C: Preliminary Geotechnical Exploration Report

PRELIMINARY GEOTECHNICAL EXPLORATION REPORT
FOR THE PROPOSED RESIDENTIAL DEVELOPMENT,
CIVIC CENTER, AND PARK AT IRWD SITE,
CITY OF LAKE FOREST, CALIFORNIA

Prepared for:

LEWIS INVESTMENT COMPANY

1156 North Mountain Avenue
Upland, California 91785

Project No. 011797-002

May 7, 2008

Leighton and Associates, Inc.

A LEIGHTON GROUP COMPANY

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To: Lewis Investment Company
1156 North Mountain Avenue
Upland, California 91785

Attention: Mr. Joe J. Stucker, Vice President, Land Sales & Disposition

Subject: Preliminary Geotechnical Exploration Report for the Proposed Residential Development, Civic Center, and Park at IRWD Site, City of Lake Forest, California

In accordance with your request, Leighton and Associates, Inc. (Leighton) has performed a preliminary geotechnical exploration for the proposed development at the Irvine Ranch Water District (IRWD) site in the city of Lake Forest, California. The Conceptual Site Plan prepared by Bassenian Lagoni Architects (BLA, 2008) was utilized to develop our conclusions and recommendations in this report.

Based on our geotechnical exploration, the site is predominantly underlain by documented and undocumented artificial fill, alluvium, and colluvium overlying sandstone of the Oso member of the Capistrano Formation. Groundwater was not encountered during our subsurface exploration. This report presents the results of our field exploration and laboratory testing and provides our conclusions and recommendations for the proposed development of the site as shown on the current conceptual site plan.

Developing the subject site for the proposed use is considered feasible from a geotechnical standpoint, provided the recommendations presented in this report are taken into consideration in the final preparation of the project plans and specifications.

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If you have any questions regarding this report, please do not hesitate to contact this office. We appreciate this opportunity to be of continued service.

Respectfully submitted,

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1.0 INTRODUCTION

1.1 Purpose and Scope

The purpose of this geotechnical exploration was to evaluate the geotechnical conditions and characteristics at the site and to provide recommendations for the design and construction of the proposed developments. The scope of our work included the following tasks:

- Pre-field activities including approval from the project biologist for the proposed boring locations and access routes to avoid the disturbance of the biologically sensitive habitats and clearance from the IRWD personnel for possible underground utilities. Leighton also obtained clearance of underground utilities from Underground Service Alert (USA) prior to commencement of field exploration.

- Review of available site-specific information, including previous geotechnical explorations and rough grading reports for the site and site vicinity and readily available publications and documents. References used in preparation of this report are listed in Section 5.0.

- Field exploration consisting of the excavation, logging, and sampling of three (3) 28-inch diameter bucket auger borings, seven (7) 8-inch diameter hollow-stem auger (HSA) borings, and sixteen (16) test pits to depths ranging from approximately 3 to 80.4 feet below the existing ground surface. The boring logs and the test pit logs are provided in Appendices A and B, respectively.

- Laboratory testing of select representative samples to characterize the engineering properties of the soils. The test results are presented in Appendix C.

- Geotechnical evaluation of the collected data and relevant engineering analyses.

- Preparation of this report summarizing our findings, conclusions, and recommendations.

1.2 Site Location and Existing Conditions

An approximately 43-acre site is located within the existing IRWD site south of Commerce Centre Drive in the city of Lake Forest, California. Serrano Creek is located approximately 0.4 miles southeast of the site. Development at the site consists of an existing administration building for the Baker Water Treatment Facility and documented fill placed in the central portion of the site. The site is bounded to the north by existing industrial development, to the east and west by residential development, and to the south by the Baker water treatment facility. The location of the site is shown in Figure 1, *Site Location Map*.

Ground surface elevations at the site vary from approximately 715 feet above Mean Sea Level (msl) at the northern portion to approximately 595 feet above msl along the southern portion.

1.3 Proposed Development

We understand that the site will be subdivided into six parcels that will consist of residential development, a civic center, and a park. The civic center and the park are currently planned in the eastern and the southern portions of the site, respectively.

Based on the conceptual site plan (BLA, 2008) provided to us, we understand that the proposed residential structures may consist of townhomes, duplexes, flats, and single family units. Associated developments are anticipated to include streets, parking lots, trails, detention basins and related improvements.

The planned site grading generally consists of cut and fill slopes facing to the north, south, east, and west around the perimeter and central portions of the site and fill placement in the central to southern portions of the site. The maximum cut is approximately 45 feet located within the northeastern portions of the site. The maximum fill to be placed over existing grades is approximately 45 to 50 feet located at the existing debris basin within the southern central portion of the site. The northern central area is underlain by up to 75 feet of documented fill placed during the rough grading at the site in the early 1990's. The current conceptual site plan (BLA, 2008) depicts fill placement over the northern central area which will increase the maximum total fill depths to approximately 120 feet upon completion of grading.

1.4 Previous Explorations and Rough Grading

Brief descriptions of the previous geotechnical explorations and rough grading activities within the current site and site vicinity are presented below in chronological order.

1987: Kenneth G. Osborne & Associates, (KGO) prepared a geotechnical investigation report, dated January 6, 1987 for the domestic water storage tank located east of the Baker Water Treatment Facility. The field investigation included eight bucket auger borings and eleven test pits.

KGO prepared a geotechnical report of rough grading, dated April 30, 1987, for a portion of the Emergency Domestic Storage Tank adjacent to the Baker Water Treatment Facility. The grading was performed in conjunction with the grading of the 7.8 million gallon reclaimed water reservoir located approximately 2 miles northeast of the site between March 1987 and April 1987.

1988: KGO prepared a geotechnical investigation report, dated August 17, 1988 for the 9,000-square-foot administration building and adjacent parking area. The field investigation consisted of six exploratory test pits excavated to depths ranging from 5 to 11 feet in depth.

KGO also prepared a geotechnical investigation report, dated September 8, 1988 for prestressed concrete water reservoirs. Proposed grading at the site consisted of excavations for the tanks and access roads, and construction of an embankment around and over the tanks after the completion of construction. The field investigation consisted of twelve exploratory bucket auger borings ranging in depth from 21 to 71 feet and ten exploratory test pits ranging from 6 to 11 feet in depth.

1989: KGO prepared a geotechnical investigation report, dated March 21, 1989 for canyon fill and stockpile for two 16-million-gallon tanks. The investigation consisted of the excavation of 11 test pits. Based on the report, the project included placement and compaction of 180,000 cubic yards of fill into an existing south-southeast trending canyon and stockpiling an additional 170,000 cubic yards of fill material for the tanks.

1990: KGO prepared a geotechnical report of rough grading, dated February 15, 1990, for the canyon fill and stockpile for two 16-million-gallon tanks between November 1989 and January 1990. Based on the report, removals in the canyon area to the north of the tanks ranged from depths of 1 to 20 feet. Excavations for the proposed tank areas ranged in depth from 2 to 60 feet below grade. Based on the report, all removals were extended into competent bedrock and observed by the engineering consultant. Fill was then properly placed and benched into competent material. Subdrains were also placed along the canyon bottom.

KGO prepared a geotechnical report of rough grading, dated September 12, 1990 for the administration building north of the Baker Water Treatment Plant. Grading for the site required conventional cuts and fills on a hillside to construct a flat area for an administration building, surrounding parking lot, and access driveway. The maximum reported depth of compacted fill at the area was approximately 15 feet. The maximum height of cut and fill slopes was approximately 12 feet and 30 feet, respectively. The building pad area was overexcavated to 2 feet below finish pad grade and extended laterally five feet from the building perimeter. Fills were benched into bedrock where the slope exceeded 5:1 (horizontal:vertical).

1991: Coleman Geotechnical prepared a geotechnical report of rough grading, dated December 24, 1991 for perimeter tank backfill and final fill slope grading adjacent to the two 16-million-gallon reservoirs for the Baker Water Treatment Plant. Fill slopes were constructed with a 2:1 (horizontal:vertical) slope.

1993: Coleman Geotechnical prepared a slope evaluation and recommendation, dated March 15, 1993 for one of the 16 million gallon storage tanks for the Baker Water Treatment Plant. Heavy rains during January and February of 1993 caused a surficial landslide in the southeast facing fill slope. The report concluded that the failure was likely caused by the loss of shear strength between the topsoil fill and underlying engineered fill.

1.5 Field Exploration

Prior to the subsurface field exploration, a site reconnaissance was performed by a professional geologist from our staff to mark the locations of borings and trenches with consideration for access of heavy equipment, avoidance of known subsurface and above ground structures, and biologically sensitive habitats. The proposed locations of our borings and trenches were observed and approved by the project biologist. Underground Service Alert (USA) was notified to locate and mark existing underground utilities prior to commencement of field exploration. Additionally, the boring and trench locations were also cleared by IRWD personnel.

Our subsurface exploration was performed from April 1 through April 4, 2008 and included the drilling of three (3) 28-inch-diameter bucket auger borings, seven (7) 8-inch-diameter hollow-stem auger (HSA) borings, and excavating sixteen (16) test pits at the project site. The bucket auger borings were drilled to a maximum depth of 50 feet below the existing grade (bgs) at locations near the northern, eastern and western corners of the site, where proposed cut slopes exposing bedrock were planned. The borings were downhole logged upon completion of drilling. The HSA borings were drilled to depths ranging from 6.5 to 80.4 feet bgs at the northern portion of the site where the previously placed canyon fills were located. Test pits were excavated across the site to depths ranging from approximately 3 to 16.2 feet. The approximate locations of the borings and test pits are depicted on Plate 1, *Geotechnical Map*.

During drilling, bulk samples were obtained from the borings and test pits, and relatively undisturbed drive samples were obtained from HSA borings for geotechnical laboratory testing and evaluation. The drive samples were obtained utilizing a modified California drive sampler, 2-3/8-inch inside diameter (I.D.), 3-inch outside diameter (O.D.), driven 18 inches with a 140 pound automatic hammer dropping 30 inches in general accordance with ASTM Test Method D3550. Standard Penetration Tests (SPT) were also performed for HSA borings using a 24-inch long 1-3/8-inch I.D. and 2-inch O.D. split spoon sampler driven 18 inches with a 140-pound hammer dropping 30 inches in general accordance with ASTM Test Method D1586. The number of blow counts per 6 inches of penetration for HSA and bucket auger borings was recorded on the boring logs (Appendix A). However, hammer weight and drop for the bucket auger drilling do not conform to the above ASTM Standards.

Logging and sampling of the borings was conducted by a geologist from our firm. Each soil sample collected was reviewed in the field, and its description was entered on the boring logs in general conformance with the Unified Soil Classification System (USCS). After logging and sampling, borings were backfilled with spoils generated during exploration and the test pits were backfilled with soil cuttings and tamped with the bottom of the bucket. Samples from field exploration were transported to our laboratory for geotechnical testing.

1.6 Laboratory Testing

Laboratory tests were performed on representative samples to determine the geotechnical properties of the subsurface materials. The following laboratory tests were conducted on selected samples:

- In-situ moisture content and density (ASTM D2216 and ASTM D2937);
- Particle-size Analysis (ASTM D422);

- Percent passing No. 200 Sieve (ASTM D1140);
- Expansion Index (ASTM D4829);
- Maximum dry density and optimum-moisture content (ASTM D1557);
- Direct Shear (ASTM D3080);
- Consolidation (ASTM D2435);
- R-Value (ASTM D2844); and
- Corrosivity Suite – Sulfate, Chloride, pH and Resistivity (California Test Methods 417, 422 and 532/643).

The laboratory tests were performed in general conformance with ASTM and/or Caltrans procedures. The results of our laboratory tests are presented in Appendix C. The results of the in-situ moisture contents and dry densities of the ring samples are presented on our geotechnical boring logs (Appendix A).

2.0 GEOTECHNICAL FINDINGS

2.1 Geologic Setting

The project site is located within the Peninsular Ranges geomorphic province, in a transitional area between the foothills of the Santa Ana Mountains and the adjacent Tustin Plain. The Peninsular Ranges geomorphic province extends 900 miles southward from the Los Angeles Basin to the tip of Baja California and is characterized by elongated northwest-trending mountain ranges separated by sediment-floored valleys. The most dominant structural features of the province are the northwest trending fault zones, most of which die out, merge with, or are terminated by the steep reverse faults at the southern margin of the Transverse Ranges geomorphic province. Section 2.6 lists the known regional faults and their approximate distance from the site. North and northeast of the site are the northwest-trending Santa Ana Mountains, a large range, which has been uplifted on its eastern side along the Whittier-Elsinore Fault Zone, producing a tilted, irregular highland that slopes westward toward the sea (Yerkes et al., 1965).

Bedrock at the site is classified as belonging to the Oso Member of the Capistrano Formation. This formation is late Miocene to early Pliocene age marine sandstone. As observed within the excavations onsite, the sandstone is fine to medium grained, poorly cemented, oxidized, friable, and contains lenses of coarser grained sand and cobble to small boulder size, very well cemented concretions. A regional geologic map for this site is shown in Figure 2.

2.2 Subsurface Geologic Conditions

The borings and test pits encountered documented and undocumented artificial fill, Quaternary-aged alluvium and colluvium, and sandstone from the Oso member of the Capistrano Formation. For purposes of this report, documented artificial fill is further broken down into subgroups, Afc1 through Afc3 based on the reports generated upon completion of rough grading at those specific areas. These materials are described in the following subsections. Geologic cross sections across the site (Sections A-A' and B-B' on Plate 1) are presented on Plate 2.

2.2.1 Artificial Fill (Afc1 and Afc1a)

The composition of artificial fill materials located within the north central portion of the site encountered during our exploration consisted mainly of medium dense to dense, brown to grayish brown, dry to slightly moist, fine to coarse grained sand to clayey, silty sand with fine to coarse gravel and small cobbles composed of sandstone rock fragments. Fill depths range from approximately 1 to 75.2 feet below existing ground surface (see Plate 1).

These fills are documented in the referenced rough grading reports (KGO, 1990a and Coleman, 1991b). Based on these reports, fill material was derived from grading activities associated with the construction of the two 16-million-gallon water tanks. In addition, a portion of the fill material was imported from a cut area on an adjacent tract located northwest of the site.

