
Appendix E-1

2004 Limited Geotechnical Investigation

**LIMITED GEOTECHNICAL INVESTIGATION
IN SUPPORT OF EIR ACTIVITIES FOR
OAK VALLEY AT CALIMESA
RIVERSIDE COUNTY, CALIFORNIA**



Prepared for

Fiesta Development, Inc.
470 E. Harrison Street
Corona, California 92879

Prepared by

HUSHMAND ASSOCIATES, INC.
15451 Red Hill Avenue, Suite A
Tustin, California 92780

PETRA GEOTECHNICAL, INC.
3185 Airway Avenue, Suite A
Costa Mesa, California 92626

June 2004

**LIMITED GEOTECHNICAL INVESTIGATION
IN SUPPORT OF EIR ACTIVITIES FOR
OAK VALLEY AT CALIMESA
RIVERSIDE COUNTY, CALIFORNIA**

Prepared for

Fiesta Development, Inc.
470 E. Harrison Street
Corona, California 92879

Prepared by

HUSHMAND ASSOCIATES, INC.
15451 Red Hill Avenue, Suite A
Tustin, California 92780

PETRA GEOTECHNICAL, INC.
3185 Airway Avenue, Suite A
Costa Mesa, California 92626

June 2004

June 7, 2004

Fiesta Development, Inc.
470 E. Harrison Street
Corona, California 92879

Attn.: Mr. Saied Naaseh

**SUBJECT: GEOTECHNICAL INVESTIGATION
IN SUPPORT OF EIR ACTIVITIES FOR
OAK VALLEY AT CALIMESA
RIVERSIDE COUNTY, CALIFORNIA**

Dear Mr. Naaseh:

Hushmand Associates, Inc. (HAI) and Petra Geotechnical, Inc. (Petra) are pleased to submit five (5) copies of the Report for the subject project, in accordance with our proposal dated March 8, 2004.

HAI/Petra appreciate the opportunity of being of service to Fiesta Development, Inc. Should you need additional information or any clarifications please call one of the undersigned.

Sincerely yours,

HUSHMAND ASSOCIATES, INC.

PETRA GEOTECHNICAL, INC.

Ben Hushmand, PhD, RCE 44777
President

Siamak Jafroudi, PhD, RGE 2024
President

HUSHMAND ASSOCIATES, INC.

Bruce Schell, CEG 1434
Associate Principal Geologist

TABLE OF CONTENTS

	<u>Page</u>
EXECUTIVE SUMMARY	
1.0 INTRODUCTION	1
1.1 Project Description.....	1
1.2 Site Description.....	1
1.3 Scope of Work	1
1.4 Methods of Investigation	2
1.5 Previous Investigations	3
1.6 Laboratory Testing.....	3
2.0 REGIONAL GEOLOGY	4
2.1 Regional Physiography	4
2.2 Regional Stratigraphy	5
2.3 Regional Structure	5
3.0 SITE GEOLOGY	7
3.1 Site Physiography	7
3.2 Site Stratigraphy.....	8
3.2.1 San Timoteo Formation	8
3.2.2 Alluvium	9
3.2.3 Relict Soil.....	9
3.2.4 Artificial Fill	10
3.3 Site Structure.....	11
4.0 SEISMOLOGY.....	12
4.1 Historical Seismicity.....	12

TABLE OF CONTENTS (continued)

	<u>Page</u>
4.2 Faults.....	13
4.3 Maximum Earthquakes	13
4.4 Deterministic Ground Motion.....	14
4.5 Probabilistic Ground Motion	14
5.0 GEOLOGICAL HAZARDS	15
5.1 Surface Faulting.....	15
5.2 Erosion	16
5.3 Landslides	16
5.4 Liquefaction	17
6.0 HYDROLOGY	17
6.1 Surface Water.....	17
6.2 Ground Water.....	18
7.0 GEOTECHNICAL ENGINEERING RECOMMENDATIONS.....	18
7.1 Earthwork Recommendations.....	19
7.1.1 General Earthwork and Grading.....	19
7.1.2 Site Clearing.....	19
7.1.3 Excavation Characteristics and Potential for Generation of Oversize Rock	19
7.1.4 Removal of Existing Fill Deposits.....	20
7.1.5 Removals and Canyon Cleanouts	20
7.1.6 Ground Water.....	20

TABLE OF CONTENTS (continued)

	<u>Page</u>
7.1.7 Canyon Subdrains	21
7.1.8 Fill Placement	21
7.1.9 Benching	21
7.1.10 Disposal of Oversize Rock.....	22
7.1.11 Processing of Cut Areas.....	22
7.1.12 Cut/Fill Transition Lots.....	22
7.1.13 Deep Fill/Shallow Fill Transitions.....	22
7.1.14 Volumetric Changes.....	23
7.2 Cut Slopes	24
7.2.1 Gross Stability of Cut Slopes.....	24
7.2.2 Stability of Temporary Backcut Slopes	25
7.3 Fill Slopes	24
7.3.1 Fill Slope Construction	25
7.3.2 Surficial Stability of Fill Slopes.....	26
7.4 Stability of Natural Slopes	24
7.4.1 Gross and Surficial Stability	27
7.5 Geotechnical Observation.....	27
7.6 Seismic Design Considerations.....	27
7.6.1 Ground Motions	27
7.6.2 Secondary Seismic Hazards	27
7.7 Preliminary Foundation Design Recommendations	28
7.7.1 Building Clearances From Ascending Slopes.....	28
7.7.2 Footing Setbacks from Descending Slopes.....	28
7.7.3 Allowable Soil Bearing Capacity.....	28
7.7.4 Footing Settlements	28

TABLE OF CONTENTS (continued)

	<u>Page</u>
7.7.5 Lateral Resistance	29
7.7.6 Footing Observations	29
7.8 Preliminary Conventional Footing and Floor Slab Recommendations	29
7.8.1 Lots with Very Low Expansion Potentials (EI between 0 and 20).....	29
7.8.2 Lots with Low Expansion Potentials (EI Between 21 and 50)	31
7.8.3 Lots with Medium Expansion Potentials (EI between 51 and 90).....	33
7.9 Preliminary Post-Tensioned Foundation Recommendations.....	35
7.9.1 Design Recommendations for Moderately Expansive Soils (S. 1816).....	36
7.9.2 Design Recommendations for Compressible Soil Conditions (Se. 1819)	37
7.9.3 Minimum Design Recommendations	37
7.9.4 Precise Grading and Drainage Recommendations.....	38
7.10 Soluble Sulfates and Soil Corrosivity	39
7.11 Preliminary Retaining Wall Design Recommendations	39
7.11.1 Allowable Bearing Values and Lateral Resistance	39
7.11.2 Active and At-Rest Earth Pressures	39
7.11.3 Drainage and Waterproofing.....	40
7.11.4 Wall Backfill.....	41
7.12 Masonry Block Walls	42
7.12.1 Construction on Level Ground.....	42
7.12.2 Construction Near the Tops of Descending Slopes	42
7.13 Exterior Concrete Flatwork.....	45
7.13.1 Thickness and Joint Spacing.....	45
7.13.2 Reinforcement.....	45
7.13.3 Subgrade Preparation	45
7.14 Preliminary Structural Pavement Sections.....	45

TABLE OF CONTENTS (continued)

	<u>Page</u>
8.0 CONCLUSIONS.....	47
9.0 INVESTIGATION LIMITATIONS	47
10.0 REFERENCES CITED.....	47

LIST OF FIGURES AND PLATES

Figure 1	Site Location Map
Figure 2	Major Active Earthquake Faults and Physiographic Features
Figure 3	Photographs Illustrating the Topography in the Eastern Part of the Site
Figure 4	Photographs of Carbonate-rich Soils Common within the Site
Figure 5	Photographs of Large Valley and Typical San Timoteo Formation
Figure 6	Regional Seismicity Map, Magnitude >4
Figure 7	Probabilistic Ground Motion Map
Plate 1	Site Geotechnical/Geological Map (<i>In pocket at back</i>)

APPENDICES

Appendix A	Logs of Exploratory Borings
Appendix B	Laboratory Test Results
Appendix C	Standard Grading Specifications

**LIMITED GEOTECHNICAL INVESTIGATION
IN SUPPORT OF EIR ACTIVITIES FOR
OAK VALLEY AT CALIMESA
RIVERSIDE COUNTY, CALIFORNIA**

EXECUTIVE SUMMARY

This Geotechnical report describes a limited geotechnical investigation performed in support of environmental impact report (EIR) activities for the Oak Valley at Calimesa development. The Oak Valley project was adopted by the County of Riverside in 1990 as Specific Plan Numbers 216 and 216A. This area is the northern part of the larger Oak Valley project as described in EIR 229 by Michael Brandman Associates dated 1988. An amendment to the approved plan is being proposed for about the northern 1539 acres to reflect changes in demographic, geopolitical, and marketing conditions since the original specific plan was approved. The amended plan continues to be a mixed-use master planned community providing for about 2996 residential dwelling units along with supportive land uses including neighborhood commercial, recreation center, mixed use, schools, parks, and open space.

The scope of work for this geotechnical investigation consisted of:

- Review of available literature and maps pertaining to soil and geologic conditions,
- Review of development plans available at the time of our investigation,
- Reviewing historical aerial photographs,
- Drilling seven exploratory hollow-stem auger borings to depths of 21.5 feet and two borings to 50 feet to evaluate subsurface soil, geologic, and ground-water conditions,
- Logging and classifying soil materials encountered in each boring,
- Performing laboratory analyses on soil samples to determine engineering properties,
- Preparing a geotechnical map, and
- Analyzing the collected geologic and engineering data.

The geologic units at the site consist of three basic units. The predominant unit is the San Timoteo Formation which underlies the entire site and can be considered the bedrock unit. This unit consists of firm materials characterized by nearly horizontal to gently northerly dipping siltstone, sandstone, and conglomerate. Although classified as a soft rock, it is very firm compared to alluvial deposits and generally will provide good foundation support. This bedrock unit is overlain by ancient soils and young alluvium. The ancient soils (map unit Qrp) overlie the San Timoteo Formation on the mesas and ridge tops, and they comprise firm materials consisting of sandy clays and a carbonate-rich layer. The Young Alluvium (map unit Qya) occupies the valleys and consists of unconsolidated sands and silts with gravels; in the smaller valleys, the alluvium is thin but in the central parts of the larger valleys it may be on the order of 50 feet thick.

No ground water was encountered in any of the borings drilled for this investigation. Regional data suggest that ground water is deep within the San Timoteo Formation and therefore it does not appear to represent a major issue for site development. Further investigations may, however, encounter local perched ground-water bodies.

The site is within seismically active southern California between two major earthquake faults, the San Andreas and the San Jacinto faults. The controlling fault for seismic design on the site is the San Jacinto fault that lies about 3 miles west of the western property line. Design ground motion acceleration values for this fault will be in the 0.6 to 0.7g range.

There is a geological fault within the property. This fault extends onto the site from the southeast and can be traced to the central part of the site. The potential for surface rupture on this fault has not been resolved, especially with respect to school sites (Planning Areas 18 and 19) which are held to higher, more-rigid standards than residential facilities. It is still very possible that the fault can be proven to be inactive if trenching is conducted in the Qrp unit just east of Covington Canyon (Planning Area 14).

The previous site investigations assumed that all valleys were susceptible to liquefaction during earthquakes, but this investigation found the smaller valleys and large portions of the larger valleys have only very thin alluvium and a shallow depth to firm bedrock (QTs). Combined with the fact that no shallow ground water was found, many of these areas are very likely to be suitable for building.

In summary, development of the site is considered feasible from a geotechnical point of view. Most building sites will be free of hazard from landslide, settlement, and slippage providing that the general recommendations outlined in the Geotechnical Report are incorporated into the design criteria and project specifications, and that good engineering practices are followed. However, the project is still in the conceptual stage and specific grading and development plans have not been developed. More-specific recommendations for individual sites and facilities will require additional more-extensive geotechnical and geological investigations.

**LIMITED GEOTECHNICAL INVESTIGATION
IN SUPPORT OF EIR ACTIVITIES FOR
OAK VALLEY AT CALIMESA
RIVERSIDE COUNTY, CALIFORNIA**

1.0 INTRODUCTION

1.1 Project Description

This report describes a limited geotechnical investigation performed in support of environmental impact report (EIR) activities for the Oak Valley at Calimesa development. The Oak Valley project was adopted by the County of Riverside in 1990 as Specific Plan Numbers 216 and 216A. The development was a golf/recreation-oriented, master-planned community of mixed residential, commercial, recreational, and community uses. An amendment to the plan is being proposed to reflect changes in demographic, geopolitical, and marketing conditions since the original specific plan was approved. The amended plan continues to be a mixed-use master planned community providing for about 2996 residential dwelling units along with supportive land uses including neighborhood commercial, recreation center, mixed use, schools, parks, and open space. A major component of the original plan, a golf course, is no longer being proposed. Specifics of the proposed amendment are presented in the amendment document by The Keith Companies (TKC, 2003).

1.2 Site Description

The project area comprises about 1539 acres as shown on Plate 1 and Figure 1. This area is the northern part of the larger Oak Valley project as described in EIR 229 by Michael Brandman Associates (MBA, 1988) and which constituted about 6725 acres primarily between the I-10 freeway and San Timoteo Canyon Road and from the San Bernardino/Riverside county line on the north near Yucaipa to the Beaumont area on the south. This area was previously referred to as the Phase 1 area of the Oak Valley project as described in the EIR (MBA, 1988). Access to the site is primarily from County Line Road and Sandalwood Drive exits from the I-10 Freeway.

1.3 Scope of Work

In accordance with our proposal (HAI/Petra, 2004), the scope of work for this geotechnical investigation consisted of:

- Collection and review of readily available literature and maps pertaining to soil and geologic conditions within and adjacent to the site,
- Review of development plans available at the time of our investigation,
- Obtaining and reviewing historical aerial photographs to better define surface features within and adjacent to the site,
- Drilling 8 to 10 exploratory hollow-stem auger borings to depths ranging from approximately 20 to a maximum of 50 feet to evaluate subsurface soil, geologic, and ground-water conditions,
- Logging and classifying soil materials encountered in each boring,
- Collecting representative bulk and/or undisturbed soil samples for laboratory analysis,
- Performing appropriate laboratory analyses on soil samples which may include in-situ and maximum dry-density; in-situ and optimum moisture content; Atterberg limits; grain-size distribution; expansion potential; shear strength; consolidation and collapse potential, corrosivity (pH, chloride, sulfate, and minimum resistivity),
- Preparing a geotechnical map, and
- Performing geologic and engineering analyses on all data collected.

1.4 Methods of Investigation

Preliminary investigations included review of published geological literature and project reports. Pertinent reports are referenced within the text of this report.

Stereographic aerial photographs were analyzed to characterize the geologic setting of the site. Several vintages of photos taken between 1979 to 1995 were reviewed. These were at a scale of about 1:24,000.

Reconnaissance level geological/geotechnical mapping was conducted on 9 April 2004 by a senior, California-licensed, Engineering Geologist aided by previous maps (e.g., Dames & Moore, 1987; Rasmussen Associates, 1984, 1988) and the aerial photographs. The map was field checked on 19 April and 3 May 2004. The resulting map is presented as Plate 1 of this report.

Nine boreholes were drilled on 19 April 2004 using a Mobile 61, 8-inch-diameter, hollow-stem auger drill rig, outfitted with an automatic trip hammer. Undisturbed samples were collected using a 2½ inch-diameter modified California ring sampler. The boring locations were selected to provide representative sampling of all geological units within the site; these locations are shown on the geotechnical map (Plate 1). Seven of the borings were drilled on the mesas and ridge tops; samples were collected at 2½, 5, 10, 15, and 20 feet. Two of the borings were drilled in the valleys to depths of 51.5 feet. Samples were placed in plastic canisters and sealed with plastic tape for transporting to the laboratory and storage.

Bulk samples were collected from the upper 5 feet at selected boreholes. The boreholes were logged by a California-licensed Engineering Geologist. The logs of the boreholes are presented in Appendix A.

Laboratory testing was conducted in the Hushmand Associates (HAI) laboratory. Testing included standard engineering property testing on the ring and bulk samples (see Section 6 and Appendix B).

1.5 Previous Investigations

Geological investigations in the region have been performed since the 1920s (e.g., Frick, 1921); these studies are summarized in the EIR (MBA, 1988). There is little published information available on the specific project area. The EIR is a principal source of background information for this geotechnical investigation and in-depth review of the previous work was not conducted unless it was considered germane to this investigation. Of particular importance were the investigations by Dames & Moore (1987) and of Rasmussen Associates (1984, 1988) both of which conducted fault trenching investigations. Also important is research conducted since the EIR was prepared; specifically, the work of Albright (1999) and Kendrick et al (2002).

1.6 Laboratory testing

Soil samples collected during field investigation were delivered to Hushmand Associates, Inc. Laboratory to be examined and tested. Selected soil samples were tested to evaluate their physical characteristics including in-situ conditions and engineering properties.

The tests were performed in accordance with the following testing procedures:

- In-place moisture and dry density (ASTM D2937),
- Modified proctor compaction (ASTM D1557),
- Collapse potential (ASTM D5333), and
- Corrosion potential (including pH, minimum resistivity, soluble sulfates and soluble chlorides tests, in accordance with Cal DOT Standard Test Nos. 643, 532, 417-B and 422).

Laboratory test results are presented in Appendix B.

2.0 REGIONAL GEOLOGY

2.1 Regional Physiography

The site area is in the northeast part of the Peninsular Ranges geologic/physiographic province. The Peninsular Ranges province comprises a series of northwest-southeast trending mountains and valleys extending from the Los Angeles-San Gabriel-Upper Santa Ana River (San Bernardino) valleys at the southern margin of the Transverse Ranges geologic/physiographic province. The Peninsular Ranges province consists of three major blocks separated by major northwest-southeast trending faults such as the Whittier-Elsinore and San Jacinto fault systems (Figure 2). The eastern margin of the Peninsular Ranges is along the San Andreas fault and the Salton Trough geologic/physiographic province.

The site area is within the eastern block of the Peninsular Ranges province between the San Jacinto and San Andreas faults. This area comprises an elevated upland terrain transected by a complex arrangement of geologic faults which create an area of diverse physiographic elements. The principal elements are the San Timoteo Badlands, Crafton Hills, Yucaipa Valley, Beaumont Plain, and San Gorgonio Pass (Figures 1 and 2). These elements are bounded by the San Jacinto Mountains and Salton Trough on the south, the San Bernardino Mountains on the northeast, the San Bernardino Valley on the northwest and the Perris Uplands block on the west.

These upland areas are dissected primarily by southwest-flowing creeks which are tributaries to the San Timoteo Creek which flows northwesterly into the San Ana River. The major creek in the site region is Yucaipa Creek which flows west-southwesterly just north of the site area. The site area is dissected and incised by local creeks forming a system of flat-lying ridges and mesas bordered by shallow to deep, steep-sided canyons. The site area is within the northwest part of the Beaumont Plain adjacent to the San Timoteo Badlands. Although the site area has been referred to as being within the San Timoteo Badlands, badlands topography is not as well-developed within the project area as in the area to the west between San Timoteo Canyon and the San Jacinto Valley.

Elevations in the site region and nearby Beaumont Plain to the east and southeast are generally in the 2000 to 2500 foot range. These elevations rise dramatically to above 11,500 feet at San Gorgonio Peak in the San Bernardino Mountains to the northeast (Figure 3), and to Mount San Jacinto at more than 10,800 feet to the southeast. The Crafton Hills to the north of the site rise to about 3500 feet at Zanja Peak. Maximum elevations in the San Timoteo Badlands are around 2000 feet (+/- a few hundred feet). San Timoteo Canyon rises from about 1000-1500 feet near the southern margin of the Upper Santa Ana River Valley on the north, to about 2200 feet in the southeast. Just west of the site area, the canyon is about 1600 to 1700 feet elevation.

2.2 Regional Stratigraphy

The region is characterized by late Tertiary and Quaternary-aged sediments and sedimentary rocks overlying ancient crystalline basement rocks. The basement rocks consist of Mesozoic plutonic rocks (e.g. granites, diorites) and Paleozoic/Precambrian-aged metamorphic rocks (e.g. gneiss). Two distinct basement complexes are present in the project region; to the north are the San Bernardino and San Gabriel mountains where the crystalline basement contains plutonic igneous rocks of late-Mesozoic age and older metamorphic rocks. The basement rocks in the San Jacinto Mountains and underlying the Perris block are part of the southern California batholith are also of late-Mesozoic age as well as pre Mesozoic metamorphic sedimentary and volcanic rocks.

Sedimentary rocks in the site vicinity consist of Miocene- to Pleistocene-age conglomerate, sandstone, siltstone, and shale of nonmarine fluvial (river) origin and local clay beds of possible lacustrine (lake) origin (Dibblee, 1981). In the San Timoteo Badlands, the oldest sedimentary rocks are the late Miocene-Pliocene age Mount Eden Formation which is overlain by the Pliocene-Pleistocene San Timoteo Formation (Shuler, 1953; Dibblee, 1981; Albright, 1999). Both of these units were deposited in a basin which extended north from what is now the San Jacinto Valley and San Timoteo Badlands into the Upper Santa Ana River Valley and eastward into the Beaumont Plain-San Gorgonio Pass region.

These sedimentary rocks are overlain within most valleys by Quaternary-age alluvium and alluvial-fan deposits. Quaternary deposits may have also covered much of the Miocene-Pliocene strata but most of these have been largely removed and reduced by erosion. The Quaternary deposits are divided into old alluvium and young alluvium. The valleys in the region are filled with late-Quaternary-age young alluvium consisting of valley alluvium, stream-channel, stream-terrace, colluvium, and alluvial-fan deposits.

2.3 Regional Structure

The geological structure along the project is quite complex. This region lies between the San Andreas fault on the northeast and the San Jacinto fault on the southwest (Figure 2). These two major faults are highly active and converge northwesterly. As they become closer, several faults cross between them. The region where all of these faults intersect has been referred to as a tectonic knot. Matti et al (1992b) believe the structural complexity in the region is a result of transferring strain across a right step between the San Jacinto and the San Andreas faults.

The older structural history of the region is evident in the crystalline basement rocks. Geological relationships show igneous rocks to be intruded into ancient rocks (Paleozoic and Precambrian age) in Mesozoic time (approximately 100 million years ago). Sometime later, probably

relatively recently in geologic time, both groups of rocks were brought to the surface by geological forces.

Most major faults in the region are strike-slip faults (e.g., San Jacinto, San Andreas, Banning) but reverse faults such as the San Gorgonio Pass fault and normal faults also occur. The Redlands, Reservoir Canyon, and the Yucaipa Valley faults to the north of the project area, are normal faults within the Crafton Hills horst and graben fault system (Matti et al, 1992b). The Redlands and Reservoir Canyon faults dip northwesterly and displace Quaternary-age sediment downward on the northwest. The Yucaipa Valley fault system forms a graben with sediments on the north side of the valley displaced down to the southeast and sediments on the south side of the valley displaced down to the northwest, forming Yucaipa Valley in between. The normal faulting may be related to arching over northerly dipping thrust faults. Although late-Holocene-age surface displacements are rare, segments of the normal fault systems displace essentially all of the stratigraphic units in the region and are testimony to the ongoing active nature of tectonics in the region.

The southern edge of the tectonic knot is marked by the San Gorgonio Pass fault and the Banning fault (Figure 1). The Banning fault is a major branch of the strike-slip San Andreas fault that projects to just northeast of the site in close association with the San Gorgonio Pass fault which is a northerly dipping oblique-reverse fault. The Banning fault is not believed to be active west of Banning (Rasmussen, 1982).

The San Gorgonio Pass fault or a similar fault called the Cherry Valley fault extends onto the site from the east. The fault is recognized by tilted and folded beds, offset early to middle Quaternary strata, and geomorphic scarps, and therefore is potentially active; this fault is described in more detail below in Section 3.3

The Beaumont Plain fault consists of two generally north-northwest trending faults in older alluvium (Figure 2). These faults are part of a series of faults which displace late Pleistocene-age alluvial deposits in the vicinity of Beaumont. Matti et al (1985) indicate these faults have normal, dip-slip separation. The faults have strong geomorphic expression including 7 to 16-foot-high, east-facing scarps aligned with linear drainages and benches (Dames & Moore, 1987). There is no surficial expression of faulting in younger alluvium which overlies the fault traces. Based on these relationships, the Beaumont Plain fault zone is considered to be potentially active.

Smaller local faults, the Shadybrook, Singleton Ranch, and the Haskell Ranch lineament faults have been mapped south of the site area. Field investigations by Rasmussen and Associates (1978, 1983) and Dames and Moore (1987) could not preclude these faults and suggested that they may be potentially active. These faults do not impact the site.

3.0 SITE GEOLOGY

3.1 Site Physiography

The Oak Valley project area is along the western edge of the Beaumont Plain and near the eastern edge of a region referred to as the San Timoteo Badlands. The site is generally a well-vegetated grassland and somewhat more similar to a savanna or wooded savanna (Figure 3) than to true badlands which are characterized by little vegetation. The locally rugged relief in parts of the site is probably mainly a result of the area being capped by a hard, erosion-resistant, impermeable, ancient soil with a well-developed carbonate horizon (Figure 4). This hard surface protects the area from erosion, but once the cap is breached, the underlying uncemented materials are susceptible to erosion and formation of steep-sided canyons.

The site is characterized by flat mesas and ridges transected by northerly, westerly, and southwesterly draining valleys and canyons. The mesas and ridges are all at about the same elevation and represent a formerly flat to undulatory, gently southwest-sloping surface (Plate 1, Figure 3). Elevations are highest in the northeast and northern parts of the site where the ridge tops range from just above 2300 feet elevation in the northeast to about 2200 feet in the northwest part of the property. Elevations gradually decrease southwesterly to just below 2200 feet along the southern property margin, an elevation difference across the site of about 100 feet.

The valleys in the northern part of the property drain toward the north and are tributaries of Yucaipa Creek (Plate 1, Figure 1). Valleys in the east, west, and south drain primarily southwesterly and are tributaries of San Timoteo Creek. The two major valleys within the site are Covington Canyon which drains southwesterly from the northeast corner of the site to the south-central part of the site, and Burns Canyon which traverses the southeast corner of the site and drains southwesterly subparallel to Covington Canyon (Plate 1). Maximum relief between canyons and the adjacent ridge tops is commonly in the 100 to 130 foot range with a maximum of about 180 feet in Covington Canyon near the southern property line.

Valley profiles range from sharp and V-shaped in the smaller valleys to somewhat broad and U-shaped across the larger valleys (Photo 5). Although commonly steep, the valley walls are mostly covered with slopewash (colluvium) which provides soil that supports vegetation well. The slopewash and the vegetation obscure the nature of the underlying bedrock material.

The flat floors of the larger valleys are incised in some places by narrow second-order ravines or arroyos with nearly vertical walls (Figure 5B). In this report, the term arroyo is reserved for these recent, vertically sided, narrow incisions into the valley bottoms. In the largest canyons these arroyos are up to about 40 or 50 feet deep, cutting through the valley alluvium and into the San Timoteo Formation providing some of the better views of the bedrock character (Figure 5B).

Some of the smaller canyons also have these second-order arroyos but these are mostly on the order of 10 feet (+/- 5 feet) deep, but also commonly expose San Timoteo bedrock in the bottom.

3.2 Site Stratigraphy

The geologic units at the site consist of three basic units. The distribution of these units is shown on Plate 1. The predominant unit is the San Timoteo Formation (map symbol QTs) which underlies the entire site and can be considered the bedrock unit. The crystalline basement rocks described under the Regional Stratigraphy section are not exposed on the site and are too deep beneath the site (possibly as much as 5000 feet) to be significant for any of the facilities associated with this project. The San Timoteo Formation is overlain by young Quaternary alluvium (Qya) in the canyons and gullies. These deposits consist primarily of local erosional and weathering byproducts of QTs which have washed into the valleys from the surrounding slopes and mesas. The third unit (Qrp) is not really a stratigraphic deposit but rather a soil formed by long-term, deep weathering of the San Timoteo Formation. These units are described further below.

3.2.1 San Timoteo Formation

The San Timoteo Formation (map symbol QTs on Plate 1) is Pliocene (4-5 million years) to Pleistocene (0.5 to 0.8 million years) in age and commonly contains vertebrate fossils (Reynolds and Reeder, 1981; Smith 1983; Albright 1999). No fossils were found onsite during this investigation and none are reported in the previous investigations. The formation is divided into three members (Shuler, 1953). The formation onsite represents the upper member. The middle and lower members are present in the deep subsurface, and the entire formation along with probable older sediments reach a total thickness of over 5,000 feet beneath parts of the Oak Valley development. An oil well drilled in 1922 on the Shutt Ranch just west of the site reached a total depth of 5,358 feet in sedimentary rock, and a well drilled in 1933 on the Haskell Ranch south of the site reached 3,180 feet in sedimentary rocks (Dames & Moore, 1987).

