GEOCON
WEST, INC.

GEOTECHNICAL
ENVIRONMENTAL
MATERIALS

UPDATED GEOTECHNICAL INVESTIGATION

KTM DEVELOPMENT
NEC OF HWY 79 AND BOREL ROAD
FRENCH VALLEY AREA
RIVERSIDE COUNTY, CALIFORNIA

PREPARED FOR

KTM NORTH AMERICA, INC.
MURRIETA, CALIFORNIA

AUGUST 18, 2017
PROJECT NO. T2788-22-01

GEOCON
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GEOTECHNICAL ■ ENVIRONMENTAL ■ MATERIALS

Project No. T2788-22-01
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KTM North America, Inc.
38429 Innovation Court
Murrieta, California 92563

Attention: Ms. Cheryl Webb

Subject: UPDATED GEOTECHNICAL INVESTIGATION
KTM DEVELOPMENT
NEC HWY 79 AND BOREL ROAD
FRENCH VALLEY AREA
RIVERSIDE COUNTY, CALIFORNIA

Dear Ms. Webb:

In accordance with your authorization of Proposal IE-1910 dated April 26, 2017, Geocon West, Inc. (Geocon) herein submits the results of our updated geotechnical investigation for the proposed KTM development to be located on approximately 53 acres immediately west of the French Valley Airport northeast of the intersection of Borel Road and Highway 79 in the French Valley area of Riverside County, 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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UPDATED GEOTECHNICAL INVESTIGATION

1. PURPOSE AND SCOPE

This report presents the results of our updated geotechnical investigation for the proposed KTM development proposed for approximately 53 acres immediately northeast of Borel Road and Highway 79 in the French Valley area of Riverside County, California (see *Vicinity Map*, Figure 1). The purpose of the updated investigation was to evaluate subsurface soil and geologic conditions underlying the area of proposed construction and, based on conditions encountered, to provide preliminary conclusions and recommendations pertaining to the geotechnical aspects of design and construction.

Geocon performed a geotechnical investigation on the site in 2007 which included the excavation of 13 test pits, four seismic refraction traverses, and laboratory testing. At that time, a light industrial/commercial development was being considered for the site. The locations of the field work, geotechnical logs, seismic refraction report, and laboratory test results are included herein for ease of reference. The previous geotechnical work is depicted on the *Geotechnical Map* (see Figure 2).

The scope of our recent work included a site reconnaissance, aerial photograph review, literature review, infiltration testing, laboratory testing, engineering analyses, and the preparation of this report. The approximate locations of the infiltration tests (IT) are presented on the *Geotechnical Map* (see Figure 2). *Appendix A* presents a discussion of the field investigation and logs of the excavations. The pertinent logs from the previous investigation and the results of the seismic refraction survey are also included in *Appendix A*.

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 pertinent laboratory testing from the previous investigation is also included in *Appendix B.*

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 approximately 53-acre site is located immediately northeast of Borel Road and Highway 79 (Winchester Road) in the French Valley area of Riverside County, California. The site is bounded on the south by Borel Road, the west by Highway 79, the east by French Valley Airport, and the north by Sparkman Way. The site descends to the south and west with a high elevation of approximately 1335 in the northern area to 1320 along the southern boundary and 1315 within a drainage at the southwestern area of the site. Fill has been placed within the central portion of the site resulting in two level pads. The site is currently undeveloped and is utilized for agriculture.

Based on the aerial photograph review, the site was undeveloped and plowed prior to 1995. The fill was placed on the site between 1995 and 1997 and appears to have been derived from the French Valley Airport north of Sparkman Way. Since 1997 the site has remained similar to today's conditions with two areas of undocumented fill north and south of a central channel with natural topography in the far northern and southern portions of the site.

Grading plans were not available at the time of this report. Based upon current site topography and surrounding grades we anticipate site grades to be changed from 5 to 15 feet to provide level building pads for the proposed development. We anticipate that grading will incorporate a bedrock cut slope up to approximately 15 feet in height descending to the site from the southern boundary. Fill slopes may also be created during grading and are anticipated to be 15 feet or less in height.

The details of site development are not known at this time; however, we understand that a KTM headquarters building will be constructed on a portion of the site. We anticipate that additional commercial or light industrial development and possibly a moto-cross track will also be constructed.

We anticipate that the buildings at the site will consist of one or more concrete tilt-up structures with spread footing foundations and concrete slab-on-grade floors. We anticipate the future buildings would be single-story, approximately 20-foot-high structures with metal roofs. It is anticipated that column loads for these structures will be up to 100 kips and wall loads will be up to 8 kips per linear foot. Preliminary geotechnical recommendations for design of these structures are provided herein. This report and preliminary recommendations should be reviewed once plans for the industrial development are available and additional geotechnical work may be necessary at that time.

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

3. GEOLOGIC SETTING

The site is located within the Perris Block of the Peninsular Ranges Geomorphic Province. The Perris Block is characterized by granitic highlands which display three elevated erosional surfaces surrounded by alluviated valleys. The Peninsular Ranges are bound by the Transverse Ranges (San Gabrielle 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. Overall the Province is characterized by Cretaceous-age granitic rock and a lesser amount of Mesozoic-age metamorphic rock overlain by terrestrial and marine sediments. Faulting within the province is typically northwest trending and includes the San Andreas, San Jacinto, Elsinore, and Newport-Inglewood faults. Locally, the site is within the northern portion of the Temecula Valley, north of the intersection of the Wildomar and Murrieta Hot Springs faults. Localized faulting is mapped as separating the Cretaceous-age granitic rocks on the northeast from the Quaternary-age Pauba fanglomerate on the southwest. Undocumented fill, alluvium, colluvium and older alluvium overlie granitic bedrock in the vicinity of the site. The regional geology is depicted on Figure 3, *Regional Geologic Map*.

4. GEOLOGIC MATERIALS

4.1 General

Site geologic materials encountered consist of undocumented artificial fill, younger alluvium, colluvium and older alluvium over Cretaceous-age gabbroic bedrock (Kennedy & Morton, 2003). The descriptions of the soil and geologic conditions are shown on the excavation logs located in *Appendix A* and described herein in order of increasing age.

4.2 Undocumented Artificial Fill (afu)

Undocumented artificial fill is located within a majority of the site with exception of approximately the southern 25 percent of the property. Based on Google images, the fill was placed prior to 1997 and appears to have been derived from the airport northeast of Sparkman Way. No geotechnical documentation was provided that would indicate this fill was placed under observation and testing by a geotechnical firm, therefore, it is considered undocumented. The fill soils consist of layers of silty to clayey sands, clays, and silts which were generally brown, loose to dense, dry to moist, and contained some porosity. We found fill depths north of the channel to be 5 to 14 feet and south of the channel to be 5 to 12 feet.

4.3 Younger Alluvium (Qal)

Younger alluvium was encountered within a drainage in the southwestern portion of the site to depths of 5.5 feet. The soil consists of soft to loose clays and silty sands which were wet during our field exploration in 2007.

4.4 Colluvium (Qcol)

Colluvium was encountered above the bedrock in approximately the southern 25 percent of the site. The soil consists of brown clayey sand to clay which were medium dense to stiff and slightly moist in 2007. Depths of colluvium were found to be 3 to 5.5 feet.

4.5 Older Alluvium (Qova)

Older alluvium is mapped across the site (Kennedy & Morton, 2003) and was encountered beneath the undocumented fill in the central and northern portions of the site. The soil consisted of red-brown silty sand and grey clay which was moist, well indurated and difficult to dig. Carbonate was observed on ped facies indicating a pre-Holocene age for the unit.

4.6 Cretaceous-age Gabbroic Bedrock (Kgb)

Cretaceous-age gabbroic bedrock underlies the site at depth and is present within 3 to 5.5 feet of the surface in approximately the southern 25 percent of the site. The unit was excavatable with a backhoe during Geocon's 2007 investigation. Seismic refraction traverses indicate the unit is rippable to depths of 20 feet below existing ground surface.

5. GEOLOGIC STRUCTURE

The geologic structure consists of generally massive granitic bedrock underlying the site with horizontal to gently dipping colluvial and alluvial soils. No jointing or foliation attitudes are depicted on the regional geologic maps in the vicinity of the site.

6. GROUNDWATER

Groundwater was not encountered during this or the previous investigation in 2007 in our explorations conducted to a maximum depth of 15 feet below grade. California Department of Water Resources well data indicates groundwater has been measured at depths of about 45 feet below the ground surface at elevation 1280 to 1285 in wells less than ¼ mile northwest of the site (Wells 07S03W12H001S and 07S03W12J002S), and groundwater was measured at a depth of 8 feet below the ground surface in a well at the elevation of 1288 approximately ½ mile east of the site near the California Aqueduct in 1968 (Well 07S02W07J001S). During the rainy season, localized perched water conditions may develop above less permeable units that may require special consideration during grading operations. Further, groundwater will likely travel along bedrock joints and could reach the surface in an artesian condition within and adjacent to the site. Groundwater elevations are dependent on seasonal precipitation, irrigation, and land use, among other factors, and vary thus.

7. GEOLOGIC HAZARDS

7.1 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 (CA DC, 2017a; RCIT, 2017) or a Riverside County Fault Hazard 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 (Morton & Kennedy, 2003).

The closest active fault to the site is the Elsinore fault located approximately 3.8 miles southwest of the site. Faults within a 50-mile radius of the site are listed in Table 7.1.1. Historic earthquakes in southern California of magnitude 6.0 and greater, their magnitude, distance, and direction from the site are listed in Table 7.1.2.

TABLE 7.1.1
ACTIVE FAULTS WITHIN 50 MILES OF THE SITE

Fault Name	Maximum Magnitude (Mw)	Geometry (Slip Character)	Slip Rate (mm/yr)	Information Source	Distance from Site (mi)	Direction from Site
San Jacinto (San Jacinto Valley)	6.9	RL-SS	12.0	a	N	19
Elsinore (Glen Ivy)	6.8	RL-SS	5.0	a	NW	21
San Jacinto (Anza)	7.2	RL-SS	12.0	a	SE	50
Elsinore (Temecula)	6.8	RL-SS	5.0	a	SW	4
San Jacinto (San Bernardino)	6.7	RL-SS	12.0	a	N	35
San Andreas Fault (San Bernardino Segment)	7.5	RL-SS	24.0	a	N	37
Chino Fault	6.7	RL-R-O	1.0	a	NW	38
Whittier Fault	6.8	RL-R-O	2.5	a	NW	50
Pinto Mountain Fault	7.2	LL-SS	2.5	a	NE	39
San Jacinto (Coyote Creek)	6.8	RL-SS	4.0	a	SE	45
Cucamonga Fault	6.9	R	5.0	a	NW	50
Newport-Inglewood (Offshore)	7.1	RL-SS	1.5	a	SW	37
Elsinore (Julian)	7.1	RL-SS	5.0	a	SE	47

Geometry: BT = blind thrust, LL = left lateral, N = normal, O = oblique, R = reverse, RL = right lateral, SS = strike slip.

Information Sources: a = Cao, T., Bryant, W.A., Rowshandel, B., Branum, D., and Wills, C.J., 2003, The Revised 2002 California Probabilistic Seismic Hazard Maps, including Appendices A, B, and C, dated June; b = online Fault Activity Map of California website, maps.conservation.ca.gov/cgs/fam/, as of 1/2017.

n/a = data not available

TABLE 7.1.2
HISTORIC EARTHQUAKE EVENTS WITH RESPECT TO THE SITE

Earthquake (Oldest to Youngest)	Date of Earthquake	Magnitude	Distance to Epicenter (Miles)	Direction to Epicenter
San Jacinto	April 21, 1918	6.8	14	NE
Loma Linda Area	July 22, 1923	6.3	30	NNW
Long Beach	March 10, 1933	6.4	48	W
Buck Ridge	March 25, 1937	6.0	52	ESE
Imperial Valley	May 18, 1940	6.9	59	NE
Desert Hot Springs	December 4, 1948	6.0	50	ENE
Arroyo Salada	March 19, 1954	6.4	63	ESE
Borrego Mountain	April 8, 1968	6.5	69	ESE
San Fernando	February 9, 1971	6.6	98	NW
Joshua Tree	April 22, 1992	6.1	58	ENE
Landers	June 28, 1992	7.3	62	NE
Big Bear	June 28, 1992	6.4	48	NNE
Northridge	January 17, 1994	6.7	98	WNW
Hector Mine	October 16, 1999	7.1	89	NE

7.2 Seismic Design Criteria

The following table summarizes 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).

2016 CBC SEISMIC DESIGN PARAMETERS

Parameter	Value	2016 CBC Reference
Site Class	D	Section 1613.3.2
MCE_R Ground Motion Spectral Response Acceleration – Class B (short), S_S	1.800g	Figure 1613.3.1(1)
MCE_R Ground Motion Spectral Response Acceleration – Class B (1 sec), S_1	0.706g	Figure 1613.3.1(2)
Site Coefficient, F_A	1.000	Table 1613.3.3(1)
Site Coefficient, F_V	1.500	Table 1613.3.3(2)
Site Class Modified MCE_R Spectral Response Acceleration (short), S_{MS}	1.800g	Section 1613.3.3 (Eqn 16-37)
Site Class Modified MCE_R Spectral Response Acceleration – (1 sec), S_{M1}	1.059g	Section 1613.3.3 (Eqn 16-38)
5% Damped Design Spectral Response Acceleration (short), S_{DS}	1.200g	Section 1613.3.4 (Eqn 16-39)
5% Damped Design Spectral Response Acceleration (1 sec), S_{D1}	0.706g	Section 1613.3.4 (Eqn 16-40)

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

ASCE 7-10 PEAK GROUND ACCELERATION

Parameter	Value	ASCE 7-10 Reference
Mapped MCE_G Peak Ground Acceleration, PGA	0.680g	Figure 22-7
Site Coefficient, F_{PGA}	1.000	Table 11.8-1
Site Class Modified MCE_G Peak Ground Acceleration, PGA_M	0.68g	Section 11.8.3 (Eqn 11.8-1)

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

7.3 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 lack of shallow groundwater, the dense consistency of the soils, and granitic bedrock underlying the site, the potential for liquefaction and associated ground deformations beneath the site is nil.

7.4 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.

Fill and alluvial soils obtained during our 2007 investigation were tested for consolidation and hydrocollapse potential. The undocumented artificial fill soils exhibited a collapse potential of 1.3% while the older alluvial soils exhibited a collapse potential of 0.3% when loaded to the anticipated post-grading pressures. The test results indicate that the undocumented artificial fill and older alluvial soils are classified as have a slight (0.1 to 2.0%) degree of specimen collapse in accordance with ASTM D5333.

7.5 Landslides

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

7.6 Rock Fall Hazards

Rock falls are not a design consideration for the site.

7.7 Slope Stability

Grading along the southern boundary of the site will likely result in a bedrock cut slope inclined as steep as 2:1 (h:v) and as high as 15 feet. Fill slopes may also result from grading and are anticipated to be inclined as steep as 2:1 (h:v) and 15 feet or less in height. In general, it is our opinion that cut slopes into the bedrock or fill slopes constructed to a maximum height of 15 feet and with an inclination of 2:1 (h:v) or less will possess Factors of Safety of 1.5 or greater under static loading and 1.1 or greater under seismic loading (see Figures 4 and 5). Specific slope stability analyses should be performed if graded fill slopes over 15 feet or steeper than 2:1 (h:v) 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 Riverside County and the 2016 California Building Code (CBC).

7.8 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 40 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 3.5 miles west of Lake Skinner. The site is not located within the flood plain for either lake, therefore a seiche emanating from either reservoir is not a design consideration.

7.9 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. Lake Skinner dam is located 3.5 miles east of the site. The site is not located within a Lake Skinner inundation area (Metropolitan Water District of Southern California, 1992). Therefore, inundation due to dam failure is not a design consideration.

8. SITE INFILTRATION

The infiltration tests were performed to assist in design of the site stormwater best management practices (BMPs) to be used for the project. The test locations were determined by Mr. Mike Gentile of CASC Engineering.

Geocon excavated three test pits to a depth of approximately 5 feet below existing grades. Infiltration testing was performed on August 3, 4, and 7, 2017, in general conformance with the applicable test methods presented in Appendix A of the *Riverside County – Low Impact Development BMP Design Handbook* (Handbook), Section 2.2.2 for double-ring infiltrometers. The test locations are depicted on the *Geotechnical Map*, Figure 2. Site soils consisted of fill above older alluvium (IT-2 and IT-3) and alluvium over granitic bedrock (IT-1). We did not encounter groundwater during our infiltration test or during our previous geotechnical exploration in 2007 conducted to depths of 15 feet.

The double-ring infiltrometer testing was conducted using graduated mariotte tubes to maintain a constant head within the tests apparatus and measure the water volume. Results of the infiltration testing are presented in Table 1 below. The infiltration test readings and a plot of the test results are included in Appendix A. The recommended infiltration rate in Table 1 was evaluated using the inner ring flow.

TABLE 8.1
INFILTRATION TEST RATES

Test ID	IT-1	IT-2	IT-3
Depth to Infiltration Test, ft	5.0	5.0	5.0
Soil Type	Kgb	SC-SM	CL
Infiltration Rate (in/hr):	0.28	0.022	0.006

It is likely the project area contains soils with varying infiltration rates. Please note that the Handbook requires that a factor of safety of 3 be applied to the infiltration rate based on these testing methods.

9. CONCLUSIONS AND RECOMMENDATIONS

9.1 General

9.1.1 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.

9.1.2 Potential geologic hazards at the site include seismic shaking, potentially compressible undocumented artificial fill, young alluvium, and colluvium, and moderately expansive soils. Based on our investigation and available geologic information, active, potentially active, or inactive faults are not present underlying or trending toward the site.

9.1.3 The undocumented artificial fill, young alluvium, and colluvium 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.

9.1.4 The site fill, alluvium, and colluvial soils are underlain by older alluvium and granitic bedrock. We did not encountered refusal during excavations and seismic refraction data indicates removals should be attainable with grading equipment in good working order to depths of approximately 20 feet.

9.1.5 Oversize material (greater than six-inches) was observed during our subsurface investigation. If oversize material is encountered it should be disposed of in accordance with *Appendix C*.

9.1.6 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.

9.1.7 Groundwater was not encountered during our exploration on the site to depths of 15 feet. Groundwater is not anticipated within the depths of the planned excavations; however, it is possible that perched water will be encountered during grading during the rainy seasons, and may require special considerations during grading.

9.1.8 Although the majority of on-site soils consist of silty or clayey sands, some granular material, having little to no cohesion and subject to caving in un-shored excavations, should be anticipated at the site. It is the responsibility of the contractor to ensure that excavations and trenches are properly shored and maintained in accordance with OSHA rules and regulations to maintain the stability of adjacent existing improvements.

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

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

9.1.11 Fill slopes and cut slopes are not expected to exceed 15 feet in height and should be constructed at a gradient of 2:1 or flatter. If slope heights or inclinations greater than those assumed herein are incorporated into the project, Geocon should be provided the opportunity to review the slopes for stability.

9.1.12 Changes in the design, location or elevation of improvements, as outlined in this report, should be reviewed by this office. Once grading plans become available, they should be reviewed by this office to determine the necessity for review and possible revision of this report.

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

9.2 Soil Characteristics

9.2.1 Based on the material classifications and laboratory testing by Geocon, site soils generally possess a medium expansion potential (expansion index [EI] of 51 to 90), and are considered "expansive" as defined by 2016 California Building Code (CBC) Section 1803.5.3. Table 9.2.1 presents soil classifications based on the EI.

TABLE 9.2.1
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	

9.2.2 Due to the variability of the materials classifications of the site soils, we anticipate that materials with a “low” to “high” expansion potential will be encountered during earthwork. Site grading should include the placement of soils with an expansion index of 60 or less within the upper 4 feet of building pad areas. Soils with an expansion index greater than 60 should not be placed within 4 feet of the proposed foundations, flatwork or paving improvements. Additional testing for expansion potential should be performed during grading and once final grades are achieved.

9.2.3 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 0.003% equating to an exposure class of S0 (Not Applicable) to concrete structures as defined by 2016 CBC Section 1904.3 and ACI 318. Table 9.2.3 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 9.2.3
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

9.2.4 Laboratory testing indicates the site soils have a minimum electrical resistivity of 3,000 ohm-cm, possess 50 parts per million chloride, 0.003% sulfate (30 parts per million), and have a pH of 7.6. Based on the laboratory test results, the site would not be classified as “corrosive” in accordance with the Caltrans Corrosion Guidelines (Caltrans, 2012).