2.2.2 Artificial Fill (Afc2)

The artificial fill material within the area of the water treatment facility tanks is documented by Coleman Geotechnical (1991a). This fill material was not investigated during Leighton's current subsurface exploration, however, the fill material is expected to consist generally of engineered fill derived from native silty sands and sand from the cut and stockpile areas located northeast of the tank sites (Coleman, 1991a).

2.2.3 Artificial Fill (Afc3)

The artificial fill material within the area of the administration building for the water treatment plant is documented in a rough grading report (KGO, 1990b). This fill material was not encountered in our boring, BA-2, located southwest corner of the parking lot. Based on KGO's report, paved areas were overexcavated to bedrock, scarified, moisture conditioned, and compacted. Fill materials in this area are expected to generally consist of onsite derived sand and silty sands with a trace of clay and concretions (KGO, 1990b).

2.2.4 Undocumented Artificial Fill (Afu)

Based on the conceptual site plan provided to us (BLA, 2008), the area in the southern portion of the site is proposed for a private park. Undocumented fill was encountered in this area to depths of 16 feet and greater. Undocumented fill overlying in-place alluvium is present in the northeast portion of the property. In addition, undocumented fill over colluvium was encountered in the southwestern portion of the site. The undocumented fill material generally consists of loose, dark brown to grey, dry to moist, fine to coarse grained sand to silty clayey sand, with fine to coarse gravel, cobbles and small boulder sized concretionary sandstone and concrete debris.

2.2.5 Quaternary Alluvium (Qal)

Quaternary alluvium, as encountered at the test pit location T-5, consists generally of crudely interfingered loose zones of light yellowish brown to orange-greyish brown, moist, fine to coarse grained sand with very well oxidized gravel-sized sandstone connections.

2.2.6 Quaternary Colluvium (Qcol)

Colluvium, as encountered at the test pit locations T-11 through T-14, consists generally of loose, dark brown, fine to coarse grained silty to clayey sand to sandy clay with occasional cobble sized sandstone concretions.

2.2.7 Bedrock: Capistrano Formation: Oso Member (Tco)

Bedrock encountered in the boring and test pit locations belongs to the Oso Member of the Capistrano Formation. The sandstone is generally hard, medium grey to tan and orange brown where oxidized, poorly cemented, friable, fine to coarse grained, and contains cobble sized concretions to fine grained, hard, grayish brown micaceous silty sandstone.

2.3 Soil Characteristics

Important soil characteristics obtained from our laboratory test results that are relevant to the proposed developments are summarized below.

2.3.1 Expansion Potential

Laboratory testing of a selected soil sample indicates low expansion potential (per ASTM D4829) with tested EI value of 21. Onsite soils are anticipated to be primarily very low to low in expansion.

2.3.2 Compressibility

Our review of the consolidation test results of the fill soils in the canyon fill area (Afc1) at different depths up to 45 feet below the existing grade indicates that these soils have relatively low compressibility when subjected to the anticipated overburden pressure and slight collapse potential upon inundation.

2.3.3 Corrosivity Potential

In general, soil environments that are detrimental to concrete have high concentrations of soluble sulfates and/or pH values of less than 5.5. Section 4.3 of ACI 318 (ACI, 2005), as referred in CBC, 2007, provides specific guidelines for the concrete mix-design when the soluble sulfate content of the soil exceeds 0.1 percent by weight or 1,000 parts per million (ppm). The minimum amount of chloride ions in the soil environment that are corrosive to steel, either in the form of reinforcement protected by concrete cover or plain steel substructures, such as steel pipes, is 500 ppm per California Test 532.

For screening purposes, three representative onsite soil samples within upper 10 feet of the existing grade were tested for corrosion suite (soluble sulfate, chloride, pH and resistivity). The summary of the test results and corresponding hazard levels are presented in the following Table 1 and test results are included in Appendix C. These limited test results indicate that the subsurface soils have "negligible" soluble sulfate contents and low chloride contents. However, the soils are considered to have moderate to severe corrosion potential to buried ferrous metal.

Table 1 – Summary of the Corrosivity Test Results

Test Parameter	Test Results	General Classification of Hazard
Water-soluble sulfate content	45 to 79 ppm	Negligible sulfate exposure to buried concrete (per ACI 318)
Water-soluble chloride Content	44 to 68 ppm	Non-corrosive to buried concrete (per Caltrans Specifications)
pH	8.18 to 8.32	Alkaline, relatively passive to buried metals
Minimum resistivity (in saturated condition)	2,024 to 4,680 ohm-cm	Moderately to severely corrosive to buried ferrous pipes (per ASTM[1])

[1] ASTM STP 1013 titled *Effects of Soil Characteristics on Corrosion* (February, 1989).

2.4 Groundwater Conditions

Groundwater was not encountered at any of our borings or test pits to the maximum depth of 80.4 feet. Based on the CDMG report (CDMG, 2001), historically high groundwater table is estimated to be at a depth of approximately 10 to 20 feet below ground surface within the canyon bottoms. Subdrains placed ruing the previous grading within the onsite canyon (KGO, 1990b), are expected to control any subsurface water infiltrating the previously placed fill material along bedrock/fill contacts. During grading for the Domestic Storage Tank (KGO, 1987b), seepage was encountered within the north and west sides of the canyon during removals. Based on the above information and the relatively granular nature of the bedrock and the reported subdrains, groundwater is not expected to be a constraint to development within the site.

2.5 Mass Movements

No landslides are known to be located at the site or were observed during our field exploration. However, based on the report prepared by Coleman Geotechnical, (Coleman, 1993) a fill slope located adjacent to one of two 16 million gallon storage tanks for the Baker Water Treatment Plant experienced surficial failure. This slope failure was attributed to heavy rains during the period March 1993. The surficial failure was less than 3 feet thick and consisted generally of topsoil overlying engineered fill.

2.6 Principal Seismic Hazard

The site is not located within an Alquist-Priolo Special Studies Zone (Hart, 1992) and no active faults are known to underlie the site. The principal seismic hazard that could affect the site is ground shaking resulting from an earthquake occurring along any one of several major active faults in the region. The known regional faults that could produce the most significant ground shaking at the site include the Chino-Central Avenue (Elsinore segment), Elsinore-Glen Ivy, Newport-Inglewood (Offshore segment), and Elsinore-Whittier faults, located approximately 17.2, 18.8, 20.0, and 21.9 kilometers from the site, respectively.

The intensity of ground shaking at a given location depends primarily upon the earthquake magnitude, the distance from the source, and the site response characteristics. The peak horizontal ground accelerations (PHGA) for the site were estimated using probabilistic seismic hazard analysis. These analyses require information regarding fault geometry, the magnitude of the earthquake the fault can produce, and the attenuation relationship. The computer program FRISKSP (Blake, 2000) was used for the analyses based on an averaging of attenuation relationships by Abrahamson and Silva (1997), Campbell (1997), and Sadigh et al. (1997) for alluvial soils.

The results of the analyses suggest a PHGA of approximately 0.34g at the site for a hazard level of 10 percent probability of exceedance in 50 years (recurrence interval of 475 years) and approximately 0.57g for 2 percent probability of exceedance in 50 years (recurrence interval of 2,475 years). The latter hazard level corresponds to the Maximum Considered Earthquake (MCE) event per CBC, 2007.

2.7 Secondary Seismic Hazards

Secondary seismic hazards induced by an earthquake that are not expressions of fault rupture at the surface but involve loss of strength of the underlying material include liquefaction, lateral spreading, lurching, seismic settlement, earthquake induced flooding and earthquake induced landsliding. Each of these is discussed in detail below.

Liquefaction: Liquefaction is a seismic phenomenon in which loose, saturated, fine-grained granular soils behave similarly to a fluid when subjected to high-intensity ground shaking. Liquefaction occurs when three general conditions exist: 1) shallow groundwater; 2) low density, fine, clean sandy soils; and 3) high-intensity ground motion.

Effects of liquefaction on level ground can include sand boils, settlement, and bearing capacity failures below structural foundations.

A review of the State of California Seismic Hazard Zones Map for the El Toro Quadrangle (CDMG, 2001) indicates that the project site is not within a liquefaction hazard zone as shown in Figure 3, *Seismic Hazard Map*. Due to shallow bedrock conditions and deeper groundwater levels, liquefaction is not expected to be a significant consideration for the proposed development.

Lateral Spreading: Seismically-induced lateral spreading involves lateral movement of earth materials as a result of liquefaction. Lateral spreading differs from slope failure in that it involves lateral movement in areas of low topographic gradient to level ground due to lack of lateral support for liquefiable horizons in the soil. Lateral spreading is often manifested by near-vertical cracks with predominantly horizontal movements of the soil mass involved. The potential for lateral spreading to occur as a result of liquefaction is considered to be low due to the presence of bedrock within the subject site and the adjacent parcels.

Lurching: Lurching is the relative displacement of adjacent land surfaces during an earthquake. As the seismic motion encounters a cliff or bluff, a stream bank, or even a fill slope at nearly right angles it may cause displacement of the material in the unsupported direction (Richter, 1958). Lurching may also be caused by liquefaction of a zone beneath the otherwise intact surface. Visible evidence of lurching includes ground cracking and fissuring generally in a relatively parallel fashion to a stream bank or slope face. Ground cracking caused by lurching is not related to the fault rupture. Ground lurching may occur on the slopes within the borders of the site, depending on the direction of seismic waves.

Seismic Settlement: Seismic densification of dry soils is a phenomenon in which loose, dry soils, primarily sands and silty sands densify and settle when subjected to earthquake shaking. In Southern California, evidence of seismically-induced densification and resultant settlement of dry soils has been observed in the 1971 San Fernando and 1994 Northridge Earthquakes. The granular soils underlying the site are generally moist and have a low collapse potential; therefore, the potential for seismic densification is considered low.

Earthquake-Induced Landsliding: Seismically-induced landslides and other slope failures are common occurrences during or soon after earthquakes. Review of the State of California Seismic Hazard Zones Map for the El Toro Quadrangles (CDMG, 2001) indicates that the western portion of the site is located within earthquake-induced landslide hazard zone as shown in Figure 3, *Seismic Hazard Map*. The potential for earthquake induced landslides impacting the proposed site is currently considered to be moderate. However, proposed grading will mitigate this impact.

Earthquake-Induced Flooding: Earthquake-induced flooding is caused by dam failures or other water-retaining structure failures as a result of seismic shaking. Two 16-million-gallon reservoirs are located on the southern end of the project site. These reservoirs are located down-gradient from the proposed developments, therefore, the potential for earthquake-induced flooding is considered to be low for the proposed development located to the north of reservoir. The potential for earthquake-induced flooding, however, does exist for the southern portion of the site at the proposed park site if the existing reservoirs failed during an earthquake.

Seiches: Seiches are waves generated in enclosed bodies of water in response to ground shaking. Two large bodies of water are located in the immediate vicinity of the park site. The potential for seiches does exist for the proposed park site if the existing reservoirs failed during an earthquake.

Tsunamis: Tsunamis are waves generated in large bodies of water by fault displacement or major ground movement. Based on the inland location of the site, tsunamis do not pose a threat to this site.

3.0 FINDINGS AND CONCLUSIONS

A summary of our findings and conclusions related to the proposed developments at the site are presented below:

- Shallow bedrock and bedrock outcroppings of Capistrano Formation (Tco) were encountered across the site. Depths of previously placed canyon fill overlying bedrock in the north-central portion of the site varied from approximately 11 to 75 feet. Elsewhere within the site boundaries, fills, both documented and undocumented, alluvium (Qal), and colluvium (Qcol) overlie bedrock.

- Groundwater was not encountered in the borings or trenches to the maximum explored depth of 80.4 feet.

- Laboratory testing of a selected soil sample indicates a low expansion potential (per ASTM D4829). Onsite soils are anticipated to contain primarily a low to very low expansion potential.

- Documented fill soils at the site consist primarily of sandy soils. Laboratory test results indicate that these fill soils will have relatively low compressibility when subjected to the anticipated overburden pressure and slight collapse potential upon inundation.

- Soil corrosivity test results indicate that the onsite soils contain "negligible" soluble sulfate contents and low chloride contents for buried concrete. However, these soils contain moderate to severe corrosion potential to buried ferrous metals (e.g., utility pipes).

- Minimal removal of documented fills (Afc1 and Afc3) on the order of 3 to 5 feet are recommended. Elsewhere, removals of the fills, alluvium, and colluvium should extend to the underlying competent bedrock which may be on the order of approximately 10 feet to greater than 16 feet below the existing grade.

- Onsite soils free of organics and oversized particles (3 inches or smaller in the maximum dimension) are suitable to be used as engineered fill. Oversize particles to be generated from the proposed cut into bedrock as well as currently existing ripraps consisting cobble sized sandstone in a detention basin along the central portion of the site may be used in the fills provided recommendations for rock fill specifications in this report (Section 4.2.7).

- Based on the conceptual site plan, cut slopes into bedrock of maximum height of approximately 45 feet – primarily south and west facing – are planned across the site to facilitate road access and attain the proposed pad elevations. Bedrocks at the proposed cut slope face are anticipated to contain poorly cemented cobble sized concretions within friable sandy matrix which may be susceptible to severe erosion over time. In order to maintain surficial stability, remedial measures, such as proper landscaping and/or erosion control matting, may be required.

- The gross stability of proposed south and west facing cut slopes may be affected by unfavorable bedding orientations at certain locations. These cut slopes may require some form of stabilization.

- Grading along the western boundary of the site may require encroachment onto the adjacent property.

- The proposed park area in the southern portion of the site is underlain by undocumented fill up to a depth 16 feet. Remedial excavation for structures in the park area, if any, can be evaluated once final design plans are provided

- The proposed structures may be supported on conventional shallow foundation.

Based upon this study, we conclude that the proposed development is feasible from a geotechnical standpoint, provided that the recommendations presented in this report are properly incorporated in the design and construction of the project.

4.0 GENERAL RECOMMENDATIONS

4.1 General

The recommendations presented in the subsequent sections of this report have been formulated based upon the conceptual site plan (BLA, 2008) and our findings from field exploration and laboratory test results. Variances in the subsurface soil conditions described herein may be encountered during construction which should be brought to our attention to evaluate the conditions and the impact upon these recommendations. Based upon review of the final grading plan as well as field evaluation of the exposed subsurface conditions during construction, revisions to the following recommendations may be necessary.

