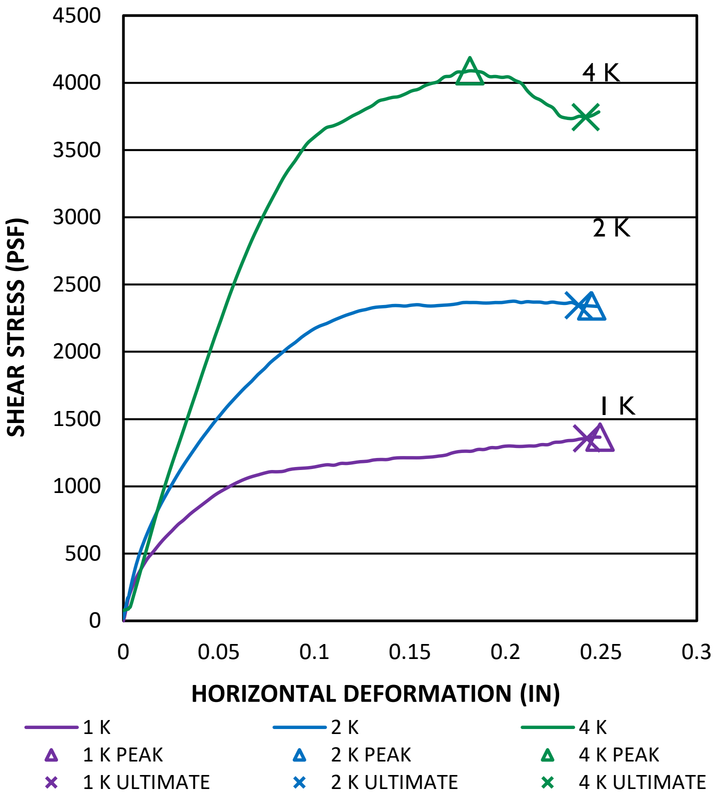
NORMAL STRESS TEST LOAD	1 K	2 K	4 K	AVERAGE
ACTUAL NORMAL STRESS (PSF):	1000	2000	4000	--
WATER CONTENT (%):	12.0	12.1	12.8	12.3
DRY DENSITY (PCF):	96.2	99.1	98.6	98.0

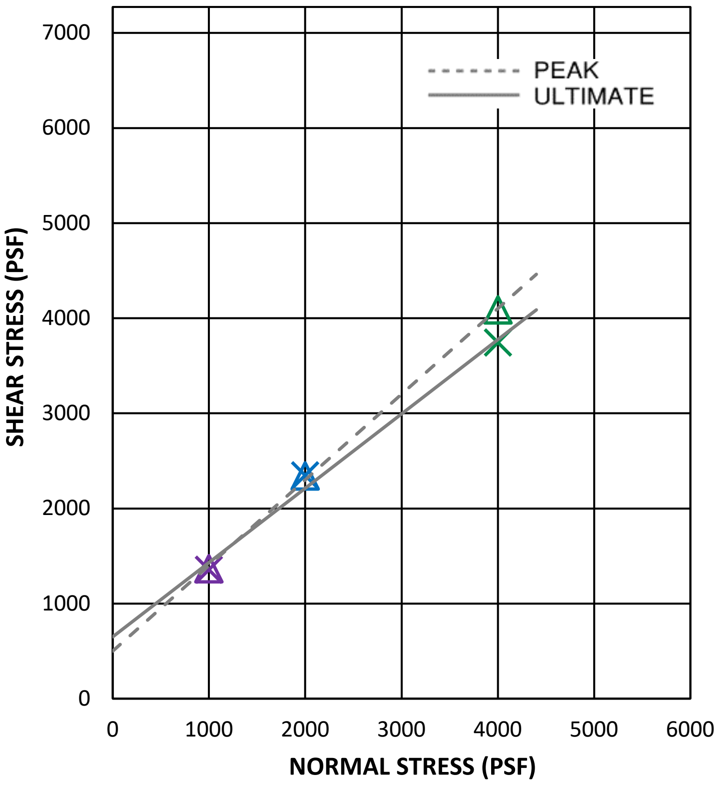
AFTER TEST CONDITIONS

NORMAL STRESS TEST LOAD	1 K	2 K	4 K	AVERAGE
WATER CONTENT (%):	22.3	21.9	22.1	22.1
PEAK SHEAR STRESS (PSF):	1365	2340	4089	--
ULT.-E.O.T. SHEAR STRESS (PSF):	1355	2347	3745	--