TABLE 9.2.4
CALTRANS CORROSION GUIDELINES

Corrosion Exposure	Resistivity (ohm-cm)	Chloride (ppm)	Sulfate (ppm)	pH
Corrosive	<1,000	500 or greater	2,000 or greater	5.5 or less

9.2.5 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.

9.3 Grading

9.3.1 Grading should be performed in accordance with the *Recommended Grading Specifications* contained in *Appendix C* and the Grading Ordinances of Riverside County.

9.3.2 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.

9.3.3 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.

9.3.4 Undocumented artificial fill, young alluvium, and colluvium within the limits of grading should be removed to expose competent older alluvium or bedrock. The depth of removals is generally anticipated to be 3 to 14 feet in depth below existing ground surface based on the subsurface excavation logs. Anticipated removal depths are depicted on the *Geotechnical Map* (see Figure 2). 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 older alluvial soils with no visible porosity or bedrock 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 within building areas, the excavations should be extended at least 2 feet below the bottom of the planned foundations and 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.

9.3.5 Removals in pavement and sidewalk areas should extend at least 2 feet 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.

9.3.6 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 to a minimum depth 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.

9.3.7 Geocon should observe the removal bottoms to check the competency 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.

9.3.8 The fill placed within 4 feet of proposed foundations should possess an expansion index (EI) of 60 or less.

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

9.3.10 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 approximately 2 percent above 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.

9.3.11 Import fill (if necessary) should consist of granular materials with an expansion index (EI) of 50 or less, non-corrosive, 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.

9.3.12 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.

9.4 Graded Slopes

9.4.1 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 2 percent above optimum moisture content. Rocks greater than 6 inches in maximum dimension should not be placed within 15 feet of slope face.

9.4.2 Finished slopes should be landscaped with drought-tolerant vegetation having variable root depths and requiring minimal landscape irrigation. Some of the site soils are granular and have little to no cohesion, so the slope surfaces may be 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.

9.4.3 Proposed slopes are anticipated to be grossly stable; however, 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.

9.4.4 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.

9.4.5 Excessive watering should be avoided, and 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.

9.5 Earthwork Grading Factors

9.5.1 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 undocumented fill is anticipated to be on the order of 5 to 10 percent, young alluvium is anticipated to shrink 10 to 15 percent, and colluvium is anticipated to shrink 5 to 10 percent when compacted to at least 90 percent of the laboratory maximum dry density. Shrinkage of older alluvium at the site is anticipated to be on the order of 0 to 5 percent when compacted to at least 90 percent of the laboratory maximum dry density. Bedrock is anticipated to bulk from 0 to 5 percent. 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.

9.6 Utility Trench Backfill

9.6.1 Utility trenches should be properly backfilled in accordance with the requirements of the County of Riverside 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. 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. 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.

9.6.2 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.

9.7 Foundation and Concrete Slabs-On-Grade Recommendations

9.7.1 The foundation recommendations presented herein are for the proposed building subsequent to the recommended grading. It is our understanding that planned buildings will be supported on conventional shallow foundations with a concrete slab-on-grade deriving support in at least 2 feet of newly placed engineered fill.

9.7.2 Foundations for the structures may consist of either continuous strip footings and/or isolated spread footings. Conventionally reinforced continuous footings should be at least 18 inches wide and extend at least 24 inches below lowest adjacent pad grade. Isolated spread footings should have a minimum width of 24 inches and should extend at least 24 inches below lowest adjacent pad grade. Figure 5 presents a wall/column footing dimension detail depicting lowest adjacent pad grade.

9.7.3 Following remedial grading, foundations for the buildings may be designed for an allowable soil bearing pressure of 2,000 psf (dead plus live load). This soil bearing pressure may be increased by 150 psf and 250 psf for each additional foot of foundation width and depth, respectively, up to a maximum allowable bearing value of 3,000 psf. The allowable bearing pressure may be increased by one-third for transient loads due to wind or seismic forces.

9.7.4 The maximum expected static settlement for the planned structure supported on conventional foundation systems with the above allowable bearing pressure, and deriving support in engineered fill is estimated to be 1 inch and to occur below the heaviest loaded structural element.

9.7.5 Settlement of the foundation system is expected to occur on initial application of loading. Differential settlement is not expected to exceed ½ inch over a horizontal distance of 40 feet.

9.7.6 Once the design and foundation loading configuration proceeds to a more finalized plan, the estimated settlements within this report should be reviewed and revised, if necessary.

9.7.7 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.

9.7.8 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.

9.7.9 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.

9.7.10 Building slabs-on-grade deriving support in newly placed engineered fill, not subject to vehicle loading, should be a minimum of 4 inches thick and should be reinforced with a minimum of No. 3 steel reinforcing bars placed 24 inches on center in both horizontal directions. Steel reinforcing should be positioned vertically near the slab midpoint.

9.7.11 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.

9.7.12 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 4 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.

9.7.13 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.

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

9.8 Exterior Concrete Flatwork

9.8.1 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 60 or less. Subgrade soils should be compacted to 90 percent relative compaction at 2 percent above optimum moisture. Slab panels should be a minimum of 4 inches thick and when in excess of 8 feet square should be reinforced with 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.

9.8.2 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.

9.8.3 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.

9.8.4 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.

9.9 Conventional Retaining Walls

9.9.1 The recommendations presented herein are generally applicable to the design of rigid concrete or masonry retaining walls having a maximum height of 10 feet. If walls higher than 10 feet or other types of walls are planned, Geocon should be consulted for additional recommendations.

9.9.2 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.

9.9.3 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 55 pcf. If restrained walls which retain sloping backfill are planned, Geocon should be contacted for additional recommendations.

9.9.4 The wall pressures provided above assume that the proposed retaining walls will support relatively undisturbed older alluvium soils, granitic bedrock, or engineered fill derived from selectively graded onsite soils with an EI of 60 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. Imported fill should be observed, tested, and approved by Geocon West, Inc. prior to bringing soil to the site.

9.9.5 In addition to the recommended earth pressure, 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.

9.9.6 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.

9.9.7 The structural engineer should determine the seismic design category for the project in accordance with Section 1613 of the CBC. If the project possesses a seismic design category of D, E, or F, 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).

9.9.8 A seismic load of 10 pcf should be used for design of walls that support more than 6 feet of backfill in accordance with Section 1803.5.12 of the 2016 CBC. The seismic load is applied as an equivalent fluid pressure along the height of the wall and the calculated loads result in a maximum load exerted at the base of the wall and zero at the top of the wall. This seismic load should be applied in addition to the active earth pressure. The earth pressure is based on half of two-thirds of PGA_M calculated from ASCE 7-10 Section 11.8.3.

9.9.9 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.

9.9.10 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 140N (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. Alternatively, a drainage panel, such as a Miradrain 6000 or equivalent, can be placed along the back of the wall. The options are shown on Figure 10. 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 50 or less) with no hydrostatic forces or imposed surcharge load. If conditions different than those described are expected or if specific drainage details are desired, Geocon should be contacted for additional recommendations.

9.9.11 Wall foundations should be designed in accordance with the above foundation recommendations.

9.10 Lateral Design

9.10.1 To resist lateral loads, a passive pressure exerted by an equivalent fluid weight of 200 pounds per cubic foot (pcf) should be used for the design of footings or shear keys poured neat against newly compacted fill. The allowable passive pressure assumes a horizontal surface extending at least 5 feet, or three times the surface generating the passive pressure, whichever is greater. The upper 12 inches of material in areas not protected by floor slabs or pavement should not be included in design for passive resistance.

9.10.2 If friction is to be used to resist lateral loads, an allowable coefficient of friction between newly compacted fill soil and concrete of 0.25 should be used for design.

9.11 Preliminary Pavement Recommendations

9.11.1 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 requirements, when final Traffic Indices and R-Value test results of subgrade soil are completed. Based on our experience with similar soils we have estimated an R-value of 15 for the site. Preliminary flexible pavement sections are presented in Table 9.11.1. We have provided pavement thicknesses for typical roadway classifications. The civil engineer should select the appropriate roadway classification and traffic index based on the anticipated traffic. Geocon should be contacted for additional recommendations if other traffic indices are appropriate for the site roadways.

TABLE 9.11.1
PRELIMINARY FLEXIBLE PAVEMENT SECTIONS

Roadway Classification	Assumed Traffic Index	Assumed Subgrade R-Value	Asphalt Concrete (inches)	Crushed Aggregate Base (inches)
Roadways Servicing Light-Duty Vehicles Local Streets	5.5	15	4.0	7.5
Roadways Servicing Heavy Truck Vehicles Collector Streets	7.0	15	4.0	13.0

9.11.2 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 2 percent above optimum moisture content beneath pavement sections.

9.11.3 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/Riverside County *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.

9.11.4 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 8.11.7.

TABLE 9.11.4
RIGID PAVEMENT DESIGN PARAMETERS

Design Parameter	Design Value
Modulus of subgrade reaction, k	75 pci
Modulus of rupture for concrete, M_R	550 psi
Traffic Category, TC	C and D
Average daily truck traffic, ADTT	100 and 700

9.11.5 Based on the criteria presented herein, the PCC pavement sections should have a minimum thickness as presented in Table 8.11.8.

TABLE 9.11.5
RIGID PAVEMENT RECOMMENDATIONS

Roadway Classification	Portland Cement Concrete (inches)
Roadways (TC=C)	7.0
Truck Areas (TC=D)	8.5

9.11.6 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 2 percent above 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.

9.11.7 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 9-inch-thick slab would have an 11-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.

9.11.8 In order 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 in accordance with the referenced ACI report.

9.11.9 Performance of the pavements 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.

9.12 Temporary Excavations

9.12.1 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.

9.12.2 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.

9.12.3 Excavations on the order of 5 to 10 feet in vertical height may be required during grading operations and utility installation. 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.

9.12.4 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.

9.12.5 Where sufficient space is available, temporary unsurcharged embankments may be sloped back at a uniform 1.5:1 (h:v) slope gradient or flatter for heights up to 20 feet. A uniform slope does not have a vertical portion.

9.12.6 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.

9.13 Site Drainage and Moisture Protection

9.13.1 Proper 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.

9.13.2 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.

9.13.3 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.

9.13.4 If not properly constructed, there is a potential for distress to improvements and properties located hydrologically down gradient or adjacent to infiltration areas. Factors such as the amount of water to be detained, its residence time, and soil permeability have an important effect on seepage transmission and the potential adverse impacts that may occur if the storm water management features are not properly designed and constructed. We have not performed a hydrogeology study at the site. Down-gradient and adjacent structures may be subjected to seeps, movement of foundations and slabs, or other impacts as a result of water infiltration.

9.14 Plan Review

9.14.1 Geocon should review the grading, structural, and foundation plans for the project prior to final submittal to verify that the plans have been prepared in substantial conformance with the recommendations of this report. 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.

LIST OF REFERENCES

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14. CASC Engineering, *APN 963-030-002 Constraints Map*, undated.

15. Continental Aerial Photographs:

Date	Photo Number(s)	Scale 1" = ft
4/19/99	C136-42 102 & 103	2000
10/15/97	C117-42 75&76	2000
10/4/95	CAP 155 & 156	2000
5/14/93	C91-7 9&10	2000
5/4/90	90116 9, 10, 11	3000
1/13/89	89019 22, 23, 24	4000
11/30/89	89264 2 & 3	4000
9/21/89	89203 3	4000
7/30/86	86184 155 & 156	2000
5/9/67	IHH 11, 12, 13	2000
5/23/49	10f 64 & 65	2000
5/23/49	9f 148 & 149	2000

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17. Jennings, Charles W. and Bryant, William A., 2010, *Fault Activity Map of California*, California Division of Mines and Geology Map No. 6.

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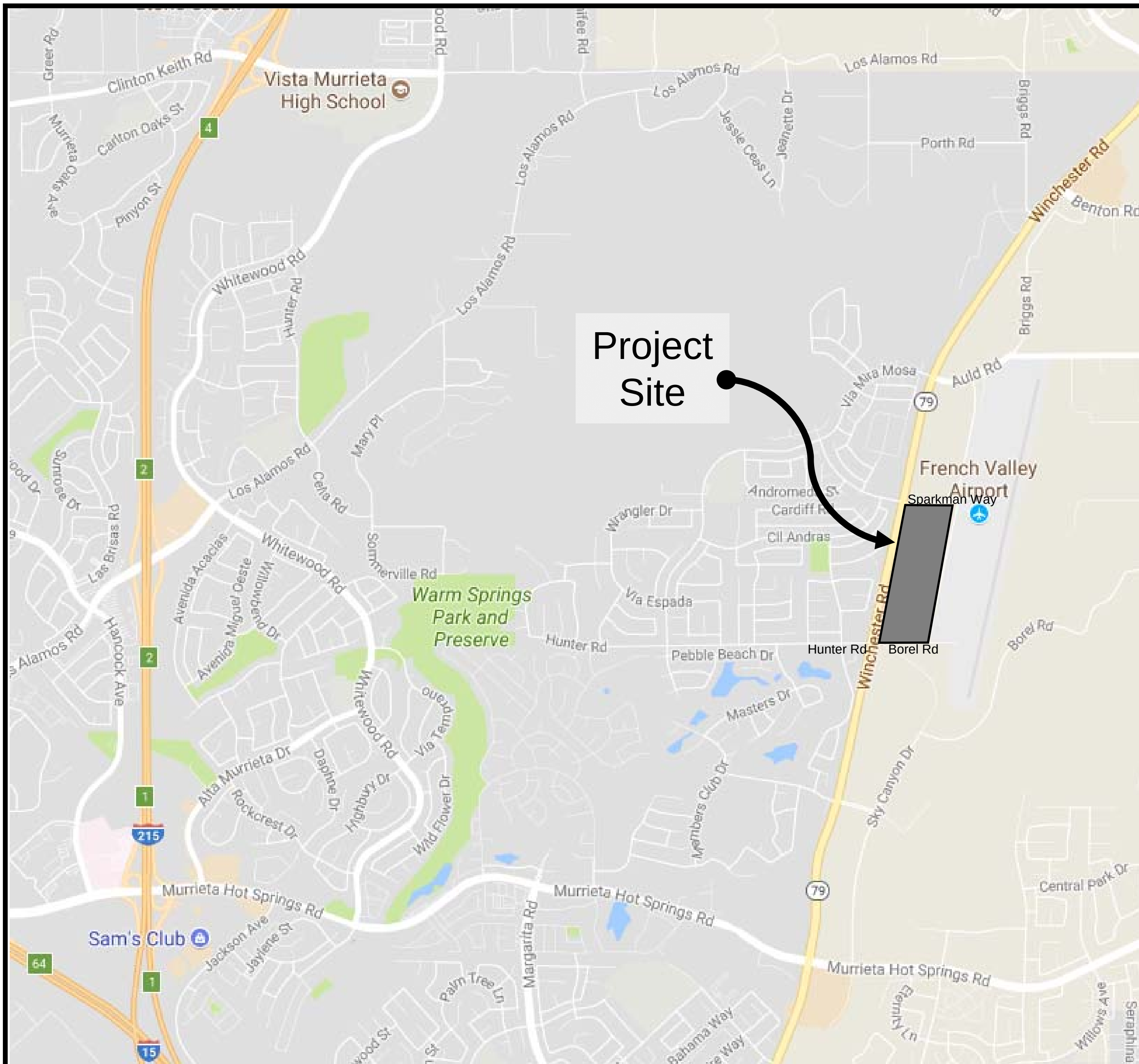

SOURCE: Google Maps, 2017

NOT TO SCALE

VICINITY MAP

GEOCON WEST, INC.

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KTM DEVELOPMENT

NE CORNER OF HWY 79 AND BOREL ROAD

FRENCH VALLEY AREA

RIVERSIDE COUNTY, CALIFORNIA

AMO			AUGUST, 2017	PROJECT NO. T2788-22-01	FIG. 1

GEOCON LEGEND

Locations are approximate

.... PROJECT BOUNDARY

IT-3 .... INFILTRATION TEST LOCATION

T-13 .... TEST PIT LOCATION, 2007

S-4 .... SEISMIC REFRACTION TRAVERSE

5 .... ANTICIPATED REMOVAL DEPTH, IN FEET

afu .... UNDOCUMENTED ARTIFICIAL FILL

Qal .... YOUNG ALLUVIUM

Qcol .... COLLUVIUM

Qvoa .... OLDER ALLUVIUM

Kgr .... GRANITIC BEDROCK

.... GEOLOGIC CONTACT

Source: CASC Engineering and Consulting, *APN 963-030-002 Constraints Map – Draft* (undated).

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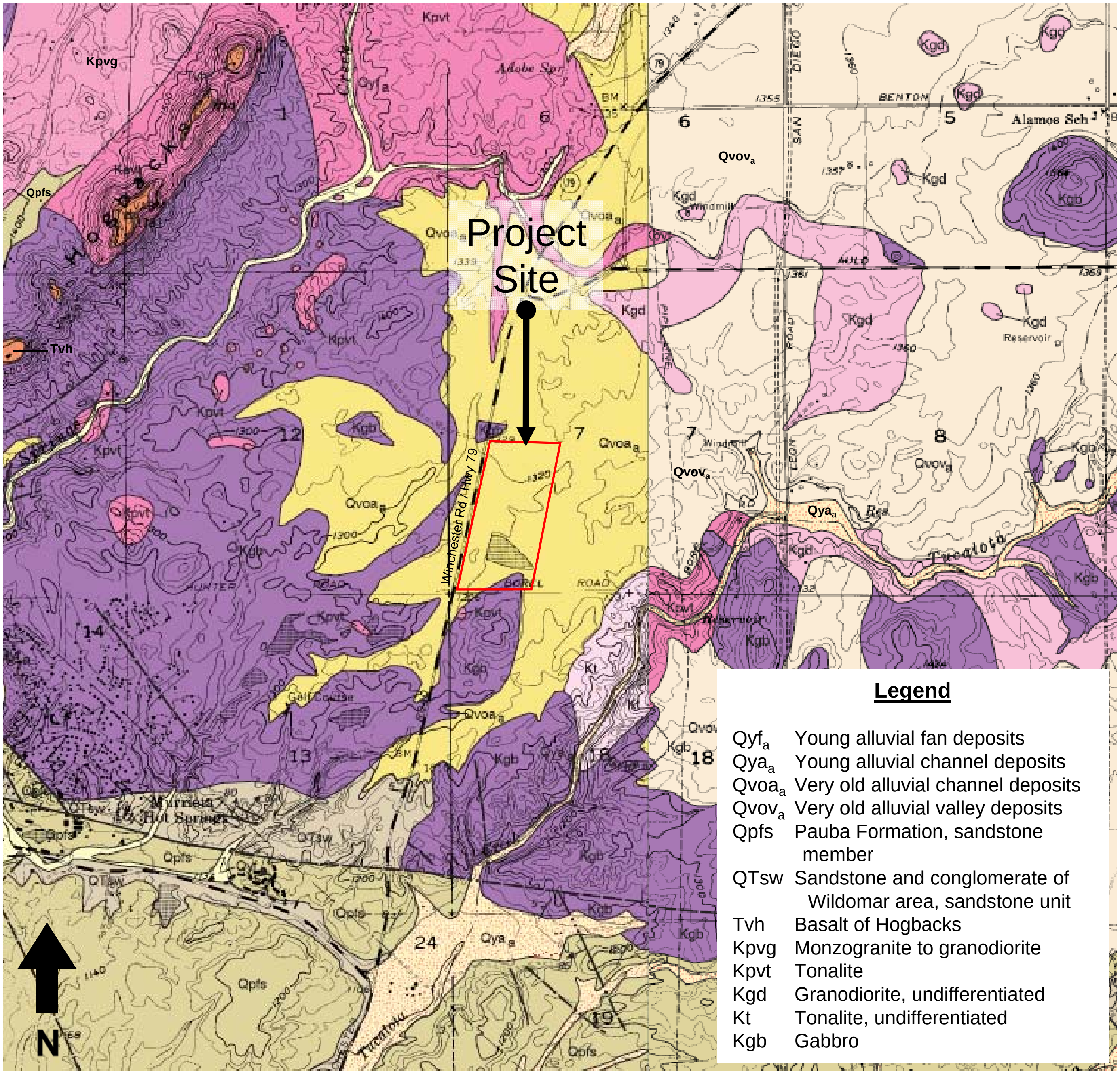
GEOTECHNICAL MAP

KTM DEVELOPMENT
NE CORNER OF HWY 79 AND BOREL ROAD
FRENCH VALLEY AREA
RIVERSIDE COUNTY, CALIFORNIA

AMO			AUGUST, 2017	PROJECT NO. T2788-22-01	FIG. 2



0 2380 4760

SCALE 1" = 2380'

SOURCE: Kennedy, M.P., and D.M. Morton, 2003, Preliminary Geologic Map of the Murrieta 7.5' Quadrangle, Riverside County, California, v. 1.0, USGS OFR 03-189; and Morton, D.M., and M.P. Kennedy, 2003, Geologic Map of the Bachelor Mountain 7.5' Quadrangle, Riverside County, California, v. 1.0, USGS OFR 03-103.

