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GEOTECHNICAL
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MATERIALS

LIMITED GEOTECHNICAL INVESTIGATION

NORTHEAST GATEWAY ESCONDIDO
NORTHEAST OF VALLEY PARKWAY AND BEVEN DRIVE
ESCONDIDO, CALIFORNIA

PREPARED FOR

MERIDIAN
DEVELOPMENT

OCTOBER 21, 2021
PROJECT NO. G2818-52-01

PL22-0145

Project No. G2818-52-01
October 21, 2021

Meridian Development
9988 Hibert Street, Suite 210
San Diego, California 92131

Attention: Mr. Guy Asaro

Subject: LIMITED GEOTECHNICAL INVESTIGATION
NORTHEAST GATEWAY ESCONDIDO
NORTHEAST OF VALLEY PARKWAY AND BEVEN DRIVE
ESCONDIDO, CALIFORNIA

Dear Mr. Asaro:

In accordance with your request and authorization of our Proposal No. LG-21433 dated September 7, 2021, we herein submit the results of our limited 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. This report is considered limited because we did not perform necessary geotechnical borings and liquefaction analysis at this time.

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.

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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LIMITED GEOTECHNICAL INVESTIGATION

1. PURPOSE AND SCOPE

This report presents the results of our limited geotechnical investigation for the proposed subdivision located in the Lake Wohlford and East Grove area in the City of Escondido, California (see Vicinity Map, Figure 1).

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 18 exploratory trenches to a maximum depth of about 14 feet, 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 subject site consist of an approximate 32-acre property located east of East Valley Parkway, South of Lake Wohlford Road, and north of Beven Drive. The property currently consists of 4 single-family residences with ancillary barns and structures and a commercial building on the west with accompanied driveways, utilities and landscaping. In addition, there are large areas of undeveloped land. Based on aerial photographs, it appears that the majority of the land has been used primarily for agricultural purposes, consisting mostly of citrus orchards. Existing grades within the proposed development are relatively flat and gently sloping with drainage directed to the west with elevations of approximately 725 to 780 feet Mean Sea Level (MSL). There is a natural ascending slope to the east that exposes large granitic outcrops and varies in elevation from approximately 780 to 1,600 feet MSL. The Existing Site Plan shows the current site conditions.

Existing Site Map

We understand the project will consist of demolishing residences and will be redeveloped to accommodate about 59-single family residential structures with accompanied roadways, utilities and landscaping. A storm water management basin is planned on the southwestern portion of the site. The new proposed development will be accessed from Beven Drive on the south and Foxley Drive on the north. The Preliminary Site Plan shows the preliminary building pads and improvements.

Proposed 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.

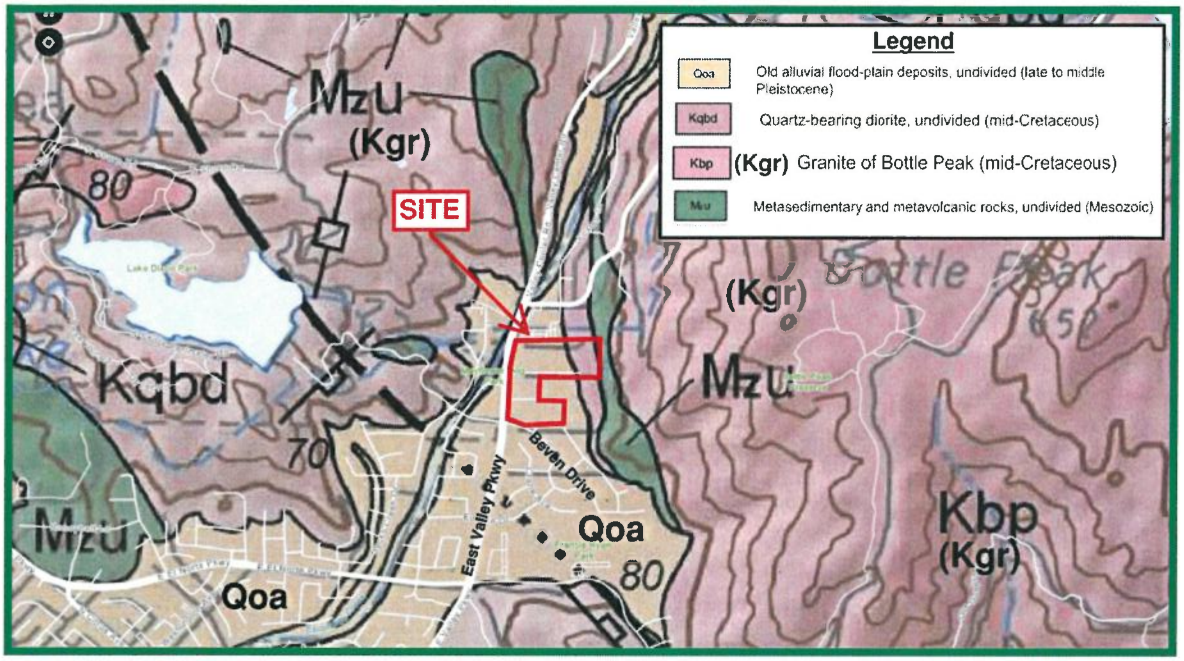
3. REVIEW OF GEOTECHNICAL REPORT

Geocon provided engineering recommendations and testing services for the grading operations for the Eureka Ranch single-family development located south of the subject site. The Geotechnical Investigation for the development titled *Update Geotechnical Investigation for Eureka Ranch,* *Escondido California*, dated May 20, 2005 (Project number 07436-52-01) consisted of geotechnical borings, trenching, seismic lines, and liquefaction analyses. The general geologic conditions prior to mass grading consisted of surficial soils composed of topsoil and alluvium overlying formational material identified as Granitic Rock. The previous investigation encountered alluvial soils up to approximately 50 feet thick near the southwestern edge of the subject property. Groundwater was encountered between 13 and 28 feet below the ground surface. The liquefaction analysis identified the potential for liquefaction in alluvial layers at depths between 22 and 35 feet with a maximum settlement of up to 2¼ inches within the liquefiable layers. Remedial grading for the development consisted of the removal and re-compaction of the upper 3 to 12 feet of surficial soils. The structures were designed using post-tensioned foundation systems to account for the settlement due to liquefaction.

4. GEOLOGIC SETTING

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

The site is located in the eastern portion of the Peninsular Range Geomorphic Province. Older alluvium is mapped on the majority of the central and western portions of the site. Cretaceous -aged Granitic and Mesozoic-aged Metavolcanic and Metasedimentary rocks are mapped on the eastern portions of the site. The Regional Geologic Map shows the geologic units in the area of the site.



Regional Geologic Map

5. SOIL AND GEOLOGIC CONDITIONS

We encountered four surficial soil units (consisting of undocumented fill, topsoil, alluvium and colluvium) and one formational unit (consisting of Granitic Rock). The Regional Geologic Map indicates the site to be underlain by Metavolcanic and Metasedimentary rock; however, we did not encounter it during our investigation. The occurrence, distribution, and description of each unit encountered is shown on the Geologic Map, Figure 1, and on the boring logs in Appendix A. The Geologic Cross-Sections, Figure 2, shows the approximate subsurface relationship between the geologic units. We prepared the geologic cross-sections using interpolation between exploratory excavations and observations; therefore, actual geotechnical conditions may vary from those illustrated and should be considered approximate. The surficial soils and geologic units are described herein in order of increasing age.

5.1 Undocumented Fill (Qudf)

We encountered shallow undocumented fill in Trenches T-9 and T-18 to depths ranging from about 2 to 2.5 feet. In general, the fill consists of loose to medium dense, dry to damp, silty to clayey sand. 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.

5.2 Topsoil (Unmapped)

Holocene-age topsoil is present as a relatively thin veneer locally overlying the alluvium and was encountered on the majority of trenches on the central and western portions of the site. The topsoil is about 1 to 3 feet thick across the site and can be characterized as loose to medium dense, dry to damp, light brown to brown, clayey to silty, fine to coarse sand. Remedial grading of the topsoil will be necessary in areas to support proposed fill or structures. The topsoil can be reused for new compacted fills.

5.3 Older Alluvium (Qoa)

Older alluvium exists below the topsoil and undocumented fill as encountered in Trenches T-1 through T-9 and T-12 through T-18. The older alluvium ranges in thickness from approximately 2 feet to the final depths explored of 14 feet. Based on previous studies in the area the thickness of the alluvium near East Valley Parkway is up to 50 feet thick. The older alluvium typically consists of medium dense to dense, light brown to brown, silty to clayey sand with a trace of gravel and granitic clasts. The soil possesses a "very low" to "medium" expansion index (expansion index of 90 or less). The upper portion of the older alluvium is considered unsuitable for the support of foundations or structural fills and will require removal during remedial grading operations. The removal depths will vary across the site.

5.4 Colluvium (Qcol)

We observed colluvium in Trenches T-10 and T-11 overlying the granitic rock near the toe of the ascending slope on the eastern portion of the site about 8 feet thick. Colluvial Deposits typically accumulates at the base of steep slopes from erosion and mass wasting and consists of a heterogenous, unsorted material. The colluvial deposits consist of loose to medium dense, reddish brown, silty, fine to coarse sand with some angular gravel to cobble sized native clasts. The colluvium is considered unsuitable for the support of foundations or structural fills and will require removal during remedial grading operations.

5.5 Granitic Rock (Kgr)

Cretaceous-age granitic rock underlies the surficial soil on the property. Geologic literature indicates that the eastern portion of the site is underlain by Granite of Bottle Peak (Gbp) which we generalized and will be referencing the granitic rock as (Kgr). We encountered the granitic rock in trenches T-10 through T-15 on the northeastern portion of the site at depths ranging from 2 to 8 feet. The granitic rock encountered generally varies from weak to moderately strong and moderately weathered to highly weathered rock. We encountered practical refusal in Trenches T-12, T-13 and T-14 at depths of 9, 7 and 5 feet, respectively. Based on our analysis of the current excavations, we expect the proposed grading of the building pads and streets will be possible without blasting or rock breaking depending on the future planned removal depths. However, localized corestones and strong rock should be expected during the construction operations. The granitic rock is generally suitable for support of proposed fill and structural loads. In addition, the granitic rock is considered stable for construction of the proposed cut slopes if free of loose rock after excavation.

6. GROUNDWATER

We did not encounter groundwater or seepage during our site investigation. The geotechnical investigation to the south encountered groundwater between 13 to 28 feet below existing ground surface (elevations of 694 feet to 728 feet MSL). Additionally, 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 do not expect groundwater to be encountered during construction of the proposed development.

7. GEOLOGIC HAZARDS

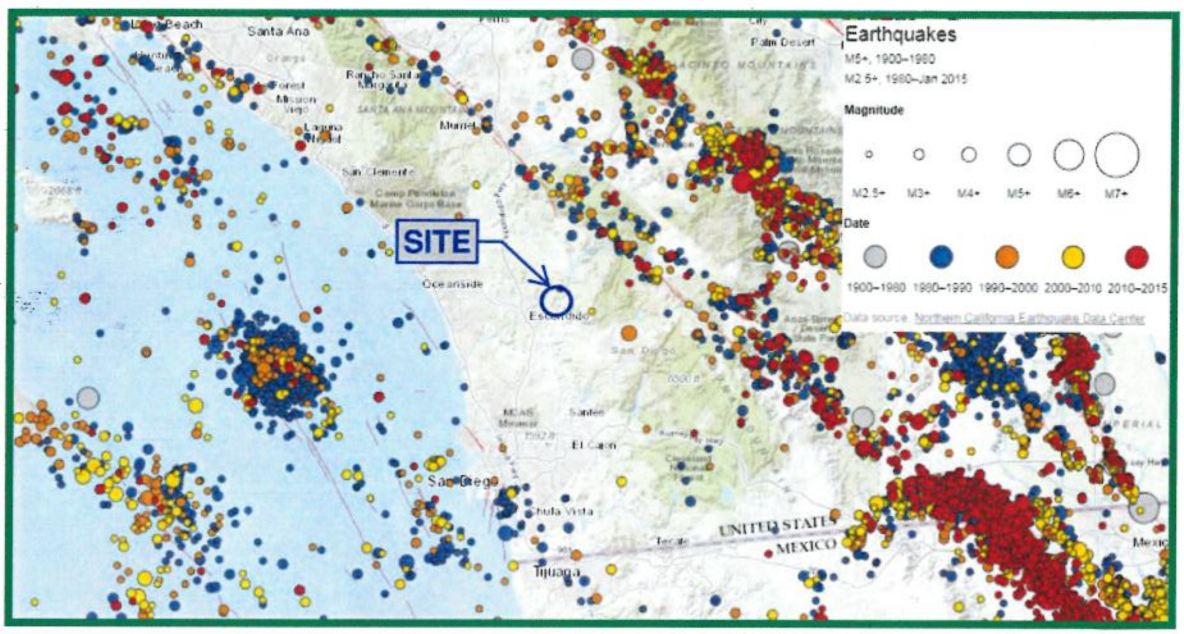
7.1 Regional Faulting and Seismicity

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. The Regional Geologic Map indicates a fault on the neighboring lot to the south; however, we did not observe the fault during the geotechnical investigation or during the grading operations. An active fault is defined by the California Geological Survey (CGS) as a fault showing evidence for activity within the last 11,700 years. The site is not located within a State of California Earthquake Fault Zone.

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

Faults in Southern California

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



Earthquakes in Southern California

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

7.2 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.

7.3 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.

Based on our previous soil investigation, the presence of groundwater within the upper 50 feet, and our knowledge of the area, we expect the potential for liquefaction exists on the site. Based on the report for the property to the south, we expect the potential for settlement is about 2 to 3 inches if unmitigated during future improvements. We should perform geotechnical borings and/or cone penetrometer tests (CPT) during the future investigation so we can evaluate liquefaction and settlement potential for the site.

7.4 Hydrocollapse

Hydrocollapse is the tendency of unsaturated soil structure to collapse upon saturation resulting in the overall settlement of the effected soil and overlying foundations or improvements supported thereon. Potentially compressible surficial soil underlying the proposed structures and existing fill is typically removed and recompacted during remedial site grading. However, if compressible soil is left in-place, a potential for settlement due to hydrocollapse of the soil exists. The potential for hydrocollapse can be mitigated by remedial grading and the use of stiffer foundation systems. Based on the limited laboratory test results, the potential for hydrocollapse ranges from 2.2 percent to 3.5 percent with an average of 2.85 percent within the older alluvium. If about 10 feet is left in place, we expect the amount of settlement due to hydrocollapse is approximately 3.4 inches. We should evaluate the hydrocollapse potential further in the upcoming geotechnical investigation.

7.5 Storm Surge, Tsunamis, and Seiches

Storm surges are large ocean waves that sweep across coastal areas when storms make landfall. Storm surges can cause inundation, severe erosion and backwater flooding along the water front. The site is located over 18 miles from the Pacific Ocean and is at an elevation of about 725 feet or greater above Mean Sea Level (MSL). Therefore, the potential of storm surges affecting the site is considered low.

A tsunami is a series of long period waves generated in the ocean by a sudden displacement of large volumes of water. Causes of tsunamis include underwater earthquakes, volcanic eruptions, or offshore slope failures. The 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.

