UPDATE
GEOTECHNICAL INVESTIGATION

VALIANO (EDEN HILLS) –
48-ACRE FINES
SAN DIEGO COUNTY, CALIFORNIA

GEOCON
INCORPORATED

GEOTECHNICAL
ENVIRONMENTAL
MATERIALS

PREPARED FOR

INTEGRAL PARTNERS FUNDING, LLC
ENCINITAS, CALIFORNIA

MAY 13, 2014
PROJECT NO. G1416-52-03

Project No. G1416-52-03
May 13, 2014

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

Attention: Mr. Gil Miltenberger

Subject: UPDATE GEOTECHNICAL INVESTIGATION
VALIANO (EDEN HILLS) – 48-ACRE FINES
SAN DIEGO COUNTY, CALIFORNIA

Dear Mr. Miltenberger:

In accordance with the authorization of our Change Order dated December 4, 2012, 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

Michael C. Ertwine	Ali Sadr	Shawn Foy Weedon
PG 9027	CEG 1778	GE 2714

PROFESSIONAL GEOLOGIST ★ STATE OF CALIFORNIA ★ MICHAEL C. ERTWINE No. 9027

PROFESSIONAL GEOLOGIST ★ STATE OF CALIFORNIA ★ ALI SADR No. 1778 CERTIFIED ENGINEERING GEOLOGIST

REGISTERED PROFESSIONAL ENGINEER ★ STATE OF CALIFORNIA ★ SHAWN WEEDON No. 2714 GEOTECHNICAL

MCE:AS:SFW:dmc

(e-mail) Addressee
(2/del) Fuscoe Engineering
Attention: Mr. Ken Kozlik

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UPDATE GEOTECHNICAL INVESTIGATION

1. PURPOSE AND SCOPE

This report presents the results of our update geotechnical investigation for the Eden Hills 48-acre Fines 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, Figure 2 (map pocket).

The scope of our investigation included a geologic reconnaissance, 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 and published geologic reports. 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. *Preliminary Geotechnical Investigation, Eden Hills - 48- Acre Fines, San Diego County, California,* prepared by Geocon Incorporated, dated December 12, 2012 (Project No. G1416-52-03).

3. *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 13, 2013 (Project No. G1416-52-03).

4. *Planning and Development Services (PDS) Planning and SEQA Comments,* dated December 4, 2013 (Project No. PDS2013-SP-13-001).

Our field investigation included geologic mapping, excavating 12 backhoe trenches, and performing three seismic traverses. Appendix A provides a discussion of the field investigation and logs of the backhoe trenches. The Geologic Map (Figure 2, map pocket) presents the approximate locations of the exploratory excavations. We performed laboratory tests on soil samples obtained from the exploratory excavations to evaluate pertinent physical and chemical properties for engineering analysis. Appendix B presents the results of the laboratory tests. We contracted Southwest

Geophysics to perform the seismic refraction survey. Appendix C present the resulting seismic refraction report.

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 of land with the assessor's parcel numbers (APNs) 232-500-18, through -23 and 253-031-41 totaling approximately 48 acres of partially developed land. The property is bounded to the north by Mount Whitney Road, to east by Country Club Drive, to the west by essentially undeveloped land, and to the south by the existing residence and an equestrian park. An SDG&E easement traverses through the southern portion of the site. A residential development occupies the southeastern corner of the site with a barn, mobile home trailer, and equestrian stables and corrals. The site consists of a lone nob which slopes to the east, north and south with three converging drainages which confluence within the central portion of the property. We understand the central drainage is located within the protected National Wetlands Inventory. A pond exists in the southeastern portion of the property, west of the equestrian area.

Elevations at the site range from approximately 614 feet above mean sea level (MSL) near the south central portion of the site to 716 feet above MSL near the central portion of the property. A residential structure and the operational equestrian park are located within the project site. The conceptual development plan provided for our use indicates that it will consist of the construction of a residential subdivision with backbone and in-tract improvements.

Based on the grading plans, Lots 229 through 297 will be constructed on the property with associated roadways, improvements, and landscaping. In addition, a community park is planned on the southeastern portion, open space in the center, and water quality basins on the north, east, and southeast. We understand the grading of the site will consist of maximum cuts and fills of approximately 28 feet and 16 feet, respectively, with cut and fill slopes having a maximum height of about 25 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 and alluvium. The formational units include Cretaceous-age granitic rock (Tonalite) and Mesozoic-age Metamorphic Rock Undivided. The surficial soil types and geologic units encountered are described herein in order of increasing age. The approximate extent of these deposits, excluding topsoil, is shown on the Geologic Map, Figure 2.

3.1 Undocumented Fill (Qudf)

Although we did not encounter undocumented fill in the backhoe trenches, we were able to map it in several locations on the property. The areas are typically associated with the existing residence and equestrian activities. It is not unusual to encounter pockets of undocumented fill and debris at random locations placed during agricultural or historic farming 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 to reddish, silty to clayey, fine to medium sand. Topsoil is compressible in its present condition and remedial grading is required within areas of planned development.

3.3 Colluvium (Qc)

We encountered colluvial deposits along the flanks of the slopes, overlying the formational rock units. These deposits generally consist of loose to dense clayey sand. Colluvium is a surficial accumulation of thickened topsoil and detritus transported by gravity. Based on our exploratory trenches, the thickness of colluvium can range from a few feet to more than 8 feet. Due to the unconsolidated condition of the colluvium, remedial grading will be required to provide suitable support for placement of compacted fill or structural improvements.

3.4 Alluvium (Qal)

We expect alluvium exists in the central drainage at the site. We were not allowed to enter the central drainage area during our investigation. Alluvium generally varies in depth depending upon the size of the canyon. We expect the alluvium consists of sandy clay, silty sand and clayey sand with occasional boulders with an estimated thickness which could range from a few feet to more than 20 feet within the center of the canyon. A review of the conceptual development plan indicates that grading in the alluvial areas will be limited.

3.5 Granitic Rock, Tonalite (Kgr)

Cretaceous-age Granitic Rock (Tonalite) associated with the Peninsular Range Batholith underlies the surficial materials in the center and northern portions of the property. The mostly massive granitic rock is characterized as medium- to coarse-grained, dark gray to grayish brown, and weak to moderately 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 using very heavy excavation effort. The results of our seismic refraction study indicate that the bedrock becomes very strong and likely requires blasting during mass grading and improvement installations below the relatively weathered zones. The rippability characteristics of the granitic rock are discussed herein. Granitic units generally exhibit good bearing and slope stability characteristics.

The soil derived from excavations within the decomposed granitic rock typically possesses a "very low" to "low" expansion potential (expansion index of 50 or less). The rock generally excavates as silty, medium- to coarse-grained sand 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.

