Appendix O

Preliminary Geotechnical Investigation for Alexan Escondido

GEOCON
INCORPORATED

GEOTECHNICAL
ENVIRONMENTAL
MATERIALS

PRELIMINARY GEOTECHNICAL INVESTIGATION

PROPOSED MULTI-STORY APARTMENT HOUSING DEVELOPMENT
855 BROTHERTON ROAD
ESCONDIDO, CALIFORNIA

PREPARED FOR

TCR
TRAMMELL CROW RESIDENTIAL

SEPTEMBER 15, 2022
PROJECT NO. G3009-52-01

Project No. G3009-52-01
September 15, 2022

Trammell Crow Residential
5790 Fleet Street, Suite 140
Carlsbad, California 92008

Attention: Mr. Conner Noon

Subject: PRELIMINARY GEOTECHNICAL INVESTIGATION
PROPOSED MULTI-STORY APARTMENT HOUSING DEVELOPMENT
855 BROTHERTON ROAD
ESCONDIDO, CALIFORNIA

Dear Mr. Noon:

In accordance with your request and authorization of our Proposal No. LG-22316 dated July 7, 2022, we herein submit the results of our geotechnical investigation for the subject project. We performed our investigation to evaluate the underlying soil and geologic conditions and potential geologic hazards, and to assist in the design of the proposed 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 grading plans have been prepared and building limits have been defined.

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 multi-story, multi-family, apartment housing development located at 855 Brotherton Road in Escondido, California (see Vicinity Map).

Vicinity Map

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

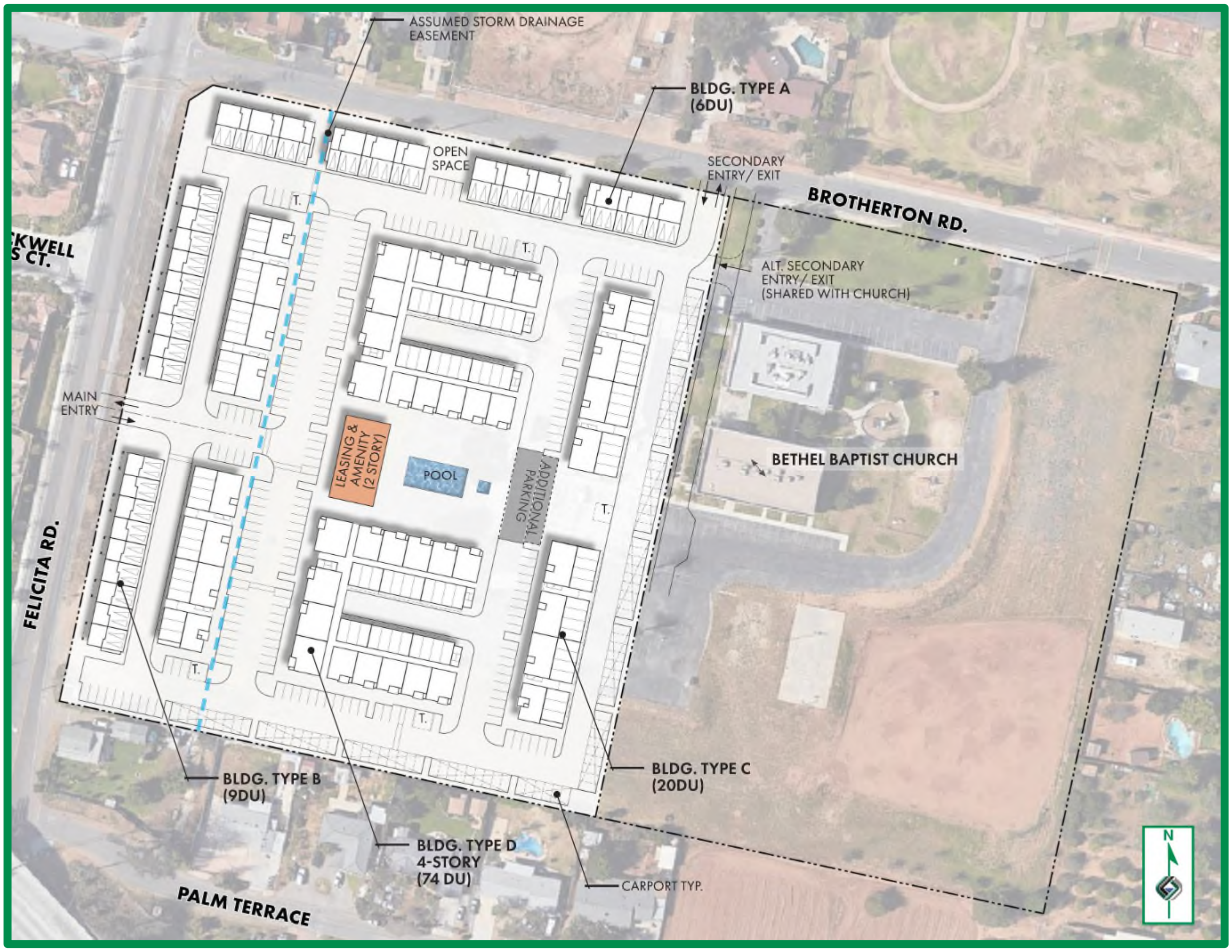
The scope of this investigation included reviewing readily available published and unpublished geologic literature (see List of References), performing engineering analyses and preparing this report. We also advanced 8 exploratory borings and 2 infiltration borings to a maximum depth of about 20 feet, performed percolation/infiltration testing, sampled soil, and performed laboratory testing. Appendix A presents the exploratory boring logs and details of the field investigation. The details of the laboratory tests and a summary of the test results are shown in Appendix B and on the boring logs in Appendix A.

2. SITE AND PROJECT DESCRIPTION

The property is east of Felicita Road, south of Brotherton Road, west of an existing private school building, and north of residential properties in the City of Escondido, California. The developed portion of the site consists of an existing church building that sits atop a nearly level pad with relative flat parking and drivelane surfaces immediately surrounding the building. Undeveloped land exists to the west and south of the existing church complex. Existing elevations around the building and pavement areas range from approximately 656 to 662 feet above Mean Sea Level (MSL). Moderately steep fill slopes descend to the west and south from the west and south pavement area boundaries with approximate inclinations ranging from 2:1 to 3:1 (horizontal to vertical) and a maximum height of about 15 feet. The site appears to be terraced into what was originally gently to moderately sloping, west descending natural ground. It appears the site generally drains to the southwest. The Existing Site Map shows the current site conditions.

Existing Site Map

Based on review of the conceptual plans provided, we understand the project will consist of demolishing the existing building and improvements and constructing several multi-story, multi-family residential apartment buildings with associated improvements and landscaping. Storm water BMPs are not included in the current plans. The Conceptual Site Plan is provided herein.



Conceptual Site Plan

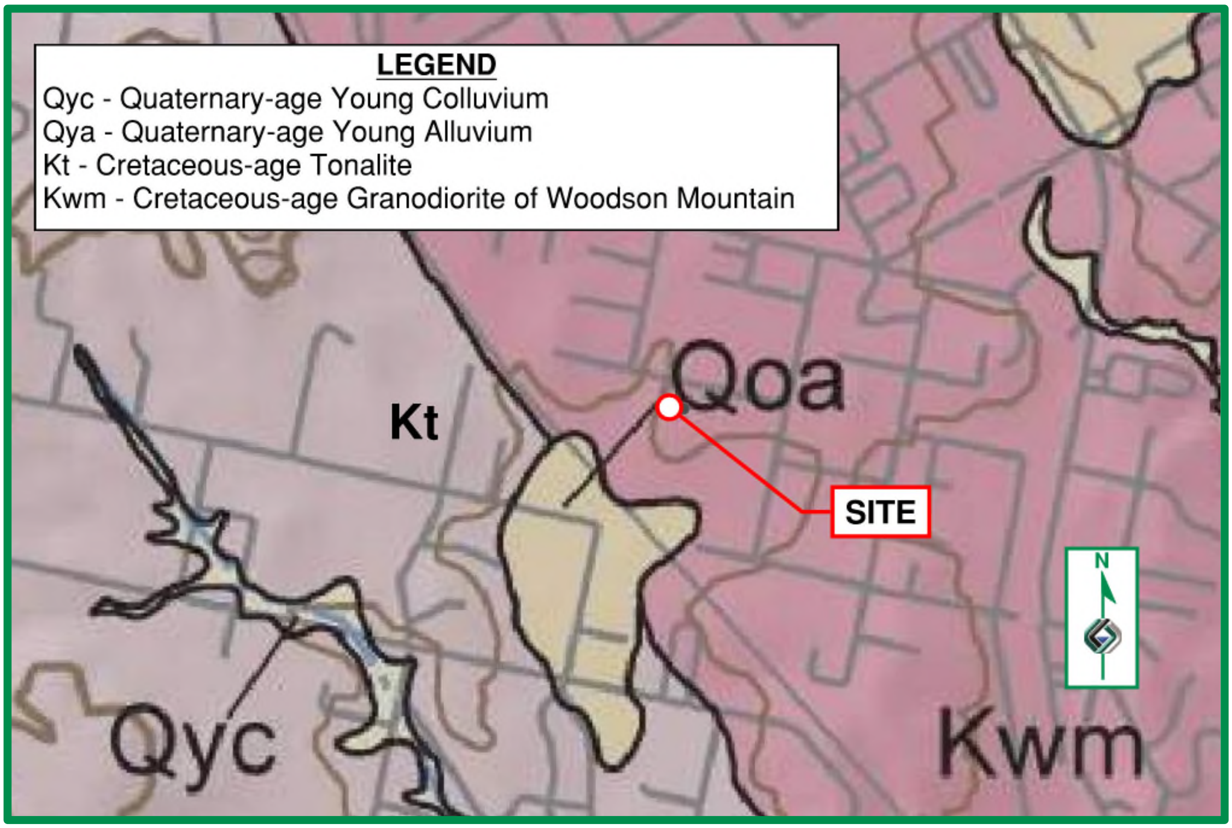
The locations, site descriptions, and proposed development are based on our site reconnaissance, review of published geologic literature, field investigations, and discussions with project personnel. If development plans differ from those described herein, Geocon Incorporated should be contacted for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

3. 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 active Rose Canyon/Inglewood Fault Zone to the west and Elsinore Fault Zone to the northeast.

The site is located on the northeastern portion of the coastal plain. Locally mapped Cretaceous-age Granitoid Bedrock known as the Granodiorite of Woodson Mountain (Kennedy and Tan, 2007; Map Symbol: Kwm) was encountered on the site underlying existing undocumented fill. The Regional Geologic Map shows the geologic units in the area of the site.



Regional Geologic Map

4. SOIL AND GEOLOGIC CONDITIONS

We encountered one surficial soil unit during exploratory drilling (consisting of undocumented fill) and one formational unit (consisting of Granitoid bedrock). 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 surficial soil and geologic units are described herein in order of increasing age.

4.1 Undocumented Fill (Qudf)

We encountered undocumented fill in our exploratory borings to depths ranging from about 2 to 13 feet. In general, the fill consists of loose to medium dense, dry to moist, silty and clayey, fine- to medium-grained sand. Laboratory testing indicates the encountered fill possesses a “very low” to “low” expansion potential (expansion index of 50 or less). The undocumented fill is not considered suitable in its current condition for the support of foundations or structural fill and remedial grading will required. The undocumented fill can be reused for new compacted fill during grading operations provided it is generally free of roots and debris.

4.2 Granodiorite of Woodson Mountain (Kwm)

Cretaceous-age granitoid bedrock underlies the undocumented fill and extend below the maximum depth explored of about 20 feet. The granitoid bedrock, as encountered, is generally slightly weathered, moderately weak, and intensely fractured, and excavates as a dry to damp, clayey, well graded sand. Additionally, the granitoid bedrock appears to become less weathered with increasing depth. We did not encounter drilling refusal in the exploratory borings. The excavated granitoid bedrock can be used for new compacted fill during grading operations provided it is generally free of roots and debris. The undisturbed granitoid bedrock is generally suitable for support of proposed fill and structural loads.

