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Description:	N/A
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Organization:	Dudek
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MEMORANDUM

To:	California Energy Commission Staff
From:	Compass Energy Storage LLC
Subject:	Geotechnical Evaluation Report (TN #255561-6) References
Date:	June 26, 2025

Compass Energy Storage LLC (applicant) prepared a Geotechnical Evaluation Report for the proposed Compass Energy Storage Project, dated April 2024, titled Appendix 4-4A Geotechnical Evaluation Report Part 1, and docketed as Tracking Number 255561-6. The Geotechnical Evaluation Report referenced several geotechnical studies performed by prior projects and other studies performed by the applicant. As per California Energy Commission staff request, the following Geotechnical Evaluation Report references are being filed to the project docket (24-OPT-02). Each reference can be navigated via the bookmarks with the document. Page numbers are also provided below for navigation purposes. Due to file size requirements, the referenced reports are provided in three parts.

Referenced Reports	Location within this Document (PDF page number)
Part 1	
NMG Geotechnical, 2001. Geotechnical Due Diligence Investigation for Proposed Middle School, Rancho Capistrano, San Juan Capistrano, California, Project No. 01012-02, dated September 28, 2001.	Page 2-99
Part 2	
Leighton and Associates, Inc., 2009. Preliminary Geotechnical Investigation, The Orchards at Capistrano, San Juan Capistrano, Project No. 012383-001, dated April 30, 2009.	Page 2-486
Lowney and Associates performed an evaluation in 2003 consisting of excavation, sampling, and logging of one small diameter hollow stem boring and six large diameter borings.	Boring logs are available within Leighton and Associates, Inc. 2009 (Page 75-161)
Part 3	
Terracon, 2021. Geotechnical Engineering Report, Broad Reach Power Compass BESS, San Juan Capistrano, CA, Terracon Project No. 60211570, dated November 3, 2021.	Page 2-157
Geosyntec Consultants, Inc., 2021. Geotechnical and Geomorphic Evaluations, Compass Energy Storage Project, San Juan Capistrano, California, dated June 29, 2021.	Page 158-199
Corrosivity Study: HDR, 2023, Soil Corrosivity Study, Saddleback Valley Compass Substation, San Juan Capistrano, California, HDR#23-0169LAB, dated March 16, 2023.	Page 200-211
Hydrology study: Chang Consultants, 2024, Sediment Transport Analysis for the Compass Energy Storage Project, dated March 21, 2024.	Page 212-367
Thermal Resistivity study: Geotherm USA, 2023, Thermal Analysis of Native Soil Samples, San Juan Capistrano, California, PO No. 22011-01, dated March 7, 2023.	Page 368-372

PRELIMINARY GEOTECHNICAL INVESTIGATION
THE ORCHARDS AT CAPISTRANO
SAN JUAN CAPISTRANO, CALIFORNIA

VOLUME I

Prepared for:

Continuing Life Communities, LLC
1940 Levante Street
Carlsbad, California 92009

Project No. 012383-001

April 30, 2009

Leighton and Associates, Inc.
A LEIGHTON GROUP COMPANY

Leighton and Associates, Inc.
A LEIGHTON GROUP COMPANY

April 30, 2009

Project No. 012383-001

To: Continuing Life Communities, LLC
1940 Levante Street
Carlsbad, California 92009

Attention: Mr. Ryan Currie

Subject: Preliminary Geotechnical Investigation, The Orchards at Capistrano, San Juan Capistrano, California

In accordance with your request and authorization, we are pleased to present the results of our preliminary geotechnical investigation of the site proposed for The Orchards at Capistrano development in San Juan Capistrano, California. The accompanying report presents a summary of our site investigation activities and provides geotechnical conclusions and recommendations relative to the proposed site development. Additional studies will be required as plans become developed.

Data from our site investigations and observations, as well as from previous reports on the site and vicinity, have been incorporated into this report. Based on our site investigation, analysis and the current conceptual site development, we consider this site geotechnically suitable for the proposed retirement community development provided the conclusions and recommendations (including structural setbacks) included herein are incorporated into the design and construction of the proposed development.

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If you have any questions regarding our report, please contact this office. We appreciate this opportunity to be of service.

Respectfully submitted,

LEIGHTON AND ASSOCIATES, INC.

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INTRODUCTION

1.1 Purpose and Scope

This report presents the results of our preliminary geotechnical investigation of The Orchards at Capistrano site, a proposed retirement community development in San Juan Capistrano (Figure 1). Site development plans have not yet been finalized and this report is being prepared to aid in their development. This report includes a summary or our reviews of previous site investigations by Leighton and by others, as well as the result of our recent studies. A major focus of this report was the characterization and evaluation of the site geology, concentrating on the existing onsite and offsite slopes and their influence upon proposed site grading and development.

Our scope of services included:

- Review of pertinent available geotechnical literature on the subject site and the bordering properties, including previous geotechnical reports, geologic maps, and aerial photographs (Appendix A). This information has been reevaluated relative to field mapping and investigation we performed at the site as part of this study.

- Subsurface exploration program consisting of the excavation, sampling, and downhole logging of 4 large-diameter exploratory borings (LB-1 through LB-4). Logs of the borings are presented in Appendix B.

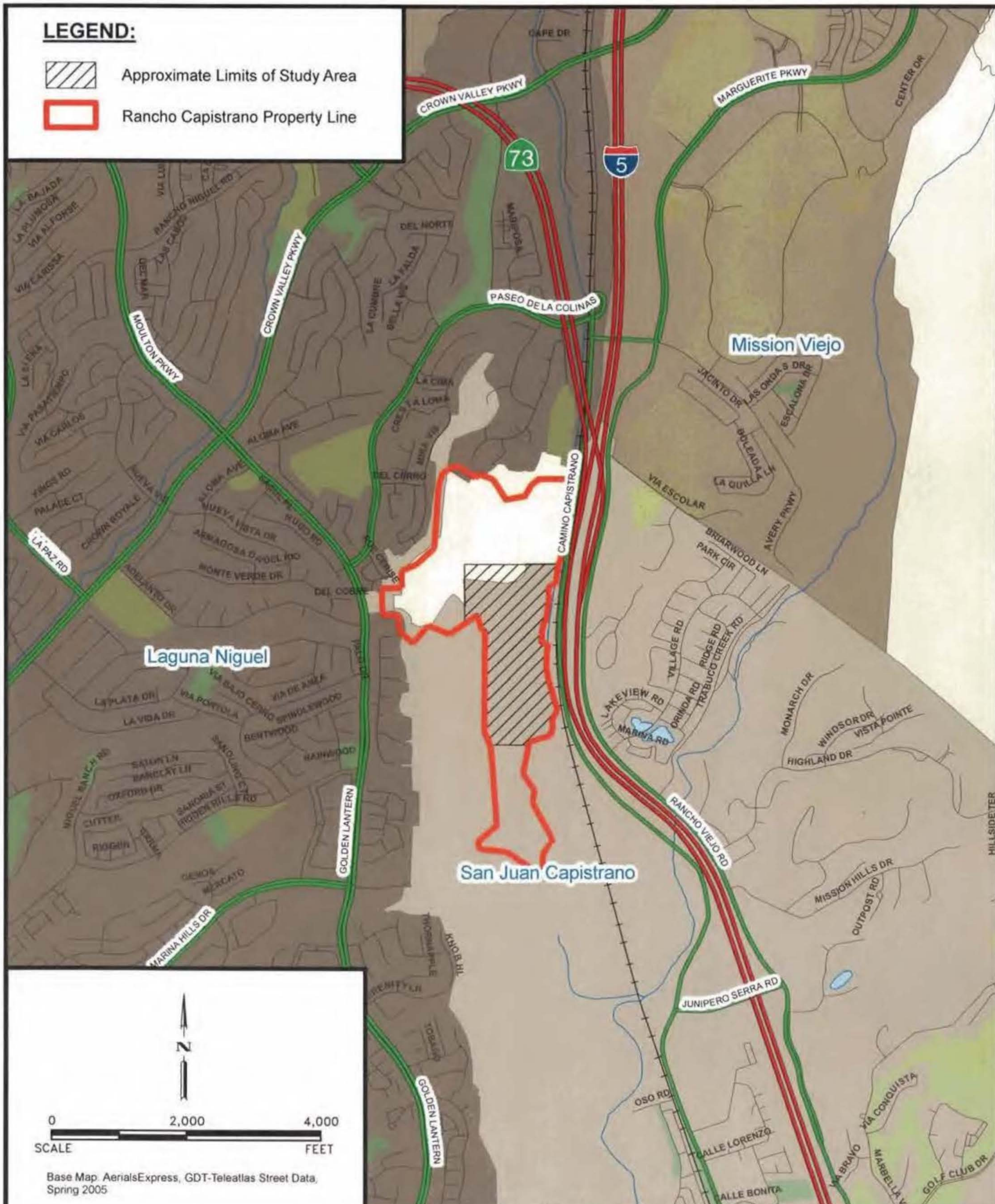
- Development of 100-scale Geotechnical Map (Plate 1). The map shows the site geology along with the current and previous exploratory locations.

- Preparation of Geotechnical Cross-Sections A-A', B-B', and C-C' (Plate 2). The location of the cross-sections are shown on Plate 1.

- Limited laboratory testing of samples obtained from our subsurface exploration. Laboratory test results are presented in Appendix C.

- Slope stability analysis was conducted on Geotechnical Cross-Sections in order to evaluate the existing factors of safety and to formulate preliminary design measures to be incorporated into the planning process. The results of our slope stability analyses are included as Appendix D.

- Review of the faulting and seismicity of the site. Analysis of the seismicity is included in Appendix E.



The Orchards
San Juan Capistrano, California

SITE LOCATION MAP

Project No.
012383-001

Date
April 2009

Figure 1

- Preparation of this report describing the site geotechnical conditions based on the results of our review and subsurface investigations. This report also provides preliminary recommendations for needed structural setbacks, remedial grading measures, and generalized grading concerns based on the current conditions. Foundation design considerations are also presented. General Earthwork and Grading Specifications are provided in Appendix F.

1.2 Site Description and Proposed Development

The Orchards will be developed within an irregularly shaped parcel known as Rancho Capistrano located west of Camino Capistrano in San Juan Capistrano, California (Figure 1). Topographically, the portion of the site proposed for development as The Orchards generally consists of a relatively flat parcels south of the existing Rancho Capistrano retreat and event facility, The area is bordered on the east by the LOSSAN rail corridor and Camino Capistrano. The Oso Creek drainage crosses the property along the eastern margin. The westerly property line is along the toe of the large (roughly 400 feet in elevation) undeveloped natural slope. This natural slope is bisected by several natural drainages. Previous site development has included the placement of a significant amount of fill soils within the northernmost drainage on the west side of the site. A portion of this area is currently used as an unpaved parking lot. In addition, two unoccupied single family homes and several buried utilities are present along the western margin of the site. The Rancho Capistrano property to the north of the site are occupied by the Rancho Capistrano facilities which include conference buildings, school building, retreat residence buildings, athletic facilities, and church, and a small lake.

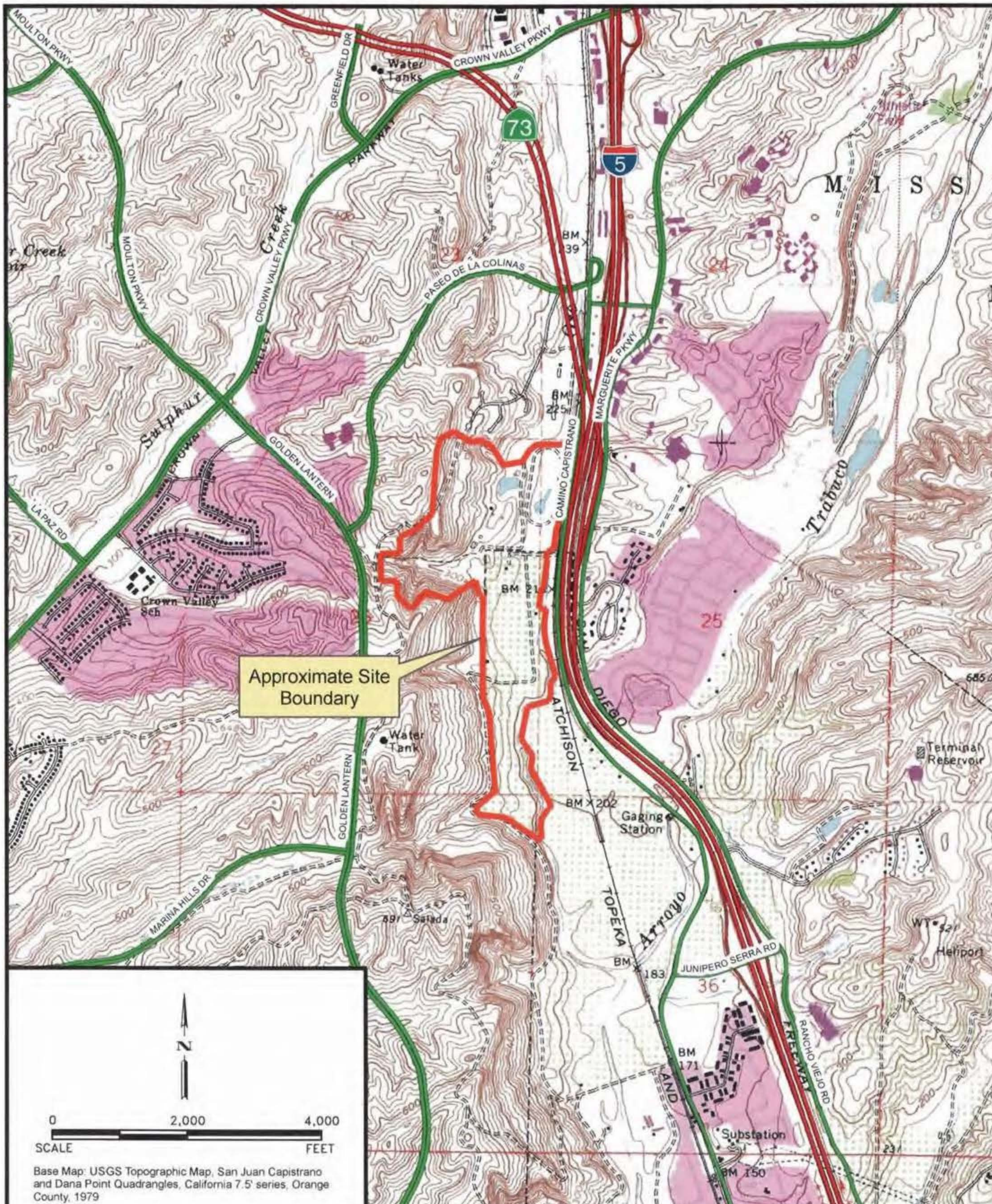
The area proposed for development is limited to the generally flat areas in the south central portions of the site. The approximate limits of the proposed development area are shown on Figures 1 and 3 and Plate 1.