3.6 Metamorphic Rock – Undivided (Mzu)

We encountered Cretaceous/Jurassic-age (Mesozoic) metasedimentary/metavolcanic rock which underlies the surficial materials and is located within the southern portion of the site. The mostly massive metamorphic rock is characterized as fine coarse-grained, dark gray to grayish brown, weak to moderately weak in strength 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. Below the relatively weathered zones, the bedrock becomes very strong and may require 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 metamorphic rock are discussed herein. These units generally exhibit good bearing and slope stability characteristics.

The soil derived from excavations within the decomposed granitic rock typically possesses a "very low" to "low" expansion potential (expansion index of 50 or less). The rock generally excavates as silty, fine- to coarse-grained sand that should provide suitable foundation support in either a natural or properly compacted condition.

4. RIPPABILITY AND ROCK CONSIDERATIONS

We performed three Seismic refraction traverses, to aid in evaluating the rippability characteristics of the rock in proposed major cut area. 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 3 seismic traverses within the central nob to evaluate the rippability characteristics of the bedrock at the proposed major cut. The report prepared by Southwest Geophysics is presented in Appendix C.

Based on this study, we expect that the majority of the significant excavations within the central portion of the nob will experience very difficult ripping and/or blasting as excavations extend beyond the rippable weathered mantle. The northern and southern flanks of the ridge appear to be more rippable based on the results from the seismic line SL-3.

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 the 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 oversized materials may be necessary to satisfy the placement requirements of this report.

5. GROUNDWATER

We did not encounter groundwater or seepage at the surface during our site investigation to show on the Geologic Maps. However, water may be encountered within the alluvium in the central portion of the project. Due to the geologic conditions and the natural and artificial water sources inherent to the property, the perched groundwater condition is expected to occur and fluctuate seasonally. Subdrain systems may be necessary to intercept and convey groundwater migrating along the geological contacts. Groundwater and seepage is dependent on seasonal precipitation, irrigation, land use, among other factors, and varies as a result. Proper surface drainage will be important to future performance of the project. The remedial grading in the low lying alluvial areas within the east central portion of the site, will be limited due to presence of groundwater. As a result some compressible alluvial soils will likely be left in place.

In addition, canyon subdrains will be installed during the grading operations to mitigate the potential for groundwater build up. We will also recommend installing drains where seepage is encountered during the grading operations. The planned drains will be surveyed for location and elevation and will be shown on the as-graded geologic map subsequent to the grading operations. We opine groundwater will not affect the planned residences if the recommendations of the referenced report are followed.

6. GEOLOGIC HAZARDS

6.1 Faulting and Seismicity

Review of geologic literature indicates active or potentially faults do not traverse the property. The property is not located within a State of California Earthquake Fault Zone. 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. Based on this database, the nearest known active fault is the Newport-Inglewood/Rose Canyon Faults, 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/Rose Canyon Faults 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/Rose Canyon Faults are 7.5 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-2008, and Chiou-Youngs (2007) NGA USGS2008 acceleration-attenuation relationships.

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 2007 (g)
Newport-Inglewood (Offshore)	13	7.5	0.19	0.16	0.19
Rose Canyon	13	6.9	0.15	0.14	0.19
Elsinore	18	7.8	0.11	0.08	0.09
Coronado Bank	28	7.4	0.15	0.10	0.12
Palos Verdes, Connected	28	7.7	0.13	0.10	0.11
Earthquake Valley	32	6.8	0.08	0.06	0.05
San Jacinto	42	7.9	0.09	0.07	0.09
San Joaquin Hills	46	6.7	0.06	0.06	0.05
Palos Verdes	47	7.3	0.06	0.05	0.05

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 faults slip rate. The program accounts for earthquake magnitude as a function of fault rupture length, 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 2008, Campbell-Bozorgnia (2008) NGA USGS 2008, and Chiou-Youngs (2007) USGS2008 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, 2007 (g)
2% in a 50 Year Period	0.36	0.35	0.39
5% in a 50 Year Period	0.27	0.26	0.28
10% in a 50 Year Period	0.21	0.20	0.20

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

The information presented in Tables 6.1.2 and 6.1.3 provides the probability analyses of site accelerations that could affect the property. For example, a 2 percent in a 50 year period acceleration of 0.39g means that there is a 2 percent probability that an earthquake will cause a higher acceleration than 0.39g at the property within the next 50 years.

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. The potential for liquefaction within the area of the planned structures is considered very low due to the dense formational material encountered.

The area susceptible for liquefaction is at the vicinity of the alluvium in the central portion of the property. However, this area is designated as open space and structures are not planned in this area. We expect we can remove the alluvium during the remedial grading operations to expose the underlying granitic rock. Subsequent to the grading operations, the planned residences will possess compacted fill overlying granitic rock. Therefore, based on a review of the current grading plans and our previous reports, the house pads are not in a location susceptible for liquefaction during a seismic event.

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.

6.4 Seiches and Tsunamis

Seiches are caused by the movement of an inland body of water due to the movement from seismic forces. The potential of seiches to occur is considered to be very low due to the absence of a nearby inland body of water. The site is not located near an inland body of water. Therefore, the risk of a seiche from flooding is considered low.

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 State of California Tsunami Inundation Map for Emergency Planning (CGS, 2009) does not show the property within a tsunami inundation zone. The site is located approximately 11 miles from the Pacific Ocean at an elevation of approximately 650 feet above Mean Sea Level. Therefore, the risk of tsunamis affecting the site is negligible.

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, 48-Acre Fines 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 fill, 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 weathered granitic rock and metamorphic rock may also require remedial grading depending on the condition as exposed during grading operations.

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.

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 in the central part of 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 and/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 Soil Characteristics

7.2.1 We expect the on-site soil is considered to be "non-expansive" and "expansive" (expansion index [EI] of 20 or less and greater than 20, respectively) as by 2013 California Building Code (CBC) Section 1803.5.3. Table 7.2.1 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 7.2.1
EXPANSION CLASSIFICATION BASED ON EXPANSION INDEX

Expansion Index (EI)	Expansion Classification	2013 CBC Expansion Classification
0 – 20	Very Low	Non-Expansive
21 – 50	Low	Expansive
51 – 90	Medium	
91 – 130	High	
Greater Than 130	Very High	

7.2.2 We performed laboratory tests on samples of the site materials to aid to evaluate the percentage of water-soluble sulfate content. Results from the laboratory water-soluble sulfate content tests are presented in Appendix B and indicate that the soil tested possesses “not applicable sulfate exposure to concrete structures 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.

7.2.3 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.