5. GROUNDWATER

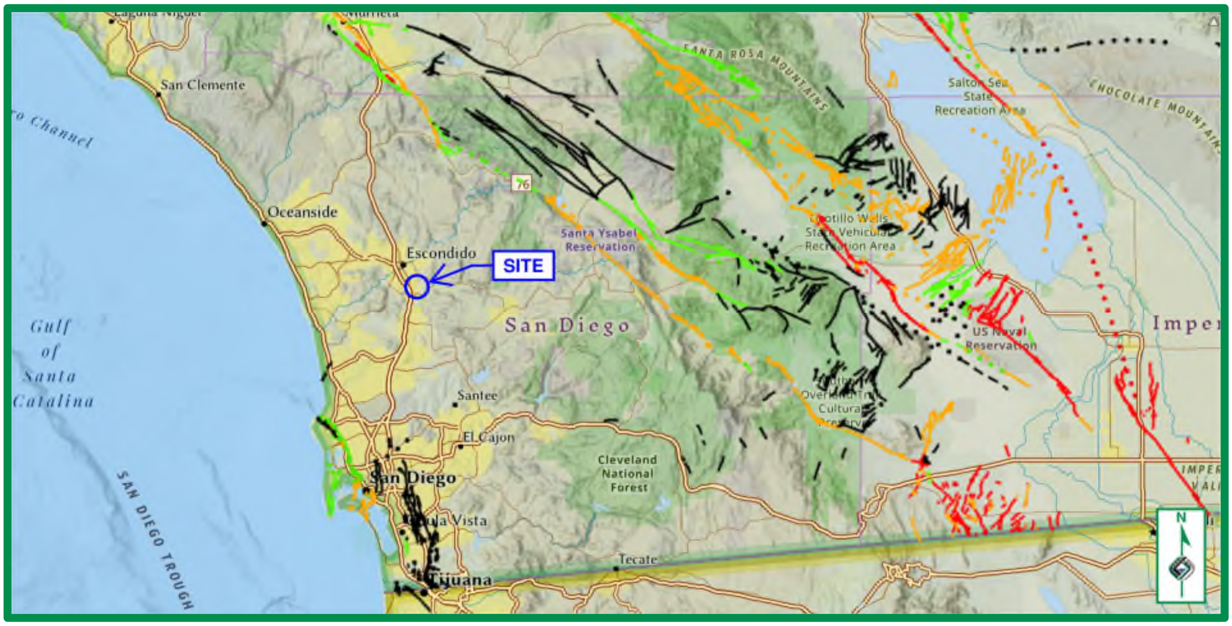
We did not encounter groundwater or seepage during our site investigation. However, it is not uncommon for shallow seepage conditions to develop where none previously existed when sites are irrigated or infiltration is implemented. Seepage is dependent on seasonal precipitation, irrigation, land use, 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 40 to 50 feet below existing grade. We do not expect groundwater to be encountered during construction of the proposed development.

6. GEOLOGIC HAZARDS

6.1 Regional Faulting and Seismicity

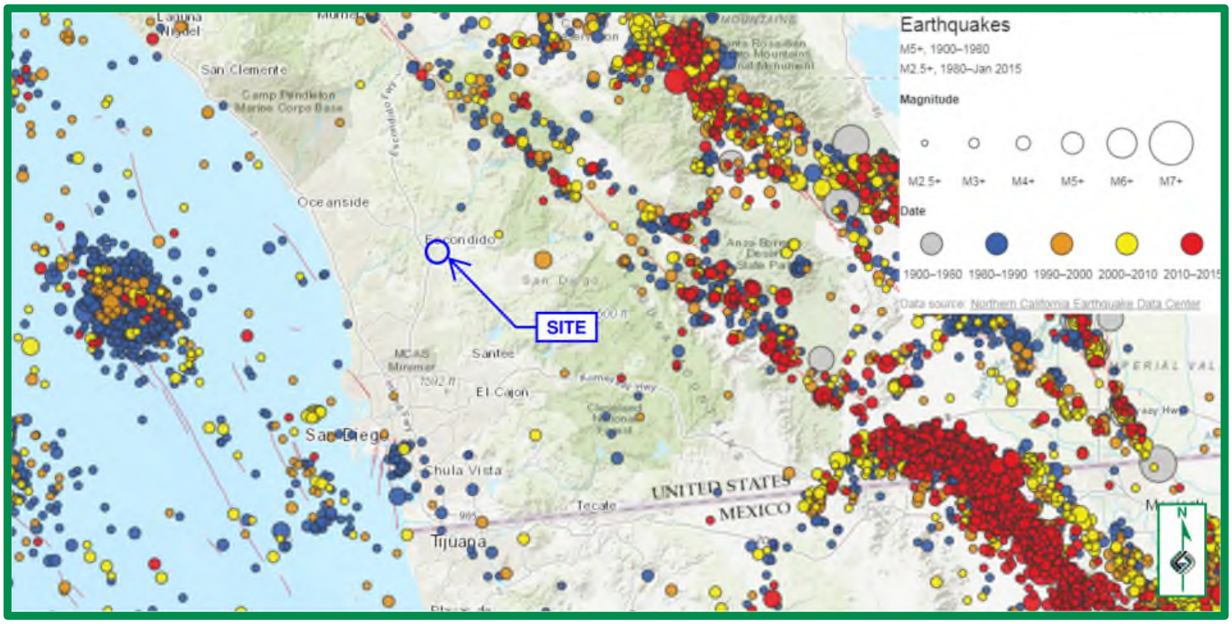
A review of the referenced geologic materials and our knowledge of the general area indicate the site is not underlain by active, potentially active, or inactive faults. An active fault is defined by the California Geological Survey (CGS) as a fault showing evidence for activity within the last 11,700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

6.2 Ground Rupture

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

6.3 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 granitoid bedrock, liquefaction potential for the site is considered very low.

6.4 Storm Surge, Tsunamis, and Seiches

Storm surges are large ocean waves that sweep across coastal areas when storms make landfall. Storm surges can cause inundation, severe erosion and backwater flooding along the waterfront. The site is located over

12 miles from the Pacific Ocean and is at an elevation of about 640 feet or greater above MSL. Therefore, the potential of storm surges affecting the site is considered low.

A tsunami is a series of long period waves generated in the ocean by a sudden displacement of large volumes of water. Causes of tsunamis include underwater earthquakes, volcanic eruptions, or offshore slope failures. The potential for the site to be affected by a tsunami is negligible due to the distance from the Pacific Ocean and the site elevation.

A seiche is a run-up of water within a lake or embayment triggered by fault- or landslide-induced ground displacement. The site is not located in the vicinity of or downstream from such bodies of water. Therefore, the risk of seiches affecting the site is negligible.

6.5 Slope Stability

It is anticipated that temporary and/or permanent cut slopes will be utilized along the eastern development boundary. Slope stability analyses for the proposed cut slopes with inclinations as steep as 2:1 (horizontal to vertical) indicate a calculated factor of safety of at least 1.5 under static conditions for both deep-seated and surficial failure. Table 6.5.1 presents the slope stability analysis for the proposed sloping conditions.

TABLE 6.5.1
SLOPE STABILITY EVALUATION

Parameter	Value
Slope Height, H	20 Feet
Slope Inclination, I (Horizontal to Vertical)	2:1
Total Soil Unit Weight, γ	130 pcf
Friction Angle, ϕ	31 Degrees
Cohesion, C	525 psf
Slope Factor $\gamma_{C\phi}$= $(\gamma H \tan\phi)/C$	3.0
NCf (From Chart)	14
Factor of Safety = $(N_{Cf}C)/(\gamma H)$	2.8

Table 6.5.2 presents the surficial slope stability analysis for the proposed sloping conditions.

TABLE 6.5.2
SURFICIAL SLOPE STABILITY EVALUATION

Parameter	Value
Slope Height, H	∞
Vertical Depth of Saturation, Z	5 Feet
Slope Inclination, I (Horizontal to Vertical)	2:1 (26.6 Degrees)
Total Soil Unit Weight, γ	130 pcf
Water Unit Weight, γ_W	62.4 pcf
Friction Angle, ϕ	31 Degrees
Cohesion, C	525 psf
Factor of Safety = $(C+(\gamma+\gamma_W)Z\cos^2 I \tan\phi)/(\gamma Z \sin I \cos I)$	2.6

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.

6.6 Landslides

We did not observe evidence of previous or incipient slope instability at the site during our study and the encountered very dense granitoid bedrock materials are generally not considered susceptible to deep-seated slope instability. Published geologic mapping indicates landslides are not present on or adjacent to the site. Therefore, in our professional opinion, the potential for a landslide is not a significant concern for this project.

6.7 Erosion

Provided the engineering recommendations herein are followed and the project civil engineer prepares the grading plans in accordance with generally accepted regional standards, we do not expect erosion to be a major impact to site development. In addition, we expect the proposed development would not increase the potential for erosion if properly designed.

7. CONCLUSIONS AND RECOMMENDATIONS

7.1 General

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

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

7.1.3 The undocumented fill is potentially compressible and unsuitable in its present condition for the support of new compacted fill or proposed settlement-sensitive improvements. Remedial grading of these materials should be performed as discussed herein. The underlying very dense granitoid bedrock is suitable for support of new compacted fill or proposed settlement-sensitive improvements.

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

7.1.5 Excavation of the existing fill should generally be possible with moderate effort using conventional, heavy-duty equipment during grading and trenching operations. Based on the drilling characteristics within the explored portions of the encountered granitoid bedrock, we anticipate that rippability is variable and ranges between moderate, near the surface, to difficult, with increasing depth. We do not expect a significant amount of oversized rock material would be generated within the upper portions of the encountered granitoid bedrock. However, although not encountered during exploratory drilling, localized very hard corestones may exist within the granitoid bedrock that may require heavy duty breaking equipment. We do not expect a rock breaking and blasting program will be required for the proposed grading operations. However, the grading contractor should be prepared to handle localized strong rock areas and rock corestones, if encountered.

7.1.6 In general, cut slopes composed of granitoid bedrock should possess factors of safety at least 1.5 at inclinations of 2:1 (horizontal to vertical), or flatter. We should observe the geologic structure of cut slopes composed of granitic rock during grading operations to evaluate stability.

7.1.7 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.

7.1.8 We should prepare a storm water management investigation under a separate report to help evaluate the potential for infiltration on the property. The project civil engineer should use that report to help design the storm water management devices.

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

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

7.2 Excavation and Soil Characteristics

7.2.1 Excavation of the existing fill should be possible with moderate effort using conventional, heavy-duty equipment during grading and trenching operations. Based on the drilling characteristics within the explored depths of the encountered granitoid bedrock, we expect the rippability is variable and ranges between moderate (near the surface) to difficult increasing depth. We do not expect a significant amount of oversized rock material would be generated within the upper portions of the encountered granitoid bedrock. However, although not encountered during exploratory drilling, localized very hard corestones may exist within the granitoid bedrock that may require heavy duty breaking equipment. We do not expect a rock breaking and blasting program will be required for the proposed grading operations. However, the grading contractor should be prepared to handle localized strong rock areas and rock corestones, if encountered. Oversized rock (rocks greater than 12 inches in dimension) may be generated with the granitic rock materials that can be incorporated into landscape use or placed in deep compacted fill areas, if available. The grading and improvement contractors should review this report and evaluate the proper equipment to use for the planned excavations.

7.2.2 Our laboratory test results indicate that the fill soils encountered during the field investigation are considered to be “non-expansive” and “expansive” (expansion index [EI] of 20 or less and EI greater than 20, respectively) as defined by 2019 California Building Code (CBC) Section 1803.5.3. Based on the test results, we expect a majority of the soil encountered possess a “very low” to “low” expansion potential (EI of 50 or less) in accordance with ASTM D 4829. Table 7.2 presents soil classifications based on the expansion index.

TABLE 7.2
EXPANSION CLASSIFICATION BASED ON EXPANSION INDEX

Expansion Index (EI)	ASTM D 4829 Expansion Classification	2019 CBC Expansion Classification
0 – 20	Very Low	Non-Expansive
21 – 50	Low	Expansive
51 – 90	Medium	
91 – 130	High	
Greater Than 130	Very High	

7.2.3 We performed laboratory tests on samples of the site materials to evaluate the percentage of water-soluble sulfate content. Appendix B presents results of the laboratory water-soluble sulfate content tests. The test results indicate the on-site materials at the locations tested possess "S0" sulfate exposure to concrete structures as defined by 2019 CBC Section 1904 and ACI 318-19 Chapter 19. The presence of water-soluble sulfates is not a visually discernible characteristic; therefore, other soil samples from the site could yield different concentrations. Additionally, over time landscaping activities (i.e., addition of fertilizers and other soil nutrients) may affect the concentration.

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

7.3 Grading

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

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

7.3.3 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.

7.3.4 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.

7.3.5 We expect the proposed buildings will be on grade (i.e. no subterranean levels) and constructed on a shallow foundation system embedded in properly compacted fill. The undocumented fill within the area of the planned development limits should be removed to expose the underlying granitic rock. In addition, the building pads should be undercut at least 3 feet (such that at least 3 feet of fill exists below proposed grades). The removals should also extend to a depth of at least 2 feet below the proposed foundations (whichever results in a deeper excavation). The excavations should then be backfilled with properly compacted fill to achieve design grade. The excavations should extend at least 10 feet laterally outside of the proposed foundation zones, where feasible. The excavations should extend at least 2 feet laterally outside of the improvement area, where feasible. Table 7.3.1 provides a summary of the remedial grading recommendations.

TABLE 7.3.1
SUMMARY OF REMEDIAL GRADING RECOMMENDATIONS

Area	Remedial Grading Excavation Requirements
Building Pads	Excavate Undocumented Fill to Expose Granitic Rock
	Excavate at Least 3 Feet Below Proposed Grade
	Excavate at Least 2 Feet Below Foundations
Site Improvements	Excavate Undocumented Fill to Expose Granitic Rock
Lateral Grading Limits	10 Feet Outside of Buildings
	2 Feet Outside of Improvement Areas
Exposed Bottoms of Excavations	Scarify Upper 12 Inches

7.3.6 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 excavations may be required if saturated or loose material is encountered. A representative of Geocon should be on-site during excavations to evaluate the limits of the remedial grading.

7.3.7 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).

