GEOLOGIC HAZARDS EVALUATION
AND
PRELIMINARY GEOTECHNICAL ENGINEERING STUDY

Faria Preserve
San Ramon
Contra Costa County, California

OCTOBER 2004

Prepared for

The Faria Preserve
CCSR
194 Francisco Lane, Suite 202
Fremont, CA 94539

Prepared by

EARTH SYSTEMS CONSULTANTS
Northern California
47853 Warm Springs Boulevard
Fremont, California 94539

© 2004 Earth Systems Consultants Northern California
Unauthorized use or copying of this document is strictly prohibited
without the express written consent of Earth Systems Consultants Northern California

Earth Systems Consultants
Northern California

47853 Warm Springs Blvd.
Fremont, CA 94539-7400
(510) 353-0320
FAX (510) 353-0344

The Faria Preserve
CCSR
194 Francisco Lane, Suite 202
Fremont, California 94539

Attention: Mr. Michael Conley

Subject: Faria Preserve
Proposed Residential Subdivisions
San Ramon, California
GEOLOGICAL HAZARDS EVALUATION AND PRELIMINARY GEOTECHNICAL ENGINEERING STUDY

Dear Mr. Conley:

In accordance with your authorization, Earth Systems Consultants Northern California (ESCNC) is submitting herewith the results of our geological hazards evaluation and preliminary geotechnical engineering study of the Faria Preserve development located in San Ramon, Contra Costa County, California. This report presents the results of our investigation procedures and our conclusions and recommendations pertaining to the geological and geotechnical engineering aspects of the project. It is ESCNC's opinion that the project is feasible, from a geologic and geotechnical aspect, as currently planned.

If you have any questions, or if we can be of further service, please contact our office.

Very truly yours,

EARTH SYSTEMS CONSULTANTS
Northern California

Gary M. Pischke
Senior Geologist
Certified Engineering Geologist 1501

Bill E. Zehrbach
Principal Engineer
Geotechnical Engineer 926

REGISTERED PROFESSIONAL ENGINEER
BILL E. ZEHRBACH
No. 926
GEOTECHNICAL
STATE OF CALIFORNIA

GMP/BEZ: gdw

Distribution: 15 to Addressee

REGISTERED GEOLOGIST
GARY PISCHKE
No. 1501
CERTIFIED
ENGINEERING
GEOLOGIST
STATE OF CALIFORNIA

TABLE OF CONTENTS

TABLE OF CONTENTS
(continued)

Page No.

TABLE OF CONTENTS
(continued)

FIGURES

TABLE OF CONTENTS
(continued)

I. EXECUTIVE SUMMARY

This report presents the results of our geologic hazards evaluation and preliminary geotechnical engineering study of the proposed Faria Preserve development in San Ramon. The purpose of this study was to characterize the geology, including landslide and fault hazards of the study area, and to assess the geotechnical aspects of the project with emphasis on characterization of landslides and colluvial slopes, settlement of new fills, seismic hazards, including liquefaction potential and dynamic settlements of engineered fills, and characterization of excavated materials. Earth Systems Consultants Northern California (ESCNC) excavated 21 test pits and 6 exploratory trenches, and drilled 31 test borings to assist in our evaluation of the study area. It is our opinion that there are no geologic constraints that would preclude the construction of single-family or multi-family residential units, other planned development, or roadways and associated utilities on the site, as generally planned, provided that the mitigation measures recommended herein are implemented in the project design and construction. A potential exists for differential settlement to occur in the deeper fills where fill thickness variations can occur. Specific mitigation measures should be incorporated into the design of foundations and other improvements in those areas most likely to be affected by differential settlement.

This report presents data and conclusions with respect to the feasibility of development on the subject site and provides a design level report pertaining to grading issues for inclusion with the Vesting Tentative Map submittal. In addition, this report also addresses a range of site characterization and related issues identified in consultation with the City of San Ramon's EIR sub-consultants, Treadwell & Rollo (T&R) and Gilpin Geosciences, Inc. (GGI).

T&R and GGI received copies of the exploratory program including maps and a boring summary spreadsheet. The proposed exploratory program was also discussed with T&R and GGI. Further discussion of the program and subsequent report were made with GGI personnel in the field in September 2004 during review of the exploratory trenches.

The Faria Preserve development will consist of a 290-acre development on the eastern portion of the 450 acre Faria Ranch property. The topography consists of a series of northwest-southeast trending ridges and valleys. The Calaveras fault crosses the eastern portion of the site. The topography is controlled by the underlying geologic structure of the area, with steeply dipping beds of two geologic formations, Pliocene non-marine sedimentary rocks including the Orinda Formation and Miocene marine sandstones including the Briones Formation.

The Orinda Formation is comprised of sandstone, conglomerate, siltstone and mudstone. The Briones Formation is comprised primarily of two units, massive sandstone and fossiliferous sandstone. The more erosion resistant sandstone and conglomerate units comprise the ridges on the site while the more erodable siltstone, mudstone, and claystone units underlie the valleys. The potential for landsliding on the portion of the site currently planned for residential development is considered moderate to high; this constraint will be mitigated through implementation of corrective grading and drainage control measures, including use of subdrains as discussed in this report.

The location of the Calaveras fault has been identified on the east side of the site in the location shown on the attached State Fault Hazard Zone Map, formerly the Alquist-Priolo (AP) Special Studies Zones Map. Several splays from the main trace were also identified by the State. The potential for ground rupture along the documented traces of the Calaveras fault is considered high. Therefore, a building setback of 50 feet has been recommended for either side of the Calaveras fault and related splays. This recommended setback adjoins the northeast corner of Neighborhood D and the easternmost portion of Neighborhood A along the access road. The location of all buildings shown in the current plans for the Faria Preserve comply with this setback requirement.

Trenching was also performed to evaluate the potential existence of the Las Trampas thrust fault that was mapped by Crane (1988) west of the main Calaveras fault zone. No evidence was found to indicate that the thrust fault is present as mapped. No setback or other mitigation is required for the area related to the presence of the Las Trampas thrust fault as mapped on the site.

There is a significant potential for strong to very strong ground shaking at the site as a result of an earthquake on one of the active faults in the San Francisco Bay area. A moderate to major earthquake on the Calaveras or Hayward fault, or a major earthquake on the San Andreas fault, could cause severe ground shaking at this site. This report provides measures to mitigate for ground shaking.

Secondary seismic effects include seismically induced landsliding, liquefaction, lurch cracking, lateral spreading and seismically induced subsidence. The potential for seismically induced landsliding under pre-mitigation conditions is moderate to high in the drainages, particularly along the slopes of the main drainage channel. The potential for liquefaction is low throughout most of the area. One alluvial deposit of potentially liquefiable soil was encountered in Boring B8 at a depth of 19.5 to 24.5 feet. This area is planned to receive approximately 50 feet of engineered fill that will provide sufficient overburden pressure to eliminate the liquefaction potential. The potential for lurching and lateral spreading due to strong ground shaking is considered moderate in the site drainages under the current pre-mitigation condition. This report provides specific recommendations to avoid such impacts through the incorporated mitigation. M

The residual, alluvial and colluvial soils on the site have a moderate to high shrink swell potential when subjected to seasonal moisture changes. The sandstone bedrock materials have a low expansion potential in an undisturbed state. However, when they are mechanically broken down and placed as fill, as called for in the grading plan, the expansion potential ranges from low to moderate. The siltstone, claystone and mudstone units found at the site have a moderate to high expansion potential, especially when reconstituted as fill materials, as also called for in the grading plan. Special grading recommendations have been made to reduce the expansion potential of the site soils on the proposed development. In general, this will include placing the expansive material in the deeper portions of the fill covered with the less expansive sandstone and conglomeratic materials.

The alluvial, colluvial and landslide deposits are stiff to hard. The compressibility characteristics of these materials vary from low to moderate, especially under the high surcharge loads to be encountered under the deeper fills. The amount of surface settlement under proposed fill loads will be on the order of 4 to 24 inches without our recommended mitigation, depending on the thickness of the surficial deposits and the depth of the fill. Subexcavation and recompaction of these deposits is required in the areas of proposed development where settlements from these units cannot be tolerated. Removal and replacement of these deposits will eliminate the potential settlement due to consolidation of the natural soil deposits. When these deposits are recompacted they would be subject to hydro-consolidations as discussed below

The engineered fills to be constructed on the site are also potentially subject to settlement as the result of a phenomenon known as hydro-consolidation. The magnitude and effects of this settlement will be partially mitigated through use of a variety of measures, as recommended in this report, including: (a) higher compaction standards; (b) placement of fills at a moisture content well above optimum; and (c) contour grading of the underlying natural topography to reduce large differential fill conditions. However, because the settlements cannot be completely eliminated, the remaining potential settlement will be considered in the design of streets, gravity utilities and structures as outlined in this report. Hydro-consolidation will result in potential surface settlement on the order of 0.5 to 1 percent of the overall fill thickness for fills greater than 50 to 75 feet thick.

Recent studies have indicated that compacted fills are also susceptible to consolidation resulting from ground shaking. The same mitigation measures used to reduce hydro-consolidation related settlement will reduce dynamic consolidation settlements. An analysis of the proposed fills indicates that seismically induced settlement of the fills will potentially be on the order of 0.3 to

2 inches for corresponding fill depths of 25 to 100 feet. These values are based on a major earthquake on the nearby Calaveras fault, a low probability event.

Static consolidation settlement will be mitigated by removal and recompaction. Settlements due to hydro-consolidation are likely. Settlement due to dynamic compaction has a low probability. These two settlements would be cumulative if both were to occur. The potential for settlement of fills up to about 50 feet thick will be negligible. For design purposes it is recommended that gravity utilities and surface drainage consider cumulative surface settlements due to hydro-consolidation and dynamic consolidation, as a percent of underlying fill thickness, of 0.15 percent for fills 50 to 75 feet thick; 0.5 percent for fills 75 to 100 feet thick; and 1 percent for fills 100+ feet thick.

Static and dynamic stability analyses indicate that the proposed cuts and fills shown on the current development plan are stable when constructed at the proposed slope ratio of 3 to 1 (horizontal to vertical). The stability analyses indicate that the calculated factors of safety are above the minimum 1.5 for the static condition and 1.1 for the pseudo-static condition. Slope gradients steeper than 3 to 1 can also be utilized in localized cut and fill areas, subject to further stability analysis.

This report recommends removal of the majority of the landslide deposits for settlement reasons, in order to eliminate their impact on stability of the graded slopes. Earthen berms have been recommended to protect the development on the west side of the site from the debris flow potential on the steeper slopes of the westerly ridge. A buttress fill will be constructed to increase the calculated factor of safety for the buttressed landslide in the northwest portion of the site. The recommended buttress fill will provide mitigation to assure the stability of planned homes and improvements to the south within Neighborhood A.