As noted above, the Oso Creek channel is present along the eastern margin of the site. The banks of this drainage are locally oversteepened to near vertical gradients with heights up to roughly 50 feet. It is our understanding that channel improvements are not currently being proposed and that the proposed development will set back from the drainage. We understand evaluation of the site hydrology and scour potential is to be performed by others.

The approximate location of the site and proposed development area has been plotted on a United States Geological Survey (USGS) Topographic Map to illustrate the relationship of the site topography to the surrounding areas. This Regional Topographic Map is included as Figure 2. An aerial photograph of the site is provided as Figure 3.

FEMA Flood Plain mapping shows a 500-year flood plain that extends into the site along the creek on the north portion of the Rancho San Juan property. More recent flood control measure implemented by Orange County Flood Control Division within Oso Creek likely will result in some narrowing of the zone.

Proposed development plans are not yet finalized however the development will consist of a senior housing facility with independent living units, clusters of villas and a health center. Buildings are anticipated to be one to three stories in heights constructed with post-tensioned slabs and wood frame and stucco construction. One of the limitations of proposed site development is the assumed inability to grade off site along the hillside to the west or onsite within Oso Creek. The generalized area proposed for development is shown on the attached Plate No. 1. We note that based on the final design studies the limits of this area may vary somewhat.



Base Map: USGS Topographic Map, San Juan Capistrano and Dana Point Quadrangles, California 7.5' series, Orange County, 1979

The Orchards San Juan Capistrano, California	REGIONAL TOPOGRAPHIC MAP	Project No. 012383-001 Date April 2009 Figure 2

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The Orchards **San Juan Capistrano, California**	**SITE AERIAL PHOTO**	Project No. **012383-001** Date **April 2009** Figure 3

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2.0 SITE INVESTIGATION

2.1 Current Site Investigation

Our subsurface investigation consisted of the geologic mapping of the site and bedrock exposures on the adjacent slope to the west. In addition, our study included excavation, logging and sampling of 4 large-diameter borings (utilizing a bucket-auger drill rig). The borings were excavated to a maximum depth of approximately 141.5 feet. The large-diameter borings were surface logged and representative bulk and relatively undisturbed samples were obtained at frequent intervals. Where high ground water did not create unsafe conditions the large-diameter borings were entered by our geologist and downhole logged. Third party logging of Boring LA-2 was conducted by a representative of Lawson and Associates on behalf of the City of San Juan Capistrano. Several attempts to downhole log additional large-diameter borings in the northern western portion of the site were attempted but encountered unsafe conditions due to ground water seepage. Logs of the borings are presented in Appendix B. The approximate locations of the borings and cross-sections are shown on the Geotechnical Map (Plate 1) and on the Geologic Cross-Sections A-A' through C-C' (Plate 2). Subsequent to the subsurface investigation, the borings were backfilled.

2.2 Previous Geotechnical Investigations

As part of our study, we reviewed previous geotechnical reports that were prepared for this and adjacent properties by Leighton and by others. Published geologic maps of the site show a flat alluvial filled valley (the area where development is currently being proposed) bordered to the west by a steep (offsite) natural hillside that is mapped as Tertiary-aged Capistrano Formation bedrock overlain by large ancient landslides.

Previous studies performed to date have been accomplished to evaluate the existence and extent of the ancient landslides. Initial studies were conducted by Leighton to evaluate some of these features just to the south of the area currently proposed for development (Leighton, 1980). This report concluded that large landslides were present to the south. A preliminary Geotechnical Investigation for Rancho Capistrano was performed in 1982 (EJN, 1982) for the Rancho Capistrano development which is located on the northern edge of the area currently considered for development. That report concluded that although ancient landslides do exist on the hillside to the west, development of the lower and flatter portions of the site was feasible. In 1984 additional studies were performed upslope of the subject area (Medal, 1984). That report included another report by R.J. Shlemon (Shelmon, 1984) that also addressed the geomorphic features on the hillside. These reports state that while some landslide features are present on the lower portions of the slope the "offset high level terrain" was related to ancient stream-cut terraces and not large ancient landslides. More recent studies have in general concluded that these features be treated as landslides.

In 2001 and 2003 additional geotechnical studies were performed on the site. These included studies by NMG Geotechnical (NMG, 2001), and Lowney and Associates (Lowney, 2003). NMG included the drilling of large diameter borings on the hillside upslope of the property line. In their boring LB-2, which was drilled to a depth of over 200 feet, they encountered a rupture surface of what was concluded to be a large deep-seated landslide and concluded that deep-seated landslides were present on the upper portions of the slope but that near the property line the slides were more shallow. Lowney in their study agreed that the lower portions of the hillside are mantled by smaller landslide deposits and also performed a significant evaluation of the soils on the flatter portion of the site determining that the site was suitable for the support of proposed structures with minimal remedial grading.

An additional geotechnical studies was previously performed on the eastern portion of the site as part of the Oso Creek Channel Improvements (Ninyo and Moore, 2000).

2.3 Laboratory Testing

Limited laboratory testing was performed on representative samples obtained during the drilling program. Laboratory testing included soil classification by particular size analysis and plasticity. Laboratory test results are included as Appendix C.

3.0 SUMMARY OF GEOTECHNICAL CONDITIONS

3.1 Regional Geologic Setting

The site is situated in the coastal section of the Peninsular Range Province, a California geomorphic province with a long and active geologic history, including deep marine sedimentation followed by uplift, fluvial and marine erosion, and deposition. The Regional Geologic Map (Morton and Miller, 2006) is shown in Figure 4 to illustrate the location of the site in relation to the surrounding geology.

The area proposed for development primarily consists of a large relatively flat alluvial valley. The western edge of the site and the offsite property to the west is generally underlain by claystones, siltstones, and very fine sandstones of the Capistrano Formation (Morton, 1974). The Capistrano Formation was deposited in the Capistrano embayment, a Miocene-Pliocene age marine embayment which extended from the coast near San Clemente northward to the Santa Ana Mountains. Its initial development is marked by a conformable change in sedimentation from the well-laminated diatomaceous shale of the Monterey Formation to the less well-bedded micaceous siltstone and sandstone of the Capistrano Formation.

As a results of regional tectonic activity, a gentle (5 to 15 degree) west dipping regional bedding attitude is present in the area. Processes inferred to be coincident and subsequent to tectonic uplift include the jointing of the bedrock materials along moderately to steeply dipping planes, the precipitation of gypsum within such fractures and other voids, and the oxidation of the materials, generally within 20 to 40 feet of the ground surface as the elements expose the bedrock to repeated wetting and drying cycles. Quaternary streams have cut deep, steep-sided valleys into the emergent bedrock. Landsliding on steep natural slopes in the region resulting from undercutting and steepening due to stream erosion combined with localized weak clay beds within the bedrock has resulted in numerous ancient landslides in the area.

3.2 Site-Specific Geology

The majority of the site is underlain by moderate to deep depths of alluvial soils. Landsliding is present along the western margin of the site and offsite slope to the west. These landslides are failing along the bedding within the underlying Capistrano Formation. Quaternary-aged Terrace deposits have also been mapped along the eastern edge of the site near Camino Capistrano.

The bedrock unit underlying the entire site is considered to be the Capistrano Formation based on prior geologic mapping of the subject area (Edgington, 1974; Morton, 1974; Morton and Miller, 1981; and Morton and Miller, 2006). More recent geotechnical investigations of the site (Leighton 1980; EJN 1982; Medall Aragon, 1984; Ninyo and

Moore, 2000; NMG, 2001; Cotton Shires, 2003; and Lowney, 2003) have generally concurred with this interpretation.

A brief description of the onsite units (youngest to oldest) encountered during this and prior investigations is presented below. The approximate areal distribution of these mapped units is depicted on the Geotechnical Map (Plate 1).

3.2.1 Artificial Fill (Af)

Localized deposits of undocumented fill soils are present on the site. These undocumented fills consist of minor fills associated with the past agricultural activity, end dumped fills placed into the Oso Creek Channel, and fills associated with the infilling of the canyon area on the northern end of the site that is currently utilized as an unpaved parking area. Based on our observations the fill generally consists of varying amounts of fine sand, silt, and clay derived from native onsite materials. The approximate limits of the most significant fill areas are outlined on the Geologic Maps. These fills are considered undocumented and will require removal and recompaction in areas of proposed development.

3.2.2 Topsoil/Colluvium (Qcol)

The topsoil encountered during our field investigation mantled the slope areas along the western edge of the site. The topsoil, as observed, consisted predominantly of a light gray to brown, damp to moist, stiff to hard, sandy to silty clay. These soils are generally massive, porous, and contained scattered roots and organics. The potentially compressible topsoil is estimated to be from 2 to 3 feet in thickness across the western edge of the site; however, localized areas of thicker accumulations of topsoil may be encountered during grading. Topsoil/colluvial soils on the sides of the canyon areas, particularly in the margins of the canyon bottoms, can be expected to be significantly deeper in extent and locally variable. Topsoil and colluvium are considered potentially compressible and not suitable for the support of additional fill or structural loads.

3.2.3 Alluvium (Qal)

Alluvial deposits are present throughout a majority of the area proposed for development. These soils are moderately to poorly consolidated and potentially compressible near the existing ground surface. These soils predominantly consist of light to medium gray and dark brown silts and sands with occasional pebble and cobble lenses. Based on previous site investigations, alluvial soils in the areas proposed for development range in depth of up to nearly 100 feet. Previous site investigations (NMG, 2001 and Lowney, 2003) which included numerous small diameter borings and CPT's have characterized the upper portion of this material as potentially compressible (the upper 5-10 feet) while the deeper alluvial soils were moderately well consolidated.

At this time we estimate the alluvial soil within the area of planned grading will require removal and recompaction on the order of 5 to 10 feet in depth to mitigate shallow compressibility. Please note that localized deeper areas may be encountered during grading due to the possible existence of subsurface irregularities.

3.2.4 Terrace Deposits (Qt)

Quaternary-aged terrace deposits underlie adjacent areas along the eastern boundary. These soils were previously encountered in borings by others (Ninyo and Moore, 2000) to depths up to 60 feet. These deposits were generally described as silty sands and gravels with cobble beds. This unit is outside the area proposed for development and is not anticipated to be encountered.

3.2.5 Landslide Deposits (Qls)

Several ancient landslides have been previously identified within and adjacent to the subject property. Based our review of the previous geotechnical studies, logging of exposures upslope of the property, and downhole logging of boring LAB-2; it is our opinion that the adjacent offsite slope is underlain by large ancient landslides, These slides toe out near the property line as shown on geologic cross-sections A-A', B-B', and C-C' (Plate 2). The topographic break near the upper portion of the offsite slope is likely in our opinion likely related to ancient landsliding.

Based on our mapping, review of logs by others, and logging of large-diameter exploratory boring LAB-2 (which was also logged by the City's geotechnical consultant), the landslides were generally encountered to depths of less than 100 feet on the site and are failing along bedding.

The landslide deposits include relatively undisturbed blocks of formational material and weathered formational material. The material is generally moderately fractured and jointed at depth and highly weathered near the surface and at the toe of the landslide complexes. The landslide basal rupture surfaces, typically consist of dark gray to gray, striated, plastic, slightly silty, paper thin to 1/4-inch thick remolded clay seams. In general, the landslide basal rupture surface appears to correspond to a weak clay seam. The landslide material above the basal rupture surface generally resembles the underlying Capistrano Formation, but is somewhat more fractured and jointed than the bedrock materials. Based on both previous borings by others and our recent deep boring (LAB-2), the basal rupture surface of the landslides is consistent with the regional slightly westerly dip observed in the area. Stabilization or avoidance of these large predominately offsite ancient landslides will be required as part of the development process.

On the eastern side of the site, numerous surficial soil failures were observed within the oversteepened banks of the Oso Creek Channel. Failures of this type likely occur during or shortly after heavy storm activity when peak flows have occurred and some additional sloughing and slumping of the channel slopes should be anticipated. As previously stated, the current development concepts call for avoidance of this area by use of setbacks. Plate No. 1 illustrates the conceptual creek setback. The final location of this line will be determined based on studies by the site civil engineer and surface hydrologist during planning. Additional geotechnical studies will likely be required to finalize the creek setback location.

3.2.6 Capistrano Formation (Tc)

The Capistrano Formation, as encountered in current and past subsurface explorations generally consists of an oxidized (weathered) siltstone underlain by a relatively unoxidized (unweathered) siltstone. The oxidized portion of the siltstone generally extends below the surface on the order of 5 to 40 feet and consists of medium olive-gray to gray-brown, sandy to clayey siltstone. The oxidized portion is generally thicker on the offsite ridgetops and flatter site areas, and thinner on the steeper hillsides. The oxidized siltstone is locally characterized by abundant gypsum and caliche-filled fractures and joints. The unoxidized siltstone consists of dark olive-gray, stiff to very stiff, sandy to clayey, micaceous siltstone. The unoxidized siltstone is generally massive to poorly bedded with a petroliferous odor as is typical of the unoxidized Capistrano Formation.

3.3 Geologic Structure

While the area proposed for development is underlain by relatively massive deposits of alluvial soils, the adjacent slope is underlain by massive to poorly bedded siltstone which has been gently uplifted and locally tilted so that bedding dips are generally a few degrees in the westerly direction. Clay seams and/or landslide rupture surfaces encountered in our subsurface exploration generally trend roughly parallel with the bedding planes. For purposes of analysis, we have generally considered clay seams and rupture surfaces found on site to be planar and continuous for conservatism. Our experience indicates that such features can be laterally discontinuous, or irregular, as a result of faulting, folding or other geologic disruptions.

Jointing is well developed in the oxidized siltstone of the Capistrano Formation. Joints tend to strike subparallel to the overlying slope face and dips at both low to moderate and high angles in the direction of the slope. Joints are often gypsum filled and clay-lined within the oxidized portion. Numerous well developed joints were observed upslope of the site within large open exposures.

3.4 Ground Water and Surface Water

In general only minor ground water seepage was encountered in Boring LAB-2. Other borings drilled along the westerly property line encountered localized seeps and perched zones of seepage within otherwise non-saturated zones that prohibited downhole logging. The areas of seepage appear to be generally controlled by geologic contacts between units and joints or fractures in the bedrock or landslide material creating perched water conditions within locally permeable sand lenses. Within the areas proposed for development ground water has been encountered in numerous borings within the alluvial soils and is expected to be encountered at depths roughly 10 to 30 feet below existing site grades.

