GEOLOGIC RECONNAISSANCE

MISSION TRAILS REGIONAL PARK
SAN DIEGO, CALIFORNIA

GEOCON
INCORPORATED

GEOTECHNICAL
ENVIRONMENTAL
MATERIALS

PREPARED FOR

KTU+A
SAN DIEGO, CALIFORNIA

APRIL 9, 2014
PROJECT NO. G1330-52-01

Project No. G1330-52-01
April 9, 2014

KTU+A
3916 Normal Street
San Diego, California 92103

Attention: Mr. Mark Carpenter

Subject: GEOLOGIC RECONNAISSANCE
MISSION TRAILS REGIONAL PARK
SAN DIEGO, CALIFORNIA

Dear Mr. Carpenter:

In accordance with your request, we have prepared this soil and geologic reconnaissance for the proposed addition of off-street parking lots at four locations within Mission Trails Regional Park (MTRP) in San Diego County, California. The accompanying report describes the site soil and geologic conditions, discusses potential geotechnical constraints, and provides preliminary recommendations to assist in planning and development studies. The site is considered suitable for the planned improvements provided the recommendations of this report are followed.

Should you have any 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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GEOLOGIC RECONNAISSANCE

1. PURPOSE AND SCOPE

This report presents the results of a geologic reconnaissance for the proposed addition of four off-street parking areas within the Mission Trails Regional Park (MTRP). The proposed four off-street parking areas are located within the City of San Diego and as well as a portion the City of Santee, California (see Vicinity Map, Figure 1). Specifically, the proposed off-street parking areas are situated in the Mission Gorge – Oak Grove and the Cowles Mountain sections of the park, respectively. The purpose of this study is to provide preliminary soil and geologic information for the subject project, identify known geologic hazards that may adversely impact the proposed project, and provide preliminary recommendations for construction. A future geotechnical investigation should be performed to provide final engineering recommendations that would include performing fieldwork, laboratory testing, and engineering analyses prior to submittal of construction documents.

The scope of this geologic reconnaissance included a review of readily available published and unpublished geologic literature, performing a site visit, preliminary engineering analyses, and preparing this report. Additionally, we reviewed previous geotechnical investigations prepared by Geocon Incorporated for the Mission Trails Regional Park Visitors Center, (see List of References). Our area of study is based on the *Public Draft, Master Plan Update for Mission Trails Regional Park*, prepared by the City of San Diego Park and Recreation Department.

We prepared the Geologic Maps (Figures 2 through 5), based on the project plans prepared by KTU+A. The map depicts the proposed parking stalls, existing topography and mapped geologic contacts based on our reconnaissance. The conclusions and preliminary recommendations presented herein are based on an analysis of the data reviewed as part of this study and our experience with similar soil and geologic conditions.

2. SITE AND PROJECT DESCRIPTION

We understand the proposed four off-street parking areas are located within two sections of MTRP identified as the Mission Gorge – Oak Grove, and Cowles Mountain sections of the park (See Vicinity Map, Figure 1). We understand one proposed off-street parking area is within the Mission Gorge – Oak Grove section, and three proposed off-street parking areas within the Cowles Mountain section of Mission Trails Regional Park.

The proposed Mission Gorge – Oak Grove parking area is along the paved portion of Father Junipero Serra Trail approximately 150 feet north of Mission Gorge Road and south of the Visitor and Interpretive Center (see Figure 2). The Mission Gorge – Oak Grove site currently consists of natural vegetation with relatively flat topography. Existing parking for the hiking trail is present along Father

Junipero Serra Trail trailhead entrance. Two picnic tables are accessed along a hiking trail along the western margin. Based on the plans, a new parking lot will be installed in the area of the existing picnic tables, east of the access roadway to the visitor's center.

One of the proposed Cowles Mountain parking areas is located approximately 600 feet northeast of the intersection of Goldcrest Drive and Mission Gorge Road (see Figure 3). This site currently consists of a narrow access road for the San Diego County Water Authority (SDWCA) pipeline and a concrete vault. Parking for hiking is not currently available in this area. The planned improvements consist of providing a relatively small parking area within the area of the existing access road.

The second proposed Cowles Mountain parking area is located approximately 250 northwest of the intersection of Boulder Lake Avenue and Barker Way within the open space at the Barker Street trailhead (see Figure 4). The Barker Way site currently consists of native vegetation and gently sloping topography. A shallow concrete-lined brow ditch and drainage channel exist adjacent to Barker Way. Parallel parking is present along the relatively narrow Barker Way roadway. Parking and a cul-de-sac is planned west of Barker Way and north of existing residences. The access to the parking area would be from Barker Way. We understand a comfort station/restroom is also proposed at the Barker Way parking area

The third proposed parking located within the Cowles Mountain portion of the park is situated behind the existing curb along the shoulder of Mesa Road at the intersection of Mesa Heights Court (see Figure 5). This location is currently non-vegetated and appears utilized as a surface drainage swale, utility easement, and hiking route to the Big Rock Trailhead. A small, gravel parking area is located at the southern portion of the shoulder. The preliminary plans indicate the shoulder area and existing curb would be removed to construct diagonal parking along the western portion of Mesa Road.

We understand that proposed improvements would consist of constructing off-street parking consisting of gravel or decomposed granite surfaced parking areas. Based on review of the referenced draft master plan report and the plans you provided, we understand a comfort station/restroom is proposed at the Barker Way parking area. Based on conversations with KTU+A, we understand the off-street parking areas will generally mirror the existing topography to create a relatively flat site gently contoured to drain the proposed sites. We expect minor grading would be necessary to construct the offsite parking which would consist of minor cuts and fills on the order of 4 to 6 feet, respectively. A geotechnical investigation can be performed to provide final engineering and grading recommendations.

The locations and descriptions of the site and proposed improvements are based on a site reconnaissance and a review of the referenced tentative map. If development plans differ significantly from those described herein, Geocon Incorporated should be contacted for review of the final plans especially with regard to changes in finish grades of the parking areas.

3. GEOLOGIC SETTING

The site is located in the coastal plain of the Peninsular Ranges 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. Crystalline basement rocks exist along the western side of the Peninsular Ranges and are dominated by pre-batholithic andesitic Metavolcanic Rock previously known as the Santiago Peak Volcanics with a late Jurassic and early Cretaceous age. The Metavolcanic Rock was intruded during the early to mid-Cretaceous by a variety of granitic to gabbroic plutons of the Southern California batholith. The coastal plain of San Diego County is underlain by a thick sequence of relatively undisturbed and non-conformable sedimentary rocks that range in age from Upper Cretaceous through the Pleistocene with intermittent deposition. Geomorphically, the coastal plain is characterized by a stair-stepped series of marine terraces, which are younger to the west and have been dissected by west flowing rivers that drain the Peninsular Ranges to the east. The coastal plain is a relatively stable block that is dissected by relatively few faults consisting of the potentially active La Nacion Fault Zone and the active Rose Canyon Fault Zone. The Peninsular Ranges are 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 on the central portion of the coastal plain. Mesozoic-age plutonic Metavolcanic Rock and intrusive Cretaceous-age Granitc Rock make up a majority of the geologic units expected at the site. Marine sedimentary units make up a small portion of the geologic units encountered on the site, which overlie the crystalline basement rock.

4. SOIL AND GEOLOGIC CONDITIONS

Based on our review of readily available published and unpublished geologic literature, our experience in the area, we expect three surficial soil types and three geologic formations underlie the proposed sites. The approximate limits of the geologic units are shown on the Geologic Map, Figures 2 through 5. The surficial soil and geologic formations are discussed herein in order of increasing age.

4.1 Undocumented Fill (Qudf)

Based on our site reconnaissance, we expect undocumented fill is localized to the proposed off-street parking area along Mission Gorge Road as shown on Figure 3. We expect the maximum thickness of the undocumented fill to be approximately 10 to 12 feet at the SDCWA location. We expect compacted fill was placed during the construction of the Mission Gorge Road and the SDWCA easement. We also expect undocumented fill to be exposed southeast of the Barker Street parking area within the existing residential subdivision and City of San Diego roadways (see Figure 4). Additionally, a relatively thin layer of compacted fill was likely placed along the shoulder of Mesa Road associated with the installation of underground utilities and is exposed at grade; however, we did not map the fill material because we did not perform field investigation. We expect the undocumented fill can be utilized for re-use for compacted fill and minor remedial grading may be required. We expect the fill materials consist of clayey to silty sand with gravel and cobble likely associated with excavations from the adjacent geologic units.