7.3.8 After the remedial excavations, the site should then be brought to final subgrade elevations with fill compacted in layers. In general, the existing fill soil and processed granitoid bedrock 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 D1557. Fill materials placed below optimum moisture content may require additional moisture conditioning prior to placing additional fill. Excavations within the granitic rock will locally generate dry to damp sand that will require the addition of significant amounts of water to achieve optimum moisture content. The upper 12 inches of subgrade soil underlying pavement areas should be compacted to a dry density of at least 95 percent of the laboratory maximum dry density near to slightly above optimum moisture content shortly before paving operations.

7.3.9 Import fill (if necessary) should consist of the characteristics presented in Table 7.3.2. 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 7.3.2
SUMMARY OF IMPORT FILL RECOMMENDATIONS

Soil Characteristic	Values
Expansion Potential	"Very Low" to "Low" (Expansion Index of 50 or less)
Particle Size	Maximum Dimension Less Than 3 Inches
	Generally Free of Debris

7.4 Subdrains

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

7.5 Temporary Excavations

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

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

7.6 Seismic Design Criteria – 2019 California Building Code

7.6.1 Table 7.6.1 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).

TABLE 7.6.1
2019 CBC SEISMIC DESIGN PARAMETERS

Parameter	Value	2019 CBC Reference
Site Class	C	Section 1613.2.2
MCE_R Ground Motion Spectral Response Acceleration – Class B (short), S_S	0.875g	Figure 1613.2.1(1)
MCE_R Ground Motion Spectral Response Acceleration – Class B (1 sec), S_1	0.321g	Figure 1613.2.1(2)
Site Coefficient, F_A	1.2	Table 1613.2.3(1)
Site Coefficient, F_V	1.5*	Table 1613.2.3(2)
Site Class Modified MCE_R Spectral Response Acceleration (short), S_{MS}	1.05g	Section 1613.2.3 (Eqn 16-36)
Site Class Modified MCE_R Spectral Response Acceleration – (1 sec), S_{M1}	0.481g*	Section 1613.2.3 (Eqn 16-37)
5% Damped Design Spectral Response Acceleration (short), S_{DS}	0.7g	Section 1613.2.4 (Eqn 16-38)
5% Damped Design Spectral Response Acceleration (1 sec), S_{D1}	0.321g*	Section 1613.2.4 (Eqn 16-39)

*See following paragraph.

7.6.2 Using the code-based values presented in this Table 7.7.1, in lieu of a performing a ground motion hazard analysis, requires the exceptions outlined in ASCE 7-16 Section 11.4.8 be followed by the project structural engineer. Per Section 11.4.8 of ASCE/SEI 7-16, a ground motion hazard analysis should be performed for projects for Site Class "E" sites with Ss greater than or equal to 1.0g and for Site Class "D" and "E" sites with S1 greater than 0.2g. Section 11.4.8 also provides exceptions which indicates that the ground motion hazard analysis may be waived provided the exceptions are followed.

7.6.3 Table 7.6.2 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 7.6.2
ASCE 7-16 PEAK GROUND ACCELERATION

Parameter	Value	ASCE 7 16 Reference
Mapped MCE_G Peak Ground Acceleration, PGA	0.377g	Figure 22-9
Site Coefficient, F_{PGA}	1.2	Table 11.8-1
Site Class Modified MCE_G Peak Ground Acceleration, PGA_M	0.452g	Section 11.8.3 (Eqn 11.8-1)

7.6.4 Conformance to the criteria in Tables 7.6.1 and 7.6.2 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.

7.6.5 The project structural engineer and architect should evaluate the appropriate Risk Category and Seismic Design Category for the planned structures. The values presented herein assume a Risk Category of II and resulting in a Seismic Design Category D. Table 7.6.3 presents a summary of the risk categories in accordance with ASCE 7-16.

TABLE 7.6.3
ASCE 7-16 RISK CATEGORIES

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

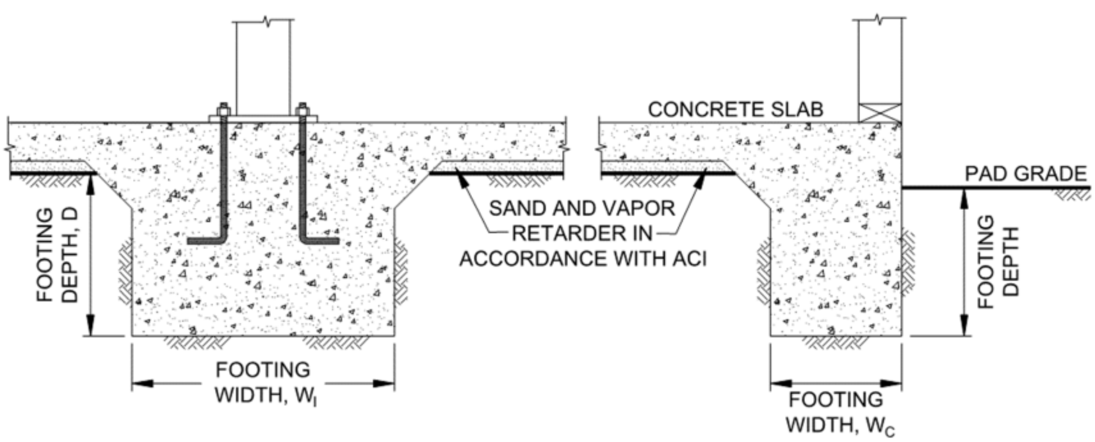
7.7 Shallow Foundations

7.7.1 The proposed structures can be supported on a shallow foundation system founded in properly compacted fill. Foundations for the structure should consist of continuous strip footings and/or isolated spread footings. Table 7.7 provides a summary of the foundation design recommendations.

TABLE 7.7
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
Bearing Capacity Increase	500 psf per Foot of Depth
	300 psf per Foot of Width
Maximum Allowable Bearing Capacity	4,500 psf
Estimated Total Settlement	½ Inch
Estimated Differential Settlement	½ Inch in 40 Feet
Footing Size Used for Settlement	10-Foot Square
Design Expansion Index	50 or less

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



Wall/Column Footing Dimension Detail

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

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

- For fill slopes less than 20 feet high, building footings should be deepened such that the bottom outside edge of the footing is at least 7 feet horizontally from the face of the slope.
- When located next to a descending 3:1 (horizontal to vertical) fill slope or steeper, the foundations should be extended to a depth where the minimum horizontal distance is equal to H/3 (where H equals the vertical distance from the top of the fill slope to the base of the fill soil) with a minimum of 10 feet but need not exceed 40 feet. The horizontal distance is measured from the outer, deepest edge of the footing to the face of the slope. An acceptable alternative to deepening the footings would be the use of a post-tensioned slab and foundation system or increased footing and slab reinforcement. Specific design parameters or recommendations for either of these alternatives can be provided once the building location and fill slope geometry have been determined.
- If swimming pools are planned, Geocon Incorporated should be contacted for a review of specific site conditions.
- Swimming pools located within 10 feet of the top of cut or fill slopes are not recommended. Where such a condition cannot be avoided, the portion of the swimming pool wall within 10 feet of the slope face be designed assuming that the adjacent soil provides no lateral support. This recommendation applies to fill slopes up to 30 feet in height, and cut slopes regardless of height. For swimming pools located near the top of fill slopes greater than 30 feet in height, additional recommendations may be required and Geocon Incorporated should be contacted for a review of specific site conditions.
- Although other improvements, which are relatively rigid or brittle, such as concrete flatwork or masonry walls, may experience some distress if located near the top of a slope, it is generally not economical to mitigate this potential. It may be possible, however, to incorporate design measures that would permit some lateral soil movement without causing extensive distress. Geocon Incorporated should be consulted for specific recommendations.

7.7.5 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.

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

7.8 Concrete Slabs-On-Grade

7.8.1 Concrete slabs-on-grade for the structures should be constructed in accordance with Table 7.8.

TABLE 7.8
MINIMUM CONCRETE SLAB-ON-GRADE RECOMMENDATIONS

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

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

7.8.3 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.

7.8.4 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.

7.8.5 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.

7.8.6 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.

7.8.7 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.

7.8.8 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.

7.9 Exterior Concrete Flatwork

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

TABLE 7.9
MINIMUM CONCRETE FLATWORK RECOMMENDATIONS

Expansion Index, EI	Minimum Steel Reinforcement* Options	Minimum Thickness
EI $\leq$ 90	6x6-W2.9/W2.9 (6x6-6/6) Welded Wire Mesh	4 Inches
	No. 3 Bars 18 Inches on Center, Both Directions	

*In excess of 8 feet square.

7.9.2 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 D1557.

7.9.3 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.

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

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

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

7.10 Retaining Walls

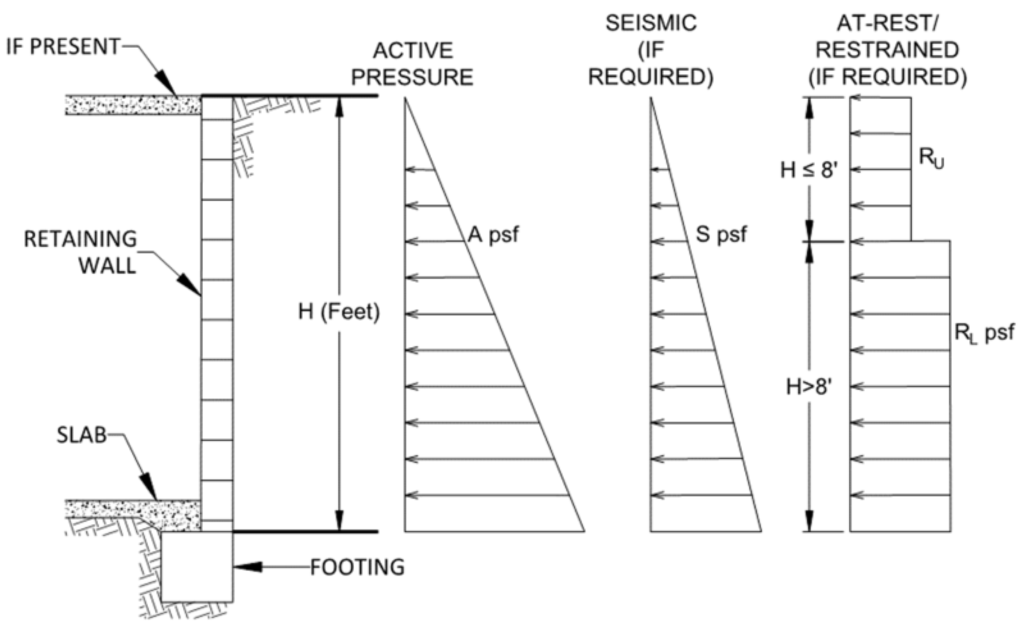
7.10.1 Retaining walls should be designed using the values presented in Table 7.10.1. Soil with an expansion index (EI) of greater than 50 should not be used as backfill material behind retaining walls.

TABLE 7.10.1
RETAINING WALL DESIGN RECOMMENDATIONS

Parameter	Value
Active Soil Pressure, A (Fluid Density, Level Backfill)	35 pcf
Active Soil Pressure, A (Fluid Density, 2:1 Sloping Backfill)	50 pcf
Seismic Pressure, S	15H psf
At-Rest/Restrained Walls Additional Uniform Pressure, R_U (0 to 8 Feet High)	7H psf
At-Rest/Restrained Walls Additional Uniform Pressure, R_L (8+ Feet High)	13H psf
Expected Expansion Index for the Subject Property	EI≤50

H equals the height of the retaining portion of the wall

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



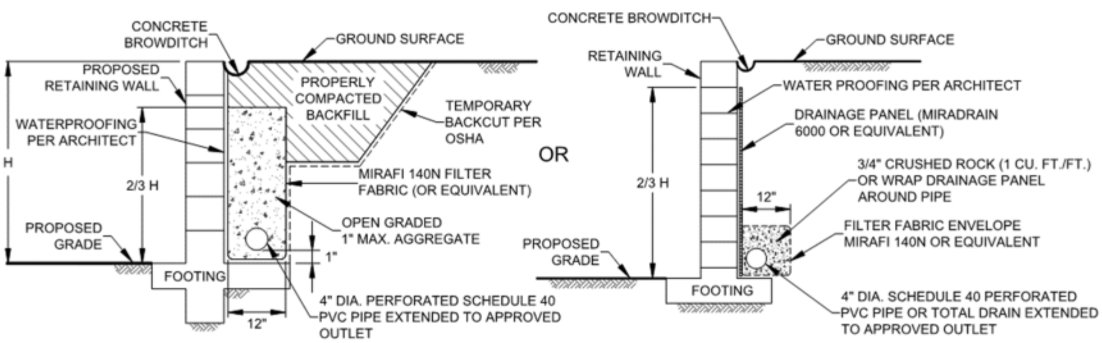
Retaining Wall Loading Diagram

7.10.3 Unrestrained walls are those that are allowed to rotate more than 0.001H (where H equals the height of the retaining portion of the wall) at the top of the wall. Where walls are restrained from movement at the top (at-rest condition), an additional uniform pressure should be applied to the wall. For retaining walls subject to vehicular loads within a horizontal distance equal to two-thirds the wall height, a surcharge equivalent to 2 feet of fill soil should be added to the upper 10 feet of the retaining wall.