RESULTS

PEAK	COHESION, C (PSF)	500
	FRICTION ANGLE (DEGREES)	42
ULTIMATE	COHESION, C (PSF)	650
	FRICTION ANGLE (DEGREES)	38





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AASHTO T-236

TORREYANA LIFE SCIENCE PROJECT

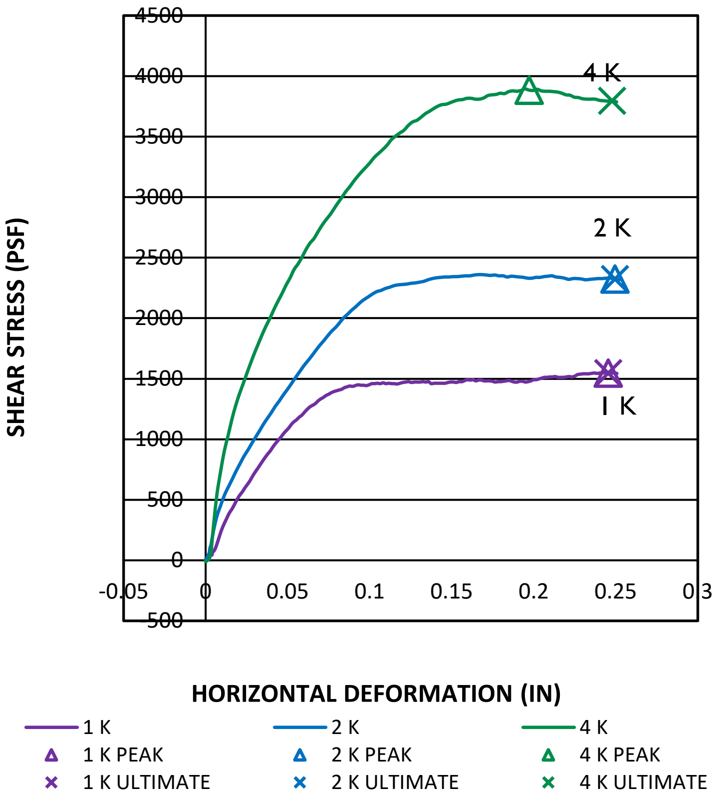
PROJECT NO.: G2972-42-01

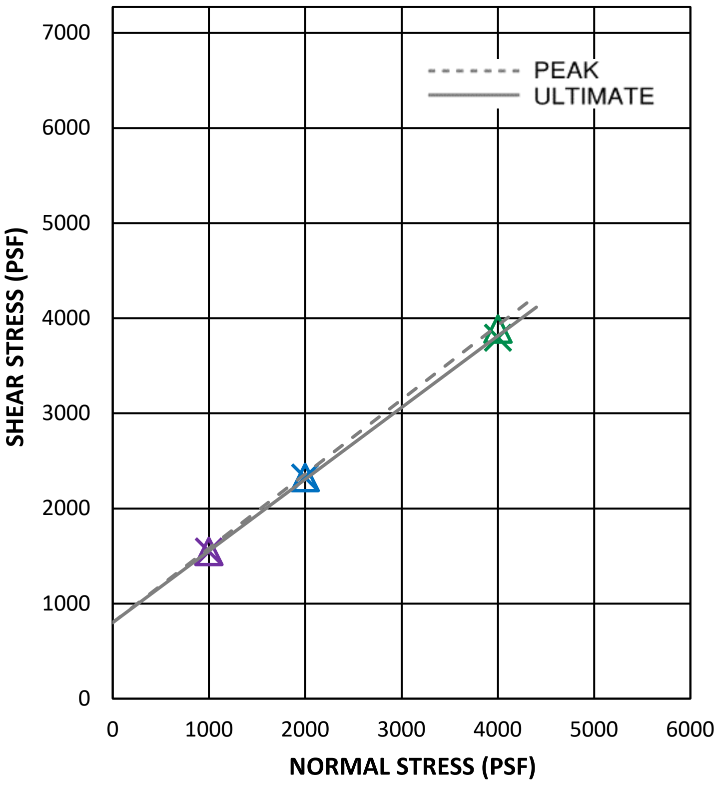
SAMPLE NO.:	**B3-9**	GEOLOGIC UNIT:	**Ta**
SAMPLE DEPTH (FT):	**35'**	NATURAL/REMOLDED:	**N**

INITIAL CONDITIONS				
NORMAL STRESS TEST LOAD	**1 K**	**2 K**	**4 K**	**AVERAGE**
ACTUAL NORMAL STRESS (PSF):	1000	2000	4000	--
WATER CONTENT (%):	13.5	13.7	13.7	13.6
DRY DENSITY (PCF):	100.4	97.9	98.6	99.0

AFTER TEST CONDITIONS				
NORMAL STRESS TEST LOAD	**1 K**	**2 K**	**4 K**	**AVERAGE**
WATER CONTENT (%):	23.3	24.0	23.8	23.7
PEAK SHEAR STRESS (PSF):	1547	2324	3884	--
ULT.-E.O.T. SHEAR STRESS (PSF):	1547	2324	3797	--

RESULTS		
PEAK	COHESION, C (PSF)	800
	FRICTION ANGLE (DEGREES)	38
ULTIMATE	COHESION, C (PSF)	800
	FRICTION ANGLE (DEGREES)	37





GEOCON
INCORPORATED

GEOTECHNICAL CONSULTANTS
6960 FLANDERS DRIVE - SAN DIEGO, CALIFORNIA 92121-2974
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AASHTO T-236

TORREYANA LIFE SCIENCE PROJECT

PROJECT NO.: G2972-42-01

APPENDIX C

APPENDIX C

PREVIOUS FIELD INVESTIGATION AND LABORATORY TESTING
(GEOCON, 1992)

FOR

TORREYANA LIFE SCIENCE PROJECT
11011 TORREYANA ROAD
SAN DIEGO, CALIFORNIA

PROJECT NO. G2972-42-01

PROJECT NO. 01549-02-04A

BORING B 1

ELEV. (MSL) 331 DATE COMPLETED 11/19/92

EQUIPMENT BEAVER RIG

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0								
2	B1-1 ■			SM	**FILL** Medium dense, slightly moist, mottled orangish-brown and light tan, Silty, fine to medium <u>SAND</u> trace clay	17	108.8	7.8
4	B1-2 ■			SM	**LINDAVISTA FORMATION** Very dense, slightly moist, orangish brown, Silty, fine to medium <u>SANDSTONE</u>	78/8"	102.8	5.4
6	B1-3 ■					50/5"		
					BORING TERMINATED AT 6.5 FEET			

Figure A-1 **Log of Boring B 1, page 1 of 1**

IDECA

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

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

PROJECT NO. 01549-02-04A

BORING B 2

ELEV. (MSL) 331 DATE COMPLETED 11/19/92

EQUIPMENT BEAVER RIG

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0								
				SM	**FILL** Loose to medium dense, slightly moist, light orange-tan, Silty, fine to medium SAND			
2	B2-1 ■					12	108.3	11.4
4	B2-2 ■				-Becomes very dense, moist, mottled, with light brown at 3.5 feet	55	117.3	7.7
6								
	B2-3 ■			SM	**LINDAVISTA FORMATION** Very dense, slightly moist, mottled, orange and gray, Silty, fine to medium SANDSTONE	90/9"		

BORING TERMINATED AT 7.5 FEET

Figure A-2 **Log of Boring B 2, page 1 of 1**

IDECA

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

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

PROJECT NO. 01549-02-04A

BORING B 3

ELEV. (MSL) 330 DATE COMPLETED 11/19/92

EQUIPMENT BEAVER RIG

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0		■ ■		GM	**CLASS 2 AGGREGATE BASE** - 5 inches			
	B3-1 ■			SM	**LINDAVISTA FORMATION** Very dense, slightly moist, mottled orange brown and light tan, Silty, fine to medium <u>SANDSTONE</u>	30/6"		
					BORING TERMINATED AT 1.5 FEET			

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

IDECA

SAMPLE SYMBOLS

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

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PROJECT NO. 01549-02-04A

BORING B 4

ELEV. (MSL) 331 DATE COMPLETED 11/19/92

EQUIPMENT HAND EXCAVATED

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0				GM	**CLASS 2 AGGREGATE BASE - 5 inches**			
	B4-1	☒		SM	**LINDAVISTA FORMATION** Very dense, slightly moist, mottled orangish-brown and light tan, Silty, fine to medium <u>SANDSTONE</u>			
					BORING TERMINATED AT 1.5 FEET			

Figure A-4 **Log of Boring B 4, page 1 of 1**

IDECA

SAMPLE SYMBOLS

☐ ... SAMPLING UNSUCCESSFUL

◧ ... STANDARD PENETRATION TEST

■ ... DRIVE SAMPLE (UNDISTURBED)

☒ ... DISTURBED OR BAG SAMPLE

▤ ... CHUNK SAMPLE

▼ ... WATER TABLE OR SEEPAGE

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

PROJECT NO. 01549-02-04A

BORING B 5

ELEV. (MSL) 334 DATE COMPLETED 11/19/92

EQUIPMENT BEAVER RIG

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0				SM	**FILL** Loose, slightly moist, Silty, fine <u>SAND</u>			
	B5-1 ■			SM	**LINDAVISTA FORMATION** Very dense, slightly moist, gray-tan, Silty, fine <u>SANDSTONE</u>	50/6"	111.5	6.6
					BORING TERMINATED AT 1.5 FEET			

