GEOTECHNICAL INVESTIGATION

GREENHILLS RANCH – PHASE 2
SAN DIEGO COUNTY, CALIFORNIA

GEOCON
INCORPORATED
GEOTECHNICAL
CONSULTANTS

PREPARED FOR

GOODMAN IRREVOCABLE FAMILY TRUST
c/o ATLAS INVESTMENTS
LOS ANGELES, CALIFORNIA

JULY 20, 2009
PROJECT NO. G1117-52-01

GEOCON
INCORPORATED

GEOTECHNICAL CONSULTANTS

Project No. G1117-52-01
July 20, 2009

Goodman Irrevocable Family Trust
c/o Atlas Investments
11661 San Vicente Boulevard, Suite 701
Los Angeles, California 90049

Attention: Mr. Steve Goodman

Subject: GREENHILLS RANCH – PHASE 2
SAN DIEGO COUNTY, CALIFORNIA
GEOTECHNICAL INVESTIGATION

Dear Mr. Goodman:

In accordance with your authorization of our Proposal No. LG-09128, dated May 20, 2009, we herein submit the results of our geotechnical investigation for the subject site. The accompanying report presents the results of our study and our conclusions and recommendations pertaining to the geotechnical aspects of the proposed development. The study also includes an evaluation of the geologic units, geologic hazards, and a discussion on rock rippability. The site is considered suitable for development provided the recommendations of this report are followed.

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

Very truly yours,

GEOCON INCORPORATED

John Hoobs
CEG 1524

PROFESSIONAL GEOLOGIST JOHN HOOBS No. 1524 CERTIFIED ENGINEERING GEOLOGIST STATE OF CALIFORNIA

Shawn Weedon
GE 2714

REGISTERED PROFESSIONAL ENGINEER SHAWN WEEDON No. 2714 Exp.06/30/11 GEOTECHNICAL STATE OF CALIFORNIA

JH:SW:sc

(4) Addressee
(5) Grabhorn Engineering Corporation
Attention: Mr. Joel Paulson

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GEOTECHNICAL INVESTIGATION

1. PURPOSE AND SCOPE

This report presents the results of our geotechnical investigation for the proposed Greenhills Ranch – Phase 2 development located in San Diego County, California (see Vicinity Map, Figure 1). The purpose of the investigation is to evaluate subsurface soil and geologic conditions at the site and, based on the conditions encountered, provide preliminary recommendations pertaining to the geotechnical aspects of developing the property. The proposed development will include the construction of single family residential homes and related infrastructure improvements. Plans for development as presently proposed are presented on Figure 2, Geologic Map (map pocket).

The scope of our investigation included geologic mapping, subsurface exploration, laboratory testing, engineering analyses, and the preparation of this report. As a part of our investigation, we have reviewed aerial stereo photographs, geologic maps, published geologic reports, and previous geotechnical reports related to the property. A summary of the background information reviewed for this study is presented in the *List of References*.

The field investigation performed for this report included geologic mapping and the excavation of 14 exploratory excavator trenches and two seismic refraction survey lines. A discussion of the field investigation and logs of the exploratory excavator trenches are presented in Appendix A. The results of the seismic refraction surveys prepared by Southwest Geophysics are presented in Appendix B. The approximate locations of the exploratory excavations and seismic surveys performed within the current development limits are presented on the Geologic Map, Figure 2. Appendix C presents a rippability summary of the existing materials based on the field data. We performed laboratory tests on soil samples obtained from the exploratory excavations to evaluate pertinent physical and chemical properties for engineering analysis. The results of the laboratory testing are presented in Appendix D.

Grabhorn Engineering Corporation provided the topographic information and proposed grading and development plans used during our field investigation to assist in preparing 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 Greenhills Ranch – Phase 2 project is located in the community of Lakeside in the County of San Diego, California. The property is located west of Lake Jennings Park Road, northwest of Audubon Road and north of the northern terminus of Adlai Road. The Helix Water District's treatment plant and

Lake Jennings is located to the northeast of the property. Residential homes are located to the south and open space is located to the north.

The roughly 59-acre property consists of a central ridge that slopes to drainages flowing to the north and south. Site elevations range from approximately 660 feet MSL adjacent to Adlai Road on the southern portion of the property to approximately 782 feet above mean sea level (MSL) at the apex of the ridge. Proposed pad elevations range from 664 feet MSL to 773 feet MSL. The site is not within the downstream drainage path of Lake Jennings which has a dam height of about 700 feet MSL. Vegetation consists of native scrub and grassland locally disturbed by existing use. The site is currently occupied by a single family residence serviced by a septic system on the western portion of the property and an active horse training facility that has several paddocks, corrals and storage sheds. A second single-family residence is located on proposed Lots 14 and 15 and is also planned to be demolished. Three existing residential homes located on the northwest side of Audubon Road will remain in place with planned improvements to their driveways. An existing SDG&E easement is located on the eastern portion of the site with numerous overhead power lines. A 48-inch Helix Water District waterline is located within Audubon Road along with additional underground and overhead utilities. In addition, a well house is located on proposed Lot 8 that will require destruction.

The proposed development includes constructing 60 single-family detached residential homes with a minimum lot size of about 6,000 square feet. Audubon Road will be realigned and will loop around the development to provide vehicular circulation. In addition, connection to Lake Jennings Park Road via Greenhills Way will provide eastern access to the project. Maximum cut and fill depths will be on the order of 35 and 30 feet, respectively. Proposed cut and fill slopes will be constructed at inclinations of 2:1 (horizontal:vertical) with a maximum height of about 50 and 60 feet, respectively. Several rear and side yard and toe and top of slope retaining walls with a maximum height of about 18 feet are proposed. We expect the walls would consist of concrete block walls and/or mechanically stabilized earth walls.

The locations and descriptions herein are based on a site reconnaissance, review of the referenced plans, and project information provided by the client and civil engineer. If development plans should change significantly, a revision of this report will be required to provide updated recommendations.

3. GEOLOGIC MATERIALS

We encountered three surficial soil types and two geologic formations during our investigation. The surficial soil units consist of undocumented fill, topsoil, and colluvium. The geologic formations are granitic and metamorphic rocks that have been subjected to various degrees of weathering and range in age from Cretaceous to Triassic. The formational units include the Cretaceous-age Granitic Rock (Kgr) and Triassic-age Julian Schist (Trm). The California Geological Survey's El Cajon Quadrangle geologic map indicates the geology as Cretaceous-age metavolcanic rock. However, based on our observations during trenching, this unit more closely resembles the quartzite schists of the Julian Schist. The formational and surficial units are discussed herein in order of increasing age. The approximate lateral extent of the surficial soil and formational materials is presented on the Geologic Map, Figure 2 (map pocket). Figure 3 presents a Geologic Cross-Section depicting the approximate subsurface relationships between the geologic units.

3.1 Undocumented Fill (Qudf)

Undocumented fill has been placed at several locations associated with the horse facility and an access road on the east side of the property. In general, the undocumented fill consists of loose to medium dense, dry to damp, silty, fine to medium sand with cobble and boulders to about 2½-feet in diameter and locally containing trash and roots. The undocumented fill has a "very low" expansion potential and is considered compressible. In its present condition, the undocumented fill is not suitable for support of additional fill or structures, and remedial grading will be necessary. The undocumented fill soil is generally suitable for reuse as compacted fill provided it is substantially free of debris and trash. Stockpiles of horse manure located along the north side of the property and localized areas of manure mixed within the undocumented fill will not be suitable for use as compacted fill and will require export from the site. The undocumented fill is suitable for use as capping material if the oversize material is screened.

3.2 Topsoil (unmapped)

Holocene-age topsoil is present as a thin veneer locally overlying formational materials across the site. The topsoil has a maximum thickness of approximately 5 feet and can be characterized as loose to medium dense, dry to damp, reddish brown, silty, fine to medium sand with cobble and boulders up to approximately 2-feet in diameter. The topsoil has a "very low" expansion potential and is considered compressible. Topsoil is considered unsuitable for support of compacted fill and structural loads in its present condition and will require remedial grading. Topsoil is suitable for use as compacted fill and as capping material if the oversize material is screened. Due to the relatively thin thickness, topsoil is not shown on the geologic map.

3.3 Colluvium (Qc)

Holocene-age colluvium, derived from weathering of the underlying bedrock materials at higher elevations and deposited by gravity and sheet-flow, is present on the side slopes and the upper portions of the drainages. The colluvium can be characterized as loose, dry to damp, reddish brown, silty, fine to medium sand with varying amounts of gravel and cobble to 1½-foot diameter. The thickness of colluvium generally ranges from approximately 5 to 7 feet, but may be thicker along the lower portions of canyons and natural slopes. The colluvium is considered unsuitable for support of compacted fill and structural loads in its present condition and will require remedial grading. Colluvium is suitable for use as compacted fill soil and as capping material if the oversize material is screened.

3.4 Granitic Rock (Kgr)

Cretaceous-aged Granitic Rock is located on the western portion of the site and is characterized as moderately strong to very strong, moderately weathered to fresh, yellow to greenish gray, fine to medium grained rock. Minor widely spaced joints were observed in a general northeast direction. This unit will be very difficult to excavate and significant portions will be non-rippable. The majority of the cuts within this unit may require blasting to excavate to the proposed grades and undercut elevations. Excavation will result in the generation of oversize material requiring special placement techniques and fill placement restrictions or crushing. The granitic rock is suitable for the support of compacted fill and structural loads. A detailed excavation discussion of the rock units is presented in the *Rock Rippability* section of this report. Placement of oversize rock materials is discussed in the *Conclusions and Recommendations* section of this report.

3.5 Julian Schist (Trm)

Triassic-age Julian Schist composed of metamorphosed quartizite and pelitic schist rock is present in the central and eastern portions of the site. The metamorphic rock is generally moderately strong to very strong, moderately to slightly weathered with some fractures and joints. This unit will likely be very difficult to excavate and the majority of the cuts within this unit may require blasting to excavate to the proposed grades and undercut elevations. Excavation will result in the generation of oversized material. The metamorphic rock is suitable for the support of compacted fill and structural loads.

4. GEOLOGIC STRUCTURE

The geologic structure within the rock units is characterized as a hard rock mass displaying a relatively consistent, northeast-southwest trending foliation and joint pattern with dips generally ranging between 75 degrees to near vertical to the southeast with the schist and dipping to the northwest in the granitic rock. The dominant structural feature within the rock mass is jointing. Joints are surfaces, fractures or partings within a rock mass that do not show evidence of displacement. Jointing within the rock mass was formed as a result of regional tectonic stresses. The joints are generally moderately to poorly developed and are typically spaced 1 to 3 feet apart. Geologic structure within the hard rock units can be variable and should be individually evaluated for each proposed cut slope during grading.

5. GROUNDWATER

We did not encounter groundwater or 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 could develop 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.

6. GEOLOGIC HAZARDS

6.1 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 a State of California Earthquake Fault Zone.

According to the computer program *EZ-FRISK* (Version 7.31), 10 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 18 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 0.19g, respectively. Table 6.1.1 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 6.1.1
DETERMINISTIC SEISMIC 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	18	7.2	0.19	0.14	0.17
Elsinore-Julian	25	7.5	0.18	0.12	0.15
Earthquake Valley	29	6.9	0.13	0.09	0.09
Coronado Bank	31	7.7	0.17	0.11	0.15
Elsinore-Coyote Mountain	32	7.2	0.14	0.09	0.10
Elsinore-Temecula	37	7.2	0.13	0.08	0.09
Newport-Inglewood Offshore	37	7.2	0.12	0.08	0.09
San Jacinto-Coyote Creek	46	7.2	0.10	0.07	0.07
San Jacinto-Borrego	47	7.0	0.09	0.06	0.06
San Jacinto-Anza	48	7.6	0.12	0.08	0.09

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 6.1.2 presents the site-specific probabilistic seismic hazard parameters including acceleration-attenuation relationships and the probability of exceedence.

TABLE 6.1.2
PROBABILISTIC SEISMIC HAZARD PARAMETERS

Probability of Exceedence	Peak Ground Acceleration		
	Boore-Atkinson, 2007 (g)	Campbell-Bozorgnia, 2008 (g)	Chiou-Youngs, 2008 (g)
2% in a 50 Year Period	0.50	0.40	0.51
5% in a 50 Year Period	0.40	0.31	0.38
10% in a 50 Year Period	0.32	0.25	0.30

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 6.1.3 presents the calculated results from the *Probabilistic Seismic Hazards Mapping Ground Motion Page* from the CGS website.

TABLE 6.1.3
PROBABILISTIC SITE PARAMETERS FOR SELECTED FAULTS
CALIFORNIA GEOLOGIC SURVEY

Calculated Acceleration (g) Firm Rock	Calculated Acceleration (g) Soft Rock	Calculated Acceleration (g) Alluvium
0.23	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) and other currently adopted county of San Diego codes.

6.2 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 the earth surface. The potential for ground rupture is considered to be negligible due to the absence of active faults at the subject site.

6.3 Liquefaction and Seismically Induced Settlement

Liquefaction typically occurs when a site is located in a zone with seismic activity, on-site soils are cohesionless silts or clays with low plasticity, groundwater is encountered within 50 feet of the surface, and soil relative densities are less than about 70 percent. If the four previous criteria are met, a seismic event could result in a rapid pore-water pressure increase from the earthquake-generated ground accelerations. Seismically induced settlement may occur whether the potential for liquefaction exists or not. The potential for liquefaction and seismically induced settlement occurring at the site is considered to be very low due to the lack of near-surface permanent groundwater within 50 feet from proposed grade and the dense nature of the proposed compacted fill and formational rock materials.

6.4 Expansive Soil

The majority of the geologic units will likely possess a "very low" expansion potential (Expansion Index of 20 or less). However, some clayey portions within the surficial units may exhibit a "very low" to "low" expansive potential (Expansion Index of 50 or less). We do not expect special grading requirements due to expansive soil.