4.2 Earthwork

Earthwork is anticipated to consist of site preparation, remedial excavations, and grading for the proposed developments. Earthwork should be accomplished under the observation and testing of the geotechnical consultant and their representatives in accordance with the recommendations contained herein and the current grading ordinance of the city of Lake Forest.

4.2.1 Site Preparation

Prior to construction, the site should be stripped of vegetation, debris, any deleterious materials, the existing IRWD building structure and pavements in the central western portion of the site. Currently, riprap consisting of cobble sized sandstone covers a detention basin along the central portion of the site. These oversize materials need to be removed and disposed off the site or may be used in the fill as discussed in Section 4.2.7. Any existing utility and irrigation lines should be removed if they interfere with the proposed construction. The cavities resulting from removal of existing building foundations and utility lines should be removed to competent material and properly backfilled and compacted.

4.2.2 Overexcavation

Depending on the subsurface soil conditions, we recommend the following remedial excavations that are specific to the encountered geologic units (see Plate 1):

- Fill Soils, Afc1 – At a minimum, the upper 3 feet of the fill soils in the northern portion of the site should be removed.

- Fill Soils, Afc1A and Afc2 - These fill soils in the central and southeastern portions of the site, within the canyon below the debris basin and along the northeast trending drainage, should be removed up to the underlying competent bedrock. Depth of removals in these areas may be on the order of approximately 10 feet to greater than 16 feet below the existing grade.

- Fill Soils, Afc3 – At a minimum, the upper 5 feet of these fill soils in the central western portion of the site which were placed during construction of the existing IRWD Administration Building and pavements at the site - should be removed.

- Undocumented Fill (Afu), Alluvium (Qal), and Colluvium (Qcol) –These soils should be removed to the competent bedrock in proposed structural areas.

Upon remedial excavation, minimum depths of compacted fill for the proposed developments are provided below.

Building Footprints: Building pad areas in general should have a minimum 4 feet of compacted fill underneath the finish pad grade. In shallow bedrock areas where cuts are proposed, the building may be supported on competent bedrock. In order to reduce the potential for differential settlement in areas of transition (fill-bedrock), we recommend that compacted fill below the pad grade in the bedrock portion of the pad be a minimum depth of 4 feet or one half of the maximum fill depth across the pad, whichever is deeper. The lateral limit of overexcavation and compacted fill should be established at a minimum distance of 5 feet horizontally beyond the building footprint.

Pavements and Concrete Flatwork: For pavements and at-grade, exterior concrete flatworks (e.g., sidewalks, courtyards, pool decks, trash enclosures, etc.), a minimum of 24 inches compacted fill should be placed below the design finish grade except the areas where competent bedrock is exposed. Laterally, these compacted fills should extend a minimum of 24 inches beyond the pavement and flatwork edges.

Fill Slopes: Proposed fill slopes and potential stabilization fills for the proposed cut slopes at the site should be constructed with the appropriate key section with benching into competent onsite soils or bedrock. Preliminary guidelines for keys and benches are shown in Figure 4, *Keying and Benching Standard Details*.

4.2.3 Fill Placement and Compaction

Exposed subgrade soil surfaces, including all excavation or removal bottoms, should be observed by a representative of the geotechnical consultant prior to placement of fill. Competent excavation bottoms should be scarified to a minimum depth of 8 inches, moisture-conditioned to above the optimum-moisture content and then compacted to a minimum of 90 percent relative compaction (per ASTM D1557)

All fill soil should be placed in loose lifts of 6 to 8 inches in thickness, moisture-conditioned to slightly above the optimum-moisture content, and compacted to a minimum of 90 percent relative compaction, as determined by ASTM Test Method D1557. Aggregate base in the pavement areas should be compacted to a minimum of 95 percent relative compaction (per ASTM D1557).

4.2.4 Fill Materials

The onsite soil free of organics, debris and oversize particles (e.g., cobbles, boulders, rubble, etc.) larger than 3 inches in the largest dimension is suitable to be used as fill. Oversize particles larger than 3 inches in the largest dimension may be used in the fill as discussed in Section 4.2.7.

Import soils and/or borrow sites should be evaluated by the geotechnical consultant prior to importation. Import soils should be uncontaminated, granular in nature, free of organic material, have very low expansion potential (with an Expansion Index less than 21 per ASTM D4829) and have a low corrosion impact to the proposed improvements.

4.2.5 Subdrainage

Subdrains will be necessary in canyon fills where fills exceed 10 feet in thickness and in fill-over-cut keyways. Fills generally become saturated at or near the contact with impermeable bedrock and the subdrains should outlet this excess water to suitable discharge areas. Schematics showing subdrain details are provided in Figure 5, *Canyon Subdrain Standard Details*. The connection between the perforated and non-perforated pipe should be sealed with a minimum 6-inch thick, concrete cut-off wall placed a minimum of 2-feet beyond the perimeter of the gravel "burrito". All outlets should be protected with a concrete apron and cover. As-built subdrain locations should be surveyed by the project civil engineer and land surveyor.

4.2.6 Rippability

Based on our findings from field exploration, we anticipate that general bedrock excavation to depths of up to 45 feet will be rippable with conventional heavy earth moving equipment in good operating condition (Caterpillar D9L or greater with single barrel ripper and rock teeth). However, for most excavations over 10 to 15 feet of depth into bedrock, localized areas of heavy ripping should be anticipated.

4.2.7 Rock Fills

We anticipate that the relatively deep cuts into bedrock will generate oversized rock. Within the upper 4 feet of finish grade or within utility trenches, fill soils should not contain rock greater than 3 inches in the largest dimension in order to facilitate foundation and utility trench excavation. For fill soils between 4 and 10 feet below finish grade, the fill may contain rock up to 8 inches in the largest dimension and should be mixed with sufficient soil to eliminate voids. Below a depth of 10 feet from finish grade, rocks up to the largest dimension of 24 inches may be incorporated into the fill provided adequate fines to fill all voids are present. Rocks greater than 24 inches in the largest dimension may be placed on a case-by-case basis in non-structural fill areas. The outer 10 feet of all fill slopes (measured vertically from the slope face) should not contain rocks greater than 8 inches. A schematic of oversize rock placement is presented in Figure 6, *Oversize Rock Disposal Standard Details*.

We anticipate that a minimum of approximately 35 to 40 percent fines will be necessary to adequately fill all voids. Soil filling voids in rock fills should be flooded during placement with a sufficient amount of water to wash soil into all voids. Material filling voids should be placed to a minimum of 90 percent relative compaction (per ASTM D1557).

4.2.8 Shrinkage and Bulking

The volume change of excavated onsite soils upon recompaction is expected to vary with materials, density, in-situ moisture content, location and compaction effort. The in-place and compacted densities of soil materials vary and accurate overall determination of shrinkage and bulking is difficult to make. Based on our field exploration and laboratory test results, we anticipate the following estimates for shrinkage and bulking (when recompacted to an average of 92-percent relative compaction per ASTM D1557) for different geologic units:

Table 2 – Summary of Shrinkage and Bulking Estimates

Geologic Unit	Shrinkage (%)	Bulking (%)
Documented Fill, Afc1	0 - 5	-
Documented Fill, Afc1A, Afc2, and Afc3	5 - 10	-
Undocumented Fill, Afu; Alluvium, Qal; Colluvium, Qcol	10 - 15	-
Bedrock, Tco	-	0 - 5

Mapping of the above geologic units is shown in Plate 1.

4.3 Seismic Design Parameters

This site is not located within a currently designated Alquist-Priolo Earthquake Fault Zone. However, strong ground shaking due to seismic activity is anticipated at the site. Based on subsurface geologic conditions, the project site may be classified as Site Class C or D per Section 1613.5.2 of CBC, 2007. The areas where engineered fill depths overlying bedrock will be 10 feet or less may be classified as Site Class C. The area where the depth of fill soils is greater than 10 feet may be classified as Site Class D. Site specific seismic design parameters for Site Class C and D according to CBC, 2007 are presented in Table 3 and 4 below, respectively.

Table 3 - Seismic Design Parameters
(Fill depth over bedrock 10 feet or less)

Categorization/Coefficient	Design Value
Site Class	C
Mapped MCE[1] (5% damped) spectral response acceleration parameter at short period (0.2 sec), S_S	1.40g
Mapped MCE[1] (5% damped) spectral response acceleration parameter at long period (1.0 sec), S_1	0.50g
Short period (0.2 sec) site coefficient, F_a	1.0
Long period (1.0 sec) site coefficient, F_v	1.3
Design (5% damped) spectral response acceleration parameter at short period (0.2 sec), S_{DS}	0.94g
Design (5% damped) spectral response acceleration parameter at long period (1.0 sec) sec, S_{D1}	0.43g

[1] MCE is the Maximum Considered Earthquake (see Section 2.6)

Table 4 - Seismic Design Parameters
(Fill depth greater than 10 feet)

Categorization/Coefficient	Design Value
Site Class	D
Mapped MCE[1] (5% damped) spectral response acceleration parameter at short period (0.2 sec), S_S	1.40g
Mapped MCE[1] (5% damped) spectral response acceleration parameter at long period (1.0 sec), S_1	0.50g
Short period (0.2 sec) site coefficient, F_a	1.0
Long period (1.0 sec) site coefficient, F_v	1.5
Design (5% damped) spectral response acceleration parameter at short period (0.2 sec), S_{DS}	0.94g
Design (5% damped) spectral response acceleration parameter at long period (1.0 sec) sec, S_{D1}	0.50g

[1] MCE is the Maximum Considered Earthquake (see Section 2.6)

Based on the short and long period response accelerations, S_{DS} and S_{D1}, and the anticipated occupancy category II (per Section 1604.5 of CBC, 2007), the proposed building structures are determined to be in seismic design category D (per Section 1613.5.6 of CBC, 2007). The above parameters should be considered as the minimum for the seismic analysis of the subject site. Additional seismic analyses may be necessary based on structural requirements.

4.4 Foundation Design

Based upon our findings from subsurface exploration and laboratory test results, the proposed residential and civic center building structures may be supported by conventional spread footings (continuous strip and/or isolated column) bearing on a zone of newly placed, properly compacted fill or competent bedrock. Preliminary design parameters for conventional spread footings are described in the following:

Minimum Footing Dimensions and Embedment: Continuous (strip) footings for up to two-story buildings should be embedded a minimum of 18 inches while the continuous footings for three- to four-story buildings should be should be embedded a minimum of 24 inches. Isolated column footings should be embedded a minimum of 24 inches. These minimum embedments are measured below the lowest adjacent grade that is considered as the top of interior slabs-on-grade or the finished exterior grade, excluding landscape topsoil, whichever is lower. Footings located adjacent to utility trenches or vaults should be embedded below an imaginary 1:1 (horizontal:vertical) plane projected upward and outward from the bottom edge of the trench or vault, towards the footing. Continuous/strip footings should have a minimum width of 18-inches, while column footings should have a minimum width of 24 inches.

Allowable Vertical Bearing: For footings founded on newly placed, properly compacted fill soil, an allowable vertical bearing capacity of 1,500 pounds per square foot (psf) may be used for design for a minimum embedment of 18 inches below the lowest adjacent grade as defined above. For footings founded on the competent bedrock, an allowable vertical bearing capacity of 2,000 psf may be used for the design for a minimum embedment of 18 inches below the lowest adjacent grade as defined above. These allowable bearing pressures may be increased by 500 psf for each additional foot of embedment, to a maximum vertical bearing value of 3,500 psf.

The above bearing values may be increased by one-third when considering short-term seismic or wind loads.

Lateral Loads: Lateral loads may be resisted by friction between the footings and the supporting subgrade and passive pressures acting against foundations poured neat against properly compacted fill. A maximum allowable frictional resistance of 0.35 may be used for design of concrete structures poured on properly compacted fill. We recommend that an allowable passive pressure based on an equivalent fluid pressure of 250 pounds-per-cubic-foot (pcf) be used in design. These friction and passive values have already been reduced by a factor-of-safety of 1.5. The lateral passive resistance is taken into account only if it is ensured that the soil against embedded structures will remain intact with time. When combining passive pressure and friction for computing resistance to lateral loads, no reduction is needed to any of these components.

Settlement Estimates: Detailed structural loadings for the proposed building structures were not available to us during preparation of this report. Existing compacted fill soils in the canyon fill area (Map symbol Afc_1, See Plate 1) consist primarily of sandy soils. Since these fills have been in place over a relatively long period of time, settlements due to fill overburden are anticipated to have occurred. Based on anticipated structural loads, the proposed building structures may be designed for a total settlement of 1 inch and differential settlement of ½ inch over a horizontal distance of 30 feet provided site grading follow the recommendations of this report. The above settlements and angular distortions include both the static and dynamic settlements. Since settlement is a function of footing size and contact bearing pressure, differential settlement can be expected between adjacent columns or walls where a large differential loading condition exists. These settlement estimates should be reviewed by the geotechnical consultant when foundation plans and loads for the proposed structures become available.

4.5 Foundation Setbacks

Based on our findings of the site soil conditions, we recommend that structural foundations on top of descending slopes be set back a minimum horizontal distance of 5 feet or H/3 (H is the height of slope in feet), whichever is greater, but not exceeding a maximum of 40 feet. Setback of structural footings from the toe of ascending slopes should follow Section 1805.3.1 of CBC, 2007. These setbacks are measured horizontally from the bottom of the leading edge of the footing to the slope face.