Based on the forgoing conclusions and the detailed recommendations contained in the body of this report, it is our opinion that development of the Faria Preserve may be carried out as reflected in the current Vesting Tentative Map application. It is anticipated that conventional foundation designs can be utilized in cut areas and in shallow fill areas. Special foundations designed to accommodate differential settlement, such as waffle slabs, post tensioned slabs with stiffener ribs, or stiffened foundations with underpinning piers, will be required near cut/fill transitions and where there are deep fills or high differential fill thicknesses under individual buildings, to accommodate the potential settlements and differential settlement.

II. INTRODUCTION

This report presents the results of our geologic hazard evaluation, and preliminary geotechnical engineering investigation performed for the proposed development of the Faria Preserve located adjacent to the City of San Ramon, Contra Costa County, California. See Figure 1-Study Area Location map for the general location of the site and Figure 2-Study Area Map for the general shape of the subject property. The geologic hazards phase of this study was focused on evaluating the local and regional geologic conditions as they may impact the proposed graded cuts and fills, planned roads, proposed building areas, and related facilities for the site. The geologic phase incorporates data from previous reports by Berlogar Geotechnical Consultants and Wahler Associates. The fault investigation phase evaluated the location of the Calaveras fault on the eastern portion of the site, and included an investigation evaluating the Las Trampas thrust fault mapped by Crane (1988) on the northeastern portion of the site. The preliminary geotechnical engineering portion of this study has evaluated the pertinent engineering properties of the on-site soil and bedrock formations. This information has been used in conjunction with known geologic conditions to enable ESCNC to provide preliminary geotechnical recommendations for site development. The conclusions and recommendations presented in this report are based upon data acquired and evaluated during the course of this study.

This study utilized data from the December 2002 ESCNC Geological Hazards and Preliminary Geotechnical Engineering Investigation and the May 2004 Supplemental Fault Investigation of the Los Trampas thrust fault. It also incorporates work our firm has been compiling in cooperation with T&R and GGI since July of this year.

II.A. Purpose of Study

The purposes of this study were to: (1) obtain information on the subsurface conditions within the project site; (2) evaluate the data; and (3) provide a design level report for inclusion with the Vesting Tentative Map submittal. Our objective has been to work closely with the City's sub-consultants, T&R and GGI, to include in this report a full analysis of site characterization and potential impact issues for use in the specific plan EIR.

This report addresses potential impacts and corresponding mitigation measures with regard to the following issues: (a) the location and extent of the Calaveras fault and related traces and any associated necessary setbacks; (b) the potential for deep-seated landslides in the central valley; (c) risks associated with off-site originating landslides or debris flows; (d) risks associated with potential liquefaction at depth; (e) risks associated with seismically-induced settlement; (f) characterization of soils and rock within deep cut and fill areas; and (g) the feasibility of grading operations utilizing conventional equipment based on analysis of the geologic formations present on the site.

II.B. Location and Description of Site

The site is located in southwestern Contra Costa County, California, adjacent to the City of San Ramon. It is west of Highway 680 and north of Crow Canyon Road. Bollinger Canyon Road runs in a north-south direction to the west of the subject study area (See Figure 1). The site is bounded to the north by a relatively undeveloped hillside, to the west by existing residential development and the remainder of the Lands of Faria, to the south by existing residential developments, and to the east by existing commercial and light industrial developments.

The site is characterized by a series of northwest-southeast trending ridges and valleys. The three predominant ridges are the eastern ridge, the central ridge, and the western ridge. The two main drainages are the eastern drainage, where the Calaveras fault zone is located, and the main or central drainage, between the central and western ridges (See Figure 2). The ground surface varies in elevation from a high of 998.7 in the northwestern portion of the site to 524 at the extreme eastern corner of the property. Refer to Figure 3 Site Plan/Geology for the topography of the site.

The ridges are covered with seasonal grasses and scattered shrubs while the drainages have scattered groves of oak and cottonwood trees. An existing 5.1 MG East Bay Municipal Utility District water tank is located in the southeastern corner of the site.

II.C. Site Development

The Lands of Faria is 450 acres of property located in Contra Costa County, adjacent to the northwest corner of the City of San Ramon. A 290-acre portion of this property (the Faria Preserve) is situated within San Ramon's Northwest Specific Plan Area. Located within the City of San Ramon's Urban Growth Boundary, the Faria Preserve is planned for annexation and development in accordance with the City's Northwest Specific Plan. The development plan indicates that roadways providing access to the site will connect with existing roadways at Bollinger Canyon Road in the southwest corner of the development, and Purdue Road on the east side of the development.

Within the development, there will be four residential neighborhoods, an educational facility, a community park, place of worship, and open space. Neighborhood A, located in the central and northwestern portion of the development will consist of 42.1 acres of low density, single-family, detached housing. The 17.6-acre Neighborhood B, located near the southeast corner of the development, will consist of a more compact single-family housing community. Neighborhoods C and D occupy sites of 11.8 and 2.1 acres, respectively at the southeast corner of the property. The 6.2-acre place of worship site is located in the southwest corner of the development. A 12.7-acre improved community park will adjoin a natural hillside and the 1.6-acre educational use site. The remaining portions of the 290-acre site will be devoted to open space and related public facility uses. Refer to Figure 4-Proposed Site Development, for the general location of the proposed development areas.

Grading will be required to create the proposed development plan. The tentative grading scheme is shown on Figure 4. In general grading will consist of cutting the ridges and filling of the main (western) drainage and lesser drainages. Cuts will be on the order of 10 to 90 feet with four small localized areas where cuts will be on the order of 100 feet. Cut slopes will be up to 158 feet in height from toe to crest. Fills will be on the order of 10 to 70 feet thick, with small localized fill depths on the order of 80 to 110 feet in the central portion of the main drainage and

80 to 90 feet in a small area located at the southeast corner of the site. Fill slopes will be constructed up to 152 feet in face height due to the use of 3 to 1 slope gradients. The deeper fills will be placed in the western and easternmost drainages. Several detention basins and two water storage reservoirs are also proposed as part of the proposed plan.

II.D. Scope of Geologic Hazards Evaluation and Preliminary Geotechnical Engineering Study

This geologic hazard evaluation and preliminary geotechnical engineering study encompassed the following work:

1. Review of available published and unpublished data concerning geologic and soil conditions within and adjacent to the site that could have an impact on the proposed development. This included review of data acquired by other engineering firms.

2. Review and interpretation of stereo aerial photographs dating from 1958 to 2002.

3. Geologic mapping of the site.

4. Excavation and logging of 21 test pits to evaluate geologic structure, lithology, extent of landsliding, and characteristics of alluvial and colluvial soils.

5. Subsurface exploration and identification of the subsurface soil, bedrock, and groundwater conditions within the Calaveras fault zone and related traces by means of excavating and logging six exploratory trenches to depths of up to 12 feet with a backhoe. Exploratory operations were conducted by a staff geologist and a Certified Engineering Geologist

6. Excavation, logging and selective sampling of 31 exploratory borings to depths of up to 84 feet, to evaluate thickness of colluvium, depth of landslide masses, liquefaction potential, and

rippability of bedrock. Nine borings were drilled for continuous core evaluation of the deepest landslides on the project. Four borings were drilled to evaluate liquefaction potential within alluvial deposits of the site. Ten borings were drilled to evaluate the rippability potential within the proposed cut areas. Two borings were drilled in the alluvial and colluvial areas of the site. Three supplemental borings were drilled in bedrock cut areas to obtain undisturbed samples of the bedrock materials for laboratory testing. Two supplemental borings were drilled in landslide areas and one in an alluvial deposit area to obtain additional samples for laboratory testing. The boring program was conducted with two rigs: a large truck mounted CME61 rig for the cut evaluation, and a smaller four-wheel drive rig to evaluate the landslides, particularly within the central valley. Access for the landslide borings was provided by a bulldozed road. No large diameter boreholes are planned at this time, until completion of final plans and the required need to increase the detail of landslide evaluation.

7. Analysis of representative samples (bulk and relatively undisturbed) obtained from the borings and test pits to determine the physical and engineering properties pertinent to the scope of this study.

8. Preparation of a geologic map and geologic cross sections through the site in general and through critical areas of proposed grading with respect to interpolated and extrapolated geologic conditions.

9. Engineering and geologic analysis of the data with respect to the proposed development.

10. Compilation of the geologic and geotechnical data and preparation of a report with appropriate graphics presenting our findings, conclusions and recommendations for the proposed development.

This scope of work incorporates additional analysis of data collected through the preceding work, and developed in cooperation with T&R and GGI, in order to provide an overall characterization of the following potential site development constraints:

- Determination of fault locations
- Landslides and colluvial slopes
- Upslope landslide hazards
- Seismic hazards including liquefaction potential
- Settlement of new fills
- Slope Stability
- Characteristics of excavated material

The scope of our services did not include determination of soil corrosion potential nor environmental assessment or investigation for the presence or absence of hazardous or toxic materials in the soil, surface water, groundwater or air, on, below, or around the site. These issues will be addressed separately.

II.E. Background Information

Wahler Associates (WA, 1987) performed an investigation of the southern and central portion of Faria Ranch in their report "Preliminary Soils and Geologic Investigation, Faria Ranch Development, San Ramon, California". Berlogar Geotechnical Consultants (BGC, 1991) performed an investigation on the southwestern portion of the subject area. Test pit and boring logs from the previous studies by Wahler Associates and Berlogar Geotechnical Consultants are included in Appendix A. The location of Wahler Associates and Berlogar Geotechnical Consultants test pits are shown on Figure 3. BGC boring locations are also shown on Figure 3.

The discussions of field conditions, geology and geotechnical considerations by both WA and BGC were reviewed for consistency with the findings of the present study. The results from the test pits and geologic evaluation from both reports are generally consistent with our interpretation of site conditions.

II.F. Geologic Setting

The site is located on the eastern side of the San Francisco Bay in the Coast Ranges Geomorphic Province of California. The Coast Ranges are comprised of several mountain ranges and structural valleys that trend northwest-southeast, parallel to strike-slip faults such as the San Andreas, Hayward, Calaveras and Greenville faults. The subject site is located on the southern terminus of the northwest-southeast trending Las Trampas Ridge. The Calaveras fault is mapped crossing the eastern portion of the study area. The Las Trampas thrust fault (Crane, 1988) is mapped crossing the eastern portion of the study area, and is considered a northern splay of the Calaveras fault.

