GEOCON
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GEOTECHNICAL
ENVIRONMENTAL
MATERIALS

UPDATE GEOTECHNICAL REPORT

TRACT 29476
WILDOMAR, CALIFORNIA

PREPARED FOR

MARKHAM DEVELOPMENT MANAGEMENT GROUP, INC.
TEMECULA, CALIFORNIA

JUNE 4, 2014
PROJECT NO. T2283-22-03

GEOCON
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Project No. T2283-22-03
June 4, 2014

Markham Development Management Group, Inc.
41635 Enterprise Circle North, Suite B
Temecula, California 92590

Attention: Ms. Sherrie Munroe

Subject: UPDATE GEOTECHNICAL REPORT
TRACT 29476
WILDOMAR, CALIFORNIA

Dear Ms. Munroe:

In accordance with your authorization and our proposal (IE-1255), we have prepared this Update Geotechnical Investigation for Tract No. 29476, a proposed residential development located in the City of Wildomar, California. Original recommendations were presented in *Geotechnical Investigation, Tract 29476, Murrieta Area, Riverside County, California,* prepared by Geocon Inland Empire Inc. and dated January 28, 2005 (Project No. T2283-12-02). The accompanying report presents the results of our review and includes our conclusions and recommendations pertaining to the geologic and geotechnical aspects of developing the property as presently proposed. It is our opinion that the site is geotechnically suitable for development of the proposed residential subdivision, provided the recommendations of this report are followed.

Should you have questions regarding this report, or if we may be of further service, please contact the undersigned at your convenience.

Very truly yours,

GEOCON WEST, INC.

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UPDATE GEOTECHNICAL REPORT

1. PURPOSE AND SCOPE

This report presents the findings of our Update geotechnical report for Tract No. 29476, a proposed 28 lot residential subdivision located in the Murrieta area, Riverside County, California (see Vicinity Map, Figure 1). Original recommendations were presented in *Geotechnical Investigation, Tract 29476, Murrieta Area, Riverside County, California,* prepared by Geocon Inland Empire Inc. and dated January 28, 2005 (Project No. T2283-12-02). Recommendations presented in the previous report are still applicable to the site, unless superseded by recommendations within this report. The purposes of this study were to; review the previous recommendations and the current grading plans, perform a site reconnaissance, and provide updated recommendations relative to the geotechnical aspects of developing the site including remedial grading, bedrock rippability, foundation and retaining wall design criteria, slope stability, seismic design parameters, and grading specifications.

The previous field investigation included the excavation of 10 exploratory backhoe trenches. Exploratory excavations were advanced through the surficial soils, where possible, and into formational materials to aid in determining the necessary remedial grading and to delineate the geologic units within areas of proposed development. Samples obtained from the exploratory excavations were examined and logged. Details of the previous field investigation including the logs are produced in Appendix A. The approximate locations of the exploratory excavations are depicted on the undated grading plans entitled *Rough Grading Plans Tract No. 29476* prepared by Markham Development Management Group, Inc. and sent to our office on May 20, 2014, see *Geologic Map*, Figure 2.

Laboratory testing was performed on samples of soil obtained from the exploratory excavations to determine in-situ density and moisture content, maximum density/optimum moisture, direct shear properties, expansion potential, sulfate content, resistivity and pH for use in engineering analyses. Details of the laboratory testing are presented in Appendix B.

2. SITE AND PROJECT DESCRIPTION

Tract No. 29476 consists of a rectangular-shaped parcel occupying approximately 18 acres of land located north of Via Sarah and west of David Lane in the City of Wildomar, California. The property is bounded on the east by David Lane and residential housing, on the south by undeveloped land, residential housing and Via Sarah, on the north by undeveloped land; and on the west by a natural drainage and rural residential housing. A large portion of the property will remain undeveloped, with the 28 residential lots proposed on the easternmost hilltops.

Following the completion of the previous geotechnical investigation, Tract 22948 to the east and south was built including Via Sarah, David Lane, and the residential homes north of Via Sarah and east of

David Lane. Construction of David Lane included construction of a fill slope with Tract 29476 in the vicinity of proposed lots 19 through 23. Details of the this work including compaction test results and the approximate location of canyon drains are available in the report entitled *Final Report of Testing and Observation Services Performed During Site Grading, Monticello, Tract No. 22948, Riverside County, California, BGR 020918 B,* prepared by Geocon Inland Empire, Inc. and dated October 17, 2005 (project no. T2213-12-02).

Topographically, the overall property is characterized by moderately steep bedrock hills and ridges which have been incised by relatively narrow drainages. Drainage is directed to the southwest toward Clinton Keith Road. Elevations range from approximately 1,470 feet Mean Sea Level (MSL) in the vicinity of proposed Lot 24 to approximately 1,563 feet MSL in the vicinity of proposed Lot 6.

Review of the referenced grading plans indicates that site development will consist of mass grading the eastern portion of the property to construct a residential subdivision comprised of 28 building pads, and associated streets. The western portion of the property will not be developed. The primary access to the subdivision will be through the adjacent property to the east and south, Tract 22948.

Proposed grading is to consist of cuts and fills of up to approximately 48 feet and 55 feet, respectively, not inclusive of any remedial grading. Cut slopes are proposed at a maximum inclination of 1.5:1 (horizontal:vertical) with a maximum height of approximately 10 feet. Fill slopes are proposed for the site at an inclination of 2:1 (horizontal:vertical) to a maximum height of 80 feet. Fill slopes between the lots are proposed at an inclination of 2:1 (horizontal:vertical) and a maximum height of 6 feet.

The descriptions of the site and proposed development are based on a site reconnaissance, observations during the field investigation, and review of the referenced reports, geologic publications, and grading plans. If project details differ significantly from those described, Geocon should be contacted for review and possible revision to this report.

3. SOIL AND GEOLOGIC CONDITIONS

Soil and geologic conditions were identified by observation within the exploratory excavations, and review of geologic literature. Surficial deposits encountered or observed during the previous investigation include alluvium, and colluvium. The formational unit encountered consists of granitic bedrock. Following completion of the previous geotechnical investigation, compacted fill soil was placed on the site during construction of David Lane. Each of the surficial deposits and the formational unit are discussed below. The approximate aerial extent of these deposits is depicted on the *Geologic Map*, Figure 2.

3.1 Documented Fill (Qdf)

Fill soil associated with the construction of David Lane was placed within the tract boundaries in the vicinity of proposed lots 19 through 23. Placement of the fill was observed and tested by Geocon Inland Empire, Inc. The results of the testing and observation were presented in the report entitled *Final Report of Testing and Observation Services Performed During Site Grading, Monticello, Tract No. 22948, Riverside County, California, BGR 020918 B,* dated October 17, 2005 (project no. T2213-12-02). Tests indicate that the fill soil was compacted to a minimum dry density of at least 90 percent of the maximum laboratory dry density. Fill soil placed within this area was described as predominantly silty fine to coarse sand with varying amounts of clay and gravel. Lesser amounts of clayey fine to coarse sand and gravelly fine to coarse sand were also placed within this area. Canyon drains were placed at the bottom of the cleanouts. Terrace drains and slope down drains were constructed at the face of the finished slope. The upper portion of the fill soil has been disturbed by plant growth and water infiltration and is unsuitable for the support of settlement sensitive structures or additional fill. We estimate that the upper 2 to 3 feet will require remedial grading.

3.2 Alluvium (Qal)

Alluvium was encountered and observed within the drainage areas of the property. The alluvial soil was encountered to depths of 1 to 2 feet within the upper areas of the canyons, the soil is likely thicker within the lower areas of the canyons. The alluvium consisted of loose, wet, dark brown silty coarse sand. Alluvium is not considered suitable to provide support for engineered fill or structural loads and should be removed prior placement of fill or settlement sensitive site improvements. It should be noted that trenching within some of the tributary drainages was not possible due to the steep topography.

3.3 Colluvium (not a mapped unit)

Colluvium was encountered to a depth of 6 inches along the hillsides and saddles of the property. Due to the relatively shallow depth of the colluvium, it is not a mapped unit on the *Geologic Map*, Figure 2. The colluvium consisted of loose, moist, orange-brown, silty medium to coarse sand. Colluvium is not considered suitable to provide support for engineered fill and/or structural loads and should be removed prior placement of fill and/or settlement sensitive site improvements.

3.4 Granitic Bedrock (Kgr)

Granitic bedrock comprises the hill tops within the property and underlies the entire site at depth. The rock consists of a light colored, coarse grained, massive granitic rock known as tonalite. Aplite dikes were observed within the site and made excavation with a small backhoe extremely difficult. The observed width of the dikes was generally less than one foot. Although equipment will encounter difficulty in excavating the dike material, it is expected to be rippable due to its narrow width. Gabbroic cobbles were observed on an adjacent property, therefore, it is anticipated that isolated areas of a fine-grained dark grey gabbro are present within the granitic rock. The majority of the rock encountered was soft to hard and excavated with ease to moderate effort from 4.5 to 12 feet below the original ground. Siesmic refraction data indicates that bedrock in the vicinity of Lots 1 through 16 is rippable to depths in excess of 30 feet. Air track borings were advanced within Tract 22948, immediately east of this property. The closest airtrack borings to the subject property indicate the bedrock is likely rippable to depths in excess of 40 feet. However, blasting may be necessary at depth in the far southern area of the property. Large (3 to 10-foot diameter) boulders were observed within a drainage and it is likely that oversized rock or core stones will be generated during grading. Oversized rock will require special handling and placement during grading operations, in accordance with the *Recommended Grading Specifications.*

4. GROUNDWATER

Groundwater was not encountered during the previous investigation to the maximum depth of exploration of 12 feet. Well data from a well located a few hundred feet southwest of the site (Well #06S03W32P001S) at elevation 1,480 feet MSL indicates groundwater was 66 feet below the surface in 1968. Some seepage along the soil/bedrock contact was observed within the drainage areas and should be expected following periods of precipitation. If seepage is encountered it will likely be manageable and should not adversely impact the proposed development.