4.6 Lateral Lot Extension

The magnitude of lateral lot extension ("lot stretching") due to slope creep is a function of a number of factors including slope height, aspect, irrigation regime, and composition of the slope. As with all fill slopes, some degree of slope creep/lot stretching should be expected for this site. Slope creep and lot stretching are expected to be particularly prevalent within approximately 5 to 15 feet of the crest of descending slopes. The effects of slope creep and lot stretching are considerably less within the main portion of the lot and are not expected to influence the proposed residential buildings. Based on our experience, with consideration to the lot length, site fill materials, the depth of fill and height of the descending slope, we estimate that long-term lateral extension of any lot above a slope that is higher than approximately 20 feet will be on the order of 1 inch within 10 feet from the slope crest and less than 1 inch beyond this 10-foot zone. The lateral extension value for slopes less than 20 feet is anticipated to be less than 1 inch. The actual amount of movement will also be a function of the homeowners and/or homeowner association's irrigation practices. Yard improvements such as decorative walkways, patios, and other landscaping features should be constructed with flexibility to accommodate the effects of creep/lot stretching. Concrete flatwork and structures within the foundation setback zone should be designed and constructed in accordance with the recommendations presented in this report.

4.7 Slabs-on-Grade (Building Floors)

Slab-on-grade floors utilized with conventional foundations should be designed with a minimum thickness as indicated by the project structural engineer consistent with a modulus of subgrade reaction of 150 pounds-per-cubic-inch (pci) and reinforced in accordance with the structural engineer's recommendations. A slip-sheet or equivalent should be used if crack-sensitive floor coverings (such as ceramic tiles, etc.) are to be placed directly on the concrete slab-on-grade.

Within areas of interior slab-on-grade floors where moisture sensitive flooring will be placed, we recommend placement of a minimum of 10-mil thick Visqueen (or equivalent) membrane as moisture retarder under the slab. To facilitate uniform curing of concrete and to provide protection of this membrane during construction, clean sand (Sand Equivalent of 30 or greater (California Standard Test Method 217)), minimum 2 inches thick, should be placed on above and below this membrane prior to placement of concrete.

Moisture retarders do not completely eliminate moisture vapor movement from the underlying soils up through the slabs or from the unbonded water in the concrete. To reduce moisture vapor emissions that may result in delamination and other tile damage, we suggest the following, only for areas where moisture sensitive floor coverings are anticipated:

- *Concrete:* A concrete mix design with a low water to cement ratio (less than 0.45) may be used. Water should not be added to this mix during placement. The concrete should be cured in a manner to eliminate slab curling.

- *Post Curing:* Before floor coverings are placed, any bond breaker coating and all other contaminates should be removed from the slab-on-grade surface. Shot blasting the slab surface may be required. Once the building has been enclosed, and environmental controls (heating and air conditioning) are installed and operational, the slab-on-grade should then be tested for moisture vapor emission, in accordance with ASTM E 1907-97.

- *Floor Coverings:* Floor coverings and adhesives should be compatible, and the manufacture's requirements should be followed. The tested moisture vapor emission rate (MVER) should be below the specified rate for the floor covering products used (e.g., MVER<5), before the product is placed. Expansion gaps hould be provided where floor tiles are placed adjacent to walls under molding, and along appropriate grids for large expanses of tile.

Cracking of concrete is normal as it cures due to drying and shrinkage, and should be expected. However, cracking is often aggravated by a high water/cement ratio, 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 slump concrete can reduce the potential for shrinkage cracking. Concrete placement during hot weather should be minimized due to the potential for slab curling.

4.8 Concrete Flatwork

To reduce the potential for uncontrolled cracking, all exterior concrete flatworks on grade (e.g., sidewalks, courtyards, pool decks, trash enclosures, etc.) should be a minimum of 4 inches thick and provided with construction or weakened plane joints at frequent intervals (e.g., every 6 feet or less). Reinforcement of the concrete should also be considered to further reduce unsightly cracking.

4.9 Retaining Walls

Although not noted on the conceptual grading plan (HPS, 2008), we anticipate that retaining walls will be planned for the project. Any type of retaining walls should be designed for lateral earth pressures. The magnitude of these pressures depends on the amount that the wall can yield horizontally under load. If the wall can yield enough to mobilize full shear strength of backfill soils, then the wall 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. Retaining walls backfilled with very low expansive soils (EI values less than 21 per ASTM D4829) should be designed using the following equivalent fluid pressures:

Table 5 - Retaining Wall Design Earth Pressures (Static, Drained)

Loading Conditions	Equivalent Fluid Pressure (pcf)	
	Level Backfill	2:1 (H:V) Backfill
Active	39	59
At-Rest	59	90
Passive[1]	250	-

[1] Allowable passive resistance. Maximum value not to exceed 2,500 psf at depth.

Unrestrained (yielding) cantilever walls should be designed for the active equivalent-fluid pressure value provided above for very low to low expansive soils that are free draining. In the design of walls restrained from movement at the top (non-yielding) such as utility vaults, wall corners, the at-rest equivalent fluid pressure should be used.

In addition to the above lateral forces due to retained earth, surcharge loads behind the retaining wall on or in the backfill within a 1:1 (horizontal:vertical) plane projection up and out from the retaining wall toe, should be considered as lateral and vertical surcharge. Unrestrained (cantilever) retaining walls should be designed to resist one-third of these surcharge loads applied as a uniform horizontal pressure on the wall. Restrained wall sections should also be designed to resist an additional uniform horizontal-pressure equivalent to one-half of uniform vertical surcharge-loads.

Retaining wall foundations should be at least 18 inches wide, embedded a minimum of 18 inches below the lowest adjacent grade, and bearing on a minimum of 2 feet of properly compacted fill soils (see Section 4.2.3). Allowable vertical bearing and maximum allowable frictional resistance for retaining wall foundations should follow the recommendations in Section 4.4 of this report. Non-standard wall designs should be reviewed by Leighton prior to construction to verify that the proper soil parameters have been incorporated into the wall design.

All retaining walls should be provided with appropriate drainage. The outlet pipe should be sloped to drain to a suitable outlet. Typical wall drainage design is illustrated in Figure 7, *Retaining Wall Backfill and Subdrain Details*, for non-expansive backfill. Wall backfill should be compacted by mechanical methods to a minimum of 90 percent relative compaction (ASTM D1557). Walls should not be backfilled until wall concrete attains the 28-day compressive strength and/or as determined by the Structural Engineer that the wall is structurally capable of supporting backfill. Lightweight compaction equipment should be used, unless other wise approved by the Structural Engineer.

4.10 Temporary Excavations

All temporary excavations, including utility trenches, retaining wall excavations, and other excavations should be performed in accordance with project plans, specifications and all Occupational Safety and Health Administration (OSHA) requirements.

Excavations 5 feet or deeper should be laid back or shored in accordance with OSHA requirements before personnel or equipment are allowed to enter. OSHA allows the sides of unbraced excavations, up to a maximum height of 20 feet, to be cut to a ¾H:1V slope for Type A soils, 1H:1V for Type B soils, and 1½H:1V for Type C soils. Shoring, if needed, can be designed using the appropriate lateral earth pressures provided in Section 4.8.

The onsite bedrock within the planned excavation depths generally conform to OSHA soil Type B while the onsite soils overlying bedrocks are anticipated to conform to OSHA soil Type C. OSHA regulations are applicable in areas with no restriction of surrounding ground deformations.

No surcharge loads should be permitted within a horizontal distance equal to the height of cut or 5 feet, whichever is greater from the top of the slope, unless the cut is shored and these surcharge loads are considered in the design of the shoring system. Excavations that extend below an imaginary plane inclined at 45 degrees below the edge of any adjacent existing site foundation should be properly shored to maintain support of the adjacent structures.

During construction, the soil conditions should be regularly evaluated to verify that conditions are as anticipated. The contractor should be responsible for providing the "competent person" required by OSHA standards to evaluate soil conditions. Close coordination between the competent person and the geotechnical engineer should be maintained to facilitate construction while providing safe excavations.

Typical cantilever shoring where deflection of the shoring will not impact the performance of adjacent structures may be designed based on the active fluid pressures 39 pcf. Braced or tie back shoring is recommended in areas where the shoring will be located close to existing structures to limit shoring deflections. Braced shoring can be designed using a uniform rectangular soil pressure of 25H psf, where H is equal to the depth of the excavation being shored. Braces should be installed and pre-loaded as the excavation progresses to reduce shoring deflections.

4.11 Slope Stability

Based on the conceptual site plan (HPS, 2008), cut slopes into bedrock of maximum height of approximately 45 feet – primarily south and west facing – are planned across the site to facilitate access roads and to attain the proposed pad elevations. Bedrock exposed at the proposed cut slope face is anticipated to contain poorly cemented cobble sized concretions within friable sandy matrix which may be susceptible to severe erosion over time. In order to maintain surficial stability, remedial measures such as proper landscaping and/or erosion control matting, may be required. In addition, the gross stability of the proposed south and west facing cut slopes may be affected by unfavorable bedding orientations at certain locations. These cut slopes may require some form of stabilization. Any slope section, cut or fill, that is steeper than 2:1 (horizontal:vertical) should be analyzed for gross stability.

Cut and fill slopes should be provided with appropriate surface drainage features and landscaped with drought-tolerant, slope-stabilizing vegetation as soon as possible after grading to reduce the potential for erosion.

4.12 Pavement Design

4.12.1 Asphalt Concrete Pavements

Our laboratory tests of two representative bulk samples – one consisting of weathered sandstone at depths of 3 to 6 feet and the other from within upper 5 feet of the canyon fill area – indicated R-values of 66 and 50, respectively. Due to wide variations of near surface soil types – from colluvium to weathered bedrock – and anticipating blending of these materials at different proportions during site grading for pavement subgrades, we utilized an average R-value of 35 for preliminary design purposes. Considering this R-value and following the Orange County Highway Design Manual, minimum asphalt pavement sections for different Traffic Indices (TIs) are listed in Table 6 below.

Table 6 - Asphalt Pavement Section Thickness

Traffic Index (TI)	Asphalt Concrete (inches)	Aggregate Base[1] (inches)
6.0 or less	4.0[2]	6.0[2]
7.0	5.0	7.0
8.0	5.5	8.5

[1] Minimum design R-value of aggregate base is 78.
[2] County's minimum requirements.

Appropriate Traffic Index (TI) data should be selected by the project civil engineer or traffic engineering consultant and appropriate R-value of the subgrade soils will need to be determined after completion of rough grading to finalize the pavement design. Final pavement sections should be in general accordance with the city standards.

The stability of compacted pavement subgrade soils will be reduced with the increase of soil moisture. If pavement areas are adjacent to heavily watered landscape areas, we recommend some measure of moisture control to be taken to prevent the subgrade soils from being saturated. It is recommended that the concrete curb separating the landscaping area from the pavement be extended below the aggregate base to reduce the potential for irrigation water entering the aggregate base. In lieu of the curb extension, a moisture barrier may be used. Concrete swales should be designed in roadway or parking areas subject to concentrated surface runoff.

Subgrade soils in the upper 24 inches of the driveways and parking areas should be properly compacted to at least 90 percent relative compaction (ASTM D1557) and should be moisture-conditioned to above optimum moisture contents, and kept in this condition until the pavement section is constructed. Minimum relative compaction requirements for aggregate base should be 95 percent of the maximum laboratory density (ASTM D1557).

Asphalt concrete and aggregate base should conform to Section 203-6 of the *Standard Specifications for Public Works Construction* (Green Book), 2003 Edition. Crushed aggregate base or crushed miscellaneous base can conform to Sections 200-2.2 and 200-2.4 of the *Standard Specifications for Public Works Construction* (Green Book), 2003 Edition, respectively.

4.12.2 Portland Cement Concrete (PCC) Pavements

Portland Cement Concrete (PCC) pavements should be considered in areas where impact loading from truck wheels is anticipated such as trash enclosure aprons, driveway approach, parking lot approach sections and fire truck lane for the proposed Civic Center and park areas, etc. For preliminary planning purposes, a minimum thickness of 6-inches may be assumed for PCC pavements. All PCC pavements should have a minimum 28-day concrete compressive strength of 3,500 psi and have appropriate joints and saw cuts in accordance with either Portland Cement Association (PCA) or American Concrete Institute (ACI) guidelines. A minimum of 4 inches thick layer of Class 2 aggregate base at 95 percent relative compaction (per ASTM D1557) should be considered beneath the PCC paving. Underlying the aggregate base layer, subgrade soils in the upper 24 inches should be properly compacted to at least 90 percent relative compaction (ASTM D1557) and should be moisture-conditioned to above the optimum moisture contents. Use of concrete cutoff or edge barriers should be considered at the perimeter of the common parking or driveway areas when they are adjacent to either open (unfinished) or landscaped areas.

4.13 Cement Type and Corrosion Measures

Preliminary laboratory test results indicate that onsite soils at shallow depth have "negligible" soluble sulfate content (per Section 4.3 of ACI 318). Accordingly, common Type II cement may be used for concrete in contact with onsite soils. The concrete should be designed for negligible sulfate exposure in accordance with ACI 318 (ACI, 2005).

The resistivity test result of the site soil indicates that these soils have moderate to severe corrosion potential to buried ferrous metals. If buried ferrous pipes are planned for the project, further resistivity tests of the soil samples should be performed and specific corrosion protection measures should be recommended by a qualified corrosion engineer.

4.14 Surface Drainage

Adequate surface drainage is a very important factor in reducing the likelihood of adverse behavior of foundations, hardscape, and slopes. Surface drainage should be sufficient to prevent ponding of water anywhere on the site, and especially near structures and top-of-slopes. Positive surface drainage should be provided and maintained to direct surface water away from structures and slopes and towards suitable drainage collection facilities and outlets.

4.15 Utility Trenches

Utility trenches should be backfilled with compacted fill in accordance with Sections 306-1.2 and 306-1.3 of the *Standard Specifications for Public Works Construction*, ("Greenbook"), 2003 Edition or corresponding sections in the later editions. Fill material should be placed in horizontal layers of thickness compatible to the type of equipment being used and should be compacted to at least 90 percent relative compaction (ASTM D1557) by mechanical means only. Utility pipes should be placed on properly placed bedding materials extended to a depth in accordance to the pipe manufacturer's specification. The pipe bedding should extend to least 12 inches over the top of the pipeline. The bedding material may consist of compacted free-draining sand, gravel, or crushed rock. If sand is used, the sand should have a Sand Equivalent (California Standard Test Method 217) of 30 or greater.

