Appendix C

Preliminary Geotechnical Investigation

PRELIMINARY GEOTECHNICAL INVESTIGATION

LUSK BUSINESS PARK REDEVELOPMENT
SAN DIEGO, CALIFORNIA

GEOCON
INCORPORATED

GEOTECHNICAL
ENVIRONMENTAL
MATERIALS

PREPARED FOR

SAN DIEGO PORTFOLIO 2, LLC
SAN DIEGO, CALIFORNIA

JULY 15, 2022
PROJECT NO. G2896-52-01

Project No. G2896-52-01
July 15, 2022

San Diego Portfolio 2, LLC
9330 Scranton Road, Suite 120
San Diego, California 92121

Attention: Mr. Eric Hotovy

Subject: PRELIMINARY GEOTECHNICAL INVESTIGATION
LUSK BUSINESS PARK REDEVELOPMENT
SAN DIEGO, CALIFORNIA

Dear Mr. Hotovy:

In accordance with your request and authorization of our Proposal No. LG-22031 dated January 19, 2022, we herein submit the results of our preliminary 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 buildings and associated improvements.

The accompanying report presents the results of our study and conclusions and recommendations pertaining to geotechnical aspects of the proposed project. The site is suitable for the proposed buildings and improvements provided the recommendations of this report are incorporated into the design and construction of the planned project. We should update this report once a grading plan has been developed.

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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PRELIMINARY GEOTECHNICAL INVESTIGATION

1. PURPOSE AND SCOPE

This report presents the results of our preliminary geotechnical investigation for the proposed bio-medical research and laboratory office buildings development located in the Sorrento Valley area of San Diego, California (see Vicinity Map).

Vicinity Map

The purpose of the preliminary 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 provided recommendations for remedial grading, shallow foundations, concrete slab-on-grade, concrete flatwork, and retaining walls.

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 13 exploratory borings to a maximum depth of about 41 feet, advanced 6 cone penetrometer tests (CPT) to a maximum depth of 45 feet, sampled soil and performed laboratory testing. Appendix A presents the exploratory boring and CPT data and details of the field investigation. The details of the laboratory tests and a summary of the test results are shown in Appendix B and on the boring logs in Appendix A. Our field investigation for this project was performed concurrently with our investigation for the Barnes Canyon Road Redevelopment project, and as a result, the numbering of our CPTs does not begin at CPT-1. Our storm water management investigation will be provided under separate cover.

2. SITE AND PROJECT DESCRIPTION

The subject property is situated on the south side of Lusk Boulevard in the Sorrento Valley area of San Diego, California. The site comprises the property addresses of 6640, 6650, 6540, 6440, 6450 and 6370 Lusk Boulevard. The property is occupied by six, two-story commercial structures with driveways, surface parking, and accommodating utilities and landscaping. We expect the existing structures are supported on conventional shallow foundations with a concrete slab-on-grade. The property is split into two separate relatively flat pads, with elevations ranging from about 320 to 330 feet above Mean Sea Level (MSL) on the western half and about 340 to 350 feet above Mean Sea Level (MSL) on the eastern half. A fill slope descends along the southern edge of the properties that ranges from about 40 to 70 feet in height. Slope buttresses were constructed at three separate locations along the descending slope due to slope instability concerns of the cut-slope areas during the original site grading in the 1980s. The Existing Site Map shows the current site configuration.

Existing Site Map

We understand the project will consist of demolishing the existing structures and improvements at the site and constructing several bio-medical office and laboratory buildings, parking structures, and other related improvements. We understand that the offices buildings are currently planned to consist of 10 to 15 stories above grade over 1 to 2 levels subterranean and the parking structures will be 7 levels above grade with 2-levels subterranean. The site development will also include new utilities, sidewalks, amenity space, and other associated improvements. Additionally, we understand that the proposed development will connect to the Sorrento Tech Center development which is proposed to the south at the base of the descending slope. Information and recommendations regarding improvements within the slope area will be incorporated into the geotechnical investigation for Sorrento Tech Center.

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 SITE GRADING

The existing parcels located at 6370, 6440, 6450, 6540, 6640 and 6650 Lusk Boulevard are located within what was originally identified as the Lusk Industrial Park development during the original grading and development. Mass grading of the development occurred in the early 1980s which created a sheet-graded pad with fills of up to approximately 65 feet deep. The development originally consisted of a hillside topography with two southward flowing drainage courses on the western and central portions of the site. Prior to mass grading operations, elevations ranged of a low of approximately 260 feet above MSL within the southwestern portion of the site to a high of approximately 350 MSL on the east portion of the site.

The general geologic conditions prior to mass grading consisted of surficial soil composed of topsoil, alluvium and colluvium overlying formational materials of the Scripps Formation. Due to adverse geologic conditions identified during the mass grading operations, the southern descending slope was constructed as a buttress and stability fill within the cut portions of the slope. The constructed buttress geometry and conditions are provided in Table 3 (as reported by Moore & Taber). Information regarding the presence of canyon and/or buttress subdrains within the project area is not provided within the referenced reports.

TABLE 3
EXISTING BUTTRESS CONDITIONS

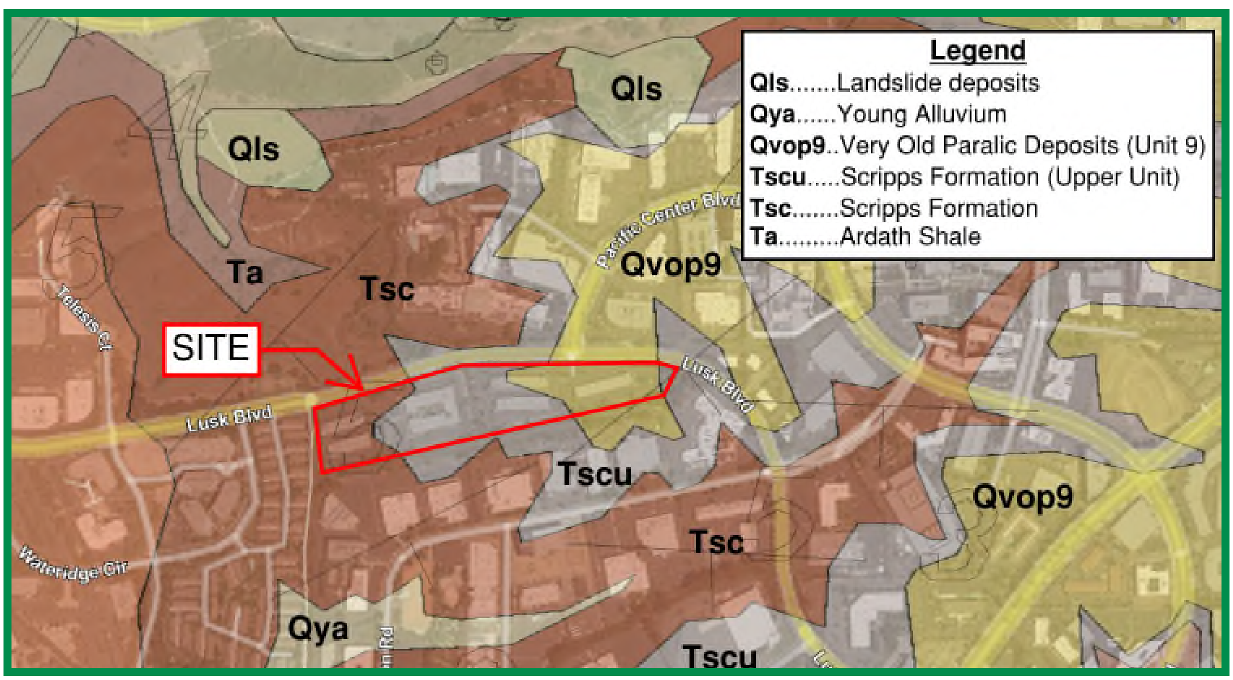
Buttress Location	Observed Adverse Competent of Dip (degrees)	Finished Slope Height (feet)	Constructed Base Width (feet)
Buttress No. 1 - South of 6370 and 6450 Lusk Blvd.	0 – 15	50 – 68	82 – 120
Buttress No. 2 - South of 6540 Lusk Blvd.	0 – 22	46 – 56	35 – 55
Buttress No. 3 - South of 6650 Lusk Blvd.	0 – 15	25	20 - 50

Moore and Taber provided the testing observation services during the grading operations that consisted of performing laboratory and compaction testing. The field density test results indicate the fill soil observed by Moore and Taber was placed at a dry density of at least 90 percent of the laboratory maximum dry density.

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 Map

The site is located on the western portion of the coastal plain. Marine sedimentary units make up the geologic sequence encountered on the site and consist of Eocene aged-Scripps Formation and Ardath Shale. Geologic literature indicates that the site is underlain with Quaternary-aged Very Old Paralic Deposits and Scripps Formation; However, the Very Old Paralic deposits were not encountered during our investigation and were likely removed during original grading operations. The Regional Geologic Map shows the geologic units in the area of the site.

5. SOIL AND GEOLOGIC CONDITIONS

We encountered one surficial soil unit (consisting of previously placed fill) and one formational unit (consisting of Scripps Formation). The occurrence, distribution, and description of each unit encountered is shown on the Geologic Map, Figure 1 and on the boring logs in Appendix A. The Geologic Cross-Sections, Figure 2, 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 herein in order of increasing age.

5.1 Previously Placed Fill (Qpf)

We encountered previously placed fill in our borings to depths ranging from about 2 to 35 feet. However, we expect that fill depths on the order of 60 to 65 feet are present within the deep canyon fill areas on the southwestern portion of the site and within the buttress fill on the southeastern portion of the site. In general, the fill consists of firm to very stiff, moist, clayey to sandy silt and medium dense, moist, clayey to silty sand and possesses a "very low" to "medium" expansion index (expansion index of 90 or less). The upper portions of the previously placed fill is not considered suitable in its current condition for the support of building foundations or structural fill and remedial grading will required. Additionally, we expect that long-term settlement of the deeper fill materials will occur and have provided recommendations for mitigation or accommodation of the estimated settlements herein. The previously placed fill can be reused for new compacted fill during grading operations provided it is generally free of roots and debris.

5.2 Scripps Formation (Tsc)

The Eocene aged Scripps Formation exists below the previously placed fill. This unit consists of predominantly yellowish brown to gray, clayey to sandy siltstone and silty to sandy claystone that is moist to wet and very stiff to hard in consistency. The formational materials possess some localized areas of moderately cemented concretionary beds. The formational materials possesses a "low" to "medium" expansion potential (expansion index of 21 to 90) and possess "S0" to S2" water-soluble sulfate classifications. The formation is generally considered suitable for support of properly compacted structural fill and improvements.

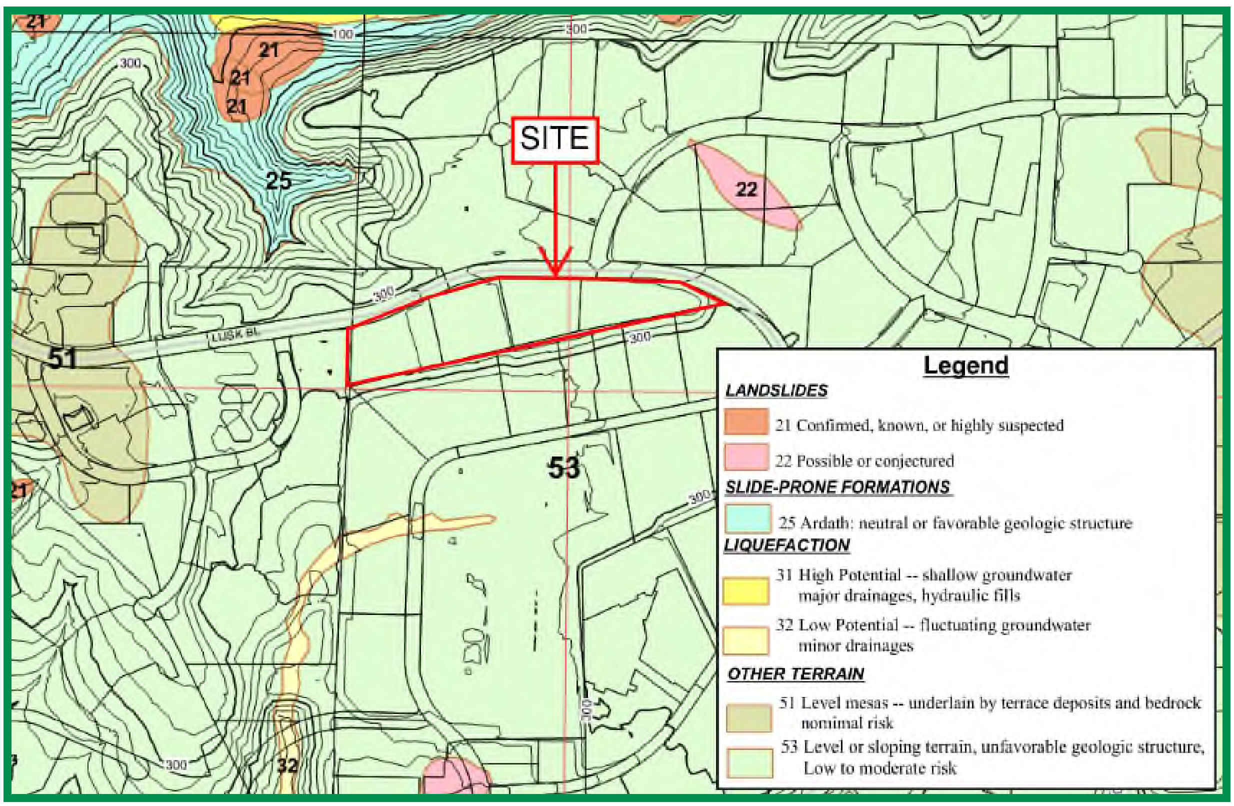
6. GROUNDWATER

We did not encounter groundwater or seepage during our site investigation to the maximum depth explored of 45 feet. However, it is not uncommon for shallow seepage conditions to develop where none previously existed when sites are irrigated or infiltration is implemented. Seepage is dependent on seasonal precipitation, irrigation, land use, among other factors, and varies as a result. Proper surface drainage will be important to future performance of the project. We expect groundwater is deeper than about 100 feet below existing grade. We do not expect groundwater to be encountered during construction of the proposed development.

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 53: level or sloping terrain, unfavorable geologic structure, Low to moderate Risk* (as shown on the Hazard Category Map). Based on a review of the map, a fault does not traverse the planned development area.

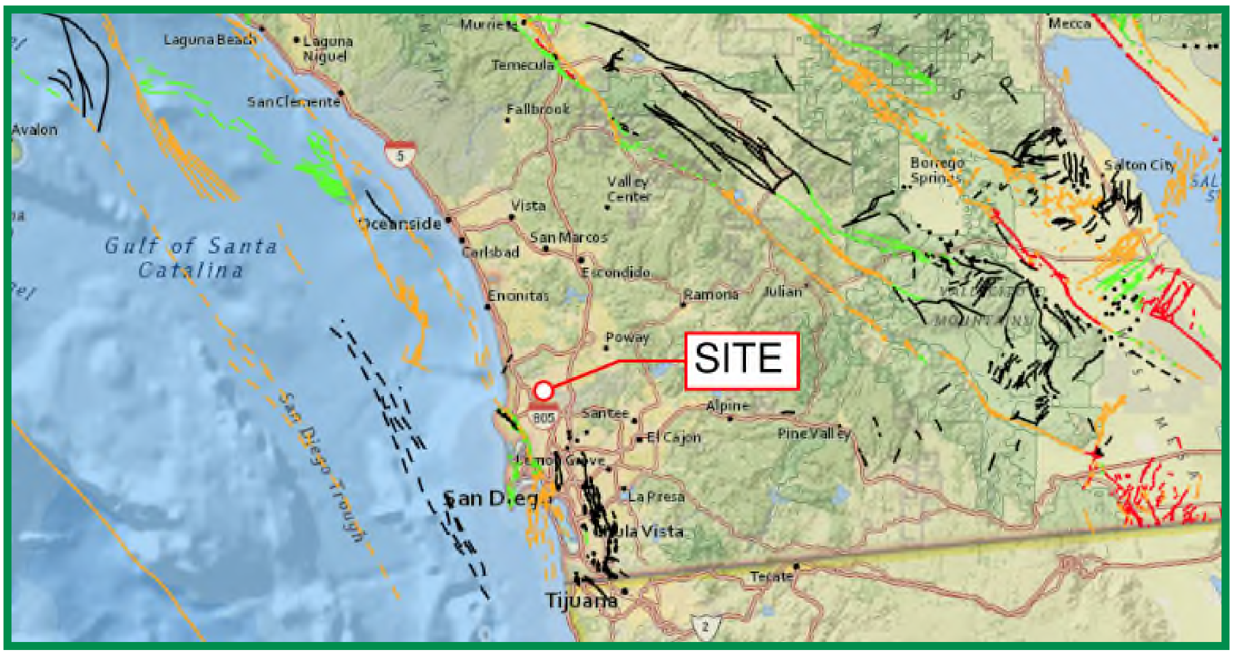


Hazard Category Map

7.2 Faulting and Seismicity

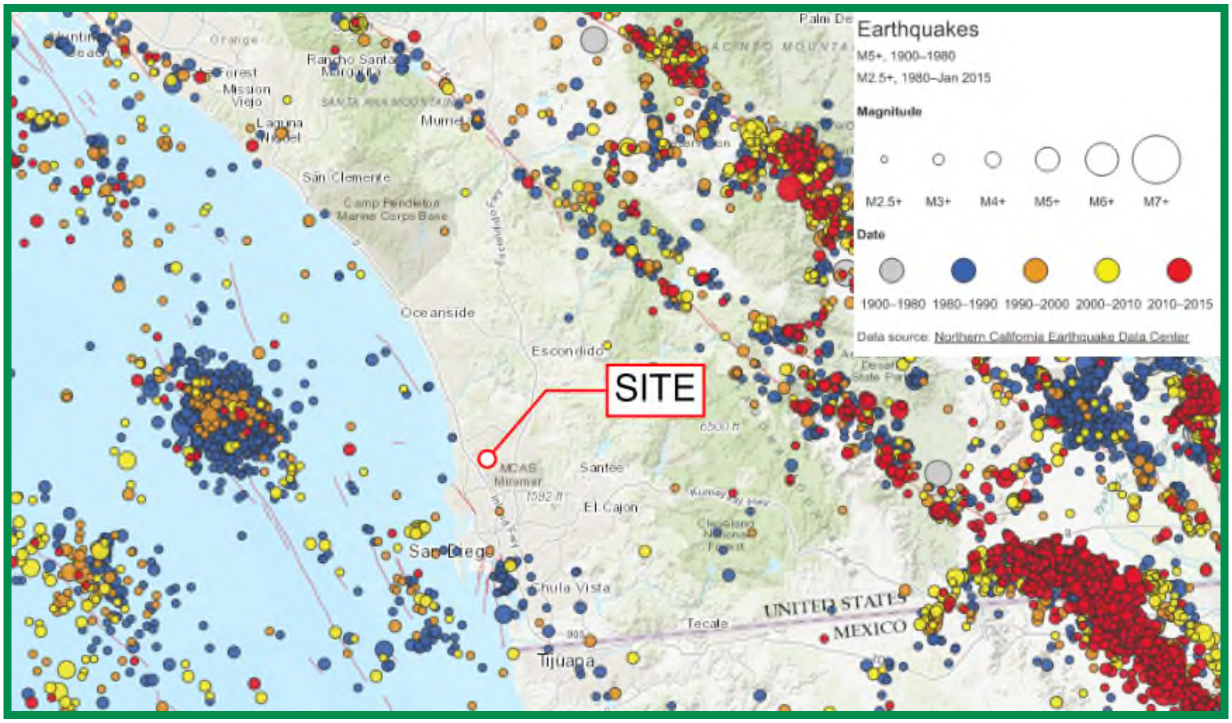
A review of the referenced geologic materials and our knowledge of the general area indicate that the site is not underlain by active, potentially active, or inactive faults. An active fault is defined by the California Geological Survey (CGS) as a fault showing evidence for activity within the last 11,700 years. The site is not located within a State of California Earthquake Fault Zone.

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



Faults in Southern California

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



Earthquakes in Southern California

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

7.3 Ground Rupture

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

7.4 Liquefaction

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

7.5 Tsunamis and Seiches

A tsunami is a series of long-period waves generated in the ocean by a sudden displacement of large volumes of water. The site is located approximately 3 miles from the Pacific Ocean at an elevation of approximately 280 feet Mean Sea Level (MSL). The risk of a tsunami and storm surges affecting the site is considered negligible due to the distance of the site from the ocean and relatively high elevation.

Seiches are standing wave oscillations of an enclosed water body after the original driving force has dissipated. Driving forces are typically caused by seismic ground shaking. The site is not located near a body of water; therefore, the risk of a seiche affecting the site is considered negligible.

7.6 Landslides

We did not observe evidence of previous or incipient slope instability at the site during our study. Published geologic mapping indicates potential landslide in north-facing descending natural slopes situated to the northeast of the subject site but are not present on the site. Therefore, we opine the potential for a landslide is not a significant concern for this project.

7.7 Slope Stability

The largest slope at the subject site is the approximately 50- to 70-foot high ascending slope on the southern side of the property that is comprised of previously placed fill used as a buttress (identified herein as Buttress No. 1) due to unfavorable geologic bedding of the native materials. The depth of fill along the buttressed slope is unknown but the approximate base width of the current descending fill slope is reported to range between 82 and 120 feet into the slope.

We performed slope stability analyses using the two-dimensional computer program GeoStudio created by Geo-Slope International Ltd. We calculated the factor of safety for the existing slopes for rotational-mode and block-mode analyses using the Spencer's method. Output of the computer program including the calculated factor of safety and the failure surface is presented in Appendix C.

We used average drained direct shear strength parameters based on laboratory tests and our experience with similar soil types in nearby areas for the slope stability analyses. Our calculations indicate the existing slopes possess calculated factors of safety (FOS) of at least 1.5 under static conditions, for both deep-seated failure and shallow sloughing conditions. We performed the slope stability analyses for 82- and 120-foot wide keyway buttress fill for the largest slope on the property. We should provide additional slope stability analyses for the proposed site grading and associated slope reconfigurations once plans are prepared.

Table 7.7 presents the surficial slope stability analysis for a the proposed sloping conditions.

TABLE 7.7
SURFICIAL SLOPE STABILITY EVALUATION

Parameter	Value
Slope Height, H	∞
Vertical Depth of Saturation, Z	5 Feet
Slope Inclination, I (Horizontal to Vertical)	1.5:1 (33.7 Degrees)
Total Soil Unit Weight, γ	125 pcf
Water Unit Weight, γ_W	62.4 pcf
Friction Angle, ϕ	28 Degrees
Cohesion, C	400 psf
Factor of Safety = $(C+(\gamma+\gamma_W)Z\cos^2 I \tan\phi)/(\gamma Z \sin I \cos I)$	1.8

We performed the slope stability analyses based on the interpretation of geologic conditions encountered during our field investigation and the results of our laboratory testing. The soil strength parameters used for the bedding plane shear material in our analyses is based on the original modeling and laboratory testing performed by Moore & Taber during construction of the site buttresses. We should evaluate the geologic conditions during the grading operations to check if the conditions observed during grading are consistent with our interpretations. Additional slope stability analyses and modifications to the proposed buttresses may be required during the grading operations.

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

7.8 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.

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 should update this report once a development plan has been prepared; therefore, the recommendations prepared herein should be considered preliminary.

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 Our field investigation indicates the site is underlain with previously placed fill overlying Tertiary-age Scripps Formation. The upper portions of the previously placed fill is potentially compressible and unsuitable in their present condition for the support of additional compacted fill or settlement-sensitive improvements. The formational materials considered 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. Seepage within surficial soils and formational materials may be encountered during the grading operations, especially during the rainy seasons.

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

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 We opine the planned development can be constructed in accordance with our recommendations provided herein. We do not expect the planned development will destabilize or result in settlement of adjacent properties if properly constructed.

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

8.2 Excavation and Soil Characteristics

8.2.1 Excavation of the in-situ soil should be possible with moderate to heavy effort using conventional heavy-duty equipment. Excavation of the formational materials will require very heavy effort and may generate oversized material using conventional heavy-duty equipment during the grading operations.

8.2.2 The soil encountered in the field investigation is considered to be "expansive" (expansion index [EI] of greater than 20) as defined by 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" to "S2" sulfate exposure to concrete structures as defined by 2019 CBC Section 1904 and ACI 318-14 Chapter 19. We expect that the majority of the existing fill materials and formational materials likely possesses a S2 sulfate exposure, and that concrete structures in contact with these materials should be designed accordingly. 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 potential of hydrogen (pH) and resistivity laboratory 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 Grading

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

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

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

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

8.3.5 We understand the site development will consist of several large R&D laboratory buildings situated either at existing surface elevations or over subterranean levels, along with ancillary structures and other associated site improvements. The majority of the site is underlain by relatively shallow previously placed fill ranging from about 5 to 10 feet in thickness, with an exception for deeper fills ranging from 10 to 50 feet associated with previous canyon fills and slope buttresses as shown on the Geologic Map, Figure 1. We expect that the proposed structures will supported by either a shallow foundation system bearing entirely in formational materials, a shallow foundation system bearing entirely in compacted fill materials, or a deep foundation system bearing in formational materials. The design team will need to determine the foundation systems for each building based on the allowable differential settlement tolerances and bearing loads determined by the project structural engineer once the grading plan is available.