5. GEOLOGIC HAZARDS

5.1 Faulting and Seismicity

The site, like the rest of southern California, is located within a seismically active region near the active margin between the North American and Pacific tectonic plates. The principal source of seismic activity is movement along the northwest-trending regional faults such as the San Andreas, San Jacinto and Elsinore fault zones. These fault systems are estimated to produce up to approximately 55 millimeters of slip per year between the plates.

By definition of the State Mining and Geology Board, an active fault is one which has had surface displacement within the Holocene Epoch (roughly the last 11,000 years). This definition is used in delineating Earthquake Fault Zones as mandated by the Alquist-Priolo Geologic Hazards Zones Act of 1972 and as revised in 1994 and 1997 as the Alquist-Priolo Earthquake Fault Zoning Act and Earthquake Fault Zones. The intent of the act is to require fault investigations on sites located within Earthquake Fault Zones to preclude new construction of certain habitable structures across the trace of active faults. Based on our review of the referenced literature, the site is not located within an Earthquake Fault Hazard Zone. The site could, however, be subjected to significant shaking in the event of a major earthquake on the Elsinore Fault or other nearby regional faults. Structures for the site should be constructed in accordance with current CBC seismic codes and local ordinances.

5.2 Liquefaction

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. Due to the presence of shallow granitic bedrock, and the lack of near surface groundwater, the potential for seismically induced soil liquefaction occurring at the site is considered to be very low.

5.3 Rock Fall Hazard

Due to the distance from bedrock hills of higher elevation and the relatively few boulders that were observed within the property, the rock fall hazard within the site is considered very low.

6. CONCLUSIONS AND RECOMMENDATIONS

6.1 General

6.1.1 No soil or geologic conditions were encountered at the site that would preclude the development of the property as presently proposed provided that the recommendations of this report are followed.

6.1.2 Alluvium, colluvium, and the upper portion of the documented fill are considered unsuitable in their present condition for support of structural loads or the placement of compacted fill and will require removal, moisture conditioning, and compaction.

6.1.3 The granitic bedrock is considered suitable to receive compacted fill or structural loads.

6.1.4 In general, the on-site soil consist of medium to coarse silty sands with variable amounts of clay, gravel and cobbles, is generally classified as "non-expansive" (EI < 20), as defined by 2013 California Building Code (CBC) Section 1803.5.3, and exhibits moderate shear strength characteristics. The on-site surficial soil and processed bedrock material are considered suitable for use as fill. It is recommended that material with an Expansion Index greater than 50 be kept at least 3 feet below proposed finish grade where possible.

6.2 Soil and Excavation Characteristics

6.2.1 The alluvium, colluvium, and documented fill can be excavated with conventional heavy-duty grading equipment. Based on the seismic refraction data and air track boring data from the adjacent site, rippable excavations made within the granitic bedrock are anticipated to be possible to depths of 40 feet below existing grade in most locations. Localized areas of non-rippable bedrock may be encountered at the southern area of the property. It is expected that the excavatable bedrock materials will generally be rippable to marginally rippable using a D9 dozer with a single-shank ripper. Some excavation difficulties should be expected during utility trench construction even in areas that are rippable with a D9 dozer. Consideration should be given to over-excavating utility corridors to at least one foot below the deepest utility within the roadways and backfilling with compacted soil to allow for future excavation with conventional equipment. Oversize rock should be placed in accordance with the *Recommended Grading Specifications* presented in Appendix C and the County of Riverside criteria.

6.2.2 Excavations should be performed in conformance with OSHA requirements, Excavations made adjacent to property lines or the existing improvements should not be left open during hours when construction is not being performed.

6.2.3 Expansion Index testing indicates that the site soil is generally classified as “non-expansive” (EI < 20), as defined by 2013 California Building Code (CBC) Section 1803.5.3. Laboratory Expansion Index testing should be performed on soil exposed at finish grade subsequent to the completion of grading to verify the at-grade expansion characteristics. Typically one test for every 3 or 4 lots is performed depending on the soil types encountered

6.2.4 Laboratory testing indicates that the samples tested yielded water-soluble sulfate contents with *negligible* sulfate ratings as defined by 2013 CBC, Section 1904.3 and ACI 318. Resistivity testing indicates soil samples are *mildly corrosive* in accordance with criteria presented by the National Association of Corrosion Engineers. These test results are presented in Appendix B. These tests are general indications only and additional testing should be performed at finish grade (materials within 3 feet of rough pad grade elevations) subsequent to completion of mass grading.

6.3 Grading

6.3.1 Grading should be performed in accordance with the *Recommended Grading Specifications* contained in Appendix C and the requirements of Riverside County. Where the recommendations of this section conflict with those of Appendix C the recommendations of this section take precedence.

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

6.3.3 Site preparation should begin with the removal of deleterious material and vegetation. The depth of removal should be such that material exposed in cut areas or soils to be used as fill are relatively free of organic matter. The removal of trees and the majority of the root structure prior to grading will reduce the amount of manually removing remnant roots during grading. Material generated during stripping and site demolition should be exported from the site.

6.3.4 Alluvium and colluvium not removed by planned grading should be completely removed to bedrock. Estimated removal depths are plotted onto the 40-scale grading plans adjacent to the trench excavations. Actual removal depths should be determined by the Geocon geologist at the time of grading, based on the above indicated criteria.

6.3.5 During remedial grading temporary slopes should be planned for an inclination no steeper than 1:1 (horizontal:vertical). Grading should be scheduled to backfill against these slopes as soon as practical. Removals along the limits of grading should include excavation of unsuitable soil that would adversely affect the performance of the planned fill, i.e., extend removals within a zone defined by a line projected down and out at a slope of 1:1 from the limit of grading to intersect with approved left-in-place soil or bedrock.

6.3.6 After removal of surficial soil, the exposed ground surface should be scarified, moisture conditioned, and compacted. Fill soil may then be placed and compacted in layers to the design finish grade elevations. Fill, including backfill and scarified ground surfaces, should be compacted to at least 90 percent of the laboratory maximum dry density and near optimum moisture content, as determined by ASTM Test Procedure D 1557. **Fill soil placed 50 feet or more below finished grade elevations should be compacted to at least 95 percent of the laboratory maximum dry density and near optimum moisture content.**

6.3.7 Settlement monuments should be constructed on lots in which 50 feet or more of fill has been placed. The monuments should be constructed as soon as possible after fill placement and should be surveyed on a weekly basis for approximately 6 weeks or until settlement has stopped.

6.3.8 Lots graded with a cut/fill transition will require undercutting to reduce the potential for differential settlement. The cut portion of the cut/fill transition should be undercut to a depth of at least 3 feet and replaced with properly compacted low expansive fill. The bottom of the undercut should be sloped at a minimum of 1 percent towards the adjacent street. In areas where a steep transition exists, additional removal will be required such that the maximum fill differential across any one building pad will be less than H/4, where H is the maximum fill thickness.

6.3.9 Overexcavation of cut lots should be performed to reduce the difficulty of excavating footing and plumbing trenches within the bedrock. Cut lots should be overexcavated h/3 (where h is the maximum fill thickness) or a minimum of three feet with the bedrock sloped 1 percent or more toward the street.

6.4 Bulking and Shrinkage Factors

6.4.1 Estimates of embankment bulking and shrinkage factors are based on comparing laboratory compaction tests with the density of the material in its natural state as encountered in the exploratory excavations. It should be emphasized that variations in natural soil density, as well as in compacted fill density, render shrinkage value estimates very approximate. As an example, the contractor can compact the fill soil to any relative compaction of 90 percent or higher of the maximum laboratory density. Thus, the contractor has approximately a 10 percent range of control over the fill volume. Based on the limited work performed to date, it is our opinion that the following shrinkage and bulking factors can be used as a basis for estimating the amount of shrinkage or bulking of the on-site materials when excavated from their natural state and placed as compacted fills.

TABLE 6.4.1
SHRINK/BULK FACTORS

Soil Unit	Shrink/Bulk Factor
Alluvium	10 - 15 percent shrink
Colluvium	5 - 10 percent shrink
Granitic Bedrock	5 - 15 percent bulk

6.5 Seismic Design Criteria

6.5.1 We used the computer program *U.S. Seismic Design Maps*, provided by the USGS. Table 8.3.1 summarizes site-specific design criteria obtained from the 2013 California Building Code (CBC; Based on the 2012 International Building Code [IBC] and ASCE 7-10), Chapter 16 Structural Design, Section 1613 Earthquake Loads. The short spectral response uses a period of 0.2 second. The improvements should be designed using a Site Class C. We evaluated the Site Class based on the discussion in Section 1613.3.2 of the 2013 CBC and Table 20.3-1 of ASCE 7-10. The values presented in Table 6.5.1 are for the risk-targeted maximum considered earthquake (MCE_R).