4.16 Geotechnical Observation During Construction

All grading and excavation should be performed under the observation and testing of the geotechnical consultant at the following stages:

- Upon completion of site clearing;
- During site earthwork;
- During preparation of subgrades;

- During fill placement for cut slope stabilization, as needed, and construction of fill slopes;
- During excavation and backfilling of all utility trenches;
- During construction of any temporary shoring, if needed;
- During placement of aggregate base and asphalt concrete for pavement areas; and
- When any unusual or unexpected geotechnical conditions are encountered.

4.17 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 observe the subsurface conditions during grading and construction of the project, in order to confirm that our findings are representative for the site.

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Important Information about Your Geotechnical Engineering Report

Subsurface problems are a principal cause of construction delays, cost overruns, claims, and disputes.

While you cannot eliminate all such risks, you can manage them. The following information is provided to help.

Geotechnical Services Are Performed for Specific Purposes, Persons, and Projects

Geotechnical engineers structure their services to meet the specific needs of their clients. A geotechnical engineering study conducted for a civil engineer may not fulfill the needs of a construction contractor or even another civil engineer. Because each geotechnical engineering study is unique, each geotechnical engineering report is unique, prepared *solely* for the client. No one except you should rely on your geotechnical engineering report without first conferring with the geotechnical engineer who prepared it. *And no one — not even you* — should apply the report for any purpose or project except the one originally contemplated.

Read the Full Report

Serious problems have occurred because those relying on a geotechnical engineering report did not read it all. Do not rely on an executive summary. Do not read selected elements only.

A Geotechnical Engineering Report Is Based on A Unique Set of Project-Specific Factors

Geotechnical engineers consider a number of unique, project-specific factors when establishing the scope of a study. Typical factors include: the client's goals, objectives, and risk management preferences; the general nature of the structure involved, its size, and configuration; the location of the structure on the site; and other planned or existing site improvements, such as access roads, parking lots, and underground utilities. Unless the geotechnical engineer who conducted the study specifically indicates otherwise, do not rely on a geotechnical engineering report that was:

- not prepared for you,
- not prepared for your project,
- not prepared for the specific site explored, or
- completed before important project changes were made.

Typical changes that can erode the reliability of an existing geotechnical engineering report include those that affect:

- the function of the proposed structure, as when it's changed from a parking garage to an office building, or from a light industrial plant to a refrigerated warehouse,
- elevation, configuration, location, orientation, or weight of the proposed structure,
- composition of the design team, or
- project ownership.

As a general rule, *always* inform your geotechnical engineer of project changes—even minor ones—and request an assessment of their impact. *Geotechnical engineers cannot accept responsibility or liability for problems that occur because their reports do not consider developments of which they were not informed.*

Subsurface Conditions Can Change

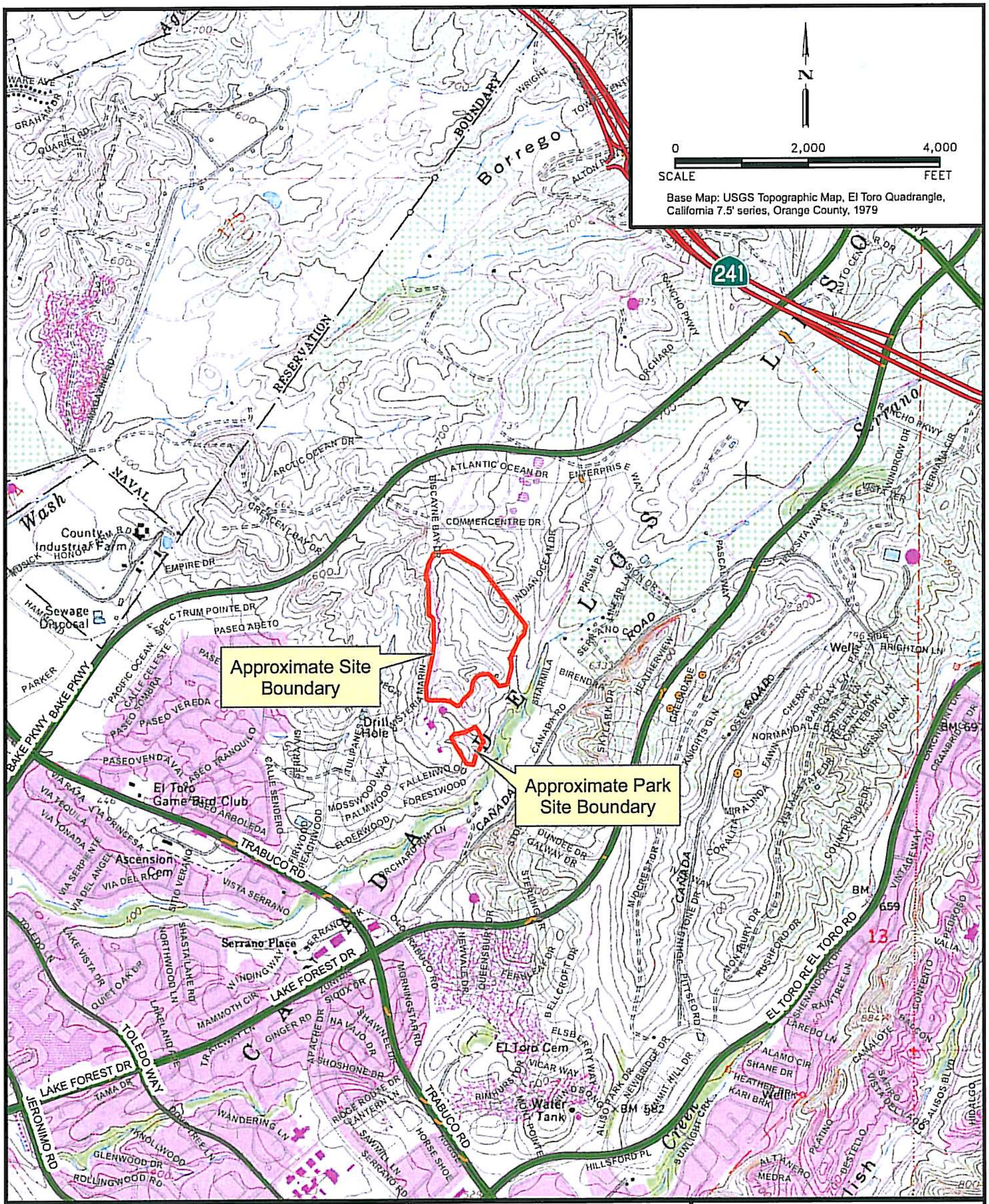
A geotechnical engineering report is based on conditions that existed at the time the study was performed. *Do not rely on a geotechnical engineering report* whose adequacy may have been affected by: the passage of time; by man-made events, such as construction on or adjacent to the site; or by natural events, such as floods, earthquakes, or groundwater fluctuations. *Always* contact the geotechnical engineer before applying the report to determine if it is still reliable. A minor amount of additional testing or analysis could prevent major problems.

Most Geotechnical Findings Are Professional Opinions

Site exploration identifies subsurface conditions only at those points where subsurface tests are conducted or samples are taken. Geotechnical engineers review field and laboratory data and then apply their professional judgment to render an opinion about subsurface conditions throughout the site. Actual subsurface conditions may differ—sometimes significantly—from those indicated in your report. Retaining the geotechnical engineer who developed your report to provide construction observation is the most effective method of managing the risks associated with unanticipated conditions.

A Report's Recommendations Are *Not* Final

Do not overrely on the construction recommendations included in your report. *Those recommendations are not final,* because geotechnical engineers develop them principally from judgment and opinion. Geotechnical engineers can finalize their recommendations only by observing actual