8.3.6 **Shallow Foundation – Fill Option:** We expect that some of the proposed structures will be supported on a shallow foundation system bearing entirely in compacted fill materials. For portions of the building pad, the upper 3 feet of the fill materials below pad grade or 2 feet below bottom of foundations (whichever results in a deeper excavation) should be excavated and compacted fill placed. The excavations should extend at least 10 feet outside the perimeter of the proposed buildings, where possible.

8.3.7 **Shallow Foundation – Formation Option:** We expect that settlement-sensitive structures will be supported on a shallow foundation system bearing entirely in formational materials where present near proposed pad elevations. The excavations can be performed to the proposed pad elevation and additional excavations will not be required where formational materials are exposed. Where fill is encountered at pad grade, the upper 3 feet below proposed pad grades should be excavated, moisture conditioned as necessary and recompacted but the removals should be limited to the formational materials. Isolated sections of the building may require deepening of the shallow foundations to expose formational material where deeper fills are present.

8.3.8 **Deep Foundation Option:** In areas where deep foundations will be used due to deep fills, the upper 3 feet of the surficial materials below proposed pad grades should be excavated, moisture conditioned as necessary and recompacted. The grading should be limited to the formational materials, where encountered.

8.3.9 In areas of proposed improvements outside of the building areas (including ancillary structures), the upper 2 to 3 feet of existing soil should be processed, moisture conditioned as necessary and recompacted. Deeper removals may be required in areas where loose or saturated materials are encountered. The removals should extend at least 2 feet outside of the improvement area, where possible. Table 8.3.1 provides a summary of the grading recommendations.

TABLE 8.3.1
SUMMARY OF GRADING RECOMMENDATIONS

Area	Remedial Excavation Recommendations
Shallow Foundation – Fill Option	Excavate Upper 3 Feet Below Finish Grade and 2 Feet Below Foundations
Shallow Foundation – Formation Option	Excavate to Pad Elevation and Expose Formational Materials
	Excavate Upper 3 Feet Below Pad Grade and Limit to Formational Materials
Building Pads – Deep Foundation Option	Excavate Upper 3 Feet of Surficial Materials Below Proposed Pad Grade and Place Compacted Fill
	Limit Excavations to Formational Materials
Site Development and Ancillary Structures	Process Upper 2 to 3 Feet of Existing Materials
Lateral Grading Limits	10 Feet Outside of Buildings
	2 Feet Outside of Improvement Areas
Exposed Bottoms of Remedial Grading	Scarify Upper 12 Inches

8.3.10 The bottom of the excavations should be sloped 1 percent to the adjacent street or deepest fill. Prior to fill soil being placed, the existing ground surface should be scarified, moisture conditioned as necessary, and compacted to a depth of at least 12 inches. Deeper removals may be required if saturated or loose fill soil is encountered. A representative of Geocon should be on-site during removals to evaluate the limits of the remedial grading.

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

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

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

TABLE 8.3.2
SUMMARY OF IMPORT FILL RECOMMENDATIONS

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

8.4 Excavation Slopes, Shoring and Tiebacks

8.4.1 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.

8.4.2 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.

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

8.4.4 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.

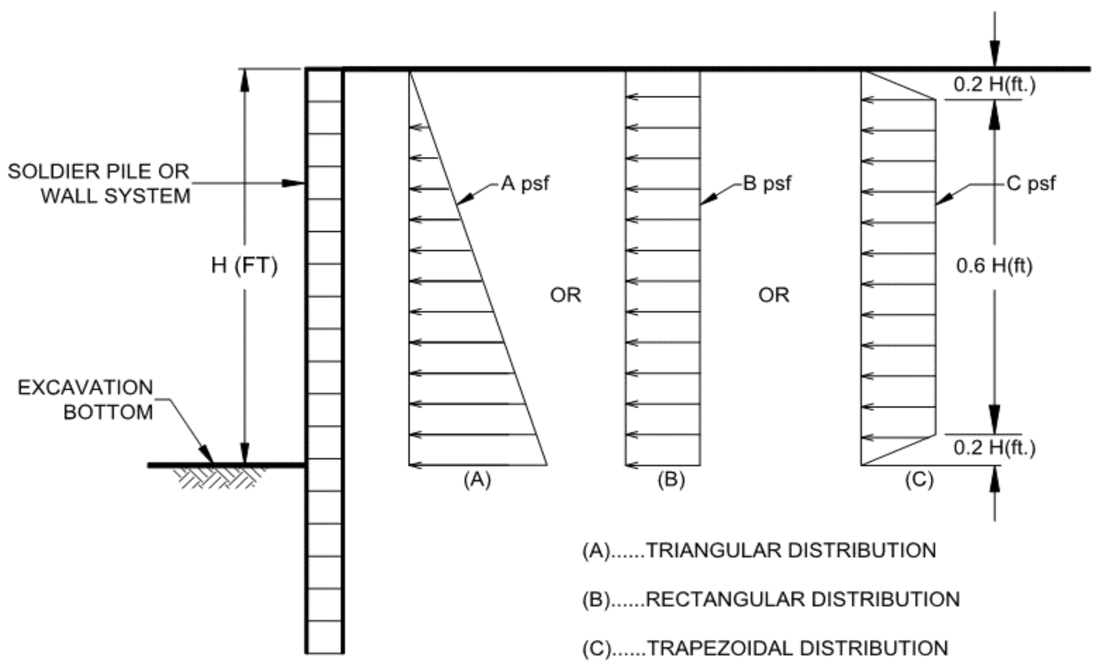
8.4.5 In general, ground conditions are moderately suited for soldier pile and tieback anchor wall construction techniques. However, gravel, cobble, and oversized material may be encountered in the existing materials that could be difficult to drill. Additionally, if cohesionless sands are encountered,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

8.4.6 Temporary shoring should be designed using a lateral pressure envelope acting on the back of the shoring as presented in Table 8.4.1 assuming a level backfill and that the excavations of primarily situated partially within fill materials (upper 10 to 20 feet) and formational materials beneath. 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 8.4.1
SUMMARY OF TEMPORARY SHORING WALL RECOMMENDATIONS

Parameter	Value		
Maximum Design Retained Height, H	20 feet	30 feet	40 feet
Triangular Distribution, A	22H psf	25H psf	28H psf
Rectangular Distribution, B	14H psf	16H psf	18H psf
Trapezoidal Distribution, C	17H psf	20H psf	22H psf
Passive Pressure, P	350D + 500 psf		
Effective Zone Angle, E	30 degrees		
Maximum Design Lateral Movement	1 Inch		
Maximum Design Vertical Movement	½ Inch		

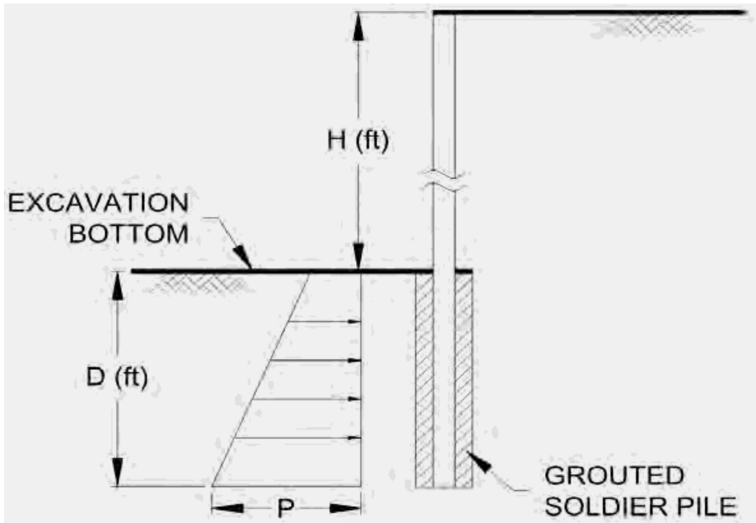
H equals the height of the retaining portion of the wall in feet
D equals the embedment depth of the retaining wall in feet



Active Pressures on Temporary Shoring

8.4.7 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.

8.4.8 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.



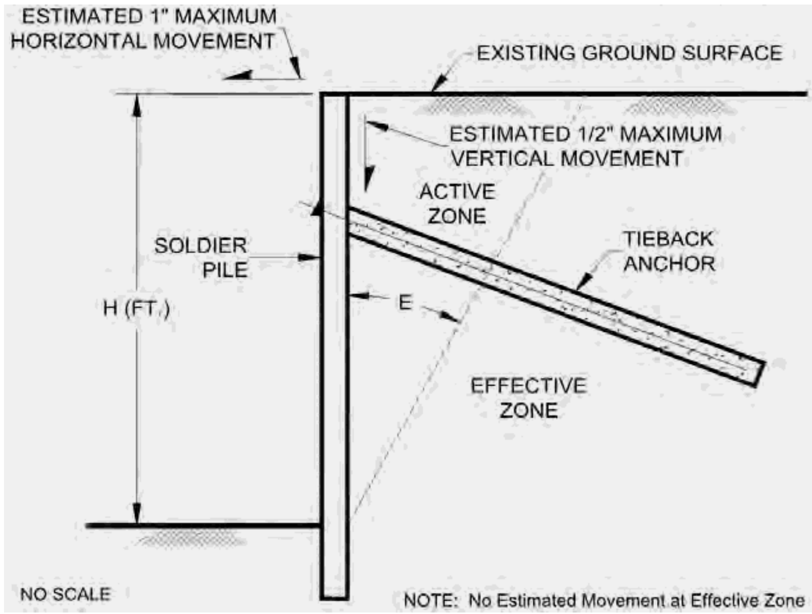
Passive Pressures on Temporary Shoring

8.4.9 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.

8.4.10 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.

8.4.11 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.

8.4.12 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

8.4.13 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.

8.4.14 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 8.4.2.

TABLE 8.4.2
SOIL STRENGTH PARAMETERS FOR TEMPORARY SHORING

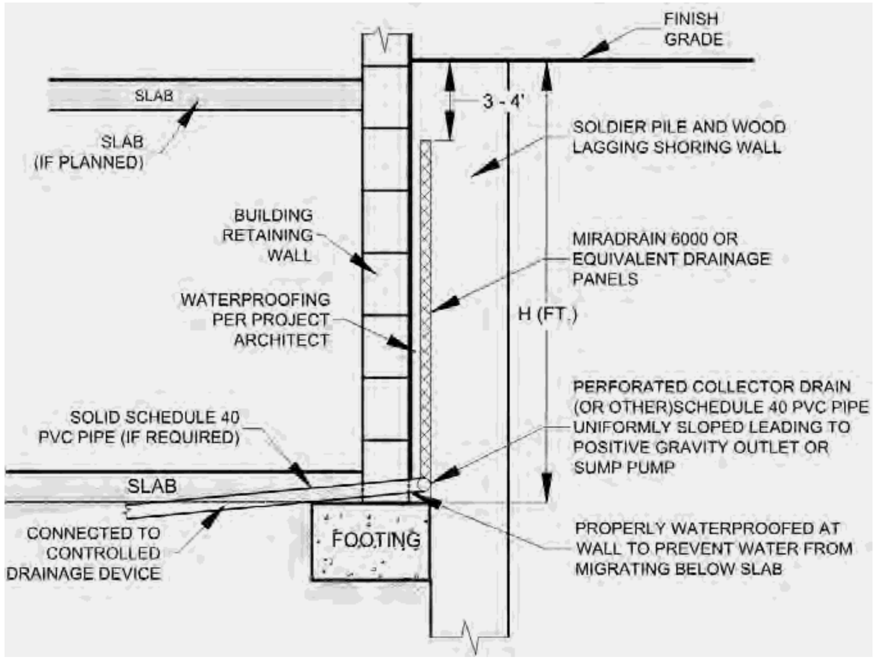
Description	Unit Weight (pcf)	Cohesion (psf)	Friction Angle (Degrees)
Previously Placed Fill & Compacted Fill (Qpf/Qcf)	125	400	28
Scripps Formation (Tsc)	125	500	32

8.4.15 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.

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

8.4.17 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.

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



Shoring Retaining Wall Drainage Detail

8.5 Soil Nail Wall

8.5.1 A soil nail wall can be used as an alternative to temporary shoring followed by construction of a permanent basement wall. 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.

8.5.2 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.

8.5.3 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).

8.5.4 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.

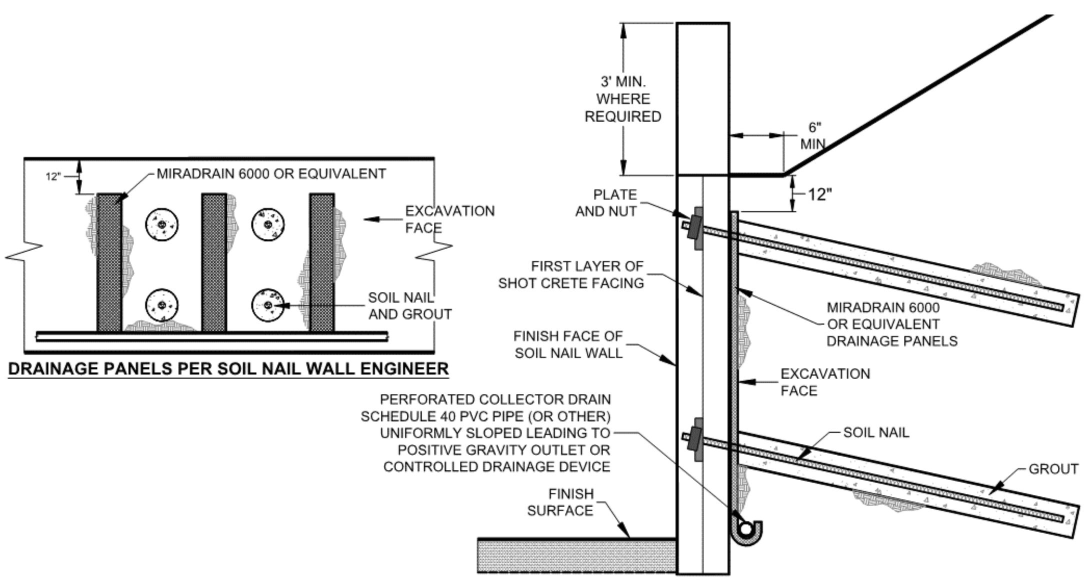
8.5.5 The soil strength parameters listed in Table 8.5.1 can be used in design of the soil nails. The bond stress is dependent on drilling method, diameter, and construction method. Therefore, the designer should evaluate the bond stress based on the existing soil conditions and the construction method.

TABLE 8.5.1
SOIL STRENGTH PARAMETERS FOR SOIL NAIL WALLS

Description	Cohesion (psf)	Friction Angle (degrees)	Estimated Ultimate Bond Stress (psi)*
Previously Placed Fill & Compacted Fill (Qpf/Qcf)	400	28	10
Scripps Formation (Tsc)	500	32	20

*Assuming gravity fed, open hole drilling techniques.

8.5.6 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

8.6 Seismic Design Criteria – 2019 California Building Code

8.6.1 Table 8.6.1 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 8.6.1
2019 CBC SEISMIC DESIGN PARAMETERS

Parameter	Value		2019 CBC Reference
Site Class	C	D	Section 1613.2.2
Thickness of Fill Materials Below Pad	< 20 feet	≥ 20 feet	---
MCE_R Ground Motion Spectral Response Acceleration – Class B (short), S_S	1.040g	1.040g	Figure 1613.2.1(1)
MCE_R Ground Motion Spectral Response Acceleration – Class B (1 sec), S_1	0.370g	0.370g	Figure 1613.2.1(2)
Site Coefficient, F_A	1.200	1.200	Table 1613.2.3(1)
Site Coefficient, F_V	1.500*	1.930*	Table 1613.2.3(2)
Site Class Modified MCE_R Spectral Response Acceleration (short), S_{MS}	1.248g	1.248g	Section 1613.2.3 (Eqn 16-36)
Site Class Modified MCE_R Spectral Response Acceleration – (1 sec), S_{M1}	0.555g*	0.714g*	Section 1613.2.3 (Eqn 16-37)
5% Damped Design Spectral Response Acceleration (short), S_{DS}	0.832g	0.832g	Section 1613.2.4 (Eqn 16-38)
5% Damped Design Spectral Response Acceleration (1 sec), S_{D1}	0.370g*	0.476g*	Section 1613.2.4 (Eqn 16-39)

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

8.6.2 Table 8.6.2 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 8.6.2
ASCE 7-16 PEAK GROUND ACCELERATION

Parameter	Value		ASCE 7 16 Reference
Site Class	C	D	Section 1613.2.2
Mapped MCE_G Peak Ground Acceleration, PGA	0.460g	0.460g	Figure 22-9
Site Coefficient, F_{PGA}	1.200	1.200	Table 11.8-1
Site Class Modified MCE_G Peak Ground Acceleration, PGA_M	0.552g	0.552g	Section 11.8.3 (Eqn 11.8-1)

8.6.3 Conformance to the criteria in Tables 8.6.1 and 8.6.2 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.

8.7 Settlement Due to Fill Loads

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

8.7.2 We expect that the proposed buildings could be underlain by a maximum fill thickness on the order of 50 feet. The settlement of compacted fill is expected to continue over a relatively extended time period resulting from both gravity loading and hydrocompression upon wetting from rainfall and/or landscape irrigation. The previously placed fill has existed for approximately 40 years; therefore, a majority of the expected settlement has likely occurred.

8.7.3 Due to the variable fill thickness, a potential for differential settlement across the proposed buildings exist and special foundation design consideration as discussed herein will be necessary. Based on measured settlement of similar fill depths on other sites and the time period since the fill was placed, we estimate that maximum settlement of the compacted fill will be approximately 0.1 percent for the existing and proposed compacted fills. The Geologic Map, Figure 1, provides the approximate elevation of the base of the fill as measured in our geotechnical borings.

8.7.4 Table 8.7 presents the estimated total fill thickness and associated settlements for the site soils. We will provide an updated table for each proposed building including differential fill thicknesses and settlements once the grading plans are available for our review. These settlement magnitudes should be considered in design of the foundation system and adjacent flatwork that connects to the proposed buildings.

TABLE 8.7
EXPECTED SETTLEMENT OF FILL SOIL

Maximum Depth of Fill (Feet)	Estimated Maximum Settlement (Inches)
10	0.2
20	0.3
30	0.4
40	0.5
50	0.6

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

8.8 Shallow Foundations

8.8.1 The proposed structures can be supported on a shallow foundation system founded in properly compacted fill (assuming the structures can tolerate the fill-related settlement provided herein) or formational materials. Foundations for the structures should consist of continuous strip footings and/or isolated spread footings. Footings should be deepened such that the bottom outside edge of the footing is at least 7 feet horizontally from the face of the slope. Table 8.8 provides a summary of the foundation design recommendations.

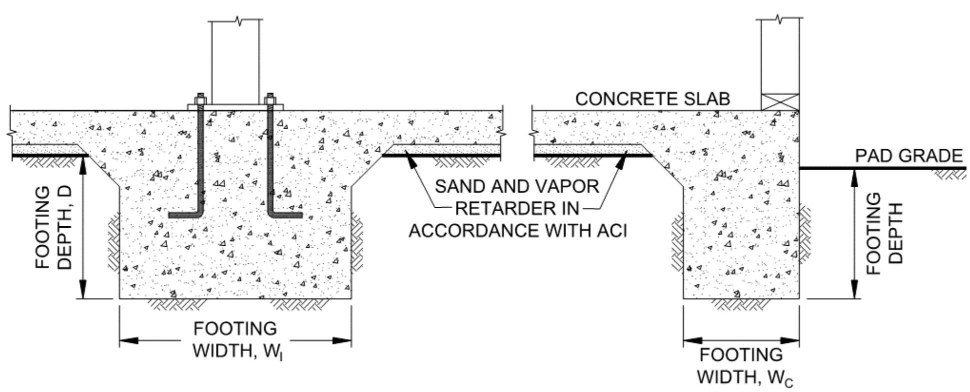
TABLE 8.8
SUMMARY OF FOUNDATION RECOMMENDATIONS

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 *	2,500 psf – Compacted Fill
	6,000 psf – Formation
Bearing Capacity Increase	300 psf per Foot of Depth or Width
Maximum Allowable Bearing *	5,000 psf – Compacted Fill
	10,000 psf – Formation
Estimated Total Settlement	1 Inch
Estimated Differential Settlement	½ Inch in 40 Feet
Footing Size Used for Settlement	10-Foot Square
Design Expansion Index	90 or less

* Allowable bearing capacity increase of 1,200 psf can be used for each additional subterranean level (assumed to be at least 10 feet below grade per level).

8.8.2 The total differential settlement of the proposed structures should include both the building-load related settlements provided in Table 8.8 and the estimated fill-load related settlements provided in Table 8.7.

8.8.3 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

8.8.4 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.

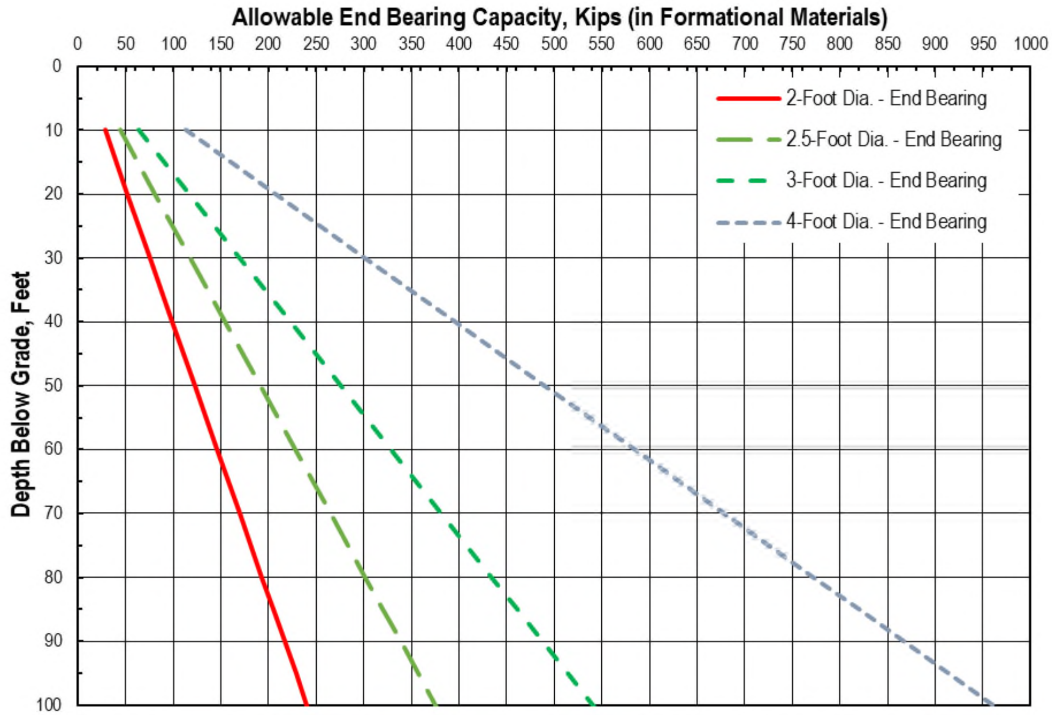
8.8.5 We should observe the foundation excavations prior to the placement of reinforcing steel and concrete to check that the exposed soil conditions are similar to those expected and that they have been extended to the appropriate bearing strata. Foundation modifications may be required if unexpected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

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

8.9 Drilled Pier Recommendations

8.9.1 We understand that drilled piers may be used for foundation support within areas with existing or proposed fill materials present at proposed grades. The foundation recommendations herein assume that the piers will extend through the fill into the underlying formational materials. The piers should be embedded at least 10 feet long and at least 5 feet within the formational materials.

8.9.2 Piers can be designed to develop support by end bearing within the formational materials and skin friction within the formational materials and portions of the fill soil. The end bearing capacity can be determined by the End Bearing Capacity Chart. These allowable values possess a factor of safety of at least 2 for skin friction and end bearing.



End Bearing Capacity Chart

8.9.3 Piers can be designed to develop support by end bearing within the formational materials and skin friction within the formational materials and portions of the fill soil using the design parameters presented in Table 8.9.