7.6 Landslides

We did not observe evidence of previous or incipient slope instability at the site during our study. 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.

7.7 Rock Fall Hazard

The potential for rock fall impact to the proposed development was evaluated due to the moderate to steep terrain and localized areas of large granitic outcrops located east of the subject property. A low risk is defined as having no potential impact to proposed development and mitigation will not be required. A medium risk is defined as having some potential impact to proposed development and mitigation may be required. A high risk is an area that rock fall is eminent and significant mitigation will be required. We did not observe evidences of rock falls or areas that would be classified as having a medium to high risk. Therefore, in our professional opinion the potential for rockfall is considered low for the subject development.

7.8 Erosion

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

8. CONCLUSIONS AND RECOMMENDATIONS

8.1 General

8.1.1 We do not expect the geologic conditions would preclude the proposed development, provided we perform an updated geotechnical investigation when development plans have been prepared. Supplementary recommendations are presented herein and should be updated once the future investigation has been completed.

8.1.2 The site may be subject to geologic hazards, including: moderate to strong seismic shaking, liquefaction, seismically induced settlement, consolidation settlement and hydrocollapse. A future geotechnical investigation would be necessary to identify and to provided recommendations for the mitigation of these geologic hazards.

8.1.3 Our field investigation indicates that the site is underlain by topsoil, undocumented fill, colluvium, older alluvium, and granitic rock. The topsoil, undocumented fill, and upper portions of the older alluvium and colluvium are potentially compressible and unsuitable in their present condition for the support of compacted fill or settlement-sensitive improvements. Remedial grading of these materials should be performed as discussed herein. The granitic rock and dense portions of the older alluvium are considered suitable for the support of proposed fill and structural loads. The topsoil, alluvium and colluvium can be reused for new compacted fill during grading operations provided it is generally free of roots, debris and oversized rock.

8.1.4 We did not encounter groundwater during our subsurface exploration and we do not expect it to be a constraint to project development. However, our previous reports indicate groundwater to fluctuate between 13 to 28 feet below the existing surface. Seepage within surficial soils and rock materials may be encountered during the grading operations, especially during the rainy seasons.

8.1.5 Excavation of the topsoil, undocumented fill, older alluvium and colluvium should generally be possible with moderate to heavy effort using conventional, heavy-duty equipment during grading and trenching operations. We expect very heavy effort with possible refusal in localized areas for excavations into the granitic rock. The rippability of the granitic rock is variable and ranges between moderate to difficult. We do not expect a rock breaking and blasting program will be required for the proposed grading operations depending on the future grading plans. However, the grading contractor should be prepared to handle localized strong rock areas and rock corestones, if encountered.

8.1.6 In general, cut slopes composed of granitic rock 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 hard rock during grading operations to help evaluate stability.

8.1.7 Due to the potential of liquefaction on the western portion of the property, the proposed structures should be founded on mat foundations or post-tensioned shallow foundations designed to resist the potential for differential settlement. Recommendations for foundation systems are presented herein.

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

8.1.9 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. However, a detailed liquefaction analysis should be performed to evaluated the potential for settlement due to liquefaction.

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

8.2 Excavation and Soil Characteristics

8.2.1 Excavation of the in-situ soil should be possible with moderate to heavy effort using conventional heavy-duty equipment. Excavation of the granitic rock will require very heavy effort and may generate oversized material using conventional heavy-duty equipment during the grading operations. Oversized rock (rocks greater than 12-inches in dimension) may be generated within the existing materials that can be incorporated into landscape use or deep compacted fill areas, if available.

8.2.2 The soil encountered in the field investigation is considered to be "expansive" (expansion index [EI] of greater than 20) as defined by 2019 California Building Code (CBC) Section 1803.5.3. We expect a majority of the soil encountered possess a "very low" to "medium" expansion potential (EI of 90 or less) in accordance with ASTM D 4829. Table 8.2 presents soil classifications based on the expansion index.

TABLE 8.2
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	

8.2.3 We performed laboratory tests on samples of the site materials to evaluate the percentage of water-soluble sulfate content. Appendix B presents results of the laboratory water-soluble sulfate content tests. The test results indicate the on-site materials at the locations tested possess "S0" sulfate exposure to concrete structures as defined by 2019 CBC Section 1904 and ACI 318-19 Chapter 19. 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.

8.2.4 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.

8.3 Grading

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

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

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

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

8.3.5 The topsoil, undocumented fill, colluvium and the upper portions of the older alluvium within the site boundary should be excavated to competent soil prior to placing properly compacted fill. In addition, the site should be graded such that there is a minimum of 5 feet of compacted fill below proposed grade. The estimated minimum removal depths of unsuitable soil are shown on the Geologic Map, Figure 1. We should evaluate the actual excavation depths during the grading operations prior to placing compacted fill. We expect the removal depths will vary from about 2 to 8 feet below existing grade, depending on the finish grade elevations. The removals should extend at least 10 feet and 2 feet outside the proposed foundation zones and improvement areas, where possible. The existing soil may be reused for new compacted fill provided it is generally free of roots, debris, and oversized rock. Table 8.3.1 provides a summary of the grading recommendations.

TABLE 8.3.1
SUMMARY OF GRADING RECOMMENDATIONS

Area	Removal Requirements
Building Pads	Excavation of unsuitable material (2 to 8 feet below existing grade) or 5 Feet Below Pad Grade (Whichever is Deeper)
Site Improvements	Process Upper 1 to 2 Feet of Existing Materials
Grading Limits	10 feet Outside Proposed Building
Exposed Bottoms of Remedial Grading	Scarify Upper 12 Inches

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

8.3.7 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).

8.3.8 The contractor should be careful during the remedial grading operations to avoid a "pumping" condition at the base of the removals. Where recompaction of the excavated bottom will result in a "pumping" condition, the bottom of the excavation should be tracked with low ground pressure earthmoving equipment prior to placing fill. If needed to improve the stability of the excavation bottoms, reinforcing fabric or 2- to 3-inch crushed rock can be placed prior to placement of compacted fill.

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

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

TABLE 8.3.2
SUMMARY OF IMPORT FILL RECOMMENDATIONS

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

8.4 Earthwork Grading Factors

8.4.1 Estimates of embankment shrink-swell factors are based on comparing laboratory compaction tests with the density of the material in its natural state and on experience with similar soil types. Variations in natural soil density, as well as in compacted fill, render shrinkage value estimates very approximate. As an example, the contractor can compact fills to any density of 90 percent or higher of the laboratory maximum dry density. Thus, the contractor has at least a 10 percent range of control over the fill volume. Based on work performed to date and considering the above discussion, the earthwork factors shown in Table 8.4 may be used as a basis for estimating how much the on-site soil may shrink or bulk when removed from their natural state and placed as compacted fill and these values should be considered approximate.

TABLE 8.4
EARTHWORK GRADING FACTORS

Soils Unit	Shrink-Bulk Factors
Undocumented fill, topsoil, alluvium, and colluvium	5 to 15 Percent Shrinkage
Granitic Rock-Rippable	5 to 15 Percent Bulk
Granitic Rock-Nonrippable	15 to 25 Percent Bulk

8.4.2 Removal of the root systems of the existing trees will be an important consideration in determining the amount of shrinkage that will occur in the topsoil. Based on our experience with similar projects, 0.3 to 0.5 feet of subsidence (shrinkage) can occur due to removal of the roots. This subsidence should be accounted for within the areas where trees exist and should not be applicable to those areas with little to no trees.

8.5 Subdrains

8.5.1 With the exception of retaining wall drains, we do not expect the installation of other subdrains. We should be contacted to provide recommendations for wick drains, if proposed.

8.6 Temporary Excavations

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

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

8.7 Seismic Design Criteria – 2019 California Building Code

8.7.1 Table 8.7.1 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.

8.7.2 The existing site possesses Site Classes C, D and E. We expect the property also possesses Site Class F where liquefiable materials exist; however, we expect a Site Class E can be used if the proposed structures possess a period of less than 0.5 second. Site Class C can be used where we have less than 20 feet of fill and older alluvium. Site Class D should be used where deeper older alluvium exists and where we do not have liquefaction. We estimated the Site Class based on the discussion in Section 1613.2.2 of the 2019 CBC and Table 20.3-1 of ASCE 7-16. The values presented herein are for the risk-targeted maximum considered earthquake (MCE_R). Sites designated as Site Class D, E and F may require additional analyses if requested by the project structural engineer and client.

TABLE 8.7.1
2019 CBC SEISMIC DESIGN PARAMETERS

Parameter	Value			2019 CBC Reference
Site Class	C	D	E	Section 1613.2.2
Site Characteristics	Less Than 20 Feet of Fill and Older Alluvium	20+ Feet of Fill and Older Alluvium – No Liquefaction	Liquefaction Potential Exists	--
MCE_R Ground Motion Spectral Response Acceleration – Class B (short), S_S	0.961g	0.961g	0.961g	Figure 1613.2.1(1)
MCE_R Ground Motion Spectral Response Acceleration – Class B (1 sec), S_1	0.347g	0.347g	0.347g	Figure 1613.2.1(2)
Site Coefficient, F_A	1.200	1.116	1.300	Table 1613.2.3(1)
Site Coefficient, F_V	1.500	1.953*	2.612*	Table 1613.2.3(2)
Site Class Modified MCE_R Spectral Response Acceleration (short), S_{MS}	1.153g	1.072g	1.249g	Section 1613.2.3 (Eqn 16-36)
Site Class Modified MCE_R Spectral Response Acceleration – (1 sec), S_{M1}	0.520g	0.678g*	0.906g*	Section 1613.2.3 (Eqn 16-37)
5% Damped Design Spectral Response Acceleration (short), S_{DS}	0.768g	0.715g	0.833g	Section 1613.2.4 (Eqn 16-38)
5% Damped Design Spectral Response Acceleration (1 sec), S_{D1}	0.347g	0.452g*	0.604g*	Section 1613.2.4 (Eqn 16-39)

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

8.7.3 Table 8.7.2 presents the mapped maximum considered geometric mean (MCE_G) seismic design parameters for projects located in Seismic Design Categories of D through F in accordance with ASCE 7-16.

TABLE 8.7.2
ASCE 7-16 PEAK GROUND ACCELERATION

Parameter	Value			ASCE 7-16 Reference
Site Class	C	D	E	--
Mapped MCE_G Peak Ground Acceleration, PGA	0.416g	0.416g	0.416g	Figure 22-9
Site Coefficient, F_{PGA}	1.200	1.184	1.368	Table 11.8-1
Site Class Modified MCE_G Peak Ground Acceleration, PGA_M	0.499g	0.492g	0.569g	Section 11.8.3 (Eqn 11.8-1)

8.7.4 Conformance to the criteria in Tables 8.7.1 and 8.7.2 for seismic design does not constitute any kind of guarantee or assurance that significant structural damage or ground failure will not occur in the event of a large earthquake. The primary goal of seismic design is to protect life, not to avoid all damage, since such design may be economically prohibitive.

8.7.5 The project structural engineer and architect should evaluate the appropriate Risk Category and Seismic Design Category for the planned structures. The values presented herein assume a Risk Category of II and resulting in a Seismic Design Category D. Table 8.7.3 presents a summary of the risk categories in accordance with ASCE 7-16.

TABLE 8.7.3
ASCE 7-16 RISK CATEGORIES

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

8.8 Foundation and Concrete Slabs-On-Grade Recommendations

8.8.1 The foundation recommendations herein are for proposed one- to three-story residential structures. The foundation recommendations have been separated into three categories based on either the maximum and differential fill thickness or Expansion Index. The foundation category criteria are presented in Table 8.8.1.

TABLE 8.8.1
FOUNDATION CATEGORY CRITERIA

Foundation Category	Maximum Fill Thickness, T (Feet)	Differential Fill Thickness, D (Feet)	Expansion Index (EI)
I	$T<20$	--	$EI\leq50$
II	$20\leq T<50$	$10\leq D<20$	$50<EI\leq90$
III	$T\geq50$	$D\geq20$	$90<EI\leq130$
	Older Alluvium Exists Below Building Pad		

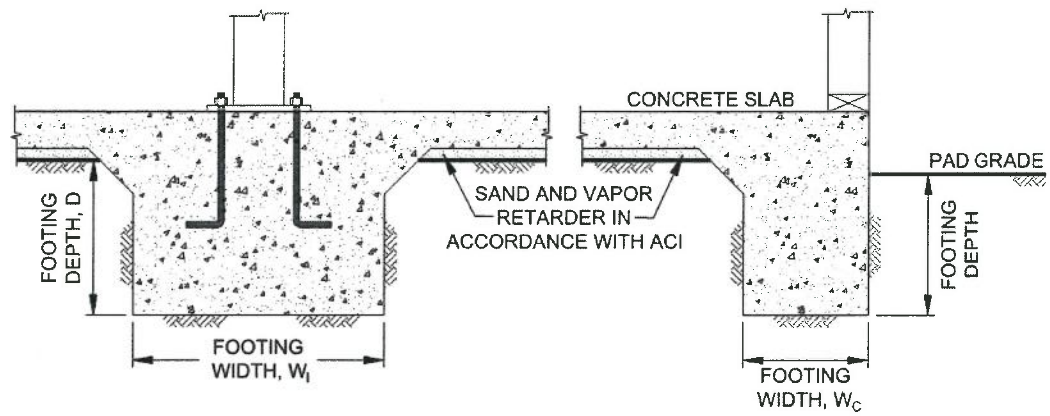
8.8.2 We will provide final foundation categories for each building or lot after finish pad grades have been achieved and we perform laboratory testing of the subgrade soil.

8.8.3 Table 8.8.2 presents minimum foundation and interior concrete slab design criteria for conventional foundation systems for ancillary structures.

TABLE 8.8.2
CONVENTIONAL FOUNDATION RECOMMENDATIONS BY CATEGORY FOR ANCILLARY STRUCTURES

Foundation Category	Minimum Footing Embedment Depth, D (inches)	Minimum Continuous Footing Reinforcement	Minimum Footing Width (Inches)
I	12	Two No. 4 bars, one top and one bottom	12 – Continuous, W_C 24 – Isolated, W_I
II	18	Four No. 4 bars, two top and two bottom	
III	24	Four No. 5 bars, two top and two bottom	

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



Wall/Column Footing Dimension Detail

8.8.5 The proposed structures can be supported on a shallow foundation system founded in the compacted fill. Table 8.8.3 provides a summary of the foundation design recommendations.

