GEOTECHNICAL INVESTIGATION AND PERCOLATION TESTING

MR 56 COMMERCIAL SITE
NWC HIGHWAY 74 AND BRIGGS ROAD
MENIFEE, CALIFORNIA

GEOCON
WEST, INC.

GEOTECHNICAL
ENVIRONMENTAL
MATERIALS

PREPARED FOR

MR 56 LLC
C/O THE RANCON GROUP
MURRIETA, CALIFORNIA

APRIL 24, 2017
PROJECT NO. T2765-22-01

Project No. T2765-22-01
April 24, 2017

MR 56 LLC
c/o The Rancon Group, Inc.
41391 Kalmia Street, Suite 200
Murrieta, California 92562

Attention: Mr. Dan Long

Subject: GEOTECHNICAL INVESTIGATION
AND PERCOLATION TESTING
MR 56 COMMERCIAL SITE
NWC HIGHWAY 74 AND BRIGGS ROAD
MENIFEE, CALIFORNIA

Dear Mr. Long:

In accordance with your authorization of Proposal IE-1813 dated March 14, 2017, Geocon West, Inc. (Geocon) herein submits the results of our geotechnical investigation and percolation testing for the proposed commercial development located at the northwest corner of the intersection of Highway 74 and Briggs Road, Menifee, California. The accompanying report presents our findings, conclusions and recommendations pertaining to the geotechnical aspects of the proposed development. Based on the results of our investigation, it is our opinion that the site can be developed as proposed, provided the recommendations of this report are followed and implemented during design and construction.

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 WEST, INC.

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GEOTECHNICAL INVESTIGATION AND PERCOLATION TESTING

1. PURPOSE AND SCOPE

This report presents the results of our geotechnical investigation and percolation testing for the proposed commercial development, located at the northwest corner of the intersection of Highway 74 and Briggs Road, Menifee, California as depicted on the *Vicinity Map*, Figure 1. The purpose of the investigation was to evaluate subsurface soil and geologic conditions underlying the area of proposed construction and, based on conditions encountered, to provide conclusions and recommendations pertaining to the geotechnical aspects of design and construction.

The scope of our investigation included a site reconnaissance, subsurface exploration, percolation testing, laboratory testing, engineering analyses, and the preparation of this report. The site was explored on March 28, 2017 by excavating ten 8-inch diameter borings using a truck-mounted hollow-stem auger drilling machine. Six of the borings were converted to percolation test holes. The borings were excavated to depths between 5 and 50 feet below the ground surface. The approximate locations of the exploratory excavations are presented on the *Geotechnical Map*, Figure 2. *Appendix A* presents a discussion of the field investigation, logs of the excavations, and percolation test data.

Laboratory tests were performed on selected soil samples obtained during the investigation to determine pertinent physical and chemical soil properties. *Appendix B* presents a summary of the laboratory test results.

The recommendations presented herein are based on analysis of the data obtained during the investigation and our experience with similar soil and geologic conditions. References reviewed to prepare this report are provided in the *List of References* section.

If project details vary significantly from those described above, Geocon should be contacted to determine the necessity for review and possible revision of this report.

2. SITE AND PROJECT DESCRIPTION

The site is located at the northwest intersection of Highway 74 and Briggs Road, in the city of Menifee, California. The area of proposed construction is bound on the south by Highway 74, on the east by Briggs Road, and of the north and west by vacant parcels, currently being used as agricultural land. The site is relative flat and has an elevation of 1,527 feet above mean sea level (MSL) in the northeast corner and 1,519 MSL in the southwest corner. Drainage is by sheet flow to the southwest. The site is currently being used to grow wheat.

This phase of development will consist of approximately 4-acres and include a gas station with convenience store and car wash, and a fast food restaurant in the southeast portion of the site. Several water quality basins around the perimeter of the site will drain into an interim detention basin of the southwestern area of the site.

Due to preliminary nature of the design, wall and column loads were not available. It is anticipated that column loads for the proposed structures will be up to 50 kips, and wall loads will be up to 3 kips per linear foot. Once the design phase and foundation loading configuration proceeds to a more finalized plan, the recommendations within this report should be reviewed and revised, if necessary.

If project details differ significantly from those described, Geocon should be contacted for review and possible revision to this report.

3. GEOLOGIC SETTING

The project site is in the Romoland area (recently incorporated into the city of Menifee) of southwestern Riverside County within the Peninsular Ranges Geomorphic Province (Province). The Peninsular Ranges are bound by the Transverse Ranges (San Gabriel and San Bernardino Mountains) to the north, and the Colorado Desert Geomorphic Province to the east. The Province extends westward into the Pacific Ocean and southward to the tip of Baja California. Geologic units within the Peninsular Ranges consist of granitic and metamorphic bedrock highlands and deep and broad alluvium filled valleys. Faulting within the Province is typically northwest trending and includes the San Andreas, San Jacinto, Elsinore, and Newport-Inglewood faults. Specifically, the site is located on an old alluvial fan emanating from the surrounding Lakeview Mountains. The site is underlain by older alluvial fan deposits observed underlying a thin layer of topsoil.

4. GEOLOGIC MATERIALS

4.1 General

Site geologic materials encountered consist of topsoil and Quaternary age old alluvial fan deposits. Detailed stratigraphic profiles are provided on the boring logs in *Appendix A* and are described herein in order of increasing age, and follow the nomenclature of Morton, 2003 (see *List of References*).

4.2 Topsoil

Topsoil was encountered to depths between 2½ and 5 feet below existing ground surface during our investigation. As encountered, the topsoil generally consists of dark brown silty sand that is loose to medium dense, and slightly moist to moist. Deeper topsoil may exist between excavations and in other portions of the site that were not directly explored.

4.3 Old Alluvial Fan Deposits (Qof)

Old alluvial fan deposits were encountered beneath the topsoil in all the borings. The old alluvial fan deposits consist of brown silty sand that is medium dense to very dense, and slightly moist to moist. Trace amounts of clay and calcium carbonate stringers were observed.

5. GROUNDWATER

Groundwater was not encountered in our borings, excavated to a maximum depth of 50 feet below the existing ground surface. The California Department of Water Resources (CDWR) well data indicates groundwater has been measured at depths between 72 to 114 feet below the ground surface in nearby wells. Based on the reported historic high groundwater level in the area (CDWR), the depth of the proposed construction, and the absence of groundwater observed in our borings, it is unlikely that groundwater will be encountered during construction. However, it is common for perched groundwater to seasonally occur in the area or for groundwater conditions to develop where none previously existed, especially in impermeable fine-grained soils which are heavily irrigated or after seasonal rainfall. In addition, recent requirements for stormwater infiltration could result in shallower seepage conditions in the immediate site vicinity. Proper surface drainage of irrigation and precipitation will be critical for future performance of the project. Recommendations for drainage are provided in the S*urface Drainage* section of this report.

6. GEOLOGIC HAZARDS

6.1 Surface Fault Rupture

The numerous faults in southern California include active, potentially active, and inactive faults. The criteria for these major groups are based on criteria developed by the California Geological Survey (CGS, formerly known as CDMG) for the Alquist-Priolo Earthquake Fault Zone Program (Bryant and Hart, 2007). By definition, an active fault is one that has had surface displacement within Holocene time (about the last 11,000 years). A potentially active fault has demonstrated surface displacement during Quaternary time (approximately the last 1.6 million years), but has had no known Holocene movement. Faults that have not moved in the last 1.6 million years are considered inactive.

The site is not within a currently established State of California Alquist-Priolo Earthquake Fault Zone (CGS, 2016) or County of Riverside Fault Zone for surface fault rupture hazards. No active or potentially active faults with the potential for surface fault rupture are known to pass directly beneath the site. Therefore, the potential for surface rupture due to faulting occurring beneath the site during the design life of the proposed development is considered low. However, the site is located in the seismically active southern California region, and could be subjected to moderate to strong ground shaking in the event of an earthquake on one of the many active southern California faults

The closest active fault to the site is the Casa Loma strand of the San Jacinto Fault Zone located approximately 7.4 miles to the northeast (CDMG, 1986). Other nearby active faults are the Glen Ivy North and Wildomar segments of the Elsinore Fault Zone, located 11.3 and 12.3 miles to the southwest, respectively. The active San Andreas Fault Zone is located approximately 22.5 miles northeast of the site.

6.2 Seismicity

As with all southern California, the site has experienced historic earthquakes from various regional faults. The seismicity of the region surrounding the site was formulated based on research of an electronic database of earthquake data. A partial list of moderate to major magnitude earthquakes that have occurred in the southern California area within the last 100 years is included in Table 6.2.1 below.

TABLE 6.2.1
LIST OF HISTORIC EARTHQUAKES

Earthquake (Oldest to Youngest)	Date of Earthquake	Magnitude	Distance to Epicenter (Miles)	Direction to Epicenter
San Jacinto	April 24, 1918	6.8	8	E
Loma Linda Area	July 22, 1923	6.3	19	NNW
Long Beach	March 10, 1933	6.4	48	W
Buck Ridge	March 25, 1937	6.0	55	ESE
Imperial Valley	May 18, 1940	6.9	53	ENE
Desert Hot Springs	December 4, 1948	6.0	45	ENE
Arroyo Salada	March 19, 1954	6.4	68	ESE
Borrego Mountain	April 8, 1968	6.5	74	ESE
San Fernando	February 9, 1971	6.6	92	WNW
Joshua Tree	April 22, 1992	6.1	54	ENE
Landers	June 28, 1992	7.3	54	NE
Big Bear	June 28, 1992	6.4	37	NE
Northridge	January 17, 1994	6.7	94	WNW
Hector Mine	October 16, 1999	7.1	80	NE

The site could be subjected to strong ground shaking in the event of an earthquake. However, this hazard is common in southern California and the effects of ground shaking can be mitigated if the proposed structures are designed and constructed in conformance with current building codes and engineering practices.

6.3 Seismic Design Criteria

The following table summarizes site-specific design criteria obtained from the 2016 California Building Code (CBC; Based on the 2015 International Building Code [IBC] and ASCE 7-10), Chapter 16 Structural Design, Section 1613 Earthquake Loads. The data was calculated using the computer program *U.S. Seismic Design Maps*, provided by the USGS. The short spectral response uses a period of 0.2 second. We evaluated the Site Class based on the discussion in Section 1613.3.2 of the 2016 CBC and Table 20.3-1 of ASCE 7-10. The values presented below are for the risk-targeted maximum considered earthquake (MCE_R).

TABLE 6.3.1
2016 CBC SEISMIC DESIGN PARAMETERS

Parameter	Value	2016 CBC Reference
Site Class	C	Section 1613.3.2
MCE_R Ground Motion Spectral Response Acceleration – Class B (short), S_S	1.500g	Figure 1613.3.1(1)
MCE_R Ground Motion Spectral Response Acceleration – Class B (1 sec), S_1	0.600g	Figure 1613.3.1(2)
Site Coefficient, F_a	1.0	Table 1613.3.3(1)
Site Coefficient, F_V	1.3	Table 1613.3.3(2)
Site Class Modified MCE_R Spectral Response Acceleration (short), S_{MS}	1.500g	Section 1613.3.3 (Eqn 16-37)
Site Class Modified MCE_R Spectral Response Acceleration (1 sec), S_{M1}	0.780g	Section 1613.3.3 (Eqn 16-38)
5% Damped Design Spectral Response Acceleration (short), S_{DS}	1.000g	Section 1613.3.4 (Eqn 16-39)
5% Damped Design Spectral Response Acceleration (1 sec), S_{D1}	0.520g	Section 1613.3.4 (Eqn 16-40)

Table 6.3.2 presents additional seismic design parameters for projects located in Seismic Design Categories of D through F in accordance with ASCE 7-10 for the geometric mean maximum considered earthquake (MCE_G).

TABLE 6.3.2
2016 CBC SITE ACCELERATION DESIGN PARAMETERS

Parameter	Value	ASCE 7-10 Reference
Mapped MCE_G Peak Ground Acceleration, PGA	0.500g	Figure 22-7
Site Coefficient, F_{PGA}	1.0	Table 11.8-1
Site Class Modified MCE_G Peak Ground Acceleration, PGA_M	0.500g	Section 11.8.3 (Eqn 11.8-1)

The Maximum Considered Earthquake Ground Motion (MCE) is the level of ground motion that has a 2 percent chance of exceedance in 50 years, with a statistical return period of 2,475 years. According to the 2016 California Building Code and ASCE 7-10, the MCE is to be utilized for the evaluation of liquefaction, lateral spreading, seismic settlements, and it is our understanding that the intent of the Building code is to maintain "Life Safety" during a MCE event. The Design Earthquake Ground Motion (DE) is the level of ground motion that has a 10 percent chance of exceedance in 50 years, with a statistical return period of 475 years.

Deaggregation of the MCE peak ground acceleration was performed using the USGS online BETA Unified Hazard Tool, 2008 Conterminous U.S. Dynamic edition. The result of the deaggregation analysis indicates that the predominant earthquake contributing to the MCE peak ground acceleration is characterized as a 7.05 magnitude event occurring at a hypocentral distance of 13.9 kilometers from the site.

Deaggregation was also performed for the Design Earthquake (DE) peak ground acceleration, and the result of the analysis indicates that the predominant earthquake contributing to the DE peak ground acceleration is characterized as a 7.03 magnitude occurring at a hypocentral distance of 16.2 kilometers from the site.

Conformance to the criteria in Tables 6.3.1 and 6.3.2 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.

6.4 Liquefaction Potential

Liquefaction is a phenomenon in which loose, saturated, relatively cohesionless soil deposits lose shear strength during strong ground motions. Primary factors controlling liquefaction include intensity and duration of ground motion, gradation characteristics of the subsurface soils, in-situ stress conditions, and the depth to groundwater. Liquefaction is typified by a loss of shear strength in the liquefied layers due to rapid increases in pore water pressure generated by earthquake accelerations.

The current standard of practice, as outlined in the "Recommended Procedures for Implementation of DMG Special Publication 117, Guidelines for Analyzing and Mitigating Liquefaction in California" and "Special Publication 117A, Guidelines for Evaluating and Mitigating Seismic Hazards in California" requires liquefaction analysis to a depth of 50 feet below the lowest portion of the proposed structure. Liquefaction typically occurs in areas where the soils below the water table are composed of poorly consolidated, fine to medium-grained, primarily sandy soil. In addition to the requisite soil conditions, the ground acceleration and duration of the earthquake must also be of a sufficient level to induce liquefaction.

Based on the dense nature of the old alluvial deposits, the potential for liquefaction and seismically-induced settlement at the site is considered negligible.

6.5 Collapsible Soils

Hydroconsolidation 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 soils underlying the site are typically removed and recompacted during remedial site grading. However, if compressible soil is left in-place, a potential for settlement due to hydroconsolidation of the soil exists.

Based on the results of our laboratory testing, the onsite soils do not exhibit a potential for significant collapse upon saturation. Furthermore, remedial grading (removal of topsoil and upper older alluvial fan deposits) is recommended to further reduce the potential effects of collapsible soils in the near surface layers.

6.6 Landslides

There are no steep slopes on or adjacent to the site. Therefore, landslides are not a design consideration for the site.

6.7 Rock Fall Hazards

Rock falls are not a design consideration for the site.

6.8 Slope Stability

Fill slopes are anticipated to be less than 10 feet in vertical height and graded to inclinations of 2:1. In general, it is our opinion that proposed fill slopes will possess adequate factors of safety for global and surficial stability. Cut slopes are not anticipated at the site. Specific slope stability analyses should be performed if graded fill slopes over 10 feet are planned at the site. Fill keys should be constructed in accordance with the standard grading specifications in *Appendix C*. Grading of fill slopes should be designed in accordance with the requirements of the local building codes of the County of Riverside and the 2016 California Building Code (CBC).