6.5 Landslides

Examination of aerial stereo photographs in our files, our geologic reconnaissance, and review of available geotechnical and geologic reports for the site vicinity indicate that landslides are not present at the property or at a location that could impact the subject site.

6.6 Slope Stability

We reviewed the proposed slope configurations, as depicted on the Geologic Map, to evaluate both surficial and global stability based on the current geologic information. The portions of the site planned for development are generally underlain by Quaternary-age surficial soil and Cretaceous and Triassic-age granitic and metamorphic rock. The stability of graded cut slopes composed of bedrock is highly dependent on the degree of weathering and the geologic structure of the slope face.

In general, it is our opinion that permanent, graded fill slopes or cut slopes at the site with gradients of 2:1 (horizontal to vertical) or flatter would possess calculated Factors of Safety of 1.5 or greater. The majority of bedrock cut slopes should be comprised of *good quality* (from Hoek and Bray, 1981), moderately strong to very strong granitic and metamorphic rock. Based on the results of our slope stability analyses, cut slopes composed of moderately to slightly weathered granitic and metamorphic rock should possess Factors of Safety of 1.5 or greater against large-scale, deep-seated slope failures at their proposed slope inclinations. Graded fill slopes constructed of granular soil derived from on site topsoil, undocumented fill or the granular portions of the formational rock materials should possess Factors of Safety of 1.5 or greater against deep-seated and surficial slope failures at their proposed slope inclinations. Figures 4, 5, and 6 present the slope stability analysis for cut and fill slopes and surficial slope stability analysis, respectively.

Because of the potential presence of adverse geologic structures, the geologic structure of permanent and temporary slopes composed of bedrock material should be analyzed in the field in detail by the geotechnical engineer and/or engineering geologist during the grading operations. Additional recommendations for slope stabilization may be necessary if adverse geologic structure is encountered. Grading of cut and fill slopes should be designed in accordance with the requirements of the County of San Diego building codes or the 2007 California Building Code (CBC).

6.7 Rock Fall Hazard

The proposed grading of the property will remove the natural slopes on the site. The hard rock slopes located below the proposed development do not have inclinations that would create a rock fall hazard affecting the site or adjacent properties. Therefore, the risk of rock fall hazard affecting the site is considered very low.

6.8 Tsunamis and Seiches

A tsunami is a series of long period waves generated in the ocean by a sudden displacement of large volumes of water. Causes of tsunamis include underwater earthquakes, volcanic eruptions, or offshore slope failures. The first order driving force for locally generated tsunamis offshore southern California is expected to be tectonic deformation from large earthquakes (Legg, *et al.*, 2002). The County of San Diego Hazard Mitigation Plan (2004) maps zones of high risk for tsunami run-up for coastal areas throughout the county. The site is not included within one of these high risk hazard areas. The site is approximately 20 miles from the Pacific Coast and ranges between approximately 650 feet and 784 feet above MSL. Therefore, we consider the risk associated with tsunamis to be negligible.

A seiche is a run-up of water within a lake or embayment triggered by fault- or landslide-induced ground displacement. The site is not located down stream of Lake Jennings. It is our opinion that the proposed site elevations are sufficient to mitigate the risk of a seiche. Therefore, the potential of seiches affecting the site is considered very low.

7. ROCK RIPPABILITY

7.1 Seismic Refraction Surveys

Southwest Geophysics performed a seismic refraction survey to evaluate the bedrock rippability along two seismic lines. The locations of the seismic traverses are presented on the Geologic Map, Figure 2, and the 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 7.1.1. 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 with a single ripper tooth bucket. It has been our experience that it can often be more cost effective to blast marginally rippable bedrock.

TABLE 7.1.1
SUMMARY OF ESTIMATED RIPPABILITY FROM SEISMIC REFRACTION

Excavation Method	Seismic Velocity (ft/s)	Estimated Rippability
Mass Grading	Less than 5,000	Rippable
	5,000 to 6,000	Marginal Ripping (Possible Blasting)
	Greater than 6,000	Non-Rippable (Pre-Blasting Required)
Trenching	Less than 3,800	Rippable
	3,800 to 4,300	Marginal Ripping
	Greater than 4,300	Non-Rippable

The results of the seismic refraction surveys indicate that velocities less than approximately 3,000 ft/s are likely associated with surficial soil. Velocities between 3,000 and 5,000 ft/s are likely associated with moderately weathered bedrock. Velocities between 5,000 and 7,000 ft/s are likely associated with slightly weathered bedrock, with higher velocities associated with non-weathered bedrock. Rippability is highly dependent upon the degree of weathering, fracturing, and jointing within bedrock and the rippability of the various soil and rock units is, correspondingly, variable. Appendix C presents a summary of the seismic refraction surveys indicating rippable and non-rippable depths estimated using approximately 5,000 ft/s as a cut-off between rippable and marginally to non-rippable for the corresponding geologic units during grading operations. The results of the seismic refraction surveys indicate a highly variable velocity in relation to depth as indicated on the tomography seismic profiles. This indicates that the rock materials are affected by weathering, jointing and fractures that give a variable depth and direction of rippability.

We performed exploratory trenches and seismic traverses in the rock units located generally in proposed cut areas, to evaluate rock rippability characteristics. The rippability of the rock units is variable, and generally limited to the depth of the weathered mantle. Proposed excavations within the rock may require blasting as excavations extend beyond the rippable weathered mantle. Based on a review of the excavator trenches and seismic shear wave velocities of 5,000 ft/s, the thickness of the rippable granitic and metamorphic rock mantle is estimated to vary from 2 to 15 feet. Estimates for the rippable depth at each of the exploratory excavation and seismic line locations are presented on Table C-I in Appendix C and on the Geologic Map, Figure 2.

Non-economic ripping and/or blasting should also be expected in areas of concentrated rock outcroppings located at the surface. Estimates of the expected volume of hard rock materials generated from proposed excavations should be evaluated by the contractor based on the information from each excavation and seismic traverse location. Roadway/utility corridor and lot undercutting criteria should also be considered when calculating the volume of hard rock. Proposed cuts in hard rock areas can be expected to generate oversized material (rocks greater than 12 inches in dimension) which will necessitate typical hard rock handling and placement procedures during grading operations.

The grading contractor is responsible for performing their own independent rippability analysis based on their experience on grading and blasting rock materials and the type of equipment used. The data presented in this report can be used in this analysis, however the information on the depth of rippable rock and capping material provided herein should not be used for bidding purposes. The analysis should include calculations on the amount of soil and rock materials generated during grading to evaluate if sufficient quantities of capping material exist for building pads, roadways and slope zones and if sufficient areas exist for the placement of blasted rock materials taking into account the bulk and shrink of the existing materials. Based on this grading analysis, grade adjustment may reduce the total cost of grading the site. If insufficient capping material exists on the site, then crushing of rock materials may be necessary to meet the capping specifications.

7.2 Capping Material

Capping material refers to select material placed within 3 feet from building pad grade, 8 feet from roadway grade, and at least 2 feet below the deepest utility within roadways consisting of "soil" fill with an approximate maximum particle dimension of 6 inches and a minimum of 40 percent soil passing the ¾ inch sieve and, in the upper 3 feet of pad grade, 20 percent soil passing the No. 4 screen. In general, capping material can be readily obtained from the surficial units. However, screening will be necessary to remove cobble and rock fragments larger than 6 inches in diameter within portions of the surficial units and the highly weathered portions of the rock materials. The availability of capping material easily obtained from the rock units is considerably less and screening and possible crushing operations may be necessary to generate sufficient capping material. Estimates for the thicknesses of capping material at each of the exploratory excavation and seismic line locations are presented in Appendix C and on the Geologic Map, Figure 2.

8. CONCLUSIONS AND RECOMMENDATIONS

8.1 General

8.1.1 It is our opinion that no soil or geologic conditions were encountered during the investigation that would preclude the proposed development of Greenhills Ranch – Phase 2 provided the recommendations presented herein are followed and implemented during construction.

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

8.1.3 The existing onsite surficial soil units including undocumented fill, topsoil, and colluvium materials are potentially compressible and unsuitable in their present condition for the support of compacted fill or settlement-sensitive improvements. Remedial grading of the unsuitable surficial soil units will be required and recommendations for remedial grading are provided herein. The rock units are suitable for the support of compacted fill and structural loads.

8.1.4 We did not encounter groundwater during our subsurface exploration and we do not expect it to be a constraint to project development. However, seepage may be encountered during the grading operations during rainy seasons.

8.1.5 We expect the rippability of the surficial units will range between easy and moderate. These units will generate some oversized material during grading and some screening and/or crushing should be expected. The rippability of the rock units is variable and ranges between moderate to very difficult. Rock breaking and blasting should be expected during grading within the bedrock units.

8.1.6 In general, cut slopes composed of hard rock should possess Factors of Safety greater than 1.5 at inclinations of 2:1 (horizontal to vertical), or flatter. The geologic structure of cut slopes composed of hard rock should be evaluated during grading operations by the geotechnical consultant. The planned proposed fill slopes should possess Factors of Safety greater than 1.5 at inclinations of 2:1 (horizontal to vertical), or flatter.

8.1.7 The proposed structures and site retaining walls may be supported on conventional foundations bearing in either competent formational materials or engineered fill. Geocon Incorporated should evaluate the foundation systems when the locations of these structures have been finalized. Transitioning foundations and slabs from bedrock to engineered fill should not occur. Where engineered fill will be utilized for foundation support, foundations should be underlain by properly compacted fill. Bedrock over-excavations will be required where engineered fill is to be utilized for foundation support. General recommendations for the design of shallow foundations are provided herein.

8.1.8 Due to the existence of hard rock at or near the proposed grades at many locations throughout the site, the building pads, streets, and utility corridors underlain by hard rock should be overexcavated to facilitate the excavation of footings, subgrade, and utility trenches. Recommendations for overexcavation operations are provided herein.

8.1.9 Canyon subdrains will not be required on this project due to the lack of well developed drainages within the limits of grading. However, if a previously installed canyon subdrain is encountered within the undocumented fill on the south side of the project, then the subdrain will need to be maintained prior to the placement of compacted fill.

8.1.10 Proper drainage should be maintained in order to preserve the engineered properties of the fill. Recommendations for site drainage are provided herein.

8.2 Soil Characteristics

8.2.1 The soil encountered in the field investigation is generally considered to be "non-expansive" (Expansion Index [EI] of 20 or less) as defined by 2007 California Building Code (CBC) Section 1802.3.2. Table 8.2 presents soil classifications based on the expansion index. Based on laboratory tests of samples we obtained during the trenching operations, we expect the on-site material possesses a "very low" expansion potential (Expansion Index of 20 or less). Additional testing for expansion potential should be performed once final grades are achieved.

TABLE 8.2
SOIL CLASSIFICATION BASED ON EXPANSION INDEX

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

8.2.2 We performed laboratory tests on samples of the site materials to evaluate the percentage of water-soluble sulfate content. Results from the laboratory water-soluble sulfate content tests are presented in Appendix D and indicate that the on-site materials at the locations tested possess "negligible" sulfate exposure to concrete structures as defined by 2007 CBC Section 1904.3 and ACI 318. The presence of water-soluble sulfates is not a visually discernible characteristic; therefore, other soil samples from the site could yield different concentrations. Additionally, over time landscaping activities (i.e., addition of fertilizers and other soil nutrients) may affect the concentration.

8.2.3 We performed laboratory tests on a sample of the site materials encountered to check the corrosion potential to subsurface metal structures and reinforcing steel. The laboratory corrosion test results are presented in Appendix D.

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

8.3 Seismic Design Criteria

8.3.1 We used the computer program *Seismic Hazard Curves and Uniform Hazard Response Spectra,* provided by the USGS to calculate the seismic design criteria. Table 8.3 summarizes site-specific design criteria obtained from the 2007 CBC, Chapter 16 *Structural Design,* and Section 1613 *Earthquake Loads.* The short spectral response has a period of 0.2 second.

TABLE 8.3
2007 CBC SEISMIC DESIGN PARAMETERS

Parameter	Value		IBC-06 Reference
Site Class	C	D	Table 1613.5.2
Fill Thickness, T	T<20 Feet	T≥20 Feet	--
Spectral Response – Class B (short), S_S	1.007g	1.007g	Figure 1613.5(3)
Spectral Response – Class B (1 sec), S_1	0.344g	0.344g	Figure 1613.5(4)
Site Coefficient, F_a	1.000	1.097	Table 1613.5.3(1)
Site Coefficient, F_v	1.456	1.713	Table 1613.5.3(2)
Maximum Considered Earthquake Spectral Response Acceleration (short), S_{MS}	1.007g	1.104g	Section 1613.5.3 (Eqn 16-37)
Maximum Considered Earthquake Spectral Response Acceleration (1 sec), S_{M1}	0.500g	0.589g	Section 1613.5.3 (Eqn 16-38)
5% Damped Design Spectral Response Acceleration (short), S_{DS}	0.671g	0.736g	Section 1613.5.4 (Eqn 16-39)
5% Damped Design Spectral Response Acceleration (1 sec), S_{D1}	0.334g	0.392g	Section 1613.5.4 (Eqn 16-40)

8.3.2 Conformance to the criteria in Table 8.3 for seismic design does not constitute any kind of guarantee or assurance that significant structural damage or ground failure will not occur if a large earthquake occurs. The primary goal of seismic design is to protect life, not to avoid all damage, since such design may be economically prohibitive.

8.4 Grading

8.4.1 Grading should be performed in accordance with the *Recommended Grading Specifications* contained in Appendix E and the County of San Diego Grading Ordinance.

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

8.4.3 Site preparation should begin with the removal of deleterious material, debris and vegetation. The depth of removal should be such that material exposed in cut areas or soil to be used as fill is relatively free of organic matter and debris. Material generated during stripping and/or site demolition should be exported from the site. The undocumented fill will require removal of the trash and some portions of the undocumented fill will not be suitable for reuse as fill soil due to high concentrations of horse manure.

8.4.4 Abandoned foundations, septic systems and buried utilities should be removed and the resultant depressions and/or trenches should be filled as necessary with properly compacted material as part of the remedial grading. In addition, the water well located on proposed Lot 8 should be properly destroyed in accordance with the requirements of the county health department.