TABLE 8.8.3
SUMMARY OF FOUNDATION RECOMMENDATIONS

Parameter	Value
Allowable Bearing Capacity	2,000 psf
Estimated Total Settlement	1 Inch
Estimated Differential Settlement	½ Inch in 40 Feet

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

8.8.7 The concrete slab-on-grades for ancillary structures should be a designed in accordance with Table 8.8.4.

TABLE 8.8.4
CONVENTIONAL SLAB-ON-GRADE RECOMMENDATIONS BY CATEGORY FOR ANCILLARY STRUCTURES

Foundation Category	Minimum Concrete Slab Thickness (inches)	Interior Slab Reinforcement	Typical Slab Underlayment
I	4	6 x 6 - 10/10 welded wire mesh at slab mid-point	3 to 4 Inches of Sand/Gravel/Base
II	4	No. 3 bars at 24 inches on center, both directions	
III	5	No. 3 bars at 18 inches on center, both directions	

8.8.8 The planned residential structures should be supported on post-tensioned concrete slab and foundation systems. The post-tensioned systems (foundation dimensions and embedment depths, slab thickness and steel placement) should be designed by a structural engineer experienced in post-tensioned slab design and design criteria of the Post-Tensioning Institute (PTI) DC 10.5-12 *Standard Requirements for Design and Analysis of Shallow Post-Tensioned Concrete Foundations on Expansive Soils* or *WRI/CRSI Design of Slab-on-Ground Foundations*, as required by the 2019 California Building Code (CBC Section 1808.6.2). Although this procedure was developed for expansive soil conditions, it can also be used to reduce the potential for foundation distress due to differential fill settlement. The post-tensioned design should incorporate the geotechnical parameters presented in Table 8.8.3 for the particular Foundation Category designated. The parameters presented in Table 8.8.5 are based on the guidelines presented in the PTI DC 10.5 design manual.

TABLE 8.8.5
POST-TENSIONED FOUNDATION SYSTEM DESIGN PARAMETERS – PROPOSED RESIDENTIAL STRUCTURES

Post-Tensioning Institute (PTI) DC10.5 Design Parameters	Foundation Category		
	I	II	III
Thornthwaite Index	-20	-20	-20
Equilibrium Suction	3.9	3.9	3.9
Edge Lift Moisture Variation Distance, e_M (Feet)	5.3	5.1	4.9
Edge Lift, y_M (Inches)	0.61	1.10	1.58
Center Lift Moisture Variation Distance, e_M (Feet)	9.0	9.0	9.0
Center Lift, y_M (Inches)	0.30	0.47	0.66

8.8.9 The foundations for the post-tensioned slabs should be embedded in accordance with the recommendations of the structural engineer. If a post-tensioned mat foundation system is planned, the slab should possess a thickened edge with a minimum width of 12 inches and extend below the clean sand or crushed rock layer.

8.8.10 If the structural engineer proposes a post-tensioned foundation design method other than PTI, DC 10.5:

- The deflection criteria presented in Table 8.8.5 are still applicable.
- Interior stiffener beams should be used for Foundation Categories II and III.
- The width of the perimeter foundations should be at least 12 inches.
- The perimeter footing embedment depths should be at least 12 inches, 18 inches and 24 inches for foundation categories I, II, and III, respectively. The embedment depths should be measured from the lowest adjacent pad grade.

8.8.11 Foundation systems for the lots that possess a foundation Category I and a "very low" expansion potential (expansion index of 20 or less) can be designed using the method described in Section 1808 of the 2019 CBC. If post-tensioned foundations are planned, an alternative, commonly accepted design method (other than PTI) can be used. However, the post-tensioned foundation system should be designed with a total and differential deflection of 1 inch. Geocon Incorporated should be contacted to review the plans and provide additional information, if necessary.

8.8.12 If an alternate design method is contemplated, Geocon Incorporated should be contacted to evaluate if additional expansion index testing should be performed to identify the lots that possess a "very low" expansion potential (expansion index of 20 or less).

8.8.13 Our experience indicates post-tensioned slabs may be susceptible to excessive edge lift from tensioning, regardless of the underlying soil conditions. Placing reinforcing steel at the bottom of the perimeter footings and the interior stiffener beams may mitigate this potential. The structural engineer should design the foundation system to reduce the potential of edge lift occurring for the proposed structures.

8.8.14 During the construction of the post-tension foundation system, the concrete should be placed monolithically. Under no circumstances should cold joints form between the footings/grade beams and the slab during the construction of the post-tension foundation system unless designed by the structural engineer.

8.8.15 Isolated footings outside of the slab area, if present, should have the minimum embedment depth and width recommended for conventional foundations for a particular Foundation Category. The use of isolated footings, which are located beyond the perimeter of the building and support structural elements connected to the building, are not recommended for Category III. Where this condition cannot be avoided, the isolated footings should be connected to the building foundation system with grade beams in both directions. In addition, consideration should be given to connecting patio slabs, which exceed 5 feet in width, to the building foundation to reduce the potential for future separation to occur.

8.8.16 Interior stiffening beams should be incorporated into the design of the foundation system in accordance with the PTI design procedures.

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

8.8.18 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). The vapor retarder used should be specified by the project architect or developer based on the type of floor covering that will be installed and if the structure will possess a humidity controlled environment.

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

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

- For fill slopes less than 20 feet high or cut slopes regardless of height, footings should be deepened such that the bottom outside edge of the footing is at least 7 feet horizontally from the face of the slope.
- When located next to a descending 3:1 (horizontal:vertical) fill slope or steeper, the foundations should be extended to a depth where the minimum horizontal distance is equal to H/3 (where H equals the vertical distance from the top of the fill slope to the base of the fill soil) with a minimum of 7 feet but need not exceed 40 feet. The horizontal distance is measured from the outer, deepest edge of the footing to the face of the slope. A post-tensioned slab and foundation system or mat foundation system can be used to reduce the potential for distress in the structures associated with strain softening and lateral fill extension. 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 7 feet of the top of cut or fill slopes are not recommended. Where such a condition cannot be avoided, the portion of the swimming pool wall within 7 feet of the slope face be designed assuming that the

adjacent soil provides no lateral support. 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 which would permit some lateral soil movement without causing extensive distress. Geocon Incorporated should be consulted for specific recommendations.

8.8.21 The recommendations of this report are intended to reduce the potential for cracking of slabs and foundations due to expansive soil (if present), differential settlement of fill 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 by limiting the slump of the concrete, proper concrete placement and curing, and by the placement of crack control joints at periodic intervals, in particular, where re-entrant slab corners occur.

8.8.22 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 when establishing crack-control spacing. Additional steel reinforcing, concrete admixtures and/or closer crack control joint spacing should be considered where concrete-exposed finished floors are planned.

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

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

8.9 Exterior Concrete Flatwork

8.9.1 Exterior concrete flatwork not subject to vehicular traffic should be constructed in accordance with the recommendations presented in Table 8.9. The recommended steel reinforcement would help reduce the potential for cracking.

TABLE 8.9
MINIMUM CONCRETE FLATWORK RECOMMENDATIONS

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

*In excess of 8 feet square.

8.9.2 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 D 1557.

8.9.3 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.

8.9.4 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.

8.9.5 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.

8.9.6 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.

8.10 Retaining Walls

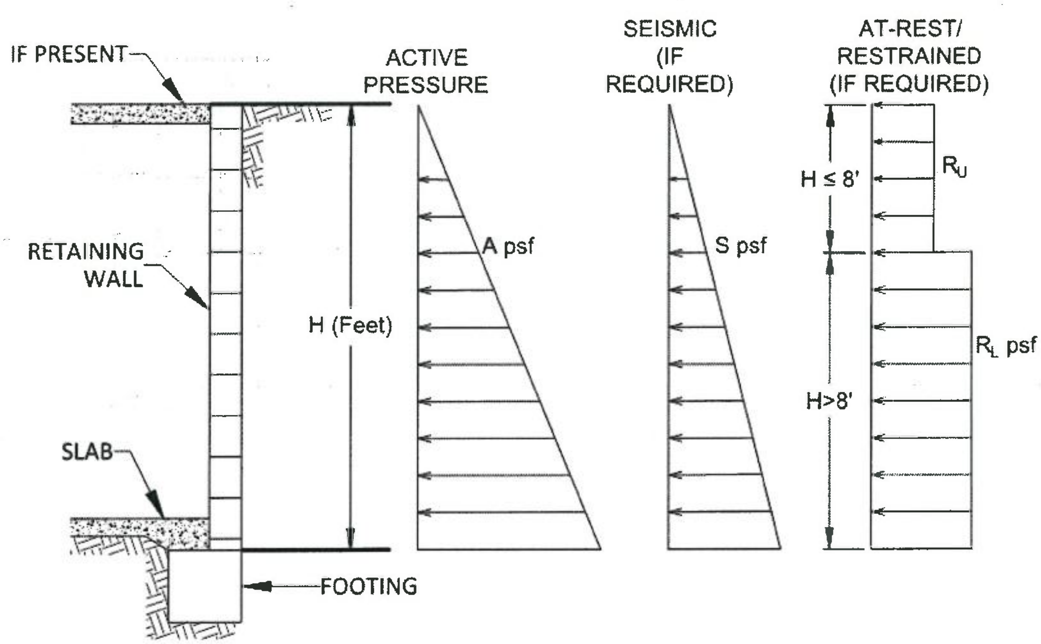
8.10.1 Retaining walls should be designed using the values presented in Table 8.10.1. Soil with an expansion index (EI) of greater than 50 should not be used as backfill material behind retaining walls.

TABLE 8.10.1
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 (0 to 8 Feet High)	7H psf
At-Rest/Restrained Walls Additional Uniform Pressure (8+ Feet High)	13H psf
Expected Expansion Index for the Subject Property	EI≤ 50

H equals the height of the retaining portion of the wall

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



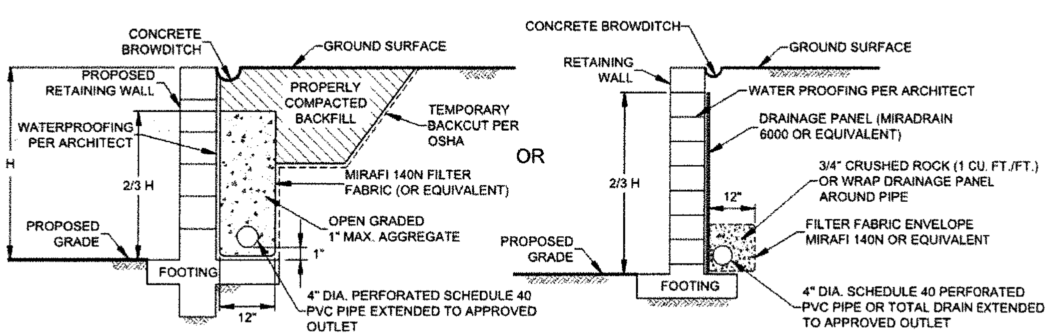
Retaining Wall Loading Diagram

8.10.3 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.

8.10.4 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 7-10. For structures assigned to Seismic Design Category of D, E, or F, retaining walls that support more than 6 feet of backfill should be designed with seismic lateral pressure in accordance with Section 1803.5.12 of the 2019 CBC. The seismic load is dependent on the retained height where H is the height of the wall, in feet, and the calculated loads result in pounds per square foot (psf) exerted at the base of the wall and zero at the top of the wall.

8.10.5 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.

8.10.6 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

8.10.7 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.

8.10.8 In general, wall foundations should be designed in accordance with Table 8.10.2. 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 8.10.2
SUMMARY OF RETAINING WALL FOUNDATION RECOMMENDATIONS

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

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

8.10.10 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.

8.10.11 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.

8.10.12 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.

8.11 Lateral Loading

8.11.1 Table 8.11 should be used to help design the proposed structures and improvements to resist lateral loads for the design of footings or shear keys. The allowable passive pressure assumes a horizontal surface extending at least 5 feet, or three times the surface generating the passive pressure, whichever is greater. The upper 12 inches of material in areas not protected by floor slabs or pavement should not be included in design for passive resistance.

TABLE 8.11
SUMMARY OF LATERAL LOAD DESIGN RECOMMENDATIONS

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

*Per manufacturer's recommendations.

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

8.12 Preliminary Pavement Recommendations

8.12.1 We calculated the flexible pavement sections in general conformance with the *Caltrans Method of Flexible Pavement Design* (Highway Design Manual, Section 608.4) using an estimated Traffic Index (TI) of 5.0, 5.5, 6.0, and 7.0 for parking stalls, driveways, medium truck traffic areas, and heavy truck traffic areas, respectively. The project civil engineer and owner should review the pavement designations to determine appropriate locations for pavement thickness. The final pavement sections for the parking lot should be based on the R-Value of the subgrade soil encountered at final subgrade elevation. We used an R-Value of 15 and 78 for the subgrade soil and base materials, respectively, for the purposes of this preliminary analysis. Table 8.12.1 presents the preliminary flexible pavement sections.

TABLE 8.12.1
PRELIMINARY FLEXIBLE PAVEMENT SECTION

Location	Assumed Traffic Index	Assumed Subgrade R-Value	Asphalt Concrete (inches)	Class 2 Aggregate Base (inches)
Parking stalls for automobiles and light-duty vehicles	5.0	15	3	8
Driveways for automobiles and light-duty vehicles	5.5	15	3	10
Medium truck traffic areas	6.0	15	3.5	11
Driveways for heavy truck traffic	7.0	15	4	13

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

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

8.12.4 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 required.

8.12.5 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 330R-08 Guide for Design and Construction of Concrete Parking Lots using the parameters presented in Table 8.12.2.

TABLE 8.12.2
RIGID PAVEMENT DESIGN PARAMETERS

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

8.12.6 Based on the criteria presented herein, the PCC pavement sections should have a minimum thickness as presented in Table 8.12.3.

TABLE 8.12.3
RIGID VEHICULAR PAVEMENT RECOMMENDATIONS

Location	Portland Cement Concrete (inches)
Automobile Parking Stalls (TC=A, ADTT=10)	5.5
Driveways (TC=C, ADTT=100)	7

8.12.7 The PCC vehicular pavement should be placed over subgrade soil that is compacted to a dry density of at least 95 percent of the laboratory maximum dry density near to slightly above optimum moisture content.

8.12.8 The rigid pavement should also be designed and constructed incorporating the parameters presented in Table 8.12.4.

TABLE 8.12.4
ADDITIONAL RIGID PAVEMENT RECOMMENDATIONS

Subject	Value
Thickened Edge	1.2 Times Slab Thickness
	Minimum Increase of 2 Inches
	4 Feet Wide
Crack Control Joint Spacing	30 Times Slab Thickness
	Max. Spacing of 12 feet for 5.5-Inch-Thick
	Max. Spacing of 15 Feet for Slabs 6 Inches and Thicker
Crack Control Joint Depth	Per ACI 330R-08
	1 Inch Using Early-Entry Saws on Slabs Less Than 9 Inches Thick
Crack Control Joint Width	¼-Inch for Sealed Joints
	⅜-Inch is Common for Sealed Joints
	$^1/_{10}$- to $^1/_8$-Inch is Common for Unsealed Joints

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

8.12.10 To control the location and spread of concrete shrinkage cracks, crack-control joints (weakened plane joints) should be included in the design of the concrete pavement slab. Crack-control joints should be sealed with an appropriate sealant to prevent the migration of water through the control joint to the subgrade materials. The depth of the crack-control joints should be determined by the referenced ACI report.

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

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

8.13 Site Drainage and Moisture Protection

8.13.1 Adequate site drainage is critical to reduce the potential for differential soil movement, erosion and subsurface seepage. Under no circumstances should water be allowed to pond adjacent to footings. The site should be graded and maintained such that surface drainage is directed away from structures in accordance with 2019 CBC 1804.4 or other applicable standards. In addition, surface drainage should be directed away from the top of slopes into swales or other controlled drainage devices. Roof and pavement drainage should be directed into conduits that carry runoff away from the proposed structure.

