UPDATE
GEOTECHNICAL INVESTIGATION

VALIANO (EDEN HILLS)
SAN DIEGO COUNTY, CALIFORNIA

GEOCON
INCORPORATED

GEOTECHNICAL
ENVIRONMENTAL
MATERIALS

PREPARED FOR

INTEGRAL PARTNERS FUNDING, LLC
ENCINITAS, CALIFORNIA

MAY 13, 2014
PROJECT NO. G1416-52-02

Project No. G1416-52-02
May 13, 2014

Integral Partners Funding, LLC
2235 Encinitas Boulevard, Suite 216
Encinitas, California 92024

Attention: Ms. Melissa Krauss

Subject: UPDATE GEOTECHNICAL INVESTIGATION
VALIANO (EDEN HILLS)
SAN DIEGO COUNTY, CALIFORNIA

Dear Ms. Krauss:

In accordance with the authorization of our Change Order dated April 2, 2014, we herein submit the results of our update geotechnical investigation for the subject site. The accompanying report presents our findings, conclusions and recommendations pertaining to the geotechnical aspects of the proposed development. The study also provides excavation and remedial grading considerations, foundation and concrete slab-on-grade recommendations, retaining wall and lateral load recommendations, 2013 CBC seismic design criteria, pavement and flatwork recommendations, and discussions regarding the local geologic hazards including faulting, liquefaction, and seismic shaking. Based on the results of this study, it is our opinion 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

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GEOTECHNICAL INVESTIGATION

1. PURPOSE AND SCOPE

This report presents the results of our geotechnical investigation for the Eden Hills property located south of SR-78 and west of I-15 in northern San Diego County, California (see Vicinity Map, Figure 1). The purpose of this update report is to provide excavation and remedial grading considerations, foundation and concrete slab-on-grade recommendations, retaining wall and lateral load recommendations, 2013 CBC seismic design criteria, pavement and flatwork recommendations, and discussions regarding the local geologic hazards including faulting, liquefaction, and seismic shaking. The proposed development will include the construction of a single family residential subdivision with associated infrastructure. Plans for development, as presently proposed, are presented on the Geologic Map, Figures 2 and 3 (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 stereoscopic aerial photographs, published 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 scope of the study also included a review of:

1. *Geotechnical Investigation, Eden Hills, San Diego County, California,* prepared by Geocon Incorporated, dated September 12, 2012 (Project No. G1416-52-02).

2. *Response to the County of San Diego, Addendum to Geotechnical Investigation Report, Valiano (Eden Hills) San Diego County, California*, prepared by Geocon Incorporated, dated June 3, 2013 (Project No. G1416-52-03).

Our field investigation performed included geologic mapping and the excavation of 56 backhoe trenches, five small-diameter borings, ten air-track borings, and six seismic traverses. A discussion of the field investigation and logs of the backhoe trenches, small-diameter borings, air-track borings are presented in Appendix A. The approximate locations of the exploratory excavations are presented on the Geologic Maps (Figures 2 and 3). 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 B. Seismic refraction survey results are presented in Appendix C.

Latitude 33 Engineering, Inc. provided a preliminary grading plan that we used as our base map for our field investigation and for 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 property consists of parcels (232-013-01, 232-013-02, 232-013-03,232-020-55, 232-492-01, 228-313-13, 228-312-014 and 228-313-18), totaling approximately 192 acres of essentially undeveloped land. The property is bounded by Mount Whitney Road to the south, La Moree Mobil Home Estate to the north, Calico Lane and several single family homes and private roads to the east and City of San Marcos property to the west. A majority of the site is used as an avocado grove. The site consists of a ridge sloping down to the east with several drainages. Elevations at the site range from approximately 667 feet above mean sea level (MSL) near the southeastern corner to 1013 feet above MSL near the northwest corner of the property. Several residential structures are located within the project site. We understand that the development will consist of the construction of a residential subdivision with backbone and in-tract improvements.

Grading of the site will consist of maximum cuts and fills of approximately 46 feet and 40 feet, respectively, with cut and fill slopes having a combined maximum height of about 50 feet and a maximum slope inclination of 2:1 (horizontal:vertical).

The locations and descriptions provided herein are based on a site reconnaissance, and review of the referenced plans and project information provided by Latitude 33, Inc.

3. SOIL AND GEOLOGIC CONDITIONS

We encountered four surficial soil types and two geologic formations during the field investigation. The surficial deposits consist of undocumented fill, topsoil, colluvium, alluvium and Terrace Deposits. The formational units include Eocene-age Santiago Formation, and Cretaceous-age granitic rock (Tonalite). Each of the surficial soil types and geologic units encountered is described in order of increasing age. The approximate extent of these deposits, excluding topsoil, is shown on the Geologic Maps, Figures 2 and 3.

3.1 Undocumented Fill (Qudf)

We encountered pockets of undocumented fill at several locations on the property. The areas are typically associated with the farm activities and access roads. It is not unusual to encounter pockets of undocumented fill and debris at random locations placed during agricultural activities. These materials are not typically suitable for receiving fill or settlement sensitive improvements and require remedial grading in the form of removal and recompaction.

3.2 Topsoil (Unmapped)

The majority of the site is irregularly blanketed by 1 to 3 feet of topsoil consisting of loose, porous, dark brown, silty to clayey, fine to medium sand. Topsoil is compressible in its present condition, and will require removal and recompaction within areas of planned development.

3.3 Colluvium (Qc)

We encountered colluvium as a surficial accumulation of soft, moderately expansive sandy clays and loose clayey sand with cobbles and occasional boulders occur on and near the toe of most slopes. These materials are thickened topsoil transported by gravity. Based on our exploratory trenches, the thickness of colluvium can range from a few feet to more than 6 feet. Due to the soft/loose and unconsolidated condition of the colluvium, removal and recompaction will be required to provide suitable support for placement of compacted fill or structural improvements.

3.4 Alluvium (Qal)

We encountered alluvial soil within the canyon drainages. The alluvium generally varies in depth depending upon the size of the canyon. The alluvium consists of sandy clay, silty sand and clayey sand with occasional boulders. Based on our exploratory trenches, the thickness of alluvium can range from a few feet to more than 20 feet. Due to the relatively unconsolidated nature of the alluvial deposits, remedial grading will be necessary in areas to receive fill or structures.