REGIONAL GEOLOGIC MAP

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KTM DEVELOPMENT
NE CORNER OF HWY 79 AND BOREL ROAD
FRENCH VALLEY AREA
RIVERSIDE COUNTY, CALIFORNIA

AMO		

AUGUST, 2017	PROJECT NO. T2788-22-01	FIG. 3

ASSUMED CONDITIONS:

SLOPE HEIGHT	H	= 15 feet
SLOPE INCLINATION	2.0 : 1.0 (Horizontal : Vertical)	
TOTAL UNIT WEIGHT OF SOIL	γ_t	= 125 pounds per cubic foot
ANGLE OF INTERNAL FRICTION	ϕ	= 22 degrees
APPARENT COHESION	C	= 235 pounds per square foot
NO SEEPAGE FORCES		

ANALYSIS:

$\lambda_{c\phi} = \dfrac{\gamma H \tan\phi}{C}$ EQUATION (3-3), REFERENCE 1

$FS = \dfrac{NcfC}{\gamma H}$ EQUATION (3-2), REFERENCE 1

$\lambda_{c\phi}$ = 3.2 CALCULATED USING EQ. (3-3)

Ncf = 16 DETERMINED USING FIGURE 10, REFERENCE 2

FS = 2.0 FACTOR OF SAFETY CALCULATED USING EQ. (3-2)

REFERENCES:

1……Janbu, N., Stability Analysis of Slopes with Dimensionless Parameters, Harvard Soil Mechanics Series No. 46,1954

2……Janbu, N., Discussion of J.M. Bell Dimensionless Parameters for Homogeneous Earth Slpes, Journal of Soil Mechanicx and Foundation Design, No. SM6, November 1967

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SLOPE STABILITY ANALYSIS

KTM DEVELOPMENT
NE CORNER OF HWY 79 AND BOREL ROAD
FRENCH VALLEY AREA
RIVERSIDE COUNTY, CALIFORNIA

CER			AUGUST, 2017	PROJECT NO. T2788-22-01	FIG. 4

ASSUMED CONDITIONS:

SLOPE HEIGHT	H = 15 feet
SLOPE INCLINATION	2.0 : 1.0 (Horizontal : Vertical)
TOTAL UNIT WEIGHT OF SOIL	γ_t = 125 pounds per cubic foot
ANGLE OF INTERNAL FRICTION	ϕ = 22 degrees
APPARENT COHESION	C = 235 pounds per square foot
PSEUDOSTATIC COEFFICIENT	k_h = 0.15
PSEUDOSTATIC INCLINATION	1.4 : 1.0 (Horizontal : Vertical)
PSEUDOSTATIC UNIT WEIGHT	γ_{ps} = 126 pounds per cubic foot

NO SEEPAGE FORCES

ANALYSIS:

$\lambda_{c\phi} = \dfrac{\gamma H \tan\phi}{C}$ EQUATION (3-3), REFERENCE 1

$FS = \dfrac{NcfC}{\gamma H}$ EQUATION (3-2), REFERENCE 1

$\lambda_{c\phi}$ = 3.3 CALCULATED USING EQ. (3-3)

Ncf = 14 DETERMINED USING FIGURE 10, REFERENCE 2

FS = 1.7 FACTOR OF SAFETY CALCULATED USING EQ. (3-2)

REFERENCES:

1……Janbu, N., Stability Analysis of Slopes with Dimensionless Parameters, Harvard Soil Mechanics Series No. 46,1954

2……Janbu, N., Discussion of J.M. Bell Dimensionless Parameters for Homogeneous Earth Slpes, Journal of Soil Mechanicx and Foundation Design, No. SM6, November 1967

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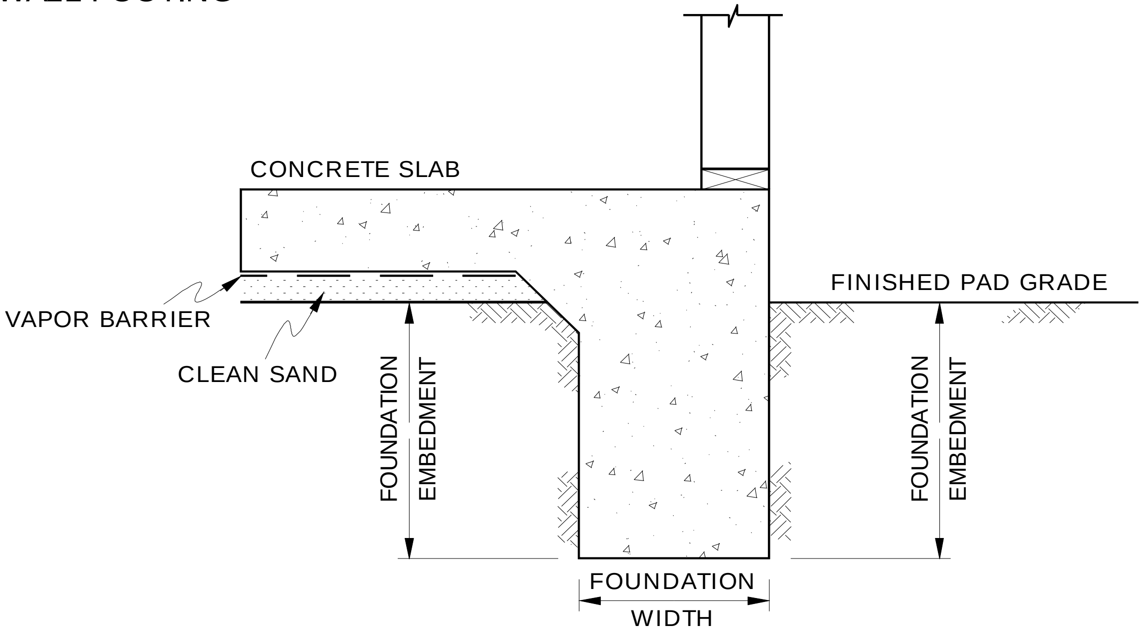
SLOPE STABILITY ANALYSIS - WITH SEISMIC

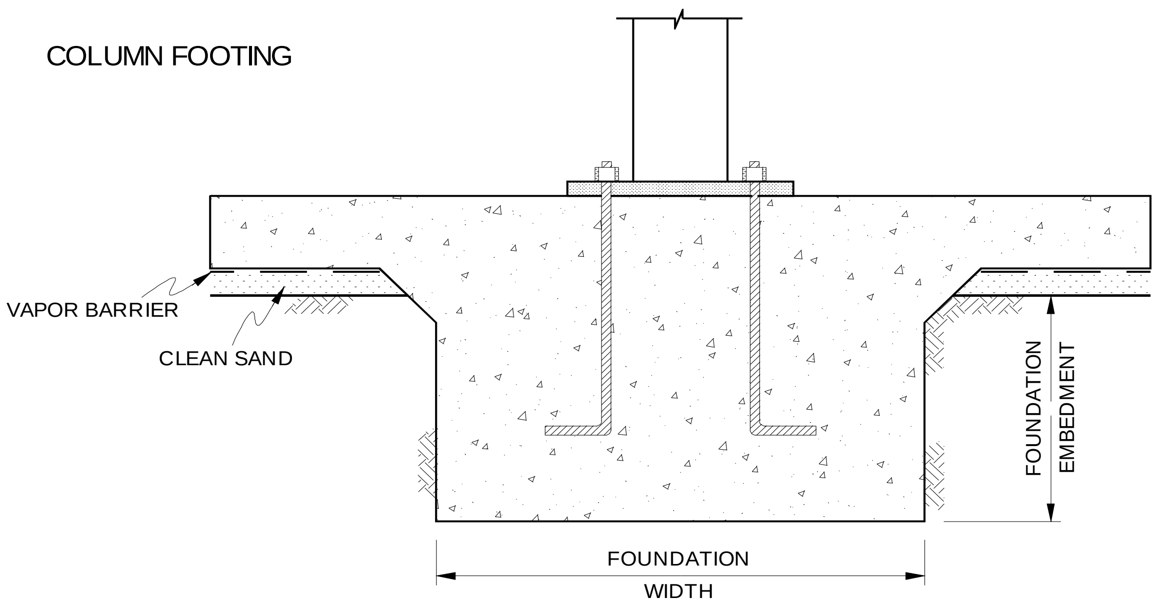
KTM DEVELOPMENT
NE CORNER OF HWY 79 AND BOREL ROAD
FRENCH VALLEY AREA
RIVERSIDE COUNTY, CALIFORNIA

CER			AUGUST, 2017	PROJECT NO. T2788-22-01	FIG. 5

WALL FOOTING



COLUMN FOOTING



NOTE: SEE REPORT FOR FOUNDATION WIDTH AND DEPTH RECOMMENDATION

NO SCALE

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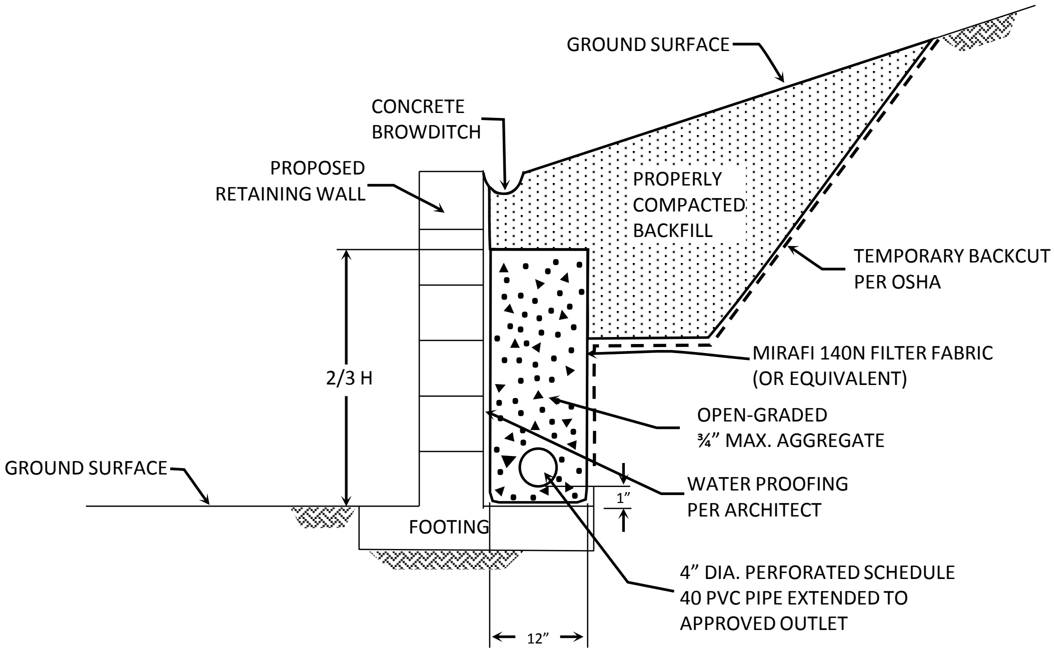
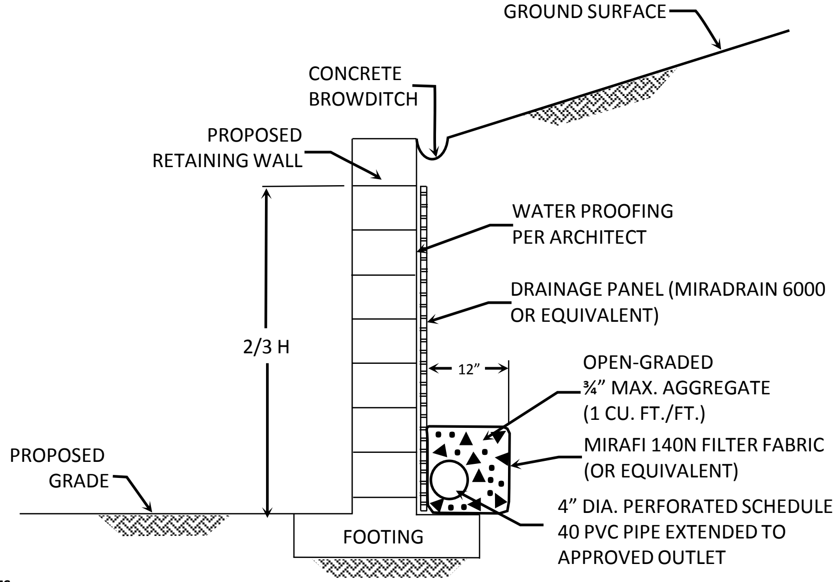
CER		

WALL / COLUMN FOOTING DETAIL

KTM DEVELOPMENT
NE CORNER OF HWY 79 AND BOREL ROAD
FRENCH VALLEY AREA
RIVERSIDE COUNTY, CALIFORNIA

AUGUST, 2017	PROJECT NO. T2788-22-01	FIG. 6





NOTES:

DRAIN SHOULD BE UNFORMLY SLOPED TO GRAVITY OUTLET
OR TO A SUMP WHERE WATER CAN BE REMOVED BY PUMPIMG

CONCRETE BROW DITCH RECOMMENDED FOR SLOPE HEIGHTS
GREATER THAN 6 FEET

NO SCALE

TYPICAL RETAINING WALL DRAIN DETAIL

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KTM DEVELOPMENT
NE CORNER OF HWY 79 AND BOREL ROAD
FRENCH VALLEY AREA
RIVERSIDE COUNTY, CALIFORNIA

CER			AUGUST, 2017	PROJECT NO. T2788-22-01	FIG. 7

APPENDIX A

APPENDIX A

EXPLORATORY EXCAVATIONS

We performed the double ring infiltration testing on August 3, 4, and 7, 2017. Our field work consisted of excavating three infiltration test pits at approximately 5 feet below existing grades and performing double ring infiltrometer testing in accordance with Riverside County LIB BMP Handbook. Upon completion, the infiltration test pits were loosely backfilled with native soils.

PROJECT NO. T2788-22-01

TRENCH IT-1

ELEV. (MSL.) **1309.5** DATE COMPLETED **08/02/2017**

EQUIPMENT **BACKHOE BUCKET** 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	IT-1 @ 0-1			SC-SM	**YOUNG ALLUVIUM (Qal)** Silty Clayey SAND, medium dense, slightly moist, dark brown; fine sand; trace medium to coarse sand; trace gravel; some clay; trace roots; visible porosity; upper 1' disked			
				CL	Sandy CLAY, stiff to hard, moist, dark brown; some fine sand			
2	IT-1 @ 2-3							
4	IT-1 @ 4-5				**GRANITIC BEDROCK (Kgr)** Medium dense, moist, olive gray; highly weathered Excavates as SC-SM, Silty Clayey SAND; fine to medium sand; some slightly indurated chunks; trace carbonate stringers			
					Total depth 5 feet No groundwater encountered; no caving Prepared for double-ring infiltrometer testing			

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

T2788-22-01 TEST PIT LOGS.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. T2788-22-01

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	**TRENCH IT-2** ELEV. (MSL.) **1326.5** DATE COMPLETED **08/02/2017** EQUIPMENT **BACKHOE BUCKET** BY: **A. ORTON**	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
					MATERIAL DESCRIPTION			
0				SM	**UNDOCUMENTED ARTIFICIAL FILL (afu)** Silty SAND, medium dense, slightly moist, brown; fine to medium sand; trace coarse sand; trace gravel; trace clay; roots near surface; upper 1' disked - @1', becomes dense; trace porosity; trace cobbles, small boulders			
2								
	IT-2 @ 3-4'			SC-SM	Silty Clayey SAND, dense, moist, dark brown; fine to coarse sand			
4								
					Total depth 5 feet No groundwater encountered; no caving Prepared for double-ring infiltrometer testing			

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

T2788-22-01 TEST PIT LOGS.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. T2788-22-01

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	TRENCH IT-3 ELEV. (MSL.) 1329.5 DATE COMPLETED 08/02/2017 EQUIPMENT BACKHOE BUCKET BY: A. ORTON	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
					MATERIAL DESCRIPTION			
0				SM	**UNDOCUMENTED ARTIFICIAL FILL (afu)** Silty SAND, medium dense, slightly moist, brown; fine to medium sand; trace coarse sand; trace gravel; trace clay; roots near surface; trace porosity; upper 1' disked			
2				SC	Clayey SAND, medium dense, slightly moist, dark brown; fine sand			
				SM	Silty SAND, medium dense, slightly moist, light brown; fine sand; trace gravel, cobbles, and boulders; trace porosity			
4	IT-3 @ 4-5			CL	Sandy CLAY, hard, moist, dark brown; fine to medium sand			
					Total depth 5 feet No groundwater encountered; no caving Prepared for double-ring infiltrometer testing			

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

T2788-22-01 TEST PIT LOGS.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

DOUBLE RING INFILTROMETER TEST DATA

Project Name:	KTM Industrial	Constants	Ring Data		Marriotte Tubes
Project No.:	T2788-22-01		Area, A_r (in^2)	Depth of Liquid (in.)	ID / Vol., V_r (in^3/in)
Test No.:	IT-1				
Test Location:	Winchester Rd.	Inner Ring:	113	11.25	Small / 3,000 ml
Test By: AMO	USCS Class: SC-SM	Annular Ring:	339	9.15	Large / 60 cm
Water Table Depth:	Penetration of Rings into Soil (in.)		Inner: 1		Outer: 1.25
Date of Test: 08/07/2017	Liquid Used: Water	pH:	Ground Temp (◦F):		at Depth:

Liquid level maintained by using small Marriotte tube for inner ring; large Marriotte tube for annular ring.

Additional Comments: Air temperature 67°F at 7:30 am; foggy.