TABLE 6.5.1
2013 CBC SEISMIC DESIGN PARAMETERS

Parameter	Value	2013 CBC Reference
Site Class	D	Section 1613.3.2
MCE_R Ground Motion Spectral Response Acceleration – Class B (short), S_S	2.070g	Figure 1613.3.1(1)
MCE_R Ground Motion Spectral Response Acceleration – Class B (1 sec), S_1	0.820g	Figure 1613.3.1(2)
Site Coefficient, F_A	1.0	Table 1613.3.3(1)
Site Coefficient, F_V	1.5	Table 1613.3.3(2)
Site Class Modified MCE_R Spectral Response Acceleration (short), S_{MS}	2.070g	Section 1613.3.3 (Eqn 16-37)
Site Class Modified MCE_R Spectral Response Acceleration (1 sec), S_{M1}	1.230g	Section 1613.3.3 (Eqn 16-38)
5% Damped Design Spectral Response Acceleration (short), S_{DS}	1.380g	Section 1613.3.4 (Eqn 16-39)
5% Damped Design Spectral Response Acceleration (1 sec), S_{D1}	0.820g	Section 1613.3.4 (Eqn 16-40)

6.5.2 Table 6.5.2 presents additional seismic design parameters for projects located in Seismic Design Categories of D through F in accordance with ASCE 7-10 for the mapped maximum considered geometric mean (MCE_G).

TABLE 6.5.2
2013 CBC SITE ACCELERATION DESIGN PARAMETERS

Parameter	Value	ASCE 7-10 Reference
Mapped MCE_G Peak Ground Acceleration, PGA	0.803g	Figure 22-7
Site Coefficient, F_{PGA}	1.0	Table 11.8-1
Site Class Modified MCE_G Peak Ground Acceleration, PGA_M	0.803g	Section 11.8.3 (Eqn 11.8-1)

6.5.3 Conformance to the criteria in Tables 6.5.1 and 6.5.2 for seismic design does not constitute any kind of guarantee or assurance that significant structural damage or ground failure will not occur if a large earthquake occurs. The primary goal of seismic design is to protect life, not to avoid all damage, since such design may be economically prohibitive.

6.6 Slopes

6.6.1 Fill slopes constructed at an inclination of 2:1 (horizontal:vertical) with the on-site soil are anticipated to be stable with respect to surficial instability. A surficial stability analysis has been performed based on an assumed 5-foot zone of saturation. This analysis is provided on Figure 3.

6.6.2 An analysis of the 80 foot high, 2:1 (horizontal:vertical) fill slope was performed to determine the stability of the slope with respect to deep seated failure. The angle of internal friction of 34 degrees; apparent cohesion of 150 psf; a moist unit weight of 130 pcf; and a pseudostatic acceleration of 0.15g were used in the analysis. Details of the analysis are presented on Figures 4 and 5 and indicate a factor of safety of 1.6 and 1.2 for the static and pseudostatic analyses respectively.

6.6.3 A key should be excavated at the base of fill slopes. The fill slopes should then be overbuilt at least 3 feet horizontally and then cut to the design finish grade. As an alternative, fill slopes may be compacted by backrolling with a sheepsfoot compactor at vertical intervals not to exceed 4 feet and then track-walked with a D-8 bulldozer, or equivalent, such that the soil is uniformly compacted to a dry density of at least 90 percent of the laboratory maximum dry denstiy to the face of the finished slope.

6.6.4 An analysis of the 20 foot high, 1.5:1 (horizontal:vertical) bedrock cut slope was performed to determine the stability of the slope with respect to deep seated failure. The angle of internal friction of 42 degrees; apparent cohesion of 150 psf; a moist unit weight of 130 pcf; and a pseudostatic acceleration of 0.15g were used in the analysis. Details of the analysis are presented on Figures 6 and 7 and indicate a factor of safety of 2.2 and 1.8 for the static and pseudostatic analyses respectively.

6.6.5 Slopes should be planted, drained and maintained to reduce erosion. Due to the very granular nature of the majority of the site soil, consideration should be given to landscaping the slopes relatively soon after completion to reduce the potential for surficial erosion.

6.6.6 If soil with strength parameters less than those presented in Figures 3 through 5 are proposed to construct the site slopes, Geocon should be contacted to provide additional recommendations. Bedrock cut slopes should be mapped during construction by the project geologist. If out of slope joints or bedding are observed, additional stability recommendations will be provided.

6.7 Subdrains

6.7.1 The hill and canyon topography of the site and the fill over granitic bedrock conditions at the conclusion of site grading will contribute to the accumulation of post-construction irrigation water along the fill/bedrock contact within canyons. Therefore, the use of canyon subdrains will be necessary to reduce the potential for adverse impacts associated with seepage conditions. Figure 8 depicts a typical canyon subdrain detail. Preliminary subdrain locations have been noted on the *Geologic Map*. The precise locations of the subdrains should be determined by the Geocon geologist during grading based on the conditions encountered. Existing subdrains associated with the David Lane construction should be extended to an approved outlet.

6.7.2 Prior to outletting, the final 20-foot segment of subdrain should consist of non-perforated drain pipe. At the non-perforated/perforated interface, a seepage cut-off wall should be constructed on the down slope side of the junction in accordance with Figure 9. Subdrains that discharge into a natural drainage course or open space area should be provided with a permanent headwall structure in accordance with Figure 10.

6.7.3 Upon completion of remedial excavations and subdrain installation, the project civil engineer should survey the drain locations and elevations and prepare an “as-built” map depicting the locations. The final outlet and connection locations should be determined during grading.

6.8 Foundation

6.8.1 The foundation recommendations presented herein are for proposed residential structures. We separated the foundation recommendations into three categories based on either the maximum and differential fill thickness or Expansion Index. We anticipate the majority of structures will be Category I or II due to the geometry of the underlying fill and native soil. The foundation category criteria for the anticipated conditions are presented in Table 6.8.1. Final foundation categories will be evaluated once site grading has been completed.

TABLE 6.8.1
FOUNDATION CATEGORY CRITERIA

Foundation Category	Maximum Fill Thickness, T (Feet)	Differential Fill Thickness, D (Feet)	Expansion Index (EI)
I	T<20	D<10	EI≤50
II	20≤T<50	10≤D<20	50<EI≤90
III	T≥50	D≥20	90<EI≤130

6.8.2 Post-tensioned concrete slab and foundation systems may be used for the support of the proposed structures. The post-tensioned systems should be designed by a structural engineer experienced in post-tensioned slab design and design criteria of the Post-Tensioning Institute (PTI), as required by the 2013 California Building Code (CBC Section 1808.6). Although this procedure was developed for expansive soil conditions, we understand it can also be used to reduce the potential for foundation distress due to differential fill settlement. The post-tensioned design should incorporate the geotechnical parameters presented on Table 6.8.2 for the particular Foundation Category designated. The parameters presented in Table 6.8.2 are based on the guidelines presented in the PTI, Third Edition design manual. The foundations for the post-tensioned slabs should be embedded in accordance with the recommendations of the structural engineer.

TABLE 6.8.2
POST-TENSIONED FOUNDATION SYSTEM DESIGN PARAMETERS

Post-Tensioning Institute (PTI) Third Edition Design Parameters	**Foundation Category**		
	I	**II**	**III**
Thornthwaite Index	-20	-20	-20
Equilibrium Suction	3.9	3.9	3.9
Edge Lift Moisture Variation Distance, e_M (feet)	5.3	5.1	4.9
Edge Lift, y_M (inches)	0.61	1.10	1.58
Center Lift Moisture Variation Distance, e_M (feet)	9.0	9.0	9.0
Center Lift, y_M (inches)	0.30	0.47	0.66

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

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

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

6.8.6 If the structural engineer proposes a post-tensioned foundation design method other than the 2013 CBC:

- The criteria presented in Table 6.8.2 are still applicable.
- Interior stiffener beams should be used for Foundation Categories II and III.
- The width of the perimeter foundations should be at least 12 inches.
- The perimeter footing embedment depths should be at least 12 inches, 18 inches and 24 inches for foundation categories I, II, and III, respectively. The embedment depths should be measured from the lowest adjacent pad grade.

6.8.7 Our experience indicates post-tensioned slabs are susceptible to excessive edge lift, regardless of the underlying soil conditions. Placing reinforcing steel at the bottom of the perimeter footings and the interior stiffener beams may mitigate this potential. Because of the placement of the reinforcing tendons in the top of the slab, the resulting eccentricity after tensioning reduces the ability of the system to mitigate edge lift. The structural engineer should design the foundation system to reduce the potential of edge lift occurring for the proposed structures.

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

6.8.9 As an alternate to post-tensioned foundation systems, conventional shallow foundation with a concrete slab-on-grade may be used for support of the proposed structures. Conventional shallow foundations may be designed for an allowable soil bearing pressure of 2,000 pounds per square foot (psf) (dead plus live load). This bearing pressure may be increased by one-third for transient loads due to wind or seismic forces. We estimate the total settlements under the imposed allowable loads to be about 1 inch with differential settlements on the order of ½ inch over a horizontal distance of 40 feet. Table 6.8.9 presents minimum foundation and interior concrete slab design criteria for conventional foundation systems.