Figure A-5 **Log of Boring B 5, page 1 of 1**

IDECA

SAMPLE SYMBOLS

☐ ... SAMPLING UNSUCCESSFUL ◧ ... STANDARD PENETRATION TEST ■ ... DRIVE SAMPLE (UNDISTURBED)

☒ ... DISTURBED OR BAG SAMPLE ▤ ... CHUNK SAMPLE ▼ ... WATER TABLE OR SEEPAGE

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

PROJECT NO. 01549-02-04A

BORING B 6

ELEV. (MSL) 336 DATE COMPLETED 11/19/92

EQUIPMENT BEAVER RIG

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0				SM	**FILL** Medium dense, moist, brown and gray, Silty, fine to medium SAND; some sandstone chunks (gravel size) in fill			
2	B6-1				-Interbeds of stiff, moist, mottled gray and brown, fine sandy silt (SM/ML)	31	111.0	9.1
4								
	B6-2					25	112.1	10.6
6								
8								
	B6-3					40	108.7	17.1
10					-Some siltstone chunks (gravel size) in fill at 9.5 feet			
12								
	B6-4				-Trace organics at 13.5 feet	30	114.3	9.5
14								
	B6-5 ⊠			SM	Medium dense, moist, brown, Silty, fine to medium SAND			
16	B6-6					27	112.4	5.0
18				SM	**LINDAVISTA FORMATION** Medium dense, slightly moist, orange brown, Silty, fine to coarse SANDSTONE			

Figure A-6 **Log of Boring B 6, page 1 of 2**

IDECA

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

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

PROJECT NO. 01549-02-04A

BORING B 6

ELEV. (MSL) 336 DATE COMPLETED 11/19/92

EQUIPMENT BEAVER RIG

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
20					Medium dense, slightly moist, orange brown, Silty, fine to coarse SANDSTONE (Continued)			
22								
24	B6-7 ■				-Very dense, slightly moist, orange-tan, silty, fine to coarse sandstone at 24 feet	50/5"	113.6	4.3
					BORING TERMINATED AT 25 FEET			

Figure A-7 **Log of Boring B 6, page 2 of 2**

IDECA

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

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

PROJECT NO. 01549-02-04A

BORING B 7

ELEV. (MSL) 337 DATE COMPLETED 11/19/92

EQUIPMENT BEAVER RIG

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0					**FILL**			
	☒ B7-1			SM	Loose, moist, Silty, fine to medium SAND, some thin lenses of silty clay and clayey silt			
2								
					-Becomes medium dense at 3 feet			
4								
	■ B7-2					24	114.7	13.3
6								
8	■ B7-3					48	121.8	10.6
10					**LINDAVISTA FORMATION**			
	■ B7-4			SM	Very dense, slightly moist, Silty, fine to coarse SANDSTONE	50/6"	103.5	3.4
12								
	■ B7-5					44/6"	101.6	5.9

BORING TERMINATED AT 13.5 FEET

Figure A-8 **Log of Boring B 7, page 1 of 1**

IDECA

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

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

PROJECT NO. 01549-02-04A

BORING B 8

ELEV. (MSL) 338 DATE COMPLETED 11/19/92

EQUIPMENT HAND EXCAVATED

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0								
	B8-1 ⊠				**FILL** Loose, moist, brown, Silty, fine to medium SAND, minor debris consisting of concrete chunks and brick			
					BORING TERMINATED AT 1.5 FEET			

Figure A-9 Log of Boring B 8, page 1 of 1

IDECA

SAMPLE SYMBOLS

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

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

APPENDIX B

LABORATORY INVESTIGATION

Laboratory tests were performed in accordance with generally accepted American Society for Testing and Materials (ASTM) test methods or suggested procedures. Undisturbed samples were tested for their in-place dry density and moisture content, and shear strength characteristics. An R-Value (Resistance Value) test was performed on one disturbed, bulk sample to provide preliminary pavement design recommendations.

The results of our laboratory tests are presented in tabular form on Tables B-I and B-II. The in-place dry density and moisture content of selected samples are also presented on the logs of the exploratory trenches (Appendix A).

TABLE B-I

SUMMARY OF IN-PLACE MOISTURE-DENSITY AND DIRECT SHEAR TEST RESULTS

Sample No.	Dry Density (pcf)	Moisture Content (%)	Unit Cohesion (psf)	Angle of Shear Resistance (degrees)
B1-1	108.8	7.8	---	---
B1-2	102.8	5.4	---	---
B2-1	108.3	11.4	---	---
B2-2	117.3	7.7	---	---
B5-1	111.5	6.6	---	---
B6-1	111.0	9.1	---	---
B6-2	112.1	10.6	---	---
B6-3	108.7	17.1	---	---
B6-4	114.3	9.5	200	47
B6-6	112.4	5.0	---	---
B6-7	113.6	4.3	---	---
B7-2	114.7	13.3	---	---
B7-3	121.8	10.6	---	---
B7-4	103.5	3.4	---	---
B7-5	101.6	5.9	---	---

Project No. 01549-02-04A
December 17, 1992

TABLE B-II

SUMMARY OF R-VALUE TEST RESULTS

Sample No.	R-Value
B4-1	73

APPENDIX D

APPENDIX D

STORM WATER MANAGEMENT INVESTIGATION

We understand storm water management devices are being proposed in accordance with the *2021 City of San Diego Storm Water Standards* (SWS). If not properly constructed, there is a potential for distress to improvements and properties located hydrologically down gradient or adjacent to these devices. Factors such as the amount of water to be detained, its residence time, and soil permeability have an important effect on seepage transmission and the potential adverse impacts that may occur if the storm water management features are not properly designed and constructed. We have not performed a hydrogeological study at the site. If infiltration of storm water runoff occurs, downstream properties may be subjected to seeps, springs, slope instability, raised groundwater, movement of foundations and slabs, or other undesirable impacts as a result of water infiltration.

Hydrologic Soil Group

The United States Department of Agriculture (USDA), Natural Resources Conservation Services, possesses general information regarding the existing soil conditions for areas within the United States. The USDA website also provides the Hydrologic Soil Group. Table D-I presents the descriptions of the hydrologic soil groups. In addition, the USDA website also provides an estimated saturated hydraulic conductivity for the existing soil.

TABLE D-I
HYDROLOGIC SOIL GROUP DEFINITIONS

Soil Group	Soil Group Definition
A	Soils having a high infiltration rate (low runoff potential) when thoroughly wet. These consist mainly of deep, well drained to excessively drained sands or gravelly sands. These soils have a high rate of water transmission.
B	Soils having a moderate infiltration rate when thoroughly wet. These consist chiefly of moderately deep or deep, moderately well drained or well drained soils that have moderately fine texture to moderately coarse texture. These soils have a moderate rate of water transmission.
C	Soils having a slow infiltration rate when thoroughly wet. These consist chiefly of soils having a layer that impedes the downward movement of water or soils of moderately fine texture or fine texture. These soils have a slow rate of water transmission.
D	Soils having a very slow infiltration rate (high runoff potential) when thoroughly wet. These consist chiefly of clays that have a high shrink-swell potential, soils that have a high water table, soils that have a claypan or clay layer at or near the surface, and soils that are shallow over nearly impervious material. These soils have a very slow rate of water transmission.