Time Interval	Time (hr.:min.)	Δt min. / total	Small Marriotte Volume (V, cm^3)	Small Marriotte ΔV (test & total)	Large Marriotte Height (H, cm)	Large Marriotte ΔH (test & total)	Ambient Air Temp (◦F)†	Infiltration Rate, I** Inner (in./hr.)	Infiltration Rate, I** Outer (in./hr.)	Remarks
1 - Start	7:50 AM	10	3000	1790	56.8	9.5	67	5.8	1.8	Light breeze; foggy
End	8:00 AM	10	1210	1790	47.3	9.5	70			
2 - Start	8:00 AM	10	1210	690	47.3	9.0	70	2.2	1.7	
End	8:10 AM	20	520	2480	38.3	18.5	70			
3 - Start	8:10 AM	10	520	380	38.3	8.2	70	1.2	1.6	
End	8:20 AM	30	140	2860	30.1	26.7	70			
4 - Start	8:20 AM	10	1450	370	30.1	8.5	70	1.2	1.6	Partially filled small tube
End	8:30 AM	40	1080	3230	21.6	35.2	71			
5 - Start	8:30 AM	10	1080	270	21.6	6.4	71	0.9	1.2	Cloudy; fog lifting
End	8:40 AM	50	810	3500	15.2	41.6	74			
6 - Start	8:40 AM	20	810	460	35.5	12.5	74	0.7	1.2	Part. filled large tube; sunny
End	9:00 AM	70	350	3960	23.0	54.1	72			
7 - Start	9:00 AM	20	2490	330	23.0	12.4	72	0.53	1.2	Partially filled small tube
End	9:20 AM	90	2160	4290	10.6	66.5	76			
8 - Start	9:20 AM	20	2160	370	40.3	10.3	76	0.60	1.0	Partially filled large tube
End	9:40 AM	110	1790	4660	30.0	76.8	77			
9 - Start	9:40 AM	20	1790	340	30.0	10.6	77	0.55	1.0	
End	10:00 AM	130	1450	5000	19.4	87.4	80			
10 - Start	10:00 AM	60	2940	1030	46.9	28.1	80	0.56	0.90	Filled both tubes
End	11:00 AM	190	1910	6030	18.8	115.5	84			
11 - Start	11:00 AM	60	2800	620	44.8	24.7	84	0.33	0.80	Mod. breeze; filled both tubes
End	12:00 PM	250	2180	6650	20.1	140.2	89			
12 - Start	12:00 PM	60	2180	660	40.9	22.3	89	0.36	0.72	Partially filled large tube
End	1:00 PM	310	1520	7310	18.6	162.5	92			
13 - Start	1:00 PM	60	1520	600	48.9	22.5	92	0.32	0.72	Partially filled large tube
End	2:00 PM	370	920	7910	26.4	185.0	95			
14 - Start	2:00 PM	60	2670	380	42.9	19.1	95	0.21	0.61	Filled both tubes
End	3:00 PM	430	2290	8290	23.8	204.1	95			
15 - Start	3:00 PM	60	2290	510	44.3	20.5	95	0.28	0.66	Filled large tube
End	4:00 PM	490	1780	8800	23.8	224.6	95			

*Flow, $Q_f = \Delta H \times V_r$ **Infiltration Rate, $I = (Q_f/A_r)/\Delta t$ † Proxy for Liquid Temperature

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AMO

INFILTRATION DATA

KTM DEVELOPMENT
NE CORNER OF HWY 79 AND BOREL ROAD
FRENCH VALLEY AREA
RIVERSIDE COUNTY, CALIFORNIA

AUGUST, 2017 | PROJECT NO. T2788-22-01 | FIG A-4

DOUBLE RING INFILTROMETER TEST DATA

Project Name:	KTM Industrial	Constants	Ring Data		Marriotte Tubes	
Project No.:	T2788-22-01		Area, A_r (in^2)	Depth of Liquid (in.)	ID	Vol., V_r (in^3/in)
Test No.:	IT-2					
Test Location:	Winchester Rd.	Inner Ring:	113	11.5	Small	3,000 ml
Test By: AMO	USCS Class: SC/CL	Annular Ring:	339	11.5	Large	60 cm
Water Table Depth:	Penetration of Rings into Soil (in.)		Inner:	1.5	Outer:	1.75
Date of Test: 08/04/2017	Liquid Used: Water	pH:	Ground Temp (◦F):		at Depth:	

Liquid level maintained by using small Marriotte tube for inner ring; large Marriotte tube for annular ring.

Additional Comments: Sunny

Time Interval	Time (hr.:min.)	Δt min. / total	Small Marriotte Volume (V, cm^3)	Small Marriotte ΔV (test & total)	Large Marriotte Height (H, cm)	Large Marriotte ΔH (test & total)	Ambient Air Temp (◦F)†	Infiltration Rate, I** Inner (in./hr.)	Infiltration Rate, I** Outer (in./hr.)	Remarks
1 - Start	7:30 AM	5	1750	1420	31.1	4.5	73	9.2	1.7	Sunny; still
End	7:35 AM	5	330	1420	26.6	4.5	73			
2 - Start	7:35 AM	9	3000	3000	44.6	20.2	73	10.8	4.3	Filled both tubes
End	7:44 AM	14	0	4420	24.4	24.7	73			
3 - Start	7:44 AM	11	2900	1550	37.8	15.1	73	4.6	2.7	Slight breeze; filled both tubes
End	7:55 AM	25	1350	5970	22.7	39.8	73			
4 - Start	7:55 AM	10	1350	180	22.7	11.1	73	0.58	2.1	
End	8:05 AM	35	1170	6150	11.6	50.9	76			
5 - Start	8:05 AM	30	1170	10	43.9	15.9	76	0.011	1.0	Filled large tube
End	8:35 AM	65	1160	6160	28.0	66.8	78			
6 - Start	8:35 AM	30	1160	10	28.0	4.5	78	0.011	0.29	
End	9:05 AM	95	1150	6170	23.5	71.3	80			
7 - Start	9:05 AM	30	1150	10	23.5	1.2	80	0.011	0.077	
End	9:35 AM	125	1140	6180	22.3	72.5	84			
8 - Start	9:35 AM	30	1140	30	22.3	1.3	84	0.032	0.084	
End	10:05 AM	155	1110	6210	21.0	73.8	86			
9 - Start	10:05 AM	30	1110	40	21.0	1.3	86	0.043	0.084	
End	10:35 AM	185	1070	6250	19.7	75.1	88			
10 - Start	10:35 AM	30	1070	30	19.7	1.2	88	0.032	0.077	
End	11:05 AM	215	1040	6280	18.5	76.3	89			
11 - Start	11:05 AM	30	1040	20	18.5	1.2	89	0.022	0.077	Moderate breeze
End	11:35 AM	245	1020	6300	17.3	77.5	92			
12 - Start	11:35 AM	30	1020	20	17.3	1.2	92	0.022	0.077	
End	12:05 PM	275	1000	6320	16.1	78.7	93			
13 - Start	12:05 PM	30	1000	20	16.1	1.2	93	0.022	0.077	
End	12:35 PM	305	980	6340	14.9	79.9	93			
14 - Start										
End										
15 - Start										
End										

*Flow, Q_f = ΔH x V_r **Infiltration Rate, I = (Q_f/A_r)/Δt † Proxy for Liquid Temperature

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

INFILTRATION DATA

KTM DEVELOPMENT
NE CORNER OF HWY 79 AND BOREL ROAD
FRENCH VALLEY AREA
RIVERSIDE COUNTY, CALIFORNIA

AUGUST, 2017 | PROJECT NO. T2788-22-01 | FIG A-5

DOUBLE RING INFILTROMETER TEST DATA

Project Name: KTM Industrial	Constants	Ring Data		Marriotte Tubes	
Project No.: T2788-22-01		Area, A_r (in^2)	Depth of Liquid (in.)	ID	Vol., V_r (in^3/in)
Test No.: IT-3					
Test Location: Winchester Rd.	Inner Ring:	113	12.25	Small	3,000 ml
Test By: AMO — USCS Class: SC/CL	Annular Ring:	339	12.25	Large	60 cm
Water Table Depth:	Penetration of Rings into Soil (in.)	Inner:	1.5	Outer:	1.25
Date of Test: 08/03/2017 — Liquid Used: Water	pH:	Ground Temp (◦F):		at Depth:	

Liquid level maintained by using small Marriotte tube for inner ring; large Marriotte tube for annular ring.

Additional Comments: Air temp 78°F at 7:23 am. It was very warm overnight.

Time Interval	Time (hr.:min.)	Δt min. / total	Small Marriotte Volume (V, cm^3)	Small Marriotte ΔV (test & total)	Large Marriotte Height (H, cm)	Large Marriotte ΔH (test & total)	Ambient Air Temp (◦F)†	Infiltration Rate, I** Inner (in./hr.)	Infiltration Rate, I** Outer (in./hr.)	Remarks
1 - Start	8:46 AM	8	350	300	42.1	3.8	84	1.2	0.9	Slightly overcast; still
End	8:54 AM	8	50	300	38.3	3.8	84			
2 - Start	8:55 AM	10	3000	90	38.2	3.0	84	0.29	0.58	Filled small tube
End	9:05 AM	18	2910	390	35.2	6.8	86			
3 - Start	9:05 AM	10	2910	50	35.2	1.8	86	0.16	0.35	Sunny; slight breeze
End	9:15 AM	28	2860	440	33.4	8.6	86			
4 - Start	9:15 AM	10	2860	4	33.4	1.2	86	0.013	0.23	
End	9:25 AM	38	2856	444	32.2	9.8	86			
5 - Start	9:25 AM	10	2856	3	32.2	0.5	86	0.010	0.097	
End	9:35 AM	48	2853	447	31.7	10.3	86			
6 - Start	9:35 AM	30	2853	13	31.7	1.4	86	0.014	0.090	
End	10:05 AM	78	2840	460	30.3	11.7	89			
7 - Start	10:05 AM	30	2840	5	30.3	1.1	89	0.005	0.071	
End	10:35 AM	108	2835	465	29.2	12.8	92			
8 - Start	10:35 AM	30	2835	5	29.2	1.5	92	0.005	0.097	
End	11:05 AM	138	2830	470	27.7	14.3	94			
9 - Start	11:05 AM	30	2830	10	27.7	1.1	94	0.011	0.071	
End	11:35 AM	168	2820	480	26.6	15.4	95			
10 - Start	11:35 AM	60	2820	10	26.6	2.3	95	0.005	0.074	Light to mod. Gusty winds
End	12:35 PM	228	2810	490	24.3	17.7	97			
11 - Start	12:35 PM	60	2810	15	24.3	2.5	97	0.008	0.080	
End	1:35 PM	288	2795	505	21.8	20.2	99			
12 - Start	1:35 PM	60	2795	13	21.8	2.4	99	0.007	0.077	Moderate breeze
End	2:35 PM	348	2782	518	19.4	22.6	100			
13 - Start	2:35 PM	60	2782	12	19.4	2.6	100	0.006	0.084	
End	3:35 PM	408	2770	530	16.8	25.2	98			
14 - Start										
End										
15 - Start										
End										

*Flow, Q_f = ΔH x V_r **Infiltration Rate, I = (Q_f/A_r)/Δt † Proxy for Liquid Temperature

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INFILTRATION DATA
KTM DEVELOPMENT
NE CORNER OF HWY 79 AND BOREL ROAD
FRENCH VALLEY AREA
RIVERSIDE COUNTY, CALIFORNIA

AUGUST, 2017 | PROJECT NO. T2788-22-01 | FIG A-6

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	TRENCH T 1 ELEV. (MSL.) ______ DATE COMPLETED 07-20-2007 EQUIPMENT JD 510 BACKHOE WITH 24" BUCKET BY: P. THERIAULT	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
					MATERIAL DESCRIPTION			
0				SM SC ML	**UNDOCUMENTED FILL-** *afu* Layered light and dark brown, loose to dense, damp to moist, Silty, fine to medium SAND to Clayey SAND to Sandy SILT; roothairs, upper 2" spread out 3/4" base; trace gravel; upper 1' disturbed			
2	T1-1							
4	T1-2			CL	Becomes stiff, medium brown, moist, fine to medium, Sandy CLAY	100/6"		
6	T1-3			SM	**OLDER ALLUVIUM-** *Qoal* Dense, brownish red, moist, Silty, fine to medium SAND			
					TRENCH TERMINATED AT 6½ FEET No groundwater encountered Removal to 5 feet			

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

07178-42-01.GPJ

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

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

GEOCON

PROJECT NO. 07178-42-01

TRENCH T 2

ELEV. (MSL.) ______ DATE COMPLETED 07-20-2007

EQUIPMENT JD 510 BACKHOE WITH 24" BUCKET BY: P. THERIAULT

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 ML	**UNDOCUMENTED FILL**- *afu* Medium dense, to stiff, mottled light brown and gray-brown, slightly moist, Silty, fine to medium sand, to fine to medium Sandy SILT; trace gravel, upper 1' disturbed			
2								
4	T2-1					52/3"		
6				ML	**OLDER ALLUVIUM**- *Qoal* Very dense, moist, gray, fine, Sandy SILT; difficult digging			

TRENCH TERMINATED AT 6½ FEET
No groundwater encountered
Removal to 5½ feet

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

07178-42-01.GPJ

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

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

GEOCON

PROJECT NO. 07178-42-01

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	**TRENCH T 3** ELEV. (MSL.)______ DATE COMPLETED 07-20-2007 EQUIPMENT JD 510 BACKHOE WITH 24" BUCKET BY: P. THERIAULT	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
					MATERIAL DESCRIPTION			
0				SM ML	**UNDOCUMENTED FILL-** *afu* Medium dense, layered brown and light brown, slightly moist, Silty, fine to medium SAND and Sandy SILT; upper 1' disturbed			
2								
4	T3-1					37/3"		
6					-Becomes mostly sandy silt			
				SM	Silty SAND; some cobble			
8				CL	Gray CLAY			
				SM	Silty SAND			
10				SM	**OLDER ALLUVIUM-** *Qoal* Dense, medium brown, moist, Silty, fine SAND, some medium , some clay; difficult digging			
12								
					GRANITIC BEDROCK- *Kgr* Weathered, soft, moist, gray/white; excavates as fine to coarse sand with gravel, difficult digging			
					TRENCH TERMINATED AT 13½ FEET No groundwater encountered			

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

07178-42-01.GPJ

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

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

GEOCON

PROJECT NO. 07178-42-01

TRENCH T 4

ELEV. (MSL.)______ DATE COMPLETED 07-20-2007

EQUIPMENT JD 510 BACKHOE WITH 24" BUCKET BY: P. THERIAULT

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 ML CL	**UNDOCUMENTED FILL-** *afu* Medium dense to stiff, slightly moist, layered light brown to dark brown, Silty, fine to medium SAND to Sandy SILT and Sandy CLAY; some gravel; upper 1' disturbed			
4	T4-1					37/3"		
10				CL	Stiff, moist, fine to coarse, Sandy CLAY			
13					**GRANITIC BEDROCK-** *Kgr* Fine grained, yellow-brown, soft (weathered), friable; difficult digging at 13½'			
14					TRENCH TERMINATED AT 14 FEET No groundwater encountered Removal to 13 feet			

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

07178-42-01.GPJ

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

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

GEOCON

PROJECT NO. 07178-42-01

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	TRENCH T 5 ELEV. (MSL.)______ DATE COMPLETED 07-20-2007 EQUIPMENT JD 510 BACKHOE WITH 24" BUCKET BY: P. THERIAULT	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
					MATERIAL DESCRIPTION			
0 – 14	T5-1 (4)			SM CL	**UNDOCUMENTED FILL-** *qfu* Medium dense to stiff, slightly moist, layered light brown and dark brown, Silty fine to medium SAND to Sandy CLAY; some gray clay	43/3"		
14 – 15	T5-2			SM	**OLDER ALLUVIUM-** *Qoal* Dense, moist, mottled gray and yellowish brown, Silty, Gravelly, fine to medium, SAND; some coarse sand; well indurated			

TRENCH TERMINATED AT 15 FEET
No groundwater encountered
Removal to 14 feet

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

07178-42-01.GPJ

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

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

GEOCON

PROJECT NO. 07178-42-01

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	TRENCH T 6 ELEV. (MSL.) ______ DATE COMPLETED 07-20-2007 EQUIPMENT JD 510 BACKHOE WITH 24" BUCKET BY: P. THERIAULT	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
					MATERIAL DESCRIPTION			
0				SM CL	**UNDOCUMENTED FILL-** *afu* Medium dense, slightly moist, layered brown, light brown and gray, Silty, fine to medium SAND with lesser amounts of Sandy CLAY			
2								
4	T6-1					40/3"		
6								
8								
10				SM	**OLDER ALLUVIUM-** *Qoal* Very dense, moist, reddish yellowish brown, Silty, fine to medium SAND; well indurated; difficult digging; some carbonate stringers on ped surfaces			
					TRENCH TERMINATED AT 10 FEET No groundwater encountered Removal to 9½ feet			

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

07178-42-01.GPJ

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

GEOCON

TRENCH T 7

ELEV. (MSL.) ______ DATE COMPLETED 07-20-2007

EQUIPMENT JD 510 BACKHOE WITH 24" BUCKET BY: P. THERIAULT

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 CL	**UNDOCUMENTED FILL-** *afu* Medium dense to stiff, slightly moist, layered brown, light brown and gray, Silty, fine to medium SAND to Sandy CLAY; upper 1' disturbed			
2								
4	T7-1					50/4"		
6								
8								
10				SM	**OLDER ALLUVIUM-** *Qoal* Very dense, moist, reddish yellowish brown, Silty, fine to medium SAND; difficult digging; well indurated; some carbonate stringers on ped surfaces			

TRENCH TERMINATED AT 10½ FEET
No groundwater encountered
Removal to 9½ feet

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

07178-42-01.GPJ

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

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

GEOCON

PROJECT NO. 07178-42-01

TRENCH T 8

ELEV. (MSL.)_______ DATE COMPLETED 07-20-2007

EQUIPMENT JD 510 BACKHOE WITH 24" BUCKET BY: P. THERIAULT

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 CL	**UNDOCUMENTED FILL-** *afu* Medium dense, slightly moist, layered light brown, dark brown and gray, Silty, fine to medium SAND to Sandy CLAY; roothairs near surface; upper 1' disturbed			
4	T8-1					32/3"		
12					**GRANITIC BEDROCK-** *Kgr* Moderately hard, moist, brownish yellow; excavates as a silty, sandy gravel; difficult digging			

TRENCH TERMINATED AT 6½ FEET
No groundwater encountered
Removal to 5 feet

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

07178-42-01.GPJ

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

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

GEOCON

PROJECT NO. 07178-42-01

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	TRENCH T 9 ELEV. (MSL.)_____ DATE COMPLETED 07-20-2007 EQUIPMENT JD 510 BACKHOE WITH 24" BUCKET BY: P. THERIAULT	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
					MATERIAL DESCRIPTION			
0 2 4				SC	**COLLUVIUM-** *Qcol* Medium dense, slightly moist, brownish red, Clayey, fine to coarse SAND, with gravel; trace cobble; upper 1' disturbed			
6		+ +			**GRANITIC BEDROCK-** *Kgr* Weathered, soft, moist, gray; excavates as a gravelly, fine to coarse sand with some silt -Becomes moderately hard; difficult digging at 5'			
					TRENCH TERMINATED AT 6½ FEET No groundwater encountered Removal to 5 feet			

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

07178-42-01.GPJ

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

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

GEOCON

PROJECT NO. 07178-42-01

TRENCH T 10

ELEV. (MSL.) ______ DATE COMPLETED 07-20-2007

EQUIPMENT JD 510 BACKHOE WITH 24" BUCKET BY: P. THERIAULT

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				SC	**COLLUVIUM-** *Qcol* Medium dense, slightly moist, brown, Clayey, fine to medium SAND, some coarse sand; upper 1' disturbed			
4					**GRANITIC BEDROCK-** *Kgr* Weathered, soft, yellow, friable; excavates as sandy cobble -Difficult digging at 3½'			
					TRENCH TERMINATED AT 4½ FEET No groundwater encountered Removal to 3½ feet			

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

07178-42-01.GPJ

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

PROJECT NO. 07178-42-01

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	TRENCH T 11 ELEV. (MSL.)_____ DATE COMPLETED 07-20-2007 EQUIPMENT JD 510 BACKHOE WITH 24" BUCKET BY: P. THERIAULT	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
					MATERIAL DESCRIPTION			
0				CL	**COLLUVIUM-** *Qcol* Stiff, slightly moist, brown, fine to medium, Sandy CLAY; trace cobble			
2								
					-Moist; some cobble at 3'			
4					**GRANITIC BEDROCK-** *Kgr* Weathered, moist, gray, fine-grained; excavates as a gravelly sand, with cobbles -Difficult digging at 5'			
					TRENCH TERMINATED AT 5½ FEET No groundwater encountered Removal to 4 feet			

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

07178-42-01.GPJ

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

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GEOCON

PROJECT NO. 07178-42-01

TRENCH T 12

ELEV. (MSL.)______ DATE COMPLETED 07-20-2007

EQUIPMENT JD 510 BACKHOE WITH 24" BUCKET BY: P. THERIAULT

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0				CL	**ALLUVIUM** - *Qal* Soft, wet, brown, fine to medium, Sandy CLAY; upper 1' disturbed			
				SM	-Boulder (30") at 1'			
2	T12-1				Whitish, fine to medium, Silty SAND			
				CL	Fine to medium, Sandy CLAY			
4								
6	T12-2				**GRANITIC BEDROCK**- *Kgr* Weathered, soft, moist, black and white with orange staining; excavates as a sandy cobble; difficult excavation			

TRENCH TERMINATED AT 6½ FEET
No groundwater encountered
Removal to 5½ feet

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

07178-42-01.GPJ

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

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

GEOCON

PROJECT NO. 07178-42-01

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	TRENCH T 13 ELEV. (MSL.)_______ DATE COMPLETED 07-20-2007 EQUIPMENT JD 510 BACKHOE WITH 24" BUCKET BY: P. THERIAULT MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0				SC	**COLLUVIUM-** *Qcol* Medium dense, slightly moist, brown, Clayey, fine to medium SAND, some coarse sand; upper 1' disturbed			
2				CL	Stiff, moist, dark brown, Sandy CLAY			
4								
6					**GRANITIC BEDROCK-** *Kgr* Weathered, gray and orange, fine-grained, moist, friable -Difficult digging at 6'			
					TRENCH TERMINATED AT 6½ FEET No groundwater encountered Removal to 5½ feet			

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

07178-42-01.GPJ

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

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

GEOCON

TERRA GEOSCIENCES

SEISMIC REFRACTION SURVEY

FLEMMING PROPERTY

CITY OF MURRIETA, CALIFORNIA

Project No. 272232-1

July 20, 2007

Prepared for:

GEOCON, Inc.
41571 Corning Place
Suite 101
Murrieta, CA 92562-7065

GEOCON, Inc.
41571 Corning Place
Suite 101
Murrieta, CA 92562-7065

Attention: Mr. Paul Theriault, Project Geologist

Regarding: Seismic Refraction Survey
Flemming Property
City of Murrieta, California
GEOCON Project Number 07871-42-01

INTRODUCTION

As requested, this firm has performed a geophysical survey using the seismic refraction method for the above-referenced site along four selected areas as delineated by you. The purpose of this investigation was to assess the general seismic velocity characteristics of the underlying earth materials and to aid in evaluating whether high velocity earth materials (non-rippable) are present along local areas which could possibly indicate areas of potential excavation difficulties.