TABLE 6.8.9
CONVENTIONAL FOUNDATION RECOMMENDATIONS BY CATEGORY

Foundation Category	Minimum Footing Embedment Depth (inches)	Continuous Footing Reinforcement	Interior Slab Reinforcement
I	12	Two No. 4 bars, one top and one bottom	6 x 6 - 10/10 welded wire mesh at slab mid-point
II	18	Four No. 4 bars, two top and two bottom	No. 3 bars at 24 inches on center, both directions at slab mid-point
III	24	Four No. 5 Bars, two top and two bottom	No. 3 bars at 18 inches on center both directions at slab mid-point

6.8.10 The embedment depths presented in Table 6.8.9 should be measured from the lowest adjacent pad subgrade for both interior and exterior footings. The conventional foundations should have a minimum width of 12 inches and 24 inches for continuous and isolated footings, respectively.

6.8.11 Isolated footings, if present, should have the minimum embedment depth and width recommended for conventional foundations for a particular foundation category. The use of isolated footings, which are located beyond the perimeter of the building and support structural elements connected to the building, are not recommended for Category III. Where this condition cannot be avoided, the isolated footings should be connected to the building foundation system with grade beams.

6.8.12 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.

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

- Building footings should be deepened such that the bottom outside edge of the footing is at least 7 feet horizontally from the face of the slope.
- Geocon should be contacted to review the pool plans and the specific site conditions to provide additional recommendations, if necessary.
- Swimming pools located within 7 feet of the top of cut or fill slopes are not recommended. Where such a condition cannot be avoided, the portion of the swimming

pool wall within 7 feet of the slope face be designed assuming that the adjacent soil provides no lateral support

- Although other improvements, which are relatively rigid or brittle, such as concrete flatwork or masonry walls, may experience some distress if located near the top of a slope, it is generally not economical to mitigate this potential. It may be possible, however, to incorporate design measures that would permit some lateral soil movement without causing extensive distress. Geocon Incorporated should be consulted for specific recommendations.

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

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

6.9 Retaining Walls and Lateral Loads

6.9.1 Retaining walls not restrained at the top and having a level backfill surface should be designed for an active soil pressure equivalent to the pressure exerted by a fluid density of 30 pounds per cubic foot (pcf). Where the backfill will be inclined at no steeper than 2.0 to 1.0, an active soil pressure of 40 pcf is recommended. These soil pressures assume that the backfill materials within an area bounded by the wall and a 1:1 plane extending upward from the base of the wall possess an Expansion Index of less than 50. For those lots with finish grade soil having an Expansion Index greater than 50 or where backfill soil does not conform to the above criteria, Geocon should be consulted for additional recommendations.

6.9.2 Unrestrained walls are those that are allowed to rotate more than 0.001H (where H equals the height of the retaining wall portion of the wall in feet) at the top of the wall. Where walls are restrained from movement at the top, an additional uniform pressure of 7H psf should be added to the above active soil pressure.

6.9.3 Retaining walls should be provided with a drainage system adequate to prevent the buildup of hydrostatic forces and should be waterproofed as required by the project civil engineer or landscape architect. The use of drainage openings through the base of the wall (weep holes, etc.) is not recommended where the seepage could be a nuisance or otherwise adversely impact the property adjacent to the base of the wall. The above recommendations assume a properly compacted granular (Expansion Index less than 50) backfill material with no hydrostatic forces or imposed surcharge load. If conditions different than those described are anticipated, or if specific drainage details are desired, Geocon should be contacted for additional recommendations.

6.9.4 In general, wall foundations having a minimum depth and width of one foot may be designed for an allowable soil bearing pressure of 2,000 psf, provided the soil within 3 feet below the base of the wall has an Expansion Index of less than 50. The proximity of the foundation to the top of a slope steeper than 3:1 could impact the allowable soil bearing pressure. Therefore, Geocon should be consulted where such a condition is anticipated.

6.9.5 For resistance to lateral loads, an allowable passive earth pressure equivalent to a fluid density of 300 pcf is recommended for footings or shear keys poured neat against properly compacted granular fill soil or undisturbed natural soil. 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 not protected by floor slabs or pavement should not be included in the design for lateral resistance. An allowable friction coefficient of 0.4 may be used for resistance to sliding between soil and concrete. This friction coefficient may be combined with the allowable passive earth pressure when determining resistance to lateral loads.

6.9.6 The recommendations presented above are generally applicable to the design of rigid concrete or masonry retaining walls having a maximum height of 8 feet. In the event that walls higher than 8 feet or other types of walls are planned, such as crib-type walls, Geocon should be consulted for additional recommendations.

6.10 Slope Maintenance

6.10.1 Slopes that are steeper than 3:1 (horizontal to vertical) may, under conditions which are both difficult to prevent and predict, be susceptible to near surface (surficial) slope instability. The instability is typically limited to the outer three feet of a portion of the slope and usually does not directly impact the improvements on the pad areas above or below the slope. The occurrence of surficial instability is more prevalent on fill slopes and is generally preceded by a period of heavy rainfall, excessive irrigation, or the migration of subsurface seepage. The disturbance and/or loosening of the surficial soils, as might result from root growth, soil expansion, or excavation for irrigation lines and slope planting, may also be a significant contributing factor to surficial instability. It is, therefore, recommended that, to the maximum extent practical: (a) disturbed/loosened surficial soil be either removed or properly recompacted, (b) irrigation systems be periodically inspected and maintained to eliminate leaks and excessive irrigation, and (c) surface drains on and adjacent to slopes be periodically maintained to preclude ponding or erosion. Although the incorporation of the above recommendations should reduce the potential for surficial slope instability, it will not eliminate the possibility, and, therefore, it may be necessary to rebuild or repair a portion of the project's slopes in the future.

6.11 Drainage

6.11.1 Adequate drainage provisions are imperative. Under no circumstances should water be allowed to pond adjacent to footings. The building pads should be properly finish graded after the buildings and other improvements are in place so that drainage water is directed away from foundations, pavements, concrete slabs, and slope tops to controlled drainage devices.

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 or corrosive 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 present date. 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.

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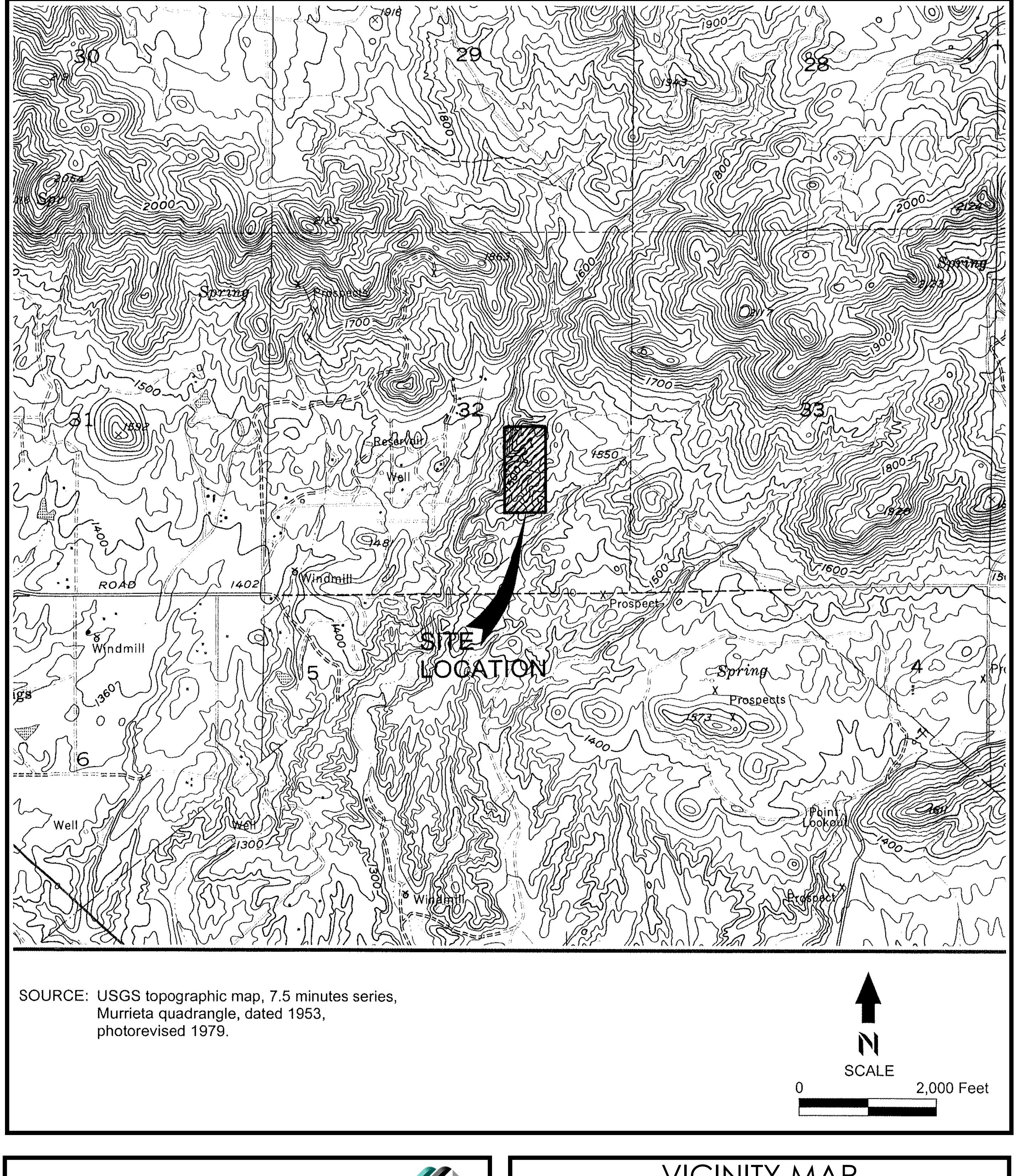

SOURCE: USGS topographic map, 7.5 minutes series,
Murrieta quadrangle, dated 1953,
photorevised 1979.