4.2 Topsoil (unmapped)

We expect a thin veneer of topsoil is exposed at the ground surface above the alluvium and formational materials. The topsoil is likely composed of clayey sand and silty to sandy clay with abundant gravel and cobble at the Barker Street, and the Mission Gorge Oak Grove sites. The topsoil could possess a “medium” to “high” expansion potential (expansion index of 51 to 130) and may require removal in areas that will receive additional fill and/or settlement-sensitive improvements.

4.3 Alluvium (Qal)

Based on our field observations, we expect alluvium may be present with the SDWCA easement located north of the proposed parking location (See Figure 3). We also expect alluvial soil with a maximum depth of approximately 4 to 6 feet below the existing exists within the drainage channel at the Barker Street location (see Figure 4). Alluvium is mapped to the west of the Mesa Road location as shown on Figure 5. These deposits typically consist of loose to medium dense, silty, fine to coarse sand with abundant gravel and cobble. We expect the alluvium possesses a relatively shallow thickness of 2 to 5 feet. The alluvium is subject to consolidation settlement and is likely not suitable for the support of structural fill and settlement-sensitive improvements. Saturated soil may be encountered within the alluvium that would require moisture conditioning/drying or mixing with dry soil if to be used as compacted fill.

4.4 Friars Formation (Tf)

Based on a review of Kennedy and Tan (2008), we expect Eocene-age Friars Formation to be encountered along the northern portion of the Mesa Road location (see Figure 5). We expect the Friars Formation to be relatively flat-lying, light yellowish brown to light gray, fine to coarse sandstone and interlayers of siltstone and claystone. The sandstone portion of the Friars Formation typically possesses a “very low” to “low” expansion potential (expansion index of 50 or less). However, the siltstone and claystone portion of the Friars Formation can possess a “medium” to “very high” expansion potential (expansion index greater than 50 to greater than 130) and relatively low shear strength. The Friars Formation is known to exhibit highly cemented zones that may result in excavation difficulty during construction of site improvements (i.e. underground utility lines). Although blasting is not likely, moderate to heavy ripping should be expected in portions of this formation to facilitate excavation. Generation of oversize materials requiring special handling and placement techniques should also be expected. In general, the Friars Formation is suitable for the support of compacted fill and structural loads.

4.5 Mission Valley Formation (Tmv)

Based on a review of Kennedy and Tan (2008), we expect Eocene-age Mission Valley Formation to be present east of the Barker Road location (see Figure 4). We expect the Mission Valley Formation to be dense to very dense, light brown to light gray sandstone with interbedded siltstones and claystones. The Mission Valley Formation often exhibits highly cemented zones, which may result in excavation difficulty during construction of site improvements. Moderate to heavy ripping and possible coring should be expected if these zones are encountered in portions of this formation to facilitate excavation. Generation of oversize materials requiring special handling and possible exportation should also be expected. In general, the Mission Valley Formation is suitable for the support of compacted fill and structural loads.

4.6 Granitic Rock (Kgu, Kt)

Early Cretaceous-age granitic rock associated with the Peninsular Range Batholith is exposed at the ground surface and on outcrops. We expect Granitic Rock to underlie the proposed Mission Gorge Road site, Barker Way site, and the Mesa Road site. Based on the review of Kennedy and Tan (2008) the granitic rock in this area is described as “Granodiorite and Tonalite, undivided” (Kgu) and “Tonalite” (Kt). We expect the granitic rock is at various stages of weathering and possesses a medium- to coarse-grained phaneritic texture with corestones interspersed within the formational unit. Granitic rock generally excavates to silty, fine- to coarse-grained sand with rock fragments and typically exhibits “very low” to “low” expansion potential (expansion index of 50 or less) with a relatively high shear strength. Although blasting is not likely for the planned improvements, moderate to heavy ripping should be expected in portions of this formation to facilitate excavation. Generation of oversize materials and “floaters” requiring special handling (including using rock breaking equipment) and placement techniques should also be expected. In general, the Granitic Rock is suitable for the support of compacted fill and structural loads.

4.7 Metavolcanic Rock (Mzu)

Based on review of Kennedy and Tan (2008), we expect the Mesozoic-age low- to high-metamorphic grade Metavolcanic Rock underlies the Mission Trails Oak Grove site (see Figure 2). Generally, the rock consists of moderately weak to strong, highly weathered to fresh, grayish brown Metavolcanic Rock. Although blasting is likely not planned, moderate to heavy ripping should be expected in portions of this formation to facilitate excavation. Generation of oversize materials requiring special handling (including using rock breaking equipment) and placement techniques should also be expected. In general, the Metavolcanic Rock is suitable for the support of compacted fill and structural loads.

5. GROUNDWATER

We did not encounter seepage or springs during our site reconnaissance. We do not expect groundwater to impact site development. It is not uncommon for groundwater or seepage conditions to develop where none previously existed. Groundwater elevations are dependent on seasonal precipitation, irrigation, and land use, among other factors, and vary as a result. Proper surface drainage of irrigation and rainwater will be important to future performance of the project. Depending upon seasonal conditions at the time of grading, specialized equipment to excavate the surficial soils and drying or mixing with other on-site materials to reduce the moisture content prior to placement, as compacted fill may be required.

6. GEOLOGIC HAZARDS

6.1 Geologic Hazard Category

The City of San Diego Seismic Safety Study, Geologic Hazards and Faults, Map Sheets 27, 28, and 32 defines the site with a Hazard Category 53: *Level or sloping terrain, unfavorable geologic structure, low to moderate risk.*

6.2 Faulting and Seismicity

A review of geologic literature and experience with the soil and geologic conditions in the general area indicate that known active, potentially active, or inactive faults are not located at the site. An active fault is defined by the California Geological Survey (CGS) as a fault showing evidence for activity within the last 11,000 years. The site is not located within a State of California Earthquake Fault Zone.

According to the computer program *EZ-FRISK* (Version 7.62), six known active faults are located within a search radius of 50 miles from the western most parking area at the Mission Gorge – Oak Grove section of the park. We used the 2008 USGS fault database that provides several models and combinations of fault data to evaluate the fault information. The nearest known active faults are the Newport-Inglewood/Rose Canyon Fault system, located approximately 9 miles west of the proposed sites and is the dominant source of potential ground motion. Earthquakes that might occur on the Rose Canyon Fault Zone or other faults within the southern California and northern Baja California area are potential generators of significant ground motion at the site. The estimated deterministic maximum earthquake magnitude and peak ground acceleration for the Newport-Inglewood Fault are 7.5 and 0.27g, respectively. The estimated deterministic maximum earthquake magnitude and peak ground acceleration for the Rose Canyon Fault are 6.9 and 0.21g, respectively. Table 6.2.1 lists the estimated maximum earthquake magnitude and peak ground acceleration for these and other faults in relationship to the site location. We used acceleration attenuation relationships developed by Boore-Atkinson (2008) NGA USGS2008, Campbell-Bozorgnia (2008) NGA USGS, and Chiou-Youngs (2007) NGA USGS2008 acceleration-attenuation relationships in our analysis.

TABLE 6.2.1
DETERMINISTIC SITE PARAMETERS

Fault Name	**Distance from Site (miles)**	**Maximum Earthquake Magnitude (Mw)**	**Peak Ground Acceleration**		
			Boore-Atkinson 2008 (g)	**Campbell-Bozorgnia 2008 (g)**	**Chiou-Youngs 2008 (g)**
Newport-Inglewood	9	7.5	0.25	0.22	0.27
Rose Canyon	9	6.9	0.21	0.20	0.21
Coronado Bank	22	7.4	0.15	0.11	0.13
Palos Verdes Connected	22	7.7	0.17	0.12	0.15
Elsinore	33	7.9	0.13	0.09	0.12
Earthquake Valley	37	6.8	0.07	0.06	0.05

It is our opinion the site could be subjected to moderate to severe ground shaking in the event of an earthquake along any of the faults listed in Table 6.2.1 or other faults in the southern California/ northern Baja California region. We do not consider the site to possess a greater risk than that of the surrounding developments.