8.4.5 Topsoil, colluvium, undocumented fill, and highly weathered or decomposed rock within the site boundary should be removed to expose firm formational materials. The estimated removal depths are depicted on the Geologic Map, Figure 2. The actual depth of removal should be evaluated by the geotechnical engineering consultant during the grading operations. The bottom of the excavations should be scarified, moisture conditioned as necessary, and properly compacted.

8.4.6 The geotechnical engineering consultant should observe the removal bottoms to check that a suitable base has been achieved prior to the placement of compacted fill.

8.4.7 The site should be brought to final subgrade elevations with fill placed and compacted in layers. The layers of fill should be no thicker than will allow for adequate bonding and compaction. Fill, including backfill and scarified ground surfaces, should be compacted to a dry density of at least 90 percent of the laboratory maximum dry density near to slightly above optimum moisture content in accordance with ASTM D 1557. Fill materials placed below optimum moisture content may require additional moisture conditioning prior to placing additional fill.

8.4.8 To reduce the potential for differential settlement, the building pads with cut-fill transitions should be undercut at least 3 feet, sloped 1 percent to the adjacent street or deepest fill, and replaced with properly compacted "very low" to "low" expansive fill soil. Where the thickness of the fill below the building pad exceeds 15 feet, the depth of the undercut should be increased to one-fifth of the maximum fill thickness.

8.4.9 Building pads underlain by hard rock units at grade should also be undercut to facilitate future trenching operations for foundations and utilities. Building pads should be undercut a minimum of 3 feet and replaced with properly compacted fill and undercuts should be sloped a minimum of 1 percent and drain toward the adjacent on-site streets.

8.4.10 Roadways 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. Figure 7 presents a typical detail for the overexcavation of street sections.

8.4.11 Recommendations for the handling and disposal of oversized rock are presented in Figure 8 and in Appendix E. In general, structural fill placed and compacted at the site should consist of material that can be classified into four zones:

Zone A: Material placed within 3 feet from building pad grade, 8 feet from roadway grade, and to at least 1 foot below the deepest utility within roadways consisting of "soil" fill with an approximate maximum particle dimension of 6 inches and a minimum of 40 percent soil passing the ¾ inch sieve and, in the upper 3 feet of pad grade, a minimum of 20 percent passing the No. 4 sieve.

Zone B: Material placed below 8 feet from grade (below *Zone A* and *C*) may consist of "rock" fill or "soil/rock" fill (as defined in Appendix E). Blasted rock should generally consist of 2 foot minus rock material with occasional rock up to 4 foot in maximum dimension. Alternatively, "soil" fill may be placed in Zone B containing rock with a maximum dimension of 2 feet, placed in windrows. Rocks up to 4 feet in maximum dimension can be placed in a properly compacted soil matrix with rocks separated at least 12 feet apart.

Zone C: Within 3 to 8 feet of pad grade and between 5 and 15 feet from face of slope, fill material should consist of "soil" fill with an approximate maximum particle dimension of 1 foot. Rocks up to 2 feet in maximum dimension may be placed, provided they are distributed in a matrix of compacted "soil" fill.

Zone D: Within the outer 5 feet of fill slopes, the fill should consist of rock up to 1 foot in maximum dimension in a matrix of compacted "soil" fill.

8.4.12 Import fill (if necessary) should consist of granular materials with a "very low" to "low" expansion potential (EI of 50 or less) generally free of deleterious material and rock fragments larger than 6 inches and should be compacted as recommended herein. Geocon Incorporated should be notified of the import soil source and should perform laboratory testing of import soil prior to its arrival at the site to evaluate its suitability as fill material.

8.4.13 Cut slopes exposing rock with unfavorable geologic structure may require stability fills. The backcut for the stability fills should commence at least 10 feet from the top of the proposed finish-graded slope and should extend at least 3 feet into formational material below the lowest adjacent pad grade.

8.4.14 Cut slope excavations should be observed during grading operations to check that geologic conditions do not differ significantly from those expected.

8.4.15 The outer 5 feet of fill slopes should be composed of properly compacted granular "soil" fill to reduce the potential for surficial sloughing. In general, soil with an Expansion Index of 50 or less or at least 35 percent sand-size particles should be acceptable as "soil" fill. Fill slopes should be overbuilt at least 2 feet and cut back or be compacted by backrolling with a loaded sheepsfoot roller at vertical intervals not to exceed 4 feet. The slopes should be track-walked shortly after the completion of each slope such that the fill is compacted to a dry density of at least 90 percent of the laboratory maximum dry density near to slightly above optimum moisture content to the face of the finished sloped. Rock fills may be placed to the face of the fill slope without affecting overall slope stability; however, the landscape planting material will be highly restricted to ground cover. In addition, toe of slope brow ditches will need to be added to capture the nuisance water in areas where seepage can affect adjacent properties or lots. Rock subdrains may be required at the toe of rock fills to capture nuisance water and direct it to an approved outlet location.

8.4.16 Placement of rock fills should be planned in the deeper fill areas to facilitate rock disposal. Overexcavation of fill areas may be required to accommodate the necessary rock volumes generated during blasting. Capping material used for placement near finish grade within roadways, building pads, and slope zones should be stockpiled during remedial grading operations. Overexcavation of weathered rock units that generate capping material may be required to achieve sufficient volumes to achieve finish grade.

8.4.17 Rock fill placement should be performed in accordance with the Recommended Grading Specifications provided in Appendix E. Blasting of rock material should be performed to maximize rock breakage to 2-foot minus material. Rock fill placement should generally be limited to 2-foot-thick horizontal layers and compacted using rock trucks. Significant volumes of water are typically required during rock fill placement.

8.4.18 Finished slopes should be landscaped with drought-tolerant vegetation having variable root depths and requiring minimal landscape irrigation. In addition, the slopes should be drained and properly maintained to reduce erosion. Rock cut and fill slopes will have restricted plant growth and will likely be limited to ground cover plant material.

8.5 Earthwork Grading Factors

8.5.1 Estimates of bulking and shrinkage factors are based on empirical judgments comparing the material in its natural state as encountered in the exploratory excavations to a compacted state. Variations in natural soil density and in compacted fill density, render shrinkage value estimates very approximate. As an example, the contractor can compact the fill to a dry density of 90 percent or higher of the laboratory maximum dry density. Thus, the contractor has an approximately 10 percent range of control over the fill volume. Bulking of rock units is a function of rock density, structure, overburden pressure, and the physical behavior of blasted material. Based on our experience, the shrinkage and bulking factors presented in Table 8.5 can be used as a basis for estimating how much the on-site soil may shrink or swell (bulk) when excavated from their natural state and placed as compacted fill. Please note that these estimates are for preliminary quantity estimates only. Due to the variations in the actual shrinkage/bulking factors, a balance area should be provided to accommodate these variations.

TABLE 8.5
SHRINKAGE AND BULK FACTORS

Soil Unit	Shrink/Bulk Factor
Undocumented Fill (Qudf)	10-15% shrink
Topsoil (unmapped)	10-15% shrink
Colluvium (Qc)	10-15% shrink
Granitic and Metamorphic Rock (Kgr and Trm) – rippable	10-15% bulk
Granitic and Metamorphic Rock (Kgr and Trm) – blasted	20-30% bulk

8.6 Shallow Foundation and Concrete Slab-On-Grade Recommendations

8.6.1 The foundation recommendations herein are for proposed one- to three-story residential structures. Foundation recommendations for other types of structures will be provided in subsequent geotechnical investigations when the locations and building loads are known. The foundation recommendations have been separated into three categories based on the maximum and differential fill thickness and Expansion Index. The foundation category criteria are presented in Table 8.6.1.

TABLE 8.6.1
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

8.6.2 Table 8.6.2 presents minimum foundation and interior concrete slab design criteria for conventional foundation systems.

TABLE 8.6.2
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

8.6.3 The embedment depths presented in Table 8.6.2 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 on Figure 9.

8.6.4 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 in Section 9.3 of the American Concrete Institute's (ACI) *Guide for Concrete Slabs that Receive Moisture-Sensitive Flooring Materials* (ACI 302.2R-06).

8.6.5 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 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 8.6.3 for the particular Foundation Category designated. The parameters presented in Table 8.6.3 are based on the guidelines presented in the PTI, Third Edition design manual.

TABLE 8.6.3
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

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

8.6.7 If the structural engineer proposes a post-tensioned foundation design method other than PTI, Third Edition:

- The deflection criteria presented in Table 8.6.3 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.

8.6.8 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.

8.6.9 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.

8.6.10 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. The estimated maximum total and differential settlement for the planned structures due to foundation loads is 1 inch and ½ inch, respectively.

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

8.6.12 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 patio slabs, which exceed 5 feet in width, to the building foundation to reduce the potential for future separation to occur.

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

8.6.14 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 and retaining walls, building and retaining wall 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 building foundations should be extended to a depth where the minimum horizontal distance is equal to H/3 (where H equals the vertical distance from the top of the fill slope to the base of the fill soil) with a minimum of 7 feet but need not exceed 40 feet. The horizontal distance is measured from the outer, deepest edge of the footing to the face of the slope. An acceptable alternative to deepening the footings would be the use of a post-tensioned slab and foundation system or increased footing and slab reinforcement. Specific design parameters or recommendations for either of these alternatives can be provided once the building location and fill slope geometry have been determined.
- If swimming pools are planned, Geocon Incorporated should be contacted for a review of specific site conditions.
- Swimming pools located within 7 feet of the top of cut or fill slopes are not recommended. Where such a condition cannot be avoided, the portion of the swimming pool wall within 7 feet of the slope face be designed assuming that the adjacent soil provides no lateral support. This recommendation applies to fill slopes up to 30 feet in height, and cut slopes regardless of height. For swimming pools located near the top of fill slopes greater than 30 feet in height, additional recommendations may be required and Geocon Incorporated should be contacted for a review of specific site conditions.
- Although other improvements, which are relatively rigid or brittle, such as concrete flatwork or masonry walls, may experience some distress if located near the top of a slope, it is generally not economical to mitigate this potential. It may be possible, however, to incorporate design measures that would permit some lateral soil movement without causing extensive distress. Geocon Incorporated should be consulted for specific recommendations.

8.6.15 The concrete slab-on-grade recommendations are based on soil support characteristics only. The project structural engineer should evaluate the structural requirements of the concrete slabs for supporting expected loads.

8.6.16 The recommendations presented herein are intended to reduce the potential for cracking of slabs and foundations as a result of differential movement. However, even with the incorporation of the recommendations presented herein, foundations and slabs-on-grade will still crack. The occurrence of concrete shrinkage cracks is independent of the soil supporting characteristics. Their occurrence may be reduced and/or controlled by limiting the slump of the concrete, the use of crack control joints and proper concrete placement and curing. Crack control joints should be spaced at intervals no greater than 12 feet. Literature provided by the Portland Concrete Association (PCA) and American Concrete Institute (ACI) present recommendations for proper concrete mix, construction, and curing practices, and should be incorporated into project construction.

8.7 Concrete Flatwork

8.7.1 Exterior concrete flatwork not subject to vehicular traffic should be constructed in accordance with the recommendations herein. Slab panels should be a minimum of 4 inches thick and, when in excess of 8 feet square, should be reinforced with 6 x 6 - W2.9/W2.9 (6 x 6 - 6/6) welded wire mesh to reduce the potential for cracking for subgrade soil with an Expansion Index of 50 or less. If subgrade soil has an Expansion Index greater than 50, the reinforcement should be increased to No. 3 reinforcing bars spaced 18 inches on center in both directions. In addition, concrete flatwork should be provided with crack control joints to reduce and/or control shrinkage cracking. Crack control spacing should be determined by the project structural engineer based upon the slab thickness and intended usage. Criteria of the American Concrete Institute (ACI) should be taken into consideration when establishing crack control spacing. Subgrade soil for exterior slabs not subjected to vehicle loads should be compacted in accordance with criteria presented in the grading section prior to concrete placement. Subgrade soil should be properly compacted and the moisture content of subgrade soil should be evaluated prior to placing concrete.

8.7.2 Even with the incorporation of the recommendations within this report, the exterior concrete flatwork has a likelihood of experiencing some uplift due to expansive soil beneath grade; therefore, the welded wire mesh should overlap continuously in flatwork to reduce the potential for vertical offsets within flatwork. Additionally, flatwork should be structurally connected to the curbs, where possible, to reduce the potential for offsets between the curbs and the flatwork.

8.7.3 Where exterior flatwork abuts the structure at entrant or exit points, the exterior slab should be dowelled into the structure's foundation stemwall. This recommendation is intended to reduce the potential for differential elevations that could result from differential settlement or minor heave of the flatwork. Dowelling details should be designed by the project structural engineer.

8.8 Conventional Retaining Walls

8.8.1 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 no steeper than 2:1 (horizontal:vertical), an active soil pressure of 50 pcf is recommended. These soil pressures assume that the backfill materials within an area bounded by the wall and a 1:1 plane extending upward from the base of the wall possess an EI of 50 or less. For those lots with finish grade soils having an EI greater than 50 and/or where backfill materials do not conform to the criteria herein, Geocon Incorporated should be consulted for additional recommendations.

8.8.2 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 in feet) at the top of the wall. Where walls are restrained from movement at the top and are 8 feet or less in height, an additional uniform pressure of 7H psf should be added to the above active soil pressure. Where the wall height exceeds 8 feet, the additional uniform pressure should be increased to 14H psf.

8.8.3 The structural engineer should determine the seismic design category for the project. If the project possesses a seismic design category of D, E, or F, the proposed retaining walls should be designed with seismic lateral pressure added to the active pressure. The seismic load exerted on the wall should be a triangular distribution with a pressure of 19H (where H is the height of the wall, in feet, resulting 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 0.29g calculated form the 2007 California Building Code (S_{DS}/2.5) and applying a pseudo-static coefficient of 0.5.