The San Timoteo Formation is chiefly of fluvial (river) origin with local lacustrine (lake) deposits and is composed of beds of siltstone, sandstone, silty sandstone, claystone, and poorly sorted gravelly to bouldery sandstone and conglomerate (Figure 5B). Gravels in the unit are composed of quartz, plutonic (e.g., granite, diorite), metasedimentary, metaigneous, and metavolcanic rock types. The rock fragments are subangular to subrounded indicating short transport distance. These deposits were principally derived from rocks to the north and northeast in the San Gabriel and San Bernardino Mountains (Albright, 1999). The sediments in outcrop are generally friable to moderately indurated, and easily to moderately erodible. Bedding is generally poorly developed, gradational, and lenticular (Figure 5B).

3.2.2 Alluvium

The canyons and valleys within the site contain young alluvium of probable Holocene age. This alluvium appears to be primarily locally derived and is mapped as Qya on Plate 1. The material is generally composed of dark reddish-brown and dark brown, nonindurated, sand to silty sand with minor amounts of gravelly and bouldery sands. The alluvium is poorly bedded.

The young alluvium is generally expected to be relatively thin in small tributary canyons and thicker in the larger drainages and at the canyon mouths. The arroyos in the narrower parts of smaller valleys indicate thicknesses of only about 5 feet to as much as 20 feet. Two borings drilled in the middle parts of two of the larger canyons indicated about 40 feet of alluvium, but the relationships were not clear because the alluvium is derived from the San Timoteo Formation (QTs), and therefore is very similar. When the alluvium overlies weathered QTs, it is difficult to distinguish between them, especially in small-diameter borehole samples.

Parts of some of the most-recently incised arroyos have thin, loose, stream-channel sand and gravel alluvium in the very bottom of the channel. These deposits are the youngest deposits and are generally of historical or modern age. However, these are so thin and so widely scattered that they were not individually mapped and are included with the Qya map unit on Plate 1.

3.2.3 Relict Soil

The mesas and ridges throughout much of the site are underlain by red clayey and white carbonate-rich soils. These materials are mapped as Qrp on Plate 1. This material is primarily a product of long-term weathering of the San Timoteo Formation, although locally there may be some remnants of old alluvial deposits that once covered the Qrp surface and which underwent similar weathering and soil formation. Areas occupied by this Qrp unit have been mapped as old alluvial deposits (Qoa) by several previous investigators but close scrutiny reveals that the material is conformable with and generally grades imperceptibly into the underlying San Timoteo Formation. Some additional Holocene-latest Pleistocene weathering and soil formation has overprinted some of the older relict soils but the younger soils are relatively thin (1-2 feet).

The relatively flat surface of the tops of the ridges and mesas occupied by the Qrp unit are remnants of an ancient geomorphic surface which was stable for a long period of time, undergoing very little deposition or erosion. After lying stable for several tens of thousands to a few hundred thousand years, the area underwent uplift which led to erosion, downcutting, and some minor tectonic deformation.

The uppermost part of the Qrp unit is characterized by dark red (2.5YR and 5YR hues¹) silty and sandy clays with scattered pebbles. This red zone represents an argillic B soil horizon. The dark red color gradually gives way with depth to yellowish-brown or strong-brown (7.5YR and 10YR hues) silts, silty sands, and gravelly sands which represent less-weathered but strongly oxidized San Timoteo Formation. The lowest part of the Qrp unit and the uppermost part of the San Timoteo Formation consist of white to very-pale-brown (10YR) calcareous silts, sands, and gravelly sands (Figure 4). This lower zone represents a pedogenic carbonate soil horizon or K Horizon. This carbonate horizon occupies much of the ground surface in the eastern and northeastern part of the site where it commonly is characterized by nearly horizontal, hard, impermeable layers (Figure 3). These carbonate soils represent a Stage III or Stage IV plugged K Horizon.

All together the Qrp unit ranges from a few inches thick to several feet thick (locally up to about 20 feet thick). Much of the site area, primarily the northeastern part is characterized by the white surface cap consisting of the calcareous zone (Figure 4B). The calcareous materials were probably overlain by an A soil horizon and the dark red B soil horizon at one time but these have been stripped away by erosion as the region underwent uplift and the resulting incision that led to the present valley-and-ridge topography. The high degree and great thickness of mature soil development within this unit indicates great age, perhaps on the order of a few hundred thousand years. This unit is similar to the Qvoa and Q₃ unit of Matti et al (1992a), Kendrick et al (2003), and Albright (1999) who estimate an age on the order of about 300,000 to 500,000 years.

The geotechnical properties of the Qrp unit vary depending upon which of the zones is present. In general the unit is very firm and hard and will provide good foundation support. The red clayey upper zone exhibits a polygonal cracking indicative of a high clay content which may have an expansive character. The light-colored calcareous zone may be corrosive to some materials.

3.2.4 Artificial fill

There are a few local areas with artificial fill within the site. Generally these are small berms or dams built to retain water for livestock. These are widely scattered throughout the site (Plate 1); some of the larger ones are identified on Plate 1 but many are too small to be adequately shown at the scale of Plate 1. The only area with significant fill is the valley just south of the trailer park in the eastern part of the site near 7th Street. This area presently serves as an equipment storage area and has been a borrow area. These activities have resulted in dirt, brush, and trash having been disturbed and pushed into the arroyo. Several other areas along the creeks also have local accumulations of trash; like many rural areas, it appears to have been a common practice to

¹ The numerical designation of colors used in this report represents the Munsell Soil color system, a standard for geological and geotechnical investigations.

dump trash into small canyons and creek beds. These areas are small and not shown on Plate 1, and generally not significant to the project.

3.3 Site Structure

Oak Valley is located on the east limb of a regional asymmetric, northwest-trending anticline in the San Timoteo Formation. The axis of this fold is located west and southwest of the project boundaries and parallels the San Jacinto fault. Bedding in the San Timoteo Formation on the west limb of the fold dips 15 to 50 degrees southwest. On the east limb, including the area beneath Oak Valley, bedding typically strikes northwesterly to northeasterly and generally has gentle dips ranging from nearly horizontal to about 5 degrees to the north with local increases in dip to about 15 degrees. Steeper dips occur only in association with deformation along the north side of the Cherry Valley fault in the eastern part of the area (Plate 1).

A gentle anticline and syncline is exposed in the arroyo walls of Covington Canyon in the central part of the site. This field reconnaissance also mapped two small northeasterly dipping faults in the walls of the arroyo near the folds (Plate 1).

The only fault known in the site area is the Cherry Valley fault. This fault extends into the southeast corner of the site area (Figure 2 and Plate 1) from offsite to the east (Matti et al, 1985, 1992; Dames and Moore, 1987; Rasmussen and Associates, 1984, 1988). There is uncertainty and has been some discussion in previous reports as to whether the fault should be called the Cherry Valley fault or the western extension of the San Gorgonio Pass fault. Rasmussen Associates (1988) believe it is the Cherry Valley fault or a branch of the Cherry Valley fault.

Field investigations of the Cherry Valley fault by Dames & Moore (1987) were inconclusive regarding the recency of its activity; they considered it to be potentially active based on the work of Matti et al (1985) that indicated that the San Gorgonio fault displaces Holocene deposits east of the site, and recommended a restricted-building zone or setback zone. Subsequent work onsite by Rasmussen Associates (1988) suggested that the fault is overlain by the unfaulted mature soil (Qrp) and, therefore they concluded that the fault should not be considered active.

Our investigation revealed that the fault can be recognized by an eroded and degraded geomorphic scarp associated with folding and displaced San Timoteo Formation strata. In the southeast corner of the site, the scarp is 70 to 80 feet high (Figure 3B and Plate 1). Westerly, the scarp height steadily diminishes to 20-30 feet high and disappears east of the Qrp mesa just east of Covington Canyon (Plate 1). In Covington Canyon, two faults are present in the arroyo walls. These have similar low- to moderate-angle dips (28 and 42 degrees), and are in direct alignment with the scarp to the east. The faults in the arroyo extend upward to young alluvium but do not appear to offset the alluvium, albeit exposures are somewhat obscured near the Qya/QTs contact.

The QTs strata in proximity to the faulting in the arroyo are folded and indicate post QTs (i.e., late Quaternary-age) tectonic deformation. The faults in the arroyo are relatively simple, sharp fractures without much evidence of strong shearing, and thus do not appear to represent a major regional fault. The amount of displacement could not be determined, but it appears to be greater than the depth of the arroyo which is more than about 20-30 feet at that locality.

Reevaluation of the Rasmussen Associates (1988) work indicates that they based their conclusions on the interpretation that the fault is overlain by the Qrp surface which is several hundred thousand years old. However, their main trench did not conclusively identify any major fault but that several minor fractures and folded QTs strata were not directly overlain by the Qrp. It seems evident that the scarp and folding occurred after formation of the Qrp surface, and that uplift along the fault or even fault displacement led to removal of the Qrp along the fault zone. These relationships suggest that the faulting occurred in late-Quaternary time after formation of the Qrp soil/surface, i.e., less than the 300,000-500,000 years ago. This would result in classifying the fault as a late-Quaternary-age, or potentially active, feature. Whether the fault has been active in early Holocene time, and therefore would be active, can not be established based on the present data.

4.0 SEISMOLOGY

4.1 Historical Seismicity

The site is in seismically active southern California and is subject to shaking from both local and distant earthquakes (Figure 6). The seismicity map shows only the larger ($M > 4$) events in historical time. The site is along the boundary between the North American and Pacific plates which comprise major faults. The San Andreas Fault, the major fault of the plate boundary system, has not had substantial earthquake activity along this segment in historical time. In spite of the paucity of recent earthquakes on the San Andreas Fault, geologic evidence indicates numerous Holocene displacements. In contrast, historical seismic activity has been abundant on the San Jacinto fault to the west of the site, and it is one of the most seismically active faults in the world. The San Jacinto system (which includes the Imperial fault) has generated about a dozen moderate- to large-magnitude ($M > 6$) earthquakes in historical time. Small earthquakes are quite common in the area between the San Andreas and San Jacinto faults, i.e., in proximity to the Banning, San Geronio Pass, and Crafton Hills fault systems.

The largest historical earthquakes in the area were the 1923 event on the San Jacinto fault ($M_S = 6.3$), the 1948 event near Desert Hot Springs ($M_S = 6.5$), and the 1986 North Palm Springs earthquake ($M_L = 5.9$). The earlier events are poorly located so the sources are uncertain, whereas the 1986 event probably occurred on the Banning fault.

Microseismicity in the Yucaipa Valley area may suggest normal faulting but alternate interpretations support reverse faulting as well (Matti et al, 1992). A small earthquake (M= 4.5) occurred just northwest of the project area in 1998. This event is notable because it yielded a normal-fault focal mechanism, and it may have been associated with the Crafton Hills fault system where it intersects the San Jacinto fault.

4.2 Faults

Figure 2 shows the location of the major active faults in the site region. Table 1 lists the nearest active faults and their estimated maximum earthquakes. The California Geological Survey has not designated any Earthquake Fault Zones (formerly known as Alquist-Priolo Special Studies zones) within the site area that require special considerations such as restricted building or setback zones (MBA, 1988). However, setback zones may be prudent on the Cherry Valley fault (see Section 5.1)

**TABLE 1
MAXIMUM EARTHQUAKES**

Fault Name	Type of Slip⁽¹⁾	Maximum Earthquake	Distance From Site (km)
San Andreas	ST	8.0	10
San Jacinto - Claremont Segment	ST	7.5	5
San Gorgonio Pass	RO	7.0	12
Beaumont Plain	N	6.75	6
Crafton Hills - Yucaipa Valley Segment	N	6.5	1.4
Cherry Valley	RE	6.25	0

Notes: 1) ST = Strike Slip RE = reverse; RO = reverse oblique; N = normal; XX = Unknown

4.3 Maximum Earthquakes

The largest earthquakes (Upperbound Earthquakes) estimated for the major faults in the region are listed on Table 1.

4.4 Deterministic Ground Motion

The California Building Code (which is based on the 1997 Uniform Building Code) indicates the site is within seismic zone IV. Oak Valley is largely located in the Riverside County Groundshaking Zone V. Major active and potentially active fault zones in the site region are listed on Table 1.

The seismic design parameters from the California Building Code (= UBC) are listed on Table 2.

**TABLE 2
CBC SEISMIC DESIGN PARAMETERS**

Seismic Zone 4	Z=0.4
Fault Type	B
Distance	~ 5 (to west property line); 10 km (to east property line)
Near Source Factors	Na = 1.0 Nv = 1.0 (east) to 1.2 (west)
Soil Profile Type	Soil Profile Type (SC)
Seismic Coefficients	Ca = (0.40) (Na) = 0.40 Cv = (0.56) (Nv) = 0.56 (east) to 0.67 (west)

Note that the site is about 5 km wide in the east-west direction so the west side of the site is about 5 km closer the San Jacinto fault than the east side which is about 10 km away. This leads to different design values for facilities depending on where they are located.

4.5 Probabilistic Ground Motion

Seismic design methods and codes have been refined somewhat since the EIR was prepared. The California Geological Survey (CGS) in association with the U.S. Geological Survey (USGS) has developed a method for probabilistic seismic hazard analysis. Figure 7 is a map showing the site relative to the regional peak ground motion accelerations determined by the CGS.

According to the CGS method, ground motions with a 10% probability of being exceeded in 50 years (~ 475 year return period) expressed as a fraction of the acceleration due to gravity (g) are listed on Table 3 for the east and west sides of the site area. Three values of ground motion are shown, peak ground acceleration (Pga), spectral acceleration (Sa) at a short period (0.2 second)

and a moderately long period (1.0 sec). Each ground motion value is shown for the two different site conditions within the site; soft rock (San Timoteo Formation) and alluvium (Qya).

As shown on the Table 3, the ground motions on the west side of the property are a little larger than ground motions on the east side. This is because the ground motion is controlled by the San Jacinto fault which is closer to the west side of the property.

**TABLE 3
PROBABILISTIC GROUND MOTIONS**

GROUND MOTION	SOFT ROCK	ALLUVIUM
<i>WEST SIDE OF SITE AREA</i>		
Pga	0.69	0.69
Sa 0.2 sec	1.659	1.659
Sa 1.0 sec	0.77	0.888
<i>EAST SIDE OF SITE AREA</i>		
Pga	0.623	0.623
Sa 0.2 sec	1.493	1.493
Sa 1.0 sec	0.713	0.823

5.0 GEOLOGICAL HAZARDS

5.1 Surface Faulting

There is one fault known within the site. The Cherry Valley fault extends into the southeast corner of the site and crosses the site to at least Covington Canyon (Plate 1). The age of latest displacement on this fault is uncertain (see Section 3.3). It appears to have been very active in late-Quaternary time but whether that activity continued into the early Holocene time has not been established. The fault is not known to displace latest Holocene young Alluvium (Qya) but it is not overlain by Qya in many places. The degree of scarp degradation indicates that the fault is not highly active. However, the possibility of early Holocene displacement cannot be ruled out based on the present data. In view of the uncertainty and its projection through two proposed school sites, we recommend 100 foot setback zones on each side of the fault. Before habitable structures, including schools, can be built within this zone, additional fault studies (preferably trenching) should be conducted.

5.2 Erosion

The steep-sided canyons and the mesa/valley-type topography of the site vicinity indicate that the geologic formations in the area, specifically the San Timoteo Formation (QTs), are susceptible to erosion. Although the San Timoteo Formation materials are quite dense and hard compared to the Qya deposits, they are not cemented and are friable. Consequently when they are subjected to flowing water they are erodible.

5.3 Landslides

There are few landslides within the site region. Although there is a potential for landsliding within the site, the hazard in general does not appear to be great. Bedding in the predominant geologic formation, the San Timoteo Formation, is poorly developed, and the formation is not highly fractured so there are few planes of weakness that would promote widespread landsliding. Furthermore, the orientation of the bedding is generally horizontal to very low angle (< 5 degrees) which is not prone to substantial landsliding.

A few small slides are observed along the major stream channels where erosion and undercutting occasionally destabilize the adjacent slopes (Plate 1). Undoubtedly there are more small slides and earth flows that are not shown on the map. Slopes throughout the site are commonly steep; the EIR (MBA, 1987) provides maps showing areas with slope gradients greater than 15% and 25% which includes most of the slopes along the sides of the canyons and valleys. Any building planned for the areas above or below these slopes should account for the landsliding hazard by setting structures back from the tops of the slopes and away from the bottom of the slopes in accordance with applicable building codes (see Geotechnical Engineering Recommendations Section 7.0).

Many of the slides appear to be shallow earth flows or debris flows. These are generally shallow (less than about 10 feet thick). These features appear to be relatively random and not structurally controlled; rather they appear to be related to downslope flow of thicker accumulations of slope wash within small side slope gullies and swales. During intense periods of rainfall they become saturated. Although these are generally not devastating events, the larger ones can damage buildings and facilities that lie in their path. These types of features also commonly fail during strong seismic events.

Rasmussen Associates (1984) reports evidence for deep-seated and large translational and/or rotational landslides. Some of the deep-seated landslides were interpreted to represent ancient landslides with now have little geomorphic expression. Some of the older, larger and deep-seated landslides identified by Rasmussen Associates exhibit little geomorphic expression and

therefore other similar subtle landslides may exist on the site. Reactivation of older landslides on-site is a potential hazard and could be aggravated in some situations by grading.

If structures for human occupancy are to be placed adjacent to the known or suspected landslides or next to the steep slopes, subsurface borings or test pits will be necessary to determine the extent and subsurface geometry of the landslides. If any development, including roadways, is planned in the immediate vicinity of the landslides or the steep slopes, mitigating measures such as buttressing of the slopes or reducing slope gradients, should be implemented (see Section 7).

5.4 Liquefaction

State of California Seismic Hazard Zones Maps have not been issued for the quadrangles where the subject site is located. Previous investigations (e.g., EIR) indicated that all of the valleys had a high liquefaction potential. However, based on our field investigation, liquefaction and dynamic settlement resulting from the effects of strong ground shaking are not expected to be widespread at the site due to the great depth to ground water and the relative density of the underlying soils. Many of the smaller valleys identified in previous reports as having a liquefaction potential are underlain by firm bedrock (QTs) at shallow depths (e.g. 5 feet) and therefore might easily be developed providing that the recommendations given in Section 7 are followed.

6.0 HYDROLOGY

6.1 Surface Water

There are no major perennial streams passing through the site. The site is crossed by several ephemeral streams that may flow during times of heavy precipitation such as the winter rainy season or during the occasional summer monsoonal thunder showers. Drainage within these local channels in the northern part of the site is toward the north whereas drainage in the rest of the site is toward the southwest. The two largest valleys are Covington Valley in the central part of the site and Burns Valley in the southeastern part of the site (Plate 1).

There are no lakes within the site but there are several small dams and water-retaining berms that have been used for stock watering. At the time of this investigation, these were nearly all dry and many were dysfunctional.

Severe erosion that led to the arroyo cutting within some existing canyons (e.g. Covington Canyon) indicate that substantial volumes of runoff occasionally occur at the site during prolonged and intense precipitation. However, the flow appears to have been completely

contained within the existing valleys so the flooding hazard for sites built on high ground appears minimal.

6.2 Ground Water

Water wells south of the site produce water from deep within the San Timoteo Formation (Rasmussen Associates, 1984). Water levels in three wells are currently from greater than 100 feet below the ground surface. Data from the South Mesa No. 5 well near the northern property line indicates the water table in June 1984 was approximately 270 feet below the surface. A regional hydrogeologic study by Bloyd (1971) suggests that the water table beneath the site slopes to the west at a gradient of approximately 150 feet per mile. Integration of the well data with water gradient interpretations by Bloyd (1971) suggests that water levels could be approximately 140 feet below the lowest ground surface in the easternmost portion of the site. There has been no analysis of the possible affect of faults on the ground water, and data are of insufficient quantity to determine any such effects. The presence of thick caliche zones near the reverse/thrust fault suggests that this fault has been a ground-water barrier in the past.

Areas of artesian ground-water conditions have occurred both north and south of the site area during the early 1900s (Mendenhall, 1905). Yucaipa Valley was an artesian ground water basin a few decades ago but over pumping has severely reduced ground water levels in the valley. Artesian conditions also occurred in San Timoteo Canyon

No ground water was encountered in borings drilled for this investigation which extended to depth of about 50 feet. No springs or evidence for an existing shallow ground-water body was observed during this investigation. The fine-grained units of the San Timoteo Formation exposed in the very bottom of several local canyons were moist where protected from direct sunlight, perhaps as a result of rains within the past couple months. Ground water may become perched following prolonged periods of high precipitation. Such perching is expected to be local because of the poor lateral continuity of strata.

7.0 GEOTECHNICAL ENGINEERING RECOMMENDATIONS

Based on our subsurface investigation and analysis, development of the site is considered feasible from a geotechnical point of view. It is our opinion that most building sites will be free of hazard from landslide, settlement and slippage provided that the following general recommendations are incorporated into the design criteria and project specifications. However, the project is still in the conceptual stage and specific grading and development plans have not been developed. More-specific recommendations will require additional more-extensive geotechnical investigations.

7.1 Earthwork Recommendations

7.1.1 General Earthwork and Grading

All earthwork and grading should be performed in accordance with the recommendations of this report and all applicable requirements of the Grading Code of the County of Riverside. Grading should also be performed in accordance with applicable provisions of the attached "Standard Grading Specifications" in Appendix C, unless specifically revised or amended herein.

7.1.2 Site Clearing

All weeds, grasses, brush, shrubs, trees and similar vegetation existing within areas to be graded should be stripped and removed from the site. Clearing operations should include the removal of all trash and debris. Trees and large shrubs, when removed, should be grubbed out so as to include their stumps and major root systems, and these organic materials removed from the site. During site grading, laborers should clear from fill soils any roots, tree branches, and other deleterious materials missed during initial clearing and grubbing operations.

The project geotechnical consultant should be notified at the appropriate times to provide observation and testing services during clearing operations to verify compliance with the above recommendations. In addition, should any buried structures or unusual or adverse soil conditions be encountered during grading that are not described or anticipated herein, these conditions should be brought to the immediate attention of the project geotechnical consultant for corrective recommendations.

7.1.3 Excavation Characteristics and Potential for Generation of Oversize Rock

Based on our limited exploratory borings, surficial deposits of man-made fill, colluvium, alluvium, older alluvium, and sedimentary bedrock materials of the San Timoteo Formation within the lower topographic areas of the site are expected to be readily excavatable with conventional heavy duty earthmoving equipment.

The hills and mesas of the site are underlain primarily by the San Timoteo Formation consisting of sandstone, siltstone, and conglomerate. Light to heavy ripping can be expected in this formation. The formation contains abundant cobbles and some boulders which may produce significant numbers of oversized rock. Cuts made into the San Timoteo Formation can be expected to generate a significant amount of cobbles greater than 6 inches in diameter and some boulders to about 2 and possibly 3 feet in diameter with the total percentage of oversize rock being on the order of 5 percent or less.

Any rock exceeding 12 inches in maximum dimension should either be disposed of offsite or buried in the deeper fills in accordance with Plate SG-4, Appendix C (Standard Grading Specifications). The "Disposal of Oversize Rock" is discussed in a subsequent section.

7.1.4 Removal of Existing Fill Deposits

All existing deposits of artificial fill should be removed to competent underlying native soils or bedrock, however, the actual limits and depths of removal of existing unsuitable fill materials will have to be determined during grading. The excavated fill materials may be replaced as properly compacted fill within the site provided that they are first cleared of any trash, construction debris or oversize rock.

7.1.5 Removals and Canyon Cleanouts

All existing low-density surficial soils in areas to receive compacted fill should be removed to underlying competent bedrock or competent native soils approved by the project geotechnical consultant. In general, low-density surficial soils include any artificial fill deposits, alluvium, colluvium, and highly weathered surficial bedrock formation. Throughout the majority of the site, recommended depths of remedial removal will extend into competent bedrock or competent native soils. Competent materials are defined as undisturbed, relatively unweathered and non-porous bedrock materials and dense native soils possessing an in-place relative compaction of at least 85 percent and a degree of saturation of at least 70 percent; however, where these materials exhibit a relative compaction of 90 percent or greater, no specific degree of saturation is necessary.

Similar removals should also be performed in areas of shallow cut where low-density surficial soil deposits or highly weathered bedrock are not removed in their entirety. Based on our boring and laboratory data, depths of removal are expected to vary from approximately 5 to in excess of 25 feet. The actual depths and horizontal limits of removals will have to be determined during future geotechnical investigations and grading.

7.1.6 Ground Water

Static ground water is not expected to be encountered during grading; however, minor amounts of perched ground water overlying dense bedrock or fine-grained materials may be encountered during canyon cleanouts, especially if grading is performed during the winter months. Temporary diversion and control of locally perched ground water may be necessary during installation of canyon subdrains and initial placement of compacted fill, particularly in the lower portions of the canyon drainages. If encountered, drying of wet or saturated soils excavated from

canyon bottom areas may be necessary to obtain near-optimum moisture content in order to achieve proper compaction.

7.1.7 Canyon Subdrains

To mitigate the potential build-up of hydrostatic pressures below compacted fills due to infiltration of surface waters, subdrains should be installed along the axes of all major canyons and tributaries to be filled where the depth of fill exceeds approximately 15 feet. Subdrains should be constructed in accordance with Plate SG-1, Appendix C. Those portions of the ends of the subdrains that are underlain by compacted fill materials rather than suitable native materials should be constructed with solid pipe rather than perforated pipe. Actual subdrain locations should be determined in the field during grading based on exposed geologic conditions; however, the project geotechnical consultant should generally designate the location of the subdrain systems on the approved tract grading plans.

7.1.8 Fill Placement

Following removal of unsuitable surficial materials and weathered bedrock, exposed bottom surfaces in areas approved for placement of fill should first be scarified to a depth of 6 inches, watered or air dried as necessary to achieve near optimum moisture conditions, and compacted to a minimum relative compaction of 90 percent.

All fills should be placed in 6- to 8-inch-thick maximum lifts, watered or air dried as necessary to achieve near optimum moisture conditions, and then compacted to a minimum relative compaction of 90 percent.

The laboratory maximum dry density and optimum moisture content for each change in soil type should be determined in accordance with Test Method ASTM D 1557-00.

7.1.9 Benching

Fills placed against canyon walls, on natural slope surfaces inclining at 5:1, horizontal to vertical, or steeper, and against temporary backcut slopes associated with construction of stabilization fills should be placed on a series of level benches excavated into competent bedrock or competent native soil materials. These benches should be provided at vertical intervals of approximately 3 to 5 feet. Typical benching details are shown on Plates SG-5 through SG-8, Appendix C.

7.1.10 Disposal of Oversize Rock

As noted previously, oversize rock is expected to be encountered during grading operations. Oversize rock is defined as hard boulders or irreducible cemented bedrock fragments exceeding 12 inches in maximum dimension. Oversize rock generated during grading operations should be removed from the site or placed in the lower portions of the deeper fills utilizing the typical detail shown on Plate SG-4, Appendix C. Any oversize materials buried on site should be placed individually or in wind rows, and in a manner to avoid nesting, and then completely covered with finer-grained on-site earth materials. The finer-grained materials should be thoroughly watered and rolled to ensure closure of all voids. Oversize rock should not be placed within the upper 10 feet of finish grade within the building areas or street areas where they may interfere with footing and utility trenches, or in areas where they may interfere with the future construction of swimming pools and/or spas.

7.1.11 Processing of Cut Areas

In shallow cut areas where unsuitable surficial materials are not removed in their entirety, these materials should be overexcavated to underlying competent materials and then brought back to grade with properly compacted fill. In deep cut areas where competent materials are exposed at grade, no special remedial work such as scarification or recompaction will be required.

Earth materials within the site is expected to range from non-expansive to highly expansive. Due to the potential for distress to building foundations due to the adverse effects of expansion and shrinkage, it is recommended that all building pads be underlain by soil or bedrock materials that exhibit very low to medium expansion potentials. Highly expansive soils should not be placed within 4 feet of proposed grade within building pad areas. In addition, building pads that are cut down to grade and expose highly expansive earth materials should be over-excavated to a depth of at least 3 feet below pad grade and replaced with fill materials exhibiting very low to medium expansion potentials.