III. GEOLOGIC HAZARDS EVALUATION

III.A. Geologic Literature Review

According to geologic maps by Wagner (1978), Dibblee (1980), Crane (1988), Wagner, et al, (1991) and Graymer, et al, (1994), the site is underlain by the Pliocene and Miocene age sedimentary rocks of the Contra Costa and San Pablo Groups. Wagner (1978) mapped the site as underlain by Tertiary sedimentary rocks including the Briones and Orinda Formations. Dibblee mapped Pliocene non-marine sedimentary rocks including the Orinda formation on the western half of the project site. The Pliocene age rocks are described as weakly indurated greenish gray mudstone, siltstone, sandstone and pebble conglomerate. The Miocene marine sedimentary rocks including the Briones formation are mapped on the eastern portion of the site. The Miocene age rocks are described as gray to tan wacke sandstone, locally pebbly, containing clam shell beds. The sandstone bedrock forms ridges that generally trend N50W. The underlying geology, as mapped by Dibblee, is shown as Figure 5. (Note: the geologic reports do not completely distinguish between the claystone, mudstone and siltstone rocks within the study area. The main distinguishing factor is the grain size increases from the claystone to the siltstone. The mudstone is a combination of claystone and siltstone.)

Crane shows the northeastern to central portion of the site crossed by the Las Trampas thrust fault, which divides the Briones formation west of the fault from Tertiary (Tr) siltstone and sandstone east of the fault. Crane characterized the Las Trampas thrust as a splay of the Calaveras fault. The underlying geology as mapped by Crane is shown on Figure 6. The mapped location of the Las Trampas thrust fault is also shown on Figure 3.

The site is shown on the Graymer, et al (1994) geologic map of Contra Costa County bedrock (Assemblage II) as underlain by the Tertiary Briones Formation- undivided (Tbr), the Neroly Formation – blue sandstone (Tn), and an un-named upper Tertiary sedimentary and volcanic rocks (Tus). The geology as compiled by Graymer, et al, is shown on Figure 7.

Several slope failures and debris flows are mapped in the project area by Nilsen (1975). Landslides are mapped along both drainages within the study area. The landslides as mapped by Nilsen are shown on Figure 8.

The Fault Hazard Zone Map (CDMG, Special Studies Zone Map, 1982) for the Diablo Quadrangle is included as Figure 9. The subject site is located between two active regional faults. The Hayward fault is located approximately 8 miles (12.8 km) southwest of the subject site (Wagner, et al, 1991). The Calaveras fault crosses the eastern portion of the study area. The regional fault map and earthquake summary for the Bay Area are shown on Figures 10 and 11.

Additional background review was performed on literature available for the Calaveras fault. Fault evaluation reports (FER) for the Calaveras fault were reviewed in more detail (Hart, 1981a, 1981b). A study by ENGEO (1978) was reviewed for the adjacent site on trend of the fault to the southeast of the site. The site study by ENGEO encountered a trace of the Calaveras fault. ENGEO recommended a 50 foot setback from the fault trace. The additional review of the FER studies indicated that the Calaveras fault might be located further east than mapped. The FER maps show various interpretations of the fault location including the trace shown on the Fault Hazard Zone Map.

The site is located within the seismically active central California Coast Ranges geomorphic province. The major active faults recognized in this region of California are the San Andreas, Hayward, and Calaveras faults. See Figure 10, the Regional Fault and Earthquake map, for the location of these faults relative to the site. Active faults are defined by the State of California as exhibiting well-defined evidence of displacement within Holocene time, or the last 11,000 years (Hart, 1997). The definitions of "potentially active" vary widely. An accepted definition of potentially active is a fault showing evidence of displacement older than 11,000 years and younger than 2,000,000 years (Pleistocene Epoch). "Potentially active" is no longer used as criteria for zoning. The terms "sufficiently active" and "well-defined" are now used by the California Division of Mines and Geology as criteria for zoning faults under the Alquist-Priolo

Act (Hart, 1997). "Inactive" faults are classified as not having been active for at least two million years.

A number of strong earthquakes, that have damaged man-made structures, have occurred on the active faults in the Bay region within the last 200 years (Figure 11). Especially notable are the 6.8M (estimated magnitude) 1868 Hayward earthquake, the 1906 8.3M San Francisco earthquake, the 1926 Monterey Bay 6.1M doublet, the 6 August 1979 5.8M Coyote Lake earthquake, the 24 April 1984 6.2M Morgan Hill (Halls Valley) earthquake, and the 17 October 1989 7.1M Loma Prieta earthquake. The epicenter of the 1989 Loma Prieta earthquake was approximately 58 miles (93 km) south of the site. The epicenter of the 24 April 1984 Morgan Hill (Magnitude 6.2) earthquake was approximately 32 miles (51 km) southeast of the site. The above-referenced earthquakes damaged man-made structures over a large part of the region (Plafker and Galloway, 1989). The major faults in the area are capable of generating an earthquake of at least 7.0 in magnitude and could cause strong ground shaking at the subject site (Hall, et al, 1974). The largest earthquake likely to be generated on the San Andreas Fault in the Santa Cruz Mountains region is estimated to be a magnitude 8.5 (Hall, et al, 1974).

The Calaveras fault is considered active from San Ramon to Hollister (Hart, 1984). Three earthquakes of Richter magnitude 5.8 and larger have occurred on the Calaveras Fault since 1900 (Stover, 1984) (Figure 11). The 1979 earthquake resulted in ground shaking intensity of VII in the site vicinity (Toppozada, et al, 1981). The 1984 Morgan Hill earthquake produced ground shaking equivalent to a modified Mercalli intensity of V-VI in the Gilroy area (Stover, 1984). The Calaveras Fault crosses the eastern portion of the site and could rupture during a major event. The site would be subject to higher intensity shaking due to close proximity of the fault zone. Figure 12 is a reproduction of the modified Mercalli intensity scale.

Major earthquakes were reportedly centered on the Hayward fault in 1858 and 1868 (Figure 10). The 1868 earthquake (approximate Richter magnitude of 6.8) was centered in Hayward, approximately 10 miles (16 km) southwest of the site and produced an estimated ground shaking intensity of IX on the modified Mercalli intensity scale (Toppozada et al, 1981). The characteristics and earthquake history of the Hayward fault are described in detail by Steinbrugge, et al (1987) in the "Earthquake Planning Scenario for a 7.5 Magnitude Earthquake on the Hayward Fault," by the California Division of Mines and Geology. The accounts of the 1868 earthquake as reported by Lawson (1908) are reiterated in that publication. The 1868 event produced ground rupture along a nearly straight fault trace extending from the Berkeley Hills to Mission San Jose. The fault trace and ground rupture were reportedly well defined from San Leandro southward to Agua Caliente Creek, near the present intersection of Mission Boulevard and Highway 680 in Fremont. The Calaveras fault reportedly ruptured in the 1861 earthquake, which may include the trace of the fault on the subject site.

III.B. Seismicity

Estimates of the potential ground shaking characteristics of Bay Area localities have been published by the Association of Bay Area Governments (ABAG, 1995). The ground shaking intensities that could be produced by an earthquake on one of the faults most likely to impact the site are summarized in Table 1 below. The potential ground shaking amplification at the site due to the physical characteristics of the underlying shallow bedrock ranges from moderately low to moderately high (ABAG, 1995).

Table 1

POTENTIAL GROUND SHAKING INTENSITIES (ABAG, 1995)

Causative Fault	Earthquake Magnitude	Relative Ground Shaking Intensity (Modified Mercalli Intensity Scale)
Calaveras	6.9	IX
Hayward (entire length)	7.3	IX
San Andreas (peninsular segment)	7.1	VII-VIII

Maximum (peak horizontal) ground acceleration is one of the basic parameters used to characterize the ground shaking potential at a given site. Actual ground accelerations at a locality are influenced by topography, geologic structure, condition of subsurface materials, and groundwater level. The peak ground accelerations presented in Table 2 are based upon the estimated upper bound earthquake (previously called a "maximum credible earthquake") at the near-point of the causative fault, or are based upon data recorded during known seismic events and extrapolated to the subject site. The table lists known active faults in the San Francisco Bay region that could impact the site and their estimated seismic parameters. These estimations were generated using the EQFAULT computer program (Blake, 2000) applying statistical analysis and attenuation relationships determined by Campbell and Bozorgnia (1994) for a soft rock site. This method of seismic analysis is a deterministic approach, where each active fault within the region that may be reasonably expected to generate strong ground shaking at the site is evaluated.

Table 2

SEISMIC DATA FOR BAY AREA ACTIVE FAULTS

	San Andreas	Calaveras	Hayward	Green Valley
Distance and Direction from Site to fault (mi/km)	27/43.5 SW	0/0 NE	8.7/14 SW	7.5/12.1 NE
Upper Bound Earthquake[1]	8.0M	7.5M	7.5M	7.7M
Maximum Probable Earthquake[1]	7.3M	6.2M	6.5M	6.8M
Maximum Credible Peak Horizontal Ground Surface Acceleration[2, 3, 4, 5]	0.19g	0.56g	0.34g	0.35g

Table 2

(continued)

SEISMIC DATA FOR BAY AREA ACTIVE FAULTS

	San Andreas	Calaveras	Hayward	Green Valley
Maximum Probable Peak Horizontal Ground Surface Acceleration[2, 3, 4, 5]	0.11g	0.34g	0.21g	0.21g
Repeatable (effective) High Ground Acceleration[4, 5]	0.09g	0.26g	0.18g	0.18g

NOTES:
1 Richter Earthquake Magnitude
2 From: Campbell and Bozorgnia (1994)
3 Assumes that the earthquake occurs on the near point of the fault, the probability of which is low
4 From: Ploessel and Slossen (1974)
5 Acceleration is expressed as percent gravity, a/g

At the present time, it is not possible to predict the occurrence or magnitude of earthquakes. The Working Group on California Earthquake Probabilities (2003) has estimated that there is a 62% probability that one or more major earthquakes will occur in the Bay Area within the next 30 years. They estimate a 21% probability of a magnitude 7± earthquake on the Peninsula segment of the San Andreas Fault, a 27% probability of a magnitude 7 earthquake on the Hayward Fault and an 11% probability of a magnitude 7 earthquake on the Calaveras Fault during the 30 year period beginning in 2002. The recorded historic seismicity and interpretation of existing data indicates that it is probable that the site will experience moderate to strong ground motion generated by at least one earthquake with a Richter magnitude of 6.4 to 8.5, and probably by a number of earthquakes of lesser magnitude. Earthquake probabilities for the Bay Area are shown on Figure 13.

III.C. Aerial Photograph Interpretation

Aerial photographs from 1958 through 2002 were reviewed prior to the field investigation for landslides, faults and features related to bedrock structure. Many landslides were visible on the older air photos for the site, which were subdued topographically in the recent mapping evaluation. Different ages of landslide events were noted on the air photos. The air photo interpreted landslides were included on the site geology map, Figure 3. The photographs indicate multiple photo-lineaments related to the Calaveras fault. The fault trace is partially obscured by landslides and slide related scarps. The CDMG, 1982, mapped trace was identified on the air photos as a linear feature along landslide scarps and related to springs. The eastern drainage appears to be linear, suggesting fault origin, rather than contact between rock types. The photo-lineaments are shown on Figure 3. Aerial photographs reviewed for this study are listed at the end of the References Cited.