Residential Development
IRWD Site
Lake Forest, California

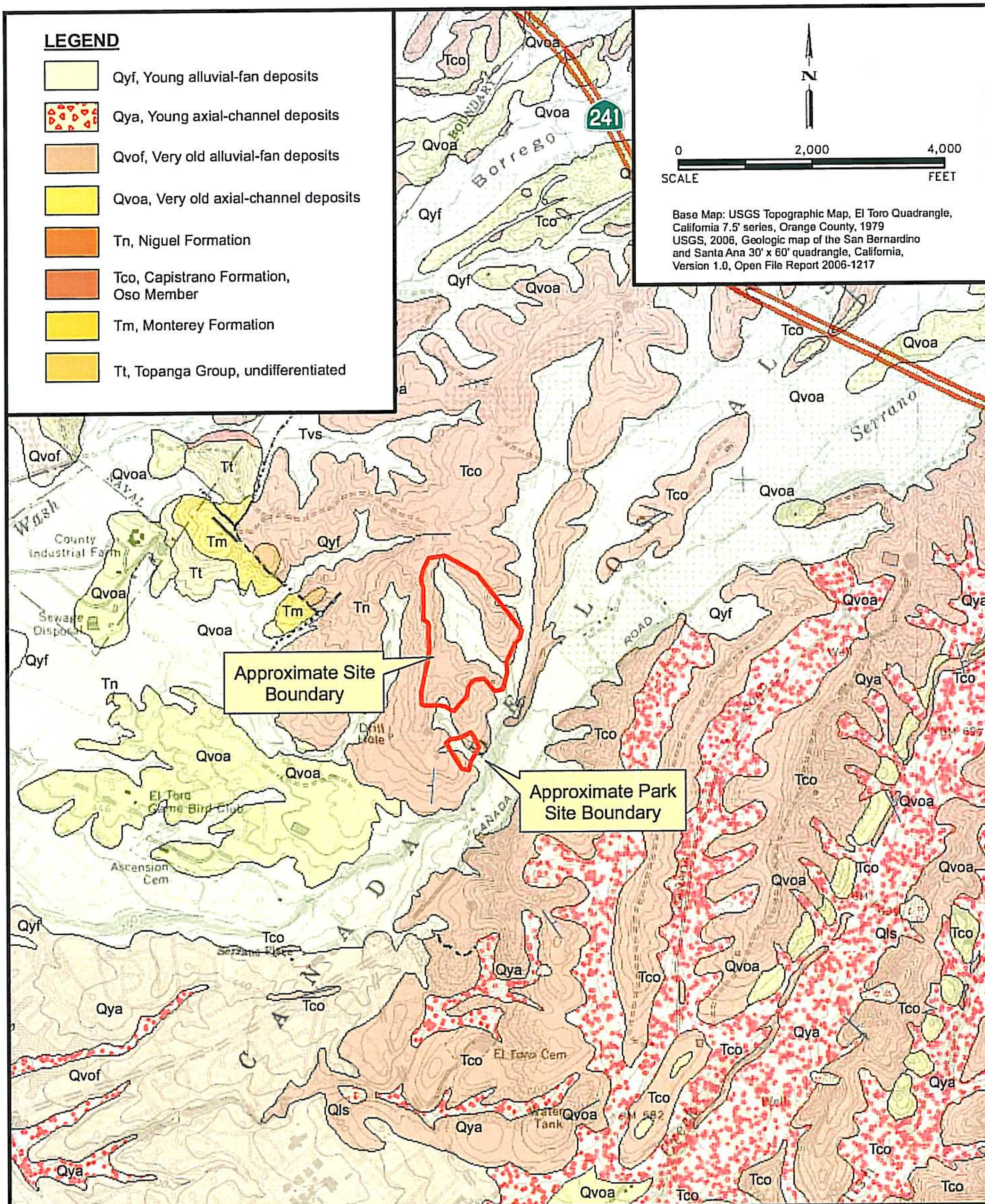
SITE LOCATION MAP

Project No.
011797-002

Date
May 2008

Figure 1

\\GIS\Administration\ArcGISTemplates\NEW_GDT_SiteLocationMap.mxd



Residential Development
IRWD Site
Lake Forest, California

REGIONAL GEOLOGIC MAP

Project No.
011797-002

Date
May 2008

Figure 2

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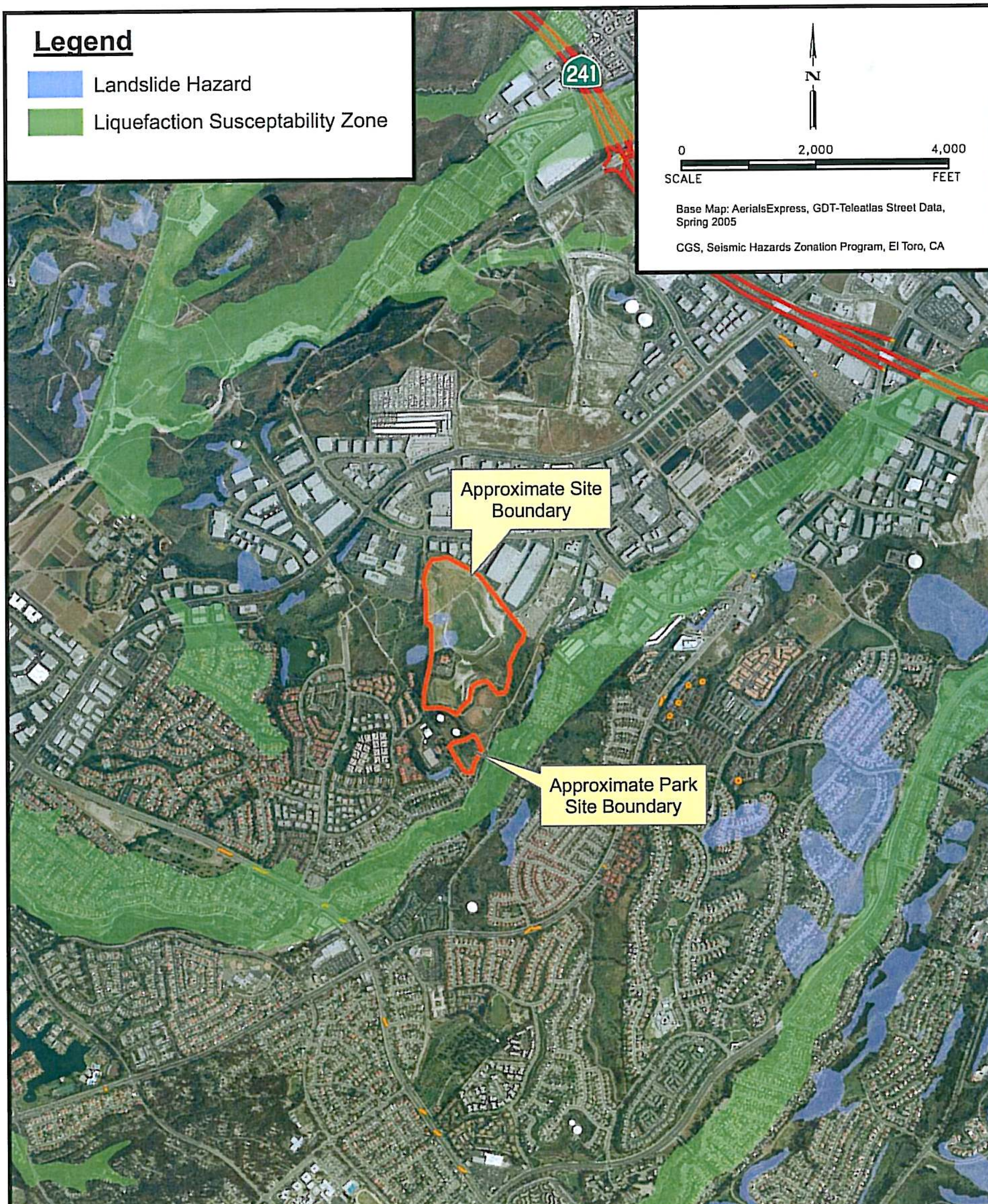
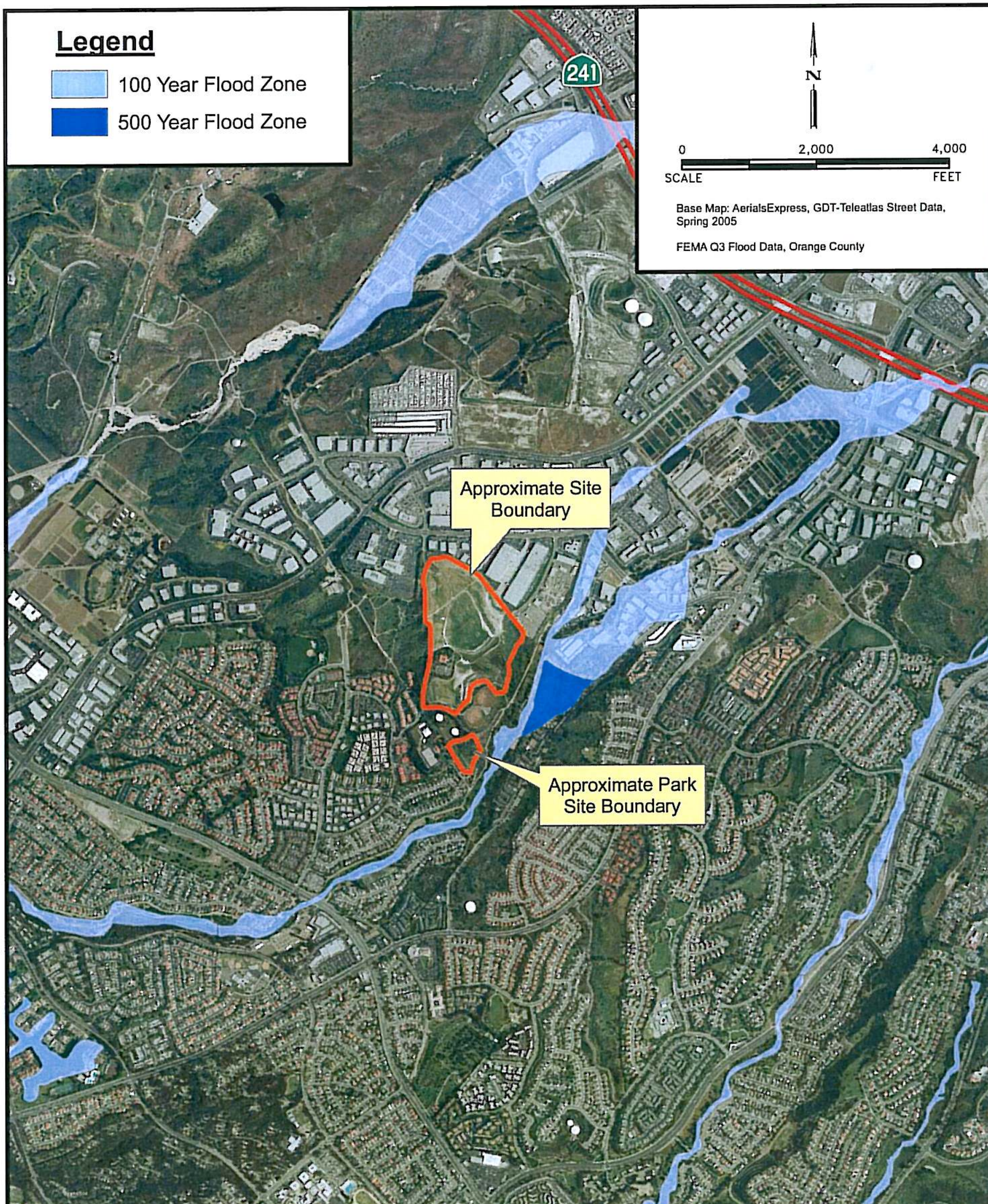


Residential Development IRWD Site Lake Forest, California	SEISMIC HAZARD MAP	Project No. 011797-002 Date May 2008 Figure 3

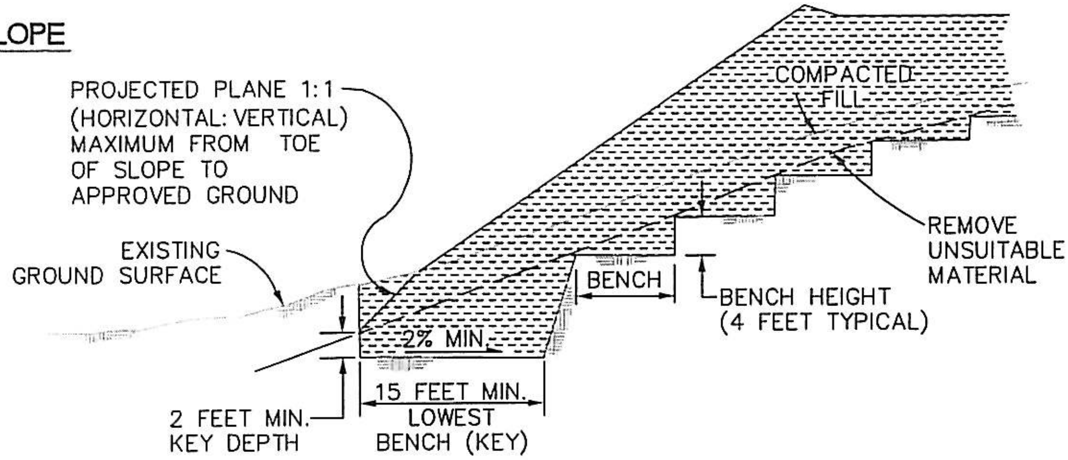
\\GIS\Administration\ArcGISTemplates\NEW_GDT_SiteLocationMap.mxd



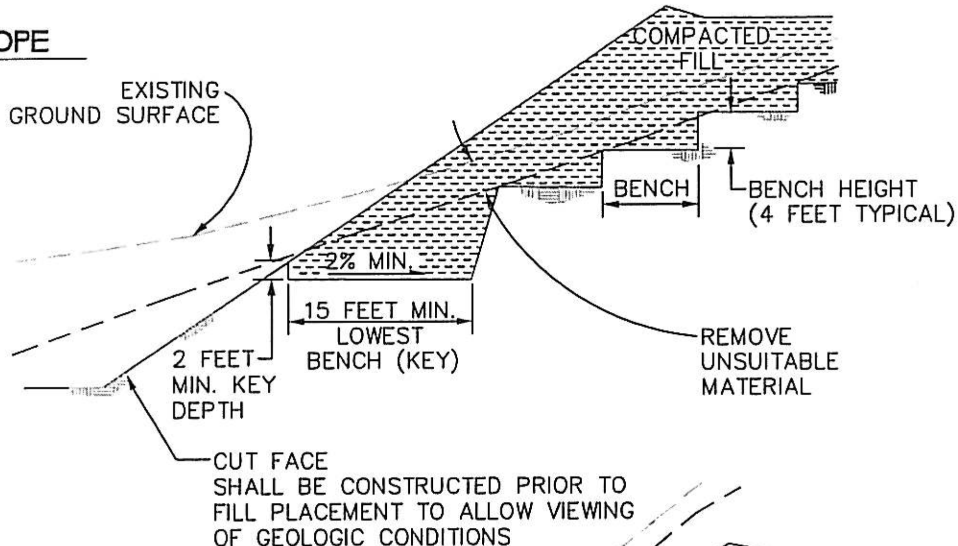
Residential Development IRWD Site Lake Forest, California	**FLOOD HAZARD MAP**	Project No. 011797-002 Date May 2008 Figure 4

\\GIS\Administration\ArcGISTemplates\NEW_GDT_SiteLocationMap.mxd

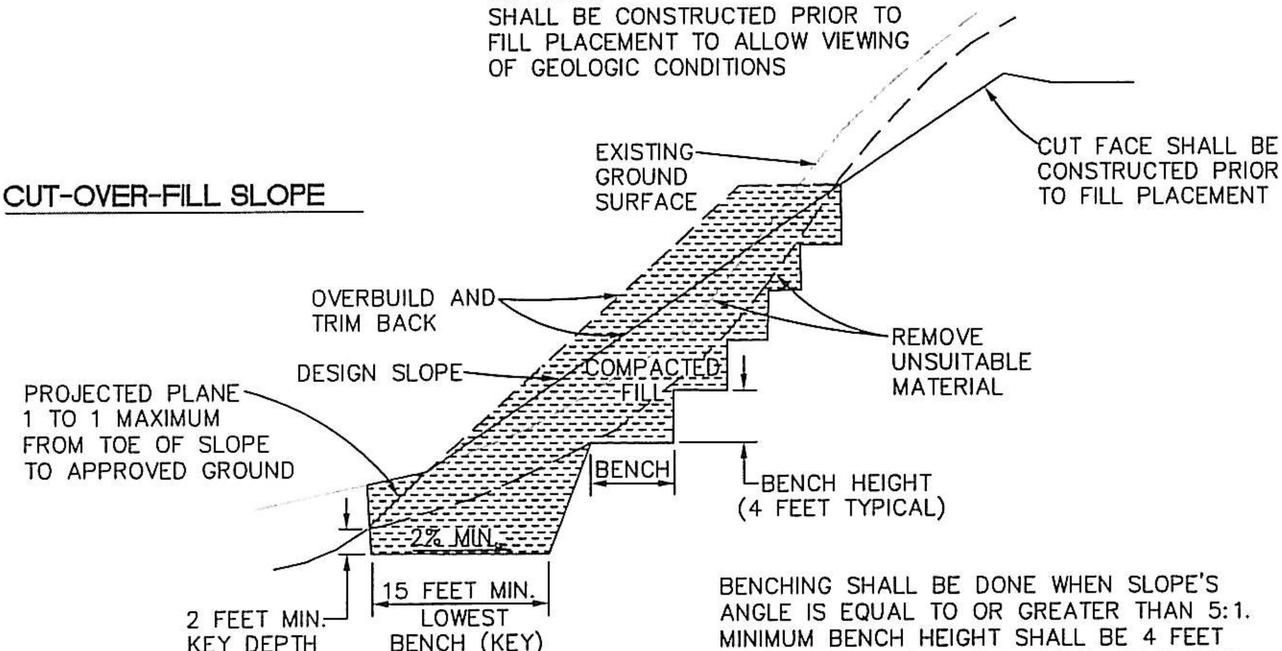
FILL SLOPE



FILL-OVER-CUT SLOPE



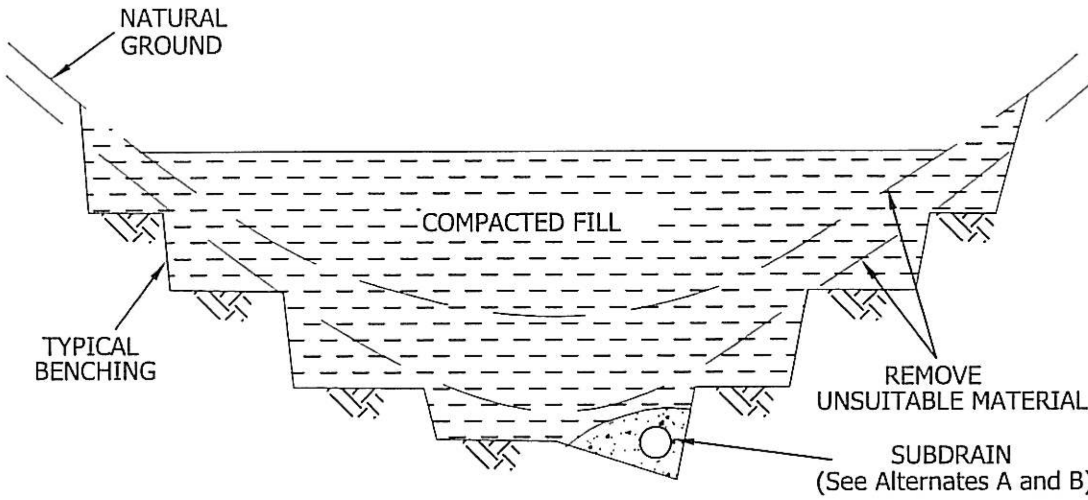
CUT-OVER-FILL SLOPE



BENCHING SHALL BE DONE WHEN SLOPE'S ANGLE IS EQUAL TO OR GREATER THAN 5:1. MINIMUM BENCH HEIGHT SHALL BE 4 FEET AND MINIMUM FILL WIDTH SHALL BE 9 FEET.