7.2.4 We expect the excavation of the surficial soil (undocumented fill, topsoil, alluvium, colluvium, weathered granitic rock) can be performed with light to moderate effort with conventional heavy-duty earthmoving equipment. Excavation of the weathered granitic rock and metamorphic rock are 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.

7.3 Grading

7.3.1 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.

7.3.2 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.

7.3.3 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.

7.3.4 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, using boulders 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.

7.3.5 Potentially compressible surficial soil (undocumented fill, topsoil, colluvium, alluvium, weathered granitic and metamorphic rocks) 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 3 depicts construction detail for lateral extent of removal of unsuitable materials.

7.3.6 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 fill with a “very low” to “low” expansion potential (EI of 50 or less). Where the thickness of the fill below the building pad exceeds 15 feet, the depth of the undercut for cut-fill transition lots, should be increased to one-fifth of the maximum fill thickness to a maximum depth of 10 feet. The large sheet-graded pads should be undercut at least 6 feet to allow for future re-grading of the pads.

7.3.7 Building pads underlain by hard rock units at grade should also be undercut to facilitate future trenching. Building pads that expose hard rock should be undercut a minimum of 3 feet and replaced with properly compacted fill and the base of the undercut should be sloped a minimum of 1 percent toward the adjacent street. Roadways underlain by hard rock should be undercut a minimum of 1 foot below the deepest utility line. Figure 4 presents a typical detail for the overexcavation of streets.

7.3.8 Recommendations for the handling and disposal of oversized rock in fill areas are presented in Figure 5 and in Appendix D. 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 should consist of "soil" fill with an approximate maximum particle dimension of 6 inches with a minimum of 40 percent of the soil passing the ¾-inch sieve. In addition, the upper 3 feet of pad grade should have at least 20 percent of the soil 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 H). 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. Rocks up to 4 feet in maximum dimension can be individually placed in a properly compacted soil matrix with rocks separated at least 8 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.

7.3.9 If perched groundwater is encountered during remedial grading within the surficial soil, top loading of wet material may be required. This condition may potentially occur within the canyon drainages, especially during the rainy season. The excavated materials should then be moisture conditioned as necessary to near optimum moisture content prior to placement as compacted fill.

7.3.10 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 the laboratory maximum dry density near to slightly above optimum moisture content, as determined in accordance with ASTM D 1557. Fill materials excessively above or below optimum moisture content may require additional moisture conditioning prior to placing additional fill.

7.3.11 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:

- 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.

7.4 Earthwork Grading Factors

7.4.1 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 7.4 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 7.4

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

7.5 Slope Stability

7.5.1 We performed slope stability analyses 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. In addition, these slopes should not have an adverse effect on the performance of the building pads. Therefore, from a geotechnical engineering standpoint, the building pads will be safe for human occupancy provided the recommendations provided in the referenced report are adhered to during the construction operations. Slope stability calculations for deep-seated and surficial fill slope stability are presented on Figures 6 and 7, respectively.

7.5.2 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 soil and/or soil/rock fill. 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 "soil" fill. Soil of questionable strength to satisfy surficial stability should be tested in the laboratory for acceptable drained shear strength.

7.5.3 Fill slopes should be overbuilt at least 3 feet horizontally, and cut back to the design finish grade. As an alternative, slopes can 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 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 slope.

7.5.4. Cut slopes excavated in the granitic rock 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 25 feet.

7.5.5 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. Loose rock fragments greater than 6 inches in maximum dimension should not 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.

7.5.6 The cut slope excavations should be observed during grading by an engineering geologist to evaluate 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.

7.5.7 Finished 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.

7.6 Seismic Design Criteria

7.6.1 We used the computer program *U.S. Seismic Design Maps*, provided by the USGS. Table 7.6.1 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 7.6.1
2013 CBC SEISMIC DESIGN PARAMETERS

Parameter	Value			2013 CBC 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	1.000	1.000	1.000	Figure 1613.3.1(1)
Spectral Response – Class B (1 sec), S_1	0.390	0.390	0.390	Figure 1613.3.1(2)
Site Coefficient, F_a	1.000	1.000	1.100	Table 1613.3.3(1)
Site Coefficient, F_v	1.000	1.408	1.619	Table 1613.3.3(2)
Maximum Considered Earthquake Spectral Response Acceleration (0.2 sec), S_{MS}	1.000	1.000	1.100	Section 1613.3.3 (Eqn 16-37)
Maximum Considered Earthquake Spectral Response Acceleration – (1 sec), S_{M1}	0.390	0.550	0.632	Section 1613.3.3 (Eqn 16-38)
5% Damped Design Spectral Response Acceleration (0.2 sec), S_{DS}	0.667	0.667	0.733	Section 1613.3.4 (Eqn 16-39)
5% Damped Design Spectral Response Acceleration (1 sec), S_{D1}	0.260	0.367	0.421	Section 1613.3.4 (Eqn 16-40)

7.6.2 Table 7.6.2 presents additional seismic design parameters for projects located in Seismic Design Categories of D through F in accordance with ASCE 7-10 for the mapped maximum considered geometric mean (MCE_G).

TABLE 7.6.2
2013 CBC SITE ACCELERATION DESIGN PARAMETERS

Parameter	Value	ASCE 7-10 Reference
Mapped MCE_G Peak Ground Acceleration, PGA	0.374g	Figure 22-7
Site Coefficient, F_{PGA}	1.126	Table 11.8-1
Site Class Modified MCE_G Peak Ground Acceleration, PGA_M	0.421g	Section 11.8.3 (Eqn 11.8-1)

7.6.3 Conformance to the criteria in Tables 7.6.1 and 7.6.2 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.

7.7 Foundation and Concrete Slabs-On-Grade Recommendations

7.7.1 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 7.7.1.

TABLE 7.7.1
FOUNDATION CATEGORY CRITERIA

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

7.7.2 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.

7.7.3 Table 7.7.2 presents minimum foundation and interior concrete slab design criteria for conventional foundation systems.

TABLE 7.7.2
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

7.7.4 The embedment depths presented in Table 7.7.2 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 8.

7.7.5 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.

7.7.6 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.

7.7.7 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.

7.7.8 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 7.7.3 for the particular Foundation Category designated. The parameters presented in Table 7.7.3 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 7.7.3
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

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

- The criteria presented in Table 7.7.3 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.

7.7.10. 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.

7.7.11 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.

7.7.12 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.

7.7.13 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.

7.7.14 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.

7.7.15 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.

7.7.16 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.

7.7.17 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.

7.7.18 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.

7.7.19 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.

7.7.20 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.

7.7.21 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.

7.7.22 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.