Based on our review of the boring log data, mapping of the subsurface geology, we have modeled the existing ground water conditions and illustrated these levels on the Geotechnical Cross-Sections. In general, we have illustrated ground water to a reasonable maximum elevation based on a combination of natural drainage levels and degrees of seepage encountered in geotechnical borings. The Capistrano Formation is not considered to possess a static ground water table.

Surface water was observed in the Oso Creek drainage courses and can be seasonally associated with re-entrant tributaries. Surface water appears to drain as runoff in the higher portions of the site during rainy periods and concentrates in the onsite drainage courses.

According to Federal Emergency Management Agency (FEMA), flood insurance rate mapping (Figure 7), northerly portions of the site along the creek are within the 500-year flood plain. More recent flood control measures by the County of Orange within Oso Creek will result in some narrowing of that mapped zone.

4.0 FAULTING AND SEISMICITY

4.1 Faulting

Our discussion of faults on the site is prefaced with a discussion of California legislation and policies concerning the classification and land-use criteria associated with faults. By definition of the California Mining and Geology Board, an active fault is a fault which has had surface displacement within Holocene time (about the last 11,000 years). The state geologist has defined a potentially active fault as any fault considered to have been active during Quaternary time (last 1,600,000 years). This definition is used in delineating Earthquake Fault Zones as mandated by the Alquist-Priolo Geologic Hazards Zones Act of 1972 and most recently an interim revision in 2007 (Bryant and Hart, 2007). The intent of this act is to assure that unwise urban development and certain habitable structures do not occur across the traces of active faults. The subject site is not located within any State mapped Earthquake Fault Zones as created by the Alquist-Priolo Act.

Our review of available geologic literature (Appendix A) indicates that there are no known major or active faults on or in the immediate vicinity of the site. The nearest active regional fault is the offshore segment of the San Joaquin Hills Thrust located approximately 3.9 miles (6.2 kilometers) north of the site. Table 1 summarizes the distance to the regional faults identified by the California Geological Survey (CGS, 2003).

Table 1 Seismic Parameter for Known Active Faults (CGS, 2003)					
Fault	Geometry	Closest Distance from Fault to Site		Maximum Moment Magnitude	Average Slip Rate (mm/yr)
		Miles	Kilometers		
San Joaquin Hills	Reverse 235W	3.0	4.8	6.6	0.5
Newport Inglewood (offshore)	Right Lateral Strike Slip	6.3	10.2	7.1	1.5
Newport-Inglewood	Right Lateral, Strike Slip	15.1	24.3	7.1	1.0
Chino-Central Ave.	Right Lateral, Strike Slip	18.1	29.2	6.7	1.0
Elsinore (Glen Ivy)	Right Lateral Strike Slip	18.3	29.5	6.8	5.0
Elsinore (Temecula)	Right Lateral Strike Slip	20.5	33.0	6.8	5.0
Palos Verdes	Right Lateral, Strike Slip	22.0	35.4	7.3	3.0
Whittier	Right Lateral Reverse Oblique 75NE	22.4	36.1	6.8	2.5
Coronado Bank	Right Lateral, Strike Slip	23.2	37.3	7.6	3.0
Puente Hills Blind Thrust	Reverse 25N	29.4	47.3	7.1	0.7
Rose Canyon	Right Lateral Strike Slip	31.8	51.2	7.2	1.5
San Jose	Left Lateral Reverse Oblique 75NW	36.9	59.4	6.4	0.5
Elsinore (Julian)	Right Lateral Strike Slip	39.8	64.0	7.1	5.0
Sierra Madre	Reverse 45N	41.0	66.0	7.2	2.0
Cucamonga	Reverse 45N	41.1	66.2	6.9	5.0
San Jacinto - San Jacinto Valley	Right Lateral Strike Slip	41.7	67.1	6.9	12.0

The location of the site with respect to the regional faults are shown on the Regional Fault Map (Figure 5).

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4.2 Seismic Design Parameters

The following geotechnical design parameters have been determined in accordance with the 2007 California Building Code utilizing the USGS Ground Motion Parameter Calculator Version 5.09.

Table 2 USGS Seismic Design Parameters	
Site Class	D
Site Coefficients	F_a = 1.0 F_v = 1.503
Mapped Spectral Accelerations	S_S = 1.403g S_1 = 0.497g
Site Modified Spectral Accelerations	S_{MS} = 1.403g S_{M1} = 0.748g
Design Spectral Accelerations	S_{DS} = 0.935g S_{D1} = 0.498g

For considerations in seismic design, a site-specific hazard analysis was performed utilizing the Software EZ-FRISK Version 7.32 (Risk, 2008). Based on that analysis, Table 3 presents site-specific design parameters. For our analysis, we utilized the State of California Fault Model (CGS, 2003) and the attenuation relationships by Abraham and Situa - Deep Soil (1997), Boore-Joyner-Fumal (1997), and Sadigh-Soil (1997). A shear wave velocity of 200m/s was considered for the attenuation relationship Boore-Joyner-Fumal. Directivity effects were considered using the methodology on Somerville et. al. (1997) and Abrahamson (2000). Summary plots of the analysis are provided in Appendix E.

Table 3 Site-Specific Seismic Design Parameters	
Design Spectral Accelerations	S_{DS} = 0.955g S_{D1} = 0.601g

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4.3 Secondary Seismic Hazards

Secondary effects that can be associates with severe ground shaking following a relatively large earthquake include shallow ground rupture, soil liquefaction and dynamic settlement, lateral spreading seiches and tsunamis. These secondary effects of seismic shaking are discussed in the following sections.

4.3.1 Design Ground Motion

For consideration in liquefaction analysis, and based on deaggregation of the Maximum Considered Earthquake event, a magnitude M6.6 is associated with the Design Earthquake Ground Motion of 0.38g.

4.3.2 Shallow Ground Rupture

No active faults are mapped crossing the site, and the site is not located within a mapped Alquist-Priolo Earthquake Fault Zone (Bryant and Hart, 2007). Shallow ground rupture due to shaking from distant seismic events is not considered a significant hazard, although it is a possibility at any site.

4.3.3 Liquefaction, Seismically-Induced Settlement, Lateral Spreading

Liquefaction and seismically-induced settlement of soils can be caused by strong vibratory motion due to earthquakes. Both research and historical data indicate that loose, saturated, granular soils are susceptible to liquefaction and liquefaction-induced settlement while the stability of clay-rich soils, silty clay and clay are generally not adversely affected by vibratory motion (Seed and Idriss, 1982; Tokimatsu and Seed, 1987; Youd and Gilstrap, 1999). Liquefaction is typified by a reduction in shear strength in the affected soil layer, thereby causing the soil to temporarily behave as a viscous liquid. This effect may be manifested by excessive settlements and sand boils at the ground surface.

Based on the State of California Seismic Hazard Zone Maps (CGS, 2001), the alluvial materials within the property limits may be susceptible to liquefaction (Figure 6). The bedrock areas of the site are not considered susceptible to liquefaction because of their high density characteristics, age, and lack of a shallow ground water table (Youd et. al., 2001). In the broad valley bottom which is underlain by deep alluvial soils previous site specific studies have indicated that because of the density and relatively fine grained nature of the alluvium that the potential for liquefaction is low. Where loose alluvial soils are present within the upper 5 to 10 feet below existing site grades, we anticipate that complete removal and re-compaction of the alluvial soils and compressible landslide material will be performed. Further studies are recommended to verify these findings.

Previous reports have stated that lateral spreading has not been completely evaluated adjacent to the Oso Creek Channel. Due to the discontinuous nature of the lenses that comprise the alluvium, we do not suspect lateral spreading to be a hazard. While a setback of this area is planned additional investigation of this phenomena should also be performed as site plans are being finalized. In the event that lateral spreading is a concern, stabilization measures may be necessary.

4.3.4 Seismically-Induced Landsliding

The property is located at the base of a large offsite slope contained within landslide prone areas shown on State of California Seismic Hazard Maps (CGS, 2001). Based on this quadrangle map, the site borders an area where previous occurrence of landslides movement or geotechnical conditions indicate a potential for permanent ground displacements as a result of seismically-induced slope movement.

For the proposed local municipal and private improvements, evaluation in accordance with the County of Orange grading ordinance has been utilized. Accordingly, in accordance with County of Orange Grading Manual, pseudo-static analysis was not performed within slide or formational masses due to the presence of bedding angles less than 12 degrees (County of Orange, 2002).

4.3.5 Tsunamis and Seiches

Based on the distance between the site and large, open bodies of water, and the elevation of the site with respect to sea level, the possibility of seiches and/or tsunamis is considered to be low.

5.0 GEOTECHNICAL DESIGN CONSIDERATIONS

The following design considerations are based on our current knowledge of the site conditions. As previously noted, site development plans are not yet available and this report is to aid in their development. Based on our review of the site conditions and our geotechnical observations to date, the most significant geotechnical constraints of the site are potentially compressible soils, the primarily offsite landslides and related slope stability concerns, deep alluvial soils, expansive soils, possibly corrosive soils and the Oso Creek Channel stability. A detailed discussion of these conditions is provided below.

5.1 Potentially Compressible Soils

Topsoil, colluvium, the weathered upper portion of alluvial soils, weathered formational material, and portions of the ancient landslide deposits are considered potentially compressible in their present state. Topsoil and colluvium are anticipated to vary in thickness from 2 to 10 feet. Colluvial accumulations are generally thicker near the flatter lower portions of the slopes and in the tributary drainages. Alluvial soils in the broad on site valley is up to 100 feet in depth and will require removal and recompacton of the upper 5 to 10 feet in areas proposed for development.

Numerous ancient landslides have been mapped in the areas of proposed development. These landslides have been identified and are shown on the Geotechnical Map and Cross-Sections. Other, less distinct and surficial slumps or landslides may also exist on site. Near-surface, weathered and disturbed slide debris are also considered potentially compressible and may not support proposed fills without significant differential settlement due to future moisture infiltration (generally on the order of 5 to 15 feet). In addition, cuts through landslide areas may reactivate the existing landslides unless sufficient factor of safety exists.

5.2 Offsite Landslides

The adjacent offsite property contains many large ancient landslides that extend only slightly onto the site. Because of the inability to grade or construct improvements off the site, it is not possible to stabilize these landslides. As a result the best mitigation is avoidance. The site development should be planned to create the proposed development with minimal grading of the onsite portions of these slides.

The landslides that border the tract were evaluated from a gross stability standpoint and stabilization measures consisting of ground anchors, stability keys and buttresses were evaluated as potential fixes but because of the site limitations are not feasible. As a result our stability analysis included the determination of the minimum factor of safety line from which a structural setback was developed as is shown on the geotechnical map. The area to the west of the line is not suitable for structures; however, this area is considered suitable for driveways and underground utilities.

For analysis purposes, a passive wedge comprises the portion of the analyzed failure surface at the toe where the surface departs a relatively long weak bed. For horizontal ground surface conditions, the critical inclination of the passive wedge from horizontal can be approximated as $45 - \phi/2$. This equates to a critical passive wedge inclination of 33° from horizontal for fill materials with a $\phi = 24°$. Because of the slope of the ground surface at the toe, this inclination can vary slightly from section to section depending on the location of the passive wedge along each section. For Wedge type surfaces, the algorithms incorporated within the block specified search routine of SlopeW software was utilized to identify the critical passive wedge inclination for the surface being considered. Because the critical passive wedge inclination represents the limiting shear strength condition, consideration of flatter inclinations is not appropriate. As discussed by Duncan (2005) and others, this approach of considering the critical passive wedge as an independent analysis has sound engineering basis.

Proposed development areas east of the structural westerly setback line are considered to have a factor of safety of at least 1.5.

5.3 Slope Stability

Because site grading plans have not yet been developed, the heights of proposed cut and fill slopes are unknown. However, because primarily only the flat area of the site is proposed for development, cut and fill slopes are only anticipated to heights of less than 5 and 20 feet. Our analyses indicate that 2:1 (horizontal:vertical) cut and fill slopes constructed up to theses heights will possess a surficial and deep seated stability factor of safety of at least 1.5 (static), and as such.

Natural slopes of the Oso Creek Channel and within undeveloped portions of the site may be prone to localized instability. Although our review and analyses indicate such areas are outside the influence of the proposed residential structures, other types of improvements constructed in areas where grading abuts natural slopes should be considered susceptible to natural slope instability.

5.4 Expansive Soils

The sandy soils of the Capistrano Formation typically have a low to medium expansion potential (Expansion Index between 20 and 90). The alluvial soils and topsoil (which comprises the majority of the onsite soils) generally have a medium to high expansion potential (Expansion Index between 50 and 130). The siltstone of the Capistrano

Formation (which comprises the majority of the onsite soils) is considered to have a medium to very high expansion potential (Expansion Index Above 50). Expansive soils within the Capistrano Formation typically contain illite as the predominant clay mineral with less abundant quantities of montmorillonite (Kile and McMillin, undated). Special foundation design for expansive soils can mitigate the effects of the expansive soils.

5.5 Soil Corrosivity

Following are corrosivity test results from within and adjacent to the site (Lowney, 2003 and Ninyo and Moore, 2000).

Based on our professional experience on nearby sites and California Building Code criteria, soluble sulfate content of the some of the onsite soils, is expected to present a severe potential for corrosion of concrete and certain other materials in contact with that soil. Additionally, due to their fine-grained nature and associated low electrical resistivity levels, the onsite soils will likely pose a corrosive environment to buried metallic improvements. ACI 318-08 provides additional guidance on concrete mix design requirements to mitigate corrosion. Measures to mitigate corrosion can be provided by a qualified corrosion engineer.

Table 4
Corrosivity Test Results

Sample Location	Sample Depth (feet)	pH	Minimum Resistivity (ohm-cm)	Soluble Sulfate Content (ppm)	Chloride Content (ppm)
LF-1	0-5	7.2	570	1,739	248
LF-10	0-5	7.0	1,200	228	ND
B-2	39-42	7.0	365	1,860	110
B-3	23-26	8.0	630	130	95

6.0 CONCLUSIONS

Based on the results of our geotechnical evaluation of the site, it is our opinion that the proposed development is feasible from a geotechnical standpoint provided the following conclusions and recommendations are incorporated into the project plans and specifications. The following is a summary of the geotechnical factors that may affect development of the site.

- Numerous ancient landslides have been mapped primarily offsite but extending onto the subject site. These landslides which include surficial soils and bedrock failures have been identified and the approximate extents of these features are shown on the Geotechnical Map (Plates 1). Because of the primarily offsite nature of these slides, stabilization is not practical and avoidance is recommended. Plate No. 1 presents a structural setback line for this area.