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

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

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

8.13.5 We should prepare a storm water infiltration feasibility report of storm water management devices are planned. We do not expect infiltration will be feasible on the property due to the presence of hydrocollapse in the existing soil.

8.14 Future Geotechnical Investigation

8.14.1 We should perform a future geotechnical investigation to evaluate the liquefaction hazards on the subject site. The additional investigation should include borings and/or cone penetration tests (CPTs) to approximately 60 feet deep to help evaluate the potential liquefaction potential on the western portion of the site.

8.14.2 The future investigation would also include laboratory tests on selected soil samples to evaluate shear strength, consolidation, in-situ dry density/moisture content, plasticity index and gradation of the soil encountered.

8.14.3 The updated geotechnical investigation report would present our findings, conclusions, and recommendations regarding the geotechnical aspects of structures as presently proposed. The proposed report would include ground modification recommendations, foundation and concrete slab on-grade design criteria, current California Building Code seismic design parameters, excavation characteristics, geologic hazard analyses, remedial grading measures, preliminary pavement sections, concrete flatwork and retaining wall recommendations.

8.15 Grading and Foundation Plan Review

8.15.1 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.

8.16 Testing and Observation Services During Construction

8.16.1 Geocon Incorporated should provide geotechnical testing and observation services during the grading operations, foundation construction, utility installation, retaining wall backfill and pavement installation. Table 8.1 presents the typical geotechnical observations we would expect for the proposed improvements.

TABLE 8.16
EXPECTED GEOTECHNICAL TESTING AND OBSERVATION SERVICES

Construction Phase	Observations	Expected Time Frame
Grading	Base of Removal	Part Time During Removals
	Geologic Logging	Part Time to Full Time
	Fill Placement and Soil Compaction	Full Time
Foundations	Foundation Excavation Observations	Part Time
Utility Backfill	Fill Placement and Soil Compaction	Part Time to Full Time
Retaining Wall Backfill	Fill Placement and Soil Compaction	Part Time to Full Time
Subgrade for Sidewalks, Curb/Gutter and Pavement	Soil Compaction	Part Time
Pavement Construction	Base Placement and Compaction	Part Time
	Asphalt Concrete Placement and Compaction	Full Time

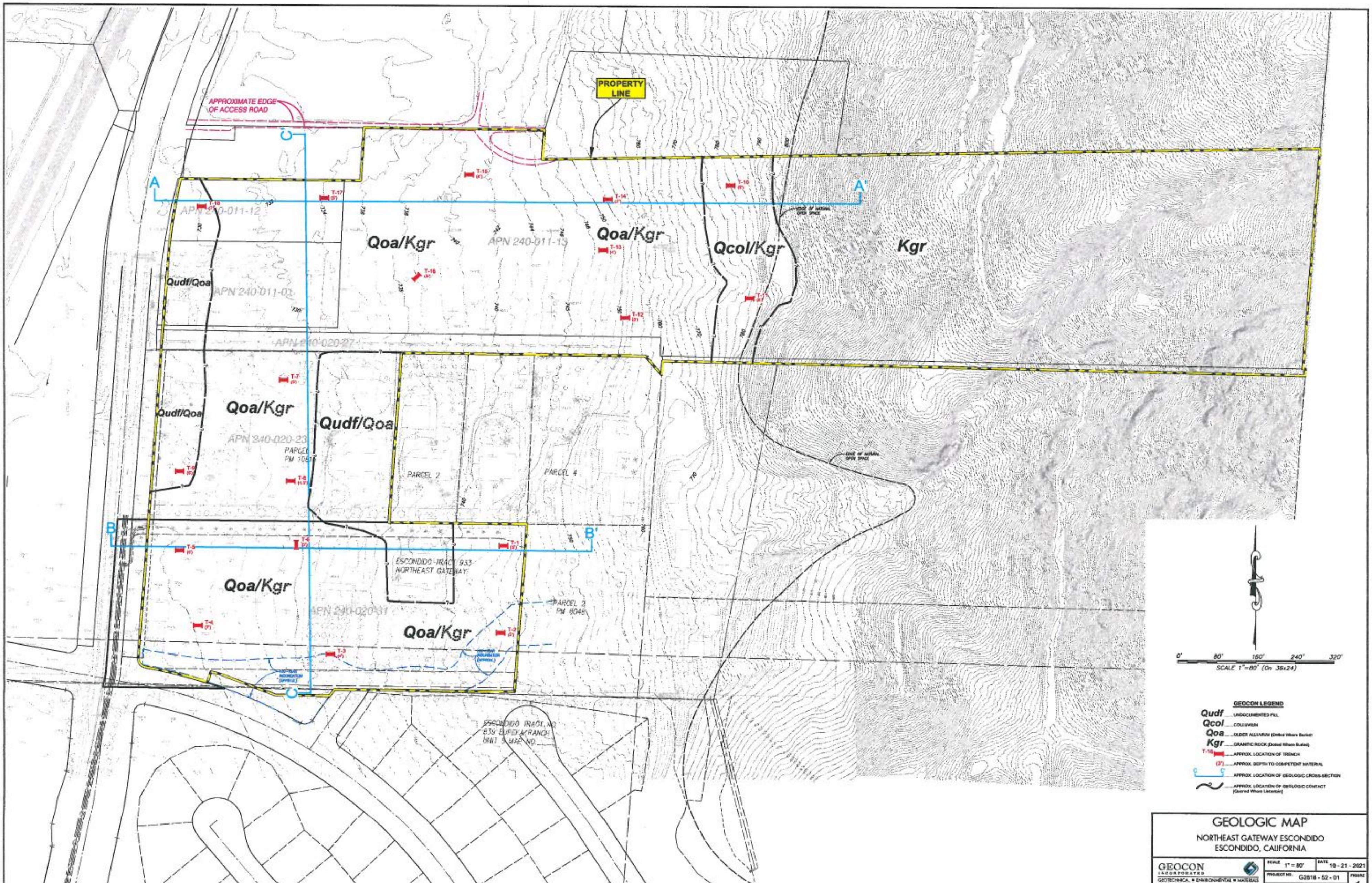
LIMITATIONS AND UNIFORMITY OF CONDITIONS

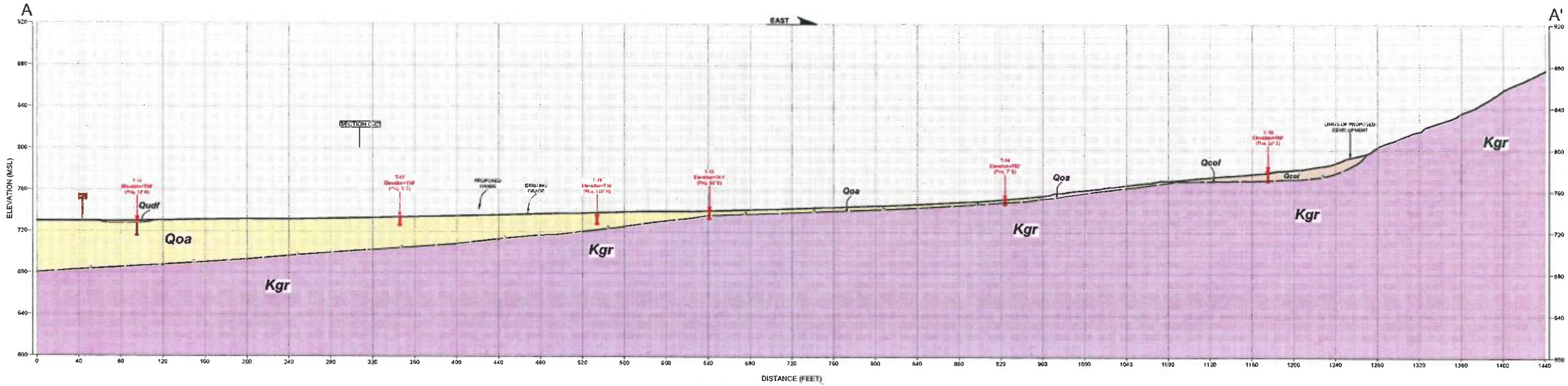
1. The firm that performed the geotechnical investigation for the project should be retained to provide testing and observation services during construction to provide continuity of geotechnical interpretation and to check that the recommendations presented for geotechnical aspects of site development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

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

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

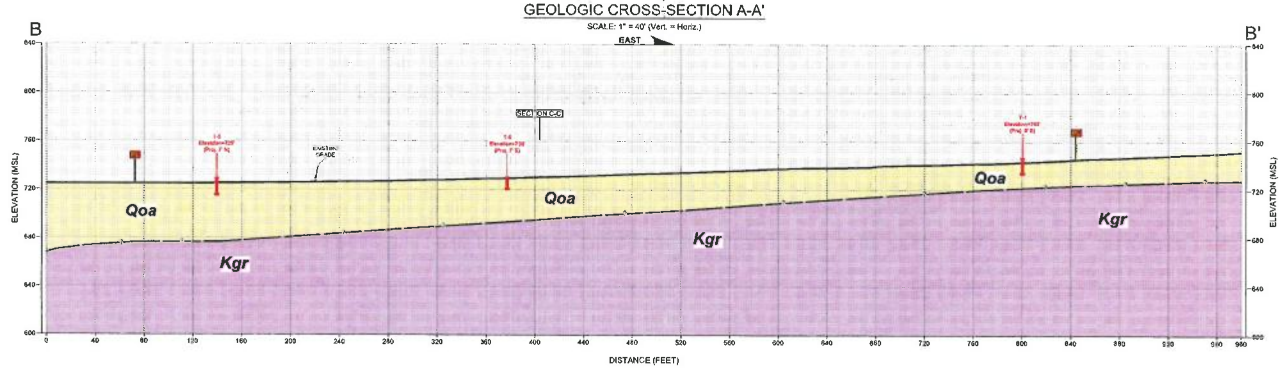
4. The findings of this report are valid as of the present date. However, changes in the conditions of a property can occur with the passage of time, whether they be due to natural processes or the works of man on this or adjacent properties. In addition, changes in applicable or appropriate standards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


PROPERTY LINE
APPROXIMATE EDGE OF ACCESS ROAD
A
A'
B
B'
C
C'
Qoa/Kgr
Qcol/Kgr
Kgr
Qudf/Qoa
APN 240-011-12
APN 240-011-13
APN 240-011-01
APN 240-020-27
APN 240-020-23
APN 240-020-31
PARCEL 2
PARCEL 4
PARCEL 2 PM 6048
ESCONDIDO TRACT 933 NORTHEAST GATEWAY
ESCONDIDO TRACT NO 839 EUREKA RANCH UNIT 5 MAP NO
T-1
T-2
T-3
T-4
T-5
T-6
T-7
T-8
T-9
T-10
T-11
T-12
T-13
T-14
T-15
T-16
T-17
T-18
0'
80'
160'
240'
320'
SCALE 1"=80' (On 36x24)
GEOCON LEGEND
Qudf ...UNDOCUMENTED FILL
Qcol ...COLLUVIUM
Qoa ...OLDER ALLUVIUM (Dotted Where Buried)
Kgr ...GRANITIC ROCK (Dotted Where Buried)
T-16 ...APPROX. LOCATION OF TRENCH
(7') ...APPROX. DEPTH TO COMPETENT MATERIAL
...APPROX. LOCATION OF GEOLOGIC CROSS-SECTION
...APPROX. LOCATION OF GEOLOGIC CONTACT (Queried Where Uncertain)
GEOLOGIC MAP
NORTHEAST GATEWAY ESCONDIDO
ESCONDIDO, CALIFORNIA
GEOCON INCORPORATED
GEOTECHNICAL ■ ENVIRONMENTAL ■ MATERIALS
SCALE 1" = 80'
DATE 10 - 21 - 2021
PROJECT NO. G2818 - 52 - 01
FIGURE 1
SHEET 1 OF 1



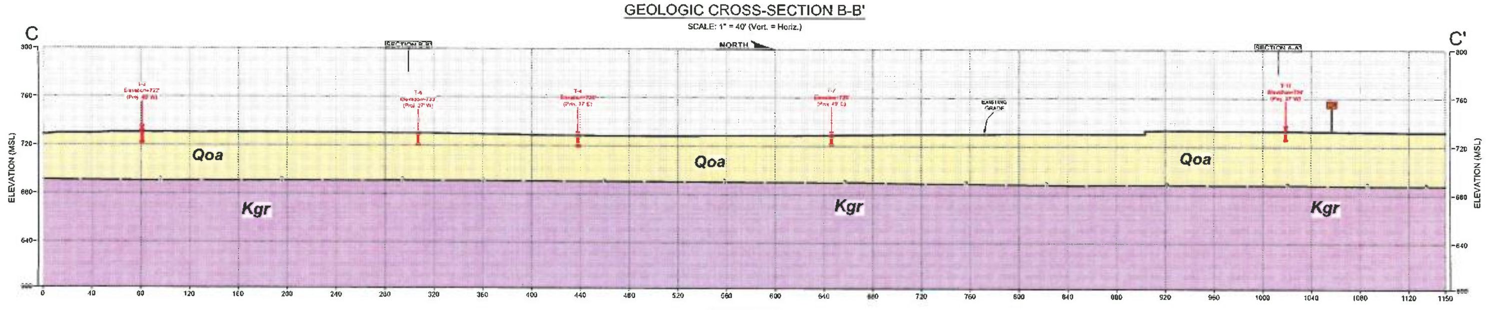
GEOLOGIC CROSS-SECTION A-A'

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



GEOLOGIC CROSS-SECTION B-B'

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



GEOLOGIC CROSS-SECTION C-C'

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

GEOCON LEGEND

Qudf ......UNDOCUMENTED FILL
Qcol ......COLLUVIUM
Qoa ......OLDER ALLUVIUM
Kgr ......GRANITIC ROCK
T-18 ......APPROX. LOCATION OF TRENCH
......APPROX. LOCATION OF GEOLOGIC CONTACT (Dotted Where Uncertain)

GEOLOGIC CROSS - SECTIONS

NORTHEAST GATEWAY ESCONDIDO
ESCONDIDO, CALIFORNIA

GEOCON INCORPORATED

SCALE 1" = 40'	DATE 10 - 21 - 2021
PROJECT NO. G2818 - 52 - 01	FIGURE 2
SHEET 1 OF 1	

APPENDIX A

APPENDIX A

FIELD INVESTIGATION

We performed the trenching operations on September 22, 2021 to depths ranging from approximately 5 to 14 feet below existing grade using a John Deere 310L EP back-hoe with MCM Construction performing the work. The Geologic Map, Figure 1, shows the approximate locations of the exploratory trenches. The trench logs are presented in this Appendix. We located the trenches in the field using a measuring tape and existing reference points; therefore, actual boring locations may deviate slightly.

We obtained bulk, ring, and chunk samples during our subsurface exploration in the trenches. We obtained the ring samples during our subsurface exploration using hand tools and a knocker bar. We obtained the samples at appropriate depths, placed them in moisture-tight containers, and transported them to the laboratory for testing. The type of sample is noted on the exploratory trench logs. We estimated elevations shown on the trench logs either from a topographic map or by using a benchmark. Each excavation was backfilled as noted on the trench logs.

We visually examined, classified, and logged the soil encountered in the trenches 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.