6.9 Tsunamis and Seiches

A tsunami is a series of long period waves generated in the ocean by a sudden displacement of large volumes of water. 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 (Legg, *et al.*, 2003). The site is located approximately 56 miles from the nearest coastline; therefore, the negligible risk associated with tsunamis is not a design consideration.

A seiche is a run-up of water within a lake or embayment triggered by fault- or landslide-induced ground displacement. The site is located approximately 7 miles south of Lake Perris and 6 miles northeast of Diamond Valley Lake. The site is located up gradient from Diamond Valley Lake, therefore a seiche emanating from Diamond Valley Lake is not a design consideration. Due to the distance from Perris Lake, a seiche is not a design consideration for the site.

6.10 Dam Inundation

Dam inundation is the flooding of an area downstream of a dam as the result of dam failure. Causes of inundation include earthquakes or over filling of a dam. Lakes near the site that have dams include Perris, Diamond Valley and Canyon Lakes. Canyon and Diamond Valley Lakes are down gradient from the site, therefore inundation due to dam failure is not a design consideration. Perris Lake is up gradient; however, inundation in the event of dam failure would follow the San Jacinto River channel southeast towards Canyon Lake. Therefore, inundation due to dam failure of Perris Lake is not a design consideration (Department of Water Resources, 1975).

7. SITE INFILTRATION

Percolation testing was performed in accordance with the Riverside County Flood Control and Water Conservation District *Low Impact Development Best Management Practices Handb*ook *(Handbook).* The percolation tests were run in accordance with Appendix A, Section 2.3 Shallow Percolation Test Procedure. This method requires two percolation tests and one deep excavation per basin. We utilized a truck-mounted drill rig to excavate the geotechnical and percolation borings. Percolation testing was performed in five areas of the site that are anticipated to receive stormwater infiltration to provide preliminary infiltration values for project planning and design.

The percolation test pits P-1 through P-6 were excavated in the anticipated areas of future stormwater infiltration structures. The percolation test locations are depicted on the *Geotechnical Map*, Figure 2. Boring logs and percolation test data are presented in *Appendix A*. No groundwater was observed within the borings.

Percolation borings were excavated to depths of approximately five feet using a truck-mounted drill rig. Two inches of gravel was placed at the base of each test hole. A 3-inch diameter perforated pipe was placed within each test hole and gravel was placed in the annular space between the sidewall and the 3-inch pipe. Each test location was pre-saturated with five gallons of water, and the percolation testing began approximately 24 hours after the holes were pre-saturated.

Calculations to convert the percolation test rate to infiltration test rate are in accordance with the Handbook Section 2.3, the Porchet Method. Please note that the Handbook requires a factor of safety of 3 be applied to the tested infiltration rates indicated below.

Table 7.1 provides a summary of the infiltration test results. Infiltration tests were performed near the anticipated depth of the proposed basin; however, we understand that other BMPs may be used infiltrate stormwater, including surface bioswales.

TABLE 7.1
INFILTRATION TEST RATES

Parameter	P-1	P-2	P-3	P-4	P-5	P-6
Soil Type	Normal	Normal	Normal	Normal	Normal	Normal
Change in head over time (in): ΔH						
Time Interval (min): Δt	30	30	30	30	30	30
Radius of test hole (in): r	4	4	4	4	4	4
Average head over time interval (in): Havg	40.1	41.2	40.4	41.7	43.9	43.1
Tested Infiltration Rate (in/hr): It	0.05	0.01	0.06	0.01	0.03	0.03

8. CONCLUSIONS AND RECOMMENDATIONS

8.1 General

8.1.1 It is our opinion that soil or geologic conditions were not encountered during the investigation that would preclude the proposed development of the project provided the recommendations presented herein are followed and implemented during design and construction.

8.1.2 Potential geologic hazards at the site include seismic shaking. Based on our investigation and available geologic information, active, potentially active, or inactive faults are not present underlying or trending toward the site.

8.1.3 The topsoil and upper portion of the older alluvial fan deposits are considered unsuitable for the support of compacted fill or settlement-sensitive improvements. Remedial grading of the upper soils will be required as discussed herein. Newly placed engineered fill is considered suitable to support additional fill, proposed structures, and improvements.

8.1.4 The site is underlain by older alluvial fan deposits. We did not encountered refusal during boring excavation and removals should be attainable with grading equipment in good working order.

8.1.5 Oversize material (greater than six-inches) was not encountered during our subsurface investigation. However, occasional cobble and boulders may be encountered during grading. If oversize material is encountered it should be disposed of in accordance with *Appendix C*.

8.1.6 Moisture contents are expected to vary based on the season and amount of precipitation. Special handling of the soil should be anticipated, particularly if grading occurs during the rainy season, as drying back of the existing materials may be necessary prior to their use as fill.

8.1.7 Although groundwater was not encountered during our subsurface investigation, it is possible that perched water will be encountered during grading during the rainy seasons, and may require special considerations during grading.

8.1.8 Proper drainage should be maintained to preserve the engineering properties of the fill in the graded areas. Recommendations for site drainage are provided herein.

8.1.9 Once grading plans become available, they should be reviewed by this office to determine the necessity for review and possible revision of this report.

8.1.10 Fill slopes and cut slopes are not expected to exceed 10 feet in height and should be constructed at a gradient of 2:1 or flatter. If slope heights greater than those assumed herein are incorporated into the project, Geocon should be provided the opportunity to review the slopes for stability.

8.1.11 Recommended grading specifications are provided in *Appendix C*.

8.2 Soil Characteristics

8.2.1 It is the responsibility of the contractor to ensure that excavations and trenches are properly shored and maintained in accordance with applicable OSHA rules and regulations to maintain safety and maintain the stability of adjacent existing improvements.

8.2.2 Onsite excavations must be conducted in such a manner that potential surcharges from existing structures, construction equipment, and vehicle loads are resisted. The surcharge area may be defined by a 1:1 projection down and away from the bottom of an existing foundation or vehicle load. Penetrations below this 1:1 projection will require special excavation measures such as sloping or shoring. Excavation recommendations are provided in the *Temporary Excavations* section of this report.

8.2.3 Based on the material classifications and laboratory testing by Geocon, site soils consisting of topsoil and older alluvial fan deposits generally possess a low expansion potential (EI = 0 to 49) and are considered "expansive" as defined by 2016 California Building Code (CBC) Section 1803.5.3. Table 8.2.3 presents soil classifications based on the EI.

TABLE 8.2.3
SOIL CLASSIFICATION BASED ON EXPANSION INDEX

Expansion Index (EI)	Expansion Classification	2016 CBC Expansion Classification
0 – 20	**Very Low**	**Non-Expansive**
21 – 50	**Low**	**Expansive**
51 – 90	Medium	
91 – 130	High	
Greater Than 130	Very High	

8.2.4 Based on the material classifications and laboratory testing, site soils are generally anticipated to possess a low expansion potential (EI of 50 or less). If any medium to highly expansive soils are encountered or imported to the site, they should not be placed within four feet of the proposed foundations, flatwork or paving improvements. Additional testing for expansion potential should be performed once final grades are achieved.

8.2.5 Laboratory tests were completed on a sample of the site materials to evaluate the percentage of water-soluble sulfate content. Results from the laboratory water-soluble sulfate content tests indicate that the on-site materials at the location tested possess a sulfate content of 0.085% equating to an exposure class of S0 (Not Applicable) to concrete structures as defined by 2016 CBC Section 1904.3 and ACI 318. Table 8.2.5 presents a summary of concrete requirements set forth by 2016 CBC Section 1904.3 and ACI 318. The presence of water-soluble sulfates is not a visually discernible characteristic; therefore, other soil samples from the site could yield different concentrations. Additionally, over time landscaping activities (i.e., addition of fertilizers and other soil nutrients) may affect the concentration.

TABLE 8.2.5
REQUIREMENTS FOR CONCRETE
EXPOSED TO SULFATE-CONTAINING SOLUTIONS

Sulfate Exposure	Exposure Class	Water-Soluble Sulfate Percent by Weight	Cement Type	Maximum Water to Cement Ratio by Weight	Minimum Compressive Strength (psi)
Negligible	**S0**	**0.00-0.10**	**--**	**--**	**2,500**
Moderate	S1	0.10-0.20	II	0.50	4,000
Severe	S2	0.20-2.00	V	0.45	4,500
Very Severe	S3	> 2.00	V+ Pozzolan or Slag	0.45	4,500

8.2.6 Laboratory testing indicates the site soils have a minimum electrical resistivity of 830 ohm-cm, possess 270 parts per million chloride, 0.085% sulfate (850 parts per million), and have a pH of 7.0. As shown in Table 8.2.6, based on the resistivity test results, the site would be classified as “corrosive” in accordance with the Caltrans Corrosion Guidelines (Caltrans, 2012).

TABLE 8.2.6
CALTRANS CORROSION GUIDELINES

Corrosion Exposure	Resistivity (ohm-cm)	Chloride (ppm)	Sulfate (ppm)	pH
Not Corrosive	>1,000	<500	<2,000	>5.5
Corrosive	**<1,000**	**500 or greater**	**2,000 or greater**	**5.5 or less**

8.2.7 Geocon does not practice in the field of corrosion engineering. Therefore, further evaluation by a corrosion engineer should be performed if improvements that could be susceptible to corrosion are planned.

8.3 Grading

8.3.1 Grading should be performed in accordance with the *Recommended Grading Specifications* contained in *Appendix C* and the Grading Ordinances of the City of Menifee.

8.3.2 Prior to commencing grading, a preconstruction conference should be held at the site with the county inspector, 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.

8.3.3 Site preparation should begin with the removal of deleterious material, debris, buried trash, and vegetation. The depth of 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.

8.3.4 Any topsoil and unsuitable old alluvial fan deposits within the limits of grading should be removed to expose competent older alluvial fan deposits. Depth of removals is anticipated to be generally about 4 feet in depth below existing ground surface based on the subsurface excavation logs. However, excavations of up to 5 feet may be required in localized areas to remove all topsoil and/or loose soils. The actual depth of removal should be evaluated by the engineering geologist during grading operations. In general, removals should extend to a depth at which moderately dense soils with no visible porosity are encountered. For the purposes of this project moderately dense soils are defined as in-situ, natural soils which have a dry density of at least 85 percent of maximum density based on ASTM D1557. Where over excavation and compaction is to be conducted, the excavations should be extended laterally a minimum distance of 5 feet beyond the building footprint or for a distance equal to the depth of removal, whichever is greater. Where the lateral over-excavation is not possible, structural setbacks or deepened footings may be required. Removals in pavement and sidewalk areas should extend at least 1 foot beneath the pavement or flatwork subgrade elevation. The bottom of the excavations should be scarified to a depth of at least 1 foot, moisture conditioned as necessary, and properly compacted.

8.3.5 The cut portion in cut fill transition areas within proposed structural areas should be over excavated to remove the differential support conditions. Over excavations should extend a minimum of H/3 where H is the deepest fill in the building area. The over excavation should extend 5 feet horizontally from the outside edge of the structural area.

8.3.6 Geocon should observe the removal bottoms to check the competence at the bottom of the removal. Deeper excavations may be required if dry, loose, soft, or porous materials are present at the base of the removals.

8.3.7 The fill placed within 4 feet of proposed foundations should possess a “low” expansion potential (EI of 50 or less).

8.3.8 If perched groundwater or saturated materials are encountered during remedial grading, extensive drying and mixing with dryer soil will be required. The excavated materials should then be moisture conditioned as necessary to optimum moisture content prior to placement as compacted fill.

8.3.9 The site should be brought to finish grade elevations with fill compacted in layers. Layers of fill should be 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 at optimum moisture content as determined by ASTM D 1557. Fill materials placed below optimum moisture content may require additional moisture conditioning prior to placing additional fill.

8.3.10 Import fill (if necessary) should consist of granular materials with a “low” expansion potential (EI of 50 or less), less corrosive than the site soils, generally free of deleterious material and contain rock fragments no larger than 6 inches. Geocon 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.

8.3.11 All trench and foundation excavation bottoms must be observed and approved in writing by the Geotechnical Engineer, prior to placing bedding materials, fill, steel, gravel or concrete.

8.4 Graded Slopes

8.4.1 If constructed, fill slopes should be overbuilt at least 2 feet and cut back to grade. The slopes should be track-walked at the completion of each slope such that the fill is compacted to a dry density of at least 90 percent of the laboratory maximum dry density at optimum moisture content. Rocks greater than 6 inches in maximum dimension should not be placed within 15 feet of slope face.

8.4.2 Finished slopes should be landscaped with drought-tolerant vegetation having variable root depths and requiring minimal landscape irrigation. The site soils are granular and generally have little to no cohesion, so the slope surfaces will be highly susceptible to erosion. Therefore, the slopes should be drained and properly maintained to reduce the potential for surface erosion. Water should not be allowed to flow down slopes. Construction of earth berms, lined v-ditches or similar are recommended.

8.4.3 Proposed slopes are anticipated to be grossly stable, natural factors may result in slope creep and/or lateral fill extension over time. Slope creep is due to alternate wetting and drying of fill soils resulting in downslope movement. Slope creep occurs throughout the life of the slope and may affect improvements within about 10 feet of the top of slope, depending on the slope height. Slope creep can result in differential settlement of the structures supported by the slope. Lateral fill extension (LFE) occurs when expansive soils within the slope experience deep wetting due to rainfall or irrigation. LFE is mitigated as much as practical during grading by placing expansive soils at slightly greater than optimum moisture content.

8.4.4 Landscaping activities should avoid over steepening of slopes or grade changes along slopes. Backfill of irrigation lines should be compacted to 90 percent of the maximum dry density as evaluated by ASTM D1557. Vegetation should be light weight with variable root depth.

8.4.5 Excessive watering should be avoided; only enough irrigation to support vegetation suitable to the prevailing climate should be applied. Irrigation of natural, ungraded slopes should not be performed. Drainage or irrigation from adjacent improvements should not be directed to the tops of slopes. Drainage should be directed toward streets and approved drainage devices. Areas of seepage may develop after periods of heavy rainfall or irrigation.

8.5 Earthwork Grading Factors

8.5.1 Estimates of shrinkage factors are based on empirical judgments comparing the material in its existing or natural state as encountered in the exploratory excavations to a compacted state. Variations in natural soil density and in compacted fill density render shrinkage value estimates very approximate. As an example, the contractor can compact the fill to a dry density of 90 percent or higher of the laboratory maximum dry density. Thus, the contractor has an approximately 10 percent range of control over the fill volume. Based on our experience and the densities measured during our investigation, the shrinkage of onsite topsoil and older alluvial fan deposits is anticipated to be less than 5 percent when compacted to at least 90 percent of the laboratory maximum dry density. Please note that this estimate is for preliminary quantity estimates only. Due to the variations in the actual shrinkage/bulking factors, a balance area should be provided to accommodate variations.

8.6 Utility Trench Backfill

8.6.1 Utility trenches should be properly backfilled in accordance with the requirements of city of Menifee and the latest edition of the *Standard Specifications for Public Works Construction* (Greenbook). The pipes should be bedded with well graded crushed rock or clean sands (Sand Equivalent greater than 30) to a depth of at least one foot over the pipe. The bedding material must be inspected and approved in writing by the Geotechnical Engineer (a representative of Geocon). We recommend that jetting only be performed if trench wall soils have an SE of 15 or greater. <u>The use of well graded crushed rock is only acceptable if used in conjunction with filter fabric to prevent the gravel from having direct contact with soil.</u> The remainder of the trench backfill may be derived from onsite soil or approved import soil, compacted as necessary, until the required compaction is obtained. The use of 2-sack slurry and controlled low strength material (CLSM) are also acceptable as backfill. However, consideration should be given to the possibility of differential settlement where the slurry ends and earthen backfill begins. These transitions should be minimized and additional stabilization should be considered at these transitions.

8.6.2 In accordance with Eastern Municipal Water District (EMWD) requirements, utility excavation bottoms must be observed and approved in writing by the Geotechnical Engineer (a representative of Geocon), prior to placing bedding materials, fill, gravel, concrete, or geogrid.