3.5 Terrace Deposits (Qt)

We encountered a relatively thin layer of Terrace Deposits within the east central portion of the site underlying colluvium. These materials consist of medium dense to dense, damp to moist, light brown silty sand, occasionally slightly cemented. Remedial grading for this unit may be limited to the near surface materials that may possess pinhole pores. The remaining Terrace Deposits are considered suitable to receive fill or structures.

3.6 Santiago Formation (Ts)

We encountered the Eocene-age Santiago Formation below the undocumented fill in Boring B-5 and Trenches T-51, T-52 and T-56. The Santiago Formation, in general, consists of dense to very dense, massive, light brown, and gray, silty, fine to coarse sandstone and hard, olive-gray and brown claystone. The Santiago Formation is considered suitable for receiving fill loads and settlement sensitive structures. We do not expect to encounter the Santiago Formation during grading operations; however, we may encounter formational materials during trenching operations for utilities.

3.7 Granitic Rock, Tonalite (Kgr)

Cretaceous-age Granitic Rock (Tonalite) associated with the Peninsular Range Batholith underlies the surficial materials and is the predominant material throughout the project. The mostly massive granitic rock is characterized as coarse-grained, dark gray to grayish brown, and moderately strong to very strong. Additionally, the granitic rock is moderately to highly fractured and at various stages of weathering. The near surface materials are highly to moderately weathered and can be excavated by a very heavy excavation effort. The results of our airtrack drilling and the seismic refraction study indicates that bellow the relatively weathered zones, the bedrock becomes very strong and likely requires blasting during mass grading and improvement installations. As a result, oversize rocks will be generated that require special handling during the grading operation.

The rippability characteristics of the granitic rock are discussed in the *Rippability and Rock Considerations* section of this report. Granitic units generally exhibit good bearing and slope stability characteristics.

The soil derived from excavations within the decomposed granitic rock typically consist of low-expansive, silty, medium- to coarse-grained sands that should provide suitable foundation support in either a natural or properly compacted condition. We expect that excavations within the granitic rock will generate boulders and oversize materials (rocks greater than 12 inches in dimension) that will require special handling and placement.

4. RIPPABILITY AND ROCK CONSIDERATIONS

We performed a subsurface exploration program consisting of 10 air-track borings and 6 seismic refraction traverses, to aid in evaluating the rippability characteristics of the rock in proposed major cut areas. Air-track borings, utilizing an Ingersoll Rand ECM 370 equipped with a 3 ½ -inch diameter bit, were advanced in selected cut areas. Drill penetration rates were used to evaluate rock rippability and to estimate the depth at which difficult excavation will occur. Rock rippability is a function of natural weathering processes that can vary vertically and horizontally over short distances depending on jointing, fracturing, and/or mineralogic discontinuities within the bedrock. Southwest Geophysics performed 6 seismic traverses within the rock units to augment the air-track information. The report prepared by Southwest Geophysics is presented in Appendix C.

A frequently used guideline to compare rock rippability to drill penetration rate is that a penetration rate of approximately 0 to 20 seconds per foot (spf) generally indicates rippable material, 20 to 30 spf indicates marginally to non-rippable material, and greater than 30 spf indicates non-rippable rock. These general guidelines are typically based on drill rates using a rotary percussion drill rig similar to an Ingersoll Rand ECM 370 with a 3½-inch diameter drill bit.

The penetration rates (recorded in seconds per foot) for each air-track boring are presented in Appendix A, Figures A-62 through A-71.

The estimated thickness of rippable material for each boring in the rock units using 20 spf as the boundary between rippable and marginal to non-rippable rock is presented on the Geologic Map. The estimate is derived from a literal interpretation of the penetration rate from each boring log, based on the first occurrence where the penetration rate reaches 20 spf. Perspective contractors should use their own judgment to identify the penetration rate boundary between productive and non-productive ripping, and rippable and non-rippable rock.

Based on this study, we expect that the majority of the significant excavations within the development will experience very difficult ripping and/or blasting as excavations are extended beyond the rippable weathered mantle. Based on a penetration rate of 20 spf, and the seismic refraction profiles, the thickness of the rippable granitic rock mantle varies between 7 and 18 feet. A review of the seismic refraction data also indicates the depth of the "easy rippability" ranges from 5 to 9 feet and at two locations "moderate rippable" materials extended to approximately 18 to 32 feet (see Appendix C).

Blasting techniques can be expected to generate oversized rock (rocks greater than 12-inches in dimension), that will necessitate typical hard rock handling and placement procedures during grading operations.

Heavy ripping and/or blasting should also be expected at the surface in areas of concentrated rock outcroppings. Estimates of the anticipated volume of hard rock materials generated from proposed excavations should be evaluated based on the information from each boring, drill penetration rate and seismic refraction criteria acceptable to the contractor. Roadway/utility corridors 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 fragments.

Earthwork construction should be carefully planned to efficiently utilize available rock placement areas. Oversize materials should be placed in accordance with rock placement procedures presented in Appendix D of this report and governing jurisdictions. Crushing of oversight materials may be necessary to satisfy the placement requirements of this report.

5. GROUNDWATER

We encountered perched groundwater and/or seepage within exploratory trenches T-8, T-38 and T-39 and exploratory boring B-3 during our study. The groundwater/seepage was encountered within the surficial alluvial deposits, suggesting that the surface runoff of rainwater and irrigation is "perched" along the less impervious rock material and is migrating along the contact.

Due to the geologic conditions and the natural and artificial water sources inherent to the property, the presumed perched groundwater condition is expected to fluctuate seasonally. Subdrain systems will be necessary to intercept and convey groundwater migrating along the geological contacts. In particular, subdrains will be required in the main drainages and tributaries throughout the site. In addition, drained stability fills may be required in areas adjacent to avocado orchard or where natural seeps, springs, or runoff daylight near the ultimate graded surface. We will also recommend installing drains where seepage is encountered during the grading operations.

The low lying alluvial areas within the east central portion of the site, at the vicinity of boring B-3 and trench T-8 is designated as open space and will not be developed.

6. GEOLOGIC HAZARDS

6.1 Faulting and Seismicity

Review of geologic literature indicates no active or potentially faults traverse the property. The property is not located within a State of California Earthquake Fault Zone. The regional geologic map of the area shows an inactive fault in the central portion of the site. This fault trends east- northeast and considered inactive and not a constraint to the property. We do not expect building setbacks will be required for the fault.

