APPENDIX F
Preliminary Geotechnical Investigation

PRELIMINARY GEOTECHNICAL INVESTIGATION

COLLIER PARK EXPANSION
PASADENA AVENUE
AND PALM AVENUE
LA MESA, CALIFORNIA

GEOCON
INCORPORATED

GEOTECHNICAL
CONSULTANTS

PREPARED FOR

CITY OF LA MESA
LA MESA, CALIFORNIA

DECEMBER 10, 2010
PROJECT NO. G1222-52-03

GEOCON
INCORPORATED

GEOTECHNICAL · CONSULTANTS

Project No. G1222-52-03
December 10, 2010

City of La Mesa
Engineering Department
8130 Allison Avenue
La Mesa, California 91942

Attention: Mr. Michael Kinnard

Subject: COLLIER PARK EXPANSION
PASADENA AVENUE AND PALM AVENUE
LA MESA, CALIFORNIA
PRELIMINARY GEOTECHNICAL INVESTIGATION

Dear Mr. Kinnard:

In accordance with your authorization of our Proposal No. LG-10244 dated August 30, 2010, we herein submit the results of our preliminary geotechnical investigation for the subject site. We performed our investigation to observe the soil and geologic conditions that may affect the proposed park expansion on the property. We understand proposed development will consist of grading to accommodate a club house, parking lot/open space, a plaza, a cooking/dining area, and additional park improvements. The accompanying report presents the results of our study and provides preliminary recommendations to assist in planning and development studies. The findings of this study indicate that the site is suitable for development provided the recommendations of this report are followed.

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

Very truly yours,

GEOCON INCORPORATED

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TABLE OF CONTENTS

1. PURPOSE AND SCOPE 1

2. SITE AND PROJECT DESCRIPTION 1

3. SOIL AND GEOLOGIC CONDITIONS 2
   - 3.1 Topsoil (Unmapped) 2
   - 3.2 Undocumented Fill (Qudf) 2
   - 3.3 Metasedimentary and Metavolcanic Rock Undivided (Mzu) 3

4. GROUNDWATER 3

5. GEOLOGIC HAZARDS 3
   - 5.1 Faulting and Seismicity 3
   - 5.2 Ground Rupture 5
   - 5.3 Rippability of Rock – Air-Track Borings 5
   - 5.4 Rippability of Rock – Seismic Refraction Survey 6

6. FUTURE GEOTECHNICAL STUDIES 7

7. CONCLUSIONS AND RECOMMENDATIONS 8
   - 7.1 General 8
   - 7.2 Excavation and Soil Characteristics 8
   - 7.3 Seismic Design Criteria 10
   - 7.4 Preliminary Grading Recommendations 11
   - 7.5 Foundation and Concrete Slabs-On-Grade Recommendations 12
   - 7.6 Retaining Walls 17
   - 7.7 Lateral Loading 18
   - 7.8 Preliminary Pavement Recommendations 18
   - 7.9 Site Drainage and Moisture Protection 21
   - 7.10 Grading and Foundation Plan Review 22

LIMITATIONS AND UNIFORMITY OF CONDITIONS

MAPS AND ILLUSTRATIONS
- Figure 1, Vicinity Map
- Figure 2, Geologic Map
- Figure 3, Wall/Column Footing Dimension Detail
- Figure 4, Typical Retaining Wall Drain Detail

APPENDIX A
- FIELD INVESTIGATION
- Figures A-1 – A-6, Logs of Air-Track Borings AT-1 through AT-6

APPENDIX B
- SEISMIC REFRACTION SURVEY

APPENDIX C
- RECOMMENDED GRADING SPECIFICATIONS

LIST OF REFERENCES

PRELIMINARY GEOTECHNICAL INVESTIGATION

1. PURPOSE AND SCOPE

This report presents the results of a preliminary geotechnical investigation for proposed improvements to Collier Park located in La Mesa, California. The purpose of the preliminary investigation is to evaluate and provide preliminary soil and geologic information at the site and identify known geologic hazards that may adversely impact the proposed development. In addition, we performed a preliminary evaluation of rippability characteristics of the volcanic rock. A design-level geotechnical investigation should be performed when grading plans for the site have been developed and the locations of the proposed buildings are known.

The scope of this investigation included a review of stereoscopic aerial photographs and readily available published and unpublished geologic literature (see *List of References*). We conducted a field investigation which included geologic mapping and the excavation of 6 exploratory air track borings. We also retained Southwest Geophysics to perform a geophysical survey of the areas of proposed development. A discussion of the field investigation and logs of the exploratory borings are presented in Appendix A. The geophysical survey is presented in Appendix B.

Warner Architecture Design provided the topographic information used during our field investigation and for the preparation of the Geologic Map. References to elevations presented in this report are based on the referenced topographic information. Geocon does not practice in the field of land surveying and is not responsible for the accuracy of such topographic information.

2. SITE AND PROJECT DESCRIPTION

The subject site is located east of Palm Avenue, west of 4th Street, and north and south of Pasadena Avenue in La Mesa, California (see Vicinity Map, Figure 1). Pasadena Avenue is in the central portion of the park. A relatively steep slope and an unpaved parking area are located north of Pasadena Avenue. A gentle sloping trail area is located on the eastern portion south of Pasadena Avenue and the main park area is on the western portion south of Pasadena Avenue. The main park area consists of a tennis court, paved parking and driveway areas, grass and landscaping, and climbing structures. The topography of the property ranges from relatively flat to steep slopes. Elevations range from about 500 feet above mean sea level (MSL) to about 545 feet MSL from south to north.

Based on the referenced plans, we understand the area of the planned expansion will be located on the northern portion of the park (north of Pasadena Avenue). Improvements are also planned on the eastern portion of the park known as "History Hill." A café is planned along the central west margin of the site. Based on our review of conceptual architectural plans and drawings, we understand that

the site will be regraded to accommodate a club house, parking lot/open space, a plaza, a cooking/dining area, and additional park improvements. We expect a maximum cut of about 25 to 30 feet would be required for the planned grading operations and a fill embankment of about 10 feet thick. This would result in a retaining wall on the northern portion of the property with a maximum height of about 25 feet. We expect the proposed structures will be founded in compacted fill or rock material.

A Geologic Map, shown in Figure 2, depicts the existing geologic conditions and approximate locations of the exploratory air track borings and seismic lines. We located the borings in the field using a measuring tape and existing reference points; therefore, actual locations may deviate slightly.

The locations, site descriptions and proposed development are based on a site reconnaissance, review of published geologic literature, our field investigation, a review of proposed architectural drawing and discussions with you. Geocon Incorporated should be contacted prepare a final geotechnical investigation when the development plans are available.

3. SOIL AND GEOLOGIC CONDITIONS

During our field investigation, we encountered two surficial soil deposits and one geologic unit at the site. The surficial deposits consist of topsoil (unmapped) and undocumented fill (Qudf). The surficial unit is underlain by formational rock material at various stages of weathering. In addition, the rock is exposed at the surface on the property and on cut slopes. The estimated occurrence and distribution of the surficial and formational rock units are discussed herein and are depicted on the Geologic Map, Figure 2.

3.1 Topsoil (Unmapped)

A relatively thin layer of topsoil covers the rock throughout the site. This material consists of loose, dry to moist, reddish brown, silty sand with some gravel and organic debris. We did not map the topsoil on the Geologic Map (Figure 2) due to the relatively thin nature of the deposit. Topsoil is not considered suitable in its current condition and remedial grading would be required.

3.2 Undocumented Fill (Qudf)

We encountered undocumented fill in air-track borings AT-5 and AT-6 to the maximum depth explored of 19 feet. Based on the rapid rate of penetration by the air track boring and some of exposed cuttings on the drill bit, the undocumented fill consists loose to medium dense, moist, brown to reddish brown and olive brown, clayey, fine to medium sand with some gravel. However, we suspect that some oversize rock material is buried in the fill due to the variable drill rates. We expect the undocumented fill was placed during the original grading of the park site and the installation of

the storm drain which we understand to traverse the northeastern corner of the site. Since, we have not been able to review engineering reports pertaining to fill placement, the fill is considered undocumented. The undocumented fill may not be suitable for the support of the proposed improvements or structural fill in its present condition. Remedial grading in the form of partial removal may be required. The depth and extent of the removals should be evaluated during future geotechnical studies. The approximate limit of the undocumented fill is presented on the Geologic Map, Figure 2.

3.3 Metasedimentary and Metavolcanic Rock Undivided (Mzu)

We encountered Cretaceous/Jurassic-age (Mesozoic) metasedimentary and metavolcanic rock in air-track borings AT-1 through AT-4 within the northern and eastern margins of the site. Contact and dynamic metamorphism has created metamorphic textures such as schistose foliation and recrystallization to form quartzites. The rock material is generally moderately strong to strong, intensely to slightly weathered, and moderately to slightly jointed. Moderately to slightly weathered and slightly jointed rock will likely be very difficult to excavate or be nonrippable and large portions will likely require blasting to excavate. The lateral extent of the volcanic rock should be evaluated during future geotechnical studies. We expect excavations within this unit will likely result in the generation of oversized material. Rippability of this unit is described in detail in subsequent sections of this report.

4. GROUNDWATER

We did not encounter a static groundwater table or subsurface seepage in the exploratory excavations performed for this study. It is not uncommon for groundwater seepage conditions to develop where none previously existed due to the permeability characteristics of the geologic units encountered on site. During the rainy season, perched water conditions are likely to develop within the drainage areas that may require special consideration during grading operations. Groundwater elevations are dependent on seasonal precipitation, irrigation, and land use, among other factors, and vary as a result.

5. GEOLOGIC HAZARDS

5.1 Faulting and Seismicity

A review of the referenced geologic materials and our knowledge of the general area indicate that the site is not underlain by active, potentially active, or inactive faults. An active fault is defined by the California Geological Survey (CGS) as a fault showing evidence for activity within the last 11,000 years. The site is not located within State of California Earthquake Fault Zone.

According to the computer program *EZ-FRISK* (Version 7.50), 7 known active faults are located within a search radius of 50 miles from the property. The nearest known active fault is the Rose

Canyon Fault, located approximately 15 miles west of the site and is the dominant source of potential ground motion. Earthquakes that might occur on the Rose Canyon Fault Zone or other faults within the southern California and northern Baja California area are potential generators of significant ground motion at the site. The estimated deterministic maximum earthquake magnitude and peak ground acceleration for the Rose Canyon Fault are 7.2 and 0.26g, respectively. Table 5.1.1 lists the estimated maximum earthquake magnitude and peak ground acceleration for the most dominant faults in relationship to the site location. We calculated peak ground acceleration (PGA) using Boore-Atkinson (2008) NGA USGS2008, Campbell-Bozorgnia (2008) NGA USGS, and Chiou-Youngs (2008) NGA acceleration-attenuation relationships.

TABLE 5.1.1
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)
Rose Canyon	15	7.2	0.23	0.25	0.26
Coronado Bank	22	7.7	0.16	0.12	0.15
Elsinore (Julian)	34	7.5	0.09	0.07	0.07
Newport-Inglewood (offshore)	37	7.2	0.08	0.06	0.06
Earthquake Valley	39	6.9	0.06	0.05	0.04
Elsinore (Coyote Mountain)	41	7.2	0.07	0.05	0.04
Elsinore (Temecula)	43	7.2	0.06	0.05	0.04

We used the computer program *EZ-FRISK* to perform a probabilistic seismic hazard analysis. The computer program *EZ-FRISK* operates under the assumption that the occurrence rate of earthquakes on each 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) NGA USGS, Campbell-Bozorgnia (2008) NGA USGS, and Chiou-Youngs (2008) in the analysis. Table 5.1.2 presents the site-specific probabilistic seismic hazard parameters including acceleration-attenuation relationships and the probability of exceedence.

TABLE 5.1.2
PROBABILISTIC SEISMIC HAZARD PARAMETERS

Probability of Exceedence	Peak Ground Acceleration		
	Boore-Atkinson, 2008 (g)	Campbell-Bozorgnia, 2008 (g)	Chiou-Youngs, 2008 (g)
2% in a 50 Year Period	0.40	0.41	0.50
5% in a 50 Year Period	0.32	0.30	0.36
10% in a 50 Year Period	0.23	0.23	0.26

The California Geologic Survey (CGS) has a program that calculates the ground motion for a 10 percent of probability of exceedence in 50 years based on an average of several attenuation relationships. Table 5.1.3 presents the calculated results from the *Probabilistic Seismic Hazards Mapping Ground Motion* Page from the CGS website.

TABLE 5.1.3
PROBABILISTIC SITE PARAMETERS FOR SELECTED FAULTS
CALIFORNIA GEOLOGIC SURVEY

Calculated Acceleration (g) Firm Rock	Calculated Acceleration (g) Soft Rock	Calculated Acceleration (g) Alluvium
0.22	0.25	0.29

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

5.2 Ground Rupture

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

5.3 Rippability of Rock – Air-Track Borings

We conducted air-track borings within the proposed area of development to evaluate rock rippability characteristics. In general, rock rippability is highly dependent on the degree of weathering and fracturing. In addition, rock rippability is a function of natural weathering processes that can change

vertically and horizontally over short distances depending on jointing, fracturing, and/or mineralogic discontinuities within the rock unit.

Air-track drill penetration rates can be used to evaluate rock rippability and to estimate the depth at which excavation difficulty will occur. Table 5.3 presents a summary of estimated rippability based on air-track drill penetration rates. Rippability is generally described as rock that can be excavated with a Caterpillar D-9 dozer with a single shank. These general guidelines are typically based on drill rates using a rotary percussion drill rig similar to an IR-370 with a 4-inch drill bit. The results of the air-track borings are presented in Appendix A.