- The location of the structural setback line is largely influenced by site grades. If grades are proposed to be significantly altered the line location may shift. Grades along the toe of the landslides west of the westerly setback should not be lowered.

- We anticipate the proposed development will be located within areas that are underlain by alluvial soils and undocumented fill soils some of which are potentially compressible. We anticipate that the upper 5-10 feet of alluvial soils and all undocumented fill soils will require removal and recompaction during site development.

- Additional evaluation of the liquefaction potential of the onsite alluvial soils should be performed as part of the continued planning process as should the potential for lateral spreading of soils adjacent to the Oso Creek Channel during a seismic event.

- Topsoil and colluvial soils present on the steep (> 2H:IV) natural slopes are considered to be susceptible to mudflow and surficial slumping during periods of heavy rainfall. Based on our review of the site topography and cross-section analyses, natural slopes within influence of the site are commonly 2:1 (H:V) or flatter, and therefore less susceptible to surficial instability. Natural slope stability considerations are further discussed in Section 6.3.3.

- It is anticipated that any planned cut slopes will require remedial stabilization measures to mitigate potential instability. A detailed discussion on slope stability is provided in Section 6.3.

- We anticipate the onsite soils on the site should be generally rippable with conventional heavy-duty earthwork equipment. However, localized areas of concretions and cemented layers within the Capistrano Formation may require heavy ripping during excavation and additional processing and/or special handling.

- Localized zones of perched seepage and/or ground water may be encountered in removal areas or buttress excavations. Subdrains are recommended beneath all buttress areas. Recommendations for subdrains can be provided after grading plans are finalized.

- Based on laboratory test results, professional experience in adjacent areas, and visual classification, the soils may possess a moderate to very high expansion potential. The presence of highly expansive soils at finish grade elevations combined with areas of differential fill thickness will require the use of special foundation and slab construction techniques (i.e., post-tensioned slabs and footings, as well as pre-moistening of the slab subgrade soils).

- Active and potentially active faults are not known to exist on or in the immediate vicinity of the site and are not anticipated to be encountered during grading.

- Based on deaggregation, a magnitude M6.6 and a Design Earthquake Ground Motion of 0.38g are considered representative for the site.

- Based on laboratory testing and our professional experience in adjacent areas, the prevailing soil on site may be highly corrosive to concrete and buried metals. The corrosivity of the onsite soil should be tested during grading and a corrosion engineer should be retained to design mitigative measures for materials that may be affected by corrosive site conditions.

7.0 RECOMMENDATIONS

We recommend that as development plans become available that they be reviewed by Leighton. Additional geotechnical studies will be required prior to site grading. For planning purposes the follow recommendations are provided.

7.1 Earthwork

We anticipate that earthwork at the site will consist of site preparation, excavation, and placement of compacted fill. We recommend that earthwork on the site be performed in accordance with the following recommendations and the General Earthwork and Grading Specifications for Rough Grading included in Appendix F. In case of conflict, the following recommendations shall supersede those in Appendix F.

7.1.1 Site Preparation

Prior to grading, all areas to receive structural fill or engineered structures should be cleared of surface and subsurface obstructions, including any existing debris and undocumented fill soils, and stripped of vegetation. Removed vegetation and debris should be properly disposed of off-site. Holes resulting from removal of buried obstructions which extend below finish site grades should be replaced with suitable compacted fill material. All areas to receive fill and/or other surface improvements should be scarified to a minimum depth of 8 inches, brought to optimum or above optimum moisture conditions, and recompacted to at least 90 percent relative compaction (based on ASTM Test Method D1557).

7.1.2 Excavations and Oversize Material

Excavations of the onsite materials may generally be accomplished with conventional heavy-duty grading equipment.

All excavation practices should be conducted in accordance with OSHA requirements. We anticipate that scattered amounts of oversize material may be generated during site grading where existing rubble containing fills are present. Our standard recommendations for treatment of oversize material are included in the attached General Earthwork and Grading Specifications for Rough Grading (Appendix F).

7.1.3 Fill Placement and Compaction

The onsite soils are generally suitable for use as compacted fill provided they are relatively free of organic material, debris, and rock fragments larger than 8 inches in maximum dimension. All fill soils should be brought to at least 2 percent above the optimum moisture content and compacted in uniform lifts to at least 90 percent relative compaction based on laboratory standard ASTM Test Method D1557. The optimal lift thickness required to produce a uniformly compacted fill will depend on the type and size of compaction equipment used. In general, fill should be placed in lifts not exceeding 8 inches in thickness.

Please note that the soil of Capistrano Formation (or soils derived from this unit i.e. alluvium) typically possesses a moisture content well above or below optimum and may require moisture conditioning and/or blending prior to use as compacted fill. Fills placed on slopes steeper than 5:1 (horizontal to vertical) should be keyed and benched into competent formational soils are indicated in the General Earthwork and Grading Specifications for Rough Grading in Appendix E. Placement and compaction of fill should be performed in general accordance with the current City of San Juan Capistrano grading ordinances, sound construction practice, and the General Earthwork and Grading Specifications for Rough Grading presented in Appendix F.

7.1.4 Transition Lots and Overexcavation

We recommend the proposed structures be planned such that they are entirely underlain by a relatively uniform thickness of properly compacted fill. Overexcavation should extend laterally at least 5 feet beyond the building footprint.

7.1.5 Expansive Soils and Selective Grading

Our experience with similar materials on adjacent sites indicate that the onsite soils possess a moderate to very high expansion potential. The presence of these highly expansive materials within 5 feet from finish grade will require special foundation and slab considerations.

As an alternative to the special foundation recommendations, you may elect to overexcavate building pads underlain by expansive soils a minimum of 5 feet below finish pad grade and replace with properly compacted fill possessing a lower expansion potential. Should this alternative be chosen, the overexcavation should extend a minimum of 5 feet beyond the perimeter of the improvements (including flatwork), and be graded such that water does not accumulate beneath the structures. Since the prevailing onsite soils are generally expansive, low expansive soils for pad capping will likely require selective grading or import operations.

7.1.6 Corrosive Soils

Corrosive soils are common throughout this general area and additional testing of the finished grade soils is recommended during site grading. A corrosion engineer should be retained for design of measures to mitigate corrosion.

7.1.7 Canyon Subdrains

Although grading of canyon areas is not anticipated at this time, ground water may accumulate in the offsite drainages that trend downhill toward the site. In order to help reduce the potential for ground water accumulation in the proposed fill areas, we recommend subdrains be installed in the base of removals along the western limits of grading during site removals. In general, subdrains should be placed at the base of removals and consist of six inch diameter perforated pipe surrounded by a minimum of 9-cubic feet per linear foot of 3/4-inch gravel wrapped in fabric with a minimum fall of at least 2 percent. These drains should extend to a suitable collective drainage system.

The actual need and/or location of subdrainage should be based on the evaluation of the configuration of the canyon bottoms by the geotechnical consultant after the removals of compressible soils have been completed.

The installed subdrains should be surveyed for alignment and grade by a representative of the project civil engineer. Sufficient time should be allowed for the surveys prior to commencement of filling over the subdrain. The subdrain outlets should be installed to discharge water into positive drainage devices.

7.2 Removal of Potentially Compressible Soils

As discussed in Section 3.2, portions of the site are underlain by potentially compressible soils, which may settle under the surcharge of fill and/or foundation loads. These materials include alluvium, colluvium, topsoils, and landslide debris, not already removed by grading activities to date. Compressible materials not removed by the planned grading should be excavated, moisture conditioned, and then recompacted prior to additional fill placement or construction. The actual depth and extent of the required removals should be determined during grading operations by the geotechnical consultant. However, estimated removal depths are summarized below.

7.2.1 Topsoil

Areas to receive fill which are on slopes flatter than 5:1 (horizontal to vertical), where normal benching would not completely remove the topsoils, or where design cuts do not remove the topsoil should be stripped to firm bedrock removing all significant topsoil prior to fill placement. Topsoil is expected to be generally 2 to 4 feet thick, although localized deeper areas may be encountered during grading.

7.2.2 Colluvium/Alluvium

In areas to receive fill, or where design cuts expose alluvium and colluvium, the alluvial and colluvial soils on the site should be removed to suitable bedrock material. Removal of alluvium and colluvium near the canyon bottoms will generally require overexcavation depths on the order of 5 to 15 feet; however, localized areas may require deeper removals.

7.2.3 Landslides

In general, the shallow portions of the slide areas along the western boundary are considered potentially compressible and should be stripped to a depth of approximately 5 to 20 feet below existing ground level to remove highly disturbed and weathered material exposing competent fractured bedrock-like material. Where stripping is performed, removed material should remain over the slide area to avoid destabilizing uphill areas. Localized deeper removals should be anticipated. The actual depth of stripping or overexcavation should be determined during grading based on field observations.

Competency of the landslide material to be left in-place is best evaluated during grading based on visual observation with testing to determine the density and moisture of the material. Based on our subsurface explorations, it is expected that deep-seated landslide materials will be left in place where observed to be compact and competent based on visual observation. To be left-in-place, highly weathered landslide material should have a minimum 85 percent relative compaction and a minimum 85 percent degree of saturation. In addition, normal benching should be performed during fill placement. In areas where shallow landslide material extends below proposed cuts, the disturbed and/or weathered landslide material should be removed and recompacted.

7.2.4 Existing Undocumented Fill

Where encountered within the limits of planned grading, the existing undocumented fills should be completely removed prior to placement of additional fill. These materials can be utilized as fill materials provided they are moisture conditioned and free of deleterious materials. All trash and deleterious material should be removed and disposed off-site.

7.3 Slope Stability

The site and adjacent offsite slopes were analyzed for gross stability utilizing a computer program that utilized limit equilibrium methodology (SLOPE/W). The results of the stability analyses are provided in Appendix D.

The strength parameters utilized in our analyses are based upon the results of our testing and testing by other consultants of the geotechnical properties of the Capistrano Formation and its associated landslides in this portion of Orange County. For ease of understanding, the strength parameters, we have utilized in our analyses are shown on the accompanying diagram showing Assigned Shear Strength Parameters (Figure 8).

Our slope stability analysis is modeled on what we have observed to be the primary failure mechanisms for slopes within this portion of the Capistrano Formation. Typically, major landslides in this formation fail as a block-type failure. This block failure is nearly always controlled by the presence of a weak clay bed along the basal planar rupture surface. These clays are recognizable as distinct beds within the otherwise typically massive silt and clays of the bedrock. We have assigned strength parameters of $c=0$ and $\phi=8°$ for these clay surfaces where they form the basal rupture surface of a landslide. This value represents in part the results of residual strength testing of the clays by Leighton and others.

For conservatism, our analysis modeled the oxidized and unoxidized Capistrano Formation with anisotropic strength functions considering clay seam strengths within a reasonable range of bedding attitudes based on our review, investigations, and professional experience. These anisotropic models used are presented in Appendix D. The anisotropic models consider the clay seams to be planar and continuous. Although the bedding structure, including clay seams, are assumed to be planar and continuous, our experience indicates that such features can be laterally discontinuous, or irregular, as a result of faulting, folding, depositional discontinuities, or other geologic conditions.

Because the offsite landslides cannot be effectively stabilized we have developed a minimum factor of safety line by utilization of a "passive wedge" theory as previously discussed. We have then conservatively set back a distance of 100 feet from this line to create the structural setback line shown on Plate No. 1.

7.3.1 Fill Slopes

The materials anticipated for use in fill slope grading will predominantly consist of clayey silt derived from the on site alluvium or Capistrano Formation. In general, our analysis, assuming homogeneous slope conditions, indicates the proposed fill slopes have a calculated factor of safety of 1.5 or greater with respect to potential, deep rotational failure.

The proposed slopes should be constructed in accordance with the recommendations of this report, the attached General Earthwork and Grading Specifications for Rough Grading (Appendix F) and City of San Juan Capistrano grading ordinances.

7.3.2 Cut Slopes

In order to reduce the potential for cut slope instability if cut slopes are proposed, we recommend the construction of buttresses or replacement stability fills to create manufactured fill slopes where interior slope faces are proposed. A typical detail for stability fill construction is provided in the attached General Earthwork and Grading Specifications for Rough Grading (Appendix F).

We recommend that the geotechnical consultant document and geologically map all excavation during grading, including cut slopes. The purpose of this mapping is to substantiate the geologic conditions considered in our analysis. Additional investigation and stability analysis should be anticipated if adverse conditions are encountered.

7.3.3 Natural Slopes

The natural slopes upslope of the developed areas may be subject to surficial failures and possible mudflows after periods of heavy rainfall. In areas where the natural slopes are steeper than 2 to 1 (horizontal to vertical) or where runoff is concentrated into a natural drainage, the risk is the highest. Bordering improvements such as concrete v-ditches, fences, scenic trails etc., should be considered susceptible offsite natural slope instability.

7.3.4 Surficial Slope Stability

The materials anticipated for use in fill slope grading will generally consist of onsite, silty clay and clayey silt. Our analysis (Appendix D), assuming a 3-foot seepage zone parallel to the slope face, of the anticipated compacted fill indicates that the proposed slopes will resist surficial failures. All slopes should be constructed in accordance with the General Earthwork and Grading Specifications for Rough Grading (Appendix F) and the City of San Juan Capistrano grading ordinances. Where remedial grading necessitates the construction of manufactured slopes not illustrated on the grading plan, we recommend such slopes be reconstructed at no steeper than 2:1 (H:V) and include intermittent terraces or V-ditches for surficial drainage, in accordance with the designs presented on the grading plan and City of San Juan Capistrano grading ordinances. Erosion and/or surficial failure potential of fill slopes may be reduced if the following measures are implemented during design and construction of the slopes.

- Slope Face Compaction and Finishing

  Due to the fine-grained nature of the prevailing soils, special compaction procedures will be necessary in order for the specified compaction to be achieved out to the slope face. During fill placement, frequent backrolling with sheepsfoot compactors out onto the slope face and backrolling the completed slope with a short-shank sheepsfoot is recommended. This would be in lieu of grid rolling. Alternatively, fill slopes may be overbuilt and trimmed back to expose the properly compacted slope face.

- Slope Landscaping and Drainage

  We recommended that all graded slopes be landscaped with drought-tolerant, slope stabilizing vegetation as soon as possible to minimize the potential for erosion. In addition to the site drainage recommendations, we recommend terrace drains be provided at intervals of 30 vertical feet or less and be constructed in accordance with current City of San Juan Capistrano specifications. Design of surface drainage provisions is within the purview of the project civil engineer.