8.6.3 As discussed in the *Groundwater* section of this report (see Section 7), groundwater was not encountered during this or the previous geotechnical investigation. The groundwater depths recorded during the explorations are representative of the groundwater conditions at the times of exploration and may not be representative of the groundwater regime including seasonal and long-term cyclical fluctuations. Furthermore, Geocon is not aware of any long-term monitoring data of the groundwater conditions across the site.

8.6.4 Recommendations for *Temporary Excavations* and *Shoring* are provided in Sections 10.13 and 10.14.

8.7 Foundation and Concrete Slabs-On-Grade Recommendations

8.7.1 We understand the proposed development has not yet been finalized. The foundation recommendations presented herein are for the proposed gas station convenience store and car wash, and a fast food restaurant. The proposed single-story commercial structures may be supported on conventional shallow foundations with a concrete slab-on-grade.

8.7.2 Foundations for the buildings may consist of either continuous strip footings and/or isolated spread footings. Conventionally reinforced continuous footings should be at least 12 inches wide and extend at least 18 inches below lowest adjacent pad grade. Isolated spread footings should have a minimum width of 24 inches and should extend at least 18 inches below lowest adjacent pad grade.

8.7.3 Foundations may be designed for an allowable soil bearing pressure of 3,500 pounds per square foot (psf) (dead plus live load). This value may be increased by 800 psf for each additional foot in depth and 350 psf for each additional foot of width to a maximum value of 4,500 psf. The allowable bearing pressure may be increased by one-third for transient loads due to wind or seismic forces.

8.7.4 The maximum expected static settlement for a single-story commercial structure supported on a conventional foundation system deriving support in engineered fill is estimated to be less than ½ inch and to occur below the heaviest loaded structural element. Settlement of the foundation system is expected to occur on initial application of loading. Differential settlement is not expected to exceed ¼ inch over 20 feet.

8.7.5 Steel reinforcement for continuous footings should consist of at least four No. 4 steel reinforcing bars placed horizontally in the footings, two near the top and two near the bottom. Steel reinforcement for the spread footings should be designed by the project structural engineer.

8.7.6 If depth increases are utilized for the exterior wall footings, this office should be provided a copy of the final construction plans so that the excavation recommendations presented herein could be properly reviewed and revised if necessary.

8.7.7 The embedment depths should be measured from the lowest adjacent pad grade for both interior and exterior footings. Figure 3 presents a wall/column footing dimension detail depicting lowest adjacent pad grade.

8.7.8 Foundations near slopes should be deepened such that the bottom outside edge of the footing is at least 7 feet horizontally from the face of the slope.

8.7.9 The above foundation dimensions and minimum reinforcement recommendations are based on soil conditions and building code requirements only, and are not intended to be used in lieu of those required for structural purposes.

8.7.10 Foundation excavations should be observed and approved in writing by the Geotechnical Engineer (a representative of Geocon West, Inc.), prior to the placement of reinforcing steel and concrete to verify that the exposed soil conditions are consistent with those anticipated. If unanticipated soil conditions are encountered, foundation modifications may be required.

8.7.11 Resistance to lateral loading may be provided by friction acting at the base of foundations, slabs and by passive earth pressure. An allowable coefficient of friction of 0.35 may be used with the dead load forces in newly compacted fill.

8.7.12 Passive earth pressure for the sides of foundations and slabs poured against newly placed engineered fill may be computed as an equivalent fluid having a density of 260 pounds per cubic foot with a maximum earth pressure of 2,600 pounds per square foot. When combining passive and friction for lateral resistance, the passive component should be reduced by one-third.

8.7.13 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 such concrete placement.

8.7.14 Slabs-on-grade that may receive moisture-sensitive floor coverings or may be used to store moisture-sensitive materials should be underlain by a vapor retarder placed directly beneath the slab. The vapor retarder and acceptable permeance should be specified by the project architect or developer based on the type of floor covering that will be installed. The vapor retarder design should be consistent with the guidelines presented in Section 9.3 of the American Concrete Institute's (ACI) Guide for Concrete Slabs that Receive Moisture-Sensitive Flooring Materials (ACI 302.2R-06) and should be installed in general conformance with ASTM E1643 (latest edition) and the manufacturer's recommendations. A minimum thickness of 15 mils extruded polyolefin plastic is recommended; vapor retarders which contain recycled content or woven materials are not recommended. The vapor retarder should have a permeance of less than 0.01 perms demonstrated by testing before and after mandatory conditioning. The vapor retarder should be installed in direct contact with the concrete slab with proper perimeter seal. If the California Green Building Code requirements apply to this project, the vapor retarder should be underlain by 4 inches of clean aggregate. It is important that the vapor retarder be puncture resistant since it will be in direct contact with angular gravel. As an alternative to the clean aggregate suggested in the Green Building Code, it is our opinion that the concrete slab-on-grade may be underlain by a vapor retarder over 4 inches of clean sand (sand equivalent greater than 30), since the sand will serve as a capillary break and will minimize the potential for punctures and damage to the vapor barrier.

8.7.15 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. Placement of 3 inches and 4 inches of sand is common practice in southern California for 5-inch and 4-inch thick slabs, respectively. The foundation engineer should provide appropriate concrete mix design criteria and curing measures that may be utilized to assure proper curing of the slab to reduce the potential for rapid moisture loss and subsequent cracking and/or slab curl. We suggest that the foundation engineer present 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.

8.7.16 The recommendations of this report are intended to reduce the potential for cracking of slabs due to expansive soil (if present), differential settlement of existing soil or soil with varying thicknesses. However, even with the incorporation of the recommendations presented herein, foundations, walls, and slabs-on-grade placed on such conditions may still exhibit some cracking due to soil movement and/or shrinkage. The occurrence of concrete shrinkage cracks is independent of the supporting soil characteristics. Their occurrence may be reduced and/or controlled by limiting the slump of the concrete, proper concrete placement and curing, and by the placement of crack control joints at periodic intervals, in particular, where re-entrant slab corners occur.

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

8.8 Exterior Concrete Flatwork

8.8.1 Exterior concrete flatwork not subject to vehicular traffic should be constructed in accordance with the recommendations herein assuming the subgrade materials possess an Expansion Index of 50 or less. Subgrade soils should be compacted to 90 percent relative compaction. Slab panels should be a minimum of 4 inches thick and when in excess of 8 feet square should be reinforced with 6x6-W2.9/W2.9 (6x6-6/6) welded wire mesh or No. 3 reinforcing bars spaced 18 inches center-to-center in both directions to reduce the potential for cracking. In addition, 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.

8.8.2 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 or differential settlement. The steel reinforcement should overlap continuously in flatwork to reduce the potential for vertical offsets within flatwork.

8.8.3 Where exterior flatwork abuts the structure at entrant or exit points, the exterior slab should be dowelled into the structure's foundation stem wall. 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.

8.8.4 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.

8.9 Conventional Retaining Walls

8.9.1 The recommendations presented herein are generally applicable to the design of rigid concrete or masonry retaining walls having a maximum height of 6 feet. If walls higher than 6 feet or other types of walls are planned, Geocon should be consulted for additional recommendations.

8.9.2 Retaining wall foundations may be designed in accordance with the recommendations provided in the *Foundation and Concrete Slabs-On-Grade Recommendations* section of this report (see Section 8.6).

8.9.3 Retaining walls with a level backfill surface that are not restrained at the top should be designed utilizing a triangular distribution of pressure (active pressure) of 35 pcf. Restrained walls are those that are not 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 and are retaining a level soil backfill, walls may be designed utilizing a triangular distribution of pressure (at-rest pressure) of 50 pcf. If restrained walls which retain sloping backfill are planned, Geocon should be contacted for additional recommendations.

8.9.4 The wall pressures provided above assume that the retaining wall will be properly drained preventing the buildup of hydrostatic pressure. If retaining wall drainage is not implemented, the equivalent fluid pressure to be used in design of undrained walls is 90 pcf. The value includes hydrostatic pressures plus buoyant lateral earth pressures.

8.9.5 The wall pressures provided above assume that the proposed retaining walls will support relatively undisturbed alluvial soils or engineered fill derived from onsite soils, with an EI of 50 or less. If import soil will be used to backfill proposed retaining walls, revised earth pressures may be required to account for the geotechnical properties of the import soil used as engineered fill. This should be evaluated once the use of import soil is established. All imported fill shall be observed, tested, and approved by Geocon West, Inc. prior to bringing soil to the site.

8.9.6 Additional active pressure should be added for a surcharge condition due to sloping ground, vehicular traffic, or adjacent structures. Once the design becomes more finalized, an addendum letter can be prepared revising recommendations and addressing specific surcharge conditions throughout the project, if necessary.

8.9.7 In addition to the recommended earth pressure, the upper 10 feet of the retaining walls adjacent to the street or driveway areas should be designed to resist a uniform lateral pressure of 100 psf, acting as a result of an assumed 300 psf surcharge behind the wall due to normal street traffic. If the traffic is kept back at least 10 feet from the walls, the traffic surcharge may be neglected.

8.9.8 The structural engineer should determine the seismic design category for the project. If the project possesses a seismic design category of D, E, or F, proposed retaining walls in excess of 6 feet in height should be designed with seismic lateral pressure (Section 1803.5.12 of the 2016 CBC). The seismic load should be applied in addition to the active earth pressure. The seismic load exerted on the wall should be a triangular distribution with a pressure of 10H (where H is the height of the wall, in feet, resulting in pounds per square foot [psf]) exerted at the bottom of the wall and zero at the top of the wall. The earth pressure is based on half of two thirds of PGA_M calculated from ASCE 7-10 Section 11.8.3.

8.9.9 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.

8.9.10 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 backfill (EI of 90 or less) with no hydrostatic forces or imposed surcharge load. Figure 4 presents a typical retaining wall drainage detail. If conditions different than those described are expected or if specific drainage details are desired, Geocon should be contacted for additional recommendations.

8.10 Preliminary Pavement Recommendations

8.10.1 The final pavement sections for roadways should be based on the R-Value of the subgrade soils encountered at final subgrade elevation. Streets should be designed in accordance with the County of Riverside *Design Standards* when final Traffic Indices and R-Value test results of subgrade soil are completed. Laboratory testing indicated that the site soils exhibited an R-value of 14. Preliminary flexible pavement sections are presented in Table 8.10.1.

TABLE 8.10.1
PRELIMINARY FLEXIBLE PAVEMENT SECTIONS

Location	Assumed Traffic Index	Assumed Subgrade R-Value	Asphalt Concrete (inches)	Crushed Aggregate Base (inches)
Light-Duty Vehicles – Local Street	5.5	14	4.0	8.0
Heavy Truck Vehicles – Industrial Collector	7.5	14	4.5	14.0

8.10.2 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 at optimum moisture content beneath pavement sections.

8.10.3 The crushed aggregated base and asphalt concrete materials should conform to Section 200-2.2 and Section 203-6, respectively, of the *Standard Specifications for Public Works Construction* (Greenbook) and the latest edition of the City of Menifee *Design Standards*. Base materials should be compacted to a dry density of at least 95 percent of the laboratory maximum dry density at optimum moisture content. Asphalt concrete should be compacted to a density of 95 percent of the laboratory Hveem density in accordance with ASTM D 1561.

8.10.4 A rigid Portland cement concrete (PCC) pavement section should be placed in driveway aprons and cross gutters and where desired to support heavy vehicle loads. We calculated the rigid pavement section in general conformance with the procedure recommended by the American Concrete Institute report ACI 330R, *Guide for Design and Construction of Concrete Parking Lots* using the parameters presented in Table 8.10.4.

TABLE 8.10.4
RIGID PAVEMENT DESIGN PARAMETERS

Design Parameter	Design Value
Modulus of subgrade reaction, k	100 pci
Modulus of rupture for concrete, M_R	550 psi
Traffic Category, TC	C and D
Average daily truck traffic, ADTT	100 and 700

8.10.5 Based on the criteria presented herein, the PCC pavement sections should have a minimum thickness as presented in Table 8.10.5.

TABLE 8.10.5
RIGID PAVEMENT RECOMMENDATIONS

Location	Portland Cement Concrete (inches)
Roadways (TC=C)	6.5
Truck Areas (TC=D)	8.0

8.10.6 The PCC 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 at optimum moisture content. This pavement section is based on a minimum concrete compressive strength of approximately 3,500 psi (pounds per square inch). Base material will not be required beneath concrete improvements.

8.10.7 A thickened edge or integral curb should be constructed on the outside of concrete slabs subjected to wheel loads. The thickened edge should be 1.2 times the slab thickness or a minimum thickness of 2 inches, whichever results in a thicker edge, and taper back to the recommended slab thickness 4 feet behind the face of the slab (e.g., a 7-inch-thick slab would have a 9-inch-thick edge). Reinforcing steel will not be necessary within the concrete for geotechnical purposes with the possible exception of dowels at construction joints as discussed herein.

8.10.8 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 not exceed 30 times the slab thickness with a maximum spacing of 15 feet for the 7-inch-thick or greater slabs (e.g., a 9-inch-thick slab would have a 15-foot spacing pattern). The depth of the crack-control joints and need for sealing of the joints should be determined by the referenced ACI report.

8.10.9 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.

8.10.10 The performance of pavement is highly dependent on providing positive surface drainage away from the edge of the pavement. Ponding of water on or adjacent to the pavement surfaces will likely result in pavement distress and subgrade failure. Drainage from landscaped areas should be directed to controlled drainage structures. Landscape areas adjacent to the edge of asphalt pavements are not recommended due to the potential for surface or irrigation water to infiltrate the underlying permeable aggregate base and cause distress. Where such a condition cannot be avoided, consideration should be given to incorporating measures that will significantly reduce the potential for subsurface water migration into the aggregate base. If planter islands are planned, the perimeter curb should extend at least 6 inches below the level of the base materials.

8.11 Temporary Excavations

8.11.1 Excavations up to 5 feet in vertical height may be required during grading operations. The contractor’s competent person should evaluate the necessity for layback of vertical cut areas. Vertical excavations up to 5 feet may be attempted where loose soils or caving sands are not present, and where not surcharged by existing structures or vehicle/construction equipment loads.

8.11.2 Vertical excavations greater than 5 feet may require sloping or slot-cutting measures in order to provide a stable excavation. It is anticipated that sufficient space is available to complete the majority of the required earthwork for this project using sloping measures. If necessary, shoring recommendations will be provided in an addendum.

8.11.3 Where sufficient space is available, temporary unsurcharged embankments may be sloped back at a uniform 1:1 (h:v) slope gradient or flatter. A uniform slope does not have a vertical portion.

8.11.4 Where sloped embankments are utilized, the top of the slope should be barricaded to prevent vehicles and storage loads at the top of the slope within a horizontal distance equal to the height of the slope. If the temporary construction embankments are to be maintained during the rainy season, berms are suggested along the tops of the slopes where necessary to prevent runoff water from entering the excavation and eroding the slope faces. The contractor's personnel should inspect the soil exposed in the cut slopes during excavation so that modifications of the slopes can be made if variations in the soil conditions occur. Excavations should be stabilized within 30 days of initial excavation.

8.12 Site Drainage and Moisture Protection

8.12.1 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 2016 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.

8.12.2 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.

8.12.3 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.

8.13 Plan Review

8.13.1 Geocon should review the grading and structural foundation plans for the project prior to final submittal. Additional analyses may be required after review of the project plans.

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 should be notified so that supplemental recommendations can be given. The evaluation or identification of the potential presence of hazardous materials was not part of the scope of services provided by Geocon.

2. This report is issued with the understanding that it is the responsibility of the owner, or of 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.