According to the results of the computer program *EZ-FRISK* (Version 7.62), 9 known active faults are located within a search radius of 50 miles from the property. We used the 2008 USGS fault database that provides several models and combinations of fault data to evaluate the fault information. The nearest known active fault is the Newport-Inglewood (Offshore)/Rose Canyon Fault, located approximately 13 miles west of the site and is the dominant source of potential ground motion. Earthquakes that might occur on the Newport-Inglewood (Offshore)/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 Newport Inglewood (Offshore)/Rose Canyon Faults are 7.5 and 0.19 g, respectively. We used Boore-Atkinson (2008) NGA USGS2008, Campbell-Bozorgnia (2008) NGA USGS 2008, and Chiou-Youngs (2008) NGA acceleration-attenuation relationships in the calculation of the peak ground accelerations (PGA). Table 6.1.1 lists the estimated maximum earthquake magnitudes and PGA's for the most dominant faults for the site location.

TABLE 6.1.1
DETERMINISTIC SPECTRA SITE PARAMETERS

Fault Name	Distance from Site (miles)	Maximum Earthquake Magnitude (Mw)	Peak Ground Acceleration		
			Boore-Atkinson 2008 (g)	Campbell-Bozorgnia 2008 (g)	Chiou-Youngs 2008 (g)
Newport-Inglewood (Offshore)	13	7.5	0.24	0.18	0.23
Rose Canyon	13	6.9	0.20	0.19	0.20
Elsinore	16	7.8	0.24	0.17	0.13
Coronado Bank	28	7.4	0.15	0.10	0.12
Palos Verde, Connected	29	7.7	0.17	0.11	0.14
Earthquake Valley	32	6.8	0.12	0.08	0.07
San Jacinto	42	7.9	0.14	0.09	0.12
San Joaquin Hills	45	6.7	0.09	0.08	0.07
Palos Verde	47	7.3	0.10	0.07	0.07

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 mapped Quaternary fault is proportional to the fault slip rate. The program accounts for fault rupture length as a function of earthquake magnitude. 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 USGS2008, Campbell-Bozorgnia (2008) NGA USGS 2008, 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, 2008 (g)	Campbell-Bozorgnia, 2008 (g)	Chiou-Youngs, 2008 (g)
2% in a 50 Year Period	0.49	0.39	0.44
5% in a 50 Year Period	0.3.8	0.30	0.33
10% in a 50 Year Period	0.31	0.24	0.26

The California Geologic Survey (CGS) provides a computer program that calculates the ground motion for a 10 percent of probability of exceedence in 50 years based on the average value 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
0.25	0.27	0.31

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

6.2 Liquefaction

Liquefaction typically occurs when a site is located in a zone with seismic activity, on-site soil are cohesionless/low plasticity silt and clay, 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. Based on a review of the current grading plans, the potential for liquefaction to occur in the proposed development is very low.

6.3 Landslides

Examination of aerial 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.

7. CONCLUSIONS AND RECOMMENDATIONS

7.1 General

7.1.1 It is our opinion that no soil or geologic conditions were encountered during the investigation that would preclude the proposed development of the Eden Hills project provided the recommendations presented herein are followed and implemented during construction.

7.1.2 The site is underlain by surficial units that include undocumented fills, topsoil, colluvium, and alluvium. These surficial deposits are unsuitable in their present condition for support of fill and/or structural loads and will require remedial grading where improvements are planned. The upper portion of the Terrace Deposits may also require remedial grading depending on their condition as exposed during grading.

7.1.3 Potential geologic hazards at the site include seismic shaking. Based on our investigation and available geologic information, active or potentially active faults are not present underlying or trending toward the site. The regional geologic map of the area shows an inactive fault in the central portion of the site. This fault trends east- northeast and considered inactive and not a constraint to the property. We do not expect building setbacks will be required for the fault.

7.1.4 The presence of hard rock within proposed cut areas will require special consideration during site development. We expect that the majority of the proposed excavations will require moderate to heavy ripping with conventional heavy-duty equipment. Blasting is expected within the rock unit exposed throughout the site. In addition, heavy ripping and blasting will generate oversize materials and corestones that will require special handling and fill placement procedures. Oversize materials should be placed in accordance with Appendix D of this report.

7.1.5 The proposed residential structures and retaining walls may be supported on conventional or post tensioned foundation systems bearing in either competent formational materials or properly compacted fill.

7.1.6 Proper drainage should be maintained in order to preserve the engineering properties of the fill in both the building pads and slope areas. Recommendations for site drainage are provided herein.

7.2 Seepage/Groundwater Mitigation

7.2.1 The geologic units encountered on the site have permeability characteristics and/or fracture systems that could be susceptible under certain conditions to water seepage. Light to strong seepage were encountered in three of the trenches and one of the borings, all within the alluvial deposits. Although we did not observe seepage or springs within the granitic bedrock, we recommend that periodic observations be made by the soil engineer or engineering geologist during grading and/or construction for the presence of groundwater. The recommendations that follow provide for the removal of colluvial, alluvial and undocumented fill soils and the placement of a "canyon" subdrain within the bottom of the removal areas to reduce the potential for groundwater buildup within the canyon fills.

7.3 Subdrains

7.3.1 The geologic units encountered on the site have permeability characteristics and/or fracture systems that could be susceptible to groundwater transmission. Subdrains are recommended to collect subsurface water within areas of planned development. Subdrains should be connected to the storm drain system or any other approved drainage. The recommended canyon subdrain locations are presented on the Geologic Map, Figure 2 and 3. Figure 4 depicts a typical canyon subdrain detail.

7.3.2 The recommended subdrain locations shown on Figure 2 and 3 are based on anticipated site conditions prior to grading and are subject to change depending on the conditions encountered in the field.

7.3.3 The final segment of the subdrain should consist of non-perforated drainpipe. At the non-perforated/perforated interface, a seepage cutoff wall should be constructed on the downslope side of the junction in accordance with Figure 5. Subdrains that discharge into a natural drainage course or open space should be provided with a permanent headwall structure in accordance with Figure 6.

7.3.4 The final grading plans should show the location of all proposed subdrains. Upon completion of remedial excavations and subdrain installation, the project civil engineer should survey the drain locations and prepare an "as-built" map depicting their location and elevation.