III.D. Geologic Reconnaissance

Field reconnaissance was conducted concurrent with the subsurface investigation. The field traverses concentrated on evaluating structure, geologic contacts, and access for subsurface investigation. Field traverses and test pits logs from WA were incorporated in the mapping shown on Figure 3.

III.E. Subsurface Exploration

The field reconnaissance, geologic mapping, and subsurface investigation were conducted in October and November 2002, March 2004, and August and September 2004. The subsurface investigation consisted of three phases of work: (a) twenty-one test pits excavated by a backhoe in the proposed fill areas, cut areas and landslides; (b) six trenches excavated by backhoe, including four along the traces of the Calaveras fault, and two across the mapped Las Trampas thrust fault; and (c) thirty-one test borings excavated with truck mounted drill rigs throughout the site. The test pit and trench logs are included in Appendix B as Figures B1 through B10, and Figures B11 through B15, respectively. The logs of test borings are presented in Figures C4 through C34 in Appendix C. Figure 3 shows the approximate locations of the test pits, fault trenches, and test borings. The present investigation did not include subsurface exploration in the area on the southwestern side of the study area. This area had been previously investigated by WA and BGC. Their test pits and boring logs are included in Appendix A. The geology of the site as interpreted by WA and BGC is also shown on Figure 3.

Test Pits - The test pits TP-1 through TP-5, TP-11, TP-14, and TP-15 were excavated by KJM Enterprises along the central drainage and western ridge in October 2002. The test pits were placed in areas of thick alluvium, colluvium and/or landslide deposits. The locations of the test pits are shown on Figure 3. The following is a description of the test pits and geology.

TP-1 was excavated in a landslide on the northwestern side of the study area. The test pit encountered two nested slides with two distinct planes over soil, possible alluvial material. The test pit was excavated to a depth of 28 feet. No groundwater was encountered. TP-2 was excavated on a hillside to evaluate a possible landslide feature. The test pit encountered colluvium over weathered bedrock. TP-3 was excavated in the center of a landslide complex. The test pit encountered multiple slide planes and blocks of displaced bedrock. Apparent bedrock was observed in the bottom of the test pit at 20 feet. TP-4 was excavated in an alluvial fan complex mapped by Nilsen, 1975, as a landslide. The test pit encountered a sequence of fan deposits over alluvial soil. A possible slide plane was observed at 15 feet. TP-5 was excavated in the western lobe of a landslide at the western edge of the eastern ridge. The test pit encountered thick colluvium and landslide debris to a total depth of 18 feet. TP-11 was excavated in the eastern lobe of the landslide. The test pit encountered landslide material over weathered shale/siltstone bedrock (Tps). TP-14 was excavated in the area of the proposed keyway to be placed in the eastern portion of the central drainage. The test pit encountered alluvial deposits and possible debris flow/landslide deposits comprised of clays with rounded sands, sandstone pebbles, and gravels. Groundwater seeps were encountered in the test pit at 19 feet, with standing water in the bottom at 25 feet. TP-15 was excavated in a landslide complex on the eastern edge of the western ridge. The test pit encountered colluvial clays over landslide debris to the explored depth of 20 feet.

Test pits TP-6 through TP-10, TP-12 and TP-13 were excavated by a KJM Enterprises backhoe along the eastern ridge and drainage in October 2002. Three test pits (TP-6, TP-7 and TP-9) were located in possible landslides. Four test pits (TP-8, TP-10, TP-12, and TP-13) were located to evaluate general soil and bedrock conditions. The locations of the test pits are shown on Figure 3. The following is a description of the test pits and geology.

TP-6 was excavated on the slope of the eastern ridge near the mapped trace of the Calaveras Fault. The test pit encountered colluvial soils over weathered massive sandstone (Tmss). TP-7 was excavated in a mapped landslide along the eastern drainage. The test pit encountered landslide material comprised of weathered claystone on a distinct slide plane over weathered clayey sandstone. The test pit was excavated to a depth of 23 feet. TP-8 was excavated at the head of a colluvium filled drainage. The test pit encountered colluvial soils over sandstone (Tmss). TP-9 was excavated in a possible mapped landslide. The test pit encountered colluvial soils with siltstone and sandstone clasts over weathered sandstone (Tmss). TP-10 was excavated on the northern slope of the eastern ridge. The test pit encountered soil over weathered mudstone and siltstone (mapped within Tmss). TP-12 was excavated at the upper end of a secondary drainage to the eastern drainage. The test pit encountered colluvial soil with sandstone clasts over claystone (mapped within Tmss). TP-13 was excavated on the ridge between the secondary drainage and the eastern drainage. The test pit encountered soil over weathered siltstone and mudstone (mapped within Tmss).

Test pits TP-16 through TP-21 were excavated by KJM Enterprises in August 2004 to supplement the 15 test pits excavated in October 2002. The test pits were placed in areas of thick alluvium, colluvium and/or landslide deposits. The locations of the test pits are shown on Figure 3. The following is a description of the test pits and geology.

TP-16 was excavated in a mapped colluvial deposit on the southeastern side of the central valley. The test pit encountered five feet of colluvium over sandstone. The test pit was excavated to a depth of seven feet. No groundwater was encountered. TP-17 was excavated northwest of TP-17 in a colluvial swale, possibly related to a landslide. The test pit encountered colluvium over landslide debris. TP-18 was excavated in the landslide complex on the western flank of the central valley. The test pit encountered one slide plane over landslide debris and possible alluvium. No apparent bedrock was observed in the bottom of the test pit at 10 feet. TP-19 was excavated on the ridge at the edge of the planned cut/ fill contact. The test pit encountered soil over weathered mudstone. No landslide planes were observed. TP-20 was excavated on the western flank of the eastern ridge, east of the Calaveras fault zone. The test pit encountered thick colluvium with floaters of Briones sandstone to a total depth of seven feet. TP-21 was excavated in the western-most drainage channel. Test pit TP-21 encountered colluvium over alluvial deposits. No groundwater was encountered in the test pits.

<u>Fault Trenches</u> - Three fault trenches were excavated on the trend of the Calaveras fault by KJM Enterprises in November 2002. Trench T1-2002 was located on the northern portion of the fault; T2-2002 was located near the southern boundary; and T3-2002 was located in the central portion of the eastern drainage. The approximate locations of the fault trenches are shown on Figure 3. The following is a summary of the observations from the three fault trenches.

Trench T1-2002 was excavated in an area where two splays of the Calaveras fault were mapped on the Fault Hazard Zone Map (Hart, 1981a, 1981b) (Figures 3 and 9). The trench excavation encountered fossiliferous Briones Formation bedrock on the east side of the eastern drainage. A fault splay was encountered at the mapped location of the fault along the drainage at station 0+30. The excavation continued in sandstone and soil blocks between fault splays to station 1+35. The trench shows that the sandstone ridge in the drainage is defined by fault splays. West of station 1+35, the trench encountered a colluvial sequence with few to no visible shears. At station 1+50, the trench exposed landslide planes and thick landslide material. From stations 1+50 to 2+60, the trench encountered landslide material with multiple slide planes exposed in the excavation. No fault related shears were observed in this portion of the trench. The northern trench wall caved in from stations 2+20 to 2+50 and was logged only from the surface.

Trench T2-2002 was excavated in the fault zone (Wagner, 1978; Hart, 1981b) on the southern portion of the subject site where the Calaveras fault is mapped in the center of the creek drainage and along the eastern slopes of the ridge. The trench encountered colluvial and alluvial material from stations 0+00 to 1+90. No disrupted or sheared zones were encountered in this portion of the trench. Multiple shears and clay seams associated with the Calaveras fault were encountered from stations 1+90 to 2+20. A landslide plane was observed above the shear zone and extended into the hillside at station 2+22. Landslide material was encountered from station 2+00 to the end of the trench at station 2+45. The landslide appeared to have overridden the fault zone at this location.

Trench T3-2002 was excavated along the confluence of the secondary drainage and eastern drainage in the central portion of the site, crossing the mapped trace of the Calaveras fault. Fault related shears and clay seams were encountered in the trench from stations 0+05 to 0+75. A landslide plane was observed above sheared Tmss sandstone. The trench encountered multiple landslide planes and appeared to be within the headscarp area of a southerly trending slide. Landslide debris appeared to have ponded against a possible fault scarp at station 0+25. Landslide material was encountered in the trench from station 0+75 to the end at station 1+05.

Additional field reconnaissance, geologic mapping, and subsurface investigation was conducted in March 2004. The subsurface investigation consisted of two trenches excavated by backhoe perpendicular to the mapped trace of the Los Trampas thrust fault (Crane, 1988) by KJM Enterprises.

Trench T4-2004 was located in a flat portion of the ridge across the trace of the mapped thrust fault. T4A-2004 was located approximately 60 feet north of the main trench T4 (See Figure 3). The following is a summary of the observations from the two fault trenches.

Trench T4-2004 was excavated in an area where the thrust fault is mapped by Crane crossing the ridge (Figure 6). The trench excavation encountered bedrock or weathered bedrock across the floor of the trench. The western 45 feet of trench encountered siltstone and mudstone bedrock, which was highly sheared and folded. The trench wall caved in from stations 0+15 through 0+40, and was logged only from the surface. Shears were observed in the mudstone at stations 0+25 to 0+30. The shears appeared to be oriented east-west. From station 0+45, the trench encountered increasingly weathered bedrock overlain by colluvium. From stations 0+50 to 0+90, the soil section appeared to be part of a shallow landslide. A landslide plane and clay seam were observed crossing the trench wall from stations 0+80 to 0+90. From station 0+90 to the end of the trench at station 1+40, the soil column contained abundant calcium carbonate stringers and pods. The bedrock was blocky and weathered, but similar in composition to the bedrock exposed between stations 0+00 to 0+45. No shears or clay seams were observed in the eastern portion of the trench from stations 0+90 to 1+40. No thrust fault was observed in the central portion of the trench cutting the overlying soils or landslide material. The soils, landslide materials and bedrock in this trench are in a proposed cut area. Mitigation of the observed hazards will be by removal of the material.

Trench T4A-2004 was excavated approximately 60 feet north of trench T4. The trench was excavated to evaluate the shear features observed in the bedrock at the western end of trench T4. Trench T4A encountered mudstone and siltstone with abundant iron and manganese staining along fractures. Clay seams were observed in the mudstone. The seams were oriented N65E, steeply dipping to the south. No shears were observed extending up into the soil. The mudstone was similar to the bedrock observed in trench T4. No thrust fault evidence was observed in the trench.

