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# GEOTECHNICAL INVESTIGATION

## PORTOLA CENTER SOUTH
## TENTATIVE TRACT NO. 15353
## LAKE FOREST, CALIFORNIA

PREPARED FOR

**SUNRANCH CAPITAL PARTNERS**
**SAN DIEGO, CALIFORNIA**

**JULY 6, 2012**
**PROJECT NO. G1218-52-01A**

Project No. G1218-52-01A
July 6, 2012

SunRanch Capital Partners, LLC
610 West Ash Street, Suite 1500
San Diego, California 92101

Attention: Mr. Scott Molloy

Subject: GEOTECHNICAL INVESTIGATION
PORTOLA CENTER SOUTH
TENTATIVE TRACT NO. 15353
LAKE FOREST, CALIFORNIA

Dear Mr. Molloy:

In accordance with the authorization of our change order dated November 18, 2011, we have performed a geotechnical investigation for the subject project. The accompanying report presents the findings of our study and our conclusions and recommendations relative to the geotechnical aspects of developing the property as presently proposed. Based on the results of our investigation, it is our opinion that the site can be developed as planned, 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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Attention: Mr. Joe Wightman

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# GEOTECHNICAL INVESTIGATION

## 1. PURPOSE AND SCOPE

This report presents the results of a geotechnical investigation for the proposed Portola Center South Tentative Tract No. 15353 development located south of Glenn Ranch Road at Saddleback Ranch Road in the City of Lake Forest, California (see Vicinity Map, Figure 1). The purpose of the investigation is to evaluate subsurface soil and geologic conditions at the site and, based on the conditions encountered provide recommendations pertaining to the geotechnical aspects of developing the property. We understand the Portola Center South development will consist of a 626-unit, single-family and multi-family townhome residential subdivision with commercial/mixed use and four park areas. Plans for the proposed development 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 aerial photographs, geologic maps, published geologic reports, and previous geotechnical reports related to the property. A summary of the background information reviewed for this study is presented in the *List of References*.

The field investigation performed for the Portola Center project, which was divided into north and south, included geologic mapping and the excavation of 66 exploratory borings, 40 exploratory test pits, and one fault trench. Appendix A presents a discussion of the field investigation and logs of the exploratory borings, test pits, and fault trench for the Portola Center project. We performed laboratory tests on soil samples obtained from the borings and test pits to evaluate pertinent physical and chemical properties for engineering analysis. The results of the laboratory testing are presented on the boring logs in Appendix A and Appendix B.

Hunsaker & Associates Irvine, Incorporated provided proposed development plans dated June 12, 2012 and topographic information obtained in 2011 for this project. We used these plans during our field investigation for the preparation of the Geologic Map and Geologic Cross-Sections. References to elevations presented in this report are based on the referenced topographic information. Geocon Incorporated 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

Portola Center South is located south of Glenn Ranch Road, west of El Toro Road, and south of the southern terminus of Saddleback Ranch Road in the City of Lake Forest, California (see Vicinity Map, Figure 1). The property is bordered by open space on the west, south and east and Portola

Center North and Glenn Ranch Road to the north. The site consists of both graded and natural topography. A large-sheet-graded pad is located in the central portion of the site created by the placement of fill soil within former canyon drainages. Natural canyon drainages and ridge topography still exist on the northern, southern, and eastern portions of the site that will require excavation within formational materials, remedial grading, and the placement of canyon subdrains and fill to achieve the proposed grades. A large drainage basin is located in the western corner of the site, southwest of the intersection of Glenn Ranch Road and Saddleback Ranch Road. The basin accommodates storm water flows from the Portola Hills planned community to the north of the project site and, to a lesser extent, flows from the Portola North site. Drainage from the basin is ultimately directed toward Aliso Creek. Surface drainage from the natural canyons flows to the west and south also into Aliso Creek. Elevations range from a high of approximately 1,150 feet above mean sea level (MSL) at the peak of the ridge on the north portion of the site to a low of approximately 885 feet MSL in the southwest portion below Lots 188 and 189 and at the western edge of the existing basin below Lot G slope and the sports park.

Portola Center South is approximately 96 acres. The development plan for this area is a detached single-family residential neighborhood, multi-family townhomes, and commercial/mixed use including four public parks with local streets and infrastructure. A total of 309 single-family homes, 260 multi-family units, and 57 commercial/mixed use units are proposed. The single-family lots range in size from 2,975 to 6,829 square-feet. A 6.3 acre sports park is proposed on the northern portion of the site and three smaller parks of 2.1, 1.8, and 0.5 acres are proposed within the development. Access to the site will be from two driveways from Glenn Ranch Road. The existing basin on the north portion of the site will be replaced with a new basin located on the west portion of the property. Several areas of hydro-modification and water quality features are proposed. Private open space of 17.5 acres located within and around the project site is generally used for internal slopes, habitat restoration and fuel modification zone purposes. Internal public streets comprise 14.7 acres of the project. Glenn Ranch Road and the detention basin account for 6.3 acres of the site.

Site grading will consist of cuts and fills with a maximum depth of 122 feet and 115 feet, respectively. Fill slopes, some containing integrated mechanically stabilized earth (MSE) retaining walls, are proposed to maximum heights of approximately 95 feet located at the southeast portion of the site below Lots 56 through 62 and along portions of the southwestern boundary of the site, including below the proposed sports park. Cut slopes will have a maximum height of 35 feet located below Glenn Ranch Road. Cut and fill slopes within Portola Center South are proposed with maximum slope inclinations of 2:1 (horizontal:vertical). Several retaining wall heights would range up to a maximum of about 30 feet. The retaining systems will consist of mechanically stabilized earth (MSE) walls and will be an important factor in designing and constructing the planned development. The total cut grading yardage is approximately 1,889,880 cubic yards with approximately 412,000 cubic yards of remedial grading. Select MSE wall backfill material obtained onsite within sandstone units will be required behind the MSE walls to provide adequate slope stability factors-of-safety. We also expect that 413,246 cubic yards of additional sandstone material will be exported to Portola Center North with the same volume of fine-grained soils imported back from the north. The project civil engineer estimates 368,000 cubic yards of select backfill will be required within the reinforced grid zone of the MSE walls.

The locations, site descriptions, and proposed development herein are based on a site reconnaissance, review of published geologic literature, our field investigation, and discussions with you as the project applicant, the City of Lake Forest, the City's third party reviewers, and Hunsaker & Associates Irvine. If development plans differ from those described herein, Geocon Incorporated should be contacted for review of the plans and possible revisions to this report.

## 3. PREVIOUS SITE DEVELOPMENT

A significant portion of the property has been subject to prior investigations and grading within the previous 25 years associated with the overall construction of Glenn Ranch Road, Saddleback Ranch Road, and the Portola Hills neighborhoods to the north and east of the Portola Center North property. The site is identified as Tentative Tract No. 15353, which combined the existing approved Tentative Tract Nos. 13491 (Phase I), 13491 (Phase II), and 13490. The large sheet-graded pad includes Phase II of the existing approved Tentative Tract No. 13491. The Saddleback Ranch Road and Glenn Ranch Road alignments were graded during the late 1980's and early 1990's. Grading included filling two large drainages, cutting ridges and constructing fill slopes to the south of Glenn Ranch Road, installing improvements at various locations, and constructing a bridge spanning Aliso Creek. Several landslides, which were mapped by Fife (1974) and Morton and Miller (1981), were removed during the grading of the road alignments and sheet graded pad areas, and a drained stability fill slope was constructed on the east portion of the site.

Geotechnical reports regarding the grading activities at the site were prepared by Pacific Soils Engineering, Inc. The reports reviewed as a part of this study are presented in the *List of References*. We were able to obtain sufficient data to adequately document the placement of properly compacted fills, construction of a stability fill, construction of canyon subdrains, and remedial grading bottom elevations and landslide removals on the project. According to the geotechnical grading reports, remedial grading of surficial soil and landslide debris was performed within the graded portions of the site, fill was placed with engineering observation and compaction testing, subdrains were installed within the previous canyon drainages in the area of existing fill, and a drained stability fill was constructed. As such, the fill placed at the site is considered "Engineered Artificial Fill" (afe). Some undocumented fill also exists on the western and eastern portions of the site and a stockpile of scattered boulders is present in the central portion. The approximate lateral extent of the previously placed fill, undocumented fill, and the approximate subdrain locations with their pipe diameters are presented on the Geologic Map.

## 4. REGIONAL GEOLOGIC SETTING

The site is part of a larger structurally geologic complicated area of southern California. The regional structure of the area is dominated by homoclinal structure dipping to the southwest and south that involves a full range of clastic sedimentary rocks and layered volcanic rocks present from late Jurassic to late Miocene.

The study area comprises a part of the southwestern flank of the Santa Ana Mountains, which is a portion of the Peninsular Range province of southern California. A sequence of Tertiary-age sedimentary rocks, including the Topanga, Puente, and Capistrano Formations, as well as younger sediments, were deposited in a marine basin that was subsequently faulted and downwarped during later Miocene time into a north-trending structural trough known as the Capistrano Embayment. The embayment extended north to the Santa Ana Mountains and received a thick sequence of sediments. Broad, gentle folding, complex north-south faulting, and regional uplift in the last 4 million years then brought these bedrock units to the surface. At present, the bedrock formations are locally capped by Quaternary surficial units including Terrace Deposits, alluvium, colluvium, topsoil, landslide debris, and man-made engineered fill and undocumented fill.

The oldest rocks in the area, the Jurassic units, are exposed at the higher elevations of the Santa Ana Mountains. Often referred to as the basement complex or subjacent series, the Bedford Canyon Formation and the Santiago Peak Volcanic or Mesozoic Metavolcanic Rocks are generally mildly metamorphosed, complexly structured rocks, which supplied most of the material for the younger sedimentary formational units overlying them to the west.

Faults in the region displace rocks at least as late as the Miocene and probably younger. The Cristianitos fault, shown on the Geologic Map, Figures 2 and 3, extends adjacent to the western boundary of the property, and is structurally the most significant fault in the local region. However, the Cristianitos fault is considered inactive from a seismicity standpoint.

Based on review of the California Division of Mines and Geology reports and maps and previous Pacific Soils Engineering, Inc. reports and maps, the Portola Hills area is and was underlain by several large landslide complexes. The nearest remaining large landslide to the site is located to the north within the Whiting Ranch Wilderness Park. The landslides in the local area have been heavily altered by erosion and are likely related to periods of significantly higher rainfall during the geologic time period of the Wisconsin Glacial Episode. Movement may have been initiated by seismic activity. Similar landslides in South Orange County have been dated by radiocarbon methods at 10,000 to

17,000 years before present. Most major bedrock landslides in the vicinity have failed as block-glide landslides in stratified siltstone and shale layers within the La Vida and Soquel members of the Puente Formation, and because of a regional southwest dip, most of the landslides are on west to southwesterly facing slopes.

The primary geologic unit encountered on the site is the upper Miocene-age Puente Formation which has been regionally subdivided into four members based on its type section in the Puente Hills. The basal unit is the La Vida member consisting of deep marine shale, mudstone, and thin turbidite sandstone beds. The Soquel member conformably overlies the La Vida member and consists of interfingering siltstone and graded sandstone layers. The next member in the sequence is the Yorba consisting of fine-grained deposits of siltstone and mudstone. The upper member consists of the Sycamore Canyon composed of a wide variety of soils consisting of mudstone, sandstone, and conglomerate beds. The lower three members of the Puente Formation are present on Portola Center South.