7.4 Soil Characteristics

7.4.1 We expect the on-site soil is considered to be "expansive" and "non-expansive" (expansion index [EI] of greater than 20 and 20 or less, respectively) as defined by 2013 California

Building Code (CBC) Section 1803.5.3. Table 7.4 presents soil classifications based on the expansion index. A majority of the soil tested in our lab possess a “very low” to “low” expansion potential (expansion index of 50 or less). However, some soil encountered during grading may have an Expansion Index between 51 and 90 based on our observation.

TABLE 7.4

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

7.4.2 We expect the excavation of the surficial soil (undocumented fill, topsoil, alluvium, colluvium, weathered Terrace Deposits) can be performed with light to moderate effort with conventional heavy-duty earthmoving equipment. Unweathered Terrace Deposits and Santiago Formation will require moderate to heavy effort. Excavation of the weathered granitic rock is expected to require a very heavy effort to excavate. Less weathered and fresh bedrock may require blasting or specialized rock breaking techniques to efficiently excavate and handle.

7.4.3 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 Table B-IV and indicate that the on-site materials at the locations tested possess “negligible” sulfate exposure to concrete structures as defined by 2013 CBC Section 1904 and ACI 318-11 Sections 4.2 and 4.3. The presence of water-soluble sulfates is not a visually discernible characteristic; therefore, other soil samples from the site could yield different concentrations. Additionally, over time landscaping activities (i.e., addition of fertilizers and other soil nutrients) may affect the concentration.

7.4.4 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.

7.5 Grading

7.5.1 Development of the site as proposed will generate a relatively large volume of oversize rock as well as shot rock. Earthwork considerations which need to be addressed within the development schedule are summarized as follows:

- Grading and blasting operations should be designed to generate a sufficient quantity of granular material for use as low expansive capping material. Where this is not possible, rock crushing may be required.
- Consideration should be given to stockpiling select materials to be utilized for capping.
- Oversize rock should be placed in deeper fill areas in accordance with the *Recommended Grading Specifications* presented in Appendix D.
- Appropriate remediation and/or backfilling of any old abandoned mines or prospect pits should be implemented during site development

7.5.2 Grading should be performed in accordance with the *Recommended Grading Specifications* contained in Appendix D. Where the recommendations of Appendix D conflict with this report, the recommendations of this report should take precedence.

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

7.5.4 Site preparation should begin with the removal of all deleterious material and vegetation. The depth of removal should be such that material exposed in cut areas or soil to be used as fill is relatively free of organic matter. Material generated during stripping and/or site demolition should be exported from the site.

7.5.5 We expect that during the excavation of the major cut areas, a moderate quantity of surface and “floater” rocks in excess of 4 feet in size will be generated. In our opinion, these rocks are suitable for placement in the fill areas, provided the placement is accomplished in accordance with the recommendations presented in Appendix D. In addition, utilization of as many boulders as practical for landscape purposes should be considered. Excavation of the deeper bedrock units should produce a relatively lower quantity of large “floater” rocks, if an efficient blasting program is implemented. The proposed blasting program should also be planned to assist in the generation of “fines”. If possible, the grading operation should attempt to reserve sufficient “soil” fill for use in capping the “rock” fills as discussed in Appendix D and to replace trench excavation material which is considered too rocky to be used as trench backfill.

7.5.6 Potentially compressible surficial soil (undocumented fill, topsoil, colluvium, alluvium, weathered Terrace Deposits) within areas of planned grading should be removed to firm natural ground prior to placing additional fill and/or structural loads. The actual extent of unsuitable soil removals should be evaluated in the field by the soil engineer and/or engineering geologist. Overly wet, surficial materials will require drying and/or mixing with drier soil to facilitate proper compaction. Figure 7 depicts construction detail for lateral extent of removal of unsuitable materials.

7.5.7 The site should then be brought to final subgrade elevations with structural fill compacted in layers. In general, soil native to the site are suitable for re-use as fill if free from vegetation, debris and other deleterious material. Layers of fill should be no thicker than will allow for adequate bonding and compaction. Fill materials, including backfill and scarified ground surfaces, should be compacted to at least 90 percent of maximum dry density, at or above, optimum moisture content, as determined in accordance with ASTM D 1557. Fill materials near and/or below optimum moisture content may require additional moisture conditioning prior to placing additional fill.

7.5.8 Consideration should be given to undercutting and replacing with compacted "soil fill" (no rocks greater than 12 inches in maximum dimension) any proposed cut areas located on the hard bedrock outcrops. The maximum dimensions should be reduced to 6 inches within building pad areas. The depth of undercutting should be at least 3 feet below the ultimate pad finish grade and 1 foot below the deepest utility lines, or, alternatively, to sufficient depth such that underground utilities can be constructed without the need for additional blasting or heavy ripping after the mass grading has been accomplished. Within rock cut areas where only landscaping is proposed, the project landscape architect should be consulted to determine an appropriate depth of undercut, if any. To reduce the potential for differential settlement, we recommend that the cut portions of the cut/fill transition building pads be undercut at least 3 feet and replaced with properly compacted fill soils. The undercut should extend from the back of the pad to the street and be graded at a gradient of at least 1 percent toward the street.

7.5.9 Deeper undercutting of street areas should be considered to facilitate the excavation of underground utilities where the streets are located in cut areas composed of marginally- to non-rippable rock. If other subsurface improvements (i.e. landscape zones, swimming pools, retaining wall footings) are planned beyond areas that are not undercut, consideration should be given to undercutting these areas as well.

7.5.10 Where practical, the upper 3 feet of building pads and 12 inches in pavement areas should be composed of properly compacted fill with "very low" to "low" expansive soil. The more highly expansive fill soil should be placed in the deeper fill areas and properly compacted. "Very low" to "low" expansive soil are defined as those soil that have an Expansion Index of 50 or less as defined by 2010 California Building Code (CBC) Section 1803.5.3.

7.5.11 To the extent possible, oversize materials greater than 12 inches in maximum dimension should not be placed within 10 feet of finish grade and a minimum of 2 feet below the deepest utility. Material greater than 6 inches in maximum dimension should not be placed within 3 feet of finish grade in building pad areas. This restriction may be modified within the requirements of the County of San Diego at the discretion of the owner.