TABLE 5.3
SUMMARY OF ESTIMATED RIPPABILITY FROM AIR-TRACK BORINGS

Drill Penetration Rate (seconds/foot)	Estimated Rippability
0 to 20	Rippable
20 to 30	Marginal Ripping (Possible Blasting)
Greater than 30	Non-Rippable (Blasting Likely)

The penetration rates (recorded in seconds per foot) for each air track boring are presented in Appendix A. The geologic materials encountered and the estimated thickness of rippable material at the air track boring locations, using 20 seconds per foot as the boundary between rippable and marginal to non-rippable rock. The estimate is derived from a literal interpretation of the penetration rate from each boring. Perspective contractors formulating construction cost estimates should consider their experience with productive and non-productive ripping when evaluating the penetration rate boundary between rippable and non-rippable rock.

5.4 Rippability of Rock – Seismic Refraction Survey

Southwest Geophysics performed a seismic refraction survey to evaluate the rock rippability along two seismic lines. The locations of the seismic traverses are presented on the Geologic Map, Figure 2 and their report is presented in Appendix B.

Based on our experience, we have summarized the estimated rippability characteristics for various excavation methods related to seismic velocity in Table 5.4. Estimates for mass grading rippability are based on using a D-9 Caterpillar Tractor equipped with a single shank hydraulic ripper. Estimates for trenching rippability are based on using a Caterpillar 345 excavator. It is often found to be more cost effective to blast marginally rippable bedrock.

TABLE 5.4
SUMMARY OF ESTIMATED RIPPABILITY FROM SEISMIC REFRACTION

Excavation Method	Seismic Velocity (ft/s)	Estimated Rippability
Mass Grading*	0 to 2,000	Easy
	2,000 to 4,000	Moderate
	4,000 to 5,500	Difficult (Possible Blasting)
	5,500 to 7,000	Very Difficult (Probable Blasting)
	Greater than 7,000	Non-Rippable (Pre-Blasting Required)
Trenching	Less than 3,800	Rippable
	3,800 to 4,300	Marginal Ripping
	Greater than 4,300	Non-Rippable

*Based on Seismic Refraction Survey.

The results of the seismic refraction surveys indicate that velocities less than approximately 3,000 ft/s are likely associated with surficial soil and highly weathered rock. Velocities between 3,000 and 5,000 ft/s are likely associated with moderately weathered rock. Velocities between 5,000 and 7,000 ft/s are likely associated with slightly weathered rock, with higher velocities associated with competent rock. Rippability is highly dependent upon the degree of weathering, fracturing, and jointing within the rock and the rippability of the various soil and rock units is, correspondingly, variable.

The grading contractor should use the information presented herein to evaluate the rippability of the existing materials. The rippability can vary depending on the type and strength of the equipment used. The contractors should rely on their experience to evaluate the rippability of the materials prior to submitting their bids/proposals.

6. FUTURE GEOTECHNICAL STUDIES

Based on our review of published geologic maps and our geologic reconnaissance of the site, an additional geotechnical investigation of the property will be necessary to further evaluate the subsurface conditions and to provide recommendations for design of the proposed project. The geotechnical investigation may include performing an additional subsurface investigation consisting of the excavation, sampling, and logging of subsurface excavations and laboratory testing to aid in the preparation of foundation and retaining wall design criteria, seismic design and recommendations for remedial grading.

7. CONCLUSIONS AND RECOMMENDATIONS

7.1 General

7.1.1 From a geotechnical standpoint, it is our opinion that the site is suitable for the proposed park improvements, provided the recommendations presented herein are implemented in design and construction of the project. The recommendations provided herein are preliminary. An additional geotechnical investigation will be required when development plans are available.

7.1.2 Our field investigation indicates that the site is underlain by topsoil, undocumented fill, and formational rock. The surficial soil is considered unsuitable to receive additional fill or settlement sensitive structures and will require remedial grading. We expect grading will consist of cuts and fills to establish grade for the proposed building and parking area and to provide surface drainage for the park site.

7.1.3 We did not observe groundwater or seepage in our exploratory excavations to the total depths explored. We do not expect groundwater to be encountered during construction of the proposed development.

7.1.4 With the exception of possible seismic shaking, we did not observe significant geologic hazards and do not know hazards to exist on the site that would adversely affect the proposed project. Special seismic design considerations other than those recommended herein are not required.

7.1.5 The proposed structures can likely be supported on conventional shallow footings founded in compacted fill. Preliminary recommendations for both conventional shallow foundations and post tensioned foundation system are presented herein.

7.1.6 Subsurface conditions observed may be extrapolated to reflect general soil/geologic conditions at the site; however, some variations in subsurface conditions between boring locations should be expected.

7.2 Excavation and Soil Characteristics

7.2.1 We expect that the surficial soil deposits can be excavated with a relatively low to moderate effort using conventional heavy-duty drilling/grading equipment. We expect moderate to very heavy effort for excavations within the weathered rock (rippable). Blasting, rock breaking or rock coring may be required if excavations are to extend into the less weathered, and fresh rock (marginal to nonrippable). Table 7.2.1 summarizes the excavation characteristics at the air-track locations based on the results of the field

investigations. In addition, the grading contractor should review the data presented herein to evaluate the rippability classification.

TABLE 7.2.1
EXCAVATION CHARACTERISTICS

Excavation No.	Geologic Formation	Estimated Depth of Layer (feet)	Estimated Rippability
AT-1	Rock Material (Mzu)	0-4	Rippable
		5-19	Marginally Rippable
AT-2	Rock Material (Mzu)	0-12	Rippable
		13-19	Marginally Rippable
AT-3	Rock Material (Mzu)	0-10	Rippable
		11-19	Marginally Rippable
		20-32	Non-Rippable
		33-40	Marginally-Rippable
AT-4	Rock Material (Mzu)	0-16	Rippable
		17-23	Marginally Rippable
		24-35	Rippable
		36-40	Marginally Rippable
AT-5	Undocumented Fill (Qudf)	0-19	Rippable
AT-6	Undocumented Fill (Qudf)	0-19	Rippable

7.2.2 We did not perform laboratory testing during our preliminary geotechnical investigation; however, we expect onsite soils are "non-expansive" (expansion index [EI] less than 20) and "expansive" (EI greater than 20) as defined by 2007 California Building Code (CBC) Section 1802.3.2. Table 7.2.2 presents soil classifications based on the expansion index. We expect a majority of the soil possesses a "very low" to "low" expansion potential (EI of 50 or less). Laboratory testing should be performed when the grading plan is complete for the design level geotechnical investigation.

TABLE 7.2.2
SOIL CLASSIFICATION BASED ON EXPANSION INDEX

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

7.2.3 Materials within the areas of planned structures should be evaluated for the percentage of water-soluble sulfate content. The presence of water-soluble sulfate 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.

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

7.3 Seismic Design Criteria

7.3.1 We used the computer program *Seismic Hazard Curves and Uniform Hazard Response Spectra*, provided by the USGS to calculate the seismic design criteria. Table 7.3 summarizes site-specific design criteria obtained from the 2007 California Building Code (CBC), Chapter 16 *Structural Design*, Section 1613 *Earthquake Loads*. The short spectral response has a period of 0.2 second. A soil Site Class C and D has been assumed for the proposed structures that may possess a fill thickness of less than 20 feet and proposed structures with a fill thickness greater than or equal to 20 feet, respectively.

TABLE 7.3
2007 CBC SEISMIC DESIGN PARAMETERS

Parameter	Value		2007 CBC Reference
Site Class	C	D	Table 1613.5.2
Fill Thickness, T (feet)	T<20	T≥20	–
Spectral Response – Class B (short), S_S	1.014g	1.014g	Figure 1613.5(3)
Spectral Response – Class B (1 sec), S_1	0.365g	0.365g	Figure 1613.5(4)
Site Coefficient, F_a	1.000	1.095	Table 1613.5.3(1)
Site Coefficient, F_v	1.435	1.671	Table 1613.5.3(2)
Maximum Considered Earthquake Spectral Response Acceleration (short), S_{MS}	1.014g	1.110g	Section 1613.5.3 (Eqn 16-37)
Maximum Considered Earthquake Spectral Response Acceleration – (1 sec), S_{M1}	0.523g	0.609g	Section 1613.5.3 (Eqn 16-38)
5% Damped Design Spectral Response Acceleration (short), S_{DS}	0.676g	0.740g	Section 1613.5.4 (Eqn 16-39)
5% Damped Design Spectral Response Acceleration (1 sec), S_{D1}	0.349g	0.406g	Section 1613.5.4 (Eqn 16-40)

7.3.2 Conformance to the criteria in Table 7.3 for seismic design does not constitute any kind of guarantee or assurance that significant structural damage or ground failure will not occur if a maximum level earthquake occurs. The primary goal of seismic design is to protect life and not to avoid all damage, since such design may be economically prohibitive.

7.4 Preliminary Grading Recommendations

7.4.1 The grading recommendations presented herein (Appendix C) are considered preliminary and should be used for planning purposes only. Finalized grading recommendations should be provided after a detailed geotechnical investigation is performed.

7.4.2 A pre-construction conference with the owner, contractor, civil engineer, and soil engineer in attendance should be held at the site prior to the beginning of grading operations. Special soil handling requirements can be discussed at that time.

7.4.3 Earthwork should be observed and compacted fill tested by representatives of Geocon Incorporated.

7.4.4 Grading of the site should commence with the removal of existing improvements from the areas to be graded. Deleterious debris and unacceptable contaminated soil, if encountered, should be exported from the site and should not be mixed with the fill soil. Existing underground improvements within the proposed building areas should be removed and the resulting depressions properly backfilled in accordance with the procedures described herein.

7.4.5 Topsoil and highly weathered or decomposed formational rock materials (if encountered) within the planned expansion area should be removed to expose firm formational rock materials. The actual depth of removal should be evaluated by the geotechnical engineering consultant during the grading operations. In addition, the existing formational rock materials should be undercut at least 3 feet and replaced with compacted fill. The undercuts will facilitate future trenching/landscaping at the planned finish grade.

7.4.6 Roadways and utility areas underlain by hard rock units at grade should be undercut a minimum of 8 feet for the areas inside of the public right-of-way (including joint utility structures and sidewalk areas). The undercut zone should include the areas within 1 foot of the lowest utility or drain line.

7.4.7 The existing upper 4 feet of undocumented fill within the area of planned structures or flatwork improvements should be removed and replaced with compacted fill. The actual

depth of removal should be evaluated by the Geotechnical Engineer during grading operations. A deeper removal my be determined subsequent to performing the supplemental geotechnical investigation. Prior to the placement of compacted fill, the exposed ground surface should be scarified where practical, moisture conditioned as necessary, and compacted.

7.4.8 The bottom of the excavations should be scarified to a depth of at least 8 inches (where possible), moisture conditioned as necessary, and properly compacted. To the extent practical, excavated soils with an Expansion Index greater than 50 should be kept at least 3 to 4 feet below finish grades in areas of the structural fill. Sheet-graded pads should be capped with at least 6 feet of low expansive soil to accommodate minor regrading.

7.4.9 If the remedial grading is limited due to the presence of utility lines or boundary conditions, partial removal and recompaction along with other corrective measures should be implemented to accommodate the potential settlement. Geocon Incorporated should be contacted if this issue exists.

7.4.10 The site should then be brought to final grade elevations with structural fill. Excavated soil generally free of deleterious debris can be placed as fill and compacted in layers to the design finish grade elevations. Fill and backfill soil should be placed in horizontal loose layers approximately 6 to 8 inches thick, moisture conditioned as necessary, and compacted to a dry density of at least 90 percent of the laboratory maximum dry density near to slightly above optimum moisture content as determined by ASTM D 1557. Rock greater than 1 foot in maximum dimension should not be placed within 3 feet of finish grade or 1 foot of the deepest utilities.

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

7.5 Foundation and Concrete Slabs-On-Grade Recommendations

7.5.1 The foundation recommendations herein are for proposed one- to two-story structures. The foundation recommendations have been separated into three categories based on either the maximum and differential fill thickness or Expansion Index. The foundation category criteria are presented in Table 7.5.1.

TABLE 7.5.1
FOUNDATION CATEGORY CRITERIA

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

7.5.2 Final foundation categories for each structure will be provided after finish pad grades have been achieved and laboratory testing of the subgrade soil has been completed.

7.5.3 Table 7.5.2 presents minimum foundation and interior concrete slab design criteria for conventional foundation systems.

TABLE 7.5.2
CONVENTIONAL FOUNDATION RECOMMENDATIONS BY CATEGORY

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

7.5.4 The embedment depths presented in Table 7.5.2 should be measured from the lowest adjacent pad grade for both interior and exterior footings. The conventional foundations should have a minimum width of 12 inches and 24 inches for continuous and isolated footings, respectively. A wall/column footing dimension detail is presented in Figure 3.

7.5.5 Concrete slabs on grade should be underlain by 4 inches of clean sand (3 inches for a 5-inch-thick slab) to reduce the potential for differential curing, slab curl, and cracking. Slabs that may receive moisture-sensitive floor coverings or may be used to store moisture-sensitive materials should be underlain by a vapor retarder placed near the middle of the sand bedding. The vapor retarder used should be specified by the project architect or developer based on the type of floor covering that will be installed. The vapor retarder design should be consistent with the guidelines presented of the American Concrete

Institute's (ACI) Guide for Concrete Slabs that Receive Moisture-Sensitive Flooring Materials (ACI 302.2R-06).

7.5.6 As an alternative to the conventional foundation recommendations, consideration should be given to the use of post-tensioned concrete slab and foundation systems for the support of the proposed structures. The 2007 CBC has updated the design requirements for post-tensioned foundation systems. The post-tensioned systems should be designed by a structural engineer experienced in post-tensioned slab design and design criteria of the Post-Tensioning Institute (PTI), Third Edition, as required by the 2007 California Building Code (CBC Section 1805.8). Although this procedure was developed for expansive soil conditions, we understand it can also be used to reduce the potential for foundation distress due to differential fill settlement. The post-tensioned design should incorporate the geotechnical parameters presented on Table 7.5.3 for the particular Foundation Category designated. The parameters presented in Table 7.5.3 are based on the guidelines presented in the PTI, Third Edition design manual.

