PRELIMINARY GEOTECHNICAL INVESTIGATION

PROPOSED MORONGO ENTERTAINMENT CENTER PORTIONS OF SECTIONS 8 AND 9 OF T3S R2E, CABAZON AREA, RIVERSIDE COUNTY, CALIFORNIA

GEOCON
WEST, INC.

GEOTECHNICAL
ENVIRONMENTAL
MATERIALS

PREPARED FOR

MSA CONSULTING, INC.
RANCHO MIRAGE, CALIFORNIA

APRIL 4, 2013
PROJECT NO. T2533-22-01

GEOCON
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Project No. T2533-22-01
April 4, 2013

MSA Consulting, Inc.
34200 Bob Hope Drive
Rancho Mirage, California 92270

Attention: Mr. Marvin Roos

Subject: PRELIMINARY GEOTECHNICAL INVESTIGATION
PROPOSED MORONG ENTERTAINMENT CENTER
PORTIONS OF SECTIONS 8 AND 9, T3S R2E
CABAZON AREA, RIVERSIDE COUNTY, CALIFORNIA

Dear Mr. Roos:

In accordance with your authorization of our proposal IE-1036 dated May 3, 2012, we have performed a preliminary geotechnical investigation of the subject property for the proposed Morongo Entertainment Center. The site encompasses portions of Sections 8 and 9 of Township 3 South Range 2 East and is located northeast of the existing Morongo Casino in the Cabazon Area of Riverside County, California. The accompanying report presents the results of our study 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 suitable for development, provided the recommendations of this report are followed and implemented during design and construction.

Should you have questions regarding this geotechnical investigation, 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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PRELIMINARY GEOTECHNICAL INVESTIGATION

1. PURPOSE AND SCOPE

This report presents the findings of a preliminary geotechnical investigation for the approximately 310-acre site located northeast of the existing Morongo Casino, in the Cabazon Area of Riverside County, California, see *Vicinity Map*, Figure 1. The purpose of the investigation was to evaluate the geologic conditions, identify potential geologic hazards, review previous investigations in the vicinity and, based on the conditions encountered; provide recommendations regarding the geotechnical aspects of developing the property as presently proposed.

The scope of the investigation included a site reconnaissance, review of aerial photographs and pertinent geologic literature (*References*), and a preliminary geotechnical site investigation, and percolation testing. The scope of our investigation included the excavation of 15 geotechnical test pits to maximum depths of 15 feet. A majority of the test pits were excavated in the proposed festival area. The test pit logs are presented in Appendix A. Laboratory test data is presented in Appendix B. Percolation testing was performed in four of the test pits at locations indicated by MSA Consulting, Inc. Percolation Test Data is provided in Appendix C. Approximate locations of the exploratory excavations are depicted on the *Site Plan*, Figure 2.

2. SITE AND PROJECT DESCRIPTION

The irregularly-shaped site encompasses the eastern portion of Section 8 and the western portion of Section 9 of Township 3 South, Range 2 East. The site is bounded on the north by a pipeline utility easement, on the south by Seminole Parkway, and on the west and east by undeveloped land. The site is composed of alluvial fan deposits derived from the San Bernardino Mountains to the north. The site is currently used as grazing land for cattle. Telephone poles traverse the site in the southern portion, and bisect the site north-south. Two petroleum pipelines traverse the site in the southern portion. Electrical lines and a petroleum pipeline traverse the site along the northern boundary. A 25-foot-wide irrigation easement bisects the site from northwest to southeast. The western boundary is flanked by an irrigation pipeline. In the central portion of the site is a 5-acre Cabazon Water Company water tank (not a part of the site). The site is relatively homogeneous with sparse to moderate shrubs, and minor braided streambeds. A barbed wire fence surrounds the property, as well as bisecting the southwest corner, near the access point. Access to the site is available from a locked gate maintained by the Morongo Band of Mission Indians, off of Seminole Parkway. Existing site elevations range from approximately 2120 feet Mean Sea Level (MSL) in the northwest corner of the property descending to 1849 feet MSL in the southeast corner of the site. Several southeast flowing drainages are present in the eastern portion of the site.

The propose development will consist of an approximately 68-acre entertainment area within the north central portion of the site. The entertainment venue will include an amphitheater, backstage area, festival grounds, tent area and several flexible use areas. Parking will be located south of the festival grounds to Seminole Parkway. Recreational vehicle camping area will be located west of the festival grounds and the eastern portion of the site will be utilized for tent/car camping. Tunnels maybe constructed in association with the festival improvements. As of the date of this report a grading plan has not been prepared. However, minor cuts and fills are anticipated to be needed to achieve designs presented in MSA Consulting, Inc.'s *Conceptual Site Plan, Alt 3*.

The descriptions of the site and proposed development are based on a site reconnaissance, observations and a review of the conceptual plan. If project details differ significantly from those described, Geocon should be contacted for review and possible revisions to this report.

3. SOIL AND GEOLOGIC CONDITIONS

Geologic units anticipated at the site include localized areas of undocumented fill and alluvial fan deposits. The undocumented fill was not encountered within the test pits excavated for this study, however, based on our site observations fill will likely be encountered in association with utility trench backfill and in localized areas within the property. It is not a designated geologic unit and therefore, is not discussed in detail below. The geologic nomenclature generally follows that of Dibblee (*References*) and is discussed below.

3.1 Alluvial Fan (Qf)

Quaternary-age alluvial fan deposits were encountered in all of the test pits excavated for this study. The fan consists of mainly coarse deposits emanating from the San Bernardino Mountains north of the site. The predominant soil types include sandy gravel with variable amounts of silt, cobbles, and boulders (30%) up to 24 inches in diameter. Excavation encountered clast-supported boulder layers with little sand matrix. A significant quantity of oversized rock should be anticipated during grading. The upper 2 to 3 feet of the unit is bioturbated by roots and rodent burrows. The soil was generally dry and loose to medium dense. The upper two to three feet of this unit are not considered suitable for support of structural fill or proposed structures and should be excavated and properly compacted where structures or surface improvements are planned.

The soil conditions described in the previous paragraphs are generalized. A more detailed description is provided on the test pit logs included in Appendix A. The logs include soil type, color, moisture, consistency, and classification of the soil encountered at specific locations and elevations.

4. GROUNDWATER

The site is located within the Whitewater Subbasin. Groundwater was not encountered during our subsurface investigation to depths of 15 feet. Well data from wells within Section 7, approximately 1 mile west of the site indicates groundwater is on the order of 430 to 500 feet below ground surface (Water Data Library). It is anticipated that a large quantity of water will be required during grading operations on the site.

5. GEOLOGIC HAZARDS

5.1 Surface Fault Rupture Hazard

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, formerly known as California Division of Mines and Geology (CDMG), for the Alquist-Priolo Earthquake Fault Zone Program (Hart, 1999). 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 Alquist-Priolo Earthquake Fault Zone for surface fault rupture hazards. Nor is it within a Riverside County Fault Hazard Zone. The San Gorgonio Fault Zone is located approximately ⅓ mile north of the site and immediately northwest of the site. The thrust fault is easily recognized in aerial photographs preceding the adjacent retail outlet development. We did not observe indications of on-site faulting during our aerial photograph review. There was no evidence of on-site faulting encountered during our exploration of the site.

5.2 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 (Harden, 1998). The San Gorgonio Fault Zone is a compressional zone as the San Andreas bends to the west. The compressional stresses through the pass have resulted in thrust faulting at the base of the foothills and across alluvial fans in the vicinity of the site.

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

5.3 Estimation of Peak Ground Accelerations

The seismic exposure of the site may be investigated in two ways. The deterministic approach recognizes the Maximum Earthquake, which is the theoretical maximum event that could occur along a fault. The deterministic method assigns a maximum earthquake to a fault derived from formulas that correlate the length and other characteristics of the fault trace to the theoretical maximum magnitude earthquake. The probabilistic method considers the probability of exceedance of various levels of ground motion and is calculated by consideration of risk contributions from regional faults.

5.3.1 Deterministic Analysis

According to the computer program *EZ-FRISK (Version 7.51)* 19 known active faults are located within 50 miles of the property. The nearest known active fault considered by *EZFRISK* is the San Andreas fault, located approximately 0.4 mile northeast of the site. The San Andreas fault is the dominant source of potential ground motion for the site. Earthquakes that might occur on the San Andreas fault or other faults within the southern California and northern Baja California area are potential generators of significant ground motion at the site. The estimated maximum earthquake magnitude and peak ground acceleration for the San Andreas fault are 8.2 and 0.60g, respectively. Table 5.3.1 lists the estimated maximum earthquake magnitude and peak ground acceleration for the ten most dominant faults in relation to the site location. We calculated peak ground acceleration (PGA) using Boore-Atkinson (2008), Campbell-Bozorgnia (2008), and Chiou-Youngs (2008).

TABLE 5.3.1
DETERMINISTIC SPECTRA SITE PARAMETERS

Fault Name	Distance from Site (miles)	Maximum Earthquake Magnitude (Mw)	Peak Ground Acceleration		
			Boore-Atkinson (2008) (g)	Campbell-Bozorgnia (2008) (g)	Chiou-Youngs (2008) (g)
Southern San Andreas	<1	8.2	0.51	0.44	0.60
Pinto Mountain	10	7.3	0.26	0.22	0.27
San Jacinto	12	7.9	0.25	0.19	0.25
Mojave Shear Zone	17	7.6	0.22	0.16	0.25
North Frontal (East)	19	7.0	0.19	0.13	0.16
Burnt Mountain	21	6.8	0.16	0.11	0.11
North Frontal (West)	24	7.2	0.17	0.11	0.14
Eureka Peak	26	6.7	0.13	0.09	0.08
Landers	27	7.4	0.16	0.11	0.12
Helendale-South Lockhart	27	7.4	0.16	0.11	0.12

In the event of a major earthquake on the referenced faults or other significant faults in the southern California and northern Baja California area, the site could be subjected to moderate to severe ground shaking. With respect to this hazard, the site is considered comparable to others in the general vicinity.

5.3.2 Probabilistic Analysis

We performed a site-specific probabilistic seismic hazard analysis using the computer program *EZ-FRISK*. Geologic parameters not addressed in the deterministic analysis are included in this analysis. The program operates under the assumption that the occurrence rate of earthquakes on each mappable Quaternary fault is proportional to the faults slip rate. The program accounts for fault rupture length as a function of earthquake magnitude, and site acceleration estimates are made using the earthquake magnitude and distance from the site to the rupture zone. The program also accounts for uncertainty in each of following: (1) earthquake magnitude, (2) rupture length for a given magnitude, (3) location of the rupture zone, (4) maximum possible magnitude of a given earthquake, and (5) acceleration at the site from a given earthquake along each fault. By calculating the expected accelerations from considered earthquake sources, the program calculates the total average annual expected number of occurrences of site acceleration greater than a specified value. We utilized acceleration-attenuation relationships suggested by Boore-Atkinson (2008), Campbell-Bozorgnia (2008), and Chiou-Youngs (2008) in the analysis. Table 5.3.2 presents the site-specific probabilistic seismic hazard parameters including acceleration-attenuation relationships and the probability of exceedence.