8.8.4 Retaining walls should be provided with a drainage system adequate to prevent the buildup of hydrostatic forces and waterproofed as required by the project architect. The soil immediately adjacent to the backfilled retaining wall should be composed of free draining material completely wrapped in Mirafi 140 (or equivalent) filter fabric for a lateral distance of 1 foot for the bottom two-thirds of the height of the retaining wall. The upper one-third should be backfilled with less permeable compacted fill to reduce water infiltration. The use of drainage openings through the base of the wall (weep holes) is not recommended where the seepage could be a nuisance or otherwise adversely affect the property adjacent to the base of the wall. The recommendations herein assume a properly compacted granular (EI of 50 or less) free-draining backfill material with no hydrostatic forces or imposed surcharge load. Figure 10 presents a typical retaining wall drain detail. If conditions different than those described are expected or if specific drainage details are desired, Geocon Incorporated should be contacted for additional recommendations.

8.8.5 In general, wall foundations having a minimum depth and width of 1 foot may be designed for an allowable soil bearing pressure of 2,000 psf, provided the soil within 4 feet below the base of the wall has an Expansion Index of 50 or less. The proximity of the foundation to the top of a slope steeper than 3:1 could impact the allowable soil bearing pressure Therefore, retaining wall foundations should be deepened such that the bottom outside edge of the footing is at least 7 feet horizontally from the face of the slope.

8.8.6 The recommendations presented herein are generally applicable to the design of rigid concrete or masonry retaining walls having a maximum height of 20 feet. In the event that walls higher than 20 feet or other types of walls (such as crib-type walls) are planned, Geocon Incorporated should be consulted for additional recommendations.

8.8.7 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

8.9 Mechanically Stabilized Earth (MSE) Retaining Walls

8.9.1 Mechanically stabilized earth (MSE) retaining walls are alternative walls that consist of modular block facing units with geogrid reinforced earth behind the block. The reinforcement grid attaches to the block units and is typically placed at specified vertical intervals and embedment lengths. For the purposes of this report, the spacing and lengths and types of the geogrid were assumed based on the expected type of soil used for the backfill, and the slope stability requirements to achieve an acceptable factor of safety.

8.9.2 The geotechnical parameters listed in Table 8.9 can be used for design of the MSE walls.

TABLE 8.9
GEOTECHNICAL PARAMETERS FOR MSE WALLS

Parameter	Reinforced Zone	Retained Zone	Foundation Zone
Angle of Internal Friction	32 degrees	32 degrees	32 degrees
Cohesion	200 psf	200 psf	200 psf
Wet Unit Density	130 pcf	130 pcf	130 pcf

8.9.3 The soil parameters presented in Table 8.9 are based on our experience and direct shear-strength tests performed during the geotechnical investigation and represent some of the on-site materials. The wet unit density values presented in Table 8.9 can be used for design but actual in-place densities may range from approximately 90 to 135 pounds per cubic foot. Geocon Incorporated has no way of knowing whether these materials will actually be used as backfill behind the wall during construction. It is up to the wall designers to use their judgment in selection of the design parameters. As such, once backfill materials have been selected and/or stockpiled, sufficient shear tests should be conducted on samples of the proposed backfill materials to check that they conform to actual design values. Results should be provided to the designer to re-evaluate stability of the walls. Dependent upon test results, the designer may require modifications to the original wall design (e.g., longer reinforcement embedment lengths and/or steel reinforcement).

8.9.4 The foundation zone is the area where the footing is embedded, the reinforced zone is the area of the backfill that possesses the reinforcing fabric, and the retained zone is the area behind the reinforced zone.

8.9.5 Wall foundations having a minimum depth and width of one foot may be designed for an allowable soil bearing pressure of 2,000 psf. The allowable soil bearing pressure may be increased by an additional 300 psf for each additional foot of width and depth to a maximum allowable bearing capacity of 4,000 psf. The values presented herein are for dead plus live loads and may be increased by one-third when considering transient loads due to wind or seismic forces.

8.9.6 Backfill materials within the reinforced zone should be compacted to a dry density of at least 90 percent of the laboratory maximum dry density near to slightly above optimum moisture content in accordance with ASTM D 1557. This is applicable to the entire embedment width of the reinforcement. Typically, wall designers specify no heavy compaction equipment within 3 feet of the face of the wall. However, smaller equipment (e.g., walk-behind, self-driven compactors or hand whackers) can be used to compact the materials without causing deformation of the wall. If the designer specifies no compactive effort for this zone, the materials are essentially not properly compacted and the reinforcement grid within the uncompacted zone should not be relied upon for reinforcement, and overall embedment lengths will have to be increased to account for the difference. The reinforcing grid should be pulled tight from the face of the wall during placement to remove any slack in the grid.

8.9.7 Select backfill materials may be required to be in accordance with the MSE retaining wall system. Materials as outlined in the specifications of the retaining wall plans may be generated and stockpiled during grading, if encountered, or may require import. Geocon should perform laboratory tests during the backfill materials to check that soil properties are in accordance with the retaining wall plans and specifications. Based on the results of our field investigation and laboratory testing, materials from the surficial soil and formational rock materials may be a potential source of granular material to create select backfill. Substantial screening of the existing units will likely be require prior to use as backfill materials.

8.9.8 The wall should be provided with a drainage system sufficient to prevent excessive seepage through the wall and the base of the wall, thus preventing hydrostatic pressures behind the wall.

8.9.9 Geosynthetic reinforcement must elongate to develop full tensile resistance. This elongation generally results in movement at the top of the wall. The amount of movement is dependent upon the height of the wall (e.g., higher walls rotate more) and the type of reinforcing grid used. In addition, over time the reinforcement grid has been known to exhibit creep (sometimes as much as 5 percent) and can undergo additional movement. Given this condition, the owner should be aware that structures and pavement placed within the reinforced and retained zones of the wall may undergo movement. In addition, reinforcing grid placed near finish grade will require building and pool restrictions on future home owners.

8.9.10 The MSE wall contractor should provide the estimated deformation of wall and adjacent ground in associated with wall construction. The calculated horizontal and vertical deformations should be determined by the wall designer. The estimated movements should be provided to the project structural engineer to determine if the planned structures can tolerate the expected movements. In addition, a third-party structural plan review should be considered for the MSE walls subsequent to the completion of the plans.

8.10 Lateral Loads

8.10.1 For resistance to lateral loads, an allowable passive earth pressure equivalent to a fluid density of 350 pcf is recommended for footings or shear keys poured neat against properly compacted granular fill or undisturbed formational materials. The allowable passive pressure assumes a horizontal surface extending away from the base of the wall at least 5 feet or three times the height of the surface generating the passive pressure, whichever is greater. The upper 12 inches of material not protected by floor slabs or pavement should not be included in the design for lateral resistance.

8.10.2 An allowable friction coefficient of 0.4 may be used for resistance to sliding between soil and concrete. This friction coefficient may be combined with the allowable passive earth pressure when determining resistance to lateral loads.

8.11 Preliminary Pavement Recommendations

8.11.1 We calculated the flexible pavement sections in general conformance with the *Caltrans Method of Flexible Pavement Design* (Highway Design Manual, Section 608.4) using an estimated Traffic Index (TI) of 4.5, 5.0, 6.0, 7.0, and 8.0. The project civil engineer and owner should review the pavement designation to determine appropriate locations for pavement thickness. The final pavement sections for roadways should be based on the R-Value of the subgrade soils encountered at final subgrade elevation. Streets should be designed in accordance with the County of San Diego specifications when final Traffic Indices and R-value test results of subgrade soil are completed. Based on the results of our laboratory R-Value testing, we have assumed an R-Value of 20 for the subgrade soil for the purposes of this preliminary analysis. Preliminary flexible pavement sections are presented in Table 8.11.1.

TABLE 8.11.1
PRELIMINARY FLEXIBLE PAVEMENT SECTIONS

Location	Assumed Traffic Index	Assumed Subgrade R-Value	Asphalt Concrete (inches)	Class 2 Aggregate Base (inches)
Residential	4.5	20	3	6
Residential Collector	5.0	20	3	8
Rural Collector	6.0	20	3.5	10
Heavy Truck Traffic	7.0	20	4.0	12
Lake Jennings Road	8.0	20	5.0	14

8.11.2 The upper 12 inches of the subgrade soil should be compacted to a dry density of at least 95 percent of the laboratory maximum dry density near to slightly above optimum moisture content beneath pavement sections.

8.11.3 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.

8.11.4 A rigid Portland Cement concrete (PCC) pavement section placed within cross-gutters should have a minimum thickness of 7 inches. We calculated the rigid pavement section in general conformance with the procedure recommended by the American Concrete Institute report ACI 330R-01 *Guide for Design and Construction of Concrete Parking Lots* using the parameters presented in Table 8.11.2.

TABLE 8.11.2
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-1 and C
Average daily truck traffic, ADTT	10 and 100

8.11.5 Based on the criteria presented herein, the PCC pavement sections should have a minimum thickness as presented in Table 8.11.3.

TABLE 8.11.3
RIGID PAVEMENT RECOMMENDATIONS

Location	Portland Cement Concrete (inches)
Automobile Parking Areas (TC=A-1)	6
Heavy Truck and Fire Lane Areas (TC=C)	7

8.11.6 The PCC pavement should be placed over subgrade soil that is compacted to a dry density of at least 95 percent of the laboratory maximum dry density 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). Base materials will not be required beneath concrete improvements including cross-gutters, curb and gutters, and sidewalks.

8.11.7 A thickened edge or integral curb should be constructed on the outside of concrete slabs subjected to wheel loads. The thickened edge should be 1.2 times the slab thickness or a minimum thickness of 2 inches, whichever results in a thicker edge, at the slab edge and taper back to the recommended slab thickness 3 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 below.

8.11.8 To control the location and spread of concrete shrinkage cracks, crack-control joints (weakened plane joints) should be included in the design of the concrete pavement slab. Crack-control joints should not exceed 30 times the slab thickness with a maximum spacing of 15 feet (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.

8.11.9 To provide load transfer between adjacent pavement slab sections, a trapezoidal-keyed construction joint should be installed. As an alternative to the keyed joint, dowelling is recommended between construction joints. As discussed in the referenced ACI guide, dowels should consist of smooth; ⅞-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. Other alternative recommendations for load transfer should be provided by the project structural engineer.

8.11.10 The performance of asphalt concrete 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 to proposed improvements. Drainage from landscaped areas should be directed to controlled drainage structures. Landscape areas adjacent to the edge of asphalt pavements are not recommended due to the potential for surface or irrigation water to infiltrate the underlying permeable aggregate base and cause distress. Where such a condition cannot be avoided, consideration should be given to incorporating measures that will significantly reduce the potential for subsurface water migration into the aggregate base. If planter islands are planned, the perimeter curb should extend at least 6 inches below the level of the base materials.

8.12 Site Drainage and Moisture Protection

8.12.1 Adequate site drainage is critical to reduce the potential for differential soil movement, erosion and subsurface seepage. Under no circumstances should water be allowed to pond adjacent to footings. The site should be graded and maintained such that surface drainage is directed away from structures in accordance with 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.

8.12.2 In the case of basement walls or building walls retaining landscaping areas, a water-proofing system should be used on the wall and joints, and a Miradrain drainage panel (or similar) should be placed over the waterproofing. A perforated drainpipe of schedule 40 or better should be installed at the base of the wall below the floor slab and drained to an appropriate discharge area. Accordion-type pipe is not acceptable. The project architect or civil engineer should provide detailed specifications on the plans for all waterproofing and drainage.

8.12.3 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.

8.12.4 Landscaping planters adjacent to paved areas are not recommended due to the potential for surface or irrigation water to infiltrate the pavement's subgrade and base course. We recommend that area drains to collect excess irrigation water and transmit it to drainage structures or impervious above-grade planter boxes be used. In addition, where landscaping is planned adjacent to the pavement, we recommend construction of a cutoff wall along the edge of the pavement that extends at least 6 inches below the bottom of the base material.

8.12.5 If detention basins, bioswales, retention basins, or water infiltration devices are being considered, Geocon Incorporated should be retained to provide recommendations pertaining to the geotechnical aspects of possible impacts and design. Distress may be caused to planned improvements and properties 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.

8.13 Grading and Foundation Plan Review

8.13.1 Geocon Incorporated should review the grading plans and foundation plans for the project prior to final design submittal to evaluate whether additional analysis and/or recommendations are required.