7.6 Earthwork Grading Factors

7.6.1 Estimates of embankment shrink-bulk factors are based on comparing laboratory compaction tests with the density of the material in its natural state and experience with similar soil types. It should be emphasized that variations in natural soil density (e.g. undocumented fills), as well as in compacted fill, render shrinkage value estimates very approximate. As an example, the contractor can compact fills to any relative compaction of 90 percent or higher of the laboratory maximum dry density. Thus, the contractor has at least a 10 percent range of control over the fill volume. Based on the work performed to date and considering the above discussion, the following earthwork factors presented on Table 7.6 may be used as a basis for estimating how much the on-site soil may shrink or swell when removed from their natural state and placed in compacted fills.

TABLE 7.6

Soil Unit	Shrink-Swell Factors
Undocumented Fill, Topsoil, Alluvium, and Colluvium,	5 to 10 Percent Shrinkage
(Rippable) Santiago Formation, and Granitic rock (Tonalite)	10 to 15 Percent Bulk
(Non-rippable) Granitic rock (Tonalite)	20 to 25 Percent Bulk

7.7 Slope Stability

7.7.1 Slope stability analyses were performed utilizing average drained direct shear strength parameters obtained from our laboratory testing and our experience with similar soil conditions. These analyses indicate that the proposed 2:1 (horizontal:vertical) fill slopes constructed of on-site materials should have calculated factors of safety of at least 1.5 under static conditions for both deep-seated failure and shallow sloughing conditions. The surficial slope stability analyses indicate the planned 2:1 slopes possess a factor of safety of at least 1.5 as required by current County of San Diego guidelines as presented in the referenced reports. The localized sloughing may occur due to heavy rain fall, over-irrigation, allowing water flowing from the top of the slope and lack of maintenance. These surficial instabilities, if they occur, should be immediately repaired and fixed to reduce the potential for progressive failure. Slope stability calculations for deep-seated and surficial fill slope stability are presented on Figures 8 and 9, respectively.

7.7.2 Fill slopes should be constructed such that the materials, within a zone measured horizontally back from the face of slope for a distance equal to the height of slope, are generally comprised of granular soils and/or soil/rock fills with minimum equivalent strength parameters of Ø' = 32 degrees and C' = 300 psf. However, the outer 15 feet of fill slopes will be restricted to materials composed of properly compacted granular "soil" fill to reduce the potential for surface sloughing. In general, soil with an Expansion Index of less than 90 or at least 35 percent sand size particles should be acceptable as "granular" fill. Soil of questionable strength to satisfy surficial stability should be tested in the laboratory for acceptable drained shear strength.

7.7.3 Fill slopes should be overbuilt at least 3 feet horizontally, and cut back to the design finish grade. As an alternative, slopes should be compacted by backrolling with a loaded sheepsfoot roller at vertical intervals not to exceed 4 feet and should be track-walked at the completion of each slope such that the fill soil are uniformly compacted to at least 90 percent relative compaction to the face of the finished slope.

7.7.4 Cut slopes excavated in the granitic rocks do not lend themselves to conventional stability analyses. However, the results of our field investigation, our experience in the general area and the examination of the existing slopes adjacent to the property indicate that the proposed 2:1 (horizontal:vertical) cut slopes should be stable with respect to deep-seated failure and surficial sloughage up to the proposed maximum height of 50 feet.

7.7.5 Where cuts exceed the maximum depth of the weathered portion of the granitic rocks, heavy blasting will be required to break the fresh, very hard rock. Overblasting of cut slopes should not be permitted. Loose rock and blasting debris should be removed from the faces of finish graded cut slopes. No loose rock fragments greater than 6 inches in maximum dimension should be left on the slope surface. Tops of cut slopes should be cleared of loose boulders and should be "rounded" within the exposed topsoil horizon.

7.7.6 The cut slope excavations should be observed during grading by an engineering geologist to verify that soil and geologic conditions do not differ significantly from those anticipated. In the event that adverse conditions are observed, stabilization recommendations can be provided.

7.7.7 All slopes should be landscaped with drought-tolerant vegetation, having variable root depths and requiring minimal landscape irrigation. In addition, all slopes should be drained and properly maintained to reduce erosion.

7.8 Seismic Design Criteria

7.8.1 We used the computer program *Seismic Hazard Curves and Uniform Hazard Response Spectra*, provided by the United States Geologic Survey (USGS), to evaluate design parameters. Table 7.8.1 summarizes site-specific design criteria obtained from the 2013 California Building Code (CBC; Based on the 2012 International Building Code [IBC] and ASCE 7-10), Chapter 16 Structural Design, Section 1613 Earthquake Loads. The short spectral response uses a period of 0.2 second. The planned buildings and improvements can be designed using a Site Class B where the fill thickness is less than 10 feet, Site Class C where the fill soil is 10 feet or greater and less than 35 feet or Site Class D for building pads with fill 35 feet or greater. We will evaluate the structure site class for each residential building once the final grading has been completed.

TABLE 7.8.1
2013 CBC SEISMIC DESIGN PARAMETERS

Parameter	Value	Value	Value	IBC-06 Reference
Site Class	B	C	D	Section 1613.3.2
Fill Thickness, T (feet)	T<10	10≤T <35	T≥35	--
Spectral Response – Class B (0.2 sec), S_S	1.006	1.006	1.006	Figure 1613.3.1(1)
Spectral Response – Class B (1 sec), S_1	0.393	0.393	0.393	Figure 1613.3.1(2)
Site Coefficient, F_a	1.000	1.000	1.097	Table 1613.3.3(1)
Site Coefficient, F_v	1.000	1.407	1.614	Table 1613.3.3(2)
Maximum Considered Earthquake Spectral Response Acceleration (0.2 sec), S_{MS}	1.006	1.000	1.104	Section 1613.3.3 (Eqn 16-37)
Maximum Considered Earthquake Spectral Response Acceleration – (1 sec), S_{M1}	0.393	0.553	0.634	Section 1613.3.3 (Eqn 16-38)
5% Damped Design Spectral Response Acceleration (0.2 sec), S_{DS}	0.671	0.671	0.736	Section 1613.3.4 (Eqn 16-39)
5% Damped Design Spectral Response Acceleration (1 sec), S_{D1}	0.262	0.369	0.423	Section 1613.3.4 (Eqn 16-40)

7.8.2 Table 7.8.1 presents additional seismic design parameters for projects located in Seismic Design Categories of D through F in accordance with ASCE 7-10 for the mapped maximum considered geometric mean (MCE_G).