3. The findings of this report are valid as of the date of this report. However, changes in the conditions of a property can occur with the passage of time, whether they are 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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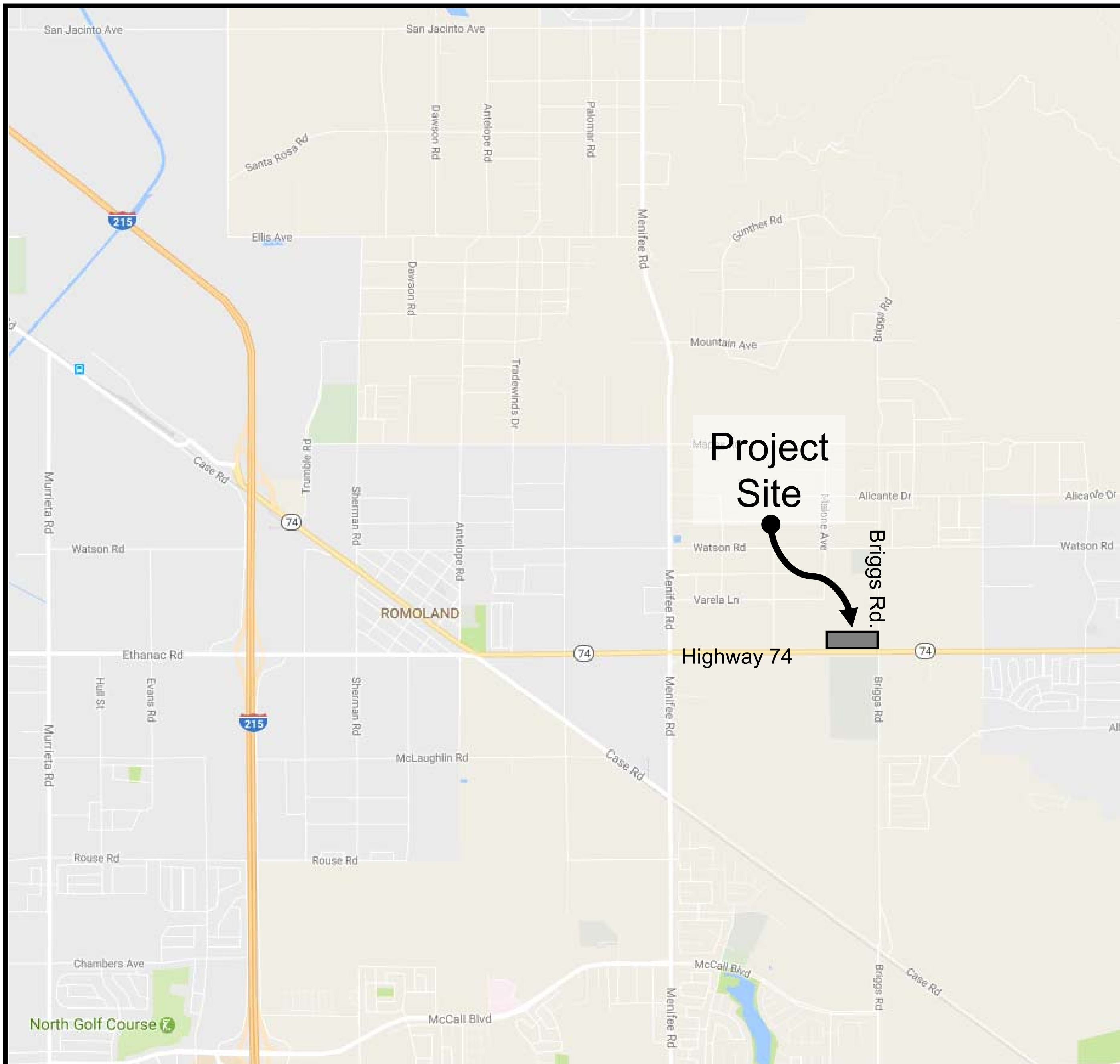

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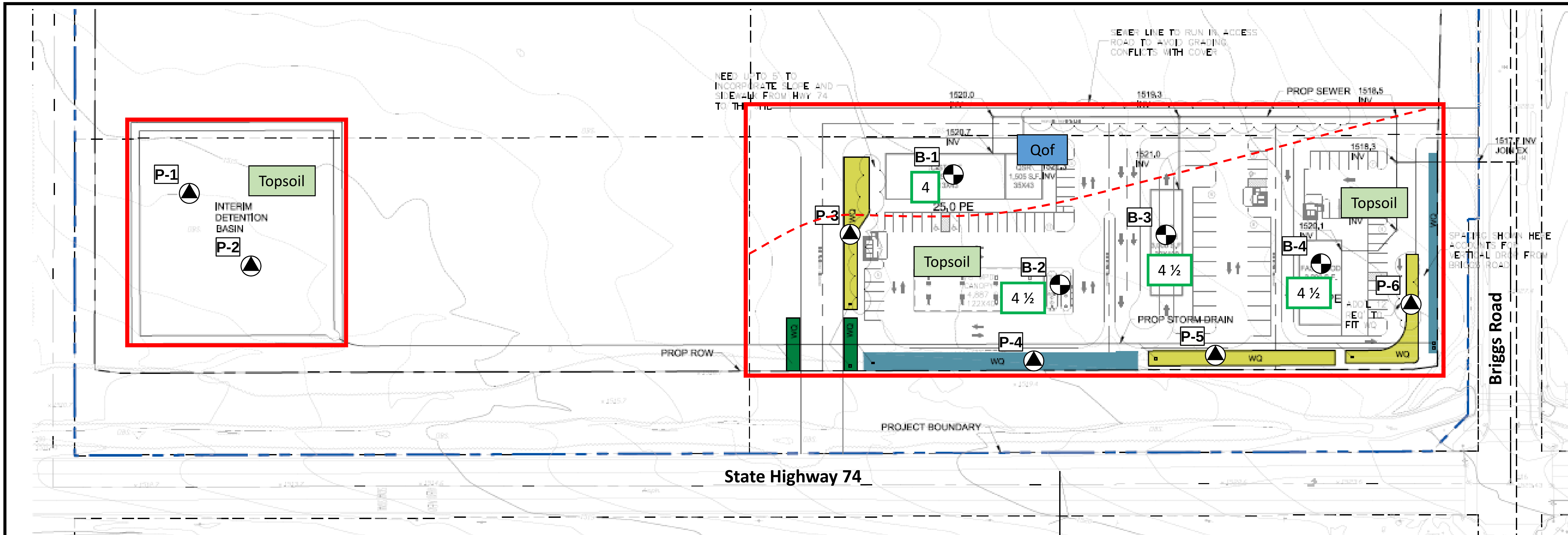
SOURCE: Google Maps, 2017

VICINITY MAP

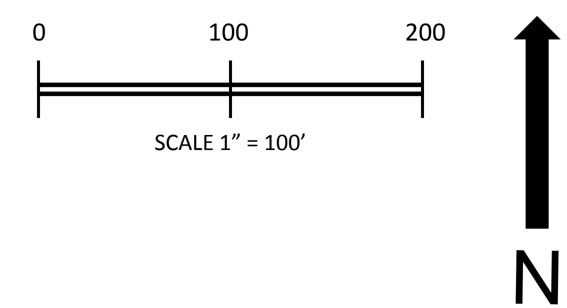
GEOCON
WEST, INC.
GEOTECHNICAL ENVIRONMENTAL MATERIALS
41571 CORNING PLACE, SUITE 101, MURRIETA, CA 92562-7065
PHONE 951-304-2300 FAX 951-304-2392

MR 56 COMMERCIAL SITE
NWC HIGHWAY 74 AND BRIGGS ROAD
MENIFEE, CALIFORNIA

AMO			APRIL, 2017	PROJECT NO. T2765-22-01	FIG. 1



Source: Anderson Consulting Engineers, Inc., undated.



GEOCON LEGEND

Locations are approximate

B-4 / 4 ½ .... BORING, WITH ANTICIPATED REMOVAL DEPTHS IN FEET

P-6 .... PERCOLATION TEST

Topsoil .... TOPSOIL

Qof .... ALLUVIUM

.... GEOLOGIC CONTACT

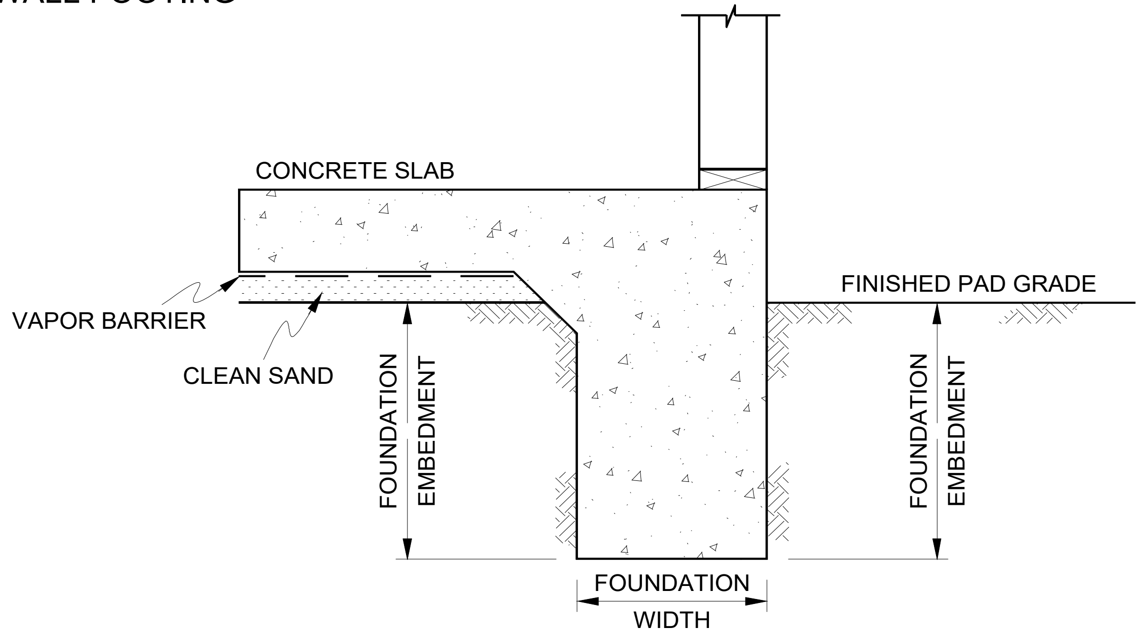
.... LIMITS OF THIS REPORT

GEOCON WEST, INC.
GEOTECHNICAL ENVIRONMENTAL MATERIALS
41571 CORNING PLACE, SUITE 101, MURRIETA, CA 92562-7065
PHONE 951-304-2300 FAX 951-304-2392

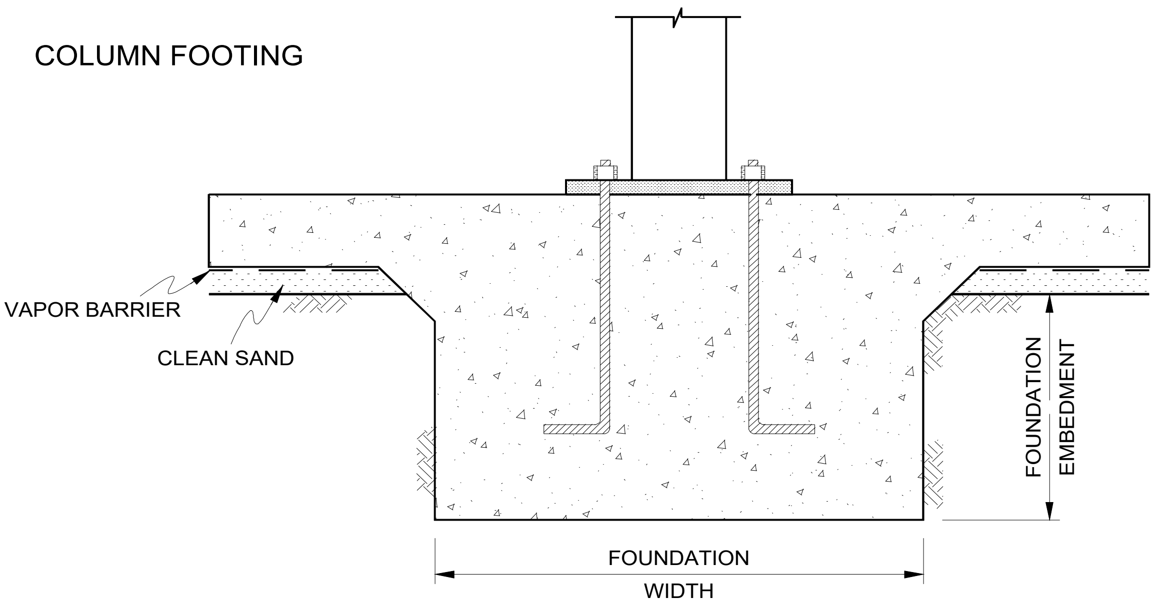
GEOTECHNICAL MAP
MR 56 COMMERCIAL SITE NWC HIGHWAY 74 AND BRIGGS ROAD MENIFEE, CALIFORNIA

AMO			APRIL, 2017	PROJECT NO. T2765-22-01	FIG. 2

WALL FOOTING



COLUMN FOOTING



NOTE: SEE REPORT FOR FOUNDATION WIDTH AND DEPTH RECOMMENDATION

NO SCALE

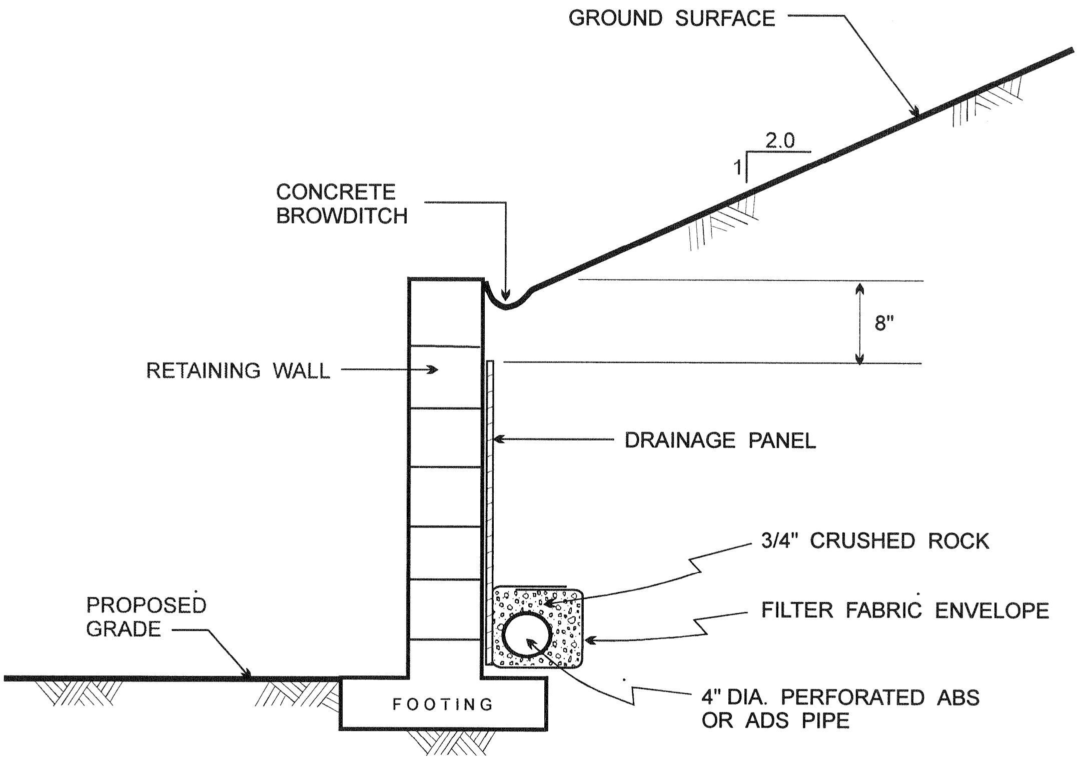
GEOCON WEST, INC.
GEOTECHNICAL ENVIRONMENTAL MATERIALS
41571 CORNING PLACE, SUITE 101, MURRIETA, CA 92562-7065
PHONE 951-304-2300 FAX 951-304-2392

AG

WALL / COLUMN FOOTING DETAIL

MR 56 COMMERCIAL SITE
NWC HIGHWAY 74 AND BRIGGS ROAD
MENIFEE, CALIFORNIA

APRIL, 2017	PROJECT NO. T2765-22-01	FIG. 3



NOTES:

1.....WALL DRAINAGE PANELS SHOULD CONSISTS OF MIRADRAIN 6000 OR EQUIVALENT

2......FILTER FABRIC SHOULD CONSIST OF MIRAFI 140N OR APPROVED EQUIVALENT

3......VOLUME OF CRUSHED ROCK SHOULD BE AT LEAST 1 CUBIC FOOT PER FOOT OF PIPE

4......CONCRETE BROWDITCH RECOMMENDED FOR SLOPE HEIGHTS GREATER THAN 6 FEET

NO SCALE

GEOCON
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WALL DRAINAGE DETAL

MR 56 COMMERCIAL SITE
NWC HIGHWAY 74 AND BRIGGS ROAD
MENIFEE, CALIFORNIA

APRIL, 2017	PROJECT NO. T2765-22-01	FIG. 4

APPENDIX A

APPENDIX A

EXPLORATORY EXCAVATIONS

We performed the field investigation on March 28, 2017. Our subsurface exploration consisted of excavating 4 geotechnical borings and 6 percolation test holes. The borings were excavated with a truck-mounted hollow stem auger drilling machine to a maximum depth of approximately 50 feet below existing ground surface. Representative and relatively undisturbed samples were obtained by driving a 3-inch O. D., California Modified Sampler into the “undisturbed” soil mass with blows from a 140 lb. auto-hammer. The California Modified Sampler was equipped with 1-inch high by 2$^{3}/_{8}$-inch diameter brass sampler rings to facilitate removal and testing. Bulk samples were also obtained.