7.1.12 Cut/Fill Transition Lots

To mitigate distress to residential structures related to the potential adverse effects of excessive differential settlement, cut/fill transitions should be eliminated from all lots where the depth of fill placed within the "fill" portion exceeds proposed footing depths (e.g., 12 inches and 18 inches for one-story and two-story structures, respectively). This should be accomplished by overexcavating the "cut" portions and replacing the excavated materials as properly compacted fill. Recommended depths of overexcavation will depend on maximum depths of compacted fill placed on the "fill" portions, but will generally follow the guidelines given below. Horizontal

limits of overexcavation should extend to within approximately 1 to 2 feet of property lines and/or the tops and toes of slopes.

<i>Depth of Fill</i>	<i>Depth of Overexcavation</i>
Up to 3 feet	Equal depth
3 to 6 feet	3 feet
Greater than 6 feet	One-half of greatest fill depth placed on the "fill" portion; to 15 feet maximum

Additional lots may require overexcavation due to cut/fill transitions created during grading such as unsuitable material removal, incised haul road areas through cut lots, construction of stabilization fills, etc. The required depths of overexcavation should be based on actual conditions encountered during grading using the guidelines presented above.

7.1.13 Deep Fill/Shallow Fill Transitions

To mitigate the detrimental effects of excessive differential settlement, deep fill/ shallow fill transitions should also be eliminated from all building areas. This should be accomplished by overexcavating the "shallow" fill portions of each building area and replacing the excavated materials as properly compacted fill. Generally, the depths of overexcavation should equal one-half the thickness of the maximum depth of fill underlying the building area to a maximum depth of 15 feet.

7.1.14 Volumetric Changes

Volumetric changes in earth quantities will occur when excavated on-site soils and bedrock materials are replaced as properly compacted fill. Following is typical values of shrinkage and bulking factors for the various geologic units present on site. These typical values are based on in-place densities of the various materials and on the estimated average degree of relative compaction achieved during grading.

Artificial Fill (Af)	Shrinkage of 15% to 20%
Alluvium/Colluvium (Qya)	Shrinkage of 15% to 20%
San Timoteo Formation (QTs)	Bulking of 0% to 3%

(All of the above are exclusive of oversize rock materials that may need to be removed from the site if not placed properly within rock disposal areas.)

The actual shrinkage or bulking that will occur during grading will depend on the average degree of relative compaction achieved.

A subsidence estimate at 0.10 to 0.15 feet may be anticipated as a result of the scarification and recompaction of the exposed ground surfaces within the removal areas.

The above estimates of shrinkage and subsidence are intended for use by project planners in determining earthwork quantities and should not be considered absolute values. Contingencies should be made for balancing earthwork quantities based on actual shrinkage and subsidence that will occur during grading. Due to uncertainties in the anticipated amounts of shrinkage or bulking of the various earth materials within the site and due to uncertainties that will occur due to the removal and disposal of oversize rock materials, it is recommended that several earthwork balance test plots be performed during the initial stages of grading. These test plots can be performed to determine the initial volume of soil excavated within the test plot area and the final volume of soil placed and compacted in the fill area in order to determine the actual as-graded shrinkage amounts of the onsite earth materials.

7.2 Cut Slopes

7.2.1 Gross Stability of Cut Slopes

Recommended remedial removal of unsuitable surficial soils is anticipated to essentially eliminate any major cut slopes exposing alluvial, or colluvial materials and is expected to result in these slopes being constructed as fill slopes. Therefore, the majority of the major cut slopes within the site will expose San Timoteo Formation sedimentary bedrock materials. In general, these cut slopes will be constructed at a slope ratio of 2:1, horizontal to vertical. For conditions where cut slopes exhibit adverse geologic conditions, such as out-of-slope bedding, they need to be removed and replaced with buttress fills. The buttress fills should be constructed in accordance with the typical detail shown on Plate SG-2, Appendix C. The bottoms of the buttress fill keys should be tilted back at a minimum of 2 percent towards the heel of the key. As the buttress fills proceed up slope, a minimum fill width of 15 feet should be maintained to allow proper compaction of the fill layers. Internal backdrains should also be installed in each buttress fill to mitigate a potential buildup of excessive hydrostatic pressures. Backdrains should be constructed in accordance with the details shown on Plates SG-2 and SG-3, Appendix C. Backdrain locations should be determined during grading based on local topography and the most feasible exit points for outlet pipe.

7.2.2 Stability of Temporary Backcut Slopes

The stability of temporary backcut slopes associated with buttress fill and stabilization fill construction is dependent on many factors which include slope angle, height, geologic structure of unsupported bedrock, shear strength along planes of weakness, groundwater conditions, nuisance water, and the length of time temporary cuts remain unsupported. Consequently, there may exist a risk of backcut failure during excavation of basal fill keys for buttress fills and stabilization fills. In order to maintain temporary stability, the backcuts should be analyzed by the project geotechnical engineer and, if needed, should be laid back at a relatively flat slope ratio. In order to minimize the potential for backcut failures, the following techniques should also be considered:

- All basal fill keys should be excavated, observed by the project geologist, and then filled in the shortest practical period of time. Keyway excavations should never be allowed to stand open for prolonged periods of time,
- Provisions should be made for preventing nuisance water and rainwater from collecting and ponding in keyway excavations,
- Grading equipment and other construction traffic should never be allowed to traverse along the tops of temporary backcut slopes, and
- In addition to the above, all OSHA requirements should be followed with respect to excavation safety.

7.3 Fill Slopes

7.3.1 Fill Slope Construction

Fill slopes are expected to be constructed at a slope ratio of 2:1, horizontal to vertical. It is expected that low to medium height fill slopes constructed with on-site soil and/or bedrock materials will be grossly stable to a maximum height of 30 feet. Higher fill slopes should be analyzed when specific plans for such slopes are available. Fill slopes should be constructed as recommended below. The surface compaction recommendations provided below for fill slopes should also be applied to buttress and stabilization slopes.

- **Fill Keys**

Fill keys excavated into competent bedrock or competent bearing soils will be required at the base of all proposed fill slopes to be constructed on slope surfaces inclining at 5:1 (H:V) or steeper. The fill keys should be excavated to a minimum depth of 2 feet into competent materials and have a minimum width equal to one-half of the slope height, or 15 feet, whichever is greater. The bottoms of the fill keys should be tilted back at a minimum of 2

percent towards the heel of the key. Internal backdrains will be required in the keyways to prevent entrapment of irrigation water and rainwater in the key bottoms. Typical details for construction of the backdrains are shown on plates SG-2 and SG-3, Appendix C.

- Surface Compaction

The finish surfaces of all fill slopes should be compacted to a minimum relative compaction of 90 percent. Final surface compaction should be achieved by overfilling the slopes during construction, backrolling the overfilled slope surfaces at vertical intervals not exceeding 4 to 5 feet, and then trimming the slopes back to the compacted inner core. Where this procedure may not be practical, surface compaction should be obtained by backrolling during construction to achieve at least 90 percent relative compaction within 6 to 8 inches of the finish surfaces. This initial back-rolling should be performed at vertical intervals not exceeding 4 to 5 feet. Final surface compaction should then be achieved by rolling the slope surface with a cable-lowered sheepsfoot and then re-rolling with a grid roller. During final surface compaction, it is critical that the moisture content of the surface soils be maintained at near optimum moisture content or slightly higher.

- Fill-Over-Cut Slopes

Where cut-to-fill transition slopes are proposed, a keyway excavated into competent bedrock or dense native soil should be provided at the contact. The keyway should be at least 15 feet wide and tilted back into the slope at a minimum gradient of 2 percent. Where cut-to-fill transition contacts vary from about vertical to a few degrees from vertical, benching of the fill portion into the cut portion will be difficult and may create a potential slip surface due to inadequate benching. Therefore, overexcavation of the cut portion and reconstruction of the entire slope with compacted fill is recommended.

7.3.2 Surficial Stability of Fill Slopes

Fill slopes are proposed to be constructed at a maximum slope ratio of 2:1, horizontal to vertical. The surfaces of the fill slopes within the site will be comprised of fill materials that consist of reconstituted native bedrock materials and (to a much lesser extent), colluvium and alluvium. Therefore, surficial slope stability calculations should be performed using strength properties of fill slope materials and for a depth of saturation of 4 feet below the slope face and assuming an infinite slope with seepage parallel to the slope face.

7.4 Stability of Natural Slopes

7.4.1 Gross and Surficial Stability

Gross and surficial stability of natural slopes left in-place should be determined once the site specific plans are available. There are many steep slopes within the subject site and these slopes have gradients in excess of 25 percent (Michael Brandman Assoc., 1988). Stability of all natural slopes should be analyzed when specific plans for such slopes are available.

7.5 Geotechnical Observation

Observation of the clearing operations, removal of low density surficial soils, keyway excavations, and general grading procedures should be performed by a representative of the project geotechnical consultant. It should be the grading contractor's responsibility to notify the project geotechnical consultant when fill areas and fill keys are ready for observation. A representative of the project geotechnical consultant should be present on site during all major grading operations to verify proper placement and adequate compaction of all fills, as well as to verify compliance with the other recommendations presented herein.

7.6 Seismic Design Considerations

7.6.1 Ground Motions

Structures within the site should be designed and constructed to resist the effects of seismic ground motions as provided in Sections 1626 through 1633 of the 1997 Uniform Building Code and the California Building Code. The method of design will be dependent on the seismic zoning, site characteristics, occupancy category, building configuration, type of structural system, and on the building height. Section 4 provides data for both deterministic and probabilistic seismic design.

7.6.2 Secondary Seismic Hazards

Secondary effects of seismic activity normally considered as possible hazards to a site include several types of ground failure as well as induced flooding. Various general types of ground failures which might occur as a consequence of severe ground shaking of the site include landsliding, ground subsidence, ground lurching, shallow ground rupture and liquefaction. The probability of occurrence of each type of ground failure depends on the severity of the earthquake, distance from faults, topography, subsoils and groundwater conditions, in addition to other factors.

7.7 Preliminary Foundation Design Recommendations

7.7.1 Building Clearances From Ascending Slopes

Building clearances from adjacent ascending slopes are unknown at this time. Such information is typically shown on precise grading plans that are not available at this time. However, to conform with Figure 18-I-1 of the 1997 Uniform Building Code, building clearances of one-half of the total slope height to a maximum of 15 feet should be maintained between the buildings and the toes of the adjacent ascending slopes.

7.7.2 Footing Setbacks from Descending Slopes

Building setbacks from adjacent slopes are unknown at the present time and will be shown on the future precise grading plans. However, to conform with Figure 18-I-1 of the 1997 Uniform Building Code, Chapter 18, Division I, building footings to be constructed on or near descending slopes should be deepened to provide a minimum footing setback of $H/3$ (one-third the slope height). The footing setbacks should be 5 feet minimum where the slope height is 15 feet or less, and vary up to 40 feet maximum where the slope height is 120 feet or more. The footing setbacks should be measured along a horizontal line projected from the lower outside bottom edges of the footings to the face of the adjacent slope.

7.7.3 Allowable Soil Bearing Capacity

Bearing capacity of shallow footings supported on site soils depends mainly on soils strength parameters, footing width and depth of embedment. Recommendations for allowable bearing capacity should be provided by the project geotechnical engineer once the building loads and other pertinent soils and structure information become available for each individual building pad. General guidelines for design of footings based on the soil expansivity and other engineering characteristics of the onsite soils are provided in the "Minimum Footing and Floor Slab Recommendations" section of this report.

7.7.4 Footing Settlements

As previously recommended, all unsuitable artificial fill, alluvium and colluvium materials and weathered bedrock material will be removed down to either competent bedrock or competent native materials and then replaced as properly compacted fill. Based on these conditions, post-grading settlements beneath the site will result from settlement of the bedrock and competent native soils to be left in-place due to their own weight and the weight of new fill materials, settlement of the proposed fills due to their own weight, and settlement of the near-surface soils due to the weight of the new buildings.

Due to the dense to very dense and well-consolidated nature of the bedrock and deeper native soils within the site, future settlement of these materials is expected to be negligible. Footing settlement due to fill self weight building loads should be calculated when fill material engineering properties, building loads magnitude and footing dimensions are known.

7.7.5 Lateral Resistance

Lateral resistance of footings is derived from footing basal friction against the underlying soils as well as passive resistance of adjacent soils. Lateral resistance information should be provided by the project geotechnical engineer once the soils strength parameters as well as the footing dimensions are available for each individual building pad.

7.7.6 Footing Observations

All footing trenches should be observed by a representative of the project geotechnical consultant to verify that they have been excavated into competent bearing soils or bedrock prior to the placement of forms, reinforcement or concrete. The excavations should be trimmed neat, level and square. All loose, sloughed or moisture-softened soils and/or any construction debris should be removed prior to placing of concrete. Excavated soils derived from footing and/or utility trenches should not be placed in building slab-on-grade areas or exterior concrete flatwork areas unless the soils are compacted to at least 90 percent of maximum dry density.

7.8 Preliminary Conventional Footing and Floor Slab Recommendations

On-site soil and bedrock materials are expected to exhibit expansion potentials that range from low to high as classified in accordance with UBC Table No. 18-I-B. However, based on the distribution of the various geologic units within the site and the anticipated grading, it is expected that upon the completion of rough grading that the majority of the lots will exhibit a low expansion potential. In addition, it was recommended within the "Earthwork Recommendations" section of this report that remedial grading be performed such that the upper 3 feet of each building pad be underlain by materials having a medium expansion potential or less. Therefore, the following preliminary recommendations are provided for foundations underlain by earth materials with expansion potentials ranging from very low to medium. During grading of the site, the weighted expansion potentials of the foundation soils existing below the building sites should be determined on a lot by lot basis. Depending on the results of these tests, the following footing and floor slab recommendations should be followed.

7.8.1 Lots with Very Low Expansion Potentials (Expansion Index between 0 and 20)

For future lots that have a very low expansion potential (expansion index between 0 and 20), the design of slab-on-ground foundations will be exempt from the procedures outlined in UBC Section 1815. However, based on the existing soil and bedrock conditions within the site, it is recommended that footings and floor slabs constructed on soils with a very low expansion potential be constructed and reinforced in accordance with the following minimum criteria. Furthermore, additional slab thickness, footing sizes and/or reinforcement should be provided as required by the project architect or structural engineer.

- Footings

Exterior continuous footings may be founded at the minimum depths indicated in UBC Table 18-I-D (i.e., 12-inch minimum depth for one-story construction, and 18-inch minimum depth for two-story construction). Interior continuous footings for both one-story and two-story construction may be founded at a minimum depth of 12 inches below the lowest adjacent final grade. All continuous footings should have a minimum width of 12 and 15 inches, for one-story and two-story construction, respectively, and should be reinforced with a minimum of two No. 4 bars, one top and one bottom.

Interior isolated pad footings should be a minimum of 24 inches square and founded at a minimum depth of 12 inches below the bottoms of the adjacent floor slabs. The pad footings should be reinforced with No. 4 bars spaced a maximum of 18 inches on centers, both ways, near the bottoms of the footings.

Exterior isolated pad footings intended for support of roof overhangs such as second-story decks, patio covers and similar construction should be a minimum of 24 inches square, and founded at a minimum depth of 18 inches below the lowest adjacent final grade. The pad footings should be reinforced with No. 4 bars spaced a maximum of 18 inches on centers, both ways, near the bottoms of the footings.

- Building Floor Slabs

Living area concrete floor slabs should be 4 inches thick, and reinforced with either 6-inch by 6-inch, No. 10 by No. 10 welded wire mesh; or with No. 3 bars spaced a maximum of 24 inches on centers, both ways. All slab reinforcement should be supported on concrete chairs or brick to ensure the desired placement near mid depth.

Living area concrete floors should be underlain with a moisture vapor retarder consisting of a polyvinyl chloride membrane such as 6-mil Visqueen, or equivalent. All laps within the

membrane should be sealed, and at least 2 inches of clean sand should be placed over the membrane to promote uniform curing of the concrete. To reduce the potential for punctures, the membrane should be placed on a pad surface that has been graded smooth without any sharp protrusions. If a smooth surface cannot be achieved by grading, consideration should be given to placing a 1-inch-thick leveling coarse of sand across the pad surface prior to the placement of the membrane.

Garage floor slabs should be 4 inches thick and reinforced in a similar manner as living area floor slabs. Garage floor slabs should also be poured separately from adjacent wall footings with a positive separation maintained with 3/8-inch-minimum felt expansion joint materials, and quartered with weakened plane joints. A 12-inch-wide grade beam founded at the same depth as adjacent footings should be provided across garage entrances. The grade beam should be reinforced with two No. 4 bars, one top and one bottom.

Presaturation of the subgrade soils below floor slabs will not be required; however, prior to placing concrete, the subgrade soils should be prewatered to promote uniform curing of the concrete and minimize the development of shrinkage cracks.

7.8.2 Lots with Low Expansion Potentials (Expansion Index Between 21 and 50)

For future lots that have a low expansion potential (expansion index between 21 and 50), the 1997 UBC specifies that slab-on-ground foundations require special design considerations in accordance with Section 1815. The design procedures outlined in Section 1815 are based on the weighted plasticity index of the different soil and/or bedrock layers existing within the upper 15 feet of each building site. Based on our preliminary laboratory testing, a worst case weighted plasticity index of 15 can be assumed for lots underlain by materials with a low expansion potential.

Section 1815.4.2 states that the weighted plasticity index of each building site should be modified (multiplied) by correction factors that compensate for the effects of sloping ground and the unconfined compressive strength of the soil and bedrock materials. Based on the buildings being constructed on level building pads and based on the estimated unconfined compressive strength of the on-site soil and bedrock materials, it is recommended that the weighted plasticity index (15) be multiplied by a factor of 1.2 in order to determine the value of the effective plasticity index (per Figure 18-III-2 of the 1997 UBC). In summary, a preliminary effective plasticity index of 18 should be used for building sites underlain by soils with a low expansion potential. However, the expansion potentials and plasticity indices of each building site should be determined on a lot by lot basis. Depending on the results of these tests, revised plasticity indices may be used for individual lots.

The design and construction recommendations that follow are based on the above soil conditions and may be considered for minimizing the effects of slightly expansive soils. These recommendations have been developed on the basis of previous experience of this firm on projects with similar soil conditions. Although construction performed in accordance with these recommendations has been found to minimize post-construction movement and/or cracking, they generally do not positively mitigate all potential effects of expansive soils and future settlement. The effective plasticity index provided above (18) should be utilized by the project structural engineer to design slab-on-ground foundations with an interior grade beam grid system in accordance with Section 1815. Based on this design, thicker floor slabs, larger footing sizes and/or additional reinforcement may be required and should govern the design if more restrictive than the minimum recommendations provided below.

- Footings

Exterior continuous footings may be founded at the minimum depths indicated in UBC Table 18-I-D (i.e., 12-inch minimum depth for one-story construction, and 18-inch minimum depth for two-story construction). Interior continuous footings for both one-story and two-story construction may be founded at a minimum depth of 12 inches below the lowest adjacent final grade. All continuous footings should have a minimum width of 12 and 15 inches, for one-story and two-story buildings, respectively, and should be reinforced with a minimum of two No. 4 bars, one top and one bottom.

Interior isolated pad footings should be a minimum of 24 inches square and founded at a minimum depth of 12 inches below the bottoms of the adjacent floor slabs. The pad footings should be reinforced with No. 4 bars spaced a maximum of 18 inches on centers, both ways, near the bottoms of the footings.

Exterior isolated pad footings intended for support of roof overhangs such as second-story decks, patio covers and similar construction should be a minimum of 24 inches square, and founded at a minimum depth of 18 inches below the lowest adjacent final grade. The pad footings should be reinforced with No. 4 bars spaced a maximum of 18 inches on centers, both ways, near the bottoms of the footings.

The spacing and layout of the interior grade beam grid system for each building should be determined by the project architect or structural engineer in accordance with UBC Section 1815.5, and the beams designed in accordance with UBC Section 1815.6.

- Building Floor Slabs

The project architect or structural engineer should evaluate minimum floor slab thickness and reinforcement in accordance with UBC Section 1815 based on the effective plasticity

index provided previously (18). Unless a more stringent design is recommended by the architect or structural engineer, we recommend a minimum slab thickness of 4 inches for both living area and garage floor slabs, and reinforcement consisting of either 6-inch by 6-inch, No. 6 by No. 6 welded wire mesh; or with No. 3 bars spaced a maximum of 18 inches on centers, both ways. All slab reinforcement should be supported on concrete chairs or brick to ensure the desired placement near mid depth.

Living area concrete floor slabs should be underlain with a moisture vapor retarder consisting of a polyvinyl chloride membrane such as 6-mil Visqueen, or equivalent. All laps within the membrane should be sealed, and at least 2 inches of clean sand should be placed over the membrane to promote uniform curing of the concrete. To reduce the potential for punctures, the membrane should be placed on a pad surface that has been graded smooth without any sharp protrusions. If a smooth surface cannot be achieved by grading, consideration should be given to placing a 1-inch-thick leveling coarse of sand across the pad surface prior to the placement of the membrane.

Garage floor slabs should be poured separately from adjacent wall footings with a positive separation maintained with 3/8-inch-minimum felt expansion joint materials, and quartered with weakened plane joints. An 18-inch-wide grade beam founded at the same depth as adjacent footings should also be provided across garage entrances. The grade beam should be reinforced with two No. 4 bars, one top and one bottom.

Prior to placing concrete, the subgrade soils below living area and garage floor slabs should be prewatered to achieve a moisture content that is at least 1.2 times the optimum moisture content. This moisture should penetrate to a depth of approximately 12 inches into the subgrade.

7.8.3 Lots with Medium Expansion Potentials (Expansion Index between 51 and 90)

For future lots that have a medium expansion potential (expansion index between 51 and 90), the 1997 UBC specifies that slab-on-ground foundations require special design considerations in accordance with Section 1815. The design procedures outlined in Section 1815 are based on the weighted plasticity index of the different soil and/or bedrock layers existing within the upper 15 feet of each building site. Based on our preliminary laboratory testing, a worst case weighted plasticity index of 30 can be assumed for lots underlain by materials with a medium expansion potential.

Section 1815.4.2 states that the weighted plasticity index of each building site should be modified (multiplied) by correction factors that compensate for the effects of sloping ground and the unconfined compressive strength of the soil and bedrock materials. Based on the buildings

being constructed on level building pads and based on the estimated unconfined compressive strength of the on-site soil and bedrock materials, it is recommended that the weighted plasticity index (30) be multiplied by a factor of 1.2 in order to determine the value of the effective plasticity index (per Figure 18-III-2 of the 1997 UBC). In summary, a preliminary effective plasticity index of 36 should be used for building sites underlain by soils with a medium expansion potential. However, the expansion potentials and plasticity indices of each building site should be determined on a lot by lot basis. Depending on the results of these tests, revised plasticity indices may be used for individual lots.

The design and construction recommendations that follow are based on the above soil conditions and may be considered for minimizing the effects of moderately expansive soils. These recommendations have been developed on the basis of previous experience of this firm on projects with similar soil conditions. Although construction performed in accordance with these recommendations has been found to minimize post-construction movement and/or cracking, they generally do not positively mitigate all potential effects of expansive soils and future settlement. The effective plasticity index provided above (36) should be utilized by the project structural engineer to design slab-on-ground foundations with an interior grade beam grid system in accordance with Section 1815. Based on this design, thicker floor slabs, larger footing sizes and/or additional reinforcement may be required and should govern the design if more restrictive than the minimum recommendations provided below.

- Footings

Exterior continuous footings should be founded at a minimum depth of 18 inches below the lowest adjacent final grade. Interior continuous footings may be founded at a minimum depth of 12 inches below the lowest adjacent final grade. All continuous footings should have a minimum width of 12 and 15 inches, for one-story and two-story buildings, respectively, and should be reinforced with a minimum of four No. 4 bars, two top and two bottoms.

Interior isolated pad footings should be a minimum of 24 inches square and founded at a minimum depth of 12 inches below the bottoms of the adjacent floor slabs. The pad footings should be reinforced with No. 4 bars spaced a maximum of 18 inches on centers, both ways, near the bottoms of the footings.

Exterior isolated pad footings intended for support of roof overhangs such as second-story decks, patio covers and similar construction should be a minimum of 24 inches square, and founded at a minimum depth of 18 inches below the lowest adjacent final grade. The pad footings should be reinforced with No. 4 bars spaced a maximum of 18 inches on centers, both ways, near the bottoms of the footings.

The spacing and layout of the interior grade beam grid system for each building should be determined by the project architect or structural engineer in accordance with UBC Section 1815.5, and the beams designed in accordance with UBC Section 1815.6.

- **Building Floor Slabs**

The project architect or structural engineer should evaluate minimum floor slab thickness and reinforcement in accordance with UBC Section 1815 based on the effective plasticity index provided previously (36). Unless a more stringent design is recommended by the architect or structural engineer, we recommend a minimum slab thickness of 4 inches for both living area and garage floor slabs, and reinforcement consisting of No. 3 bars spaced a maximum of 18 inches on centers, both ways. All slab reinforcement should be supported on concrete chairs or brick to ensure the desired placement near mid depth.

Living area concrete floor slabs should be underlain with a moisture vapor retarder consisting of a polyvinyl chloride membrane such as 6-mil Visqueen, or equivalent. All laps within the membrane should be sealed, and at least 2 inches of clean sand should be placed over the membrane to promote uniform curing of the concrete. To reduce the potential for punctures, the membrane should be placed on a pad surface that has been graded smooth without any sharp protrusions. If a smooth surface cannot be achieved by grading, consideration should be given to placing a 1-inch-thick leveling course of sand across the pad surface prior to the placement of the membrane.

Garage floor slabs should be poured separately from adjacent wall footings with a positive separation maintained with 3/8-inch-minimum felt expansion joint materials, and quartered with weakened plane joints. An 18-inch-wide grade beam founded at the same depth as adjacent footings should also be provided across garage entrances. The grade beam should be reinforced with four No. 4 bars, two top and two bottom.

Prior to placing concrete, the subgrade soils below living area and garage floor slabs should be prewatered to achieve a moisture content that is at least 1.3 times the optimum moisture content. This moisture should penetrate to a depth of approximately 18 inches into the subgrade.

7.9 Preliminary Post-Tensioned Foundation Recommendations

In lieu of conventional foundations, post-tensioned foundation systems may be utilized for the proposed buildings. Therefore, we have evaluated the engineering characteristics of the on-site soils and the anticipated soil conditions that will exist at the completion of grading for construction of post-tensioned foundation systems within the subject site. As described

previously, it is expected that upon the completion of rough grading that the majority of the lots will exhibit **low** expansion potentials. In addition, it was recommended within the Earthwork Recommendations section of this report that remedial grading be performed such that the upper 3 feet of each building pad be underlain by materials having a medium expansion potential or less. The following design and construction recommendations are based on a **medium** expansion potential for the onsite soils. The structural engineer should use the following geotechnical recommendations in order to design the proposed foundations in accordance with both Sections 1816 and 1819 of the 1997 Uniform Building Code (UBC), which are based on the design specifications of the Post-Tensioning Institute. The more restrictive recommendations should govern the final design.

7.9.1 Design Recommendations for Moderately Expansive Soils (Section 1816)

Variations in subsurface moisture play a significant role in soils volume change which directly influences slab-on-ground performance. As stated in the 1997 UBC, the Post-Tensioning Institute procedure is applicable *...only in those cases where site conditions have been corrected so that soil moisture conditions are controlled by the climate alone.* In the general region where the site is located, it is a common practice to use a Thornthwaite Moisture Index of -20, as provided by the 1997 UBC. Subsurface moisture conditions within the vicinity of the structure, however, may also be controlled by irrigation water and/or any other post-construction activities that alter surface conditions. The adverse effects of irrigation water and/or any other post-construction activities are expected to be more pronounced for the edge lift condition. Therefore, the design Thornthwaite Moisture Index for the edge lift condition has been appropriately increased, as reported herein, to partially account for these adverse effects.

Summary of Assumed Conditions

Liquid Limit (LL)	45
Plastic Limit (PL)	15
Plastic Index (PI)	30
Percent Fine Clay	50
Clay Type	Montmorillonite
Expansion Index	70 (Medium)

Summary of Design Parameters Based on Assumptions Above

Approximate Depth of Constant Suction:	
Center Lift	7 feet
Edge Lift	7 feet
Approximate Soil Suction, pF:	3.6

Approximate Moisture Velocity:	0.7 inches/month
Thornthwaite Index:	
Center Lift	-20
Edge Lift	0
Average Edge Moisture Variation Distance, e_m :	
Center Lift	5.3 feet
Edge Lift	3.7 feet
Anticipated Swell, y_m :	
Center Lift	3.2 inches
Edge Lift	1.1 inches

7.9.2 Design Recommendations for Compressible Soil Conditions (Section 1819)

Compressible soils may be encountered within the site, albeit to a limited extent. The following recommendations are provided as a guideline for conditions where a maximum differential settlement of an inch over a span of 40 feet to occur. The final design recommendations should be developed by the project geotechnical engineer when site specific information for each building pad is known.