TABLE 7.8.2
2013 CBC SITE ACCELERATION DESIGN PARAMETERS

Parameter	Value	ASCE 7-10 Reference
Mapped MCE_G Peak Ground Acceleration, PGA	0.376g	Figure 22-7
Site Coefficient, F_{PGA}	1.124	Table 11.8-1
Site Class Modified MCE_G Peak Ground Acceleration, PGA_M	0.423g	Section 11.8.3 (Eqn 11.8-1)

7.8.3 Conformance to the criteria in Tables 7.8.1 and 7.8.2 for seismic design does not constitute any kind of guarantee or assurance that significant structural damage or ground failure will not occur if a large earthquake occurs. The primary goal of seismic design is to protect life, not to avoid all damage, since such design may be economically prohibitive.

7.9 Foundation and Concrete Slabs-On-Grade Recommendations

7.9.1 The following foundation recommendations are for proposed one- to two-story residential structures. The foundation recommendations have been separated into three categories based on either the maximum and differential fill thickness or Expansion Index. The foundation category criteria are presented in Table 7.9.1.

TABLE 7.9.1
FOUNDATION CATEGORY CRITERIA

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

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

7.9.3 Table 7.9.2 presents minimum foundation and interior concrete slab design criteria for conventional foundation systems.

TABLE 7.9.2
CONVENTIONAL FOUNDATION RECOMMENDATIONS BY CATEGORY

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

7.9.4 The embedment depths presented in Table 7.9.2 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 typical wall/column footing dimension detail depicting lowest adjacent grade is shown as Figure 10.

7.9.5 The concrete slab-on-grade should be a minimum of 4 inches thick for Foundation Categories I and II and 5 inches thick for Foundation Category III.

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

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

7.9.8 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 2013 California Building Code (CBC Section 1808.6). Although this procedure was developed for expansive soil conditions, it can also be used to reduce the potential for foundation distress due to differential fill settlement. The post-tensioned design should incorporate the geotechnical parameters presented on Table 7.9.3 for the particular Foundation Category designated. The parameters presented in Table 7.9.3 are based on the guidelines presented in the PTI, Third Edition design manual. The foundations for the post-tensioned slabs should be embedded in accordance with the recommendations of the structural engineer.

TABLE 7.9.3
POST-TENSIONED FOUNDATION SYSTEM DESIGN PARAMETERS

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

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

- The criteria presented in Table 7.9.3 are still applicable.
- Interior stiffener beams should be used for Foundation Categories II and III.
- The width of the perimeter foundations should be at least 12 inches.
- The perimeter footing embedment depths should be at least 12 inches, 18 inches and 24 inches for foundation categories I, II, and III, respectively. The embedment depths should be measured from the lowest adjacent pad grade.

7.9.10. Foundation systems for the lots that possess a foundation Category I and a “very low” expansion potential (Expansion Index of 20 or less)) can be designed using the method described in Section 1808 of the 2013 CBC. If post-tensioned foundations are planned, an alternative, commonly accepted design method (other than PTI Third Edition) can be used.

However, the post-tensioned foundation system should be designed for a total and differential deflection of 1 inch. Geocon Incorporated should be contacted to review the plans and provide additional information, if necessary.

7.9.11 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.

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

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

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

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

7.9.16 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.

7.9.17 Footings that must be placed within seven feet of the top of slopes should be extended in depth such that the outer bottom edge of the footing is at least seven feet horizontally inside the face of the slope.

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

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

- For fill slopes less than 20 feet high, building footings should be deepened such that the bottom outside edge of the footing is at least 7 feet horizontally from the face of the slope.

- When located next to a descending 3:1 (horizontal:vertical) fill slope or steeper, the foundations should be extended to a depth where the minimum horizontal distance is equal to H/3 (where H equals the vertical distance from the top of the fill slope to the base of the fill soil) with a minimum of 7 feet but need not exceed 40 feet. The horizontal distance is measured from the outer, deepest edge of the footing to the face of the slope. An acceptable alternative to deepening the footings would be the use of a post-tensioned slab and foundation system or increased footing and slab reinforcement. Specific design parameters or recommendations for either of these alternatives can be provided once the building location and fill slope geometry have been determined.

- 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 should be designed assuming that the adjacent soil provides no lateral support. This recommendation applies to fill slopes up to 30 feet in height, and cut slopes regardless of height. For swimming pools located near the top of fill slopes greater than 30 feet in height, additional recommendations may be required and Geocon 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 which would permit some lateral soil movement without causing extensive distress. Geocon Incorporated should be consulted for specific recommendations.

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

7.9.21 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 placed in the middle of the slab to reduce the potential for cracking. In addition, concrete flatwork should be provided with crack control joints to reduce and/or control shrinkage cracking. Crack control spacing should be determined by the project structural engineer based on 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 checked prior to placing concrete. Base or sand bedding is not required beneath the flatwork.

7.9.22 Even with the incorporation of the recommendations within this report, exterior concrete flatwork has a potential of experiencing some movement due to swelling or settlement; therefore, welded wire mesh should overlap continuously in flatwork. Additionally, flatwork should be structurally connected to curbs, where possible.

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

7.10 Retaining Walls and Lateral Loads

7.10.1 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 expansion index of 50 or less. For those buildings with finish-grade soils having an expansion index greater than 50 and/or where backfill materials do not conform to the criteria herein, Geocon Incorporated should be consulted for additional recommendations.

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

7.10.3 The structural engineer should determine the seismic design category for the project in accordance with Section 1613 of the CBC. If the project possesses a seismic design category of D, E, or F, retaining walls that support more than 6 feet of backfill should be designed with seismic lateral pressure in accordance with Section 18.3.5.12 of the 2013 CBC. The seismic load is dependent on the retained height where H is the height of the wall, in feet, and the calculated loads result in pounds per square foot (psf) exerted at the base of the wall and zero at the top of the wall. A seismic load of 16H should be used for design. We used the peak ground acceleration adjusted for Site Class effects, PGA_M, of 0.38g calculated from ASCE 7-10 Section 11.8.3 and applied a pseudo-static coefficient of 0.33.