7.3.5 Foundation Setback from Slopes

The following foundation considerations are presented for planning purposes based on the anticipated site use. We recommend a minimum horizontal setback distance from the face of slopes for all structural foundations, footings, and other settlement-sensitive structures as indicated on Table 5 This distance is measured from the outside bottom edge of the footing, horizontally to the slope face and is based on the slope height and type of soil. However, the foundation setback distance may be revised by the geotechnical consultant on a case-by-case basis if the geotechnical conditions are different than anticipated.

Table 5

Minimum Foundation Setback from Slope Faces

Slope Height	Minimum Recommended Foundation Setback
less than 5 feet	7 feet
5 to 30 Feet	10 feet
greater than 30 feet	H/3

Please note that soils within the structural setback area possess poor lateral stability, and improvements (such as retaining walls, sidewalks, fences, pavements, etc.) constructed within this setback area may be subject to lateral movement and/or differential settlement.

7.4 Post-Tensioned Slab Foundation Design Considerations

We recommend the post-tensioned slabs be considered for this project. These slabs should be designed in accordance with the following design parameters presented in Table 3, and the 2008 edition of the California Building Code (CBSC, 2008). A post-tensioned foundation system designed and constructed in accordance with the recommendations provided in this report is expected to be structurally adequate for the support of the structures planned at the subject site, provided our recommendations for slope maintenance, surface drainage, and landscaping (presented later in this report) are carried out and maintained through the design life of the project. Adhering to the design and maintenance recommendations presented in this report will help ensure that expansive soil-related effects to the residences are limited to cosmetic distresses, with no adverse impact to the overall structural integrity of the residences.

Table 6

Post-Tensioned Slab Design Parameters

Edge Moisture Variation Distance for Edge Lift (e_m), feet	3.7
Edge Moisture Variation Distance for Center Lift (e_m), feet	7.0
Edge Lift (Y_m), inches	7.0
Center Lift (Y_m), inches	1.1
Minimum Depth of Perimeter Footing Embedment, inches	18
Acceptable Design Deflection	Per Structural Engineer

For seismic settlement, the design should consider an additional differential movement of 1 inch between the middle and edge of the slab. The differential movement should be treated as additive to the lift condition being analyzed. Some differential settlement may occur at the site as a result of seismic shaking. The post-tensioned foundations and slabs should be designed in accordance with the design acceptable deflection criteria determined by the structural engineer, architect and governing codes.

Prior to constructing the building pads, in general, the slab subgrade soil should be presoaked to obtain a moisture content between 100 to 120 percent of the optimum moisture content within the upper 18 to 24 inches, to be based on the expansion potential of the building pad. Presoaking recommendations for slab subgrade soils will be provided on a pad-by-pad basis upon completion of site grading.

Our experience indicates that use of reinforcement in slabs and foundations will generally reduce the potential for drying and shrinkage cracking. However, some cracking should be expected as the concrete cures. Minor cracking is considered normal; however, it is often aggravated by a high water/cement ratio, high water content, high concrete temperature at the time of placement, small nominal aggregate size, and rapid moisture loss due to hot, dry and/or windy weather conditions during placement and curing. Cracking due to temperature and moisture fluctuations can also be expected. The use of low water content concrete can reduce the potential for shrinkage cracking and the action of tensioning the tendons can close small shrinkage cracks. In addition to the careful control of water/cement ratios and water content of concrete, application of 50 percent of the design post-tensioning load within three to four days of slab pour may be an effective method of reducing the cracking potential.

The slab subgrade soils underlying the post-tensioned (or equivalent) foundation systems should be presoaked as indicated above, prior to placement of the moisture barrier and slab concrete.

Where moisture-sensitive finishes are planned, underslab moisture protection should be designed by the project architect. Additional guidance is contained with ACI 302.1R-04, Guide for Concrete Floor and Slab Construction and ACI 302.2R-06, Guide for Concrete Slabs that Receive Moisture Sensitive Flooring Materials.

7.5 Lateral Earth Pressures and Resistance

Embedded structural walls should be designed for lateral earth pressures exerted on them. The magnitude of these pressures depends on the amount of deformation that the wall can yield under load. If the wall can yield enough to mobilize the full shear strength of the soil, it can be designed for "active" pressure. If the wall cannot yield under the applied load, the shear strength of the soil cannot be mobilized and the earth pressure will be higher. Such walls should be designed for "at rest" conditions. If a structure moves toward the soils, the resulting resistance developed by the soil is the "passive" resistance.

For design purposes, for walls founded above the static ground water table and backfilled with a very low expansion potential (EI $\leq$ 20), the recommended equivalent fluid pressure for each case is provided below. The onsite expansive soils are not considered suitable for wall backfill. Therefore, we recommend nonexpansive, granular soil be imported for use as compacted wall backfill.

Table 7

Equivalent Fluid Weight (pcf)

Condition	Level	3:1 Slope	2:1 Slope
Active	35	50	55
At-Rest	55	60	55
Passive (Fill Soils)	250 (Maximum of 3 ksf)	150 (Sloping Down)	100 (Sloping Down)

The above values assume non-expansive backfill and free-draining conditions. If conditions other than those assumed above are anticipated, the equivalent fluid pressure values should be provided on an individual-case basis by the geotechnical engineer.

All retaining wall structures should be provided with appropriate drainage. Typical drainage design is contained in Appendix F. As an alternative, a drain board may be utilized behind the retaining wall in addition to normal waterproofing. This system may consist of Miradrain (or Geotech Systems Drainage Board) lined with filter fabric. At the wall base, we recommend that a minimum of the bottom 24 inches of the drain board and the 6-inch perforated PVC drain be wrapped in filter cloth (Mirafi 140N or equivalent). The pipe should be surrounded by approximately 4 cubic feet of clean gravel per foot of wall length. The pipe should be sloped to drain to a suitable outlet.

As previously mentioned, the walls should be backfilled with granular, nonexpansive (EI $\leq$ 20) material. The granular material backfill should be brought up to a height of approximately 2 feet below the top of the walls and capped with compacted fill consisting of native soils. The granular and native backfill soils should be compacted to at least 90 percent relative compaction (based on ASTM Test Method D1557). The top of the granular fill should extend horizontally to a minimum distance equal to one-half the wall height behind the walls. The walls should be constructed and backfilled as soon as practical after backcut excavation. Prolonged exposure of retaining wall backcut slope may result in slope instability.

To reduce the effect of soil expansion on wall footings, such as for retaining and free-standing walls, the footings should penetrate through the soil zone that is most likely prone to volume change. It is recommended the footings be embedded at least 24 inches below the lowest adjacent finish grade. At these depths, minimum 24 inch wide continuous footings may be designed for a bearing pressure of 1,500 psf. Continuous footings should be reinforced with at least two No. 5 bars top and bottom. In addition, the wall footings should be designed and reinforced with structural considerations. Plans for free-standing walls located at the top of slopes should be reviewed by the geotechnical consultant prior to construction. Foundation setbacks for retaining and free-standing walls should be in accordance with the recommendations outlined in the following section.

Soil resistance developed against lateral structural movement can be obtained from the passive pressure value provided above. Further, for sliding resistance, a friction coefficient of 0.35 may be used at the concrete and soil interface. These values may be increased by one-third when considering loads of short duration including wind or seismic loads. The total resistance may be taken as the sum of the frictional and passive resistances provided that the passive portion does not exceed two-thirds of the total resistance.

7.6 Fences and Freestanding Walls

Fences and freestanding wall recommendations are presented in the following sections.

7.6.1 Walls and Fences Not Close to the Top of Slopes

Footings for freestanding walls should be founded a minimum of 24 inches below lowest adjacent grade. To reduce the potential for unsightly cracks in freestanding walls, we recommend inclusion of construction joints at a maximum of 15-foot intervals. This spacing may be altered in accordance with the recommendations of the structural engineer, based on wall reinforcement details.

7.6.2 Walls and Fences Close to the Top of Slopes

Our experience on similar sites in older developments indicates that many back yard and side yard walls on shallow foundations near the top-of-slopes tend to tilt excessively over time as a result of slope creep. If the effects of slope creep on top-of-slope walls are not deemed acceptable, one or a combination of the options provided in the following paragraph should be utilized in the design of such structures, based on the desired level of mitigation of creep-related effects on them.

A relatively inexpensive option to address creep related problems in top-of-slope walls and fences is to allow some degree of creep damage and design the structures so that tilting or cracking will be less visually obvious, or such that they may be economically repaired or replaced. If, however, a better degree of creep mitigation is desired, the walls and fences may be provided with the deepened foundations or caissons and grade beams that meet the setback requirements for structure foundations. In addition, the inclusion of frequent (10-15 feet interval) crack control joints should be considered.

7.7 Control of Surface Water and Drainage Control

Surface drainage should be carefully taken into consideration during precise grading, landscaping, and building construction. Positive drainage (e.g., roof gutters, downspouts, area drain, etc.) should be provided to direct surface water away from structures and towards the street or suitable drainage devices. Ponding of water adjacent to structures should be avoided. Roof gutters, downspouts, and area drains should be aligned so as to transport surface water to a minimum distance of 5 feet away from structures. The performance of structural foundations is dependent upon maintaining adequate surface drainage away from structures.

Water should be transported off the site in approved drainage devices or unobstructed swales. We recommend that the minimum flow gradient for the drainage be 1 percent for area drains and paved drainage swales and 2 percent for unpaved drainage swales, and within 5 feet of structures (sloping away). Unpaved swales may have a minimum flow gradient of 1 percent if used in conjunction with area drains; however, swales with only 1 percent gradient should not be constructed within 5 feet of buildings. In places where the prospect of maintaining the minimum recommended gradient for the drainage swales and the construction of additional area drains is not feasible, provisions for specific recommendations to the homeowners may be necessary, outlining the importance of maintaining positive drainage to streets.

The impact of heavy irrigation or inadequate runoff gradient can create perched water conditions, resulting in seepage or shallow groundwater conditions where previously none existed. Maintaining adequate surface drainage and controlled irrigation will significantly reduce the potential for nuisance-type moisture problems. To reduce differential earth movements such as heaving and shrinkage due to the change in moisture content of foundation soils, which may cause distress to a residential structure and improvements, moisture content of the soils surrounding the structure should be kept as relatively constant as possible.

All area drain inlets should be maintained and kept clear of debris in order to function properly. In addition, yard landscaping should not cause any obstruction to the yard drainage. Rerouting of yard drainage pattern and/or installation of area drains should be performed, if necessary, but a qualified civil engineer or a landscape architect should be consulted prior to rerouting of drainage.

7.8 Landscaping and Post-Construction

Landscaping and post-construction practices exert significant influences on the integrity of structures founded on expansive soils. Improper landscaping and post-construction practices, which are beyond the control of the geotechnical engineer, are frequently the primary cause of distress to these structures. Recommendations for proper landscaping and post-construction practices are provided in the following paragraphs within this section. Adhering to these recommendations will help in minimizing distress due to expansive soils, and in ensuring that such effects are limited to cosmetic damages, without compromising the overall integrity of structures. The recommendations provided herein have been developed in general accordance with the guidelines provided within the Post-Tensioning Institute's (1996) recommendations for the design and construction of post-tensioned slabs-on-ground.

Initial landscaping should be done on all sides adjacent to the foundation of a structure, and adequate measures should be taken to ensure drainage of water away from the foundation. If larger, shade providing trees are desired, such trees should be planted away from structures (at a minimum distance equal to half the mature height of the tree) in order to prevent penetration of the tree roots beneath the foundation of the structure.

Locating planters adjacent to buildings or structures should be avoided as much as possible. If planters are utilized in these locations, they should be properly designed (such as with a liner) so as to prevent fluctuations in the moisture content of subgrade soils. Planting areas at grade should be provided with appropriate positive drainage. Wherever possible, exposed soil areas should be above paved grades. Planters should not be depressed below adjacent paved grades unless provisions for drainage, such as catch basins and drains, are made. Adequate drainage gradients, devices, and curbing should be provided to prevent runoff from adjacent pavement or walks into planting areas.

Watering should be done in a uniform, systematic manner as equally as possible on all sides of the foundation, to keep the soil moist. Irrigation methods should promote uniformity of moisture in planters and beneath adjacent concrete flatwork. Overwatering and underwatering of landscape areas must be avoided. Areas of soil that do no have ground cover may require more moisture, as they are more susceptible to evaporation. Ponding or trapping of water in localized areas adjacent to the foundations can cause differential moisture levels in subsurface soils and should, therefore, not be allowed. Trees located within a distance of 20 feet of foundations would require more water in periods of extreme drought, and in some cases, a root injection system may be required to maintain moisture equilibrium. During extreme hot and dry periods, close observations should be carried out around foundations to ensure that adequate moisture control is being undertaken to deter soil from separating or pulling back from the foundations.

7.9 Slope Maintenance Guidelines

It is the responsibility of the owner to maintain the slopes, including adequate planting, proper irrigation and maintenance, and repair of faulty irrigation systems. To reduce the potential for erosion and slumping of graded slopes, all slopes should be planted with ground cover, shrubs, and plants that develop dense, deep root structures and require minimal irrigation. Slope planting should be carried out as soon as practical upon completion of grading. Surface-water runoff and standing water at the top-of-slopes should be avoided. Oversteepening of slopes should be avoided during construction activities and landscaping. Maintenance of proper lot drainage, undertaking of property improvements in accordance with sound engineering practices, and proper maintenance of vegetation, including regular slope irrigation, should be performed. Slope irrigation sprinklers should be adjusted to provide maximum uniform coverage with minimal of water usage and overlap. Overwatering and consequent runoff and ground saturation should be avoided. If automatic sprinklers systems are installed, their use must be adjusted to account for rainfall conditions.

Trenches excavated on a slope face for any purpose should be properly backfilled and compacted in order to obtain a minimum of 90 percent relative compaction, in accordance with ASTM Test Method D1557. Excavations should not be made in areas of geogrid reinforcement and slope irrigation lines should be placed on face of slope. Observation/testing and acceptance by the geotechnical consultant during trench backfill are recommended. A rodent-control program should be established and maintained. Prior to planting, recently graded slopes should be temporarily protected against erosion resulting from rainfall, by the implementing slope protection measures such as polymer covering, jute mesh, etc.

7.10 Concrete

Concrete in direct contact with soil or water that contains a high concentration of soluble sulfates can be subject to chemical deterioration commonly known as "sulfate attack." Soils in the general site area typically generally possess water-soluble sulfate content greater than 0.2 percent and less than 2.0 percent. Therefore, we recommend that concrete in contact with earth materials be designed in accordance with the Section 4 of ACI 318-08 (ACI, 2008). In addition, testing and previous testing of similar soils of the Capistrano Formation indicates that the soils are corrosive to metals. We recommend measures to mitigate corrosion be implemented during design and construction per the project corrosion engineer.