TABLE 5.3.2
PROBABILISTIC SEISMIC HAZARD PARAMETERS

Probability of Exceedence	Peak Ground Acceleration		
	Boore-Atkinson 2008 (g)	Campbell-Bozorgnia 2008 (g)	Chiou-Youngs 2008 (g)
2% in a 50 Year Period	1.14	0.87	1.09
5% in a 50 Year Period	0.87	0.69	0.87
10% in a 50 Year Period	0.67	0.55	0.70

The California Geologic Survey (CGS) has a program that calculates the ground motion for a 10 percent probability of exceedence in a 50-year period based on an average of several attenuation relationships. Table 5.3.3 presents the calculated results from the Probabilistic Seismic Hazards Mapping Ground Motion Page from the CGS website.

TABLE 5.3.3
PROBABILISTIC SITE PARAMETERS FOR SELECTED FAULTS
CALIFORNIA GEOLOGIC SURVEY

Ground Motion	Calculated Acceleration (g) Firm Rock	Calculated Acceleration (g) Soft Rock	Calculated Acceleration (g) Alluvium
Pga	0.61	0.61	0.61

While listing peak accelerations is useful for comparison of potential effects of fault activity in a region, other considerations are important in seismic design, including the frequency and duration of motion and the soil conditions underlying the site. Seismic design of the structures should be performed in accordance with the 2010 California Building Code (CBC) guidelines currently adopted by the County of Riverside.

5.4 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 soil, *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. Due to the depth of the permanent water table, it is our opinion that the potential for liquefaction at this site is very low.

6. CONCLUSIONS AND RECOMMENDATIONS

6.1 General

6.1.1 It is our opinion that neither soil nor geologic conditions were encountered during the investigation that would preclude the construction of the proposed entertainment center provided the recommendations presented herein are followed and implemented during design and construction.

6.1.2 Any undocumented fill (if encountered), and the upper approximately three feet of site soil deposits are considered unsuitable in their present condition for support of proposed improvements and will require remedial grading. Removals on the order of three feet with scarification, moisture conditioning (flooding) and compaction of the removal bottom can be expected, with possible deeper excavations needed. Actual removal depths should be determined by the geotechnical engineer or their representative in the field during grading.

6.1.3 Groundwater was not encountered during the subsurface investigation and groundwater related problems are not anticipated during grading.

6.1.4 Percolation testing was performed at four locations within the site to provide preliminary infiltration rates for on-site soil. The test results indicate percolation rates on the order of 42 to 240 inches/hour can be anticipated for onsite sandy soils in a loose state for preliminary design purposes. Actual infiltration testing should be performed at the location and depth of the proposed infiltration structures once those have been determined.

6.1.5 Once a design layout has been determined and structural loads are known, a more detailed geotechnical review and investigation should be performed to address specific site improvements.

6.2 Soil and Excavation Characteristics

6.2.1 It is our opinion that undocumented fill (if encountered), and alluvium can be excavated using conventional heavy-duty grading equipment. Over-sized rock (greater than 6 inches in diameter) will be encountered. It should be placed in accordance with the *Recommended Grading Specifications* presented in Appendix D and the requirements of the County of Riverside.

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. For trenching purposes onsite soil should be considered Type "C" with back cuts and trenching protection designed accordingly.

6.2.3 The soil encountered in the field investigation is considered to be "non-expansive" (Expansion Index [EI] of 20 or less) as defined by 2010 California Building Code (CBC) Section 1803.5.3. Table 6.2.3 presents soil classifications based on the expansion index. A majority of the soil encountered appears to possess a "very low" expansion potential (expansion index of 20 or less).

TABLE 6.2.3
EXPANSIVE SOIL CLASSIFICATION BASED ON EXPANSION INDEX

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

6.2.4 We performed laboratory tests on samples of the site soil to evaluate the percentage of water-soluble sulfate content. Results from the laboratory water-soluble sulfate content tests are presented in Appendix B and indicate that the soil tested possesses "negligible" sulfate exposure to concrete structures as defined by 2010 CBC Section 1904.3 and ACI 318-08 Sections 4.2 and 4.3. 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.

6.2.5 Potential of hydrogen (pH) and resistivity testing on selected soil samples indicate a pH of 8.1 to 8.4, and a resistivity of 3,000 and 22,000 ohm-cm. These results indicate that the soil tested is "moderately corrosive" to "corrosive" for resistivity, according to NACE International.

6.2.6 Import soil, if required, should be no more corrosive than the on-site soil and exhibit an Expansion Index of 20 or less. Furthermore, it should be free of organic material and construction debris, and not contain rock larger than six inches in maximum dimension. Import material should be sampled, tested and approved by the geotechnical engineer prior to its transportation to the site.

6.2.7 Geocon does not practice in the field of corrosion engineering and mitigation. Therefore, if improvements that could be susceptible to corrosion are planned, we recommend that further evaluation by a corrosion engineer be performed. We also recommend that these results and the recommendations from the corrosion engineer be forwarded to the appropriate design team members (i.e., project architect, structural engineer, etc.) for incorporation into the plans and implementation during construction.

6.3 Grading

6.3.1 Grading should be performed in accordance with the *Recommended Grading Specifications* contained in Appendix D. Where the recommendations of this section are in conflict with those of Appendix D, 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 soil exposed in cut areas or to be used as fill is relatively free of organic matter. Material generated during stripping and site demolition should be exported from the site.

6.3.4 Although not observed during our site visit, any water wells on site should be abandoned in accordance with California Well Standards Bulletin 74-81, amended by Bulletin 74-90. In addition, any septic systems that exist within the area of the proposed improvements will require removal.

6.3.5 Undocumented fill, and alluvium in the areas of proposed improvements should be removed to expose competent alluvial soil exhibiting an in place dry density of at least 85 percent of the maximum dry density (ASTM D1557). In areas where the quantity of cobble prevents density testing with a nuclear gage, the removal bottom should be approved by the Geotechnical Engineer. We anticipate excavation of the upper three feet of soil will be necessary; however, the actual excavation depth should be determined in the field by a representative of Geocon. Undocumented fill, and alluvium may be utilized as fill provided they are free of organic material, deleterious material, and rocks larger than six inches. The client and contractor should be aware that a significant quantity of oversize rock will be generated during the grading operations.

6.3.6 During remedial grading, temporary slopes should be planned for an inclination no steeper than 1.5:1 (horizontal:vertical). Grading should be scheduled to backfill against these slopes as soon as practical.

6.3.7 Where excavation and compaction is to be conducted, the excavations should be extended laterally a minimum distance of five feet beyond the building footprint area 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 needed.

6.3.8 After removal of surficial and unsuitable soil, the exposed ground surface should be scarified, moisture conditioned (flooded), and compacted to a dry density of at least 90 percent of the laboratory maximum dry density near to slightly above optimum moisture content as determined by ASTM D 1557 (latest edition). Fill may then be placed and compacted in layers to the design finish grade elevations. Fill, including backfill and scarified ground surfaces, should be moisture conditioned to near to slightly above optimum moisture content and compacted to a dry density of at least 90 percent of laboratory maximum dry density, as determined by ASTM D 1557.

6.3.9 Where new paving is to be placed, it is recommended that all existing fill and soft or loose alluvial soil be excavated and properly compacted for paving support. Fill placed within areas to be paved should be compacted to at least 95 percent relative compaction for paving support.

6.3.10 Utility trenches should be properly backfilled in accordance with the requirements of the Green Book (latest edition). The pipe should be bedded with clean sands (Sand Equivalent greater than 30) containing no rock greater than 6 inches to a depth of at least one foot over the pipe. The use of gravel is not acceptable unless used in conjunction with filter fabric to prevent the gravel from having direct contact with soil. Jetting may only be performed if trench wall soils have an SE of 15 or greater. Laboratory test results indicate the on-site soil has a sand equivalent of 59. The remainder of the trench backfill may be derived from onsite soil (provided it is free of deleterious material and rocks greater than 6 inches) or approved import soil, compacted as necessary, until the required compaction is obtained. The use of one- or two-sack slurry is also acceptable. If the trench backfill soil contains significant gravel size particles (as is anticipated if on site soil is used), care should be taken to ensure that voids spaces between the gravel particles are filled during the placement and compaction process. Flooding of the trench backfill may be preferable to other compaction methods in this situation.

6.3.11 All excavation bottoms must be observed and approved in writing by the Geotechnical Engineer (a representative of Geocon), prior to placing bedding materials, fill, steel, gravel or concrete.

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. Variations in natural soil density, as well as in compacted fill density, render bulking and shrinkage value estimates very approximate. Based on our experience with similar soil, it is our opinion that the shrinkage and bulking factors in Table 6.4.1 can be used as a basis for estimating how much the on-site soil may shrink or swell (bulk) when excavated from its natural state and placed as compacted fill at 92 percent relative compaction (per ASTM D 1557).

TABLE 6.4.1
SHRINK/BULK FACTORS

Soil Unit	Shrink/Bulk Factor
Undocumented Fill	10 to 15 percent shrink
Alluvium	5 to 10 percent shrink

6.5 Seismic Design Criteria

6.5.1 We used the computer program *Seismic Hazard Curves and Uniform Hazard Response Spectra*, provided by the USGS. Table 6.5.1 summarizes site-specific design criteria obtained from the 2010 California Building Code (CBC; Based on the 2009 International Building Code [IBC]), Chapter 16 Structural Design, Section 1613 Earthquake Loads. The short spectral response uses a period of 0.2 second. The building structure and improvements should be designed using a Site Class D.

TABLE 6.5.1
CBC SEISMIC DESIGN PARAMETERS

Parameter	Value	CBC Reference
Site Class	D	Table 1613.5.2
Spectral Response – Class C (short), S_S	1.526 g	Figure 1613.5(3)
Spectral Response – Class C (1 sec), S_1	0.662 g	Figure 1613.5(4)
Site Coefficient, F_a	1.0	Table 1613.5.3(1)
Site Coefficient, F_v	1.5	Table 1613.5.3(2)

Maximum Considered Earthquake Spectral Response Acceleration (short), S_{MS}	1.526 g	Section 1613.5.3 (Eqn 16-36)
Maximum Considered Earthquake Spectral Response Acceleration – (1 sec), S_{M1}	0.993 g	Section 1613.5.3 (Eqn 16-37)
5% Damped Design Spectral Response Acceleration (short), S_{DS}	1.017 g	Section 1613.5.4 (Eqn 16-38)
5% Damped Design Spectral Response Acceleration (1 sec), S_{D1}	0.662 g	Section 1613.5.4 (Eqn 16-39)

6.5.2 Conformance to the criteria in Table 6.6.1 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 Foundation Design

6.6.1 Subsequent to the recommended grading, the proposed stage, restroom buildings and other lightly loaded or similar structures can be supported on shallow foundation systems deriving support in engineered fill containing no rock larger than 6 inches. Foundations for the structures may consist of continuous strip footings and isolated spread footings.