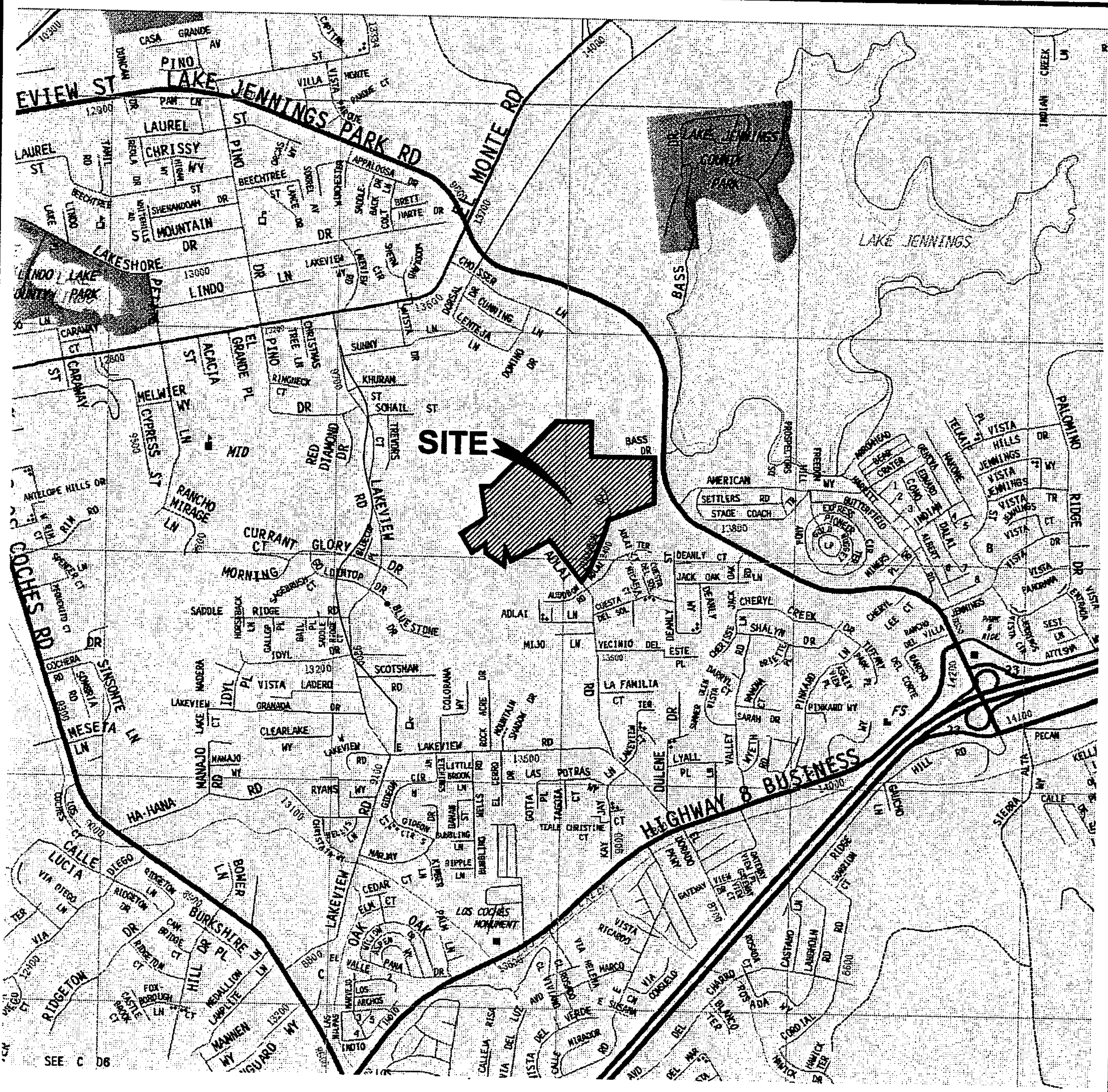
LIMITATIONS AND UNIFORMITY OF CONDITIONS

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 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 of his representative, to ensure that the information and recommendations contained herein are brought to the attention of the architect and engineer for the project and incorporated into the plans, and the necessary steps are taken to see that the contractor and subcontractors carry out such recommendations in the field.

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



SOURCE: 2007 THOMAS BROTHERS MAP
SAN DIEGO COUNTY, CALIFORNIA

N

NO SCALE

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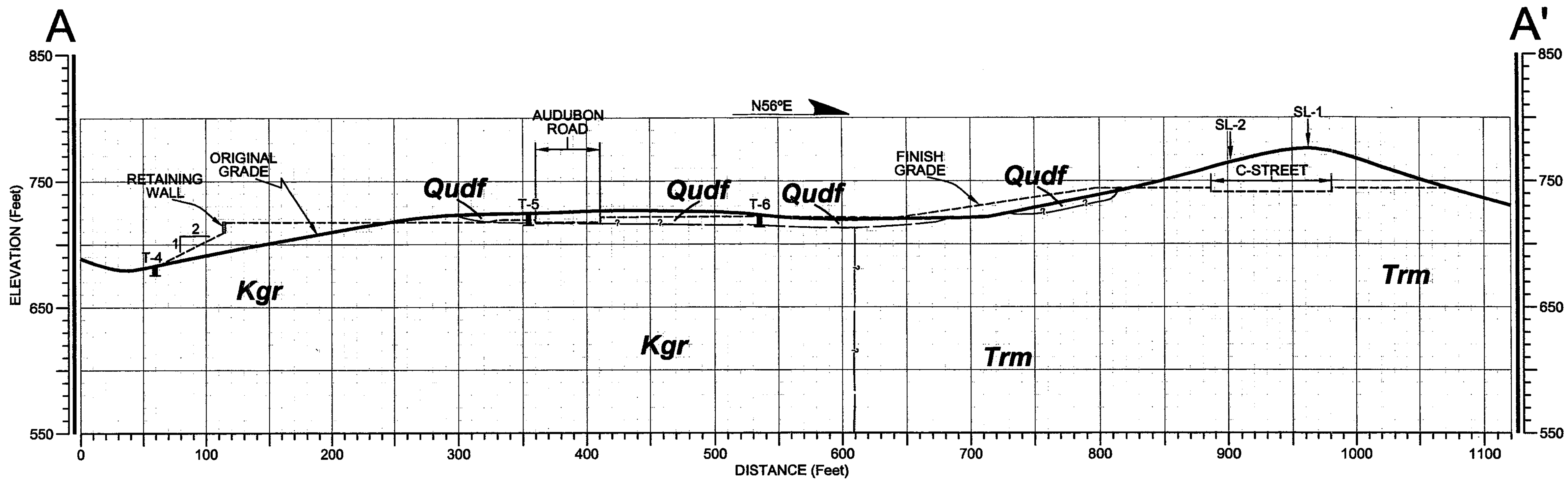
VICINITY MAP

GREENHILLS RANCH - PHASE 2
SAN DIEGO COUNTY, CALIFORNIA

DATE 07 - 20 - 2009	PROJECT NO. G1117 - 52 - 01	FIG. 1

Vicinity Map

GREENHILLS RANCH - PHASE 2
SAN DIEGO COUNTY, CALIFORNIA



GEOLOGIC CROSS - SECTION A-A'

SCALE: 1" = 100' (Horiz. = Vert.)

GEOCON LEGEND

Qudf ........UNDOCUMENTED FILL
Kgr ....GRANITIC ROCK
Trm ....JULIAN SCHIST
T-6 ....APPROX. LOCATION OF TRENCH
SL-2 ....APPROX. LOCATION OF SEISMIC TRAVERSE
? ....APPROX. LOCATION OF GEOLOGIC CONTACT (Queried Where Uncertain)

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PROJECT NO. G1117 - 52 - 01
FIGURE 3
DATE 07 - 20 - 2009

ASSUMED CONDITIONS :

SLOPE HEIGHT	H	= 50 feet
SLOPE INCLINATION	2 : 1	(Horizontal : Vertical)
TOTAL UNIT WEIGHT OF SOIL	γ_t	= 125 pounds per cubic foot
ANGLE OF INTERNAL FRICTION	ϕ	= 35 degrees
APPARENT COHESION	C	= 1,000 pounds per square foot
NO SEEPAGE FORCES		

ANALYSIS :

$\gamma_{c\phi}$	=	$\dfrac{\gamma H \tan\phi}{C}$	EQUATION (3-3), REFERENCE 1
FS	=	$\dfrac{N_{cf} C}{\gamma H}$	EQUATION (3-2), REFERENCE 1
$\gamma_{c\phi}$	=	4.4	CALCULATED USING EQ. (3-3)
Ncf	=	20	DETERMINED USING FIGURE 10, REFERENCE 2
FS	=	3.2	FACTOR OF SAFETY CALCULATED USING EQ. (3-2)

REFERENCES :

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

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

SLOPE STABILITY ANALYSIS - CUT SLOPES

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GREENHILLS RANCH - PHASE 2
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JH / RA		DSK/GTYPD	DATE 07 - 20 - 2009	PROJECT NO. G1117 - 52 - 01	FIG. 4

ASSUMED CONDITIONS :

SLOPE HEIGHT	H	= 60 feet
SLOPE INCLINATION	2 : 1 (Horizontal : Vertical)	
TOTAL UNIT WEIGHT OF SOIL	γ_t	= 125 pounds per cubic foot
ANGLE OF INTERNAL FRICTION	ϕ	= 32 degrees
APPARENT COHESION	C	= 250 pounds per square foot
NO SEEPAGE FORCES		

ANALYSIS :

$\gamma_{c\phi} = \dfrac{\gamma H \tan\phi}{C}$	EQUATION (3-3), REFERENCE 1
$FS = \dfrac{N_{cf} C}{\gamma H}$	EQUATION (3-2), REFERENCE 1
$\gamma_{c\phi} = 18.7$	CALCULATED USING EQ. (3-3)
$N_{cf} = 50$	DETERMINED USING FIGURE 10, REFERENCE 2
$FS = 1.7$	FACTOR OF SAFETY CALCULATED USING EQ. (3-2)

REFERENCES :

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

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

SLOPE STABILITY ANALYSIS - FILL SLOPES

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GREENHILLS RANCH - PHASE 2
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JH / RA		DSK/GTYPD	DATE 07 - 20 - 2009	PROJECT NO. G1117 - 52 - 01	FIG. 5

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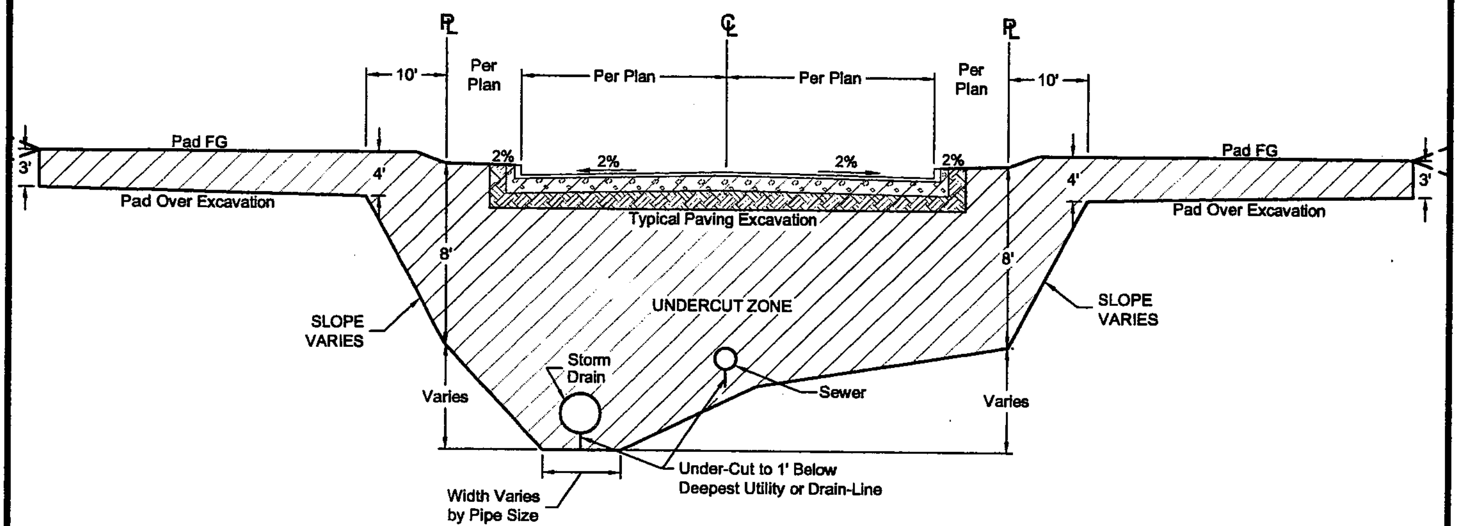

NOT TO SCALE

NOTE:
UNDERCUT ZONE SHOULD CONTAIN COMPACTED SOIL FILL WITH MAXIMUM ROCK FRAGMENTS LESS THAN 6 INCHES IN DIMENSION

TYPICAL STREET SECTION OVEREXCAVATION DETAIL

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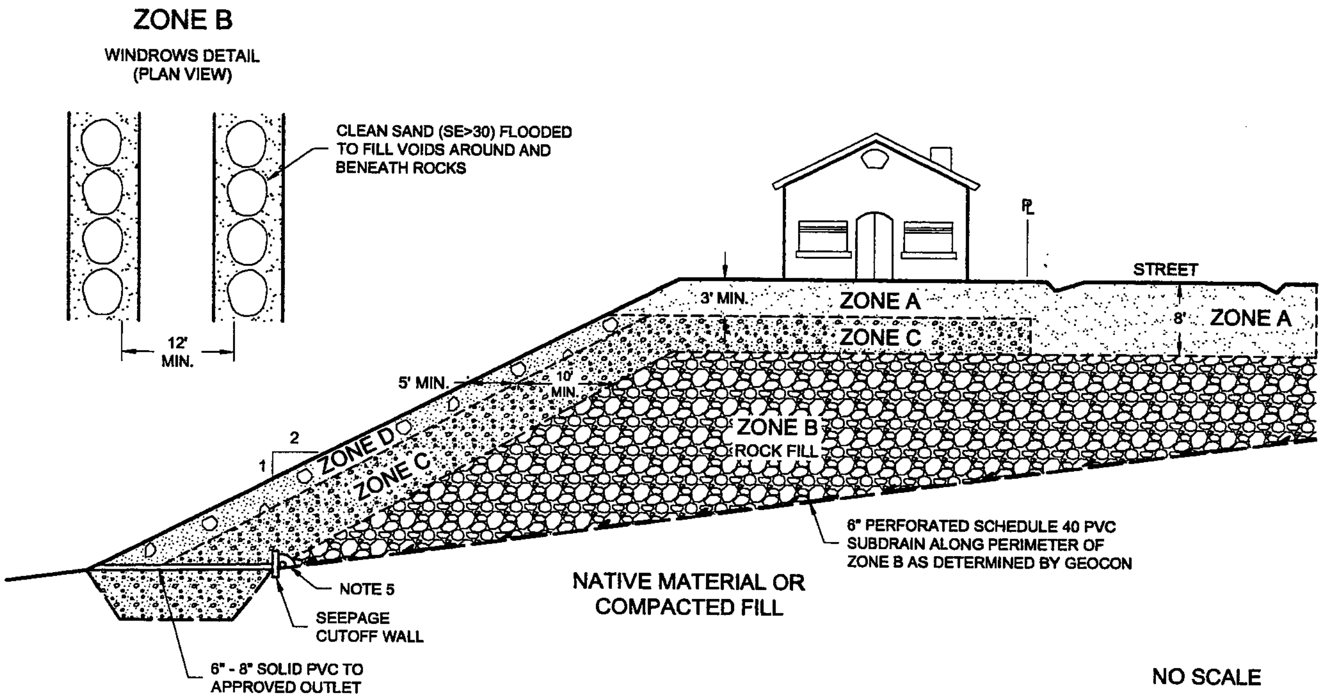
JH / RA		DSK/GTYPD	DATE 07 - 20 - 2009	PROJECT NO. G1117 - 52 - 01	FIG. 7



LEGEND

ZONE A: COMPACTED SOIL FILL. NO ROCK FRAGMENTS OVER 6 INCHES IN DIMENSION.