The bedrock materials underlying the site at depth have been mapped by Kennedy and Morton (2003) to consist of Cretaceous age granitic rock classified as brown-weathering, medium- to very coarse-grained hornblende gabbro, which is locally mantled across portions of the site by Pleistocene age older alluvial channel deposits generally consisting of moderately indurated, dissected gravel, sand, silt, and clay. We understand that this report will be included as a technical appendix to your report, therefore as requested, the locations of our geophysical survey lines were transferred onto your field map for inclusion onto your final map.

As authorized by you, the following services were performed during this study:

- **Review of available published and unpublished geologic/geophysical data in our files pertinent to the site.**

- **Performing a seismic refraction survey by a State of California Professional Geophysicist, to include four traverses along selected portions of the subject site.**

- **Preparation of this report, presenting the results of our findings and conclusions with respect to the velocity characteristics and the expected rippability potentials of the subsurface earth materials.**

Accompanying Appendices

Appendix A - Layer Velocity Profiles
Appendix B - Tomographic Models
Appendix C - Excavation Considerations
Appendix D - References

SEISMIC REFRACTION SURVEY

Methodology

The seismic refraction method consists of measuring (at known points along the surface of the ground) the travel times of compressional waves generated by an impulsive energy source and can be used to estimate the layering, structure, and seismic acoustic velocities of subsurface horizons. Seismic waves travel down and through the soils and rocks, and when the wave encounters a contact between two earth materials having different velocities, some of the wave's energy travels along the contact at the velocity of the lower layer. The fundamental assumption is that each successively deeper layer has a velocity greater than the layer immediately above it. As the wave travels along the contact, some of the wave's energy is refracted toward the surface where it is detected by a series of motion-sensitive transducers (geophones). The arrival time of the seismic wave at the geophone locations can be related to the relative seismic velocities of the subsurface layers in feet per second (fps), which can then be used to aid in interpreting both the depth and type of materials encountered.

Field Procedures

Four seismic refraction survey lines were performed each being 130-feet in length, with a target depth of around 30±-feet. A 16-pound sledge-hammer was used as an energy source to produce the seismic waves and twelve, 14-Hz geophones (with 70% damping), were spaced at 12-foot intervals along the traverse lines to detect both the direct and refracted waves. The seismic wave arrivals were digitally recorded in SEG-2 format on a Geometrics StrataVisor™ NX model signal enhancement refraction seismograph. Seven shot points were utilized along each seismic line spread using forward, reverse, and intermediate locations, in order to obtain sufficient data for velocity analysis and depth modeling purposes. The data was acquired using a sampling rate of 0.25 milliseconds with a record length of 0.08 seconds. No acquisition filters were used. Each geophone and shot location was surveyed using a hand level and ruler for relative topographic correction. During acquisition, the seismograph provides both a hard copy and screen display of the seismic wave arrivals, of which are digitally recorded on the in-board seismograph computer.

Data Reduction

The data on the paper record and/or display screen were used to analyze the arrival time of the primary seismic "P"-waves at each geophone station, in the form of a wiggle trace, or wave travel-time curve, for quality control purposes in the field. All of the recorded data was subsequently transferred to our office computer for further processing, analyzing, and printing purposes, using the computer programs **SIP** (**S**eismic refraction **I**nterpretation **P**rogram) developed by Rimrock Geophysics, Inc. (1995), and **Rayfract™** (Intelligent Resources, Inc., 1996-2007). **SIP** is a ray-trace modeling program that evaluates the subsurface using layer assignments based on time-distance curves and is better suited for layered media, using the "Seismic Refraction Modeling by Computer" method (Scott, 1973). In addition, **Rayfract™** was also used for comparative purposes. **Rayfract™** is seismic refraction tomography software that models subsurface refraction, transmission, and diffraction of acoustic waves. Both computer programs perform their analysis using exactly the same input data, which includes first-arrival P-waves and line geometry.

SUMMARY OF GEOPHYSICAL INTERPRETATION

To begin our discussion, it should be understood that the velocity data obtained during this survey represents an average of seismic velocities within any given layer. For example, high seismic velocity boulders/dikes or local lithologic inconsistencies, may be isolated within a low velocity matrix, thus yielding an average medium velocity for that layer. Therefore, in any given layer, a range of velocities could be anticipated, which can also result in a wide range of excavation characteristics.

It is also important to consider that the seismic velocities obtained within bedrock materials are influenced by the nature and character of the localized major structural discontinuities (foliation, fracturing, etc.). Generally, it is expected that higher (truer) velocities will be obtained when the seismic waves propagate along direction (strike) of the dominant structure, with a damping effect when the seismic waves travel in a perpendicular direction. Therefore, the seismic velocities obtained during our field study and as discussed below, should be considered minimum velocities at this time, as the structure of the bedrock locally is not known.

In general, the site where locally surveyed, was noted to be characterized by three major subsurface layers with respect to seismic velocities. The following velocity layer summaries have been prepared using the **SIP** analysis, with the representative Layer Velocity Profiles for each seismic survey line presented within Appendix A. These profiles generally indicate the respective "weighted average" subsurface velocities in generalized layers.

- <u>**Velocity Layer V1**</u>:

  This uppermost velocity layer (V1) is most likely comprised of topsoil, colluvium, fill materials, and/or older alluvial deposits, such as mapped by Kennedy and Morton (2003). This layer has an average weighted velocity ranging from 1,418 to 1,605 fps, which is typical for these types of surficial-mantling materials.

- <u>**Velocity Layer V2**</u>:

  The second velocity layer (V2) yielded a wide range of 2,037 to 3,397 fps, indicating high degrees of weathering and fracturing of the underlying granitic bedrock where present, moderately indurated older alluvial deposits, or possible localized artificial fill. The higher-end seismic velocities in this layer are typical for both moderately indurated sediments, and for the near surface weathered zone commonly found in granitic rocks within the southern California region, with fill materials possibly represented by the lower-end velocities (i.e., 2,037 fps).

- <u>**Velocity Layer V3**</u>:

  The third layer (V3) indicates relatively a wide range of weathered granitic bedrock, with average weighted velocities of 4,348 to 7,806 fps. This range of seismic velocities indicates the likelihood of scattered buried fresh large boulders and/or dikes within a moderately decomposed matrix or possibly a moderate to slightly weathered intact rock matrix with wide-spaced fracturing.

Using **Rayfract™**, a tomographic model for each seismic line was also prepared and analyzed for comparative purposes, as presented in Appendix B, which generally indicates the relative structure and velocity distribution. The models were prepared to display the same relative color intensities for the respective velocities so that they may be comparable across the site. Although no discrete velocity layers or boundaries are created, these models generally resemble the **SIP** analysis. **Rayfract™** allows imaging of subsurface velocity using first break energy propagation modeling. It can be seen in these tomographic models that the seismic velocity (which generally relates to hardness) of the bedrock and/or older alluvial deposits gradually increases with depth which is most likely the representative condition of the subsurface materials, along with some lateral variations suggestive of buried corestones and/or dikes. It was also noted that for the most part, the seismic velocities on the Layer Velocity Profiles (Appendix A) appears to generally correlate with the average of the velocity gradients as shown on the Tomographic Models (Appendix B).

GENERALIZED RIPPABILITY CHARACTERISTICS OF GRANITIC BEDROCK

A summary of the generalized rippability characteristics of granitic bedrock based on rippability performance charts prepared by Caterpillar, Inc. (2000 and 2004) has been provided to aid in evaluating potential excavation difficulties with respect to the seismic velocities obtained along the local areas surveyed. The velocity ranges described below are approximate and assume typical, good-working, heavy excavation equipment, such as single shank or D9R dozer, such as described by Caterpillar, Inc. (2000 and 2004); however, different excavating equipment (i.e., trenching equipment) may not correlate well with these velocity ranges. Trenching operations within granitic bedrock materials with seismic velocities generally greater than 3,500 to 4,000±-fps, typically encounter very difficult to non-productable conditions. A summary of excavation considerations has been included in Appendix C in order to provide the client with a better understanding of the complexities of excavation in granitic bedrock materials. These concepts should be understood so that proper planning and excavation techniques can be employed by the selected grading contractor.

- ***Rippable Condition (0 - 4,000 ft/sec)***:

  This velocity range indicates rippable materials which may consist of alluvial-type deposits and decomposed granitics, with random hardrock floaters. These materials will break down into slightly silty, well-graded sand, whereas floaters will require special disposal. Some areas containing numerous hardrock floaters may present utility trench problems. Large floaters exposed at or near finished grade may present problems for footing or infrastructure trenching.

- **Marginally Rippable Condition (4,000 - 8,000 ft/sec)**:

  This range of velocities indicates materials which may consist of slightly- to moderately-weathered granitics or large areas of fresh granitics separated by weathered fractured zones. These materials are generally rippable with difficulty by a Caterpillar D9R or equivalent. Excavations may produce material that will partially break down into a coarse, slightly silty to clean sand, with a high percentage of very coarse sand to pebble-sized material. Less fractured or weathered materials will probably require blasting to facilitate removal.

- **Non-Rippable Condition (8,000 ft/sec or greater)**:

  This velocity range includes non-rippable material consisting primarily of moderately fractured granitics at lower velocities and only slightly fractured or unfractured rock at higher velocities. Materials in this velocity range may be marginally rippable, depending upon the degree of fracturing and the skill and experience of the operator. Tooth penetration is often the key to ripping success, regardless of seismic velocity. If the fractures and joints do not allow tooth penetration, the material may not be ripped effectively; however, pre-blasting or "popping" may induce sufficient fracturing to permit tooth entry. In their natural state, materials with these velocities are generally not desirable for building pad grade, due to difficulty in footing and utility trench excavation. Blasting will most likely produce oversized material, requiring special disposal.

SUMMARY OF FINDINGS AND CONCLUSIONS

The raw field data was considered to be of moderately good quality which had only minor amounts of ambient "noise" that was introduced during our survey from distant vehicular traffic and periodic wind sources. Analysis of the data and picking of the primary "P"-wave arrivals was performed with little difficulty and occasional interpolation of data was necessary. Based on the results of our comparative seismic analyses of both **SIP** and **Rayfract™** (of which both software programs use exactly the same input data), the seismic refraction survey lines appear to generally coincide with one another, with some minor variances due to the methods that these programs process and integrate the input data. The anticipated excavation potentials of the velocity layers encountered locally during our survey are as follows:

- **Velocity Layer V1**:

  No major excavating difficulties are expected to be encountered within the uppermost, low-velocity layer V1 (velocity range of 1,418 to 1,605 fps). This layer is expected to be comprised of topsoil, colluvium, fill, and/or older alluvial deposits.

- **Velocity Layer V2**:

  The second layer V2 is most likely consists of highly- to moderately-weathered granitic bedrock and/or moderately indurated older alluvial deposits (velocity range of 2,037 to 3,397 fps), along with localized fill materials, of which we understand are present locally within the subject property. These materials are expected to excavate with only slight difficulty assuming appropriate good-working equipment for the proposed type of excavation. Isolated floaters (i.e., boulders, corestones, etc.) could be present within weathered granitic bedrock based on surficial exposures in the local region and could produce difficult conditions locally. Placement of infrastructures in this material may also be difficult. Although not anticipated, localized blasting in the bedrock materials due to the presence of buried boulders and dikes cannot be completely ruled out.

- **Velocity Layer V3**:

  Some excavation difficulties within the lower V3 velocity layer (velocity range of 4,348 to 7,806 fps) are anticipated, where slightly- to moderately-weathered granitic bedrock is encountered approaching the higher-end velocities. Hard excavating areas consisting of localized boulders, dikes, and/or fresher bedrock with relatively wide-spaced jointing/fracturing could be encountered during both remedial grading and placement of infrastructures, which may require some blasting to achieve desired grade. Excavations performed within the older alluvial deposits, if present, are not expected to encounter difficult conditions which would require blasting.

Based on the Tomographic Models (Appendix B) and typical excavation characteristics that have observed within granitic bedrock materials of the southern California region, anticipation of gradual increasing hardness with depth along with localized lateral variations, with respect to excavation characteristics, should be anticipated across the site. It may be expected that when ground velocities on the order of 6,000± fps or greater are encountered, increasing difficulties in excavation conditions and rippability will occur with respect to grading production. These increases may result in slower production rates from the cut excavation with an increase in the generation of oversized rock materials. This is also dependent upon the type and operating condition of the excavation equipment used, how hard the contractor is willing to work the equipment, and the structural discontinuities of the rock fabric. The decision for blasting of the rock for excavatability is sometimes made based upon economic production reasons and not solely on the rippability (velocity/hardness) characteristics of the bedrock.

CLOSURE

This survey was performed using "state of the art" geophysical techniques, computer processing, and equipment, in the localized areas delineated by you. We make no warranty, either expressed or implied. It should be noted that our data was obtained along four specific areas; therefore, other local areas within the site beyond the limits of our seismic lines may contain different velocity layers and depths not encountered during our field survey. Estimates of layer velocity boundaries are generally considered to be within 10±-percent of the depth of the contact. It should be understood that when using these theoretical geophysical principles and techniques, sources of error are possible in both the data obtained and in the interpretation. In summary, the results of this survey are to be considered as an aid to assessing the rippability potentials of the bedrock locally. This information should be carefully reviewed by the grading contractor and representative "test" excavations should be considered, so that they may be correlated with the data presented within this report.

If you should have any questions regarding this report or do not understand the limitations of this survey, please do not hesitate to contact our office.