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

7.8 Retaining Walls and Lateral Loads

7.8.1 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 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.

7.8.2 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) 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.

7.8.3 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.

7.8.4 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 9 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.

7.8.5 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.

7.8.6 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.

7.8.7 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 or undisturbed formational materials. 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. 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.

7.8.8 The recommendations presented herein 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.

7.9 Preliminary Pavement Recommendations

7.9.1 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 7.9.1.

TABLE 7.9.1
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 and Minor Arterials	6.0	20	3.5	10
Roadways	7.0	20	4	12

7.9.2 Prior to placing base materials, the upper 12 inches of the subgrade soil should be scarified, moisture conditioned as necessary, and recompacted to a dry density of at least 95 percent of the laboratory maximum dry density near to slightly above optimum moisture content as determined by ASTM D 1557. Similarly, the base material should be compacted to a dry density of at least 95 percent of the laboratory maximum dry density near to slightly above optimum moisture content. Asphalt concrete should be compacted to a density of at least 95 percent of the laboratory Hveem density in accordance with ASTM D 2726.

7.9.3 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. The asphalt concrete should conform to Section 203-6 of the *Standard Specifications for Public Works Construction (Greenbook)*.

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

7.9.5 A rigid Portland Cement concrete (PCC) pavement section should be placed in driveway entrance aprons, trash bin loading/storage areas and loading dock areas. The concrete pad for trash truck areas should be large enough such that the truck wheels will be positioned on the concrete during loading. We calculated the rigid pavement section in general conformance with the procedure recommended by the American Concrete Institute report ACI 330R-08 Guide for Design and Construction of Concrete Parking Lots using the parameters presented in Table 7.9.2.

TABLE 7.9.2
RIGID PAVEMENT DESIGN PARAMETERS

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

7.9.6 Based on the criteria presented herein, the PCC pavement sections should have a minimum thickness as presented in Table 7.9.3.

TABLE 7.9.3
RIGID PAVEMENT RECOMMENDATIONS

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

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

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

7.9.9 To control the location and spread of concrete shrinkage cracks, crack-control joints (weakened plane joints) should be included in the design of the concrete pavement slab. Crack-control joints should not exceed 30 times the slab thickness with a maximum spacing of 12.5 feet and 15 feet for the 5.5 and 7-inch-thick slabs, respectively, 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.

7.9.10 To provide load transfer between adjacent pavement slab sections, a butt-type construction joint should be constructed. The butt-type joint should be thickened by at least 20 percent at the edge and taper back at least 4 feet from the face of the slab. As an alternative to the butt-type construction joint, dowelling can be used between construction joints for pavements of 7 inches or thicker. As discussed in the referenced ACI guide, dowels should consist of smooth, 1-inch-diameter reinforcing steel 14 inches long embedded a minimum of 6 inches into the slab on either side of the construction joint. Dowels should be located at the midpoint of the slab, spaced at 12 inches on center and lubricated to allow joint movement while still transferring loads. In addition, tie bars should be installed at the as recommended in Section 3.8.3 of the referenced ACI guide. The structural engineer should provide other alternative recommendations for load transfer.

7.9.11 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.

7.10 Slope Maintenance

7.10.1 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.

7.10.2 From a geotechnical engineering standpoint, the building pads will be safe for human occupancy provided the recommendations provided in the referenced report are adhered to during the construction operations.

7.11 Site Drainage and Moisture Protection

7.11.1 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.

7.11.2 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.

7.11.3 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.

7.11.4 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.

7.11.5 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 effect 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.

7.12 Grading and Foundation Plan Review

7.12.1 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.