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	TRENCH T 1 ELEV. (MSL.) 743' DATE COMPLETED 9-22-2021 EQUIPMENT BACKHOE BY: D. THOMAS	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
					MATERIAL DESCRIPTION			
0	T1-1			SM-SC	**TOPSOIL** Loose, dry, brown, Silty to Clayey SAND			
				SM	**OLDER ALLUVIUM (Qoa)** Medium dense, dry to damp, light brown, Silty, fine to coarse SAND; trace of cobble to boulder clast			
2								
4	T1-2						110.7	2.9
					-Becomes dense and damp, less cobble/gravel at 5 feet			
6								
					-Becomes very dense, difficulty digging at 7 feet			
8								
					TRENCH TERMINATED AT 9 FEET No groundwater encountered			

Figure A-1,
Log of Trench T 1, Page 1 of 1

G2818-52-01.GPJ

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

PROJECT NO. G2818-52-01

TRENCH T 2

ELEV. (MSL.) 743' DATE COMPLETED 9-22-2021

EQUIPMENT BACKHOE BY: D. THOMAS

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0				SM-SC	**TOPSOIL** Loose, dry, dark brown, Silty to Clayey, fine SAND			
				SM	**OLDER ALLUVIUM (Qoa)** Loose to medium dense, dry to damp, light brown, Silty, fine to coarse SAND; some gravel			
2								
4	T2-1						107.5	2.6
	T2-2							
					-Becomes dense, moist, and medium brown			
6								
8								
				SC	Dense, moist, brown, Clayey, fine to coarse SAND; some gravel/cobble			
10					TRENCH TERMINATED AT 10 FEET No groundwater encountered			

Figure A-2,
Log of Trench T 2, Page 1 of 1

G2818-52-01.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. G2818-52-01

TRENCH T 3

ELEV. (MSL.) 732' DATE COMPLETED 9-22-2021

EQUIPMENT BACKHOE BY: D. THOMAS

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0				SM	**TOPSOIL** Loose, dry, light brown, Silty, fine to coarse SAND			
2				SM	**OLDER ALLUVIUM (Qoa)** Medium dense, dry, brown, Silty, fine to coarse SAND; trace of gravel			
4	T3-1			SM-SC	Dense, dry to damp, light brown, Silty to Clayey, fine to coarse SAND; some gravel		109.2	2.8
6								
	T3-2			SC	Dense, moist, brown, Clayey, fine to coarse SAND with some gravel			
8								
10					TRENCH TERMINATED AT 10 FEET No groundwater encountered			

Figure A-3,
Log of Trench T 3, Page 1 of 1

G2818-52-01.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

TRENCH T 4

ELEV. (MSL.) 725' DATE COMPLETED 9-22-2021

EQUIPMENT BACKHOE BY: D. THOMAS

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0				SM	**TOPSOIL** Loose to medium dense, dry, brown, Silty, fine to coarse SAND			
2				CL-SC	**OLDER ALLUVIUM (Qoa)** Stiff, moist, brown, Sandy CLAY to Clayey SAND; trace of gravel			
	T4-1							
4	T4-2						115.1	14.6
6								
					-Becomes dense to very dense at 7 feet			
8					TRENCH TERMINATED AT 8 FEET No groundwater encountered			

Figure A-4,
Log of Trench T 4, Page 1 of 1

G2818-52-01.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.

TRENCH T 5

ELEV. (MSL.) 725' DATE COMPLETED 9-22-2021

EQUIPMENT BACKHOE BY: D. THOMAS

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0				SM	**OLDER ALLUVIUM (Qoa)** Loose, damp, brown, Silty, fine to medium SAND; trace of gravel			
2								
					-Becomes moist at 3 feet			
4	T5-1						108.8	6.7
	T5-2							
6				SM-SC	Medium dense, moist, brown, Silty to Clayey, fine to coarse SAND; trace of gravel			
8								
10					TRENCH TERMINATED AT 10 FEET No groundwater encountered			

Figure A-5,
Log of Trench T 5, Page 1 of 1

G2818-52-01.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. G2818-52-01

TRENCH T 6

ELEV. (MSL.) 730' DATE COMPLETED 9-22-2021

EQUIPMENT BACKHOE BY: D. THOMAS

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0				SM	**TOPSOIL** Loose, dry, brown, Silty, fine to coarse SAND; with porosity			
2								
	T6-1			SM	**OLDER ALLUVIUM (Qoa)** Medium dense to dense, damp, light brown, Silty, fine to coarse SAND; trace of gravel		101.0	6.5
4								
				SC	Dense, moist, brown, Clayey, fine to coarse SAND; trace of gravel			
6								
8								
					TRENCH TERMINATED AT 9 FEET No groundwater encountered			

Figure A-6,
Log of Trench T 6, Page 1 of 1

G2818-52-01.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. G2818-52-01

TRENCH T 7

ELEV. (MSL.) 729' DATE COMPLETED 9-22-2021

EQUIPMENT BACKHOE BY: D. THOMAS

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0	T7-1			SM	**TOPSOIL** Loose, dry, brown, Silty, fine to coarse SAND			
2				SM	**OLDER ALLUVIUM (Qoa)** Medium dense, moist, light reddish brown, Silty, fine to coarse SAND; trace of gravel			
	T7-2						109.0	5.7
4								
				SC	Medium dense to dense, moist, dark brown, Clayey, fine to medium SAND			
6				SM	Dense to very dense, moist, brown, Silty, fine to coarse SAND; trace of gravel			

TRENCH TERMINATED AT 7 FEET
No groundwater encountered

Figure A-7,
Log of Trench T 7, Page 1 of 1

G2818-52-01.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. G2818-52-01

TRENCH T 8

ELEV. (MSL.) **728'** DATE COMPLETED **9-22-2021**

EQUIPMENT **BACKHOE** BY: **D. THOMAS**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0				SM	**TOPSOIL** Loose to medium dense, dry, light brown, Silty, fine to coarse SAND			
2				SM	**OLDER ALLUVIUM (Qoa)** Medium dense, slightly moist, brown, Silty, fine to coarse SAND; trace of gravel			
4					-Becomes medium dense to dense at 4.5 feet			
6								
				SM-SC	Dense, damp to moist, dark brown, Silty to Clayey, fine to medium SAND			
8					TRENCH TERMINATED AT 8 FEET No groundwater encountered			

Figure A-8,
Log of Trench T 8, Page 1 of 1

G2818-52-01.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. G2818-52-01

TRENCH T 9

ELEV. (MSL.) **726'** DATE COMPLETED **9-22-2021**

EQUIPMENT **BACKHOE** BY: **D. THOMAS**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0				SM	**UNDOCUMENTED FILL (Qudf)** Loose to medium dense, dry, brown, Silty, fine to coarse SAND; trace of construction debris			
2				SM	**OLDER ALLUVIUM (Qoa)** Loose, moist, brown, Silty, fine to coarse SAND; trace of gravel			
4	T9-1						108.6	9.3
6				SC	Medium dense, moist, brown, Clayey, fine to coarse SAND			
8				SM	Medium dense to dense, moist, Silty, fine to coarse SAND			
					TRENCH TERMINATED AT 9 FEET No groundwater encountered			

Figure A-9,
Log of Trench T 9, Page 1 of 1

G2818-52-01.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. G2818-52-01

TRENCH T 10

ELEV. (MSL.) 780' DATE COMPLETED 9-22-2021

EQUIPMENT BACKHOE BY: D. THOMAS

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0				SM	**COLLUVIUM (Qcol)** Loose, dry, light to reddish brown, Silty, fine to coarse SAND; trace of gravel to cobble native clast			
2								
	T10-1				-Becomes medium dense to dense at 3 feet		107.7	5.4
4	T10-2							
6								
					-Increase in coarse sand content at 7 feet			
8					**GRANITIC ROCK (Kgr)** Dense, damp, dark gray, GRANITIC ROCK; highly weathered, weak to moderately weak			
10					TRENCH TERMINATED AT 10 FEET No groundwater encountered			

Figure A-10,
Log of Trench T 10, Page 1 of 1

G2818-52-01.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. G2818-52-01

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	TRENCH T 11 ELEV. (MSL.) 779' DATE COMPLETED 9-22-2021 EQUIPMENT BACKHOE BY: D. THOMAS	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
					MATERIAL DESCRIPTION			
0	T11-1			SM	**COLLUVIUM (Qcol)** Loose, dry, light reddish brown, Silty, fine to coarse SAND; some angular gravel and cobble native clast			
2								
					-Becomes medium dense to dense at 3 feet			
4								
6								
8					**GRANITIC ROCK (Kgr)** Dense to very dense, damp, dark gray, GRANITIC ROCK; highly weathered, weak to moderately weak			

TRENCH TERMINATED AT 9 FEET
No groundwater encountered

Figure A-11,
Log of Trench T 11, Page 1 of 1

G2818-52-01.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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TRENCH T 12

ELEV. (MSL.) **750'** DATE COMPLETED **9-22-2021**

EQUIPMENT **BACKHOE** BY: **D. THOMAS**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0				SM	**OLDER ALLUVIUM (Qoa)** Loose, dry, light reddish brown, Silty, fine to coarse SAND; some gravel and cobble clast			
	T12-1							
2								
					-Becomes medium dense to dense at 3 feet			
4								
6					**GRANITIC ROCK (Kgr)** Very dense, damp, light gray, GRANITIC ROCK; highly weathered, moderately strong to strong			
					PRACTICAL REFUSAL AT 7 FEET No groundwater encountered			

Figure A-12,
Log of Trench T 12, Page 1 of 1

G2818-52-01.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.

PROJECT NO. G2818-52-01

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	TRENCH T 13 ELEV. (MSL.) 749' DATE COMPLETED 9-22-2021 EQUIPMENT BACKHOE BY: D. THOMAS	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
					MATERIAL DESCRIPTION			
0				SM	**OLDER ALLUVIUM (Qoa)** Loose to medium dense, dry, reddish brown, Silty, fine- to coarse-grained SAND			
2					-Becomes medium dense at 2 feet			
4					**GRANITIC ROCK (Kgr)** Very dense, damp, dark gray, GRANITIC ROCK; weathered (upper 6" highly weathered; moderately strong)			
6					PRACTICAL REFUSAL AT 6 FEET No groundwater encountered			

Figure A-13,
Log of Trench T 13, Page 1 of 1

G2818-52-01.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. G2818-52-01

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	TRENCH T 14 ELEV. (MSL.) 752' DATE COMPLETED 9-22-2021 EQUIPMENT BACKHOE BY: D. THOMAS	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
					MATERIAL DESCRIPTION			
0	T14-1			SM	**OLDER ALLUVIUM (Qoa)** Loose to medium dense, dry, light brown, Silty, fine to coarse SAND; with porosity			
2					**GRANITIC ROCK (Kgr)** Very dense, damp, dark gray, GRANITIC ROCK; moderately to highly weathered; moderately strong to strong			
4								
					PRACTICAL REFUSAL AT 5 FEET No groundwater encountered			

Figure A-14,
Log of Trench T 14, Page 1 of 1

G2818-52-01.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. G2818-52-01

TRENCH T 15

ELEV. (MSL.) **741'** DATE COMPLETED **9-22-2021**

EQUIPMENT **BACKHOE** BY: **D. THOMAS**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0				SM	**TOPSOIL** Loose, dry, brown, Silty, fine to medium SAND			
				SM	**OLDER ALLUVIUM (Qoa)** Loose to medium dense, dry, light brown, Silty, fine to coarse SAND; some granitic gravel and cobble clast, some porosity			
2								
4					**GRANITIC ROCK (Kgr)** Dense to very dense, damp, dark gray, GRANITIC ROCK; very highly weathered; weak to moderately strong			
	T15-1							
6								
8					TRENCH TERMINATED AT 8 FEET No groundwater encountered			

Figure A-15,
Log of Trench T 15, Page 1 of 1

G2818-52-01.GPJ

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

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PROJECT NO. G2818-52-01

TRENCH T 16

ELEV. (MSL.) **736'** DATE COMPLETED **9-22-2021**

EQUIPMENT **BACKHOE** BY: **D. THOMAS**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0				SM	**TOPSOIL** Loose, dry, brown, Silty to Clayey, fine to coarse SAND			
2	T16-1			SM	**OLDER ALLUVIUM (Qoa)** Medium dense, dry, light reddish brown, SIlty, fine- to coarse-grained SAND; trace gravel/cobble			
4					-Increase in coarse sand/gravel from 4 to 5 feet			
6				SC	Dense, damp, dark brown, Clayey, fine- to coarse-grained SAND			
8					-Becomes dense to very dense at 8 feet			
					TRENCH TERMINATED AT 8.5 FEET No groundwater encountered			

Figure A-16,
Log of Trench T 16, Page 1 of 1

G2818-52-01.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. G2818-52-01

TRENCH T 17

ELEV. (MSL.) **734'** DATE COMPLETED **9-22-2021**

EQUIPMENT **BACKHOE** BY: **D. THOMAS**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0				SM	**TOPSOIL** Loose, dry, light reddish brown, Silty, fine to coarse SAND			
2	T17-1			ML-CL	**OLDER ALLUVIUM (Qoa)** Stiff to very stiff, damp, reddish brown, Sandy SILT to Sandy CLAY; trace of gravel			
4								
6				SC	Dense, damp, dark brown, Clayey, fine to coarse SAND; trace of gravel			
8					TRENCH TERMINATED AT 8 FEET No groundwater encountered			

Figure A-17,
Log of Trench T 17, Page 1 of 1

G2818-52-01.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. G2818-52-01

TRENCH T 18

ELEV. (MSL.) **730'** DATE COMPLETED **9-22-2021**

EQUIPMENT **BACKHOE** BY: **D. THOMAS**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0				SM-SC	**UNDOCUMENTED FILL (Qudf)** Medium dense, dry to damp, brown, Silty to Clayey SAND; trace of construction debris			
2								
	T18-1			SM	**OLDER ALLUVIUM (Qoa)** Loose, damp, brown, Silty, fine to medium SAND; trace of gravel and coarse sand			
4								
6								
					-Becomes moist and medium dense at 7 feet			
8								
					-Trace of angular granitic cobble at 9 feet			
10								
12								
14					TRENCH TERMINATED AT 14 FEET No groundwater encountered			

Figure A-18,
Log of Trench T 18, Page 1 of 1

G2818-52-01.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 content, R-Value, unconfined compressive strength, consolidation, gradation and direct shear strength characteristics as shown herein. The in-place dry density/moisture content of the samples tested are presented on the logs in Appendix A.