7.10.4 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 11 presents a typical retaining wall drainage detail. If conditions different than those described are expected, or if specific drainage details are desired, Geocon Incorporated should be contacted for additional recommendations.

7.10.5 In general, wall foundations founded in properly compacted fill or formational materials should possess a minimum depth and width of one foot and may be designed for an allowable soil bearing pressure of 2,000 psf, provided the soil within three feet below the base of the wall has an expansion index of 90 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, Geocon Incorporated should be consulted where such a condition is expected.

7.10.6 Footings that must be placed within seven feet of the top of slopes should be extended in depth such that the outer bottom edge of the footing is at least seven feet horizontally inside the face of the slope.

7.10.7 For resistance to lateral loads, an allowable passive earth pressure equivalent to a fluid with a density of 300 pcf is recommended for footings or shear keys poured neat against properly compacted granular fill soils or undisturbed natural soils. The allowable passive pressure assumes a horizontal surface extending away from the face of the wall at least 5 feet or three times the height of surface generating the passive pressure, whichever is greater. For 2:1 (H:V) sloping conditions in front of the surface generating the passive pressure, an allowable passive earth pressure of 200 pcf is recommended. The upper 12 inches of material not protected by floor slabs or pavement should not be included in the design for lateral resistance. A 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.

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

7.11 Preliminary Pavement Recommendations

7.11.1 The final pavement sections for parking lots and roadways should be based on the R-Value of the subgrade soil 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. We calculated the flexible pavement sections in general conformance with the Caltrans Method of Flexible Pavement Design (Highway Design Manual, Section 608.4) Based on the results of our experience with similar soil types 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 7.11.1.

TABLE 7.11.1
PRELIMINARY FLEXIBLE PAVEMENT SECTIONS

Location	Assumed Traffic Index	Assumed Subgrade R-Value	Asphalt Concrete (inches)	Class 2 Aggregate Base (inches)
Parking stalls for automobiles and light-duty vehicles	5.0	20	3	7
Driveway areas within industrial pads	6.0	20	3.5	10
Roadways	7.0	20	4	12
Major Roadways	8.0	20	5	14

7.11.2 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.

7.11.3 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.

7.11.4 A rigid Portland cement concrete (PCC) pavement section should be placed in driveway entrance aprons and trash bin loading/storage areas. The concrete pad for trash truck areas should be large enough such that the truck wheels will be positioned on the concrete during loading. We calculated the rigid pavement section in general conformance with the procedure recommended by the American Concrete Institute report ACI 330R-01 Guide for Design and Construction of Concrete Parking Lots using the parameters presented in Table 7.11.2.

TABLE 7.11.2
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

7.11.5 Based on the criteria presented herein, the PCC pavement sections should have a minimum thickness as presented in Table 7.11.3.

TABLE 7.11.3
PRELIMINARY RIGID PAVEMENT RECOMMENDATIONS

Location	Portland Cement Concrete (inches)
Automobile Parking Areas	6
Trash and Heavy Truck and Fire Lane Areas	7

7.11.6 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).

7.11.7 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.

7.11.8 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.

7.11.9 To provide load transfer between adjacent pavement slab sections, a trapezoidal-keyed construction joint is recommended. 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.

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

7.12 Slope Maintenance

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

7.12.2 From a geotechnical engineering standpoint, the building pads will be constructed for the acceptable purpose for human occupancy provided the recommendations provided herein are adhered to during the construction operations.

7.13 Site Drainage and Moisture Protection

7.13.1 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 2013 CBC 1804.3 or other applicable standards. In addition, surface drainage should be directed away from the top of slopes into swales or other controlled drainage devices. Roof and pavement drainage should be directed into conduits that carry runoff away from the proposed structure.

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

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

7.13.4 If detention basins, bioswales, retention basins, water infiltration, low impact development (LID), or storm water management devices are being considered, Geocon Incorporated should be retained to provide recommendations pertaining to the geotechnical aspects of possible impacts and design.

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

7.14 Grading and Foundation Plan Review

7.14.1 Geocon Incorporated should review the grading plans and foundation plans, if prepared, for the project prior to final design submittal to check whether additional analysis and/or recommendations are required.