The property is underlain by man-made previously placed fill and dense Very Old Paralic Deposits and formational bedrock and should be classified as Soil Group D. The Hydrologic Soil Group Map presents output from the USDA website showing the limits of the soil units.

Hydrologic Soil Group Map

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

TABLE D-II
USDA WEB SOIL SURVEY – HYDROLOGIC SOIL GROUP*

Map Unit Name	Map Unit Symbol	Approximate Percentage of Property	Hydrologic Soil Group	k_{SAT} of Most Limiting Layer (Inches/ Hour)
Loamy alluvial land-Huerhuero complex, 9 to 50 percent slopes, severely eroded	LvF3	36	D	0.00 – 0.06
Terrace Escarpments	TeF	64	Info. Not Available	Info. Not Available

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

In Situ Testing

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

TABLE D-III
INFILTRATION TEST RESULTS

Test No.	Geologic Unit	Test Elevation (feet, MSL)	Field-Saturated Hydraulic Conductivity/Infiltration Rate, k_{sat} (inch/hour)	Worksheet Infiltration Rate[1] (inch/hour)
A-1	Qpf	328	0.086	0.043
A-2	Qvop	334	0.001	0.001
Average			0.044	0.022

[1]Using a Factor of Safety of 2.

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

TABLE D-IV
INFILTRATION CATEGORIES

Infiltration Category	Field Infiltration Rate, I (Inches/Hour)	Factored Infiltration Rate[1], I (Inches/Hour)
Full Infiltration	$I > 1.0$	$I > 0.5$
Partial Infiltration	$0.10 < I \leq 1.0$	$0.05 < I \leq 0.5$
No Infiltration (Infeasible)	$I < 0.10$	$I < 0.05$

[1]Using a Factor of Safety of 2.

The results of the infiltration tests indicate an average infiltration rate of 0.044 inches per hour (0.022 inches per hour with a factor of safety of 2). Therefore, based on the results of the field infiltration tests, and our experience, infiltration is considered infeasible.

GEOTECHNICAL CONSIDERATIONS

Groundwater Elevations

Groundwater was not encountered in our borings to a maximum depth of about 70 feet. We expect groundwater is at a depth of greater than 100 feet below existing grades within the project area. Therefore, infiltration due to groundwater elevations would be considered feasible.

Expansive Soils

Based on our laboratory testing, the soil encountered in the field investigation is "non-expansive" and (expansion index [EI] of 20 or less) as defined by 2019 California Building Code (CBC) Section 1803.5.3. We expect most of the soil on site will have a "very low" to "medium" expansion potential (expansion index of 90 or less). Infiltration would be feasible when considering the expansion potential of the soil.

New or Existing Utilities

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

Existing or Planned Structures

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

Slopes

Natural and compacted fill slopes descend to the east and northeast at approximate inclinations of 2:1 (horizontal to vertical) and are up to about 250 feet high. Infiltration devices should not be installed adjacent to or on slopes unless they are lined, possess a minimum setback distance of 50 feet or 1.5 times the slope height, or extend below the height of the slope.

Soil or Groundwater Contamination

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

CONCLUSIONS AND RECOMMENDATIONS

Storm Water Evaluation Narrative

We used information from our geotechnical investigation and site observations to help evaluate possible locations for infiltration based on the known geologic information on the property. We selected areas on the property underlain by less than 5 feet of fill materials overlying Very Old Paralic Deposits or Scripps Formation. The results of the infiltration testing indicate an average rate of 0.022 inches per hour (with an applied factor of safety of 2).

Storm Water Evaluation Conclusion

Based on the average results of our infiltration tests (less than 0.05 inches per hour); we opine full and partial infiltration on the property is considered infeasible and the property possesses a “No Infiltration” condition.

Storm Water Management Devices

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

Storm Water Standard Worksheets

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

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

TABLE D-V
SUITABILITY ASSESSMENT RELATED CONSIDERATIONS FOR INFILTRATION FACILITY SAFETY FACTORS

Consideration	High Concern – 3 Points	Medium Concern – 2 Points	Low Concern – 1 Point
Assessment Methods	Use of soil survey maps or simple texture analysis to estimate short-term infiltration rates. Use of well permeameter or borehole methods without accompanying continuous boring log. Relatively sparse testing with direct infiltration methods	Use of well permeameter or borehole methods with accompanying continuous boring log. Direct measurement of infiltration area with localized infiltration measurement methods (e.g., Infiltrometer). Moderate spatial resolution	Direct measurement with localized (i.e. small-scale) infiltration testing methods at relatively high resolution or use of extensive test pit infiltration measurement methods.
Predominant Soil Texture	Silty and clayey soils with significant fines	Loamy soils	Granular to slightly loamy soils
Site Soil Variability	Highly variable soils indicated from site assessment or unknown variability	Soil boring/test pits indicate moderately homogenous soils	Soil boring/test pits indicate relatively homogenous soils
Depth to Groundwater/ Impervious Layer	<5 feet below facility bottom	5-15 feet below facility bottom	>15 feet below facility bottom

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

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

Suitability Assessment Factor Category	Assigned Weight (w)	Factor Value (v)	Product (p = w x v)
Assessment Methods	0.25	2	0.50
Predominant Soil Texture	0.25	2	0.50
Site Soil Variability	0.25	2	0.50
Depth to Groundwater/ Impervious Layer	0.25	1	0.25
Suitability Assessment Safety Factor, $S_A = \sum p$			1.75

*The project civil engineer should complete Worksheet D.5-1 or Form I-9 using the data on this table. Additional information is required to evaluate the design factor of safety.

TEST NO.: **A-1**

GEOLOGIC UNIT: **Tsc**

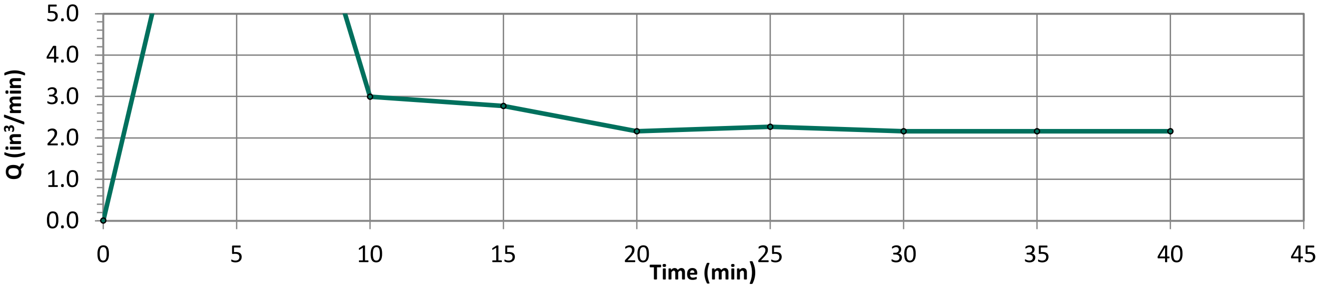
EXCAVATION ELEVATION (MSL, FT): **330**

TEST INFORMATION

BOREHOLE DIAMETER (IN):	4
BOREHOLE DEPTH (FT):	2.0
TEST/BOTTOM ELEVATION (MSL, FT):	328
MEASURED HEAD HEIGHT (IN):	6.0
CALCULATED HEAD HEIGHT (IN):	4.7
FACTOR OF SAFETY:	2.0