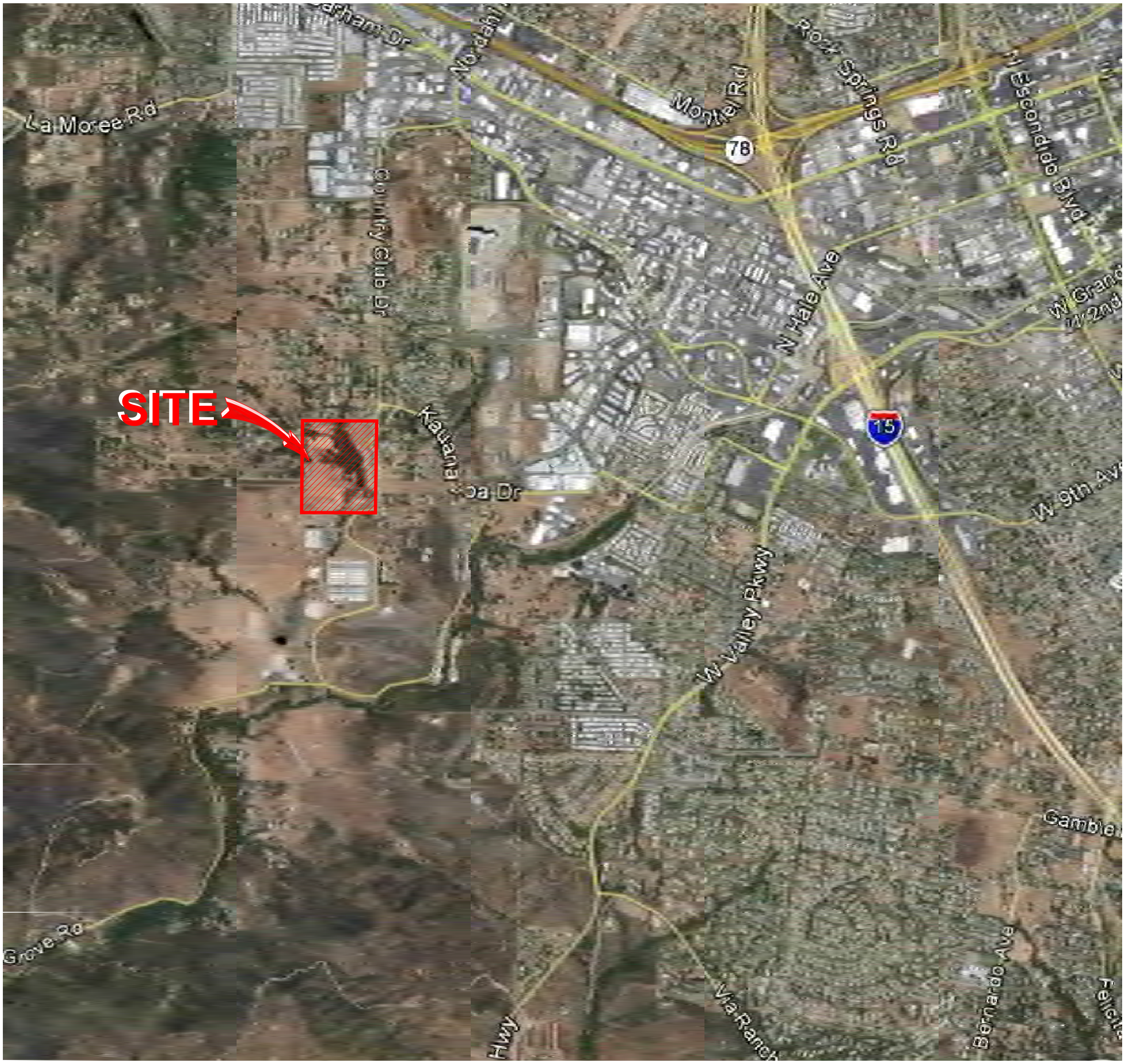
LIMITATIONS AND UNIFORMITY OF CONDITIONS

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

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

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

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



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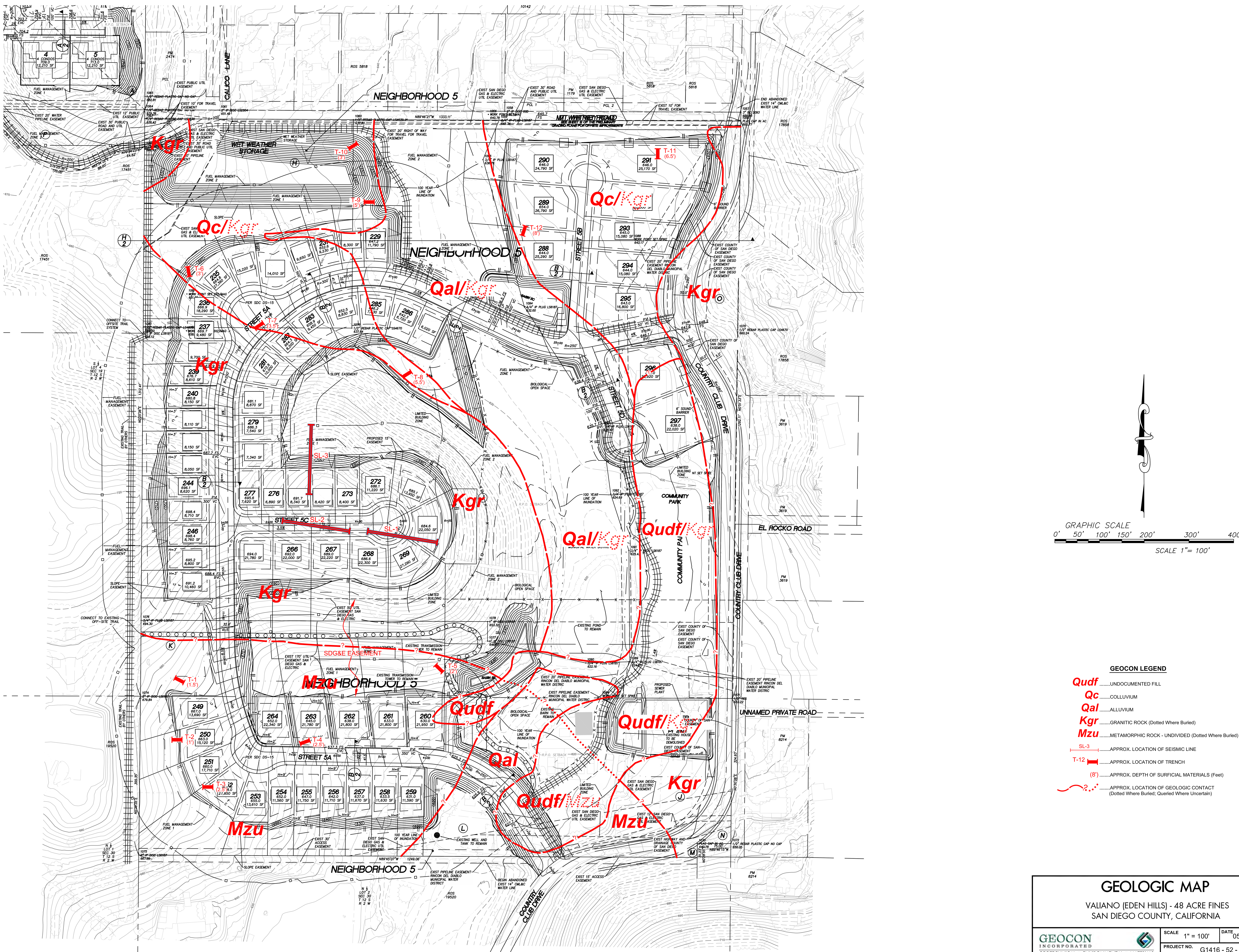
VICINITY MAP

VALIANO (EDEN HILLS) - 48 ACRE FINES
SAN DIEGO COUNTY, CALIFORNIA

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



GEOCON LEGEND

Qudf ........UNDOCUMENTED FILL

Qc ....COLLUVIUM

Qal ....ALLUVIUM

Kgr ....GRANITIC ROCK (Dotted Where Buried)

Mzu ....METAMORPHIC ROCK - UNDIVIDED (Dotted Where Buried)

SL-3 ....APPROX. LOCATION OF SEISMIC LINE

T-12 ....APPROX. LOCATION OF TRENCH

(8') ....APPROX. DEPTH OF SURFICIAL MATERIALS (Feet)

?........APPROX. LOCATION OF GEOLOGIC CONTACT (Dotted Where Buried; Queried Where Uncertain)