7.9.3 Minimum Design Recommendations

The design and construction recommendations that follow are based on the above described anticipated soil conditions and may be considered for minimizing the effects of moderately expansive soils and anticipated total and differential settlements. These recommendations have been developed on the basis of previous experience of this firm on projects with similar soil conditions. Although construction performed in accordance with these recommendations has been found to minimize post-construction movement and/or cracking, they generally do not positively mitigate all potential effects of expansive soils and future settlement. The soil parameters provided previously should be utilized by the project structural engineer to design post-tensioned foundations in accordance with Sections 1816 and 1819 of the UBC. Based on this design, thicker floor slabs, larger footing sizes and/or additional reinforcement and additional grade beams may be required and should govern the design if more restrictive than the minimum recommendations provided below:

- Perimeter footings for both one-story and two-story structures should be founded at a minimum depth of 15 inches below the lowest adjacent final ground surface. Interior footings may be founded at a minimum depth of 12 inches below the tops of the finish floor slabs.

- The thickness of the floor slabs should be determined by the project structural engineer with consideration to the worst case MEDIUM expansion potential of the on-site soils; however, we recommend a minimum slab thickness of 4 inches.
- For conditions where the perimeter footings are eliminated in favor of a thicker (mat) slab, we recommend a minimum slab thickness of 10 inches.
- All dwelling area floor slabs constructed on-grade should be underlain with a moisture vapor retarder consisting of a polyvinyl chloride membrane such as 6-mil Visqueen or equivalent. A minimum of two inches of clean sand should be placed over the membrane to promote uniform curing of the concrete.
- Presaturation of the subgrade below floor slabs will not be required; however, prior to placing concrete, the subgrade below all dwelling and garage floor slab areas should be thoroughly moistened to achieve a moisture content that is at least equal to or slightly greater than optimum moisture content. This moisture content should penetrate to a minimum depth of 12 inches below the bottoms of the slabs.
- A 12-inch-wide grade beam founded at the same depth as adjacent footings should be provided across the garage entrance.

7.9.4 Precise Grading and Drainage Recommendations

To reduce the potential adverse impact of post-construction activities on slab performance, it is recommended all applicable construction and/or grading procedures provided below be followed. These construction and/or grading procedures include construction of sloped ground away from the buildings, compacted and smooth surfaces to avoid saturation and ponding, properly graded and maintained swales, paved surfaces, impervious backfill at the sewer and water trench entrances to the structure, drip irrigation or professionally installed sprinkler systems with controlled timing, area drains and pipes, raised planters with sealed bottoms, roof gutters, downspouts, etc. These devices and/or grading practices should be installed and/or implemented so that the area within at least 5 feet of the building structure has proper and adequate drainage. Precipitation or irrigation water should be collected and diverted away from the structure to an appropriate drainage outlet to inhibit water from ponding or migrating below the slab.

Future changes to site improvements, or planting and watering practices, should not be allowed to cause cycles of drying and oversaturation of site soils adjacent to the structures. Furthermore, future homeowners should be notified that our recommendations for collection and diversion of excess irrigation water should be followed.

7.10 Soluble Sulfates and Soil Corrosivity

The results of our preliminary laboratory test performed in accordance with California Test Method No. 417 indicate on-site soils contain water soluble sulfate contents of less than 0.10 percent. Therefore, according to UBC Table 19-A-4, a negligible exposure to sulfate can be expected for concrete placed in contact with the on-site soils. Type II cement or equivalent can therefore be used for concrete to be placed in contact with on-site soils.

The results of limited in-house testing of soil pH and resistivity indicate that onsite soils and bedrock are generally neutral with respect to pH (pH= 6.6 to 6.94); soil and bedrock resistivity was found to be low (1,000 to 1,500 ohm-cm) and chloride contents were found to range from 168 to 290 parts per million. The preliminary chemical test results are included in Appendix B. The results of these tests indicate that onsite soils may be moderately corrosive to ferrous metals and copper. As such, it is recommended that additional sampling and analysis be conducted during the final stages of site grading to provide a complete assessment of soil corrosivity. Our firm does not practice corrosion engineering; therefore, we recommend that onsite soils be tested and analyzed near or at the completion of precise grading by a qualified corrosion engineer to evaluate the general corrosion potential of the onsite soils and any impact on the proposed construction.

7.11 Preliminary Retaining Wall Design Recommendations

7.11.1 Allowable Bearing Values and Lateral Resistance

Retaining wall footings may be designed using the allowable soil bearing and lateral resistance values once site specific information is available. In general, when calculating passive resistance, the upper 6 inches of the footings should be ignored in areas where the footings will not be covered with concrete flatwork, or where the thickness of soil cover over the top of the footing is less than 12 inches.

7.11.2 Active and At-Rest Earth Pressures

As of the date of this report, it is uncertain whether the proposed retaining walls on-site will be backfilled with on-site soils or imported granular materials. The active and at-rest earth pressures that are provided below for each condition should be viewed as a guideline only and be verified once site specific information is available.

- On-Site Soils Used for Backfill

If the onsite soils are used as the wall backfill, active earth pressures equivalent to fluids having densities of 45 and 75 pounds per cubic foot may be considered for design of cantilevered walls retaining a level backfill and ascending 2:1 backfill, respectively. For walls that are restrained at the top, at-rest earth pressures of 68 and 110 pounds per cubic foot (equivalent fluid pressures) should be used. The above values are for retaining walls that have been supplied with a proper subdrain system. All walls should be designed to support any adjacent structural surcharge loads imposed by other nearby walls or footings in addition to the above recommended active and at-rest earth pressures.

- Imported Sand, Pea Gravel or Rock Used for Wall Backfill

Imported clean sand exhibiting a sand equivalent value (SE) of 30 or greater, or pea gravel or crushed rock may be used for wall backfill to reduce the lateral earth pressures provided these granular backfill materials extend behind the walls to a minimum horizontal distance equal to one-half the wall height. In addition, the sand, pea gravel or rock backfill materials should extend behind the walls to a minimum horizontal distance of 2 feet at the base of the wall or to a horizontal distance equal to the heel width of the footing, whichever is greater. For the above conditions, cantilevered walls retaining a level backfill and ascending 2:1 backfill may be designed to resist active earth pressures equivalent to fluids having densities of 30 and 41 pounds per cubic foot, respectively. For walls that are restrained at the top, at-rest earth pressures equivalent to fluids having densities of 45 and 62 pounds per cubic foot are recommended for design of restrained walls supporting a level backfill and ascending 2:1 backfill, respectively. These values are also for retaining walls supplied with a proper subdrain system. Furthermore, as with native soil backfill, the walls should be designed to support any adjacent structural surcharge loads imposed by other nearby walls or footings in addition to the recommended active and at-rest earth pressures.

All structural calculations and details for the proposed retaining walls should be provided to this firm for verification purposes prior to grading and construction phases.

7.11.3 Drainage and Waterproofing

Perforated pipe and gravel subdrains should be installed behind all retaining walls to prevent entrapment of water in the backfill. Perforated pipe should consist of 4-inch-minimum diameter PVC Schedule 40, or ABS SDR-35, with the perforations laid down. The pipe should be encased in a 1-foot-wide column of 3/4-inch to 1 1/2-inch open-graded gravel. If on-site soils are used as backfill, the open-graded gravel should extend above the wall footings to a minimum

height equal to one-half the wall height, or to a minimum height of 1.5 feet above the footing, whichever is greater. If imported sand, pea gravel, or crushed rock is used as backfill, the open-graded gravel should extend above the wall footing to a minimum height of 1 foot above the footing. The open-graded gravel should be completely wrapped in filter fabric consisting of Mirafi 140N, or equivalent. Solid outlet pipes should be connected to the subdrains and then routed to a suitable area for discharge of accumulated water.

The portions of retaining walls supporting backfill should be coated with an approved waterproofing compound or covered with a similar material to inhibit infiltration of moisture through the walls.

7.11.4 Wall Backfill

Recommended active and at-rest earth pressures for design of retaining walls are based on the physical and mechanical properties of the on-site soil materials. However, since the on-site soil materials are expected to be moderately expansive, they may be difficult to compact when placed in the relatively confined areas located between the walls and temporary backcut slopes. Therefore, to facilitate compaction of the backfill, consideration should be given to using sand, pea gravel, crushed rock, or imported granular soils for backfill that exhibit a very low expansion potential (Expansion Index of less than 20).

Where on-site soils or imported sand are used for backfill, they should be placed in approximately 6- to 8-inch-thick maximum lifts, watered as necessary to achieve near optimum moisture conditions, and then mechanically compacted in place to a minimum relative compaction of 90 percent. Flooding or jetting of the backfill materials should be avoided. A representative of the project geotechnical consultant should observe the backfill procedures and test the wall backfill to verify adequate compaction.

If imported pea gravel or rock is used for backfill, the gravel should be placed in approximately 2- to 3-foot-thick lifts, thoroughly wetted but not flooded, and then mechanically tamped or vibrated into place. A representative of the project geotechnical consultant should observe the backfill procedures and probe the backfill to determine that an adequate degree of compaction is achieved.

To mitigate the potential for the direct infiltration of surface water into the backfill, imported sand, gravel or rock backfill should be capped with at least 12 inches of onsite soil. Filter fabric such as Mirafi 140N, or equivalent, should be placed between the soil and the imported gravel or rock to prevent fines from penetrating into the backfill.

7.12 Masonry Block Walls

7.12.1 Construction on Level Ground

Footings for masonry block walls proposed on level ground and at least 10 feet from the tops of any adjacent descending slopes (see section below) should be embedded at a minimum depth of 18 inches below the lowest adjacent final grade. The footings should also be reinforced with a minimum of two No. 4 bars, one top and one bottom. In order to minimize the potential for unsightly cracking related to the possible effects of differential settlement and/or expansion, positive separations (construction joints) should also be provided in the garden walls at each corner and at horizontal intervals of approximately 20 to 25 feet. The separations should be provided in the blocks and not extend through the footings. The footings should be poured monolithically with continuous rebars to serve as effective "grade beams" below the walls.

7.12.2 Construction Near the Tops of Descending Slopes

Due to the large number of steep slopes within some parts of the subject site, there may be some potential for development of a soil creep on the slopes with the passage of time. Creep is an imperceptibly slow, nearly continuous downward and outward movement of slope soils. The movement is essentially viscous under small shear stresses sufficient to produce permanent deformation but not large enough to produce a shear failure, as occurs in a landslide. For any slope gradient, the principal cause for development of a creep condition is a result of repeated cycles of swelling and contraction of slightly expansive soils over a period of time due to seasonal variations in the moisture content and is an irreversible process resulting in a loss of soil density and shear strength and subsequent buildup of small shear stresses. Experience has shown that creep can affect compacted fill slopes to vertical depths of several feet depending on the expansiveness of the soils and slope height, as well as a number of other factors. Other factors which can contribute to development of a slope creep condition include overwatering and subsequent saturation of the slope soils, rainfall intensity and duration, prolonged periods of drought, rodent activity, inadequate plant materials used for slope protection, inadequate drainage facilities, and/or lack of a proper slope maintenance program. Creep cannot be stopped or eliminated; however, a proper footing design can be provided such that the magnitude, depth and rate of creep movement will not significantly affect any retaining walls proposed along the tops or in close proximity to the descending slopes.

On slopes where creep might occur, it can be assumed to develop to a depth of approximately 2 to 3 feet on the descending slopes. This creep zone may extend laterally back into the adjacent lots approximately 6 to 10 feet. On the basis of this assumed condition, any masonry block walls proposed within approximately 10 feet of the descending slopes should be supported on either

deepened footings or caissons to mitigate the potential adverse effects of slope creep. Recommendations for deepened footings and caissons are provided below.

Deepened Footings

Footings for walls proposed on or near the top of the descending slopes should be deepened such that a minimum horizontal clearance of 10 feet is maintained between the outside bottom edges of the footings and the face of the slopes to mitigate the potential adverse effects of slope creep. It should be noted that additional footing depths may be required to resist the potential creep forces and to achieve the necessary passive resistance against lateral movement as determined by the project structural engineer based on the soil parameters provided below.

Footings for retaining walls at the above recommended minimum setbacks may be designed using the allowable bearing and lateral resistance values as determined for the site specific condition. It should be noted that the lateral resistance should be ignored for the upper portions of the wall footings located within the creep zone.

Cast-In-Place Caissons

In lieu of deepened conventional footings, cast-in-place concrete caissons and grade beams may be used to support any walls proposed along the top of the descending slopes. A general guideline for design of caissons and grade beams are provided below.

- a. Grade Beam Embedment: The tops of the grade beams may be located near the ground surface. No specific setback will be required between the outside bottom edges of the grade beams and slope face.
- b. Lateral Resistance for Grade Beams: Due to the downward and outward movement of the soils within the creep zone, lateral resistance and bearing capacity should be ignored in design of grade beams.
- c. Caisson Capacity: End bearing capacity and skin friction may be combined to determine allowable caisson capacities provided the minimum caisson diameter is 18 inches. However, when calculating skin friction, the upper portions of caissons located within the potential creep zone should be ignored.
- d. Passive Resistance for Caissons: Passive earth pressure should be determined for site specific conditions. It should be noted that lateral resistance should be ignored for the upper portions of the caissons located within the anticipated 3-foot-deep creep zone.

- e. Lateral Loading: To compensate for potential creep forces, the caissons should be designed to resist a lateral load imposed by creep affected slope materials. This lateral load should be assumed to be equal to 500 pounds per foot of embedment in the creep zone.
- f. Point of Fixity: The point of fixity for the caissons may be taken at a depth equal to 5 feet plus two times the caisson diameter (e.g., at a depth of 8 feet below grade for an 18-inch diameter caisson).
- g. Uplift: Caissons may be considered to resist uplift forces equal to the skin friction between the concrete caisson and the surrounding soils as described above.
- h. Caisson Depth and Spacing: Caisson depth and spacing should be determined by the project structural engineer based on total wall loads and lateral loading. However, minimum clear spacing between caissons should be two caisson diameters, sidewall to sidewall. In addition, maximum spacing between caissons should not exceed six caisson diameters, center to center. Further, the caissons should have a minimum depth of at least three caisson diameters below the creep zone.
- i. Caisson Locations Relative to Wall: To prevent eccentric loading, the centerlines of the caissons should correspond to the centerline of the wall.
- j. Reinforcement: Reinforcement for caissons should be determined by the project structural engineer with regard to strengthening the concrete to resist lateral forces.
- k. Geotechnical Observations: All caisson excavations should be observed by a representative of this firm to verify minimum embedments determined by the project structural engineer. The drilled holes should also be cleared of loose materials and any construction debris prior to pouring concrete.

Concrete Placement

Concrete should be placed by the tremie method and not allowed to free fall to prevent segregation of the concrete, as well as scouring or erosion of the sidewalls of drilled holes. The lower end of the tremie pipe should be continually immersed in fresh concrete and slowly withdrawn as the concrete is deposited.

7.13 Exterior Concrete Flatwork

7.13.1 Thickness and Joint Spacing

To reduce the potential of unsightly cracking related to the effects of expansive soils, concrete sidewalks, patio-type slabs and concrete subslabs to be covered with decorative pavers should be at least 4 inches thick and provided with construction joints or expansion joints every 6 feet or less. Concrete driveway slabs should be at least 5 inches thick and provided with construction joints or expansion joints every 10 feet or less.

7.13.2 Reinforcement

Consideration should be given to reinforcing all concrete patio-type slabs, subslabs, driveways and sidewalks greater than 5 feet in width with 6-inch by 6-inch, No. 6 by No. 6 welded wire fabric, or with No. 3 bars spaced 24 inches on centers, both ways. The reinforcement should be positioned near the middle of the slabs by means of concrete chairs or brick. Reinforcing dowels should also be considered across all cold joints to mitigate the potential for differential movement at the joints.

7.13.3 Subgrade Preparation

As a further measure to minimize cracking of concrete flatwork, the subgrade soils below concrete flatwork areas should first be compacted to a minimum relative compaction of 90 percent and then thoroughly wetted to achieve a moisture content that is at least equal to or slightly greater than optimum moisture content. This moisture should penetrate to a depth of 12 inches into the subgrade and maintained in the soils during placement of concrete. Pre-watering of the soils will promote uniform curing of the concrete and minimize the development of shrinkage cracks. A representative of the project geotechnical consultant should observe and verify the density and moisture content of the soils, and the depth of moisture penetration prior to pouring concrete.

7.14 Preliminary Structural Pavement Sections

In order to provide a preliminary estimate of the pavement sections for the site, a conservative R-value of 15 was assumed for the onsite soils. Based on this anticipated R-value and an anticipated Traffic Index value of 5.0 for the streets within the site, a tentative structural pavement section consisting of 3 inches of asphaltic concrete over 8 inches of aggregate base may be assumed for the streets within the site.

However, due to the remedial grading that will be performed within the site, the existing soils within the site will be reprocessed, mixed and replaced resulting in compacted fill materials that will have different subgrade strength characteristics than those that presently exist within the site. Therefore, representative samples of the subgrade soils within street areas should be obtained for R-value testing at the completion of grading. In addition, the actual traffic indices for the streets with the site should be obtained from the jurisdictional Public Works Department when they become available. A separate letter providing specific recommendations for structural pavement sections within the site will then be submitted by this firm based on the results of these tests and on the additional information.

Where aggregate base is used, the subgrade soils immediately below the aggregate base should be compacted to at least 90 percent relative compaction. If the asphaltic concrete is placed directly on the native soils, the subgrade should be compacted to a minimum relative compaction of 95 percent to a depth of 6 inches. Final subgrade compaction should be performed just prior to placing aggregate base, and after all utility trench backfills are compacted and tested.

Aggregate base materials should be Crushed Aggregate Base, Crushed Miscellaneous Base, or Processed Miscellaneous Base conforming to Section 200-2 of the Standard Specifications for Public Works Construction (Greenbook). The materials should be brought to a uniform moisture near optimum moisture, and then compacted to at least 95 percent of the maximum density determined in accordance with California Test Method No. 216. Asphaltic concrete materials and construction should conform to Section 203 of the Greenbook.

8.0 CONCLUSIONS

Based on our subsurface investigation and analysis, development of the site is considered feasible from a geotechnical point of view. It is our opinion that most building sites will be free of hazard from landslide, settlement, and slippage provided that the general recommendations are incorporated into the design criteria and project specifications and that good engineering practices are followed. However, the project is still in the conceptual stage and specific grading and development plans have not been developed. More-specific recommendations will require additional more-extensive geotechnical investigations.

Preliminary conclusions suggest that the faulting issues have not been resolved, especially with respect to school sites which are held to higher, more-rigid standards than many residential facilities. It is still very possible that the fault can be shown to be inactive if trenching is conducted in the Qrp unit (school sites) just east of Covington Canyon. Even if future fault studies are inconclusive, the loss of building sites due to the fault zone might be offset by building in some of the smaller canyons and valleys which were assumed to be liquefaction

zones but which were found to have only very thin alluvium and a shallow depth to firm bedrock (QTs).

9.0 INVESTIGATION LIMITATIONS

This report is based on the proposed project and geotechnical data as described herein. The materials encountered on the project site, described in other literature, and utilized in our laboratory investigation are believed representative of the total project area, and the conclusions and recommendations contained in this report are presented on that basis. However, soils can vary in characteristics between points of exploration, both laterally and vertically, and those variations could affect the conclusions and recommendations contained herein. As such, observation and testing by a geotechnical consultant during the construction phase of the project are essential to confirming the basis of this report. To provide the greatest degree of continuity between the design and construction phases, consideration should be given to retaining Petra Geotechnical, Inc. for construction services.

This report has been prepared consistent with the level of care being provided by other professionals providing similar services at the same locale and in the same time period. This report provides our professional opinions and as such, they are not to be considered a guaranty or warranty. This report should be reviewed and updated after a period of one year or if the site conditions, ownership or project concept changes from that described herein. This report has not been prepared for use by parties or projects other than those named or described herein and may not contain sufficient information for other parties or other purposes.

10.0 REFERENCES CITED

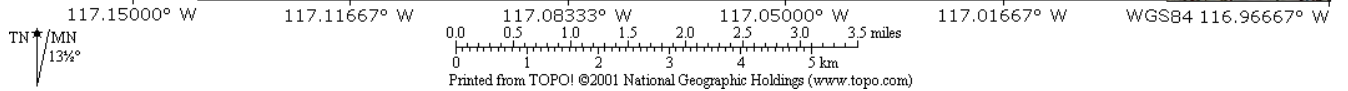
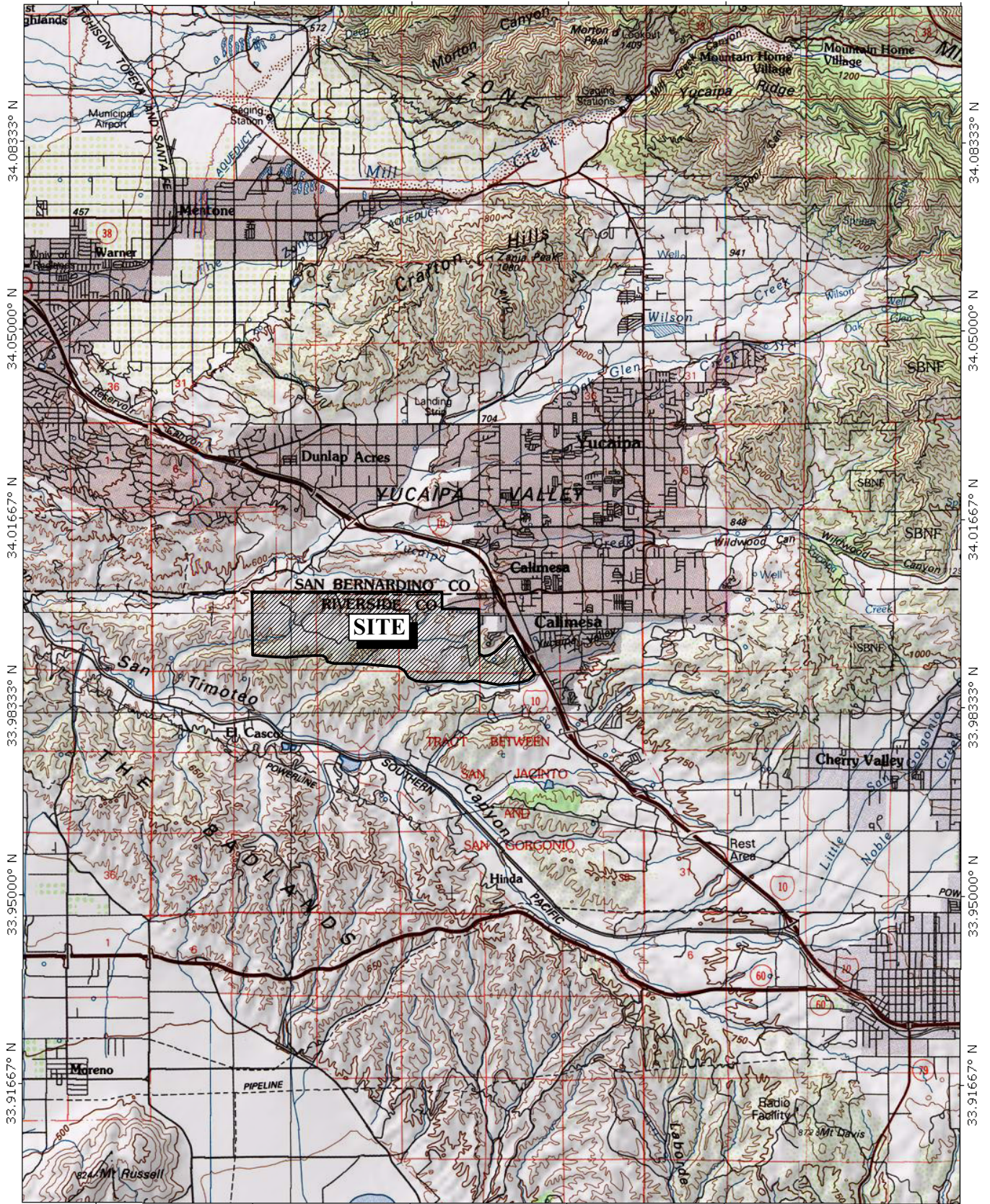
- Albright III, L.B., 1999, Magneostratigraphy and biochronology of the San Timoteo Badlands southern California, with implications for local Pliocene-Pleistocene tectonic and depositional patterns: Geological Society of America Bulletin, v. 111, p. 1265-1293
- Bloyd, R.M., Jr., 1971, Underground storage of imported water in the San Gorgonio Pass area, southern California: U.S. Geological Survey Water Supply Paper 1999-D.
- Dames & Moore, 1987, Geology and seismicity, Oak Valley, Riverside County, California: Draft Environmental Impact Report, Part V, *in* Michael Brandman Associates, 1988, Final environmental impact report no. 229 for Oak Valley specific plan no. 216 (state clearinghouse # 87033011): Prepared for County of Riverside, Riverside CA, dated October 7, 1988, Technical Appendix A: Prepared for Landmark Land Company of California, Inc, Moreno Valley, CA, dated November 13, 1987.

- Dibblee, T.W., Jr., 1981, Geology of the San Jacinto Mountains and adjacent areas: *in* South Coast Geological Society Annual Field Trip Guidebook No. 9.
- Frick, C., 1921, Extinct vertebrate faunas of the badlands of Bautista Creek and San Timoteo Canon, southern California: University of California Publications, Bulletin of the Department of Geology, v. 12, p. 277-424.
- Kendrick, K.J., Morton, D.M., Wells, S.G., and Simpson, R.W., 2003, Spatial and temporal deformation along the northern San Jacinto Fault, southern California: implications for slip rates: Bulletin of the Seismological Society of America, v. 92, p. 2782-2802.
- Matti, J.C., Morton, D.M., and Cox, B.F., Carson, S.E., and Yetter, T.J., 1992a, Geologic setting of the Yucaipa quadrangle, San Bernardino and Riverside counties, California: U.S. Geological Survey, Open File Report 92-446.
- Matti, J.C., Morton, D.M., and Cox, B.F., 1992b, The San Andreas fault system in the vicinity of the central Transverse Ranges province, southern California: U.S. Geological Survey, Open File Report 92-354
- Matti, J.C., Morton, D.M., and Cox, B.F., 1985, Distribution and geologic relations of fault systems in the vicinity of the Central Transverse Ranges, southern California: U.S. Geological Survey, Open File Report 85-365.
- Mendenhall, W.C., 1905, The hydrology of the San Bernardino Valley, California: U.S. Geological Survey Water Supply Paper 142.
- Michael Brandman Associates, 1988, Final environmental impact report no. 229 for Oak Valley specific plan no. 216 (state clearinghouse # 87033011): Prepared for County of Riverside, Riverside CA, dated October 7, 1988.
- Rasmussen, G.S., 1982, Geologic features and rate of movement along the south branch of the San Andreas fault, San Bernardino, California; geologic hazards along the San Andreas fault system, San Bernardino-Hemet-Chino California, Field Trip No. 4: *in* Neotectonics in southern California: volume and guidebook, 78th annual meeting, Cordilleran Section, Geological Society of America, p. 109-114.
- Rasmussen Associates, 1978, Preliminary engineering geology investigation of proposed 680-acre Singleton Ranch development, Calimesa, Riverside county, California: Unpublished consultants's report prepared for Construction Development Corporation.