TEST RESULTS

STEADY FLOW RATE (IN^3/MIN):	**2.160**
FIELD-SATURATED INFILTRATION RATE (IN/HR):	**0.086**
FACTORED INFILTRATION RATE (IN/HR):	**0.043**



TEST DATA

Reading	Time Elapsed (min)	Water Weight Consumed (lbs)	Water Volume Consumed (in^3)	Q (in^3/min)
1	0.00	0.000	0.00	0.00
2	5.00	2.465	68.26	13.652
3	5.00	0.540	14.95	2.991
4	5.00	0.500	13.85	2.769
5	5.00	0.390	10.80	2.160
6	5.00	0.410	11.35	2.271
7	5.00	0.390	10.80	2.160
8	5.00	0.390	10.80	2.160
9	5.00	0.390	10.80	2.160

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

AARDVARK PERMEAMETER TEST RESULTS

TORREYANA LIFE SCIENCE

PROJECT NO.: G2972-42-01

TEST NO.: **A-2**

GEOLOGIC UNIT: **Qvop**

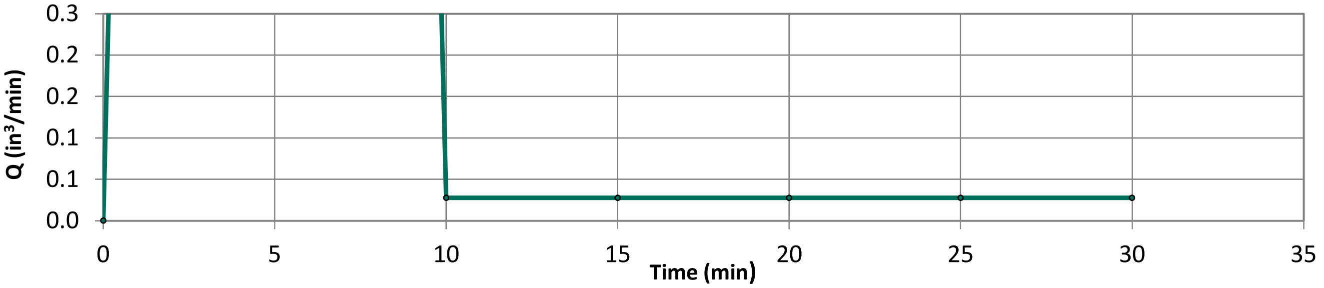
EXCAVATION ELEVATION (MSL, FT): **336**

TEST INFORMATION

BOREHOLE DIAMETER (IN):	4
BOREHOLE DEPTH (FT):	2.0
TEST/BOTTOM ELEVATION (MSL, FT):	334
MEASURED HEAD HEIGHT (IN):	6.0
CALCULATED HEAD HEIGHT (IN):	4.7
FACTOR OF SAFETY:	2.0

TEST RESULTS

STEADY FLOW RATE (IN^3/MIN):	**0.028**
FIELD-SATURATED INFILTRATION RATE (IN/HR):	**0.001**
FACTORED INFILTRATION RATE (IN/HR):	**0.001**



TEST DATA

Reading	Time Elapsed (min)	Water Weight Consumed (lbs)	Water Volume Consumed (in^3)	Q (in^3/min)
1	0.00	0.000	0.00	0.00
2	5.00	1.475	40.85	8.169
3	5.00	0.005	0.14	0.028
4	5.00	0.005	0.14	0.028
5	5.00	0.005	0.14	0.028
6	5.00	0.005	0.14	0.028
7	5.00	0.005	0.14	0.028

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AARDVARK PERMEAMETER TEST RESULTS

TORREYANA LIFE SCIENCE

PROJECT NO.: G2972-42-01

Worksheet C.4-1: Categorization of Infiltration Feasibility Condition Based on Geotechnical Conditions[9]

<table>
<tr><th colspan="2">Categorization of Infiltration Feasibility Condition based on Geotechnical Conditions</th><th>Worksheet C.4-1: Form I-8A[10]</th></tr>
<tr><td colspan="3">Part 1 - Full Infiltration Feasibility Screening Criteria</td></tr>
<tr><td colspan="2">DMA(s) Being Analyzed:</td><td>Project Phase:</td></tr>
<tr><td colspan="2">Torreyana Life Science</td><td>Design</td></tr>
<tr><td colspan="3">Criteria 1: Infiltration Rate Screening</td></tr>
<tr><td>1A</td><td colspan="2">Is the mapped hydrologic soil group according to the NRCS Web Soil Survey or UC Davis Soil Web Mapper Type A or B and corroborated by available site soil data[11]?
☐ Yes; the DMA may feasibly support full infiltration. Answer “Yes” to Criteria 1 Result or continue to Step 1B if the applicant elects to perform infiltration testing.
☐ No; the mapped soil types are A or B but is not corroborated by available site soil data (continue to Step 1B).
☐ No; the mapped soil types are C, D, or “urban/unclassified” and is corroborated by available site soil data. Answer “No” to Criteria 1 Result.
☒ No; the mapped soil types are C, D, or “urban/unclassified” but is not corroborated by available site soil data (continue to Step 1B).</td></tr>
<tr><td>1B</td><td colspan="2">Is the reliable infiltration rate calculated using planning phase methods from Table D.3-1?
☒ Yes; Continue to Step 1C.
☐ No; Skip to Step 1D.</td></tr>
<tr><td>1C</td><td colspan="2">Is the reliable infiltration rate calculated using planning phase methods from Table D.3-1 greater than 0.5 inches per hour?
☐ Yes; the DMA may feasibly support full infiltration. Answer “Yes” to Criteria 1 Result.
☒ No; full infiltration is not required. Answer “No” to Criteria 1 Result.</td></tr>
<tr><td>1D</td><td colspan="2">Infiltration Testing Method. Is the selected infiltration testing method suitable during the design phase (see Appendix D.3)? Note: Alternative testing standards may be allowed with appropriate rationales and documentation.
☐ Yes; continue to Step 1E.
☐ No; select an appropriate infiltration testing method.</td></tr>
</table>

---

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

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

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

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

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

We performed 2 infiltration within areas of the site underlain by less than 5 feet of fill. The results indicate an average rate of 0.022 inches per hour (with an applied factor of safety of 2). Therefore, full infiltration is considered infeasible at the site.