7.11 Pavement Sections

Design of pavement sections was not included within the scope of this report. Pavement sections will depend largely on the subgrade soil conditions after grade. Pavement sections in accordance with the City of San Juan Capistrano criteria can be provided upon completion of rough grading based on laboratory R-value testing of subgrade soils.

7.12 Construction Observation and Grading Plan Review

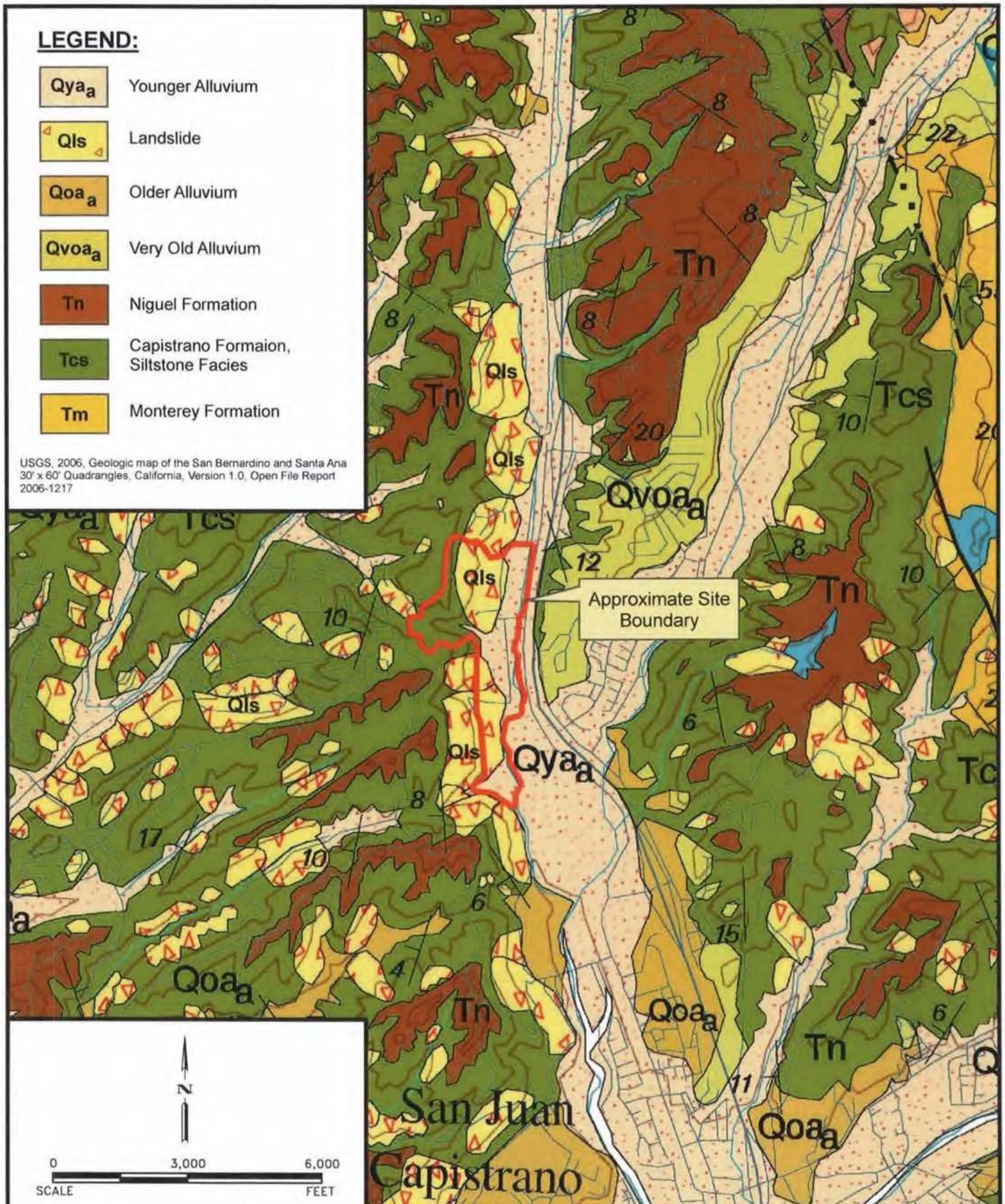
The recommendations provided in this report are based on preliminary design information and subsurface conditions disclosed by widely spaced borings. The interpolated subsurface conditions should be checked in the field during construction. Construction observation of all onsite excavations and field density testing of all compacted fill should be performed by representatives of this office so that construction is in accordance with the recommendations of this report. We recommend that cut slopes and all backcuts be geologically mapped by the geotechnical consultant during grading for the presence of potentially adverse geologic conditions.

Final project plans should be geotechnically reviewed by the geotechnical engineer to see that the recommendations provided in this report are incorporated.

7.13 Limitations

The conclusions and recommendations in this report are based in part upon data that were obtained from a limited number of observations, site visits, excavations, samples, and tests. Such information is by necessity incomplete. The nature of many sites is such that differing geotechnical or geological conditions can occur within small distances and under varying climatic conditions. Changes in subsurface conditions can and do occur over time. Therefore, the findings, conclusions, and recommendations presented in this report can be relied upon only if Leighton has the opportunity to perform addition subsurface investigation and observe the subsurface conditions during grading and construction of the project, in order to confirm that our preliminary findings are representative for the site.

FIGURES



The Orchards
San Juan Capistrano, California

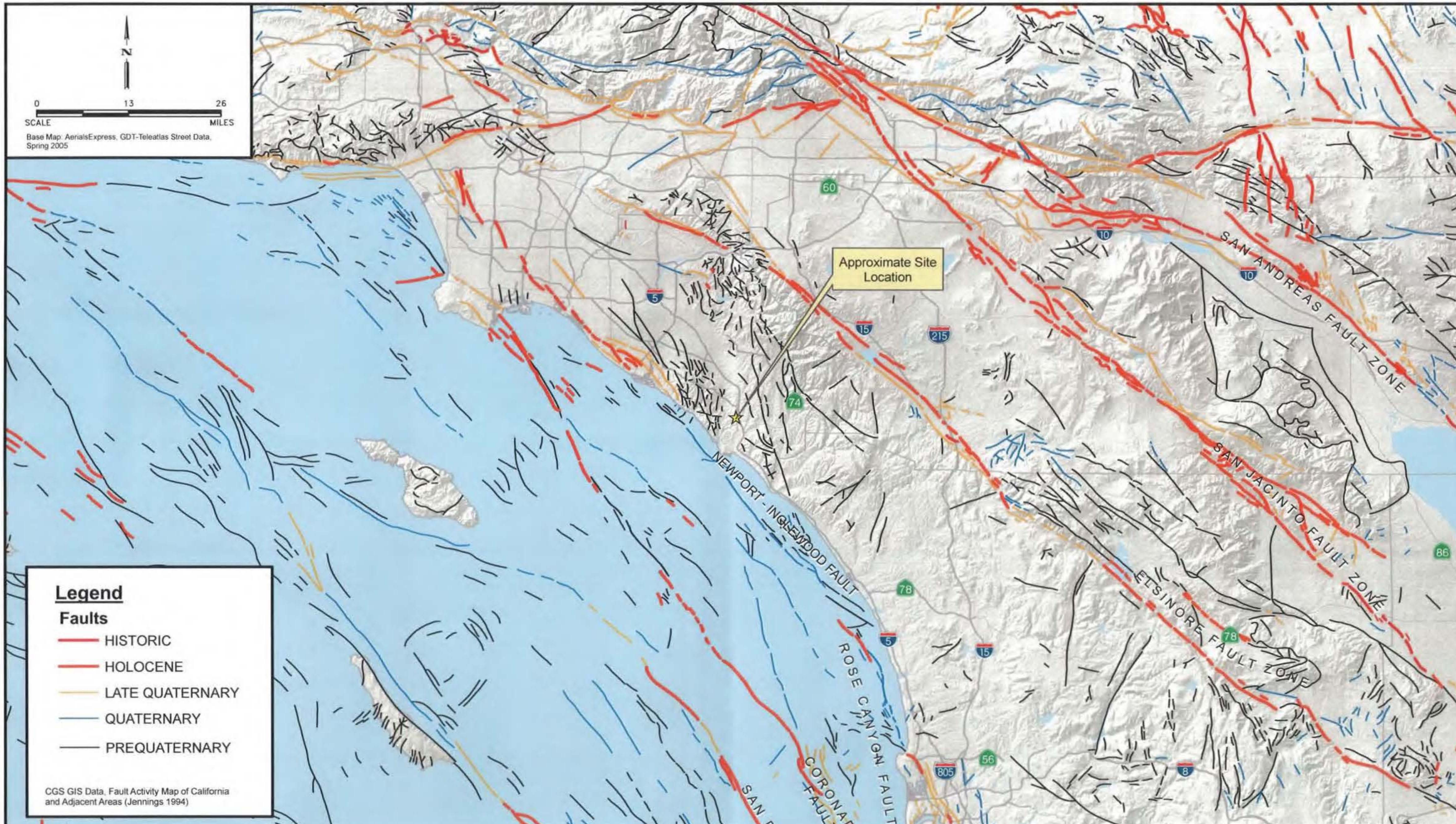
REGIONAL GEOLOGY MAP

Project No.
012383-001

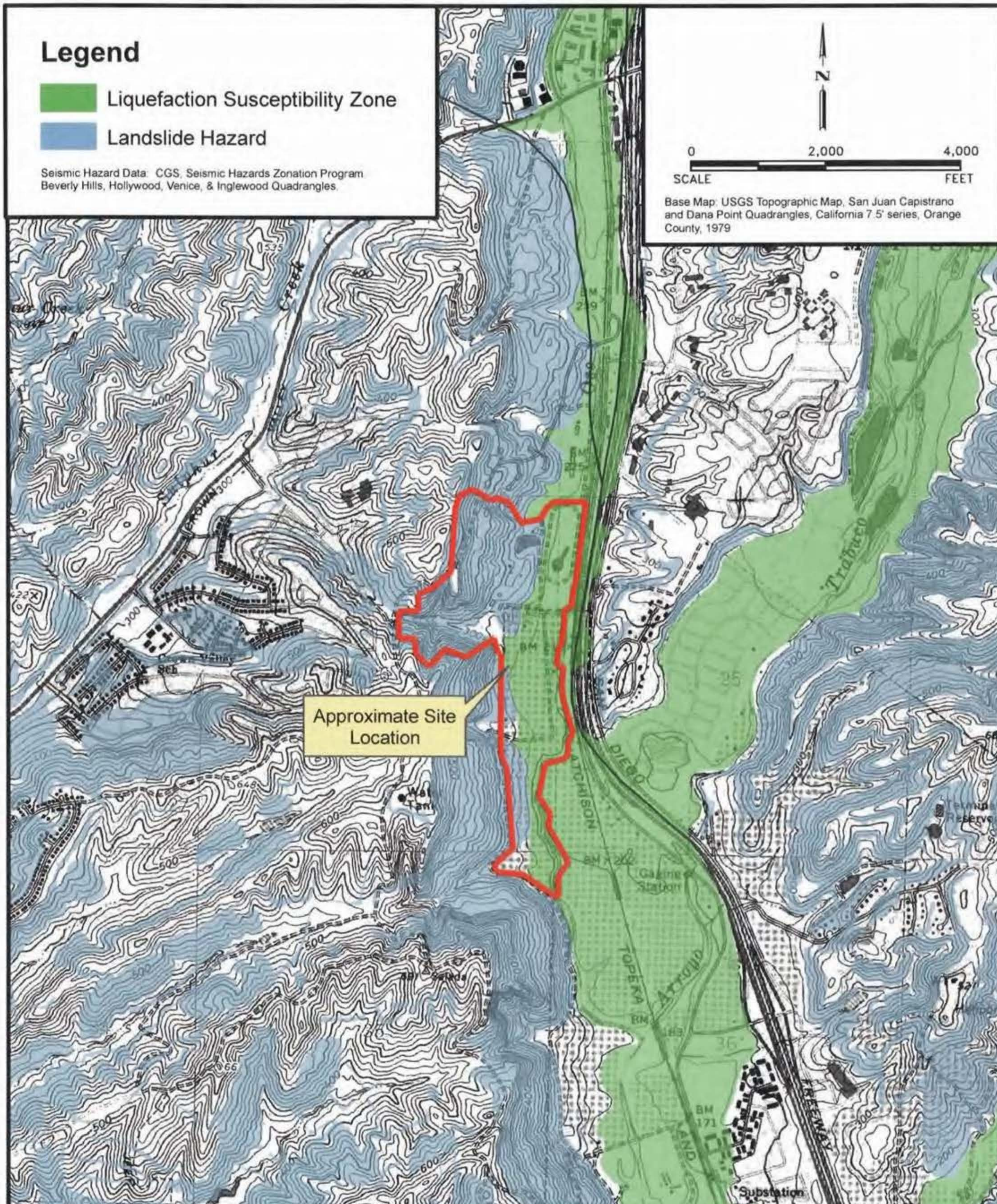
Date
April 2009

Figure 4

\\GIS\Administration\ArcGISTemplates\NEW_GDT_SiteLocationMap.mxd



The Orchards San Juan Capistrano, California	REGIONAL FAULT MAP	Project No. 012383-001 Date April 2009	Figure 5