7.10.4 The structural engineer should determine the Seismic Design Category for the project in accordance with Section 1613.3.5 of the 2019 CBC or Section 11.6 of ASCE 7-16. For structures assigned to Seismic Design Category of D, E, or F, retaining walls that support more than 6 feet of backfill should be designed with seismic lateral pressure in accordance with Section 1803.5.12 of the 2019 CBC. The seismic load is dependent on the retained height where H is the height of the wall, in feet, and the calculated loads result in pounds per square foot (psf) exerted at the base of the wall and zero at the top of the wall.

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

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



Typical Retaining Wall Drainage Detail

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

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

TABLE 7.10.2
SUMMARY OF SITE RETAINING WALL FOUNDATION RECOMMENDATIONS

Parameter	Value
Minimum Retaining Wall Foundation Width	12 inches
Minimum Retaining Wall Foundation Depth	12 Inches
Minimum Steel Reinforcement	Per Structural Engineer
Allowable Bearing Capacity	2,500 psf
Bearing Capacity Increase	500 psf per Foot of Depth
	300 psf per Foot of Width
Maximum Allowable Bearing Capacity	4,500 psf
Estimated Total Settlement	½ Inch
Estimated Differential Settlement	½ Inch in 40 Feet

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

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

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

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

7.11 Mechanically Stabilized Earth (MSE) Retaining Walls

7.11.1 Mechanized stabilized earth (MSE) retaining walls can be used on the property. MSE retaining walls are alternative walls that consist of modular block facing units with geogrid reinforced earth behind the block. The reinforcement grid attaches to the block units and is typically placed at specified vertical intervals and embedment lengths. The grid length and spacing will be determined by the wall designer. The designer should also check that sufficient horizontal distance exists to install the grids.

7.11.2 The geotechnical parameters listed in Table 7.11.1 can be used for preliminary design of the MSE walls. Laboratory testing indicates the onsite soils may be suitable for re-use as backfill within the reinforced zones behind MSE retaining wall. However, we anticipate that import soil may be needed and may be used as backfill material behind the walls. Once the import source has been determined, laboratory testing should be performed to check that the shear strength parameters used in the design of the MSE walls meet the required strength within the reinforced zone.

TABLE 7.11.1
GEOTECHNICAL PARAMETERS FOR MSE WALLS*

Parameter	Reinforced Zone	Retained Zone	Foundation Zone
Angle of Internal Friction	30 degrees	30 degrees	30 degrees
Cohesion	300 psf	300 psf	300 psf
Wet Unit Density	130 pcf	130 pcf	130 pcf

*Assumed for on-site soil.

7.11.3 The soil parameters presented in Table 7.11.1 are based on our experience and direct shear-strength tests performed during the geotechnical investigation and represent some of the on-site materials. The wet unit density values presented in Table 7.11.1 can be used for design but actual in-place densities may range from approximately 90 to 135 pounds per cubic foot. Geocon has no way of knowing which materials will actually be used as backfill behind the wall during construction. It is up to the wall designers to use their judgment in selection of the design parameters. As such, once backfill materials have been selected and/or stockpiled, sufficient shear tests should be conducted on samples of the proposed backfill materials to check that they conform to actual design values. Results should be provided to the designer to re-evaluate stability of the walls. Dependent upon test results, the designer may require modifications to the original wall design (e.g., longer reinforcement embedment lengths and/or steel reinforcement).

7.11.4 The foundation zone is the area where the footing is embedded, the reinforced zone is the area of the backfill that possesses the reinforcing fabric, and the retained zone is the area behind the reinforced zone.

7.11.5 Wall foundations should be designed in accordance with Table 7.11.2. The walls should be deepened such that the bottom outside edge of the footing is at least 7 feet horizontally from the face of the slope.

TABLE 7.11.2
SUMMARY OF MSE RETAINING WALL FOUNDATION RECOMMENDATIONS

Parameter	Value
Minimum Retaining Wall Foundation Width	12 inches
Minimum Retaining Wall Foundation Depth	12 Inches
Bearing Capacity	2,000 psf
Bearing Capacity Increase	500 psf per Foot of Depth
	300 psf per Foot of Width
Maximum Bearing Capacity	4,500 psf
Estimated Total Settlement	½ Inch
Estimated Differential Settlement	½ Inch in 40 Feet

7.11.6 Backfill materials within the reinforced zone 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. This is applicable to the entire embedment width of the reinforcement. Typically, wall designers specify no heavy compaction equipment within 3 feet of the face of the wall. However, smaller equipment (e.g., walk-behind, selfdriven compactors or hand whackers) can be used to compact the materials without causing deformation of the wall. If the designer specifies no compactive effort for this zone, the materials are essentially not properly compacted and the reinforcement grid within the uncompacted zone should not be relied upon for reinforcement, and overall embedment lengths will have to be increased to account for the difference.

7.11.7 Select backfill materials may be required to be in accordance with the MSE retaining wall system. Materials as outlined in the specifications of the retaining wall plans may be generated and stockpiled during grading, if encountered, or may require import. Geocon should perform laboratory tests during the backfill materials to check that soil properties are in accordance with the retaining wall plans and specifications.

7.11.8 The wall should be provided with a drainage system sufficient to prevent excessive seepage through the wall and the base of the wall, thus preventing hydrostatic pressures behind the wall.

7.11.9 Geosynthetic reinforcement must elongate to develop full tensile resistance. This elongation generally results in movement at the top of the wall. The amount of movement is dependent on the height of the wall (e.g., higher walls rotate more) and the type of reinforcing grid used. In addition, over time the reinforcement grid has been known to exhibit creep (sometimes as much as 5 percent) and can undergo additional movement. Given this condition, the owner should be aware that structures and pavement placed within the reinforced and retained zones of the wall may undergo movement.

7.11.10 The MSE wall contractor should provide the estimated deformation of wall and adjacent ground in associated with wall construction. The calculated horizontal and vertical deformations should be determined by the wall designer. The estimated movements should be provided to the project structural engineer to determine if the planned improvements can tolerate the expected movements.

7.11.11 The MSE wall designer/contractor should review this report, including the slope stability requirements, and incorporate our recommendations as presented herein. We should be provided the plans for the MSE walls to check if they are in conformance with our recommendations prior to issuance of a permit and construction.

7.12 Lateral Loading

7.12.1 Table 7.12 should be used to help design the proposed structures and improvements to resist lateral loads for the design of footings or shear keys. The allowable passive pressure assumes a horizontal surface extending at least 5 feet, or three times the surface generating the passive pressure, whichever is greater. The upper 12 inches of material in areas not protected by floor slabs or pavement should not be included in design for passive resistance.

TABLE 7.12
SUMMARY OF LATERAL LOAD DESIGN RECOMMENDATIONS

Parameter	Value
Passive Pressure Fluid Density	400 pcf
Coefficient of Friction (Concrete and Soil)	0.4
Coefficient of Friction (Along Vapor Barrier)	0.2 to 0.25*

*Per manufacturer's recommendations.

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

7.13 Preliminary Pavement Recommendations

7.13.1 We calculated the flexible pavement sections in general conformance with the *Caltrans Method of Flexible Pavement Design* (Highway Design Manual, Section 608.4) using an estimated Traffic Index (TI) of 5.0, 5.5, 6.0, and 7.0 for parking stalls, driveways, medium truck traffic areas, and heavy truck traffic areas, respectively. The project civil engineer and owner should review the pavement designations to determine appropriate locations for pavement thickness. The final pavement sections for the parking lot should be based on the R-Value of the subgrade soil encountered at final subgrade elevation. We have used an R-Value of 40 and 78 for the subgrade soil and base materials, respectively, for the purposes of this preliminary analysis. Table 7.13.1 presents the preliminary flexible pavement sections.

TABLE 7.13.1
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	40	3	4
Driveways for Automobiles and Light-Duty Vehicles	5.5	40	3	4
Medium Truck Traffic Areas	6.0	40	3.5	6
Driveways for Heavy Truck Traffic	7.0	40	4	7

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

7.13.3 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)*.

7.13.4 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.

7.13.5 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 7.13.2 provides the traffic categories and design parameters used for the calculations for 20-year design life.

TABLE 7.13.2
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
E	Garbage or Fire Truck Lane	75	15

7.13.6 We used the parameters presented in Table 7.13.3 to calculate the pavement design sections. We should be contacted to provide updated design sections, if necessary.

TABLE 7.13.3
RIGID PAVEMENT DESIGN PARAMETERS

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

7.13.7 Based on the criteria presented herein, the PCC pavement sections should have a minimum thickness as presented in Table 7.13.4.

TABLE 7.13.4
RIGID VEHICULAR PAVEMENT RECOMMENDATIONS

Traffic Category	Trucks Per Day	Portland Cement Concrete, T (Inches)
A = Car Parking Areas and Access Lanes	10	5½
B = Entrance and Truck Service Lanes	10	6
E = Garbage or Fire Truck Lanes	5	6½

7.13.8 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.

7.13.9 Adequate joint spacing should be incorporated into the design and construction of the rigid pavement in accordance with Table 7.13.5.

TABLE 7.13.5
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

7.13.10 The rigid pavement should also be designed and constructed incorporating the parameters presented in Table 7.13.6.

TABLE 7.13.6
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

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

7.13.12 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.

7.13.13 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.

7.13.14 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.

7.14 Site Drainage and Moisture Protection

7.14.1 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.

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

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

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

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

7.15 Grading and Foundation Plan Review

7.15.1 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.

7.16 Testing and Observation Services During Construction

7.16.1 Geocon Incorporated should provide geotechnical testing and observation services during the grading operations, foundation construction, utility installation, retaining wall backfill and pavement installation. Table 7.16 presents the typical geotechnical observations we would expect for the proposed improvements.

TABLE 7.16
EXPECTED GEOTECHNICAL TESTING AND OBSERVATION SERVICES

Construction Phase	Observations	Expected Time Frame
Ground Modification	Ground Modification Installation	Full Time
	Confirmation Testing	Part Time to Full Time
Grading	Base of Removal	Part Time During Removals
	Geologic Logging	Part Time to Full Time
	Fill Placement and Soil Compaction	Full Time
Foundations	Foundation Excavation Observations	Full 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.

THE GEOGRAPHICAL INFORMATION MADE AVAILABLE FOR DISPLAY WAS PROVIDED BY GOOGLE EARTH, SUBJECT TO A LICENSING AGREEMENT. THE INFORMATION IS FOR ILLUSTRATIVE PURPOSES ONLY; IT IS NOT INTENDED FOR CLIENT'S USE OR RELIANCE AND SHALL NOT BE REPRODUCED BY CLIENT. CLIENT SHALL INDEMNIFY, DEFEND AND HOLD HARMLESS GEOCON FROM ANY LIABILITY INCURRED AS A RESULT OF SUCH USE OR RELIANCE BY CLIENT.

PROPOSED APARTMENT DEVELOPMENT
855 BROTHERTON ROAD
ESCONDIDO, CALIFORNIA

0' 100' 200'
SCALE 1"= 100' (On 11x17)

GEOCON LEGEND

Qudf ....QUATERNARY UNDOCUMENTED FILL

Kwm ....CRETACEOUS GRANODIORITE OF WOODSON MOUNTAIN (Dotted Where Buried)

B-8 ....APPROX. LOCATION OF GEOTECHNICAL BORING

I-2 ....APPROX. LOCATION OF INFILTRATION TEST

(3') ....APPROX. DEPTH TO GRANITOID BEDROCK

GEOCON
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GEOLOGIC MAP

PROJECT NO. G3009 - 52 - 01
FIGURE 1
DATE 09 - 16 - 2022

Plotted:09/14/2022 2:00PM | By:RUBEN AGUILAR | File Location:Y:\PROJECTS\G3009-52-01 Brotherton Rd\SHEETS\G3009-52-01 GeologicMap.dwg

APPENDIX A

APPENDIX A

FIELD INVESTIGATION

We performed the subsurface investigation on August 11, 2022, consisting of the drilling, logging, and sampling of 8 exploratory borings and 2 infiltration test borings using a truck mounted Ingersoll Rand A-300 drilling rig equipped with 8-inch diameter hollow-stem augers with North County Drilling. The borings extended to maximum depths of approximately 3 to 20 feet. The locations of our exploratory borings are shown on the Geologic Map, Figure 1. The boring logs are presented in this Appendix. We located the borings in the field using a measuring tape and existing reference points; therefore, actual boring locations may deviate slightly.

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

The samplers were driven 12 inches or to refusal depth. 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 Google Earth satellite imagery. 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.