Respectfully submitted,
TERRA GEOSCIENCES

Donn C. Schwartzkopf
Principal Geophysicist
PGP 1002

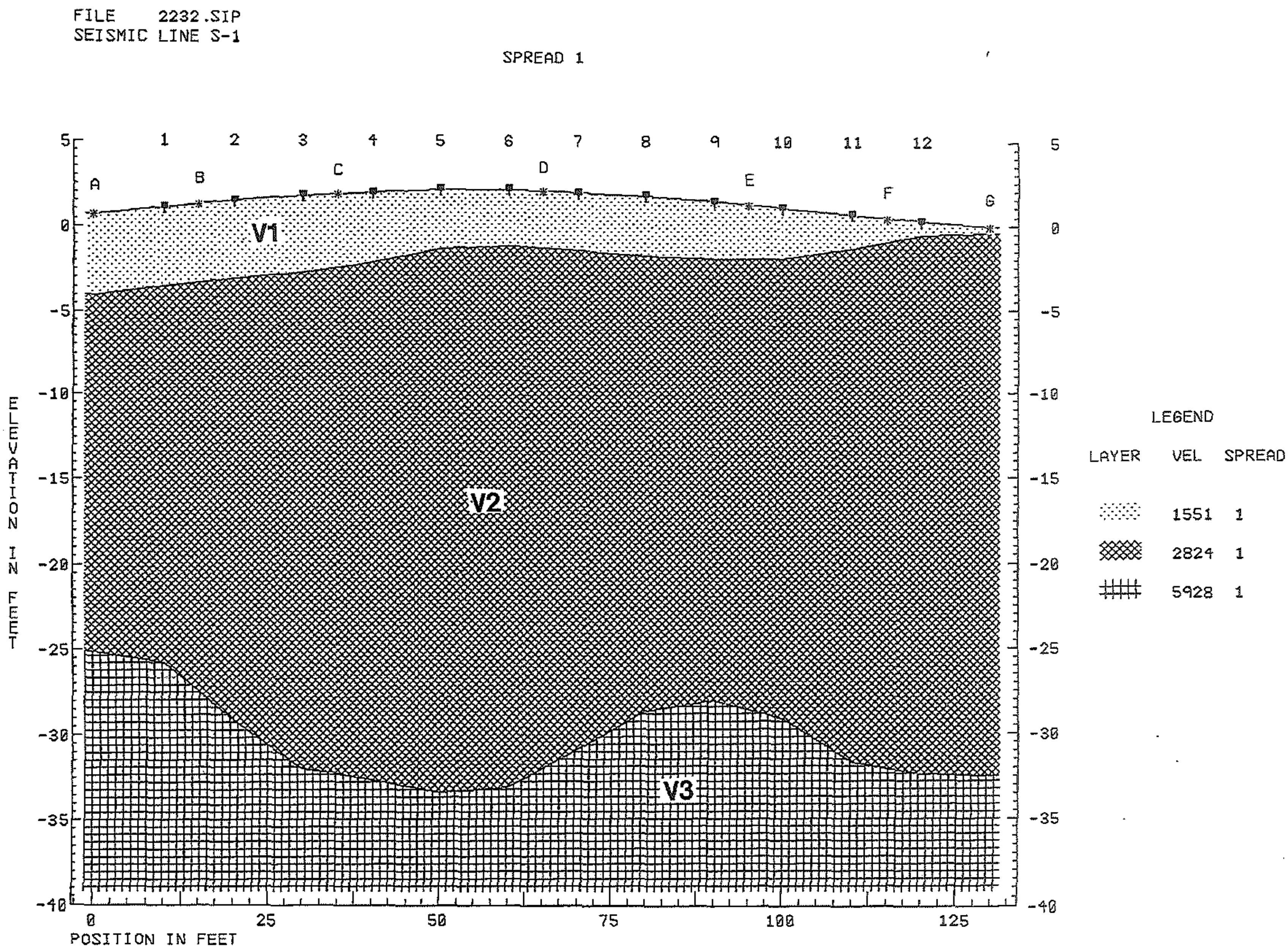
PROFESSIONAL GEOPHYSICIST
DONN C. SCHWARTZKOPF
No. 1002
STATE OF CALIFORNIA

APPENDIX A

LAYER VELOCITY PROFILES

LAYER VELOCITY PROFILE S-1

← North - South →

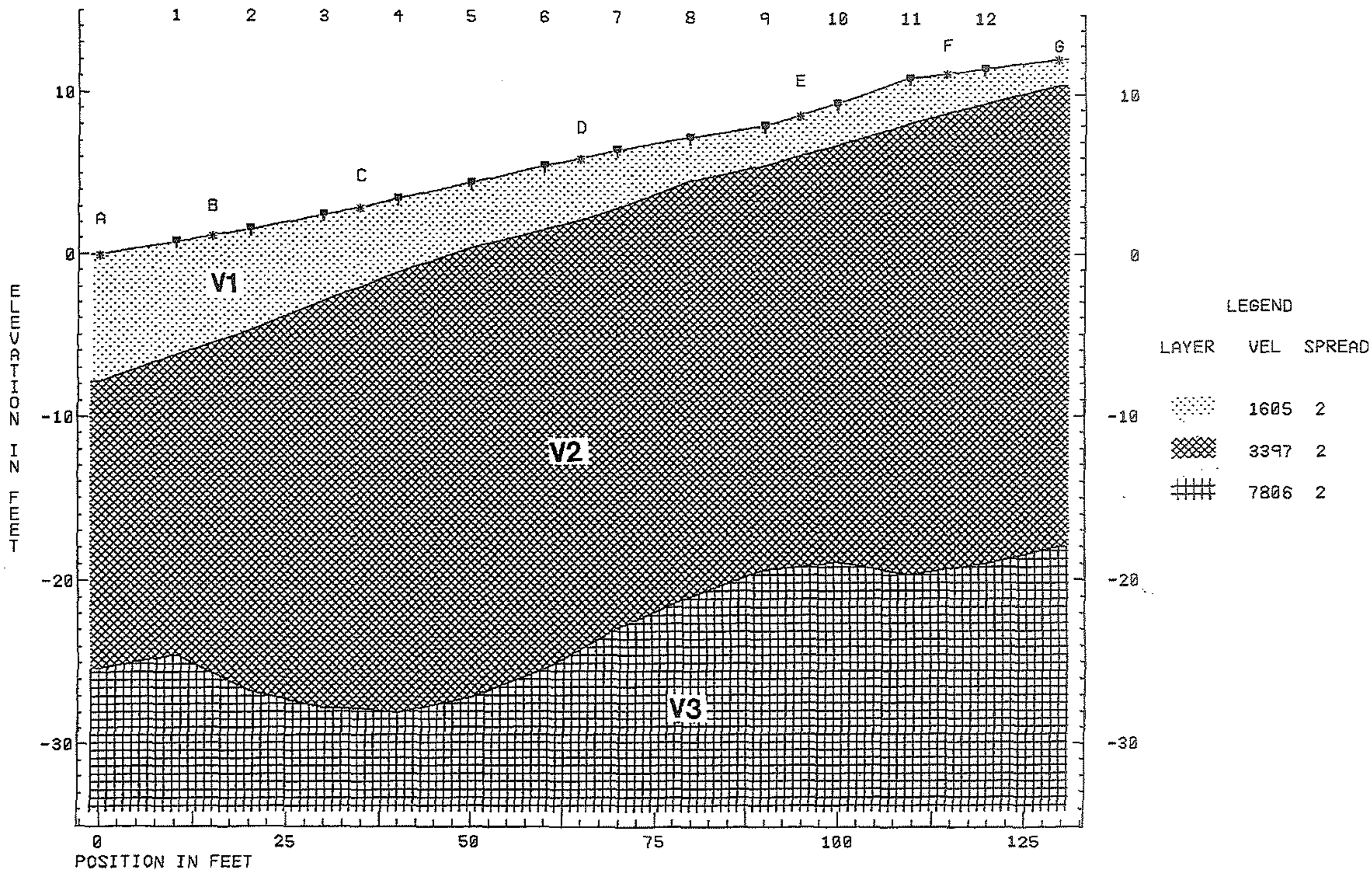


LAYER VELOCITY PROFILE S-2

← North - South →

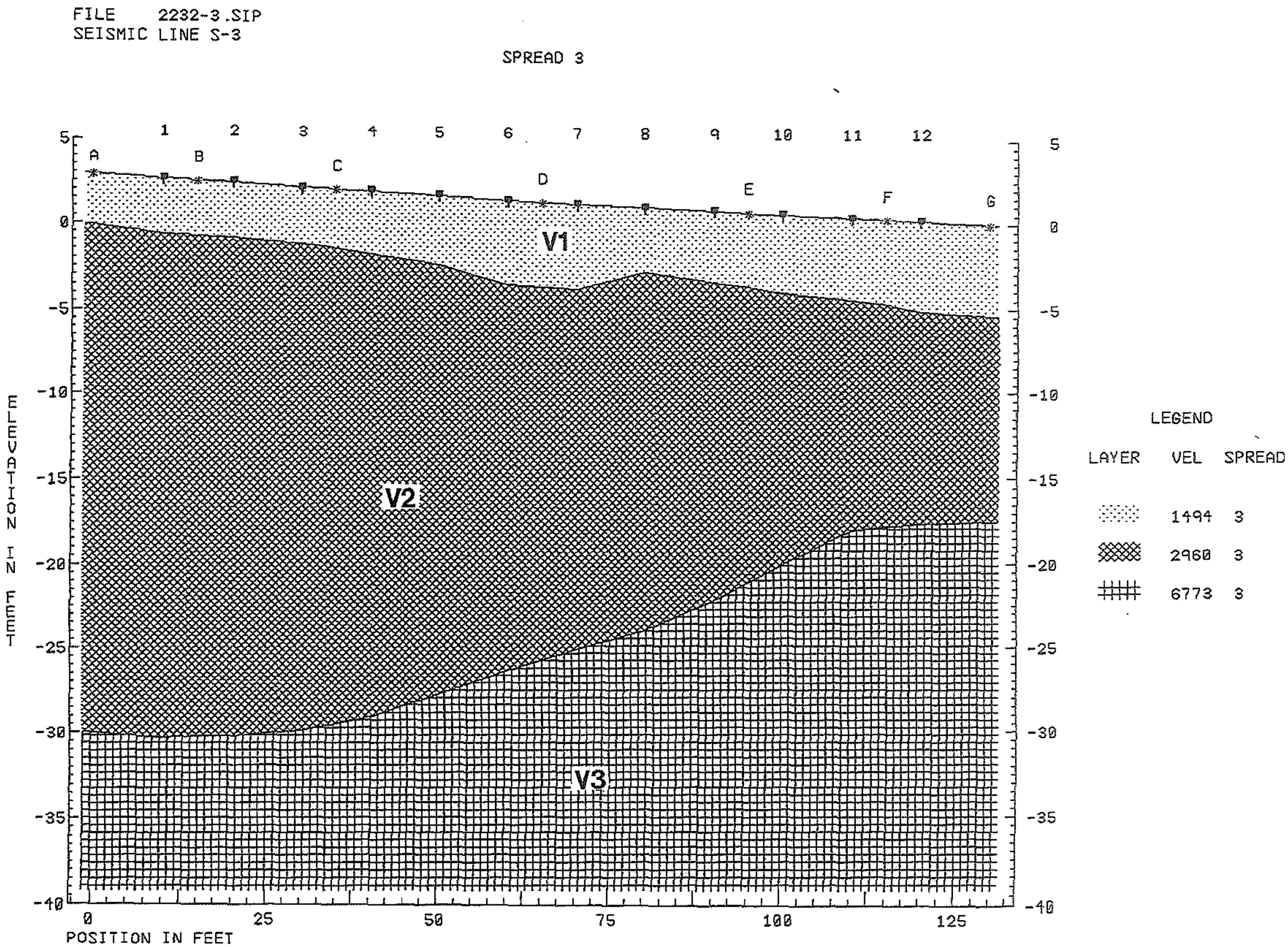
FILE 2232-2.SIP
SEISMIC LINE S-2

SPREAD 2



LAYER VELOCITY PROFILE S-3

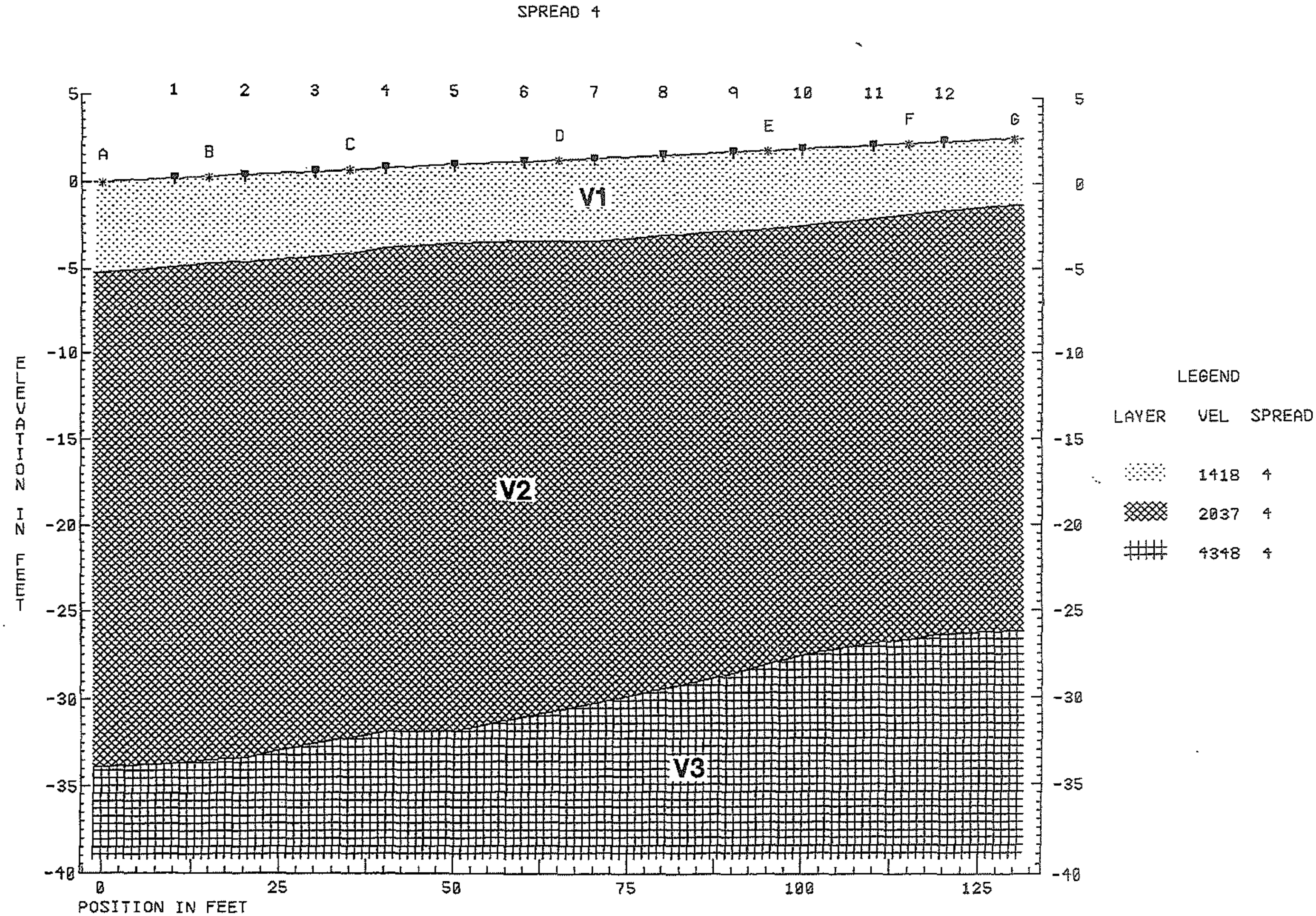
← East - West →



LAYER VELOCITY PROFILE S-4

← West - East →

FILE 2232-4.SIP
SEISMIC LINE S-4



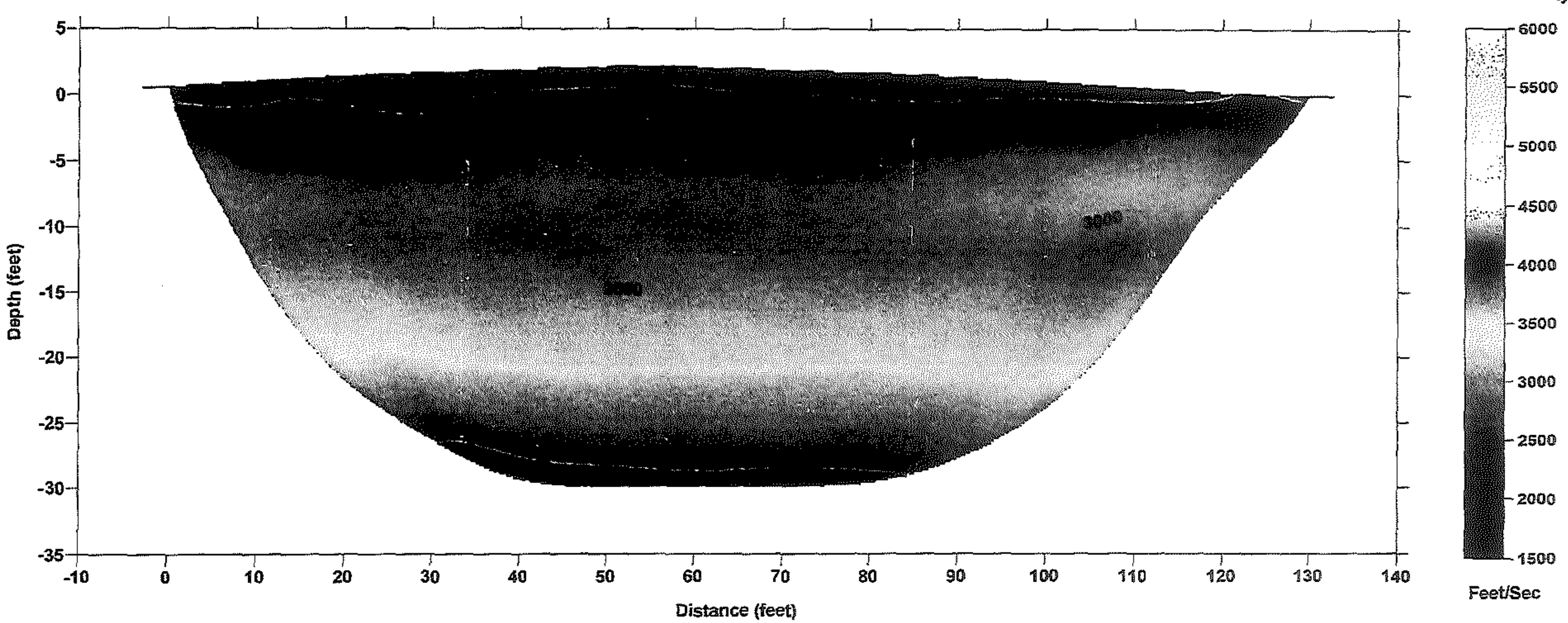
APPENDIX B

TOMOGRAPHIC MODELS

SEISMIC LINE S-1

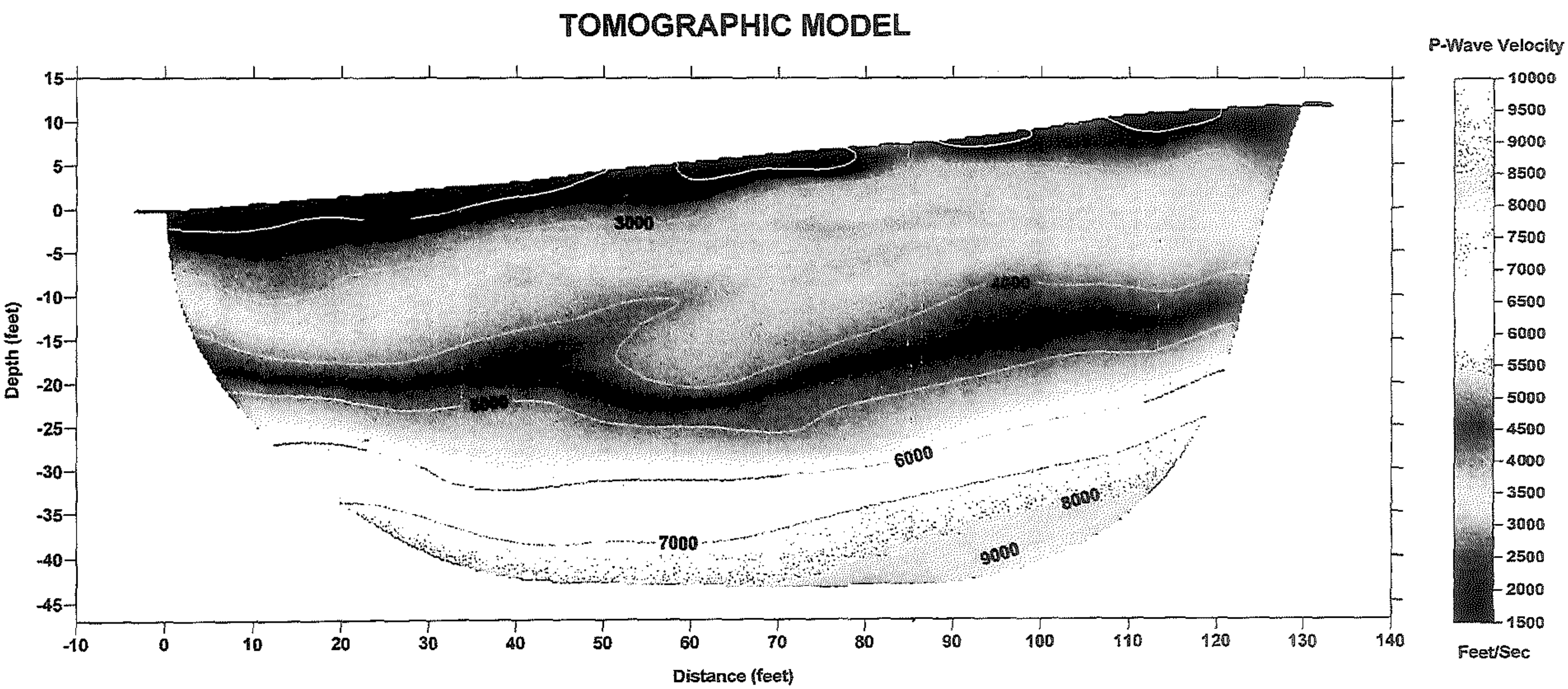
← North - South →

TOMOGRAPHIC MODEL



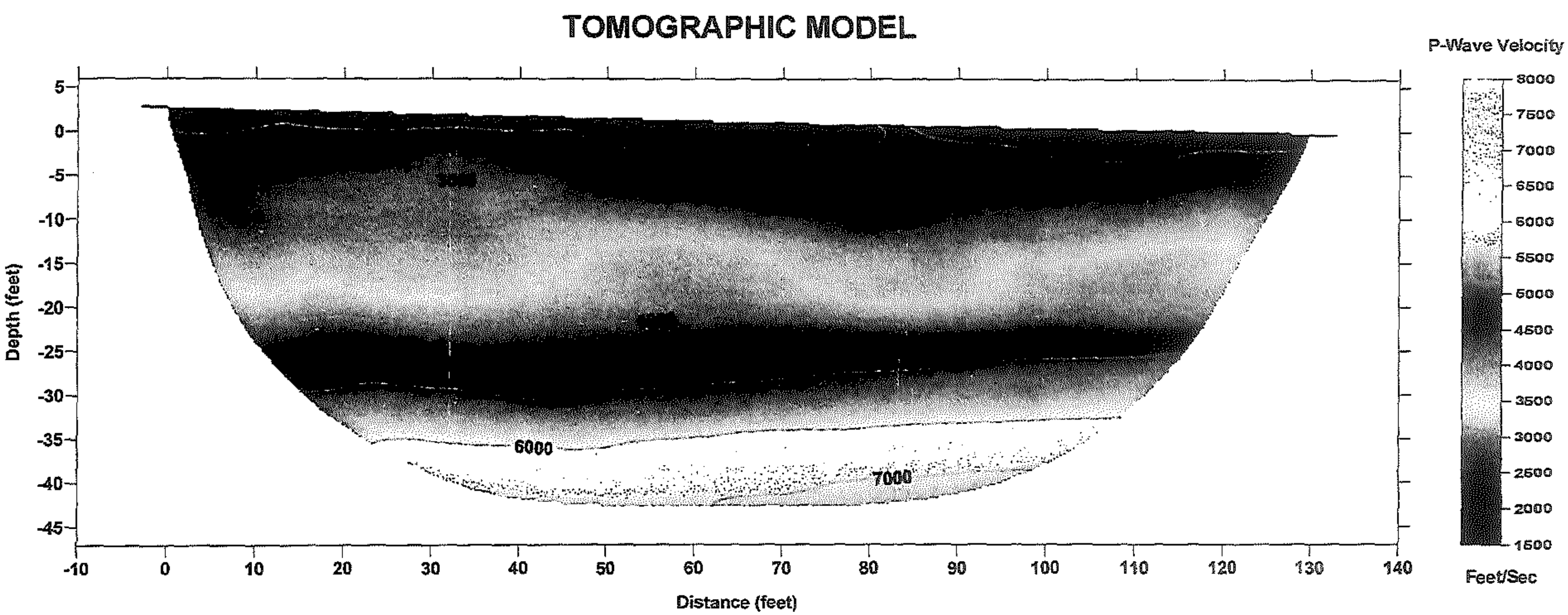
SEISMIC LINE S-2

← North - South →



SEISMIC LINE S-3

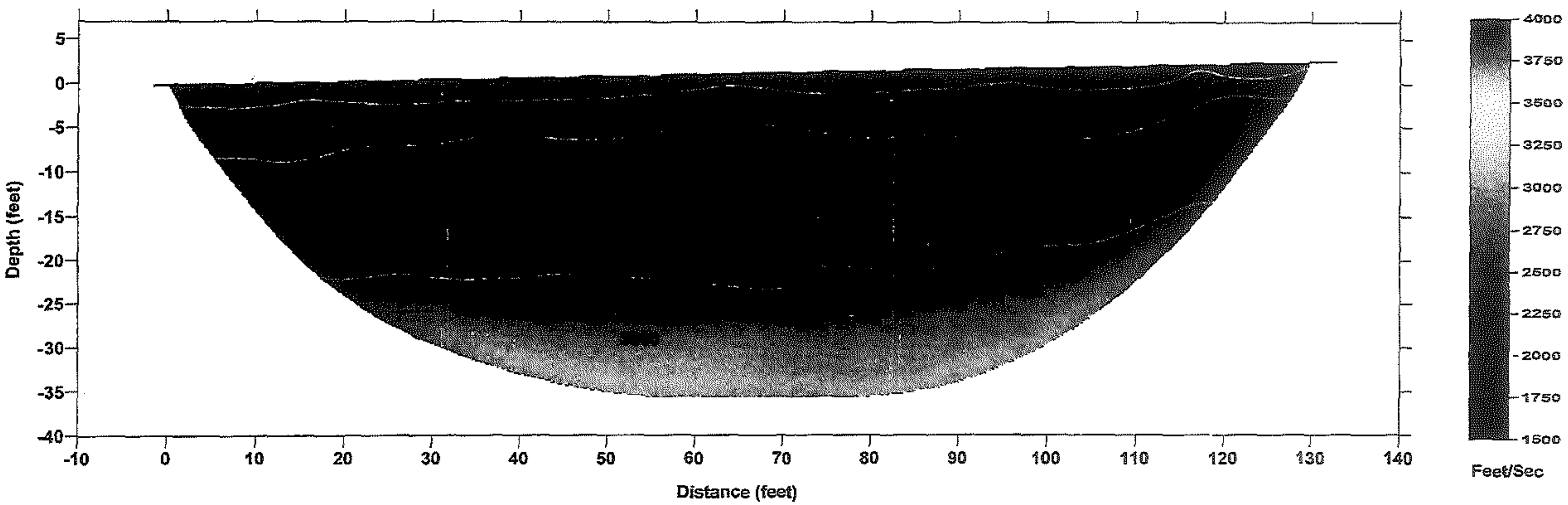
← East - West →



SEISMIC LINE S-4

← West - East →

TOMOGRAPHIC MODEL



APPENDIX C

EXCAVATION CONSIDERATIONS

EXCAVATION CONSIDERATIONS

These excavation considerations have been included to provide the client with a brief overall summary of the general complexity of hard bedrock excavation. It is considered the clients responsibility to insure that the grading contractor they select is both properly licensed and qualified, with experience in hard-bedrock ripping processes. To evaluate whether a particular bedrock material can be ripped, this geophysical survey should be used in conjunction with the geologic or geotechnical report prepared for the project which describes the physical properties of the bedrock. The physical characteristics of bedrock materials that favor ripping generally include the presence of fractures, faults and other structural discontinuities, weathering effects, brittleness or crystalline structure, stratification of lamination, large grain size, moisture permeated clay, and low compressive strength. Unfavorable conditions can include such characteristics as massive and homogeneous formations, non-crystalline structure, absence of planes of weakness, fine-grained materials, and formations of clay origin where moisture makes the material plastic.

When assessing the potential rippability of the underlying bedrock of a given site, the above geologic characteristics along with the estimated seismic velocities can then be used to evaluate what type of equipment may be appropriate for the proposed grading. When selecting the proper ripping equipment there are three primary factors to consider, which are:

- **Down Pressure available at the tip, which determines the ripper penetration that can be attained and maintained,**
- **Tractor flywheel horsepower, which determines whether the tractor can advance the tip, and,**
- **Tractor gross-weight, which determines whether the tractor will have sufficient traction to use the horsepower.**

In addition to selecting the appropriate tractor, selection of the proper ripper design is also important. There are basically three designs, being radial, parallelogram, and adjustable parallelogram, of which the contractor should be aware of when selecting the appropriate design to be used for the project. The penetration depth will depend upon the down-pressure and penetration angle, as well as the length of the shank tips (short, intermediate, and long).

Also important in the excavation process is the ripping technique used as well as the skill of the individual tractor operator. These techniques include the use of one or more ripping teeth, up- and down-hill ripping, and the direction of ripping with respect to the geologic structure of the bedrock locally. The use of two tractors (one to push the first tractor-ripper) can extend the range of materials that can be ripped. The second tractor can also be used to supply additional down-pressure on the ripper. Consideration of light blasting can also facilitate the ripper penetration and reduce the cost of moving highly consolidated rock formations.