TABLE 7.5.3
POST-TENSIONED FOUNDATION SYSTEM DESIGN PARAMETERS

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

7.5.7 If the structural engineer proposes a post-tensioned foundation design method other than the 2007 CBC:

- The criteria presented in Table 7.5.3 are still applicable.
- Interior stiffener beams should be used for Foundation Categories II and III.
- The width of the perimeter foundations should be at least 12 inches.
- The perimeter footing embedment depths should be at least 12 inches, 18 inches and 24 inches for foundation categories I, II, and III, respectively. The embedment depths should be measured from the lowest adjacent pad grade.

7.5.8 Foundation systems for the lots that possess a foundation Category I and a "very low" expansion potential (expansion index of 20 or less) can be designed using the method

described in Section 1805 of the 2007 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 Incorporated should be contacted to review the plans and provide additional information, if necessary.

7.5.9 If an alternate design method is contemplated, Geocon Incorporated should be contacted to evaluate if additional expansion index testing should be performed to further evaluate and identify the lots that possess a "very low" expansion potential (expansion index of 20 or less)

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

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

7.5.12 Category I, II, or III foundations may be designed for an allowable soil bearing pressure of 2,000 pounds per square foot (psf) (dead plus live load). This bearing pressure may be increased by one-third for transient loads due to wind or seismic forces.

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

7.5.14 For Foundation Category III, consideration should be given to using interior stiffening beams and connecting isolated footings and/or increasing the slab thickness. In addition,

consideration should be given to connecting flatwork slabs, which exceed 5 feet in width, to the building foundation to reduce the potential for future separation to occur.

7.5.15 Special subgrade presaturation is not deemed necessary prior to placing concrete; however, the exposed foundation and slab subgrade soil should be moisture conditioned, as necessary, to maintain a moist condition as would be expected in any such concrete placement.

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

- For fill slopes less than 20 feet high, 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 3:1 (horizontal:vertical) fill slope or steeper, 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.

- 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 Incorporated should be consulted for specific recommendations.

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

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

7.6 Retaining Walls

7.6.1 Retaining walls not restrained at the top and having a level backfill surface should be designed for an active soil pressure equivalent to the pressure exerted by a fluid density of 35 pounds per cubic foot (pcf). Where the backfill will be inclined at 2:1 (horizontal:vertical), an active soil pressure of 50 pcf is recommended. Soil with an expansion index (EI) of greater than 50 should not be used as backfill material behind retaining walls.

7.6.2 Unrestrained walls are those that are allowed to rotate more than 0.001H (where H equals the height of the retaining portion of the wall) at the top of the wall. Where walls are restrained from movement at the top, an additional uniform pressure of 7H psf should be added to the active soil pressure where the wall possesses a height of 8 feet or less and 12H where the wall in greater than 8 feet. For retaining walls subject to vehicular loads within a horizontal distance equal to two-thirds the wall height, a surcharge equivalent to 2 feet of fill soil should be added.

7.6.3 The use of drainage openings through the base of the wall (weep holes) is not recommended where the seepage could be a nuisance or otherwise adversely affect the property adjacent to the base of the wall. The recommendations herein assume a properly compacted granular (EI of 50 or less) free-draining backfill material with no hydrostatic forces or imposed surcharge load. Figure 4 presents a typical retaining wall drainage detail. If conditions different than those described are expected, or if specific drainage details are desired, Geocon Incorporated should be contacted for additional recommendations.

7.6.4 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 23H 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 top of the wall and zero at the base of the wall. We used a peak site acceleration of 0.30g calculated using $S_{DS}/2.5$ USGS and applying a pseudo-static coefficient of 0.5.

7.6.5 Unrestrained walls will move laterally when backfilled and loading is applied. The amount of lateral deflection is dependant on the wall height, the type of soil used for backfill, and loads acting on the wall. The retaining walls and improvements above the retaining walls

should be designed to incorporate an appropriate amount of lateral deflection as determined by the structural engineer

7.7 Lateral Loading

7.7.1 To resist lateral loads, a passive pressure exerted by an equivalent fluid density of 300 pounds per cubic foot (pcf) should be used for the design of footings or shear keys. The allowable passive pressure assumes a horizontal surface extending at least 5 feet, or three times the surface generating the passive pressure, whichever is greater. The upper 12 inches of material in areas not protected by floor slabs or pavement should not be included in design for passive resistance.

7.7.2 If friction is to be used to resist lateral loads, an allowable coefficient of friction between soil and concrete of 0.35 should be used for design.

7.8 Preliminary Pavement Recommendations

7.8.1 We expect the planned driveways will be paved with asphalt concrete pavement. The final pavement sections for the roadways should be based on the R-Value of the subgrade soil encountered at final subgrade elevation. Based on our experience, we have assumed an R-Value of 5 and 20 for the subgrade soil for the purposes of this preliminary analysis. Table 7.8.1 presents preliminary flexible pavement sections.

TABLE 7.8.1
PRELIMINARY FLEXIBLE PAVEMENT SECTION

Location	Assumed Traffic Index	Assumed Subgrade R-Value	Asphalt Concrete (inches)	Class 2 Aggregate Base (inches)
Parking stalls for automobiles and light-duty vehicles	5.0	5	3	10
		20	3	7
Driveways for automobiles and light-duty vehicles	5.5	5	3	12
		20	3	9
Driveways for heavy truck traffic	7.0	5	4	16
		20	4	12

7.8.2 Base materials should conform to Section 26-1.028 of the *Standard Specifications for The State of California Department of Transportation (Caltrans)* with a ¾-inch maximum size aggregate. Base materials should be compacted to a dry density of at least 95 percent of the laboratory maximum dry density near to slightly above optimum moisture content. The

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

7.8.3 The base thickness can be reduced if a reinforcement geogrid within the base section is used during the installation of the pavement. Geocon should be contacted for additional recommendations, if required.

7.8.4 Prior to placing base materials, the subgrade soil should be scarified, moisture conditioned as necessary, and recompacted to a dry density of at least 95 percent of the laboratory maximum dry density near to slightly above optimum moisture content as determined by ASTM D 1557. The depth of compaction should be at least 12 inches. Similarly, the base material should be compacted to a dry density of at least 95 percent of the laboratory maximum dry density near to slightly above optimum moisture content as determined by ASTM D 1557.

7.8.5 A rigid Portland cement concrete (PCC) pavement section should be placed in driveway entrance aprons, trash bin loading/storage areas and loading dock areas. The concrete pad for trash truck areas should be large enough such that the truck wheels will be positioned on the concrete during loading. We calculated the rigid pavement section in general conformance with the procedure recommended by the American Concrete Institute report ACI 330R-08 Guide for Design and Construction of Concrete Parking Lots using the parameters presented in Table 7.8.2.

TABLE 7.8.2
RIGID PAVEMENT DESIGN PARAMETERS

Design Parameter	Design Value
Modulus of subgrade reaction, k	100 pci
Modulus of rupture for concrete, M_R	500 psi
Traffic Category, TC	A and C
Average daily truck traffic, ADTT	10 and 100

7.8.6 Based on the criteria presented herein, the PCC pavement sections should have a minimum thickness as presented in Table 7.8.3.

TABLE 7.8.3
RIGID PAVEMENT RECOMMENDATIONS

Location	Portland Cement Concrete (inches)
Automobile Parking Areas (TC=A-1)	5.5
Heavy Truck and Fire Lane Areas (TC=C)	7.0

7.8.7 The PCC pavement should be placed over subgrade soil that is compacted to a dry density of at least 95 percent of the laboratory maximum dry density near to slightly above optimum moisture content. This pavement section is based on a minimum concrete compressive strength of approximately 3,000 psi (pounds per square inch).

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

7.8.9 To control the location and spread of concrete shrinkage cracks, crack-control joints (weakened plane joints) should be included in the design of the concrete pavement slab. Crack-control joints should not exceed 30 times the slab thickness with a maximum spacing of 12.5 feet and 15 feet for the 5.5 and 7-inch-thick slabs, respectively (e.g., a 7-inch-thick slab would have a 15-foot spacing pattern), and should be sealed with an appropriate sealant to prevent the migration of water through the control joint to the subgrade materials. The depth of the crack-control joints should be determined by the referenced ACI report.

7.8.10 To provide load transfer between adjacent pavement slab sections, a butt-type construction joint should be constructed. The butt-type joint should be thickened by at least 20 percent at the edge and taper back at least 4 feet from the face of the slab. As an alternative to the butt-type construction joint, dowelling can be used between construction joints for pavements of 7 inches or thicker. As discussed in the referenced ACI guide, dowels should consist of smooth, 1-inch-diameter reinforcing steel 14 inches long embedded a minimum of 6 inches into the slab on either side of the construction joint. Dowels should be located at the midpoint of the slab, spaced at 12 inches on center and lubricated to allow joint movement while still transferring loads. In addition, tie bars should be installed at the as

recommended in Section 3.8.3 of the referenced ACI guide. The structural engineer should provide other alternative recommendations for load transfer.

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

7.9 Site Drainage and Moisture Protection

7.9.1 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 2007 CBC 1803.3 or other applicable standards. In addition, surface drainage should be directed away from the top of slopes into swales or other controlled drainage devices. Roof and pavement drainage should be directed into conduits that carry runoff away from the proposed structure.

7.9.2 In the case of basement walls or building walls retaining landscaping areas, a waterproofing system should be used on the wall and joints, and a Miradrain drainage panel (or similar) should be placed over the waterproofing. The project architect or civil engineer should provide detailed specifications on the plans for all waterproofing and drainage.

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

7.9.4 Detention basins, bioswales, retention basins, or water infiltration devices without liners and subdrains are not considered suitable for the site conditions. If these devices are required, impermeable liners and subdrains should be installed to prevent water infiltrating the planned fill and existing alluvium soil. Distress may be caused to planned improvements and properties onsite and located hydrologically downstream. The distress

depends on the amount of water to be detained, its residence time, soil permeability, and other factors. We have not performed a hydrogeology study at the site. Downstream properties may be subjected to seeps, springs, slope instability, raised groundwater, movement of foundations and slabs, or other impacts as a result of water infiltration.

7.10 Grading and Foundation Plan Review

7.10.1 Geocon Incorporated should review the grading plans and foundation plans for the project prior to final design submittal and prepare a geotechnical investigation. The geotechnical investigation should include performing additional explorations on the property, performing laboratory analyses, and preparing a final report.

LIMITATIONS AND UNIFORMITY OF CONDITIONS

1. The recommendations of this report pertain only to the site investigated and are based upon the assumption that the soil conditions do not deviate from those disclosed in the investigation. If any variations or undesirable conditions are encountered during construction, or if the proposed construction will differ from that anticipated herein, Geocon Incorporated should be notified so that supplemental recommendations can be given. The evaluation or identification of the potential presence of hazardous or corrosive materials was not part of the scope of services provided by Geocon Incorporated.

2. This report is issued with the understanding that it is the responsibility of the owner or his representative to ensure that the information and recommendations contained herein are brought to the attention of the architect and engineer for the project and incorporated into the plans, and that the necessary steps are taken to see that the contractor and subcontractors carry out such recommendations in the field.

3. The findings of this report are valid as of the present date. However, changes in the conditions of a property can occur with the passage of time, whether due to natural processes or the works of man on this or adjacent properties. In addition, changes in applicable or appropriate standards may occur, whether they result from legislation or the broadening of knowledge. Accordingly, the findings of this report may be invalidated wholly or partially by changes outside our control. Therefore, this report is subject to review and should not be relied upon after a period of three years.

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

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NO SCALE

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ME / AML		DSK/GTYPD

VICINITY MAP

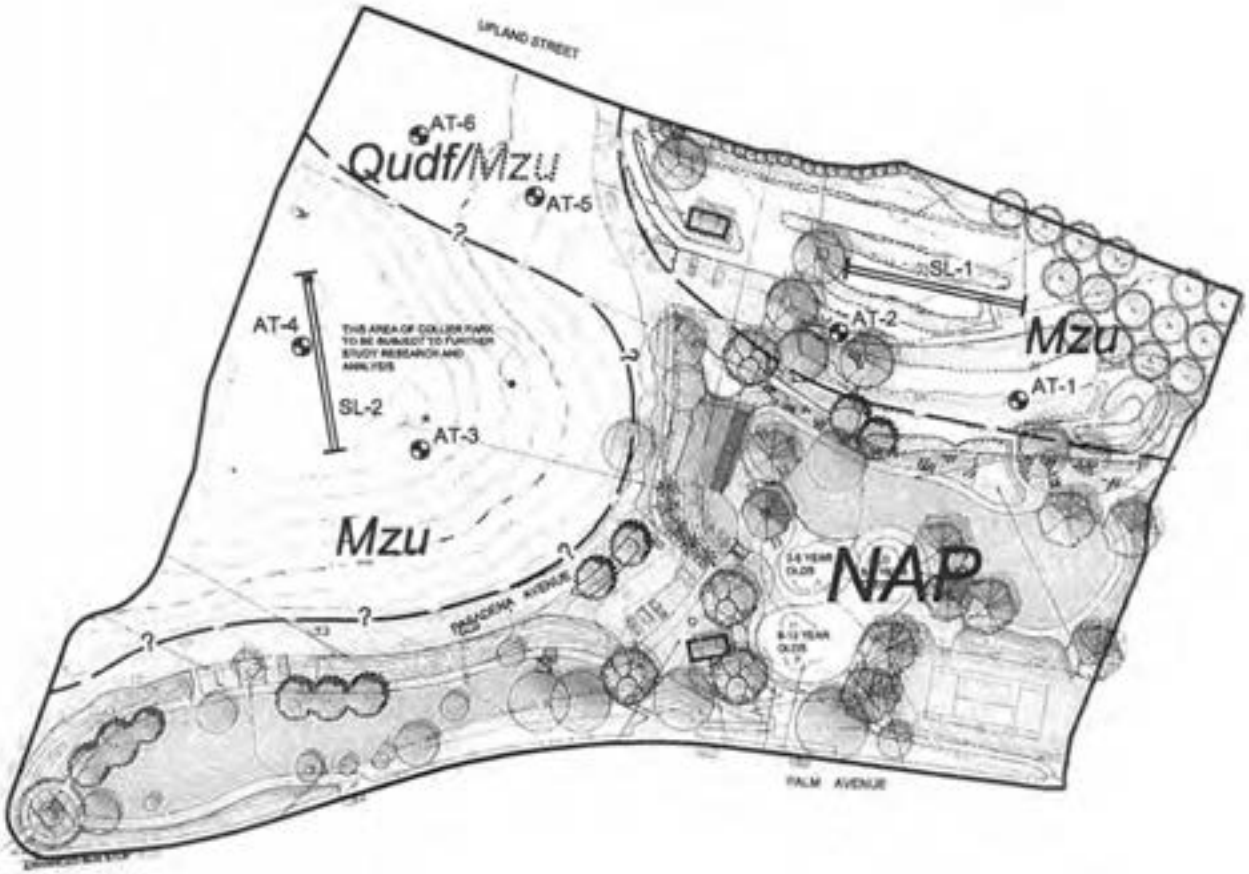
COLLIER PARK EXPANSION
PASADENA AVENUE AND PALM AVENUE
LA MESA, CALIFORNIA

DATE 12 - 10 - 2010	PROJECT NO. G1222 - 52 - 03	FIG. 1

Vicinity Map

Y:\PROJECTS\G1222-52-03 [Collier Park]\DETAILS\Vicinity Map.dwg, 12/10/2010 11:11:53 AM, bob@geo