We used the computer program *EZ-FRISK* to perform a probabilistic seismic hazard analysis. The computer program *EZ-FRISK* operates under the assumption that the occurrence rate of earthquakes on each mapped Quaternary fault is proportional to the faults slip rate. The program accounts for earthquake magnitude as a function of fault rupture length, and site acceleration estimates are made using the earthquake magnitude and distance from the site to the rupture zone. The program also accounts for uncertainty in each of following: (1) earthquake magnitude, (2) rupture length for a given magnitude, (3) location of the rupture zone, (4) maximum possible magnitude of a given earthquake, and (5) acceleration at the site from a given earthquake along each fault. By calculating the expected accelerations from considered earthquake sources, the program calculates the total average annual expected number of occurrences of site acceleration greater than a specified value. We utilized acceleration-attenuation relationships suggested by Boore-Atkinson (2008) NGA USGS 2008, Campbell-Bozorgnia (2008) NGA USGS 2008, and Chiou-Youngs (2008) in the analysis. Table 6.2.2 presents the site-specific probabilistic seismic hazard parameters including acceleration-attenuation relationships and the probability of exceedence.

TABLE 6.2.2
PROBABILISTIC SEISMIC HAZARD PARAMETERS

Probability of Exceedence	Peak Ground Acceleration		
	Boore-Atkinson 2008 (g)	Campbell-Bozorgnia 2008 (g)	Chiou-Youngs 2008 (g)
2% in a 50 Year Period	0.35	0.34	0.40
5% in a 50 Year Period	0.26	0.25	0.27
10% in a 50 Year Period	0.19	0.19	0.19

The California Geologic Survey (CGS) has a program that calculates the ground motion for a 10 percent of probability of exceedence in 50 years based on an average of several attenuation relationships. Table 6.2.3 presents the calculated results from the *Probabilistic Seismic Hazards Mapping Ground Motion* Page from the CGS website.

TABLE 6.2.3
PROBABILISTIC SITE PARAMETERS FOR SELECTED FAULTS
CALIFORNIA GEOLOGIC SURVEY

Calculated Acceleration (g) Firm Rock	Calculated Acceleration (g) Soft Rock	Calculated Acceleration (g) Alluvium
0.23	0.25	0.29

While listing peak accelerations is useful for comparison of potential effects of fault activity in a region, other considerations are important in seismic design, including the frequency and duration of motion and the soil conditions underlying the site. Seismic design of the structures should be evaluated in accordance with the 2013 California Building Code (CBC) guidelines or guidelines currently adopted by the City of San Diego.

6.3 Ground Rupture

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

6.4 Seiches and Tsunamis

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.

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 first-order driving force for locally generated tsunamis offshore southern California is expected to be tectonic deformation from large earthquakes The westernmost area of the property site is located about 15 miles from the Pacific Ocean at a lowest minimum elevation of approximately 355 feet above MSL. Therefore, the risk of tsunamis affecting the site is negligible.

6.5 Liquefaction Potential

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. The potential for liquefaction at the site is considered low due to the presence of shallow dense formational materials and the lack of permanent, near-surface groundwater.

6.6 Landslides

Examination of aerial photographs in our files, review of published geologic maps for the site vicinity, and the relatively level topography, it is our opinion that landslides are not present at the property or at a location that could impact the subject site.

7. CONCLUSIONS AND RECOMMENDATIONS

7.1 General

7.1.1 From a geotechnical engineering standpoint, it is our opinion that the site is suitable for the proposed improvements, provided the recommendations of this report and future geotechnical investigations are followed.

7.1.2 The site is expected to be underlain by undocumented fill, topsoil, and alluvium overlying Granitic and Metamorphic Rock. We expect the alluvium within the drainage channel to extend to a maximum depth of approximately 4 to 6 feet below the existing grade. We expect the maximum thickness of the undocumented fill to be approximately 10 to 12 feet.

7.1.3 We expect the planned off-street parking areas will be supported on compacted fill placed during the future grading operations.

7.1.4 Excavation of the surficial soil should generally be possible with moderate effort using conventional, heavy-duty equipment during grading and trenching operations. Moderate to very heavy effort should be expected for the weathered metamorphic and/or granitic rocks. Blasting, rock breaking or rock coring may be required where excavations are to extended into the less slightly weathered, metamorphic and/or granitic rocks. Additional geotechnical studies should evaluate the rippability of the geologic materials, if required.

7.1.5 The site is located approximately 9 miles from the nearest active fault, the Newport-Inglewood/Rose Canyon Fault system. Based on our background research, active, potentially active, or inactive faults do not extend across or trend toward the site. Risks associated with seismic activity at this site generally consist of the potential for strong seismic shaking. The site is not mapped in a High Liquefaction Hazard Zone as defined by the City of San Diego (2008).

7.1.6 We expect groundwater is relatively deep; however, surface water may flow subsequent to rain events within the drainage channel adjacent to Mission Gorge Road (see Figures 3 and 4). Groundwater could have an influence on construction operations depending on the volume of perched water, utility invert elevations, and excavation depths. Stabilization and/or dewatering may be necessary for excavations where seepage is encountered.

7.2 Soil and Excavation Characteristics

7.2.1 Excavation of the in-situ surficial soil should be possible with moderate effort using conventional heavy-duty equipment. Excavation of the formational materials will require moderate to 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 with the granitic rock materials that can be incorporated into landscape use or deep compacted fill areas, if available. Blasting or rock breaking equipment may be required if excavations into relatively fresh and strong rock are planned.

7.2.2 We expect the existing soil can be considered to be "non-expansive" and "expansive" (expansion index of 20 or less and greater than 20, respectively) as defined by 2013 California Building Code (CBC) Section 1803.5.3. Table 7.2 presents soil classifications based on the expansion index. We expect a majority of the soil encountered possess a "very low" to "medium" expansion potential (expansion index of 90 or less).

TABLE 7.2
SOIL CLASSIFICATION BASED ON EXPANSION INDEX

Expansion Index (EI)	Expansion Classification	2013 CBC Expansion Classification
0 – 20	Very Low	Non-Expansive
21 – 50	Low	Expansive
51 – 90	Medium	
91 – 130	High	
Greater Than 130	Very High	

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

7.3 Seismic Design Criteria

7.3.1 We used the computer program *U.S. Seismic Design Maps*, provided by the USGS to evaluate the seismic design criteria. Table 7.3.1 summarizes site-specific design criteria obtained from the 2013 California Building Code (CBC; Based on the 2012 International Building Code [IBC] and ASCE 7-10), Chapter 16 Structural Design, Section 1613 Earthquake Loads. The short spectral response uses a period of 0.2 second. Structures, if planned, can be designed using Site Class C.

TABLE 7.3.1
2013 CBC SEISMIC DESIGN PARAMETERS

Parameter	Value	2013 CBC Reference
Site Class	C	Section 1613.3.2
MCE_R Ground Motion Spectral Response Acceleration – Class B (short), S_S	0.892g	Figure 1613.3.1(1)
MCE_R Ground Motion Spectral Response Acceleration – Class B (1 sec), S_1	0.345g	Figure 1613.3.1(2)
Site Coefficient, F_A	1.043	Table 1613.3.3(1)
Site Coefficient, F_V	1.455	Table 1613.3.3(2)
Site Class Modified MCE_R Spectral Response Acceleration (short), S_{MS}	0.931g	Section 1613.3.3 (Eqn 16-37)
Site Class Modified MCE_R Spectral Response Acceleration (1 sec), S_{M1}	0.502g	Section 1613.3.3 (Eqn 16-38)
5% Damped Design Spectral Response Acceleration (short), S_{DS}	0.621g	Section 1613.3.4 (Eqn 16-39)
5% Damped Design Spectral Response Acceleration (1 sec), S_{D1}	0.335g	Section 1613.3.4 (Eqn 16-40)

7.3.2 Table 7.3.2 presents additional seismic design parameters for projects located in Seismic Design Categories of D through F in accordance with ASCE 7-10 for the mapped maximum considered geometric mean (MCE_G).