GEOLOGIC MAP

VALIANO (EDEN HILLS) - 48 ACRE FINES
SAN DIEGO COUNTY, CALIFORNIA

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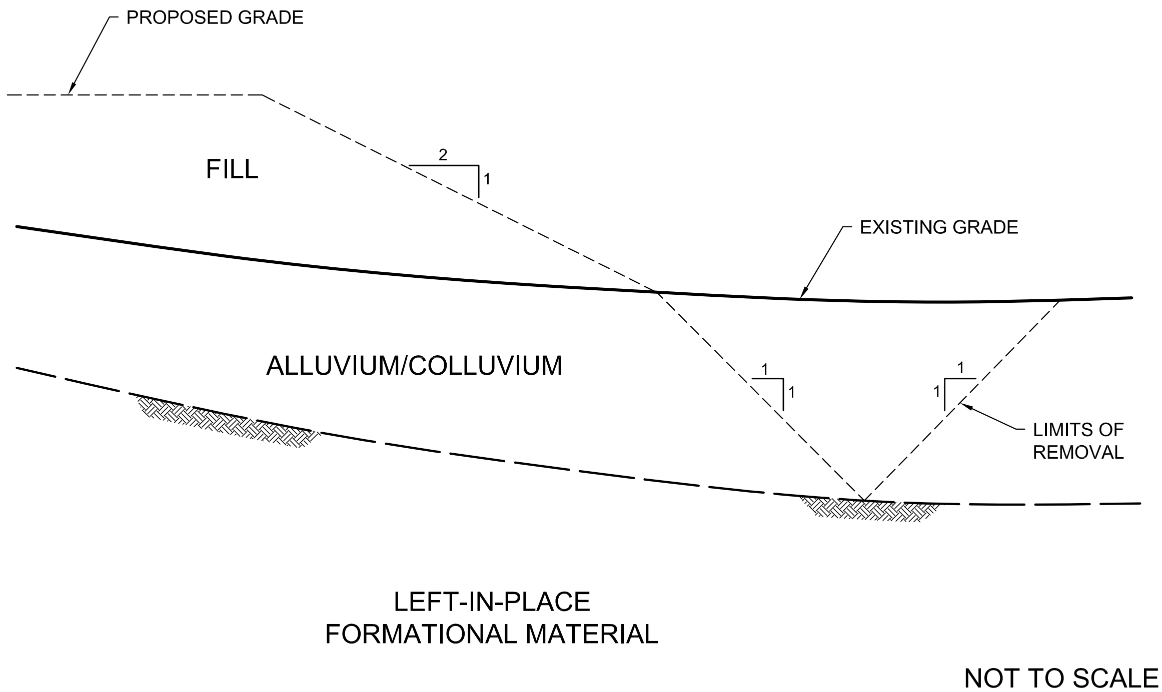
SCALE 1" = 100'	DATE 05 - 13 - 2014
PROJECT NO. G1416 - 52 - 03	FIGURE 2
SHEET 1 OF 1	



NOTE:

SLOPE OF BACKCUT MAY BE STEEPENED WITH THE APPROVAL OF THE SOILS ENGINEER WHERE BOUNDARY CONSTRAINTS LIMIT EXTENT OF REMOVALS

CONSTRUCTION DETAIL FOR LATERAL EXTENT OF REMOVAL

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VALIANO (EDEN HILLS) - 48 ACRE FINES
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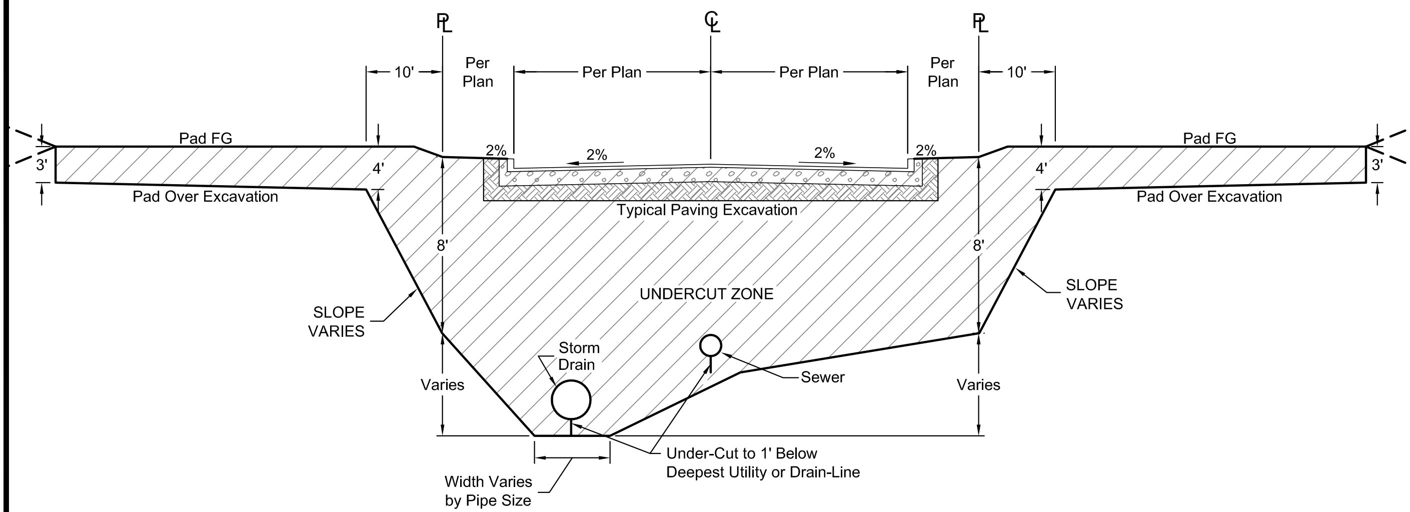
ME / RA		DSK/GTYPD	DATE 05 - 13 - 2014	PROJECT NO. G1416 - 52 - 03	FIG. 3

Y:\PROJECTS\G1416-52-03 Eden Hills & 48 Acre Fines\DETAILS\CDFLER.dwg



NOT TO SCALE

NOTE:
UNDERCUT ZONE SHOULD CONTAIN COMPACTED SOIL FILL WITH MAXIMUM ROCK FRAGMENTS LESS THAN 6 INCHES IN DIMENSION AND A MINIMUM OF 40 PERCENT SOIL PASSING THE 3/4-INCH SIEVE