## 5. SOIL AND GEOLOGIC CONDITIONS

### 5.1 General

Seven surficial soil types and three geologic formations have been mapped or were encountered during our investigation. The surficial units consist of undocumented fill, previously placed engineered fill, topsoil, alluvium, colluvium, landslide debris, and Terrace Deposits. Formational units include the Oso member of the late Miocene- to early Pliocene-age Capistrano Formation, the Yorba, Soquel and La Vida members of the late Miocene-age Puente Formation, and the early Miocene-age Topanga Formation. The formational and surficial units are discussed in order of increasing age. The approximate lateral extent of the surficial soil and formational materials are presented on the Geologic Map, Figures 2 and 3 (map pocket). The subsurface relationships between the geologic units are presented on the Geologic Cross-Sections, Figures 4 through 8 (map pocket).

### 5.2 Undocumented Fill (afu)

Undocumented fill was placed and end dumped at several locations within the graded portions of the site with thicknesses ranging from a few feet to approximately 25 feet. Some minor undocumented fill also exists on the western portion of the site and extends offsite to the southwest. In general, the undocumented fill consists of loose, damp to moist, silt and sand with rock fragments and cobbles up to approximately 1½ feet in diameter. In addition, scattered boulders of cemented formational material and concrete with a maximum size of 3 to 4 feet in diameter are present at the surface in the central portion of the site with the approximate limits shown on Figures 2 and 3. In its present condition, the undocumented fill soil is not suitable for support of additional fill or structures and remedial grading will be necessary. Undocumented fill is generally suitable for reuse as compacted fill; however, the remedial grading of the undocumented fill areas may generate some debris unsuitable for reuse as compacted fill. Oversize material generated during grading operations may be placed in the deeper fills. In addition, the oversize boulders can be buried within compacted fill if placed in accordance with our grading specifications.

### 5.3 Engineered Artificial Fill (afe)

Previously placed engineered fill underlies the sheet-graded area of the central portion of the site and south of Glenn Ranch Road south of the southern terminus of Saddleback Ranch Road and at the eastern portion of the site. Engineered fill was placed at the site under the observation of Pacific Soils Engineering, Inc. Geocon has reviewed the geotechnical reports related to the placement of the fill, a stability fill, and subdrain placement (see *List of References)*. Geocon has also performed sufficient field investigation to confirm the reported extent, depth, and suitability of the existing fill soil and geologic conditions. In general, previously placed fill consists of silty and clayey sand, silt, and clay, contains gravel- to cobble-size rock fragments, and varies from less than 5 feet to a maximum reported thickness of approximately 125 feet. The majority of the previously placed fill appears to be suitable in its present condition for the support of additional compacted fill and structural loads; however, the upper 3 to 5 feet of the soil has been disturbed due to discing, vegetation, and burrowing animals. Partial removal and recompaction of previously placed fill within areas of proposed grading and improvement should be expected.

### 5.4 Topsoil (Unmapped)

Topsoil is present as a thin veneer overlying the natural, ungraded slopes and bedrock materials across the site. The topsoil has an average thickness of approximately 2 to 3 feet based on our exploratory excavations. The topsoil consists of soft to stiff, loose to medium dense, dry to slightly moist, dark brown, porous, sandy clay to clayey sand with varying amounts of roots and rootlets. Removal of the topsoil will be necessary in areas to support fill or structures. Due to the relatively thin thickness and discontinuity of these deposits, the topsoil is not shown on the Geologic Map.

### 5.5 Alluvium (Qal)

Alluvium is stream-deposited material found in the canyon drainages and generally varies in thickness depending on the size of the canyon and extent of the drainage area. The alluvium consists of firm to stiff, light to dark brown, sandy clay and loose to medium dense, silty to clayey sand. The thickness of the alluvium encountered at the site ranged from approximately 4 feet to more than 10 feet. Alluvial deposits may be deeper in the bottom of the drainages along the southern and western margins of the site. Due to the relatively unconsolidated nature of the alluvial deposits, remedial grading will be necessary in areas to receive fill or structures.

## 5.6 Colluvium (Qcol)

Colluvium, derived from weathering of the underlying bedrock materials at higher elevations and deposited by gravity and sheet-flow, is present on the side slopes of canyons and the upper portions of the canyon drainages. The colluvium is generally stiff to hard, dry to moist, light to dark brown, sandy clay, and loose to medium dense, clayey to silty sand and clayey silt. The thickness of colluvium generally ranges from approximately 2 to 5 feet. Removal of the colluvium is required in areas that will support fill or structures. Due to the relatively thin thickness and discontinuity of the deposits, only the larger areas of colluvium are shown on the Geologic Map.

## 5.7 Landslide Debris (Qls)

Two areas of recent landslide debris exist within the site. The landslides have generally occurred within the thinly bedded siltstone and claystone layers of the La Vida and Soquel members of the Puente Formation. The landslide debris encountered during our investigation varied from a few feet to about 16 feet thick and consisted of a mixture of discontinuous rock clasts within a matrix of silt and sand. The landslides are located along the lower portions of the canyon drainages. Landslide debris is not suitable for the support of compacted fill or structures in its present condition and may be subject to further slope instability. The landslide debris should be removed and replaced with compacted fill during remedial grading operations in areas of the planned development. The landslide debris is generally suitable for use as compacted fill; however, some of the clayey portions may possess a "high" expansion potential (Expansion Index of 91 to 130) and should be placed in the deeper fill areas, where practical.

## 5.8 Terrace Deposits (Qt)

Holocene- to Pleistocene-age, fluvial-derived Terrace Deposits are located on the southwestern portion of the site. We encounter Terrace Deposits in Boring B-21 to a depth of 8 feet during our subsurface investigation. The deposits generally consist of medium dense to dense, damp to moist, brown to yellowish brown, silty sand with gravel and cobble size material. Localized areas within this unit have been reported to have cemented zones. In addition, loose sand and gravel layers are known to exist. The granular dense portions of the Terrace Deposits typically exhibit favorable shear strength and "very low" to "low" expansive characteristics (expansion index of 50 or less). The Terrace Deposits are generally suitable for the support of compacted fill and structural loads. However, layers of loose sand and gravel, if encountered, may be subject to raveling and erosion where exposed on slopes, and may be prone to settlement. The loose portions of the Terrace Deposits will require remedial grading where engineered fill or structural loads are planned, if encountered.

## 5.9 Capistrano Formation-Oso Member (Tco)

Late Miocene- to early Pliocene-age Oso Member of the Capistrano Formation is located along the natural slopes on the western portion of the site. The Capistrano Formation is in high-angle fault contact with the older Puente Formation along the Cristianitos Fault in the western portion of the site. The Oso Member of the Capistrano Formation generally consists of fine- to medium-grained sandstone that is white to light yellowish brown, poorly bedded to massive, and weakly to moderately cemented. In general, the sediments of the Oso Member exhibit favorable shear strength and "very low" to "medium" expansion characteristics (expansion index of 90 or less). The Capistrano Formation is suitable for the support of compacted fill and structural loads. Oversize material may be generated from this unit during excavation because of matrix cementation.

## 5.10 Puente Formation-Yorba Member (Tpy)

The upper Miocene-age Yorba Member of the Puente Formation is exposed on the north portion of the site at the top and along the west side of the ridge south of Glenn Ranch Road. This unit is the highest member in the sequence within the Puente Formation exposed on the site. The Yorba Member conformably overlies the older Soquel Member of the Puente Formation. The contact between the two members is generally dipping from south to west. The Yorba Member typically consists of light olive to grayish brown, thinly bedded, moderately indurated, sandy to clayey siltstone. Some of the beds contain high concentrations of evaporate minerals such as carbonates and gypsum. A thin, light gray ash bed was also present approximately 2 inches thick. Some of the siltstone beds encountered in Boring B-59 have been subject to bedding plane shearing and are weak.

In general, the sediments of the Yorba Member of the Puente Formation exhibit low to moderate shear strength and "medium" to "high" expansion characteristics (expansion index of 51 to 130). The Yorba Member is suitable for the support of compacted fill and structural loads. The La Vida Member contains minerals that may be corrosive to steel or concrete. Laboratory tests related to corrosivity are presented in Appendix B. This unit will be predominately encountered in a cut area and is suitable for use as fill material.

## 5.11 Puente Formation-Soquel Member (Tps and Tps-slt)

The upper Miocene-age Soquel Member of the Puente Formation is exposed on the majority of the natural exposures on the site and is the middle member in the sequence. The Soquel Member conformably overlies the older La Vida Member of the Puente Formation. The contact between the two members is generally dipping from south to west. The Soquel Member consists of white to light yellowish brown, massively bedded, weakly to moderately cemented, fine- to coarse-grained (arkosic) sandstone (Tps) and thinly bedded diatomaceous shale and siltstone (Tps-slt). The sandstone portions of this unit are exposed in cut slopes along Glenn Ranch Road and on the lower portions of the natural slopes on the south and western portions of the site. Siltstone layers where observed to interfinger within this unit.

Where exposed within existing cut and natural slopes, the sandstone portions of the Soquel Member (Tps) have been stable, but have been subject to minor raveling and erosion. In general, the granular sediments of the Soquel Member exhibit favorable shear strength and "very low" to "low" expansion characteristics (expansion index of 50 or less). The Soquel Member sandstone is suitable for the support of compacted fill and structural loads. The sandstone is moderately to well cemented and oversize material may be generated in this unit during grading operations because of matrix cementation. Granular material from this unit may be used as select backfill within the reinforced zone for the proposed MSE retaining walls.

The siltstone portions of the Soquel Member (Tps-slt) exhibit relatively low to moderate shear strength and "medium" to "high" expansion characteristics (expansion index of 51 to 130). The siltstone unit of the Soquel Member is suitable for the support of compacted fill and structural loads; however, stability fills will be required where siltstone is exposed in cut slopes. This unit is typically prone to slope instability and has been subject to slope failures and landslides. The stability of proposed slopes composed of Soquel Member siltstone units are evaluated in subsequent sections of this report.

### 5.12 Puente Formation-La Vida Member (Tplv)

The late Miocene-age La Vida Member of the Puente Formation is present on the natural slope areas on the southeastern portion of the site. The La Vida Member is conformably overlain by the younger Soquel Member. The contact between the two siltstone members is generally dipping from south to west and is sometimes difficult to distinguish. The La Vida Member typically consists of interbedded siltstone, shale, claystone, and sandstone beds. We observed the majority of the formation as light olive brown to grayish brown, thinly bedded, moderately indurated, sandy to clayey siltstone. Some of the beds are highly calcareous or diatomaceous and contain high concentrations of evaporate minerals such as carbonates and gypsum. Thin, light gray, ash beds are also present ranging from ½ to 8 inches thick. Deeper within the formation, the siltstone beds are generally unoxidized, very dark gray, well indurated, and shaley. Some of the claystone beds encountered in the borings have been subject to bedding plane shearing and are weak. The La Vida Member is prone to slope instability and has been subject to slope failures and landsliding.

In general, the sediments of the La Vida Member of the Puente Formation exhibit low to moderate shear strength and "medium" to "high" expansion characteristics (expansion index of 51 to 130). The La Vida Member is suitable for the support of compacted fill and structural loads; however, stability fills should be constructed where the La Vida Member is exposed in cut slopes. The stability of proposed slopes composed of La Vida Member materials are evaluated in subsequent sections of this report. The unit has locally been subject to deep weathering and slope creep. Deeply weathered and creep-affected areas are compressible and should be removed during remedial grading in areas to receive compacted fill or structural loads. The La Vida Member also contains minerals that may be corrosive to steel or concrete. Laboratory tests related to corrosivity are presented in Appendix B.