BORING B 1

ELEV. (MSL.) **663'** DATE COMPLETED **08-11-2022**

EQUIPMENT **INGERSOLL RAND A-300 W/8-INCH HSA's** BY: **A. HESPELER**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0	B1-1				**2.5 INCHES ASPHALT CONCRETE, NO BASE**			
				SM	**UNDOCUMENTED FILL (Qudf)** Medium dense, moist, light grayish brown, Silty, fine- to medium-grained SAND			
2				SW	**GRANODIORITE OF WOODSON MOUNTAIN (Kwm)** Slightly weathered, "salt and pepper" colored with some red iron oxide staining, moderately weak GRANITOID BEDROCK; very intensely fractured; excavates as Well Graded SAND			
4								
6	B1-2					50/3"		2.5
8								
10	B1-3					50/3"		3.5
12								
14					-Becomes less weathered at 14 feet			
	B1-4					50/3"		3.0
16								
18								
20					BORING TERMINATED AT 20 FEET BELOW GROUND SURFACE (No drilling refusal encountered) No groundwater encountered Backfilled with cuttings Patched with aquaphalt			

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

G3009-52-01.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. G3009-52-01

BORING B 2

ELEV. (MSL.) 661' DATE COMPLETED 08-11-2022

EQUIPMENT INGERSOLL RAND A-300 W/8-INCH HSA's BY: A. HESPELER

DEPTH IN FEET	SAMPLE NO.	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0			**2.5 INCHES ASPHALT CONCRETE, NO BASE**			
		SM	**UNDOCUMENTED FILL (Qudf)** Medium dense, moist, light grayish brown, Silty, fine- to medium-grained SAND			
2		SW	**GRANODIORITE OF WOODSON MOUNTAIN (Kwm)** Slightly weathered, "salt and pepper" colored with some red iron oxide staining, moderately weak GRANITOID BEDROCK; very intensely fractured; excavates as Well Graded SAND			
4						
	B2-1			50/4"	129.8	6.7
6						
8						
10	B2-2			50/3"	119.3	6.9
12						
14						
	B2-3			50/3"		
16						
18						
20			BORING TERMINATED AT 20 FEET BELOW GROUND SURFACE (No drilling refusal encountered) No groundwater encountered Backfilled with cuttings Patched with aquaphalt			

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

G3009-52-01.GPJ

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

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

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BORING B 3

ELEV. (MSL.) **663'** DATE COMPLETED **08-11-2022**

EQUIPMENT **INGERSOLL RAND A-300 W/8-INCH HSA's** BY: **A. HESPELER**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0					**2.5 INCHES ASPHALT CONCRETE, NO BASE**			
				SM	**UNDOCUMENTED FILL (Qudf)** Medium dense, moist, grayish brown, Silty, fine- to medium-grained SAND			
	B3-1							
2								
4								
	B3-2				-Becomes dense and reddish brown at 5 feet	69	128.1	11.4
6								
8					-Becomes dark brown at 8 feet			
10	B3-3					75	122.2	11.7
				SW	**GRANODIORITE OF WOODSON MOUNTAIN (Kwm)** Slightly weathered, "salt and pepper" colored with some red iron oxide staining, moderately weak GRANITOID BEDROCK; very intensely fractured; excavates as Well Graded SAND			
12								
14								
	B3-4					50/3"		2.7
16								
18								
20					BORING TERMINATED AT 20 FEET BELOW GROUND SURFACE (No drilling refusal encountered) No groundwater encountered Backfilled with spoils Patched with aquaphalt			

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

G3009-52-01.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. G3009-52-01

BORING B 4

ELEV. (MSL.) **657'** DATE COMPLETED **08-11-2022**

EQUIPMENT **INGERSOLL RAND A-300 W/8-INCH HSA's** BY: **A. HESPELER**

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 INCHES ASPHALT CONCRETE, NO BASE**			
				SM	**UNDOCUMENTED FILL (Qudf)** Medium dense, moist, grayish brown, Silty, fine- to medium-grained SAND			
2								
4								
	B4-1				-Becomes very dense	82	124.5	10.3
6								
8				SC	Dense, moist, dark gray, Clayey fine- to medium-grained SAND			
10	B4-2					67	123.5	13.5
12								
				SW	**GRANODIORITE OF WOODSON MOUNTAIN (Kwm)** Slightly weathered, "salt and pepper" colored with some red iron oxide staining, moderately weak GRANITOID BEDROCK; very intensely fractured; excavates as Well Graded SAND			
14								
	B4-3					50/3"		4.5
16								
18								
20					BORING TERMINATED AT 20 FEET BELOW GROUND SURFACE (No drilling refusal encountered) No groundwater encountered Backfilled with spoils Patched with aquaphalt			

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

G3009-52-01.GPJ

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

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

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PROJECT NO. G3009-52-01

BORING B 5

ELEV. (MSL.) 659' DATE COMPLETED 08-11-2022

EQUIPMENT INGERSOLL RAND A-300 W/8-INCH HSA's BY: A. HESPELER

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0					**2.5 INCHES ASPHALT CONCRETE, NO BASE**			
	B5-1			SC	**UNDOCUMENTED FILL (Qudf)** Medium dense, moist, reddish brown, Clayey, fine- to medium-grained SAND			
2				SW	**GRANODIORITE OF WOODSON MOUNTAIN (Kwm)** Slightly weathered, "salt and pepper" colored with some red iron oxide staining, moderately weak GRANITOID BEDROCK; very intensely fractured; excavates as Well Graded SAND			
4								
	B5-2					50/6"	122.4	5.8
6								
8								
10	B5-3				-No sample recovery	50/2"		
12								
14								
16					-Becomes less weathered at 16 feet			
18								
20					BORING TERMINATED AT 20 FEET BELOW GROUND SURFACE (No drilling refusal encountered) No groundwater encountered Backfilled with spoils Patched with aquaphalt			

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

G3009-52-01.GPJ

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

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

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BORING B 6

ELEV. (MSL.) **662'** DATE COMPLETED **08-12-2022**

EQUIPMENT **INGERSOLL RAND A-300 W/8-INCH HSA's** BY: **A. HESPELER**

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 INCHES ASPHALT CONCRETE, NO BASE**			
				SC	**UNDOCUMENTED FILL (Qudf)** Medium dense, moist, reddish brown, Clayey, fine- to medium-grained SAND			
2				SW	**GRANODIORITE OF WOODSON MOUNTAIN (Kwm)** Slightly weathered, "salt and pepper" colored with some red iron oxide staining, moderately weak GRANITOID BEDROCK; very intensely fractured; excavates as Well Graded SAND			
4								
	B6-1					50/5"	117.2	4.5
6								
8								
10	B6-2					50/5"	112.2	4.1
12								
14								
16								
18								
20					BORING TERMINATED AT 20 FEET BELOW GROUND SURFACE (No drilling refusal encountered) No groundwater encountered Backfilled with spoils Patched with aquaphalt			

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

G3009-52-01.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.

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BORING B 7

ELEV. (MSL.) **644'** DATE COMPLETED **08-12-2022**

EQUIPMENT **INGERSOLL RAND A-300 W/8-INCH HSA's** BY: **A. HESPELER**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0				SC	**UNDOCUMENTED FILL (Qudf)** Loose, dry, grayish brown, Clayey, fine- to medium-grained SAND			
3				SW	**GRANODIORITE OF WOODSON MOUNTAIN (Kwm)** Slightly weathered, "salt and pepper" colored with some red iron oxide staining, moderately weak GRANITOID BEDROCK; very intensely fractured; excavates as Well Graded SAND			
5	B7-1					50/6"	106.7	8.7
10	B7-2					50/6"	104.0	19.7
20					BORING TERMINATED AT 20 FEET BELOW GROUND SURFACE (No drilling refusal encountered) No groundwater encountered Backfilled with spoils			

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

G3009-52-01.GPJ

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

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

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BORING B 8

ELEV. (MSL.) 642' DATE COMPLETED 08-12-2022

EQUIPMENT INGERSOLL RAND A-300 W/8-INCH HSA's BY: A. HESPELER

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0				SC	**UNDOCUMENTED FILL (Qudf)** Loose, dry, grayish brown, Clayey, fine- to medium-grained SAND			
2				SW	**GRANODIORITE OF WOODSON MOUNTAIN (Kwm)** Slightly weathered, "salt and pepper" colored with some red iron oxide staining, moderately weak GRANITOID BEDROCK; very intensely fractured; excavates as Well Graded SAND			
4								
	B8-1					50/6"	119.3	12.9
6								
8								
10	B8-2					50/4.5"	112.0	12.4
12								
					-Becomes less weathered at 13 feet			
14								
16								
18								
20					BORING TERMINATED AT 20 FEET BELOW GROUND SURFACE (No drilling refusal encountered) No groundwater encountered Backfilled with spoils			

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

G3009-52-01.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 I 1

ELEV. (MSL.) **644'** DATE COMPLETED **08-12-2022**

EQUIPMENT **INGERSOLL RAND A-300 W/8-INCH HSA's** BY: **A. HESPELER**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0				SC	**UNDOCUMENTED FILL (Qudf)** Loose, dry, grayish brown, Clayey, fine- to medium-grained SAND			
2	I1-1			SW	**GRANODIORITE OF WOODSON MOUNTAIN (Kwm)** Slightly weathered, "salt and pepper" colored with some red iron oxide staining, moderately weak GRANITOID BEDROCK; very intensely fractured; excavates as Clayey SAND			

BOTTOM OF BORING AT 39 INCHES BELOW GROUND SURFACE
(No drilling refusal encountered)
No groundwater encountered
Backfilled with spoils

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

G3009-52-01.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. G3009-52-01

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	BORING I 2 ELEV. (MSL.) 643' DATE COMPLETED 08-12-2022 EQUIPMENT INGERSOLL RAND A-300 W/8-INCH HSA's BY: A. HESPELER	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
					MATERIAL DESCRIPTION			
0				SC	**UNDOCUMENTED FILL (Qudf)** Loose, dry, grayish brown, Clayey, fine- to medium-grained SAND			
2	I2-1			SW	**GRANODIORITE OF WOODSON MOUNTAIN (Kwm)** Slightly weathered, "salt and pepper" colored with some red iron oxide staining, moderately weak GRANODIORITE BEDROCK; very intensely fractured; excavates as Well Graded SAND			
					BOTTOM OF BORING AT 42 INCHES BELOW GROUND SURFACE (No drilling refusal encountered) No groundwater encountered Backfilled with spoils			

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

G3009-52-01.GPJ

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

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

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APPENDIX B

APPENDIX B

LABORATORY TESTING

We performed laboratory tests in accordance with generally accepted test methods of the American Society for Testing and Materials (ASTM) or other suggested procedures. We tested selected soil samples for in-place dry density/moisture content, maximum density/optimum moisture content, expansion index, water-soluble sulfate, plasticity index, R-Value, 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	Light grayish brown, Well Graded SAND (Qudf/Kwm)	140.9	6.6
B3-1	Grayish brown, Silty, fine- to coarse-grained SAND (Qudf)	138.5	7.6

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	5.7	10.5	127.7	6	Non-Expansive	Very Low
B3-1	7.1	12.8	122.5	23	Expansive	Low

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

Sample No.	Depth (feet)	Geologic Unit	Water-Soluble Sulfate (%)	ACI 318 Sulfate Exposure
B1-1	1-5	Qudf/Kmw	0.002	S0
B3-1	1-5	Qudf	0.014	S0

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

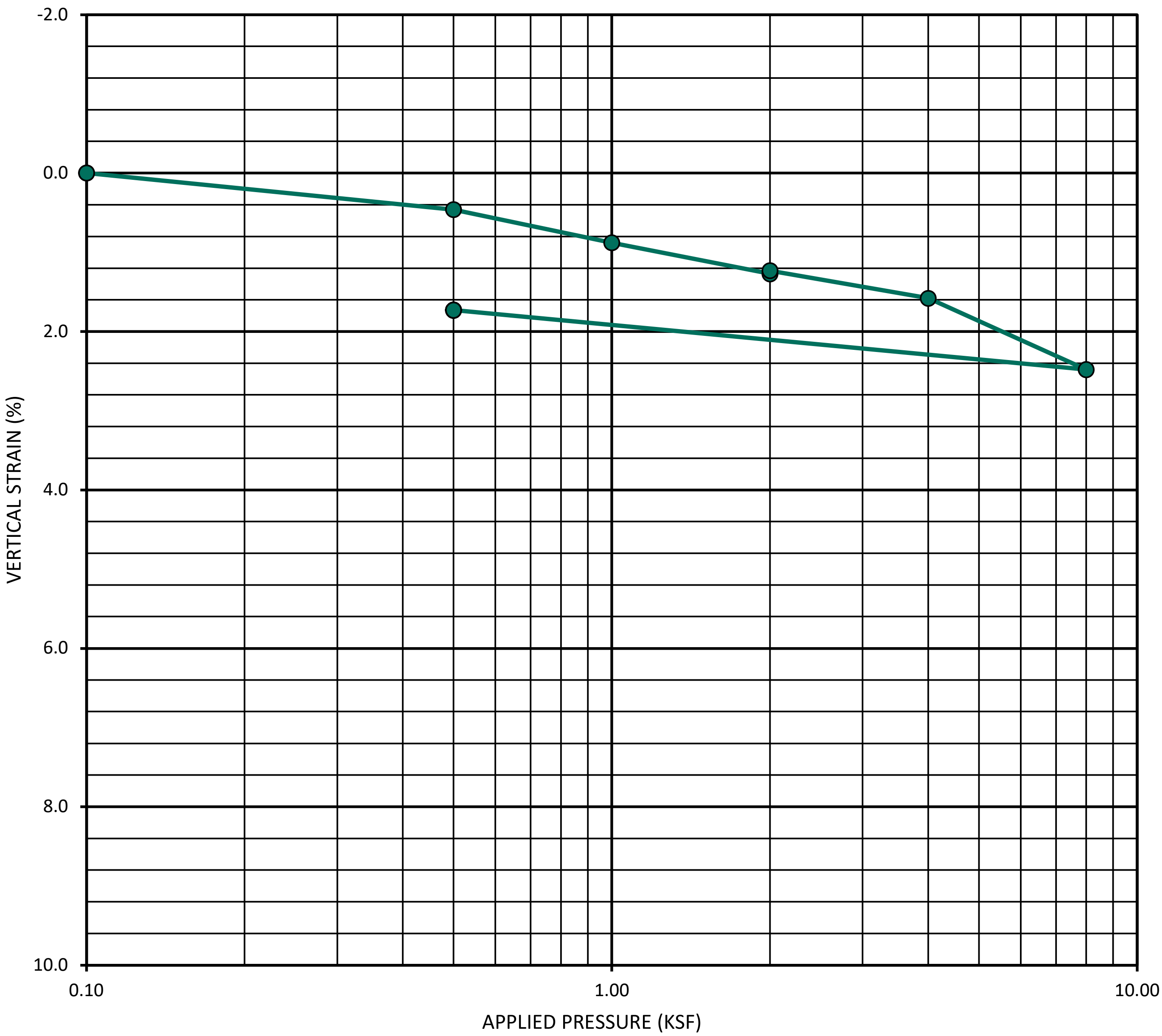
Sample No.	Depth (Feet)	Description (Geologic Unit)	R-Value
B1-1	1-5	Light grayish brown, well graded SAND (Kmw)	58