TABLE 7.3.2
2013 CBC SITE ACCELERATION DESIGN PARAMETERS

Parameter	Value	ASCE 7-10 Reference
Mapped MCE_G Peak Ground Acceleration, PGA	0.345g	Figure 22-7
Site Coefficient, F_{PGA}	1.055	Table 11.8-1
Site Class Modified MCE_G Peak Ground Acceleration, PGA_M	0.364g	Section 11.8.3 (Eqn 11.8-1)

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

7.4 Preliminary Grading Recommendations

7.4.1 Grading should be performed in accordance with the attached *Recommended Grading Specifications* (Appendix A). Where the recommendations of this section conflict with

Appendix A, the recommendations of this section take precedence. Earthwork should be observed and fill tested for proper compaction by Geocon Incorporated.

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

7.4.3 Site preparation should begin with the removal of deleterious material and vegetation. The depth of removal should be such that material exposed in cut areas or soils to be used as fill are relatively free of organic matter. Material generated during stripping and/or site demolition should be exported from the site.

7.4.4 In general, the upper 2 feet below existing grade or below proposed grade, whichever results in a deeper excavation, within area of pavement or flatwork improvements should be removed and replaced with properly compacted fill. Loose and/or soft portions of surficial soil within areas of planned grading may require deeper removals to attain proper compaction.

7.4.5 If structures are planned, the existing surficial soil should be removed to expose formational materials and replaced with properly compacted fill. The surficial soil should be removed a lateral extent of at least 5 feet outside of the planned structures. If formational materials are encountered within 3 feet of finish grade, consideration should be given to undercutting the formational materials at least 3 feet and replacing it with properly compacted fill to help with trenching operations for foundations and utilities.

7.4.6 The actual extent and depth of surficial soil requiring removal should be evaluated during the planned geotechnical investigation. Overly wet soils, as might be encountered in the vicinity of drainages, will require drying and/or mixing with drier soils to facilitate proper compaction.

7.4.7 Excavated, on-site soil if free of deleterious debris, expansive soil and large rock can be placed as fill and compacted in layers to the design finish grade elevations. Fill and backfill soil should be placed in horizontal loose layers approximately 6 to 8 inches thick, moisture conditioned as necessary, and compacted to a dry density of at least 90 percent of the laboratory maximum dry density near to slightly above optimum moisture content as determined by ASTM D 1557. The upper 12 inches of soil beneath pavement areas should be compacted to a dry density of at least 95 percent of the laboratory maximum dry density near to slightly above optimum moisture content.

7.4.8 Import fill, should consist of granular materials with a “very low” to “low” expansion potential (EI less than 50) free of deleterious material or stones larger than 3 inches and should be compacted as recommended herein. Geocon Incorporated should be notified of the import soil source and should perform laboratory testing of import soil prior to its arrival at the site to evaluate its suitability as fill material.

7.5 Site Drainage and Moisture Protection

7.5.1 The existing drainage channel at the proposed Baker Street location (see Figure ???) including the storm drain and outlets should be mitigated as a part of the proposed site improvements via appropriate storm drain, subdrain, and/or canyon subdrain system.

7.5.2 Adequate site drainage is critical to reduce the potential for differential soil movement, erosion and subsurface seepage. The site should be graded and maintained such that surface drainage is directed away from improvements in accordance with 2013 CBC 1804.3 or other applicable standards. In addition, surface drainage should be directed away from the top of slopes into swales or other controlled drainage devices. Pavement drainage should be directed into conduits that carry runoff away from the proposed improvements.

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

7.5.4 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. We recommend that area drains to collect excess irrigation water and transmit it to drainage structures or impervious above-grade planter boxes be used. In addition, where landscaping is planned adjacent to the pavement, we recommend construction of a cutoff wall along the edge of the pavement that extends at least 6 inches below the bottom of the base material.

7.5.5 If detention basins, bioswales, retention basins, or water infiltration devices are being considered, Geocon Incorporated should be retained to provide recommendations pertaining to the geotechnical aspects of possible impacts and design. Distress may be caused to planned improvements and properties located hydrologically downstream. The distress depends on the amount of water to be detained, its residence time, soil permeability, and other factors. We have not performed a hydrogeology study at the site. Downstream properties may be subjected to seeps, springs, slope instability, raised groundwater, movement of foundations and slabs, or other impacts as a result of water infiltration.

7.5.6 The existing and planned soil conditions are likely not conducive to water infiltration and infiltration should not be allowed due to the relatively shallow formational materials. Water storage devices can be installed to reduce the velocity and amount of water entering the storm drain system but liners will be required if water in contact with soil.

7.5.7 Storm water management devices, if planned, should be properly constructed to prevent water infiltration and lined with an impermeable liner (e.g. High-density polyethylene, HDPE, with a thickness of about 30 mil or equivalent Polyvinyl Chloride, PVC, liner). The devices should also be installed in accordance with the manufacturer's recommendations.

7.6 Conventional Retaining Walls

7.6.1 Retaining walls not restrained at the top and having a level backfill surface should be designed for an active soil pressure equivalent to the pressure exerted by a fluid density of 40 pounds per cubic foot (pcf). Where the backfill will be inclined at no steeper than 2:1 (horizontal to vertical), an active soil pressure of 55 pcf is recommended. These soil pressures assume that the backfill materials within an area bounded by the wall and a 1:1 plane extending upward from the base of the wall possess an EI of 90 or less. For those lots where backfill materials do not conform to the criteria herein, Geocon Incorporated should be consulted for additional recommendations.

7.6.2 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 in feet) at the top of the wall. Where walls are restrained from movement at the top, an additional uniform pressure of 7H psf should be added to the active soil pressure.

7.6.3 The structural engineer should determine the seismic design category for the project in accordance with Section 1613 of the CBC. If the project possesses a 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 18.3.5.12 of the 2013 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. A seismic load of 16H should be used for design. We used the peak ground acceleration adjusted for Site Class effects, PGA_M, of 0.36g calculated from ASCE 7-10 Section 11.8.3 and applied a pseudo-static coefficient of 0.33.

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

7.6.5 Retaining walls should be provided with a drainage system adequate to prevent the buildup of hydrostatic forces and waterproofed as required by the project architect. The soil immediately adjacent to the backfilled retaining wall should be composed of free draining material completely wrapped in Mirafi 140 (or equivalent) filter fabric for a lateral distance of 1 foot for the bottom two-thirds of the height of the retaining wall. The upper one-third should be backfilled with less permeable compacted fill to reduce water infiltration. The use of drainage openings through the base of the wall (weep holes) is not recommended 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. Figure 6 presents a 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.

7.6.6 In general, wall foundations having a minimum depth and width of 1 foot may be designed for an allowable soil bearing pressure of 2,000 psf. The proximity of the foundation to the top of a slope steeper than 3:1 could impact the allowable soil bearing pressure. Therefore, Geocon Incorporated should be consulted where such a condition is expected. The foundation should be deepened such that the bottom outside edge of the footing is at least 7 feet horizontally from the face of the slope.

7.6.7 The recommendations presented herein are generally applicable to the design of rigid concrete or masonry retaining walls having a maximum height of 8 feet. In the event that walls higher than 8 feet or other types of walls are planned, Geocon Incorporated should be consulted for additional recommendations.

7.7 Preliminary Pavement Recommendations

7.7.1 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 4.5 through 6.0 for the planned parking areas. The project civil engineer and owner should review the pavement designations to determine appropriate locations for pavement thickness. The final pavement sections for the parking lot should be based on the R-Value of the subgrade soil encountered at final subgrade elevation. We have assumed an R-Value of 20 and 78 for the subgrade soil and base materials, respectively, for the purposes of this preliminary analysis. Table 7.7.1 presents the preliminary flexible pavement sections.