COLLIER PARK EXPANSION
PASADENA AVENUE AND PALM AVENUE
LA MESA, CALIFORNIA



APPROX. SCALE: 1" = 80'

GEOCON LEGEND

Qudf — UNDOCUMENTED FILL

Mzu — METASEDIMENTARY AND METAVOLCANIC ROCK, UNDIVIDED (Dashed Where Buried)

APPROX. LOCATION OF GEOLOGIC CONTACT (Queried Where Uncertain)

AT-4 — APPROX. LOCATION OF AIR TRACK BORING

APPROX. LOCATION OF GEOPHYSICAL SURVEY, SEISMIC LINE

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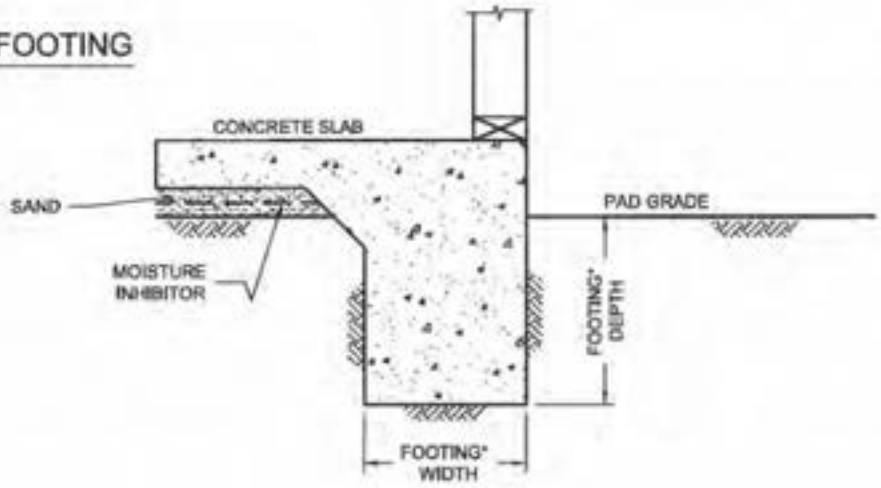
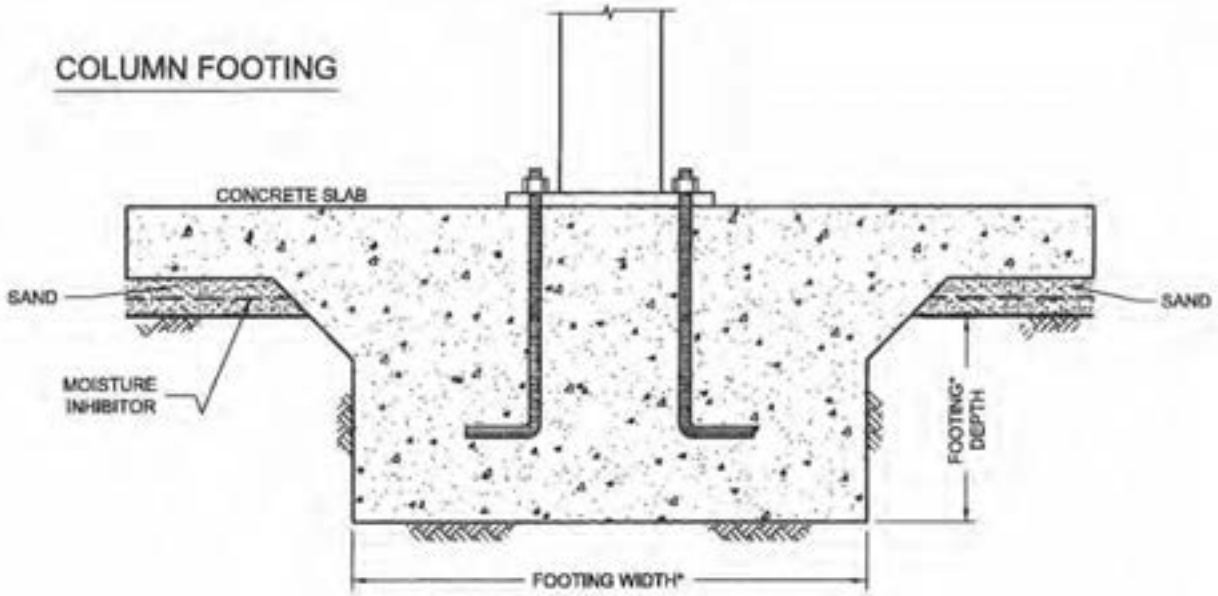
PROJECT NO. G1222-32-03
FIGURE 2
DATE 12-10-2010

GEOLOGIC MAP

WALL FOOTING



COLUMN FOOTING



NO SCALE

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

WALL / COLUMN FOOTING DIMENSION DETAIL

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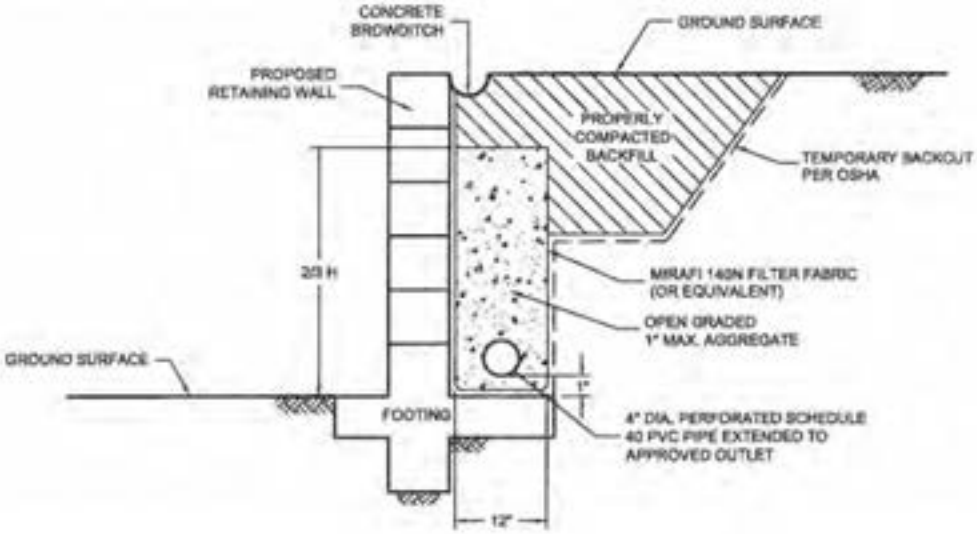
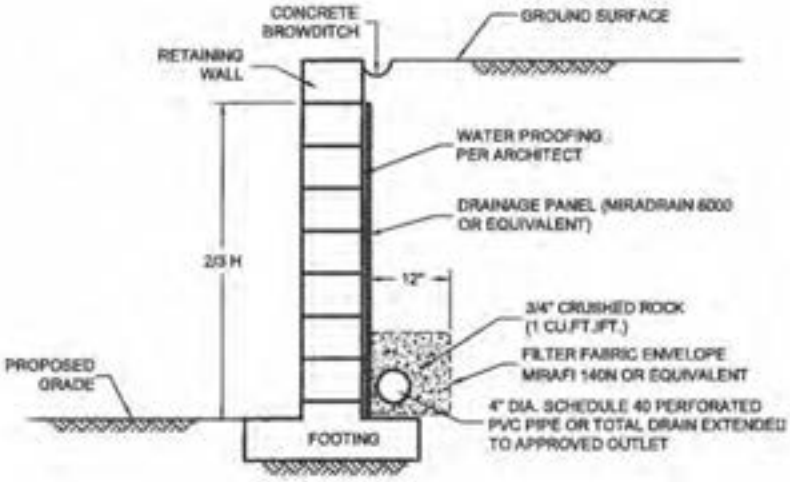
COLLIER PARK EXPANSION
PASADENA AVENUE AND PALM AVENUE
LA MESA, CALIFORNIA

SW / AML		DSK/GTYPD	DATE 12 - 10 - 2010	PROJECT NO. G1222 - 52 - 03	FIG. 3

COLFOOT2DWG/wd





NOTE :

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

NO SCALE

TYPICAL RETAINING WALL DRAIN DETAIL

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COLLIER PARK EXPANSION
PASADENA AVENUE AND PALM AVENUE
LA MESA, CALIFORNIA

SW / AML		DSK/GTYPD	DATE 12 - 10 - 2010	PROJECT NO. G1222 - 52 - 03	FIG. 4

BWD07

APPENDIX A

APPENDIX A

FIELD INVESTIGATION

We performed the field investigation on November 19, 2010, consisting of a visual site reconnaissance and the drilling of six air-track borings. The approximate locations of the exploratory borings are shown on the Geologic Map, Figure 2.

An EMC 370 pneumatic percussion, 4-inch-diameter drill with hydraulic pulldown was used for air-track holes at selected locations. A graphical representation of each of the air-track borings is presented on Figure A-1 through A-6. The graphs depict the rate (in seconds per foot) at which the drill penetrated each foot of depth examined.

If elevations are shown on the boring logs, they were evaluated either from a topographic map or by using a temporary benchmark.

PROJECT NO. G1222-52-03

COLLIER PARK EXPANSION - LA MESA

AIR TRACK BORING AT-1
Approx. Elevation - 515 Feet (MSL)
Date 11-19-2010 - Equipment: ECM 370

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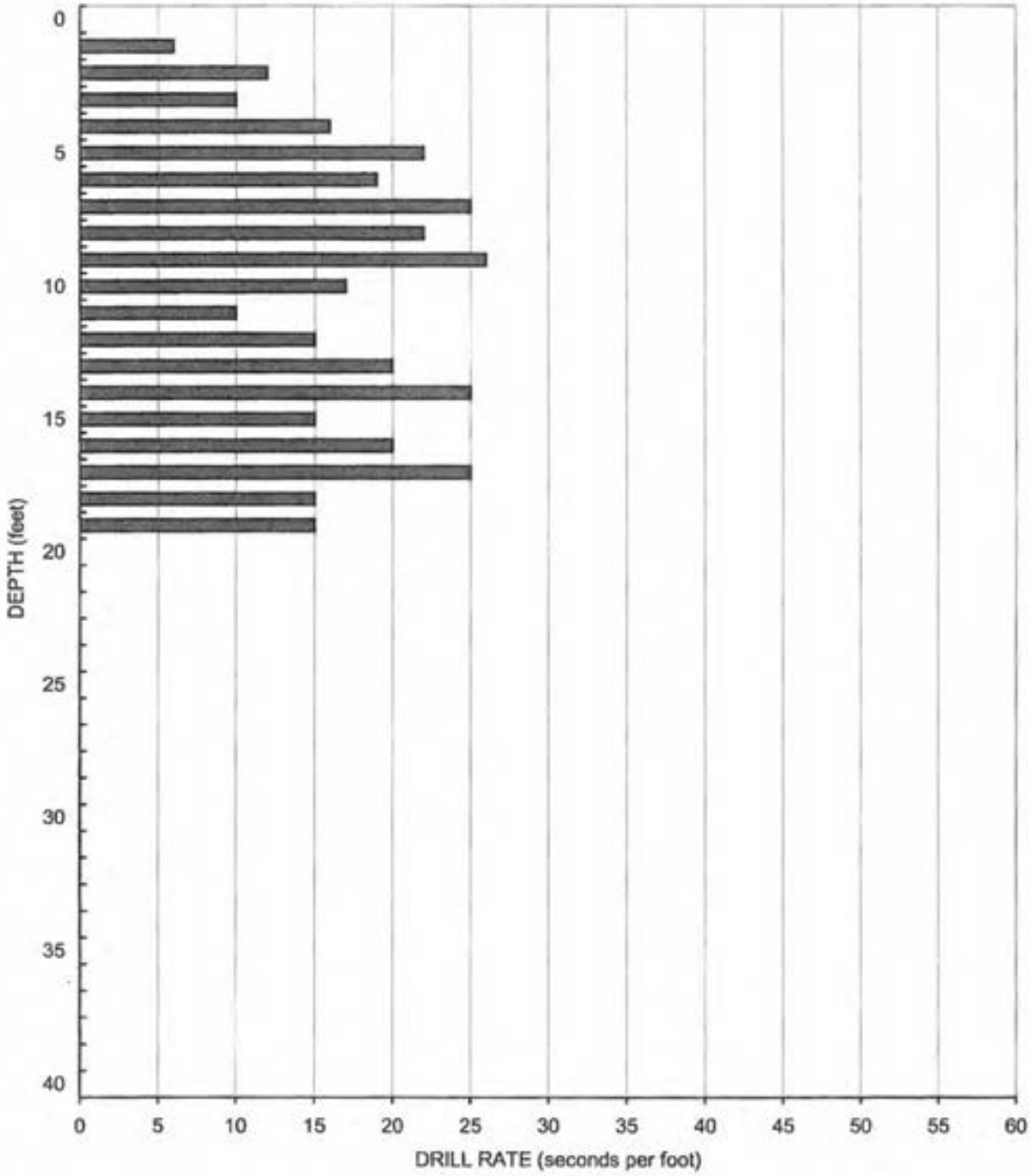