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

Sample No.	Description	Maximum Dry Density (pcf)	Optimum Moisture Content (% dry wt.)
T4-1	Stiff, Moist, Brown, Sandy CLAY to Clayey SAND	121.5	13.0
T17-1	Stiff to Very Stiff, Damp, Reddish Brown, Sandy SILT to Sandy CLAY	124.5	11.2

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				
T4-1	12.3	22.6	103.3	51	Expansive	Medium
T17-1	10.2	20.0	110.3	29	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
T4-1	3-6	Qoa	0.017	S0
T17-1	3-5	Qoa	0.011	S0

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

Sample No.	Description	R-Value
T4-1/T17-1 Mix	Brown, Silty to Clayey, fine to coarse SAND	15

SUMMARY OF LABORATORY UNCONFINED COMPRESSIVE STRENGTH TEST RESULTS
ASTM D 1558

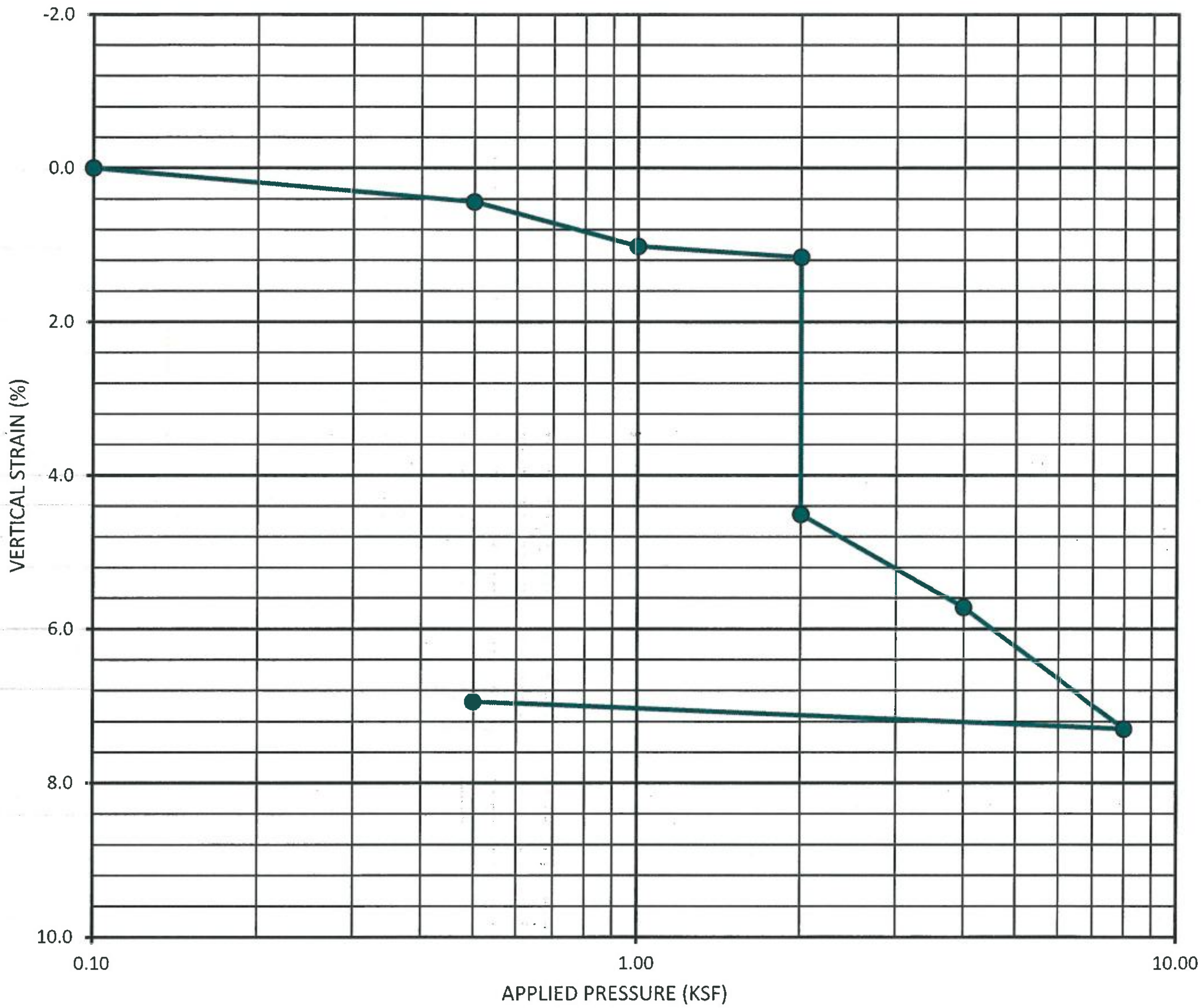
Sample No.	Depth (feet)	Geologic Unit	Hand Penetrometer Reading/Unconfined Compression Strength (tsf) and Undrained Shear Strength (ksf)
T2-1	3.5	Qoa	4.5+
T3-1	4	Qoa	4.5+
T4-2	4	Qoa	4.5+
T6-1	3	Qoa	4.5+
T7-2	3	Qoa	4.5+
T9-1	4	Qoa	3.5
T10-1	3	Qoa	4.5+

SAMPLE NO.: T1-2 @ 4'

SAMPLE DEPTH (FT): 4'

GEOLOGIC UNIT: Qoa

TEST INFORMATION	
INITIAL DRY DENSITY (PCF):	110.7
INITIAL WATER CONTENT (%):	2.9%
SAMPLE SATURATED AT (KSF):	2.0
INITIAL SATURATION (%):	15.3%



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CONSOLIDATION CURVE - ASTM D 2435

NORTHEAST GATEWAY ESCONDIDO

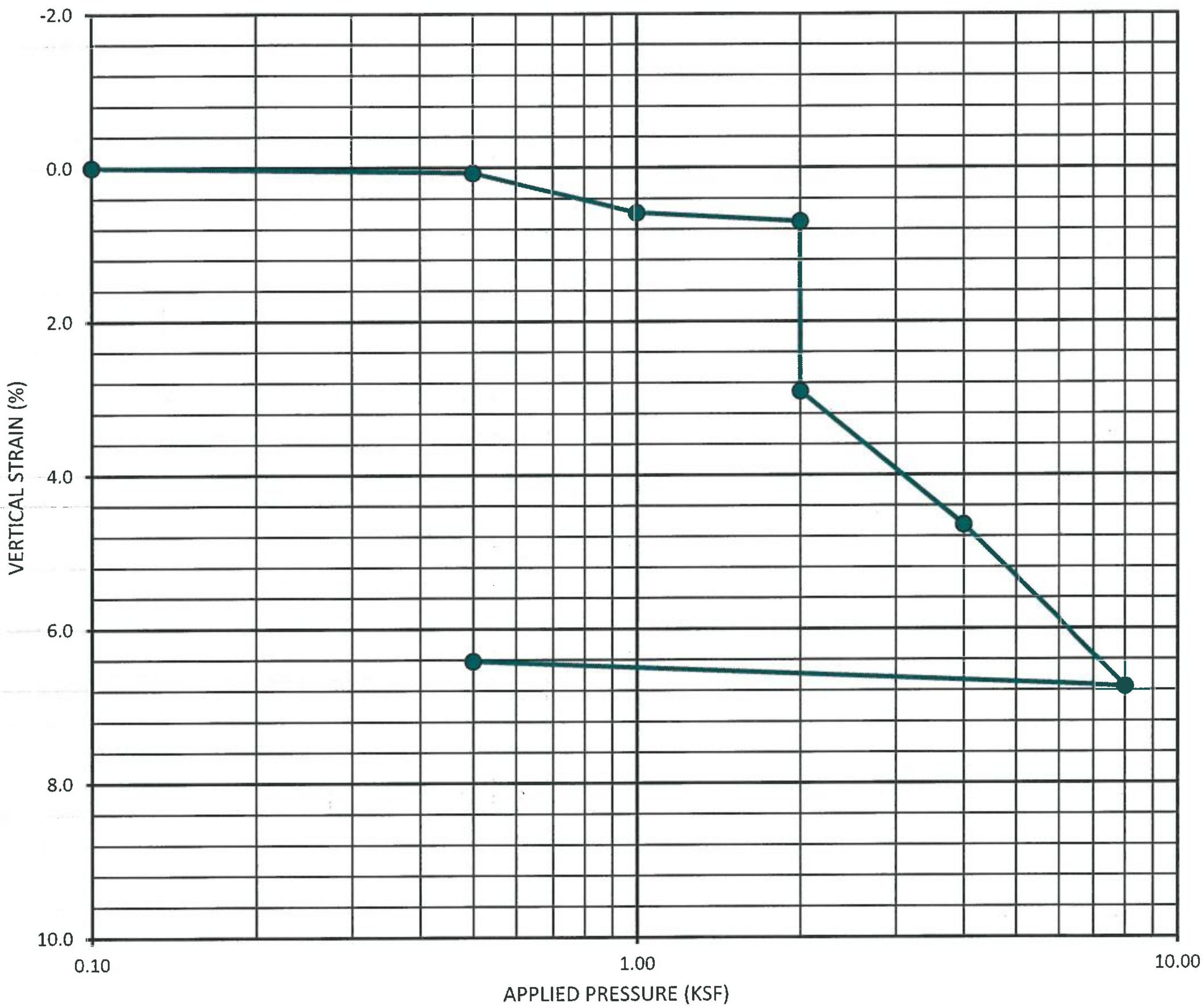
PROJECT NO.: G2818-52-01

SAMPLE NO.: T5-1

SAMPLE DEPTH (FT): 3.5'

GEOLOGIC UNIT: Qoa

TEST INFORMATION	
INITIAL DRY DENSITY (PCF):	108.8
INITIAL WATER CONTENT (%):	6.7%
SAMPLE SATURATED AT (KSF):	2.0
INITIAL SATURATION (%):	33.7%



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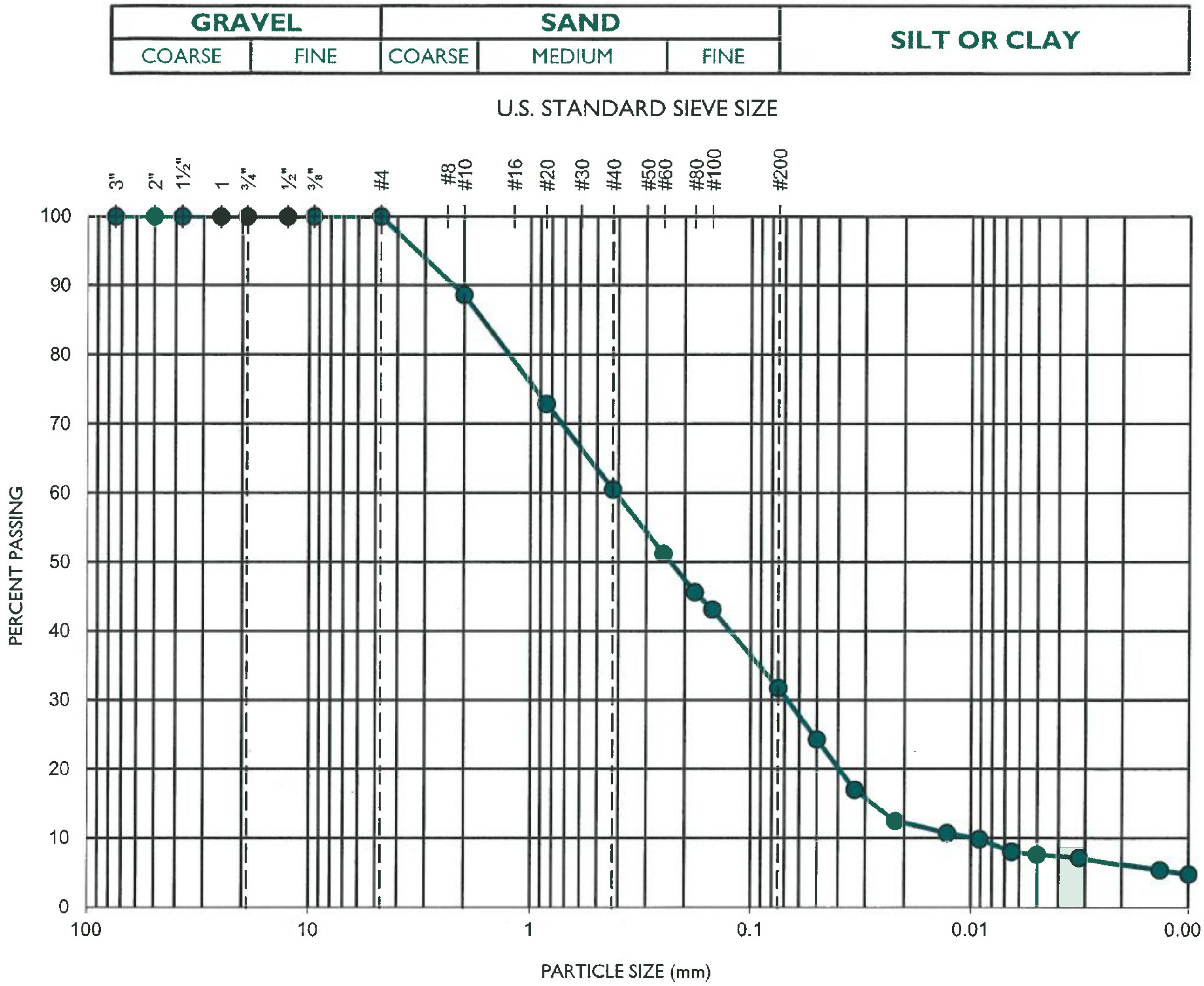
CONSOLIDATION CURVE - ASTM D 2435

NORTHEAST GATEWAY ESCONDIDO

PROJECT NO.: G2818-52-01

SAMPLE NO.:	T1-2	GEOLOGIC UNIT:	Qoa
SAMPLE DEPTH (FT.):	4'		



TEST DATA					
D_{10} (mm)	D_{30} (mm)	D_{60} (mm)	C_c	C_u	SOIL DESCRIPTION
0.00977	0.06921	0.41580	1.2	42.6	Silty SAND

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SIEVE ANALYSES - ASTM D 135 & D 422