6.6.2 Shallow foundation systems can be designed using the method described in Section 1808 of the 2010 CBC. If post-tensioned foundations are planned, an alternative, commonly accepted design method (other than PTI Third Edition) can be used. However, the post-tensioned foundation system should be designed with a total and differential deflection of 1 inch. Geocon should be contacted to review the plans and provide additional information, if necessary.

6.6.3 Conventionally reinforced continuous footings should be at least 12 inches wide and extend at least 18 inches below lowest adjacent pad grade. Isolated spread footings should have a minimum width of 2 feet and should extend at least 18 inches below lowest adjacent pad grade.

6.6.4 Footings should be sized (using the above minimum footing dimensions) based on an allowable soil bearing pressure of 2,000 pounds per square foot (psf). This allowable soil bearing capacity may be increased by 500 psf for each additional foot of footing embedment and 200 psf for each additional foot of width to a maximum value of 4,000 psf. The allowable bearing pressure value is for dead plus live loads and may be increased by one-third when considering transient loads due to wind or seismic forces. Steel reinforcement for continuous footings should consist of at least two No. 4 steel reinforcing bars placed horizontally in the footings, one near the top and one near the bottom. Steel reinforcement for the spread footings should be designed by the project structural engineer.

6.6.5 The minimum reinforcement recommended above is based on soil characteristics only and is not intended to replace reinforcement required for structural considerations.

6.6.6 Foundation recommendations for structures such as towers and tunnels should be determined when structural details are known.

6.6.7 No special subgrade preparation is deemed necessary prior to placing concrete, however, the exposed foundation and slab subgrade soil should be sprinkled, as necessary, to maintain a moist soil condition as would be expected in any such concrete placement. However, where a long period of drying of the subgrade soil has occurred, reconditioning of the surficial soil will be required. This recommendation applies to foundations as well as exterior concrete flatwork.

6.6.8 The embedment depths should be measured from the lowest adjacent pad sub-grade for both interior and exterior footings. Figure 3 depicts the depth to lowest adjacent sub-grade.

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

6.6.10 This office should be provided a copy of the final construction plans so that the excavation recommendations presented herein could be properly reviewed and revised if necessary.

6.6.11 Consideration should be given to using interior stiffening beams and connecting isolated footings as well as patio slabs which exceed 5 feet in width to the building foundation to reduce the potential for future separation to occur.

6.6.12 The estimated maximum total and differential settlement for the planned structures due to foundation loading is ½ inch and ¼ inch, respectively.

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

- For fill slopes less than 20 feet high or cut slopes regardless of height, 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.

- When located next to a descending fill slope, the foundations should be extended to a depth where the minimum horizontal distance is equal to H/3 (where H equals the vertical distance from the top of the fill slope to the base of the fill soil) with a minimum of 7 feet but need not exceed 40 feet. The horizontal distance is measured from the outer, deepest edge of the footing to the face of the slope. An acceptable alternative to deepening the footings would be the use of a post-tensioned slab and foundation system or increased footing and slab reinforcement. Specific design parameters or recommendations for either of these alternatives can be provided once the building location and fill slope geometry have been determined.

- If swimming pools are planned, Geocon should be contacted for a review of specific site conditions.

- Swimming pools located within 7 feet of the top of cut or fill slopes are not recommended. Where such a condition cannot be avoided, the portion of the swimming pool wall within 7 feet of the slope face should be designed assuming that the adjacent soil provides no lateral support. This recommendation applies to fill slopes up to 30 feet in height, and cut slopes regardless of height. For swimming pools located near the top of fill slopes greater than 30 feet in height, additional recommendations may be required and Geocon should be contacted for a review of specific site conditions.

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

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

6.7 Light Standard and Signage Foundations

6.7.1 It is anticipated that light standards and signage will be installed within the complex.

6.7.2 Typical light standards are between 15 and 20 feet in height and are supported on pile foundations. Cast-in-place friction piles may be utilized for support of the proposed light standards and should be a minimum of 24 inches in diameter. Design of the anticipated signage was not available at the time of this report. If signage loading is similar to that of the assumed light standards presented in this section, foundation recommendations presented here are applicable. Foundation for other signage designs may be provided upon request once design and anticipated loads are known.

6.7.3 The friction piles do not require the complete removal of all loose earth materials from the bottom of the excavation, since end-bearing capacity is not being considered; however, a cleanout of the excavation bottom will be required and must be observed and approved by the Geotechnical Engineer (a representative of Geocon West, Inc.).

6.7.4 Piles should be embedded a minimum of six feet below the ground surface and may be embedded into newly placed engineered fill or competent underlying alluvium. All drilled pile excavations should be continuously observed by personnel of this firm to verify adequate penetration into the recommended bearing materials. Piles may be assumed fixed at an embedment depth of five feet below the ground surface. The coefficient of friction may be taken as 0.40 based on uniform contact between the concrete and newly placed engineered fill, and as 0.30 based on uniform contact between the concrete and competent alluvial soil at or below a depth of three feet. The downward capacity may be determined using a frictional resistance of 200 pounds per square foot where piles are in contact with the recommended bearing materials. An allowable bearing capacity of 3,000 psf may be used to calculate the end bearing capacity of the pile. A one-third increase in the capacity may be used for wind or seismic loads.

6.7.5 Passive earth pressure for cast in place piles poured against properly compacted engineered fill may be computed as an equivalent fluid having a density of 275 pcf. Passive earth pressure for cast in place piles poured against undisturbed alluvium may be computed as an equivalent fluid having a density of 200 pcf. The maximum allowable earth pressure is 3,000 pcf. An effective width of 3 times the pile diameter may be utilized in design to account for passive soil arching; provided piles are spaced a minimum of 4 times the pile diameter.

6.7.6 All drilled pile excavations should be continuously observed by personnel of this firm to verify adequate penetration into the recommended bearing materials. The capacity presented is based on the strength of the soil. The compressive and tensile strength of the pile sections should be checked to verify the structural capacity of the piles.

6.7.7 Casing may be required if caving is encountered. If casing is used, extreme care should be employed so that the pile is not pulled apart as the casing is withdrawn. At no time should the distance between the surface of the concrete and the bottom of the casing be less than five feet. Continuous observation of the drilling and pouring of the piles by the Geotechnical Engineer (a representative of Geocon), is required.

6.7.8 Closely spaced piles should be drilled and filled alternately, with the concrete permitted to set at least eight hours before drilling an adjacent hole. Pile excavations should be filled with concrete as soon after drilling and inspection as possible; the holes should not be left open overnight unless approved by the Geotechnical Engineer.

6.8 Helical Anchors

6.8.1 It is anticipated that helical anchors will be utilized for the entertainment center tents proposed at the site.

6.8.2 Helical anchors should be installed such that the depth of the shallowest helix is at least 5 diameters of the largest helix. Actual anchor depth should be determined in the field by a representative of Geocon based and the measured torque installation. If anchors with multiple helix blades are utilized, the helix blades should be at least three helix diameters apart. The minimum horizontal center to center spacing is 3 diameters of the largest helix.

6.8.3 Anchor loads are not known at the present time; however if anchors with a helix diameter of 1 foot are utilized, we estimate that an allowable tensile capacity of 2.2 tons per helix blade may be utilized for the design of the tent anchors. Once tent design has progressed to the point where anchor loads may be estimated, specific helical anchor recommendations may be provided.

6.8.4 Installation of the helical anchors should be observed by a representative of Geocon. Installation torque should be monitored during installation of the anchors to verify capacity of each anchor. The minimum installation torque will depend upon the design capacity and the installation torque-capacity ratio (K) but should not be less than 3,000 ft-lbs. The installation torque-capacity ratio (K) is dependent upon the anchor construction and should be provided by the anchor manufacturer.

6.9 Concrete Slabs-on-Grade

6.9.1 Building (interior) concrete slabs-on-grade should have a minimum thickness of 4 inches. Slabs-on-grade should be reinforced with at least No. 3 steel reinforcing bars spaced 24 inches on center in both horizontal directions and placed mid-height in the slab, or as deemed necessary by the structural engineer. The slabs should be underlain by at least 4 inches of fine gravel or coarse sand. The actual slab thickness and steel reinforcement should be determined by the structural engineer based on the expected floor loading and usage.

6.9.2 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.

6.9.3 The bedding sand thickness should be determined by the project foundation engineer, architect, or developer. However, we should be contacted to provide recommendations if the bedding sand is thicker than 6 inches. The foundation design engineer should provide appropriate concrete mix design criteria and curing measures to assure proper curing of the slab by reducing the potential for rapid moisture loss and subsequent cracking or slab curl. We suggest that the foundation design engineer present the concrete mix design and proper curing methods on the foundation plans. It is critical that the foundation contractor understands and follows the recommendations presented on the foundation plans.

6.9.4 Exterior slabs (not subject to traffic loads) should be at least 4 inches thick and reinforced with No. 3 reinforcing bars at 18 inches on center in both horizontal directions to reduce the potential for cracking. In addition, concrete flatwork should be provided with crack-control joints to reduce 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.

6.9.5 The recommendations of this report are intended to reduce the potential for cracking of slabs due to differential settlement of fill soil of varying thickness. 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. The occurrence of concrete shrinkage cracks is independent of the supporting soil characteristics. Their occurrence may be reduced 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.

6.10 Retaining Walls and Lateral Loads

6.10.1 Retaining walls that are allowed to rotate more than 0.001H at the top of the wall (where H equals the height of the retaining portion of the wall in feet) and having a level backfill surface should be designed for an active soil pressure equivalent to the pressure exerted by a fluid density of 43 pounds per cubic foot (pcf). Where the backfill will be inclined at no steeper than 2:1 (horizontal:vertical), an active soil pressure of 66 pcf is recommended. These soil pressures assume that the backfill 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 20. For those areas with finish grade soil having an Expansion Index greater than 20 or where backfill materials do not conform to the above criteria, Geocon should be consulted for additional recommendations.

6.10.2 Where walls are restrained from movement at the top, an additional uniform pressure of 7H psf (where H equals the height of the retaining portion of the wall in feet) should be added to the above active soil pressure.

6.10.3 The structural engineer should determine the seismic design category for the project. If the project possesses a seismic design category of D, E, or F, the proposed retaining walls should be designed with seismic lateral pressure. A seismic load of 18H should be used for design. The seismic load is dependent on the retained height where H is the height of the wall, in feet, and the calculated loads result in pounds per square foot (psf) exerted at the base of the wall and zero at the top of the wall. We used a peak site acceleration of 0.41g calculated from Section 1803.5.12 of the 2010 California Building Code ($S_{DS}/2.5$) and applied a pseudo-static coefficient of 0.5.

6.10.4 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 architect (see Figure 4). 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 20) 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.

6.10.5 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 consists of properly compacted fill and has an Expansion Index of less than 20. 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.

6.10.6 For resistance to lateral loads, an allowable passive earth pressure equivalent to a fluid density of 275 pcf is recommended for footings or shear keys poured neat against properly compacted granular fill soil. A factor of safety of 1.5 has been applied to these calculations. 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. If a vapor barrier is placed below the slab a friction coefficient of 0.15 may be used. This friction coefficient may be combined with the allowable passive earth pressure when determining resistance to lateral loads.