An additional fault trench was excavated across the trend of the Calaveras fault by KJM Enterprises in August 2004. Trench T5-2004 was located on the southern segment of the fault adjacent to the proposed Neighborhood D, the senior housing component of the site. The following is a summary of the observations from the fault trench.

The trench excavation encountered bedrock on the western side within the colluvial filled swale. A possible fault splay was encountered from stations 1+10 to 1+27. This possible splay appeared to be restricted to the bedrock and did not extend into the overlying soil or weathered bedrock. Fault related features include clay shears along a depositional feature, soil infill, and clast filled sedimentary features. A sand unit with alternating gray clay and red brown sand also occurred at this zone. The field interpretation was more sedimentary in origin than tectonic for many of the features. However, groundwater was encountered at this point; the rock type changed from channel sands to a silty sandstone to sandy siltstone at the contact; and minor iron rich shears were observed. The zone was interpreted as a combination of older fault features modified by subsequent erosion. The soils and bedrock are consistent from station 1+25 to station 2+30. At this station, the soil thickness increases partly due to shallow landsliding and partly due to an apparent paleoscarp. The Calaveras fault is mapped by the California Geologic Survey on the Fault Hazards Zone Maps (CDMG, 1982) at stations 2+30 to 2+60. A groundwater barrier was encountered at station 2+45. The trench to the east encountered increasingly thick colluvial soils and no groundwater to a depth of 18 feet below ground surface (bgs). The thick colluvium and depth to groundwater was confirmed in boring B2. The trench was continued to the east of the thick colluvial soil section, and encountered the main active trace of the Calaveras fault. The fault separated younger colluvial soils (most likely Holocene in age) from a Miocene age sandstone (Tmss) ridge at station 3+20. The fault trace was observed to continue to the surface, where the trace was observed as a flower structure in the overlying soil. This trace may be related to the 1861 earthquake rupture. A possible secondary fault was observed further east. The section of Tmss bedrock continued to the east, sloping northeastward. The bedrock was further faulted or possibly disrupted by landslide movement further east in the trench. The trench east of station 3+75 encountered thick colluvial soils and landslide debris.

IV. INVESTIGATIVE RESULTS

The site elevations range from 524 feet in the eastern edge of the property to 998.7 feet on the western ridge (Note: the elevations on the latest version of the base map have been corrected from the December 2002 report). The underlying geology defines the ridges and drainages on the site. The easternmost ridge is underlain by variably (southwest to southeast) dipping Briones Formation sandstone separated from the eastern ridge by the Calaveras fault, which delineates the eastern drainage. The eastern and central ridge is underlain by undivided Briones massive and fossiliferous sandstones and interbedded siltstone variably dipping to the southwest. The central drainage and western ridge are underlain by the Orinda Formation or equivalent sandstone, siltstone and claystone dipping to the southwest. The Orinda Formation has a pebble conglomerate member, which forms a separate spur off the elevation 998.7 high point on the western ridge. The general structural alignment of the bedrock, faults and landslides are shown on Figure 14, Cross sections A-A' and B-B'. The two sections for the site are oriented northeast-southwest to cross the structural grain and fault trends of the site.

IV.A. Fault Location Studies

The December 2002 investigation of the Calaveras fault was performed to identify fault evidence along mapped traces of the fault. The FER by Hart (1981), was reviewed prior to the fault investigation. The report indicated a fault trace westerly of other mapped traces shown along a linear drainage. The investigation proceeded with the evaluation of the drainage and mapped traces (Wagner, J. R., 1978; Dibblee, 1980, Hart, 1981a, 1981b; Crane, 1988; and Wagner, et al, 1981). The locations of the interpreted fault locations from the studies by Wagner (1978), Hart (1981a, 1981b), and Crane (1988) are shown on Figure 3. Photo-lineament and field evaluations are also included on the map (Figure 3).

Crane (1988) shows the Las Trampas thrust fault partially mapped within the alignment of the fault on the site, and trending westerly along the central ridge. However, as discussed in the May 2004 report, the fault trenches T4-2004 and T4A-2004 did not locate fault related features associated with the Las Trampas thrust fault. The shears observed in both trenches appeared to be related to regional deformation of the mudstone bedrock and trended east–west rather than northwest-southeast. The thrust fault mapped by Crane 1988 is an interpretation of the northern trend of the Calaveras fault. The Calaveras fault was observed in trenches located on the eastern portion of the property. The fault was observed as a strike-slip type rather than a thrust. Based upon the fault trenches, the Las Trampas thrust fault is not present on the site, as mapped. The bedrock in the trenches was similar on both sides of the mapped thrust fault location. No setback or other mitigation is required for the area related to the interpreted fault.

An additional fault trench, T5-2004, has been excavated adjacent to residential Neighborhood D. The results of the trenching indicate a possible older fault feature within the bedrock near this location. This feature appears at a contact between different bedrock materials, and may be associated with a groundwater barrier. Altered sediments and apparent past fault related features were observed at this feature in the trench. The overlying soil and sediment indicate that this feature is not active and is most likely a Miocene age feature. A groundwater barrier and distinct colluvial soil thickening were observed at the Fault Hazard Zone Map (CDMG, 1982; Hart, 1981b) location of the Calaveras fault (second fault trace, active). A landslide crosses the fault related feature at the trench location and has obscured the surface soil overlying the fault trace. East of the thick colluvial soil, a bedrock ridge was encountered in the eastern portion of the trench. The active trace (third fault trace) of the Calaveras fault was located on the western side of the bedrock ridge. The fault was observed as a distinct fault contact between dark brown clayey colluvial soils, and light reddish brown sandstone (Tmss). The fault was observed to form a flower structure into the surface soils and can be considered the active trace of the Calaveras fault.

The previous mapping efforts were reviewed and tested as part of this report. Fault traces were identified east of the mapped trace in the three trenches. Additional information from offsite parcels was also evaluated with respect to the location of the fault trace. The main trace of the Calaveras fault was encountered on site in trench T5-2004. A study by ENGEO (1978, 1983) south of Deerwood Road has located the fault trace which aligns with the fault located in trench T5-2004. A fifty foot setback is recommended for this Calaveras fault trace, and has been incorporated into the placement of buildings and lots within each of the residential neighborhoods shown on the Vesting Tentative Map. None of the proposed residential structures are located within the proposed setback zone. The FER studies indicate that additional fault traces are most likely situated east of the main trace, (further separated from proposed residential structures). Other studies by Rogers, et al, (1992) describe the Calaveras fault as further east than the FER trace. A 50-foot setback from the two locations of the active trace, from the 2002 fault investigation, and from interpreted lineaments of the Calaveras fault is shown on Figure 3. The structures shown on the Vesting Tentative Map within Neighborhood D are located outside the respective 50-foot setbacks from each of these fault traces, as shown on Figure 3.

Additional studies would be required if future modifications to the Vesting Tentative Map are proposed which would result in the placement of habitable structures within the 50-foot fault setback zone. The Neighborhood D, senior housing, is planned on the southern portion of the site within the fault rupture hazard zone, but outside of the recommended setback. The eastern access road and utilities and the eastern portion of Neighborhoods A and C, are within the western edge of the fault rupture hazard zone. The roadway crosses through the fault setback zone. However, all proposed habitable structures within these neighborhoods are outside of the 50-foot setback zone. No further mitigation is required for structures outside the setback zone.

IV.B. Characterization of Landslide and Colluvial Slopes

Eleven borings were drilled to characterize the thicker landslides located within the central valley. The original test pits, excavated in 2002, were planned to evaluate only the surface extent of the landslides and were not intended to provide deeper information at the time. Borings B9, B15, and B17 through B23, were drilled within the landslide areas to evaluate the landslide materials. The borings were continuously cored to evaluate the depth of the basal landslide plane and other deeper planes within the underlying bedrock. Geotechnical samples were also collected from the borings drilled within the landslide masses. Two additional borings, B30 and B31, were drilled within mapped landslide areas to collect geotechnical samples for laboratory testing.

Boring B9, drilled in the northwest corner of the site, encountered landslide debris to a depth of approximately 26 feet below the ground surface (bgs). A landslide plane was interpreted at 18 feet bgs and above the claystone at 26.5 feet. The boring B15 log shows a slide plane at 12.5 feet, over siltstone and conglomerate. Two borings, B17 and B18, were placed within the large 600 by 600 foot landslide interpreted by GGI. The large landslide appears as a bench on the eastern side of the central valley. The boring B17 results indicate sandy clay to 11 feet bgs over silty sand to 14 feet bgs. The upper material is interpreted as landslide debris. Boring B18 encountered mudstone at 5 feet bgs, which continued to the total depth drilled. No landslide planes were encountered. Borings B19, B22 and B23 were drilled in the northwest mapped landslide along the valley margin. Boring B19 observed landslide material to a depth of 11.5 feet bgs. The deeper claystone increased in stiffness with depth. Boring B22 and B23 encountered landslide material to approximately 29 feet bgs and 41 feet bgs, respectively. Borings B20 and B21 were drilled on the landslides mapped along the eastern central portion of the site. B20 and B21 encountered landslide debris to depths of 35 feet bgs and 19 feet bgs, respectively. Both borings found groundwater at approximately 13 to 14 feet. Boring B30 encountered colluvium and possible old landslide deposits to a depth of 17.5 feet, underlain by Tps mudstone. Boring B31 encountered colluvium/alluvium (possible landslide material) to a depth of 14 feet, over highly weathered mudstone (possible landslide material) to 24.5 feet, underlain by highly weathered in-place Tps mudstone.

The deepest identified landslide deposit was the northwestern slide with a depth of 41 feet bgs from B23. The other landslides were encountered with depths ranging from 14 to 26 feet. Except for the large complex on the northwest side, the landslide masses do not appear to be deep seated features. The extent of the landslides is shown on the Geologic Map, Figure 3. The depth of landsliding is depicted on Cross-sections A-A' and B-B' (Figure 14).

IV.C. Upslope Landslide Hazards

The general dip of the bedding observed within the test pits and field mapping, in the western portion of the property, show the formations dipping to the southwest at 25 to 42 degrees. Based upon the configuration of beds shown in the cross sections, the general bedding dip does not appear to form an adverse condition. Most of the landslides in this area appear to be shallow colluvial failures. The upslope landslide hazards will be mitigated by grading and other engineering design features as discussed in this report. Additional mitigation measures will include debris basins and berms placed along the western portion of the proposed development. The location of the proposed berms and debris basins within the western portion of the proposed development are shown on Figure 4. Engineering level design of the debris basins and berms will be finalized based on approval of the Vesting Tentative Map

IV.D. Discussion/Evaluation of Geologic and Seismic Hazards

This geologic hazards evaluation and fault investigation was conducted to ascertain the local and regional geologic conditions and to evaluate potential geologic hazards related to those identified conditions that may affect development on the Faria Preserve as shown on the Vesting Tentative Map. In general, geologic hazards include landsliding, debris flows, and the hazards associated with earthquakes. Earthquake-related hazards include ground rupture along fault traces and hazards associated with ground shaking. The hazards related to ground shaking include lateral spreading, lurching, liquefaction, landsliding, ground vibration, and ridge-top cracking.