ZONE B: BLASTED ROCK FILL GENERALLY CONSISTING OF 2 FOOT MINUS MATERIAL WITH OCCASIONAL INDIVIDUAL ROCK UP TO 4 FEET MAXIMUM DIMENSION
ALTERNATE: ROCKS 2 TO 4 FEET IN MAXIMUM DIMENSION CAN BE PLACED IN WINDROWS IN COMPACTED SOIL FILL POSSESSING A SAND EQUIVALENT OF AT LEAST 30.

ZONE C: ROCKS UP TO 2 FEET IN MAXIMUM DIMENSION IN A MATRIX OF COMPACTED SOIL FILL WITHIN BUILDING PADS AND SLOPE AREAS ONLY.

ZONE D: ROCKS UP TO 1 FOOT IN MAXIMUM DIMENSION IN A MATRIX OF COMPACTED SOIL FILL.

NOTES

1. COMPACTED SOIL FILL IN UPPER 8 FEET SHALL CONTAIN AT LEAST 40 PERCENT SOIL PASSING THE 3/4 - INCH SIEVE (BY WEIGHT) AND IN THE UPPER 3 FEET OF PAD GRADE AT LEAST 20% SOIL PASSING THE NO. 4 SIEVE (BY WEIGHT) AND COMPACTED IN ACCORDANCE WITH SPECIFICATIONS FOR STRUCTURAL FILL.
2. CONTINUOUS OBSERVATION REQUIRED BY GEOCON DURING ROCK PLACEMENT.
3. ROCK FILL (LESS THAN 40 PERCENT SOIL SIZES) MAY BE PERMITTED IN DESIGNATED AREAS UPON THE RECOMMENDATION OF THE GEOTECHNICAL ENGINEER AND APPROVAL BY THE CITY ENGINEER.
4. DEPTH OF ZONE A SHOULD BE AT LEAST 8 FEET AND EXTENDED TO AT LEAST 1 FOOT BELOW DEEPEST UTILITY WITHIN ROADWAYS.
5. 6" PERFORATED SCHEDULE 40 PVC SUBDRAIN ALONG THE TOE AND PORTIONS OF THE PERIMETER OF ZONE B.
6. BASE OF ZONE B SHOULD SLOPE A MINIMUM OF 3 PERCENT.

OVERSIZE ROCK DISPOSAL DETAIL

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GREENHILLS RANCH - PHASE 2
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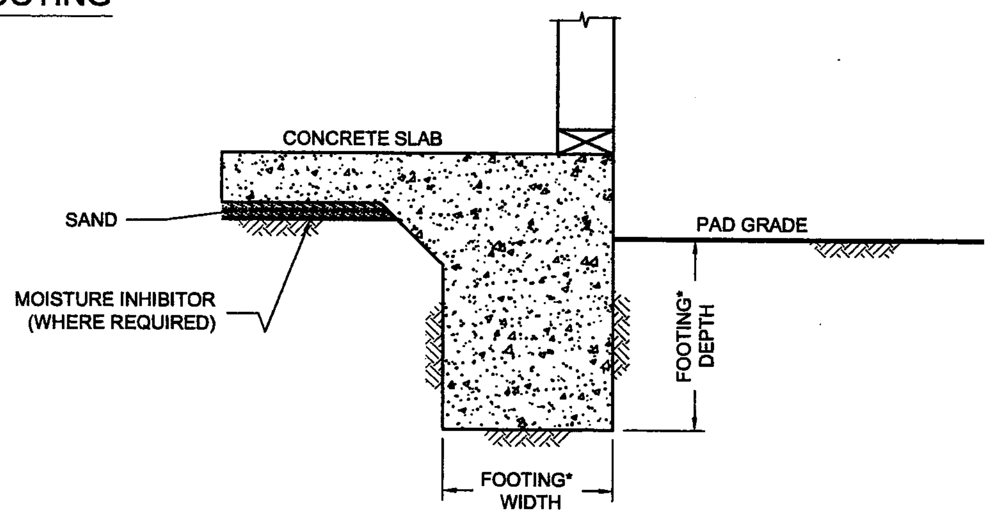
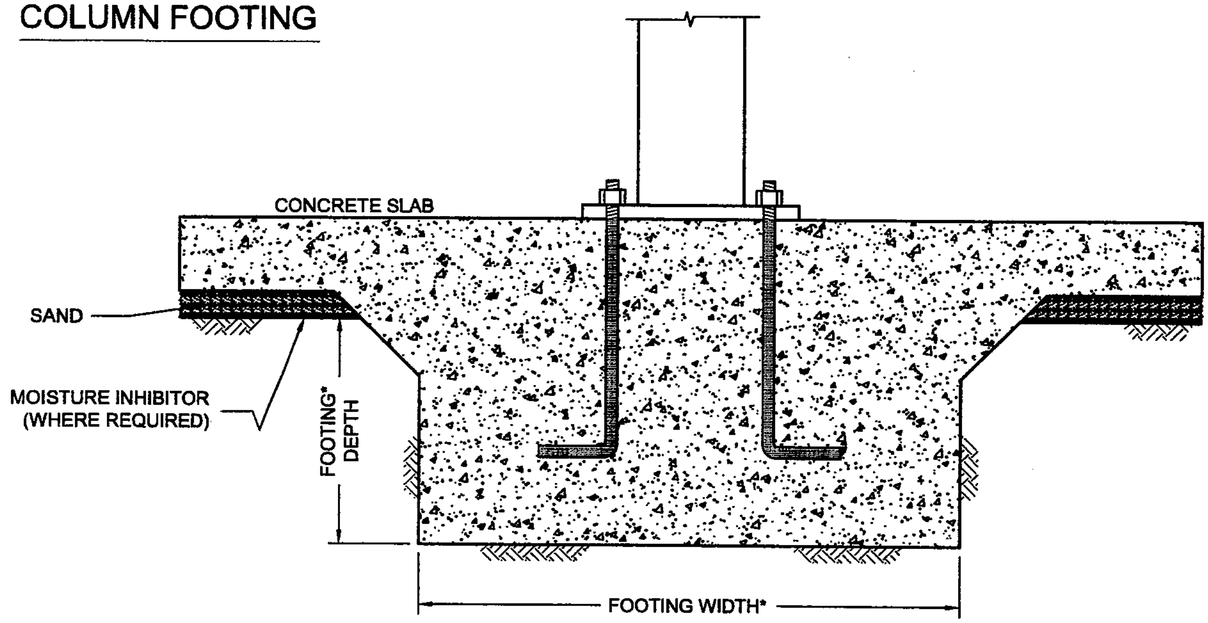
WALL FOOTING



COLUMN FOOTING



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

NO SCALE

WALL / COLUMN FOOTING DIMENSION DETAIL

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GREENHILLS RANCH - PHASE 2
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JH / RA		DSK/GTYPD	DATE 07 - 20 - 2009	PROJECT NO. G1117 - 52 - 01	FIG. 9

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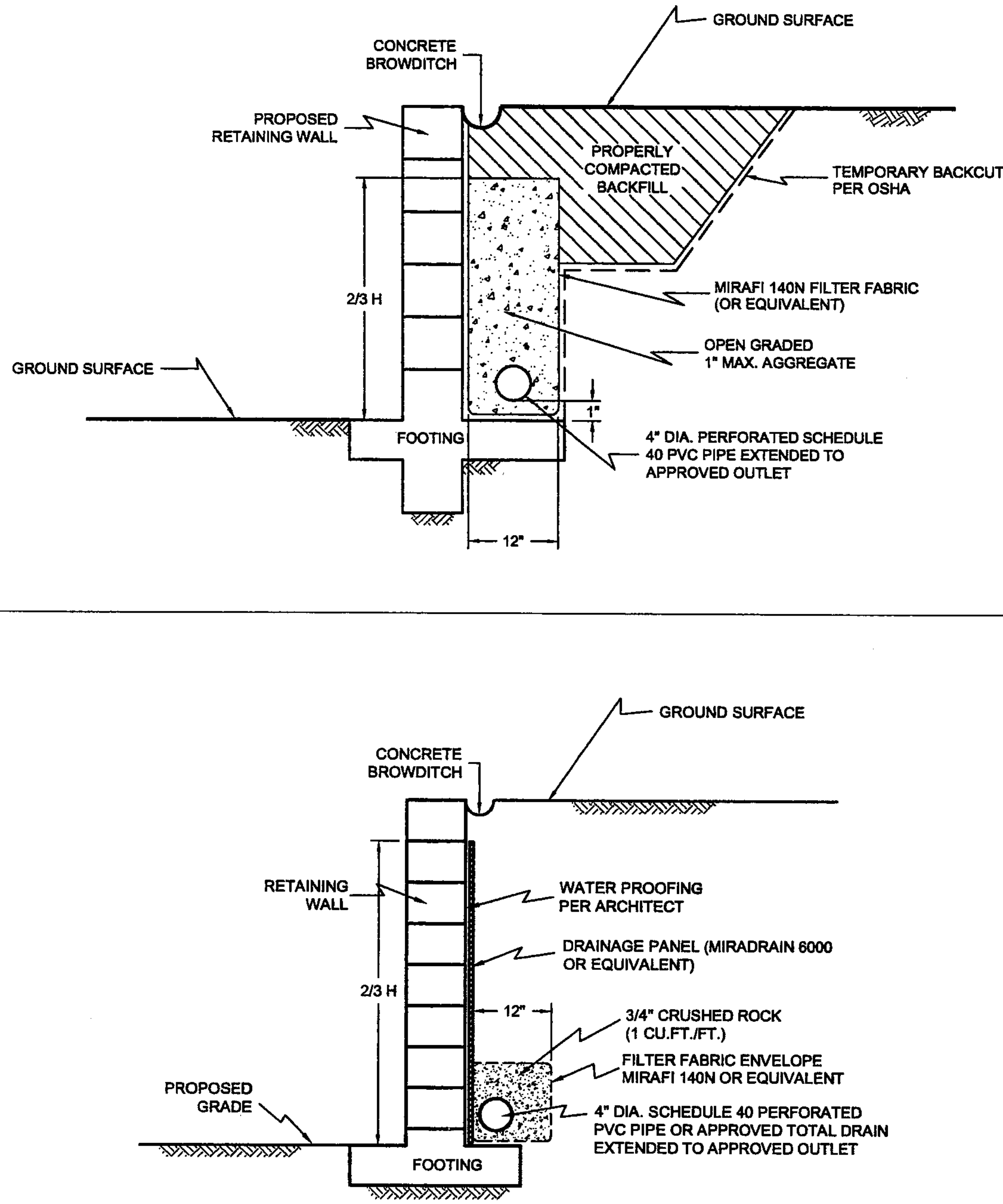

NOTE :

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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GREENHILLS RANCH - PHASE 2
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JH / RA		DSK/GTYPD	DATE 07 - 20 - 2009	PROJECT NO. G1117 - 52 - 01	FIG. 10

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APPENDIX A

APPENDIX A

EXPLORATORY EXCAVATIONS

The field investigation included the excavation of 14 excavator trenches. The approximate locations of the excavations are shown on the Geologic Map, Figure 2. The excavator trenches were excavated to a maximum depth of approximately 12 feet using a Komatsu PC 200 LC excavator with a 24-inch wide bucket. We logged and sampled the soil and geologic conditions as trenching proceeded.

We visually examined, classified and logged the soil conditions encountered in the excavations in general accordance with the Unified Soil Classification System (USCS). Logs of the exploratory excavator trenches are presented on Figures A-1 through A-14. The logs depict the general soil and geologic conditions encountered and the depth at which we obtained samples.