AT001.xls

FIGURE A1

PROJECT NO. G1222-52-03

COLLIER PARK EXPANSION - LA MESA

AIR TRACK BORING AT-2
Approx. Elevation - 515 Feet (MSL)
Date 11-19-2010 - Equipment: ECM 370

GEOCON
INCORPORATED

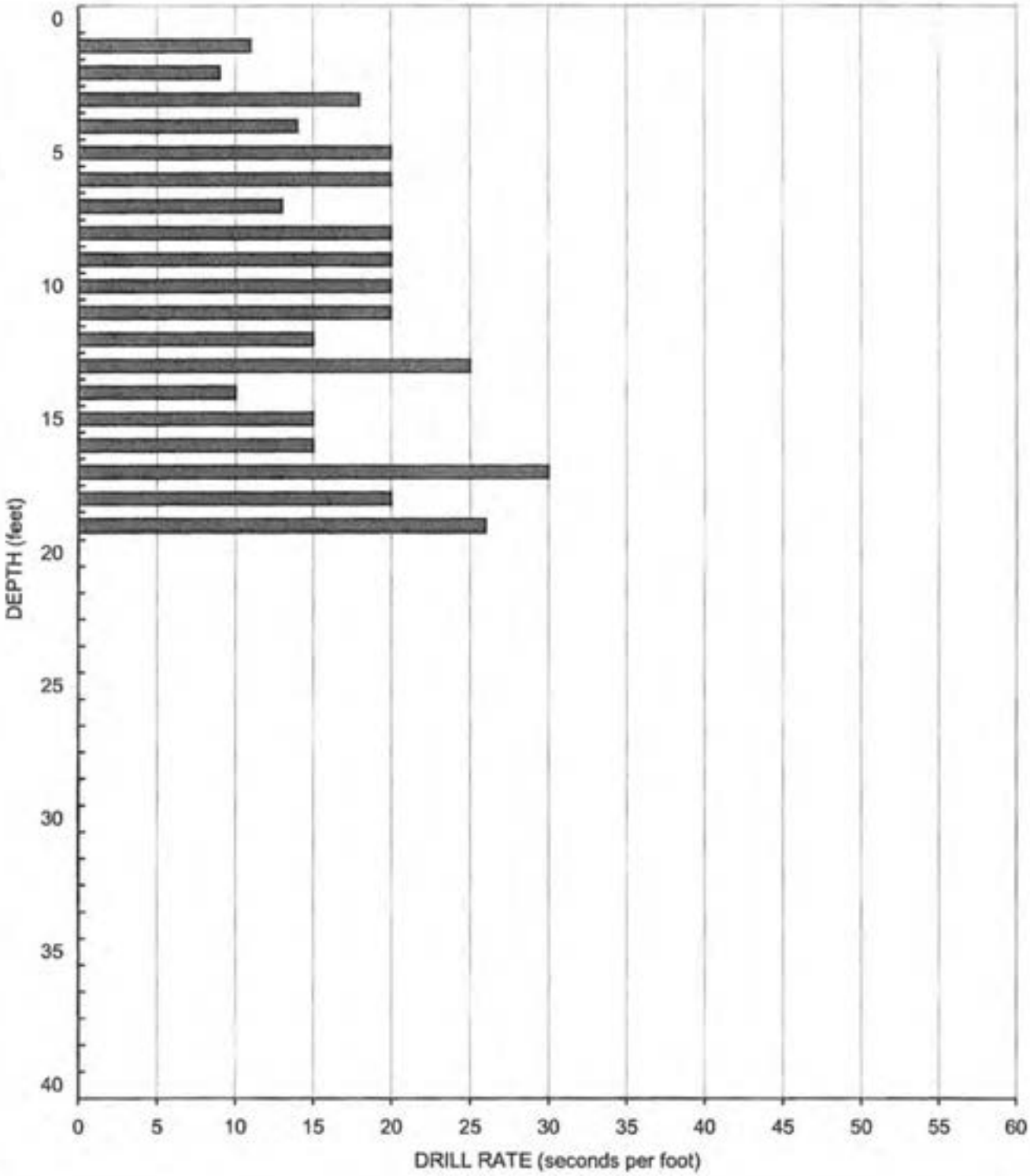

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FIGURE A2

PROJECT NO. G1222-52-03

COLLIER PARK EXPANSION - LA MESA

AIR TRACK BORING AT-3
Approx. Elevation - 545 Feet (MSL)
Date 11-19-2010 - Equipment: ECM 370

GEOCON
INCORPORATED

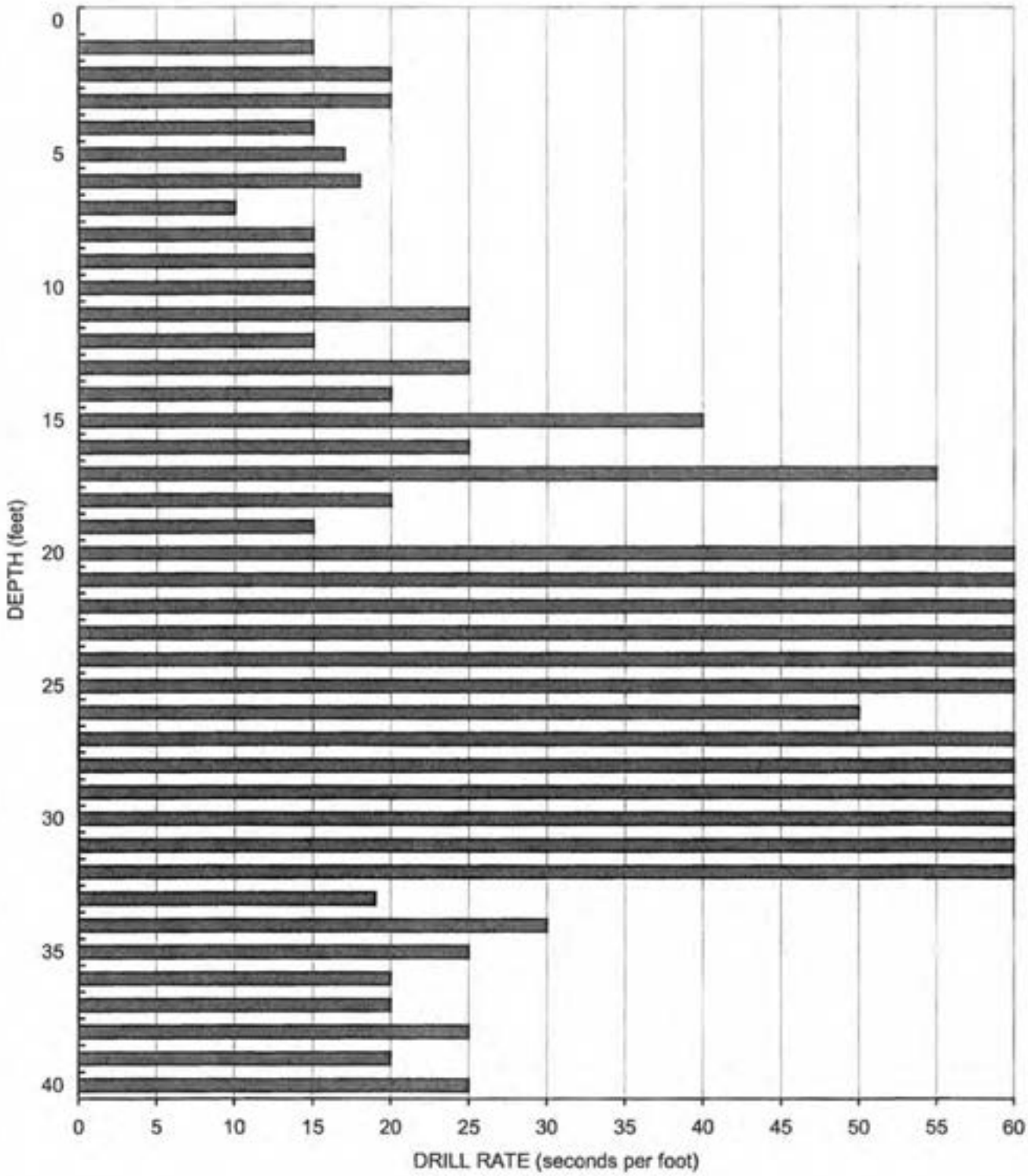

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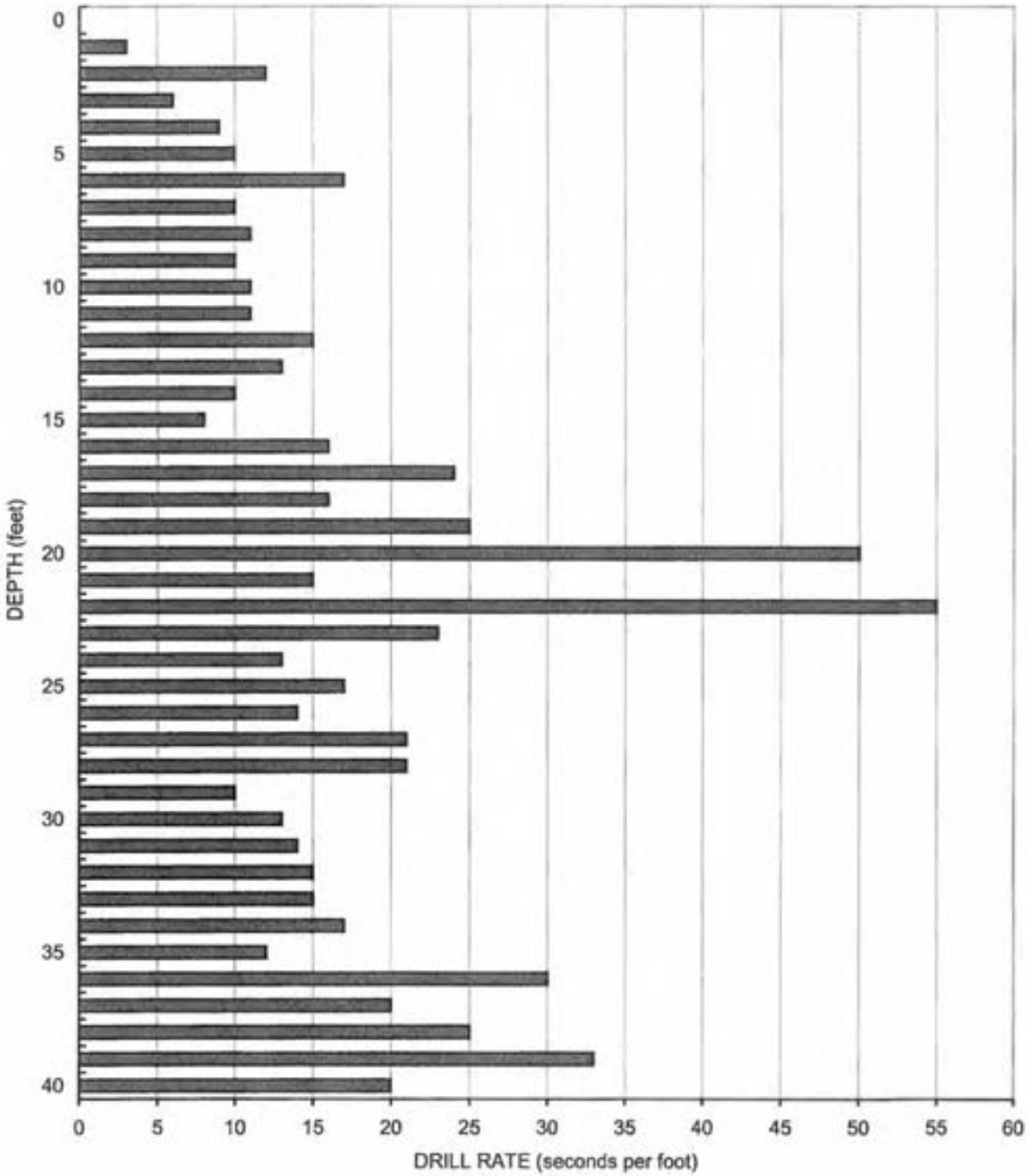
FIGURE A3

PROJECT NO. G1222-52-03

COLLIER PARK EXPANSION - LA MESA

AIR TRACK BORING AT-4
Approx. Elevation - 543 Feet (MSL)
Date 11-19-2010 - Equipment: ECM 370

GEOCON
INCORPORATED



AT004.xls

FIGURE A4

COLLIER PARK EXPANSION - LA MESA

AIR TRACK BORING AT-5
Approx. Elevation - 520 Feet (MSL)
Date 11-19-2010 - Equipment: ECM 370