All of the combined factors above should be considered by both the client and the grading contractor, to insure that the proper selection of equipment and ripping techniques are used for the proposed grading.

APPENDIX D

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REFERENCES

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APPENDIX B

APPENDIX B

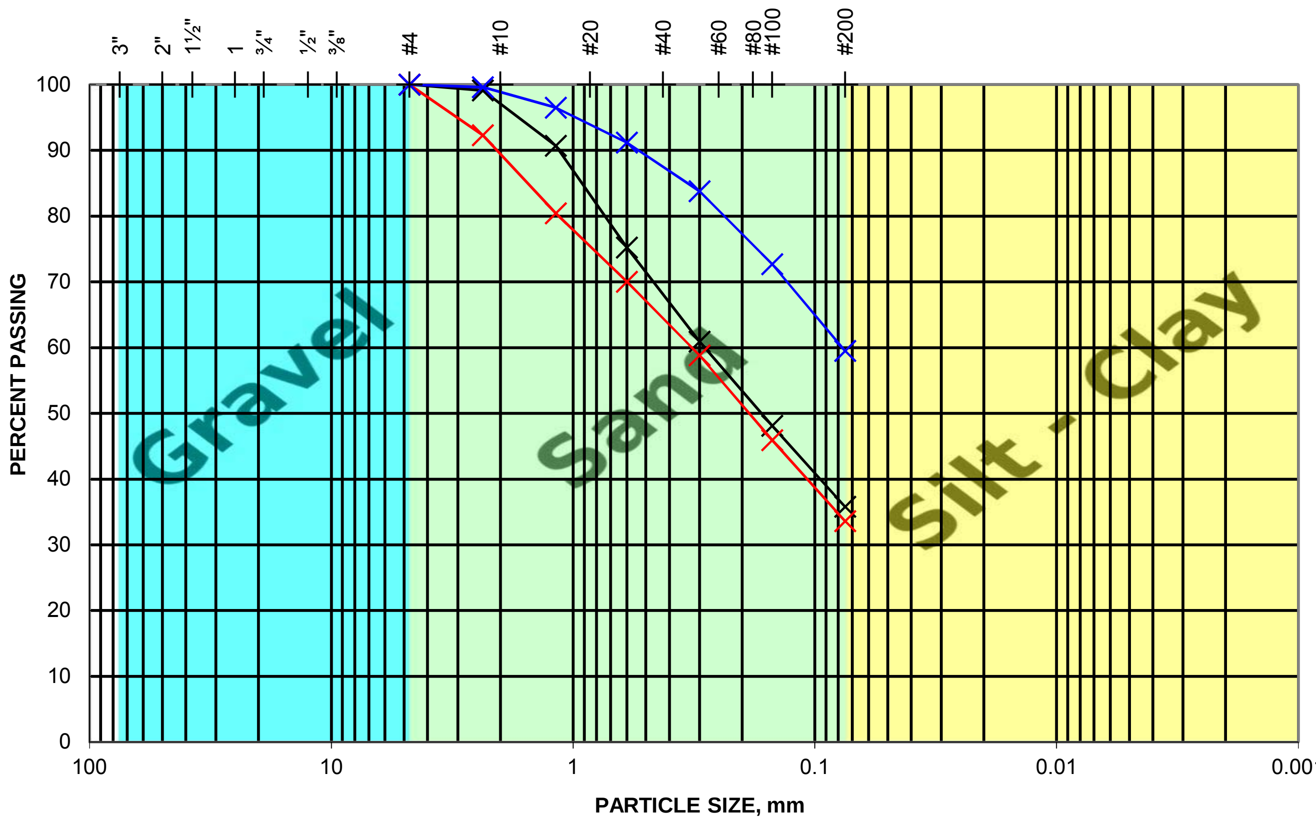
LABORATORY TESTING

We performed laboratory tests in accordance with current generally accepted test methods of ASTM International (ASTM) or other suggested procedures. The results of the laboratory tests are presented in *Appendix B.*

SUMMARY OF CORROSIVITY TEST RESULTS

Sample No.	Chloride Content (ppm)	Sulfate Content (%)	pH	Resistivity (ohm-centimeter)
IT-1 @ 0-1’	50	--	7.6	3,000

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



SAMPLE ID	SAMPLE DESCRIPTION
IT-1 @ 4-5'	SC-SM - Silty Clayey SAND
IT-2 @ 3-4'	SC-SM - Silty Clayey SAND
IT-3 @ 4-5'	CL - Sandy CLAY

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RIVERSIDE COUNTY, CALIFORNIA

AMO			AUGUST, 2017	PROJECT NO. T2788-22-01	FIG B-2

APPENDIX B

LABORATORY TESTING

Laboratory tests were performed in accordance with generally accepted test methods of the American Society for Testing and Materials (ASTM) or other suggested procedures. Selected ring samples were tested to determine their in-place density and moisture content. Disturbed bulk samples were tested to determine compaction (maximum dry density and optimum moisture content), remolded direct shear strength, expansion characteristics, and water soluble sulfate content. The results of laboratory tests performed are summarized in tabular and graphical form herewith.

TABLE B-I
SUMMARY OF LABORATORY MAXIMUM DRY DENSITY AND OPTIMUM MOISTURE CONTENT TEST RESULTS
ASTM D 1557-02

Sample No.	Description	Maximum Dry Density (pcf)	Optimum Moisture Content (% dry wt.)
T1-1	Brown, fine to medium sandy Clay	121.7	12.9
T5-1	Brown, silty Clay with little sand	111.3	17.1
T12-1	Grayish brown, clayey, fine to medium Sand	127.7	11.2
T12-2	Gray, sandy, fine to coarse Gravel with trace clay	121.7	13.6

TABLE B-II
SUMMARY OF LABORATORY DIRECT SHEAR TEST RESULTS
ASTM D 3080-03

Sample No.	Dry Density (pcf)	Moisture Content (%)	Unit Cohesion (psf)	Angle of Shear Resistance (degrees)
T1-1	109.1	13.3	235	22

Sample remolded to approximately 90 percent maximum dry density near optimum moisture content

TABLE B-III
SUMMARY OF LABORATORY EXPANSION INDEX TEST RESULTS
ASTM D 4829-88

Sample No.	Moisture Content		Dry Density (pcf)	Expansion Index
	Before Test (%)	After Test (%)		
T1-1	10.6	24.9*	110.0	60
T12-1	10.7	27.1	108.1	61

TABLE B-IV
SUMMARY OF LABORATORY WATER–SOLUBLE SULFATE TEST RESULTS
CALIFORNIA TEST 417

Sample No.	Water Soluble Sulfate (%)
T1-1	0.003

GEOCON

INLAND EMPIRE 41571 Corning place Suite 101MURRIETA CA 92562

MOISTURE DENSITY& MOISTURE CONTENT

PROJECT NAME	FLEMMING PROPERTY
PROJECT NUMBER	7178-42-01
DATE	7/26/2007
TECHNICIAN	JD

SAMPLE	T2-1	T3-1	T4-1	T5-2	T7-1			
HT. OF SAMPLE	2	1	1	1	1			
GROSS WET WT	363.5	192.2	177.2	199.6	184.0			
TARE	88.5	44.7	44.0	44.0	43.7			
RING DIAMETER	2.420	2.420	2.420	2.420	2.420	2.375	2.375	2.375
WET DENSITY	113.9	122.2	110.3	128.9	116.2			
WET WEIGHT	100.0	100.0	100.0	100.0	100.0			
DRY WEIGHT	82.3	82.1	83.9	87.0	87.9			
% MOISTURE	21.5	21.8	19.2	14.9	13.8			
DRY DENSITY	93.7	100.3	92.6	112.1	102.1			
	ML- DARK YELLOWISH BROWN CLAYEY SILT WITH TRACE FINE SAND	CL- DARK BROWN SILTY CLAY WITH TRACE FINE SAND	CL- DARK BROWN SILTY CLAY WITH TRACE FINE SAND	CL- DARK BROWN SILTY CLAY WITH TRACE FINE SAND	CL- DARK BROWN SILTY CLAY WITH TRACE FINE SAND			

GEOCON INCORPORATED

DIRECT SHEAR TEST REPORT

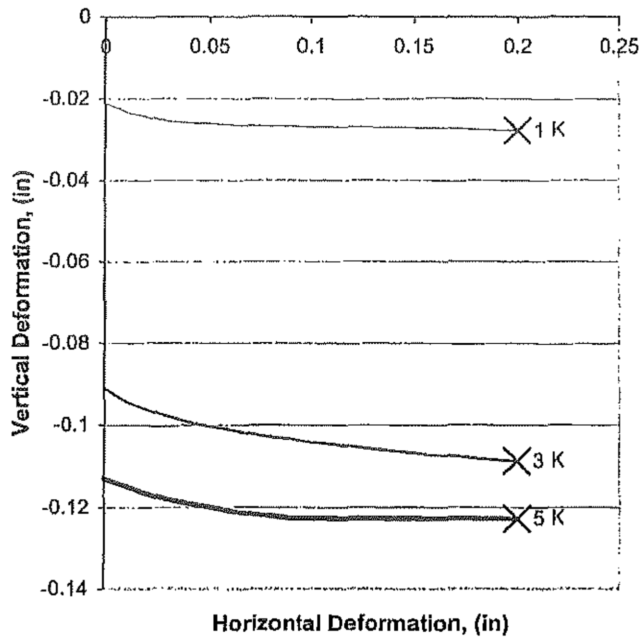
FLEMMING PROPERTY

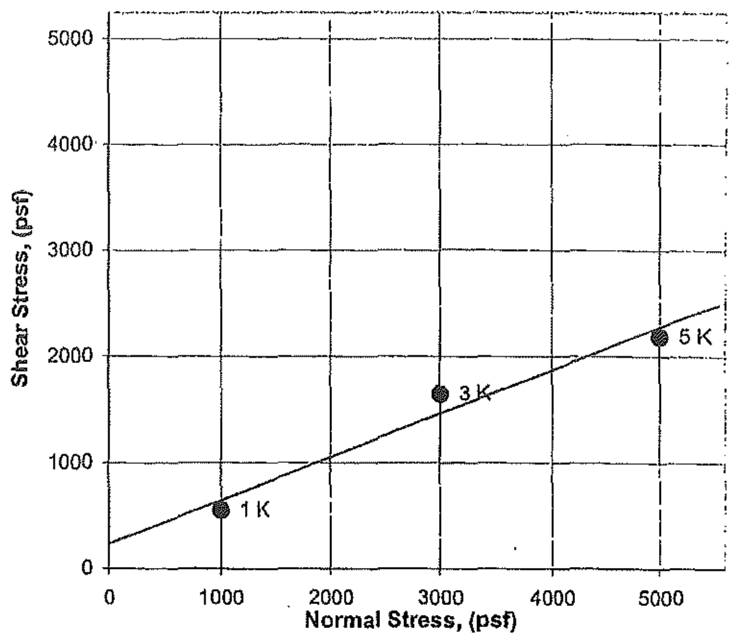
7178-42-01 **Date:** Thursday, July 26, 2007 **By:** JD

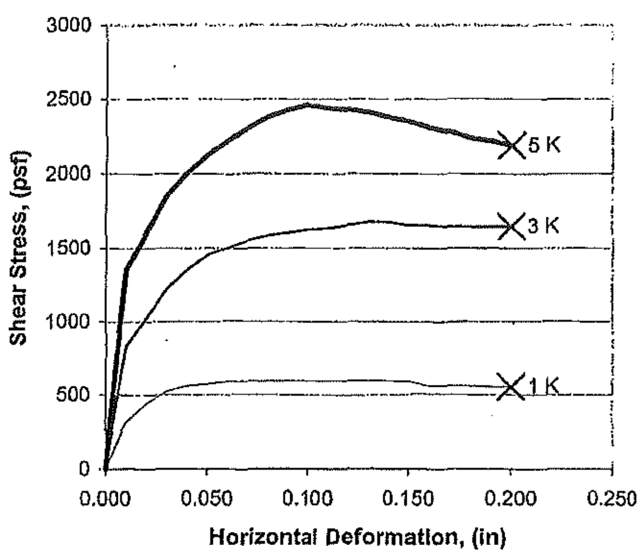
Sample No.: T1-1 **Natural or Remold:** Remolded

Description: CL- brown (f-m) sandy clay

Remarks:







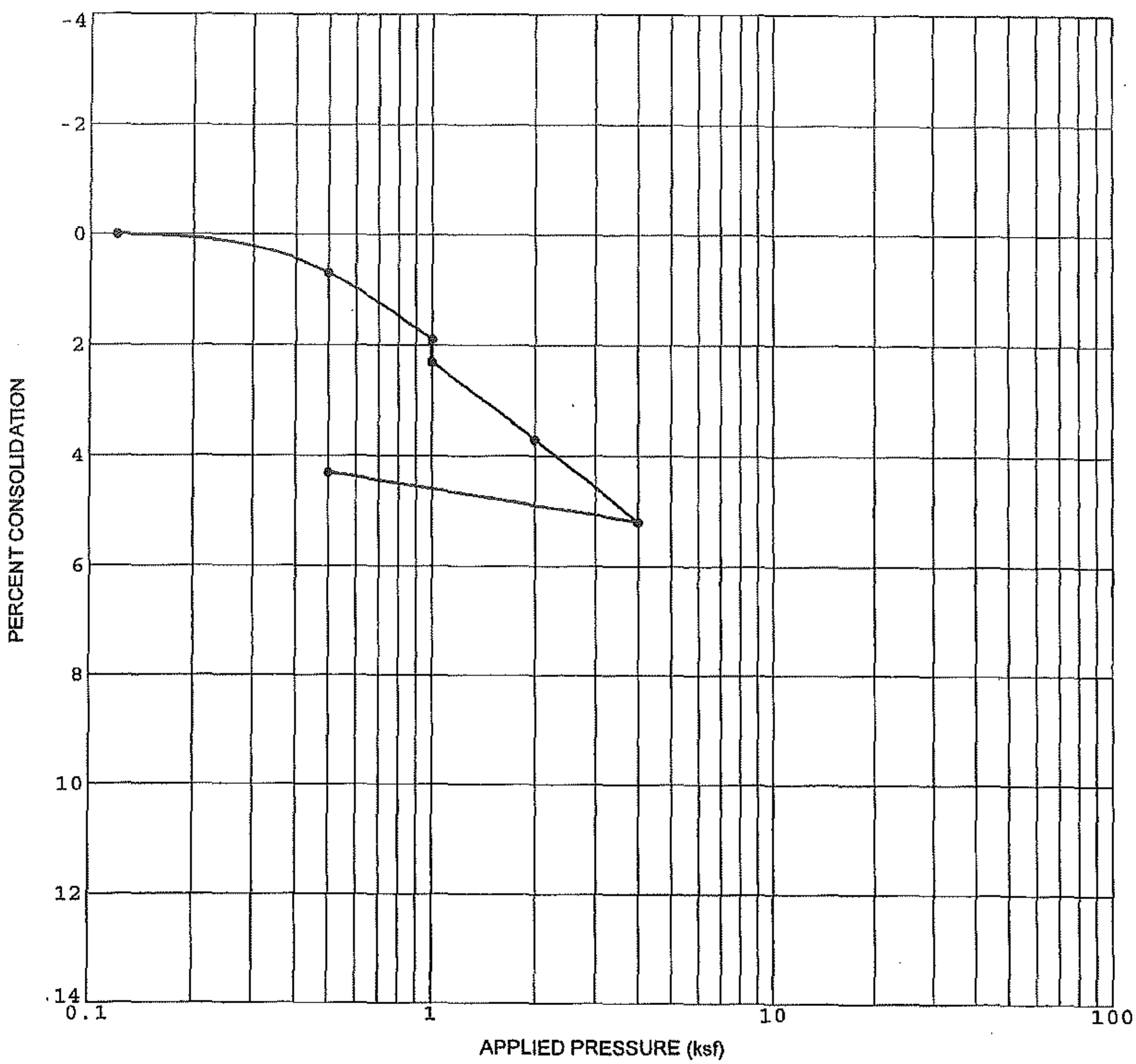
φ (Degrees)	22.2
c (psf)	235
Tan φ	0.409
Method	Calc