TABLE 7.7.1
PRELIMINARY FLEXIBLE PAVEMENT SECTION

Location	Estimated Traffic Index	Assumed Subgrade R-Value	Asphalt Concrete (inches)	Class 2 Aggregate Base (inches)
Parking stalls for automobiles and light-duty vehicles	4.5	20	3.0	6
	5.0	20	3.0	7
	5.5	20	3.0	9
	6.0	20	3.5	10

7.7.2 We understand the pavement may consist of full-depth base sections, full-depth cement-treated base (CTB), or full-depth decomposed granite (DG) sections. Table 7.7.2 presents the recommended pavement sections for full-depth base and DG.

TABLE 7.7.2
PRELIMINARY ALTERNATIVE PAVEMENT SECTION

Location	Estimated Traffic Index	Assumed Subgrade R-Value	Option 1 Full-Depth Base Thickness (inches)	Option 2 Full-Depth Cement-Treated Base (Inches)	Option 3 Full-Depth DG Thickness (inches)
Parking stalls for automobiles and light-duty vehicles	4.5	20	13	8.5	15.5
	5.0	20	14	9.5	17.5
	5.5	20	15.5	10.0	19
	6.0	20	17	11.0	20.5

7.7.3 Additional laboratory testing should be performed during future geotechnical studies. The upper 12 inches of the subgrade soil should be compacted to a dry density of at least 95 percent of the laboratory maximum dry density based on ASTM D 1557 near to slightly above optimum moisture content beneath the selected pavement section.

7.7.4 Class 2 base should conform to Section 26-1.02B of the *Standard Specifications for the State of California Department of Transportation (Caltrans)* and should be compacted to a minimum of 95 percent of the maximum dry density at near optimum moisture content. The asphalt concrete should conform to Section 203-6 of the *Standard Specifications for*

Public Works Construction (Green Book). Additionally, cement-treated base will be required if the pavement areas are deemed to be within the City of San Diego jurisdiction.

7.7.5 The asphalt concrete should conform to Section 203-6 of the *Standard Specifications for Public Works Construction (Greenbook)*. The asphalt concrete should be compacted to a density of at least 95 percent of the laboratory Hveem density in accordance with ASTM D 2726.

7.7.6 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.

7.7.7 If a full-depth DG pavement will be installed, the owner should consider incorporating a stabilizing agent within the DG material. The stabilizing agent helps reduce the erosion potential of the DG if water flows over the surface.

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

7.7.9 The performance of asphalt concrete pavement is highly dependent upon providing positive surface drainage away from the edge of the pavement. Ponding of water on or adjacent to the pavement will likely result in pavement distress and subgrade failure. If planter islands are proposed, the perimeter curb should extend at least 12 inches below the subgrade elevation of the adjacent pavement or below proposed subgrade elevations, whichever is deeper. In addition, the surface drainage within the planter should be such that ponding will not occur. The use of water quality basins increases the potential for a groundwater condition the pavement section.

7.8 Site Drainage and Moisture Protection

7.8.1 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 improvements. The site should be graded and maintained such that surface drainage is directed away from structures in accordance with 2013 CBC 1804.3 or other applicable standards. In addition, surface drainage should be directed away from the top of slopes into swales or other controlled drainage devices. Roof and pavement drainage should be directed into conduits that carry runoff away from the proposed structure.

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

7.8.3 If detention basins, bioswales, retention basins, water infiltration, low impact development (LID), or storm water management devices are being considered, Geocon Incorporated should be retained to provide recommendations pertaining to the geotechnical aspects of possible impacts and design. Distress may be caused to planned improvements and properties located hydrologically downstream if water infiltrates the soil. The distress depends on the amount of water to be detained, its residence time, soil permeability, and other factors. We have not performed a hydrogeology study at the site. If infiltration of storm water runoff was incorporated into the project design, downstream properties may be subjected to seeps, springs, slope instability, raised groundwater, movement of foundations and slabs, or other undesirable impacts as a result of this water infiltration.

7.9 Future Geotechnical Investigation

7.9.1 A geotechnical investigation can be performed consisting of 2 to 3 trenches utilizing a rubber tire backhoe, or hand excavations, at each of the proposed parking areas. The field investigation would consist of sampling the soil conditions during excavation of the exploratory trenches, to observe the soil conditions encountered, and evaluate the surficial deposits and depth of groundwater.

7.9.2 We should perform laboratory tests on selected soil samples to evaluate maximum dry density and optimum moisture content, resistance value (R-Value), in-situ dry density and moisture content. Similar laboratory tests should also be performed on imported fill soil samples.

7.9.3 The geotechnical investigation report should present our findings, conclusions, and recommendations regarding the geotechnical aspects of grading and improvements as presently proposed. Excavation characteristics, geologic hazard analyses, and remedial grading measures at the site would be included in the report.

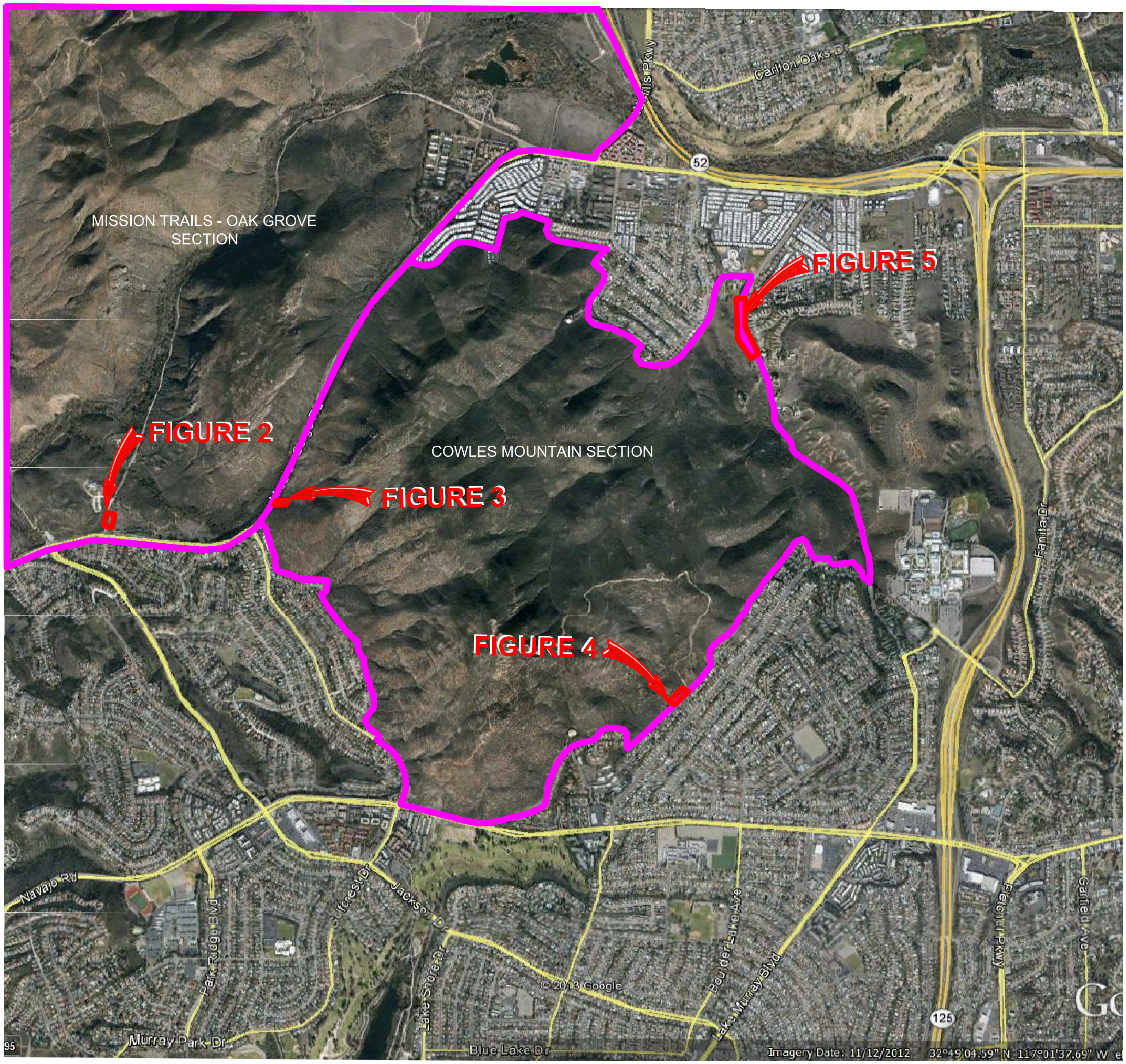
LIMITATIONS AND UNIFORMITY OF CONDITIONS

1. 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.