6.10.7 The recommendations presented above are generally applicable to the design of rigid concrete or masonry retaining walls having a maximum height of 10 feet. In the event that walls higher than 10 feet or other types of walls are planned, such as crib-type walls, Geocon should be consulted for additional recommendations.

6.11 Tunnels

6.11.1 We understand that the proposed site development may include tunnels for access to the stage area. The locations of the tunnels are not known at the present time. Specific recommendations should be provided in a geotechnical investigation report when the locations and anticipated construction methods of the tunnels are known. The following recommendations are preliminary and may be used for preliminary design and cost analysis of the project only.

6.11.2 For preliminary design purposes, the tunnels may be designed for a soil bearing pressure of 2,000 psf, for foundations deriving support in engineered fill. At a minimum, continuous footings should be at least 12 inches wide and extend 18 inches below the lowest adjacent pad grade. Isolated spread footings should have a minimum width of 2 feet and should extend at least 18 inches below the lowest adjacent pad grade. Additional foundation recommendations are provided in Section 6.6.

6.11.3 The tunnel sidewalls should be designed for an active soil pressure equivalent to the pressure exerted by a fluid density of 43 pcf plus an additional uniform pressure of 7H psf where H equals the depth to the bottom of the tunnel from the ground surface. These recommendations are generally applicable to tunnel side walls restrained from movement. Additionally, any structural or traffic loads within a 1:1 cone of influence extending up from the bottom of the tunnel should be accounted for in the design lateral pressures. Specific recommendations should be provided when the locations of the tunnels and surrounding improvements are known.

6.12 Preliminary Pavement Recommendations

6.12.1 As a minimum the upper 12 inches of soil should be moisture conditioned to near or slightly above optimum moisture content and property compacted to at least 95 percent relative compaction, as determined by ASTM D 1557 (latest edition).

6.12.2 The following pavement sections are based on an R-Value test result of 83 Once site grading activities are complete an R-Value test should be performed on a soil sample from the street subgrade area to confirm soil properties prior to placing pavement. Pavement thicknesses were determined following procedures outlined in the *California Highway Design Manual* (Caltrans).

6.12.3 The traffic index to be used for pavement section design should be provided by the project civil engineer.

PRELIMINARY PAVEMENT DESIGN SECTIONS

Estimated Traffic Index (TI)	Asphalt Concrete (inches)	Class 2 Aggregate Base (inches)
5	3.0	3.0
6	3.5	3.0
7	4.0	3.0

6.12.4 Asphalt concrete should conform to Section 203-6 of the "*Standard Specifications for Public Works Construction*" (Green Book). Class 2 aggregate base materials should conform to Section 26-1.02A of the "*Standard Specifications of the State of California, Department of Transportation*" (Caltrans). Crushed Miscellaneous Base should conform to Section 200-2.4 of the "*Standard Specifications for Public Works Construction*" (Green Book).

6.12.5 Unless specifically designed by a qualified structural engineer, where concrete paving will be utilized for support of vehicles, it is recommended that the concrete be a minimum of 5 inches thick and reinforced with No. 3 steel reinforcing bars placed 24 inches on center in both horizontal directions. Concrete paving supporting vehicular traffic should be underlain by a minimum of 4 inches of aggregate base and a properly compacted subgrade. The subgrade should be compacted to at least 95 percent relative. The base material should be compacted to at least 95 percent relative compaction, respectively, as determined by ASTM D 1557 (latest edition).

6.12.6 The performance of pavements is highly dependent upon providing positive surface drainage away from the edge of pavements. Ponding of water on or adjacent to the pavement will likely result in saturation of the subgrade materials and subsequent cracking, subsidence and pavement distress. If planters are planned adjacent to paving, it is recommended that the perimeter curb be extended at least 12 inches below the bottom of the aggregate base to minimize the introduction of water beneath the paving.

6.13 Slope Maintenance

6.13.1 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 or loosening of the surficial soil, 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 or 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. Irrigation lines should be placed on the slope face, not along the slope in trenches. 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 portions of the projects slopes in the future.

6.14 Site Drainage and Moisture Protection

6.14.1 Adequate site drainage is critical to reduce the potential for differential soil movement, erosion and subsurface seepage. Under no circumstances should water be allowed to pond adjacent to footings. The site should be graded and maintained such that surface drainage is directed away from structures in accordance with 2010 CBC 1804.3 or other applicable standards. In addition, surface drainage should be directed away from the top of slopes into swales or other controlled drainage devices. Roof and pavement drainage should be directed into conduits that carry runoff away from the proposed structure.

6.15 Plan Review

6.15.1 The soil engineer and engineering geologist should review the grading plans prior to finalization to verify their compliance with the recommendations of this report and determine the necessity for additional analyses or recommendations. The geotechnical engineer should also be provided the opportunity to review the structural foundation plans prior to finalizing to verify substantial conformance with the recommendations of this report.

LIMITATIONS AND UNIFORMITY OF CONDITIONS

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

2. The recommendations of this report pertain only to the site investigated and are based upon the assumption that the soil conditions do not deviate from those disclosed in the investigation. If any variations or undesirable conditions are encountered during construction, or if the proposed construction will differ from that anticipated herein, Geocon West, Inc. 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 West, Inc.

3. 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.

4. The findings of this report are valid as of the present date. However, changes in the conditions of a property can occur with the passage of time, whether they 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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Riverside County Flood Control and Water Conservation District Aerial Photographs.

Aerial Photograph	Date	Scale
5-49/5-50/5-51	4/13/05	1" = 1600 ft
5-50/5-51/5-52	3/11/00	1" = 1600 ft
5-50/5-51	1/28/95	1" = 1600 ft
5-52/5-53/5-54	2/22/90	1" = 1600 ft
1550/1551	2/7/84	1" = 2000 ft
185/186	5/24/74	1" = 2000 ft

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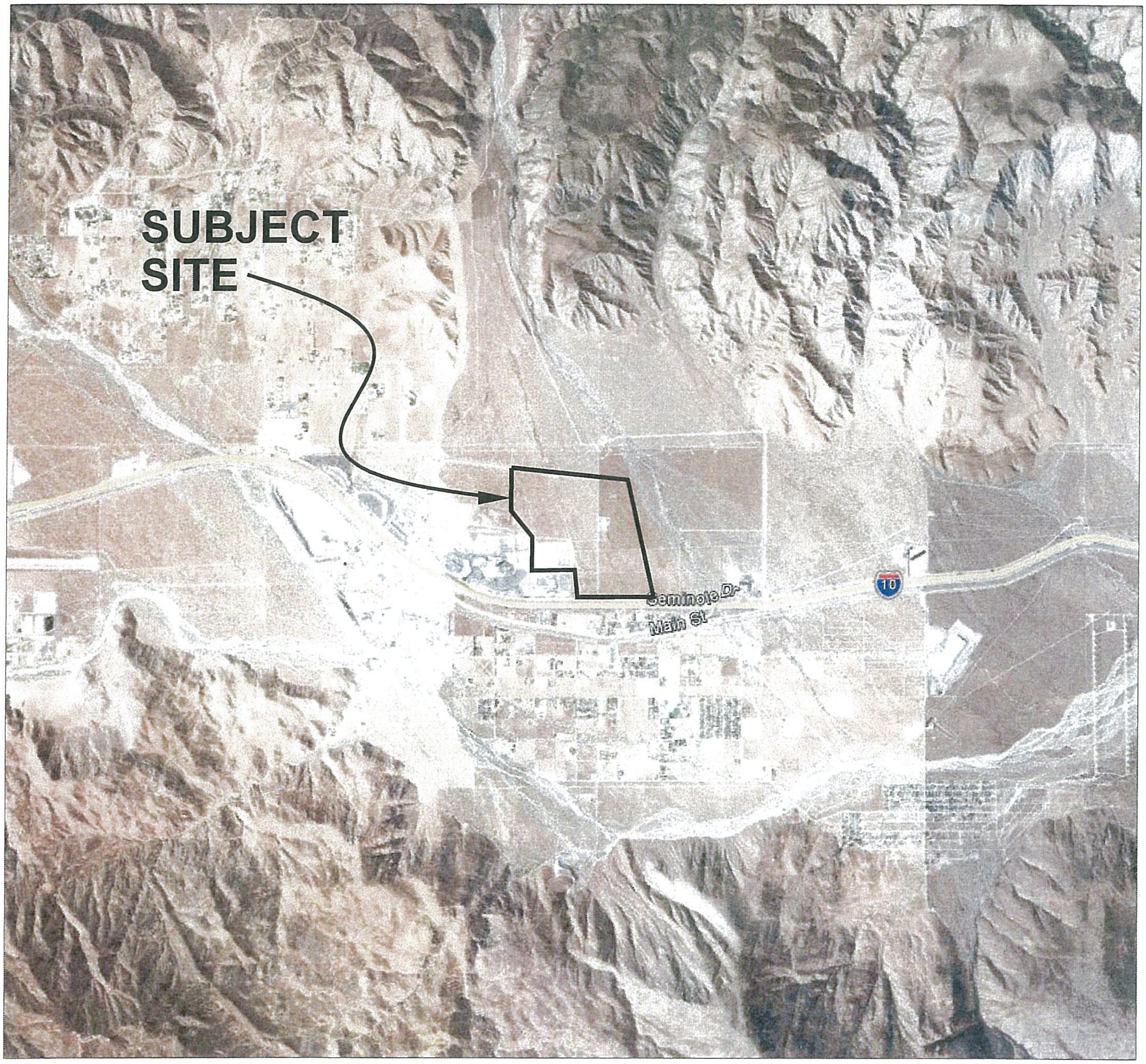

REFERENCE: GOOGLE EARTH, 2012

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JL		2000

VICINITY MAP

PRELIMINARY GEOTECHNICAL INVESTIGATION
MORONGO ENTERTAINMENT CENTER
RIVERSIDE COUNTY, CALIFORNIA

August 2012	PROJECT NO. T2533-22-01	FIG. 1

0 600' 1200'

LEGEND

T11	Approximate Location of Test Pit
P4	Approximate Location of Percolation Test

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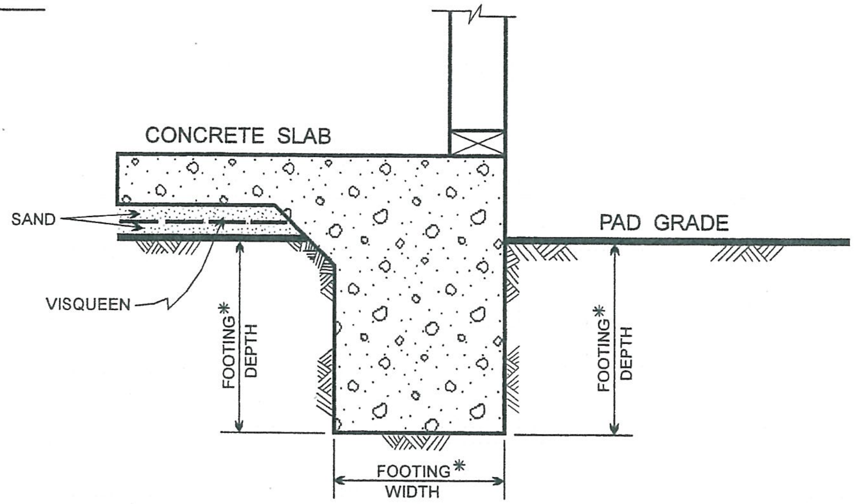
JL		2000