### 5.13 Topanga Formation (Tt)

The middle Miocene-age Topanga Formation is mapped in the southeast margins of the site underlying the La Vida Member of the Puente Formation (Fife, 1974 and Morton and Miller, 1981). We did not encounter the Topanga Formation during our subsurface investigation, but this unit typically consists of moderately to well cemented, fine- to medium-grained sandstone. We do not expect to encounter the Topanga Formation during the proposed site development.

## 6. GEOLOGIC STRUCTURE

The geologic structure within the project area is characterized by a series of regional fault blocks within the Tertiary-age sedimentary units, which have been tilted generally to the south and west to form dipping bedding. Bedding attitudes observed within formational materials encountered during the investigation range from 2 to 33 degrees generally dipping from south to west with most dips ranging 10 to 25 degrees from horizontal. Bedding plan shear (BPS) dips and directions tend to be more variable when measured within each borings but are believed to be generally parallel to bedding when compared regionally between borings. However, BPS are commonly discontinuous between adjacent borings. Bedding and structural orientations measured during our field investigation are presented on the Geologic Map (Figures 2 and 3) and in the boring and test pit logs in Appendix A. Interpretations of subsurface structure are depicted on the Geologic Cross-Sections (Figures 4 through 8).

The granular portions of the sandstone formational units within the Puente and Capistrano Formations (Tps and Tco) are typically massive to poorly bedded. The interbedded siltstone and sandstone units within the Puente Formation (Tplv) and the siltstone units of the Soquel Member (Tps-slt) typically are thinly bedded (less than 2 inches) and are frequently jointed or fractured. Sheared claystone beds exist within the siltstone units, generally along bedding (referred to as bedding plane shears) and frequently with "out-of-slope" orientations. Shear zones create a possibility for slope instability and, where encountered in cut slopes during grading, will necessitate slope stabilization measures. Adverse geologic structure does not present a significant geologic hazard to the proposed development provided the recommended use of buttresses, stability fills, shear pins and/or soils nails are incorporated into design and construction.

The Cristianitos Fault extends along the western portions of the site, juxtaposing the underlying Puente Formation with the younger Oso Member of the Capistrano Formation. Dips should be expected to be considerably steeper in the vicinity of the Cristianitos Fault. Pacific Soils Engineering (1996) mapped fault strands and shear zones encountered during previous grading operations at several locations on the site, but are not expected to impact the proposed development.

## 7. GROUNDWATER

A review of the Seismic Hazard Evaluation of the El Toro 7.5 Minute Quadrangle, Orange County, California (California Division of Mines and Geology, 2000), indicates that the site is not located within a groundwater basin. The site is located within the southern portion of Portola Hills and is underlain by bedrock units that are not considered water bearing. Groundwater information presented in this document is generated from data collected in the early 1900's to present.

With the exception of the existing debris basin and drainage structure located on the northwest portion of the site, we did not observe evidence of near surface water, such as seeps, springs, or phreatophytes within the existing drainages. We did not encounter a static groundwater table in the exploratory excavations performed for this investigation. However, we did encounter localized layers of seepage within the exploratory borings. We do not expect groundwater to adversely impact the development of the property. It is not uncommon for groundwater seepage conditions to develop where none previously existed due to the permeability characteristics of the geologic units encountered on site. During the rainy season, perched water conditions are likely to develop within the drainage areas that may require special consideration during grading operations. Groundwater elevations are dependent on seasonal precipitation, irrigation, and land use, among other factors, and vary as a result. Proper surface and subsurface drainage will be critical to future performance of the development.

## 8. GEOLOGIC HAZARDS

### 8.1 Faulting

The numerous faults in Southern California include active, potentially active, and inactive faults. The criteria for these major groups are based on criteria developed by the California Division of Mines and Geology (CDMG). By definition, an active fault is one that has had surface displacement within Holocene time (about the last 11,000 years). A potentially active fault has demonstrated surface displacement during Quaternary time (approximately the last 1.6 million years), but has had no known Holocene movement. Faults that have not moved in the last 1.6 million years are considered inactive. The site is not located within a State of California Earthquake Fault Zone (CDMG, 2010). The location of the site with respect to local active and potentially active faults is shown on Figure 9, Regional Fault Map.

Active or potentially active faults with the potential for surface fault rupture are not known to pass directly beneath the site. Therefore, the potential for surface rupture due to faulting occurring beneath the site during the design life of the proposed development is considered low. The site, however, is located in the seismically active Southern California region, and could be subjected to moderate to strong ground shaking in the event of an earthquake on one of the many active Southern California faults.

The San Joaquin Hills Thrust, located approximately 5½ miles west of the site, is the closest known active fault. The San Joaquin Hills Thrust is a recently discovered blind thrust fault (fault with no surface expression) having an expected maximum earthquake magnitude (Mw) of 7.1. The fault extends roughly between Huntington Beach and Dana Point, is not exposed at the ground surface, and is typically identified at depths greater than 3 kilometers. This fault and other blind thrust faults are not exposed at the surface and do not present a potential surface fault rupture hazard; however, these active features are capable of generating future earthquakes and ground shaking.

The Cristianitos Fault has been mapped extending along the western margins of the site (see Figures 2 and 3). We encountered the fault within the fault trench (FT-1) excavated as a part of this study. The fault offsets the Oso Member of the Capistrano Formation and the Soquel Member of the Puente Formation. The fault trends roughly north and dips at high-angles to nearly vertical. A log of Geocon's fault trench FT-1 is presented in Appendix A. We observed continuous "A" and "AB" topsoil units extending across the fault trace with no evidence of offset. Pacific Soils Engineering, Inc. (1996) performed a fault trench north of Glenn Ranch Road in Portola Center North. The fault trench encountered the Cristianitos Fault as a zone of faulting approximately 80 feet wide composed of approximately nine thin fault strands offsetting beds within the Soquel Member of the Puente Formation. Evidence was not observed within the recent and previous fault trenches, and no evidence is present in the literature that suggests the fault offsets Holocene-age material. The Cristianitos Fault is locally overlain by Quaternary terrace deposits ranging in age from an estimated 34,000 to 120,000 years before present and has not been offset by faulting (Shlemon, 1987). The onshore portion of the Cristianitos Fault is considered "inactive" by the State Geologist. We do not expect the Cristianitos Fault to affect the proposed development and structural setbacks will not be required.

## 8.2 Seismicity

According to the computer program *EZ-FRISK (Version 7.62),* 27 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 San Joaquin Hills Thrust, located approximately 5½ miles west of the site, is the nearest known active fault and is the dominant source of potential ground motion. Earthquakes that might occur on the San Joaquin Hills Thrust or other faults within the southern California and northern Baja California area are potential generators of significant ground motion at the site. The estimated maximum earthquake magnitude and peak ground acceleration for the San Joaquin Hills Thrust are 7.1 and 0.40g, respectively. The location of the site in relation to historic earthquake activity is presented in Figure 10, California Seismicity Map. Table 8.2.1 lists the estimated maximum earthquake magnitude and peak ground acceleration for the 10 most dominant faults in relation 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 (2008) NGA acceleration-attenuation relationships.

**TABLE 8.2.1**
**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) |
| San Joaquin Hills Thrust | 5½ | 7.1 | 0.28 | 0.38 | 0.40 |
| Chino | 10 | 6.8 | 0.21 | 0.19 | 0.21 |
| Elsinore | 11 | 7.85 | 0.26 | 0.21 | 0.28 |
| Newport Inglewood | 15 | 7.5 | 0.20 | 0.15 | 0.19 |
| Puente Hills (Coyote Hills) | 20 | 6.9 | 0.13 | 0.13 | 0.12 |
| Puente Hills | 22 | 7.1 | 0.13 | 0.13 | 0.14 |
| Puente Hills (Santa Fe Springs) | 28 | 6.7 | 0.09 | 0.09 | 0.07 |
| Palos Verdes | 29 | 7.3 | 0.12 | 0.08 | 0.08 |
| Palos Verdes Connected | 29 | 7.7 | 0.14 | 0.10 | 0.12 |
| San Jose | 29 | 6.7 | 0.09 | 0.07 | 0.06 |

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

We performed a site-specific probabilistic seismic hazard analysis using *EZ-FRISK*. Geologic parameters not addressed in the deterministic analysis are included in this analysis. The program operates under the assumption that the occurrence rate of earthquakes on each mapped Quaternary fault is proportional to the faults' slip rate. The program accounts for fault rupture length as a function of earthquake magnitude, and site acceleration estimates are made using the earthquake magnitude and distance from the site to the rupture zone. The program also accounts for uncertainty in each of following: (1) earthquake magnitude, (2) rupture length for a given magnitude, (3) location of the rupture zone, (4) maximum possible magnitude of a given earthquake, and (5) acceleration at the site from a given earthquake along each fault. By calculating the expected accelerations from considered earthquake sources, the program calculates the total average annual expected number of occurrences of site acceleration greater than a specified value. We utilized acceleration-attenuation relationships suggested by Boore-Atkinson (2008), Campbell-Bozorgnia (2008) and Chiou-Youngs (2008) in the analysis. Table 8.2.2 presents the site-specific probabilistic seismic hazard parameters including acceleration-attenuation relationships and the probability of exceedence.

**TABLE 8.2.2**
**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 | 0.53 | 0.52 | 0.58 |
| 5% in a 50 Year Period | 0.41 | 0.40 | 0.44 |
| 10% in a 50 Year Period | 0.33 | 0.31 | 0.33 |

The California Geologic Survey (CGS) has a program that calculates the ground motion for a 10 percent probability of exceedence in a 50-year period based on an average of several attenuation relationships. Table 8.2.3 presents the calculated results from the Probabilistic Seismic Hazards Mapping Ground Motion Page from the CGS website.

**TABLE 8.2.3**
**PROBABILISTIC SITE PARAMETERS FOR SELECTED FAULTS**
**CALIFORNIA GEOLOGIC SURVEY**

| Calculated Acceleration (g) Firm Rock | Calculated Acceleration (g) Soft Rock | Calculated Acceleration (g) Alluvium |
|---|---|---|
| 0.34 | 0.36 | 0.39 |

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

## 8.3 Liquefaction

Liquefaction typically occurs when a site is subjected to strong seismic shaking, on-site soils are cohesionless or are silt and clay with low plasticity, groundwater is encountered within 50 feet of the surface, and soil relative densities are less than about 70 percent. If the four criteria are met, a seismic event could result in a rapid pore water pressure increase from the earthquake-generated ground accelerations. According to mapping produced by the State of California (California Division of Mines and Geology, 2001), there are no areas susceptible to liquefaction mapped at the site. The potential for liquefaction is considered to be very low due to the presence of drained compacted fill, dense formational units, and the absence of a permanent groundwater table in the upper 50 feet. The site location is presented on Figure 11 in relation to areas with a potential for liquefaction, based on the Seismic Hazard Zones map prepared by the California Division of Mines and Geology (CDMG, 2010).

## 8.4 Landslides

Based on our field reconnaissance and our subsurface investigation, two areas of recent landslide deposits exist at the site, originating in the La Vida Member and the siltstone portions of the Soquel Member of the Puente Formation. The approximate limits and dimensions of the landslides are depicted on the Geologic Map (Figures 2 and 3) and the Geologic Cross-Sections (Figures 4 through 8).

Pacific Soils Engineering Inc. (1996) encountered a landslide 1,350 feet north of the site at its closest point within the Whiting Ranch Wilderness Park. We prepared geologic cross-sections provided within the Portola Center North report to illustrate the relationship between the existing landslide and the proposed development to the north. This landslide, in its present orientation, does not pose a geologic hazard to proposed development of the site in the current configuration or if the offsite landslide were to re-activate. Previous mapping performed by Fife (1974) and Morton and Miller (1981) indicate that several large areas of landslide debris previously existed at the site prior to development. According to our review of the prior geotechnical investigations by Pacific Soils Engineering, these landslides have been removed during the previous grading operations. The site location is presented on Figure 11 in relation to areas with a potential for earthquake-induced landslides, based on the Seismic Hazard Zones map prepared by the California Division of Mines and Geology (CDMG, 2001).

