

ENGINEERING GEOLOGY
AND
GEOTECHNICAL ENGINEERING REPORT
FOR
744 VEREDA DEL CIERVO
PROPOSED RESIDENCE
SANTA BARBARA COUNTY, CALIFORNIA

Consulting Engineers and Geologists

Project: 744 Vereda Del Ciervo
Proposed Residence
Santa Barbara County, Calif

As authorized, we have performed an en
a proposed residence to be located at 74
The accompanying Engineering Geolo
results of our research, as well as
geotechnical aspects of project design.

We have appreciated the opportunity to
have any questions, or if we can be of fu

Respectfully submitted,

EARTH SYSTEMS SOUTHERN CA

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Todd J. Tranby
Engineering Geologist

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Regional Geology Map (USGS)

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Geologic Cross Section

Boring Log

Symbols Commonly Used on Boring Logs

Unified Soil Classification

APPENDIX B

Laboratory Testing

Tabulated Test Results

Composite Shear Diagrams

Individual Test Results

Soil Chemistry Results

Table 1805.4.2 with Footnotes

2007 California Building Code (CBC) & ASCE 7-05 Seismic Parameters

APPENDIX C

Slope Stability Analyses

performed for a proposed residence at 744 Vereda Del Ciervo in Santa Barbara County, California. It is assumed herein that the proposed residence will be one- to two-story, of wood-framed and/or of masonry/concrete construction with raised and/or slab-on-grade floors. Structural considerations for building column loads of up to 25 kips with maximum wall loads of 2.0 kips per lineal foot were used as a basis for the recommendations of this report. If actual loads vary significantly from these assumed loads, Earth Systems Southern California should be notified since reevaluation of the recommendations contained in this report may be required.

The exact location and shape of the proposed residence has not been determined by the Client. When this information is available it should be provided to this office for review and possible additional recommendations.

Purpose and Scope of Work

The purpose of the geotechnical study that led to this report was to analyze the geology and soil conditions of the site with respect to the proposed construction. These conditions include potential geohazards, surface and subsurface soil/bedrock types, slope stability, expansion potential, settlement potential, bearing capacity, and the presence or absence of subsurface water. The scope of our work included:

1. Reconnaissance of the site.
2. Reviewing pertinent geologic literature.
3. Review of the following aerial photographs taken of the site at stored at the Map and Imagery Laboratory at the UCSB Campus.

<u>Date</u>	<u>Flight and Frame Numbers</u>	<u>Scale</u>
1929	C-430, B-20 & 21	1:24,000
1938	C4950, SE 166 & 168	1:24,000
1943	BTM-1943, SB-55 & -56	1:20,000
1947	GS-EM, 4-97 & -98	1:24,000
1954	BTM-1954, 6K-28 & -29	1:20,000
1959	HA-GN, 10 & 11	1:12,000

1989	PW-SB-7, 50 & 51	1:24,000
1997	PW-SB-10, 60	1:24,000
2003	PW-SB-14, 94 & 94B	1:12,000

4. Drilling, downhole logging and sampling of one 24-inch diameter bucket auger boring to study bedrock, soil and groundwater conditions.
5. Laboratory testing of bedrock/soil samples obtained from the subsurface exploration to determine their physical and engineering properties.
6. Consulting with owner representatives.
7. Analyzing the geotechnical data.
8. Preparing this report.

Contained in this report are:

1. Descriptions and results of field and laboratory tests that were performed.
2. Discussions pertaining to the local bedrock, soil and groundwater conditions.
3. Conclusions and recommendations pertaining to site grading and structural design.

Site Setting

The site of the proposed improvements is located at 744 Vereda Del Ciervo in Santa Barbara County, California (see Vicinity Map and Geologic Map / Site Plan in Appendix A). The site is vacant of structures. The subject property is an approximately rectangular-shaped polygon with sides measuring about 350 to 450 feet in the east-west direction and about 175 and 225 feet in the north-south direction. The new residence will be constructed in the southwestern corner of the site. The proposed building site lies at the toe of a west-facing, 2:1 (horizontal to vertical) gradient, ascending natural slope. This slope is part of an east-west trending ridge spur that connects with the north-south trending ridgeline located east of the subject lot. The slope gradient is relatively flat at the proposed building site. Vereda Del Ciervo lies along the west side of the subject lot. The site is covered with natural grasses and brush with some trees near the proposed building site. No springs or seeps were noted on the ground surface during our field study.

produces the east-west trending faults which deforms early Pleistocene to Tertiary aged marine and non-marine sedimentary bedrock units. These sedimentary bedrock units underlie the property (see Regional Geologic Maps by Dibblee and USGS in Appendix A). The site does not lie within any special study zones for fault rupture hazard or landslides. No faults or landslides were encountered in the general area of the proposed building site during field studies, although several landslides are located east of the proposed building pad (see attached Geologic Map / Site Plan).