SITE PLAN

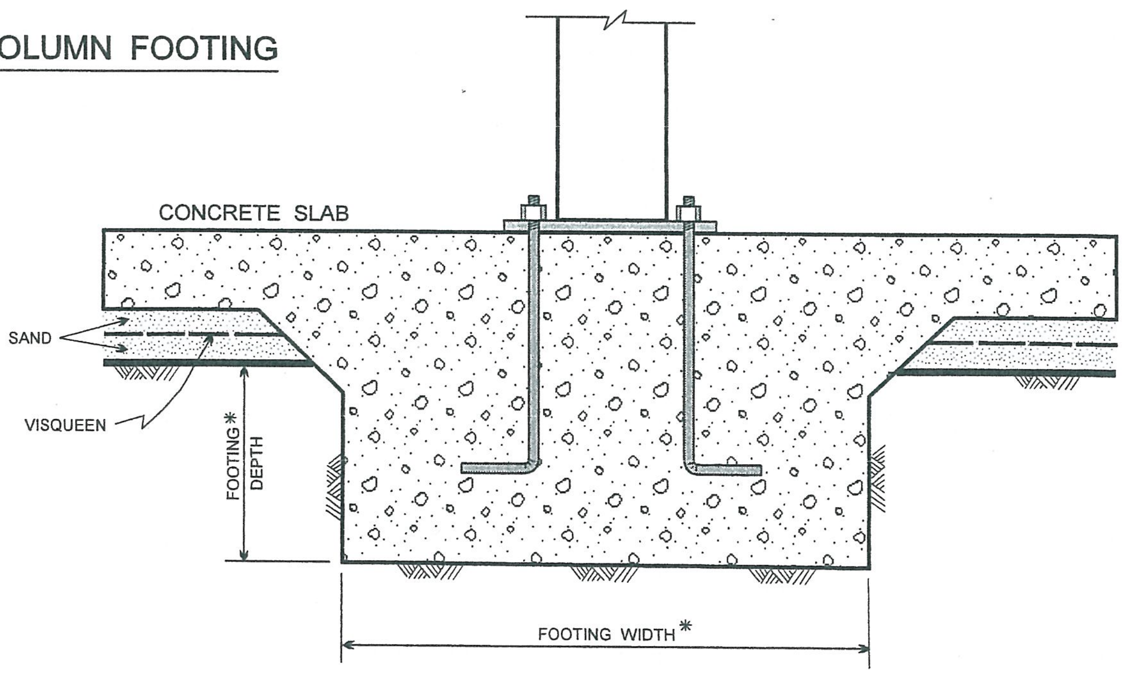
PRELIMINARY GEOTECHNICAL INVESTIGATION
MORONGO ENTERTAINMENT CENTER
RIVERSIDE COUNTY, CALIFORNIA

8 / 2012	PROJECT NO. T2533-22-01	FIG. 2

WALL FOOTING



COLUMN FOOTING



*......SEE REPORT FOR FOUNDATION WIDTH AND DEPTH RECOMMENDATION

NO SCALE

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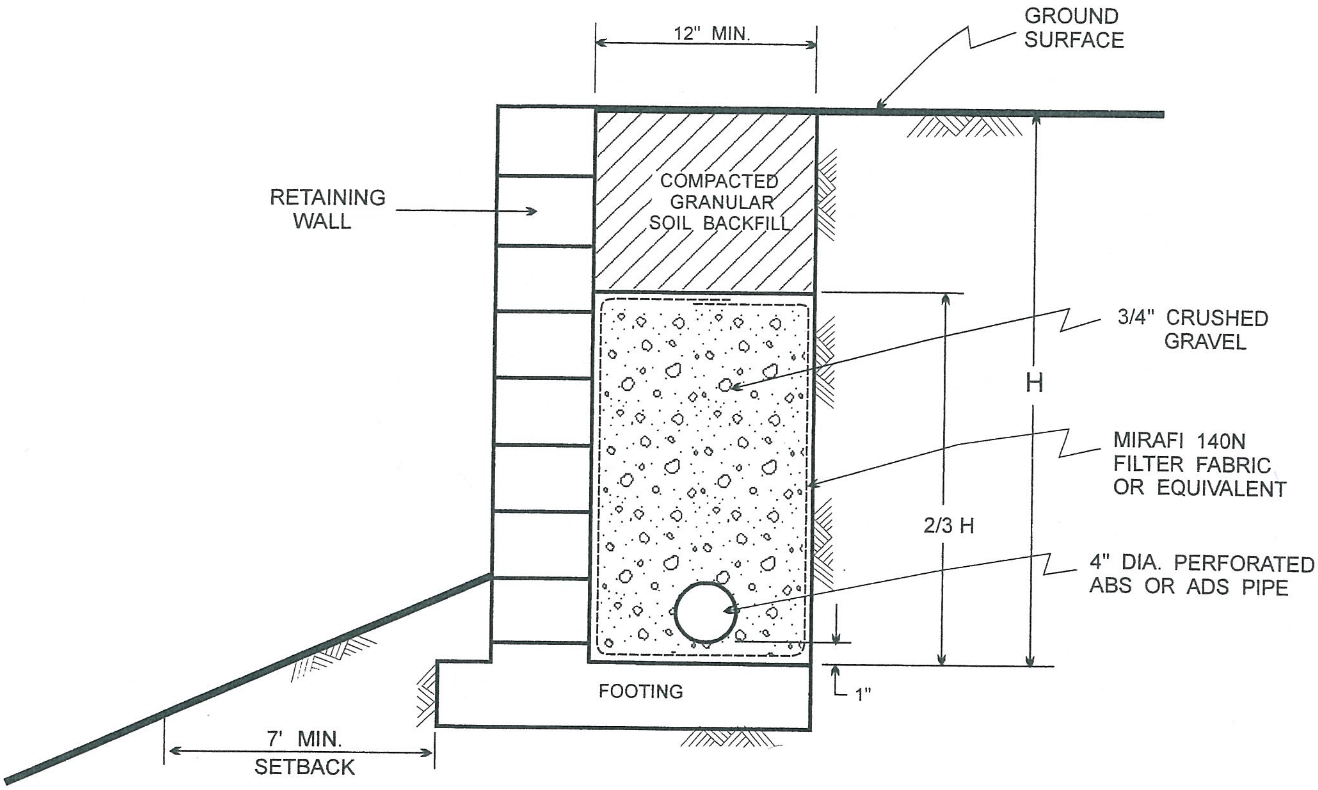
KC / KC

WALL/COLUMN FOOTING DIMENSION DETAIL

PROPOSED MORONGO ENTERTAINMENT CENTER
PORTIONS OF SECTIONS 8 AND 9, T3S R2E
CABAZON AREA, RIVERSIDE COUNTY, CALIFORNIA

DATE 08/2012	PROJECT NO. T2533-22-01	FIG. 3



NOTES :

1......PREFABRICATED DRAINAGE PANELS, SUCH AS MIRADRAIN 7000 OR EQUIVALENT, MAYBE USED IN LIEU OF PLACING GRAVEL TO HEIGHT OF 2/3 THE TOTAL WALL HEIGHT

2......DRAIN SHPOULD BE UNIFORMLY SLOPED AND MUST LEAD TO A POSITIVE GRAVITY OUTLET

3......TEMPORARY EXCAVATION SLOPES SHOULD BE CONSTRUCTED AND/OR SHORED IN ACCORDANCE WITH CAL-OSHA REGULATIONS

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KC / KC

WALL DRAINAGE DETAIL

PROPOSED MORONGO ENTERTAINMENT CENTER
PORTIONS OF SECTIONS 8 AND 9, T3S R2E
CABAZON AREA, RIVERSIDE COUNTY, CALIFORNIA

DATE 08/2011	PROJECT NO. T2533-22-01	FIG. 4

APPENDIX A

FIELD INVESTIGATION

The field investigation was performed on August 2, 2012, and consisted of a site reconnaissance and excavation of 15 geotechnical test pits utilizing a 4X4 rubber-tire backhoe. The test pits were excavated to a maximum depth of approximately 15 feet below existing grade. 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 D 2488). We obtained bulk soil samples during our subsurface exploration and transported them to the laboratory for testing. Four of the test pits were utilized to conduct percolation testing in accordance with Riverside County Department of Environmental Health requirements. The test pits were loosely backfill upon completion. Logs of the excavations are presented on Figures A-1 through A-15. The logs depict the soil and geologic conditions encountered and the depth at which samples were obtained. The percolation Test data is presented in Appendix C. The approximate locations of the exploratory excavations are shown on the *Site Plan*, Figure 2.

PROJECT NO. T2533-22-01

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	**TRENCH P-1** ELEV. (MSL.) **1893** DATE COMPLETED **8-2-12** EQUIPMENT **Backhoe w/24" bucket** BY: **PDT**	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
					MATERIAL DESCRIPTION			
0 – 5	P-1@4.5			SM/G	**ALLUVAIL FAN DEPOSITS (Qf)** Silty SAND with gravel and cobble, well graded, loose, dry, grayish brown, fine to coarse sand; trace boulders; rootlets; micaceous; surface shrubs; some bioturbation (krotovina) in upper 18" -Becomes light brownish gray			
					Total Depth 5 feet No water Caving from 1-5 feet Loosely backfilled with cuttings 8-2-12			

Figure A12,
Log of Trench P-1, Page 1 of 1

T2533-22-01 BORING LOGS.GPJ

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

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

GEOCON

PROJECT NO. T2533-22-01

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	TRENCH P-2 ELEV. (MSL.) **1876** DATE COMPLETED **8-2-12** EQUIPMENT **Backhoe w/24" bucket** BY: **PDT**	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
					MATERIAL DESCRIPTION			
0				SW/G	**ALLUVIAL FAN DEPOSITS (Qf)** Gravelly SAND, well graded, loose, dry, light brownish gray, fine to coarse sand; some cobbles (up to 8"); little silt; surface shrubs; roots to 2 feet; some bioturbation (krotovina) in upper 18"			
2								
4				GW/S	Sandy GRAVEL, well graded, dry, medium dense, light brown, fine to coarse sand, some silt			
	P-2@4.5			SP/M/G	SAND with silt and gravel, poorly graded, loose, dry, light brownish gray, fine to coarse sand; some cobbles (up to 8")			

Total Depth 5 feet
No water
Caving from 1.5 to 4.5 feet
Loosely backfilled with cuttings 8-2-12

Figure A13,
Log of Trench P-2, Page 1 of 1

T2533-22-01 BORING 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. T2533-22-01