GEOCON
INCORPORATED

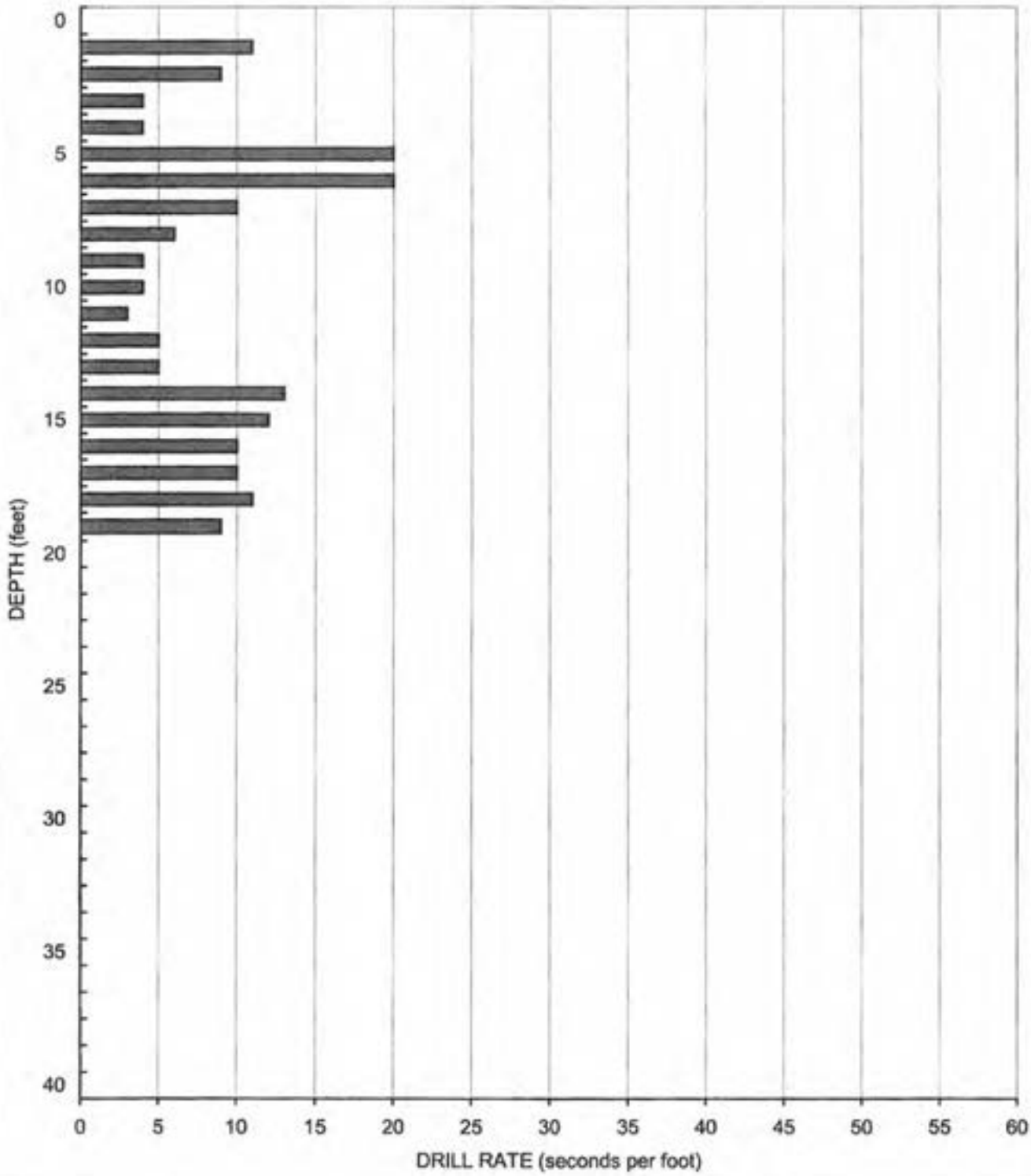


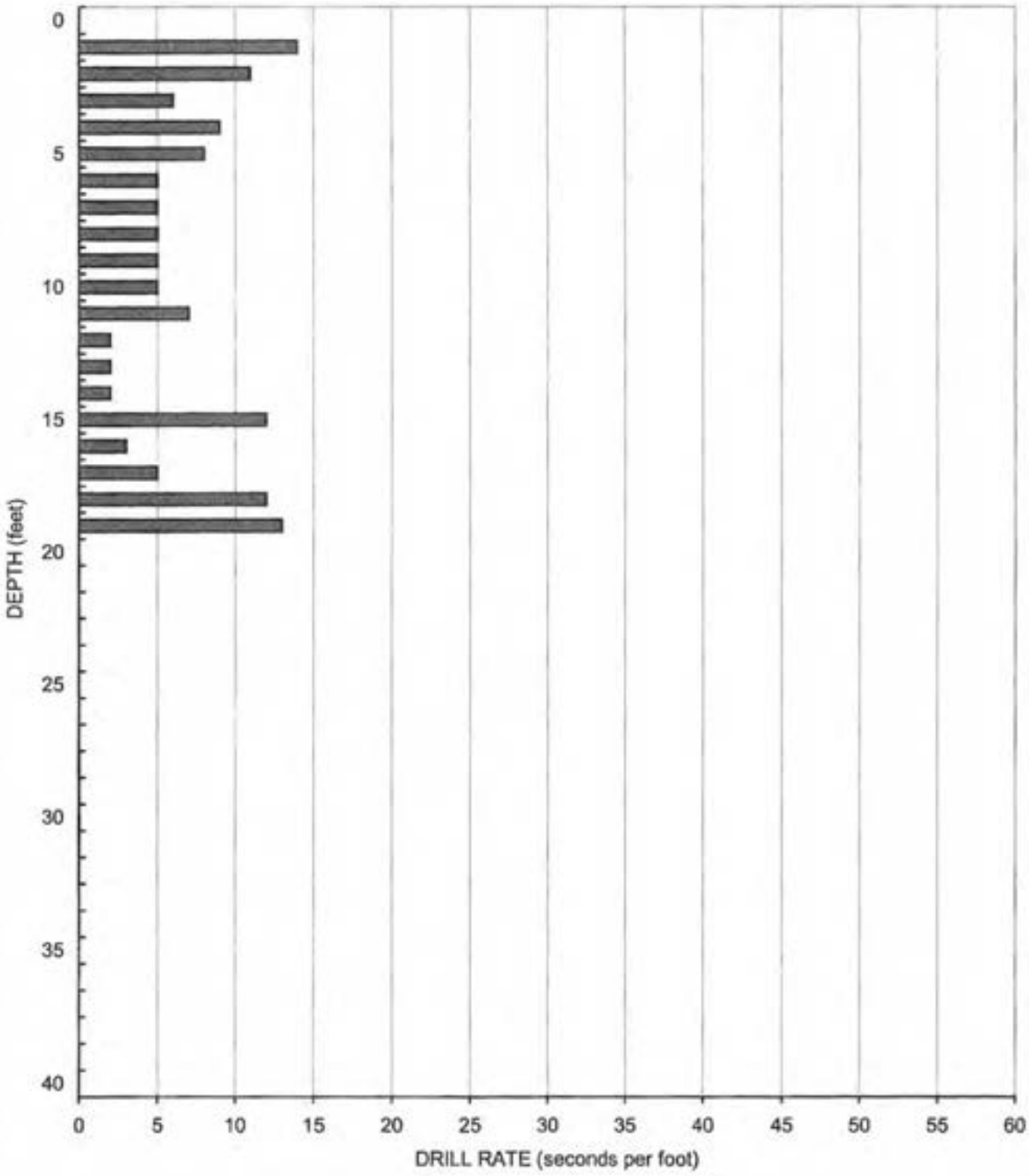
FIGURE A5

PROJECT NO. G1222-52-03

COLLIER PARK EXPANSION - LA MESA

AIR TRACK BORING AT-6
Approx. Elevation - 522 Feet (MSL)
Date 11-19-2010 - Equipment: ECM 370

GEOCON
INCORPORATED



AT006.xls

FIGURE A6

APPENDIX B

SEISMIC REFRACTION SURVEY
COLLIER PARK
LA MESA, CALIFORNIA

PREPARED FOR:
Geocon Consultants, Inc.
6970 Flanders Drive
San Diego, CA 92121

PREPARED BY:
Southwest Geophysics, Inc.
8057 Raytheon Road, Suite 9
San Diego, CA 92111

October 8, 2010
Project No. 110259

SOUTHWEST
GEOPHYSICS, INC
YOUR SUBSURFACE SOLUTION

October 8, 2010
Project No. 110259

Mr. Shawn Weedon
Geocon Consultants, Inc.
6970 Flanders Drive
San Diego, CA 92121

Subject: Seismic Refraction Survey
Collier Park
La Mesa, California

Dear Mr. Weedon:

In accordance with your authorization, we have performed a seismic refraction survey for the proposed Collier Park improvements to be located in an area of generally undeveloped land along the north and east sides of Collier Park in La Mesa, California. Specifically, our survey consisted of performing two seismic refraction lines at the subject site. The purpose of our study was to develop a subsurface velocity profile of the areas surveyed, and to assess the apparent rippability of near surface materials. This data report presents our survey methodology, equipment used, analysis, and results.

We appreciate the opportunity to be of service on this project. Should you have any questions related to this report, please contact the undersigned at your convenience.

Sincerely,
SOUTHWEST GEOPHYSICS, INC.

Patrick Lehrmann, P.G., R.Gp.
Principal Geologist/Geophysicist

Hans van de Vrugt, C.E.G., R.Gp.
Principal Geologist/Geophysicist

HV/PFL/hv

Distribution: Addressee (electronic)

REGISTERED GEOPHYSICIST
Hans van de Vrugt
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Exp
STATE OF CALIFORNIA

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TABLE OF CONTENTS

Page

1. INTRODUCTION 1
2. SCOPE OF SERVICES 1
3. SITE AND PROJECT DESCRIPTION 1
4. SURVEY METHODOLOGY 2
5. RESULTS 3
6. CONCLUSIONS AND RECOMMENDATIONS 4
7. LIMITATIONS 5
8. SELECTED REFERENCES 6

Tables

Table 1 – Rippability Classification 3
Table 2 – Seismic Traverse Results 4

Figures

Figure 1 – Site Location Map
Figure 2 – Seismic Line Location Map
Figure 3 – Site Photographs
Figure 4a – Seismic Profile, SL-1
Figure 4b – Seismic Profile, SL-2

1. INTRODUCTION

In accordance with your authorization, we have performed a seismic refraction survey for the proposed Collier Park improvements to be located in an area of generally undeveloped land along the north and east sides of Collier Park in La Mesa, California (Figure 1). Specifically, our survey consisted of performing two seismic refraction lines at the subject site. The purpose of our study was to develop a subsurface velocity profile of the areas surveyed, and to assess the apparent rippability of near surface materials. This data report presents our survey methodology, equipment used, analysis, and results.

2. SCOPE OF SERVICES

Our scope of services included:

- Performance of two seismic refraction lines at the project site.
- Compilation and analysis of the data collected.
- Preparation of this data report presenting our results and conclusions.

3. SITE AND PROJECT DESCRIPTION

The project site generally includes a west facing slope located to the east of the existing park, and a relatively small hill (knob) located to the north of the park, across Pasadena Avenue (Figures 1 and 2). Some minor grading has been performed along the west facing slope that has resulted in relatively small cut benches in the hillside. Exposures of volcanic rock were observed in the cuts. The area to the north of the park has also been graded with some relatively minor cuts producing a flat pad atop the knob. Vegetation in the area consists of trees, brush and grass; however, the study areas consisted primarily of grass. Figures 2 and 3 illustrate the study areas and general site conditions.

Based on our discussions with you, we understand that the study areas are under consideration for additional grading in order to enhance the park. Cuts up to roughly 15 feet deep are proposed in the west facing slope area, and cuts up to 30 feet are proposed in the area of the knob.

4. SURVEY METHODOLOGY

A seismic P-wave (compression wave) refraction survey was conducted at the site to evaluate the depth to bedrock and apparent rippability characteristics of the subsurface materials, and to develop a subsurface velocity profile of the areas surveyed. The seismic refraction method uses first-arrival times of refracted seismic waves to estimate the thicknesses and seismic velocities of subsurface layers. Seismic P-waves generated at the surface, using a hammer and plate, are refracted at boundaries separating materials of contrasting velocities. These refracted seismic waves are then detected by a series of surface vertical component geophones and recorded with a 24-channel Geometrics StrataView seismograph. The travel times of the seismic P-waves are used in conjunction with the shot-to-geophone distances to obtain thickness and velocity information on the subsurface materials.

Two seismic lines/profiles (SL-1 and SL-2) were conducted as part of our study. The general locations of the lines were selected by your office, and are depicted on Figure 2. Shot points were conducted at the ends, midpoint and several intermediate points along the lines (Figures 4a and 4b illustrate the shot locations, letters A through E).

The refraction method requires that subsurface velocities increase with depth. A layer having a velocity lower than that of the layer above will not be detectable by the seismic refraction method and, therefore, could lead to errors in the depth calculations of subsequent layers. In addition, lateral variations in velocity, such as those caused by core stones, fracture zones or intrusions can also result in the misinterpretation of the subsurface conditions.

In general, seismic wave velocities can be correlated to material density and/or rock hardness. The relationship between rippability and seismic velocity is empirical and assumes a homogenous mass. Localized areas of differing composition, texture, and/or structure may affect both the measured data and the actual rippability of the mass. The rippability of a mass is also dependent on the excavation equipment used and the skill and experience of the equipment operator.

The rippability values presented in Table 1 are based on our experience with similar materials and assumes that a Caterpillar D-9 dozer ripping with a single shank is used. We emphasize that the cutoffs in this classification scheme are approximate and that rock characteristics, such as fracture spacing and orientation, play a significant role in determining rock rippability. These characteristics may also vary with location and depth.

For trenching operations, the rippability values should be scaled downward. For example, velocities as low as 3,500 feet/second may indicate difficult ripping during trenching operations. In addition, the presence of boulders, which can be troublesome in a narrow trench, should be anticipated.

Table 1 – Rippability Classification

Seismic P-wave Velocity	Rippability
0 to 2,000 feet/second	Easy
2,000 to 4,000 feet/second	Moderate
4,000 to 5,500 feet/second	Difficult, Possible Blasting
5,500 to 7,000 feet/second	Very Difficult, Probable Blasting
Greater than 7,000 feet/second	Blasting Generally Required

It should be noted that the rippability cutoffs presented in Table 1 are slightly more conservative than those published in the Caterpillar Performance Handbook (Caterpillar, 2004). Accordingly, the above classification scheme should be used with discretion, and contractors should not be relieved of making their own independent evaluation of the rippability of the on-site materials prior to submitting their bids.