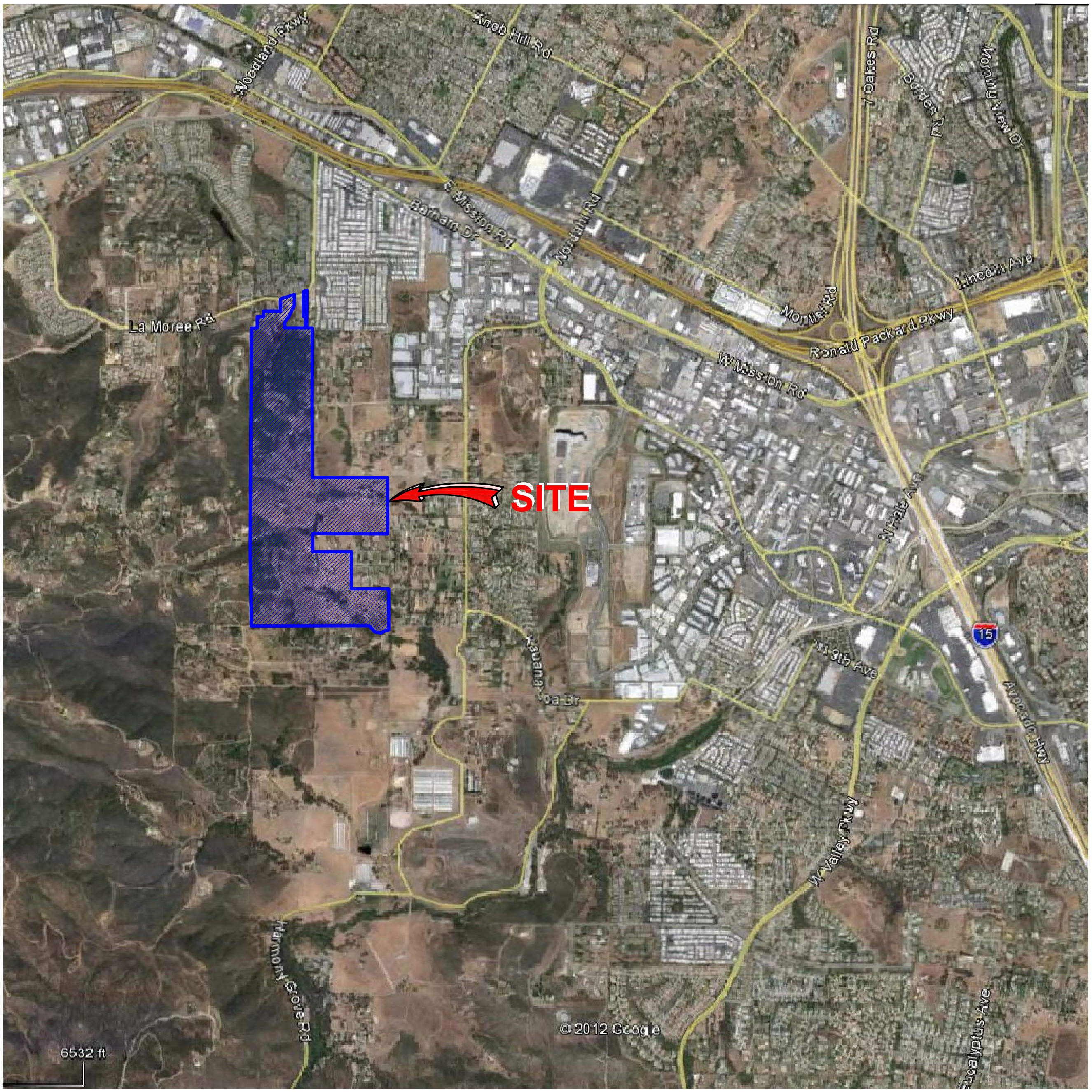
LIMITATIONS AND UNIFORMITY OF CONDITIONS

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

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

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

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



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NO SCALE

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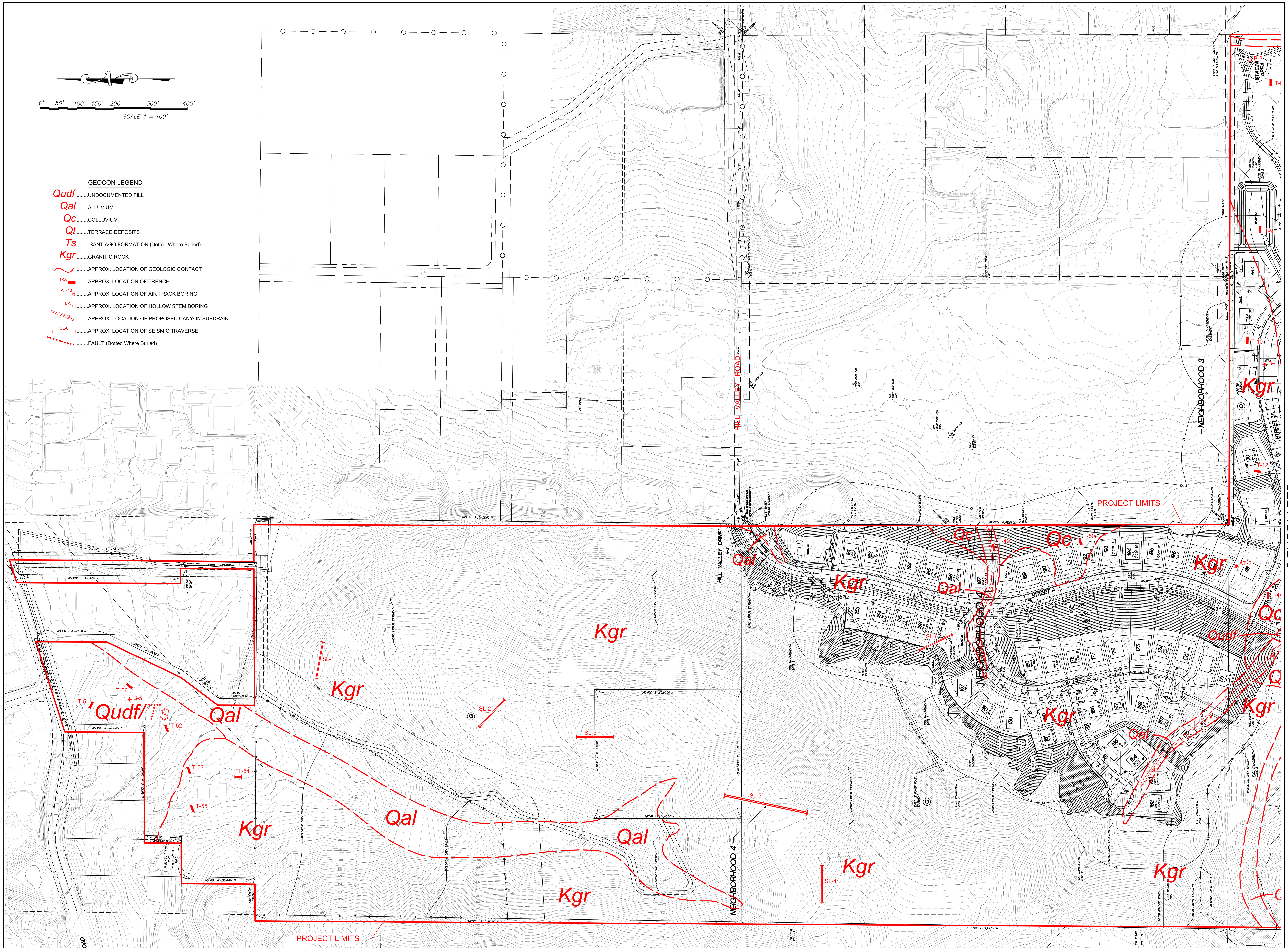
VICINITY MAP

VALIANO (EDEN HILLS)
SAN DIEGO COUNTY, CALIFORNIA

AS / RA		DSK/GTYPD	DATE 05 - 13 - 2014	PROJECT NO. G1416 - 52 - 03	FIG. 1



GEOLOGIC MAP

VALIANO (EDEN HILLS)
SAN DIEGO COUNTY, CALIFORNIA

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SCALE	1" = 100'	DATE	05 - 13 - 2014
PROJECT NO.	G1416 - 52 - 03	FIGURE	2
SHEET	1 OF 2		