TABLE 8.9
SUMMARY OF DRILLED PIER RECOMMENDATIONS

Parameter	Value
Minimum Pile Diameter	2 Feet
Minimum Pile Spacing	3 Times Pile Diameter
Minimum Foundation Embedment Depth	5 Feet in Formational Materials
Allowable End Bearing Capacity	Per Chart
Allowable Skin Friction Capacity	250 psf – Fill Materials
	1,000 psf – Formational Materials
Estimated Total Settlement	½ Inch
Estimated Differential Settlement	½ Inch in 40 Feet

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

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

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

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

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

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

8.10 Concrete Slabs-On-Grade

8.10.1 Concrete slabs-on-grade for the structures 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. 4 Bars 18 Inches on Center, Both Directions
Typical Slab Underlayment	3 to 4 Inches of Sand/Gravel/Base
Design Expansion Index	90 or less

8.10.2 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 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 Some projects remove the sand layer below the slab in parking structure areas. This is acceptable from a geotechnical engineering standpoint; however, relatively minor cracks could form due to differential curing. Therefore, the structural engineer and/or the concrete contractor should provide recommendations for proper curing techniques to help prevent cracking.

8.10.5 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.

8.10.6 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.

8.10.7 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.

8.10.8 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 ≤ 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

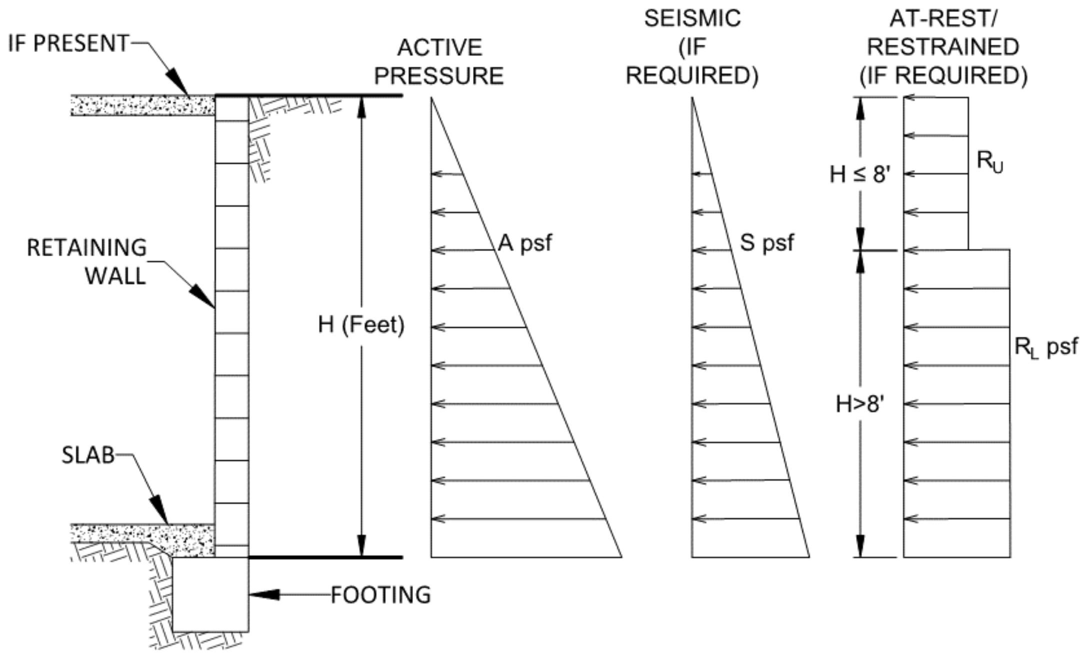
8.12.1 Retaining walls should be designed using the values presented in Table 8.12.1. Soil with an expansion index (EI) of greater than 90 should not be used as backfill material behind retaining walls.

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.



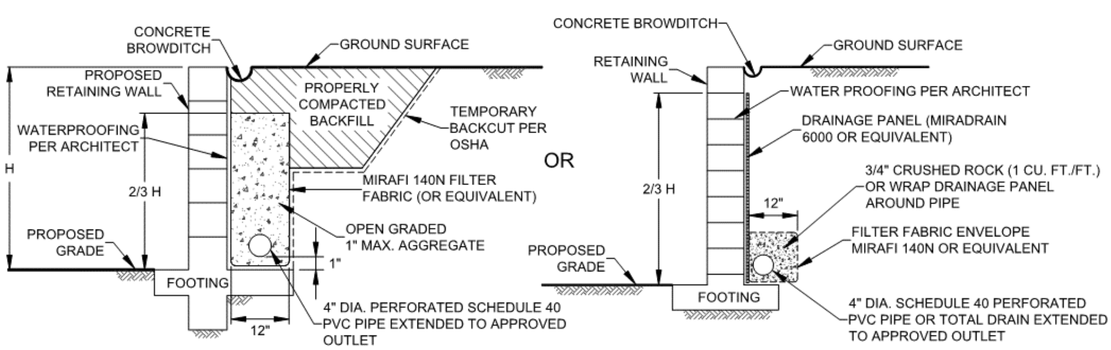
Retaining Wall Loading Diagram

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.

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 7-10. For structures assigned to Seismic Design Category of D, E, or F, retaining walls that support more than 6 feet of backfill should be designed with seismic lateral pressure in accordance with Section 1803.5.12 of the 2019 CBC. The seismic load is dependent on the retained height where H is the height of the wall, in feet, and the calculated loads result in pounds per square foot (psf) exerted at the base of the wall and zero at the top of the wall.

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.



Typical Retaining Wall Drainage Detail

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

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
Coefficient of Friction (Concrete and Soil)	0.3
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 assumed an R-Value of 10 and 78 for the subgrade soil and base materials, respectively, for the purposes of this preliminary analysis. 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	10	3	9
Driveways for Automobiles and Light-Duty Vehicles	5.5	10	3	11
Medium Truck Traffic Areas	6.0	10	3½	12
Driveways for Heavy Truck Traffic	7.0	10	4	15

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 The base thickness can be reduced if a reinforcement geogrid is used during the installation of the pavement. Geocon should be contact for additional recommendations if alternate design parameters are requested.

8.14.5 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
D	Heavy Duty Trucks (80-Kip Gross Weight)	75	15
E	Garbage or Fire Truck Lane	75	15

8.14.6 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	50 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

8.14.7 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	6
B = Entrance and Truck Service Lanes	10	6½
	50	7
D = Heavy Duty Trucks	50	7½
E = Garbage or Fire Truck Lanes	5	7½
	10	7½

8.14.8 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.

8.14.9 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
$5 \leq T < 6$	12.5
$6 \leq T$	15

8.14.10 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

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

8.14.12 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.

8.14.13 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.

8.14.14 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 Site Drainage and Moisture Protection

8.15.1 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.

8.15.2 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.

8.15.3 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.

8.15.4 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.

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

8.16 Grading and Foundation Plan Review

8.16.1 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.

8.17 Testing and Observation Services During Construction

8.17.1 Geocon Incorporated should provide geotechnical testing and observation services during the grading operations, foundation construction, utility installation, retaining wall backfill and pavement installation. Table 8.17 presents the typical geotechnical observations we would expect for the proposed improvements.

TABLE 8.17
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	Drilling Operations for Piles	Full Time
	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

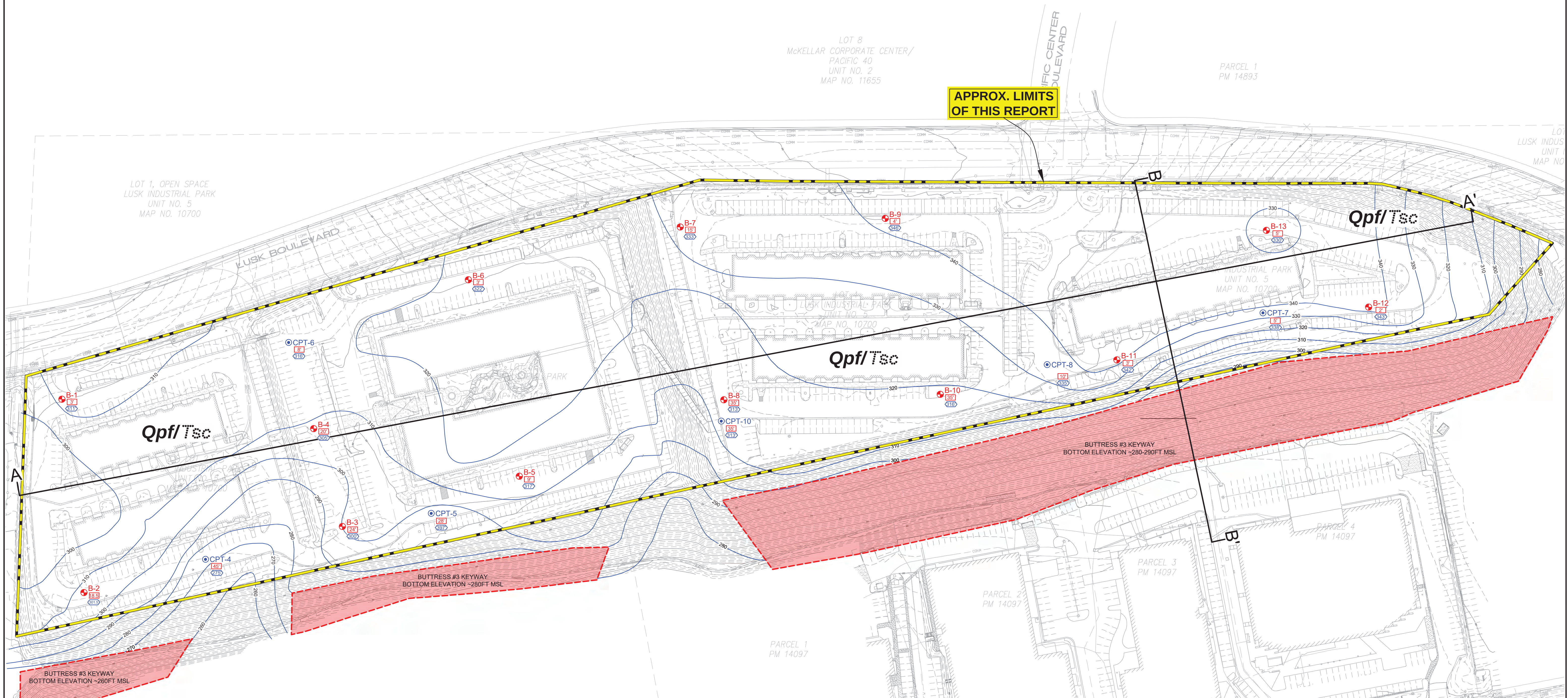
LIMITATIONS AND UNIFORMITY OF CONDITIONS

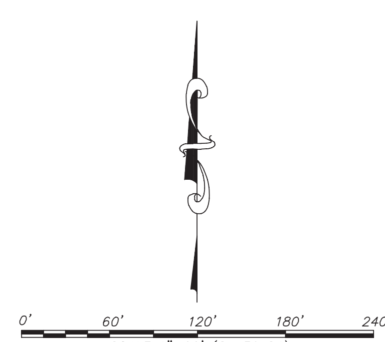
1. The firm that performed the geotechnical investigation for the project should be retained to provide testing and observation services during construction to provide continuity of geotechnical interpretation and to check that the recommendations presented for geotechnical aspects of site development 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

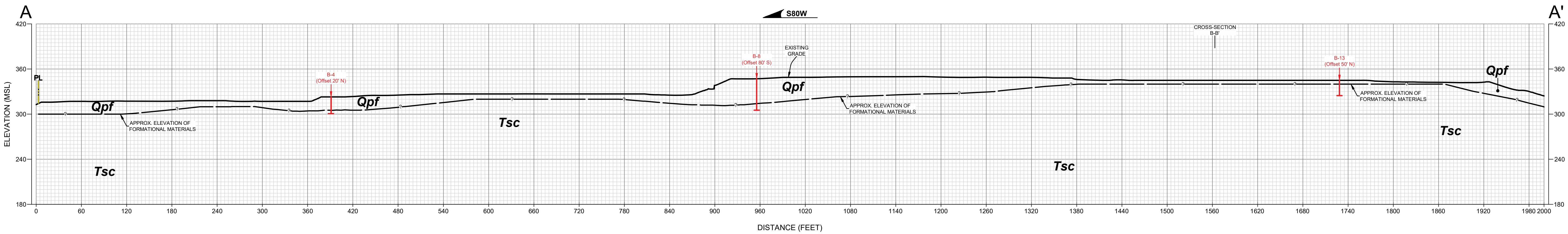
- **Qpf** ....PREVIOUSLY PLACED FILL
- **Tsc** ....SCRIPPS FORMATION (Dotted Where Buried)
- B-18 ....APPROX. LOCATION OF BORING
- CPT-10 ....APPROX. LOCATION OF CONE PENETROMETER TEST
- 70' ....APPROX. DEPTH TO FORMATION
- 275' ....APPROX. ELEVATION OF FORMATION
- 350 ....APPROX. ELEVATION OF FORMATIONAL MATERIALS
- B B' ....APPROX. LOCATION OF GEOLOGIC CROSS-SECTION
- ....APPROX. LIMITS OF BUTRESS KEYWAY

GEOLOGIC MAP

LUSK BUSINESS PARK REDEVELOPMENT
SAN DIEGO, CALIFORNIA

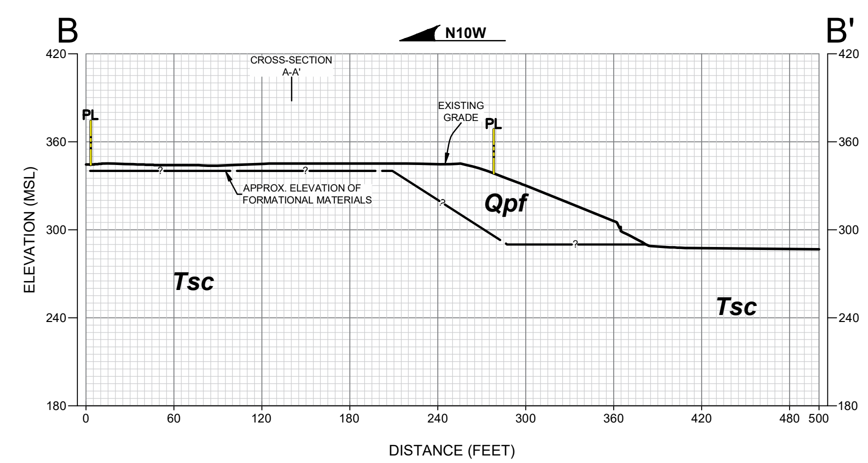
GEOCON INCORPORATED
GEOTECHNICAL ■ ENVIRONMENTAL ■ MATERIALS
6960 FLANDERS DRIVE - SAN DIEGO, CALIFORNIA 92121 - 2974
PHONE 858 558-6900 - FAX 858 558-6159

SCALE	1" = 60'
DATE	07 - 15 - 2022
PROJECT NO.	G2896 - 52 - 01
FIGURE	1
SHEET	1 OF 1



GEOLOGIC CROSS-SECTION A-A'

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



GEOLOGIC CROSS-SECTION B-B'

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

GEOCON LEGEND

Qpf ....PREVIOUSLY PLACED FILL

Tsc ....SCRIPPS FORMATION

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

B-13 ....APPROX. LOCATION OF BORING

GEOLOGIC CROSS - SECTIONS

LUSK BUSINESS PARK REDEVELOPMENT
SAN DIEGO, CALIFORNIA

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

SCALE 1" = 60'	DATE 07 - 15 - 2022
PROJECT NO. G2896 - 52 - 01	FIGURE 2
SHEET 1 OF 1	

APPENDIX A

APPENDIX A

FIELD INVESTIGATION

We performed the drilling operations on February 22 through February 24, 2022 with Baja Exploration using either a CME 75 or CME 95 truck mounted rig equipped with hollow-stem augers. The locations of the current exploratory borings are shown on the Geologic Map, Figure 1. The boring logs are presented in this Appendix. We located the in the field using a measuring tape and existing reference points; therefore, actual boring locations may deviate slightly. The geotechnical borings were drilled to depths ranging from approximately 15.5 to 41 feet below existing grade. The infiltration-test borings were drilled to depths of approximately 4 to 5 feet.

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

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

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

Kehoe Testing & Engineering performed the CPT soundings. The soil conditions encountered during the field investigation were automatically logged in a nearly continuous profile of penetration resistance as each CPT sounding was being conducted. The recorded tip stress, sleeve stress, and pore pressure of the soil is used to develop a stratigraphic interpretation of the soil profile.

BORING B 1

ELEV. (MSL.) **314'** DATE COMPLETED **02-22-2022**

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

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0					3" **ASPHALT** over 7" **BASE**			
	B1-1			SM	**PREVIOUSLY PLACED FILL (Qpf)** Medium dense, moist, brown and yellowish brown, Silty SAND			
2								
				CL	**SCRIPPS FORMATION (Tsc)** Hard, moist to wet, yellowish brown to grayish brown, Silty CLAYSTONE			
4								
	B1-2					76/11"	106.1	19.9
6								
8								
10	B1-3					82/11"		
12								
14								
	B1-4					50/6"	109.0	20.2

BORING TERMINATED AT 15.5 FEET
No groundwater encountered
Backfilled with soil cuttings

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

G2896-52-01.GPJ

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

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

GEOCON

PROJECT NO. G2896-52-01

BORING B 2

ELEV. (MSL.) 320'   DATE COMPLETED 02-22-2022

EQUIPMENT CME-75   BY: K. HAASE

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0					4" **ASPHALT** over 6" **BASE**			
				ML	**PREVIOUSLY PLACED FILL (Qpf)** Firm to stiff, moist to wet, brown to yellowish brown, Sandy SILT			
2								
4								
	B2-1				-Becomes very stiff	44	108.7	19.6
6								
8								
10	B2-2					34	104.6 108.8	20.7 20.1
12								
14								
	B2-3			CL	Hard, moist to wet, brown CLAY	54		
16								
18								
				ML	**SCRIPPS FORMATION (Tsc)** Hard, moist, yellowish brown, SILTSTONE; mottled gray			
20	B2-4					88/10"		
22								
24								
	B2-5					90/11"	108.3	20.3
26	B2-6							
28								

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

G2896-52-01.GPJ

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

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

GEOCON

PROJECT NO. G2896-52-01

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	**BORING B 2** ELEV. (MSL.) **320'** DATE COMPLETED **02-22-2022** EQUIPMENT **CME-75** BY: **K. HAASE**	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
					MATERIAL DESCRIPTION			
30	B2-7					95/9"	109.3	20.5
					BORING TERMINATED AT 31 FEET No groundwater encountered Backfilled with approx. 10.5 ft³ of grout			

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

G2896-52-01.GPJ

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

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

GEOCON

BORING B 3

ELEV. (MSL.) 324' DATE COMPLETED 02-22-2022

EQUIPMENT CME-75 BY: K. HAASE

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0					5" **ASPHALT** over 6" **BASE**			
1	B3-1			CL	**PREVIOUSLY PLACED FILL (Qpf)** Firm to stiff, moist, yellowish brown to brown, Sandy CLAY			
5	B3-2					22		
10	B3-3				-Becomes wet	32	101.2	22.9
15	B3-4				-Becomes moist	49	108.4	15.3
20	B3-5					51	112.1	17.7
23.5				ML	**SCRIPPS FORMATION (Tsc)** Hard, moist to wet, yellowish brown, SILTSTONE; mottled gray			
25	B3-6					83/11"	105.0	22.8

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

G2896-52-01.GPJ

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

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

GEOCON

BORING B 3

ELEV. (MSL.) 324' DATE COMPLETED 02-22-2022

EQUIPMENT CME-75 BY: K. HAASE

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
30	B3-7				-Becomes olive gray	92/10"	108.9	20.1
32								
34								
	B3-8				-Becomes yellowish brown; mottled gray	50/4"	112.4	19.2

BORING TERMINATED AT 35 FEET
No groundwater encountered
Backfilled with approx. 12 ft³ of grout

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

G2896-52-01.GPJ

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

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

GEOCON

BORING B 4

ELEV. (MSL.) **325'** DATE COMPLETED **02-22-2022**

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

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0				ML	**PREVIOUSLY PLACED FILL (Qpf)** Stiff to very stiff, moist to wet, brown to yellowish brown, Sandy SILT			
2								
4								
	B4-1					25	104.4	21.3
6								
8								
10	B4-2				-Becomes brown to grayish olive	24	102.0	22.6
12								
14								
	B4-3				-Becomes dark brown	33	108.6	19.4
16								
18								
				ML	**SCRIPPS FORMATION (Tsc)** Hard, moist to wet, yellowish brown, SILTSTONE; mottled gray			
20	B4-4					50/5"	109.2	12.3
22								
24								
	B4-5					50/6"	110.2	19.8

BORING TERMINATED AT 25.5 FEET
No groundwater encountered
Backfilled with approx. 8.5 ft³ of grout

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

G2896-52-01.GPJ

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

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

GEOCON

BORING B 5

ELEV. (MSL.) **326'** DATE COMPLETED **02-23-2022**

EQUIPMENT **CME-75** BY: **N. GARCIA**

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 9" **BASE**			
				ML	**PREVIOUSLY PLACED FILL (Qpf)** Stiff to very stiff, moist to wet, yellowish brown, fine Sandy SILT			
2								
4								
	B5-1					30	102.2	22.8
6								
8								
				ML	**SCRIPPS FORMATION (Tsc)** Very stiff, damp, yellowish brown, SILTSTONE; mottled olive			
10	B5-2					47		
12								
14								
	B5-3				-Becomes hard, olive gray, mottled yellow	50/6"		
16					BORING TERMINATED AT 16 FEET No groundwater encountered			

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

G2896-52-01.GPJ

SAMPLE SYMBOLS

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

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

GEOCON

PROJECT NO. G2896-52-01

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	BORING B 6 ELEV. (MSL.) 325' DATE COMPLETED 02-23-2022 EQUIPMENT CME-75 BY: N. GARCIA	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
					MATERIAL DESCRIPTION			
0					4" **ASPHALT CONCRETE** over 9" **BASE**			
2				ML	**PREVIOUSLY PLACED FILL (Qpf)** Stiff, moist, yellowish brown, fine Sandy SILT			
4				ML	**SCRIPPS FORMATION (Tsc)** Hard, wet, yellowish to grayish brown SILTSTONE			
	B6-1					79/9"	109.2	20.6
6								
8								
10	B6-2					50/6"	110.0	20.1
12								
14					-Becomes moist			
	B6-3					50/5'	112.7	15.9
					BORING TERMINATED AT 15.5 FEET No groundwater encountered			

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

G2896-52-01.GPJ

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

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

GEOCON

BORING B 7

ELEV. (MSL.) **348'** DATE COMPLETED **02-23-2022**

EQUIPMENT **CME-75** BY: **N. GARCIA**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0	B7-1				3" **ASPHALT CONCRETE** over 9" **BASE**			
				ML	**PREVIOUSLY PLACED FILL (Qpf)** Very stiff, moist to wet, yellowish brown, fine Sandy SILT			
2								
4								
	B7-2					33	105.5	21.0
6								
8								
10	B7-3				-Becomes yellowish to olive gray	26	118.0	13.6
12								
14								
	B7-4			ML	**SCRIPPS FORMATION (Tsc)** Hard, moist to wet, yellowish to olive brown SILTSTONE	90	109.1	19.7
16								
18								
	B7-5				-Becomes olive gray mottled yellow	87/10"	109.6	21.0

BORING TERMINATED AT 19.9 FEET
No groundwater encountered

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

G2896-52-01.GPJ

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

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

PROJECT NO. G2896-52-01

BORING B 8

ELEV. (MSL.) **348'** DATE COMPLETED **02-23-2022**

EQUIPMENT **CME-75** BY: **D. THOMAS**

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 7" **BASE**			
				CL	**PREVIOUSLY PLACED FILL (Qpf)** Stiff, moist, yellowish brown, Sandy to Silty CLAY			
2								
4								
	B8-1					43	112.0	16.4
6								
8								
10	B8-2				-Becomes wet -Some native clasts	43	109.7	19.1
12								
14								
	B8-3					42	106.6	20.5
16								
18								
20	B8-4				-Becomes stiff	50/6"	105.1	19.8
	B8-5						104.2	21.7
22								
24								
	B8-6					26	102.4	22.3
26								
28								

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

G2896-52-01.GPJ

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

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

GEOCON

BORING B 8

ELEV. (MSL.) **348'** DATE COMPLETED **02-23-2022**

EQUIPMENT **CME-75** BY: **D. THOMAS**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
30	B8-7			CL-SC	Stiff, moist, dark brown, Sandy CLAY to Clayey SAND	27	105.1	19.6
32								
34								
	B8-8			ML	**SCRIPPS FORMATION (Tsc)** Very stiff, moist, yellowish brown to olive gray, Sandy SILTSTONE to Clayey SILTSTONE	89	108.2	20.9
36								
38								
40	B8-9					88	105.7	23.1

BORING TERMINATED AT 41 FEET
No groundwater encountered
Backfilled with 14ft³ of bentonite/grout mixture

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

G2896-52-01.GPJ

SAMPLE SYMBOLS

... SAMPLING UNSUCCESSFUL ... STANDARD PENETRATION TEST ... DRIVE SAMPLE (UNDISTURBED)