The Orchards
San Juan Capistrano, California

SEISMIC HAZARDS MAP

Project No.
012383-001

Date
April 2009

Figure 6

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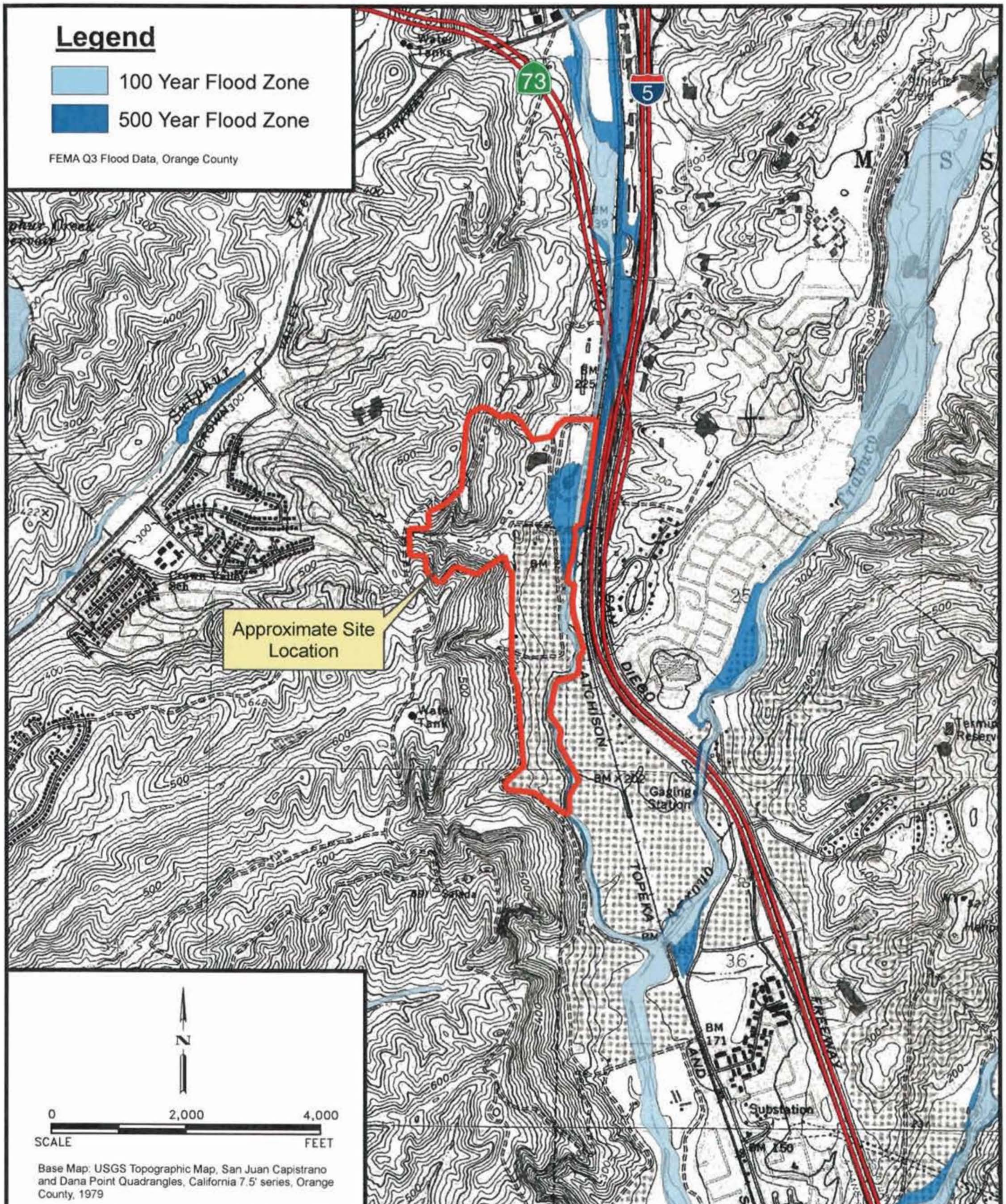
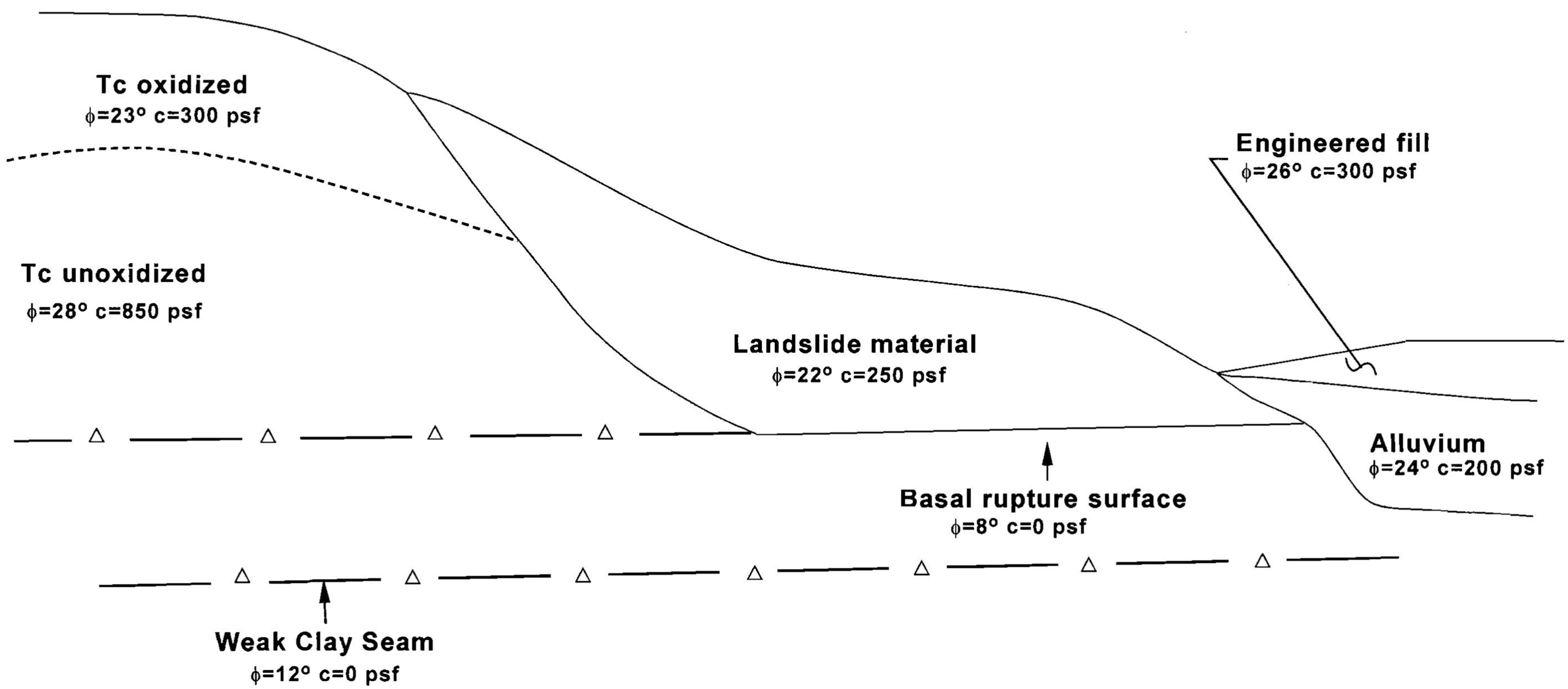


The Orchards San Juan Capistrano, California	**FLOOD HAZARD MAP**	Project No. **012383-001** Date **April 2009** Figure 7

\\GIS\Administration\ArcGISTemplates\NEW_GDT_SiteLocationMap.mxd



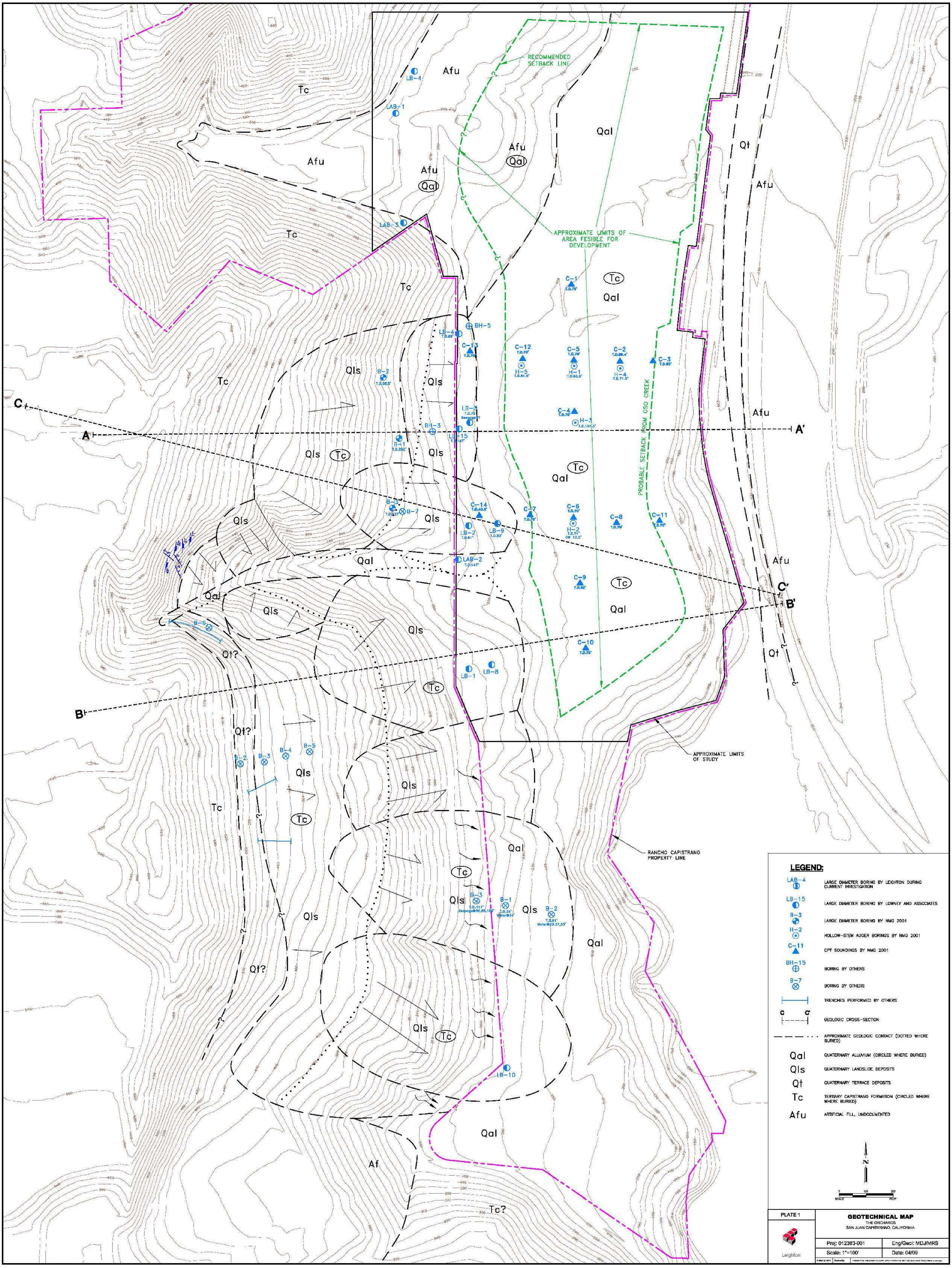
SCHEMATIC DIAGRAM SHOWING ASSIGNED SHEAR STRENGTH PARAMETERS AND ASSUMED FAILURE GEOMETRIES

The Orchards at Capistrano

Project No.	012383-001
Scale	Not to scale
Engr./Geol.	SAC/MRS
Drafted By	-
Date	April 30, 2009

Figure No. 8

PLATE 1 and 2


RECOMMENDED SETBACK LINE
APPROXIMATE LIMITS OF AREA FESIBLE FOR DEVELOPMENT
PROBABLE SETBACK FROM OSO CREEK
APPROXIMATE LIMITS OF STUDY
RANCHO CAPISTRANO PROPERTY LINE
Afu
Qal
Qls
Qt
Qt?
Tc
Tc?
Af
A
A'
B
B'
C
C'
LB-4
LAB-1
LAB-3
LAB-2
BH-5
BH-3
LB-15
LB-2
LB-9
LB-1
LB-8
LB-10
C-1
C-2
C-3
C-4
C-5
C-6
C-7
C-8
C-9
C-10
C-11
C-12
C-13
C-14
H-1
H-2
H-3
H-4
H-5
B-1
B-2
B-3
B-4
B-5
B-6
B-7
LEGEND:
LAB-4 LARGE DIAMETER BORING BY LEIGHTON DURING CURRENT INVESTIGATION
LB-15 LARGE DIAMETER BORING BY LOWNEY AND ASSOCIATES
B-3 LARGE DIAMETER BORING BY NMG 2001
H-2 HOLLOW-STEM AUGER BORINGS BY NMG 2001
C-11 CPT SOUNDINGS BY NMG 2001
BH-15 BORING BY OTHERS
B-7 BORING BY OTHERS
TRENCHES PERFORMED BY OTHERS
C C' GEOLOGIC CROSS-SECTION
APPROXIMATE GEOLOGIC CONTACT (DOTTED WHERE BURIED)
Qal QUATERNARY ALLUVIUM (CIRCLED WHERE BURIED)
Qls QUATERNARY LANDSLIDE DEPOSITS
Qt QUATERNARY TERRACE DEPOSITS
Tc TERTIARY CAPISTRANO FORMATION (CIRCLED WHERE WHERE BURIED)
Afu ARTIFICIAL FILL, UNDOCUMENTED
N
0 100 200
SCALE FEET
PLATE 1
Leighton
GEOTECHNICAL MAP
THE ORCHARDS
SAN JUAN CAPISTRANO, CALIFORNIA
Proj: 012383-001
Eng/Geol: MDJ/MRS
Scale: 1"=100'
Date: 04/09

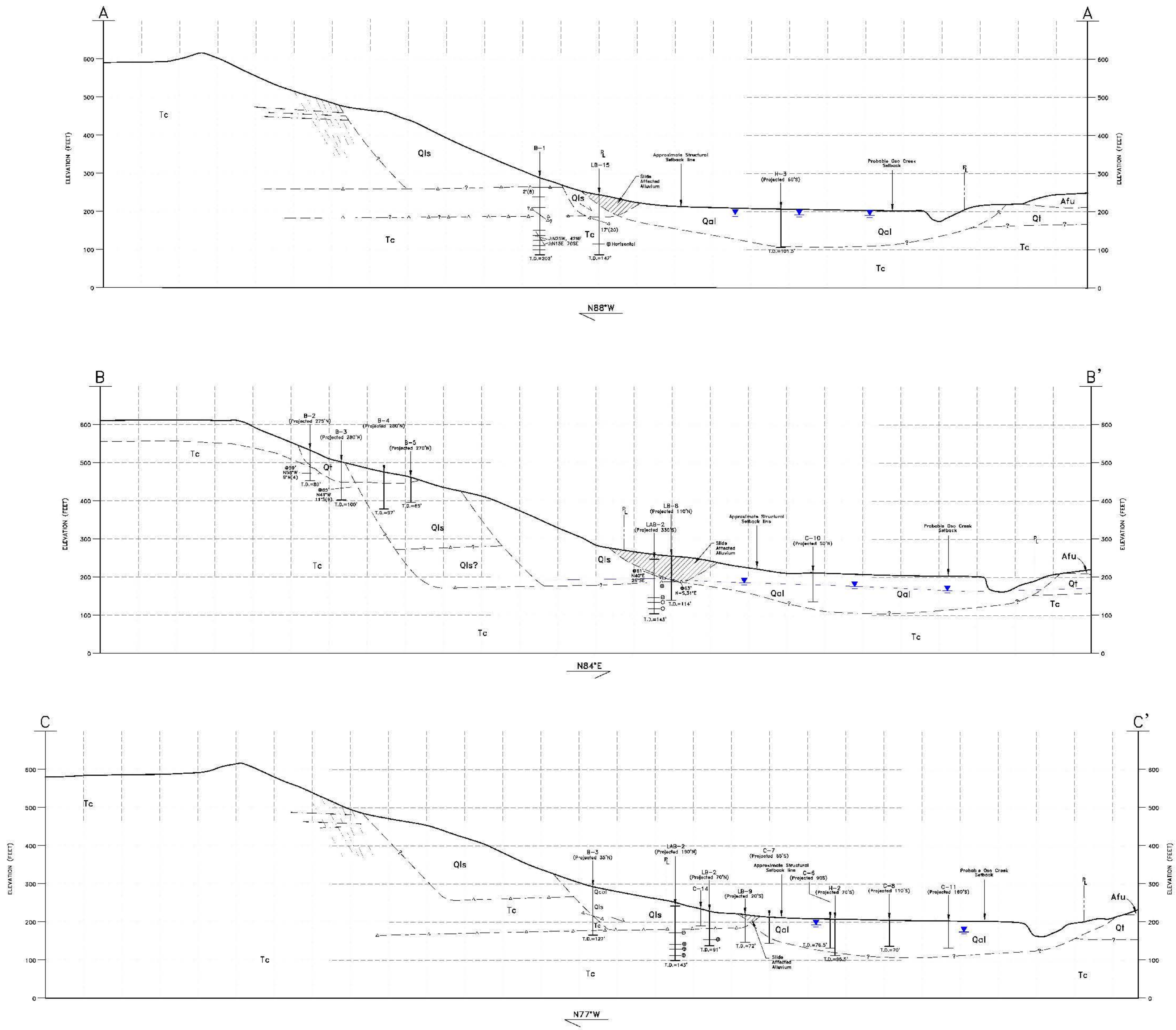


PLATE 2	**CROSS-SECTIONS A-A', B-B' AND C-C'** THE ORCHARDS SAN JUAN CAPISTRANO, CALIFORNIA	
Leighton	Proj: 012383-001	Eng/Geol: MDJ/MRS
	Scale: 1"=100'	Date: 04/09