TYPICAL STREET OVEREXCAVATION DETAIL

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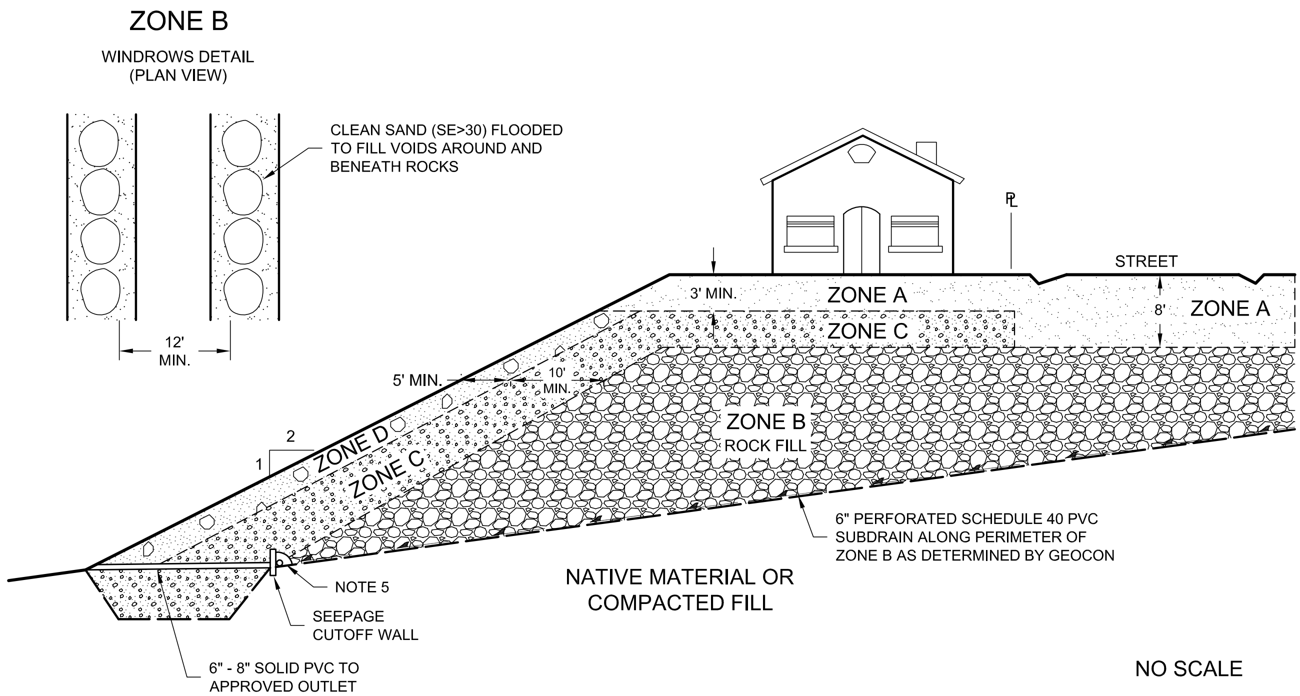
ME / RA		DSK/GTYPD	DATE 05 - 13 - 2014	PROJECT NO. G1416 - 52 - 03	FIG. 4

Y:\PROJECTS\G1416-52-03 Eden Hills & 48 Acre Fines\DETAILS\Street Section.dwg



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.

4. DEPTH OF ZONE A SHOULD BE AT LEAST 8 FEET AND EXTENDED TO AT LEAST 2 FEET 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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ASSUMED CONDITIONS :

SLOPE HEIGHT	H	= 25 feet
SLOPE INCLINATION	2 : 1	(Horizontal : Vertical)
TOTAL UNIT WEIGHT OF SOIL	γ_t	= 125 pounds per cubic foot
ANGLE OF INTERNAL FRICTION	ϕ	= 29 degrees
APPARENT COHESION	C	= 300 pounds per square foot
NO SEEPAGE FORCES		

ANALYSIS :

$\gamma_{c\phi}$	=	$\dfrac{\gamma_t H \tan\phi}{C}$	EQUATION (3-3), REFERENCE 1
FS	=	$\dfrac{N_{cf} C}{\gamma_t H}$	EQUATION (3-2), REFERENCE 1
$\gamma_{c\phi}$	=	5.8	CALCULATED USING EQ. (3-3)
Ncf	=	22	DETERMINED USING FIGURE 10, REFERENCE 2
FS	=	2.1	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.

FILL SLOPE STABILITY ANALYSIS

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ME / RA		DSK/E0000	DATE 05 - 13 - 2014	PROJECT NO. G1416 - 52 - 03	FIG. 6

ASSUMED CONDITIONS :

SLOPE HEIGHT	H	= Infinite
DEPTH OF SATURATION	Z	= 3 feet
SLOPE INCLINATION	2 : 1 (Horizontal : Vertical)	
SLOPE ANGLE	i	= 26.6 degrees
UNIT WEIGHT OF WATER	γ_w	= 62.4 pounds per cubic foot
TOTAL UNIT WEIGHT OF SOIL	γ_t	= 125 pounds per cubic foot
ANGLE OF INTERNAL FRICTION	ϕ	= 29 degrees
APPARENT COHESION	C	= 300 pounds per square foot

SLOPE SATURATED TO VERTICAL DEPTH Z BELOW SLOPE FACE

SEEPAGE FORCES PARALLEL TO SLOPE FACE

ANALYSIS :

$$FS = \frac{C + (\gamma_t - \gamma_w)\ Z\ cos^2\ i\ tan\ \phi}{\gamma_t\ Z\ sin\ i\ cos\ i} = 2.6$$

REFERENCES :

1......Haefeli, R. *The Stability of Slopes Acted Upon by Parallel Seepage*, Proc. Second International Conference, SMFE, Rotterdam, 1948, 1, 57-62

2......Skempton, A. W., and F.A. Delory, *Stability of Natural Slopes in London Clay*, Proc. Fourth International Conference, SMFE, London, 1957, 2, 378-81

SURFICIAL FILL SLOPE STABILITY ANALYSIS

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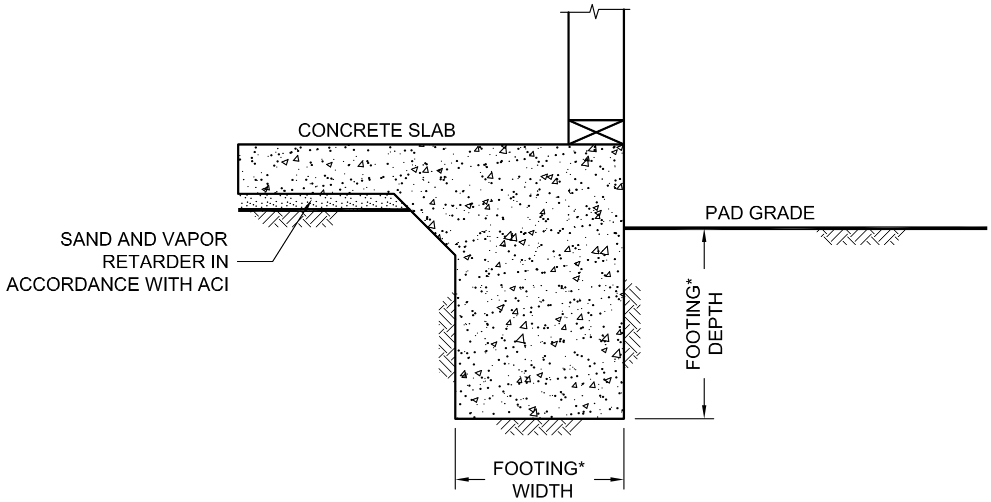