... DISTURBED OR BAG SAMPLE ... CHUNK SAMPLE ... WATER TABLE OR ... SEEPAGE

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

GEOCON

BORING B 9

ELEV. (MSL.) **350'** DATE COMPLETED **02-23-2022**

EQUIPMENT **CME-75** BY: **N. GARCIA**

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 9" **BASE**			
				ML	**PREVIOUSLY PLACED FILL (Qpf)** Stiff to very stiff, moist, yellowish brown, Sandy SILT			
2								
4				ML	**SCRIPPS FORMATION (Tsc)** Hard, moist to wet, olive gray, SILTSTONE; mottled yellowish brown			
	B9-1					87/11"	108.4	20.7
6								
8								
10	B9-2				-No recovery	90/9"		
12					-Becomes highly cemented			
14								
	B9-3				-No recovery	50/1"		
16					BORING TERMINATED AT 16 FEET No groundwater encountered			

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

G2896-52-01.GPJ

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

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

GEOCON

BORING B 10

ELEV. (MSL.) **351'** DATE COMPLETED **02-23-2022**

EQUIPMENT **CME-75** BY: **N. GARCIA**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0	B10-1				3" **ASPHALT CONCRETE** over 9" **BASE**			
				ML	**PREVIOUSLY PLACED FILL (Qpf)** Very stiff, moist to wet, yellowish to olive brown, Sandy SILT			
2								
4								
	B10-2					36	104.0	20.8
6								
8								
					-Becomes dark brown, clayey			
10	B10-3					68	107.4	19.1
12								
14								
	B10-4				-Becomes yellowish brown	22	109.9	18.6
16								
18								
20	B10-5				-Becomes yellowish to olive brown	46	106.1	21.3
22								
24								
	B10-6				-Becomes dark olive brown	31	103.6	20.1
26								
28	B10-7					50/3"		

Figure A-10,
Log of Boring B 10, Page 1 of 2

G2896-52-01.GPJ

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

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

GEOCON

BORING B 10

ELEV. (MSL.) 351' DATE COMPLETED 02-23-2022

EQUIPMENT CME-75 BY: N. GARCIA

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
30	B10-8					34	113.1	18.5
32								
34								
	B10-9			ML	**SCRIPPS FORMATION (Tsc)** Hard, wet, yellowish brown, SILTSTONE	50/5"	108.5	20.2
36								
38								
40	B10-10				-Becomes moist	50/5"	110.2	10.3
					BORING TERMINATED AT 40.5 FEET No groundwater encountered			

Figure A-10, Log of Boring B 10, Page 2 of 2

G2896-52-01.GPJ

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

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

GEOCON

PROJECT NO. G2896-52-01

BORING B 11

ELEV. (MSL.) **345'** DATE COMPLETED **02-23-2022**

EQUIPMENT **CME-75** BY: **D. THOMAS**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0					4" **ASPHALT CONCRETE** over 6" **BASE**			
				SC-SM	**PREVIOUSLY PLACED FILL (Qpf)** Dense, moist, yellowish brown, Clayey to Silty, SAND			
2								
				SM-ML	**SCRIPPS FORMATION (Tsc)** Very dense, moist, yellowish brown, Silty, fine-grained SANDSTONE to Sandy SILTSTONE			
4								
	B11-1					88	111.7	15.4
6								
8								
10	B11-2					79	112.6	17.2
12								
14								
	B11-3				-Becomes wet		107.9	20.9
16								

BORING TERMINATED AT 16 FEET
No groundwater encountered
Boring backfilled with cuttings

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

G2896-52-01.GPJ

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

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

GEOCON

BORING B 12

ELEV. (MSL.) **345'** DATE COMPLETED **02-24-2022**

EQUIPMENT **CME-75** BY: **N. GARCIA**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0					4" **ASPHALT CONCRETE** over 6" **BASE**			
				ML	**PREVIOUSLY PLACED FILL (Qpf)** Stiff, moist, yellowish brown, fine Sandy SILT			
2				ML	**SCRIPPS FORMATION (Tsc)** Hard, damp, yellowish to olive brown, SILTSTONE			
4								
	B12-1					50/3"		
6								
8								
10						50/6"	91.9	16.7
	B12-2				-Becomes olive gray mottled yellow			
12								
14								
	B12-3					50/6"	111.5	16.7
					BORING TERMINATED AT 15.5 FEET No groundwater encountered			

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

G2896-52-01.GPJ

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

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

GEOCON

PROJECT NO. G2896-52-01

BORING B 13

ELEV. (MSL.) **345'** DATE COMPLETED **02-24-2022**

EQUIPMENT **CME-75** BY: **N. GARCIA**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0					2" **ASPHALT CONCRETE** over 6" **BASE**			
				ML	**PREVIOUSLY PLACED FILL (Qpf)** Soft to firm, moist to wet, yellowish brown, fine Sandy SILT			
2								
4								
	B13-1					7	100.5	22.6
6	B13-2							
8								
10	B13-3					5		22.3
12				ML	**SCRIPPS FORMATION (Tsc)** Very stiff to hard, moist to wet, yellowish to olive brown, SILTSTONE			
14								
	B13-4				-Becomes gray	62/10"	109.6	19.9
16	B13-5							
18								
	B13-6					66/10"	110.0	19.8

BORING TERMINATED AT 19.8 FEET
No groundwater encountered

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

G2896-52-01.GPJ

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

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GEOCON

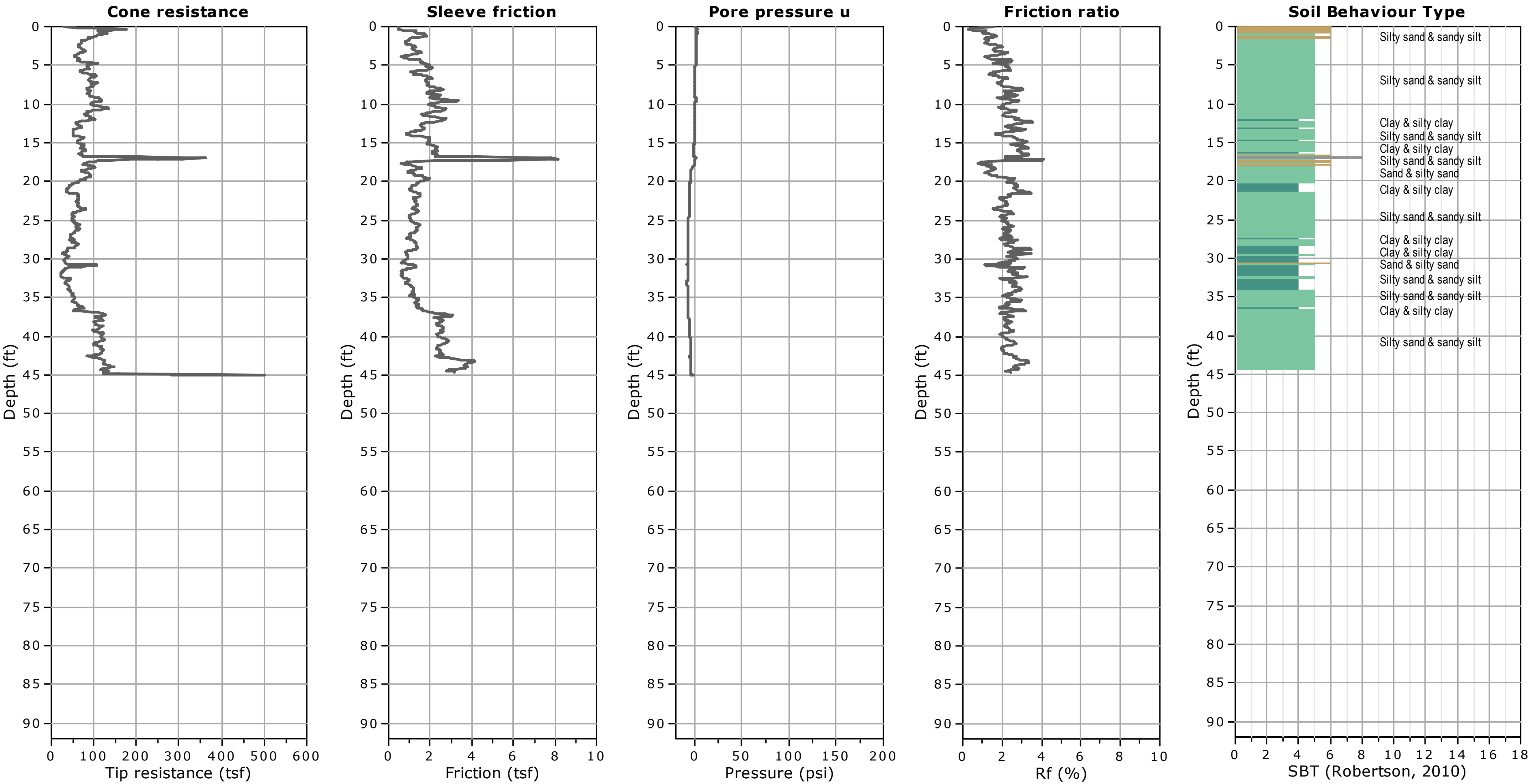
Kehoe Testing and Engineering
714-901-7270
steve@kehoetesting.com
www.kehoetesting.com

Project: **Geocon / Lusk Business Park Redevelopment**

Location: **San Diego, CA**

CPT-4

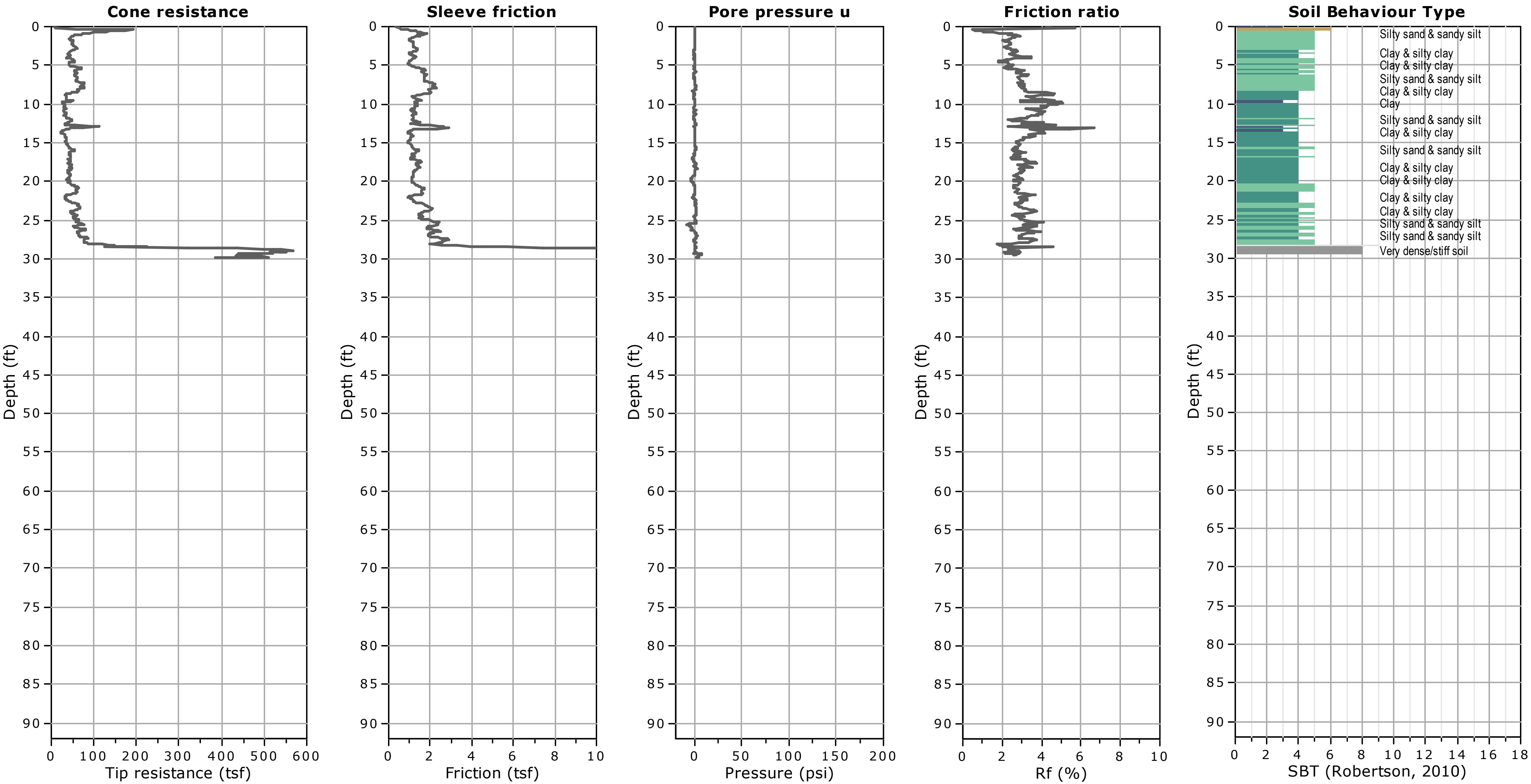
Total depth: 45.02 ft, Date: 2/28/2022



Kehoe Testing and Engineering
714-901-7270
steve@kehoetesting.com
www.kehoetesting.com

Project: Geocon / Lusk Business Park Redevelopment
Location: San Diego, CA

CPT-5
Total depth: 29.93 ft, Date: 3/1/2022



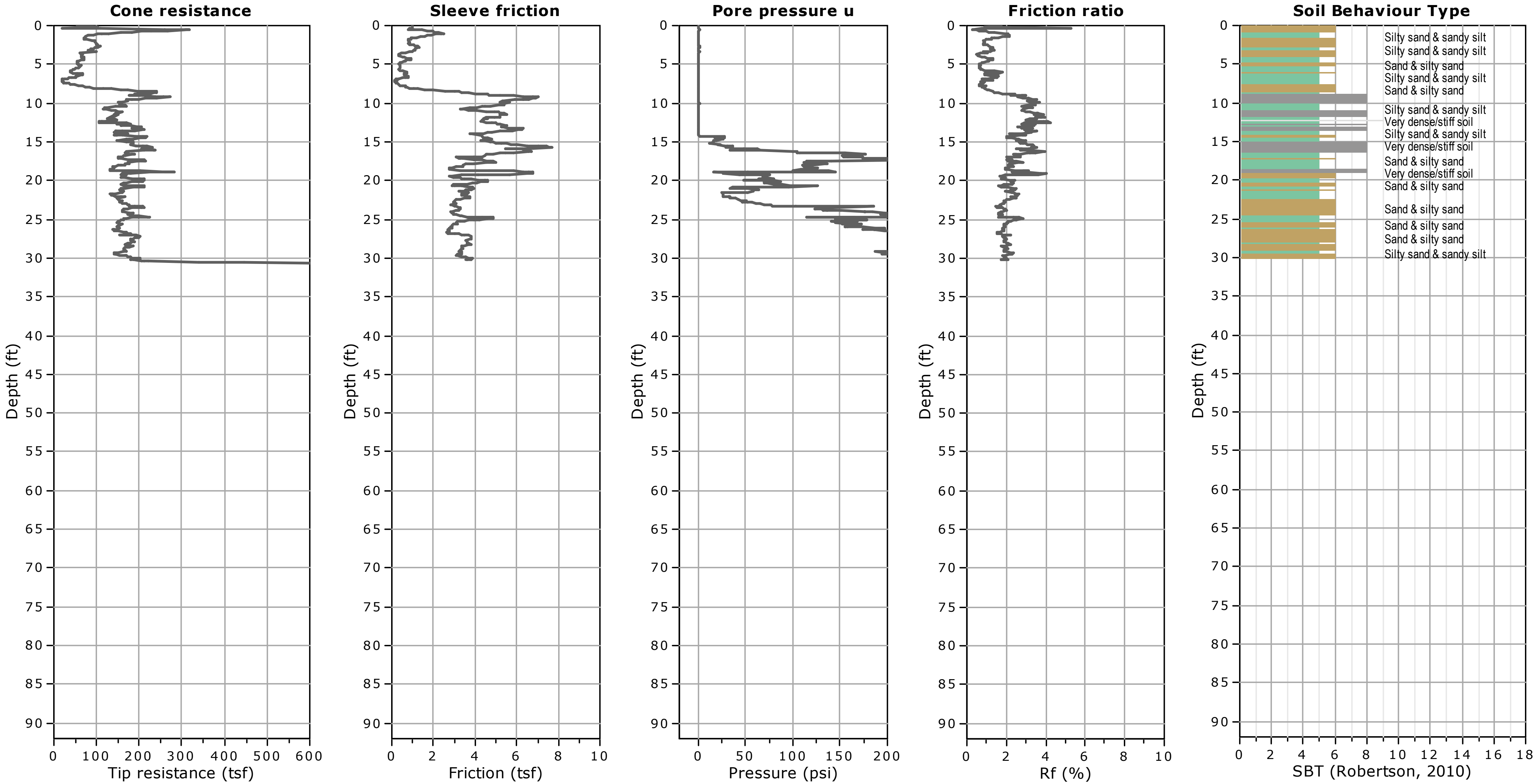
Kehoe Testing and Engineering
714-901-7270
steve@kehoetesting.com
www.kehoetesting.com

Project: **Geocon / Lusk Business Park Redevelopment**

Location: **San Diego, CA**

CPT-6

Total depth: 30.64 ft, Date: 3/1/2022



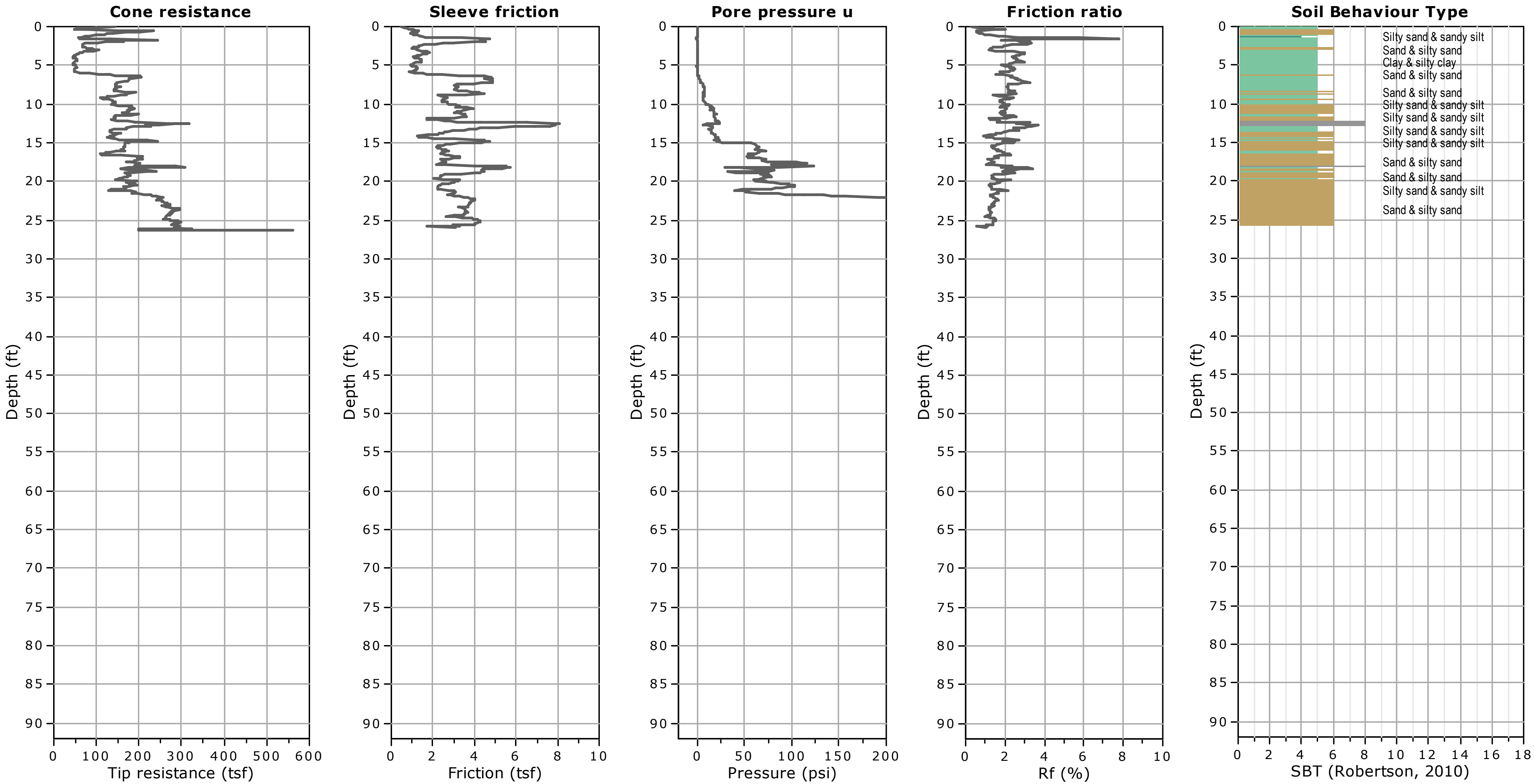
Kehoe Testing and Engineering
714-901-7270
steve@kehoetesting.com
www.kehoetesting.com

Project: Geocon / Lusk Business Park Redevelopment

Location: San Diego, CA

CPT-7

Total depth: 26.32 ft, Date: 3/1/2022



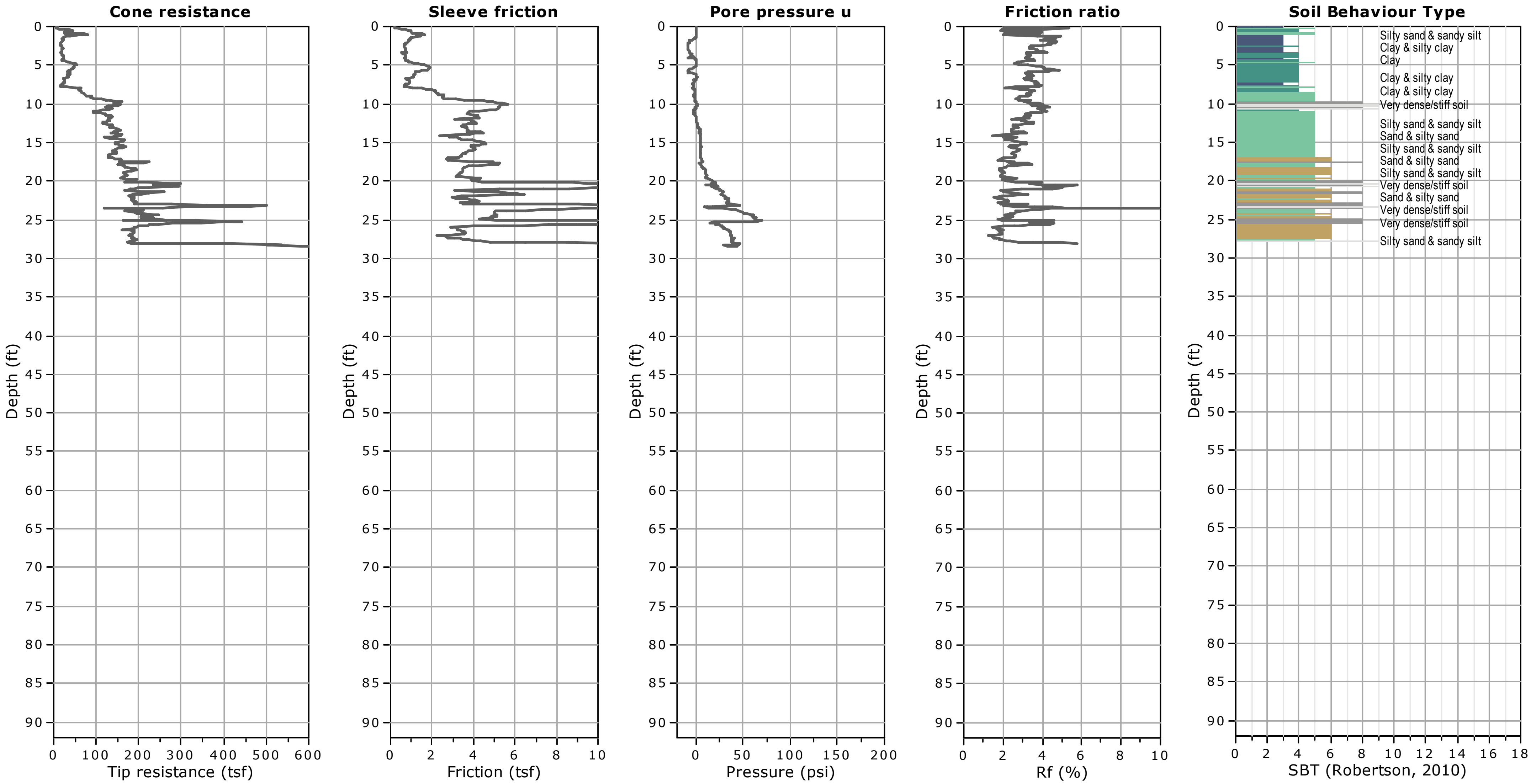
Kehoe Testing and Engineering
714-901-7270
steve@kehoetesting.com
www.kehoetesting.com