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

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

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

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

---

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

Categorization of Infiltration Feasibility Condition based on Geotechnical Conditions		Worksheet C.4-1: Form I-8A[10]
Part 2 – Partial vs. No Infiltration Feasibility Screening Criteria		
DMA(s) Being Analyzed:		**Project Phase:**
Torreyana Life Science		Design
Criteria 3 : Infiltration Rate Screening		
3A	**NRCS Type C, D, or "urban/unclassified":** Is the mapped hydrologic soil group according to the NRCS Web Soil Survey or UC Davis Soil Web Mapper is Type C, D, or "urban/unclassified" and corroborated by available site soil data? ☐ Yes; the site is mapped as C soils and a reliable infiltration rate of 0.15 in/hr. is used to size partial infiltration BMPS. Answer "Yes" to Criteria 3 Result. ☐ Yes; the site is mapped as D soils or "urban/unclassified" and a reliable infiltration rate of 0.05 in/hr. is used to size partial infiltration BMPS. Answer "Yes" to Criteria 3 Result. ☒ No; infiltration testing is conducted (refer to Table D.3-1), continue to Step 3B.	
3B	**Infiltration Testing Result:** Is the reliable infiltration rate (i.e. average measured infiltration rate/2) greater than 0.05 in/hr. and less than or equal to 0.5 in/hr? ☐ Yes; the site may support partial infiltration. Answer "Yes" to Criteria 3 Result. ☒ No; the reliable infiltration rate (i.e. average measured rate/2) is less than 0.05 in/hr., partial infiltration is not required. Answer "No" to Criteria 3 Result.	
Criteria 3 Result	Is the estimated reliable infiltration rate (i.e., average measured infiltration rate/2) greater than or equal to 0.05 inches/hour and less than or equal to 0.5 inches/hour at any location within each DMA where runoff can reasonably be routed to a BMP? ☐ Yes; Continue to Criteria 4. ☒ No: Skip to Part 2 Result.	
Summarize infiltration testing and/or mapping results (i.e. soil maps and series description used for infiltration rate). We performed 2 infiltration tests within areas of the site underlain by less than 5 feet of fill. The results indicate an average rate of 0.022 inches per hour (with an applied factor of safety of 2). Therefore, full infiltration is considered infeasible at the site.		

Categorization of Infiltration Feasibility Condition based on Geotechnical Conditions		Worksheet C.4-1: Form I-8A[10]	
Criteria 4: Geologic/Geotechnical Screening			
4A	If all questions in Step 4A are answered "Yes," continue to Step 2B. For any "No" answer in Step 4A answer "No" to Criteria 4 Result, and submit an "Infiltration Feasibility Condition Letter" that meets the requirements in Appendix C.1.1. The geologic/geotechnical analyses listed in Appendix C.2.1 do not apply to the DMA because one of the following setbacks cannot be avoided and therefore result in the DMA being in a no infiltration condition. The setbacks must be the closest horizontal radial distance from the surface edge (at the overflow elevation) of the BMP.		
4A-1	Can the proposed partial infiltration BMP(s) avoid areas with existing fill materials greater than 5 feet thick?	☐ Yes	☐ No
4A-2	Can the proposed partial infiltration BMP(s) avoid placement within 10 feet of existing underground utilities, structures, or retaining walls?	☐ Yes	☐ No
4A-3	Can the proposed partial infiltration BMP(s) avoid placement within 50 feet of a natural slope (>25%) or within a distance of 1.5H from fill slopes where H is the height of the fill slope?	☐ Yes	☐ No
4B	When full infiltration is determined to be feasible, a geotechnical investigation report must be prepared that considers the relevant factors identified in Appendix C.2.1 If all questions in Step 4B are answered "Yes," then answer "Yes" to Criteria 4 Result. If there are any "No" answers continue to Step 4C.		
4B-1	**Hydroconsolidation.** Analyze hydroconsolidation potential per approved ASTM standard due to a proposed full infiltration BMP. Can partial infiltration BMPs be proposed within the DMA without increasing hydroconsolidation risks?	☐ Yes	☐ No
4B-2	**Expansive Soils.** Identify expansive soils (soils with an expansion index greater than 20) and the extent of such soils due to proposed full infiltration BMPs. Can partial infiltration BMPs be proposed within the DMA without increasing expansive soil risks?	☐ Yes	☐ No
4B-3	**Liquefaction**. If applicable, identify mapped liquefaction areas. Evaluate liquefaction hazards in accordance with Section 6.4.2 of the City of San Diego's Guidelines for Geotechnical Reports (2011). Liquefaction hazard assessment shall take into account any increase in groundwater elevation or groundwater mounding that could occur as a result of proposed infiltration or percolation facilities. Can partial infiltration BMPs be proposed within the DMA without increasing liquefaction risks?	☐ Yes	☐ No

Categorization of Infiltration Feasibility Condition based on Geotechnical Conditions		Worksheet C.4-1: Form I-8A[10]	
4B-4	**Slope Stability**. If applicable, perform a slope stability analysis in accordance with the ASCE and Southern California Earthquake Center (2002) Recommended Procedures for Implementation of DMG Special Publication 117, Guidelines for Analyzing and Mitigating Landslide Hazards in California to determine minimum slope setbacks for full infiltration BMPs. See the City of San Diego's Guidelines for Geotechnical Reports (2011) to determine which type of slope stability analysis is required. Can partial infiltration BMPs be proposed within the DMA without increasing slope stability risks?	☐ Yes	☐ No
4B-5	**Other Geotechnical Hazards.** Identify site-specific geotechnical hazards not already mentioned (refer to Appendix C.2.1). Can partial infiltration BMPs be proposed within the DMA without increasing risk of geologic or geotechnical hazards not already mentioned?	☐ Yes	☐ No
4B-6	**Setbacks.** Establish setbacks from underground utilities, structures, and/or retaining walls. Reference applicable ASTM or other recognized standard in the geotechnical report. Can partial infiltration BMPs be proposed within the DMA using recommended setbacks from underground utilities, structures, and/or retaining walls?	☐ Yes	☐ No
4C	**Mitigation Measures.** Propose mitigation measures for each geologic/geotechnical hazard identified in Step 4B. Provide a discussion on geologic/geotechnical hazards that would prevent partial infiltration BMPs that cannot be reasonably mitigated in the geotechnical report. See Appendix C.2.1.8 for a list of typically reasonable and typically unreasonable mitigation measures. Can mitigation measures be proposed to allow for partial infiltration BMPs? If the question in Step 4C is answered “Yes,” then answer “Yes” to Criteria 4 Result. If the question in Step 4C is answered “No,” then answer “No” to Criteria 4 Result.	☐ Yes	☐ No
Criteria 4 Result	Can infiltration of greater than or equal to 0.05 inches/hour and less than or equal to 0.5 inches/hour be allowed without increasing the risk of geologic or geotechnical hazards that cannot be reasonably mitigated to an acceptable level?	☐ Yes	☐ No

Categorization of Infiltration Feasibility Condition based on Geotechnical Conditions	**Worksheet C.4-1: Form I-8A[10]**
Summarize findings and basis; provide references to related reports or exhibits.	