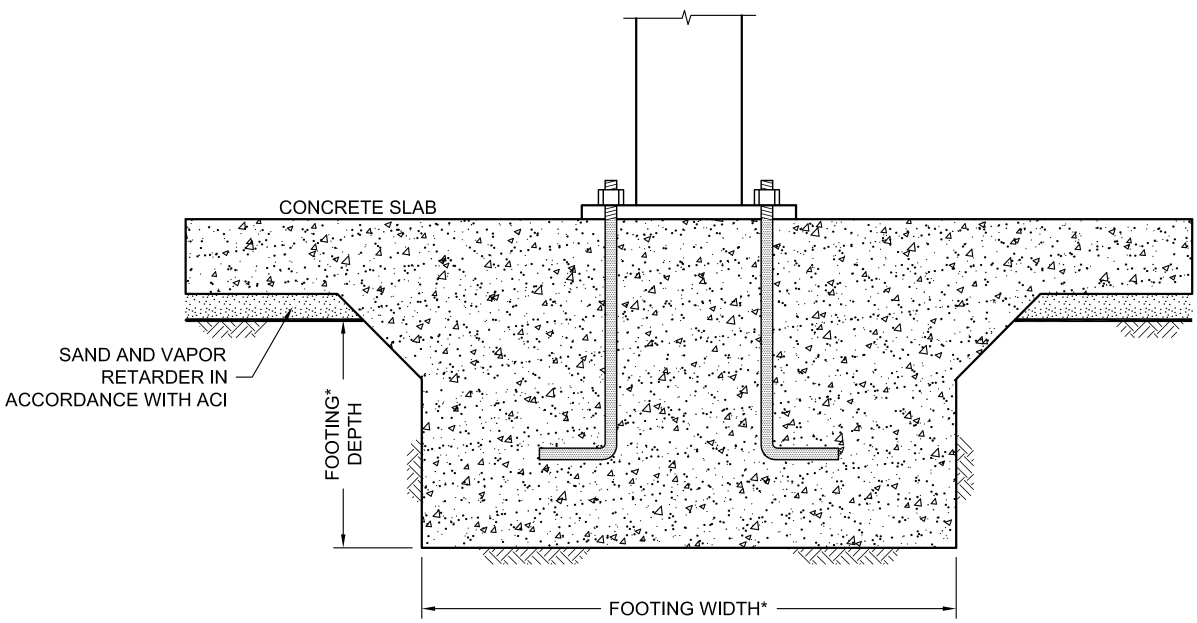

*....SEE REPORT FOR FOUNDATION WITDH AND DEPTH RECOMMENDATION

NO SCALE

WALL / COLUMN FOOTING DIMENSION DETAIL

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COLFOOT2.DWG

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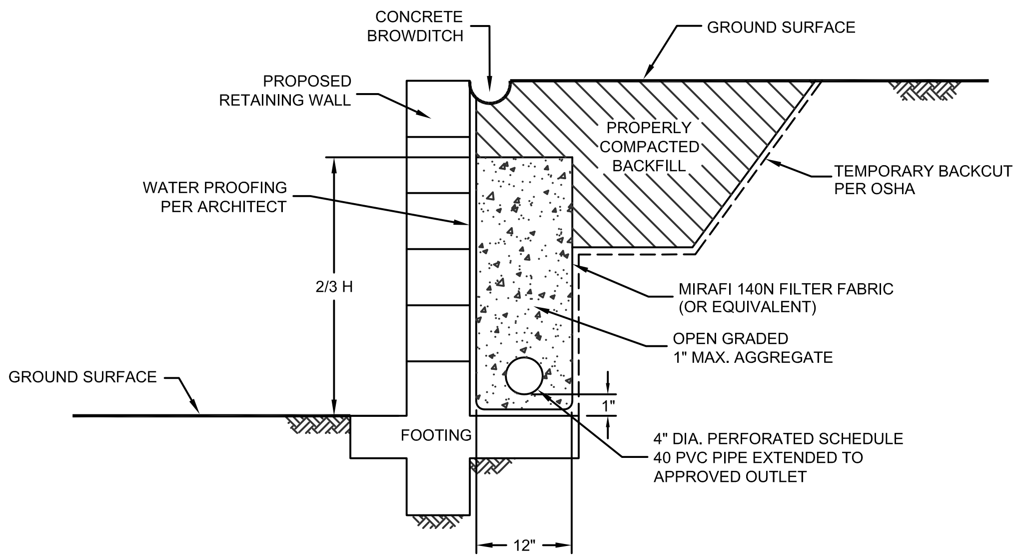

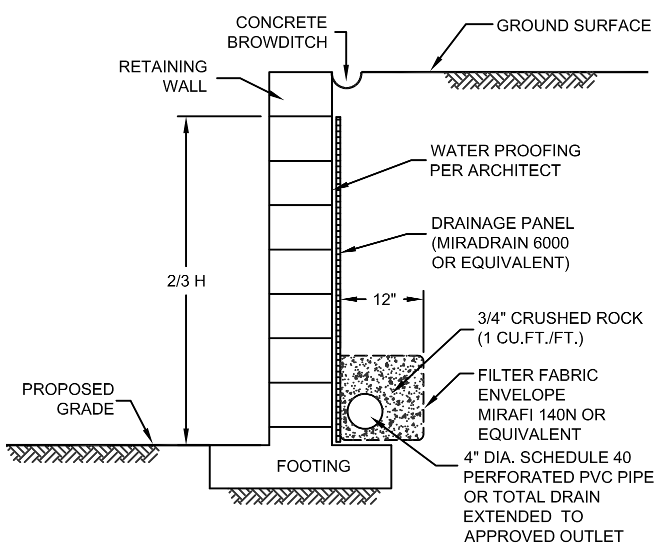

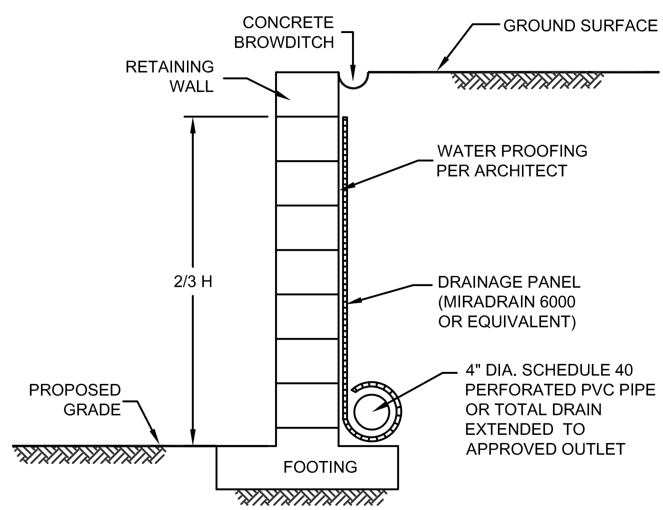

NOTE :

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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ME / RA		DSK/GTYPD	DATE 05 - 13 - 2014	PROJECT NO. G1416 - 52 - 03	FIG. 9

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