The soil conditions encountered in the borings were visually examined, classified, and logged in general accordance with the Unified Soil Classification System (USCS). Logs of the borings are presented on Figures A-1 through A-10. The logs depict the soil and geologic conditions encountered and the depth at which samples were obtained. The approximate locations of the borings are indicated the *Geotechnical Map*, Figure 2.

Percolation testing was performed on March 29, 2017 (P-1 through P-6) in accordance with the Riverside County Handbook. The percolation test data is presented on Figures A-11 through A-16.

PROJECT NO. T2765-22-01

BORING B-1

ELEV. (MSL.) 1520 DATE COMPLETED 03/28/2017

EQUIPMENT HOLLOW STEM AUGER BY: A. ORTON

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0	B-1@0-5'			SM	**TOPSOIL** Silty SAND, medium dense, moist, dark brown; fine to medium sand; trace mica; roots near surface			
2	B-1@2.5'			SM	**OLD ALLUVIAL FAN DEPOSITS (Qof)** Silty SAND, medium dense, moist, dark brown; fine to medium sand; trace coarse sand; trace clay; trace calcium carbonate stringers	42	128.6	12.5
4								
6	B-1@5'				-Becomes very dense, brown; fine to coarse sand; trace mica	82/11"	133.3	10.4
8	B-1@7.5'				-Becomes slightly moist; trace calcium carbonate stringers	50/6"	117.7	6.7
10	B-1@10'					69/11"	122.5	11.4
12								
14								
16	B-1@15'				-Trace gravel	50/5"	113.7	8.7
18								
20	B-1@20'				-No observed calcium carbonate stringers	50/6"	129.6	4.3

Total depth 21.0 feet
Groundwater not encountered
Penetration resistance for 140 lb. hammer falling 30" by auto-hammer
Backfilled with cuttings on 03/28/2017

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

T2765-22-01 MR 56 COMMERCIAL.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. T2765-22-01

BORING B-2

ELEV. (MSL.) 1521 DATE COMPLETED 03/28/2017

EQUIPMENT HOLLOW STEM AUGER BY: A. ORTON

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0	B-2@0-5'			SM	**TOPSOIL** Silty SAND, medium dense, slightly moist, dark brown; fine to medium sand; trace coarse sand; trace gravel; roots near surface			
2	B-2@2.5'					23	122.4	10.2
4				SM	**OLD ALLUVIAL FAN DEPOSITS (Qof)** Silty SAND, very dense, moist, brown; fine to coarse sand; trace mica; trace clay			
6	B-2@5'					94/10"	122.9	12.9
8	B-2@7.5'				-Becomes slightly moist	50/6"	122.8	6.1
10	B-2@10'				-Trace gravel	50/6"	137.8	6.4
12								
14								
16	B-2@15'				-Becomes medium dense, moist; fine to medium sand; trace coarse sand; trace mica; trace gravel; calcium carbonate stringers	39	115.4	8.7
18								
20	B-2@20'				-Becomes very dense, fine to coarse sand; trace mica	50/4"	129.2	6.3
22								
24								
26	B-2@25'					50/6"	113.2	6.3
28								

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

T2765-22-01 MR 56 COMMERCIAL.GPJ

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

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

GEOCON

BORING B-2

ELEV. (MSL.) 1521 DATE COMPLETED 03/28/2017

EQUIPMENT HOLLOW STEM AUGER BY: A. ORTON

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
30	B-2@30'			SM	Silty SAND, very dense, slightly moist, grayish brown; fine to coarse sand	50/4"	121.9	8.6
32								
34								
36	B-2@35'				-Becomes brown; trace mica	50/4"	126.7	7.3
38								
40	B-2@40'					50/6"	122.1	7.3
42								
44								
46	B-2@45'				-Trace gravel	50/5"	121.2	7.3
48								
50	B-2@50'					50/4"	128.7	3.8

Total depth 50.3 feet
Groundwater not encountered
Penetration resistance for 140 lb. hammer falling 30" by auto-hammer
Backfilled with cuttings on 03/28/2017

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

T2765-22-01 MR 56 COMMERCIAL.GPJ

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

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

GEOCON

BORING B-3

ELEV. (MSL.) 1523 DATE COMPLETED 03/28/2017

EQUIPMENT HOLLOW STEM AUGER BY: A. ORTON

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0	B-3@0-5'			SM	**TOPSOIL** Silty SAND, medium dense, moist, dark brown; fine to medium sand; trace coarse sand; trace mica; roots near surface			
2	B-3@2.5'			SC	Clayey SAND, medium dense, moist, dark brown; fine to coarse sand; trace gravel	35		
4					**OLD ALLUVIAL FAN DEPOSITS (Qof)** Silty SAND, very dense, moist, brown; fine to coarse sand; large mica flakes			
6	B-3@5'					50/5"	127.8	9.9
8	B-3@7.5'				-Calcium carbonate veins to 1/4" diameter	50/6"	126.4	9.3
10	B-3@10'				-Becomes dense, slightly moist; trace gravel; trace mica	52	132.6	5.6
12								
14								
16	B-3@15'				-Becomes very dense	50/6"	115.3	6.5
18								
20	B-3@20'					50/6"	124.8	8.0

Total depth 21.0 feet
Groundwater not encountered
Penetration resistance for 140 lb. hammer falling 30" by auto-hammer
Backfilled with cuttings on 03/28/2017

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

T2765-22-01 MR 56 COMMERCIAL.GPJ

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

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

GEOCON

BORING B-4

ELEV. (MSL.) **1525** DATE COMPLETED **03/28/2017**

EQUIPMENT **HOLLOW STEM AUGER** BY: **A. ORTON**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0	B-4@0-5'			SM	**TOPSOIL** Silty SAND, medium dense, moist, dark brown; fine to medium sand; trace coarse sand; roots near surface			
2	B-4@2.5'					28	124.0	9.3
4				SM	**OLD ALLUVIAL FAN DEPOSITS (Qof)** Silty SAND, very dense, slightly moist, brown; fine to coarse sand			
6	B-4@5'					50/5"	117.4	7.8
8	B-4@7.5'				-Trace mica	50/5"	120.8	6.3
10	B-4@10'					50/5"	123.5	9.1
12								
14								
16	B-4@15'				-Trace calcium carbonate stringers	50/6"	111.6	8.6
18								
20	B-4@20'					50/5"	110.4	6.1

Total depth 20.9 feet
Groundwater not encountered
Penetration resistance for 140 lb. hammer falling 30" by auto-hammer
Backfilled with cuttings on 03/28/2017

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

T2765-22-01 MR 56 COMMERCIAL.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.

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	BORING P-1 ELEV. (MSL.) 1514 DATE COMPLETED 03/28/2017 EQUIPMENT HOLLOW STEM AUGER BY: A. ORTON	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
	BULK / DRIVE				MATERIAL DESCRIPTION			
0				SM	**TOPSOIL** Silty SAND, medium dense, slightly moist, dark brown; fine to coarse sand; trace mica; roots near surface			
2								
4	P-1@3.5' P-1@4'				-Becomes loose, moist; trace gravel; trace clay; trace mica	9		
					Total depth 5 feet Groundwater not encountered Penetration resistance for 140 lb. hammer falling 30" by auto-hammer Set for percolation testing on 03/28/2017			

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

T2765-22-01 MR 56 COMMERCIAL.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. T2765-22-01

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	BORING P-2 ELEV. (MSL.) 1514 DATE COMPLETED 03/28/2017 EQUIPMENT HOLLOW STEM AUGER BY: A. ORTON	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
	BULK / DRIVE				MATERIAL DESCRIPTION			
0 2				SM	**TOPSOIL** Silty SAND, medium dense, slightly moist, dark brown; fine to coarse sand; trace gravel; trace mica; roots near surface			
4	P-2@3.5' P-2@4'			SM	**OLD ALLUVIAL FAN DEPOSITS (Qof)** Silty SAND, medium dense, moist, dark brown; fine to coarse sand; trace mica	31		

Total depth 5 feet
Groundwater not encountered
Penetration resistance for 140 lb. hammer falling 30" by auto-hammer
Set for percolation testing on 03/28/2017

Figure A-6,
Log of Boring P-2, Page 1 of 1

T2765-22-01 MR 56 COMMERCIAL.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. T2765-22-01

BORING P-3

ELEV. (MSL.) **1519** DATE COMPLETED **03/28/2017**

EQUIPMENT **HOLLOW STEM AUGER** BY: **A. ORTON**

DEPTH IN FEET	SAMPLE NO.	BULK	DRIVE	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0						SM	**TOPSOIL** Silty SAND, medium dense, moist, brown; fine to medium sand; trace coarse sand; roots near surface			
2										
4	P-3@3.5'							41		
	P-3@4'					SM	**OLD ALLUVIAL FAN DEPOSITS (Qof)** Silty SAND, medium dense, moist, brown; fine to medium sand; trace coarse sand; trace gravel; trace clay			

Total depth 5 feet
Groundwater not encountered
Penetration resistance for 140 lb. hammer falling 30" by auto-hammer
Set for percolation testing on 03/28/2017

Figure A-7,
Log of Boring P-3, Page 1 of 1

T2765-22-01 MR 56 COMMERCIAL.GPJ

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

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

GEOCON

BORING P-4

ELEV. (MSL.) 1520 DATE COMPLETED 03/28/2017

EQUIPMENT HOLLOW STEM AUGER BY: A. ORTON

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** Silty SAND, loose, slightly moist, dark brown; fine to medium sand; trace coarse sand; trace mica; roots near surface			
2								
4	P-4@3.5' P-4@4'				-Becomes moist; fine to coarse sand	13		

Total depth 5 feet
Groundwater not encountered
Penetration resistance for 140 lb. hammer falling 30" by auto-hammer
Set for percolation testing on 03/28/2017

Figure A-8,
Log of Boring P-4, Page 1 of 1

T2765-22-01 MR 56 COMMERCIAL.GPJ

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

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

GEOCON

BORING P-5

ELEV. (MSL.) **1522** DATE COMPLETED **03/28/2017**

EQUIPMENT **HOLLOW STEM AUGER** BY: **A. ORTON**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0 – 2				SM	**TOPSOIL** Silty SAND, medium dense, slightly moist, dark brown; fine to coarse sand; roots near surface			
4	P-5@3.5' (drive) P-5@4' (bulk)			ML	**OLD ALLUVIAL FAN DEPOSITS (Qof)** Sandy SILT, hard, slightly moist, brown; trace mica	78		

Total depth 5 feet
Groundwater not encountered
Penetration resistance for 140 lb. hammer falling 30" by auto-hammer
Set for percolation testing on 03/28/2017

Figure A-9,
Log of Boring P-5, Page 1 of 1

T2765-22-01 MR 56 COMMERCIAL.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. T2765-22-01

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	BORING P-6 ELEV. (MSL.) 1525 DATE COMPLETED 03/28/2017 EQUIPMENT HOLLOW STEM AUGER BY: A. ORTON	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
					MATERIAL DESCRIPTION			
0 2				SM	**TOPSOIL** Silty SAND, medium dense, moist, brown; fine to medium sand; trace coarse sand; roots near surface			
4	P-6@3.5' P-6@4'			SM	**OLD ALLUVIAL FAN DEPOSITS (Qof)** Silty SAND, dense, moist, brown; fine to medium sand; trace coarse sand; trace clay; trace mica	70		
					Total depth 5 feet Groundwater not encountered Penetration resistance for 140 lb. hammer falling 30" by auto-hammer Set for percolation testing on 03/28/2017			

Figure A-10,
Log of Boring P-6, Page 1 of 1

T2765-22-01 MR 56 COMMERCIAL.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

PERCOLATION TEST REPORT

Project Name:		MR-56 Commercial			**Project No.:**		T2765-22-01
Test Hole No.:		P-1			**Date Excavated:**		3/28/2017
Length of Test Pipe:			57.1	inches	**Soil Classification:**		SM
Height of Pipe above Ground:			0.0	inches	**Presoak Date:**		3/28/2017
Depth of Test Hole:			57.1	inches	**Perc Test Date:**		3/29/2017
Check for Sandy Soil Criteria Tested by:				SP	**Percolation Tested by:**		SP

Water level measured from bottom of hole

Sandy Soil Criteria Test

Trial No.	Time	Time Interval (min)	Total Elapsed Time (min)	Initial Water Level (in)	Final Water Level (in)	Δ in Water Level (inches)	Percolation Rate (min/inch)
1	7:40 AM 8:05 AM	25	25	45.6	45.1	0.5	52.1
2	8:05 AM 8:30 AM	25	50	45.1	43.9	1.2	20.8

Soil Criteria: Normal

Percolation Test

Reading No.	Time	Time Interval (min)	Total Elapsed Time (min)	Initial Water Head (in)	Final Water Head (in)	Δ in Water Level (inches)	Percolation Rate (min/inch)
1	9:55 AM 10:25 AM	30	30	42.5	42.2	0.2	125.0
2	10:25 AM 10:55 AM	30	60	42.2	41.8	0.5	62.5
3	10:55 AM 11:25 AM	30	90	41.8	41.3	0.5	62.5
4	11:25 AM 11:55 AM	30	120	41.3	40.8	0.5	62.5
5	11:55 AM 12:25 PM	30	150	40.8	40.3	0.5	62.5
6	12:25 PM 12:55 PM	30	180	40.3	40.0	0.4	83.3
7	12:55 PM 1:25 PM	30	210	40.0	39.4	0.6	50.0
8	1:25 PM 1:55 PM	30	240	40.1	39.6	0.5	62.5
9	1:55 PM 2:25 PM	30	270	40.0	39.6	0.4	83.3
10	2:25 PM 2:55 PM	30	300	40.3	39.8	0.5	62.5
11	2:55 PM 3:25 PM	30	330	39.8	39.4	0.5	62.5
12	3:25 PM 3:55 PM	30	360	40.3	39.8	0.5	62.5

Infiltration Rate (in/hr):	0.05	
Radius of test hole (in):	4	**Figure A-11**
Average Head (in):	40.1	