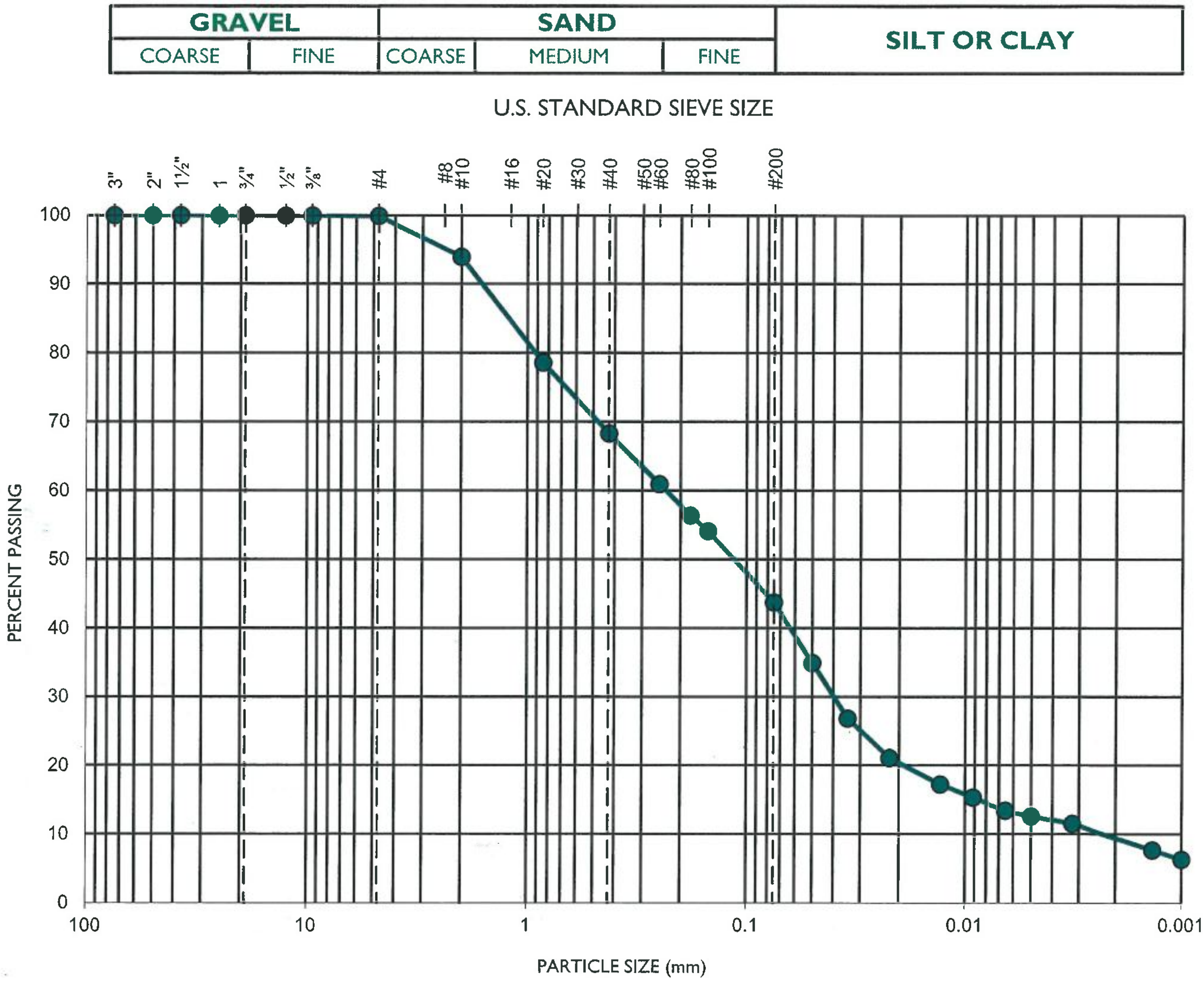
NORTHEAST GATEWAY ESCONDIDO

PROJECT NO.: G2818-52-01

SAMPLE NO.: T10-2

SAMPLE DEPTH (FT.): 3'-6'

GEOLOGIC UNIT: Qcol



TEST DATA					
D_{10} (mm)	D_{30} (mm)	D_{60} (mm)	C_c	C_u	SOIL DESCRIPTION
0.00251	0.04053	0.23578	2.8	94.1	Silty SAND

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SIEVE ANALYSES - ASTM D 135 & D 422

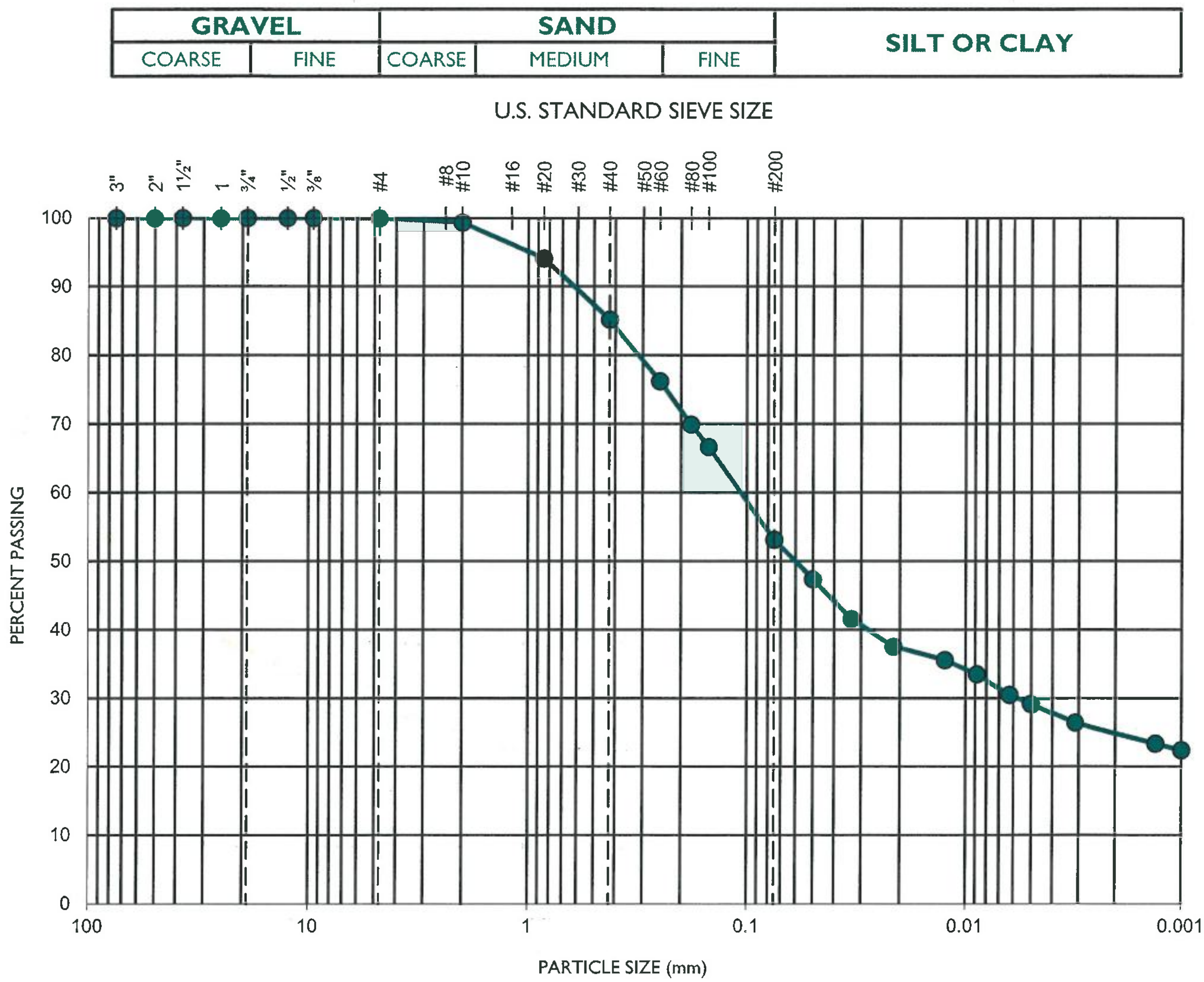
NORTHEAST GATEWAY ESCONDIDO

PROJECT NO.: G2818-52-01

SAMPLE NO.: T17-1

SAMPLE DEPTH (FT.): 2.5'-5'

GEOLOGIC UNIT: Qoa



TEST DATA					
D_{10} (mm)	D_{30} (mm)	D_{60} (mm)	C_c	C_u	SOIL DESCRIPTION
--	0.00585	0.11325	--	--	Sandy CLAY

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SIEVE ANALYSES - ASTM D 135 & D 422

NORTHEAST GATEWAY ESCONDIDO

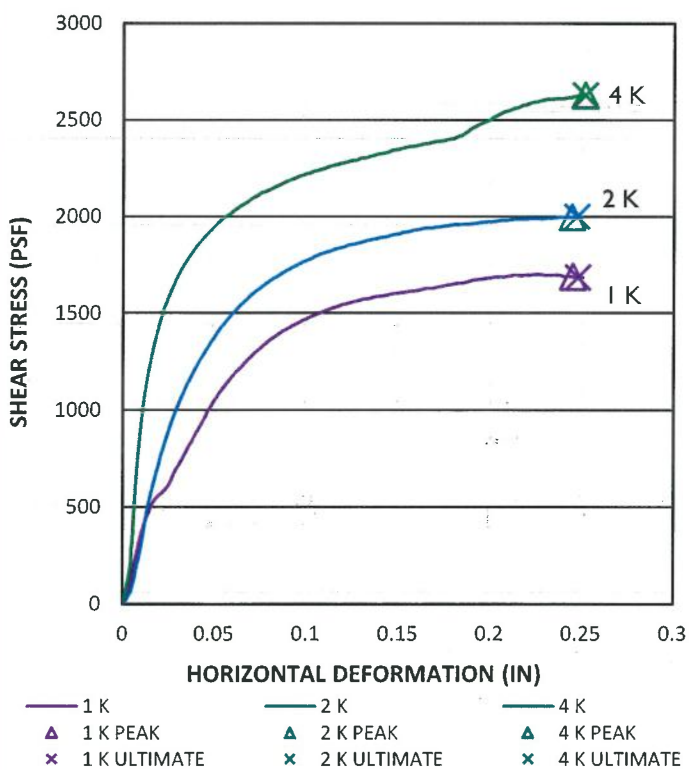
PROJECT NO.: G2818-52-01

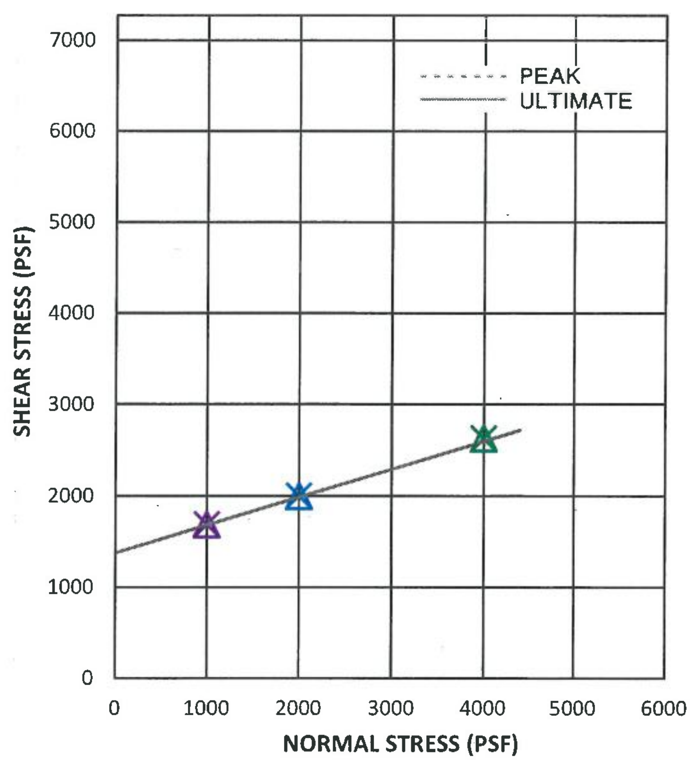
SAMPLE NO.:	T4-1	GEOLOGIC UNIT:	Qoa
SAMPLE DEPTH (FT):	3'-6'	NATURAL/REMOLDED:	R

INITIAL CONDITIONS				
NORMAL STRESS TEST LOAD	1 K	2 K	4 K	AVERAGE
ACTUAL NORMAL STRESS (PSF):	1000	2000	4000	--
WATER CONTENT (%):	13.5	12.6	12.1	12.7
DRY DENSITY (PCF):	109.1	109.7	110.9	109.9

AFTER TEST CONDITIONS				
NORMAL STRESS TEST LOAD	1 K	2 K	4 K	AVERAGE
WATER CONTENT (%):	19.2	20.5	19.8	19.9
PEAK SHEAR STRESS (PSF):	1687	1996	2626	--
ULT.-E.O.T. SHEAR STRESS (PSF):	1684	1999	2626	--

RESULTS		
PEAK	COHESION, C (PSF)	1375
	FRICTION ANGLE (DEGREES)	17
ULTIMATE	COHESION, C (PSF)	1375
	FRICTION ANGLE (DEGREES)	17





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DIRECT SHEAR - ASTM D 3080

NORTHEAST GATEWAY ESCONDIDO

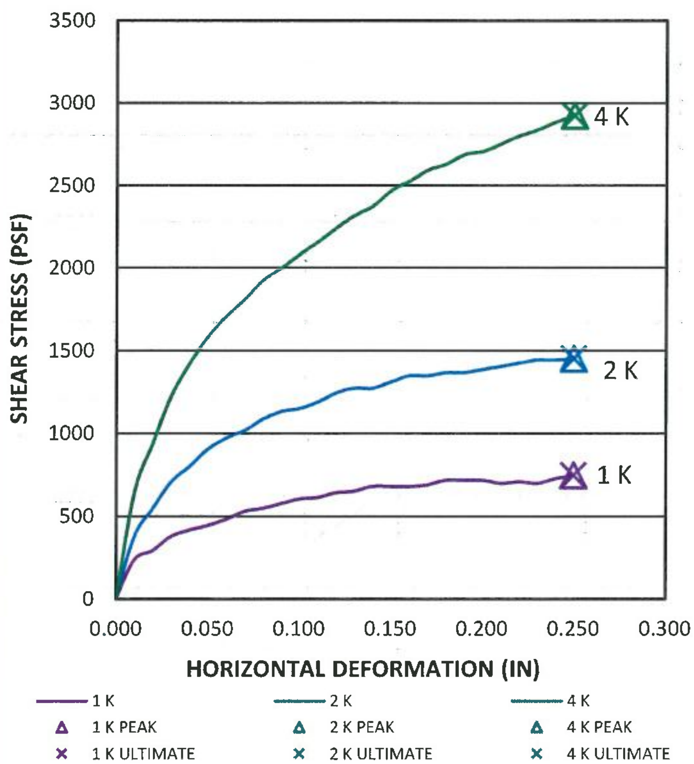
PROJECT NO.: G2818-52-01

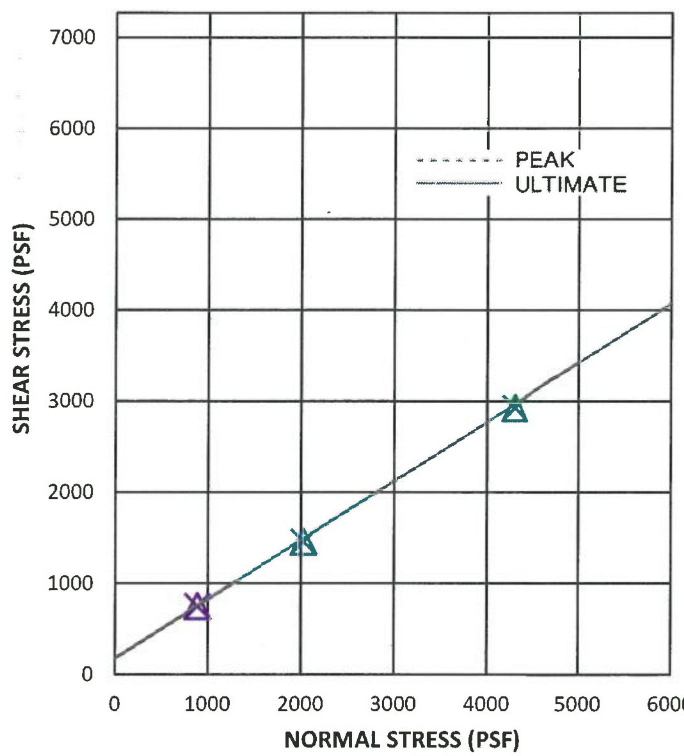
SAMPLE NO.:	T12-1	GEOLOGIC UNIT:	Qoa
SAMPLE DEPTH (FT):	1'-3'	NATURAL/REMOLDED:	R