SAMPLE NO.: **B3-2**

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

GEOLOGIC UNIT: **Qudf**

TEST INFORMATION	
INITIAL DRY DENSITY (PCF):	128.1
INITIAL WATER CONTENT (%):	11.4%
SAMPLE SATURATED AT (KSF):	2.0
INITIAL SATURATION (%):	100+



GEOCON INCORPORATED

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PHONE 858 558-6900 - FAX 858 558-6159

CONSOLIDATION CURVE ASTM D 2435

BROTHERTON RD. APARTMENTS

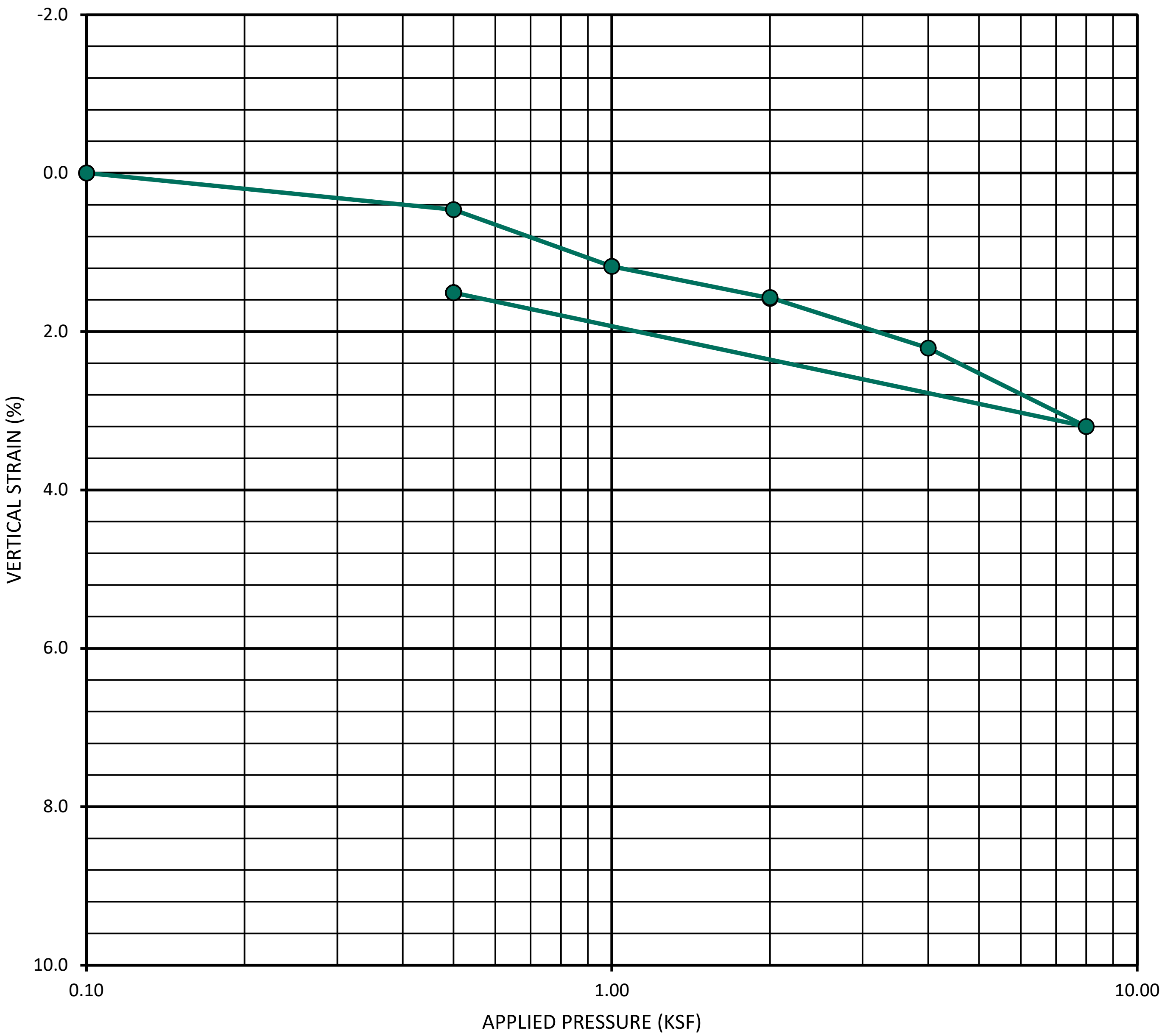
PROJECT NO.: G3009-52-01

SAMPLE NO.: **B8-1**

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

GEOLOGIC UNIT: **Kgr**

TEST INFORMATION	
INITIAL DRY DENSITY (PCF):	119.3
INITIAL WATER CONTENT (%):	12.9%
SAMPLE SATURATED AT (KSF):	2.0
INITIAL SATURATION (%):	88.0%



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CONSOLIDATION CURVE ASTM D 2435

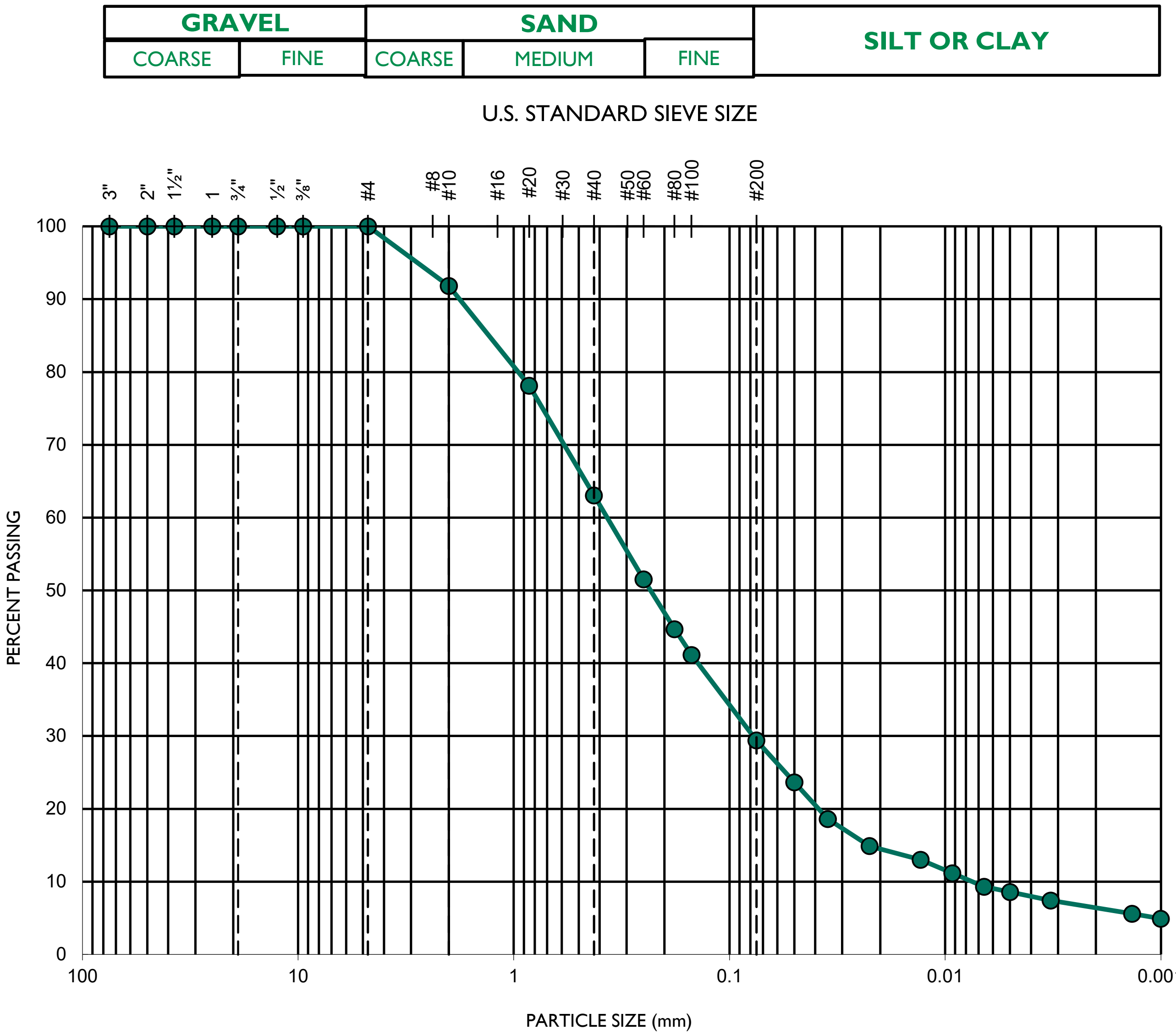
BROTHERTON RD. APARTMENTS

PROJECT NO.: G3009-52-01

SAMPLE NO.: **B1-1**

SAMPLE DEPTH (FT.): **1-5'**

GEOLOGIC UNIT: **Qudf/Kgr**



TEST DATA					
D_{10} (mm)	D_{30} (mm)	D_{60} (mm)	C_c	C_u	SOIL DESCRIPTION
0.00030	0.00310	0.01491	2.2	50.0	Silty SAND

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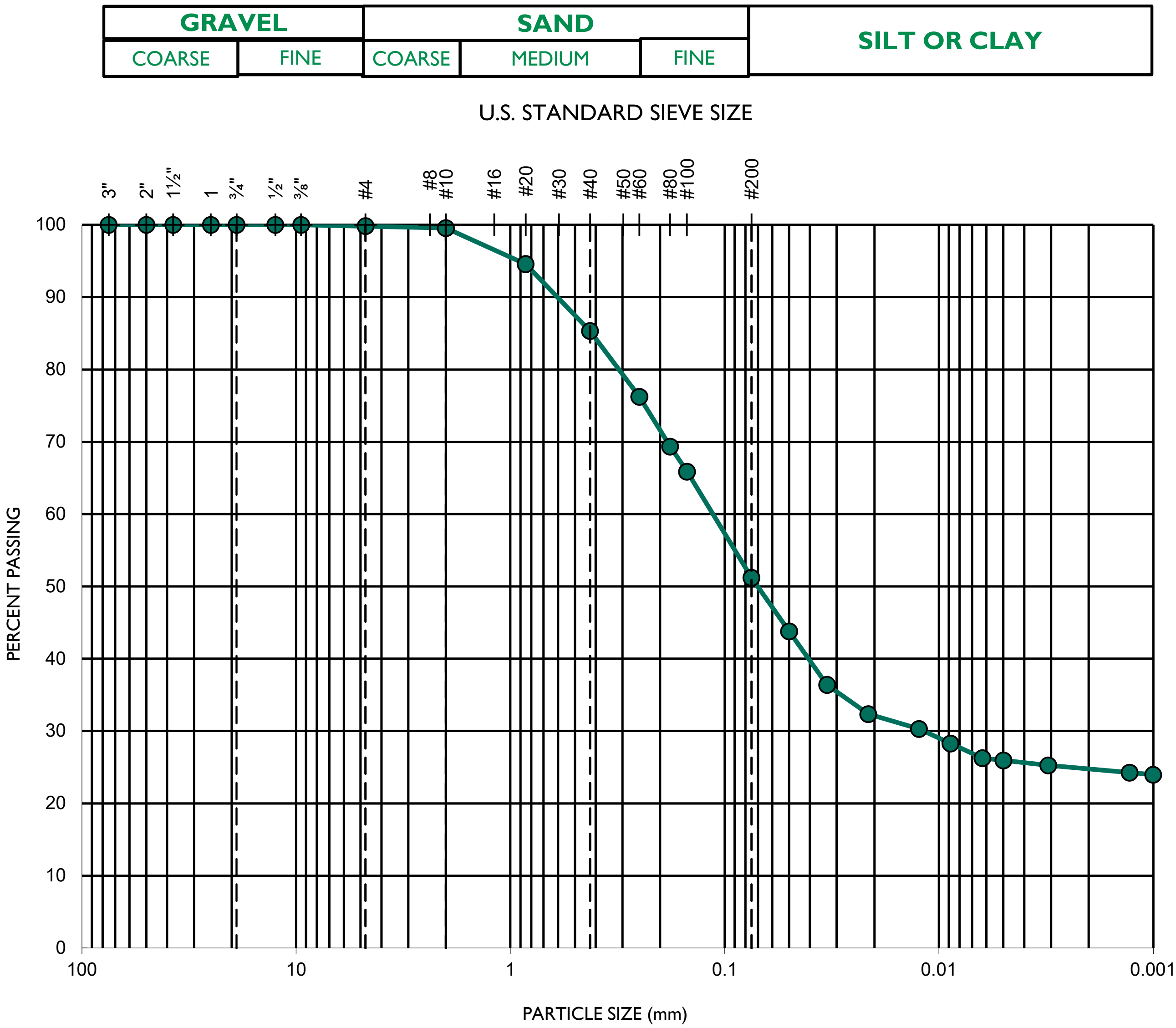
SIEVE ANALYSES ASTM D 6913