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VICINITY MAP

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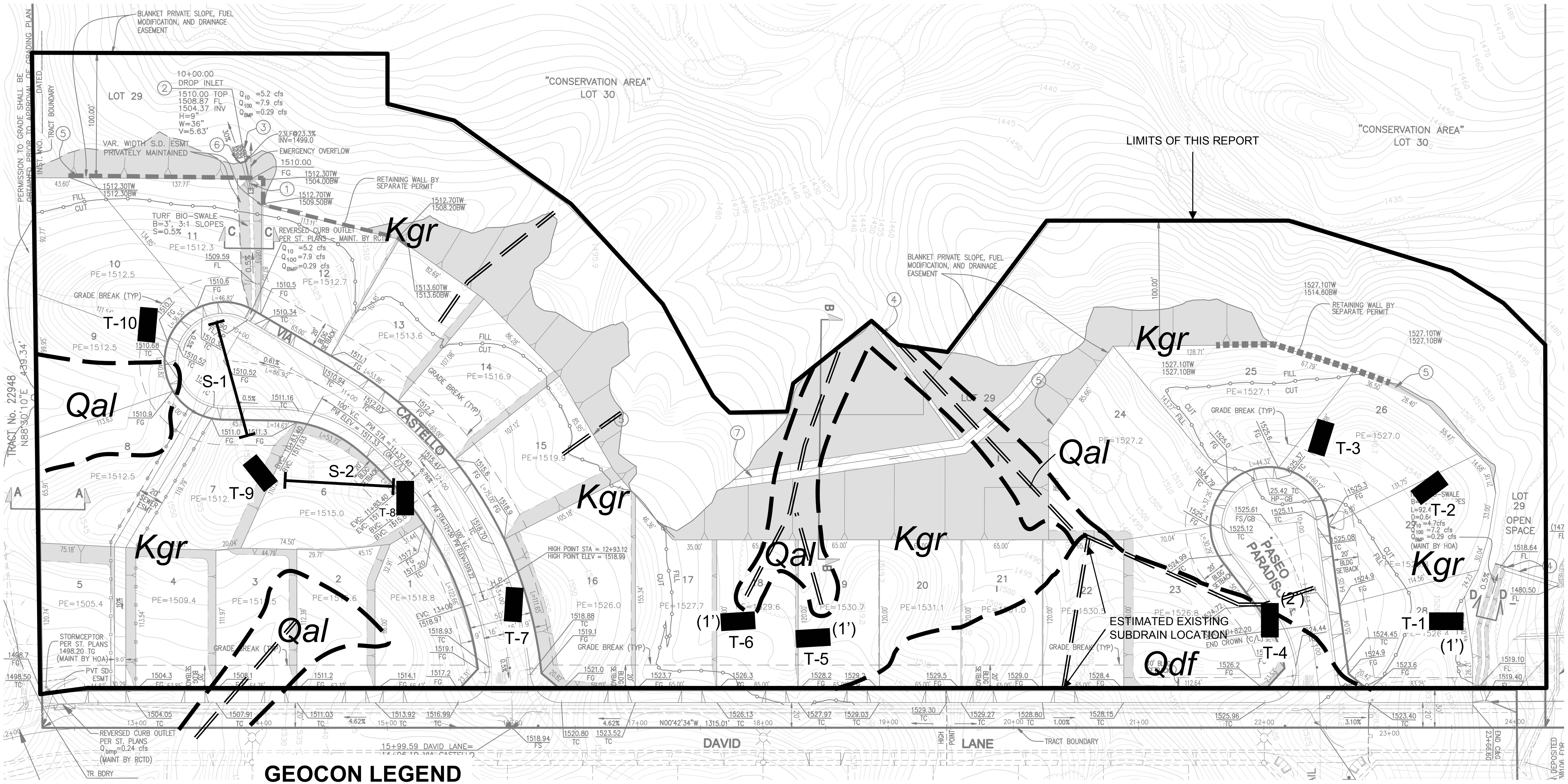
KC / KC		

DATE JUNE 2014	PROJECT NO. T2283-22-03	FIG. 1



SCALE: 1"=80'

GEOCON LEGEND

Qdf.......DOCUMENTED FILL

Qal.......ALLUVIUM

Kgr.......GRANITIC BEDROCK

.......APPROXIMATE LOCATION OF GEOLOGIC CONTACT

T-10 ▬ ....APPROX. LOCATION OF GEOCON TRENCH

S-2 ⊢ ....APPROX. LOCATION OF GEOCON SEISMIC LINE

(2') ....ESTIMATED REMOVAL DEPTH

═ ═ ....ESTIMATED SUBDRAIN LOCATION

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GEOLOGIC MAP

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DATE JUNE 2014	PROJECT NO. T2283-22-03	FIG. 2

ASSUMED CONDITIONS:

SLOPE HEIGHT H = Infinte

SLOPE INCLINATION 2.0 : 1.0 (Horizontal : Vertical)

SLOPE ANGLE i = 26.6 °

DEPTH OF SATURATION Z = 5 feet

UNIT WEIGHT OF WATER γ_w 62.4 pounds per cubic foot

TOTAL UNIT WEIGHT OF SOIL γ_t = 125 pounds per cubic foot

ANGLE OF INTERNAL FRICTION ϕ = 42 degrees

APPARENT COHESION C = 150 pounds per square foot

SLOPE SATURATED TO VERTICAL DEPTH Z BELOW SLOPE FACE.
SEEPAGE FORCES PARALLEL TO SLOPE FACE.

ANALYSIS:

$$FS = \frac{C + (\gamma_t - \gamma_w)Z \cdot \cos^2 i \cdot \tan\phi}{\gamma_t \cdot Z \cdot \sin i \cdot \cos i} = 1.5$$

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SURFICIAL SLOPE STABILITY - FILL SLOPES

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DATE JUNE 2014 | PROJECT NO. T2283-22-03 | FIG. 3

ASSUMED CONDITIONS:

SLOPE HEIGHT H = 80 feet

SLOPE INCLINATION 2.0 : 1.0 (Horizontal : Vertical)

TOTAL UNIT WEIGHT OF SOIL γ_t = 130 pounds per cubic foot

ANGLE OF INTERNAL FRICTION ϕ = 34 degrees

APPARENT COHESION C = 150 pounds per square foot

PSEUDOSTATIC COEFFICIENT k_h = 0

PSEUDOSTATIC INCLINATION 2.0 : 1.0 (Horizontal : Vertical)

PSEUDOSTATIC UNIT WEIGHT γ_{ps} = 130 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

$\lambda_{c\phi}$ = 46.8 CALCULATED USING EQ. (3-3)

Ncf = 113 DETERMINED USING FIGURE 10, REFERENCE 2

FS = 1.6 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 - FILL SLOPES

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DATE JUNE 2014 | PROJECT NO. T2283-22-03 | FIG. 4

ASSUMED CONDITIONS:

SLOPE HEIGHT $H = 80$ feet

SLOPE INCLINATION 2.0 : 1.0 (Horizontal : Vertical)

TOTAL UNIT WEIGHT OF SOIL $\gamma_t = 130$ pounds per cubic foot

ANGLE OF INTERNAL FRICTION $\phi = 34$ degrees

APPARENT COHESION $C = 150$ pounds per square foot

PSEUDOSTATIC COEFFICIENT $k_h = 0.15$

PSEUDOSTATIC INCLINATION 1.4 : 1.0 (Horizontal : Vertical)

PSEUDOSTATIC UNIT WEIGHT $\gamma_{ps} = 131$ 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

$\lambda_{c\phi} = 47.3$ CALCULATED USING EQ. (3-3)

$Ncf = 85$ DETERMINED USING FIGURE 10, REFERENCE 2

$FS = 1.2$ 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 - FILL SLOPES

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ASSUMED CONDITIONS:

SLOPE HEIGHT $H = 20$ feet

SLOPE INCLINATION 1.5 : 1.0 (Horizontal : Vertical)

TOTAL UNIT WEIGHT OF SOIL $\gamma_t = 130$ pounds per cubic foot

ANGLE OF INTERNAL FRICTION $\phi = 42$ degrees

APPARENT COHESION $C = 150$ pounds per square foot

PSEUDOSTATIC COEFFICIENT $k_h = 0$

PSEUDOSTATIC INCLINATION 1.5 : 1.0 (Horizontal : Vertical)

PSEUDOSTATIC UNIT WEIGHT $\gamma_{ps} = 130$ 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

$\lambda_{c\phi} = 15.6$ CALCULATED USING EQ. (3-3)

$Ncf = 37$ DETERMINED USING FIGURE 10, REFERENCE 2

$FS = 2.2$ 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 - CUT SLOPES

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DATE JUNE 2014 PROJECT NO. T2283-22-03 FIG. 6

ASSUMED CONDITIONS:

SLOPE HEIGHT $H = 20$ feet

SLOPE INCLINATION 1.5 : 1.0 (Horizontal : Vertical)