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	TRENCH P-3 ELEV. (MSL.) 1860 DATE COMPLETED 8-2-12 EQUIPMENT Backhoe w/24" bucket BY: PDT	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
					MATERIAL DESCRIPTION			
0 – 2				SW/G	**ALLUVIAL FAN DEPOSITS (Qf)** Gravelly SAND, well graded, loose, dry, light brownish gray, fine to coarse sand; some cobbles (up to 8"); few silt; surface shrubs; roots to 2 feet; some bioturbation (krotovina) in upper 18"			
4	P-3@4.5			GW/S	Sandy GRAVEL, well graded, dry, brownish gray, fine to coarse sand; trace cobbles			
					Total Depth 5 feet No water Caving from 1-3 feet Loosely backfilled with cuttings 8-2-12			

Figure A14,
Log of Trench P-3, Page 1 of 1

T2533-22-01 BORING LOGS.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. T2533-22-01

TRENCH P-4

ELEV. (MSL.) **1977** DATE COMPLETED **8-2-12**

EQUIPMENT **Backhoe w/24" bucket** BY: **PDT**

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				SW/G	**ALLUVIAL FAN DEPOSITS (Qf)** Gravelly SAND, well graded, loose, dry, light grayish brown, fine to coarse sand; some cobbles (up to 8"); few silt; trace boulders (up to 14"); surface shrubs; roots to 2 feet; some bioturbation (krotovina) in upper 18"			
4	P-4@4.5			SP/M/G	SAND with silt and gravel, poorly graded, medium dense, dry, light gray, fine to coarse sand; trace cobble (up 8")			

Total Depth 5 feet
No water
Caving from 1-3 feet
Loosely backfilled with cuttings 8-2-12

Figure A15,
Log of Trench P-4, Page 1 of 1

T2533-22-01 BORING LOGS.GPJ

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

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GEOCON

PROJECT NO. T2533-22-01

TRENCH T-1

ELEV. (MSL.) **2007** DATE COMPLETED **8-2-12**

EQUIPMENT **Backhoe w/24" bucket** BY: **PDT**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0				SW/M/G	**ALLUVIAL FAN DEPOSITS (Qf)** Gravelly SAND with silt, well graded, loose, dry, light brown, fine to coarse sand; some cobbles (up to 8"); surface shrubs; roots to 2 feet; some bioturbation (krotovina) in upper 18"			
2								
4				GP/S	GRAVEL with sand, poorly graded, medium dense, dry, gray, fine to coarse sand; some cobbles			
6					-Increase in cobbles (up to 10"); few boulders (up to 24") -Increase in fine to coarse sand			
8				SW/G	Gravelly SAND with cobble, well graded, medium dense, dry, light brownish gray, fine to coarse sand; trace silt, trace boulders (up to 14")			
10				GP	GRAVEL with cobble, poorly graded; medium dense, dry, light gray, some fine to coarse sand			
12								
14	T1@13							

Total Depth 15 feet
No water/caving from 1-14 feet
Loosely backfilled with cuttings 8-2-12

Figure A1,
Log of Trench T-1, Page 1 of 1

T2533-22-01 BORING LOGS.GPJ

SAMPLE SYMBOLS

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

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GEOCON

PROJECT NO. T2533-22-01

TRENCH T-2

ELEV. (MSL.) **2019** DATE COMPLETED **8-2-12**

EQUIPMENT **Backhoe w/24" bucket** BY: **PDT**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0				SW/G	**ALLUVIAL FAN DEPOSITS (Qf)** Gravelly SAND, well graded, loose, dry, light grayish brown, fine to coarse sand; some cobbles (up to 8"); trace silt; surface shrubs; roots to 2 feet; some bioturbation (krotovina) in upper 18"			
2				GP/S	GRAVEL with sand, poorly graded, loose, dry, gray, fine to coarse sand; some cobbles (up to 10")			
4				SW/G	Gravelly SAND, well graded, loose, dry, light grayish brown, fine to coarse sand			
6								
8	T2@7.5							
10					Total Depth 10 feet No water Caving from 1-8 feet Loosely backfilled with cuttings 8-2-12			

Figure A2,
Log of Trench T-2, Page 1 of 1

T2533-22-01 BORING LOGS.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. T2533-22-01

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	**TRENCH T-3** ELEV. (MSL.) **2056** DATE COMPLETED **8-2-12** EQUIPMENT **Backhoe w/24" bucket** BY: **PDT**	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
					MATERIAL DESCRIPTION			
0 – 4				SW/G	**ALLUVIAL FAN DEPOSITS (Qf)** Gravelly SAND, well graded, loose, dry, light brownish gray, fine to coarse sand; trace cobbles (up to 8"); surface shrubs; roots to 2 feet; some bioturbation (krotovina) in upper 18" -Increase in sand			
6	T3@5			GW/S	Sandy GRAVEL with cobbles, well graded, loose, dry, light brownish gray, fine to coarse sand			
8 – 10				GP/S	Cobbley GRAVEL with boulders and sand, poorly graded, loose, dry, light gray, fine to coarse sand			
					Total Depth 10 feet No water Caving from 0.5-10 feet Loosely backfilled with cuttings 8-2-12			

Figure A3,
Log of Trench T-3, Page 1 of 1

T2533-22-01 BORING LOGS.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. T2533-22-01

TRENCH T-4

ELEV. (MSL.) **2049** DATE COMPLETED **8-2-12**

EQUIPMENT **Backhoe w/24" bucket** BY: **PDT**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0				SW/G	**ALLUVIAL FAN DEPOSITS (Qf)** Gravelly SAND, well graded, loose, dry, light grayish brown, fine to coarse sand; some cobbles (up to 8"); trace silt; surface shrubs; roots to 2 feet; some bioturbation (krotovina) in upper 18" @2' increase in cobbles			
2								
	T4@3							
4								
6				GW/S	Sandy GRAVEL, well graded, loose, dry, light grayish brown, fine to coarse sand			
8				GP/S	Cobbley GRAVEL with sand, poorly graded, loose, dry, light yellowish brown, trace boulders (up to 24")			
				GW/S	Sandy GRAVEL, well graded, loose, dry, light yellowish brown, fine to coarse sand			
10				GP/S	Cobbley GRAVEL with sand, poorly graded, loose, dry, light yellowish brown, trace boulders (up to 24")			
12								
14								

Total Depth 15 feet
No water
Caving from 0-6 feet
Loosely backfilled with cuttings 8-2-12

Figure A4,
Log of Trench T-4, Page 1 of 1

T2533-22-01 BORING LOGS.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. T2533-22-01

TRENCH T-5

ELEV. (MSL.) **2082** DATE COMPLETED **8-2-12**

EQUIPMENT **Backhoe w/24" bucket** BY: **PDT**

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/G	**ALLUVIAL FAN DEPOSITS (Qf)** Silty SAND with gravel, well graded, loose, dry, light brown, fine to coarse sand; surface shrubs; roots to 2 feet; some bioturbation (krotovina) in upper 18"			
2	T5@1				-some cobbles			
4				GW/S	Sandy GRAVEL with cobbles, well graded, loose, gray, fine to coarse sand			
6				SW/G	Gravelly SAND with cobbles, well graded, loose, gray, fine to coarse sand -cobble layer, 1 foot thick			
8				GW/S	Sandy GRAVEL with cobbles, well graded, loose, gray, fine to coarse sand			

Total Depth 8.5 feet
No water
Caving from 1-6 feet
Loosely backfilled with cuttings 8-2-12

Figure A5,
Log of Trench T-5, Page 1 of 1

T2533-22-01 BORING LOGS.GPJ

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

PROJECT NO. T2533-22-01

TRENCH T-6

ELEV. (MSL.) **2099** DATE COMPLETED **8-2-12**

EQUIPMENT **Backhoe w/24" bucket** BY: **PDT**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0				SW/G	**ALLUVIAL FAN DEPOSITS (Qf)** Gravelly SAND with cobble, well graded, loose, dry, light brown, fine to coarse sand; some silt; surface shrubs; roots to 3 feet; some bioturbation (krotovina) in upper 18"			
2				GP/S	Sandy GRAVEL with cobbles, poorly graded, loose, dry, light grayish brown, fine to coarse sand; trace boulders (up to 18")			
4				GW/S	Sandy GRAVEL with cobbles, well graded, loose, dry, light gray, fine to coarse sand			
6	T6@4				-decrease in cobbles			
8					Total Depth 8 feet No water Caving from 2-5 feet Loosely backfilled with cuttings 8-2-12			

Figure A6,
Log of Trench T-6, Page 1 of 1

T2533-22-01 BORING LOGS.GPJ

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

PROJECT NO. T2533-22-01

TRENCH T-7

ELEV. (MSL.) **2079** DATE COMPLETED **8-2-12**

EQUIPMENT **Backhoe w/24" bucket** BY: **PDT**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0				SW/G	**ALLUVIAL FAN DEPOSITS (Qf)** Gravelly SAND, well graded, loose, dry, light grayish brown, fine to coarse sand; surface shrubs; roots to 3 feet; some bioturbation (krotovina) in upper 18"			
2								
4				GW/S	Sandy GRAVEL, well graded, loose, dry, light yellowish brown, fine to coarse sand, some cobbles			
6	T7@6							
8					-decrease in sand, increase in gravel			
					-increase in cobbles, some boulders Refusal on boulders			

Total Depth 9.5 feet (refusal)
No water
Caving from 3-6 feet
Loosely backfilled with cuttings 8-2-12

Figure A7,
Log of Trench T-7, Page 1 of 1

T2533-22-01 BORING LOGS.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. T2533-22-01

TRENCH T-8

ELEV. (MSL.) **2007** DATE COMPLETED **8-2-12**

EQUIPMENT **Backhoe w/24" bucket** BY: **PDT**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0				SW/G	**ALLUVIAL FAN DEPOSITS (Qf)** Gravelly SAND, well graded, loose, dry, light grayish brown, fine to coarse sand; some silt; some cobbles; surface shrubs; roots to 3 feet; some bioturbation (krotovina) in upper 18"			
2				GW/S	Sandy GRAVEL with cobbles, well graded, loose, dry, light brownish gray, fine to coarse sand -some boulders up to 24"			
4								
6				GP/S	Sandy GRAVEL with cobbles, poorly graded, loose, dry, light gray, fine to coarse sand; no boulders -boulders up to 24 "			
8								
10	T8@10							

Total Depth 11 feet
No water
Caving from 1-7 feet
Loosely backfilled with cuttings 8-2-12

Figure A8,
Log of Trench T-8, Page 1 of 1

T2533-22-01 BORING LOGS.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. T2533-22-01

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	TRENCH T-9 ELEV. (MSL.) 2010 DATE COMPLETED 8-2-12 EQUIPMENT Backhoe w/24" bucket BY: PDT	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
					MATERIAL DESCRIPTION			
0				SW	**ALLUVIAL FAN DEPOSITS (Qf)** Gravelly SAND, well graded, loose, dry, light grayish brown, fine to coarse sand; some silt; some cobbles; surface shrubs; roots to 3 feet; some bioturbation (krotovina) in upper 18"			
2				GW/S	Sandy GRAVEL with cobbles, well graded, loose, dry, light brown, fine to coarse sand; trace boulders (up to 18")			
4	T9@3			GP/S	Sandy GRAVEL with cobbles, poorly graded, loose, dry, light gray, fine to coarse			
					Total Depth 5 feet No water Caving from 1-4 feet Loosely backfilled with cuttings 8-2-12			