Siltstone portions of the Puente Formation contain out-of-slope bedding orientations and bedding plane shears and are prone to slope instability. The landslide deposits observed at the site should be removed in the areas of proposed development. The potential for future landsliding adversely affecting the proposed improvements is low, provided the recommendations presented in this report for removal and compaction of landslide debris and for stabilization of proposed slopes are followed.

## 8.5 Slope Stability

We evaluated the proposed slope configurations, as depicted on the Geologic Map, to calculate both surficial and global stability based on the current geologic information. Adverse geologic conditions including out-of-slope-bedding, bedding plane shears, and weak discontinuous claystone layers exist within the Yorba and La Vida Members of the Puente Formation (Tpy and Tplv) and the siltstone portions of the Soquel Member (Tps-slt). Slopes composed of siltstone formational material should be considered potentially unstable if weak layers or adverse bedding orientations are present. Proposed cut slopes within the granular sandstone units of the Puente (Tps) and Capistrano (Tco) Formations should be stable. Overall, the proposed cut and fill slopes can be constructed as planned; however, due to the discontinuous nature of the weak layers within the siltstone portions of the formational materials, predicting or locating isolated layers is difficult. Fill slopes, typically containing integrated MSE retaining walls, are proposed to maximum heights of approximately 95 feet throughout the development. Buttress and stability fills will be required during grading operations where out-of-slope bedding orientations or bedding plane shears detrimentally affect the stability of the proposed slopes.

We performed the slope stability analyses using the two-dimensional computer program *GeoStudio2004* created by Geo-Slope International Ltd. Stability fills will be required along cut slopes exposing siltstone units with bedding plane shears and out-of-slope bedding orientations. The approximate shear key widths for the proposed buttress slopes are presented on the Geologic Map. The proposed slopes should be stable from shallow sloughing conditions provided the recommendations for grading and drainage are incorporated into the design and construction of the proposed slopes. Buttress grading plans showing proposed subdrain locations, tie-in and outlet points, and bottom and subdrain elevations will be prepared once the 40-scale grading plans and improvement plans are available to detail this information.

Buttress fills will be required as evaluated using Cross-Sections E-E′, H-H′, L-L′ and N-N′. The computer slope stability output in Appendix C presents the approximate location of the buttresses. In addition, the approximate widths of the buttresses are presented on the Geologic Map and the Geologic Cross-Sections. We should evaluate the limits of the buttresses prior to construction of the project and after the 40-scale grading plans have been prepared.

We included preliminary information for the planned MSE walls in our slope stability analyses. The reinforcement geogrid type, length, and spacing presented on the slope stability analyses are the estimated minimum requirements for the required factor of 1.5 and 1.1 for static and seismic conditions, respectively. We should review the retaining wall plans after the walls have been designed.

## 8.6 Hydroconsolidation

Hydroconsolidation is the tendency of unsaturated soil structure to collapse upon saturation resulting in the overall settlement of the affected soil and overlying foundations or improvements supported thereon. Potentially compressible surficial soil underlying the proposed structures and existing fill is typically removed and recompacted during remedial site grading. However, if compressible soil is left in-place, a potential for settlement due to hydroconsolidation of the soil exists.

The results of the laboratory consolidation tests for the fill materials indicate a range of 0.8 percent swell to about 1.3 percent collapse with an average of zero consolidation when water is added. We calculated an approximate average degree of saturation of 80 percent on the samples obtained during our investigation. Therefore, based on the results of the laboratory tests and the calculated degree of saturation of the existing fill materials, we do not expect settlement due to hydroconsolidation will affect the planned development.

## 9. CONCLUSIONS AND RECOMMENDATIONS

### 9.1 General

9.1.1 No soil or geologic conditions were encountered that would preclude the development of the property as presently planned, provided the recommendations of this report are followed. The proposed development of the property will not have an adverse impact to adjacent properties or improvements provided the recommendations of this report are implemented.

9.1.2 Potential geologic hazards at the site include seismic shaking, fill settlement, and slope instability. Based on our investigation and available geologic information, active faults are not present underlying or trending toward the site. Pacific Soils Engineering (1996) encountered and investigated a relatively large landslide located 1,350 feet north of the site within the Whiting Ranch Wilderness Park. Based on our review of the geologic map and cross-sections, it is our opinion that this landslide does not pose a geologic hazard to the proposed development even if the landslide were to re-activate. The other minor landslide deposits observed within the site boundary will be removed during grading operations and should not impact the proposed development.

9.1.3 The surficial soil (consisting of undocumented artificial fill, the upper 3 to 5 feet of previously placed fill, topsoil, colluvium, alluvium, landslide debris, creep-affected formational material, and loose Terrace Deposits) are not considered suitable for the support of fill or structural loads in its present condition and will require remedial grading in the form of removal, moisture conditioning as necessary, and compaction within the limits of grading. The majority of the previously placed fill, the Terrace Deposits, and formational materials of the Puente and Capistrano Formations are suitable for the support of structures and compacted fill.

9.1.4 Remedial grading operations are generally not planned to extend beyond the limits of grading presented on the tentative tract map with the exception small areas along the west, south, and east portions of the site where removal of landslide debris and alluvium will extend to the property line (see Geologic Map, Figures 2 and 3).

9.1.5 In general, cut slopes composed of Terrace Deposits, sandstone formational materials, siltstone formational materials with favorable geologic structure, and properly compacted fill, should possess factors of safety of at least 1.5 at inclinations of 2:1 (horizontal to vertical), or flatter. The results of our slope stability analyses are presented in Appendix C.

9.1.6 Based on our slope stability analyses, the proposed slopes along the western, southern and eastern portions of the project are potentially unstable and will require slope stabilization consisting of the construction of buttress fills with a shear key with a maximum width of about 105 feet. Most of the slopes with MSE walls will require lengthening the reinforcement grids to achieve an appropriate factor of safety. Soil nail walls can be used where MSE wall grid reinforcement cannot be constructed due to property line constraints. The approximate buttress widths of slopes requiring stabilization are presented on the Geologic Map. Recommendations for slope stabilization are presented herein. Additional subsurface investigation and slope stability analyses may be necessary when 40-scale grading plans are finalized to provide final recommendations for buttress width design. Cut slopes exposing siltstone units with weak claystone beds, bedding plane shears, or out-of-slope bedding are potentially unstable and will require the construction of buttress and stability fills.

9.1.7 MSE walls and wall-slope combinations are expected to possess factors of safety of at least 1.5 provided the geotechnical recommendations presented in this report are followed. Selective grading will be necessary to provide backfill materials that exceed the minimum shear strength used in wall design. Close coordination between the grading and wall construction contractors and the engineering consultants will be necessary for efficient wall construction operations. Slopes incorporating MSE retaining walls may be subject to relaxation and settlement beyond the top of the slope. If estimated settlements are greater than the design tolerances of the planned residential structures and utilities, structural slope setbacks or significant construction waiting periods may be required.

9.1.8 The existing constructed slopes along and south of Glenn Ranch Road were designed and constructed with a factor of safety of at least 1.5 based our review of the grading reports and slope stability analyses. The planned grading and proposed buttresses and stability fills will provide adequate stability for the proposed development achieving a minimum factor of safety of 1.5.

9.1.9 The proposed structures and site retaining walls may be supported on shallow foundations bearing in either competent bedrock or engineered fill. Building pads with a fill/formational contact should be undercut as described herein. General recommendations for the design of shallow foundations are provided herein.

9.1.10 The on-site geologic units possess physical and chemical characteristics that may adversely affect the proposed development in their present condition. Laboratory tests indicate that the soil locally possesses a "very low" to "high" expansion potential, and moderate to severe corrosion potential. Recommendations to mitigate these adverse soil conditions are provided herein.

9.1.11 Proper surface and subsurface drainage should be maintained in order to preserve the engineered properties of the fill in the building pads, slope areas, and retaining wall areas.

9.1.12 Grading plans indicate that local area parks, multi-family, and commercial/mixed use area will be constructed to a sheet-graded condition. Preparation of update geotechnical reports will be necessary prior to the fine grading of these projects.

## 9.2 Soil and Excavation Characteristics

9.2.1 Based on the results of the field investigation and our experience in the general area, we expect the surficial soil and formational materials can generally be excavated with moderate to heavy effort using conventional heavy-duty excavation equipment. Cemented zones requiring very heavy effort to excavate may be encountered at random locations in the formational materials; however, we expect the extent will be localized. Difficult ripping conditions and the generation of oversize material should be expected within these cemented zones. Cemented zones and concretions will likely be present in the formational materials.

9.2.2 We expect the soil within the upper five feet of proposed grade to be "expansive" (Expansion Index [EI] greater than 20) as defined by 2010 California Building Code (CBC) Section 1803.5.3. Table 9.2.1 presents soil classifications based on the expansion index.

**TABLE 9.2.1**
**SOIL CLASSIFICATION BASED ON EXPANSION INDEX**

| Expansion Index (EI) | Expansion Classification | 2010 CBC Expansion Classification |
|---|---|---|
| 0 – 20 | Very Low | Non-Expansive |
| 21 – 50 | Low | Expansive |
| 51 – 90 | Medium | |
| 91 – 130 | High | |
| Greater Than 130 | Very High | |

9.2.3 We performed laboratory Expansion Index testing on several samples of material expected to be exposed near the proposed grades. The test results are summarized in Appendix B and indicate the on-site material is expected to possess an Expansion Index of 130 or less corresponding to a "very low" to "high" expansion potential. Additional testing for expansion potential should be performed during grading once final grades are achieved.

9.2.4 We performed laboratory tests on samples of the site materials to evaluate the percentage of water-soluble sulfate content. Results from the laboratory water-soluble sulfate content tests are presented in Appendix B. The results indicate that the on-site materials at the locations tested possess "moderate" to "severe" sulfate exposure to concrete structures as defined by 2010 CBC Section 1904.3 and ACI 318. Table 9.2.2 presents a summary of concrete requirements set forth by 2010 CBC Section 1904.3 and ACI 318. The presence of water-soluble sulfates is not a visually discernible characteristic; therefore, other soil samples from the site could yield different concentrations. Additionally, landscaping activities (i.e., addition of fertilizers and other soil nutrients) may affect the concentration over time.

**TABLE 9.2.2**
**REQUIREMENTS FOR CONCRETE**
**EXPOSED TO SULFATE-CONTAINING SOLUTIONS**

| Sulfate Exposure | Exposure Class | Water-Soluble Sulfate Percent by Weight | Cement Type | Maximum Water to Cement Ratio by Weight | Minimum Compressive Strength (psi) |
|---|---|---|---|---|---|
| Not Applicable | S0 | 0.00-0.10 | -- | -- | 2,500 |
| Moderate | S1 | 0.10-0.20 | II | 0.50 | 4,000 |
| Severe | S2 | 0.20-2.00 | V | 0.45 | 4,500 |
| Very Severe | S3 | > 2.00 | V+ Pozzolan or Slag | 0.45 | 4,500 |

9.2.5 We selected samples to perform potential of hydrogen (pH), resistivity, and water-soluble chloride testing to help evaluate the corrosion potential of the planned improvements. The laboratory test results are presented in Appendix B and should be considered for the design of underground structures.

9.2.6 Geocon Incorporated does not practice in the field of corrosion engineering. Therefore, a registered corrosion engineer may be retained if improvements that could be susceptible to corrosion are planned. Their study should evaluate corrosion test results and incorporate the necessary precautions to avoid premature corrosion on buried metal pipes and concrete structures in direct contact with the soils.