Project: Geocon / Lusk Business Park Redevelopment

Location: San Diego, CA

CPT-8

Total depth: 28.42 ft, Date: 3/1/2022



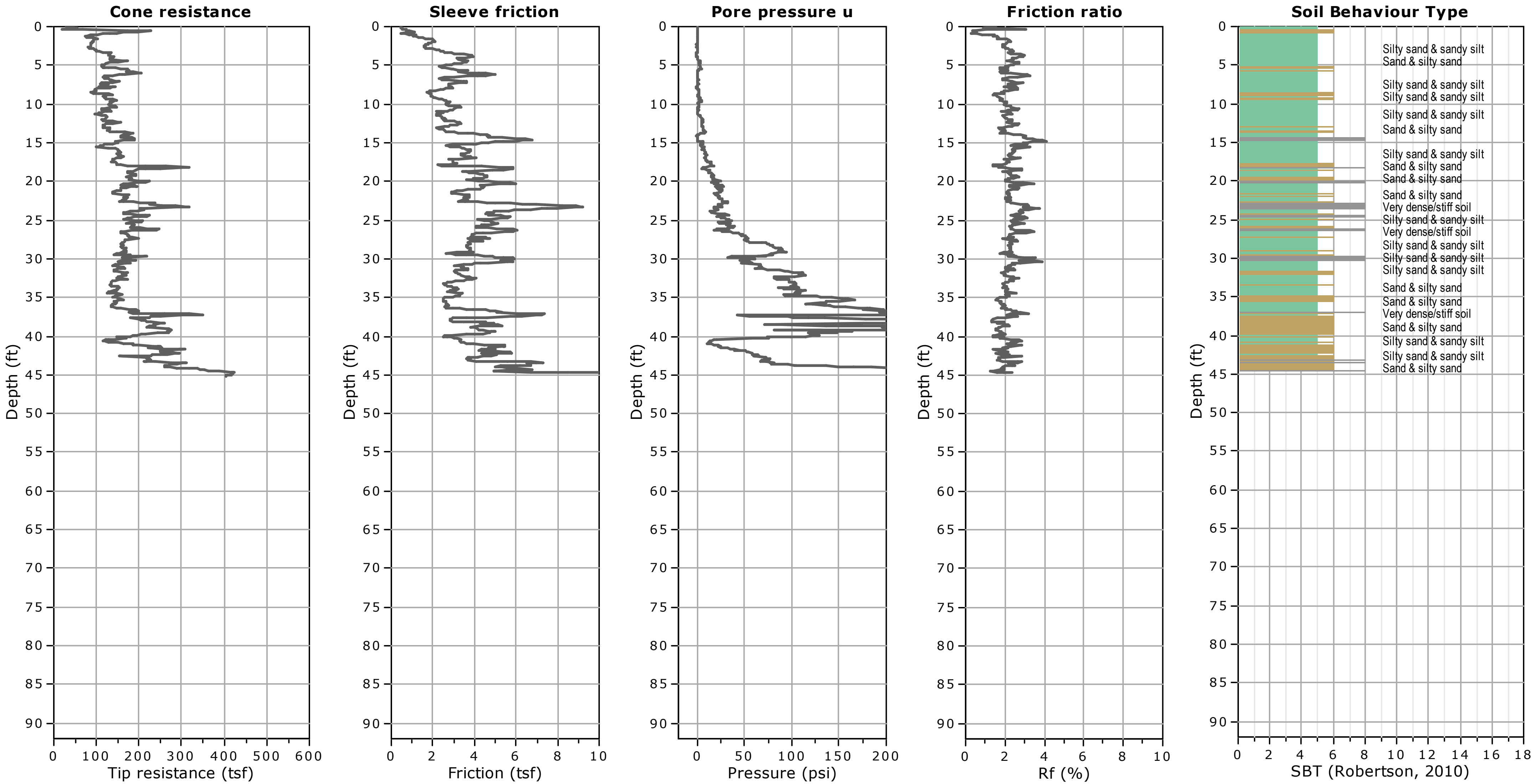
Kehoe Testing and Engineering
714-901-7270
steve@kehoetesting.com
www.kehoetesting.com

Project: Geocon / Lusk Business Park Redevelopment

Location: San Diego, CA

CPT-10

Total depth: 45.15 ft, Date: 3/1/2022



APPENDIX B

APPENDIX B

LABORATORY TESTING

We performed laboratory tests in accordance with generally accepted test methods of the American Society for Testing and Materials (ASTM) or other suggested procedures. We tested selected soil samples for in-place dry density/moisture content, maximum density/optimum moisture content, expansion index, water-soluble sulfate, pH, resistivity, water-soluble chloride ion content, unconfined compressive strength, consolidation, gradation 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 MAXIMUM DRY DENSITY AND OPTIMUM MOISTURE CONTENT TEST RESULTS ASTM D 1557

Sample No.	Description (Geologic Unit)	Maximum Dry Density (pcf)	Optimum Moisture Content (% dry wt.)
B1-1	Yellowish Brown, Silty SAND/Silty CLAY (Qpf/Tsc)	114.2	15.4
B3-1	Yellowish Brown to Brown, Sandy CLAY (Qpf)	117.6	13.5
B13-5	Gray SILTSTONE (Tsc)	118.3	12.4

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	12.7	27.2	99.2	67	Expansive	Medium
B3-1	11.3	24.9	103.0	71	Expansive	Medium
B13-5	12.0	24.7	103.2	55	Expansive	Medium

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

Sample No.	Depth (feet)	Geologic Unit	Water-Soluble Sulfate (%)	ACI 318 Sulfate Exposure
B1-1	0-5	Qpf/Tsc	0.002	S0
B3-1	0-5	Qpf	0.531	S2
B13-5	16-19	Tsc	0..403	S2

SUMMARY OF LABORATORY CHLORIDE TEST RESULTS
AASHTO T 291

Sample No.	Depth (Feet)	Geologic Unit	Chloride Ion Content (ppm)	Chloride Ion Content (%)
B1-1	0-5	Qpf/Tsc	103	0.010
B3-1	0-5	Qpf	88	0.009
B13-5	16-19	Tsc	357	0.036

SUMMARY OF LABORATORY POTENTIAL OF HYDROGEN (PH) AND RESISTIVITY TEST RESULTS
CALIFORNIA TEST NO. 643

Sample No.	Depth (Feet)	Geologic Unit	pH	Minimum Resistivity (ohm centimeters)
B1-1	0-5	Qpf/Tsc	8.49	1000
B3-1	0-5	Qpf	7.67	640
B13-5	16-19	Tsc	7.61	310

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

Sample No.	Depth (Feet)	Description (Geologic Unit)	R-Value
B1-1	0-5	Yellowish Brown, Silty SAND/Silty CLAY (Qpf/Tsc)	17

SUMMARY OF LABORATORY UNCONFINED COMPRESSIVE STRENGTH TEST RESULTS
ASTM D 1558

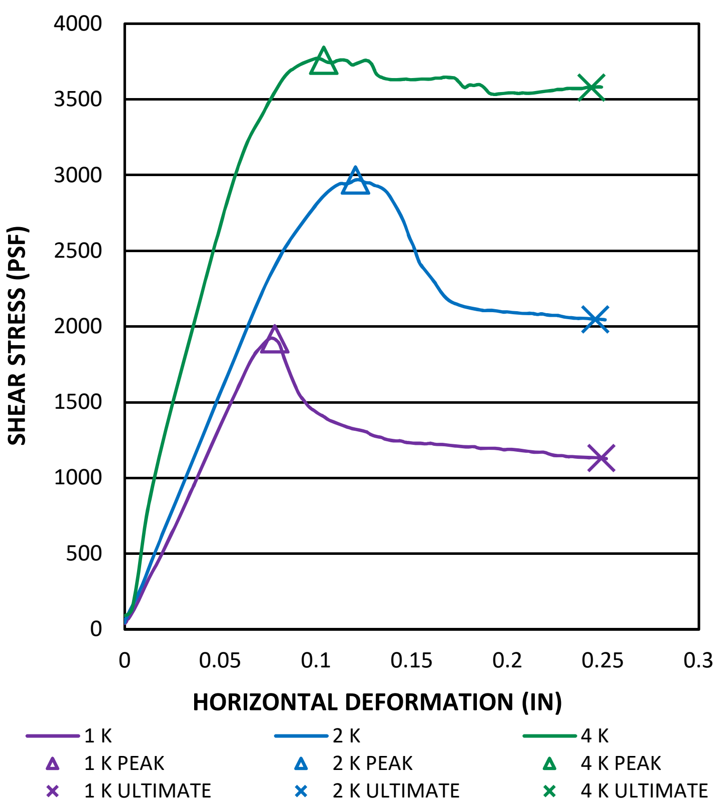
Sample No.	Depth (feet)	Geologic Unit	Hand Penetrometer Reading/Unconfined Compression Strength (tsf) and Undrained Shear Strength (ksf)
B1-4	15	Tsc	4.5+
B2-1	5	Qpf	4.5+
B2-5	25	Tsc	4.5+
B2-7	30	Tsc	4.5+
B3-4	15	Qpf	4.5+
B3-5	20	Qpf	4.5+
B3-6	25	Tsc	4.5+
B3-8	35	Tsc	4.5+
B4-1	5	Qpf	4.0
B4-2	10	Qpf	2.0
B4-3	15	Qpf	4.5+
B4-4	20	Tsc	4.5+
B4-5	25	Tsc	4.5+
B5-1	5	Qpf	4.5+
B6-2	10	Tsc	4.5+
B6-3	15	Tsc	4.5+
B7-2	5	Qpf	4.0
B7-3	10	Qpf	4.5+
B7-4	15	Tsc	4.5+
B7-5	19	Tsc	4.5+
B8-1	5	Qpf	4.5+
B8-2	10	Qpf	4.5+
B8-3	15	Qpf	4.5+
B8-4	20	Qpf	4.5+
B8-6	25	Qpf	4.5+
B8-7	30	Qpf	4.5+
B8-8	35	Tsc	4.5+
B8-9	40	Tsc	4.5+
B9-1	5	Tsc	4.5+
B10-2	5	Qpf	4.5+
B10-4	15	Qpf	4.5+
B10-5	20	Qpf	4.5+
B10-6	25	Qpf	3.5
B10-9	35	Tsc	4.5
B10-10	40	Tsc	4.5
B11-2	10	Tsc	4.5
B11-3	15	Tsc	4.5
B12-3	15	Tsc	4.5
B13-1	5	Qpf	2.5
B13-4	15	Tsc	4.5
B13-6	19	Tsc	4.5

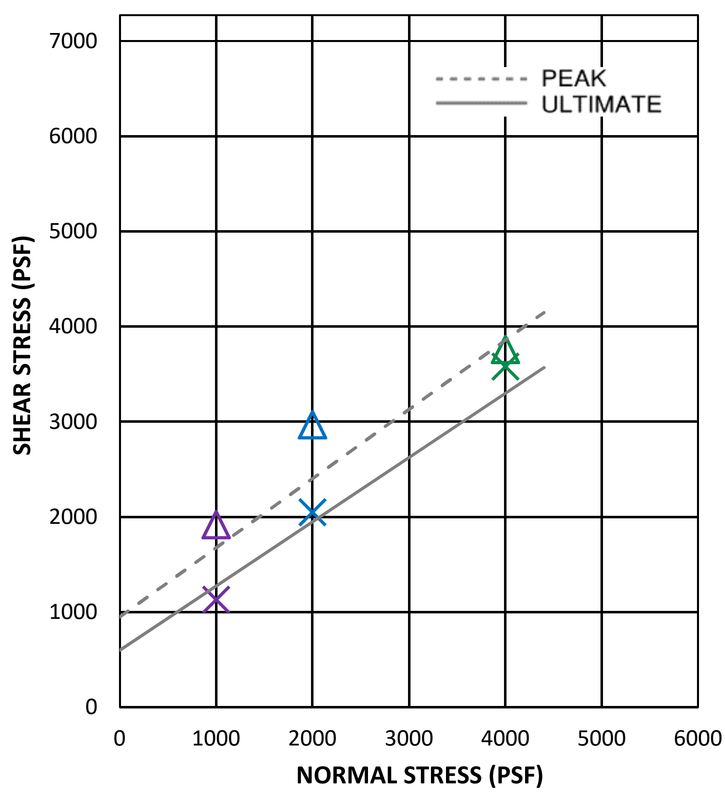
SAMPLE NO.:	B1-2	GEOLOGIC UNIT:	Tsc
SAMPLE DEPTH (FT):	5'	NATURAL/REMOLDED:	N

INITIAL CONDITIONS				
NORMAL STRESS TEST LOAD	1 K	2 K	4 K	AVERAGE
ACTUAL NORMAL STRESS (PSF):	1000	2000	4000	--
WATER CONTENT (%):	19.2	20.9	19.5	19.9
DRY DENSITY (PCF):	108.3	105.6	104.4	106.1

AFTER TEST CONDITIONS				
NORMAL STRESS TEST LOAD	1 K	2 K	4 K	AVERAGE
WATER CONTENT (%):	22.6	24.3	23.0	23.3
PEAK SHEAR STRESS (PSF):	1918	2968	3758	--
ULT.-E.O.T. SHEAR STRESS (PSF):	1131	2048	3579	--

RESULTS		
PEAK	COHESION, C (PSF)	950
	FRICTION ANGLE (DEGREES)	36
ULTIMATE	COHESION, C (PSF)	600
	FRICTION ANGLE (DEGREES)	34





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DIRECT SHEAR - ASTM D 3080

LUSK BUSINESS PARK

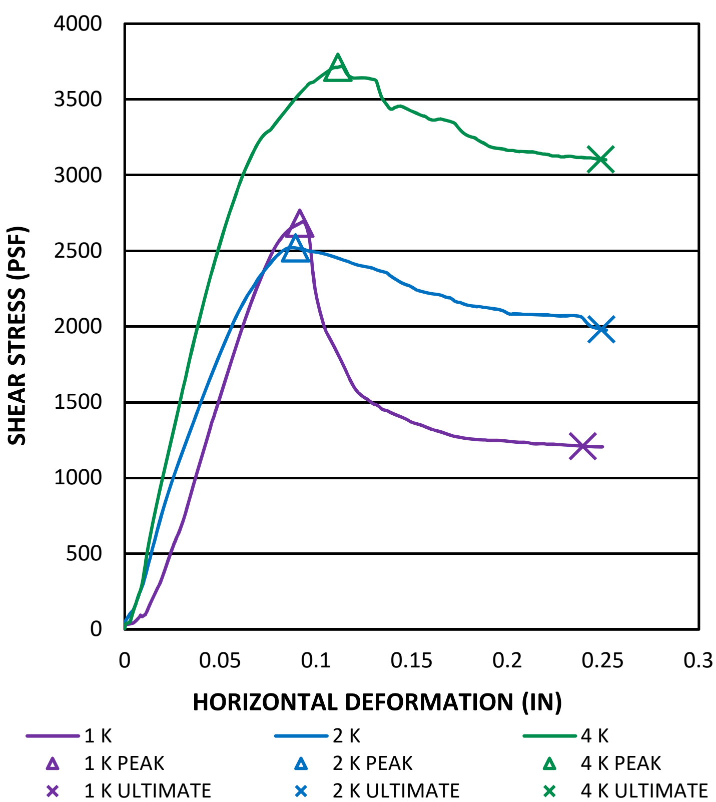
PROJECT NO.: G2896-52-01

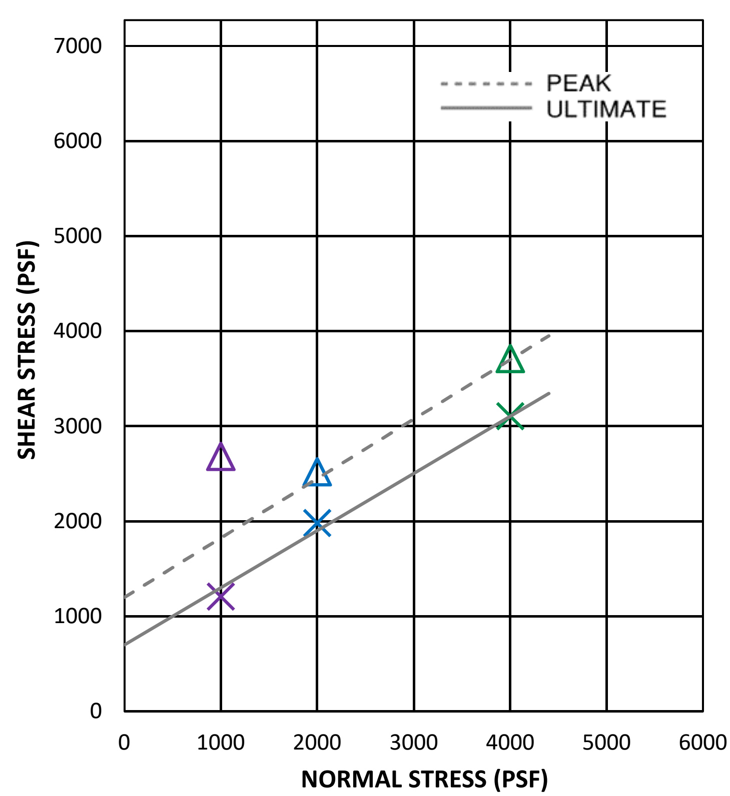
SAMPLE NO.:	B2-2	GEOLOGIC UNIT:	Qpf
SAMPLE DEPTH (FT):	10'	NATURAL/REMOLDED:	N

INITIAL CONDITIONS				
NORMAL STRESS TEST LOAD	1 K	2 K	4 K	AVERAGE
ACTUAL NORMAL STRESS (PSF):	1000	2000	4000	--
WATER CONTENT (%):	21.0	18.0	21.1	20.1
DRY DENSITY (PCF):	108.4	110.7	107.4	108.8

AFTER TEST CONDITIONS				
NORMAL STRESS TEST LOAD	1 K	2 K	4 K	AVERAGE
WATER CONTENT (%):	24.0	22.1	24.7	23.6
PEAK SHEAR STRESS (PSF):	2682	2519	3712	--
ULT.-E.O.T. SHEAR STRESS (PSF):	1209	1983	3104	--

RESULTS		
PEAK	COHESION, C (PSF)	1200
	FRICTION ANGLE (DEGREES)	32
ULTIMATE	COHESION, C (PSF)	700
	FRICTION ANGLE (DEGREES)	31





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DIRECT SHEAR - ASTM D 3080

LUSK BUSINESS PARK

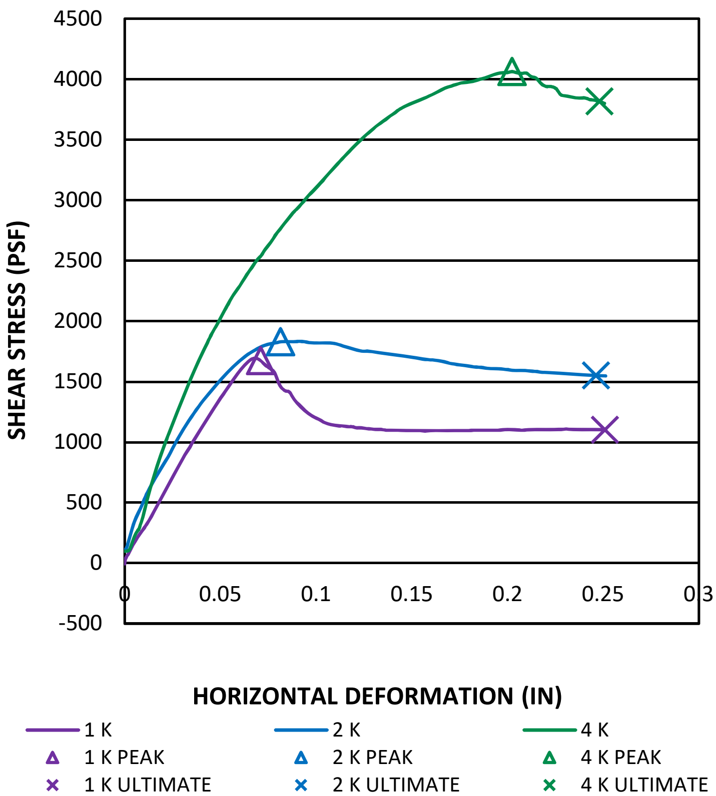
PROJECT NO.: G2896-52-01

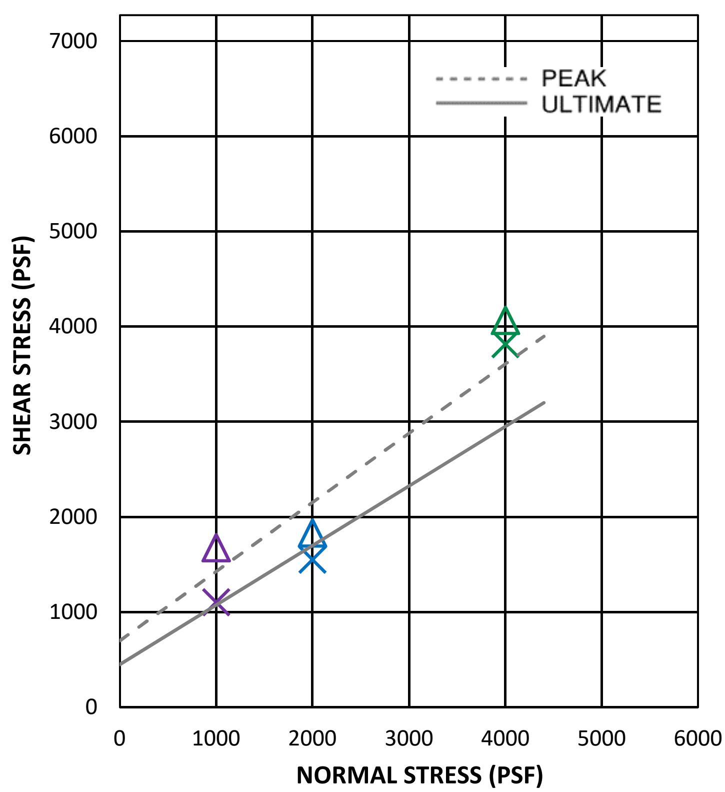
SAMPLE NO.:	**B3-7**	GEOLOGIC UNIT:	**Tsc**
SAMPLE DEPTH (FT):	**30'**	NATURAL/REMOLDED:	**N**

INITIAL CONDITIONS				
NORMAL STRESS TEST LOAD	**1 K**	**2 K**	**4 K**	**AVERAGE**
ACTUAL NORMAL STRESS (PSF):	1000	2000	4000	--
WATER CONTENT (%):	20.3	20.7	19.3	20.1
DRY DENSITY (PCF):	109.3	107.7	109.7	108.9

AFTER TEST CONDITIONS				
NORMAL STRESS TEST LOAD	**1 K**	**2 K**	**4 K**	**AVERAGE**
WATER CONTENT (%):	24.6	26.6	24.5	25.3
PEAK SHEAR STRESS (PSF):	1674	1830	4063	--
ULT.-E.O.T. SHEAR STRESS (PSF):	1102	1550	3816	--

RESULTS		
PEAK	COHESION, C (PSF)	700
	FRICTION ANGLE (DEGREES)	36
ULTIMATE	COHESION, C (PSF)	450
	FRICTION ANGLE (DEGREES)	32