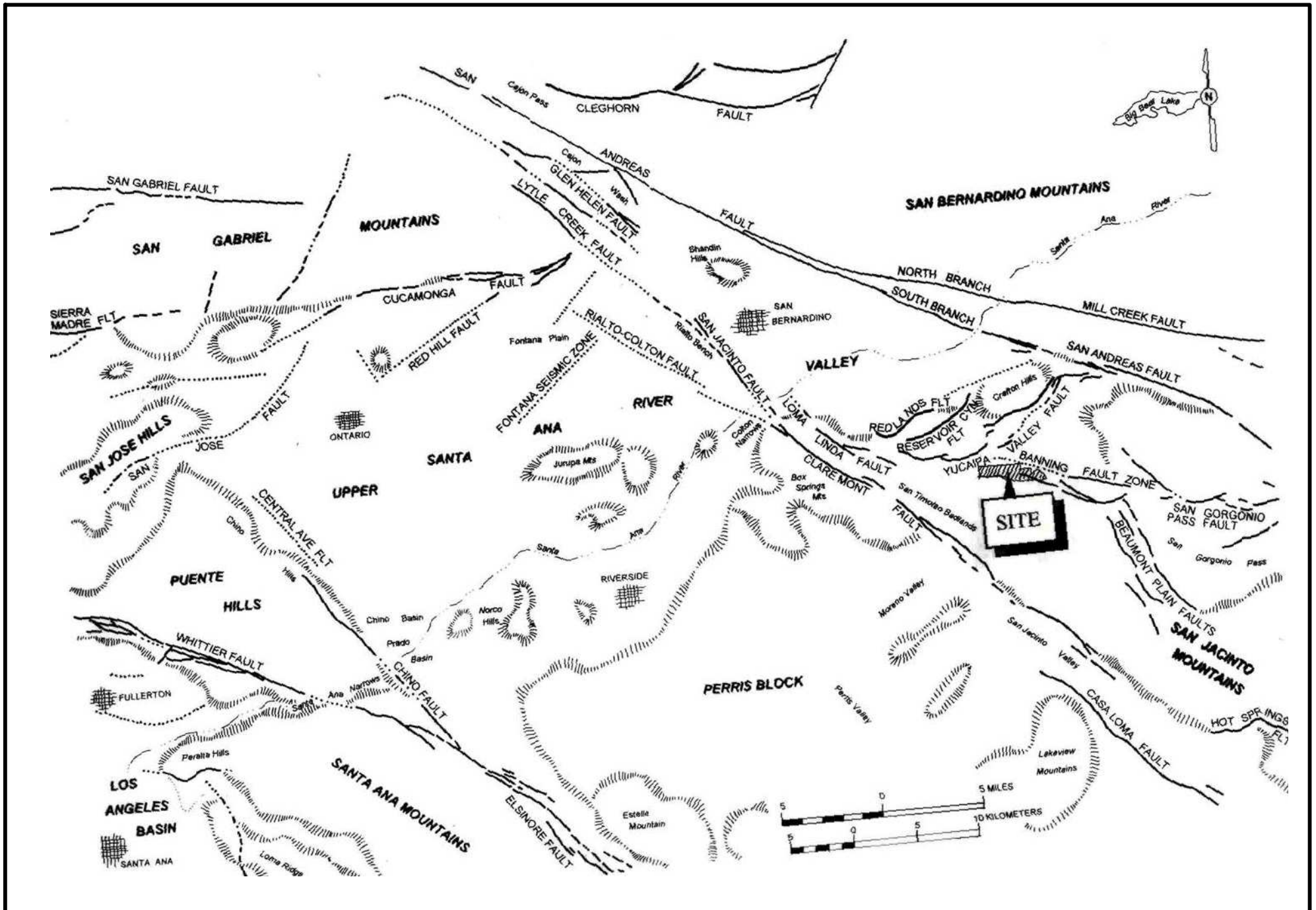
**Fiesta Development, Inc.
Oak Valley at Calimesa**

- Rasmussen Associates, 1983, Preliminary engineering geology investigation of the proposed 1350 acre Canyon Oaks Development, west of County Line Road, San Bernardino and Riverside Counties, Calimesa, California, Project No. 1789, dated December 22, 1983
- Rasmussen Associates, 1984, Preliminary engineering geology investigation of proposed 776-acre McCown Ranch development, Calimesa, Riverside County, CA, Unpublished consultant's report project No. 2093, prepared for Daniel, Mann, Johnson & Mendenhall, Los Angeles, California, dated December 5, 1984
- Rasmussen Associates, 1988, Subsurface engineering geology investigation, northeast portion of McCown Ranch, Oak Valley, Southwest of Sandalwood Drive and 7th Street, Calimesa, California: Unpublished consultant's Report Project No. 2529.4, Prepared for Landmark Land Company of California, Inc., Moreno Valley, CA, dated September 29, 1988.
- Reynolds, R.E., and Reeder, W.A., 1991, The San Timoteo Formation, Riverside county, California: San Bernardino county Museum Association Quarterly, v. 39, p. 44-48.
- Petra, 2004, Proposal for limited geotechnical investigation in support of EIR activities for Oak Valley at Calimesa, Riverside County, California: submitted to Fiesta Development, Inc, Corona CA, Project Number 1140-04, dated March 8, 2004
- Shuler, E.H, 1953, Geology of a portion of the San Timoteo Canyon badlands near Beaumont, California: University of Southern California Unpublished MS thesis.
- Smith, G.A., Reynolds, R.E., Lerch, M.K., Burford, W.T., 1983, Environmental studies at the Haskell Ranch, tentative parcels 19014 and 19015, San Timoteo Canyon, Riverside county, California: Unpublished consultant's report for Harshell Ranch and Land Company.
- The Keith Companies, Inc., 2003, Oak Valley at Calimesa, Specific Plan amendment to Oak Valley SP 216 and 216A: Unpublished report prepared for Fiesta Development, Inc, Corona, Ca.

FIGURES



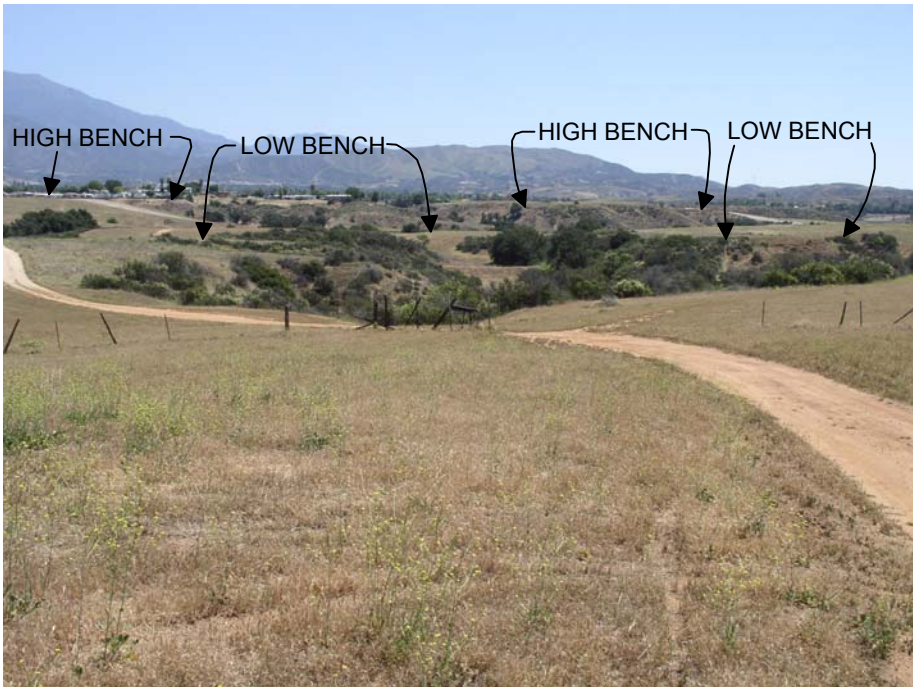
Project No. 04-0419	Oak Valley Calimesa, Riverside County, California	SITE LOCATION MAP	Figure 1
HUSHMAND ASSOC./ PETRA			



Project No. 04-0419	Oak Valley Calimesa, Riverside County, CA	MAJOR ACTIVE EARTHQUAKE FAULTS AND PHYSIOGRAPHIC FEATURES	Figure 2
HUSHMAND ASSOC./ PETRA			



3A. Looking northerly toward Yucapia, Crafton Hills, and the San Bernardino Mountains from central part of site.



3B. Looking easterly toward Calimesa and the San Bernardino Mountains. The Cherry Valley fault separates the higher bench from the lower bench.

Project No. 04-0419	Oak Valley Calimesa, Riverside County, California	PHOTOGRAPHS ILLUSTRATING THE TOPOGRAPHY IN THE EASTERN PART OF THE SITE	Figure 3
HUSHMAND ASSOC./ PETRA			



4A. Light colored material just below top of mesa is carbonate soil horizon of the Qrp unit in the western part of the site.



4B. Looking easterly from the central part of the site. The white ground is the carbonate horizon of the Qrp unit.

Project No. 04-0419	Oak Valley Calimesa, Riverside County, California	PHOTOGRAPHS OF CARBONATE-RICH SOILS COMMON WITHIN THE SITE	Figure 4
HUSHMAND ASSOC./ PETRA			

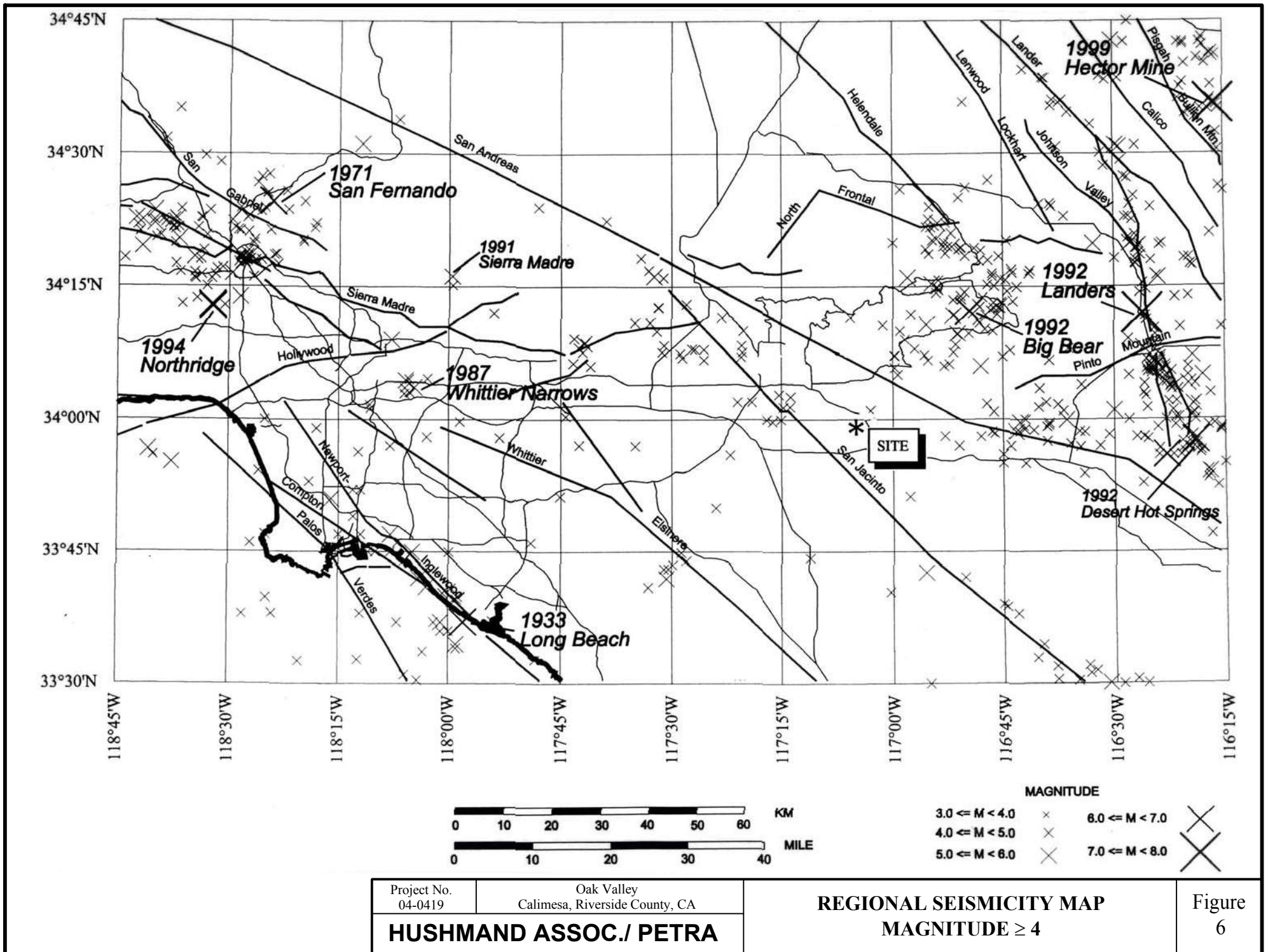


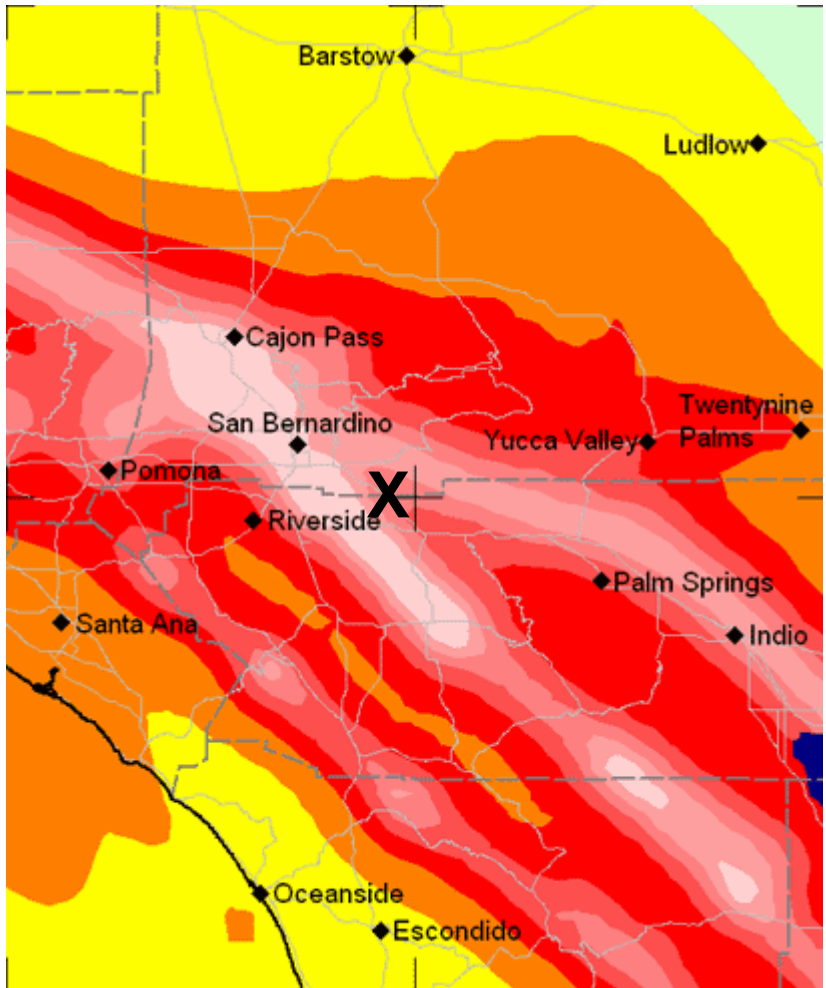
5A. Looking easterly at one of the larger valleys in the northwestern part of the site.



5B. Typical San Timoteo Formation. This exposure of poorly bedded siltstone, sandstone and conglomerate is in the arroyo incised into the bottom of Covington Canyon.

Project No. 04-0419	Oak Valley Calimesa, Riverside County, California	PHOTOGRAPHS OF LARGE VALLEY AND TYPICAL SAN TIMOTEO FORMATION	Figure 5
HUSHMAND ASSOC./ PETRA			





Shaking (%g)
Pga (Peak Ground Acceleration)

- Firm Rock*
- < 10%
 - 10 - 20%
 - 20 - 30%
 - 30 - 40%
 - 40 - 50%
 - 50 - 60%
 - 60 - 70%
 - 70 - 80%
 - > 80%

The unit "g" is acceleration of gravity.

X marks the site.

Project No. 04-0419	Oak Valley Calimesa, Riverside County, California	PROBABILISTIC GROUND MOTION MAP	Figure 7
HUSHMAND ASSOC./ PETRA			

APPENDIX A

LOGS OF EXPLORATORY BORINGS

SOIL CLASSIFICATION SYSTEM ASTM D2487

MAJOR DIVISIONS			SYMBOLS		TYPICAL DESCRIPTIONS
			GRAPH	LETTER	
COARSE GRAINED SOILS MORE THAN 50% OF MATERIAL IS LARGER THAN NO. 200 SIEVE SIZE	GRAVEL AND GRAVELLY SOILS MORE THAN 50% OF COARSE FRACTION RETAINED ON NO. 4 SIEVE	CLEAN GRAVELS (LITTLE OR NO FINES)		GW	WELL-GRADED GRAVELS, GRAVEL - SAND MIXTURES, LITTLE OR NO FINES
		GRAVELS WITH FINES (APPRECIABLE AMOUNT OF FINES)		GP	POORLY-GRADED GRAVELS, GRAVEL - SAND MIXTURES, LITTLE OR NO FINES
		GRAVELS WITH FINES (APPRECIABLE AMOUNT OF FINES)		GM	SILTY GRAVELS, GRAVEL - SAND - SILT MIXTURES
		GRAVELS WITH FINES (APPRECIABLE AMOUNT OF FINES)		GC	CLAYEY GRAVELS, GRAVEL - SAND - CLAY MIXTURES
	SAND AND SANDY SOILS MORE THAN 50% OF COARSE FRACTION PASSING ON NO. 4 SIEVE	CLEAN SANDS (LITTLE OR NO FINES)		SW	WELL-GRADED SANDS, GRAVELLY SANDS, LITTLE OR NO FINES
		CLEAN SANDS (LITTLE OR NO FINES)		SP	POORLY-GRADED SANDS, GRAVELLY SAND, LITTLE OR NO FINES
SANDS WITH FINES (APPRECIABLE AMOUNT OF FINES)			SM	SILTY SANDS, SAND - SILT MIXTURES	
FINE GRAINED SOILS MORE THAN 50% OF MATERIAL IS SMALLER THAN NO. 200 SIEVE SIZE	SILTS AND CLAYS LIQUID LIMIT LESS THAN 50		ML	INORGANIC SILTS AND VERY FINE SANDS, ROCK FLOUR, SILTY OR CLAYEY FINE SANDS OR CLAYEY SILTS WITH SLIGHT PLASTICITY	
			CL	INORGANIC CLAYS OF LOW TO MEDIUM PLASTICITY, GRAVELLY CLAYS, SANDY CLAYS, SILTY CLAYS, LEAN CLAYS	
			OL	ORGANIC SILTS AND ORGANIC SILTY CLAYS OF LOW PLASTICITY	
	SILTS AND CLAYS LIQUID LIMIT GREATER THAN 50		MH	INORGANIC SILTS, MICACEOUS OR DIATOMACEOUS FINE SAND OR SILTY SOILS	
			CH	INORGANIC CLAYS OF HIGH PLASTICITY	
			OH	ORGANIC CLAYS OF MEDIUM TO HIGH PLASTICITY, ORGANIC SILTS	
HIGHLY ORGANIC SOILS			PT	PEAT, HUMUS, SWAMP SOILS WITH HIGH ORGANIC CONTENTS	

NOTE: DUAL SYMBOLS ARE USED TO INDICATE BORDERLINE SOIL CLASSIFICATIONS

- | | | |
|--|-----------------------------|--------------------------------------|
| | BULK SAMPLE | R = R Value |
| | MODIFIED CALIFORNIA SAMPLER | CONS = Consolidation |
| | ROCK CORE | SA = Sieve (Particle Size) Analysis |
| | STANDARD PENETRATION TEST | COMP = Compaction Test |
| | NO RECOVERY IN SAMPLER | COLL = Collapse Potential |
| | SHELBY TYBE | EIT = Expansion Index Test |
| | GROUNDWATER SURFACE | SE = Sand Equivalent |
| | | UC = Unconfined Compression |
| | | DS = Direct Shear |
| | | HA = Hydrometer Analysis |
| | | #200 = Percentage Passing #200 Sieve |
| | | AL = Atterberg Limits |
| | | CORR = Corrosion Potential |
| | | PP = Pocket Penetrometer |



CLIENT FIESTA DEVELOPMENT CO. PROJECT NAME OAK VALLEY
 PROJECT NUMBER 04-0419 PROJECT LOCATION CALIMESA, RIVERSIDE COUNTY, CALIFORNIA
 DATE STARTED 4/19/04 COMPLETED 4/19/04 GROUND ELEVATION N/A HOLE SIZE 8"
 DRILLING CONTRACTOR CAL-PAC Drilling GROUND WATER LEVELS:
 DRILLING METHOD HSA AT TIME OF DRILLING None
 LOGGED BY BS CHECKED BY BH AT END OF DRILLING None
 NOTES CaCO3= Calcium Carbonate or Caliche. AFTER DRILLING None

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	CORE SAMPLE	BULK SAMPLE	SAMPLE NUMBER	BLOW COUNTS (N VALUE)	DRY UNIT WT. (pcf)	MOISTURE CONTENT (%)	OTHER LABORATORY TESTS
		SILTY CLAY: Dark red (2.5YR3/6), slightly moist, very stiff, with roots.			AU SK1			8.6	
		SILT/CLAY: Very pale brown (10YR7/4), slightly moist to dry, dense, calcareous soil (K) horizon with CaCO3 nodules. Slightly darker color (10YR6/6) at 4 1/2 ft. Brownish yellow also with calcareous nodules.			MC 1A MC 1B	13-40-46 (86)	98.2 90.7	11.9 7.5	
5		SILT/CLAY: Brownish yellow (10YR6/6) with white and very pale brown layers and zones. Stiff. Carbonate disseminated throughout and as nodules, some are hard due to cementation with CaCO3. This is still a (K) soil horizon.			MC 2	33-50/2"	92.1	8.7	
10		same: Yellowish-brown (10YR5/4) with white and very pale brown layers, spots, streaks and CaCO3 nodules. Slightly moist, stiff to hard.			MC 3	20-50/5"	101.8	14.4	
15		CLAYEY SILT: Strong brown (7.5YR5/6) with trace of white specks, slightly moist, stiff to hard, with hard bits (slightly cemented bits of same color). Low plasticity.			MC 4	15-50/5"	112	13.9	
20		SILTY CLAY and SANDY CLAY: Strong brown (7.5YR4/6) with black (Manganese oxide) spots, joint lining and filaments. Stiff to hard, moist, micaceous zones, low plasticity. Fine to coarse sand (5%) with trace of pebbles (1/4 inch size).			MC 5	8-15-26 (41)	114.7	12.5	
		Borehole terminated at 21.5 feet.							

OAK VALLEY 2 OAK VALLEY.GPJ GINT.US.GDT 10/19/04



CLIENT FIESTA DEVELOPMENT CO. PROJECT NAME OAK VALLEY
 PROJECT NUMBER 04-0419 PROJECT LOCATION CALIMESA, RIVERSIDE COUNTY, CALIFORNIA
 DATE STARTED 4/19/04 COMPLETED 4/19/04 GROUND ELEVATION N/A HOLE SIZE 8"
 DRILLING CONTRACTOR CAL-PAC Drilling GROUND WATER LEVELS:
 DRILLING METHOD HSA AT TIME OF DRILLING None
 LOGGED BY BS CHECKED BY BH AT END OF DRILLING None
 NOTES -- AFTER DRILLING None

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	CORE SAMPLE	BULK SAMPLE	SAMPLE NUMBER	BLOW COUNTS (N VALUE)	DRY UNIT WT. (pcf)	MOISTURE CONTENT (%)	OTHER LABORATORY TESTS
5		SILTY SAND: Dark yellowish brown (10YR4/4), dry, medium dense. Well graded from Silt to Pebbles. Sand 50%, Silt 40%, Coarse Sand 5%, Pebbles 5% (pebbles to 1/2 inch size) (ALLUVIUM)			AU SK1			2.1	COMP
5		SILTY SAND: Yellowish brown (10YR3/4) with Gravel, dry, medium dense, poorly bedded alluvium. Beds of Silty Sand, Sand and Gravel. Gravel to 2" size of igneous and metamorphic rocks, subangular to subrounded, hard. Sand is medium to coarse grained.			MC 1	10-9-7 (16)	101.9	6.1	
10		same: Interbedded SILTY SAND, SAND and GRAVEL, dry, medium dense, well-graded from Silt to Gravel (as above). 50% medium-coarse Sand, 30% Gravel, 20% Silt and fine Sand. Dark yellowish brown (10YR4/4). (ALLUVIUM)			MC 2	9-10-13 (23)	113.3	3.9	
15		same with 2" thick bed of moist Silty Sand with slight cohesion. Sand in this bed is fine to medium with 1-2% coarse Sand, Silt is 20-30%.			MC 3	8-13-19 (32)	119.9	3.9	
20		SAND and GRAVEL: Yellowish brown (10YR5/4), dry to slightly moist, dense. Coarser than above, predominantly coarse 40-50%, Gravel 30%, fine to medium Sand 30%. Less grading than above.			MC 4	14-19-22 (41)	122	2.3	
25		Cuttings same i.e. Brown to yellowish brown Sandy Gravel with Silt: ALLUVIUM.							

OAK VALLEY 2 OAK VALLEY.GPJ GINT.US.GDT 10/19/04



CLIENT FIESTA DEVELOPMENT CO.

PROJECT NAME OAK VALLEY

PROJECT NUMBER 04-0419

PROJECT LOCATION CALIMESA, RIVERSIDE COUNTY, CALIFORNIA

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	CORE SAMPLE	BULK SAMPLE	SAMPLE NUMBER	BLOW COUNTS (N VALUE)	DRY UNIT WT. (pcf)	MOISTURE CONTENT (%)	OTHER LABORATORY TESTS
30		SILTY SAND and SAND with trace of Gravel (1"): Dark yellowish brown (10YR3/4), moist, medium dense, some layers with slight plasticity. Generally well-graded from Clay to medium Sand with tr. small Gravel.	X		MC 5	6-6-8 (14)	109.7	12.5	
35		No change in cuttings.							
40		SAND and GRAVEL: Yellowish brown, slightly moist, medium dense, well-graded coarse Sand to Gravel; several broken rocks indicate max size > 2-1/2". Igneous and Metamorphic Rocks, subrounded to subangular, some schist rocks crumble when handled. Many rocks highly oxidized and weathered. (SAN TIMOTEO FORMATION - Highly weathered)	X		MC 6	48-50/1"	123.2	3	
45		No changes in cuttings.							
50		GRAVEL: 3" fragment of Diorite rock. Blocked sampler, material probably similar to above i.e. Sand and Gravel.	X		MC 7	50/2"	133.4	1.7	
		Borehole terminated at 50.17 feet.							

OAK VALLEY 2 OAK VALLEY.GPJ GINT US.GDT 10/19/04



CLIENT <u>FIESTA DEVELOPMENT CO.</u>	PROJECT NAME <u>OAK VALLEY</u>
PROJECT NUMBER <u>04-0419</u>	PROJECT LOCATION <u>CALIMESA, RIVERSIDE COUNTY, CALIFORNIA</u>
DATE STARTED <u>4/19/04</u> COMPLETED <u>4/19/04</u>	GROUND ELEVATION <u>N/A</u> HOLE SIZE <u>8"</u>
DRILLING CONTRACTOR <u>CAL-PAC Drilling</u>	GROUND WATER LEVELS:
DRILLING METHOD <u>HSA</u>	AT TIME OF DRILLING <u>None</u>
LOGGED BY <u>BS</u> CHECKED BY <u>BH</u>	AT END OF DRILLING <u>None</u>
NOTES <u>---</u>	AFTER DRILLING <u>None</u>

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	CORE SAMPLE	BULK SAMPLE	SAMPLE NUMBER	BLOW COUNTS (N VALUE)	DRY UNIT WT. (pcf)	MOISTURE CONTENT (%)	OTHER LABORATORY TESTS
		CLAYEY SILT with SAND: Yellowish red (5YR4/6), Argillic B Soil Horizon, dry, hard, few plant roots.							
		Same in upper rings i.e. Argillic B Soil Horizon.							
5		SANDY CLAY: Yellowish red (5YR4/6). Slightly moist, hard, prismatic soil structure with Clay-lined peds with few root filaments. Sand comprises 5-20% fine to coarse grained. Color becomes lighter with depth indicating less Clay.			MC 1	19-26-38 (64)	123.1	5.5	
		CLAYEY SAND in lower rings.							
		SAND with GRAVEL: Strong brown (7.5YR6/6) with white specks (Sand and rock grains). Dry to slightly moist, dense to hard. 50% Gravel, 40-50% Sand, locally 10% Silt and Clay. Rocks are crystalline igneous and metamorphic, subrounded to subangular, hard. Maximum size 1.5 to 2.0 inches. (SAN TIMOTEO FORMATION)			MC 2	16-17-19 (36)	123.1	3.7	
10		Same SAND and GRAVEL: Yellowish red (5YR5/8), moist, dense. Crystalline igneous (diortitic) and metamorphic rocks, subangular to subrounded. Few rocks crumble under finger pressure but most are hard. Well-graded, maximum size > 2-1/2".			MC 3	15-18-23 (41)	118.4	2.5	
15		SAND and GRAVEL: Strong brown (7.5YR5/8). Moist, dense, well-graded, medium to coarse Sand with 3% Gravel to 1" size. Poorly bedded.			MC 4	24-40-50/3"	118.1	3.8	
20		same: Reddish yellow (7.5YR6/6), moist, dense, well-graded fine to coarse with 2-3% gravel to 1" size. Poorly bedded.			MC 5	27-50/5"	107.8	2.9	
		Borehole terminated at 20.92 feet.							