## 9.3 Seismic Design Criteria

9.3.1 We used the computer program Seismic Hazard Curves and Uniform Hazard Response Spectra, provided by the USGS to calculate the seismic design parameters. Table 9.3 summarizes design criteria obtained from the 2010 CBC (based on the 2009 International Building Code [IBC]), Chapter 16 Structural Design, Section 1613 Earthquake Loads. The short spectral response uses a period of 0.2 second. The structures should be designed using Site Class C where there is less than 20 feet of fill and Site Class D where the fill thickness is 20 feet or greater. We evaluated the site class in accordance with Section 1613.5.5 of the CBC. We will evaluate the structure site class for each residential and commercial building once the final grading has been completed.

**TABLE 9.3**
**2010 CBC SEISMIC DESIGN PARAMETERS**

| Parameter | Value | | 2010 CBC Reference |
|---|---|---|---|
| Site Class | C | D | Table 1613.5.2 |
| Spectral Response – Class B (short), $S_S$ | 1.396g | 1.396g | Table 1613.5(3) |
| Spectral Response – Class B (1 sec), $S_1$ | 0.504g | 0.504g | Table 1613.5(4) |
| Site Coefficient, $F_A$ | 1.000 | 1.000 | Figure 1613.5.3(1) |
| Site Coefficient, $F_V$ | 1.300 | 1.500 | Figure 1613.5.3(2) |
| Maximum Considered Earthquake Spectral Response Acceleration (short), $S_{MS}$ | 1.396g | 1.396g | Section 1613.5.3 (Eqn 16-36) |
| Maximum Considered Earthquake Spectral Response Acceleration – (1 sec), $S_{M1}$ | 0.655g | 0.756g | Section 1613.5.3 (Eqn 16-37) |
| 5% Damped Design Spectral Response Acceleration (short), $S_{DS}$ | 0.931g | 0.931g | Section 1613.5.4 (Eqn 16-38) |
| 5% Damped Design Spectral Response Acceleration (1 sec), $S_{D1}$ | 0.437g | 0.504g | Section 1613.5.4 (Eqn 16-39) |

9.3.2 Conformance to the criteria in Tables 9.3 for seismic design does not constitute any kind of guarantee or assurance that significant structural damage or ground failure will not occur if a large earthquake occurs. The primary goal of seismic design is to protect life, not to avoid all damage, since such design may be economically prohibitive.

## 9.4 Slope Stability Analyses

9.4.1 We performed slope stability analyses using the two-dimensional computer program *GeoStudio2004* created by Geo-Slope International Ltd. We calculated the factor of safety for the planned slopes for rotational-mode and block-mode analyses using the Spencer's method. Output of the computer program including the calculated factor of safety and the failure surface is presented in Appendix C.

9.4.2 We used average drained direct shear strength parameters based on laboratory tests and our experience with similar soil types in nearby areas for the slope stability analyses. Our calculations indicate the proposed slopes, constructed of on-site materials, should have calculated factors of safety (FOS) of at least 1.5 and 1.1 under static and pseudo-static conditions, respectively, for both deep-seated failure and shallow sloughing conditions when the recommendations of this report are followed.

9.4.3 We selected Cross-Sections E-E′, F-F′, H-H′, I-I′, L-L′ , M-M′, N-N′, O-O′, and P-P′ to perform the slope stability analyses. Appendix C presents the results of the slope stability analyses.

9.4.4 The shallow landslide deposits and alluvium encountered within the site, as depicted on the Geologic Map, Figures 2 and 3 will require remedial grading in areas beyond the limits of grading within the western, southern, and eastern portions of the site generally at the toes of slopes.

9.4.5 The proposed fill slopes with MSE walls will require slope stabilization measures to achieve acceptable slope stability. The general configuration of the zones required to be reinforced are shown on figures in Appendix C. Recommendations regarding the geotechnical aspects of the proposed MSE walls are provided herein.

9.4.6 Among the slopes analyzed for acceptable calculated factors of safety, Cross-Sections E-E′, H-H′, L-L′, and N-N′ will require buttresses due to the presence of bedding plane shears, out-of-slope bedding orientations, and weak siltstone layers. Buttress designs have assumed a 1:1 (horizontal:vertical) frontcut and backcut extending down to intercept the critical bedding plane shears or weak zones.

9.4.7 MSE wall reinforcements should be designed by the wall contractor. For the purposes of this report, reinforcements with Geosynthetic grids were incorporated into the slope stability analyses as provided in Appendix C. The wall contractor should provide design details and alternatives based on the geotechnical data presented in this report. However, the required lengths and grid types presented in Appendix C should be incorporated into the design of the walls.

9.4.8 Due to the very light loads expected from the planned homes and improvements, the loads are considered negligible with no appreciable impact to the slope stability analyses and, therefore were not incorporated into the analyses.

9.4.9 Buttress and stability fill shear keys and associated subdrains should be surveyed during construction and depicted on the final as-built 40-scale grading plans.

9.4.10 Excavations including buttresses, shear keys, and stability fills should be observed during grading by an engineering geologist to evaluate whether soil and geologic conditions do not differ significantly from those expected or identified in this report.

9.4.11 We performed the slope stability analyses based on the interpretation of geologic conditions encountered during our field investigation. In certain areas, the geologic conditions such as the localized or continuous features of the bedding plane shears may need to be further defined by additional borings based on our review of the 40-scale grading plans.

## 9.5 Grading

9.5.1 Grading should be performed in accordance with the *Recommended Grading Specifications* contained in Appendix D and the City of Lake Forest Grading Ordinance. Where the recommendations of Appendix D conflict with this section, the recommendations of this section should take precedence.

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

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

9.5.4 Topsoil, colluvium, alluvium, landslide debris, undocumented fill, and the unsuitable portions of previously placed fill and formational materials within the limits of grading should be removed to expose firm, formational materials or moist, dense previously placed fill. Removals will be required beyond the toe of slope and extend to the property line to remove landslide debris and alluvium along the west, south, and east portions of the site (see Figures 2 and 3). The approximate thickness of the surficial soil is presented on the Geologic Map. We estimate that the upper approximately 3 to 5 feet of the previously placed fill will require remedial grading. We should evaluate the actual depth of removal during grading operations. The bottom of the excavation should be scarified at least 1 foot, moisture conditioned as necessary, and compacted prior to the placement of fill material. Excavated soil with an expansion index greater than 90 should be kept at least 4 feet below finish grade in areas of the structural fill, where possible.

9.5.5 To reduce the potential for differential settlement, the building pads with cut-fill transitions should be undercut at least 3 feet and sloped 1 percent to the adjacent street or deepest fill. Where the thickness of the fill below the building pad exceeds 15 feet, the depth of the undercut should be increased to one-fifth of the maximum fill thickness. In addition, cut pads that expose expansive siltstone and claystone or cemented formational materials should also be undercut at least 3 feet to mitigate soil expansion and facilitate future trenching.

9.5.6 Wet soil conditions should be expected within the existing detention basin. Remedial grading may be difficult in this area and may require top loading with the use of an excavator. The excavated materials can then be properly moisture conditioned prior to placing as fill material. This may require mixing with dryer materials to achieve proper compaction. We expect deeper removals within the basin due to the wet conditions.

9.5.7 Fill placed within the upper 40 vertical feet of proposed finish grade during the planned grading operations should be compacted to a dry density of at least 90 percent of the laboratory maximum dry density, near to slightly above optimum moisture content, as determined by ASTM Test Method D 1557. Fill placed 40 feet and deeper should be compacted to a dry density of at least 92 percent of the laboratory maximum dry density near to slightly above optimum moisture content. The siltstone and claystone soil materials should be placed at least 2 percent to 5 percent above optimum moisture content. The upper 12 inches of fill beneath the pavement structural section should be moisture conditioned and compacted to a dry density of at least 95 percent of the laboratory maximum dry density near to slightly above optimum moisture content shortly before paving operations. If construction delays or the weather results in the surface of the fill drying, the surface should be scarified and moisture conditioned before the next layer of fill is placed.

9.5.8 Cobbles or concretions greater than 1 foot in maximum dimension should not be placed within 5 feet of finish grade or 3 feet of the deepest utility. Cobbles and concretions greater than 6 inches in maximum dimension should not be placed within 3 feet of finish grade.

9.5.9 The proposed slopes will locally require buttressing and stability fills to obtain a factor of safety of at least 1.5 due to the presence of bedding plane shears and weak clay layers with out-of-slope orientations. We should perform additional slope stability analyses during preparation of the 40-scale grading plans in the areas of the buttress slopes to further evaluate the limits of the buttresses. Buttress plans will be prepared using these plans that will include proposed buttress widths, subdrain locations and elevations, tie-in and outlet points, and bottom elevations.

9.5.10 Stability fills will be required where formational siltstone/claystone is exposed in the proposed cut slopes during grading operations. A Typical Stability Fill Detail is presented on Figure 12 and should be used for design and construction of stability fills, where required. The backcut for the stability fills should commence at least 10 feet from the top of the proposed finish-graded slope and should extend at least 3 feet below adjacent pad grade, to a maximum depth of 15 feet below finish-pad grade. Lots adjacent to the stability fills may require undercutting due to the installation of the stability fill. Stability fills may also be required on cut slopes where cohesionless sand is encountered.

9.5.11 Cut slope excavations including buttresses and shear keys should be observed during grading operations to check that soil and geologic conditions do not differ significantly from those expected. During the construction of buttresses and during landslide removals, there is a risk that the temporary backcut slopes will become unstable. This risk can be reduced by grading the buttress fill in short segments and/or flattening the inclination of the temporary slopes. These excavations should be backfilled as soon as possible after establishing the shear key.

9.5.12 The outer 15 feet (or a distance equal to the height of the slope, whichever is less) of fill slopes should be composed of properly compacted granular soil to reduce the potential for surficial sloughing. In general, soil with an expansion index of 90 or less 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. The use of cohesionless soil in the outer portion of fill slopes should be avoided. Fill slopes should be overbuilt at least 3 feet and cut back to establish the finished sloped. Track walking of fill slopes will not be acceptable.

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

## 9.6 Temporary Excavations

9.6.1 The stability of the excavations is dependent on the design and construction of the shoring system. Therefore, Geocon Incorporated cannot be responsible for site safety and the stability of the proposed excavations. It is the responsibility of the contractor to follow all applicable safety standards and industry protocols when performing excavations during the construction of the proposed project.

9.6.2 Temporary slopes should be made in conformance with OSHA requirements. The undocumented fill and surficial soil should be considered a Type C soil, properly compacted fill should be considered a Type B soil (Type C soil if seepage is encountered), and the formational materials should be considered a Type A soil (Type B soil if seepage is encountered) in accordance with OSHA requirements. In general, special shoring requirement will not be necessary if temporary excavations will be less than 4 feet high. However, temporary excavation depths greater than 4 feet should be laid back at an appropriate inclination in accordance with OSHA recommendations. These excavations should not be allowed to become saturated or allowed to dry appreciably. Surcharge loads should not be permitted within a distance equal to the depth of the excavation from the top of the excavation. The top of the excavation should be a minimum of 15 feet from the edge of existing improvements. Excavations steeper than those recommended or closer than 15 feet from an existing surface improvement should be shored in accordance with applicable OSHA codes and regulations.