STRUCTURE

The subject site is underlain by about 21 feet of colluvial/alluvial deposits over Miocene-aged Vaqueros Formation bedrock. This bedrock classification is different than that mapped by Dibblee and the USGS (see attached Regional Geology maps) who both have the subject site underlain by Sespe Formation bedrock. Bedrock units encountered within the bucket auger boring had strikes of bedding ranging from about N38°W to N61°W and dips ranging from 60° to 83° to the southwest. These strikes and dips appear to be generally consistent with the regional strikes of other bedrock units in the general area of the subject site according to Dibblee and USGS.

GEOLOGIC HAZARDS

Geologic hazards that may impact a site include seismic shaking, fault rupture, landsliding, liquefaction and flooding.

Seismic Shaking

The site is located in Southern California which is within an active seismic area where large numbers of earthquakes are recorded each year. Historically, major earthquakes felt in the vicinity of the subject site have originated from faults outside the area. These include the 1812 Santa Barbara Channel Earthquake, 1857 Fort Tejon earthquake, the 1872 Owens Valley earthquake, and the 1952 Arvin-Tehachapi earthquake.

accelerations. The California Division of Mines and Geology, in concert with the U.S. Geological Survey and the scientific community, has presented results of a statewide probabilistic seismic hazard assessment (CDMG, Seismic Shaking Map Sheets, Map Sheet 48, 1999). The focus of the assessment was to generate a seismic hazard map showing zones of estimated peak ground accelerations at a hazard level of 10% probability of exceedance in 50 years. The site plots between 0.40 g to 0.50 g acceleration potential. The subject site is located within a zone indicated as having predicted earthquake magnitudes of 6.5 to 7.0. The proposed construction should be designed for the 2007 California Building Code (CBC) requirements; however, the project Structural Engineer should be made aware of the design base acceleration.

The 2007 CBC requires design by different seismic parameters than required by the 2001 CBC. The site is located at about geographic coordinates 34.45079N and 119.91609W. These values were input into the USGS's web based Seismic Hazard Curves and Uniform Response Spectra calculator to determine the site's short term (0.2 sec.) and long term (1.0 sec.) spectral accelerations. The results of the analysis are presented on a data sheet included in Appendix B.

Fault Rupture

The parcel does not lie within a State of California designated fault hazard zone. Dibblee and USGS do not map any faults crossing the subject site. No faults were observed during our field study or in the aerial photograph review. Therefore, it appears the potential for fault rupture hazard on the subject site is considered low.

Liquefaction

A major cause of damage during earthquakes is a significant reduction of soil strength or stiffness, generally referred to as liquefaction. Liquefaction can cause translational instability, bearing failure, settlement, ground loss, and other related phenomena. Translational instabilities can be slope failures or lateral spreading. Bearing failure can occur when soil strength loss is near a foundation. Settlement can occur when bearing failure is precluded, but volumetric compression occurs. Ground loss results from sand boils and is usually very localized. Liquefaction is typically a design problem only if it occurs in the upper 50 feet of the subsurface

Cohesive soils with fines content greater than 30% are generally not susceptible to liquefaction if their fines classify as clays, or they have a plasticity index greater than 30%. Generally, if a soil has a clay content greater than 20%, or the water content is less than 0.9 times the liquid limit, liquefaction can be ruled out. However, cohesive soils, if sensitive, can lose significant strength even if they cannot liquefy, and there may be a need to address this problem. Although widely believed to be non-liquefiable, gravelly soils can be susceptible to liquefaction if internal drainage is impeded. In order for liquefaction to occur, a potentially liquefiable soil must be saturated and subjected to rapid cyclic loading that is sufficiently intense to overcome a soil's internal resistance to liquefaction.