Load	1 K	3 K	5 K
INITIAL			
Water Content	12.5%	13.3%	13.9%
Dry Density (pcf)	109.9	109.1	108.5
Saturation*	65.7%	68.4%	70.5%
Height (inches)	1.00	1.00	1.00
AFTER TEST			
Water Content	22.9%	24.3%	24.9%
Dry Density (pcf)	113.0	122.4	123.7
FAILURE			
Normal Stress (psf)	1000	3000	5000
Failure Stress (psf)	552	1645	2187
Failure Deffinition	User	User	User
Displacement (in)	0.20	0.20	0.20
Rate (in/min)	0.0150	0.0150	0.0150

* Degree of saturation calculated with a specific gravity of 2.65

PROJECT NO. 07178-42-01

SAMPLE NO. T1-3



Initial Dry Density (pcf)	105.6	Initial Saturation (%)	88.6
Initial Water Content (%)	17.9	Sample Saturated at (ksf)	1.0

CONSOLIDATION CURVE

FLEMMING PROPERTY

RIVERSIDE COUNTY, CALIFORNIA

07178-42-01.GPJ

Figure B-1

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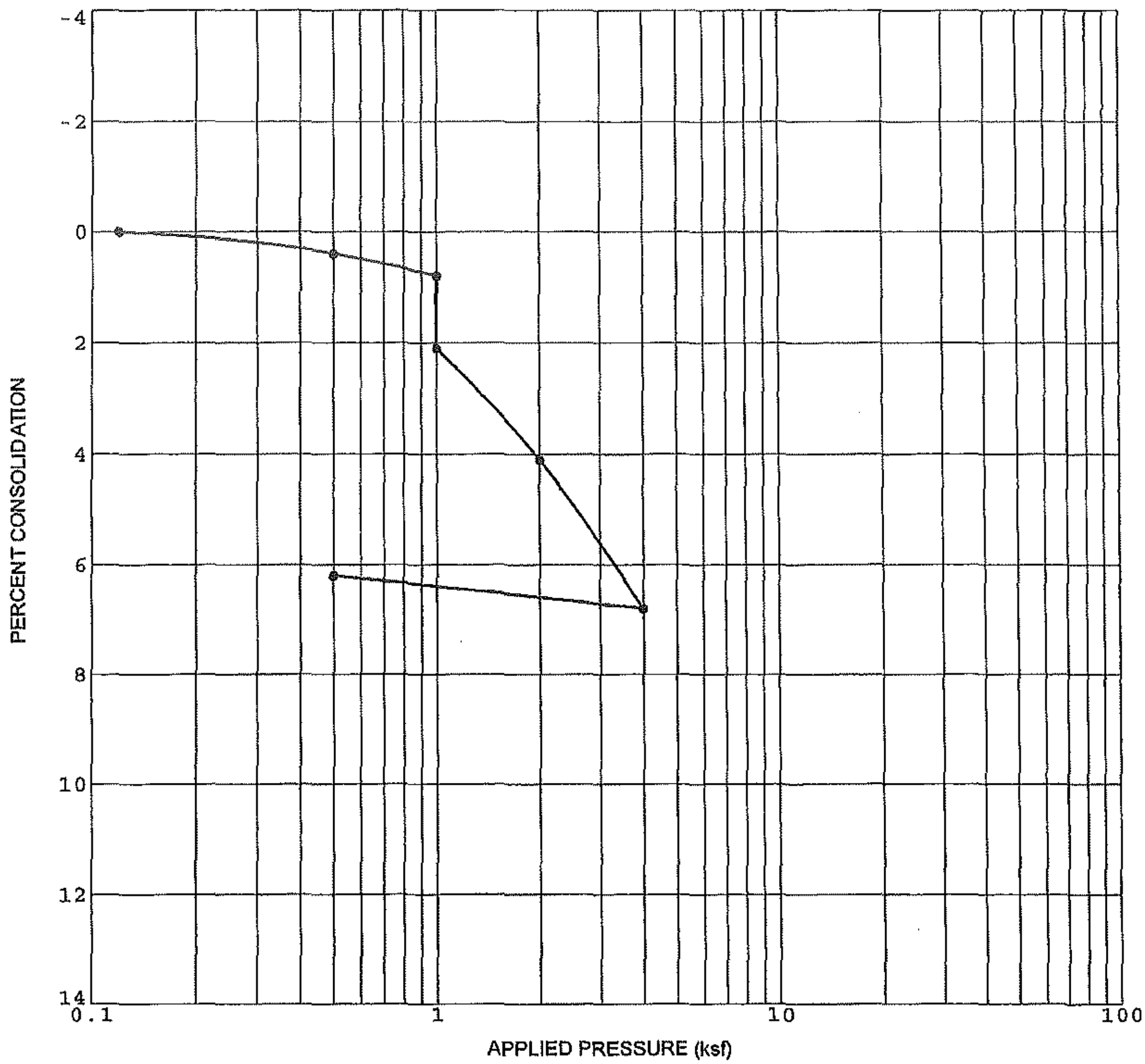
PROJECT NO. 07178-42-01

SAMPLE NO. T6-1



Initial Dry Density (pcf)	111.2		Initial Saturation (%)	72.1
Initial Water Content (%)	12.4		Sample Saturated at (ksf)	1.0

CONSOLIDATION CURVE

FLEMMING PROPERTY

RIVERSIDE COUNTY, CALIFORNIA

07178-42-01.GPJ

Figure B-2

GEOCON

APPENDIX C

APPENDIX C

RECOMMENDED GRADING SPECIFICATIONS

FOR

KTM DEVELOPMENT
NEC BOREL ROAD AND HIGHWAY 79
FRENCH VALLEY AREA
RIVERSIDE COUNTY, CALIFORNIA

PROJECT NO. T2788-22-01

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

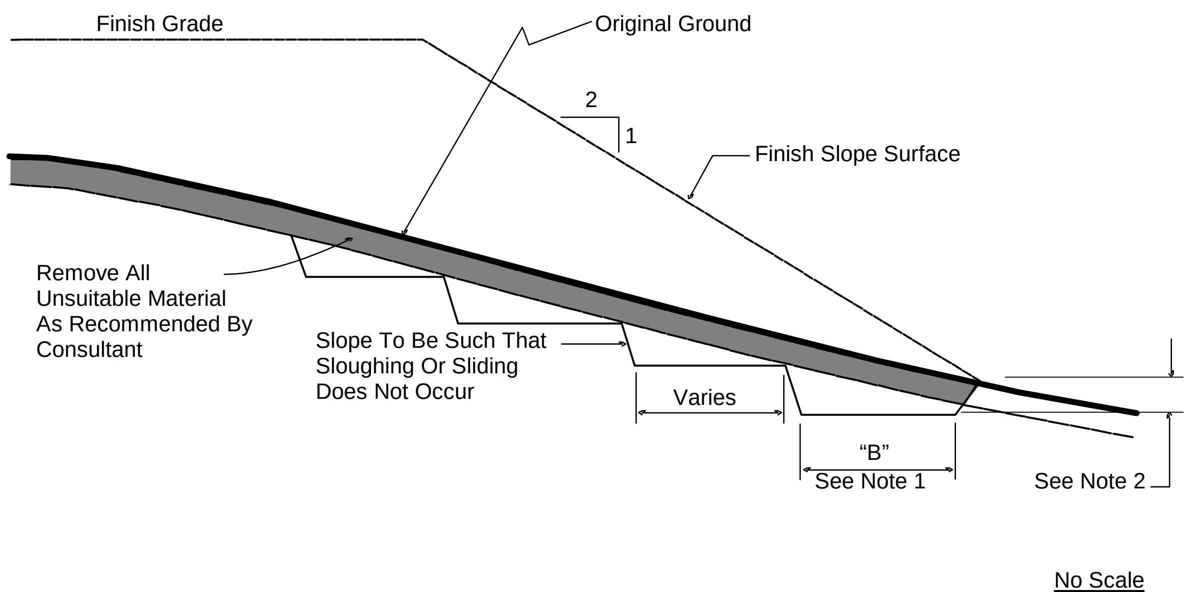
4.1 Areas to be excavated and filled shall be cleared and grubbed. Clearing shall consist of complete removal above the ground surface of trees, stumps, brush, vegetation, man-made structures, and similar debris. Grubbing shall consist of removal of stumps, roots, buried logs and other unsuitable material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

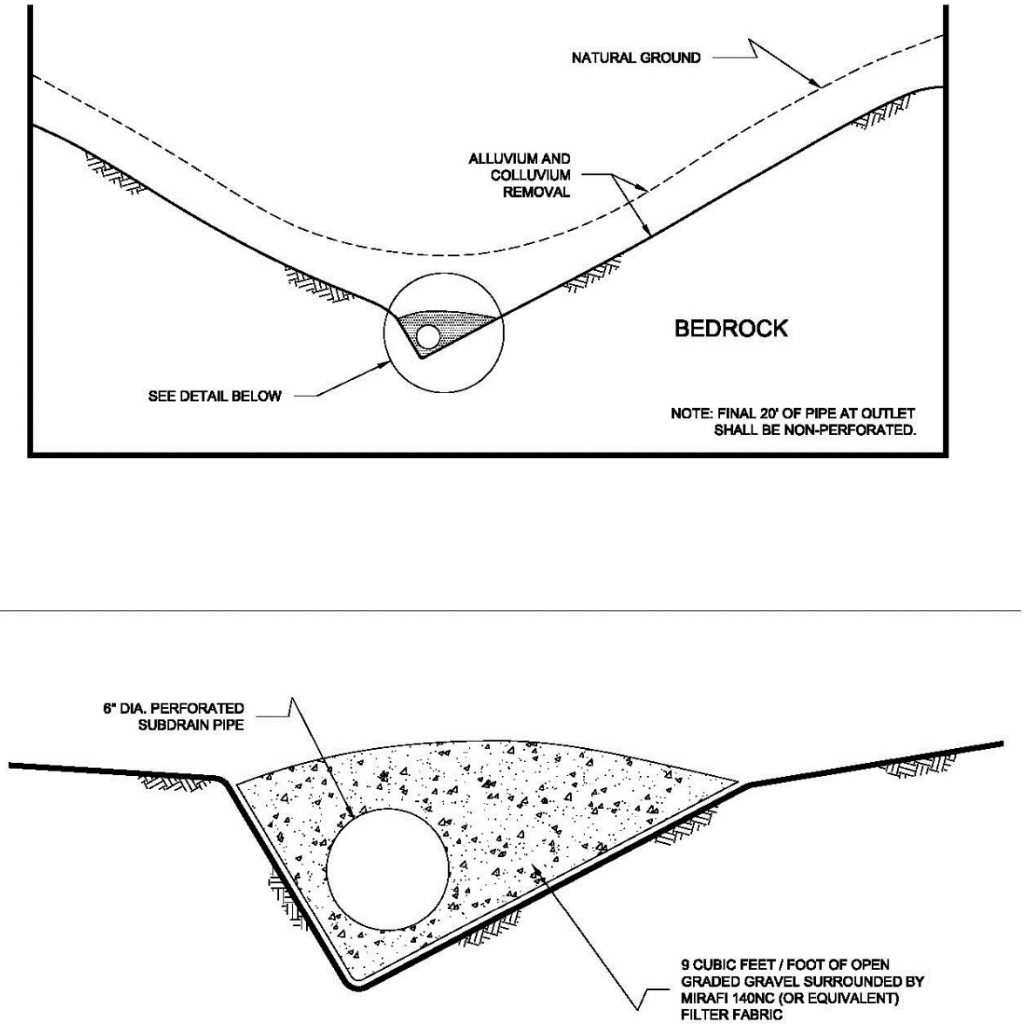
6.3.6 To reduce the potential for “piping” of fines into the *rock* fill from overlying *soil* fill material, a 2-foot layer of graded filter material shall be placed above the uppermost lift of *rock* fill. The need to place graded filter material below the *rock* should be determined by the Consultant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

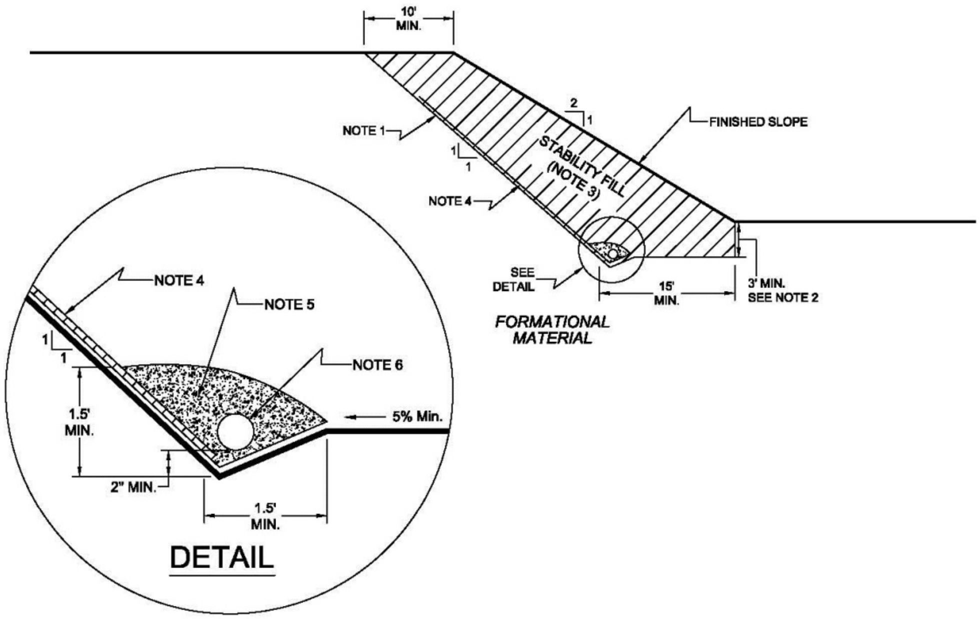
1......6-INCH DIAMETER, SCHEDULE 80 PVC PERFORATED PIPE FOR FILLS IN EXCESS OF 100-FEET IN DEPTH OR A PIPE LENGTH OF LONGER THAN 500 FEET.

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

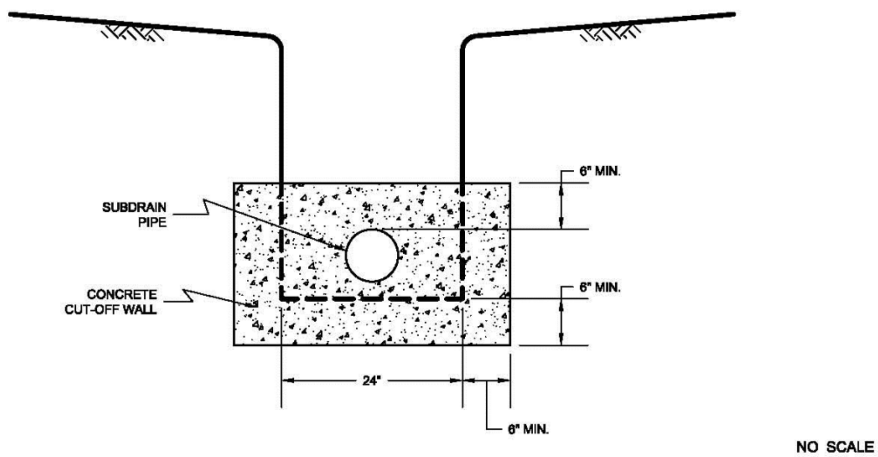
7.3 The actual subdrain locations will be evaluated in the field during the remedial grading operations. Additional drains may be necessary depending on the conditions observed and the requirements of the local regulatory agencies. Appropriate subdrain outlets should be evaluated prior to finalizing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

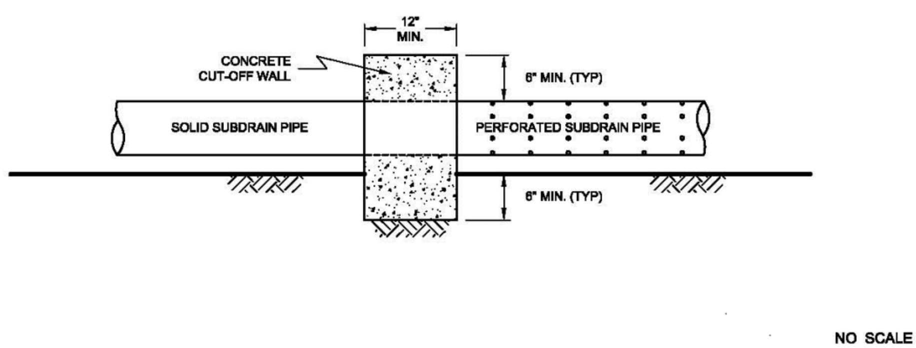
7.5 Prior to outletting, the final 20-foot segment of a subdrain that will not be extended during future development should consist of non-perforated drainpipe. At the non-perforated/ perforated interface, a seepage cutoff wall should be constructed on the downslope side of the pipe.

TYPICAL CUT OFF WALL DETAIL

FRONT VIEW

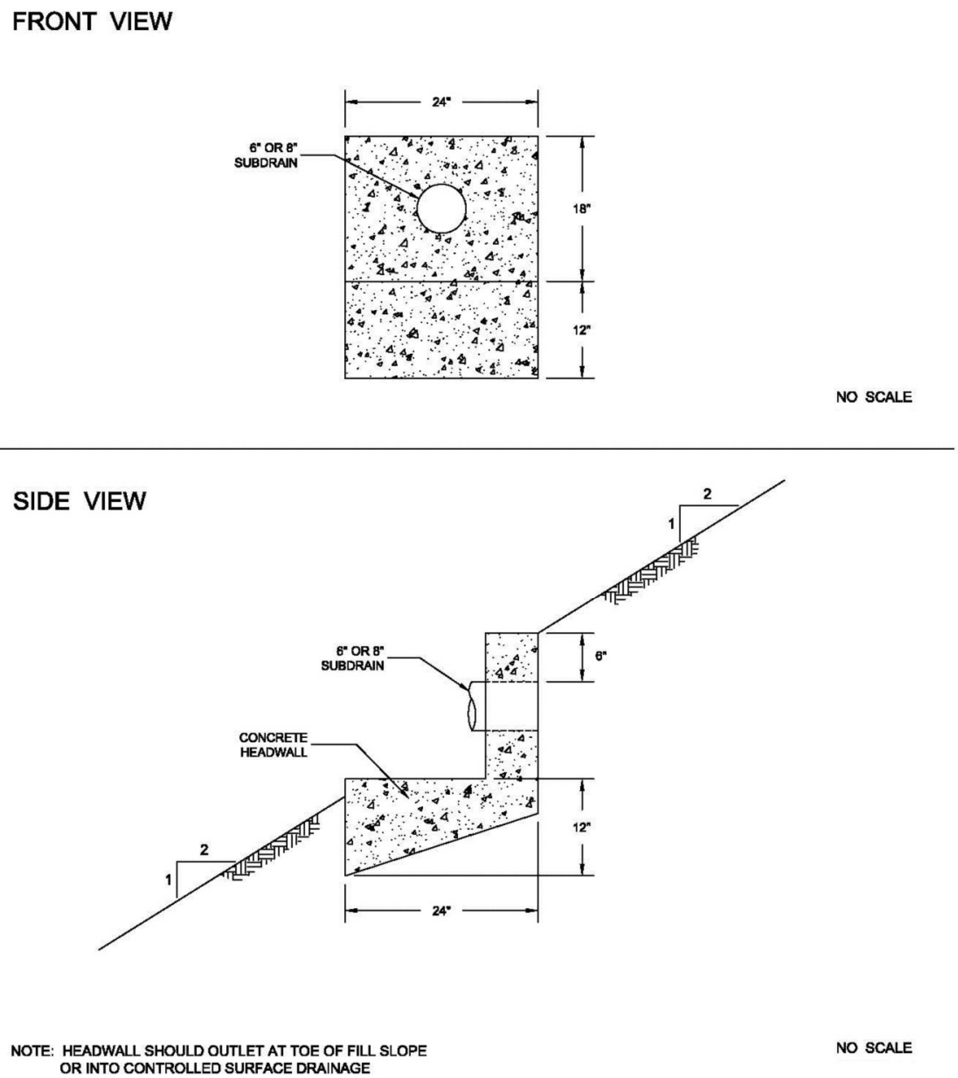


SIDE VIEW



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



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.