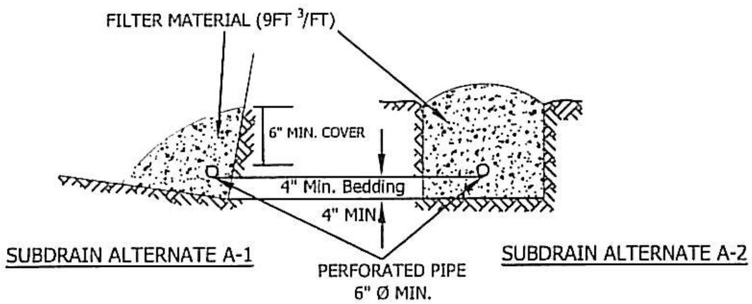
KEYING AND BENCHING	GENERAL EARTHWORK AND GRADING SPECIFICATIONS STANDARD DETAILS A	Leighton

FIGURE 5



SUBDRAIN ALTERNATE A

PERFORATED PIPE SURROUNDED WITH FILTER MATERIAL

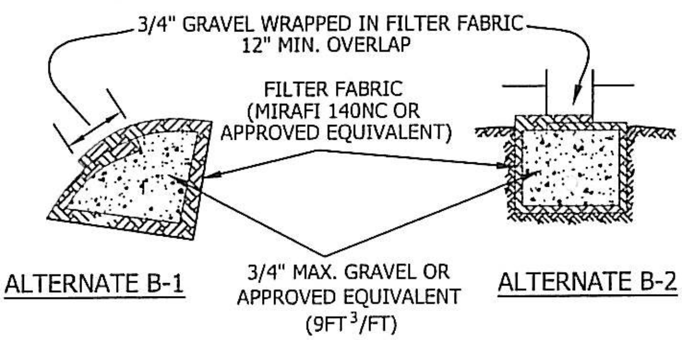


FILTER MATERIAL

FILTER MATERIAL SHALL BE CLASS 2 PERMEABLE MATERIAL PER STATE OF CALIFORNIA STANDARD SPECIFICATION, OR APPROVED ALTERNATE. CLASS 2 GRADING AS FOLLOWS:

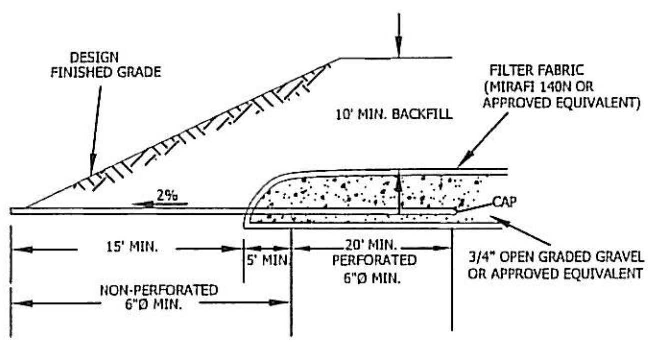
Sieve Size	Percent Passing
1"	100
3/4"	90-100
3/8"	40-100
No. 4	25-40
No. 8	18-33
No. 30	5-15
No. 50	0-7
No. 200	0-3

SUBDRAIN ALTERNATE B



PERFORATED PIPE IS OPTIONAL PER GOVERNING AGENCY'S REQUIREMENTS

DETAIL OF CANYON SUBDRAIN TERMINAL



CANYON SUBDRAIN	GENERAL EARTHWORK AND GRADING SPECIFICATIONS STANDARD DETAILS C	Leighton

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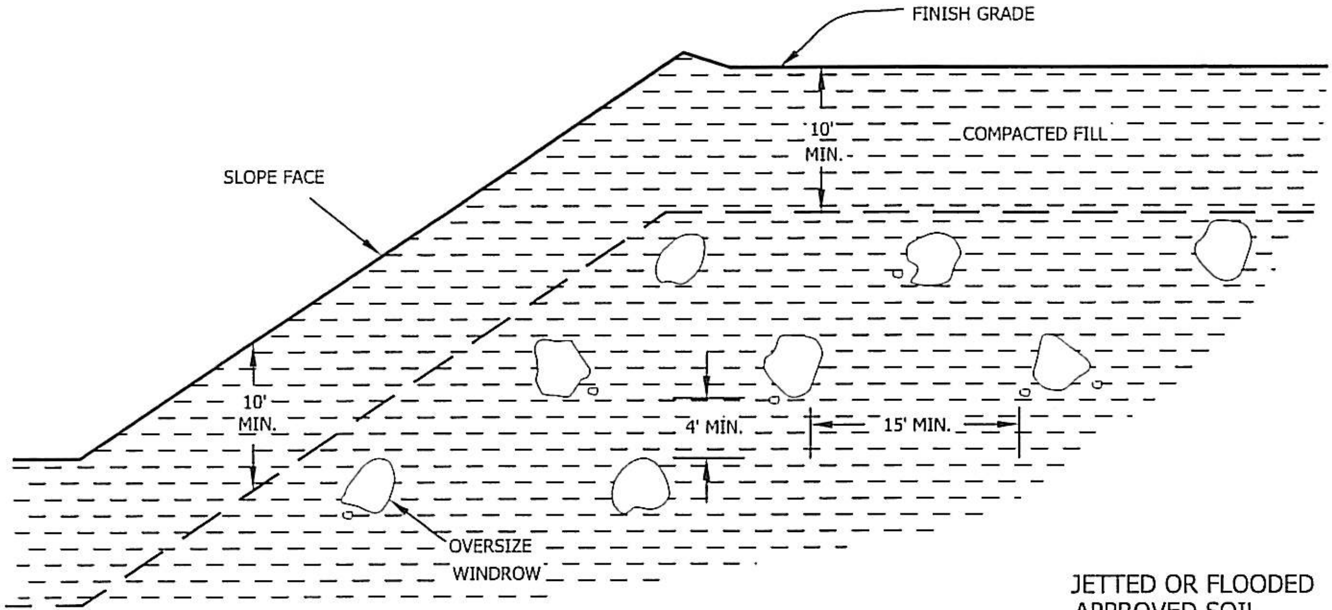
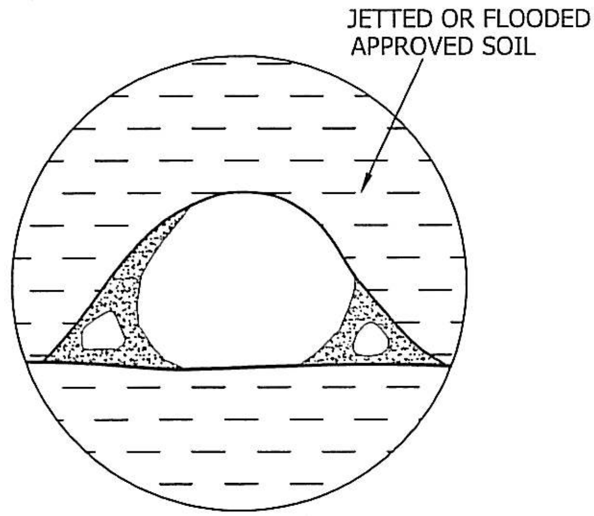
Figure 6

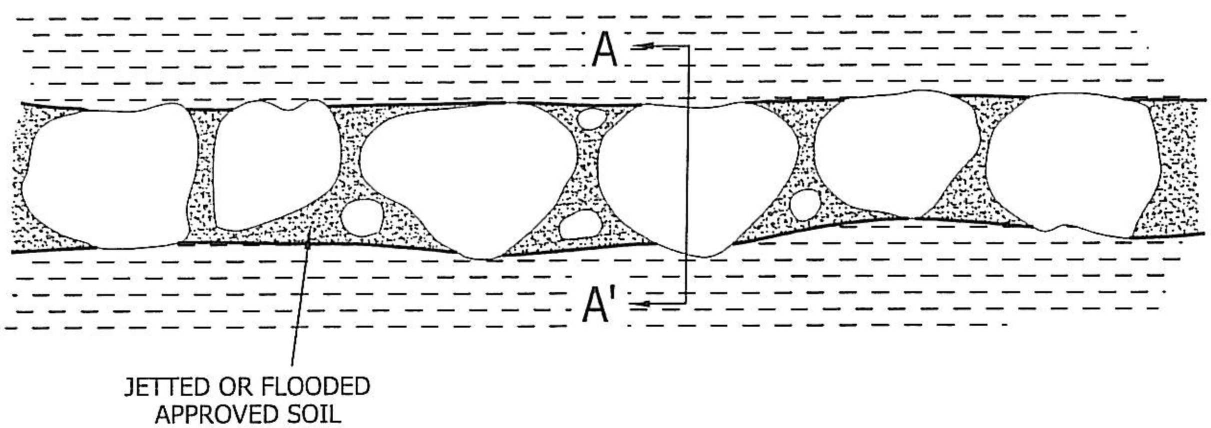




- Oversize rock is larger than 8 inches in largest dimension.
- Backfill with approved soil jetted or flooded in place to fill all the voids.
- Do not bury rock within 10 feet of finish grade.
- Windrow of buried rock shall be parallel to the finished slope face.

SECTION A-A'

PROFILE ALONG WINDROW



OVERSIZE ROCK DISPOSAL	GENERAL EARTHWORK AND GRADING SPECIFICATIONS STANDARD DETAILS B	Leighton

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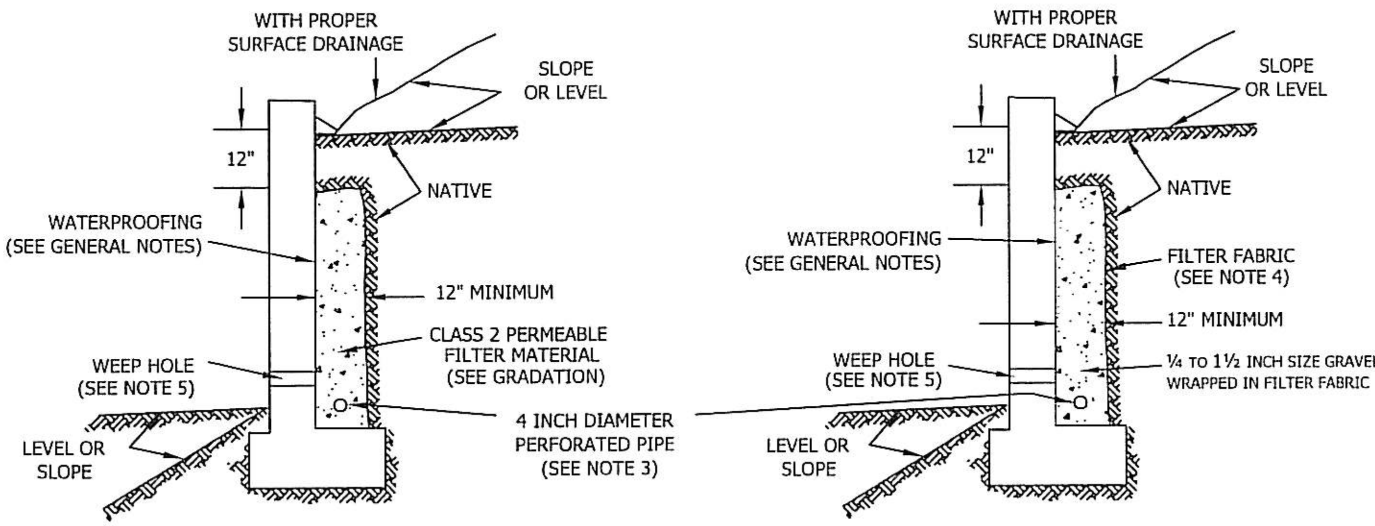
Figure 7

SUBDRAIN OPTIONS AND BACKFILL WHEN NATIVE MATERIAL HAS EXPANSION INDEX OF ≤50

OPTION 1: PIPE SURROUNDED WITH CLASS 2 PERMEABLE MATERIAL

OPTION 2: GRAVEL WRAPPED IN FILTER FABRIC



Class 2 Filter Permeable Material Gradation
Per Caltrans Specifications

Sieve Size	Percent Passing
1"	100
3/4"	90-100
3/8"	40-100
No. 4	25-40
No. 8	18-33
No. 30	5-15
No. 50	0-7
No. 200	0-3

GENERAL NOTES:

* Waterproofing should be provided where moisture nuisance problem through the wall is undesirable.
* Water proofing of the walls is not under purview of the geotechnical engineer
* All drains should have a gradient of 1 percent minimum
*Outlet portion of the subdrain should have a 4-inch diameter solid pipe discharged into a suitable disposal area designed by the project engineer. The subdrain pipe should be accessible for maintenance (rodding)
*Other subdrain backfill options are subject to the review by the geotechnical engineer and modification of design parameters.

Notes:
1) Sand should have a sand equivalent of 30 or greater and may be densified by water jetting.
2) 1 Cu. ft. per ft. of 1/4- to 1 1/2-inch size gravel wrapped in filter fabric
3) Pipe type should be ASTM D1527 Acrylonitrile Butadiene Styrene (ABS) SDR35 or ASTM D1785 Polyvinyl Chloride plastic (PVC), Schedule 40, Armco A2000 PVC, or approved equivalent. Pipe should be installed with perforations down. Perforations should be 3/8 inch in diameter placed at the ends of a 120-degree arc in two rows at 3-inch on center (staggered)
4) Filter fabric should be Mirafi 140NC or approved equivalent.
5) Weephole should be 3-inch minimum diameter and provided at 10-foot maximum intervals. If exposure is permitted, weepholes should be located 12 inches above finished grade. If exposure is not permitted such as for a wall adjacent to a sidewalk/curb, a pipe under the sidewalk to be discharged through the curb face or equivalent should be provided. For a basement-type wall, a proper subdrain outlet system should be provided.
6) Retaining wall plans should be reviewed and approved by the geotechnical engineer.
7) Walls over six feet in height are subject to a special review by the geotechnical engineer and modifications to the above requirements.