PRELIMINARY GEOTECHNICAL INVESTIGATION
THE ORCHARDS AT CAPISTRANO
SAN JUAN CAPISTRANO, CALIFORNIA

VOLUME II

Prepared for:

Continuing Life Communities, LLC
1940 Levante Street
Carlsbad, California 92009

Project No. 012383-001

April 30, 2009

Leighton and Associates, Inc.
A LEIGHTON GROUP COMPANY

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Appendices

Leighton

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APPENDIX A

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APPENDIX B

Current
Leighton Logs

GEOTECHNICAL BORING LOG LAB-1

Project No.	012383-001	**Date Drilled**	12-23-08
Project	Continuing Life/Orchards	**Logged By**	MAW
Drilling Co.	C&L Pacific Drilling	**Hole Diameter**	30"
Drilling Method	- 12" Drop	**Ground Elevation**	280'
Location		**Sampled By**	MAW

Elevation Feet	Depth Feet	Graphic Log (N S)	Attitudes	Sample No.	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	**SOIL DESCRIPTION** *The Soil Description applies only to a location of the exploration at the time of drilling. Subsurface conditions may differ at other locations and may change with time. The description is a simplification of the actual conditions encountered. Transitions between soil types may be gradual.*	Type of Tests
280	0							CL	QUATERNARY COLLUVIUM @ 0': Sandy CLAY: Dark brown, damp to moist, loose; organics	
275	5								@ 7': Silty sandy CLAY: Brown, moist, loose	
270	10									
265	15								@ 17': Ground water encountered	
260	20								@ 20': Denser possible Capistrano Formation **Total Depth = 20 Feet** **Ground water encountered at 17 Feet** **Boring backfilled 12/23/08** **Not downhole logged due to presence of ground water** **Drive Weights: 0-29-3,615 lbs.**	
255	25									
250	30									

SAMPLE TYPES:
S SPLIT SPOON
R RING SAMPLE
B BULK SAMPLE
T TUBE SAMPLE
G GRAB SAMPLE
C CORE SAMPLE

TYPE OF TESTS:
DS DIRECT SHEAR
MD MAXIMUM DENSITY
CN CONSOLIDATION
CR CORROSION
SA SIEVE ANALYSIS
SE SAND EQUIVALENT
EI EXPANSION INDEX
RV R VALUE
-200 % FINES PASSING
AL ATTERBERG LIMITS
CO COLLAPSE
PP POCKET PENETROMETER

*** This log is a part of a report by Leighton and should not be used as a stand-alone document. ***

GEOTECHNICAL BORING LOG LAB-2

Project No.	012383-001	**Date Drilled**	12-28-08
Project	Continuing Life/Orchards	**Logged By**	MDJ
Drilling Co.	C&L Pacific Drilling	**Hole Diameter**	30"
Drilling Method	- 12" Drop	**Ground Elevation**	245'
Location		**Sampled By**	MDJ

Elevation Feet	Depth Feet	Graphic Log (N S)	Attitudes	Sample No.	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	SOIL DESCRIPTION *The Soil Description applies only to a location of the exploration at the time of drilling. Subsurface conditions may differ at other locations and may change with time. The description is a simplification of the actual conditions encountered. Transitions between soil types may be gradual.*	Type of Tests
245	0							ML	QUATERNARY ALLUVIUM (Qal)	
									@ 4': Fine sandy SILT: Gray-brown, damp to moist Casing in upper 4'	
240	5									
								ML	QUARTERNARY LANDSLIDE DEPOSITS (Qls) @ 9': Concretion fragments around hole, very fine sandy SILT: Brown to light brown, damp, loose; mottled disturbed, gypsum, calcium-carbonate blebs	
235	10			B-1 9'-10'						
									@ 12'-13': Concretion fragments, calcium-carbonate blebs abundant below 13'	
230	15			R-1	2				@ 15': Fine sandy SILT with clay: Brown, damp to moist, medium stiff, moisture change at 15', clay-lined shear, random shears @ 16'-16.8': Broken concretion with random clay-lined shears	
								SM	@ 17': Silty fine SAND: Light brown and gray-brown, damp, medium dense	
								ML	@ 19': Fine sandy SILT with clay: Gray, moist, medium stiff, high plasticity	
225	20			B-1 20'-22'				SM	@ 20': Silty clay with SAND: Medium brown to gray-brown, moist,	
			sh:N50W, 12-16SE					CL	@ 22'-23': High plasticity clay layer, increase iron-oxide staining (orange and yellow)	
			N40W, 6-7°NE					ML	@ 24': Dark gray/black SILT	
220	25								@ 25': Increasing sand content and density, random shear blebs	
									@ 26': Clayey SILT: Gray-brown	
			sh:N60E 30°N						@ 27.5'-28': Highly plasticity clay layer on shear, dark gray below layer (shear)	
215	30									

SAMPLE TYPES:

- S SPLIT SPOON
- R RING SAMPLE
- B BULK SAMPLE
- T TUBE SAMPLE
- G GRAB SAMPLE
- C CORE SAMPLE

TYPE OF TESTS:

- DS DIRECT SHEAR
- MD MAXIMUM DENSITY
- CN CONSOLIDATION
- CR CORROSION
- SA SIEVE ANALYSIS
- SE SAND EQUIVALENT
- EI EXPANSION INDEX
- RV R VALUE
- -200 % FINES PASSING
- AL ATTERBERG LIMITS
- CO COLLAPSE
- PP POCKET PENETROMETER

GEOTECHNICAL BORING LOG LAB-2

Project No.	012383-001	**Date Drilled**	12-28-08
Project	Continuing Life/Orchards	**Logged By**	MDJ
Drilling Co.	C&L Pacific Drilling	**Hole Diameter**	30"
Drilling Method	- 12" Drop	**Ground Elevation**	245'
Location		**Sampled By**	MDJ

Elevation Feet	Depth Feet	Graphic Log (N S)	Attitudes	Sample No.	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	**SOIL DESCRIPTION** *The Soil Description applies only to a location of the exploration at the time of drilling. Subsurface conditions may differ at other locations and may change with time. The description is a simplification of the actual conditions encountered. Transitions between soil types may be gradual.*	Type of Tests
215	30			R-2	1			ML	QUATERNARY LANDSLIDE DEPOSITS (Qls) continued @ 30': Fine sandy clayey SILT: Dark gray, moist to very moist, soft to medium stiff; micaceous, plastic, organic odor @ 31.5': Plastic layer 2" thick	
210	35			B-3 36'				CL	@ 36.8'-37': Very moist black and gray SM/ML, organics with plastic layer	
									@ 39': Sandy silty CLAY: Dark gray (charcoal gray), moist, medium stiff, stiffer drilling conditions below 39'	
205	40		sh:E-W 50°N						@ 41': Higher plasticity CLAY, some black layers (possibly charcoal), shear dipping 30°	
									@ 44': Changes color from gray and dark gray to gray and orange, becomes damp	
200	45			R-3	3			ML	@ 45': Very fine sandy SILT with clay: Gray to dark gray, moist, medium stiff to stiff	
									@ 47': Stiffer drilling	
								SM	@ 48': Silty fine SAND: Medium gray, moist, dense, abundant calcium carbonate stringers and bleb	
195	50							ML	@ 50.5': Silty very fine sandy SILT: Gray-brown to gray, moist, medium stiff to stiff, iron-oxide staining, calcium-carbonate bleb still common	
190	55			R-4	10			ML/CL	@ 55': Silty CLAY to sandy clayey SILT: Gray to gray, brown, moist, stiff; iron-oxide staining	
			striation: 4° N78E rs:N48W, rs:4-5°SW N34°W, 4SW						@ 58.5': Rupture surface 1" to 1.5" thick plastic soft clay striated N78E	
185	60									

SAMPLE TYPES:

S	SPLIT SPOON	G	GRAB SAMPLE
R	RING SAMPLE	C	CORE SAMPLE
B	BULK SAMPLE		
T	TUBE SAMPLE		

TYPE OF TESTS:

DS	DIRECT SHEAR	SA	SIEVE ANALYSIS	-200	% FINES PASSING
MD	MAXIMUM DENSITY	SE	SAND EQUIVALENT	AL	ATTERBERG LIMITS
CN	CONSOLIDATION	EI	EXPANSION INDEX	CO	COLLAPSE
CR	CORROSION	RV	R VALUE	PP	POCKET PENETROMETER

GEOTECHNICAL BORING LOG LAB-2

Project No.	012383-001	**Date Drilled**	12-28-08
Project	Continuing Life/Orchards	**Logged By**	MDJ
Drilling Co.	C&L Pacific Drilling	**Hole Diameter**	30"
Drilling Method	- 12" Drop	**Ground Elevation**	245'
Location		**Sampled By**	MDJ

Elevation Feet	Depth Feet	Graphic Log (N S)	Attitudes	Sample No.	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	SOIL DESCRIPTION	Type of Tests
									The Soil Description applies only to a location of the exploration at the time of drilling. Subsurface conditions may differ at other locations and may change with time. The description is a simplification of the actual conditions encountered. Transitions between soil types may be gradual.	
185	60								TERTIARY CAPISTRANO FORMATION (Tc) @ 60': Very fine sandy SILTSTONE: Dark gray-brown to dark gray, damp, very stiff; micaceous, massive, hydrocarbon odor?? slower driller 59-60°	
180	65			R-5 B-6 65'	18				@ 65': Very fine sandy SILTSTONE: Dark gray, damp, very stiff to hard; massive, very fine white blebs (diatomaceous); unoxidized	
									@ 68': Horizontal at 68.4' on light gray SILTSTONE	
175	70									
									@ 74': Some claystone beds	
170	75			R-6	22				@ 75': Very fine sandy SILTSTONE with clay: Dark gray, damp, hard; very fine blebs (diatomaceous); unoxidized, very fine micaceous	
165	80									
160	85			R-7						
155	90									

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TYPE OF TESTS:
- DS DIRECT SHEAR
- MD MAXIMUM DENSITY
- CN CONSOLIDATION
- CR CORROSION
- SA SIEVE ANALYSIS
- SE SAND EQUIVALENT
- EI EXPANSION INDEX
- RV R VALUE
- -200 % FINES PASSING
- AL ATTERBERG LIMITS
- CO COLLAPSE
- PP POCKET PENETROMETER

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GEOTECHNICAL BORING LOG LAB-2

Project No.	012383-001	**Date Drilled**	12-28-08
Project	Continuing Life/Orchards	**Logged By**	MDJ
Drilling Co.	C&L Pacific Drilling	**Hole Diameter**	30"
Drilling Method	- 12" Drop	**Ground Elevation**	245'
Location		**Sampled By**	MDJ

Elevation Feet	Depth Feet	Graphic Log (N S)	Attitudes	Sample No.	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	SOIL DESCRIPTION *The Soil Description applies only to a location of the exploration at the time of drilling. Subsurface conditions may differ at other locations and may change with time. The description is a simplification of the actual conditions encountered. Transitions between soil types may be gradual.*	Type of Tests
155	90							ML	TERTIARY CAPISTRANO FORMATION (Tc) Continued	
150	95			R-8	10				@ 95': Very fine sandy SILTSTONE with clay: Dark gray, damp, hard; unoxidized, hydrocarbon odor, very fine white blebs	
									@ 99': Generally horizontal bed along laminated SILTSTONE layer	
145	100								@ 100': Concretion (well cemented, very dense)	
140	105			R-9	15				@ 105': Very fine sandy SILTSTONE with clay: Dark gray, damp, hard; unoxidized, hydrocarbon odor, very fine white blebs	
135	110			B-7 110'						
									@ 112': Subround concretion right side of hole, faint generally horizontal bedding	
130	115			R-10	9				@ 115': Very fine sandy SILTSTONE: Dark gray, damp, hard	
									@ 117'-117.5': Black/gray bed over medium gray bed, continuous around hole at approximately same depth	
125	120									

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125	120							ML	<u>TERTIARY CAPISTRANO FORMATION (Tc) (Continued)</u>	
120	125			R-11	20				@ 125': Very fine sandy SILTSTONE: Dark gray-brown, damp, hard @ 126': Generally horizontal	
115	130									
									@ 133'-134': Hit very dense concretion layer, random orientation, polished surfaces at the top of concretion, concretion appears to be intact, but mechanically broken	
110	135			R-12	10				@ 125': Very fine sandy SILTSTONE: Dark gray-brown, damp, hard	
105	140									
				R-13	10				**Downhole Logged Hole to 141.5 Feet** **No ground water encountered at time of drilling** **Backfilled on 1/5/09** **Drive Weights: 0-29-3,615 lbs, 30-57-2,395 lbs, 58-85-1,310 lbs, 86-110-1,800 lbs, 111-136-2,290 lbs, 137-162-2,780 lbs.**	
100	145									
95	150									

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