OAK VALLEY 2 OAK VALLEY.GPJ GINT.US.GDT 10/19/04



CLIENT FIESTA DEVELOPMENT CO. PROJECT NAME OAK VALLEY
 PROJECT NUMBER 04-0419 PROJECT LOCATION CALIMESA, RIVERSIDE COUNTY, CALIFORNIA
 DATE STARTED 4/19/04 COMPLETED 4/19/04 GROUND ELEVATION N/A HOLE SIZE 8"
 DRILLING CONTRACTOR CAL-PAC Drilling GROUND WATER LEVELS:
 DRILLING METHOD HSA AT TIME OF DRILLING None
 LOGGED BY BS CHECKED BY BH AT END OF DRILLING None
 NOTES --- AFTER DRILLING None

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	CORE SAMPLE	BULK SAMPLE	SAMPLE NUMBER	BLOW COUNTS (N VALUE)	DRY UNIT WT. (pcf)	MOISTURE CONTENT (%)	OTHER LABORATORY TESTS
5		SILTY CLAY with SAND: Strong brown (7.5YR5/8), dry, dense/stiff tr. coarse Sand and fine Gravel (1/4" size), 5-10% fine to medium Sand, 30% Silt, 50% Clay.			AU SK1			4.7	
		same grading down into SANDY CLAY: Yellowish red (5YR4/6). Dry, hard, 10% coarse Sand in Clay matrix. Argillic B Soil Horizon, well-developed prismatic soil structure with Clay filaments on peds (mature soil).	X		MC 1	18-24-34 (58)	114.2	10.4	
		same with GRAVEL: Less red, strong brown (7.5YR5/8), dry, hard. Lower part of B horizon. 20-30% Gravel to 1/2" size.	X		MC 2	15-15-18 (33)	117.4	4.7	
10		SAND: Reddish yellow (7.5YR6/8), slightly moist, dense. Poorly graded medium to coarse Sand, tr. small Gravel (1/4-inch).	X		MC 3	20-50/3"	110	5.8	
15		SANDY CLAY: Strong brown (7.5YR5/8), moist, stiff with white caliche veins and some black spots and streaks (MnO). Finely micaceous.	X		MC 4	28-30-50/4"	108.3	22.5	
		SAND and GRAVEL: Yellowish brown, slightly moist, dense, well-graded (SAN TIMOTEO FORMATION).							
20		SAND and GRAVEL: Yellowish brown (10YR5/4-5/6). Slightly moist, dense, well-graded from fine Sand to Gravel of 2" size. Gravel consists of igneous and metamorphic rocks, subrounded to subangular, hard.	X		MC 5	50/5"	114.7	3.7	
		Borehole terminated at 20.42 feet.							

OAK VALLEY 2 OAK VALLEY.GPJ GINT.US.GDT 10/19/04



CLIENT FIESTA DEVELOPMENT CO. **PROJECT NAME** OAK VALLEY
PROJECT NUMBER 04-0419 **PROJECT LOCATION** CALIMESA, RIVERSIDE COUNTY, CALIFORNIA
DATE STARTED 4/19/04 **COMPLETED** 4/19/04 **GROUND ELEVATION** N/A **HOLE SIZE** 8"
DRILLING CONTRACTOR CAL-PAC Drilling **GROUND WATER LEVELS:**
DRILLING METHOD HSA **AT TIME OF DRILLING** None
LOGGED BY BS **CHECKED BY** BH **AT END OF DRILLING** None
NOTES -- **AFTER DRILLING** None

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	CORE SAMPLE	BULK SAMPLE	SAMPLE NUMBER	BLOW COUNTS (N VALUE)	DRY UNIT WT. (pcf)	MOISTURE CONTENT (%)	OTHER LABORATORY TESTS
		SANDY CLAY: Brownish yellow (10YR6/6), dry, hard, some roots.			AU SK1			9.4	CORR
		same, high carbonate content.			MC 1	50/5"	85.3	12	
5		SILTY SAND/ SANDY SILT with CLAY and tr. GRAVEL: Strong Brown (7.5YR5/6), dry, hard. Lower part of Argillic B Soil Horizon.			MC 2	12-26-50/5"	128.4	7.8	
10		SAND AND GRAVEL: Strong Brown (7.5YR5/6), slightly moist, dense, well-graded from Silt to fine Gravel (10%). (SAN TIMOTEO FORMATION)			MC 3	50/5"	114.6	6.4	
15		SAND AND GRAVEL: Dark yellowish brown (10YR4/6) with white and black specks (MnO), slightly moist, dense, well-graded from Silt to Gravel (1" size). (SAN TIMOTEO FORMATION - Highly weathered)			MC 4	46-50/3"	115.6	5.2	
20		same, GRAVEL to 2" size.			MC 5	50/5"	114.3	2.8	
		Borehole terminated at 20.42 feet.							

OAK VALLEY 2 OAK VALLEY.GPJ GINT US.GDT 10/19/04



CLIENT <u>FIESTA DEVELOPMENT CO.</u>	PROJECT NAME <u>OAK VALLEY</u>
PROJECT NUMBER <u>04-0419</u>	PROJECT LOCATION <u>CALIMESA, RIVERSIDE COUNTY, CALIFORNIA</u>
DATE STARTED <u>4/19/04</u> COMPLETED <u>4/19/04</u>	GROUND ELEVATION <u>N/A</u> HOLE SIZE <u>8"</u>
DRILLING CONTRACTOR <u>CAL-PAC Drilling</u>	GROUND WATER LEVELS:
DRILLING METHOD <u>HSA</u>	AT TIME OF DRILLING <u>None</u>
LOGGED BY <u>BS</u> CHECKED BY <u>BH</u>	AT END OF DRILLING <u>None</u>
NOTES <u>---</u>	AFTER DRILLING <u>None</u>

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	CORE SAMPLE	BULK SAMPLE	SAMPLE NUMBER	BLOW COUNTS (N VALUE)	DRY UNIT WT. (pcf)	MOISTURE CONTENT (%)	OTHER LABORATORY TESTS
		SANDY CLAY: Brown, dry, hard, abundant roots and grass.							
5		SILTY SAND with GRAVEL: Dark red (2.5YR3/6) to dark reddish brown (5YR4/4), dry, hard. 10% fine Gravel to 1/2", 40% Silt, 45% Sand. Igneous and Metamorphic Rocks. Root filaments. Mature Argillic B Soil Horizon.			MC 1	38-22-30 (52)	117.6	7.1	
		SAND and GRAVEL in SANDY CLAY matrix: Yellowish brown (5YR5/6) with lighter specks (rocks). Slightly moist to dry, hard. This is lower part of mature Argillic B Horizon. Well-graded to rocks 1" size.			MC 2	12-28-28 (56)	115	4.9	
10		SANDY CLAY: Reddish yellow (7.5YR6/6), slightly moist, dense/stiff, 10-20% Sand. SANDY CLAY: White to very pale brown, slightly moist, stiff to hard, Calcareous K horizon.			MC 3	20-39-50/2"	97.6	22.2	
		SILTY CLAY: Light brown (7.5YR6/4), moist, very stiff to hard, with black specks, low plasticity.							
15		SILTY CLAY: Light yellowish brown (10YR6/4) with black specks and yellowish brown streaks (oxidation), moist, stiff to hard.			MC 4	22-30-50/3"	109.8	13.4	
20		Same with tr. medium Sand grains in upper rings.			MC 5	19-50/2"	115.1	14.5	
		SILTY SAND (at the bottom): Yellowish brown (10YR5/4), moist, dense, fine to medium Sand, 20% Silt. Borehole terminated at 20.67 feet.							

OAK VALLEY 2 OAK VALLEY.GPJ GINT.US.GDT 10/19/04



CLIENT FIESTA DEVELOPMENT CO. PROJECT NAME OAK VALLEY
 PROJECT NUMBER 04-0419 PROJECT LOCATION CALIMESA, RIVERSIDE COUNTY, CALIFORNIA
 DATE STARTED 4/19/04 COMPLETED 4/19/04 GROUND ELEVATION N/A HOLE SIZE 8"
 DRILLING CONTRACTOR CAL-PAC Drilling GROUND WATER LEVELS:
 DRILLING METHOD HSA AT TIME OF DRILLING None
 LOGGED BY BS CHECKED BY BH AT END OF DRILLING None
 NOTES --- AFTER DRILLING None

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	CORE SAMPLE	BULK SAMPLE	SAMPLE NUMBER	BLOW COUNTS (N VALUE)	DRY UNIT WT. (pcf)	MOISTURE CONTENT (%)	OTHER LABORATORY TESTS
		SILTY CLAY with SAND: Red (2.5YR4/6), dry to slightly moist, very stiff, 30% Silt, 2-3% fine Sand, tr. medium Sand.							
		same: Dark reddish brown (2.5YR3/4) to yellowish red (5YR5/6), dry, hard. This is mature Argillic B Soil Horizon, 5% medium to coarse Sand. Low plasticity.	X		MC 1	18-14-21 (35)	112.7	6.9	
5		Color change near 5'. SANDY CLAY: White to very pale brown (10YR7/3), dry to slightly moist, hard. This is K soil horizon-caliche (Carbonate), 5-10% fine Sand, tr. fine Gravel (1/4-inch). White cuttings.	X		MC 2	50/5"	102.3	11	
10		Same: GRAVEL to 2" size.	X		MC 3	23-50/5"	116.4	10.4	
15		SILT with Calcareous Nodules and SILTY SAND: Light olive brown (2.5YR5/4) mottled, slightly moist, stiff/dense, micaceous. Silt 20%, fine to medium Sand 60-70%, 10% Coarse, 5% fine Gravel. Reddish brown cuttings.	X		MC 4	24-50/5"	108.5	9.1	
20		SAND and GRAVEL: Yellowish brown (10YR5/6), moist, dense, well-graded from Silt to Gravel (max. 2-1/2" size). Gravel is igneous and metamorphic rocks, subrounded to subangular, hard to crumbly rocks. (SAN TIMOTEO FORMATION) Borehole terminated at 20.83 feet.	X		MC 5	30-50/4"	109.4	4.9	

OAK VALLEY 2 OAK VALLEY.GPJ GINT.US.GDT 10/19/04



CLIENT FIESTA DEVELOPMENT CO. PROJECT NAME OAK VALLEY
PROJECT NUMBER 04-0419 PROJECT LOCATION CALIMESA, RIVERSIDE COUNTY, CALIFORNIA
DATE STARTED 4/19/04 COMPLETED 4/19/04 GROUND ELEVATION N/A HOLE SIZE 8"
DRILLING CONTRACTOR CAL-PAC Drilling GROUND WATER LEVELS:
DRILLING METHOD HSA AT TIME OF DRILLING None
LOGGED BY BS CHECKED BY BH AT END OF DRILLING None
NOTES --- AFTER DRILLING None

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	CORE SAMPLE	BULK SAMPLE	SAMPLE NUMBER	BLOW COUNTS (N VALUE)	DRY UNIT WT. (pcf)	MOISTURE CONTENT (%)	OTHER LABORATORY TESTS
		CLAYEY SILT/SILTY CLAY: White, dry, stiff, Calcareous K horizon.			AU SK1			13	
		same: slightly moist, low plasticity.	X		MC 1	12-18-27 (45)	91.1	14.4	
5		same: mottled white to yellow (10YR7/6), slightly moist, stiff with hard calcareous nodules and carbonate disseminated throughout sample with filaments common.	X		MC 2	21-41-46 (87)	104.1	11.9	
10		same, color predominantly light yellowish brown (10YR6/4) but white zones and veins common, slightly moist, stiff with hard calcareous zones, low plasticity. White layers commonly horizontal.	X		MC 3	13-26-45 (71)	106.9	17.2	
15		SILTY CLAY/CLAYEY SILT with SAND: Yellowish brown (10YR5/6) to strong brown (7.5YR5/6), 5-10% medium to coarse Sand, tr. small pebbles (1/4") and tr. Calcareous nodules and streaks. Slightly moist, stiff to hard, low plasticity.	X		MC 4	15-26-33 (59)	112.1	15.1	
20		SILTY CLAY: Brown (7.5YR5/4) and yellowish brown (10YR5/8) with few white veins, moist, stiff, micaceous, low plasticity.	X		MC 5	10-21-30 (51)	105.8	15.8	
		Borehole terminated at 21.5 feet.							

OAK VALLEY 2 OAK VALLEY.GPJ GINT.US.GDT 10/19/04



CLIENT FIESTA DEVELOPMENT CO. PROJECT NAME OAK VALLEY
PROJECT NUMBER 04-0419 PROJECT LOCATION CALIMESA, RIVERSIDE COUNTY, CALIFORNIA
DATE STARTED 4/19/04 COMPLETED 4/19/04 GROUND ELEVATION N/A HOLE SIZE 8"
DRILLING CONTRACTOR CAL-PAC Drilling GROUND WATER LEVELS:
DRILLING METHOD HSA AT TIME OF DRILLING None
LOGGED BY BS CHECKED BY BH AT END OF DRILLING None
NOTES --- AFTER DRILLING None

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	CORE SAMPLE	BULK SAMPLE	SAMPLE NUMBER	BLOW COUNTS (N VALUE)	DRY UNIT WT. (pcf)	MOISTURE CONTENT (%)	OTHER LABORATORY TESTS
5		SANDY SILT: with scattered pebbles, dry to slightly moist, medium stiff. Grass and roots common. (VALLEY ALLUVIUM)							
		SANDY SILT: Dark yellowish brown (10YR4/4), slightly moist, medium stiff, 10-20% fine Sand. Massive (unbedded). (ALLUVIUM)	X		MC 1	5-6-10 (16)	102.8	7.6	COLL
10		same but with tr. Gravel several pieces 1/4" and one 2" size.	X		MC 2	5-7-13 (20)	111.5	7.1	
15		SILTY SAND: Reddish yellow (7.5YR6/6) with few small darker mottles. Predominantly fine Sand with 5% coarse Sand and trace of small pebbles (1/4"). Pebbles subrounded and composed of igneous and metamorphic rocks.	X		MC 3	7-9-10 (19)	117.4	6.8	COLL
20		same.	X		MC 4	8-9-14 (23)	120.2	9.2	
25		Cuttings all dark brown with occassional pebbles.							

OAK VALLEY 2 OAK VALLEY.GPJ GINT.US.GDT 10/19/04



CLIENT FIESTA DEVELOPMENT CO.

PROJECT NAME OAK VALLEY

PROJECT NUMBER 04-0419

PROJECT LOCATION CALIMESA, RIVERSIDE COUNTY, CALIFORNIA

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	CORE SAMPLE	BULK SAMPLE	SAMPLE NUMBER	BLOW COUNTS (N VALUE)	DRY UNIT WT. (pcf)	MOISTURE CONTENT (%)	OTHER LABORATORY TESTS
30		SILTY SAND with CLAY and GRAVEL: Dark yellowish brown (10YR4/6) with a few faint lighter filaments, moist, dense. Predominantly fine Sand with 5% Clay, 10-30% Silt, and 5% gravel ranging from 1/4" to 1/2" size. Hard subangular igneous and metamorphic rocks. Some cohesion, low plasticity. (ALLUVIUM)	X		MC 5	7-2-4 (6)	115.7	12.1	
35		Cuttings all dark brown with occasional pebbles.							
40		SAND and GRAVEL: Yellowish brown (10YR5/4) with white specks (rocks), slightly moist, dense. Well-graded from fine sand to gravel > 2-1/2" diameter. Predominantly igneous rocks. Very much different than brown materials above. (SAN TIMOTEO FORMATION)	X		MC 6	28-22-30 (52)	117.1	3.4	
45		Added water to help drilling.							
50		SILTY CLAY: Strong brown (7.5YR4/6), moist, very stiff to hard, finely micaceous modulus of plasticity.	X		MC 7	14-28-50/4"	117.5	16.6	
		Borehole terminated at 51.33 feet.							

OAK VALLEY 2 OAK VALLEY.GPJ GINT.US.GDT 10/19/04

APPENDIX B

LABORATORY TEST RESULTS

MOISTURE CONTENT AND DRY DENSITY OF RING SAMPLES

Client: Petra Geotechnical, Inc.
Project Name: Oak Valley
Project No.: -----

HAI Project No.: 04-0419
Date: 4/27/04

Boring No.	B1						B2					
	1A	1B	2	3	4	5	1	2	3	4	5	
Sample No.												
Depth (ft.)	2	2	5	10	15	20	5	10	15	20	30	
Sample type (Tube-Ring-Shelby)	R	R	R	R	R	R	R	R	R	R	R	
Total wt of container and soil	gr	1067.4	979.9	1002.6	1113.5	1193.8	1209.9	879.3	1115.1	969.1	1164.7	1165.7
Height of container	in	6	6	6	6	6	6	5	6	5	6	6
Diameter of container	in	2.416	2.416	2.416	2.416	2.416	2.416	2.416	2.416	2.416	2.416	2.416
Volume of container	cu.ft	0.0159	0.0159	0.0159	0.0159	0.0159	0.0159	0.0133	0.0159	0.0133	0.0159	0.0159
Weight of container	gr	274.68	275.76	280.08	272.64	272.58	278.34	229.35	264.96	219.45	263.64	274.38
Weight of soil	lbs.	1.748	1.552	1.593	1.854	2.031	2.054	1.433	1.874	1.653	1.986	1.965
Wet Density	pcf	109.8	97.5	100.1	116.5	127.6	129.0	108.0	117.7	124.6	124.8	123.4
Container No.		P 4	P 84	P 25	P 85	P 9	P 15	P 30	P 86	P 31	P 26	P 2
Weight of cont.+ wet soil	gr	160.54	145.33	120.03	166.31	160.08	159.44	142.49	157.49	135.86	156.54	169.42
Weight of cont.+ dry soil	gr	145.84	136.87	111.33	148.55	143.15	142.92	135.60	152.49	131.55	153.32	153.33
Weight of water	gr	14.70	8.46	8.70	17.76	16.93	16.52	6.89	5.00	4.31	3.22	16.09
Weight of container	gr	21.84	23.68	10.78	25.50	21.75	10.59	21.75	25.62	21.66	10.57	24.80
Weight of dry soil	gr	124.00	113.19	100.55	123.05	121.40	132.33	113.85	126.87	109.89	142.75	128.53
Moisture Content	%	11.9	7.5	8.7	14.4	13.9	12.5	6.1	3.9	3.9	2.3	12.5
Dry Density	pcf	98.2	90.7	92.1	101.8	112.0	114.7	101.9	113.3	119.9	122.0	109.7

MOISTURE CONTENT AND DRY DENSITY OF RING SAMPLES

Client: Petra Geotechnical, Inc.

Project Name: Oak Valley

Project No.: -----

HAI Project No.: 04-0419

Date: 4/27/04

Boring No.	B2		B3					B4				
	6	7	1	2	3	4	5	1	2	3	4	
Sample No.	6	7	1	2	3	4	5	1	2	3	4	
Depth (ft.)	40	50	2	5	10	15	20	2	5	10	15	
Sample type (Tube-Ring-Shelby)	R	R	R	R	R	R	R	R	R	R	R	
Total wt of container and soil	gr	993.9	417.2	1006.6	998.7	1149.8	946.5	896.7	984.7	1162.8	1113.4	1214.8
Height of container	in	5	2	5	5	6	5	5	5	6	6	6
Diameter of container	in	2.416	2.416	2.416	2.416	2.416	2.416	2.416	2.416	2.416	2.416	2.416
Volume of container	cu.ft	0.0133	0.0053	0.0133	0.0133	0.0159	0.0133	0.0133	0.0133	0.0159	0.0159	0.0159
Weight of container	gr	231.00	90.52	224.75	231.00	273.66	209.15	229.05	226.10	275.40	272.94	256.32
Weight of soil	lbs.	1.682	0.720	1.724	1.692	1.932	1.626	1.472	1.672	1.956	1.853	2.113
Wet Density	pcf	126.8	135.7	129.9	127.6	121.3	122.5	111.0	126.1	122.9	116.4	132.7
Container No.		P 5	P 83	P 23	P 35	P 19	P 37	P 18	CK	P 38	P 29	P 27
Weight of cont.+ wet soil	gr	173.08	157.45	126.74	141.56	122.31	121.48	109.49	169.80	161.80	152.66	139.93
Weight of cont.+ dry soil	gr	168.74	155.12	120.66	136.94	119.63	117.47	106.68	156.62	155.08	144.91	116.18
Weight of water	gr	4.34	2.33	6.08	4.62	2.68	4.01	2.81	13.18	6.72	7.75	23.75
Weight of container	gr	21.95	21.70	10.63	11.31	10.65	11.43	10.56	30.42	11.36	10.66	10.72
Weight of dry soil	gr	146.79	133.42	110.03	125.63	108.98	106.04	96.12	126.20	143.72	134.25	105.46
Moisture Content	%	3.0	1.7	5.5	3.7	2.5	3.8	2.9	10.4	4.7	5.8	22.5
Dry Density	pcf	123.2	133.4	123.1	123.1	118.4	118.1	107.8	114.2	117.4	110.0	108.3

MOISTURE CONTENT AND DRY DENSITY OF RING SAMPLES

Client: Petra Geotechnical, Inc.

Project Name: Oak Valley

Project No.: -----

HAI Project No.: 04-0419

Date: 4/27/04

Boring No.	B4	B5					B6					
		5	1	2	3	4	5	1	2	3	4	5
Sample No.	5	1	2	3	4	5	1	2	3	4	5	
Depth (ft.)	20	2	5	10	15	20	2	5	10	15	20	
Sample type (Tube-Ring-Shelby)	R	R	R	R	R	R	R	R	R	R	R	
Total wt of container and soil	gr	585.1	802.1	1274.6	769.3	766.8	746.3	987.1	955.7	1136.7	1172.5	1218.6
Height of container	in	3	5	6	4	4	4	5	5	6	6	6
Diameter of container	in	2.416	2.416	2.416	2.416	2.416	2.416	2.416	2.416	2.416	2.416	2.416
Volume of container	cu.ft	0.0080	0.0133	0.0159	0.0106	0.0106	0.0106	0.0133	0.0133	0.0159	0.0159	0.0159
Weight of container	gr	155.79	227.35	275.28	182.26	181.46	180.68	229.15	229.80	275.58	273.90	266.82
Weight of soil	lbs.	0.946	1.267	2.203	1.294	1.290	1.247	1.671	1.600	1.898	1.981	2.098
Wet Density	pcf	118.9	95.5	138.4	122.0	121.6	117.5	126.0	120.6	119.3	124.5	131.8
Container No.		P 33	P 21	P 20	P 103	QP	EG	JB 1	BG	JB 2	P 112	P 109
Weight of cont.+ wet soil	gr	153.55	116.77	133.36	126.97	154.04	101.86	98.12	130.65	141.54	145.32	160.60
Weight of cont.+ dry soil	gr	148.50	105.39	124.47	119.83	146.87	99.31	92.13	124.93	117.34	129.15	141.25
Weight of water	gr	5.05	11.38	8.89	7.14	7.17	2.55	5.99	5.72	24.20	16.17	19.35
Weight of container	gr	11.32	10.65	10.65	8.03	8.35	8.13	8.28	8.17	8.26	8.30	8.20
Weight of dry soil	gr	137.18	94.74	113.82	111.80	138.52	91.18	83.85	116.76	109.08	120.85	133.05
Moisture Content	%	3.7	12.0	7.8	6.4	5.2	2.8	7.1	4.9	22.2	13.4	14.5
Dry Density	pcf	114.7	85.3	128.4	114.6	115.6	114.3	117.6	115.0	97.6	109.8	115.1

MOISTURE CONTENT AND DRY DENSITY OF RING SAMPLES

Client: Petra Geotechnical, Inc.
Project Name: Oak Valley
Project No.: -----

HAI Project No.: 04-0419
Date: 4/27/04

Boring No.	B7					B8					B9	
	1	2	3	4	5	1	2	3	4	5	1	
Sample No.	1	2	3	4	5	1	2	3	4	5	1	
Depth (ft.)	2	5	10	15	20	2	5	10	15	20	5	
Sample type (Tube-Ring-Shelby)	R	R	R	R	R	R	R	R	R	R	R	
Total wt of container and soil	gr	953.6	912.2	1204.8	1129.1	914.5	1027.9	932.9	1173.7	1207.9	1162.7	894.1
Height of container	in	5	5	6	6	5	6	5	6	6	6	5
Diameter of container	in	2.416	2.416	2.416	2.416	2.416	2.416	2.416	2.416	2.416	2.416	2.416
Volume of container	cu.ft	0.0133	0.0133	0.0159	0.0159	0.0133	0.0159	0.0133	0.0159	0.0159	0.0159	0.0133
Weight of container	gr	228.55	229.10	276.48	273.90	224.00	274.98	232.50	269.34	276.30	277.50	228.25
Weight of soil	lbs.	1.598	1.506	2.047	1.885	1.522	1.660	1.544	1.994	2.054	1.952	1.468
Wet Density	pcf	120.5	113.5	128.6	118.4	114.8	104.3	116.4	125.3	129.0	122.6	110.7
Container No.		P 105	P 111	P 114	P 108	P 107	AS	P 100	P 40	P 8	P 10	P 34
Weight of cont.+ wet soil	gr	141.33	127.17	144.72	153.61	143.96	130.82	129.15	159.94	151.04	149.78	112.80
Weight of cont.+ dry soil	gr	132.75	115.41	131.83	141.44	137.60	115.36	116.33	138.16	132.60	130.75	105.65
Weight of water	gr	8.58	11.76	12.89	12.17	6.36	15.46	12.82	21.78	18.44	19.03	7.15
Weight of container	gr	8.12	8.21	8.35	8.19	8.15	8.16	8.28	11.31	10.60	10.49	11.53
Weight of dry soil	gr	124.63	107.20	123.48	133.25	129.45	107.20	108.05	126.85	122.00	120.26	94.12
Moisture Content	%	6.9	11.0	10.4	9.1	4.9	14.4	11.9	17.2	15.1	15.8	7.6
Dry Density	pcf	112.7	102.3	116.4	108.5	109.4	91.1	104.1	106.9	112.1	105.8	102.8

MOISTURE CONTENT AND DRY DENSITY OF RING SAMPLES

Client: Petra Geotechnical, Inc.
Project Name: Oak Valley
Project No.: -----

HAI Project No.: 04-0419
Date: 4/27/04

Boring No.	B9						
Sample No.	2	3	4	5	6	7	
Depth (ft.)	10	15	20	30	40	50	
Sample type (Tube-Ring-Shelby)	R	R	R	R	R	R	
Total wt of container and soil	gr	1139.3	1179.0	1221.4	1210.2	1135.0	1262.1
Height of container	in	6	6	6	6	6	6
Diameter of container	in	2.416	2.416	2.416	2.416	2.416	2.416
Volume of container	cu.ft	0.0159	0.0159	0.0159	0.0159	0.0159	0.0159
Weight of container	gr	277.38	273.72	273.48	273.30	260.94	273.24
Weight of soil	lbs.	1.900	1.996	2.090	2.065	1.927	2.180
Wet Density	pcf	119.4	125.4	131.3	129.8	121.1	137.0
Container No.		HA-012	P 17	HA-12	P 13	P 7	P 22
Weight of cont.+ wet soil	gr	178.82	155.55	179.90	173.96	143.62	155.94
Weight of cont.+ dry soil	gr	169.33	146.35	167.46	156.33	139.27	135.27
Weight of water	gr	9.49	9.20	12.44	17.63	4.35	20.67
Weight of container	gr	35.71	10.71	32.30	10.67	10.55	10.58
Weight of dry soil	gr	133.62	135.64	135.16	145.66	128.72	124.69
Moisture Content	%	7.1	6.8	9.2	12.1	3.4	16.6
Dry Density	pcf	111.5	117.4	120.2	115.7	117.1	117.5

MOISTURE CONTENT OF BAG SAMPLES

Client: Petra Geotechnical, Inc.
Project Name: Oak Valley
Project No.: -----

HAI Project No.: 04-0419
Date: 4/27/08

Boring No.	B1	B2	B4	B5	B8
Sample No.	SK 1	SK 1	SK 1	SK 1	SK 1
Depth (ft.)	0 - 3	--	--	--	--
Container No.	P 115	JB 3	CY	EP	P 106
Weight of cont.+ wet soil	gr 334.90	345.12	359.21	251.62	179.17
Weight of cont.+ dry soil	gr 308.94	338.14	343.36	230.71	159.52
Weight of water	gr 25.96	6.98	15.85	20.91	19.65
Weight of container	gr 8.37	8.28	8.23	8.18	8.22
Weight of dry soil	gr 300.57	329.86	335.13	222.53	151.30
Moisture Content	% 8.6	2.1	4.7	9.4	13.0