The Calaveras fault was located in the four trenches excavated along the mapped traces of the fault identified in the FER and related reports. The fault zone was found to be from 20 feet up to 100 feet wide (in the northerly end) with multiple splays. Based upon the offsets observed in the soil in trench T1-2002, the fault is considered active and capable of future rupture along the principal trace and related splays. Trench T1-2002 found multiple splays defining a pressure ridge between two major traces of the fault. The other two trenches, T2-2002 and T3-2002, encountered the fault covered by landslide material. However, one fault splay in T3-2002 appeared to have ruptured into surface soil. Trench T5-2004 encountered multiple possible splays and one active trace of the Calaveras fault. The trench found a 50 foot wide zone with a groundwater barrier, which is aligned with the FER mapped trace of the Calaveras fault. The active trace may be related to the 1861 earthquake rupture. Therefore, the potential for surface rupture along the trace of a fault at the eastern portion of the site is considered to be high. Air photo interpretation of the site did not reveal other photo lineaments suggestive of a fault trace on the western portions of the study area. Linear features observed in the photo were observed to be related to bedding in the rock units. Development as called for in the Vesting Tentative Map may be carried out subject to the mitigation measures outlined in the proceeding sections of this report.

The proposed development will occupy areas that are underlain at shallow depth by interbedded siltstone and claystone of the Orinda Formation, which is prone to landslides. The claystone and siltstone members of the other bedrock units (Tbr and Tmss) are also prone to landslides. Debris flows are also common in the siltstone and claystone. Multiple landslides were mapped on the site and shown to occur in the siltstone and claystone. Air photo interpretation also located many subdued landslides not visible during current field work. Debris flows were mapped on the western ridge and portions of the eastern ridge from the air photos. Therefore, the potential for landslides and debris flows is considered to be high. Risks associated with these features will be mitigated through site grading and related measures discussed in this report.

Ground vibration is a potential hazard accompanying all earthquakes to a varying degree and can damage or destroy inadequately designed structures. Future earthquakes on the San Andreas, Hayward, and Calaveras Faults will probably produce ground shaking at the site comparable to at least that produced by the 1989 Loma Prieta and 1984 Morgan Hill earthquakes. Those earthquakes caused ground shaking equivalent to a modified Mercalli Intensity IX in the vicinity of the site. Damage due to ground shaking of this intensity can be mitigated by designing to current building code standards and construction in strict accordance with approved plans and details.

Lurching and lateral spreading were observed in areas flanked by unsupported faces, such as creek channels, that exposed relatively loose and unconsolidated sediments following the 1906 San Francisco earthquake (Lawson, 1908) and the 1989 Loma Prieta earthquake (Plafker and Galloway, 1989). Lurching produces fracturing and irregular displacement of the ground surface that is sometimes associated with eruptions of sandy or muddy water jetting from the fractures. Lateral spreading is a predominantly horizontal failure of loose, unconsolidated sediments that are displaced towards an unsupported face such as a river or creek bank. These types of ground failure are associated with unconsolidated sediments and a near-surface groundwater table. The present drainage swales in the east and west portions of the site could experience lateral spreading and continuation of landslides during an earthquake in the absence of grading and related mitigation as shown on the Vesting Tentative Map and discussed in this report. Liquefaction typically occurs in areas underlain by fine to medium grained, well-sorted, saturated sands. Most of the site is underlain by bedrock or moderately consolidated cohesive sediments.

Ridge-top cracks are a phenomenon that occurred at many sites throughout the general "Summit" area near State Highway 17 in the Santa Cruz Mountains as a result of ground shaking during the Loma Prieta earthquake. The effects of topography on relative ground shaking intensity and resultant ground surface disturbance and structural damage was noted in the Santa Cruz Mountains after the 1906 San Francisco Earthquake (Lawson, 1908) and the 1989 Loma Prieta earthquake (Plafker and Galloway 1989). Sites located on ridge tops underlain by sedimentary rocks were particularly susceptible to this phenomenon during the Loma Prieta earthquake due to both topographic focusing of earthquake pressure waves and regional uplift in the general vicinity of the Santa Cruz Mountains Summit area (Plafker and Galloway 1989). The origin of the cracks is complex, and may have been caused in part by large-scale lateral spreading in the relatively soft Tertiary sedimentary rocks in the region (Plafker and Galloway 1989). The topographic effects of ground shaking and high level of ground cracking and structural damage after the Loma Prieta earthquake has been studied at Robinwood Ridge, approximately 7.5km north-northwest of the epicenter (Hartzell, et al, 1994). The study by Hartzell, et al (1994) concluded that the apparent amplification of ground shaking is a complex interaction of seismic and topographic conditions that cannot be quantified with existing data. The subject site is located on terrain of topographic relief generally comparable to that impacted by ridgetop fractures in the Santa Cruz Mountains during the Loma Prieta earthquake. However, the site is underlain at shallow depth by older sedimentary bedrock and the topography will be modified in development areas by grading. It is our opinion that the potential for ridge-top cracking ground failure will be limited to those areas outside of planned development shown in the Vesting Tentative Map, and will not adversely affect the structures or utility improvements as proposed, including roadways and water tanks.

Estimates of ground response characteristics at the site and vicinity at this site suggest that high peak accelerations can be expected during a moderate to major earthquake on the Hayward, Calaveras, or San Andreas faults. The duration of shaking and the frequency component of the vibrational waves will depend upon the magnitude and duration of the earthquake. Structures should be designed to accommodate seismic vibrations and be designed in accordance with the guidelines adopted by the most recent Uniform Building Code. The project design engineer will evaluate the adequacy of minimal seismic design criteria of the current UBC for the proposed development. Compliance with UBC standards will serve to mitigate for this potential risk.

Finally, this fault investigation included an evaluation of the existence of the potential thrust fault as mapped by Crane (1988). The geomorphic evidence from the maps suggested a possible thrust fault associated with the eastern portion of the hills. However, the bedrock observed in trench T4-2004 was similar in composition across the mapped fault trace. As documented in the preceding section of this report, no thrust fault related features were observed in the trenches. Landslide related features were observed in the area of the mapped thrust fault. We therefore have concluded that the thrust fault originally mapped by Crane does not exist on the subject site, and that no separate mitigative measures are necessary to address this issue.

V. SUMMARIZED GEOLOGIC CONCLUSIONS

The following conclusions are based on the data acquired and analyzed during the course of this geologic hazards evaluation.

- The proposed development is underlain by a thin layer of topsoil and at shallow depth by dense bedrock along the ridges, and by thick colluvium and landslide material in the drainage channels. The central valley and flanks are underlain by mudstone and claystone of the Orinda Formation equivalent (Tps). The mudstone and claystone are more prone to landsliding than the sandstone forming the ridges. The eastern portion of the site includes mudstone with the Miocene Briones equivalent sandstone (Tmss). Most of this formation encountered was stable sandstone, although a portion of the formation adjacent to the fault was observed to be mudstone and landslide prone. The potential for landsliding on the portion of the site currently planned for residential development would be considered moderate to high; however, this potential for landsliding is mitigated by corrective grading as proposed in the Vesting Tentative Map. It is our opinion that the construction of the proposed development on the site will not exacerbate existing geologic conditions at the site if the recommendations presented in the design level geotechnical engineering studies by ESCNC are implemented during the design and construction of the project. This opinion will be further amplified through review of the construction level improvement plans.

- The potential for ground rupture along the herein mapped trace of the Calaveras fault is high, based upon the findings of the ENGEO (1978, 1983) reports and the results from trenches T1-2002 and T5-2004. The active trace was encountered in Trench T5 that matches traces interpreted as the fault in trenches T1-2002, T2-2002, and T3-2002. A 50-foot setback should be suitable for the fault zone. The proposed development, as shown in the Vesting Tentative Map, complies with this recommended setback. There is a significant potential for strong to very strong ground shaking at the site as a result of an earthquake on one of the active faults in the San Francisco Bay Area. A moderate to major earthquake on the Calaveras or Hayward fault, or a major earthquake on the San Andreas fault, could cause severe ground shaking at this site. This potential risk is, however, mitigated through implementation of measures presented in this report.

- As discussed in the body of this report, a portion of Neighborhoods C and D extend into the Fault Rupture Hazard Zone, originally designated the Alquist-Priolo Special Studies Zone. Only Neighborhood D is affected by the 50-foot setback from the Calaveras fault. Neighborhood A is west of the 50-foot setback. ESCNC has carefully examined the location of proposed buildings and improvements within neighborhood D as shown on the Vesting Tentative Map, and concluded that they may be implemented subject to the mitigation measures proposed herein. No portion of the proposed buildings are in conflict with the fault trace setback as proposed.

- The eastern entrance roadway and related utility improvements will cross the Fault Rupture Zone within the southeasterly corner of the site. These facilities will be engineered to include additional protective features to mitigate risks associated with the Calaveras fault.

- The supplemental fault trenches (T4-2004 and T4A-2004) excavated across the Las Trampas thrust fault, as mapped by Crane (1988), indicate that no fault trace exists within the eastern portion of the Faria Preserve. This conclusion is supported by shears observed in both trenches related to regional deformation of the mudstone bedrock, which trended east–west rather than northwest-southeast. No thrust fault exists as mapped within the eastern portion of the site. No setback or other mitigation is required with respect to this issue.

- The landslide observed within trenches T4-2004 and T4A-2004 appears to be buttressed by colluvium and bedrock. This slide is in an area of proposed cut and will be removed during grading.

- The potential for lurching and lateral spreading due to strong ground shaking is considered moderate in the drainages and will be mitigated by removal and replacement. The potential for liquefaction is considered low. Geo-2d

- Most of the soils across the site, and much of the bedrock (especially the Orinda Formation claystone) are highly expansive. Recommended grading procedures will offset potential impacts associates with expansive soil.

VI. PRELIMINARY GEOTECHNICAL ENGINEERING STUDY

This report section presents the results of our preliminary geotechnical engineering investigation for the proposed development and related construction for the property. Preliminary grading plans have been reviewed as part of the Vesting Tentative Map for the Faria Preserve and the proposed grading scheme is shown in Figure 4.

VI.A. Purpose and Scope

The purpose of this geotechnical engineering study was to identify and evaluate the geologic and soil conditions at the site in relation to the proposed multi-use development. Conclusions in this report are based on data acquired and evaluated from this study. Recommendations are made for minimizing the observed and potentially adverse geotechnical conditions, relating to slope stability, static and dynamic settlement, and rippability. Emphasis was placed on providing the engineering characteristics and behavior of the graded product as discussed in consultation with the specific plan EIR sub-consultants Treadwell and Rollo. The report presents recommendations for site development, drainage, grading, and a generalized discussion of foundations systems for the development. This report does not contain design level recommendations for the residential, multi-family, place of worship, or educational use developments, or the detailed public works structures required on the site.