PROJECT NO. G1117-52-01

TRENCH T 1

ELEV. (MSL.) 683' DATE COMPLETED 06-09-2009

EQUIPMENT KOMATSU PC 200 LC BY: J. HOOBS

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0				SM	**UNDOCUMENTED FILL** Medium dense, dry to damp, reddish brown, Silty, fine to medium SAND with little gravel and minor roots			
2								
4								
6	T1-1				-Becomes yellowish brown and damp			
8								
10								
					JULIAN SCHIST Slightly weathered, yellow, strong SCHIST; fine- to medium-grained			
12					TRENCH TERMINATED AT 12 FEET No groundwater encountered			

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

G1117-52-01.GPJ

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

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

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PROJECT NO. G1117-52-01

TRENCH T 2

ELEV. (MSL.) 690' DATE COMPLETED 06-09-2009

EQUIPMENT KOMATSU PC 200 LC BY: J. HOOBS

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0				SM	**COLLUVIUM** Loose, damp, reddish brown, Silty, fine to medium SAND with little gravel			
2	T2-1							
4					-Becomes medium dense			
6								
8					**GRANITIC ROCK** Slightly weathered, greenish gray, strong GRANITIC ROCK; fine-grained			
					TRENCH TERMINATED AT 9 FEET No groundwater encountered			

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

G1117-52-01.GPJ

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

GEOCON

PROJECT NO. G1117-52-01

TRENCH T 3

ELEV. (MSL.) 711' DATE COMPLETED 06-09-2009

EQUIPMENT KOMATSU PC 200 LC BY: J. HOOBS

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0 – 2.5				SM	**TOPSOIL** Medium dense, damp, reddish brown, Silty, fine to medium SAND with gravel and cobble to 2 foot-diameter (40%)			
2.5 – 4					**GRANITIC ROCK** Slightly weathered, yellow, very strong GRANITIC ROCK; fine-grained			
					REFUSAL AT 4 FEET No groundwater encountered			

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

G1117-52-01.GPJ

SAMPLE SYMBOLS

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

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

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PROJECT NO. G1117-52-01

TRENCH T 4

ELEV. (MSL.) 682' DATE COMPLETED 06-09-2009

EQUIPMENT KOMATSU PC 200 LC BY: J. HOOBS

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0 – 3				SM	**TOPSOIL** Loose, damp, reddish brown, Silty, fine to medium SAND with some gravel			
3 – 6					**GRANITIC ROCK** Moderately weathered, greenish gray, moderately strong, GRANITIC ROCK; fine- to coarse-grained			
6					TRENCH TERMINATED AT 6 FEET No groundwater encountered			

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

G1117-52-01.GPJ

SAMPLE SYMBOLS

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

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

GEOCON

PROJECT NO. G1117-52-01

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	TRENCH T 5 ELEV. (MSL.) 723' DATE COMPLETED 06-09-2009 EQUIPMENT KOMATSU PC 200 LC BY: J. HOOBS	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
					MATERIAL DESCRIPTION			
0				SM	**UNDOCUMENTED FILL** Loose, dry, reddish brown, Silty, fine to medium SAND with cobble and boulder to 2½ foot-diameter (20%) and some trash			
2	T5-1							
4								
6								
					GRANITIC ROCK Fresh, yellowish brown, very strong GRANITIC ROCK; fine- to medium-grained			
8					REFUSAL AT 8 FEET No groundwater encountered			

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

G1117-52-01.GPJ

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

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

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PROJECT NO. G1117-52-01

TRENCH T 6

ELEV. (MSL.) 723' DATE COMPLETED 06-09-2009

EQUIPMENT KOMATSU PC 200 LC BY: J. HOOBS

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0				SM	**UNDOCUMENTED FILL** Loose, dry to damp, brown, Silty, fine to medium SAND with cobble and boulder to 2 foot-diameter (10%) and some trash			
2								
4								
6								
					GRANITIC ROCK Slightly weathered, yellowish brown, very strong GRANITIC ROCK; fine- to medium-grained			
8					REFUSAL AT 8 FEET No groundwater encountered			

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

G1117-52-01.GPJ

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

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

GEOCON

PROJECT NO. G1117-52-01

TRENCH T 7

ELEV. (MSL.) 709' DATE COMPLETED 06-09-2009

EQUIPMENT KOMATSU PC 200 LC BY: J. HOOBS

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0				SM	**COLLUVIUM** Loose, dry to damp, reddish brown, Silty, fine to medium SAND with gravel and cobble to 18 inch-diameter (20%)			
	T7-1							
2								
4								
					JULIAN SCHIST Moderately weathered, light reddish brown, moderately strong SCHIST; medium-grained			
6								
8	T7-2							
					TRENCH TERMINATED AT 9 FEET No groundwater encountered			

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

G1117-52-01.GPJ

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

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

GEOCON

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	TRENCH T 8 ELEV. (MSL.) 778' DATE COMPLETED 06-09-2009 EQUIPMENT KOMATSU PC 200 LC BY: J. HOOBS	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
					MATERIAL DESCRIPTION			
0				SM	**TOPSOIL** Loose, dry, reddish brown, Silty, fine to medium SAND with cobble and boulder to 2 foot-diameter (50%)			
2					**JULIAN SCHIST** Slightly weathered, yellowish brown, strong SCHIST; fine-grained			
4								
6					REFUSAL AT 6 FEET No groundwater encountered			

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

G1117-52-01.GPJ

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

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

GEOCON

PROJECT NO. G1117-52-01

TRENCH T 9

ELEV. (MSL.) 775' DATE COMPLETED 06-09-2009

EQUIPMENT KOMATSU PC 200 LC BY: J. HOOBS

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0				SM	**TOPSOIL** Medium dense, dry to damp, reddish brown, Silty, fine to medium SAND with cobble and boulder to 2-foot diameter (45%)			
2					**JULIAN SCHIST** Slightly weathered, reddish brown, strong SCHIST; fine-grained. Excavates to cobble and boulder to 2 foot-diameter			
4					REFUSAL AT 4 FEET No groundwater encountered			

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

G1117-52-01.GPJ

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

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

GEOCON

PROJECT NO. G1117-52-01

TRENCH T 10

ELEV. (MSL.) 771' DATE COMPLETED 06-09-2009

EQUIPMENT KOMATSU PC 200 LC BY: J. HOOBS

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0				SM	**TOPSOIL** Moderately dense, damp, reddish brown, Silty, fine to medium SAND with cobbles and boulders to 2 foot-diameter (45%)			
2					**JULIAN SCHIST** Moderately weathered, reddish brown, moderately strong SCHIST; fine- to medium-grained. Excavates to cobble and boulder to 2½ foot-diameter			
4					-Becomes strong at 4 feet and difficult to excavate			
					REFUSAL AT 5 FEET No groundwater encountered			

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

G1117-52-01.GPJ

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

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

GEOCON

TRENCH T 11

ELEV. (MSL.) 706' DATE COMPLETED 06-09-2009

EQUIPMENT KOMATSU PC 200 LC BY: J. HOOBS

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0	T11-1			SM	**TOPSOIL** Medium dense, dry, reddish brown, Silty, fine to medium SAND with gravel and cobble to 6 inch-diameter (10%)			
2					**JULIAN SCHIST** Slightly weathered, greenish brown, very strong SCHIST; fine- to medium-grained			
					REFUSAL AT 3 FEET No groundwater encountered			

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

G1117-52-01.GPJ

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

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

PROJECT NO. G1117-52-01

TRENCH T 12

ELEV. (MSL.) 699' DATE COMPLETED 06-09-2009

EQUIPMENT KOMATSU PC 200 LC BY: J. HOOBS

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0 – 5				SM	**TOPSOIL** Loose, dry, reddish brown, Silty, fine to medium SAND with cobble to 6 inch-diameter (10%)			
5 – 7					**JULIAN SCHIST** Moderately weathered, reddish brown, moderately strong SCHIST; fine- to medium-grained			
					TRENCH TERMINATED AT 7 FEET No groundwater encountered			

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

G1117-52-01.GPJ

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

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

GEOCON

PROJECT NO. G1117-52-01

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	TRENCH T 13 ELEV. (MSL.) 685' DATE COMPLETED 06-09-2009 EQUIPMENT KOMATSU PC 200 LC BY: J. HOOBS	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
					MATERIAL DESCRIPTION			
0				SM	**TOPSOIL** Medium dense, dry, reddish brown, Silty, fine to medium SAND with little cobble			
2								
4								
6					**JULIAN SCHIST** Moderately weathered, reddish brown, moderately strong SCHIST; fine- to medium-grained			
					TRENCH TERMINATED AT 7 FEET No groundwater encountered			

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

G1117-52-01.GPJ

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

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

GEOCON

TRENCH T 14

ELEV. (MSL.) 724' DATE COMPLETED 06-09-2009

EQUIPMENT KOMATSU PC 200 LC BY: J. HOOBS

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0				SM	**TOPSOIL** Loose, dry, reddish brown, Silty, fine to medium SAND with gravel and cobble to 12 inch-diameter (20%)			
2								
					JULIAN SCHIST Moderately weathered, reddish brown, moderately strong SCHIST; fine- to medium-grained			
4					-Becomes strong at 4 feet and difficult to excavate			
					TRENCH TERMINATED AT 5 FEET No groundwater encountered			

Figure A-14,
Log of Trench T 14, Page 1 of 1

G1117-52-01.GPJ

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

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

GEOCON

APPENDIX B

APPENDIX B

SEISMIC REFRACTION SURVEY REPORT

FROM

SEISMIC REFRACTION SURVEY,
GREENHILLS RANCH – PHASE 2,
LAKESIDE, CALIFORNIA,

PREPARED BY

SOUTHWEST GEOPHYSICS, INC.,
DATED JUNE 10, 2009 (PROJECT NO. 109123)

FOR

GREENHILLS RANCH – PHASE 2
SAN DIEGO COUNTY, CALIFORNIA

SEISMIC REFRACTION SURVEY
GREENHILLS RANCH-PHASE 2
LAKESIDE, 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

June 10, 2009
Project No. 109123

SOUTHWEST
GEOPHYSICS, INC.
YOUR SUBSURFACE SOLUTION

June 10, 2009
Project No. 109123

Mr. John Hoobs
Geocon Consultants, Inc.
6970 Flanders Drive
San Diego, CA 92121

Subject: Seismic Refraction Survey
Greenhills Ranch-Phase 2
Lakeside, California

Dear Mr. Hoobs:

In accordance with your authorization, we have performed a seismic refraction survey for the proposed Greenhills Ranch Residential Development, to be located in an area of generally undeveloped land along the northeast side of Adlai Road in the Lakeside area of San Diego County, 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

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

Hans van de Vrugt

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

REGISTERED GEOPHYSICIST
Hans van de Vrugt
No 1012
Exp. 2/28/10
STATE OF CALIFORNIA

HV/PFL/hv

Distribution: Addressee (electronic)

TABLE OF CONTENTS

1. INTRODUCTION

In accordance with your authorization, we have performed a seismic refraction survey for the proposed Greenhills Ranch Residential Development, to be located in an area of generally undeveloped land along the northeast side of Adlai Road in the Lakeside area of San Diego County, 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 south facing slopes just to the north of the intersection of Adlai Road and Audubon Road (Figure 1). The project area is situated just to the east of a single family residential property. The study area consists of undeveloped land along the top and southern flank of a moderately steep hill. Several outcrops of crystalline rock are present in and near the study area. Vegetation onsite consists of annual grass and scattered brush. Figures 2 and 3 illustrate the study area and general site conditions.

Based on our discussions with you, we understand that the study area is under consideration for grading and construction of new single family homes and associated roadways. Cuts up to roughly 45 feet deep are proposed.

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).

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 Local Blasting
5,500 to 7,000 feet/second	Very Difficult, Probable Local to General 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-1 240 feet	V1 = 1,500 V2 = 4,850 V3 = 7,850	1 – 3 1 – 38 ---	Easy Difficult, Possible Blasting Blasting Generally Required
SL-2 180 feet	V1 = 1,900 V2 = 5,750 V3 = 7,500	4 – 9 0 – 24 ---	Easy Very Difficult, Probable Blasting Blasting Generally Required

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

The results from our seismic survey 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 crystalline 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.

Based on the results of our survey, significant variations in the subsurface materials are present in the study area. Accordingly, variability in the excavatability (including excavation depth) of the subsurface materials should be expected across the project area.

Blasting will likely be required to obtain proposed excavation depths depending on the location and desired rate of production. A contractor with excavation experience in similar difficult conditions should be consulted for expert advice on excavation methodology, equipment, production rate, and possibly oversized materials.