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LUSK BUSINESS PARK

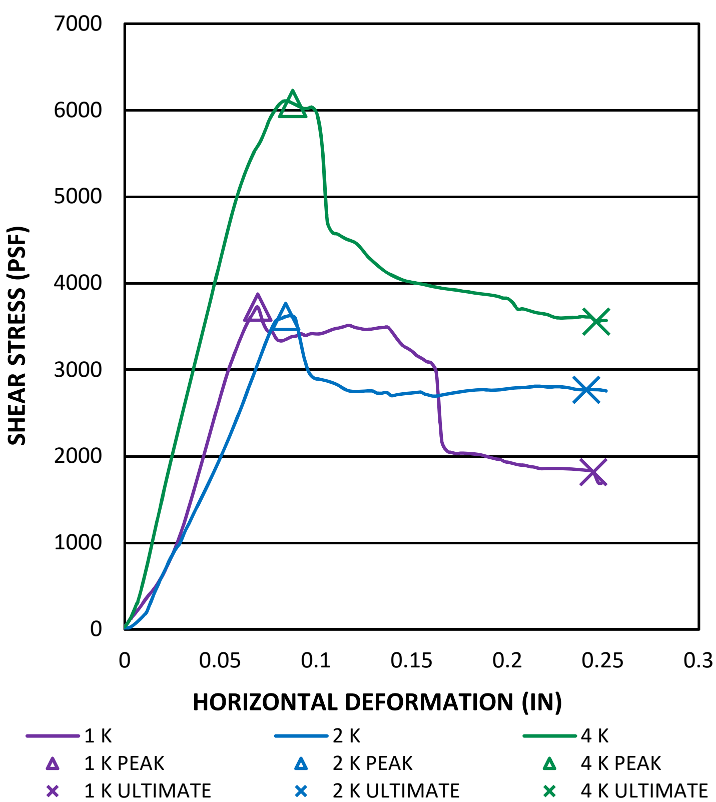
PROJECT NO.: G2896-52-01

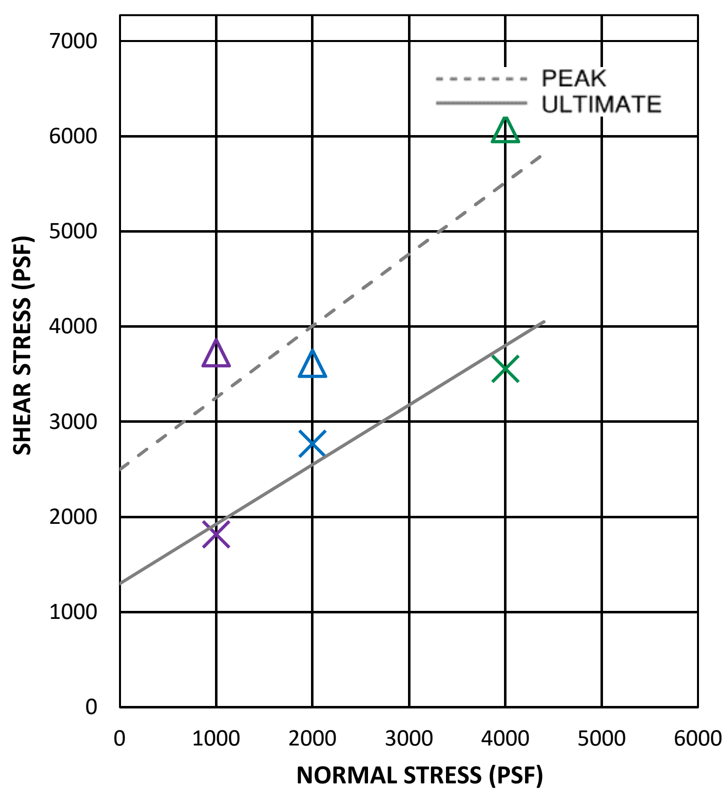
SAMPLE NO.:	B11-1	GEOLOGIC UNIT:	Tsc
SAMPLE DEPTH (FT):	5'	NATURAL/REMOLDED:	N

INITIAL CONDITIONS				
NORMAL STRESS TEST LOAD	1 K	2 K	4 K	AVERAGE
ACTUAL NORMAL STRESS (PSF):	1000	2000	4000	--
WATER CONTENT (%):	16.6	15.0	14.5	15.4
DRY DENSITY (PCF):	115.8	106.9	112.3	111.7

AFTER TEST CONDITIONS				
NORMAL STRESS TEST LOAD	1 K	2 K	4 K	AVERAGE
WATER CONTENT (%):	19.5	22.0	20.1	20.5
PEAK SHEAR STRESS (PSF):	3725	3615	6078	--
ULT.-E.O.T. SHEAR STRESS (PSF):	1817	2769	3556	--

RESULTS		
PEAK	COHESION, C (PSF)	2500
	FRICTION ANGLE (DEGREES)	37
ULTIMATE	COHESION, C (PSF)	1300
	FRICTION ANGLE (DEGREES)	32





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LUSK BUSINESS PARK

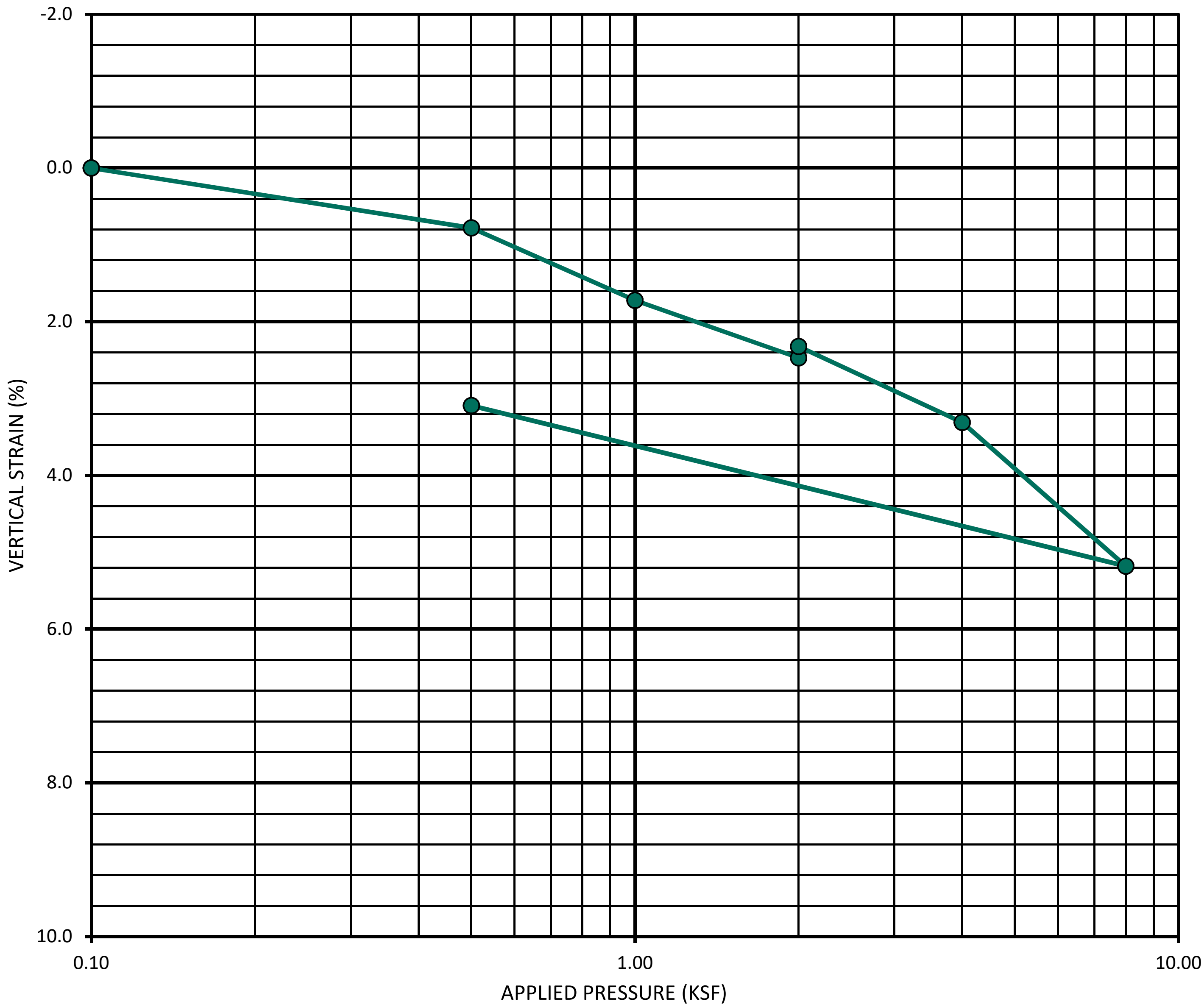
PROJECT NO.: G2896-52-01

SAMPLE NO.: **B3-3**

SAMPLE DEPTH (FT): **10'**

GEOLOGIC UNIT: **Qpf**

TEST INFORMATION	
INITIAL DRY DENSITY (PCF):	101.2
INITIAL WATER CONTENT (%):	22.9%
SAMPLE SATURATED AT (KSF):	2.0
INITIAL SATURATION (%):	95.2%



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CONSOLIDATION CURVE - ASTM D 2435

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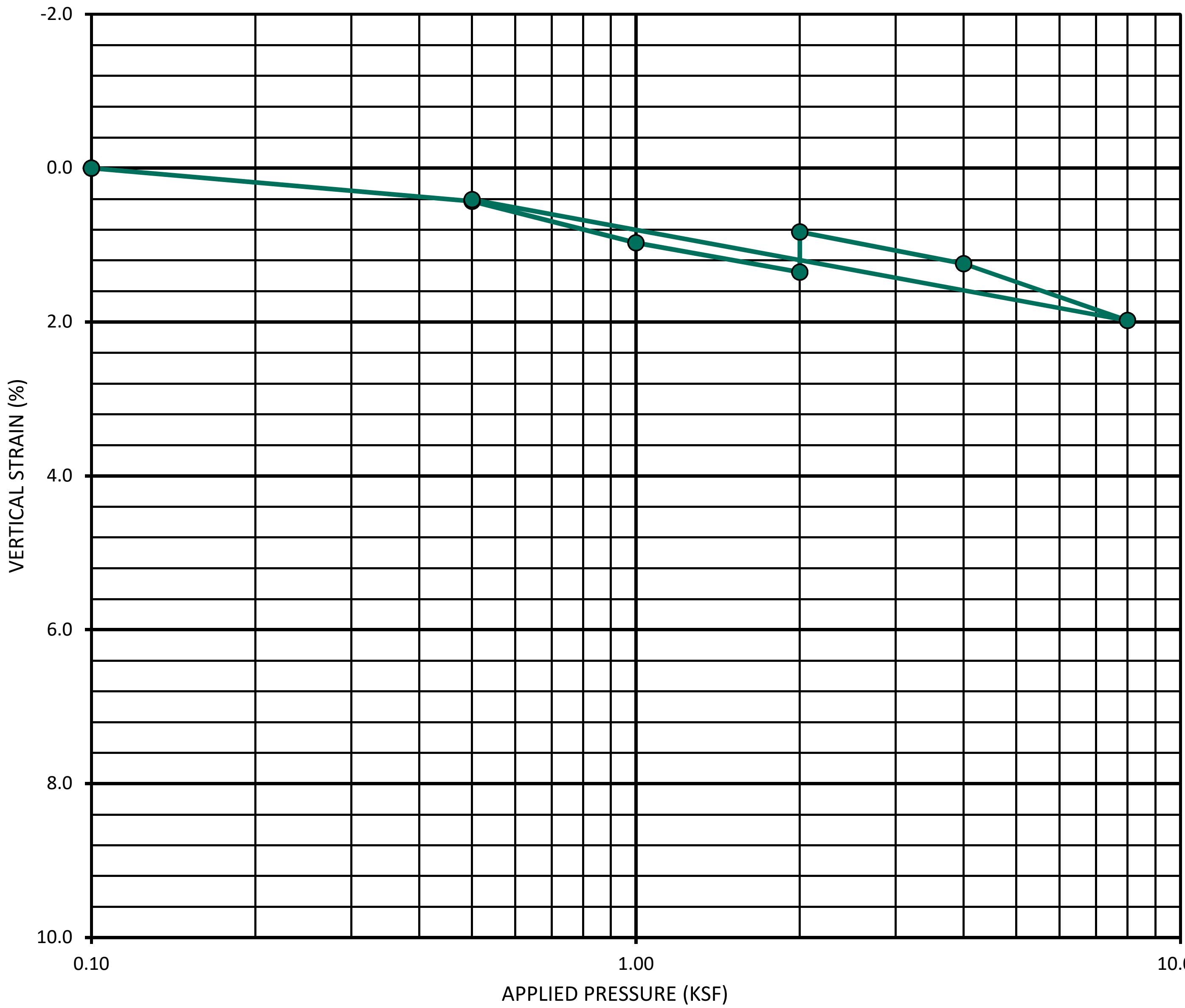
PROJECT NO.: G2896-52-01

SAMPLE NO.:	B6-1	GEOLOGIC UNIT:	Tsc
SAMPLE DEPTH (FT):	5'		

TEST INFORMATION	
INITIAL DRY DENSITY (PCF):	109.2
INITIAL WATER CONTENT (%):	20.6%
SAMPLE SATURATED AT (KSF):	2.0
INITIAL SATURATION (%):	100+



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CONSOLIDATION CURVE - ASTM D 2435

LUSK BUSINESS PARK

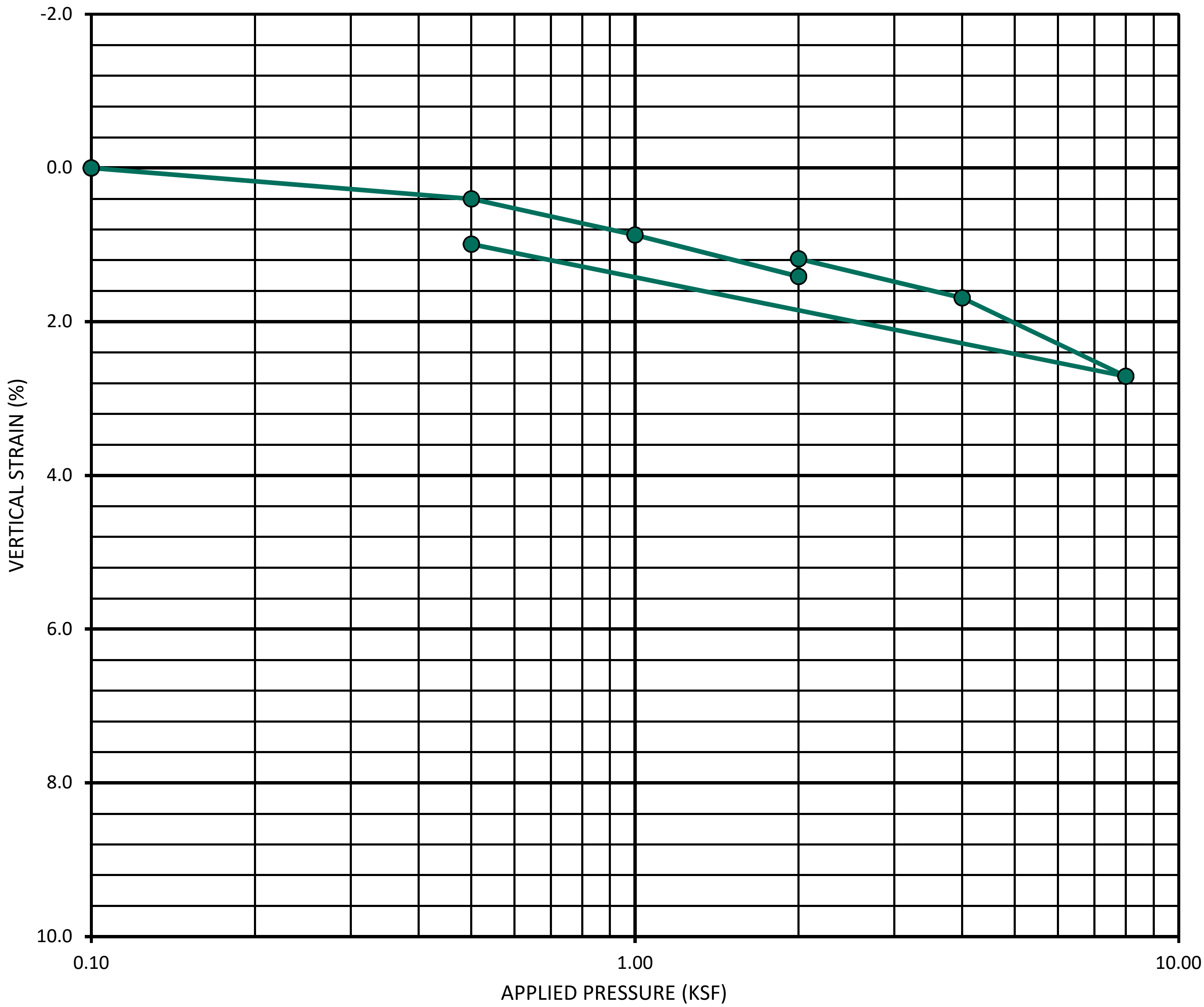
PROJECT NO.: G2896-52-01

SAMPLE NO.: **B8-4**

SAMPLE DEPTH (FT): **20'**

GEOLOGIC UNIT: **Qpf**

TEST INFORMATION	
INITIAL DRY DENSITY (PCF):	104.2
INITIAL WATER CONTENT (%):	21.7%
SAMPLE SATURATED AT (KSF):	2.0
INITIAL SATURATION (%):	97.2%



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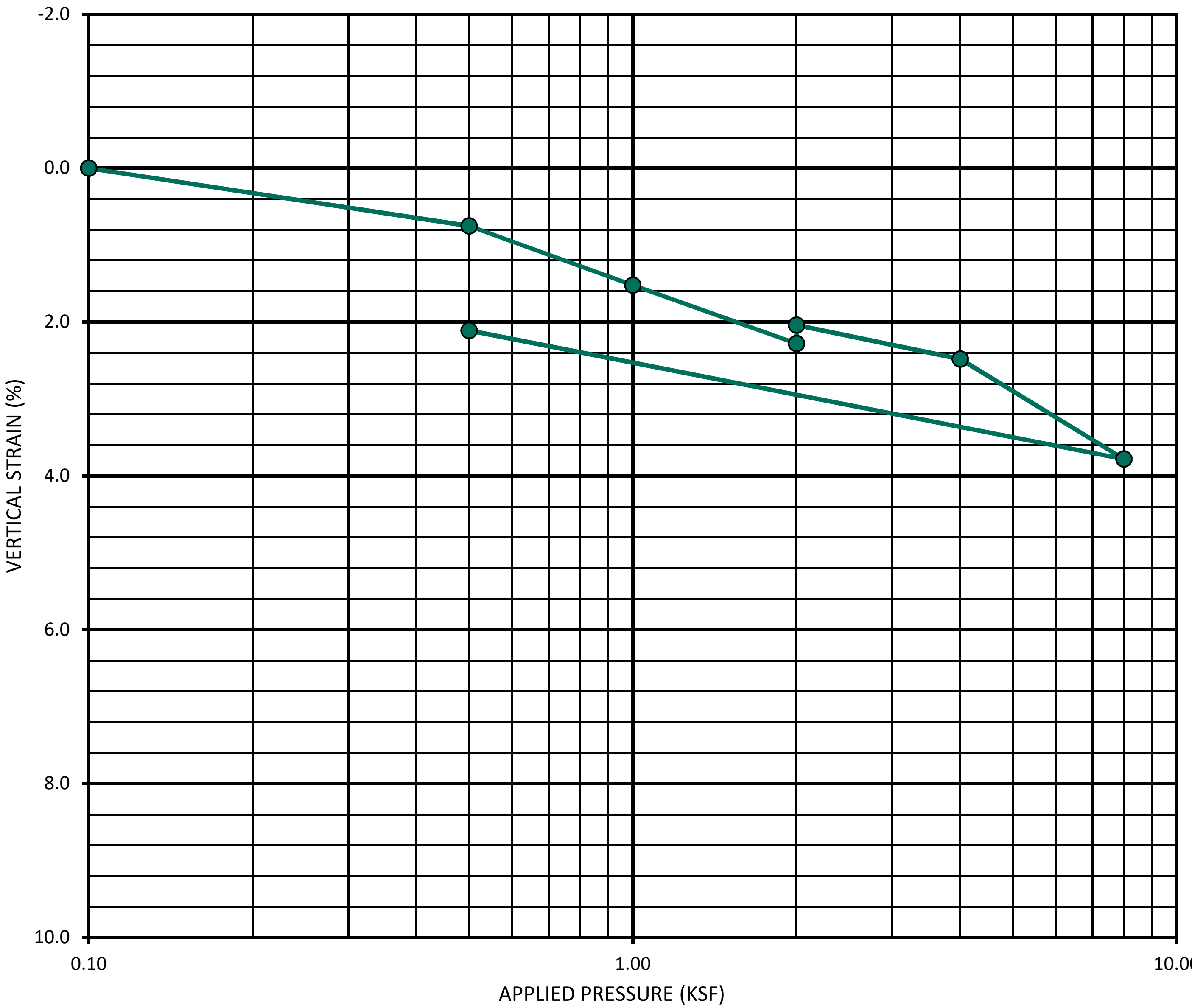
PROJECT NO.: G2896-52-01

SAMPLE NO.: **B10-8**

SAMPLE DEPTH (FT): **30'**

GEOLOGIC UNIT: **Qpf**

TEST INFORMATION	
INITIAL DRY DENSITY (PCF):	113.1
INITIAL WATER CONTENT (%):	18.5%
SAMPLE SATURATED AT (KSF):	2.0
INITIAL SATURATION (%):	100+



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CONSOLIDATION CURVE - ASTM D 2435

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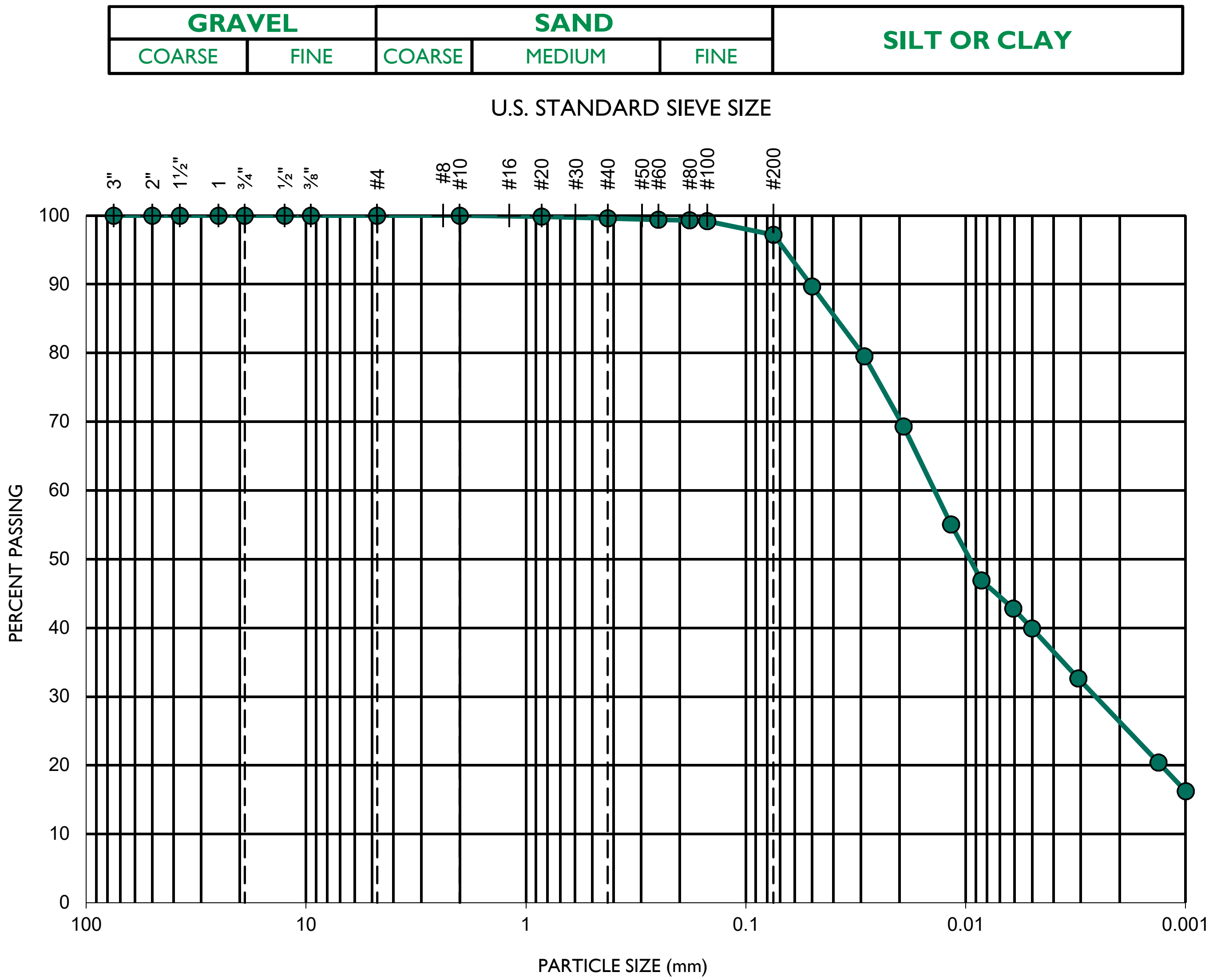
PROJECT NO.: G2896-52-01

SAMPLE NO.: B1-3

SAMPLE DEPTH (FT.): 10'

GEOLOGIC UNIT: Tsc



TEST DATA					
D_{10} (mm)	D_{30} (mm)	D_{60} (mm)	C_c	C_u	SOIL DESCRIPTION
0.00051	0.00270	0.01426	1.0	28.2	Silty CLAY

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SIEVE ANALYSES - ASTM D 6913