The geotechnical study included the following:

1. Review of geological maps and reports pertinent to the area.
2. Drilling of test borings and sampling of native soils and rock materials.
3. Laboratory testing of collected soil and rock materials.
4. Engineering analyses of accumulated data.
5. Consultation with the owner's representatives and the project design professionals.
6. Preparation of this geotechnical report section with appropriate graphics.

VI.B. Field Reconnaissance: Investigative Procedures

Reconnaissance of the site was performed by a registered Geotechnical Engineer and a Certified Engineering Geologist on many occasions between October 2002 and October 2004. The site was examined for evidence of landsliding and slope instability, spring activity and general soil and bedrock conditions.

The general land form consists of sub-parallel ridges and valleys. The ridges are flanked by landslides and colluvial deposits. Alluvial and colluvial deposits were mapped along the canyon floors and at the mouth of the canyons where the streams flow out onto the edge of the valley floor. Bedrock exposures are limited to scattered outcrops of the more resistant sandstone bedrock. Numerous landslides of varying size, geometry and apparent age are present throughout the canyon areas of the site.

Based on the site visits, study of aerial photography, topography, existing soil conditions and the proposed grading concept, a program of field exploration was developed. This program consisted of the drilling of test borings and the collection of subsurface samples. The data from the test pits and fault trenches, excavated by ESCNC, Wahler & Associates, and Berloger Geotechnical Consultants, for previous geologic studies of the site, were also utilized in this study.

VI.C. Drilling and Sampling: Investigative Procedures

A drilling program consisting of 31 test borings was performed to obtain relatively undisturbed representative samples of the soil and rock materials found on the site for laboratory testing, and to collect other data pertaining to the subsurface soil and bedrock conditions. A description of the drilling program and logs of the test borings, which show the depths and descriptions of the soil and bedrocks encountered and the vertical locations of the samples that were obtained, are presented in Appendix C. The approximate locations of the test borings are shown on Figure 3.

VI.D. Laboratory Testing: Investigative Procedures

The laboratory testing program was planned to determine some of the physical and engineering characteristics of the soil and rock materials that may be encountered and/or used during construction of the project. These tests included moisture content and density determination, grain size analyses, Atterberg Limits, swell tests, direct shear tests, laboratory compaction, consolidation, and triaxial compression tests. A description of the laboratory testing program and summaries of the results are presented in Appendix D.

The laboratory testing program did not include testing for the corrosion potential of the soils, sulfate contents of the soils, or the presence of toxic or hazardous materials that may or may not be present in the site soils.

VI.E. Engineering Analysis and Evaluation Procedures: Investigative Procedures

Engineering analyses of the collected field and laboratory data was undertaken to provide recommendations pertinent to the design and construction of the proposed hillside development. These analyses included analysis of test boring, test pit and fault trench logs for site preparation recommendations and landslide evaluation; evaluation of the direct shear and triaxial compression data for slope stability analyses; analysis of consolidation test data and triaxial compression data for determination of settlement potential of in-situ soils under fill loads; analysis of the swell and consolidation test data to evaluate the behavior of the fill materials as a result of the environmental changes from percolating irrigation water following development; evaluation of groundwater data for subsurface drainage recommendations; and evaluation of test boring logs, shear strength data, swell data and Atterberg Limits data for evaluation of foundation systems.

VI.F. Liquefaction Analysis: Investigative Procedures

Soil liquefaction is a phenomenon where saturated granular soils near the ground surface undergo a substantial loss of strength due to increased pore water pressure resulting from cyclic stress applications induced by earthquakes or other vibrations. In this process, the soil acquires mobility sufficient to permit both vertical and horizontal movements, if not confined, which may result in significant deformations. Soils most susceptible to liquefaction are loose, uniformly graded, fine-grained sands and loose silts with low cohesion. It is generally acknowledged that liquefaction will not occur if such deposits are located at a depth greater than 40 to 50 feet below the ground surface. In deposits at depths of more than 40 to 50 feet the greater overburden pressure is sufficient to prevent liquefaction from occurring.

The liquefaction analysis at the subject site was evaluated using the methodology suggested in the "Proceedings of the NCEER Workshop on Evaluation of Liquefaction Resistance of Soils", Technical Report NCEER-97-0022, edited by T.L. Youd, and I.M. Idriss, and the Recommended Procedures for Implementation of DMG Special Publication 117 from the Southern California Earthquake Center, University of Southern California.

The first step of liquefaction potential evaluation consisted of normalizing the SPT blow count to the effective overburden stress of 100Kpa or 1.044 tons per square foot. This is denoted $N_{1(60)}$ and is found through the following formula:

$$N_{1(60)} = N_m C_N C_E C_B C_R C_S$$

Where
N_m = measured standard penetration resistance
C_N = depth correction value $= (Pa/\sigma'_{vo})^{0.5}$ $0.4<C_N<2$
C_E = hammer energy ration (ER) correction factor
C_B= borehole diameter correction factor
C_R = rod length correction factor
C_S = correction factor for samplers with or without liners

Factor	**Equipment Variable**	**Term**	***Correction**
Overburden Pressure		C_N	$(Pa/\sigma'_{vo})^{0.5}$ $0.4<C_N<2$
Energy Ratio	Safety Hammer Donut Hammer Automatic Trip Hammer	C_E	0.60 to 1.17 0.45 to 1.00 0.9 to 1.6
Borehole Diameter	65 mm to 115 mm 150 mm	C_B	1.0 1.05

	200mm		1.15
Rod Length	3m to 4m 4m to 6m 6m to 10m 10m to 30m >30 m	C_R	0.75 0.85 0.95 1.0 <1.0
Sampling Method	Standard Sampler Sampler without liners	C_S	1.0 1.2

*Recommended Procedures for Implementation of DMG SP 117.

P_a is 100 Kpa or approximately one atmosphere of pressure in the same units used for σ'_{vo} (effective vertical stress).

CRR is obtained from the Simplified Base Curve recommended for determination of CRR from SPT for Magnitude 7.5 along with Empirical Liquefaction Data (after Youd and Idriss, 1977).

The seismic demand placed on the soil, expressed as the cyclic stress ratio, was calculated using the following equation:

$$CSR = (\tau_{av}/\sigma'_{vo}) = 0.65(a_{max}/g)(\sigma_{vo}/\sigma'_{vo})r_d$$

where a_{max} is the horizontal acceleration at the ground surface generated by the earthquake, g is the acceleration of gravity, σ_{vo} and σ'_{vo} are total and effective vertical overburden stresses, respectively and r_d is a stress reduction coefficient. The factor of safety against liquefaction was calculated using the following equation:

$$FS = (CRR_{7.5}/CSR)MSF$$

where $CRR_{7.5}$ is the cyclic resistance ratio for a magnitude 7.5 earthquake. The calculated safety factor was multiplied by a magnitude scaling factor for an earthquake with a magnitude different than 7.5. The safety factor was further corrected for confining pressures greater than 100 Kpa. Since the valley floor areas are relatively flat, the safety factor was not corrected for sloping ground. The soil layer was considered to be liquefiable if the calculated factor of safety was less than 1.1.

VI.G. Slope Stability: Investigative Procedures

Numerous landslides of varying size, geometry and apparent age are present throughout the site. The landslides have developed primarily in the valleys, drainage swales, creases or concave depression throughout the site, in areas underlain by the finer grained siltstone and mudstone units of both the Orinda and Briones formations. The shape, size and areal distribution of the landslides do not appear to be controlled by the bedding attitude of the underlying bedrock. The landslides were identified as generally being 14 to 26 feet deep. The deepest landslide reported was for the northwestern slide with a depth of 41 feet at boring B23. With the exception of the large complex on the northwest side of the development, the landslide masses do not appear to be deep seated features and have occurred in the colluvium or highly weathered bedrock, along the contact with the underlying less weathered bedrock.

VI.H. Slope Stability Analysis: Investigative Procedures

Strength Parameters -- Samples of undisturbed bedrock from areas of proposed cut and a sample of colluvium from the valley floor were tested in the laboratory to determine their shear strength parameters using triaxial compression testing. Bulk samples of the various materials anticipated to be used as fill were tested using direct shear tests. The results of the laboratory testing are presented in Appendix D. For purposes of analysis a friction angle of 30 degrees and a cohesion of 490 psf were assumed for weathered sandstone. No undisturbed samples of the harder, less weathered sandstone were collected. Based on blow counts and the field descriptions of the materials, a friction angle of 40 and a cohesion of 200 psf were assumed for the less weathered sandstone at depth. For valley sediments, a friction angle of 25 degrees and a unit cohesion of 510 psf were assumed. Analysis of the geologic structure indicated that the majority of the cut slopes will not be affected by adverse bedding or jointing. Section T-T' is the exception. For analysis of this slope, strength parameters of 27 degrees and unit cohesion of 500 were selected for potential failure surfaces parallel to the bedding planes. The fill slope faces were assumed to be constructed of the more weathered sandstone materials. For analysis a friction angle of 26 degrees with a unit cohesion of 600 psf were assumed for fill materials.

Model Development – Twenty six cross sections were developed at the locations shown on Figure 4. These sections were developed to evaluate the cut and fill grading of the site. Selected sections were chosen to perform stability analyses. Seven sections were selected for analysis, sections, F-F', O-O'. P-P', and T-T' in areas of high cut slopes, E-E' and H-H' representing high fill slopes, and section Z-Z' through the large deep seated landslide in the northwest corner of the property that extends beyond the development limits.

Sections, F-F', O-O', and P-P' are in areas of cut underlain by sandstone. No adverse bedding or jointing was identified or modeled. The sandstone was assumed to be weathered to a depth of 34 feet based on analysis of the boring logs drilled in the sandstones. The weathered strength parameters were assigned to this upper layer with the higher strength parameters assigned to the deeper, less weathered sandstone. Section T-T' was also in sandstone with the dip of the bedrock parallel to the section line. For this section the same bedrock profile was used. However, for potential failure planes parallel to the bedding (40 degrees) a friction angle of 27 degrees and unit cohesion of 500 psf were assumed.

Groundwater was assumed to be parallel to the original ground surface at the depth encountered in the nearby borings. When no groundwater was encountered, none was used in the analysis. Where the groundwater was projected to daylight on the proposed cuts, it was assumed to be mitigated by subdrains and modeled as being 10 feet below the finish ground surface.

Sections E-E' and H-H' are sections through valley fills. The fill was assumed to be constructed on the existing ground surface with a nominal 10 foot deep by 25-foot wide keyway extending into the valley sediments. Groundwater was assumed to be at a depth of 15 feet below the ground surface representing at least 5 feet of subexcavation and trenched subdrains 10 feet deep up the valley.