BROTHERTON RD. APARTMENTS

PROJECT NO.: G3009-52-01

SAMPLE NO.: II-1
SAMPLE DEPTH (FT.): 27-39"
GEOLOGIC UNIT: Kgr



TEST DATA					
D_{10} (mm)	D_{30} (mm)	D_{60} (mm)	C_c	C_u	SOIL DESCRIPTION
--	0.00047	0.00472	--	--	Sandy CLAY

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SIEVE ANALYSES ASTM D 6913

BROTHERTON RD. APARTMENTS

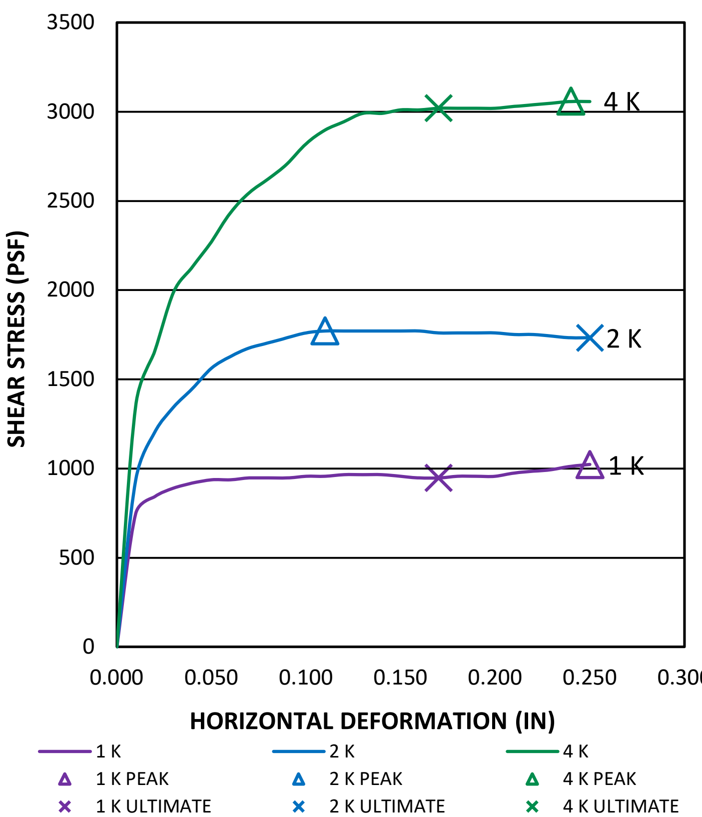
PROJECT NO.: G3009-52-01

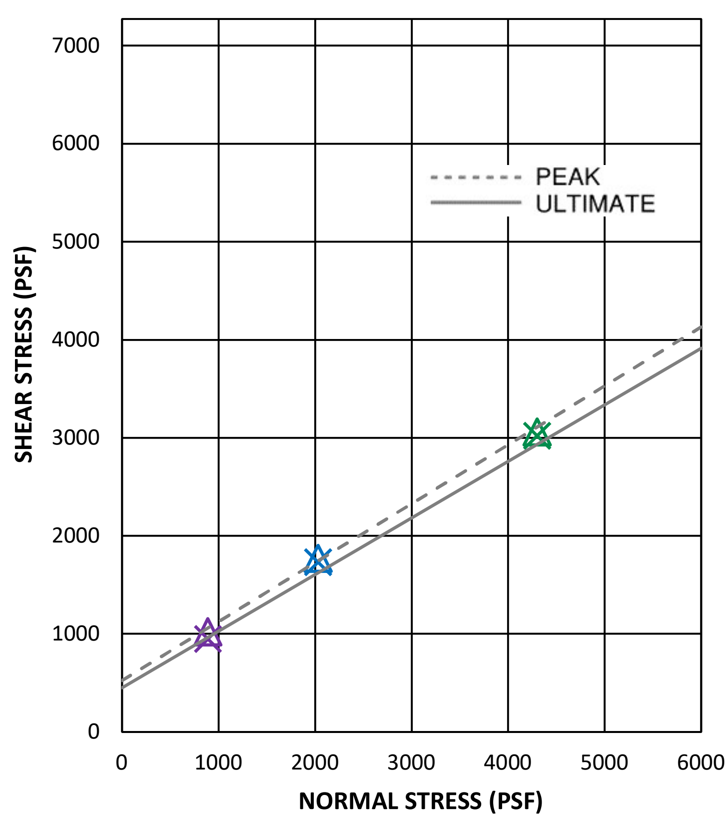
SAMPLE NO.:	B1-1	GEOLOGIC UNIT:	Qudf / Kg
SAMPLE DEPTH (FT):	1-5'	NATURAL/REMOLDED:	R

INITIAL CONDITIONS				
NORMAL STRESS TEST LOAD	1 K	2 K	4 K	AVERAGE
ACTUAL NORMAL STRESS (PSF):	890	2030	4300	--
WATER CONTENT (%):	6.4	6.6	6.1	6.3
DRY DENSITY (PCF):	127.3	127.1	127.4	127.3

WATER CONTENT (%):	11.0	11.3	11.7	11.3
PEAK SHEAR STRESS (PSF):	1023	1770	3058	--
ULT.-E.O.T. SHEAR STRESS (PSF):	947	1733	3020	--

PEAK	COHESION, C (PSF)	525
	FRICTION ANGLE (DEGREES)	31
ULTIMATE	COHESION, C (PSF)	450
	FRICTION ANGLE (DEGREES)	30





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DIRECT SHEAR AASHTO T 236

BROTHERTON RD. APARTMENTS

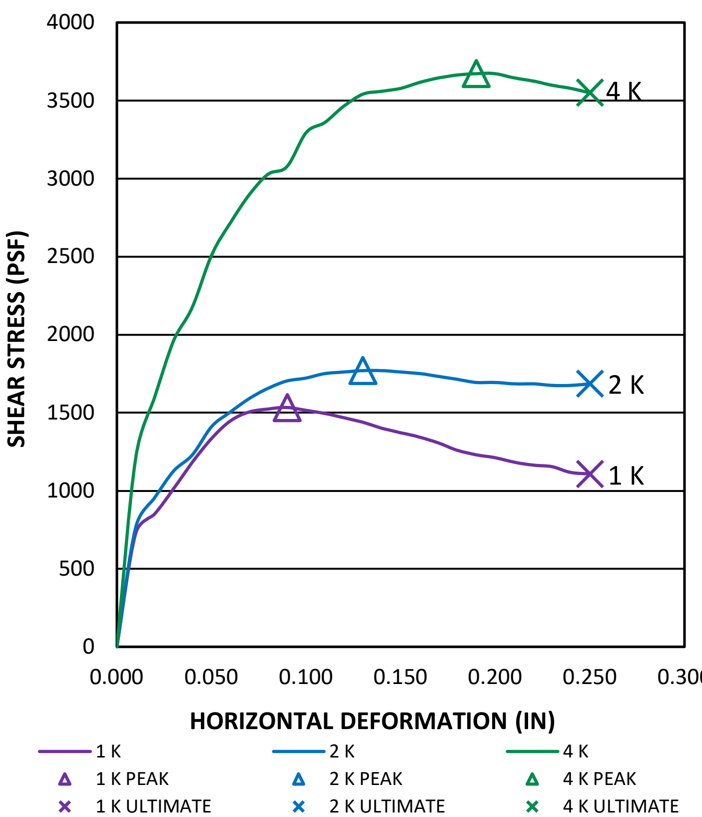
PROJECT NO.: G3009-52-01

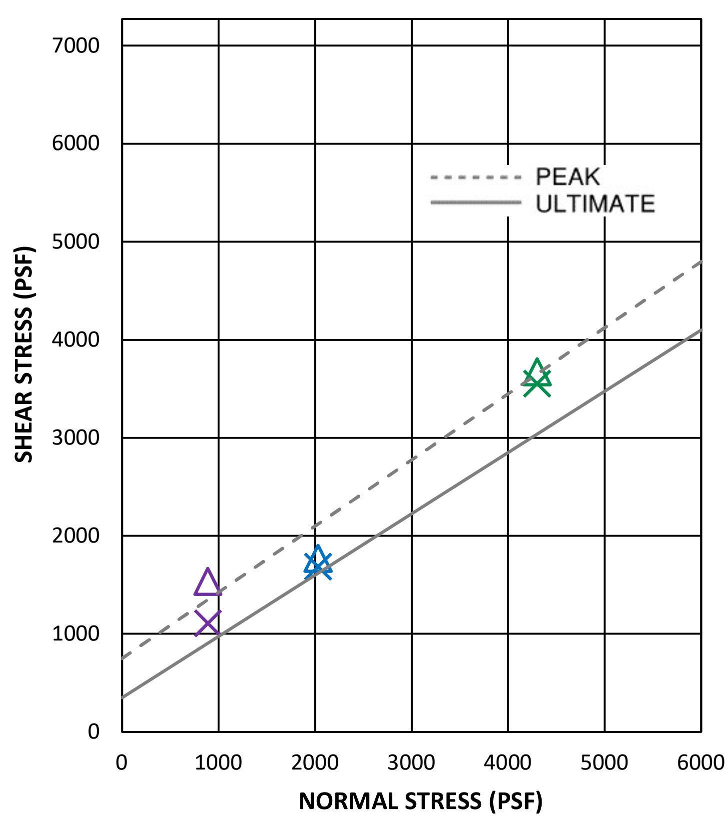
SAMPLE NO.:	B5-2	GEOLOGIC UNIT:	Kgr
SAMPLE DEPTH (FT):	5'	NATURAL/REMOLDED:	N

INITIAL CONDITIONS				
NORMAL STRESS TEST LOAD	1 K	2 K	4 K	AVERAGE
ACTUAL NORMAL STRESS (PSF):	890	2030	4300	--
WATER CONTENT (%):	5.4	6.1	5.8	5.8
DRY DENSITY (PCF):	128.2	115.1	123.8	122.4

AFTER TEST CONDITIONS				
NORMAL STRESS TEST LOAD	1 K	2 K	4 K	AVERAGE
WATER CONTENT (%):	11.5	15.5	13.0	13.3
PEAK SHEAR STRESS (PSF):	1534	1770	3673	--
ULT.-E.O.T. SHEAR STRESS (PSF):	1108	1685	3550	--

RESULTS		
PEAK	COHESION, C (PSF)	750
	FRICTION ANGLE (DEGREES)	34
ULTIMATE	COHESION, C (PSF)	350
	FRICTION ANGLE (DEGREES)	32





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DIRECT SHEAR AASHTO T 236

BROTHERTON RD. APARTMENTS

PROJECT NO.: G3009-52-01

APPENDIX C

APPENDIX C

RECOMMENDED GRADING SPECIFICATIONS

FOR

BROTHERTON RD. APARTMENTS
855 BROTHERTON RD.
ESCONDIDO, CA

PROJECT NO. G3009-52-01

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

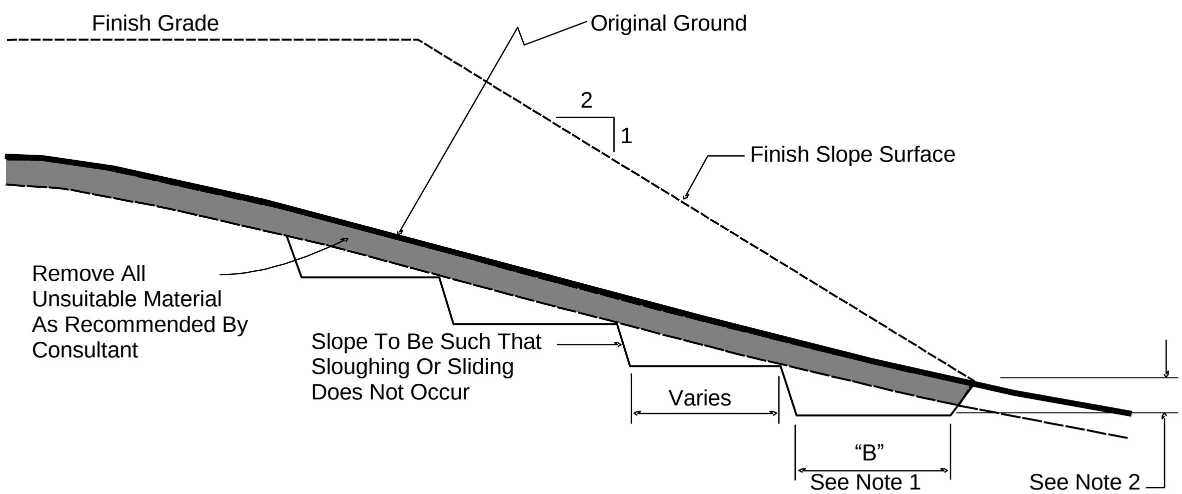
4.1 Areas to be excavated and filled shall be cleared and grubbed. Clearing shall consist of complete removal above the ground surface of trees, stumps, brush, vegetation, man-made structures, and similar debris. Grubbing shall consist of removal of stumps, roots, buried logs and other unsuitable material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