The sandstone bedrock units below 21 feet should not be acceptable to liquefaction because of their dense, cemented nature. The colluvial/alluvial soils from the ground surface down to a depth of 21 feet have a low potential for liquefaction because no groundwater was encountered in these soils during the field exploration (performed during a rainy season with about average rainfall).

Landsliding

No existing landslides were observed on or trending onto the proposed construction site during the site reconnaissance and aerial photograph review, although landslides have occurred on both the south and north sides of the ridge spur in which the subject site is located. This landsliding did not show evidence of re-activation on the aerial photographs reviewed from 1929 to 2003. Therefore, these landslides are considered to be older slide events. Any additional movement in these existing landslides should be either to the south or north, or away from the proposed building site.

Rockfall

No potential rockfall hazard was observed during our field study.

Flooding

Earthquake-induced flooding types include tsunamis, seiches, and reservoir failure. Due to the inland location of the site, hazards from tsunamis and seiches are considered extremely unlikely.

ENGINEERING GEOLOGY CONCLUSIONS AND RECOMMENDATIONS

Based on the data provided in this report, it is our opinion that the site is suitable for the proposed development from an Engineering Geology standpoint provided that the recommendations given in this report are properly implemented into the design and construction phases of the project.

SOIL/BEDROCK CONDITIONS

Near-surface soils underlying the proposed building areas generally consist of about 21 feet of colluvium/alluvium over Vaqueros Formation bedrock. The colluvium/alluvium consists of soft to stiff clayey silt with some gravel and cobbles to medium dense silty sand with some gravel to cobbles. Vaqueros Formation bedrock was encountered below the colluvium/alluvium and consisted of medium dense to very dense sandstone. Testing indicates that anticipated bearing soils lie in the "medium" expansion range of the attached Table 1805.4.2.

Samples of near-surface soils were tested for pH, resistivity, soluble sulfates and soluble chlorides. Testing indicates that anticipated bearing soils lie within the "negligible" sulfate exposure range in Table 4.3.1 in ACI 318, Section 4.3. Hence, special concrete designs do not appear necessary to combat sulfate attack. A soil resistivity measurement indicates that the soil is "moderately corrosive" to ferrous metals. The test results provided in Appendix B should be provided to the project designers for their interpretations pertaining to the corrosivity or reactivity of various construction materials (such as concrete and piping) with the soils.

ANALYSES OF SLOPE STABILITY

Gross Stability Analyses

One cross-section (Section A-A') was constructed through the natural slope areas of the site (see Geologic Map/Site Plan and Geologic Cross Section). The subsurface geometry illustrated in the cross-section was based on the County of Santa Barbara Flood Control topographic map, aerial

bedrock units were based on linear regressions of the "peak" and "ultimate" shear test results from relatively undisturbed samples obtained from BA-1 (see attached Composite Shear Diagrams in Appendix B). Density data were estimated from results of moisture/density testing.

Based on relatively steep regional bedding orientations (approximately 51 to 56 degrees to the southwest within the bedrock units, a rotational rather than a block failure mechanism was evaluated. The slopes at the subject site were analyzed using the SLOPE/W program for circular type failures. Analyses were performed for both static and pseudostatic ("seismic") stability for the three cross sections. Pseudostatic analyses utilized a horizontal earthquake factor of 0.15. In each circular analysis, 2,000 trial failure surfaces were created from radius points above the ground surface to search for the minimum factors of safety. The minimum factor-of safety point shown on the attached graphics are generally located and do not represent the actual center of the circle. All other radius points were not shown because many points would plot off the sheet. Circular failures were analyzed using the Morgenstern-Price Method.

For pseudostatic conditions, the minimum factor of safety of the slope depicted in Cross Section A-A was found to be 1.535 for a rotational type failure. For static conditions, the minimum factor of safety of the slope depicted in Cross Section A-A was found to be 1.324 for a rotational type failures.

Acceptable minimum factors of safety are typically considered to be 1.5 for static conditions, and 1.1 for pseudostatic conditions. All of the calculated factors of safety exceed the minimum values. (Plots of the slopes showing the failure surfaces and minimum factors of safety are in Appendix C. Based on these analyses, deep-seated "gross" failures along the natural slopes would not be anticipated.

provided that the recommendations contained in this report are successfully implemented into the project.