TOTAL UNIT WEIGHT OF SOIL $\gamma_t = 130$ pounds per cubic foot

ANGLE OF INTERNAL FRICTION $\phi = 42$ degrees

APPARENT COHESION $C = 150$ pounds per square foot

PSEUDOSTATIC COEFFICIENT $k_h = 0.15$

PSEUDOSTATIC INCLINATION 1.1 : 1.0 (Horizontal : Vertical)

PSEUDOSTATIC UNIT WEIGHT $\gamma_{ps} = 131$ 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

$\lambda_{c\phi} = 15.8$ CALCULATED USING EQ. (3-3)

$Ncf = 31$ DETERMINED USING FIGURE 10, REFERENCE 2

$FS = 1.8$ 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

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SLOPE STABILITY ANALYSIS - CUT SLOPES

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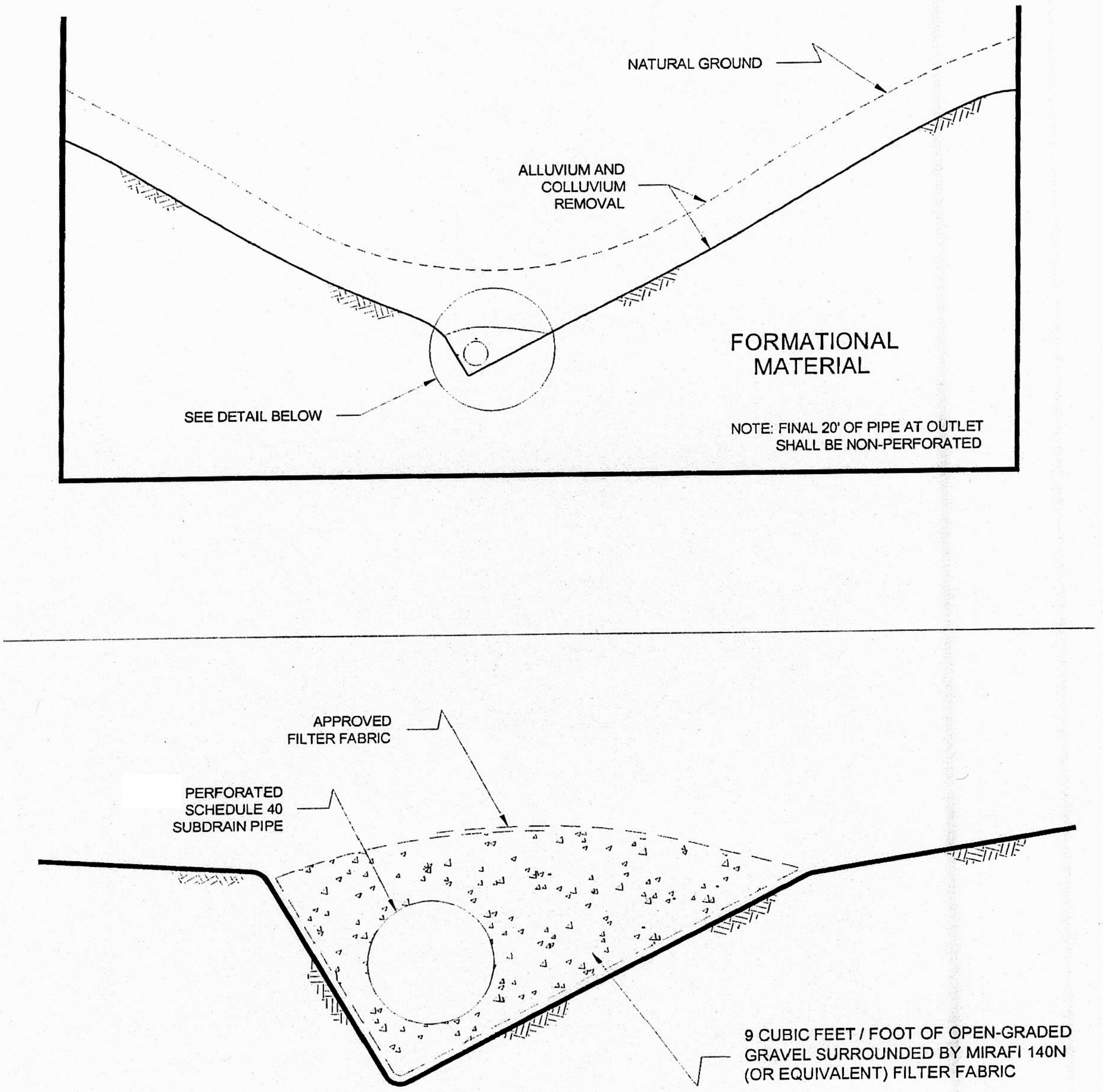
DATE JUNE 2014 | PROJECT NO. T2283-22-03 | FIG. 7



NOTES:

1...8-INCH DIAMETER SCHEDULE 40 PVC PERFORATED PIPE FOR SUBDRAINS AFTER 500 FEET OR AFTER 2 SUBDRAINS JOIN.

2...6-INCH DIAMETER SCHEDULE 40 PVC PERFORATED PIPE FOR SUBDRAINS UP TO 500 FEET LONG.

NO SCALE

TYPICAL CANYON SUBDRAIN DETAIL

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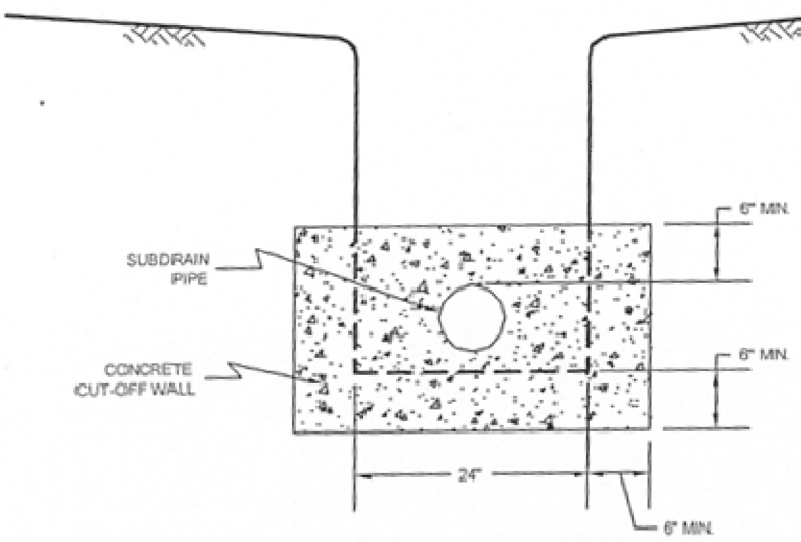
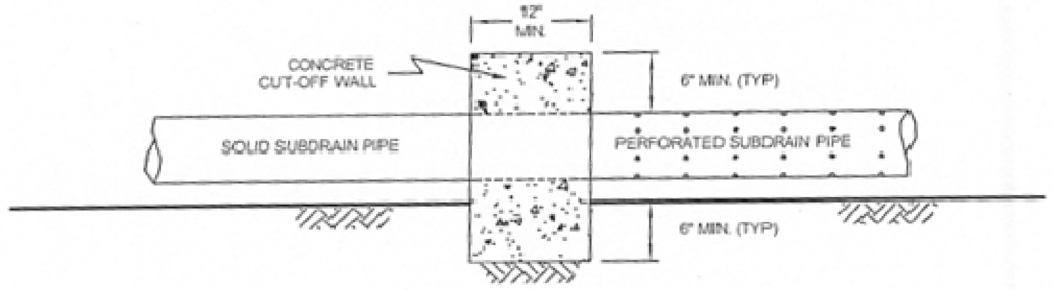
DATE JUNE 2014 | PROJECT NO. T2283-22-03 | FIG. 8