5. RESULTS

As previously indicated, two seismic traverses were conducted as part of our study. The collected data were processed using SIPwin (Rimrock Geophysics, 2003) a seismic interpretation program and analyzed using both SIPwin and SeisOpt Pro (Optim, 2008). Both programs use first arrival picks and elevation data to produce subsurface velocity models. SIPwin uses layered based modeling techniques to produce layered velocity models, where changes in velocities are depicted as

discrete contacts. SeisOpt Pro uses a nonlinear optimization technique called adaptive simulated annealing. The resulting velocity models provide a tomographic image of the estimated geologic conditions. Both vertical and lateral velocity information is contained in the tomography models. Changes in layer velocity are revealed as gradients rather than discrete contacts, which typically are more representative of actual conditions.

Table 2 lists the approximate P-wave velocities and depths calculated from the seismic refraction traverses conducted during the evaluation. The approximate locations of the seismic refraction traverses are shown on the Seismic Line Location Map (Figure 2). The layer velocity profiles are included in Figures 4a and 4b. It should also be noted that, as a general rule, the effective depth of evaluation for a seismic refraction traverse is approximately one-third to one-fifth the length of the refraction line.

Table 2 – Seismic Traverse Results[1]

Traverse No. And Length	P-wave Velocity feet/second	Approximate Depth to Bottom of Layer in feet	Apparent Rippability[2]
SL-1 125 feet	V1 = 2,750 V2 = 4,140	1 – 5 ---	Moderate Difficult, Possible Blasting
SL-2 150 feet	V1 = 2,115 V2 = 3,445 V3 = 5,380	1 – 2 16 – 30 ---	Moderate Moderate Difficult, Possible Blasting

1 Results based on models generated using SIP, 2003
2 Rippability criteria based on the use of a Caterpillar D-9 dozer ripping with a single shank

6. CONCLUSIONS AND RECOMMENDATIONS

The results from our seismic survey generally revealed three distinct layers/zones at the locations surveyed. Based on our site observations and discussions with you, the layers detected have been interpreted to be surficial soil (colluvium or topsoil) overlying volcanic bedrock with varying degrees of weathering. Figures 4a and 4b provide the velocity models for the areas calculated from both SIPwin and SeisOpt Pro. In general the two models agree, with distinct lateral velocity variations evident in the tomographic profiles. In addition, higher velocity values for the bedrock materials were calculated in the tomography analysis.

As evident in Figure 4b, the tomography model terminates at approximately 13 feet below the ground surface. This is due to the lack of significantly higher velocity materials below this depth, down to the depth explored (roughly 30 to 40 feet). Based on the overall length of the seismic spread, it is expected that the 5,000 to 5,500 feet per second materials depicted in the tomography model extend to the exploration depth (as noted above the effective depth of evaluation for a seismic refraction traverse is generally one-third to one-fifth the length of the refraction line). However, we recommend that an additional evaluation be performed to assess the actual velocity structure down to the proposed excavation depths. Such an evaluation might include additional longer seismic profiles, and/or exploratory borings.

7. LIMITATIONS

The field evaluation and geophysical analyses presented in this report have been conducted in general accordance with current practice and the standard of care exercised by consultants performing similar tasks in the project area. No warranty, expressed or implied, is made regarding the conclusions, recommendations, and opinions presented in this report. There is no evaluation detailed enough to reveal every subsurface condition. Variations may exist and conditions not observed or described in this report may be present. Uncertainties relative to subsurface conditions can be reduced through additional subsurface exploration. Additional subsurface surveying will be performed upon request.

This document is intended to be used only in its entirety. No portion of the document, by itself, is designed to completely represent any aspect of the project described herein. Southwest Geophysics, Inc. should be contacted if the reader requires additional information or has questions regarding the content, interpretations presented, or completeness of this document. This report is intended exclusively for use by the client. Any use or reuse of the findings, conclusions, and/or recommendations of this report by parties other than the client is undertaken at said parties' sole risk.

8. SELECTED REFERENCES

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Optim, Inc., 2008, SeisOpt Pro, V-5.0.

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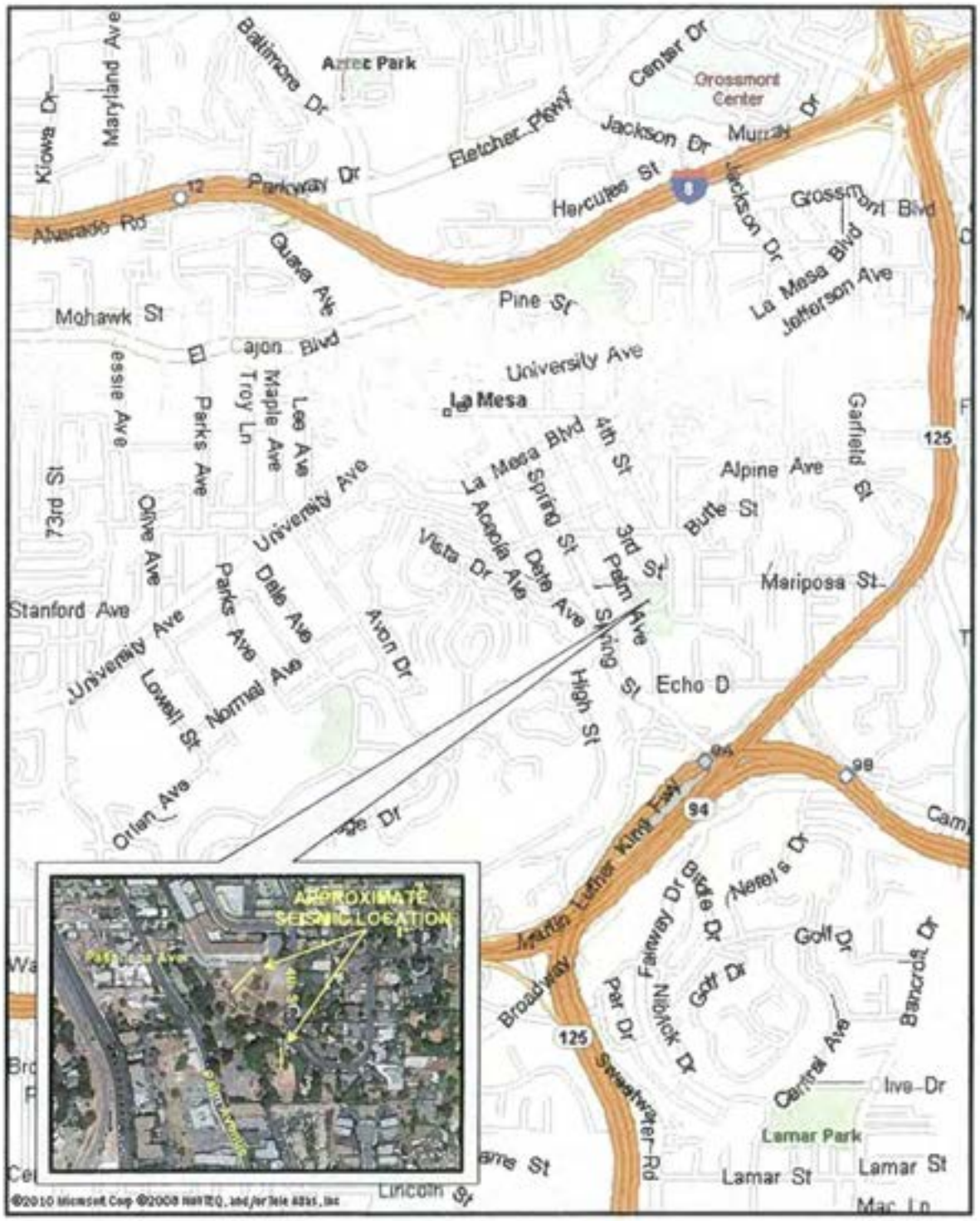
Telford, W.M., Geldart, L.P., Sheriff, R.E., and Keys, D.A., 1976, Applied Geophysics, Cambridge University Press.



SITE LOCATION MAP	N	Collier Park La Mesa, California	SOUTHWEST
		Project No.: 110259 / Date: 10/10	Figure 1

SEISMIC LINE LOCATION MAP

N

Collier Park
La Mesa, California

Project No.: 110259 | Date: 10/10

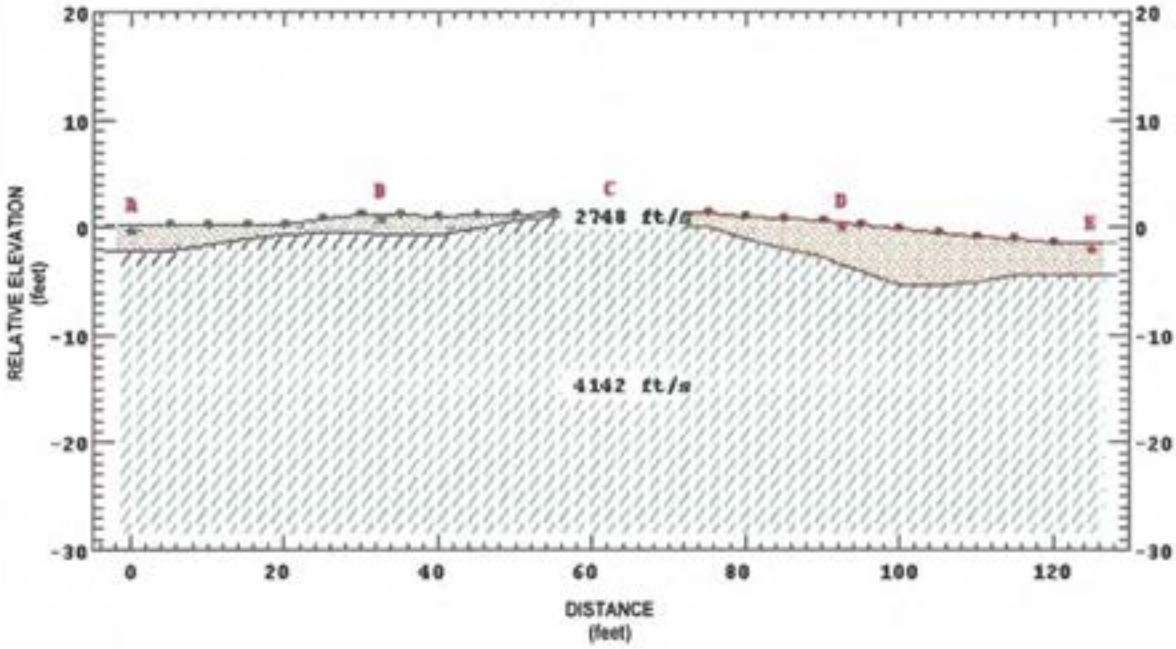
SOUTHWEST

Figure 2

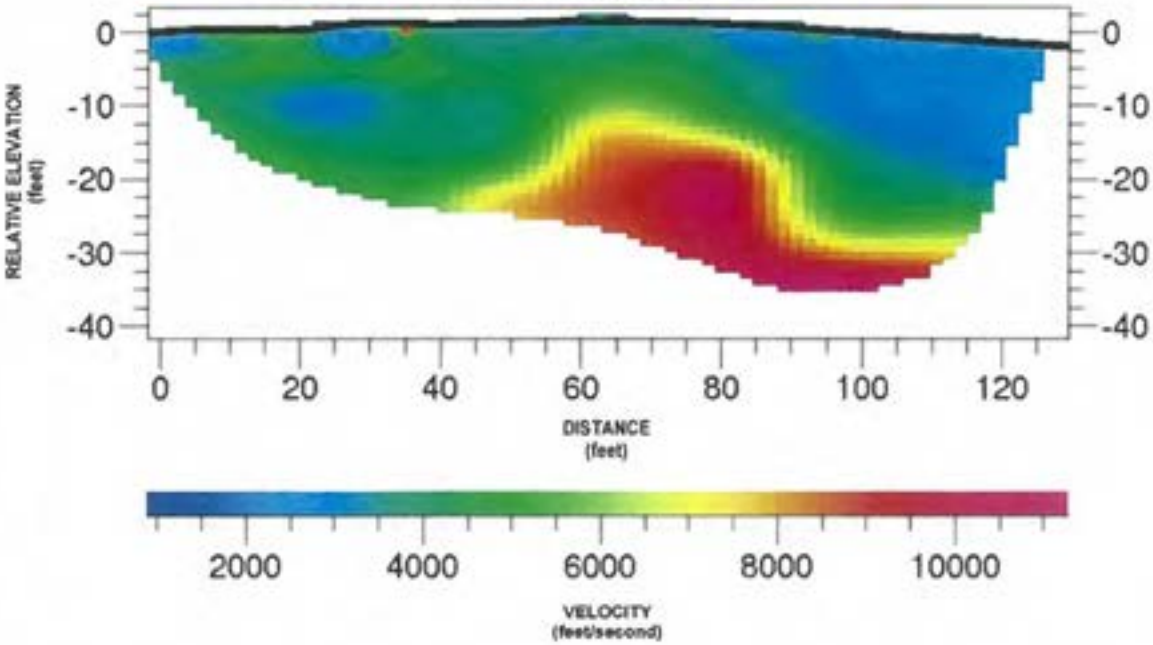
0 100 200
approximate scale in feet

SITE PHOTOGRAPHS	Collier Park La Mesa, California	SOUTHWEST GEOPHYSICS INC
	Project No.: 110259 / Date: 10/10	Figure 3