## 9.7 Settlement of Existing and Proposed Fill

9.7.1 Previously placed engineered fill encountered on the site with thicknesses ranging up to approximately 125 feet was placed at the site approximately 16 to 25 years ago for the roadways and sheet-graded pads. Geocon installed 10 surface settlement monuments in April 2007 to monitor the amount of surface settlement occurring in the previously placed engineered fill areas. The approximate locations of the settlement monuments for areas within Portola Center South are depicted on the Geologic Map (Figures 2 and 3). These monuments have been surveyed by Hunsaker & Associates Irvine, Incorporated on a periodic basis. Table 9.7 presents the results of the settlement monitoring for both Portola Center North and South. The existing fill at the locations of the monuments have experienced settlements of 0.12 to 0.84 inch with an average of 0.43 inch between April 2007 and October 2011. The percentage of settlement to fill thickness is 0.01 to 0.08 with an average of 0.04 percent. The monitoring data suggests that long term settlement due to consolidation of the fill is near completion.

**TABLE 9.7**
**SETTLEMENT OF EXISTING FILL**

| Monument Number and Location | Approximate Depth of Fill Below Monument (feet) | Monument Elevation April, 2007 (feet above MSL) | Monument Elevation October, 2011 (feet above MSL) | Elevation Differential (feet [%]) |
|---|---|---|---|---|
| SM-10001 (South) | 90 | 1013.24 | 1013.23 | -0.01 [-0.01] |
| SM-10002 (South) | 110 | 1053.71 | 1053.65 | -0.06 [-0.05] |
| SM-10003 (South) | 85 | 1069.41 | 1069.37 | -0.04 [-0.05] |
| SM-10004 (North) | 130 | 1123.10 | 1123.06 | -0.04 [-0.03] |
| SM-10005 (North) | 70 | 1130.32 | 1130.31 | -0.01 [-0.01] |
| SM-10006 (North) | 95 | 1079.88 | 1079.82 | -0.07 [-0.07] |
| SM-10007 (North) | 105 | 1057.43 | 1057.41 | -0.02 [-0.02] |
| SM-10008 (North) | 60 | 1080.93 | 1080.88 | -0.05 [-0.08] |
| SM-10009 (South) | 80 | 1019.06 | 1019.04 | -0.02 [-0.02] |
| SM-10010 (North) | 110 | 1060.70 | 1060.65 | -0.04 [-0.04] |

9.7.2 Planned grading will result in the placement of up to 115 feet of new fill for the proposed development. In addition, the maximum depth of new fill placed on existing fill soil is approximately 80 feet resulting in a maximum fill thickness of 105 feet. Based on the results of our laboratory tests, we expect the existing fill will settle up to about 1½ inches where 80 feet of new fill will be placed over the existing 25 feet of fill. The estimated settlement will occur relatively quickly during the placement of the fill; however, settlement monitoring should occur on the fill as discussed herein.

9.7.3 The post-grading settlement (hydrocompression) of properly compacted new fill with a maximum thickness of 115 feet could reach up to about 5½ inches. We expect the settlement will occur over 20+ years depending on the influx of rain and irrigation water into the fill mass. This settlement will likely be linear from the time the fill is placed to the end of the settlement period. We do not expect the settlement will impact proposed utilities with proposed gradients of 1 percent or greater.

9.7.4 Settlement deformations should be expected for MSE walls with extensive Geosynthetic reinforcements. The estimated vertical and horizontal deformations due to the construction of the planned MSE walls will be provided in a separate report. The calculated deflections should be provided to the project structural engineer to determine if the planned structures can tolerate the expected movement. Significant construction waiting periods of up to 3 to 9 months may be required if the structures cannot handle the estimated deflections.

9.7.5 Additional surface settlement and lateral deflection monuments should be installed in fill areas deeper than 30 feet subsequent to grading. The project surveyor should record the movements every two weeks until data indicates that the rate of primary fill compression is essentially non-detrimental to proposed improvements. Based on our experience, we expect the monuments will be required to be monitored for at least 90 days. At that time, we expect development can begin for settlement-sensitive underground utilities with less than one percent gradient and structures in new fill areas deeper than 30 feet. Underground utilities with a gradient of one percent or greater will not have a waiting period and can start construction after finish grade is achieved. Geocon should evaluate the locations and number of monuments once 40-scale grading plans have been developed and based on the final configuration of the proposed MSE walls and geologic conditions.

## 9.8 Earthwork Grading Factors

9.8.1 Estimates of embankment shrink-swell factors are based on comparing laboratory compaction tests with the density of the material in its natural state and experience with similar soil types. Variations in natural soil density and in compacted fill render shrinkage value estimates very approximate. As an example, the contractor can compact fill to a density 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 discussion herein, the earthwork factors in Table 9.8 may be used as a basis for estimating how much the on-site soils may shrink or bulk when removed from their natural state and placed in compacted fill.

**TABLE 9.8**
**SHRINKAGE AND BULK FACTORS**

| Soil Unit | Shrink/Bulk Factor |
|---|---|
| Undocumented Fill | 5-10 percent shrink |
| Previously Placed Fill | 0-2 percent shrink |
| Topsoil, Alluvium and Colluvium | 10-15 percent shrink |
| Landslide Debris | 10-15 percent shrink |
| Terrace Deposits | 2 percent shrink to 2 percent bulk |
| Capistrano Formation and Sandstone Units of Puente Formation | 3-5 percent bulk |
| Siltstone Units of Puente Formation | 3-5 percent bulk |

## 9.9 Subdrains

9.9.1 Conditions encountered prior to and during grading do not necessarily reveal the conditions that will be realized once construction of the proposed development is completed.

Specifically, irrigation both on site and within up gradient areas cannot be reasonably predicted. Therefore, the design and implementation of additional drainage mechanisms will be necessary. The geologic units encountered on the site have permeability characteristics and/or fracture systems that could be susceptible under certain conditions to seepage. Building pad areas adjacent to ascending slopes may experience wet to saturated soil conditions due to water migration or seepage. To reduce the potential for this to occur, a toe drain should be placed along the base of ascending slopes to collect potential seepage and convey it to a suitable outlet. The drain should be sufficiently deep to intercept the seepage (on the order of 3 feet below finish grade). The necessity for the drains should be discussed prior to grading on a slope specific basis. In addition, the project civil engineer should be consulted to evaluate the appropriate drain locations and necessary easements, building restriction zones or disclosure requirements that may be necessary. The drains should be surveyed for location and shown on the project as-built drawings. As an alternative, a small retaining wall approximately 3 to 4 feet in height that contains subsurface drainage behind the wall can be placed at the toe of ascending slopes.

9.9.2 Canyon subdrains within the major drainages and a stability fill were constructed during previous grading operations. The reported locations, pipe diameters, and elevations are presented on the Geologic Map. The existing subdrain outlets were surveyed for location by the civil engineer. The pipe size and diameter were verified in the field by Geocon Incorporated. It is our opinion that the existing subdrains were properly constructed and in adequate condition to accommodate the proposed development and addition of new fill soils.

9.9.3 Proposed grading will remove some existing drains or require the placement of additional fill soil. The outlet locations of the subdrains are shown on the geologic maps. Some of the outlets will need future tie-ins or extensions. Specific locations for future tie-ins, connection points and elevations will be analyzed once 40-scale grading and improvement plans are prepared. Two new subdrains in natural drainages will be necessary to mitigate the potential for adverse impacts associated with seepage conditions. Some removal of existing fill soil may be required to provide an adequate subdrain tie-in or removal of segments of pipe. In addition, excavations within existing fill areas may expose the upper portions of the existing canyon subdrains that will require removal and repair of the existing pipes to at least 10 feet below new finish grade. The locations of proposed canyon subdrains and subdrain extensions are presented on the Geologic Map. A typical canyon subdrain detail is presented in Figure 13. Subdrains less than 750 feet in length and located at the base of fill less than 100 feet in depth should use 6-inch-diameter schedule 40 PVC perforated pipes. Other subdrains should use 8-inch-diameter schedule 80 PVC perforated pipe. Subdrain extensions should be connected to the existing canyon subdrain at their intersection point using pipes with the same diameter. Subdrains within the buttress and stability fill keyways can use Schedule 40 PVC perforated pipes with a diameter of at least 4 inches.

9.9.4 Prior to outletting, the final 20-foot segment of subdrain should consist of non-perforated drain pipe. At the non-perforated/perforated interface, a seepage cutoff wall should be constructed on the downslope side of the junction in accordance with Figure 14. Subdrains that discharge into a natural drainage course or open space area should be provided with a permanent headwall structure in accordance with Figure 15. Coordination between the MSE wall and grading contractors will be required to allow the proper outlet of canyon subdrains and wall drains and mitigate conflicts during construction. Verification of proper flow of the existing subdrain outlets should be performed with the addition of a permanent headwalls once finish grades have been achieved.

9.9.5 The final 40-scale grading plans should show the location of 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 the existing conditions. The final outlet and connection locations should be evaluated during grading operations.

## 9.10 Foundation and Concrete Slabs-On-Grade Recommendations

9.10.1 The foundation recommendations presented herein are for proposed one- to three-story residential structures. We separated the foundation recommendations into three categories based on either the maximum and differential fill thickness or Expansion Index. The foundation category criteria are presented in Table 9.10.1. We will evaluate the Final foundation categories once site grading has been completed.

**TABLE 9.10.1**
**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 |

9.10.2 Table 9.10.2 presents minimum foundation and interior concrete slab design criteria for conventional foundation systems. This foundation system should only be used on cut lots with a very low to low expansion potential within the sandstone portions of the formational units.

**TABLE 9.10.2**
**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 at slab mid-point |
| III | 24 | Four No. 5 bars, two top and two bottom | No. 3 bars at 18 inches on center, both directions at slab mid-point |

9.10.3 The embedment depths presented in Table 9.10.2 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. Figure 16 presents a wall/column footing dimension detail.

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

9.10.5 The bedding sand thickness should be determined by the project foundation engineer, architect, and/or developer. It is typical to have 3 inches and 4 inches of sand for 5-inch thick and 4-inch thick slabs, respectively, in the southern California area. The foundation 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 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.

9.10.6 Post-tensioned concrete slab and foundation systems should be used for the support of the proposed structures on fill soils or building pads with a medium to high expansion potential. The 2010 CBC has updated the design requirements for post-tensioned foundation systems. The post-tensioned systems should be designed by a structural engineer experienced in post-tensioned slab design and design criteria of the Post-Tensioning Institute (PTI), Third Edition, as required by the 2010 California Building Code (CBC Section 1805.8). 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 9.10.3 for the particular Foundation Category designated. The parameters presented in Table 9.10.3 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 9.10.3**
**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 |

9.10.7 If the structural engineer proposes a post-tensioned foundation design method other than the 2010 CBC:

- The criteria presented in Table 9.10.3 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.

9.10.8 We recommend that a post-tensioned mat foundation system be used where the MSE wall grids within the reinforced zone extend into the building pads. The slab should possess a thickened edge with a minimum width of 12 inches and extend below the clean sand or crushed rock layer.

9.10.9 Our experience indicates post-tensioned slabs can be susceptible to 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 and the contractor should properly construct the foundation system to reduce the potential of edge lift occurring for the proposed structures.

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

9.10.11 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.

9.10.12 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 slab and support structural elements connected to the building, are not recommended. Where this condition cannot be avoided, the isolated footings should be connected to the building foundation system with grade beams.

9.10.13 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.

9.10.14 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 such concrete placement.

9.10.15 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.

- Geocon Incorporated should be contacted to review the pool plans and the specific site conditions to provide additional recommendations, if necessary.

- 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.

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

9.10.16 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.