LUSK BUSINESS PARK

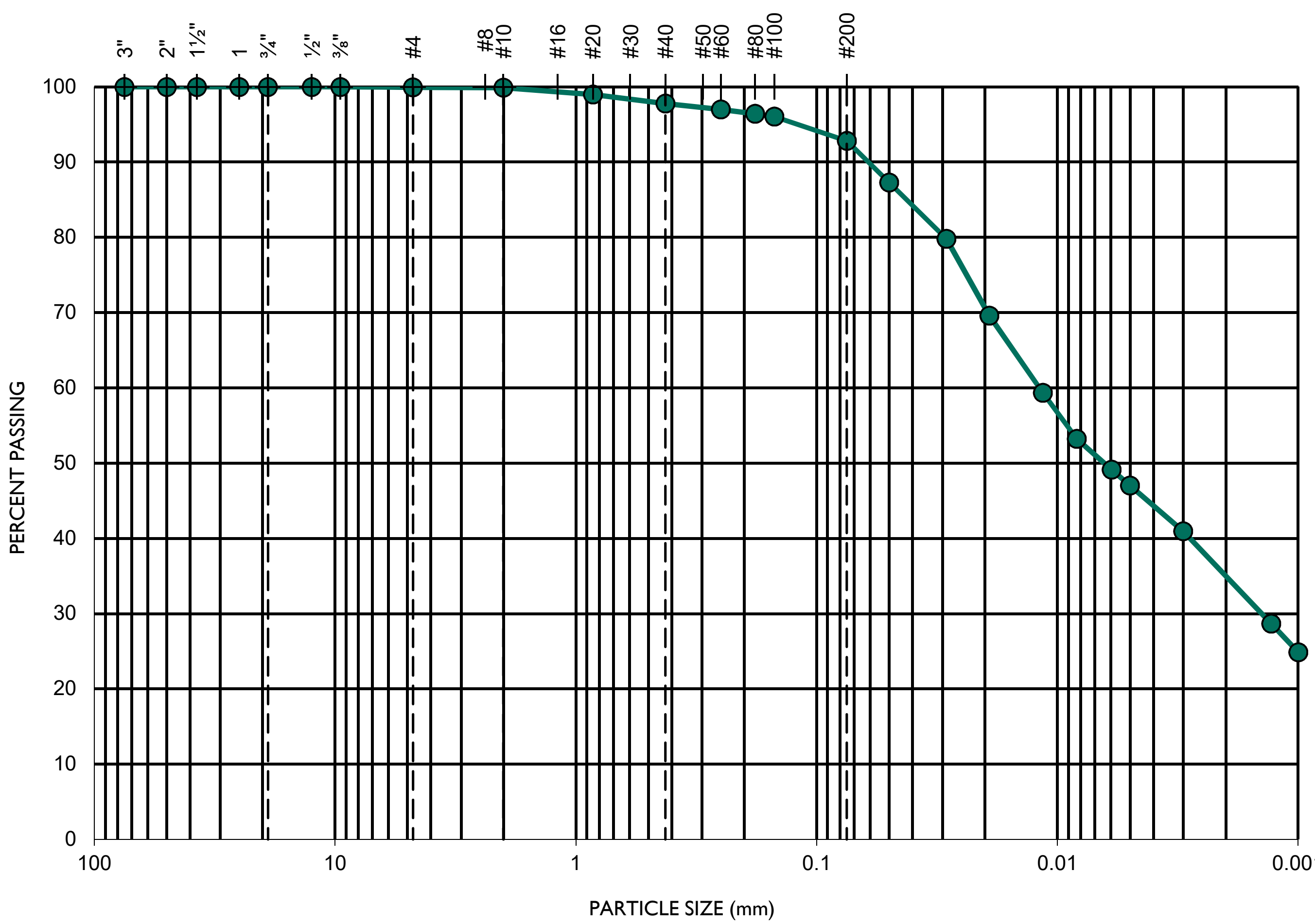
PROJECT NO.: G2896-52-01

SAMPLE NO.: **B2-3**

SAMPLE DEPTH (FT.): **15'**

GEOLOGIC UNIT: **Qpf**

GRAVEL		SAND			SILT OR CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	



TEST DATA					
D_{10} (mm)	D_{30} (mm)	D_{60} (mm)	C_c	C_u	SOIL DESCRIPTION
--	0.00148	0.01198	--	--	CLAY

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SIEVE ANALYSES - ASTM D 6913

LUSK BUSINESS PARK

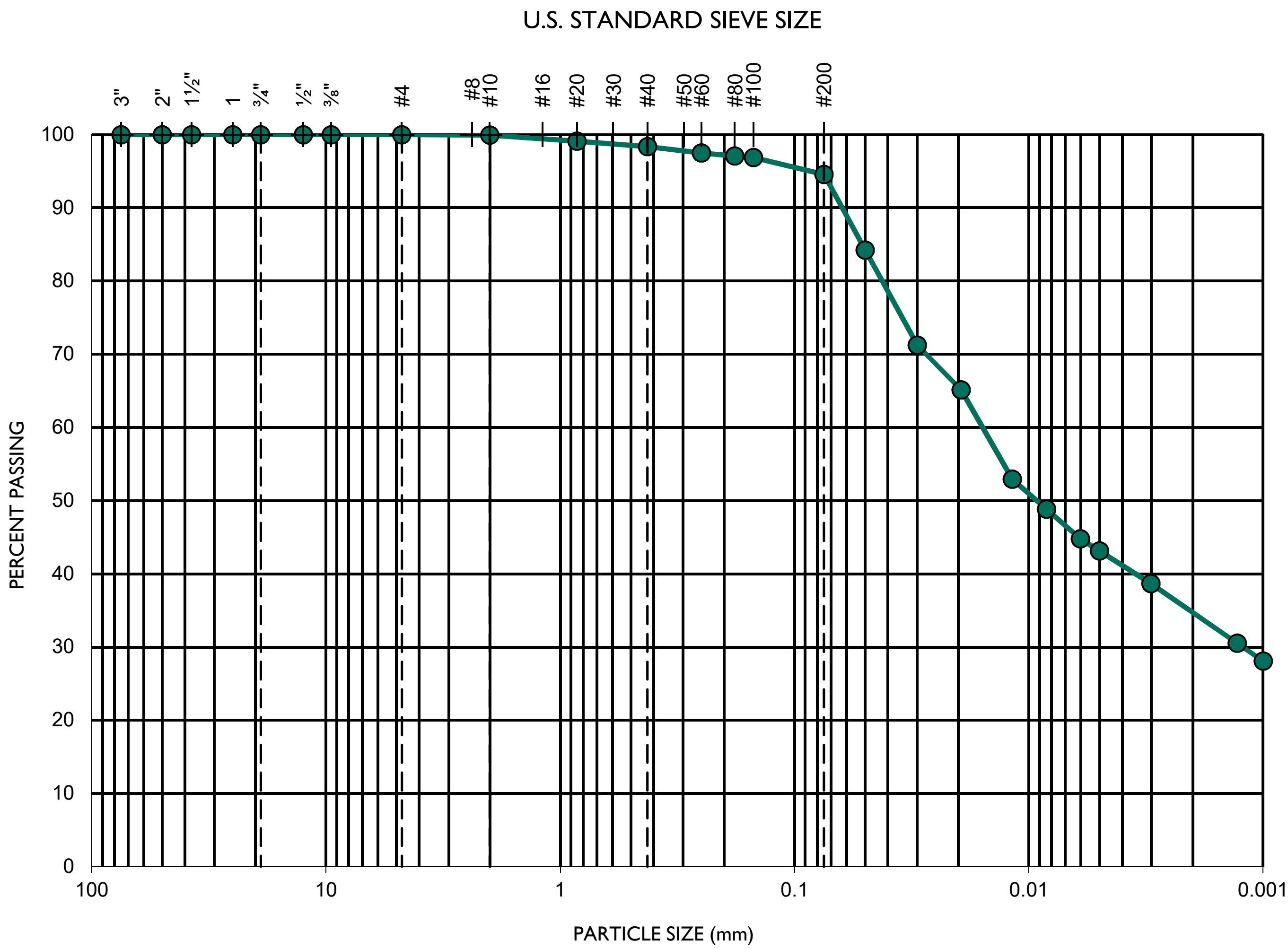
PROJECT NO.: G2896-52-01

SAMPLE NO.: **B3-2**

SAMPLE DEPTH (FT.): **5'**

GEOLOGIC UNIT: **Qpf**

GRAVEL		SAND			SILT OR CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	



TEST DATA					
D_{10} (mm)	D_{30} (mm)	D_{60} (mm)	C_c	C_u	SOIL DESCRIPTION
--	0.00123	0.01621	--	--	CLAY

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SIEVE ANALYSES - ASTM D 6913

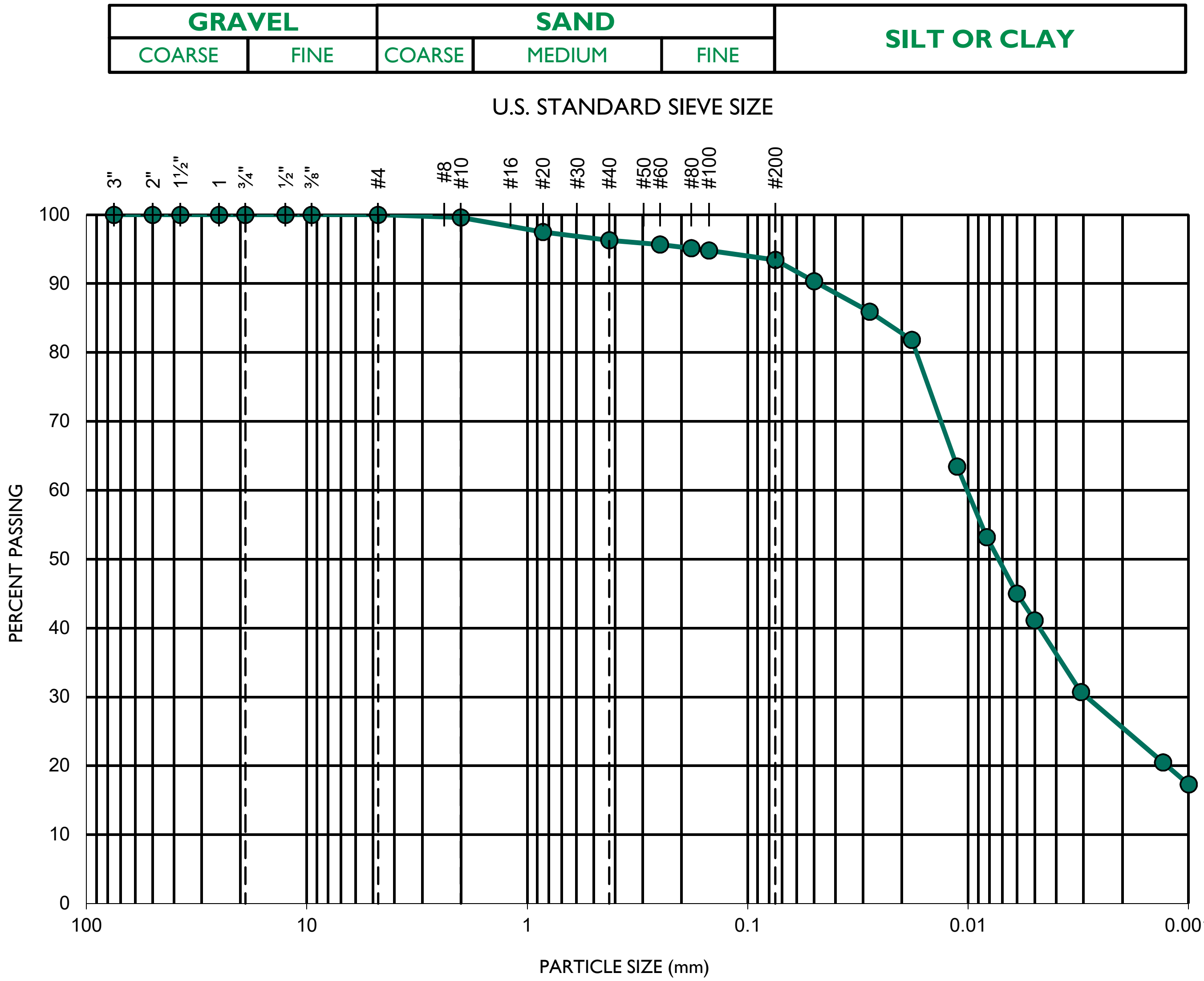
LUSK BUSINESS PARK

PROJECT NO.: G2896-52-01

SAMPLE NO.: **B8-2**

SAMPLE DEPTH (FT.): **10'**

GEOLOGIC UNIT: **Qpf**



TEST DATA					
D_{10} (mm)	D_{30} (mm)	D_{60} (mm)	C_c	C_u	SOIL DESCRIPTION
0.00030	0.00295	0.01022	2.8	34.1	Silty CLAY

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SIEVE ANALYSES - ASTM D 6913

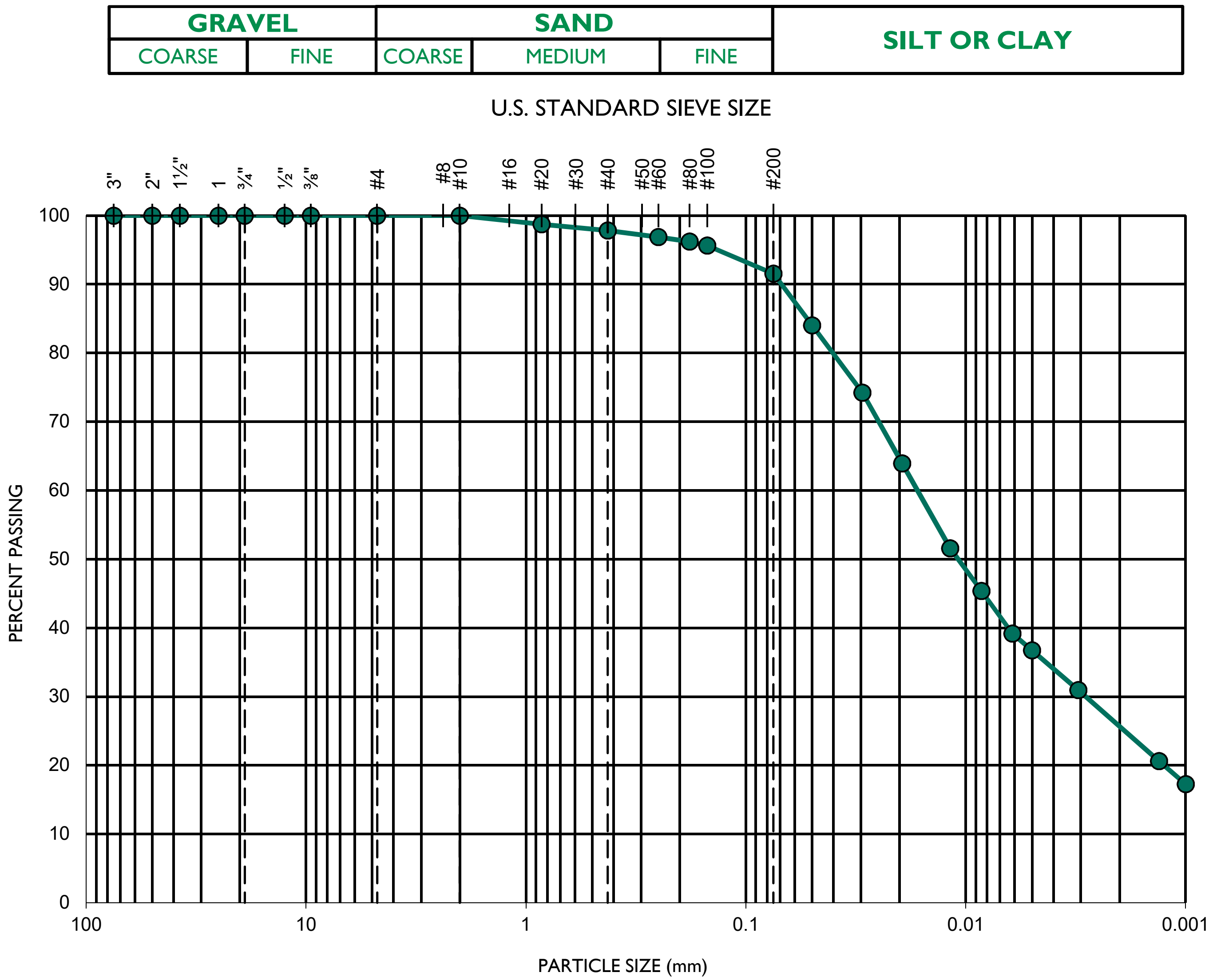
LUSK BUSINESS PARK

PROJECT NO.: G2896-52-01

SAMPLE NO.: **B8-6**

SAMPLE DEPTH (FT.): **25'**

GEOLOGIC UNIT: **Qpf**



TEST DATA					
D_{10} (mm)	D_{30} (mm)	D_{60} (mm)	C_c	C_u	SOIL DESCRIPTION
0.00032	0.00292	0.01701	1.6	53.8	Silty CLAY

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SIEVE ANALYSES - ASTM D 6913

LUSK BUSINESS PARK

PROJECT NO.: G2896-52-01

APPENDIX C

APPENDIX C

SLOPE STABILITY ANALYSIS

We performed slope stability analyses using a two-dimensional computer software *GeoStudio2018* developed by Geo-Slope International Ltd. We analyzed the critical modes of potential slip surfaces including rotational-mode and block-mode based on Spencer's method. The soil parameters used, case conditions, and the calculated factors of safety were presented herein. Plots of analyses' results, including the soil stratigraphy, potential failure surfaces, and calculated Factors of Safety, are included in this appendix.

We estimated shear strength characteristics of the existing geologic units based on laboratory direct shear tests on samples obtained during our field investigation in accordance with ASTM D 3080 and based on empirical data obtained from the referenced geotechnical literature. Additionally, the bedding plane shear strength characteristics and inclinations are based on the modeling and laboratory testing results performed by Moore & Taber during construction of the site buttresses. Table C-I presents a summary of the soil parameters used for the stability analyses.

TABLE C-I
SUMMARY OF SOIL PROPERTIES USED FOR SLOPE STABILITY ANALYSES

Geologic Unit/Material	Density (pcf)	Cohesion (psf)	Friction Angle (degrees)
Compacted Fill (Qcf)	125	400	28
Scripps Formation/Ardath Shale (Tsc/Ta)	135	500	32
Bedding Plane Shear (BPS)	135	150	8

We selected the general slope geometry of Cross-Section B-B' to perform the slope stability analyses assuming the maximum and minimum buttress key widths reported by Moore & Taber. Table C-II provides a summary of cases analyzed and calculated Factors of Safety. A minimum Factor of Safety of 1.5 under static conditions is currently required by the City of San Diego for slope stability. Results of slope stability analyses are plotted on the following figures.

TABLE C-II
SUMMARY OF SLOPE STABILITY ANALYSES

Cross Section	Figure No.	Condition of Slope Stability Analyses	Calculated Factor of Safety
B-B'	C-1	Minimum Rotational-Mode Factor of Safety of Fill Slope, 82-Foot Wide Buttress – Static	1.86
	C-2	Minimum Block-Mode Factor of Safety of Fill Slope, 82-Foot Wide Buttress – Static	1.94
	C-3	Minimum Rotational-Mode Factor of Safety of Fill Slope, 120-Foot Wide Buttress – Static	1.86
	C-4	Minimum Block-Mode Factor of Safety of Fill Slope, 120-Foot Wide Buttress – Static	3.18

Lusk Boulevard Redevelopment
Project No. G2896-52-01
Section B-B'
Name: BBc0 -82 foot buttress- entry exit Analysis-LR.gsz
Date: 07/14/2022 Time: 03:04:11 PM

Existing Condition
Static Analysis

Color	Name	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)
	BPS - Bedding Plane Shear	125	150	8
	Qcf - Compacted Fill	135	400	28
	Tsc - Scripps Formation	135	500	32

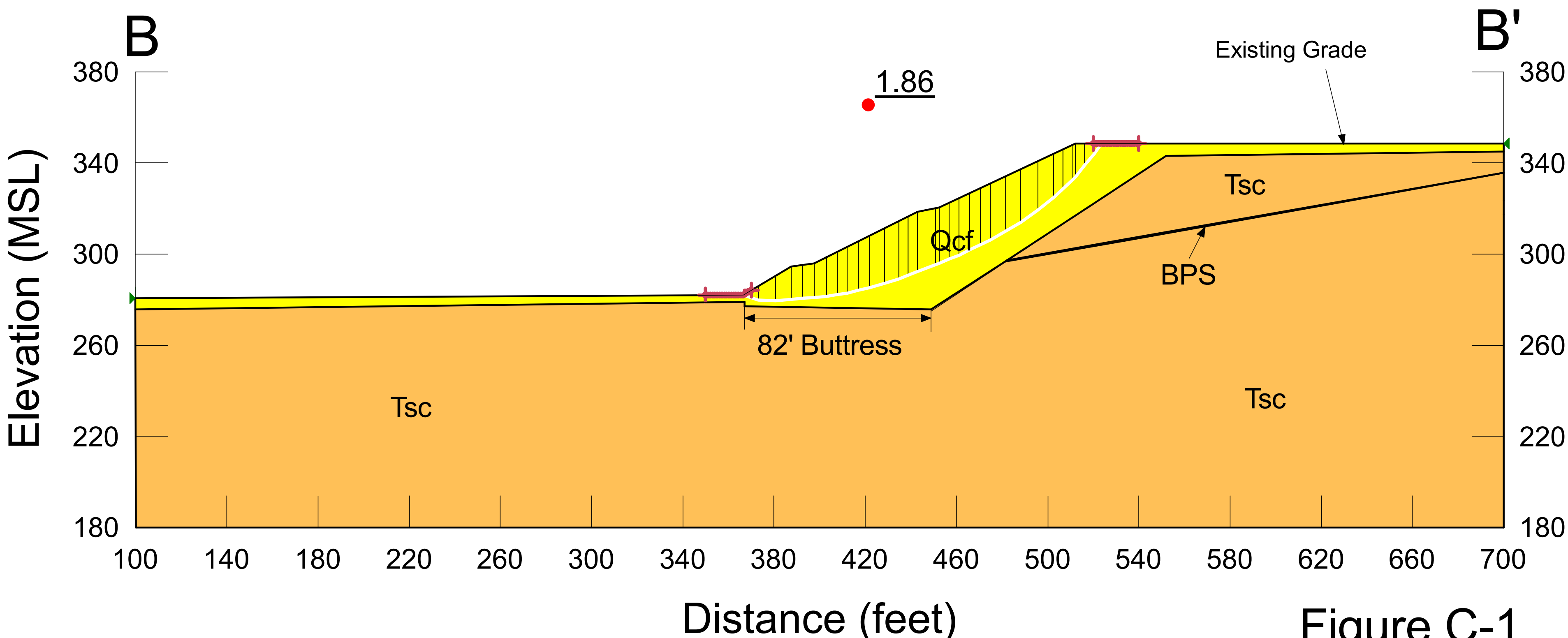


Figure C-1

Lusk Boulevard Redevelopment
Project No. G2896-52-01
Section B-B'
Name: BBc0 -82 foot buttress- Block Analysis-LR.gsz
Date: 07/14/2022 Time: 03:02:46 PM

Existing Condition
Static Analysis

Color	Name	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)
	BPS - Bedding Plane Shear	125	150	8
	Qcf - Compacted Fill	135	400	28
	Tsc - Scripps Formation	135	500	32

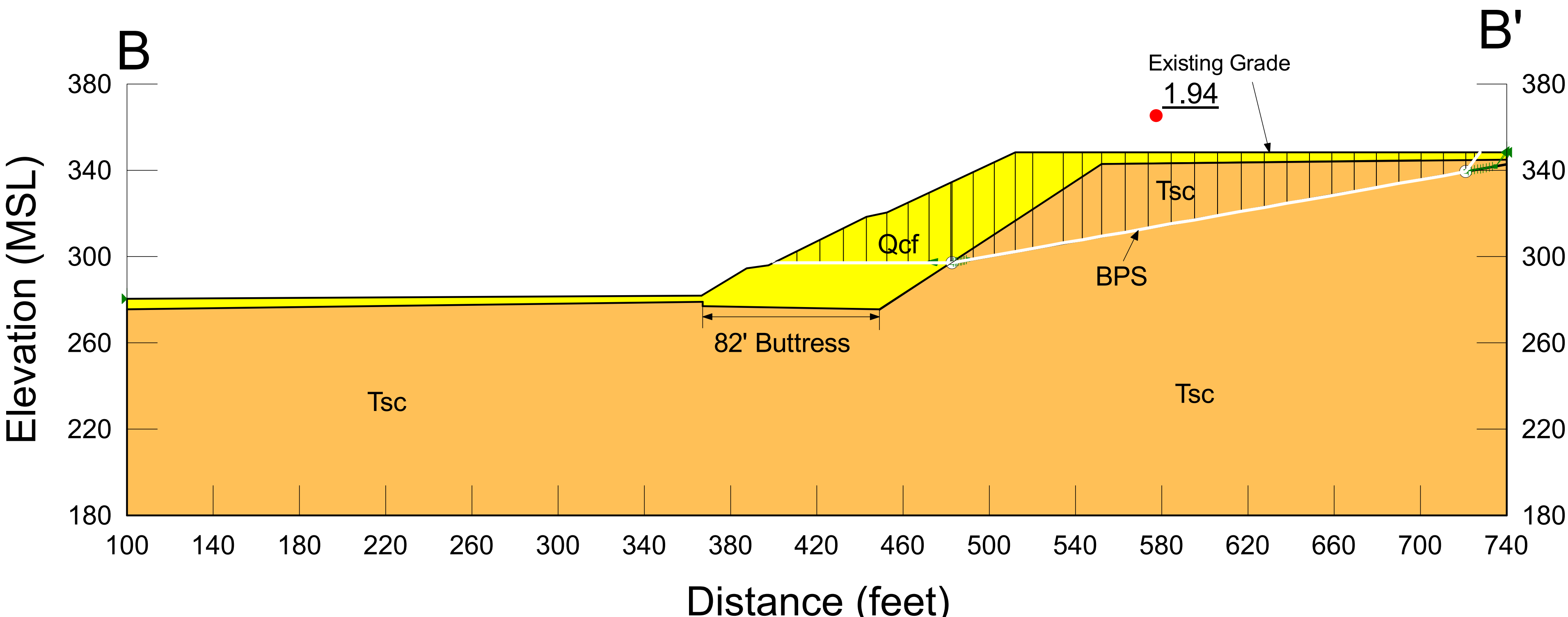


Figure C-2

Lusk Boulevard Redevelopment
Project No. G2896-52-01
Section B-B'
Name: BBc0 -120 foot buttress- entry exit Analysis-LR.gsz
Date: 07/14/2022 Time: 03:00:42 PM