2. 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 that the necessary steps are taken to see that the contractor and subcontractors carry out such recommendations in the field.

3. 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 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.

4. 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.



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NO SCALE

GEOCON INCORPORATED

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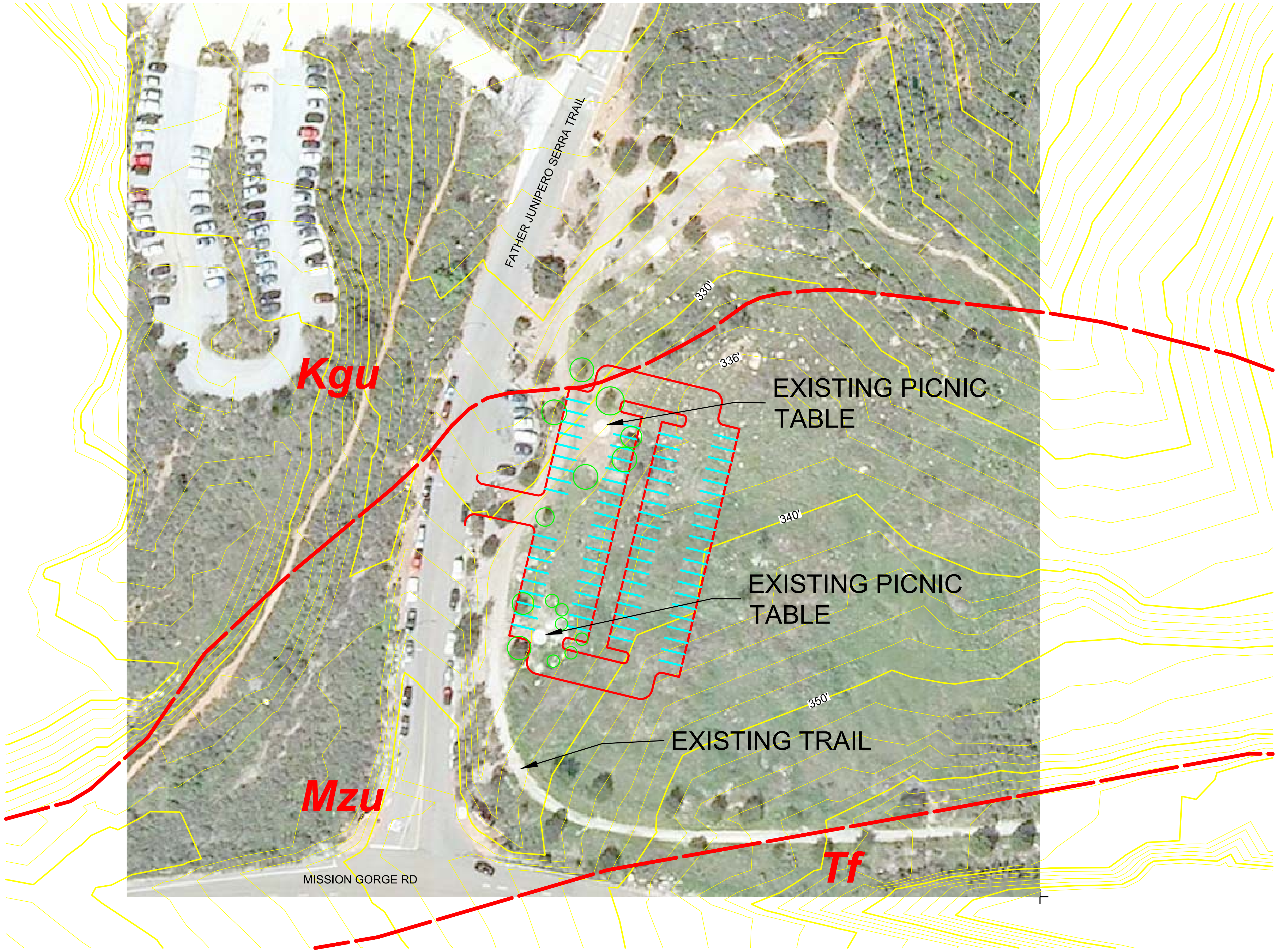
PHONE 858 558-6900 - FAX 858 558-6159

VICINITY MAP

MISSION TRAILS REGIONAL PARK
SAN DIEGO, CALIFORNIA

ME / CW		DSK/GTYPD	DATE 04 - 09 - 2014	PROJECT NO. G1330 - 52 - 01	FIG. 1

MISSION TRAILS
REGIONAL PARK
OAK GROVE
SAN DIEGO, CALIFORNIA



0' 40' 80' 160'
SCALE 1"=80'

GEOCON LEGEND

Tf ....FRIARS FORMATION

Kgu ....GRANITIC ROCK (Granodiorite)

Mzu ........METAVOLCANIC ROCK

........APPROX. LOCATION OF GEOLOGIC CONTACT

GEOLOGIC MAP

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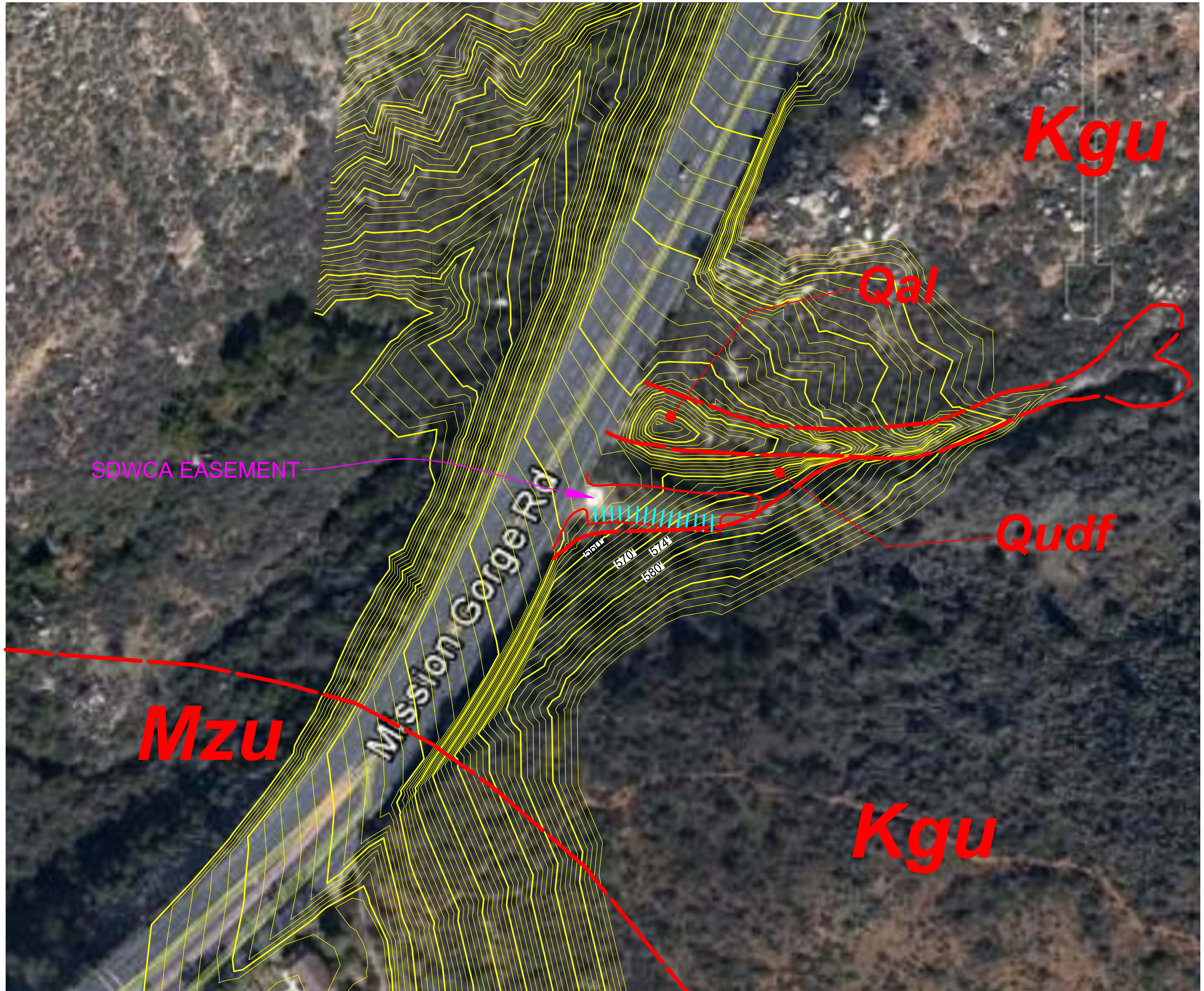
PROJECT NO. G1330 - 52 - 01
FIGURE 2
DATE 04 - 09 - 2014