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

## 9.11 Exterior Concrete Flatwork

9.11.1 Exterior concrete flatwork not subject to vehicular traffic should be constructed in accordance with the recommendations herein. Slab panels should be a minimum of 4 inches thick and when in excess of 8 feet square should be reinforced with 6x6-W2.9/W2.9 (6x6-6/6) welded wire mesh or No. 3 reinforcing bars spaced 18 inches on center in both directions 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 upon the slab thickness and intended usage. Criteria of the American Concrete Institute (ACI) should be taken into consideration when establishing crack control spacing. Subgrade soil for exterior slabs not subjected to vehicle loads should be compacted in accordance with criteria presented in the grading section prior to concrete placement. Subgrade soil should be properly compacted and the moisture content of subgrade soil should be verified prior to placing concrete.

9.11.2 Even with the incorporation of the recommendations of this report, the exterior concrete flatwork has a potential to experience some uplift due to expansive soil beneath grade. The reinforcing steel should overlap continuously in flatwork to reduce the potential for vertical offsets within flatwork. Additionally, flatwork should be structurally connected to the curbs, where possible, to reduce the potential for offsets between the curbs and the flatwork.

9.11.3 Where exterior flatwork abuts the structure at entrant or exit points, the exterior slab should be dowelled into the structure's foundation stemwall. This recommendation is intended to reduce the potential for differential elevations that could result from differential settlement or minor heave of the flatwork. Dowelling details should be designed by the project structural engineer. In addition, concrete sidewalks that are placed adjacent to curbs should be dowelled into the curb to reduce the potential for vertical offsets.

9.11.4 The recommendations presented herein are intended to reduce the potential for cracking of exterior slabs as a result of differential movement. However, even with the incorporation of the recommendations presented herein, slabs-on-grade will still crack. The occurrence of concrete shrinkage cracks is independent of the soil supporting characteristics. Their occurrence may be reduced and/or controlled by limiting the slump of the concrete, the use of crack control joints and proper concrete placement and curing. Crack control joints should be spaced at intervals no greater than 12 feet. Literature provided by the Portland

Concrete Association (PCA) and American Concrete Institute (ACI) present recommendations for proper concrete mix, construction, and curing practices, and should be incorporated into project construction.

## 9.12 Retaining Wall Recommendations

9.12.1 Retaining walls that are allowed to rotate more than 0.001H (where H equals the height of the retaining portion of the wall in feet) at the top of the wall and having a level backfill surface should be designed for an active soil pressure equivalent to the pressure exerted by a fluid density of 40 pounds per cubic foot (pcf). Where the backfill will be inclined at no steeper than 2:1 (horizontal:vertical), an active soil pressure of 55 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 90 or less.

9.12.2 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.

9.12.3 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 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 backfill material (EI of 90 or less) with no hydrostatic forces or imposed surcharge load. Figure 17 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.

9.12.4 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.

9.12.5 The structural engineer should determine the seismic design category for the project. If the project possesses a seismic design category of D, E, or F, the proposed retaining walls should be designed with seismic lateral pressure. A seismic load of 16H should be used for design. The seismic load is dependent on the retained height where H is the height of the wall, in feet, and the calculated loads result in pounds per square foot (psf) exerted at the top of the wall and zero at the base of the wall. We used a peak site acceleration of 0.37g calculated from the 2010 California Building Code ($S_{DS}$/2.5) and applying a pseudo-static coefficient of 0.33.

9.12.6 The recommendations presented herein are generally applicable to the design of rigid concrete or masonry retaining walls having a maximum height of 8 feet. In the event that walls higher than 8 feet or other types of walls (such as crib-type walls) are planned, Geocon Incorporated should be consulted for additional recommendations.

9.12.7 Unrestrained walls will move laterally when backfilled and loading is applied. The amount of lateral deflection is dependent on the wall height, the type of soil used for backfill, and loads acting on the wall. The retaining walls and improvements above the retaining walls should be designed to incorporate an appropriate amount of lateral deflection as determined by the structural engineer.

9.12.8 The recommendations presented herein are generally applicable to the design of rigid concrete or masonry retaining walls having a maximum height of 8 feet. Recommendations for MSE, soldier pile, and soil nail retaining walls are presented herein.

## 9.13 Mechanically Stabilized Earth (MSE) Retaining Walls

9.13.1 Mechanized stabilized earth (MSE) retaining walls are associated with proposed fill slopes to a maximum height of approximately 95 feet throughout the development. Combined retaining wall heights are expected to range up to a maximum of 95 feet, with a maximum geogrid length of approximately 105 feet. Mechanically stabilized earth (MSE) retaining walls are alternative walls that consist of modular block facing units with geogrid reinforced earth behind the block. The reinforcement grid attaches to the block units and is typically placed at specified vertical intervals and embedment lengths. For the purposes of this report, the spacing and lengths and types of the geogrid were assumed based on the expected type of soil used for the backfill, and the slope stability requirements to achieve an acceptable factor of safety.

9.13.2 The geotechnical parameters listed in Table 9.13 can be used for preliminary design of the MSE walls.

**TABLE 9.13**
**GEOTECHNICAL PARAMETERS FOR MSE WALLS**

| Parameter | Reinforced Zone | Retained Zone | Foundation Zone |
|---|---|---|---|
| Angle of Internal Friction | 32 degrees | 28 degrees | 28 degrees |
| Cohesion | 500 psf | 500 psf | 500 psf |
| Wet Unit Density | 120 pcf | 120 pcf | 120 pcf |

9.13.3 The soil parameters presented in Table 9.13 are based on our experience and direct shear-strength tests performed during the geotechnical investigation and represent some of the on-site materials. The wet unit density values presented in Table 9.13 can be used for design but actual in-place densities may range from approximately 90 to 135 pounds per cubic foot. Geocon has no way of knowing whether these materials will actually be used as backfill behind the wall during construction. The wall designers should use their judgment in selection of the design parameters. As such, once backfill materials have been selected and/or stockpiled, sufficient shear tests should be conducted on samples of the proposed backfill materials to check that they conform to actual design values. Results should be provided to the designer to re-evaluate stability of the walls. Dependent upon test results, the designer may require modifications to the original wall design (e.g., longer reinforcement embedment lengths and/or steel reinforcement).

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

9.13.5 Wall foundations having a minimum depth and width of one foot may be designed for an allowable soil bearing pressure of 2,000 psf. This soil pressure may be increased by 300 psf and 500 psf for each additional foot of foundation width and depth, respectively, up to a maximum allowable soil bearing pressure of 4,000 psf.

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

9.13.7 Select backfill materials will be required to be in accordance with the MSE retaining wall system. Materials as outlined in the specifications of the retaining wall plans may be generated and stockpiled during grading, if encountered, or may require import. Geocon should perform laboratory tests during the backfill materials to check that soil properties are in accordance with the retaining wall plans and specifications. Based on the results of our field investigation and laboratory testing, materials within the Puente Formation-Soquel Member (Tps) and the Capistrano Formation (Tco) will be potential sources of granular material to create select backfill.

9.13.8 The wall should be provided with a drainage system sufficient to prevent excessive seepage through the wall and the base of the wall, thus preventing hydrostatic pressures behind the wall. The perforated drainage pipe should be wrapped in an approved filter fabric.

9.13.9 Geosynthetic reinforcement must elongate to develop full tensile resistance. This elongation generally results in movement at the top of the wall. The amount of movement is dependent upon the height of the wall (e.g., higher walls rotate more) and the type of reinforcing grid used. In addition, over time the reinforcement grid has been known to exhibit creep (sometimes as much as 5 percent) and can undergo additional movement. Given this condition, the owner should be aware that structures and pavement placed within the reinforced and retained zones of the wall may undergo movement. A separate report will include the estimated vertical and horizontal deflections of the planned MSE retaining walls. The estimated movements should be provided to the project structural engineer to determine if the planned structures can tolerate the expected movements.

9.13.10 A geotechnical *in situ* monitoring program should be performed during the site grading and long term post-grading to observe the settlement of the fill slopes and the vertical and horizontal movements of MSE walls. The observation instrumentations should include settlement and lateral deflection monuments/survey points on the tops of retaining walls. Information regarding the progress of fill placement should also be recorded as a part of monitoring program.

9.13.11 MSE walls can be constructed using metallic reinforcement in the reinforced zone to prevent the significant deformations that would be expected in similar-height walls reinforced with extensive Geosynthetic reinforcements. The wall designer should evaluate the alternative with steel reinforcement during the design of the planned MSE walls.

9.13.12 Proposed retaining walls that are located near adjacent properties or property lines along the eastern boundary of the property will likely need to be supported by soil nail walls or soldier pile walls. The proposed MSE walls at these locations may not have sufficient space to install the horizontal grids to support the facing and will need to be constructed using top down methods. This supporting wall system will need to be designed by the structural engineer.

### 9.14 Soldier Pile Walls

9.14.1 Soldier pile walls can be constructed adjacent to property lines and improvements where the reinforcement grid may not be allowed to extend behind the face of the wall.

9.14.2 In general, ground conditions are moderately suited for soldier pile wall construction techniques. However, gravel, cobble, cemented zones, and oversized material may be encountered in the existing materials that could be difficult to drill. Additionally, if cohesionless sands are encountered, some raveling may result along the unsupported portions of excavations.

9.14.3 Geocon Incorporated should observe the drilled shafts for the soldier piles prior to the placement of steel reinforcement to check that the exposed soil conditions are similar to those expected and that footing excavations have been extended to the appropriate bearing strata and design depths. If unexpected soil conditions are encountered, foundation modifications may be required.

9.14.4 A wall drain system should be incorporated into the design of the soldier pile wall. Figure 18 presents a typical soldier pile wall drainage detail.

9.14.5 Lateral movement of shoring is associated with vertical ground settlement outside of the excavation. Therefore, it is essential that the soldier pile system allow very limited amounts of lateral displacement. Earth pressures acting on a lagging wall can cause movement of the shoring toward the excavation and result in ground subsidence outside of the excavation.

9.14.6 Lagging should keep pace with the excavation operations. The excavation should not be advanced deeper than three feet below the bottom of lagging at any time. These unlagged gaps of up to three feet should only be allowed to stand for short periods of time in order to decrease the probability of soil sloughing and caving and should never be unsupported overnight. Backfilling should be conducted between the back of lagging and excavation sidewalls to reduce sloughing in this zone and voids should be filled by the end of each day.

9.14.7 Prior to the commencement of excavation activities that have the potential to affect existing buildings, streets, sidewalks, and other structures/improvements, the condition of these existing structures, pavements, and/or improvements should be documented prior to the start of work. Special attention should be given to documenting existing cracks or other indications of differential settlement within these adjacent structures, pavements and other improvements. Underground utilities sensitive to settlement should be videotaped prior to construction to check the integrity of pipes. In addition, monitoring points should be established indicating location and elevation around the excavation.

## 9.15 Soil Nail Wall

9.15.1 Soil nail walls can be used where MSE walls cannot be constructed. Soil nail walls consist of installing closely spaced steel bars (nails) into a slope or excavation in a top-down construction sequence. Following installation of a horizontal row of nails, drains, waterproofing and wall reinforcing steel are placed and shotcrete applied to create a final wall.

9.15.2 The wall should be designed by an engineer familiar with the design of soil nail walls.

9.15.3 In general, ground conditions are moderately suited to soil nail wall construction techniques. However, localized gravel, cobble, cemented zones, and oversized material could be encountered in the existing materials that could be difficult to drill. Additionally, relatively clean sands may be encountered within the existing soil that may result in some raveling of the unsupported excavation.

9.15.4 A wall drain system should be incorporated into the design of the soil nail wall. Corrosion protection should be provided for the nails where the wall will be a permanent structure. Figure 19 presents a typical soil nail wall drainage detail.