PERCOLATION TEST REPORT

Project Name:		MR-56 Commercial			**Project No.:**		T2765-22-01
Test Hole No.:		P-2			**Date Excavated:**		3/28/2017
Length of Test Pipe:			56.2	inches	**Soil Classification:**		SM
Height of Pipe above Ground:			0.0	inches	**Presoak Date:**		3/28/2017
Depth of Test Hole:			56.2	inches	**Perc Test Date:**		3/29/2017
Check for Sandy Soil Criteria Tested by:				SP	**Percolation Tested by:**		SP

Water level measured from bottom of hole

Sandy Soil Criteria Test

Trial No.	Time	Time Interval (min)	Total Elapsed Time (min)	Initial Water Level (in)	Final Water Level (in)	Δ in Water Level (inches)	Percolation Rate (min/inch)
1	7:45 AM 8:10 AM	25	25	43.2	43.0	0.2	104.2
2	8:10 AM 8:35 AM	25	50	43.0	42.8	0.1	208.3

Soil Criteria: Normal

Percolation Test

Reading No.	Time	Time Interval (min)	Total Elapsed Time (min)	Initial Water Head (in)	Final Water Head (in)	Δ in Water Level (inches)	Percolation Rate (min/inch)
1	10:00 AM 10:30 AM	30	30	42.6	42.5	0.1	250.0
2	10:30 AM 11:00 AM	30	60	42.5	42.4	0.1	250.0
3	11:00 AM 11:30 AM	30	90	42.4	42.2	0.1	250.0
4	11:30 AM 12:00 PM	30	120	42.2	42.0	0.2	125.0
5	12:00 PM 12:30 PM	30	150	42.0	41.9	0.1	250.0
6	12:30 PM 1:00 PM	30	180	41.9	41.9	0.0	0
7	1:00 PM 1:30 PM	30	210	41.9	41.8	0.1	250.0
8	1:30 PM 2:00 PM	30	240	41.8	41.6	0.1	250.0
9	2:00 PM 2:30 PM	30	270	41.6	41.4	0.2	125.0
10	2:30 PM 3:00 PM	30	300	41.4	41.3	0.1	250.0
11	3:00 PM 3:30 PM	30	330	41.4	41.3	0.1	250.0
12	3:30 PM 4:00 PM	30	360	41.3	41.2	0.1	250.0

Infiltration Rate (in/hr):	0.01
Radius of test hole (in):	4
Average Head (in):	41.2

Figure A-12

PERCOLATION TEST REPORT							
Project Name:		MR-56 Commercial			**Project No.:**		T2765-22-01
Test Hole No.:		P-3			**Date Excavated:**		3/28/2017
Length of Test Pipe:			56.2	inches	**Soil Classification:**		SM
Height of Pipe above Ground:			0.0	inches	**Presoak Date:**		3/28/2017
Depth of Test Hole:			56.2	inches	**Perc Test Date:**		3/29/2017
Check for Sandy Soil Criteria Tested by:				SP	**Percolation Tested by:**		SP
Water level measured from bottom of hole							
Sandy Soil Criteria Test							
Trial No.	**Time**	**Time Interval (min)**	**Total Elapsed Time (min)**	**Initial Water Level (in)**	**Final Water Level (in)**	**∆ in Water Level (inches)**	**Percolation Rate (min/inch)**
1	7:50 AM 8:15 AM	25	25	41.0	40.0	1.1	23.1
2	8:15 AM 8:40 AM	25	50	40.0	39.2	0.7	34.7
		Soil Criteria: Normal					
		Percolation Test					
Reading No.	**Time**	**Time Interval (min)**	**Total Elapsed Time (min)**	**Initial Water Head (in)**	**Final Water Head (in)**	**∆ in Water Level (inches)**	**Percolation Rate (min/inch)**
1	10:05 AM 10:35 AM	30	30	40.4	39.6	0.8	35.7
2	10:35 AM 11:05 AM	30	60	39.6	38.9	0.7	41.7
3	11:05 AM 11:35 AM	30	90	39.8	39.1	0.7	41.7
4	11:35 AM 12:05 PM	30	120	40.4	39.6	0.8	35.7
5	12:05 PM 12:35 PM	30	150	39.6	38.9	0.7	41.7
6	12:35 PM 1:05 PM	30	180	40.8	40.1	0.7	41.7
7	1:05 PM 1:35 PM	30	210	40.1	39.4	0.7	41.7
8	1:35 PM 2:05 PM	30	240	40.0	39.5	0.5	62.5
9	2:05 PM 2:35 PM	30	270	40.1	39.5	0.6	50.0
10	2:35 PM 3:05 PM	30	300	40.6	40.0	0.6	50.0
11	3:05 PM 3:35 PM	30	330	40.0	39.4	0.6	50.0
12	3:35 PM 4:05 PM	30	360	40.7	40.1	0.6	50.0
Infiltration Rate (in/hr):			0.06				
Radius of test hole (in):			4				**Figure A-13**
Average Head (in):			40.4				

PERCOLATION TEST REPORT

Project Name:		MR-56 Commercial			**Project No.:**		T2765-22-01
Test Hole No.:		P-4			**Date Excavated:**		3/28/2017
Length of Test Pipe:			60.0	inches	**Soil Classification:**		SM
Height of Pipe above Ground:			0.7	inches	**Presoak Date:**		3/28/2017
Depth of Test Hole:			59.3	inches	**Perc Test Date:**		3/29/2017
Check for Sandy Soil Criteria Tested by:				SP	**Percolation Tested by:**		SP

Water level measured from bottom of hole

Sandy Soil Criteria Test

Trial No.	Time	Time Interval (min)	Total Elapsed Time (min)	Initial Water Level (in)	Final Water Level (in)	Δ in Water Level (inches)	Percolation Rate (min/inch)
1	7:55 AM 8:20 AM	25	25	46.7	45.0	1.7	14.9
2	8:20 AM 8:45 AM	25	50	45.0	44.6	0.4	69.4

Soil Criteria: Normal

Percolation Test

Reading No.	Time	Time Interval (min)	Total Elapsed Time (min)	Initial Water Head (in)	Final Water Head (in)	Δ in Water Level (inches)	Percolation Rate (min/inch)
1	10:10 AM 10:40 AM	30	30	43.8	43.6	0.2	125.0
2	10:40 AM 11:10 AM	30	60	43.6	43.4	0.1	250.0
3	11:10 AM 11:40 AM	30	90	43.4	43.2	0.2	125.0
4	11:40 AM 12:10 PM	30	120	43.2	43.0	0.2	125.0
5	12:10 PM 12:40 PM	30	150	43.0	42.8	0.1	250.0
6	12:40 PM 1:10 PM	30	180	42.8	42.6	0.2	125.0
7	1:10 PM 1:40 PM	30	210	42.6	42.5	0.1	250.0
8	1:40 PM 2:10 PM	30	240	42.5	42.4	0.1	250.0
9	2:10 PM 2:40 PM	30	270	42.4	42.1	0.2	125.0
10	2:40 PM 3:10 PM	30	300	42.1	42.0	0.1	250.0
11	3:10 PM 3:40 PM	30	330	42.0	41.8	0.2	125.0
12	3:40 PM 4:10 PM	30	360	41.8	41.6	0.1	250.0

Infiltration Rate (in/hr):	0.01	
Radius of test hole (in):	4	**Figure A-14**
Average Head (in):	41.7	

PERCOLATION TEST REPORT

Project Name:		MR-56 Commercial			**Project No.:**		T2765-22-01
Test Hole No.:		P-5			**Date Excavated:**		3/28/2017
Length of Test Pipe:			59.4	inches	**Soil Classification:**		SM
Height of Pipe above Ground:			4.2	inches	**Presoak Date:**		3/28/2017
Depth of Test Hole:			55.2	inches	**Perc Test Date:**		3/29/2017
Check for Sandy Soil Criteria Tested by:				SP	**Percolation Tested by:**		SP

Water level measured from bottom of hole

Sandy Soil Criteria Test

Trial No.	Time	Time Interval (min)	Total Elapsed Time (min)	Initial Water Level (in)	Final Water Level (in)	Δ in Water Level (inches)	Percolation Rate (min/inch)
1	8:55 AM 9:20 AM	25	25	43.8	43.4	0.4	69.4
2	9:20 AM 9:45 AM	25	50	43.4	43.0	0.5	52.1

Soil Criteria: Normal

Percolation Test

Reading No.	Time	Time Interval (min)	Total Elapsed Time (min)	Initial Water Head (in)	Final Water Head (in)	Δ in Water Level (inches)	Percolation Rate (min/inch)
1	10:15 AM 10:45 AM	30	30	43.7	43.2	0.5	62.5
2	10:45 AM 11:15 AM	30	60	43.9	43.6	0.4	83.3
3	11:15 AM 11:45 AM	30	90	43.8	43.4	0.4	83.3
4	11:45 AM 12:15 PM	30	120	44.0	43.7	0.4	83.3
5	12:15 PM 12:45 PM	30	150	44.5	44.0	0.5	62.5
6	12:45 PM 1:15 PM	30	180	44.0	43.7	0.4	83.3
7	1:15 PM 1:45 PM	30	210	43.9	43.6	0.4	83.3
8	1:45 PM 2:15 PM	30	240	43.9	43.6	0.4	83.3
9	2:15 PM 2:45 PM	30	270	43.9	43.6	0.4	83.3
10	2:45 PM 3:15 PM	30	300	44.0	43.7	0.4	83.3
11	3:15 PM 3:45 PM	30	330	44.4	44.0	0.4	83.3
12	3:45 PM 4:15 PM	30	360	44.0	43.7	0.4	83.3

Infiltration Rate (in/hr):	0.03	
Radius of test hole (in):	4	**Figure A-15**
Average Head (in):	43.9	

PERCOLATION TEST REPORT

Project Name:	MR-56 Commercial	**Project No.:**	T2765-22-01
Test Hole No.:	P-6	**Date Excavated:**	3/28/2017
Length of Test Pipe:	59.4 inches	**Soil Classification:**	SM
Height of Pipe above Ground:	3.0 inches	**Presoak Date:**	3/28/2017
Depth of Test Hole:	56.4 inches	**Perc Test Date:**	3/29/2017
Check for Sandy Soil Criteria Tested by:	SP	**Percolation Tested by:**	SP

Water level measured from bottom of hole

Sandy Soil Criteria Test

Trial No.	Time	Time Interval (min)	Total Elapsed Time (min)	Initial Water Level (in)	Final Water Level (in)	Δ in Water Level (inches)	Percolation Rate (min/inch)
1	9:00 AM 9:25 AM	25	25	46.7	45.8	0.8	29.8
2	9:25 AM 9:50 AM	25	50	45.8	45.0	0.8	29.8

Soil Criteria: Normal

Percolation Test

Reading No.	Time	Time Interval (min)	Total Elapsed Time (min)	Initial Water Head (in)	Final Water Head (in)	Δ in Water Level (inches)	Percolation Rate (min/inch)
1	10:20 AM 10:50 AM	30	30	44.5	43.9	0.6	50.0
2	10:50 AM 11:20 AM	30	60	43.9	43.4	0.5	62.5
3	11:20 AM 11:50 AM	30	90	43.4	43.0	0.5	62.5
4	11:50 AM 12:20 PM	30	120	43.0	42.5	0.5	62.5
5	12:20 PM 12:50 PM	30	150	43.1	42.7	0.4	83.3
6	12:50 PM 1:20 PM	30	180	42.7	42.4	0.4	83.3
7	1:20 PM 1:50 PM	30	210	43.0	42.6	0.4	83.3
8	1:50 PM 2:20 PM	30	240	42.6	42.2	0.4	83.3
9	2:20 PM 2:50 PM	30	270	43.2	42.8	0.4	83.3
10	2:50 PM 3:20 PM	30	300	43.1	42.7	0.4	83.3
11	3:20 PM 3:50 PM	30	330	42.7	42.4	0.4	83.3
12	3:50 PM 4:20 PM	30	360	43.3	43.0	0.4	83.3

Infiltration Rate (in/hr):	0.03
Radius of test hole (in):	4
Average Head (in):	43.1

Figure A-16

APPENDIX B

APPENDIX B

LABORATORY TESTING

We performed laboratory tests in accordance with current generally accepted test methods of ASTM International (ASTM) or other suggested procedures. We analyzed selected soil samples for maximum dry density and optimum moisture content, consolidation, expansion index, corrosivity, Atterberg limits, grain size distribution, R-Value, direct shear strength, and in-situ dry density and moisture content. The results of the laboratory tests are presented on Figures B-1 through B-7.

SUMMARY OF LABORATORY MAXIMUM DRY DENSITY AND OPTIMUM MOISTURE CONTENT TEST RESULTS ASTM D1557

Sample No.	Description	Maximum Dry Density (pcf)	Optimum Moisture Content (% of dry wt.)
B-3 @ 0-5’	Silty, Clayey SAND, dark brown	133.0	8.0

SUMMARY OF LABORATORY EXPANSION INDEX TEST RESULTS ASTM D4829

Sample No.	Moisture Content		Before Test Dry Density (pcf)	Expansion Index
	Before Test (%)	After Test (%)		
B-1 @ 0-5’	8.0	16.4	117.1	40

SUMMARY OF CORROSIVITY TEST RESULTS

Sample No.	Chloride Content (ppm)	Sulfate Content (%)	pH	Resistivity (ohm-centimeter)
B-4 @ 0-5’	270	0.085	7.0	830

Chloride content determined by California Test 422.
Water-soluble sulfate determined by California Test 417.
Resistivity and pH determined by Caltrans Test 643.

SUMMARY OF LABORATORY R-VALUE TEST RESULTS
ASTM D2844

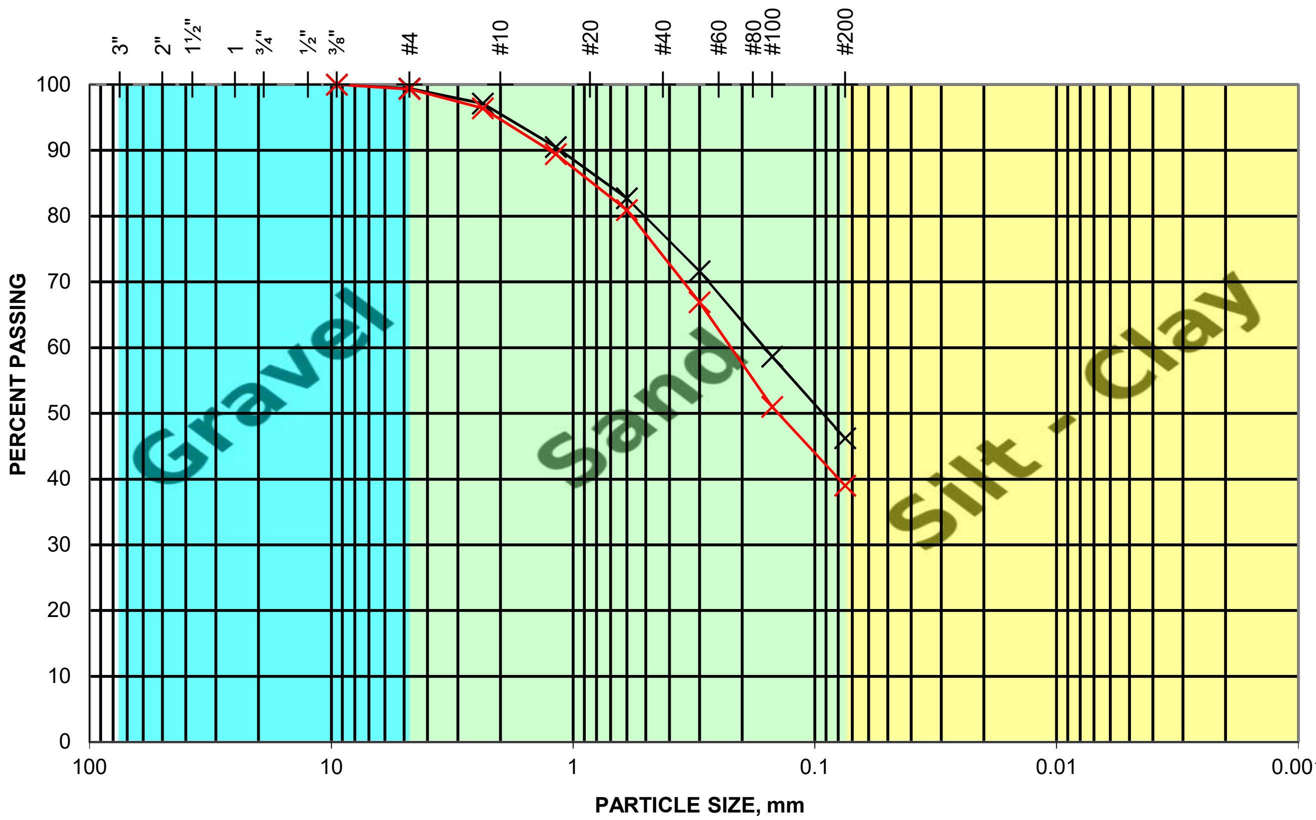
Sample No.	R-Value
B-2 @ 0-5'	14

SUMMARY OF ONE-DIMENSIONAL CONSOLIDATION (COLLAPSE) TESTS
ASTM D2435

Sample No.	In-situ Dry Density (pcf)	Moisture Content Before Test (%)	Final Moisture Content (%)	Axial Load with Water Added (psf)	Percent Collapse
B-2 @ 2.5'	122.4	10.2	11.4	2,000	0.1
B-4 @ 2.5'	124.0	9.3	11.9	2,000	0.4