NO SCALE

SIDE VIEW



NO SCALE

RECOMMENDED SUBDRAIN CUT-OFF WALL

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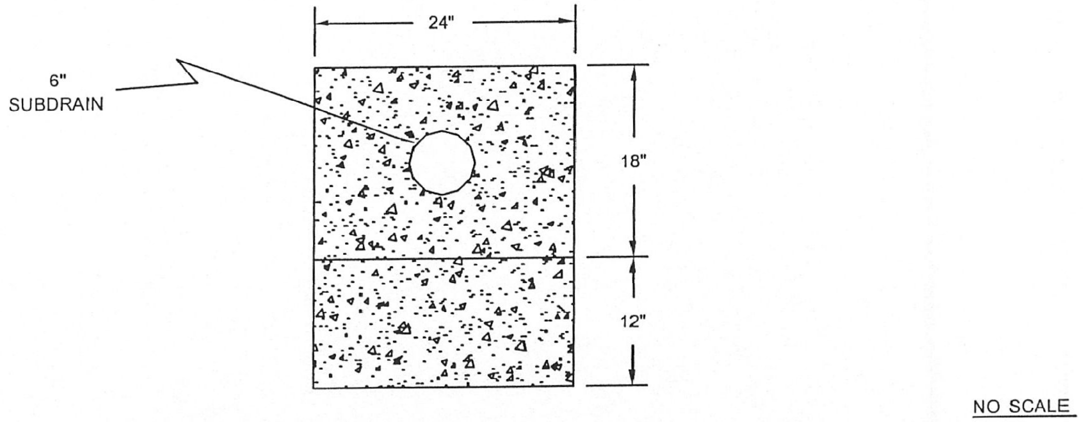
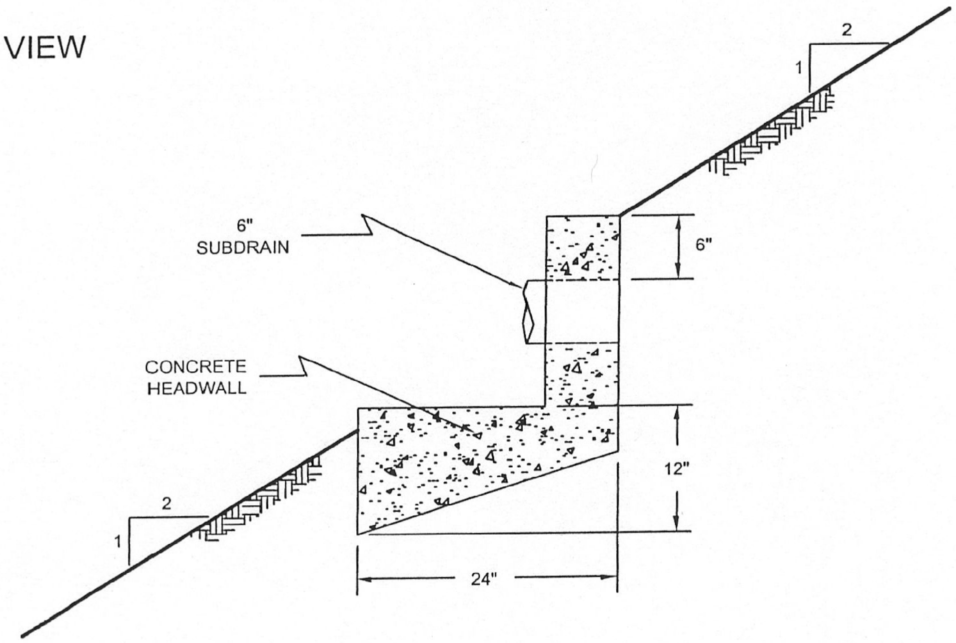
FIG. 9

FRONT VIEW



SIDE VIEW



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

NO SCALE

SUBDRAIN OUTLET HEADWALL DETAIL

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FIG. 10

APPENDIX A

APPENDIX A

FIELD INVESTIGATION

The field investigation was performed on January 17, 2005, and consisted of a site reconnaissance and the excavation of 10 backhoe trenches. Trenches were excavated with a track mounted backhoe equipped with a 24-inch bucket. Relatively undisturbed chunk samples and disturbed bulk samples were obtained from the exploratory trenches.

The soil conditions encountered in the excavations were visually examined, classified and logged in general accordance with American Society for Testing and Materials (ASTM) practice for Description and Identification of Soils (Visual-Manual Procedure D2488). Logs of the trenches are presented on Figures A-1 through A-10. The trench logs depict the soil and geologic conditions encountered and the depth at which samples were obtained. The approximate locations of the exploratory excavations are shown on the *Geologic Map* (Figure 2).

TABLE A-I
SEISMIC TRAVERSE RESULTS

Seismic Traverse No.	V1 (fps)	V2 (fps)	D1 (feet)	D2 (feet)	Anticipated Maximum Excavation in feet
S-1	2290	3950	10	30+	40
S-2	1970	2965	10	30+	48

V_1 = Velocity in feet per second of first layer of materials
V_2 = Second layer velocities
D_1 = Depth in feet to base of first layer
D_2 = Depth to base of second layer
fps = Feet per second

NOTE:

For mass grading, materials with velocities of less than approximately 5000 fps are generally rippable with a D9 Caterpillar Tractor equipped with a single shank hydraulic ripper. Velocities of 5000 to 6000 fps indicate marginal ripping and blasting. Velocities greater than 6000 fps generally require pre-blasting. For trenching, materials with velocities less than 3800 fps are generally rippable depending upon the degree of fracturing and the presence or absence of boulders. Velocities between 3800 and 4300 fps generally indicate marginal ripping, and velocities greater than 4300 fps generally indicate non-rippable conditions. The above velocities are based on a Kohring 505. The reported velocities represent average velocities over the length of each traverse, and should not generally be used for subsurface interpretation greater than 100 feet from a traverse.

PROJECT NO. T2283-12-02

TRENCH T 2

ELEV. (MSL.) 1540' DATE COMPLETED 01-17-2005

EQUIPMENT TRACK HOE

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0		+ +			**GRANITIC BEDROCK** Soft in upper 2 feet, hard below, moist, orange and white, hit pegmatite vein at 2 feet and met with refusal at 4 feet; moved trench 5 feet NW; practical refusal at 9 feet			
2		+ +						
4		+ +						
6		+ +						
8		+ +						
					TRENCH TERMINATED AT 9 FEET (Refusal) No groundwater encountered Backfilled			

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

T2283-12-01.GPJ

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

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

PROJECT NO. T2283-12-02

TRENCH T 3

ELEV. (MSL.) 1532' DATE COMPLETED 01-17-2005

EQUIPMENT TRACK HOE

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0		+ +			**GRANITIC BEDROCK** Soft, moist, orange, easy digging to 8 feet, excavates as coarse SAND with silt			
2								
4								
6								
8					TRENCH TERMINATED AT 8 FEET No groundwater encountered Backfilled			

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

T2283-12-01.GPJ

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

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

PROJECT NO. T2283-12-02

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	**TRENCH T 4** ELEV. (MSL.) **1496'** DATE COMPLETED **01-17-2005** EQUIPMENT **TRACK HOE**	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
					MATERIAL DESCRIPTION			
0 2	▨ T4-1			SM	**ALLUVIUM** Loose, wet, dark brown, Silty SAND			
					GRANITIC BEDROCK Soft, very moist, orange, highly weathered			
					TRENCH TERMINATED AT 3.5 FEET No groundwater encountered Backfilled			

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

T2283-12-01.GPJ

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

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

PROJECT NO. T2283-12-02

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	**TRENCH T 5** ELEV. (MSL.) **1504'** DATE COMPLETED **01-17-2005** EQUIPMENT **TRACK HOE**	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
					MATERIAL DESCRIPTION			
0 2					**GRANITIC BEDROCK** Soft, moist, light orange-brown, completely weathered, very easy digging			
					TRENCH TERMINATED AT 3.5 FEET No groundwater encountered Backfilled			

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

T2283-12-01.GPJ

SAMPLE SYMBOLS

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

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

PROJECT NO. T2283-12-02

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	**TRENCH T 6** ELEV. (MSL.) **1524'** DATE COMPLETED **01-17-2005** EQUIPMENT **TRACK HOE**	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
					MATERIAL DESCRIPTION			
0			▽	SM	**ALLUVIUM** Loose, wet, brown, Silty, coarse SAND			
2					**GRANITIC BEDROCK** Soft, wet. light orange, completely weathered, excavates as coarse SAND with silt			
					TRENCH TERMINATED AT 2.5 FEET Seepage at contact at 1 foot Backfilled			

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

T2283-12-01.GPJ

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

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

PROJECT NO. T2283-12-02

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	**TRENCH T 7** ELEV. (MSL.) **1544'** DATE COMPLETED **01-17-2005** EQUIPMENT **TRACK HOE**	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
					MATERIAL DESCRIPTION			
0		+ +			**GRANITIC BEDROCK** Soft in upper 4 feet, moderately hard below, moist, orange-brown, moderately difficult digging 3 to 6 feet; difficult digging below 6 feet			
2								
4								
6								
8								
10								
12					TRENCH TERMINATED AT 12 FEET (Refusal) No groundwater encountered Backfilled			

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

T2283-12-01.GPJ

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

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PROJECT NO. T2283-12-02

TRENCH T 8

ELEV. (MSL.) **1560'** DATE COMPLETED **01-17-2005**

EQUIPMENT **TRACK HOE**

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 4					**GRANITIC BEDROCK** Soft in upper 2 feet, moderately hard to hard below, moist, orange, highly weathered and locally massive; practical refusal at 4.5 feet			
					TRENCH TERMINATED AT 4.5 FEET (Refusal) No groundwater encountered Backfilled			

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

T2283-12-01.GPJ

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

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

PROJECT NO. T2283-12-02

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	**TRENCH T 9** ELEV. (MSL.) **1560'** DATE COMPLETED **01-17-2005** EQUIPMENT **TRACK HOE**	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
					MATERIAL DESCRIPTION			
0 2 4		+ +			**GRANITIC BEDROCK** Soft in upper 2 feet, moist, orange, becomes moderately difficult digging at 2.5 feet; practical refusal at 4 feet			
					TRENCH TERMINATED AT 4 FEET (Refusal) No groundwater encountered Backfilled			

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

T2283-12-01.GPJ

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

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

PROJECT NO. T2283-12-02

TRENCH T 10

ELEV. (MSL.) 1544' DATE COMPLETED 01-17-2005

EQUIPMENT TRACK HOE

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0		+ +			**GRANITIC BEDROCK** Soft in upper 3 feet, moderately hard below, moist, orange, excavates as coarse SAND with trace silt			
	T10-1	+ +					142.7	3.7
2		+ +						
		+ +		SP				
4		+ +						
		+ +						
6		+ +						
					TRENCH TERMINATED AT 6.5 FEET (Refusal) No groundwater encountered Backfilled			

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

SAMPLE SYMBOLS

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

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APPENDIX B

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. Geocon tested relatively undisturbed chunk samples to determine the in-situ moisture and density of the soil materials. Bulk samples were tested to determine the maximum density/optimum moisture, the direct shear properties, the expansion potential, soluable sulfate content, pH, and resistivity. Results of the laboratory tests are presented herin and are included in the trench logs.