Plotted:04/09/2014 4:23PM | By:JONATHAN WILKINS | File Location:Y:\PROJECTS\G1330-52-01 MISSION TRAIL PARK\SHEETS\G1330-52-01 SitePlan-MissionGorge2&BarkerWay.dwg

MISSION TRAILS
REGIONAL PARK
MISSION GORGE RD
SAN DIEGO, CALIFORNIA



0' 60' 120' 240'
SCALE 1"=120'

GEOCON LEGEND

Qudf ........UNDOCUMENTED FILL
Qal ....ALLUVIUM
Kgu ....GRANITIC ROCK (Granodiorite)
Mzu ........METAVOLCANIC ROCK
........APPROX. LOCATION OF GEOLOGIC CONTACT

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GEOLOGIC MAP

PROJECT NO. G1330 - 52 - 01
FIGURE 3
DATE 04 - 09 - 2014

MISSION TRAILS
REGIONAL PARK
BARKER WAY
SAN DIEGO, CALIFORNIA

0' 40' 80' 160'
SCALE 1"=80'

GEOCON LEGEND

Qudf ........UNDOCUMENTED FILL
Qal ....ALLUVIUM
Tmv ....MISSION VALLEY FORMATION
Kgu ....GRANITIC ROCK (Granodiorite)
........APPROX. LOCATION OF GEOLOGIC CONTACT

GEOLOGIC MAP

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PROJECT NO. G1330 - 52 - 01
FIGURE 4
DATE 04 - 09 - 2014

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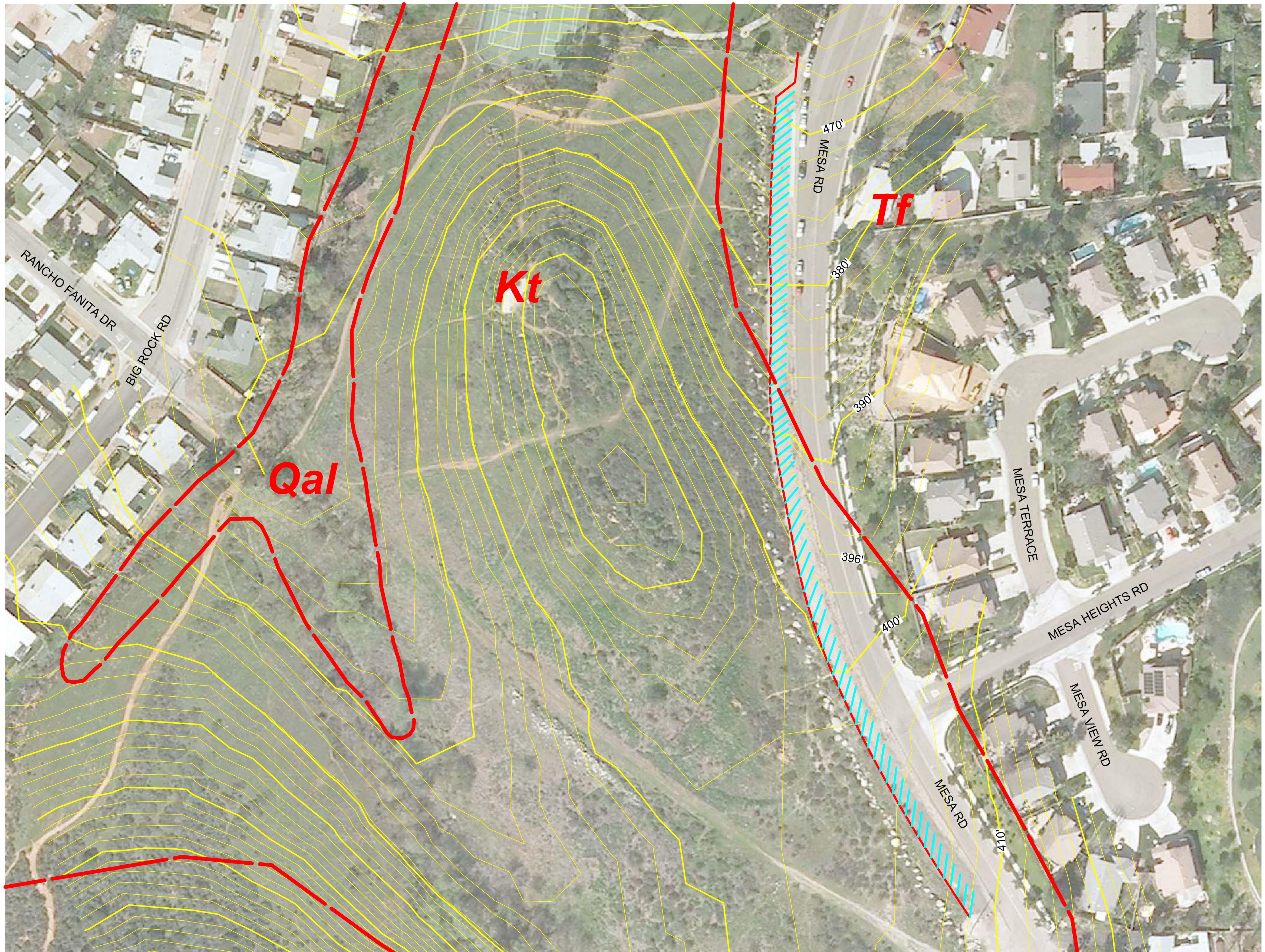
MISSION TRAILS
REGIONAL PARK
MESA ROAD
SAN DIEGO, CALIFORNIA



0' 60' 120' 240'

SCALE 1"=120'

GEOCON LEGEND

Qal ....ALLUVIUM

Tf ....FRIARS FORMATION

Kt ....GRANITIC ROCK (Tonalite)