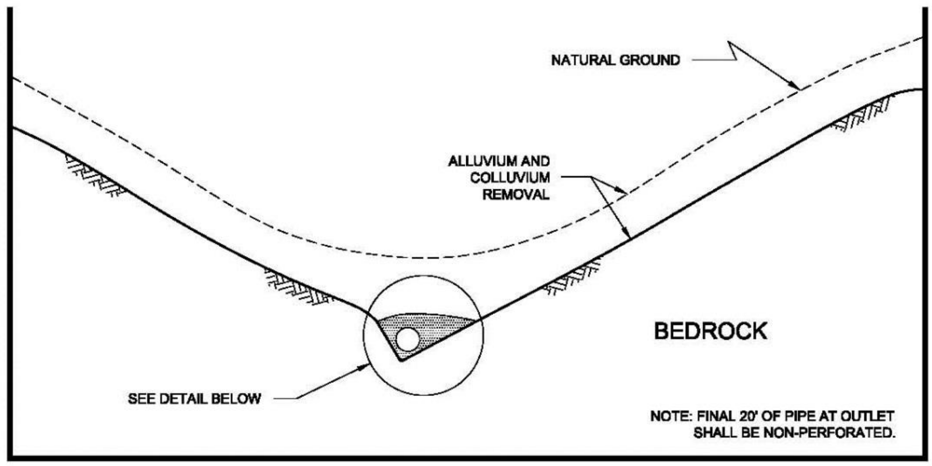
6.3.6 To reduce the potential for “piping” of fines into the *rock* fill from overlying *soil* fill material, a 2-foot layer of graded filter material shall be placed above the uppermost lift of *rock* fill. The need to place graded filter material below the *rock* should be determined by the Consultant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

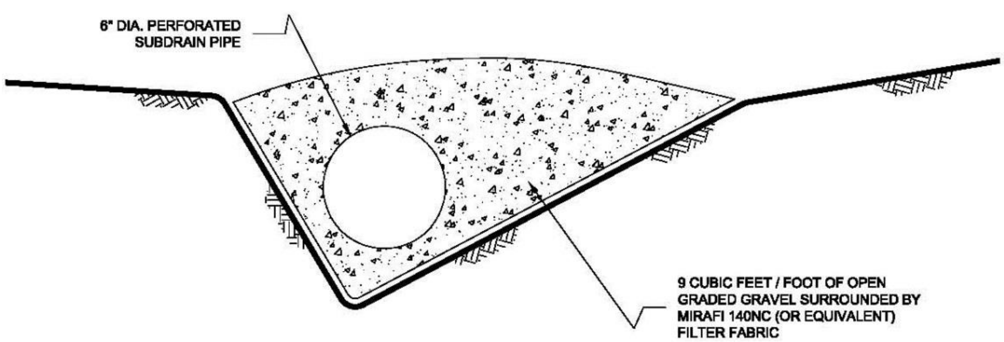
6.3.7 *Rock* fill placement should be continuously observed during placement by the Consultant.

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:

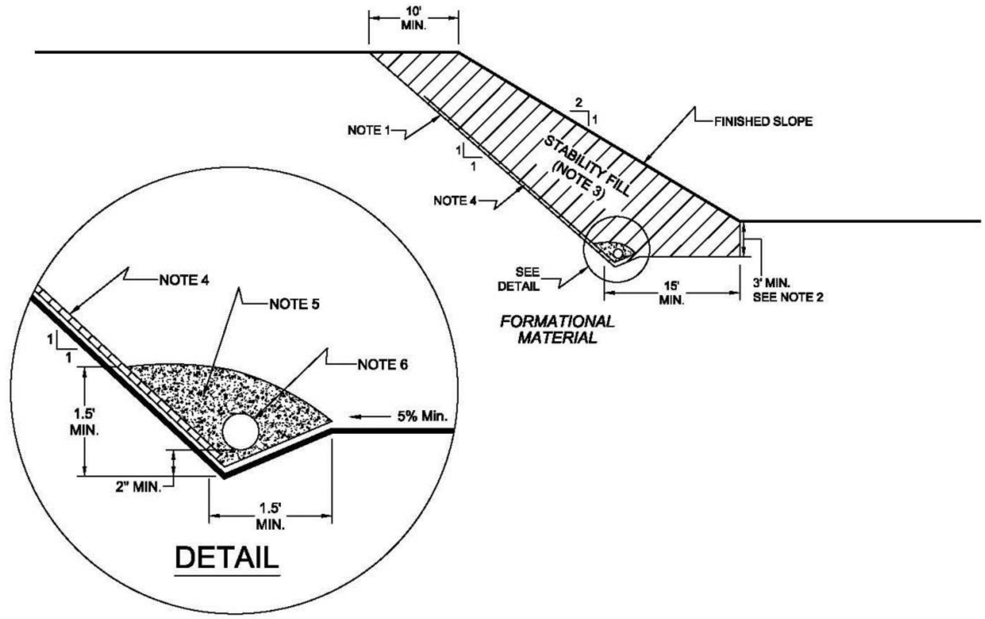
1......6-INCH DIAMETER, SCHEDULE 80 PVC PERFORATED PIPE FOR FILLS IN EXCESS OF 100-FEET IN DEPTH OR A PIPE LENGTH OF LONGER THAN 500 FEET.

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

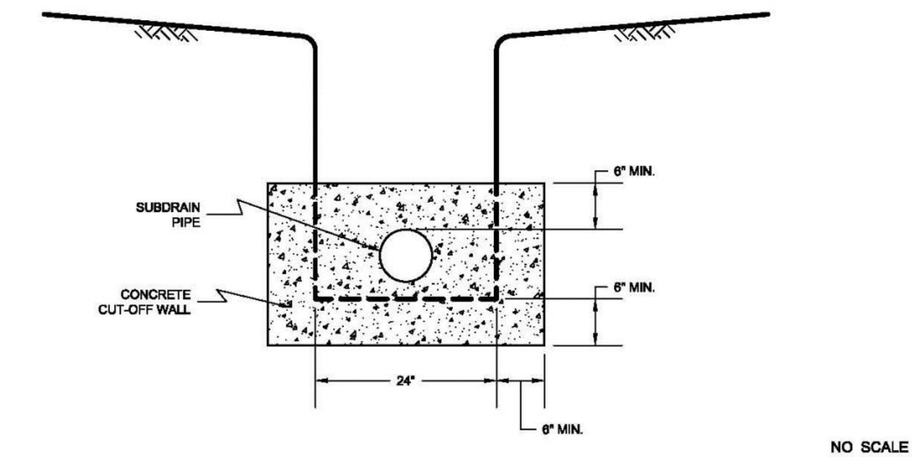
7.3 The actual subdrain locations will be evaluated in the field during the remedial grading operations. Additional drains may be necessary depending on the conditions observed and the requirements of the local regulatory agencies. Appropriate subdrain outlets should be evaluated prior to finalizing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

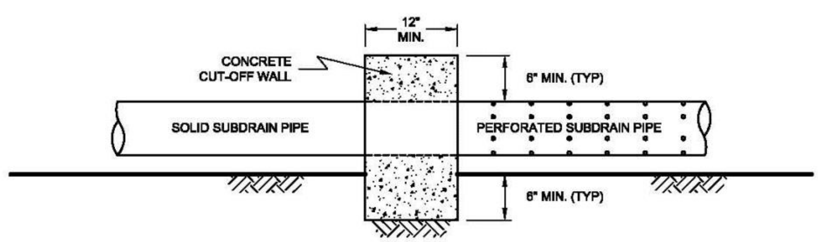
7.5 Prior to outletting, the final 20-foot segment of a subdrain that will not be extended during future development should consist of non-perforated drainpipe. At the non-perforated/ perforated interface, a seepage cutoff wall should be constructed on the downslope side of the pipe.

TYPICAL CUT OFF WALL DETAIL

FRONT VIEW



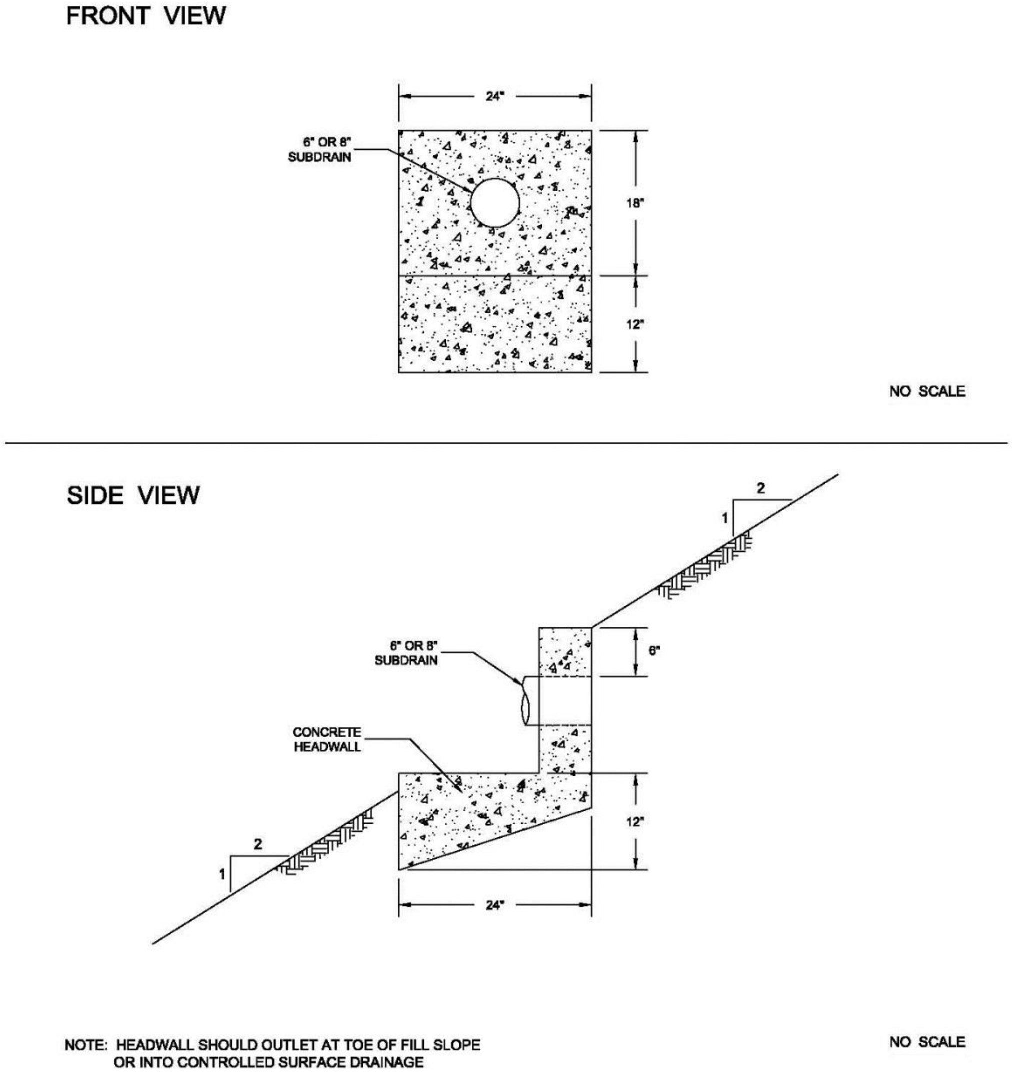
SIDE VIEW



NO SCALE

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

TYPICAL HEADWALL DETAIL



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. *ACI 318-19, Commentary on Building Code Requirements for Structural Concrete,* prepared by the American Concrete Institute, dated May 2019.

3. *ACI 330-21, Commercial Concrete Parking Lots and Site Paving Design and Construction,* prepared by the 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. County of San Diego, *San Diego County Multi Jurisdiction Hazard Mitigation Plan, San Diego, California – Final Draft,* dated 2017.

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

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

10. Kennedy, M. P., and S. S. Tan, 2007, *Geologic Map of the Oceanside 30'x60' Quadrangle, California*, USGS Regional Map Series Map No. 3, Scale 1:100,000.

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

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

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