Existing Condition
Static Analysis

Color	Name	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)
Red	BPS - Bedding Plane Shear	125	150	8
Yellow	Qcf - Compacted Fill	135	400	28
Orange	Tsc - Scripps Formation	135	500	32

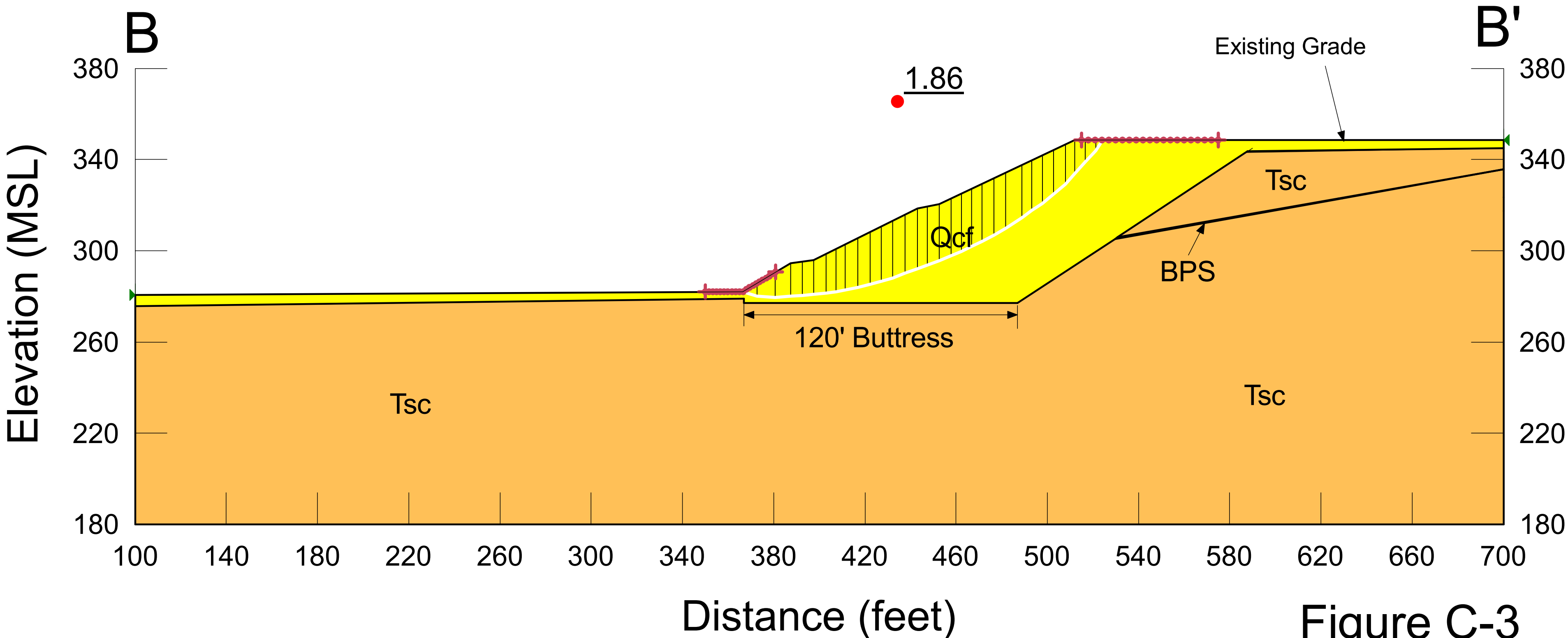


Figure C-3

Lusk Boulevard Redevelopment
Project No. G2896-52-01
Section B-B'
Name: BBc0 -120 foot buttress- Block Analysis-LR.gsz
Date: 07/14/2022 Time: 03:01:58 PM

Existing Condition
Static Analysis

	BPS - Bedding Plane Shear	125	150	8
	Qcf - Compacted Fill	135	400	28
	Tsc - Scripps Formation	135	500	32

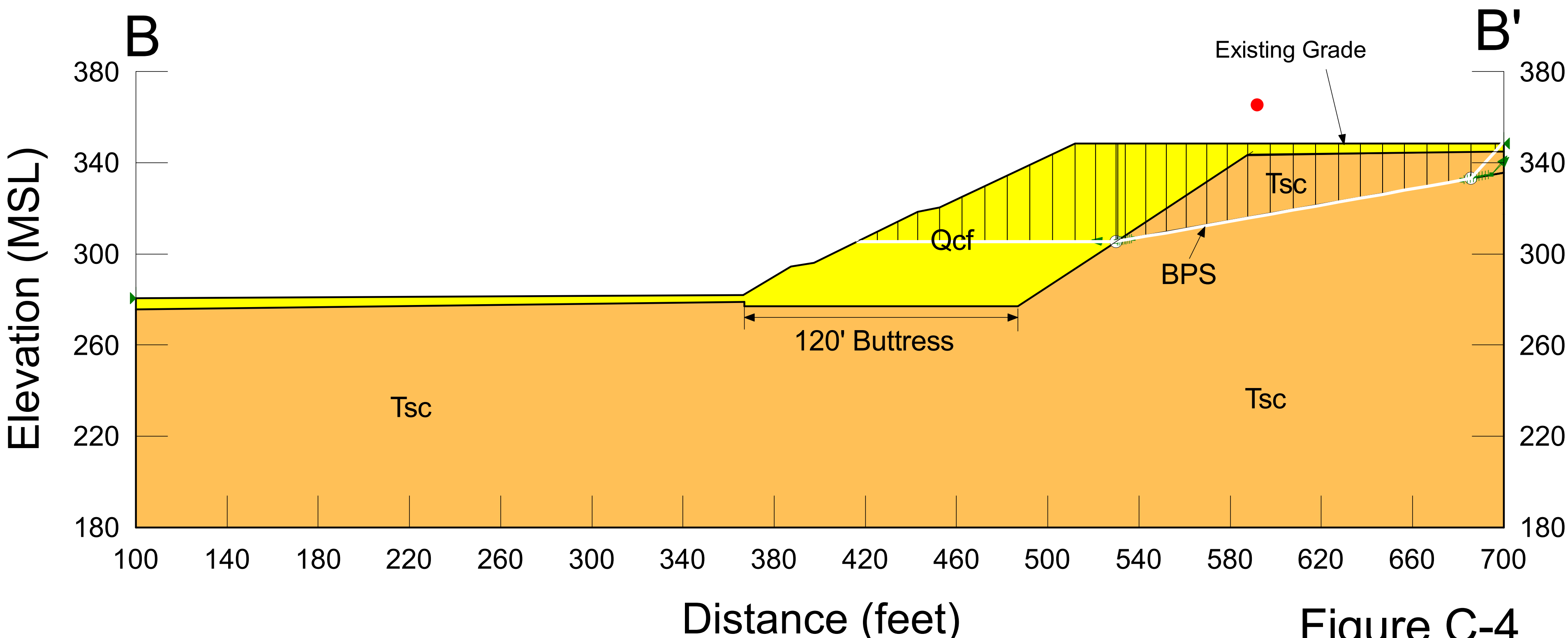


Figure C-4

APPENDIX D

APPENDIX D

RECOMMENDED GRADING SPECIFICATIONS

FOR

LUSK BUISNESS PARK REDEVELOPMENT
SAN DIEGO, CALIFORNIA

PROJECT NO. G2896-52-01

RECOMMENDED GRADING SPECIFICATIONS

1. GENERAL

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

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

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

2. DEFINITIONS

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

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

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

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

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

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

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

3. MATERIALS

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

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

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

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

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

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

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

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

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

4. CLEARING AND PREPARING AREAS TO BE FILLED

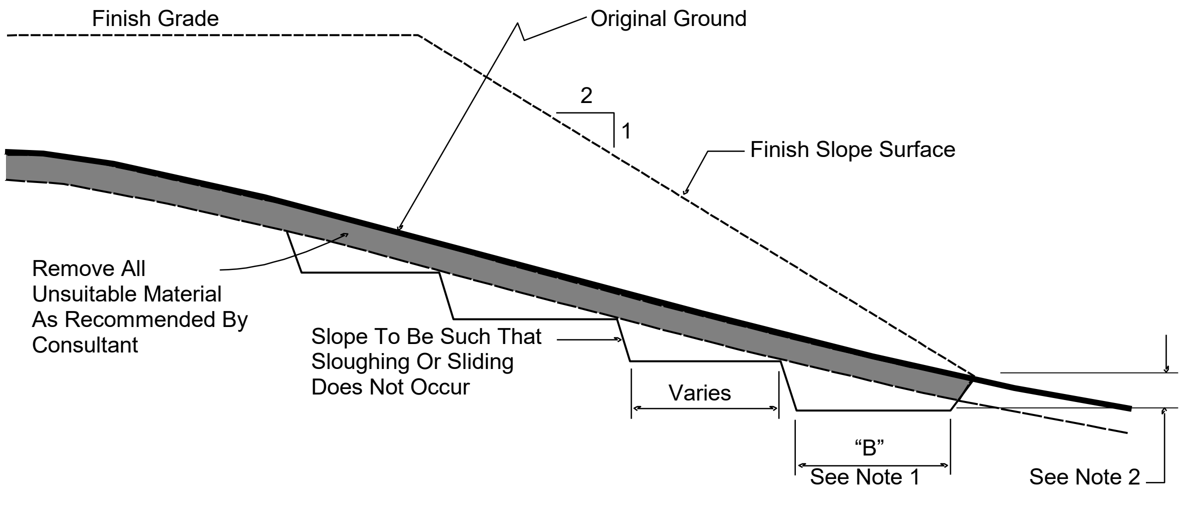
4.1 Areas to be excavated and filled shall be cleared and grubbed. Clearing shall consist of complete removal above the ground surface of trees, stumps, brush, vegetation, man-made structures, and similar debris. Grubbing shall consist of removal of stumps, roots, buried logs and other unsuitable material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

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

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

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

TYPICAL BENCHING DETAIL



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.

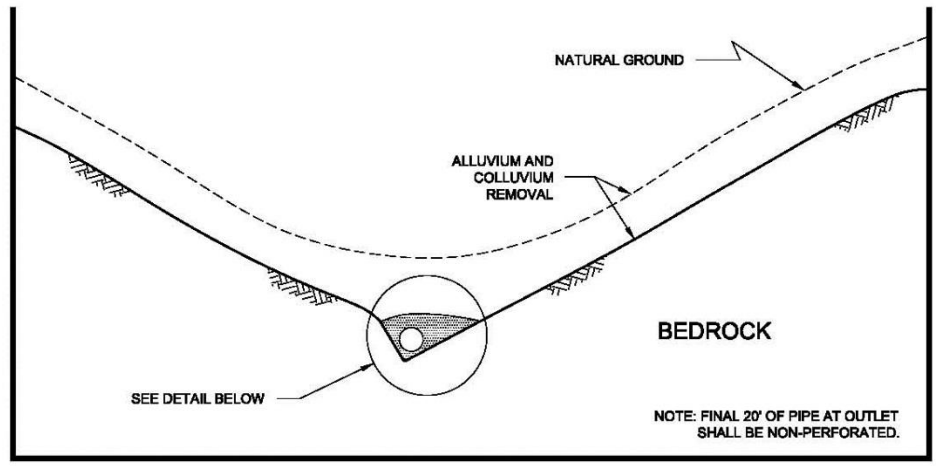
6.3.6 To reduce the potential for “piping” of fines into the *rock* fill from overlying *soil* fill material, a 2-foot layer of graded filter material shall be placed above the uppermost lift of *rock* fill. The need to place graded filter material below the *rock* should be determined by the Consultant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

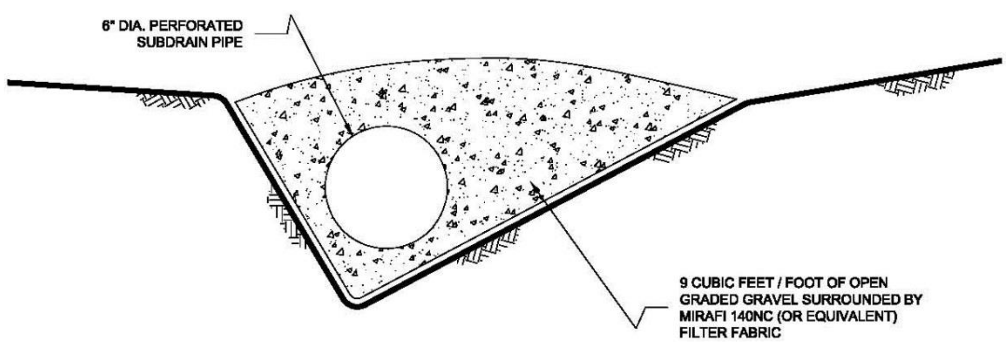
6.3.7 *Rock* fill placement should be continuously observed during placement by the Consultant.

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:

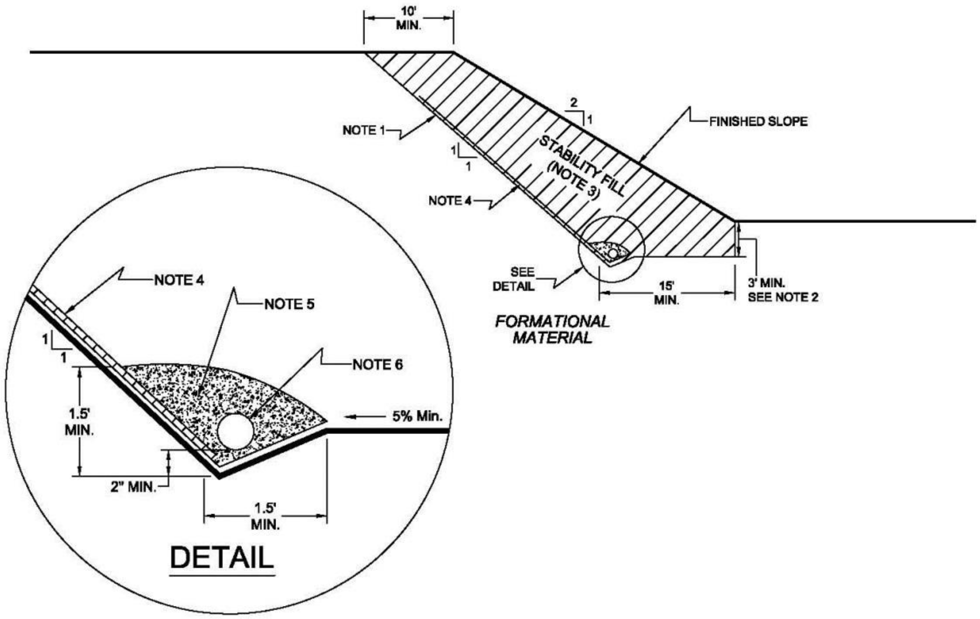
1......6-INCH DIAMETER, SCHEDULE 80 PVC PERFORATED PIPE FOR FILLS IN EXCESS OF 100-FEET IN DEPTH OR A PIPE LENGTH OF LONGER THAN 500 FEET.

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.

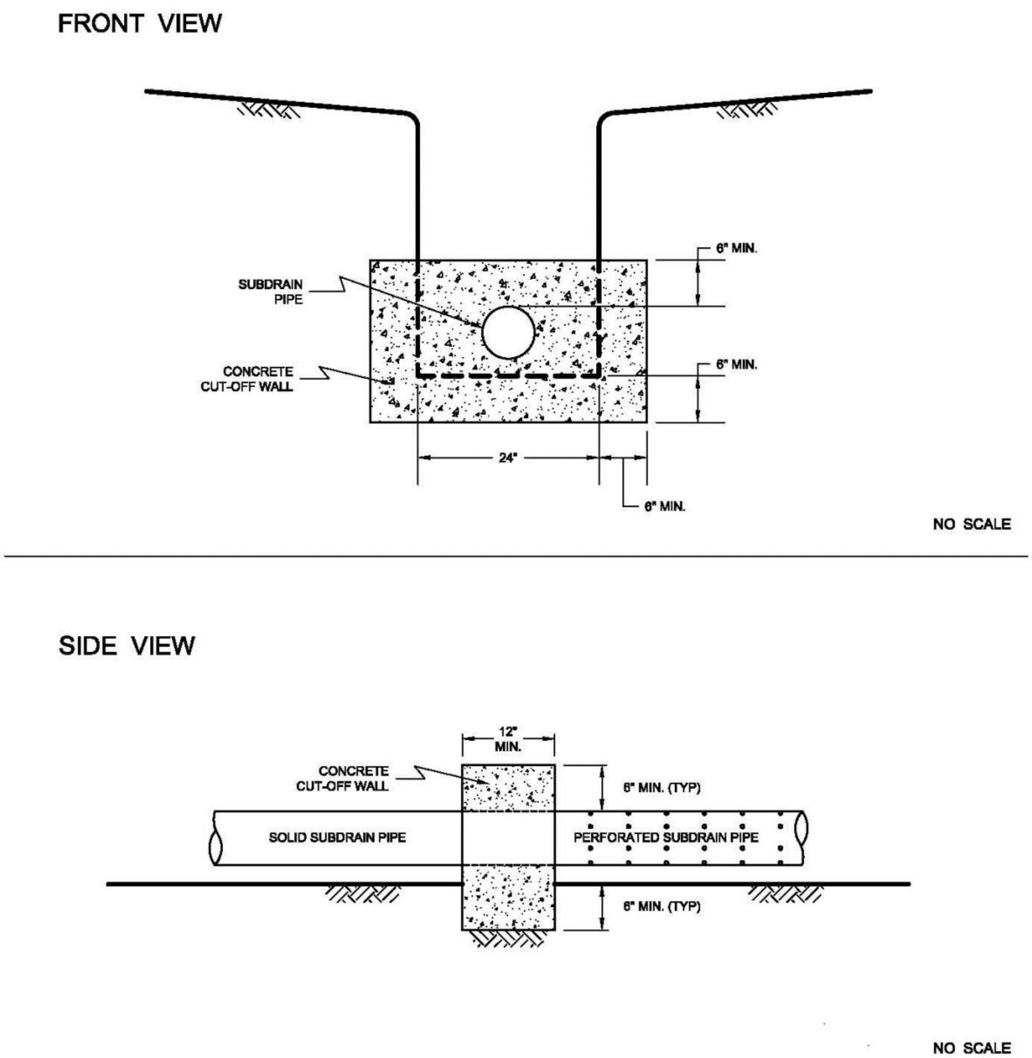
NO SCALE

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.

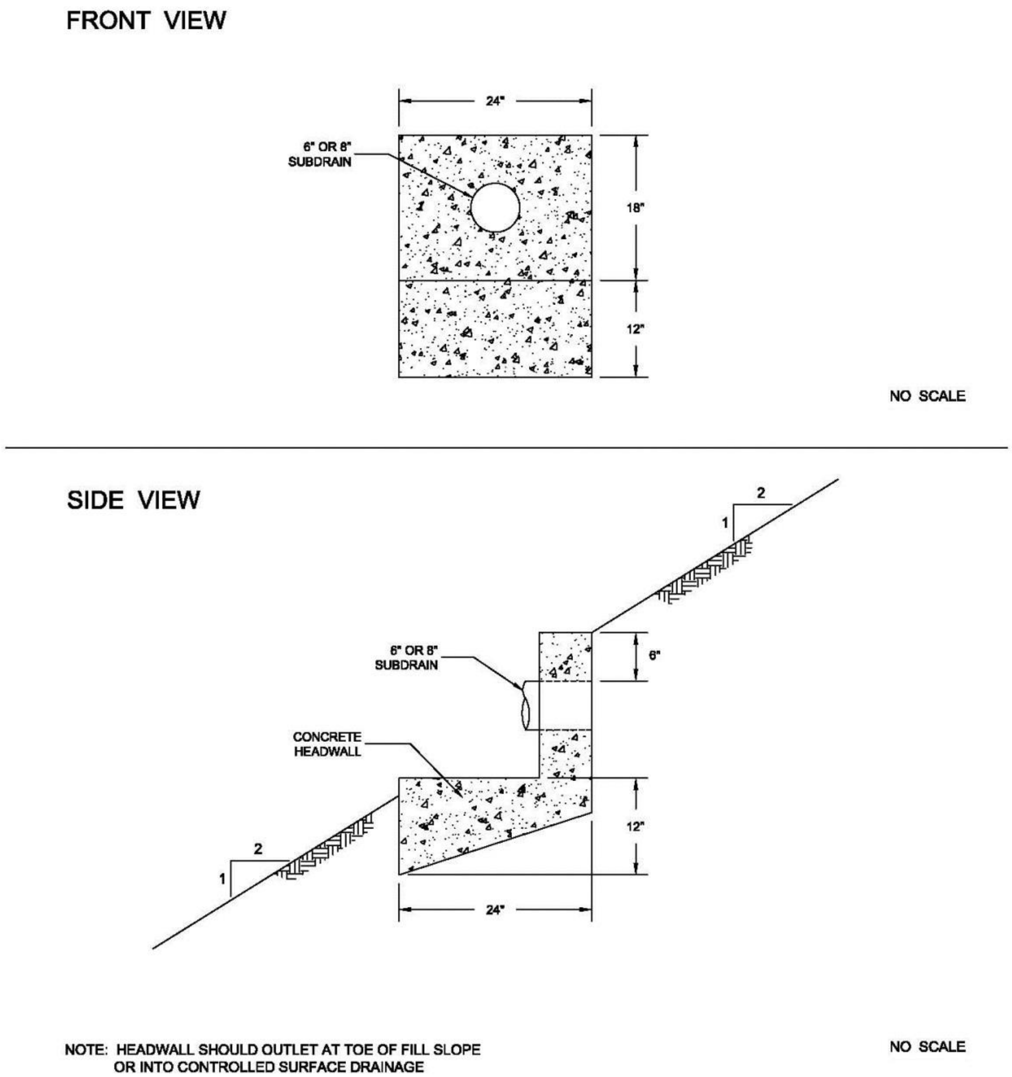
7.5 Prior to outletting, the final 20-foot segment of a subdrain that will not be extended during future development should consist of non-perforated drainpipe. At the non-perforated/ perforated interface, a seepage cutoff wall should be constructed on the downslope side of the pipe.

TYPICAL CUT OFF WALL DETAIL



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

TYPICAL HEADWALL DETAIL



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

8. OBSERVATION AND TESTING

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

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

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

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

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

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

8.6.1 Soil and Soil-Rock Fills:

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

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

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

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

9. PROTECTION OF WORK

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

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

10. CERTIFICATIONS AND FINAL REPORTS

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

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

LIST OF REFERENCES

1. *2019 California Building Code, California Code of Regulations, Title 24, Part 2, based on the 2018 International Building Code*, prepared by California Building Standards Commission, dated July 2019.

2. American Concrete Institute, *ACI 318-19, Building Code Requirements for Structural Concrete and Commentary*, dated May 2019.

3. *ACI 330-21, Commercial Concrete Parking Lots and Site Paving Design and Construction,* prepared by American Concrete Institute, dated May, 2021.

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

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

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

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

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

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

10. Moore & Taber, (1982), A*s-Built Geologic Report, Lusk Industrial Park- Unit 3, San Diego, California*, dated December 14, 1982.

11. Moore & Taber, (1982) *Final Report of Compacted Fill and Geotechnical Opinions, Unit III, Lots 87-99, Lusk Mira Mesa Business Park, San Diego, California*, dated November 2, 1982.

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

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

14. USGS computer program, Seismic Hazard Curves and Uniform Hazard Response Spectra, *http://geohazards.usgs.gov/designmaps/us/application.php.*