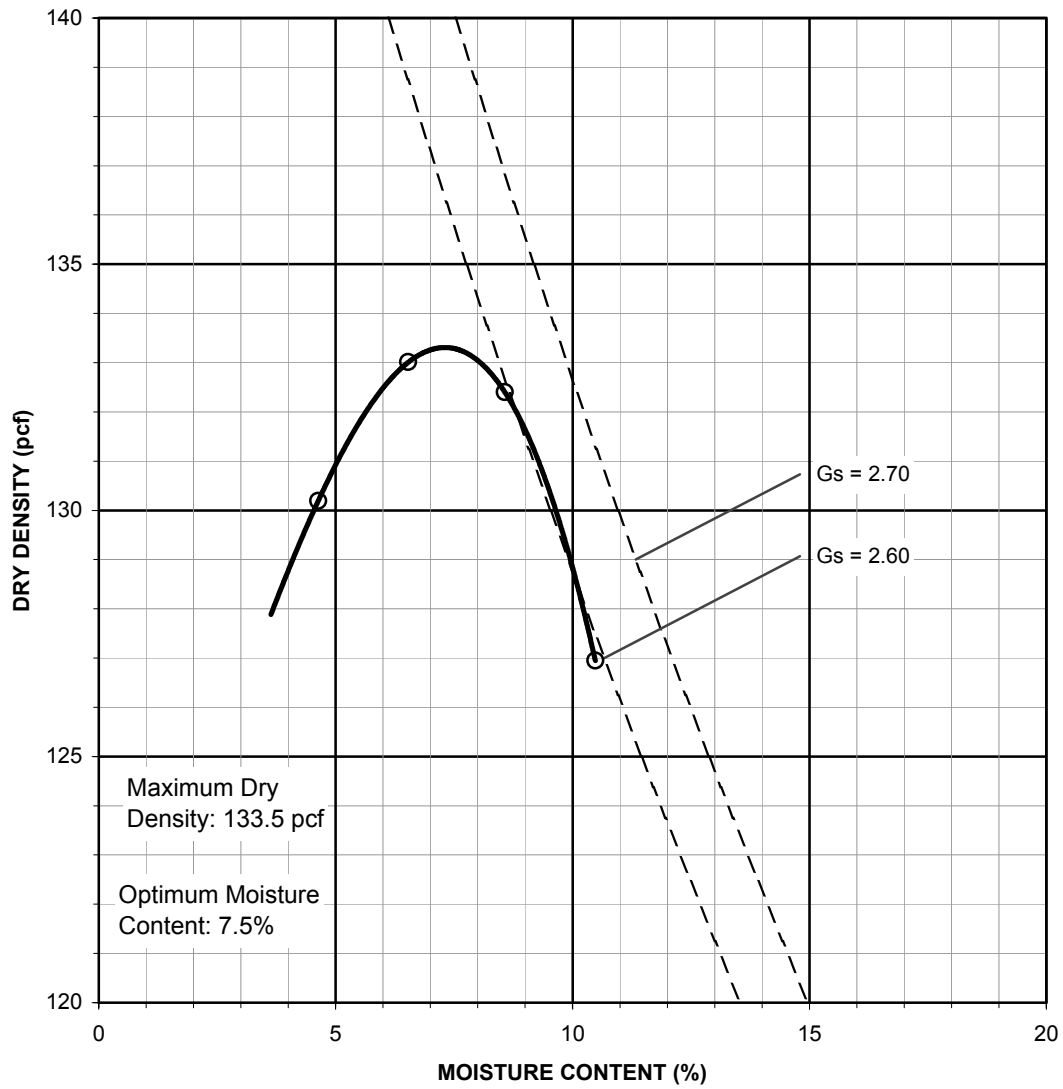


HUSHMAND ASSOCIATES, INC.
Geotechnical and Earthquake Engineers

COMPACTION CURVE ASTM D1557

Client : Petra Geotechnical, Inc.
Project Name: Oak Valley
Project No.: ---
Boring No.: B2
Sample No.: SK1 **Depth:** ---
Soil Description: Dark Yellowish Brown, Silty Sand (SM)

HAI Project No.: 04-0419
Tested by: WG/PM
Checked by: JT
Date: 4/28/2004
Procedure: A
Mold Size: 4 in



CORROSION TEST

(C-643, C-417, C-422)

Client: Petra Geotechnical, Inc

HAI Project No.: 04-0419

Project Name : Oak Valley

Tested by: PM

Project No. : ---

Checked by: JT

Boring No.: B5

Date: 5/4/04

Sample No.: SK1 **Depth:** ---

Soil Description: Brownish Yellow, Sandy Clay (CL)

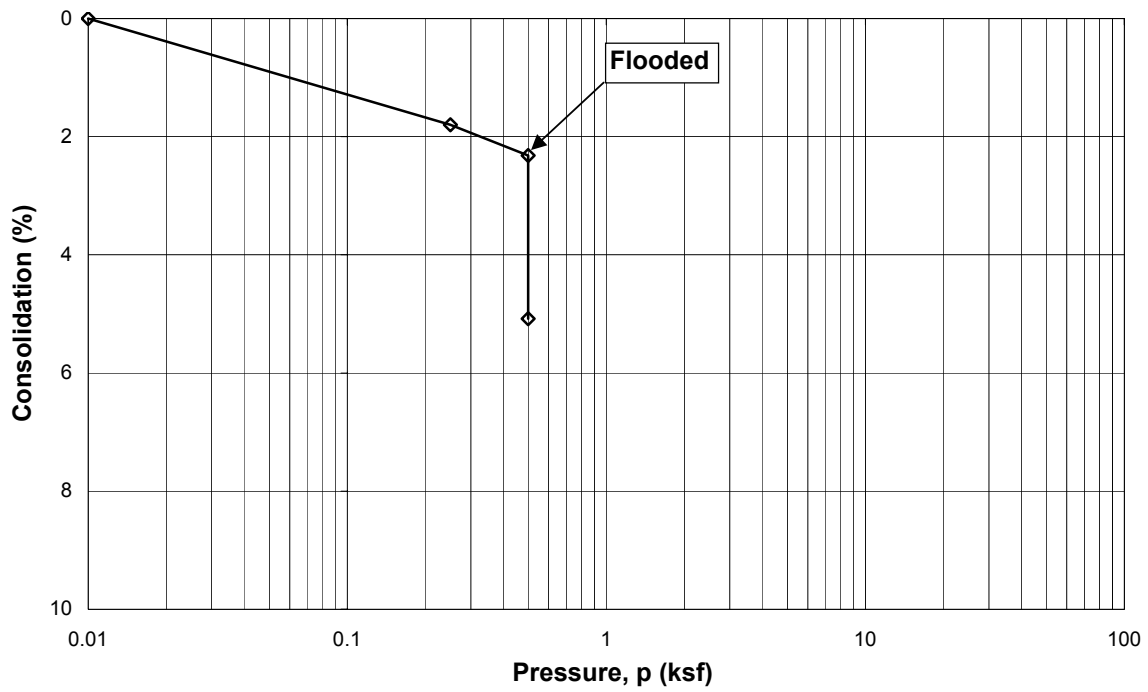
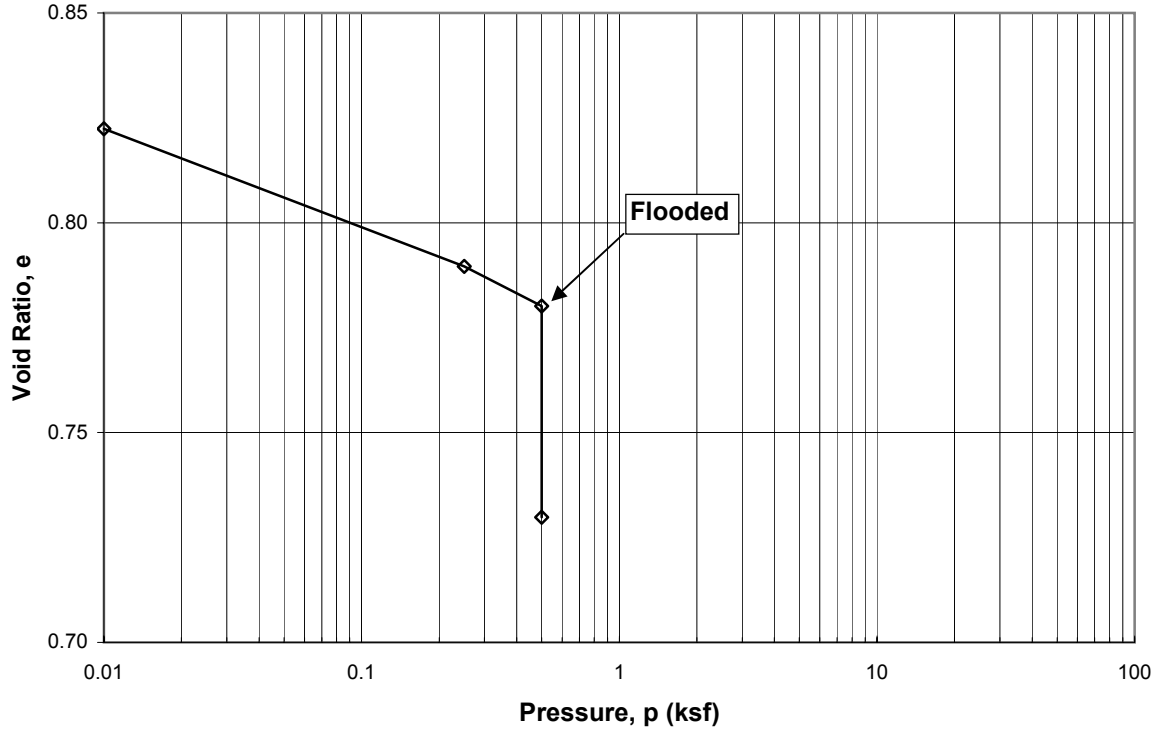
pH	7.1
-----------	-----

(ppm)	
Sulfates	67
Chlorides	46

Minimum Resistivity	(ohm-cm)	Moisture Content (%)
	2015	31.8

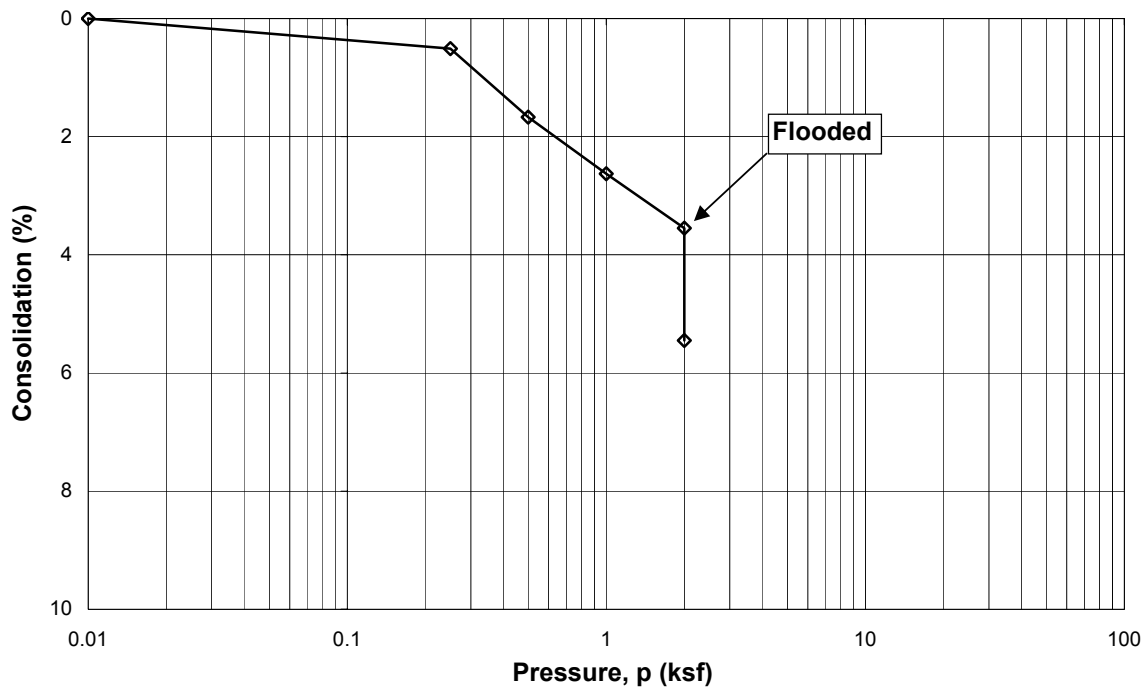
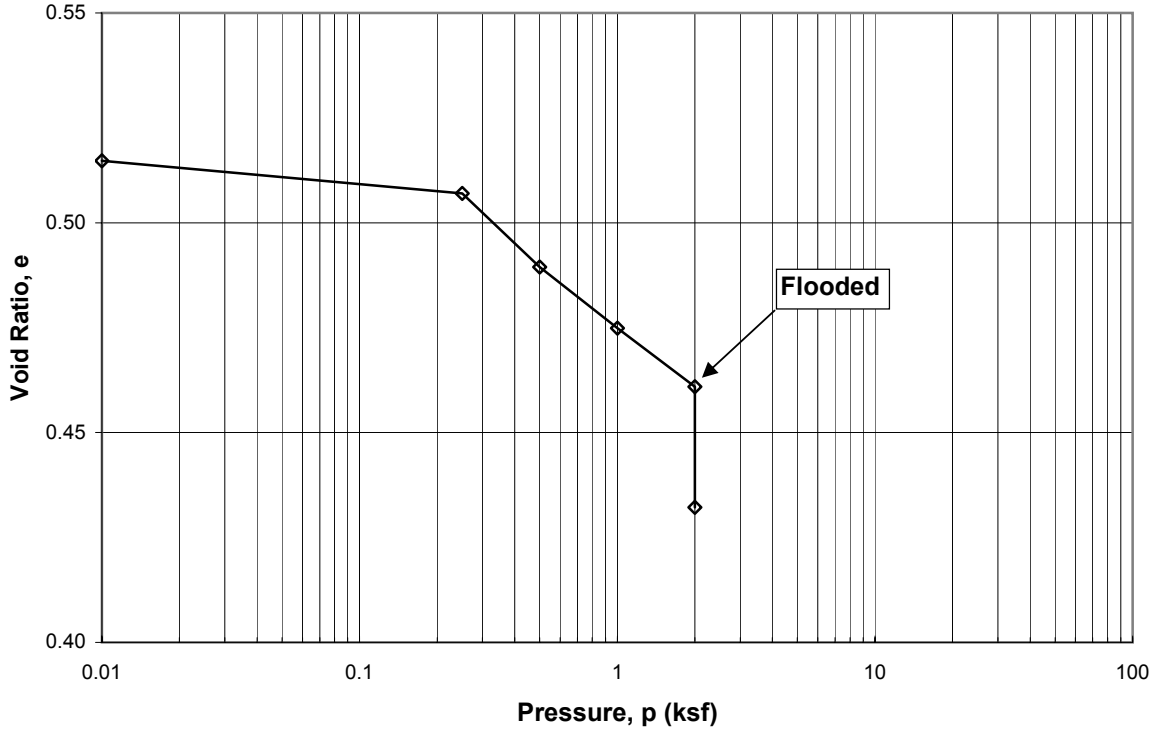
COLLAPSE POTENTIAL (ASTM D5333)

Project Name: Oak Valley **Project No.:** --
Boring No.: B9 **Sample No.:** # 1 **Depth:** 5'
Soil Description: Dark Yellowish Brown, Sandy Silt (ML)



COLLAPSE POTENTIAL (ASTM D5333)

Project Name: Oak Valley **Project No.:** --
Boring No.: B9 **Sample No.:** # 3 **Depth:** 15'
Soil Description: Reddish Yellow, Silty Sand (SM)



APPENDIX C

STANDARD GRADING SPECIFICATIONS

STANDARD GRADING SPECIFICATIONS

These specifications present the usual and minimum requirements for projects on which Petra Geotechnical, Inc. is the geotechnical consultant.

No deviation from these specifications will be allowed, except where specifically superseded in the preliminary geology and soils report, or in other written communication signed by the Soils Engineer and Engineering Geologist of record (geotechnical consultant).

I. GENERAL

- A. The Geotechnical Consultant is the Owner's or Builder's representative on the project. For the purpose of these specifications, participation by the Geotechnical Consultant includes that observation performed by any person or persons employed by, and responsible to, the *licensed Soils Engineer and Engineering Geologist* signing the soils report.
- B. The contractor should prepare and submit to the Owner and Geotechnical Consultant a work plan that indicates the sequence of earthwork grading, the number of "spreads", and the estimated quantities of daily earthwork to be performed prior to the commencement of grading. This work plan should be reviewed by the Geotechnical Consultant to schedule personnel to perform the appropriate level of observation, mapping, and compaction testing as necessary.
- C. All clearing, site preparation, or earthwork performed on the project shall be conducted by the Contractor in accordance with the recommendations presented in the geotechnical report and under the observation of the Geotechnical Consultant.
- D. It is the Contractor's responsibility to prepare the ground surface to receive the fills to the satisfaction of the Geotechnical Consultant and to place, spread, mix, water, and compact the fill in accordance with the specifications of the Geotechnical Consultant. The Contractor shall also remove all material considered unsatisfactory by the Geotechnical Consultant.

STANDARD GRADING SPECIFICATIONS

- E. It is the Contractor's responsibility to have suitable and sufficient compaction equipment on the job site to handle the amount of fill being placed. If necessary, excavation equipment will be shut down to permit completion of compaction to project specifications. Sufficient *watering apparatus will also be provided by the Contractor, with due consideration for the fill material, rate of placement, and time of year.*
- F. After completion of grading a report will be submitted by the Geotechnical Consultant.

II. SITE PREPARATION

A. Clearing and Grubbing

- 1. All vegetation such as trees, brush, grass, roots, and deleterious material shall be disposed of offsite. This removal shall be concluded prior to placing fill.
- 2. Any underground structures such as cesspools, cisterns, mining shafts, tunnels, septic tanks, wells, pipe lines, etc., are to be removed or treated in a manner prescribed by the Geotechnical Consultant.

III. FILL AREA PREPARATION

A. Remedial Removals/Overexcavations

- 1. Remedial removals, as well as overexcavation for remedial purposes, shall be evaluated by the Geotechnical Consultant. Remedial removal depths presented in the geotechnical report and shown on the geotechnical plans are estimates only. The actual extent of removal should be determined by the Geotechnical Consultant based on the conditions exposed during grading. All soft, loose, dry, saturated, spongy, organic-rich, highly fractured or otherwise unsuitable ground shall be overexcavated to competent ground as determined by the Geotechnical Consultant.

STANDARD GRADING SPECIFICATIONS

2. Soil, alluvium, or bedrock materials determined by the Soils Engineer as being unsuitable for placement in compacted fills shall be removed from the site. Any material incorporated as a part of a compacted fill must be approved by the Geotechnical Consultant.
3. Should potentially hazardous materials be encountered, the Contractor should stop work in the affected area. An environmental consultant specializing in hazardous materials should be notified immediately for evaluation and handling of these materials prior to continuing work in the affected area.

B. Evaluation/Acceptance of Fill Areas

All areas to receive fill, including removal and processed areas, key bottoms, and benches, shall be observed, mapped, elevations recorded, and/or tested prior to being accepted by the Geotechnical Consultant as suitable to receive fill. The contractor shall obtain a written acceptance from the Geotechnical Consultant prior to fill placement. A licensed surveyor shall provide sufficient survey control for determining locations and elevations of processed areas, keys, and benches.

C. Processing

After the ground surface to receive fill has been declared satisfactory for support of fill by the Geotechnical Consultant, it shall be scarified to a minimum depth of 6 inches and until the ground surface is uniform and free from ruts, hollows, hummocks, or other uneven features which may prevent uniform compaction.

The scarified ground surface shall then be brought to optimum moisture, mixed as required, and compacted to a minimum relative compaction of 90 percent.

STANDARD GRADING SPECIFICATIONS

D. Subdrains

Subdrainage devices shall be constructed in compliance with the ordinances of the controlling governmental agency, and/or with the recommendations of the Geotechnical Consultant. (Typical Canyon Subdrain details are given on Plate SG-1).

E. Cut/Fill & Deep Fill/Shallow Fill Transitions

In order to provide uniform bearing conditions in cut/fill and deep fill/shallow fill transition lots, the cut and shallow fill portions of the lot should be overexcavated to the depths and the horizontal limits discussed in the approved geotechnical report and replaced with compacted fill. (Typical details are given on Plate SG-7.)

III. COMPACTED FILL MATERIAL

A. General

Materials excavated on the property may be utilized in the fill, provided each material has been determined to be suitable by the Geotechnical Consultant. Material to be used for fill shall be essentially free of organic material and other deleterious substances. Roots, tree branches, and other matter missed during clearing shall be removed from the fill as recommended by the Geotechnical Consultant. Material that is spongy, subject to decay, or otherwise considered unsuitable shall not be used in the compacted fill.

Soils of poor quality, such as those with unacceptable gradation, high expansion potential, or low strength shall be placed in areas acceptable to the Geotechnical Consultant or mixed with other soils to achieve satisfactory fill material.

STANDARD GRADING SPECIFICATIONS

B. Oversize Materials

Oversize material defined as rock, or other irreducible material with a maximum dimension greater than 12 inches in diameter, shall be taken offsite or placed in accordance with the recommendations of the Geotechnical Consultant in areas designated as suitable for rock disposal (Typical details for Rock Disposal are given on Plate SG-4).

Rock fragments less than 12 inches in diameter may be utilized in the fill provided, they are not nested or placed in concentrated pockets; they are surrounded by compacted fine grained soil material and the distribution of rocks is approved by the Geotechnical Consultant.

C. Laboratory Testing

Representative samples of materials to be utilized as compacted fill shall be analyzed by the laboratory of the Geotechnical Consultant to determine their physical properties. If any material other than that previously tested is encountered during grading, the appropriate analysis of this material shall be conducted by the Geotechnical Consultant as soon as possible.

D. Import

If importing of fill material is required for grading, proposed import material should meet the requirements of the previous section. The import source shall be given to the Geotechnical Consultant at least 2 working days prior to importing so that appropriate tests can be performed and it's suitability be determined.

STANDARD GRADING SPECIFICATIONS

IV. FILL PLACEMENT AND COMPACTION

A. Fill Layers

Material used in the compacting process shall be evenly spread, watered, processed, and compacted in thin lifts not to exceed 6 inches in thickness to obtain a uniformly dense layer. The fill shall be placed and compacted on a horizontal plane, unless otherwise approved by the Geotechnical Consultant.

B. Moisture Conditioning

Fill soils shall be watered, dried back, blended, and/or mixed, as necessary to attain a relatively uniform moisture content at or slightly above optimum moisture content.

C. Compaction

Each layer shall be compacted to 90 percent of the maximum density in compliance with the testing method specified by the controlling governmental agency. (In general, ASTM D 1557-00, will be used.)

If compaction to a lesser percentage is authorized by the controlling governmental agency because of a specific land use or expansive soils condition, the area to received fill compacted to less than 90 percent shall either be delineated on the grading plan or appropriate reference made to the area in the soils report.

D. Failing Areas

If the moisture content or relative density varies from that required by the Geotechnical Consultant, the Contractor shall rework the fill until it is approved by the Geotechnical Consultant.

E. Benching

All fills shall be keyed and benched through all topsoil, colluvium, alluvium or creep material, into sound bedrock or firm material

STANDARD GRADING SPECIFICATIONS

where the slope receiving fill exceeds a ratio of 5 horizontal to 1 vertical, in accordance with the recommendations of the Geotechnical Consultant.

V. SLOPES

A. Fill Slopes

The contractor will be required to obtain a minimum relative compaction of 90 percent out to the finish slope face of fill slopes, buttresses, and stabilization fills. This may be achieved by either overbuilding the slope and cutting back to the compacted core, or by direct compaction of the slope face with suitable equipment, or by any other procedure which produces the required compaction.

B. Side Hill Fills

The key for side hill fills shall be a minimum of 15 feet within bedrock or firm materials, unless otherwise specified in the soils report. (See detail on Plate SG-5.)

C. Fill-Over-Cut Slopes

Fill-over-cut slopes shall be properly keyed through topsoil, colluvium or creep material into rock or firm materials, and the transition shall be stripped of all soils prior to placing fill. (see detail on Plate SG-6).

D. Landscaping

All fill slopes should be planted or protected from erosion by other methods specified in the soils report.

E. Cut Slopes

1. The Geotechnical Consultant should observe all cut slopes at vertical intervals not exceeding 10 feet.
2. If any conditions not anticipated in the preliminary report such as *perched water, seepage, lenticular or confined strata of a*

STANDARD GRADING SPECIFICATIONS

potentially adverse nature, unfavorably inclined bedding, joints or fault planes are encountered during grading, these conditions shall be evaluated by the Geotechnical Consultant, and recommendations shall be made to treat these problems (Typical details for stabilization of a portion of a cut slope are given in Plates SG-2 and SG-3.).

3. Cut slopes that face in the same direction as the prevailing drainage shall be protected from slope wash by a non-erodible interceptor swale placed at the top of the slope.
4. Unless otherwise specified in the soils and geological report, no cut slopes shall be excavated higher or steeper than that allowed by the ordinances of controlling governmental agencies.
5. Drainage terraces shall be constructed in compliance with the ordinances of controlling governmental agencies, or with the recommendations of the Geotechnical Consultant.

VI. GRADING OBSERVATION

A. General

All cleanouts, processed ground to receive fill, key excavations, subdrains, and rock disposals must be observed and approved by the Geotechnical Consultant prior to placing any fill. It shall be the Contractor's responsibility to notify the Geotechnical Consultant when such areas are ready.

B. Compaction Testing

Observation of the fill placement shall be provided by the Geotechnical Consultant during the progress of grading. Location and frequency of tests shall be at the Consultants discretion based on field conditions encountered. Compaction test locations will not necessarily be selected on a random basis. Test locations may be selected to verify adequacy of compaction levels in areas that are judged to be susceptible to inadequate compaction.

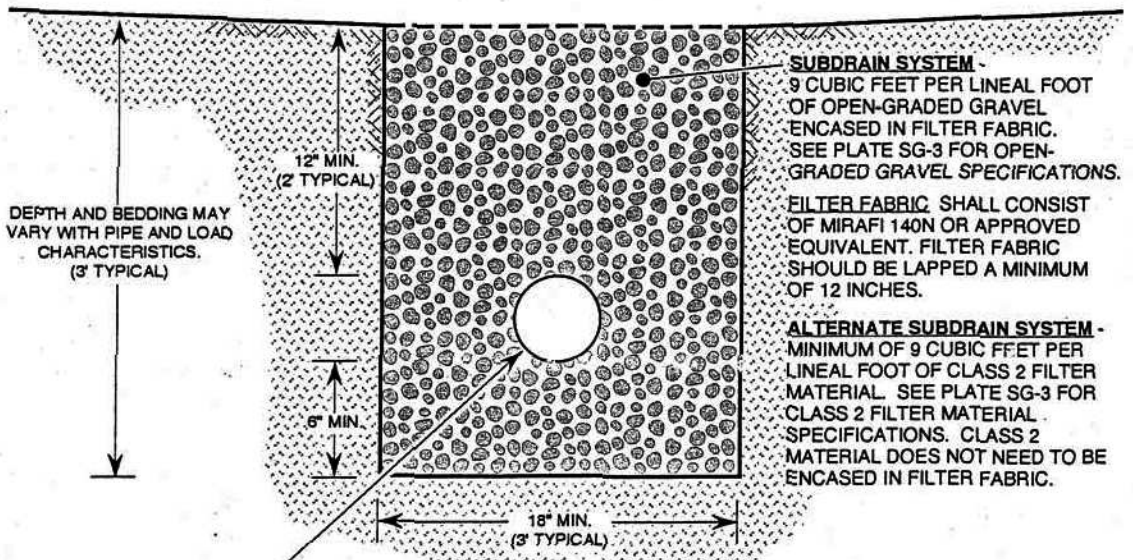
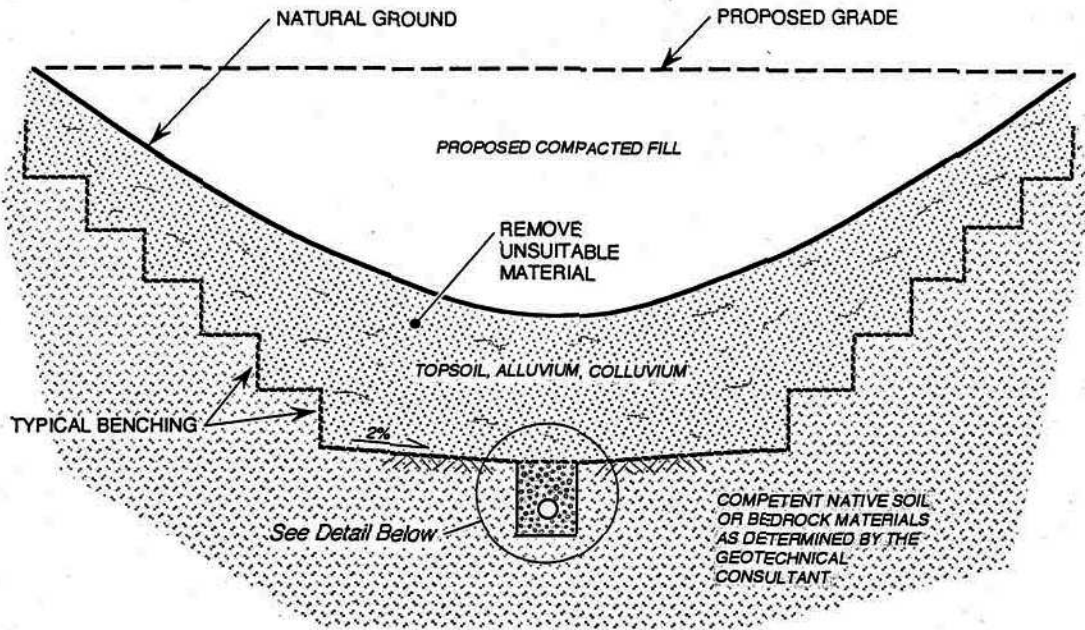
STANDARD GRADING SPECIFICATIONS

C. Frequency of Compaction Testing

In general, density tests should be made at intervals not exceeding 2 feet of fill height or every 1000 cubic yards of fill placed. This criteria will vary depending on soil conditions and the size of the job. In any event, an adequate number of field density tests shall be made to verify that the required compaction is being achieved.