RETAINING WALL BACKFILL AND SUBDRAIN DETAIL FOR WALLS 6 FEET OR LESS IN HEIGHT

WHEN NATIVE MATERIAL HAS EXPANSION INDEX OF ≤50

Leighton

Figure 8

Appendix A

GEOTECHNICAL BORING LOG BA-1

Date	4-1-08	Sheet	1 of 2
Project	Residential Development at IRWD Site	Project No.	011797-002
Drilling Co.	Al Roy	Type of Rig	EZ-BORE
Hole Diameter	28"	Drive Weight	
Drop	"		
Elevation Top of Hole	692'	Location	See Plate 1, Geotechnical Map

Elevation Feet	Depth Feet	Graphic Log (N S)	Attitudes	Sample No.	Blows Per foot	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	DESCRIPTION Logged By J Roe Sampled By	Type of Tests
	0								**Disced Field:** @0': Silty SAND, loose, fine to coarse grained sand, fine rounded gravel.	
690									**Bedrock: Oso member of the Capistrano formation (Tco):** @1': SANDSTONE, yellow brown to olive grey, moist, cobble sized concretions, fine to coarse grained sand, friable.	
				BB1						RV, CR
	5		@4.6': B N34W, 9S	R1	7/6"				@4.6': Thin oxidized SAND bed, basal contact with underlying fine Silty SANDSTONE, cobble sized Sandstone concretions. @5.7': grades to coarse SANDSTONE	
685									@erosional contact top, oxidized, irregular, fine GRAVEL and Silty SANDSTONE with cobble concretions, rounded very hard, very well cemented, grades to coarse SANDSTONE to 10 feet.	
	10		@10': B N51W, 2N	R2	6 5/1"				@10': SANDSTONE, medium grey to orange brown, hard, oxidized, moderately to poorly cemented, friable, medium grained sand, oxidized.	
680			@11.2': B N52E, 0						@11.2': Oxidized SAND bed, overlie fine Silty Sandstone, 52E, Horizontal.	
			@12.7': B N72E, 6N						@12.7': Cobble concretions, becomes coarse SANDSTONE below.	
	15			R3	10/6"				@15': Silty SANDSTONE, yellow brown to orange brown, dry, hard, fine to coarse grained sand, poorly cemented, friable.	
675			@16.3': C N66E, 3N						@16.2': Basal contact overlie yellow brown micaceous Silty SANDSTONE, fine to coarse gravel.	
			@18.7': C N49E, 4N						@18.7': Basal contact, oxidized, becomes fine gravel, greyish Silty SANDSTONE.	
	20			R4	10/6"				@20': SANDSTONE, light grey to yellow brown, dry, hard, fine grained sand, moderately oxidized.	
670			@21': C N60E, 0						@21': Coarse SANDSTONE over fine grey Silty Sandstone, micaceous, oxidized at contact, infilled vertical worm burrows, oxidized around rim, dark reddish orange sand, healed.	
			@23.9': B N75E, 2N						@23.9': Oxidized SAND bed at contact with underlying Silty SANDSTONE.	
	25		@24.5': B N33E, 0	R5	13 12/3"	116.9	5.3		@24.5 - 26.5': Coarse grained SAND, erosional contact at top and bottom, oxidized at contact, horizontal bedding.	DS
665										
	30									

SAMPLE TYPES:		TYPE OF TESTS:	
S SPLIT SPOON	G GRAB SAMPLE	AL ATTERBERG LIMITS	-200 PERCENT PASSING
R RING SAMPLE	C CORE SAMPLE	CN CONSOLIDATION	SA SIEVE ANALYSIS
B BULK SAMPLE		DS DIRECT SHEAR	CR CORROSION SUITE
T TUBE SAMPLE		MD MAXIMUM DENSITY	SU SULFATE CONTENT
			RV R-VALUE

GEOTECHNICAL BORING LOG BA-1

Date 4-1-08 | Sheet 2 of 2
Project Residential Development at IRWD Site | Project No. 011797-002
Drilling Co. Al Roy | Type of Rig EZ-BORE
Hole Diameter 28" | Drive Weight | Drop "
Elevation Top of Hole 692' | Location See Plate 1, Geotechnical Map

Logged By J Roe
Sampled By

Elevation Feet	Depth Feet	Attitudes	Sample No.	Blows Per foot	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	DESCRIPTION	Type of Tests
	32.7	@32.7': B N89W, 4N						@32.7': Oxidized coarse grained SAND bed over fine Silty SANDSTONE, becomes grey to orange brown.	
	35		R6	17 20/4"				@35': SANDSTONE, light grey to olive brown, dry, hard, fine grained sand, some silt, micaceous.	
	39.7	@39.7': B N31W, 5N						@39.7': Oxidized SAND bed over fine gravel and Silty SANDSTONE.	
	43.5	@43.5': C N87W, 0						@43.5': SANDSTONE, light yellow brown, coarse, over blue grey, fine, Silty SANDSTONE, oxidized at contact, horizontal.	
	45		R7	40/5"				@45': SANDSTONE, light grey to dark orange brown, moist, very hard, fine grained sand, friable, well oxidized along micaceous silty interbeds, moderately well cemented with iron oxide.	
	47.7							@47.7': Coarse SANDSTONE	

Elevation scale: 660, 655, 650, 645, 640, 635. Depth scale: 30, 35, 40, 45, 50, 55, 60. Graphic Log: N S

Total depth of boring: 50 feet.
Downhole logged to 48.1 feet.
No groundwater encountered during drilling.
Boring was backfilled with soil cuttings and tamped.

B = Bedding Surface
C = Contact between units

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

TYPE OF TESTS:
AL ATTERBERG LIMITS
CN CONSOLIDATION
DS DIRECT SHEAR
MD MAXIMUM DENSITY
-200 PERCENT PASSING
SA SIEVE ANALYSIS
CR CORROSION SUITE
SU SULFATE CONTENT
RV R-VALUE

GEOTECHNICAL BORING LOG BA-2

Date 4-1-08 | Sheet 1 of 2
Project Residential Development at IRWD Site | Project No. 011797-002
Drilling Co. Al Roy | Type of Rig EZ-BORE
Hole Diameter 28" | Drive Weight | Drop "
Elevation Top of Hole 675' | Location See Plate 1, Geotechnical Map

Logged By J Roe
Sampled By

Elevation Feet	Depth Feet	Graphic Log (N S)	Attitudes	Sample No.	Blows Per foot	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	DESCRIPTION	Type of Tests
675	0							SM	@0': Asphalt Concrete: 2-inch over Silty SAND with gravel, dark brown, moist, fine grained sand, some clay.	
									Bedrock: Oso member of the Capistrano formation (Tco): @1': SANDSTONE, yellow brown to orange brown, moist, moderately hard, fine to medium grained sand, poorly graded, poorly cemented, friable, thin beds of dark brown micaceous SILTSTONE.	MD, CR
			@2': B N28W, 7N	BB1					@2': Cross-bedded SANDSTONE, tan, thin beds, erosional contact, irregular contact, well oxidized at basal contact, abundant concretions.	
670	5		@5': B N21W, 26S	R1	4 6				@5': SANDSTONE, thin bed, irregular erosion contact, 2-4 inch thick, cross bedded oxidized basal contact with orange brown to olive brown Silty SANDSTONE.	
									@8': becomes massive, cobble sized iron rimmed, well cemented concretions, fine sand.	
665	10			R2	5 10				@10': SANDSTONE, medium grey to dark orange brown, slightly moist, moderately hard, fine grained sand, thin beds of well oxidized sand, friable.	
			@13.4': C N84W, 5S						@13.4': Clayey SILTSTONE, olive brown, 5-inch thick, top contact is erosional, bottom contact is planar, mottled with well oxidized fine grained sand lenses.	
660	15			R3	5 10				@15': SANDSTONE, medium grey, moist, hard, fine grained sand, friable, micaceous, silty.	
			@15.8': C N84E, 10S						@15.8': Sandy SILTSTONE, 2-inch thick, trace clay, top contact is erosional with well oxidized sand, bottom contact is planar, becomes sandstone.	
			@19.1': B N53W, 4S						@19.1': 3-inch thick, well oxidized, well cemented, sandy bed becomes fine grained sandy SILTSTONE, erosional, irregular contact.	
655	20			R4	6 12				@20': SANDSTONE, light grey, moist, hard, fine to medium grained sand, friable, micaceous, well indurated.	
			@22.5': B N80E, 4S						@22.5': Sandy SILTSTONE with trace clay, 6-inch thick interbed, well oxidized, micaceous, erosional/irregular top and bottom contact, becomes sandstone.	
650	25			R5	10 16				@25': SANDSTONE, light grey to light orange brown, dry, hard, fine grained sand, micaceous with silt.	
			@26.2': C N58W, 2S						@26.2': Wispy-thin, oxidized, fine grained SANDSTONE beds, some discontinuous, SANDSTONE lenses with loose dark brown sand, top and bottom contact are erosional, well healed burrows with sand.	
645	30									

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

TYPE OF TESTS:
AL ATTERBERG LIMITS
CN CONSOLIDATION
DS DIRECT SHEAR
MD MAXIMUM DENSITY
-200 PERCENT PASSING
SA SIEVE ANALYSIS
CR CORROSION SUITE
SU SULFATE CONTENT
RV R-VALUE

GEOTECHNICAL BORING LOG BA-2

Date 4-1-08 | Sheet 2 of 2
Project Residential Development at IRWD Site | Project No. 011797-002
Drilling Co. Al Roy | Type of Rig EZ-BORE
Hole Diameter 28" | Drive Weight | Drop "
Elevation Top of Hole 675' | Location See Plate 1, Geotechnical Map

Logged By J Roe
Sampled By

Elevation Feet	Depth Feet	Graphic Log (N S)	Attitudes	Sample No.	Blows Per foot	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	DESCRIPTION	Type of Tests
645	30		@30.7': Fault, N7E, 44NW @32.3': B N66W, 6S	R6	12 28				@30': SANDSTONE, light grey, moist, hard, fine grained sand, poorly cemented, friable, well indurated. @30.7': FAULT: 1/8-inch wide, well healed with iron oxide, well cemented along planar surface, oxidized sand below fault, minor offset along laminated bedding to 2-inches @32.3': Oxidized 2-inch thick sand bed, SANDSTONE becomes harder.	
640	35			R7	22 10/2"				@35': SANDSTONE, light grey to orange brown, moist, very hard, fine grained sand, well oxidized, poorly cemented, friable. @35.5': Irregular erosional contact with hard, light grey, coarse grained SANDSTONE. @38': Thin bed of oxidized fine grained silty SANDSTONE, contact is roughly planar, well cemented at contact, becomes friable with depth.	
635	40		@38.9': C N67W, 3S @41.2': B N80E, 4S	R8	12 22/3"				@40': SANDSTONE, light yellow brown to orange brown, dry, very hard, fine grained sand, well oxidized, poorly cemented, friable, well indurated. @41.2': Oxidized thin sandstone bed, contact is roughly planar, abundant cobble sized sandstone iron concretions.	
630	45		@43.5': Fault, N8E, 65NW @44.8': Fault, N6W, 62NE	R9	25 25/2"				@43.5': FAULT: oxidized at top of planar contact, 1/8 to 1/2-inch wide, well healed with dark grey fine grained sand, well cemented along fault plane, offsets fine grained light grey silty SANDSTONE against coarse grained yellow brown SANDSTONE. @44.8': FAULT: steeply dipping, 1/8 to 1/4-inch wide, planar, fracture zone is well healed and well cemented. @47.8': FAULT is truncated by above mentioned FAULT. @50': SANDSTONE, olive brown, dry, very hard, fine grained sand.	
625	50								Total depth of boring: 50 feet. Downhole logged to 47.8 feet. No groundwater encountered during drilling. Boring was backfilled with soil cuttings and tamped. B = Bedding Surface C = Contact between units	
620	55									
615	60									

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

TYPE OF TESTS:
AL ATTERBERG LIMITS
CN CONSOLIDATION
DS DIRECT SHEAR
MD MAXIMUM DENSITY
-200 PERCENT PASSING
SA SIEVE ANALYSIS
CR CORROSION SUITE
SU SULFATE CONTENT
RV R-VALUE

GEOTECHNICAL BORING LOG BA-3

Date 4-1-08 | Sheet 1 of 2
Project Residential Development at IRWD Site | Project No. 011797-002
Drilling Co. Al Roy | Type of Rig EZ-BORE
Hole Diameter 28" | Drive Weight | Drop "
Elevation Top of Hole 708' | Location See Plate 1, Geotechnical Map

Logged By J Roe
Sampled By

Elevation Feet	Depth Feet	Attitudes	Sample No.	Blows Per foot	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	DESCRIPTION	Type of Tests
	0							**Disced field** @0': Silty SAND with fine gravel comprised of sandstone.	
705		@2': B N35E, 4N						**Bedrock: Oso member of the Capistrano formation (Tco)** @0.7': Silty SANDSTONE, mottled medium grey to orange brown, moist, hard, fine to medium grained sand, trace clay and charcoal fragments. @1': Concretionary SAND bed, very well cemented, 1 to 3-inch thick. @2': Oxidized SANDSTONE bed, light yellow brown, dry, hard, fine grained sand, 1/2-inch thick, roughly planar, erosional top and bottom contact, friable, poorly cemented, well cemented rounded sandstone cobble concretions.	
	5	@5': B N72W, 4N	R1	5 9					CR
700		@7.4': C N78W, 0						@7.4': SANDSTONE, undulatory, horizontal contact, overlie cobble sandstone concretions.	
	10		R2	5 8				@9.9': SANDSTONE, yellow brown to orange brown, coarse grained sand, irregular erosional contact, becomes blue grey fine grained sandstone to silty sandstone with gravel sized iron nodules. @10': SANDSTONE, light yellow brown, dry, hard, fine grained sand, poorly cemented, friable, trace amount of clay.	CR
695		@12.4': B N78E, 4S						@12.4': SANDSTONE bed, iron oxide stained, becomes coarser to 14.2'	
	15	@15': B N55W, 9S	R3	4 10				@15': SANDSTONE, light orange brown, slightly moist, hard, fine grained, oxidized.	
		@16.3': C N46W, 0						@16.3': Coarse grained SANDSTONE over silty SANDSTONE, fine grained, hard, horizontal bedding.	
690									
	20	@20': B N80E, 2S	R4	6/6"				@20': SANDSTONE, yellow brown to olive brown, dry, very hard, moderately cemented layers with iron oxide as cement, becomes friable with depth, some gravel sized iron concretions, horizontal bedding.	
685		@23': B N7W, 2S						@23.9': Well cemented concretions, calcium carbonate cementation. @24.5': Concretionary SANDSTONE bed, 3-inch thick.	
	25		R5						
		@26': B N75E, 2N						@26.6': Concretionary SANDSTONE bed, horizontal, 3-inch thick.	
680									
	30								

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

TYPE OF TESTS:
- AL ATTERBERG LIMITS
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