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

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

APPENDIX E

APPENDIX E

RECOMMENDED GRADING SPECIFICATIONS

FOR

TORREYANA LIFE SCIENCE PROJECT
11011 TORREYANA ROAD
SAN DIEGO, CALIFORNIA

PROJECT NO. G2972-42-01

RECOMMENDED GRADING SPECIFICATIONS

1. GENERAL

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

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

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

2. DEFINITIONS

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

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

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

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

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

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

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

3. MATERIALS

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

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

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

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

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

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

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

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

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

4. CLEARING AND PREPARING AREAS TO BE FILLED

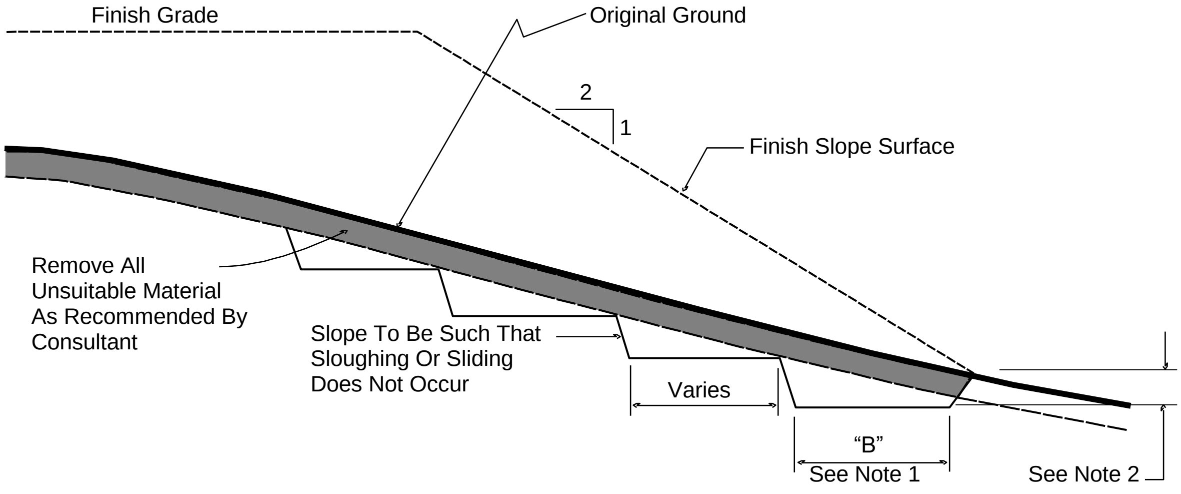
4.1 Areas to be excavated and filled shall be cleared and grubbed. Clearing shall consist of complete removal above the ground surface of trees, stumps, brush, vegetation, man-made structures, and similar debris. Grubbing shall consist of removal of stumps, roots, buried logs and other unsuitable material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

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

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

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

TYPICAL BENCHING DETAIL



No Scale

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.

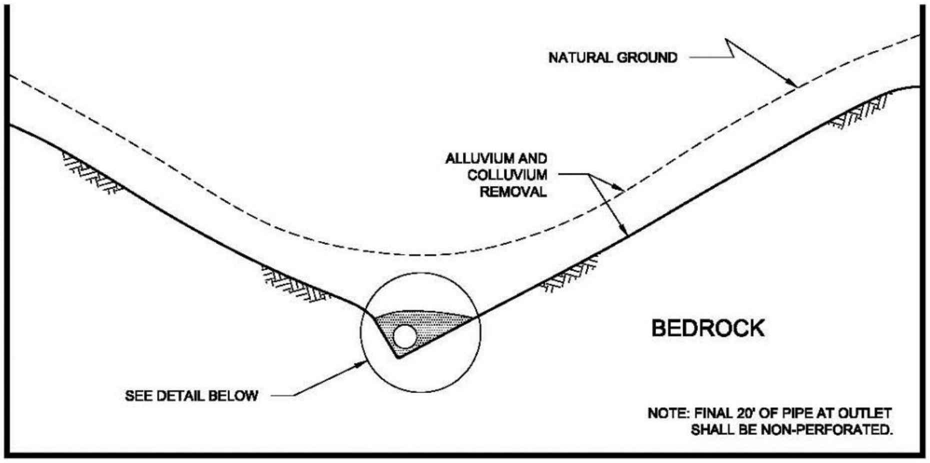
6.3.6 To reduce the potential for "piping" of fines into the *rock* fill from overlying *soil* fill material, a 2-foot layer of graded filter material shall be placed above the uppermost lift of *rock* fill. The need to place graded filter material below the *rock* should be determined by the Consultant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

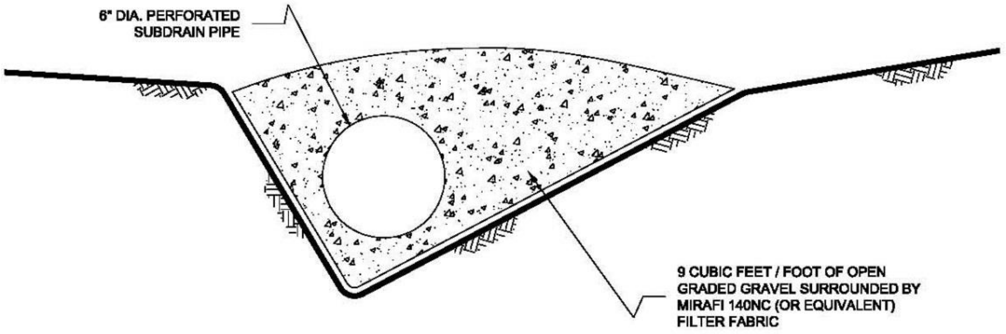
6.3.7 *Rock* fill placement should be continuously observed during placement by the Consultant.

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:

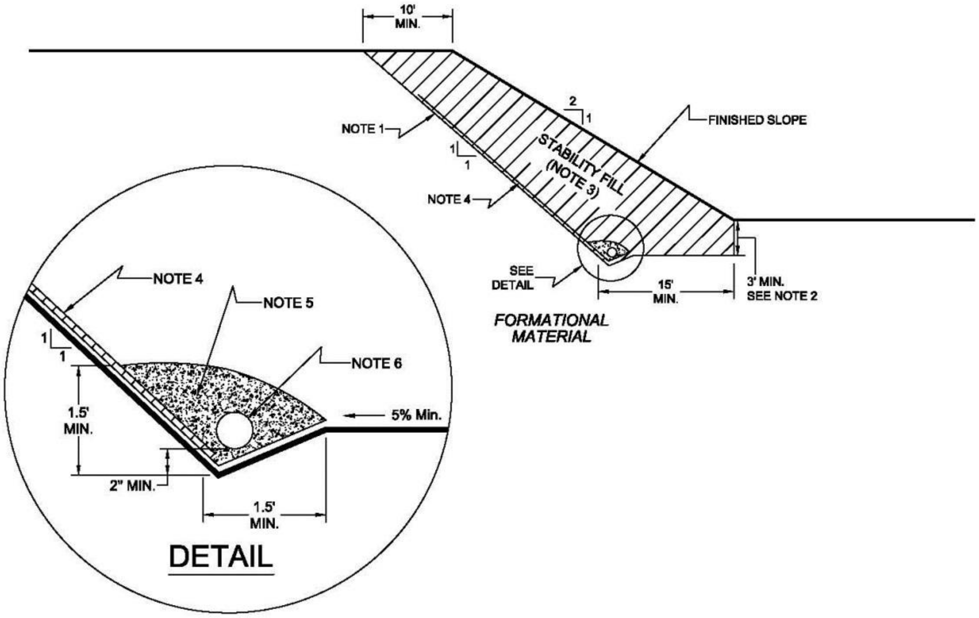
1......6-INCH DIAMETER, SCHEDULE 80 PVC PERFORATED PIPE FOR FILLS IN EXCESS OF 100-FEET IN DEPTH OR A PIPE LENGTH OF LONGER THAN 500 FEET.