9.15.5 Testing of the soil nails should be performed in accordance with the guidelines of the Federal Highway Administration or similar guidelines. At least two passing verification tests should be performed to confirm design assumptions for each soil/rock type encountered. Verification tests of soil nails should be sacrificial and should not be used to support the proposed wall. The bond length should be adjusted to allow for pullout testing of the verification nails to evaluate the ultimate bond stress. A minimum of 5 percent of the production nails should also be proof tested and a minimum of 4 sacrificial nails should be tested at the discretion of Geocon Incorporated. Consideration should be given to testing sacrificial nails with an adjusted bond length rather than testing production nails. Geocon Incorporated should observe the nail installation and perform the nail testing.

9.15.6 The soil strength parameters listed in Table 9.15 can be used in design of the soil nails.

**TABLE 9.15**
**SOIL STRENGTH PARAMETERS FOR SOIL NAIL WALLS**

| Description | Cohesion (psf) | Friction Angle (degrees) | Ultimate Bond Stress (psi) |
|---|---|---|---|
| Engineered Artificial Fill (afe) | 500 | 28 | 10 |
| Puente Formation (Tps) | 400 | 33 | 20 |
| Puente Formation – Siltstone (Tps [slt] and Tplv) | 300 | 30 | 20 |

## 9.16 Lateral Loads

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

9.16.2 An allowable friction coefficient of 0.35 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.

## 9.17 Preliminary Pavement Recommendations

9.17.1 We calculated the flexible pavement sections in general conformance with the *Caltrans Method of Flexible Pavement Design* (Highway Design Manual, Section 608.4) using an estimated Traffic Index (TI) of 5.0, 5.5, 6.0, and 7.0 for the planned roadways. The project civil and traffic engineer and developer should review the pavement designations to determine appropriate locations for pavement thickness. The final pavement sections should be based on the R-Value of the subgrade soil encountered at final subgrade elevations. Streets should be designed in accordance with applicable standards when final Traffic Indices and R-value tests on subgrade soil are completed. We have assumed R-Values of 10 and 78 for the subgrade soil and base materials, respectively, for the purpose of the preliminary analyses. Table 9.17.1 presents options for asphalt concrete over base and full-depth asphalt concrete for the planned roadways.

**TABLE 9.17.1**
**PRELIMINARY FLEXIBLE PAVEMENT SECTION**

| Assumed Traffic Index | Assumed Subgrade R-Value | Option 1 | | Option 2 |
|---|---|---|---|---|
| | | Asphalt Concrete (inches) | Class 2 Aggregate Base (inches) | Full-Depth Asphalt Concrete Thickness (inches) |
| 5.0 | 10 | 3.0 | 9 | 7.5 |
| 5.5 | 10 | 3.0 | 11 | 8.0 |
| 6.0 | 10 | 3.5 | 12 | 9.0 |
| 7.0 | 10 | 4.0 | 15 | 10.5 |

9.17.2 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 shortly before paving operations. 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 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*). The asphalt concrete should be compacted to a density of at least 95 percent of the laboratory Hveem density in accordance with ASTM D 2726.

9.17.3 A rigid Portland cement concrete (PCC) pavement section should be placed in cross-gutters, private driveways, and driveway entrance aprons. 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 9.17.2.

**TABLE 9.17.2**
**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 |

9.17.4 Based on the criteria presented herein, the PCC pavement sections should have a minimum thickness as presented in Table 9.17.3.

**TABLE 9.17.3**
**RIGID PAVEMENT RECOMMENDATIONS**

| Location | Portland Cement Concrete (inches) |
|---|---|
| Private driveways and aprons | 5.5 |
| Cross-gutters and public driveway aprons (TC=C) | 7 |

9.17.5 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).

9.17.6 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.

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

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

9.17.9 The performance of pavement is highly dependent on providing positive surface drainage away from the edge of the pavement. Water that is allowed to pond on or adjacent to roadway pavement areas 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.

## 9.18 Site Drainage and Moisture Protection

9.18.1 Adequate site drainage is critical to reduce the potential for differential soil movement, erosion and subsurface seepage. Under no circumstances should water be allowed to pond adjacent to footings. The site should be graded and maintained such that surface drainage is directed away from structures in accordance with 2010 CBC 1804.3 and guidelines of the city of Lake Forest. 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.

9.18.2 Conditions encountered prior to and during grading do not necessarily reveal the conditions that will be encountered once construction of the proposed development is completed. Specifically, irrigation both on site and within up gradient areas cannot be reasonably predicted. Therefore the design and implementation of additional drainage mechanisms may be necessitated. The geologic units encountered on the site have permeability characteristics and/or fracture systems that could be susceptible under certain conditions to groundwater seepage. Building pad areas adjacent to ascending slopes may experience wet to saturated soil conditions due to water migration or seepage. To reduce the potential for this to occur, consideration should be given to placing a subdrain along the base of the slopes to collect potential seepage and convey it to a suitable outlet. The drain should be sufficiently deep to intercept the seepage (on the order of 3 feet below finish grade). The necessity for the drains should be discussed prior to grading on a slope specific basis. In addition, the project civil engineer should be consulted to evaluate the appropriate drain locations and necessary easements, building restriction zones or disclosure requirements that may be necessary. The drains should be surveyed for location and shown on the project as-built drawings.

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

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

9.18.5 We understand the property may incorporate storm water management devices that promote water storage but not water infiltration. The existing and planned soil conditions are not conducive to water infiltration and infiltration should not be performed. In addition, if water is allowed to infiltrate the soil, seepage may occur through the planned retaining walls and could cause slope instability. Water storage devices can be installed to reduce the velocity and amount of water entering the storm drain system but liners will be required if water in contact with soil. Distress may be caused to planned improvements and properties located hydrologically downstream if water infiltrates the soil. The distress depends on the amount of water to be detained, its residence time, soil permeability, and other factors. We have not performed a hydrogeology study at the site. If infiltration of storm water runoff was incorporated into the project design, downstream properties may be subjected to seeps, springs, slope instability, raised groundwater, movement of foundations and slabs, or other undesirable impacts as a result of this water infiltration.

9.18.6 Storm water management devices should be properly constructed to prevent water infiltration and lined with an impermeable liner (e.g. High-density polyethylene, HDPE, with a thickness of about 12 mil or equivalent Polyvinyl Chloride, PVC, liner). The devices should also be installed in accordance with the manufacturer's recommendations.

9.18.7 We recommend roof drains be connected to subsurface drains that direct the water to a storm drain system. However, we understand that the SUSMP and Leadership in Engineering and Environmental Design (LEED) requests disconnecting the roof drains to help obtain certification. The water from the roof drains should be directed away from buildings. Consideration should be given to draining roofs to lined planter boxes or placing liners below the proposed landscape areas to prevent infiltration of the water. Erosion control devices should be installed at the outlets to prevent soil migration during rain events. Geocon Incorporated can be contacted for additional recommendations.

9.18.8 If detention basins, bioswales, retention basins, or water infiltration devices are being considered, Geocon Incorporated should be retained to provide recommendations pertaining to the geotechnical aspects of possible impacts and design. Distress may be caused to planned improvements and properties located hydrologically downstream. The distress depends on the amount of water to be detained, its residence time, soil permeability, and other factors. We have not performed a hydrogeology study at the site. If infiltration of storm water runoff was incorporated into project design, downstream properties may be subjected to seeps, springs, slope instability, raised groundwater, movement of foundations and slabs, or other impacts as a result of water infiltration.

## 9.19 Grading, Improvement and Foundation Plan Review

9.19.1 Geocon should review the 40-scale grading plans, improvement and MSE wall plans, and foundation plans prior to finalization to verify their compliance with the recommendations of this report and determine the need for additional comments, recommendations, and/or analysis.

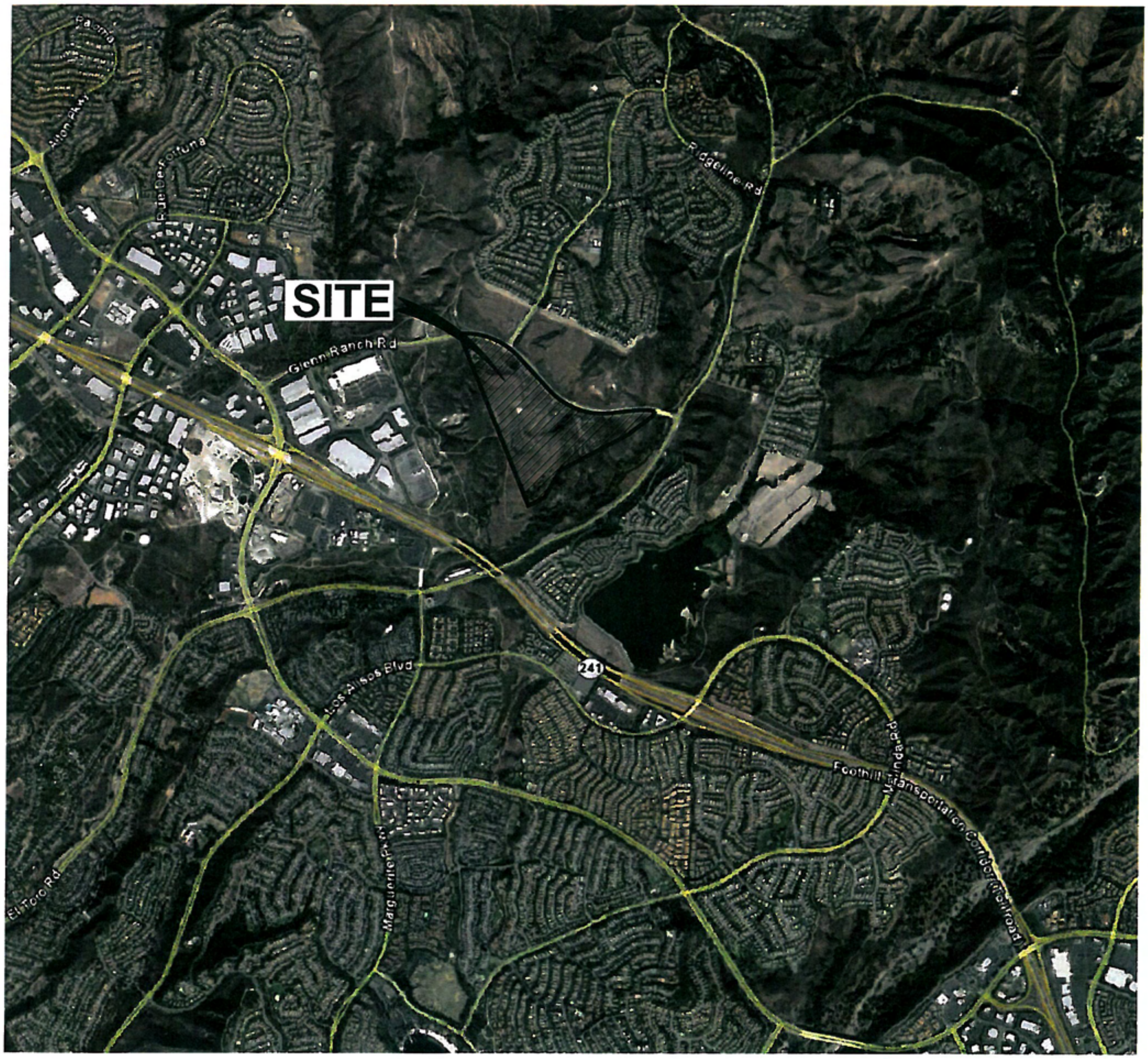
## LIMITATIONS AND UNIFORMITY OF CONDITIONS

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

PORTOLA CENTER SOUTH
TM #15353
LAKE FOREST, CALIFORNIA

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| CL / RA | | DSK/GTYPD | DATE 07 - 06 - 2012 | PROJECT NO. G1218 - 52 - 01A | FIG. 1 |

Vicinity Map

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