VII. CONSTRUCTION CONSIDERATIONS

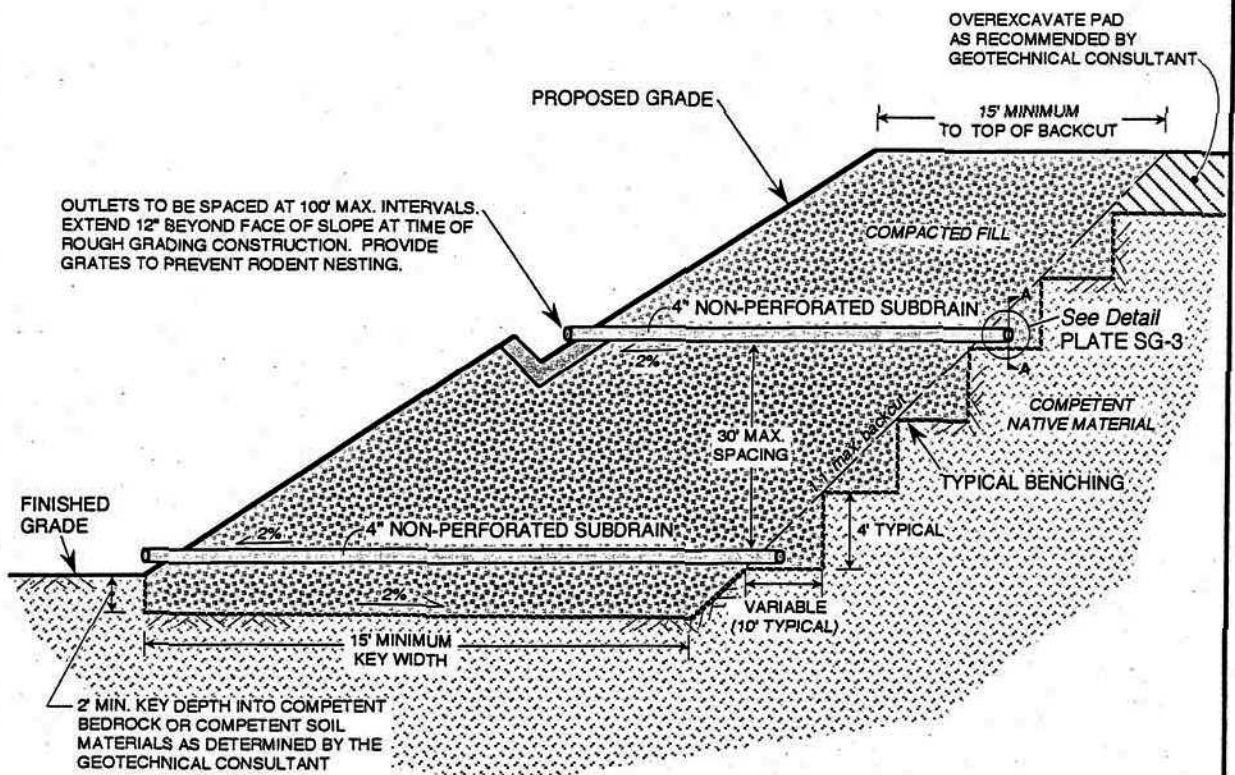
- A. Erosion control measures, when necessary, shall be provided by the Contractor during grading and prior to the completion and construction of permanent drainage controls.
- B. Upon completion of grading and termination of observations by the Geotechnical Consultant, no further filling or excavating, including that necessary for footings, foundations, large tree wells, retaining walls, or other features shall be performed without the approval of the Geotechnical Consultant.
- C. Care shall be taken by the Contractor during final grading to preserve any berms, drainage terraces, interceptor swales, or other devices of permanent nature on or adjacent to the property.



MINIMUM 6-INCH DIAMETER PVC SCHEDULE 40, OR ABS SDR-35 WITH A MINIMUM OF EIGHT 1/4-INCH DIAMETER PERFORATIONS PER LINEAL FOOT IN BOTTOM HALF OF PIPE. PIPE TO BE LAID WITH PERFORATIONS FACING DOWN.

NOTES:

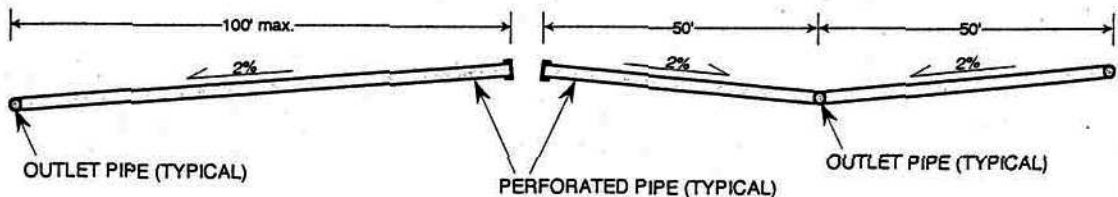
1. FOR CONTINUOUS RUNS IN EXCESS OF 500 FEET USE 8-INCH DIAMETER PIPE.
2. FINAL 20 FEET OF PIPE AT OUTLET SHALL BE NON-PERFORATED AND BACKFILLED WITH FINE-GRAINED MATERIAL.

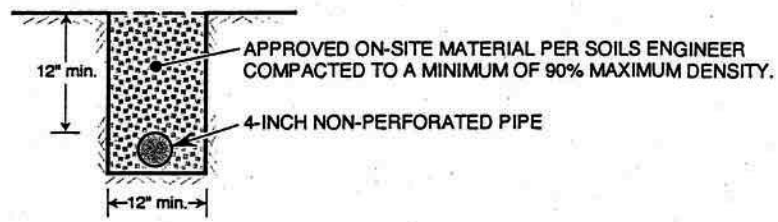
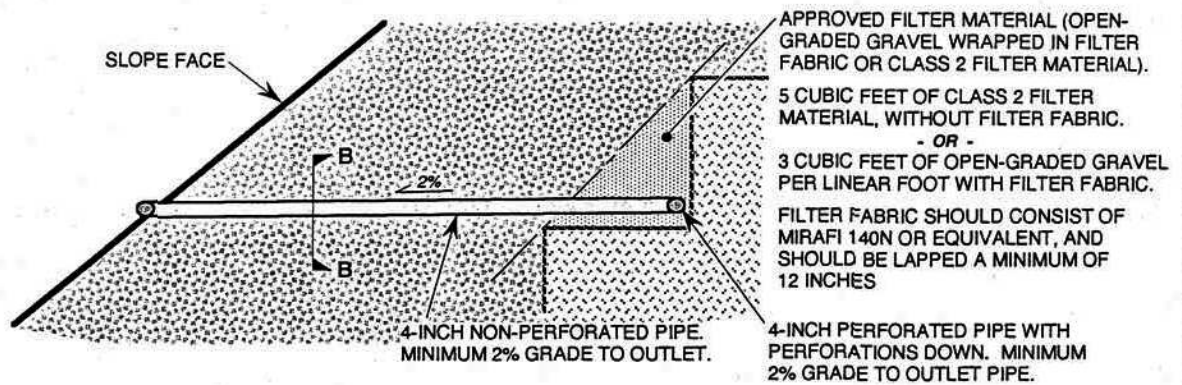


NOTES:

1. 30' MAXIMUM VERTICAL SPACING BETWEEN SUBDRAIN SYSTEMS.
2. 100' MAXIMUM HORIZONTAL DISTANCE BETWEEN NON-PERFORATED OUTLET PIPES. (See Below)
3. MINIMUM GRADIENT OF 2% FOR ALL PERFORATED AND NON-PERFORATED PIPE.

SECTION A-A (PERFORATED PIPE PROFILE)





SECTION B-B (OUTLET PIPE)

PIPE SPECIFICATIONS:

1. 4-INCH MINIMUM DIAMETER, PVC SCHEDULE 40 OR ABS SDR-35.
2. FOR PERFORATED PIPE, MINIMUM 8 PERFORATIONS PER FOOT ON BOTTOM HALF OF PIPE.

FILTER MATERIAL/FABRIC SPECIFICATIONS:

OPEN-GRADED GRAVEL ENCASED IN FILTER FABRIC.
(MIRAFAI 140N OR EQUIVALENT)

ALTERNATE:

CLASS 2 PERMEABLE FILTER MATERIAL PER CALTRANS
STANDARD SPECIFICATION 68-1.025.

OPEN-GRADED GRAVEL

SIEVE SIZE	PERCENT PASSING
1 1/2-INCH	88 - 100
1-INCH	5 - 40
3/4-INCH	0 - 17
3/8-INCH	0 - 7
No. 200	0 - 3

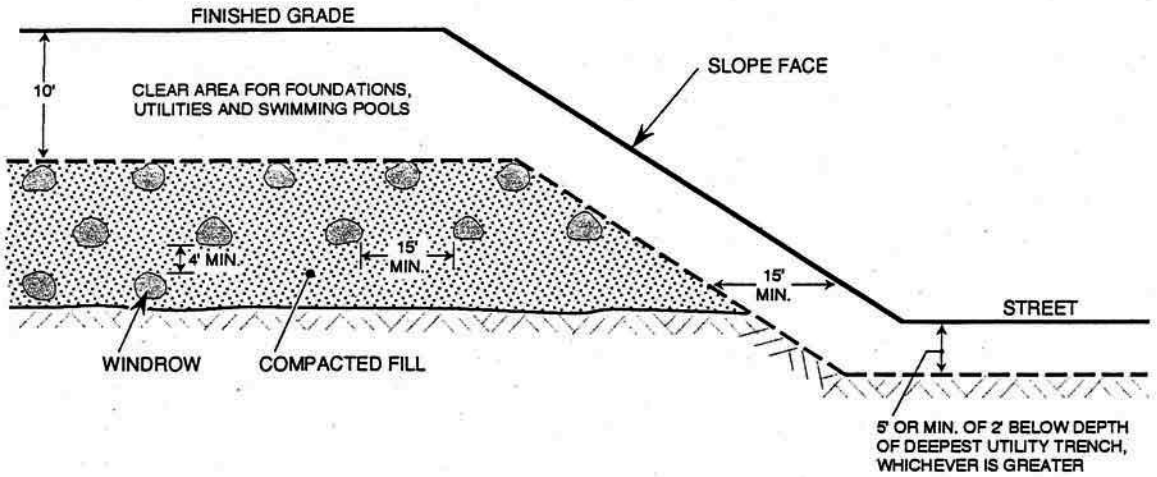
CLASS 2 FILTER MATERIAL

SIEVE SIZE	PERCENT PASSING
1-INCH	100
3/4-INCH	90 - 100
3/8-INCH	40 - 100
No. 4	25 - 40
No. 8	18 - 33
No. -30	5 - 15
No. -50	0 - 7
No. 200	0 - 3

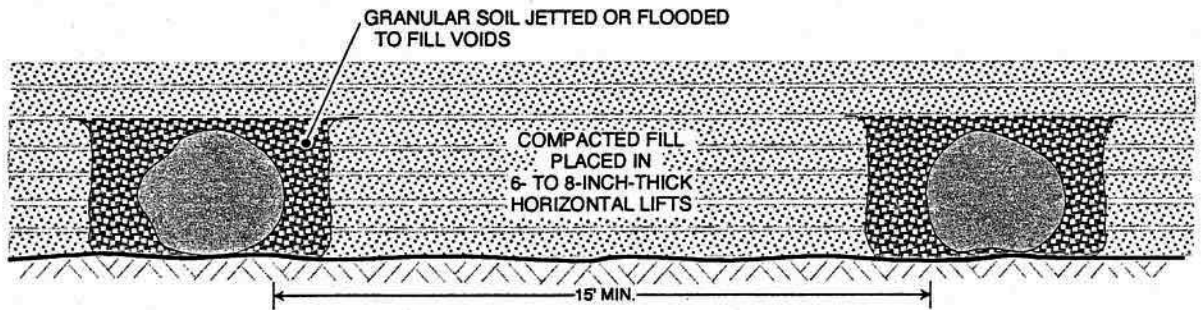


**BUTTRESS OR STABILIZATION
FILL SUBDRAIN**

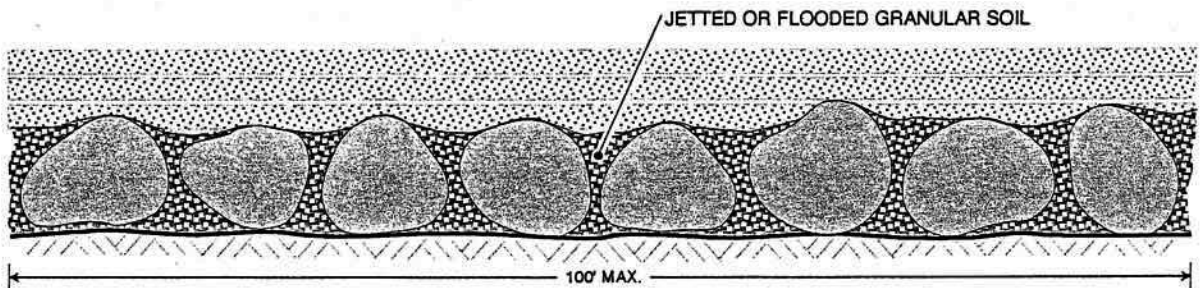
PLATE SG-3



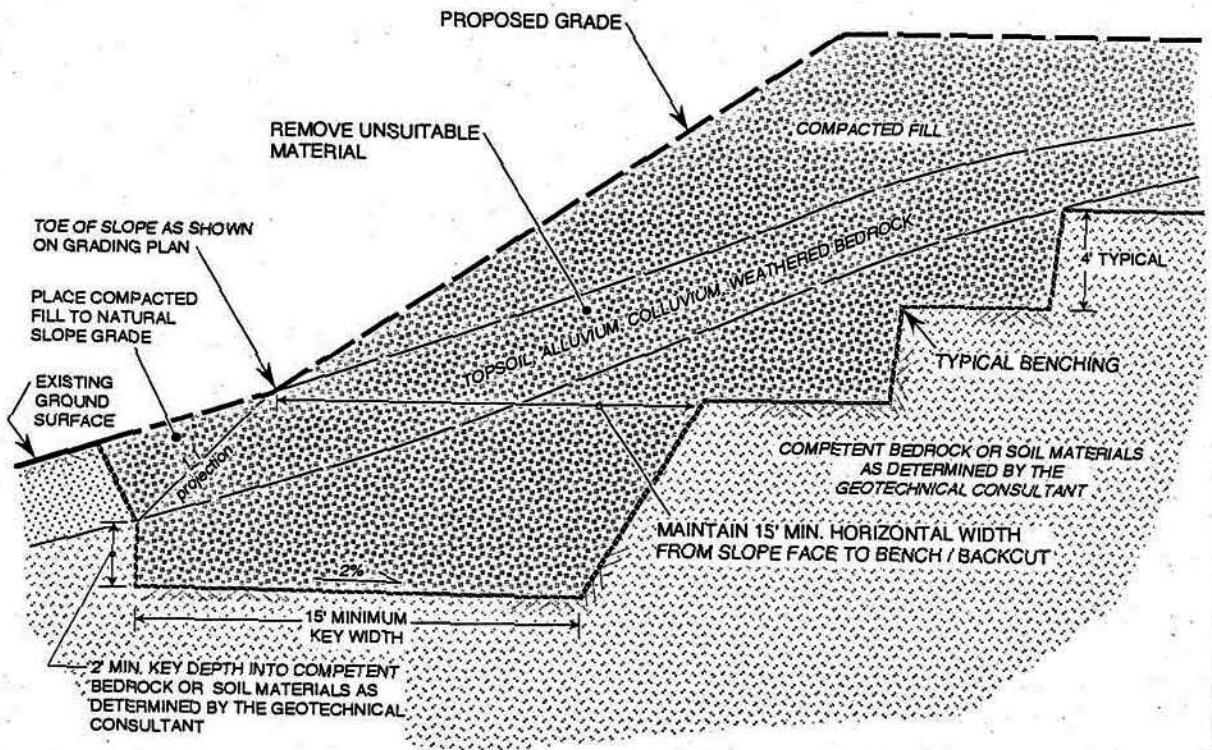
TYPICAL WINDROW DETAIL (END VIEW)



TYPICAL WINDROW DETAIL (PROFILE VIEW)

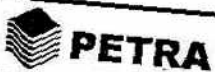
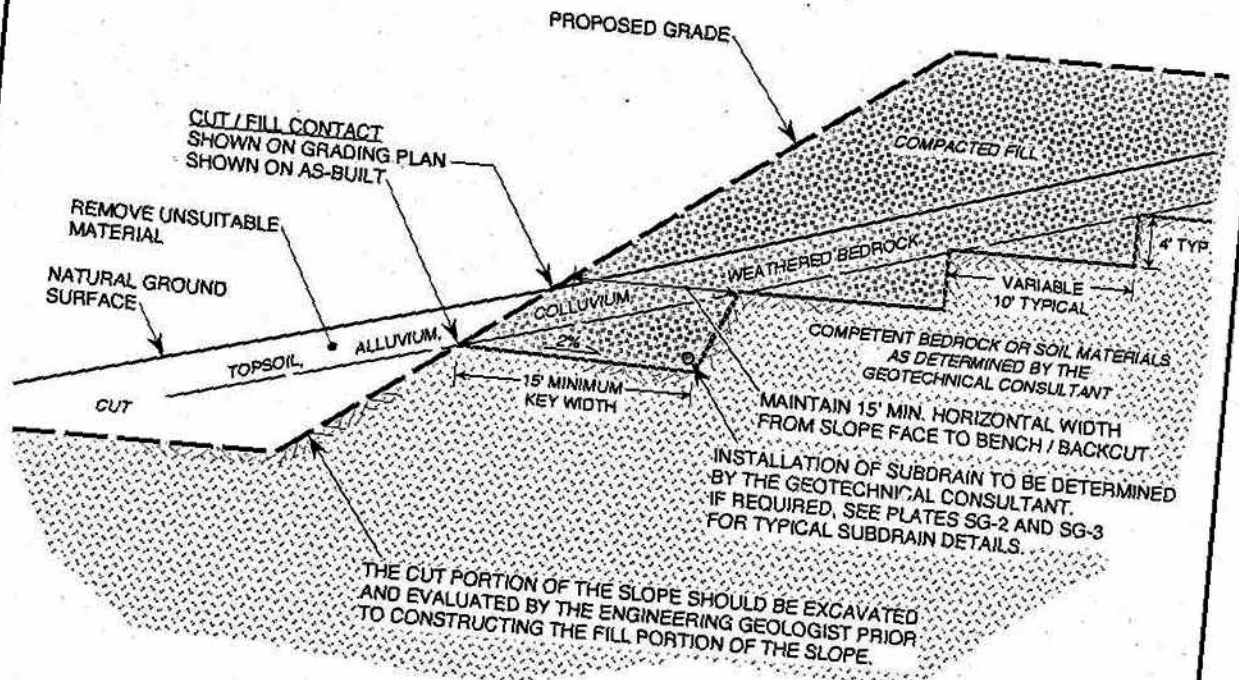


NOTE: OVERSIZE ROCK IS DEFINED AS CLASTS HAVING A MAXIMUM DIMENSION OF 12" OR LARGER



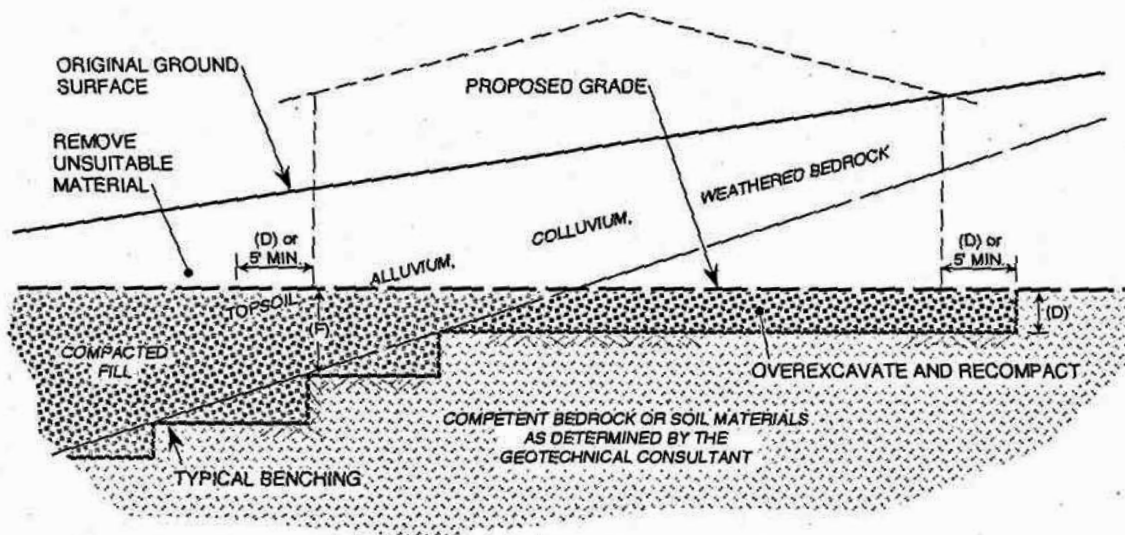
NOTES:

1. WHERE NATURAL SLOPE GRADIENT IS 5:1 OR LESS, BENCHING IS NOT NECESSARY; HOWEVER, FILL IS NOT TO BE PLACED ON COMPRESSIBLE OR UNSUITABLE MATERIAL.
2. SOILS ENGINEER TO DETERMINE IF SUBDRAIN IS REQUIRED.

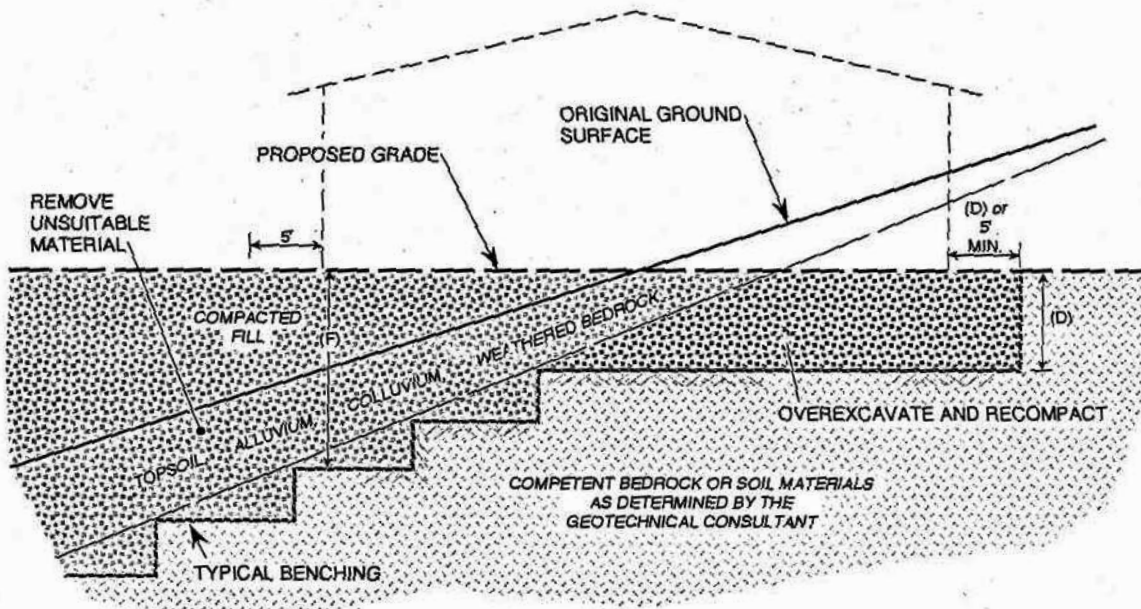


CUT LOT

UNSUITABLE MATERIAL EXPOSED IN PORTION OF CUT PAD



CUT-FILL TRANSITION LOT



MAXIMUM FILL THICKNESS (F)

FOOTING DEPTH TO 3 FEET

3 TO 6 FEET

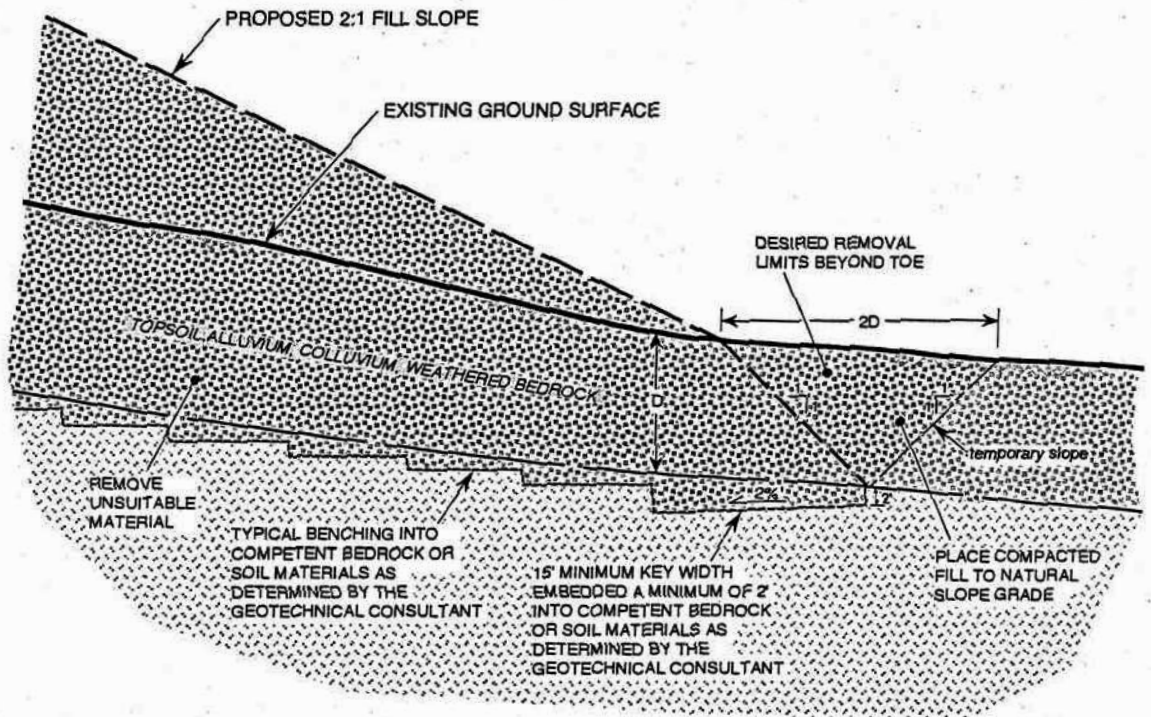
GREATER THAN 6 FEET

DEPTH OF OVEREXCAVATION (D)

EQUAL DEPTH

3 FEET

1/2 THE THICKNESS OF DEEPEST FILL PLACED WITHIN THE "FILL" PORTION (F) TO 15 FEET MAXIMUM



D = RECOMMENDED DEPTH OF REMOVAL
PER GEOTECHNICAL REPORT