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

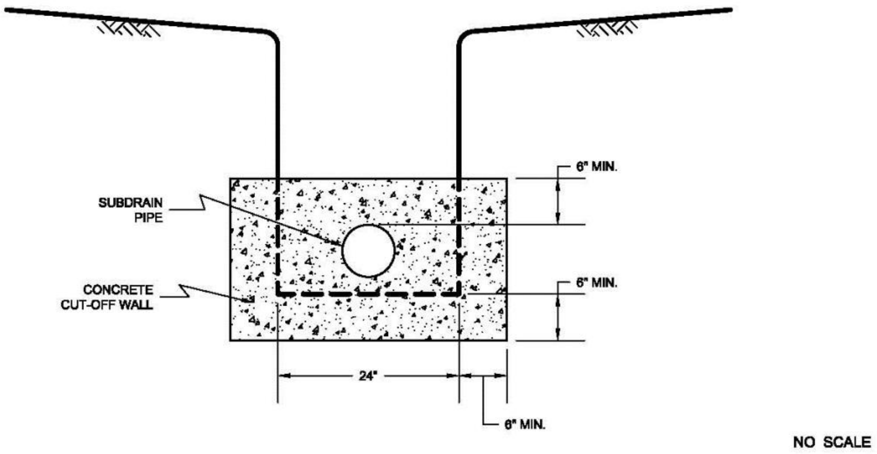
7.3 The actual subdrain locations will be evaluated in the field during the remedial grading operations. Additional drains may be necessary depending on the conditions observed and the requirements of the local regulatory agencies. Appropriate subdrain outlets should be evaluated prior to finalizing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

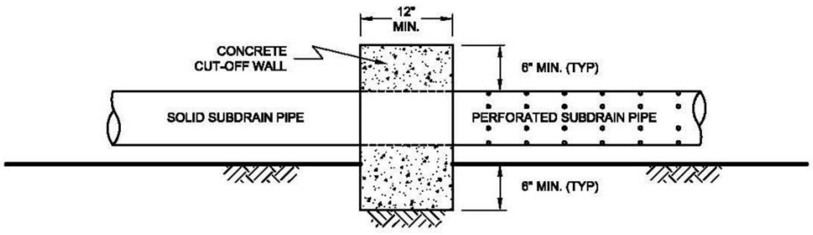
7.5 Prior to outletting, the final 20-foot segment of a subdrain that will not be extended during future development should consist of non-perforated drainpipe. At the non-perforated/ perforated interface, a seepage cutoff wall should be constructed on the downslope side of the pipe.

TYPICAL CUT OFF WALL DETAIL

FRONT VIEW



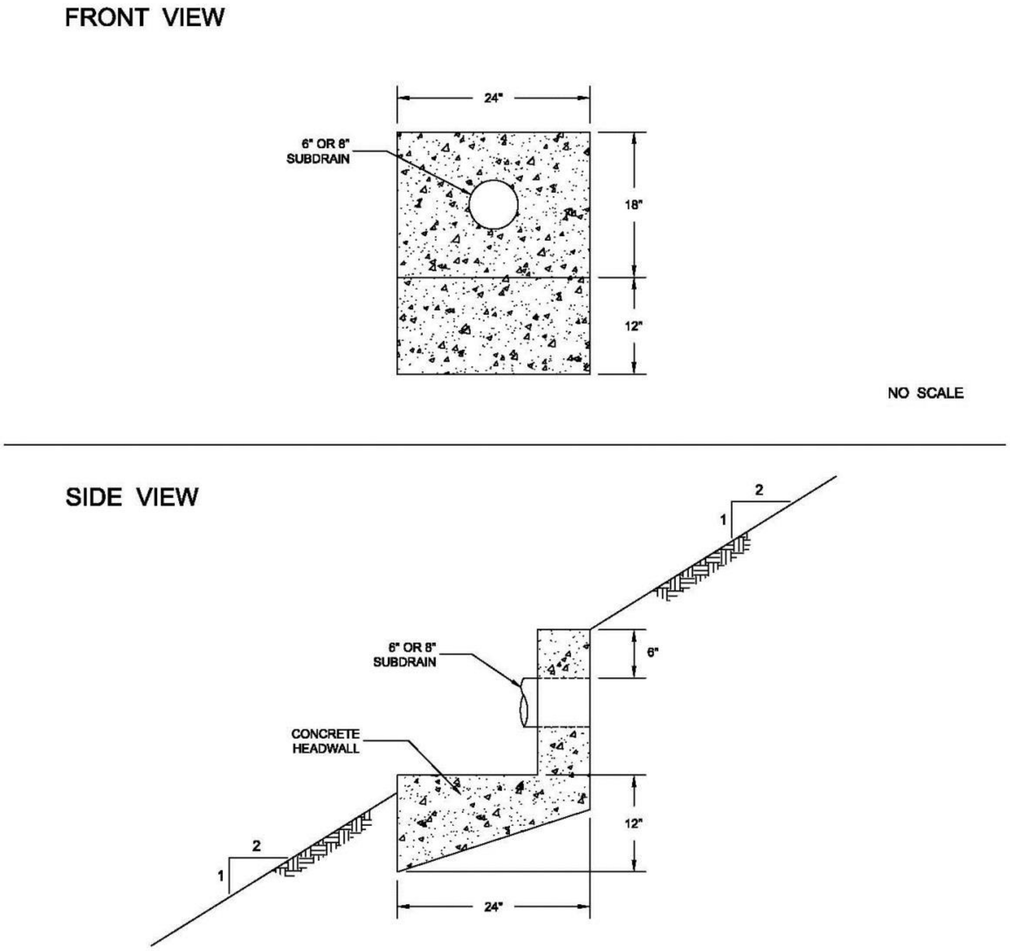
SIDE VIEW



NO SCALE

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

TYPICAL HEADWALL DETAIL



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

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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CGS (2021b), *California Tsunami Maps and Data*, web application for accessing tsunami inundation hazard, https://www.conservation.ca.gov/cgs/tsunami/maps, accessed June 7, 2022.

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Kennedy, M. P., and Tan, S. S., (2008), *Geologic Map of the San Diego 30' x 60' Quadrangle, California*, USGS Regional Geologic Map Series, 1:100,000 Scale, Map No. 3;

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