SUMMARY OF ATTERBERG LIMIT TEST RESULTS
ASTM D4318

Sample No.	Liquid Limit	Plastic Limit	Plasticity Index	USCS
B-3 @ 2.5'	25	12	13	SC



SAMPLE ID	SAMPLE DESCRIPTION
P-1 @ 4-5'	SM - Silty SAND, dark brown
P-3 @ 4-5'	SM - Silty SAND, brown

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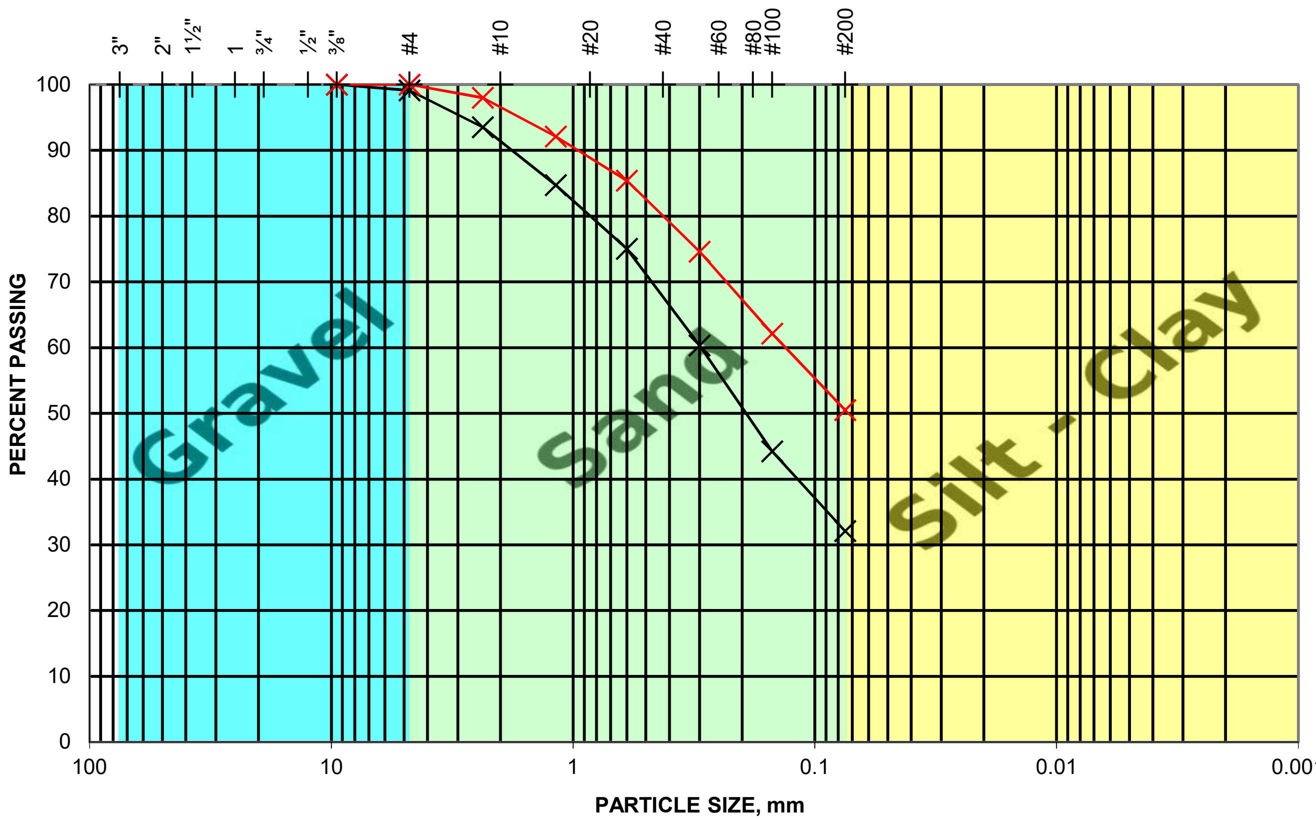
AG

GRAIN SIZE DISTRIBUTION

MR 56 COMMERCIAL SITE
NWC HIGHWAY 74 AND BRIGGS ROAD
MENIFEE, CALIFORNIA

APRIL, 2017	PROJECT NO. T2765-22-01	FIG B-3



SAMPLE ID	SAMPLE DESCRIPTION
P-4 @ 4-5'	SM - Silty SAND, dark brown
P-5 @ 4-5'	ML - Sandy SILT, brown

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GRAIN SIZE DISTRIBUTION

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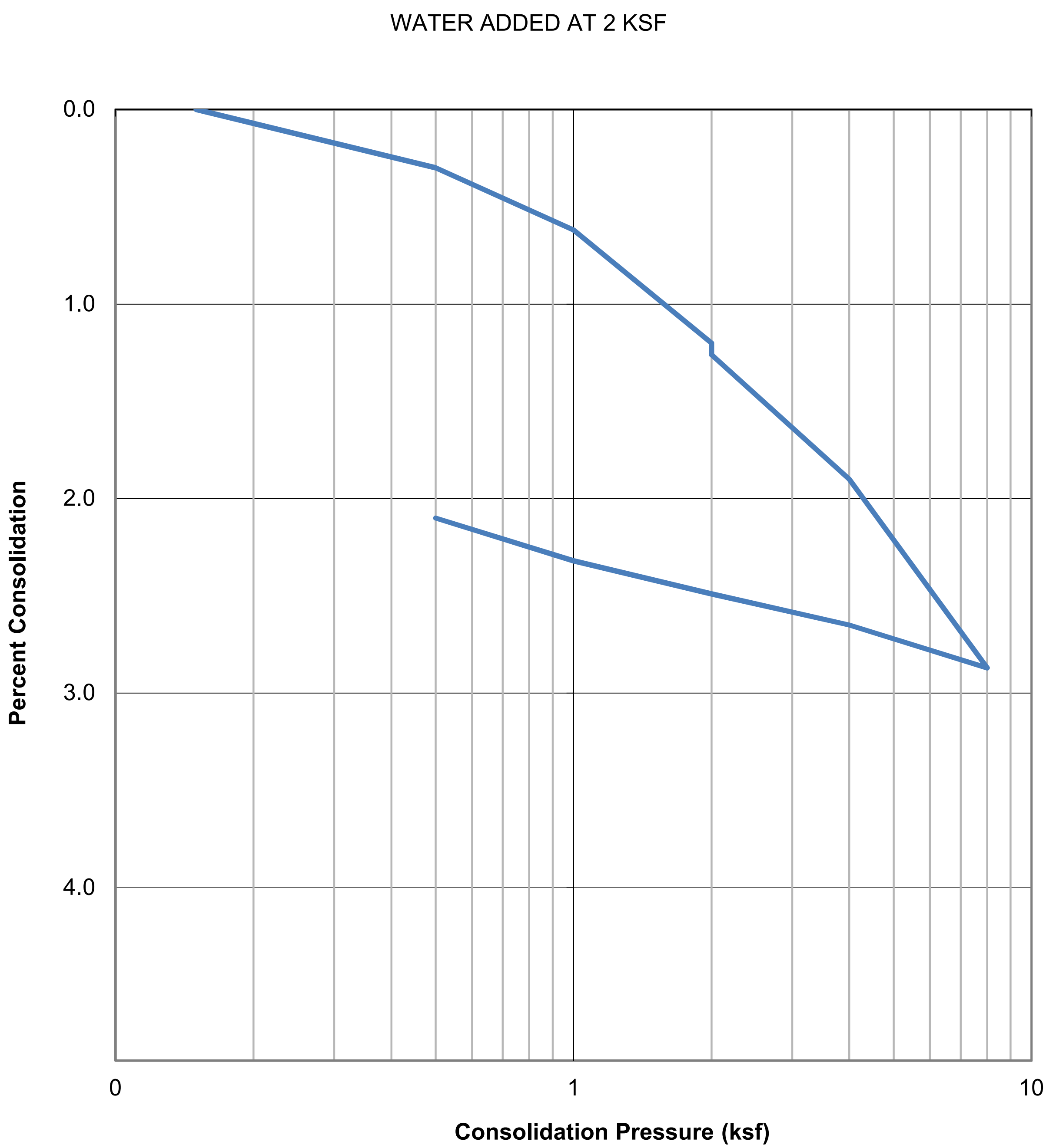
APRIL, 2017	PROJECT NO. T2765-22-01	FIG B-4



SAMPLE ID	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B-2 @ 2.5'	SM	122.4	10.2	11.4

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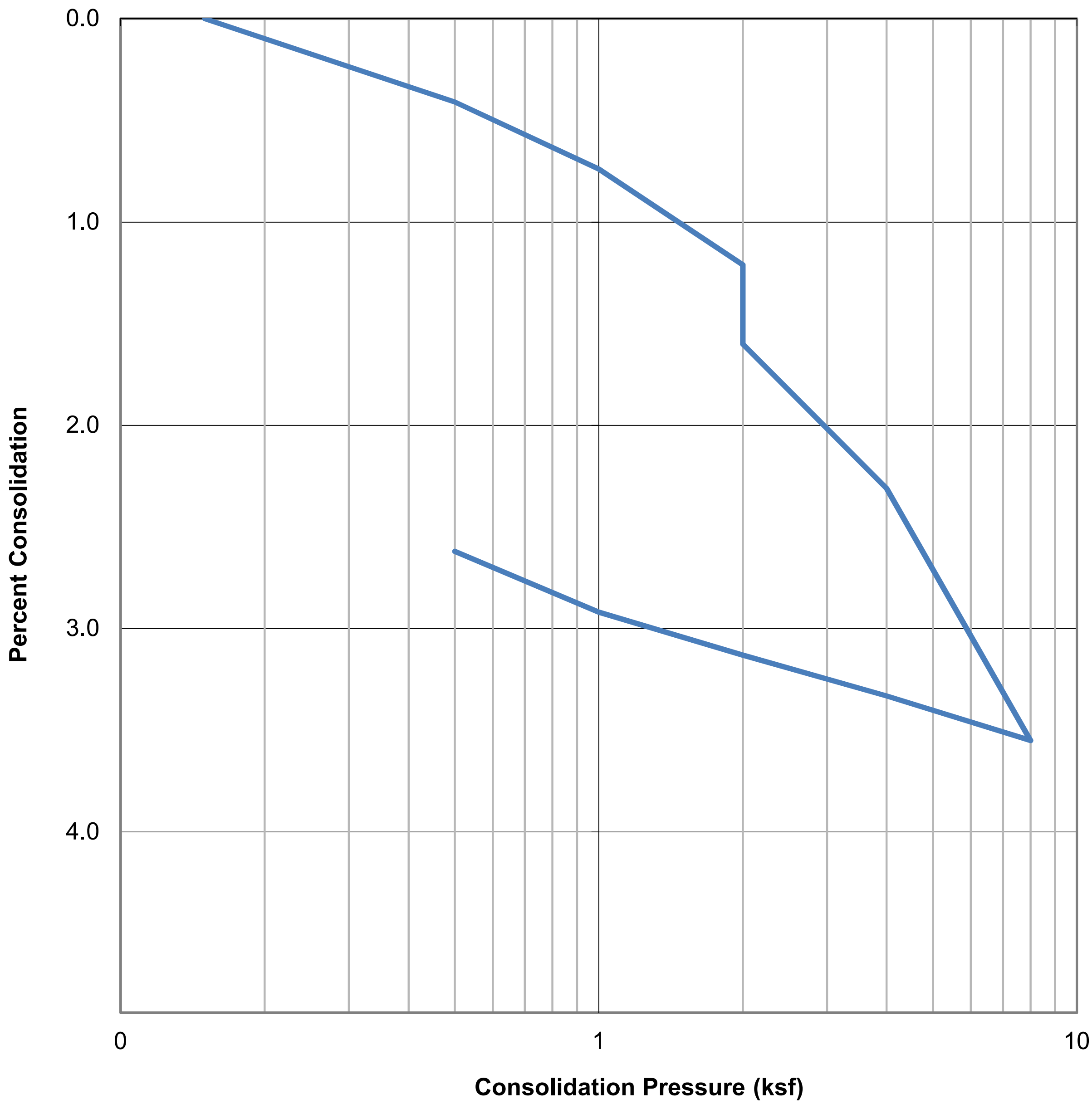
CONSOLIDATION TEST RESULTS

MR 56 COMMERCIAL SITE
NWC HIGHWAY 74 AND BRIGGS ROAD
MENIFEE, CALIFORNIA

APRIL, 2017	PROJECT NO. T2765-22-01	FIG B-5

WATER ADDED AT 2 KSF



SAMPLE ID	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B-4 @ 2.5'	SM	124.0	9.4	11.9