........APPROX. LOCATION OF GEOLOGIC CONTACT

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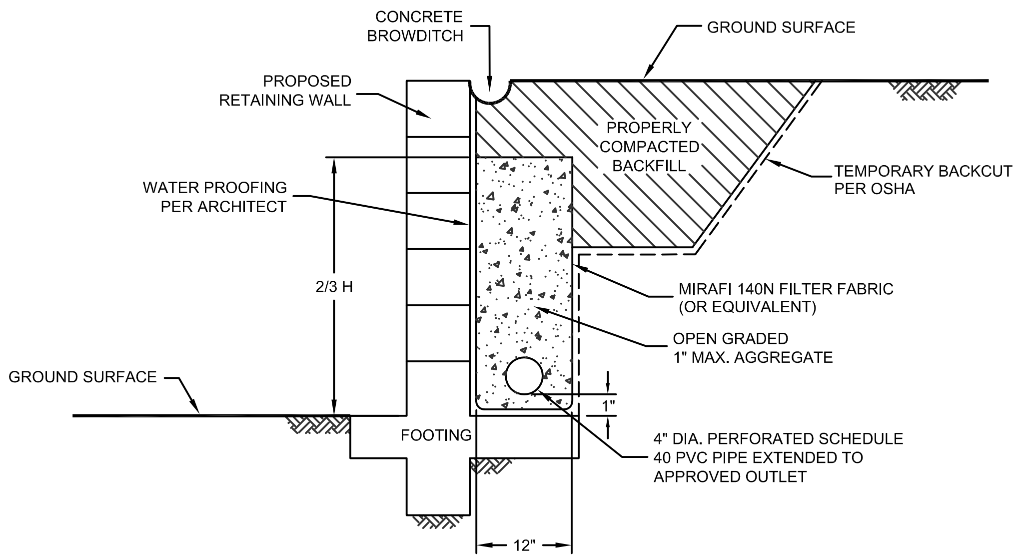
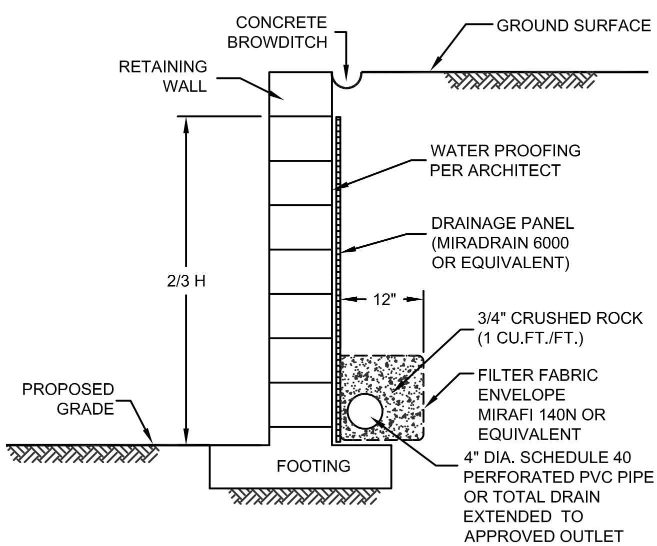
PROJECT NO. G1330 - 52 - 01

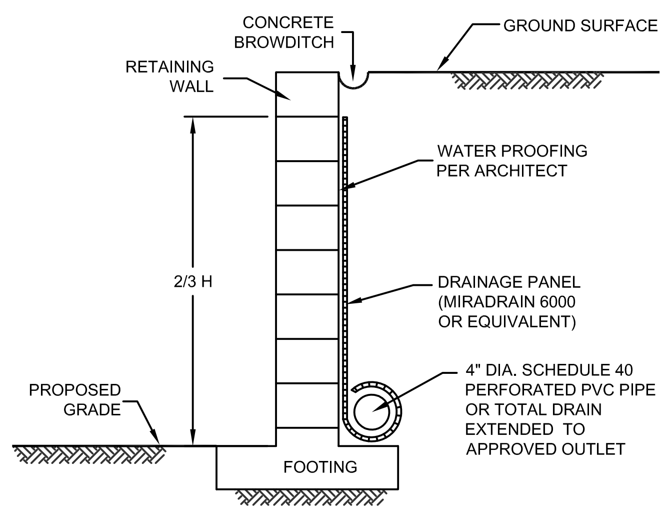
GEOLOGIC MAP

FIGURE 5

DATE 04 - 09 - 2014







NOTE :

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

NO SCALE

TYPICAL RETAINING WALL DRAIN DETAIL

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MISSION TRAILS REGIONAL PARK
SAN DIEGO, CALIFORNIA

SW / CW		DSK/GTYPD	DATE 04 - 09 - 2014	PROJECT G1330 - 52 - 01	FIG. 6

APPENDIX A

APPENDIX A

RECOMMENDED GRADING SPECIFICATIONS

FOR

MISSION TRAILS REGIONAL PARK
SAN DIEGO, CALIFORNIA
PROJECT NO. G1330-52-01

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 Incorporated. 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, 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

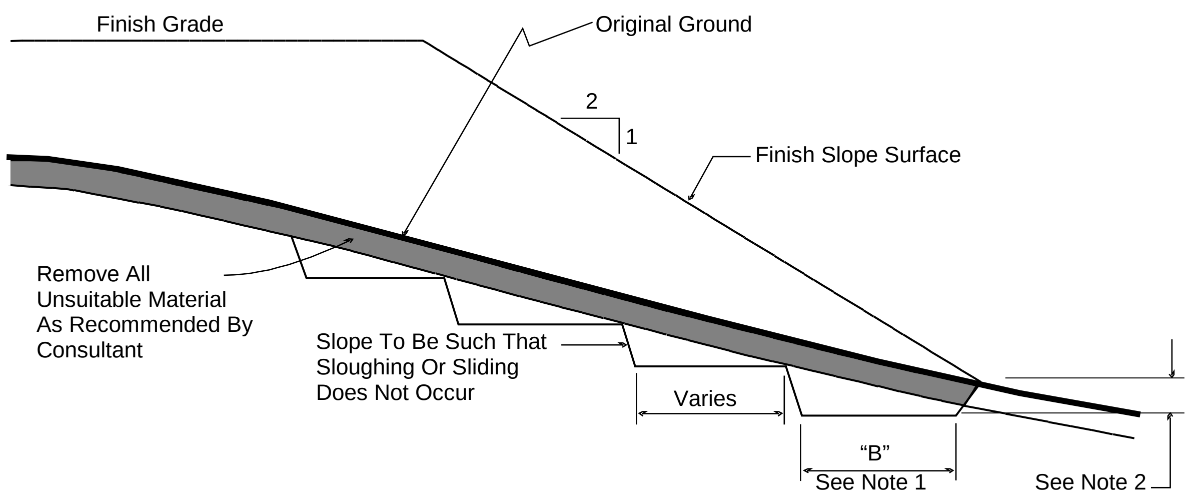
4.1 Areas to be excavated and filled shall be cleared and grubbed. Clearing shall consist of complete removal above the ground surface of trees, stumps, brush, vegetation, man-made structures, and similar debris. Grubbing shall consist of removal of stumps, roots, buried logs and other unsuitable material 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 Any asphalt pavement material removed during clearing operations should be properly disposed at an approved off-site facility. Concrete fragments that are free of reinforcing steel may be placed in fills, provided they are placed in accordance with Section 6.2 or 6.3 of this document.

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

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

TYPICAL BENCHING DETAIL



No Scale

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

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

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

5. COMPACTION EQUIPMENT

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

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

6. PLACING, SPREADING AND COMPACTION OF FILL MATERIAL

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

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

6.1.2 In general, the *soil* fill shall be compacted at a moisture content at or above the optimum moisture content as determined by ASTM D 1557-09.

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 1557-09. 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 1196-09, 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. OBSERVATION AND TESTING

7.1 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.

7.2 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.

7.3 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.

7.4 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.

7.5 The Consultant should observe the placement of subdrains, to verify that the drainage devices have been placed and constructed in substantial conformance with project specifications.

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

7.6.1 Soil and Soil-Rock Fills:

7.6.1.1 Field Density Test, ASTM D 1556-07, *Density of Soil In-Place By the Sand-Cone Method.*

7.6.1.2 Field Density Test, Nuclear Method, ASTM D 6938-08A, *Density of Soil and Soil-Aggregate In-Place by Nuclear Methods (Shallow Depth).*

7.6.1.3 Laboratory Compaction Test, ASTM D 1557-09, *Moisture-Density Relations of Soils and Soil-Aggregate Mixtures Using 10-Pound Hammer and 18-Inch Drop.*

7.6.1.4. Expansion Index Test, ASTM D 4829-08A, *Expansion Index Test*.

7.6.2 Rock Fills

7.6.2.1 Field Plate Bearing Test, ASTM D 1196-09 (Reapproved 1997) *Standard Method for Nonreparative Static Plate Load Tests of Soils and Flexible Pavement Components, For Use in Evaluation and Design of Airport and Highway Pavements.*

8. PROTECTION OF WORK

8.1 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.

8.2 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.

9. CERTIFICATIONS AND FINAL REPORTS

9.1 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.

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

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