INITIAL CONDITIONS				
NORMAL STRESS TEST LOAD	1 K	2 K	4 K	AVERAGE
ACTUAL NORMAL STRESS (PSF):	890	2030	4300	--
WATER CONTENT (%):	2.3	2.0	1.5	1.9
DRY DENSITY (PCF):	97.6	100.4	97.9	98.6

AFTER TEST CONDITIONS				
NORMAL STRESS TEST LOAD	1 K	2 K	4 K	AVERAGE
WATER CONTENT (%):	22.3	19.9	18.7	20.3
PEAK SHEAR STRESS (PSF):	745	1452	2922	--
ULT.-E.O.T. SHEAR STRESS (PSF):	745	1452	2922	--

RESULTS		
PEAK	COHESION, C (PSF)	175
	FRICTION ANGLE (DEGREES)	33
ULTIMATE	COHESION, C (PSF)	175
	FRICTION ANGLE (DEGREES)	33





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DIRECT SHEAR - ASTM D 3080

NORTHEAST GATEWAY ESCONDIDO

PROJECT NO.: G2818-52-01

APPENDIX C

APPENDIX C

RECOMMENDED GRADING SPECIFICATIONS

FOR

NORTHEAST GATEWAY ESCONDIDO
NORTHEAST CORNER OF BEVEN DRIVE AND EASTVALLEY PARKWAY, ESCONDIDO CALIFORNIA

PROJECT NO. G2818-52-01

RECOMMENDED GRADING SPECIFICATIONS

1. GENERAL

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

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

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

2. DEFINITIONS

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

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

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

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

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

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

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

3. MATERIALS

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

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

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

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

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

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

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

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

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

4. CLEARING AND PREPARING AREAS TO BE FILLED

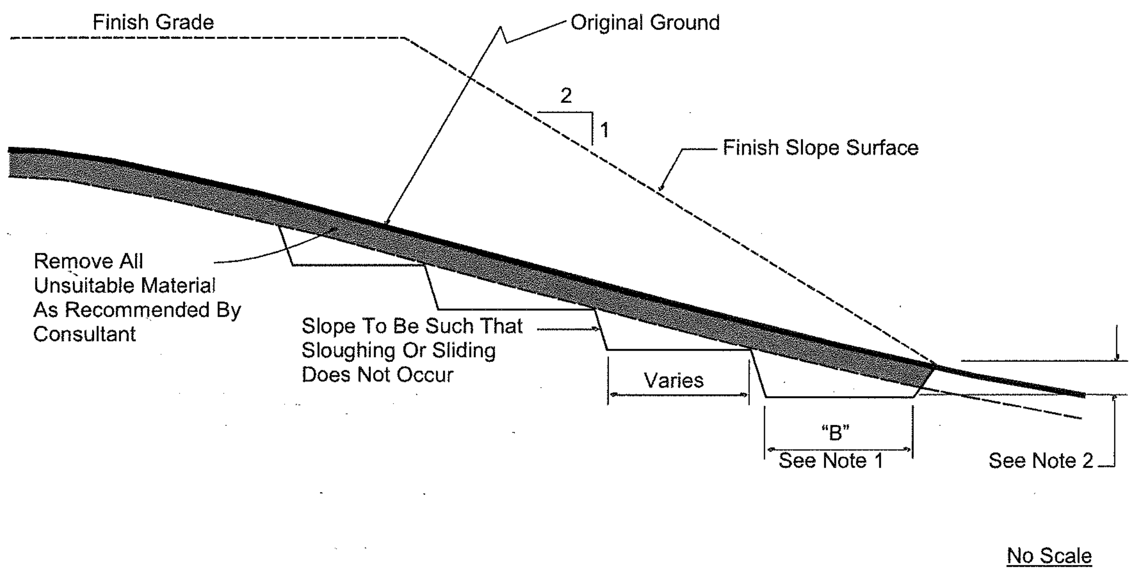
4.1 Areas to be excavated and filled shall be cleared and grubbed. Clearing shall consist of complete removal above the ground surface of trees, stumps, brush, vegetation, man-made structures, and similar debris. Grubbing shall consist of removal of stumps, roots, buried logs and other unsuitable material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

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

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

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

TYPICAL BENCHING DETAIL



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

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

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

5. COMPACTION EQUIPMENT

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

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

6. PLACING, SPREADING AND COMPACTION OF FILL MATERIAL

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

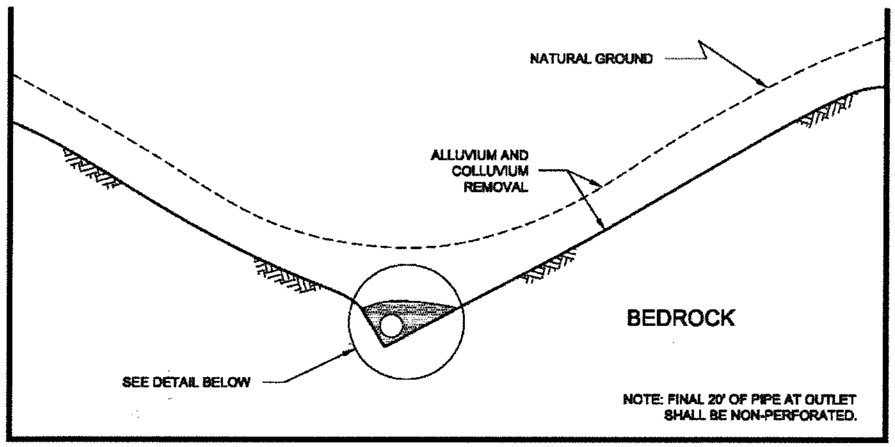
6.3.6 To reduce the potential for "piping" of fines into the *rock* fill from overlying *soil* fill material, a 2-foot layer of graded filter material shall be placed above the uppermost lift of *rock* fill. The need to place graded filter material below the *rock* should be determined by the Consultant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

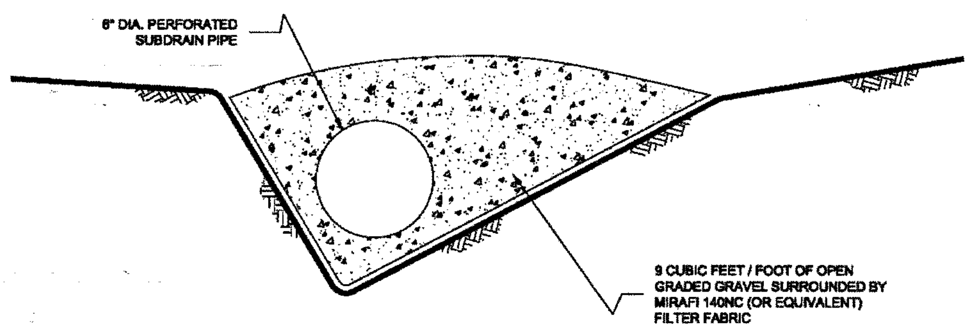
6.3.7 *Rock* fill placement should be continuously observed during placement by the Consultant.

7. SUBDRAINS

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

TYPICAL CANYON DRAIN DETAIL





NOTES:

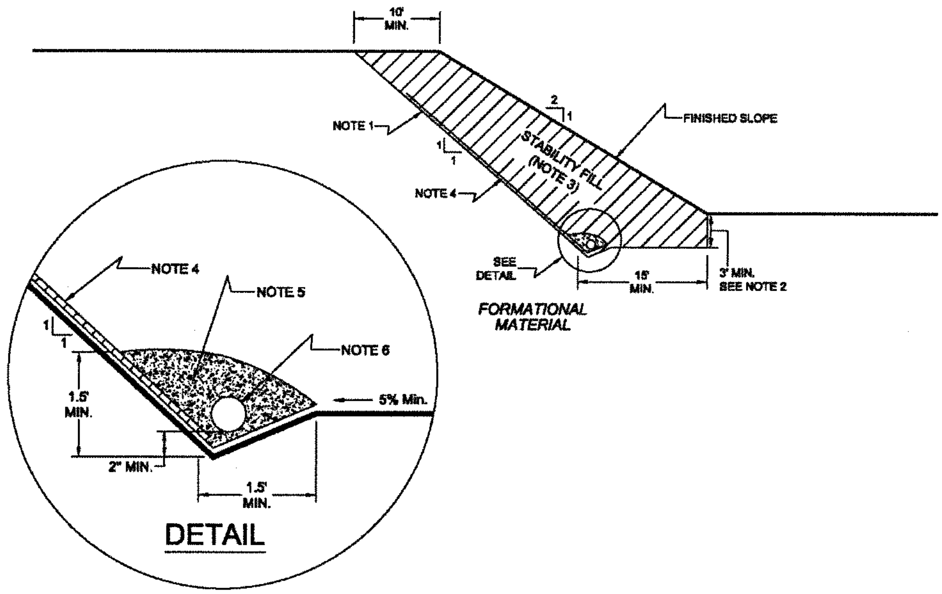
1......8-INCH DIAMETER, SCHEDULE 80 PVC PERFORATED PIPE FOR FILLS IN EXCESS OF 100-FEET IN DEPTH OR A PIPE LENGTH OF LONGER THAN 500 FEET.

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

NO SCALE

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

TYPICAL STABILITY FILL DETAIL



NOTES:

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

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

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

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

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

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

NO SCALE

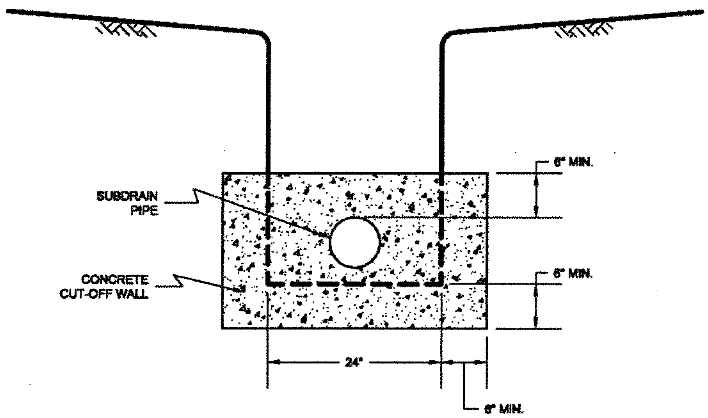
7.3 The actual subdrain locations will be evaluated in the field during the remedial grading operations. Additional drains may be necessary depending on the conditions observed and the requirements of the local regulatory agencies. Appropriate subdrain outlets should be evaluated prior to finalizing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

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

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

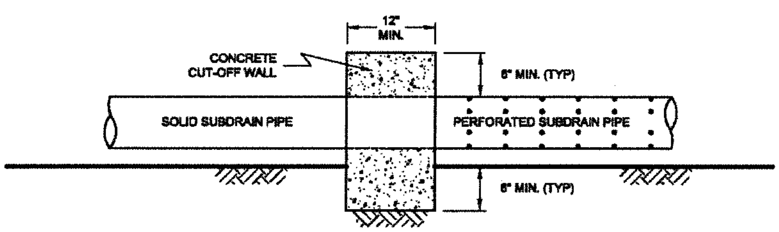
TYPICAL CUT OFF WALL DETAIL

FRONT VIEW



NO SCALE

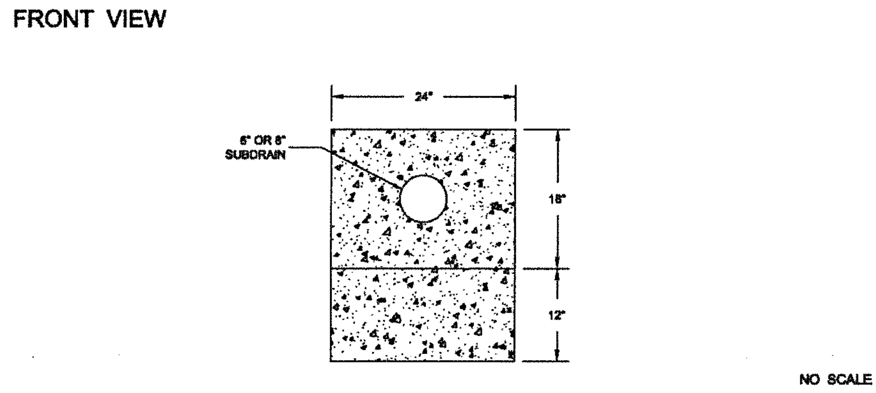
SIDE VIEW

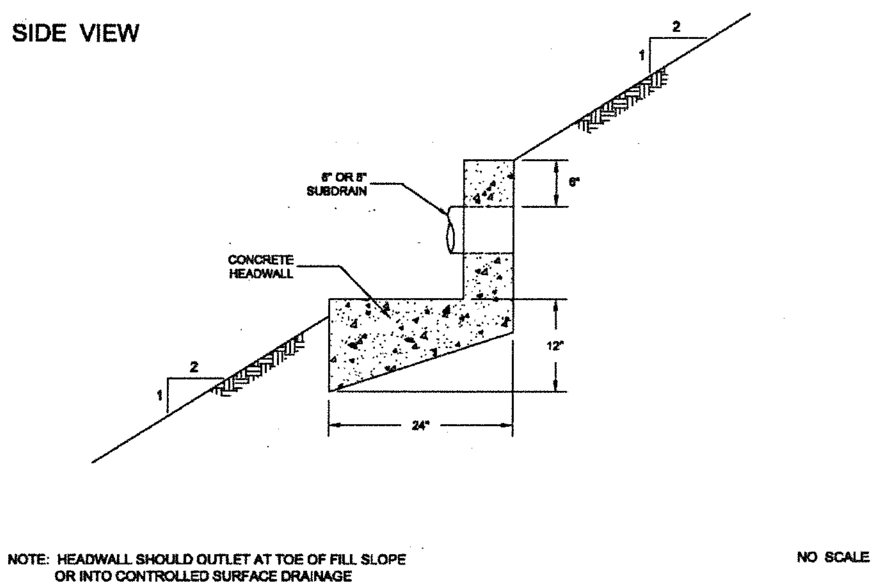


NO SCALE

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

4. American Society of Civil Engineers (ASCE), *ASCE 7-16, 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. Geocon Incorporated, *Final Report of Testing and observation Services Performed During Site Grading , Eureka Ranch, Unit 4, Tract NO. 839, Lots 1 through 73, Escondido California,* dated June 10, 2008 (Project No. 07436-52-02).

8. Geocon Incorporated, *Final Report of Testing and observation Services Performed During Site Grading , Eureka Ranch, Unit 5, Tract NO. 839, Lots 1 through 64, Escondido California,* dated August 22, 2006 (Project No. 07436-52-02).

9. Geocon Incorporated, *Updated Geotechnical Investigation, Eureka Ranch, Escondido California*, dated May 20, 2005 (Project No. 07436-52-01).

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

11. Hunsaker and Associates, San Diego Inc, *Base Map for Northeast Gateway Escondido, California, Sheet 1, Project number WO #9999-2881,* undated.

12. 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.

13. Kennedy, M. P., and S. S. Tan, 2008, *Geologic Map of the Oceanside 30'x60' Quadrangle, California*, USGS Regional Map Series Map No. 2, Scale 1:100,000.

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

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

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