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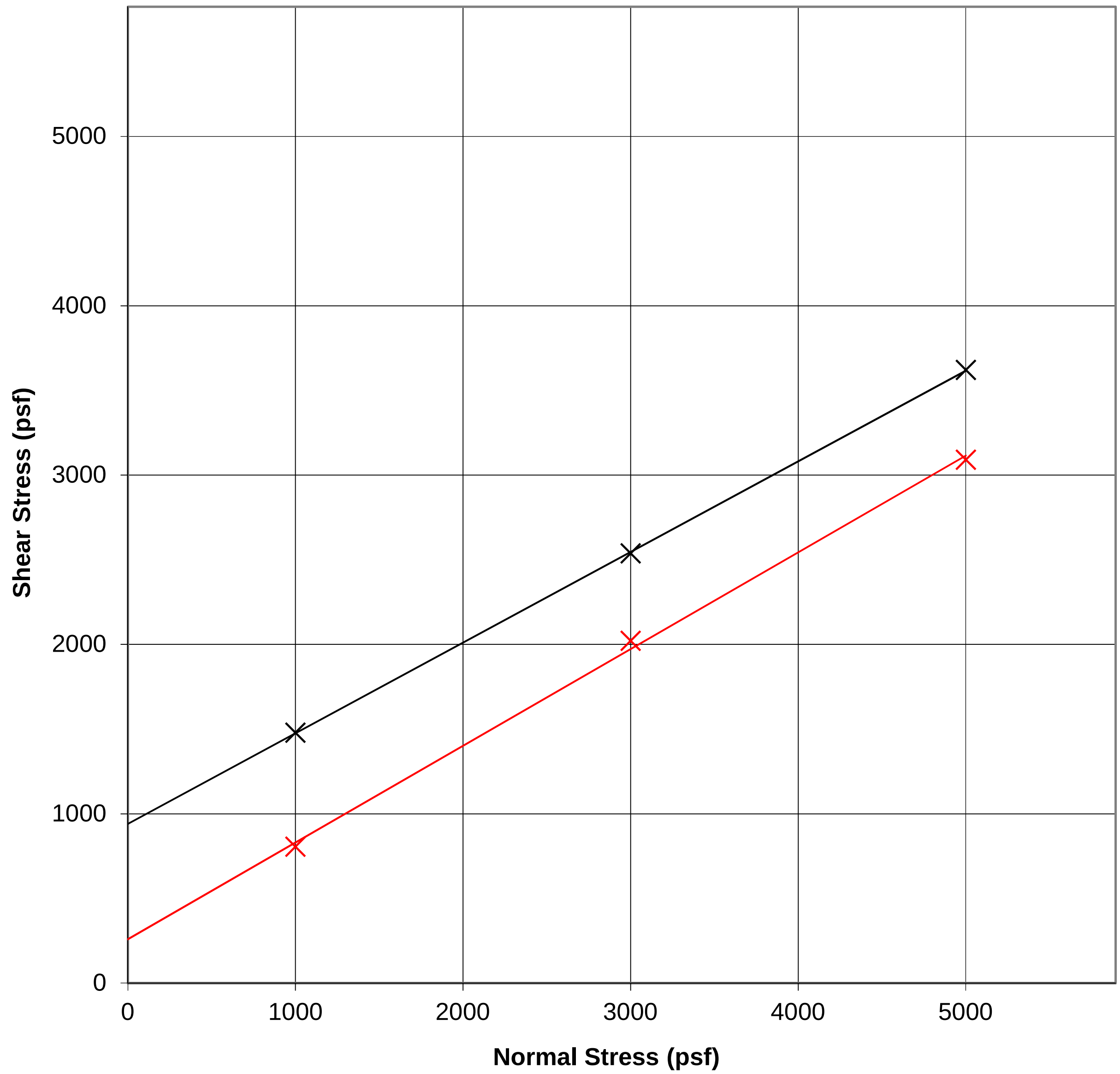
CONSOLIDATION TEST RESULTS

MR 56 COMMERCIAL SITE
NWC HIGHWAY 74 AND BRIGGS ROAD
MENIFEE, CALIFORNIA

APRIL, 2017	PROJECT NO. T2765-22-01	FIG B-6



SAMPLE ID	SOIL TYPE	INITIAL DRY DENSITY (pcf)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)	C (psf)	ϕ (deg)
B-2 @ 5'	SM	122.9	12.9	15.7	940	28
*B-3 @ 0-5'	SC-SM	120.1	7.7	12.9	260	30

*Sample remolded to approximately 90% of the test maximum dry density at optimum moisture content.

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DIRECT SHEAR TEST RESULTS

MR 56 COMMERCIAL SITE
NWC HIGHWAY 74 AND BRIGGS ROAD
MENIFEE, CALIFORNIA

APRIL, 2017 | PROJECT NO. T2765-22-01 | FIG B-7

APPENDIX C

APPENDIX C

RECOMMENDED GRADING SPECIFICATIONS

FOR

MR 56 COMMERCIAL SITE
NWC HIGHWAY 74 AND BRIGGS ROAD
MENIFEE, CALIFORNIA

PROJECT NO. T2765-22-01

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

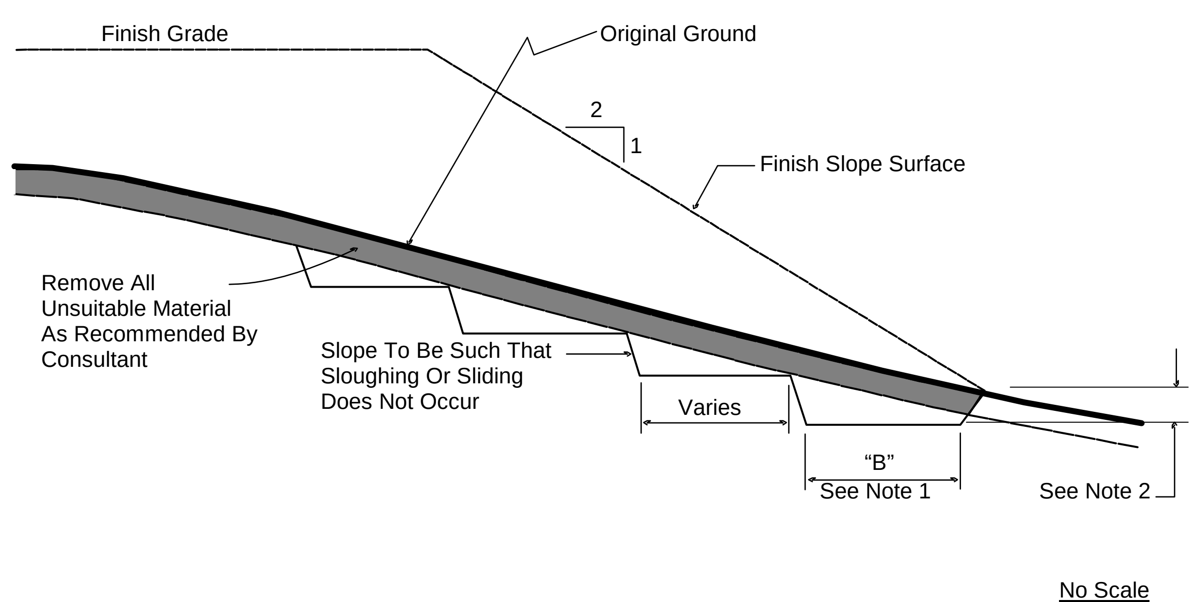
4.1 Areas to be excavated and filled shall be cleared and grubbed. Clearing shall consist of complete removal above the ground surface of trees, stumps, brush, vegetation, man-made structures, and similar debris. Grubbing shall consist of removal of stumps, roots, buried logs and other unsuitable material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

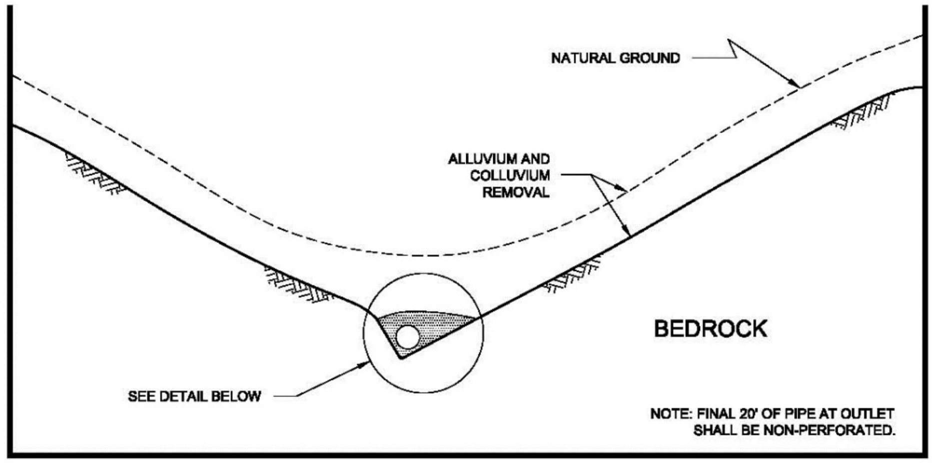
6.3.6 To reduce the potential for “piping” of fines into the *rock* fill from overlying *soil* fill material, a 2-foot layer of graded filter material shall be placed above the uppermost lift of *rock* fill. The need to place graded filter material below the *rock* should be determined by the Consultant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

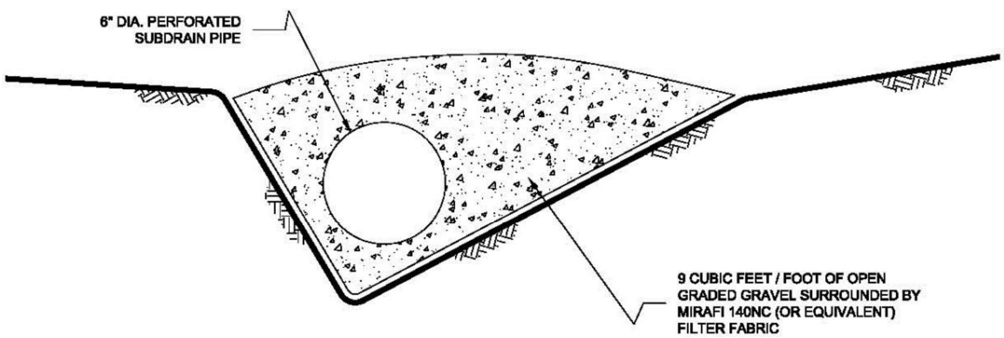
6.3.7 *Rock* fill placement should be continuously observed during placement by the Consultant.

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:

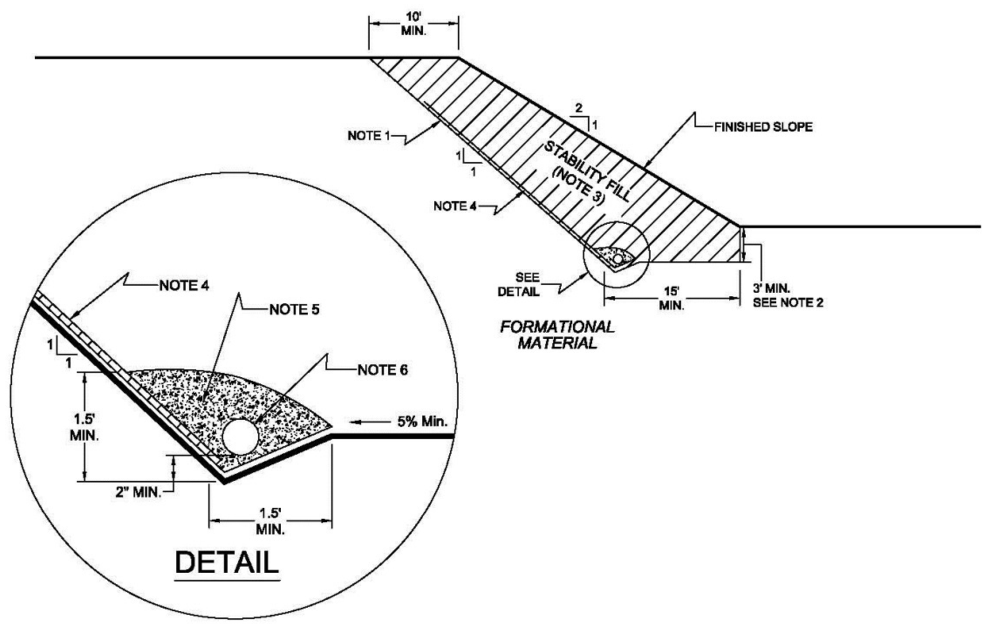
1......6-INCH DIAMETER, SCHEDULE 80 PVC PERFORATED PIPE FOR FILLS IN EXCESS OF 100-FEET IN DEPTH OR A PIPE LENGTH OF LONGER THAN 500 FEET.

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

NO SCALE

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

TYPICAL STABILITY FILL DETAIL



NOTES:

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

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

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

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

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

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

NO SCALE

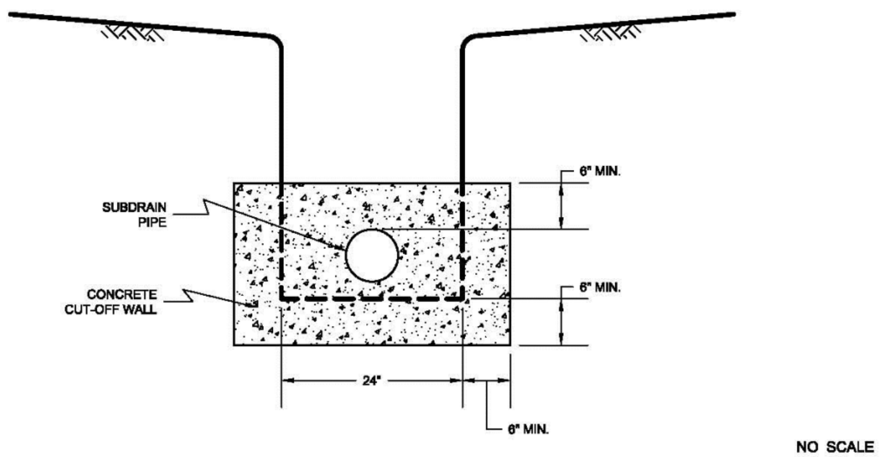
7.3 The actual subdrain locations will be evaluated in the field during the remedial grading operations. Additional drains may be necessary depending on the conditions observed and the requirements of the local regulatory agencies. Appropriate subdrain outlets should be evaluated prior to finalizing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

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

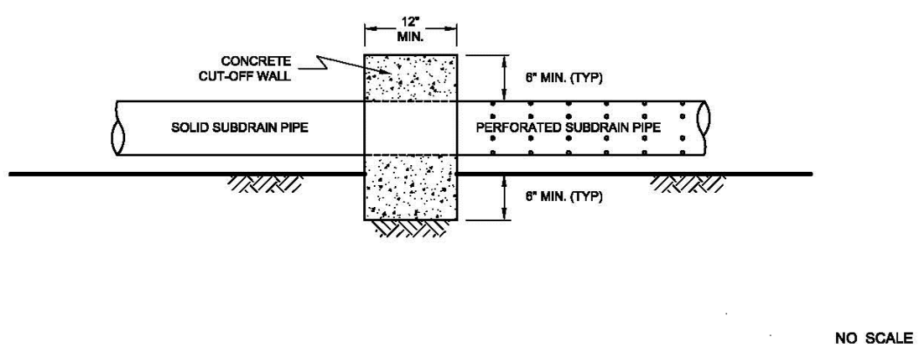
7.5 Prior to outletting, the final 20-foot segment of a subdrain that will not be extended during future development should consist of non-perforated drainpipe. At the non-perforated/ perforated interface, a seepage cutoff wall should be constructed on the downslope side of the pipe.

TYPICAL CUT OFF WALL DETAIL

FRONT VIEW



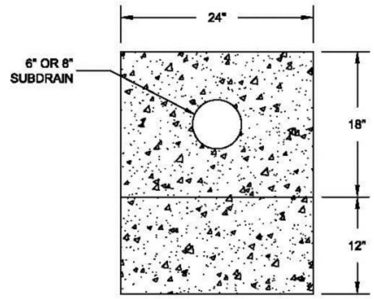
SIDE VIEW



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

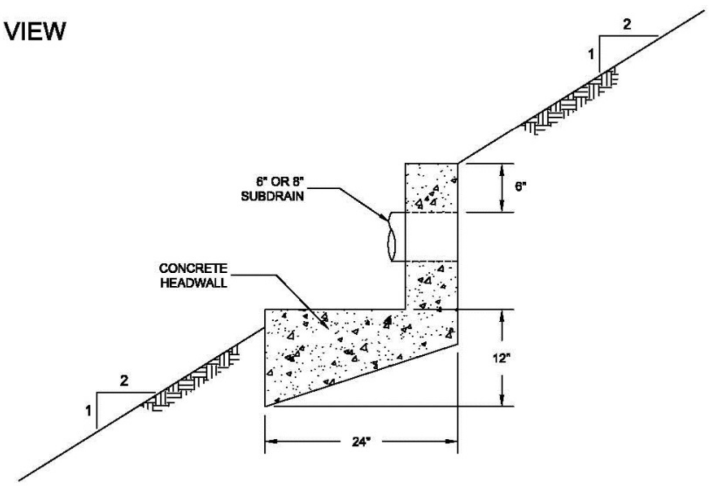
TYPICAL HEADWALL DETAIL

FRONT VIEW



NO SCALE

SIDE VIEW



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

NO SCALE

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.