Figure A9,
Log of Trench T-9, Page 1 of 1

T2533-22-01 BORING LOGS.GPJ

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

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GEOCON

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	TRENCH T-10 ELEV. (MSL.) 2029 DATE COMPLETED 8-2-12 EQUIPMENT Backhoe w/24" bucket BY: PDT	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
					MATERIAL DESCRIPTION			
0 2 4				SW/G	**ALLUVIAL FAN DEPOSITS (Qf)** Gravelly SAND, well graded, loose, dry, light brown, fine to coarse sand; some silt; some cobbles; surface shrubs; roots to 3 feet; some bioturbation (krotovina) in upper 18" -some boulders (up to 24")			
					Total Depth 5 feet No water Caving from 0-4 feet Loosely backfilled with cuttings 8-2-12			

Figure A10,
Log of Trench T-10, Page 1 of 1

T2533-22-01 BORING LOGS.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. T2533-22-01

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	TRENCH T-11 ELEV. (MSL.) **2050** DATE COMPLETED **8-2-12** EQUIPMENT **Backhoe w/24" bucket** BY: **PDT**	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
					MATERIAL DESCRIPTION			
0				GW/S	**ALLUVIAL FAN DEPOSITS (Qf)**			
	T11@1				Sandy GRAVEL, well graded, loose, dry, light brown, fine to coarse sand; surface shrubs; roots to 3 feet; some bioturbation (krotovina) in upper 18"			
2					-increase in cobbles; trace boulder (up to 24")			
4					-no boulders, trace cobbles, light gray			
					Total Depth 5 feet No water Caving from 0-4 feet Loosely backfilled with cuttings 8-2-12			

Figure A11,
Log of Trench T-11, Page 1 of 1

T2533-22-01 BORING LOGS.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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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 samples were tested for maximum density/optimum moisture content, direct shear strength, expansion characteristics, corrosivity, grain size distribution, sand equivalent, resistance value (R-Value). The results of the laboratory tests are summarized in Tables B-I through B-VIII, and Figure B-1.

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

Sample No.	Description	Maximum Dry Density (pcf)	Optimum Moisture Content (% dry wt.)
T-4 @ 3	Gray brown Silty GRAVEL with sand (GM)	129.8	9.2
T-5 @ 1	Gray brown Silty fine to coarse SAND (SM)	129.4	7.5
T-9 @ 3	Light brown, poorly graded GRAVEL with silty sand (GP-GM)	134.4	6.7

TABLE B-II
SUMMARY OF LABORATORY R-VALUE TEST RESULTS
ASTM D 2844

Sample No.	R-Value
T-6 @ 4	83

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

Sample No.	Moisture Content		Dry Density (pcf)	Expansion Index
	Before Test (%)	After Test (%)		
T-3 @ 5	27.5	59.0	120.7	0
T-11 @ 1	30.0	57.0	118.3	1

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

Sample No.	Dry Density (pcf)	Moisture Content (%)	Peak [Ultimate] Cohesion (psf)	Peak [Ultimate] Shear Angle (degrees)
T-4 @ 3*	116.2	13.6	520 [470]	31 [30]
T-5 @ 1*	114.6	13.0	200 [0]	35 [37]
T-9 @ 3*	121.0	13.0	260 [140]	34 [34]

* Sample remolded to near 90% relative compaction.

TABLE B-V
SUMMARY OF LABORATORY POTENTIAL OF HYDROGEN (pH) AND RESISTIVITY TEST RESULTS
CALIFORNIA TEST 532

Sample No.	pH	Resistivity (ohm centimeters)
T-2 @ 7	8.1	3,000
T-7 @ 6	8.4	22,000

TABLE B-VI
SUMMARY OF LABORATORY WATER-SOLUBLE CHLORIDE TEST RESULTS
AASHTO T 291

Sample No.	Water-Soluble Chloride (ppm)
T-2 @ 7.5	0.004
T-7 @ 6	0.001

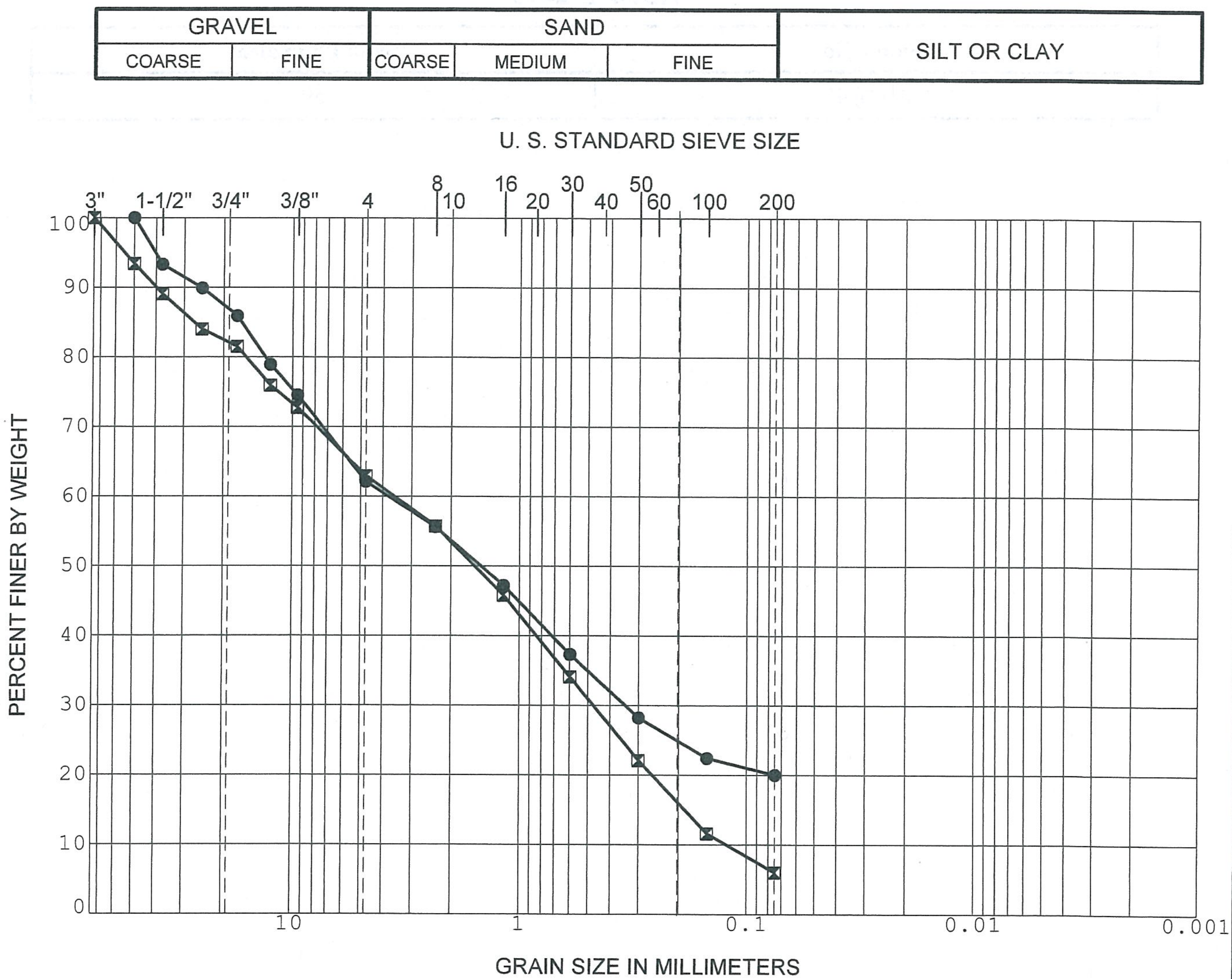
TABLE B-VII
SUMMARY OF LABORATORY WATER-SOLUBLE SULFATE TEST RESULTS
CALIFORNIA TEST 417

Sample No.	Sulfate Content (%)
T-2 @ 7.5	0.001
T-7 @ 6	0.0005

TABLE B-VIII
SUMMARY OF LABORATORY SAND EQUIVALENT TEST RESULTS
ASTM D 2419

Sample No.	Sand Equivalent
T-6 @ 4	59

PROJECT NO. T2533-22-01



	SAMPLE	DEPTH (ft)	CLASSIFICATION	NAT WC	LL	PL	PI
●	P1	5.0	(SM) Silty SAND with gravel				
⊠	P2	1.0	(SP-SM) Poorly graded SANDwith silt and gravel				
▲							

GRADATION CURVE

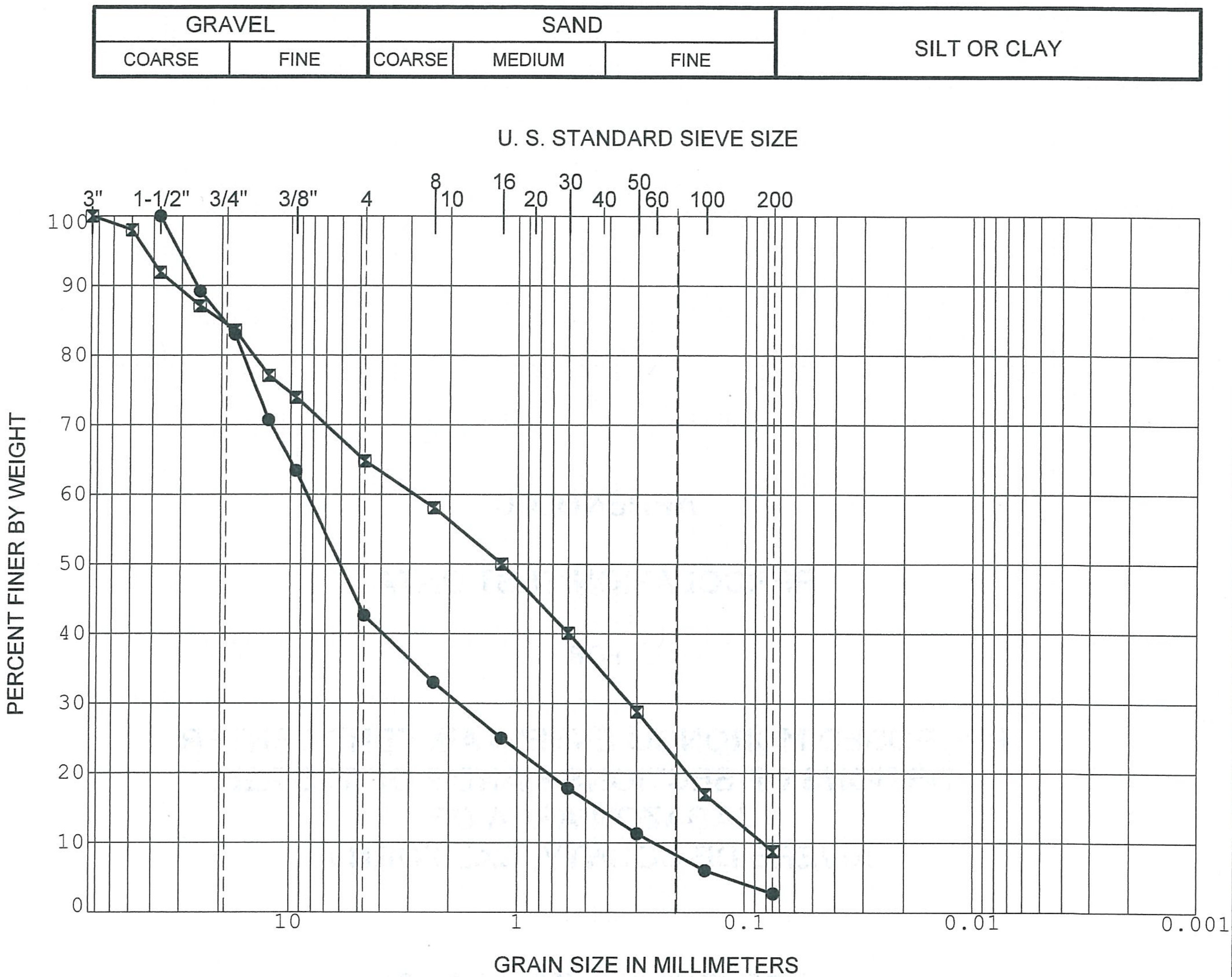
PROPOSED MORONGO ENTERTAINMENT CENTER
PORTIONS OF SECTION 8 AND 9 OF T3S, CABAZON AREA
RIVERSIDE COUNTY, CALIFORNIA