A. Grading

1. Pre-Grading Considerations

- a. Plans and specifications should be provided to Earth Systems Southern California prior to grading. Plans should include the grading plans, foundation plans, and foundation details.
- b. Roof draining systems should be designed so that water is not discharged into bearing soils or near the structures. Final site grade could be such that all water is diverted away from the structures toward either hardscapes or drain inlets, and is not allowed to pond. In landscape areas adjacent to the residence the 2007 California Building Code (Section 1803.3) requires a minimum gradient of 5% away from the edge of the residence foundation for a minimum distance of 10 feet.
- c. Improvements below ground level, such as sunken living rooms or wine cellars, should be waterproofed and drained in accordance with the project Architect's recommendations.
- d. It is recommended that Earth Systems Southern California be retained to provide Geotechnical Engineering services during site development and grading, and foundation construction phases of the work to observe compliance with the design concepts, specifications and recommendations, and to allow design changes in the event that subsurface conditions differ from those anticipated prior to the start of construction.

2. Rough Grading/Areas Of Development

- a. Grading at a minimum should conform to Appendix J of the 2007 California Building Code.
- b. The existing ground surface should be initially prepared for grading by removing all vegetation, trees, large roots, debris, other organic material and non-complying fill. Voids created by removal of such material should be properly

recommended that caissons bearing into the hard bedrock units be used to support the proposed residential structure. Hence, no overexcavation and recompaction of soils below the bottom of the foundation will be required.

- d. Slab-on-grade floors are not recommended. Floors should be structural slabs or raised floors rather than slabs-on-grade.
- e. Areas outside of the building area to receive fill, exterior slabs-on-grade, sidewalks or paving should be overexcavated to the deeper depth of either: 1) through any loose uncertified fill or topsoil, or 2) one foot below the finish subgrade. The resulting surface should then be scarified an additional 6- to 8-inches, moisture conditioned and recompacted.
- f. The bottom of all excavations should be observed by a representative of this firm prior to processing or placing fill.
- g. Voids created by dislodging cobbles and boulders during scarification should be backfilled and recompacted and dislodged cobbles larger than 8 inches in diameter should be removed from the subgrade.
- h. On-site soils may be used for fill once they are cleaned of all organic material, rock, debris and irreducible material larger than 8 inches.
- i. Fill and backfill placed at near optimum moisture in layers with loose thickness not greater than 8 inches should be compacted to a minimum of 90 percent of the maximum dry density obtainable by the ASTM D 1557 test method.
- j. Import soils used to raise site grade should be equal to, or better than, on-site soils in strength, expansion, and compressibility characteristics. Import soil can be evaluated, but will not be prequalified by the Geotechnical Engineer. Final comments on the characteristics of the import will be given after the material is at the project site.

3. Slope Construction

- a. Any construction of fill slopes should conform to the minimum standards listed in Appendix J of the 2007 California Building Code. It is recommended that the

steeper, horizontal to vertical. The keys should be tilted into the slope, should be a minimum of 12 feet wide, should be a minimum of 2 feet deep on their outside edge, and should be into firm, natural materials.

- c. Fill slopes should be overfilled, compacted, and then cut back to the planned configurations. This will yield better compaction on the slope faces than other methods.
- d. Backdrains should be placed within fill slopes to minimize the potential of seepage of water from the fill slope faces. A backdrain should consist of a minimum of 1 cubic foot of Class 2 permeable Filter Material per lineal foot of pipe surrounding a 4-inch diameter perforated PVC pipe (holes down). As an alternative to the filter material, $\frac{3}{4}$ -inch gravel can be used surrounded by a drain filter fabric. The drain should have a solid pipe extending out of the slope face to a concrete swale or a non-erosive surface. The backdrains should be placed at 10-foot vertical intervals in order to provide sufficient drainage.
- e. Fill and cut slopes are anticipated to be less than 10 feet in height.
- f. Cut slopes should be observed by an Engineering Geologist to determine if adversely oriented bedding planes are exposed in the slope face. If so, buttressing or flattening of the slope will be required.
- g. Lined drainage swales and downdrains should be provided at the tops of cut and fill slopes to divert drainage away from the slope face.

4. Utility Trenches

- a. Utility trench backfill should be governed by the provisions of this report relating to minimum compaction standards. In general, on-site service lines may be backfilled with native soils compacted to 90% of maximum density. Backfill of offsite service lines will be subject to the specifications of the jurisdictional agency or this report, whichever are greater.
- b. Backfill operations should be observed and tested by the Geotechnical Engineer to monitor compliance with these recommendations.
- c. Some difficulty may be encountered when excavating trenches because of cobbles and possibly boulders.