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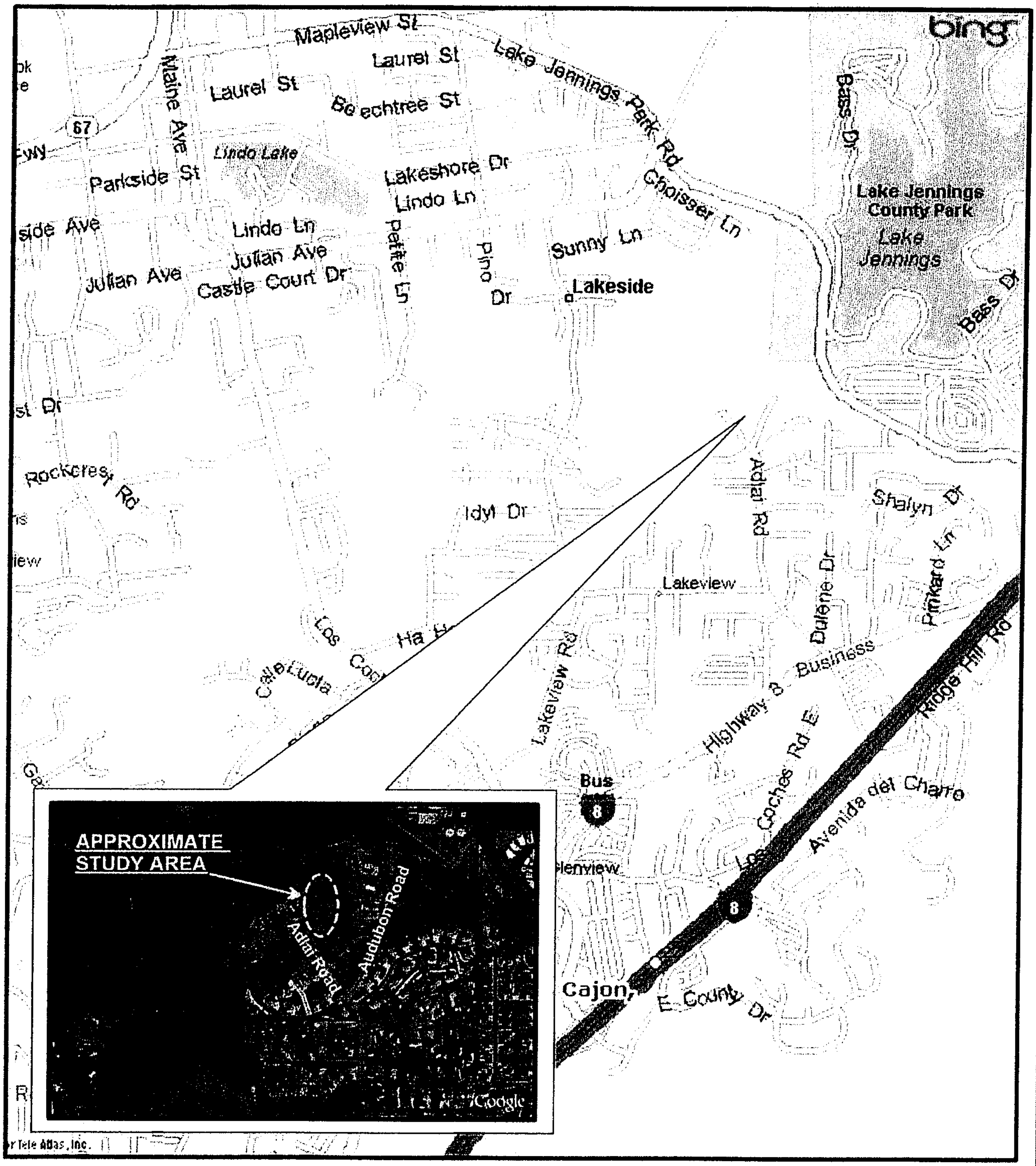
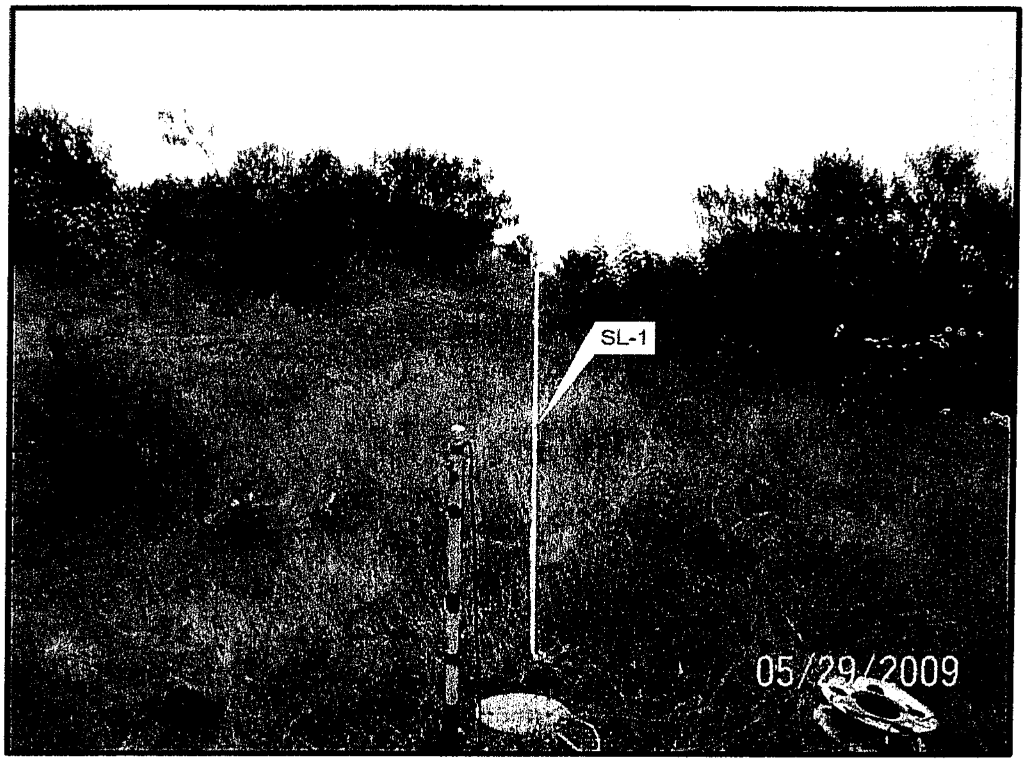
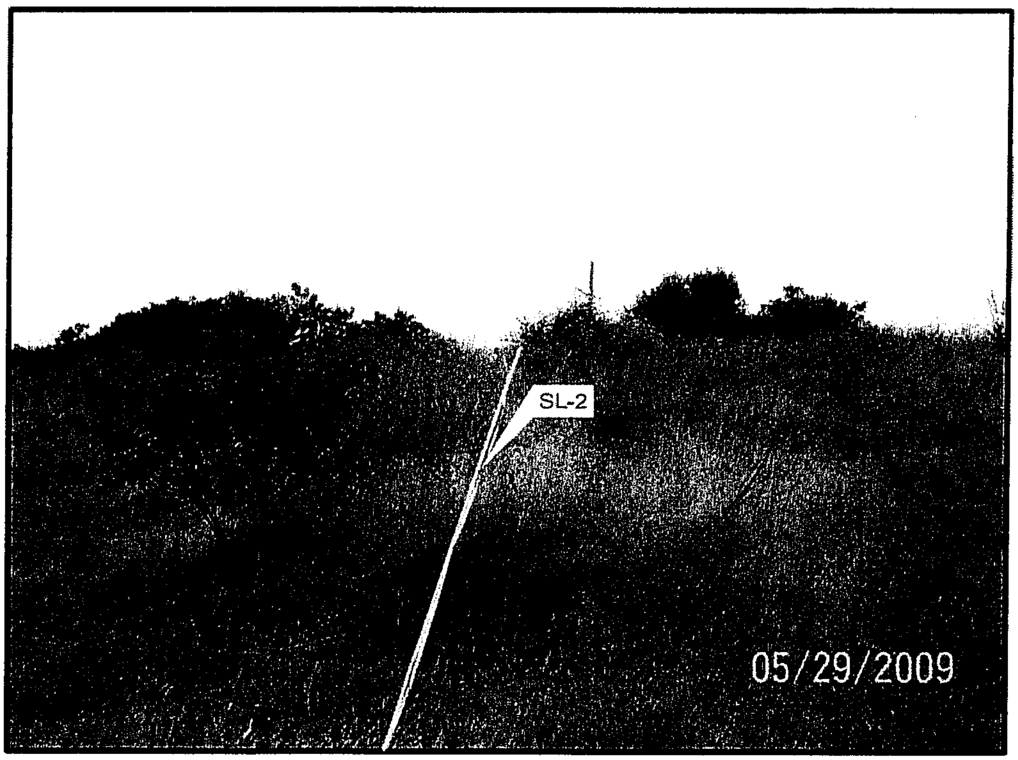
Telford, W.M., Geldart, L.P., Sheriff, R.E., and Keys, D.A., 1976, Applied Geophysics, Cambridge University Press.



SITE LOCATION MAP	N	Greenhills Ranch-Phase 2 Lakeside, California	SOUTHWEST GEOPHYSICS INC
		Project No.: 109123 / Date: 06/09	Figure 1

SEISMIC LINE LOCATION MAP	N	Greenhills Ranch-Phase 2 Lakeside, California Project No.: 109123 — Date: 06/09	SOUTHWEST GEOPHYSICS INC Figure 2	0 200 400 **approximate scale in feet**





SITE PHOTOGRAPHS	Greenhills Ranch-Phase 2 Lakeside, California	SOUTHWEST GEOPHYSICS INC
	Project No.: 109123 — Date: 06/09	Figure 3

APPENDIX C

RIPPABILITY SUMMARY

Our subsurface investigation included 2 seismic-refraction surveys and 14 excavator trenches. We used information from our subsurface exploration to estimate the approximate thickness of capping material and the depth to marginally to non-rippable rock summarized in Table C-I for each excavation location. The rippability information is also presented on the Geologic Map, Figure 2 at each excavation and seismic location.

TABLE C-I
RIPPABILITY SUMMARY

Seismic and Excavation No.	Geologic Formation	Estimated Thickness of Capping Material After Screening (feet)	Estimated Rippable Depth (feet)
SL-1	Metamorphic Rock (Trm)	1-3	2-15
SL-2	Metamorphic Rock (Trm)	4-7	4-12
T-1	Undocumented Fill and Metamorphic Rock (Qudf and Trm)	11	12+
T-2	Colluvium and Granitic Rock (Qc and Kgr)	7	9+
T-3	Topsoil and Granitic Rock (Kgr)	2 ½	4
T-4	Topsoil and Granitic Rock (Kgr)	3	6+
T-5	Undocumented fill and Granitic Rock (Qudf and Kgr)	7	8
T-6	Undocumented Fill and Granitic Rock (Qudf and Kgr)	7	8
T-7	Colluvium and Metamorphic Rock (Qc and Trm)	5	9+
T-8	Topsoil and Metamorphic Rock (Trm)	2	6
T-9	Topsoil and Metamorphic Rock (Trm)	1	4
T-10	Topsoil and Metamorphic Rock (Trm)	2	5
T-11	Topsoil and Metamorphic Rock (Trm)	2	3
T-12	Topsoil and Metamorphic Rock (Trm)	5	7+
T-13	Topsoil and Metamorphic Rock (Trm)	5	7+
T-14	Topsoil and Metamorphic Rock (Trm)	2 ½	5+

APPENDIX D

APPENDIX D

LABORATORY TESTING

We performed laboratory tests in accordance with generally accepted, current test methods of the American Society for Testing and Materials (ASTM) or other suggested procedures. We analyzed selected soil samples for maximum dry density and optimum moisture content, expansion potential, shear strength, water-soluble sulfate content, chloride ion content, pH and resistivity, and R-Value. The results of the laboratory tests are presented on Tables D-I through D-VII.

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

Sample No.	Description	Maximum Dry Density (pcf)	Optimum Moisture Content (% dry wt.)
T5-1	Brown, Silty, fine to medium SAND with trace gravel	118.5	12.0
T7-2	Brown, Silty fine to medium SAND with trace gravel	122.8	11.9

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

Sample No.	Dry Density (pcf)	Moisture Content (%)		Peak [Ultimate] Cohesion (psf)	Peak [Ultimate] Angle of Shear Resistance (degrees)
		Initial	Final		
T5-1*	106.4	12.3	17.2	270 [200]	35 [36]

*Sample remolded to a dry density of approximately 90 percent of the laboratory maximum dry density near optimum moisture content.

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

Sample No.	Moisture Content (%)		Dry Density (pcf)	Expansion Index	Expansion Classification
	Before Test	After Test			
T2-1	9.6	17.5	110.4	5	Very Low
T5-1	8.3	15.3	114.7	0	Very Low
T7-2	8.8	17.6	112.6	0	Very Low

TABLE D-IV
SUMMARY OF LABORATORY WATER-SOLUBLE SULFATE TEST RESULTS
CALIFORNIA TEST NO. 417

Sample No.	Water-Soluble Sulfate (%)	Water-Soluble Sulfate (ppm)	Sulfate Exposure*
T1-1	0.006	60	Negligible
T2-1	0.001	10	Negligible
T7-2	0.001	10	Negligible

*Reference: 2007 California Building Code

TABLE D-V
SUMMARY OF LABORATORY CHLORIDE ION CONTENT TEST RESULTS
AASHTO TEST NO. T 291

Sample No.	Chloride Ion Content (%)	Chloride Ion Content (ppm)
T7-1	0.004	41

TABLE D-VI
SUMMARY OF LABORATORY POTENTIAL OF HYDROGEN (pH) AND RESISTIVITY TEST RESULTS
CALIFORNIA TEST METHOD 643

Sample No.	pH	Minimum Resistivity (ohm-centimeters)
T7-1	7.3	10,000

TABLE D-VII
SUMMARY OF LABORATORY RESISTANCE VALUE (R-VALUE) TEST RESULTS
ASTM D 2844

Sample No.	R-Value
T11-1	19

APPENDIX E

APPENDIX E

RECOMMENDED GRADING SPECIFICATIONS

FOR

GREENHILLS RANCH – PHASE 2
SAN DIEGO COUNTY, CALIFORNIA

PROJECT NO. G1117-52-01

RECOMMENDED GRADING SPECIFICATIONS

1. GENERAL

1.1 These Recommended Grading Specifications shall be used in conjunction with the Geotechnical Report for the project prepared by Geocon 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

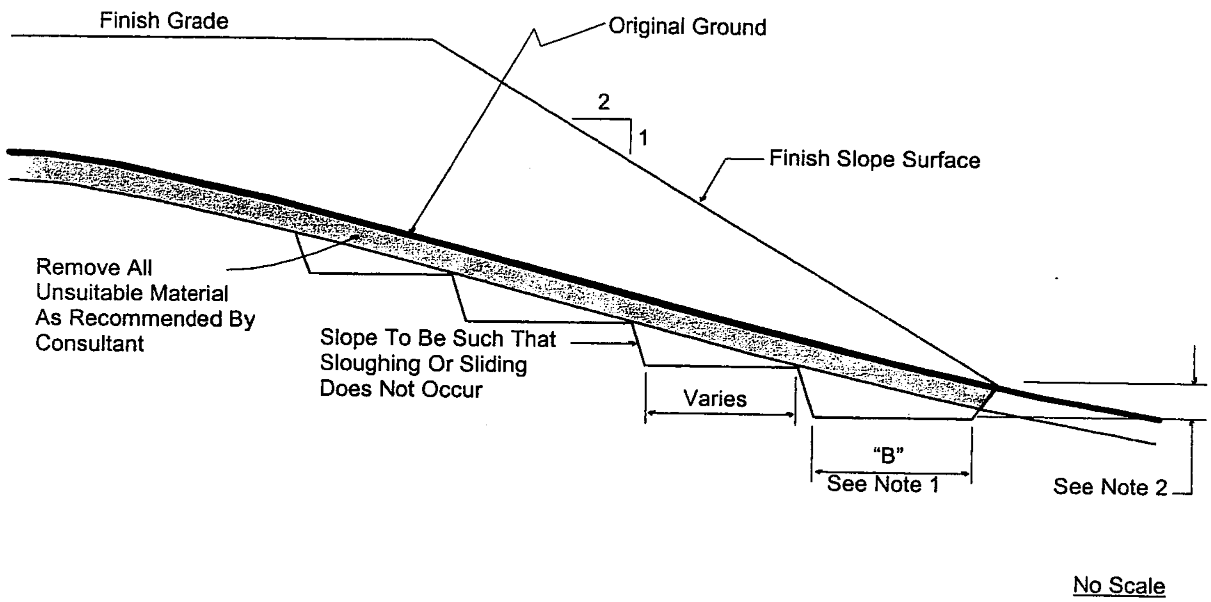
4.1 Areas to be excavated and filled shall be cleared and grubbed. Clearing shall consist of complete removal above the ground surface of trees, stumps, brush, vegetation, man-made structures, and similar debris. Grubbing shall consist of removal of stumps, roots, buried logs and other unsuitable material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