T2533-22-01.GPJ

Figure B-1

GEOCON

PROJECT NO. T2533-22-01



	SAMPLE	DEPTH (ft)	CLASSIFICATION	NAT WC	LL	PL	PI
●	P3	4.5	(GW) Well graded GRAVEL with sand				
⊠	P4	1.5	(SP-SM) Poorly graded SAND with silt and gravel				
▲							

GRADATION CURVE

PROPOSED MORONGO ENTERTAINMENT CENTER
PORTIONS OF SECTION 8 AND 9 OF T3S, CABAZON AREA
RIVERSIDE COUNTY, CALIFORNIA

T2533-22-01.GPJ

Figure B-2

GEOCON

APPENDIX C

PERCOLATION TEST DATA

FOR

PROPOSED MORONGO ENTERTAINMENT CENTER
PORTIONS OF SECTIONS 8 AND 9 OF T3S R2E,
CABAZON AREA OF
RIVERSIDE COUNTY, CALIFORNIA

PROJECT NO. T2533-22-01

LEACH LINE PERC DATA SHEET

Project: MSA CABAZON Job No: T2533-22-01

Test Hole No: P-1 Date Excavated: 8/2/2012

Depth of Test Hole: 5' Soil classification: SANDY

Check for Sandy Soil Criteria Tested By: JL Date: 8/2/12 Presoak: N/A

Actual Percolation Tested By: JL Date: 8/2/12

SANDY SOIL CRITERIA TEST

Trial No.	Time	Time Interval (Min)	Initial Water Level (inches)	Final Water Level (inches)	Δ in Water Level (inches)
1	8:27 8:28	1	3.70'	4.70'	12
2	8:32 8:35	8	3.70'	4.70'	12

Use Normal Sandy (Circle One) Soil Criteria

Time	Time Interval (min)	Total Elapsed Time (min.)	Initial Water Level (inches)	Final Water Level (inches)	Δ in Water Level (inches)	Percolation Rate (min/inch)
8:38 8:41	3	14	3.70'	4.70'	12	0.25
8:46 8:49	3	22	3.70'	4.70'	12	0.25
8:56 8:59	3	32	3.70'	4.70'	12	0.25
9:00 9:03	3	36	3.70'	4.70'	12	0.25
9:04 9:07	3	40	3.70'	4.70'	12	0.25
9:08 9:11	3	44	3.70'	4.70'	12	0.25
9:14 9:17	3	50	3.70'	4.70'	12	0.25

LEACH LINE PERC DATA SHEET

Project: MSA CABAZON Job No: 72533-22-01

Test Hole No: P-2 Date Excavated: 8/2/2012

Depth of Test Hole: 5' Soil classification: SANDY

Check for Sandy Soil Criteria Tested By: JL Date: 8/2/12 Presoak: N/A

Actual Percolation Tested By: JL Date: 8/2/12

SANDY SOIL CRITERIA TEST

Trial No.	Time	Time Interval (Min)	Initial Water Level (inches)	Final Water Level (inches)	Δ in Water Level (inches)
1	10:06 10:10	4	3.55'	4.70'	13.8
2	10:11 10:15	4	3.70'	4.70'	12

Use Normal Sandy (Circle One) Soil Criteria

Time	Time Interval (min)	Total Elapsed Time (min.)	Initial Water Level (inches)	Final Water Level (inches)	Δ in Water Level (inches)	Percolation Rate (min/inch)
10:16 10:21	5	15	3.70'	4.70'	12	0.42
10:22 10:27	5	21	3.70'	4.70'	12	0.42
10:29 10:34	5	28	3.70'	4.70'	12	0.42
10:35 10:41	6	35	3.70'	4.70'	12	0.50
10:42 10:47	5	41	3.70'	4.70'	12	0.42
10:48 10:53	5	47	3.70'	4.70'	12	0.42
10:55 11:00	5	54	3.70'	4.70'	12	0.42

LEACH LINE PERC DATA SHEET

Project: MSA CABAZON Job No: T2533-22-01

Test Hole No: P-3 Date Excavated: 8/2/2012

Depth of Test Hole: 5' Soil classification: SANDY

Check for Sandy Soil Criteria Tested By: VL Date: 8/2/12 Presoak: N/A

Actual Percolation Tested By: VL Date: 8/2/12

SANDY SOIL CRITERIA TEST

Trial No.	Time	Time Interval (Min)	Initial Water Level (inches)	Final Water Level (inches)	Δ in Water Level (inches)
1	11:44 11:51	7	3.60'	4.60'	12
2	11:53 12:03	10	3.60'	4.60'	12

Use Normal Sandy (Circle One) Soil Criteria

Time	Time Interval (min)	Total Elapsed Time (min.)	Initial Water Level (inches)	Final Water Level (inches)	Δ in Water Level (inches)	Percolation Rate (min/inch)
12:05 12:15	10	31	3.60'	4.45'	10.2	0.98
12:15 12:25	10	41	3.60'	4.39'	9.48	1.05
12:25 12:35	10	51	3.57'	4.20'	7.56	1.32
12:35 12:45	10	61	3.60'	4.18'	6.96	1.44
12:45 12:55	10	71	3.58'	4.19'	7.32	1.36
12:55 1:05	10	81	3.58'	4.20'	7.44	1.34

LEACH LINE PERC DATA SHEET

Project: MSA CABAZON Job No: T2533-22-01

Test Hole No: P-4 Date Excavated: 8/2/2012

Depth of Test Hole: 5' Soil classification: SANDY

Check for Sandy Soil Criteria Tested By: JL Date: 8/2/12 Presoak: N/A

Actual Percolation Tested By: JL Date: 8/2/12

SANDY SOIL CRITERIA TEST

Trial No.	Time	Time Interval (Min)	Initial Water Level (inches)	Final Water Level (inches)	Δ in Water Level (inches)
1	1:49 1:54	5	3.60'	4.60'	12
2	1:54 2:00	6	3.60'	4.60'	12

Use Normal Sandy (Circle One) Soil Criteria

Time	Time Interval (min)	Total Elapsed Time (min.)	Initial Water Level (inches)	Final Water Level (inches)	Δ in Water Level (inches)	Percolation Rate (min/inch)
2:00 2:06	6	17	3.60'	4.60'	12	0.5
2:06 2:13	7	24	3.60'	4.60'	12	0.58
2:13 2:20	7	31	3.57'	4.57'	12	0.58
2:20 2:28	8	39	3.55'	4.55'	12	0.66
2:28 2:36	8	47	3.60'	4.60'	12	0.66
2:36 2:45	9	56	3.60'	4.60'	12	0.75
2:45 2:54	9	65	3.60'	4.60'	12	0.75
2:54 3:03	9	74	3.59'	4.59'	12	0.75

APPENDIX D

RECOMMENDED GRADING SPECIFICATIONS

FOR

PROPOSED PALM SPRINGS AUTODROME
T4S R5E, SECTION 12,
THOUSAND PALMS AREA
RIVERSIDE COUNTY, CALIFORNIA

PROJECT NO. T2516-22-01

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 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

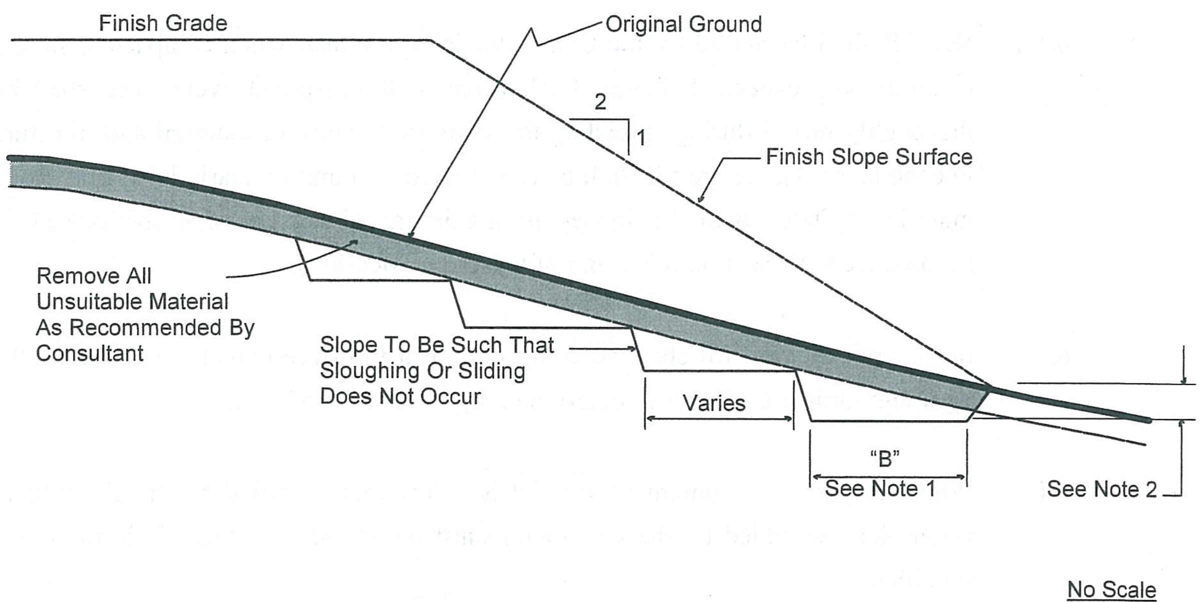
4.1 Areas to be excavated and filled shall be cleared and grubbed. Clearing shall consist of complete removal above the ground surface of trees, stumps, brush, vegetation, man-made structures, and similar debris. Grubbing shall consist of removal of stumps, roots, buried logs and other unsuitable material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