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.)
T1-2	Orange brown, Silty fine to coarse SAND	129.9	8.1

TABLE B-II
SUMMARY OF LABORATORY EXPANSION INDEX TEST RESULTS
ASTM D4829-95

Sample No.	Moisture Content		Dry Density (pcf)	Expansion Index
	Before Test (%)	After Test (%)		
T1-2	8.5	17.0	116.9	2

TABLE B-III
SUMMARY OF DIRECT SHEAR TEST RESULTS

Sample No.	Dry Density (pcf)	Moisture Content (%)	Unit Cohesion (psf)	Angle of Shear Resistance (degrees)
T1-2	117.0	8.0	150	42

Sample remolded to approximately 90 percent relative compaction near optimum moisture content.

TABLE B-IV
SUMMARY OF CHEMICAL TEST RESULTS

Sample No.	Sulfate Content (%)	pH	Resisitivity (ohm centimeters)
T1-2	0.015	6.2	5140
T4-1	0.014	Not Tested	Not Tested

Resistivity and pH determined by Cal Trans Test 532.
Water-soluable sulfate determined by California Test 417.

APPENDIX C

APPENDIX C

RECOMMENDED GRADING SPECIFICATIONS

FOR

TRACT 29476
WILDOMAR, CALIFORNIA

PROJECT NO. T2283-22-03

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 Inland Empire, Incorporated. The recommendations contained in the text of the Geotechnical Report are a part of the earthwork and grading specifications and shall supersede the provisions contained hereinafter in the case of conflict.

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

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

2. DEFINITIONS

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

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

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

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

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

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

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

3. MATERIALS

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

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

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

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, gradation and chemical characteristics of the soil.

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

4. CLEARING AND PREPARING AREAS TO BE FILLED

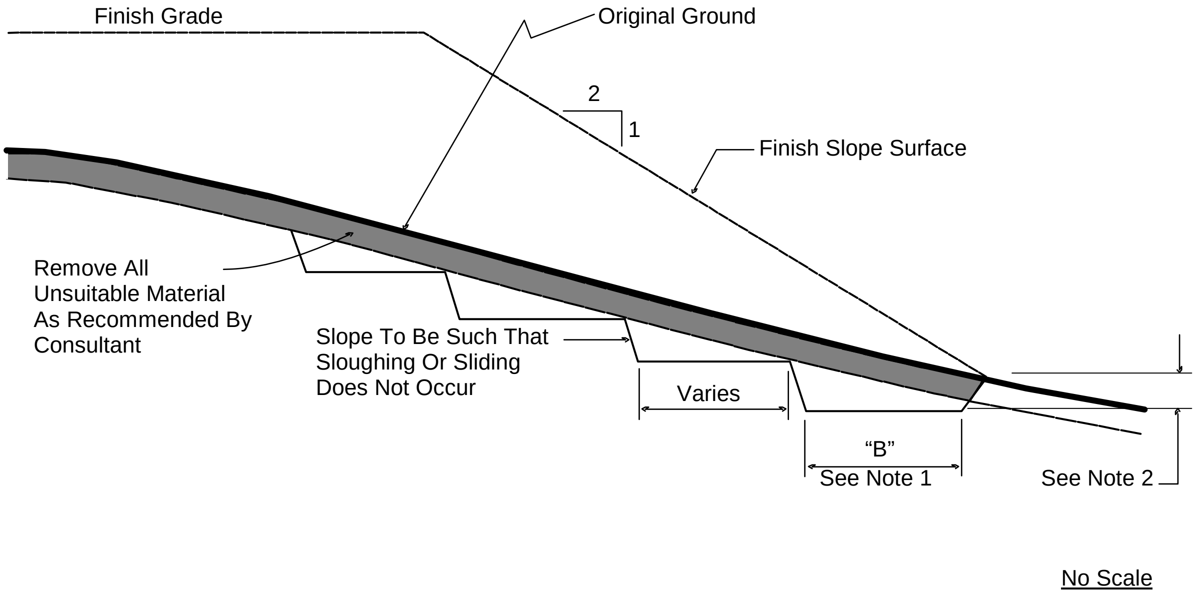
4.1 Areas to be excavated and filled shall be cleared and grubbed. Clearing shall consist of complete removal above the ground surface of trees, stumps, brush, vegetation, man-made structures, and similar debris. Grubbing shall consist of removal of stumps, roots, buried logs and other unsuitable material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

4.2 Any asphalt pavement material removed during clearing operations should be properly disposed at an approved off-site facility. Concrete fragments that are free of exposed 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 1557-02.

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 1557-02. 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 10 feet below finish grade or 3 feet below the deepest utility, whichever is deeper. In the event that placement of oversized rock is planned less than 10 feet below finish grade, 15 feet behind slope face, or 3 feet below deepest utility, Geocon should be consulted for additional recommendations.

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 1196-93, may be performed in both the compacted *soil* fill and in the *rock* fill to aid in determining the required minimum number of passes of the compaction equipment. If performed, a minimum of three plate bearing tests should be performed in the properly compacted *soil* fill (minimum relative compaction of 90 percent). Plate bearing tests shall then be performed on areas of *rock* fill having two passes, four passes and six passes of the compaction equipment, respectively. The number of passes required for the *rock* fill shall be determined by comparing the results of the plate bearing tests for the *soil* fill and the *rock* fill and by evaluating the deflection variation with number of passes. The required number of passes of the compaction equipment will be performed as necessary until the plate bearing deflections are equal to or less than that determined for the properly compacted *soil* fill. In no case will the required number of passes be less than two.

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

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

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

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

7. OBSERVATION AND TESTING

7.1 The Consultant shall be the Owner's representative to observe and perform tests during clearing, grubbing, filling, and compaction operations. In general, no more than 2 feet in vertical elevation of *soil* or *soil-rock* fill should be placed without at least one field density test being performed within that interval. In addition, a minimum of one field density test should be performed for every 2,000 cubic yards of *soil* or *soil-rock* fill placed and compacted.

7.2 The Consultant should perform a sufficient distribution of field density tests of the compacted *soil* or *soil-rock* fill to provide a basis for expressing an opinion whether the fill material is compacted as specified. Density tests shall be performed in the compacted materials below any disturbed surface. When these tests indicate that the density of any layer of fill or portion thereof is below that specified, the particular layer or areas represented by the test shall be reworked until the specified density has been achieved.

7.3 During placement of *rock* fill, the Consultant should observe that the minimum number of passes have been obtained per the criteria discussed in Section 6.3.3. The Consultant should request the excavation of observation pits and may perform plate bearing tests on the placed *rock* fills. The observation pits will be excavated to provide a basis for expressing an opinion as to whether the *rock* fill is properly seated and sufficient moisture has been applied to the material. When observations indicate that a layer of *rock* fill or any portion thereof is below that specified, the affected layer or area shall be reworked until the *rock* fill has been adequately seated and sufficient moisture applied.

7.4 A settlement monitoring program designed by the Consultant may be conducted in areas of *rock* fill placement. The specific design of the monitoring program shall be as recommended in the Conclusions and Recommendations section of the project Geotechnical Report or in the final report of testing and observation services performed during grading.

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

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

7.6.1 Soil and Soil-Rock Fills:

7.6.1.1 Field Density Test, ASTM D 1556-02, *Density of Soil In-Place By the Sand-Cone Method.*

7.6.1.2 Field Density Test, Nuclear Method, ASTM D 2922-01, *Density of Soil and Soil-Aggregate In-Place by Nuclear Methods (Shallow Depth).*

7.6.1.3 Laboratory Compaction Test, ASTM D 1557-02, *Moisture-Density Relations of Soils and Soil-Aggregate Mixtures Using 10-Pound Hammer and 18-Inch Drop*.

7.6.1.4. Expansion Index Test, ASTM D 4829-03, *Expansion Index Test*.

7.6.2 Rock Fills

7.6.2.1 Field Plate Bearing Test, ASTM D 1196-93 (Reapproved 1997) *Standard Method for Nonreparative Static Plate Load Tests of Soils and Flexible Pavement Components, For Use in Evaluation and Design of Airport and Highway Pavements.*

8. PROTECTION OF WORK

8.1 During construction, the Contractor shall properly grade all excavated surfaces to provide positive drainage and prevent ponding of water. Drainage of surface water shall be controlled to avoid damage to adjoining properties or to finished work on the site. The Contractor shall take remedial measures to prevent erosion of freshly graded areas until such time as permanent drainage and erosion control features have been installed. Areas subjected to erosion or sedimentation shall be properly prepared in accordance with the Specifications prior to placing additional fill or structures.

8.2 After completion of grading as observed and tested by the Consultant, no further excavation or filling shall be conducted except in conjunction with the services of the Consultant.

9. CERTIFICATIONS AND FINAL REPORTS

9.1 Upon completion of the work, Contractor shall furnish Owner a certification by the Civil Engineer stating that the lots and/or building pads are graded to within 0.1 foot vertically of elevations shown on the grading plan and that all tops and toes of slopes are within 0.5 foot horizontally of the positions shown on the grading plans. After installation of a section of subdrain, the project Civil Engineer should survey its location and prepare an *as-built* plan of the subdrain location. The project Civil Engineer should verify the proper outlet for the subdrains and the Contractor should ensure that the drain system is free of obstructions.

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