- cast in place piers (caissons) should be used to support the structure. The caissons should be tied together by grade beams.
- b. Caissons should bear into firm to hard bedrock as recommended elsewhere in this report. Caisson excavations should be observed by a representative of Earth Systems Southern California after drilling, but prior to placing of reinforcing steel or concrete, to verify bearing conditions.
 - c. An allowable adhesion value of 500 psf can be used to design caissons based on friction on the side of the piers. The portion of the caisson designed for side adhesion should be at a minimum horizontal distance from the slope face equal to the complete height of the slope divided by three, unless stated otherwise herein. This distance should not be less than 15 feet, but need not exceed 40 feet. The portion of the caisson designed for side adhesion should also be embedded exclusively into firm, "Vaqueros Formation bedrock" (not fill or colluvium/alluvium). The caisson excavations should be observed by a representative of this office to verify conformance with these recommendations.
 - d. Passive resistance of caissons can be estimated using a passive equivalent fluid weight of 325 pcf. This value does not include a factor of safety, which should not be less than 1.5. In determining passive resistance, none should be taken until the distance to daylight exceeds 8 feet. The effective diameter of any isolated caissons can be taken as twice the actual diameter.
 - e. It is recommended that concrete used in the caisson be placed with a slump of 4 to 6 inches in dry excavations and 6 to 8 inches when placed under water. In dry excavations the concrete can free-fall so long as it dropped vertically and does not strike the reinforcing cage. In wet excavations the concrete should be tremied to the bottom of the excavation. In no case should the concrete be allowed to free fall through water or drilling fluids. The end of the tremie should be kept several feet below the top of the concrete. The concreting should continue until clean concrete is discharged at the top of the caisson.

- b. It is recommended that perimeter slabs (walks, patios, etc.) be designed relatively independent of footing stems (i.e., free floating) so foundation adjustment will be less likely to cause cracking.
 - c. The slab designs should be provided by the project Structural Engineer.
 - d. Slabs should be underlain with a minimum of 4 inches of sand. Areas where floor wetness would be undesirable should be underlain with a vapor retarder or barrier (as specified by the project Architect or Civil Engineer) to reduce moisture transmission from the subgrade soils to the slab. The retarder/barrier should be placed per the recommendations of the project Architect or Manufacturer.
 - e. Soils underlying slabs that are in the "medium" expansion range should be premoistened to 140% of optimum moisture content to a depth of 27 inches below lowest adjacent grade. Premoistening should be confirmed by testing.
3. Frictional and Lateral Coefficients
- a. Resistance to lateral loading may be provided by friction acting on the base of foundations. A coefficient of friction of 0.53 may be applied to dead load forces. This value does not include a factor of safety.
 - b. Passive resistance acting on the sides of foundation stems equal to 308 pcf of equivalent fluid weight may be included for resistance to lateral load. This value does not include a factor of safety.
 - c. A minimum factor of safety of 1.5 should be incorporated into designs for sliding or overturning.
 - d. Passive resistance may be combined with frictional resistance provided that a one-third reduction in the coefficient of friction is used.
4. Retaining Walls
- a. Conventional cantilever retaining walls should not be backfilled with onsite expansive soils. Retaining walls backfilled with compacted imported granular soils may be designed for active pressures of 35 pcf of equivalent fluid weight for well-drained, level backfill, and 46 pcf for 2:1 sloping backfill. This backfill

be compacted to 90% of maximum dry density as determined by the ASTM D 1557 Test Method.

- c. The lateral earth pressure to be resisted by the retaining walls or similar structures should be increased to allow for surcharge loads. The surcharge considered should include the loads from any structures or temporary loads that would influence the wall design.
- d. A system of backfill drainage and waterproofing should be incorporated into the retaining wall designs. Backfill comprising the drainage system immediately behind the retaining structures should be a free-draining granular material with a filter fabric between it and the rest of the backfill soils. As an alternative, the back of the wall could be lined with a geodrain system. The backdrain should extend from the bottom of the wall to about 18 inches from finished backfill grade. In addition to waterproofing retaining walls that are a part of the buildings, waterproofing of exterior retaining walls should be considered to help mitigate efflorescence on wall faces.
- e. Compaction on the uphill side of the wall within a horizontal distance equal to one wall height should be performed by hand-operated or other light weight compaction equipment. This is intended to reduce potential "locked-in" lateral pressures caused by compaction with heavy grading equipment.
- f. Water should not be allowed to pond near the top of the wall. To accomplish this the final backfill site grade should be such that all water is diverted away from the retaining wall.