Layer Model

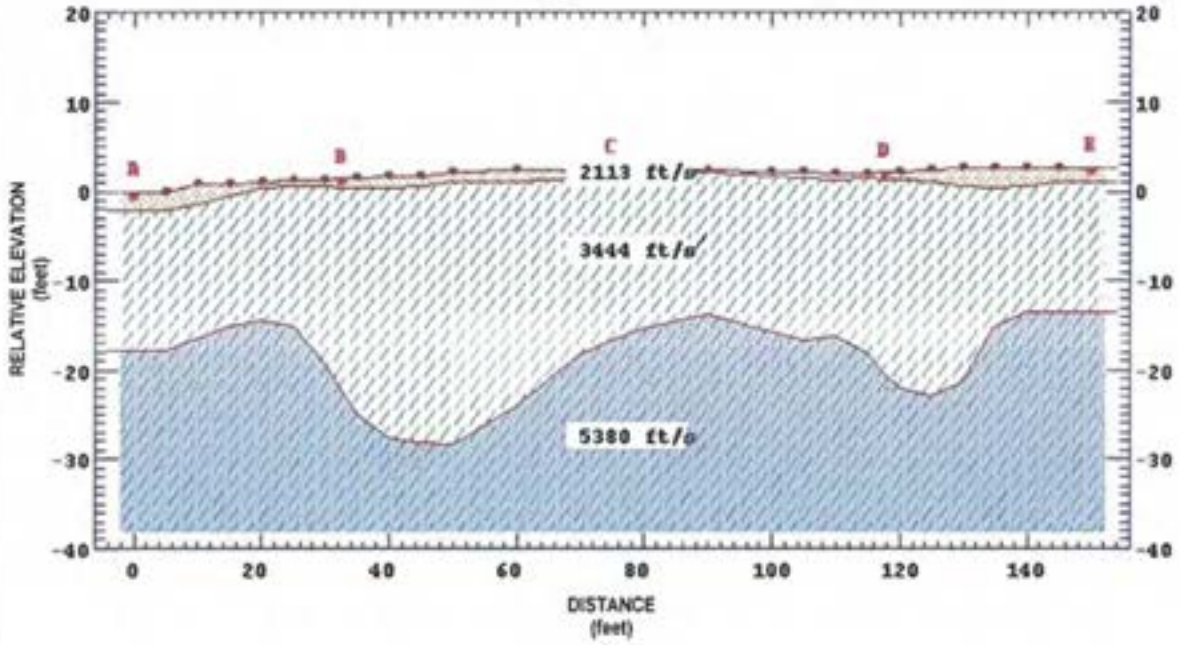


Tomography Model

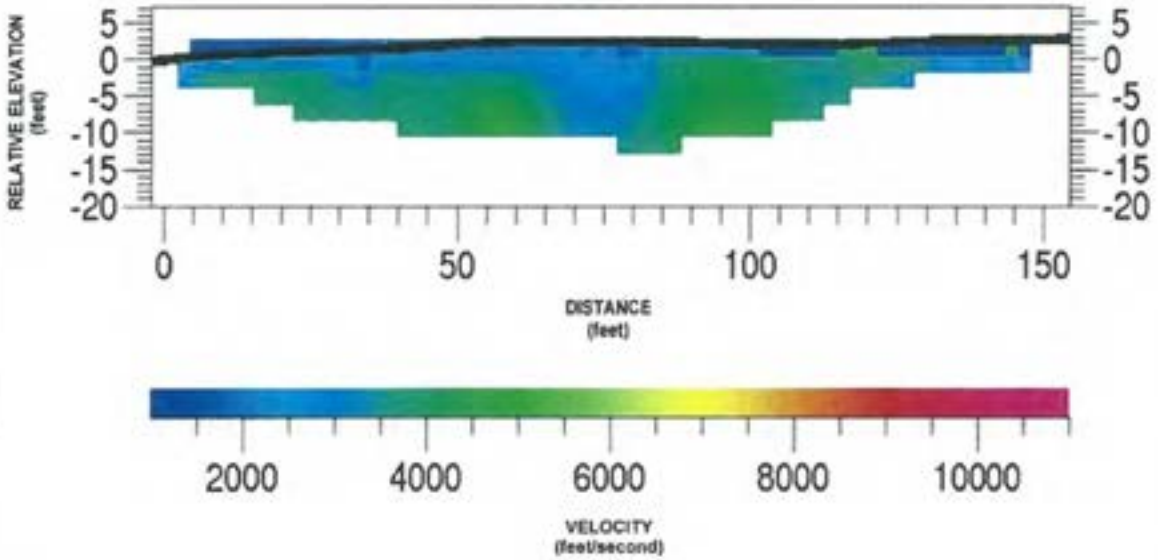


SEISMIC PROFILE SL-1	Collier Park La Mesa, California Project No. 110259 Date: 10/10	SOUTHWEST GEOPHYSICS INC Figure 4a

Layer Model



Tomography Model



SEISMIC PROFILE SL-2	Collier Park La Mesa, California	SOUTHWEST GEOPHYSICS INC.
	Project No.: 110259 / Date: 10/10	Figure 4b

APPENDIX C

APPENDIX C

RECOMMENDED GRADING SPECIFICATIONS

FOR

COLLIER PARK EXPANSION
PASADENA AVENUE AND PALM AVENUE
LA MESA, CALIFORNIA

PROJECT NO. G1222-52-03

RECOMMENDED GRADING SPECIFICATIONS

1. GENERAL

1.1 These Recommended Grading Specifications shall be used in conjunction with the Geotechnical Report for the project prepared by Geocon Incorporated. The recommendations contained in the text of the Geotechnical Report are a part of the earthwork and grading specifications and shall supersede the provisions contained hereinafter in the case of conflict.

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

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

2. DEFINITIONS

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

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

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

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

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

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

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

3. MATERIALS

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

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

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

3.1.3 **Rock fills** are defined as fills containing no rocks or hard lumps larger than 3 feet in maximum dimension and containing little or no fines. Fines are defined as material smaller than ¾ inch in maximum dimension. The quantity of fines shall be less than approximately 20 percent of the rock fill quantity.

3.2 Material of a perishable, spongy, or otherwise unsuitable nature as determined by the Consultant shall not be used in fills.

3.3 Materials used for fill, either imported or on-site, shall not contain hazardous materials as defined by the California Code of Regulations, Title 22, Division 4, Chapter 30, Articles 9 and 10; 40CFR; and any other applicable local, state or federal laws. The Consultant shall not be responsible for the identification or analysis of the potential presence of hazardous materials. However, if observations, odors or soil discoloration cause Consultant to suspect the presence of hazardous materials, the Consultant may request from the Owner the termination of grading operations within the affected area. Prior to resuming grading operations, the Owner shall provide a written report to the Consultant indicating that the suspected materials are not hazardous as defined by applicable laws and regulations.

3.4 The outer 15 feet of *soil-rock* fill slopes, measured horizontally, should be composed of properly compacted *soil* fill materials approved by the Consultant. *Rock* fill may extend to the slope face, provided that the slope is not steeper than 2:1 (horizontal:vertical) and a soil layer no thicker than 12 inches is track-walked onto the face for landscaping purposes. This procedure may be utilized provided it is acceptable to the governing agency, Owner and Consultant.

3.5 Samples of soil materials to be used for fill should be tested in the laboratory by the Consultant to determine the maximum density, optimum moisture content, and, where appropriate, shear strength, expansion, and gradation characteristics of the soil.

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

4. CLEARING AND PREPARING AREAS TO BE FILLED

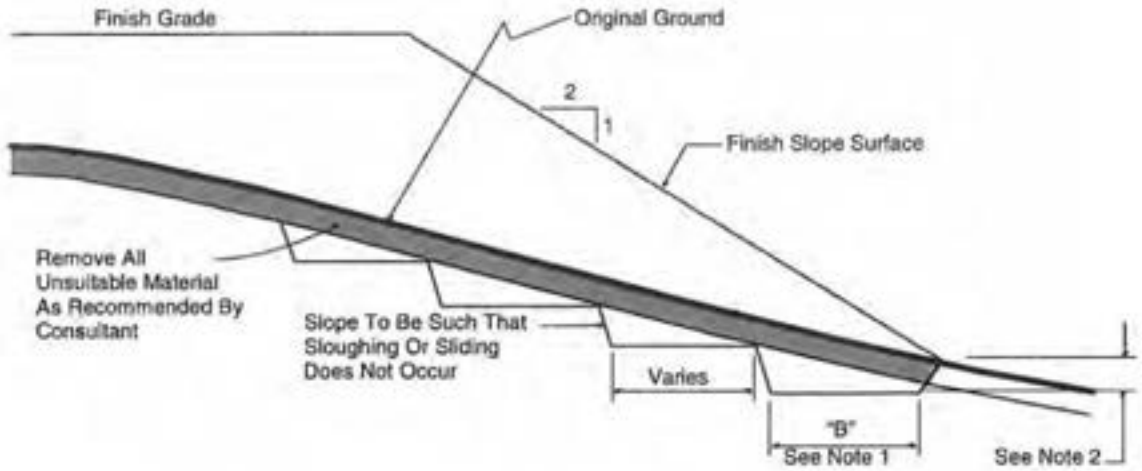
4.1 Areas to be excavated and filled shall be cleared and grubbed. Clearing shall consist of complete removal above the ground surface of trees, stumps, brush, vegetation, man-made structures, and similar debris. Grubbing shall consist of removal of stumps, roots, buried logs and other unsuitable material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

4.2 Any asphalt pavement material removed during clearing operations should be properly disposed at an approved off-site facility. Concrete fragments that are free of reinforcing steel may be placed in fills, provided they are placed in accordance with Section 6.2 or 6.3 of this document.

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

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

TYPICAL BENCHING DETAIL



No Scale

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

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

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

5. COMPACTION EQUIPMENT

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

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

6. PLACING, SPREADING AND COMPACTION OF FILL MATERIAL

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

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

6.1.2 In general, the *soil* fill shall be compacted at a moisture content at or above the optimum moisture content as determined by ASTM D 1557-02.

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 5 feet below finish grade or 3 feet below the deepest utility, whichever is deeper.

6.2.2 Rocks or rock fragments up to 4 feet in maximum dimension may either be individually placed or placed in windrows. Under certain conditions, rocks or rock fragments up to 10 feet in maximum dimension may be placed using similar methods. The acceptability of placing rock materials greater than 4 feet in maximum dimension shall be evaluated during grading as specific cases arise and shall be approved by the Consultant prior to placement.

6.2.3 For individual placement, sufficient space shall be provided between rocks to allow for passage of compaction equipment.

6.2.4 For windrow placement, the rocks should be placed in trenches excavated in properly compacted *soil* fill. Trenches should be approximately 5 feet wide and 4 feet deep in maximum dimension. The voids around and beneath rocks should be filled with approved granular soil having a Sand Equivalent of 30 or greater and should be compacted by flooding. Windrows may also be placed utilizing an "open-face" method in lieu of the trench procedure, however, this method should first be approved by the Consultant.

6.2.5 Windrows should generally be parallel to each other and may be placed either parallel to or perpendicular to the face of the slope depending on the site geometry. The minimum horizontal spacing for windrows shall be 12 feet center-to-center with a 5-foot stagger or offset from lower courses to next overlying course. The minimum vertical spacing between windrow courses shall be 2 feet from the top of a lower windrow to the bottom of the next higher windrow.

6.2.6 Rock placement, fill placement and flooding of approved granular soil in the windrows should be continuously observed by the Consultant.

6.3 *Rock* fills, as defined in Section 3.1.3, shall be placed by the Contractor in accordance with the following recommendations:

6.3.1 The base of the *rock* fill shall be placed on a sloping surface (minimum slope of 2 percent). The surface shall slope toward suitable subdrainage outlet facilities. The *rock* fills shall be provided with subdrains during construction so that a hydrostatic pressure buildup does not develop. The subdrains shall be permanently connected to controlled drainage facilities to control post-construction infiltration of water.

6.3.2 *Rock* fills shall be placed in lifts not exceeding 3 feet. Placement shall be by rock trucks traversing previously placed lifts and dumping at the edge of the currently placed lift. Spreading of the *rock* fill shall be by dozer to facilitate *seating* of the rock. The *rock* fill shall be watered heavily during placement. Watering shall consist of water trucks traversing in front of the current rock lift face and spraying water continuously during rock placement. Compaction equipment with compactive energy comparable to or greater than that of a 20-ton steel vibratory roller or other compaction equipment providing suitable energy to achieve the

required compaction or deflection as recommended in Paragraph 6.3.3 shall be utilized. The number of passes to be made should be determined as described in Paragraph 6.3.3. Once a *rock* fill lift has been covered with *soil* fill, no additional *rock* fill lifts will be permitted over the *soil* fill.

6.3.3 Plate bearing tests, in accordance with ASTM D 1196-93, 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 6938-08A, *Density of Soil and Soil-Aggregate In-Place by Nuclear Methods (Shallow Depth).*

7.6.1.3 Laboratory Compaction Test, ASTM D 1557-02, *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.

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3. California Department of Conservation, Division of Mines and Geology, *Probabilistic Seismic Hazard Assessment for the State of California*, Open File Report 96-08, 1996.

4. California Geological Survey, *Seismic Shaking Hazards in California*, Based on the USGS/CGS Probabilistic Seismic Hazards Assessment (PSHA) Model, 2002 (revised April 2003). 10% probability of being exceeded in 50 years. http://redirect.conservation.ca.gov/cgs/rghm/pshamap/pshamain.html

5. Campbell, K. W., Y. Bozorgnia, *NGA Ground Motion Model for the Geometric Mean Horizontal Component of PGA, PGV, PGD and 5% Damped Linear Elastic Response Spectra for Periods Ranging from 0.01 to 10 s*, Preprint of version submitted for publication in the NGA Special Volume of Earthquake Spectra, Volume 24, Issue 1, pages 139-171, February 2008.

6. Chiou, Brian S. J. and Robert R. Youngs, Robert R, *A NGA Model for the Average Horizontal Component of Peak Ground Motion and Response Spectra*, preprint for article to be published in NGA Special Edition for Earthquake Spectra, Spring 2008.

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8. Risk Engineering, *EZ-FRISK*, 2010.

9. Unpublished reports and maps on file with Geocon Incorporated.

10. USGS computer program, *Seismic Hazard Curves and Uniform Hazard Response Spectra*.