5. Settlement Considerations

Maximum expected settlement of less than 1 inch is anticipated for foundations and floor slabs designed as recommended and subjected to static loading. Differential settlement between adjacent load bearing members should be less than one-half the total settlement.

- but are not necessarily limited to the following:
1. Review of the building and grading plans during the design phase of the project.
 2. Observation and testing during site preparation, grading, placing of engineered fill, and foundation construction.
 3. Consultation as required during construction.

LIMITATIONS AND UNIFORMITY OF CONDITIONS

The analysis and recommendations submitted in this report are based in part upon the data obtained from the boring drilled on the site. The nature and extent of variations beyond the boring may not become evident until construction. If variations then appear evident, it will be necessary to reevaluate the recommendations of this report.

The scope of our services did not include any environmental assessment or investigation for the presence or absence of wetlands, hazardous or toxic materials in the soil, surface water, groundwater or air, on, below, or around this site. Any statements in this report or on the soil test pit logs regarding odors noted, unusual or suspicious items or conditions observed, are strictly for the information of our client.

Findings of this report are valid as of this date; however, changes in conditions of a property can occur with passage of time whether they be due to natural processes or works of man on this or adjacent properties. In addition, changes in applicable or appropriate standards may occur whether they result from legislation or broadening of knowledge. Accordingly, findings of this report may be invalidated wholly or partially by changes outside our control. Therefore, this report is subject to review and should not be relied upon after a period of 1 year.

In the event that any changes in the nature, design, or location of the construction and other improvements are planned, the conclusions and recommendations contained in this report shall

representative to insure that the information and recommendations contained herein are called to the attention of the Architect and Engineers for the project and incorporated into the plan and that the necessary steps are taken to see that the Contractor and Subcontractors carry out such recommendations in the field.

As the Geotechnical Engineers for this project, Earth Systems Southern California strives to provide our services in accordance with the generally accepted geotechnical engineering practices in this community at this time. No warranty or guarantee is expressed or implied. This report was prepared for the exclusive use of the Client and their authorized agents.

It is recommended that Earth Systems Southern California be provided the opportunity for a general review of final design and specifications in order that earthwork and foundation recommendations may be properly interpreted and implemented in the design and specifications. If Earth Systems Southern California is not accorded the privilege of making this recommended review, we can assume no responsibility for misinterpretation of our recommendations.

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CDMG, 1972 (Revised 1994), Fault Rupture Hazard Zones In California, Special Publication 42.

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FIELD STUDY

- A. On February 4, 2008, a 24-inch diameter bucket auger boring was drilled near the proposed building site to a maximum depth of 46 feet below the existing ground surface to observe the soil/bedrock profile and to obtain samples for laboratory analysis. The bucket auger boring was drilled by a track mounted rig, and was down-hole logged by a Geologist when the conditions were considered safe. The approximate location of the bucket auger boring was determined in the field by pacing and sighting, and is shown on the Geologic Map / Site Plan in this Appendix.
- B. Samples were obtained within the bucket auger boring using a 2.37-inch inside diameter ring sampler. The sampler was driven using the weight of the kelly bar.
- C. Bulk samples of the soils encountered were gathered from the auger cuttings.
- D. The final log of the boring represents our interpretation of the contents of the field log and the results of laboratory testing performed on the samples obtained during the subsurface study. The final log is included in this Appendix.

VT-24070-01

APRIL, 2008

Earth Systems
Southern California



744 VEREDA DEL CIERVO
SANTA BARBARA COUNTY, CALIFORNIA

OBLIQUE PHOTOGRAPH

**PROPOSED
BUILDING
SITE**



REGIONAL GEOLOGY MAP

744 VEREDA DEL CIERVO
SANTA BARBARA COUNTY, CALIFORNIA

Earth Systems



APRIL 2008

VT-24070-01

NOT TO SCALE

*Taken from T.W. Dibble, Jr., Geologic Map of the Dos Pueblos Quadrangle, 1987