Section Z-Z' was modeled to evaluate the effectiveness of the proposed fill, as currently designed, on buttressing the large deep landslide located in the northwest corner of the site. The landslide mass was assigned soil strengths of 25 degrees and 510 psf. The basal slide plane was modeled as a 5-foot thick zone. The strength parameters along the identified failure surface were assigned 18 degrees and 0 cohesion.

VI.I. Settlement Analysis: Investigative Procedures

The present grading scheme calls for fills of up to 110 feet thick. Settlement analyses were performed to evaluate magnitude of settlement resulting from these deep fills. Three settlement conditions were evaluated. They include static settlements of the underlying alluvial, colluvial and landslide deposits, post development settlement of the fills resulting from hydro-consolidation, and seismically induced settlement of the fills. The settlement potential of the natural soil deposits was evaluated using laboratory consolidation test data and stress/strain data from the triaxial compression tests. The potential settlement (and heave) of the compacted fills as a result of hydro-consolidation was evaluated using one-point laboratory swell/compression tests, and integrating the resulting strains over the full depth of the fill. Dynamically induced settlement of the fills was evaluated using the procedures presented in the paper "Seismic Compression of As-Compacted Fill Soils with Variable Levels of Fines Content and Fines Plasticity" by Stewart et al (2004). Calculations of the potential settlement for each of the three conditions were performed for fill thicknesses of 25, 50, 75, and 100 feet.

VI.J. Soil and Bedrock Conditions: Investigative Results

The site is underlain by the Briones Formation, or equivalent, and the Orinda Formation, or equivalent. The Briones Formation is divided into two units, fossiliferous sandstone (Tbr) along the eastern ridge and marine wacke sandstone (Tmss), including clam shell beds, underlying a portion of the eastern valley and the central ridge. The Orinda Formation and its equivalent (Tps) are non-marine sedimentary rocks consisting of mudstone, siltstone, sandstone and pebble conglomerate. Surficial soil deposits include landslide deposits and colluvium on the flanks of the valleys and alluvial deposits along the valley floors. (See Figure 3 for the mapped bedrock and surficial soil units).

A brief description of the basic soil and bedrock units follows:

Briones Formation and Equivalent (Tbr and Tmss) -- The Briones Formation and equivalent underlies the eastern ridge, the eastern drainage, and the central ridge. The eastern ridge is underlain by very hard fossiliferous sandstone (Tbr). The eastern drainage is underlain by interbedded grey to tan wacke sandstone and siltstone, while the central ridge is comprised of yellow brown wacke sandstone with interbedded mudstone and siltstone on the eastern flank (Tmss). The weathered mudstone and siltstone are hard with unconfined compressive strengths of 4 to 4.5 tsf as measured with a pocket penetrometer. In place densities range from 100 to 111 pcf.

The sandstone on the central ridge is covered with a thin layer of colluvium. The sandstone is yellow brown in color and fine to medium grained. The upper 30 to 35 feet is weathered to a clayey sandstone with in-place densities on the order of 110 to 115 pcf. The weathering decreases with depth, with in-place densities increasing to the 123 to 129 pcf range.

Orinda Formation and Equivalent (Tps) -- The Orinda Formation underlies the west flank of the central ridge, the central drainage and the western ridge. The Orinda is comprised of siltstone, mudstone and weathered sandstone units in the valley, and a sandstone/conglomerate unit on the west ridge. The finer grained siltstone and mudstone units weather to sandy clays to clayey silts with unconfined compressive strengths on the order of 2.5 to 4 tsf, becoming harder with depth and decreased weathering. The claystone materials underlying the valley have in place densities on the order of 93 to 127 pcf, depending upon the degree of weathering.

The sandstone/conglomerate unit underlying the western ridge is very hard. There is a thin 1 foot thick layer of colluvium overlying a layer of residual soil approximately 3 feet thick, described as very stiff dark yellow brown sandy clay. The sandstone is typically dark yellow brown in color, fine to coarse grained and very hard. The conglomerate unit is gray in color with rounded to semi-rounded fine gravels.

Colluvial Deposits (Qc) -- Colluvial deposits consist primarily of dark brown and dark grey to black silty to sandy clays. The deposits vary in strength with unconfined compressive strengths on the order of 1.75 to 4.5 tsf. The strengths vary according to the location of perched water within the deposits. The in-place dry densities generally range between 99 and 110 pcf. The near surface colluvial deposits were found to be lower in density, and at the time of our field studies somewhat desiccated, with dry densities on the order of 77 to 90 pcf. These lower densities generally occurred within the top 5 feet, the most active shrink/swell zone.

Alluvial Deposits (Qal) – The alluvial deposits are generally restricted to drainage bottoms. The borings revealed that the surficial soil deposits in the drainages are generally colluvium overlying Quaternary alluvium. The thickness of alluvium varied from 4 to 21 feet, with the thickest alluvial deposits in areas where there is a confluence of drainages.

The alluvial deposits are primarily stiff sandy clays to clayey sands with random deposits of more granular soil. These finer grained alluvial soils have similar characteristics to the colluvium with in-place dry densities on the order of 108 to 118 pcf and unconfined compressive strengths of 2.75 to 4.5 tsf.

More granular alluvial deposits were encountered in three locations. A layer of relatively clean, loose sand, approximately 5 feet thick was encountered in the upper end of the central drainage (boring B8). A 4-foot layer of loose silty sand with clay binder was encountered near the southern end of the central drainage (boring B14). And a 4.5-foot thick layer of dense sandy gravel was encountered near the middle portion of the central drainage (boring B16).

Landslide Deposits (Qls) -- The landslide deposits were found to consist primarily of colluvial deposits overlying the bedrock. The deeper landslides near the middle and upper portions of the central valley also included blocks of weathered mudstone and claystone within the landslide materials.

VI.K. Groundwater Conditions: Investigative Results

Groundwater was encountered in 10 of the 31 borings, ranging from 9.5 to 75 feet below the existing ground surface. Groundwater levels were 11, 14.5, and 25 feet in three of the four borings (B8, B14, and B3, respectively) drilled for liquefaction analysis. Groundwater levels were 9.5, 13, 23, and 27 feet in the four borings (B24, B21, B20, and 28 respectively) drilled to characterize the landslide and colluvial deposits. Groundwater levels were encountered in three of the borings drilled in cut areas. The groundwater levels in these borings were 34, 37, and 75 feet (B1, B11, and B7, respectively).

Groundwater in the landslide and colluvial deposits will be controlled by subdrains under the canyon fills. It is anticipated that groundwater will daylight on some of the taller cut slopes requiring mitigation. Potential mitigation measures include finger drains, hydraugers, and gallery drains to collect the water within the slope and remove it before it daylights on the slope face.

VI.L. Soil Expansion: Investigative Results

The soils derived from the mechanical breakdown of the sedimentary bedrock materials on the site vary from low to high. Laboratory expansion tests on the more cohesive potential fill materials indicate a volume increase of between 11 and 16.5 percent when samples compacted to 92 percent relative compaction at optimum moisture content were soaked under a 144 psf confining pressure, placing those materials in the high to very high expansion category. For the more granular weathered sandstone materials the swell at 144 psf was 2.3 and 6.2 percent, placing the material, in the low to moderate category. Atterberg Limits tests on the 11 samples tested indicated Plasticity Indices between 9 and 38. Ten of the eleven tests were above 18 indicating moderately low to very high expansion potential. Loads in the range of 4500 to 8100 psf were required to prevent expansion of all of the swell test samples.

VI.M. Soil Creep: Investigative Results

Creep is the slow downward movement of surficial soils resulting from the cyclic wetting and drying of the soils with changes in the seasons and the effects of gravity on the soil mass. Soil creep appears to be occurring primarily on the steeper slopes of the property. BGC concluded that soil creep was evident on slopes steeper than 3 to 1.

VI.N. Erosion: Investigative Results

The U.S. Department of Agriculture (1977) has mapped the soils on the site as a member of the Los Osos soil series. The Los Osos series is subdivided into three groups, LhE on the western ridge, LhF in the eastern drainage and eastern ridge, and LhG in the central drainage. Erosion potential is moderate in LhE soils, moderate to high in LhF soils, and high in LhG soils where soils are bare.

In general, erosion does not appear to be a problem on slopes with good grass growth. The most prevalent erosion takes place within drainage concentrations where there is a steep gradient or change in grade, or within areas of recent landsliding where the protective vegetation has been removed. Erosion generally takes the form of undercutting of the banks and headward migration of the erosion channels. As the water deepens the gullies on the slopes, the side banks are undercut, resulting in mass movement that ranges from minor sloughing to significant landsliding. Headward migration of erosion is evident in many of the incised channels in the major drainages. The gradient of flow is stepped in these channels (i.e., minor abrupt breaks in gradient, with the active stream undercutting the step, resulting in headward migration of the channel bottom due to this undercutting operation. Erosion and gullying is not as prevalent in the minor drainages.

VI.O. Settlement Potential of Natural Soil Deposits: Investigative Results

Analysis of the laboratory consolidation and triaxial stress/strain test data indicates that the naturally occurring colluvial, alluvial and landslide deposits adjacent to, or on the floor of, the drainage channels proposed for filling are compressible under the anticipated loads imposed by fills that will be up to 110 feet in depth. The amount of such potential static settlement would vary depending on the final depth of fill and the thickness of compressible soil near the drainage floor. The thickness of soil deposits varies from approximately 10 to 50+ feet, averaging approximately 22 feet. Some of the underlying bedrock is highly weathered and will behave like a stiff soil with moderate compressibility characteristics under high vertical loads. These materials will also be potentially compressible under the higher loads. The settlement analyses indicated that under a 25 foot thick fill, total calculated settlements will be on the order of 4 to 7 inches; for a 50 foot thick fill, 8 to 12 inches; for a 75 foot thick fill, 10 to 16 inches; and for a 100 foot thick fill, 16 to in excess of 24 inches. Settlement will begin as soon as the placement of fill commences and will continue beyond the completion of mass grading operations, approximately 1 to 3 years, depending on the thickness of the compressible soil and the imposed loads. Potential static settlement will be mitigated by removal of the potentially compressible soils and replacement with compacted fill. Some additional settlement will nevertheless continue to occur as a result of other factors, as discussed below. However the settlements due to hydro-consolidation will be smaller in magnitude (1 to 5 inches) than the settlements due to consolidation of the natural soils (4 to 24 inches).

VI.P. Settlement Potential of Compacted Fills Due to Hydro-Consolidation: Investigative Results

Analysis of the laboratory consolidation/swell test date indicates that the compacted fills will have a tendency to swell near the surface and consolidate at depth when exposed to water. The magnitude of the settlement will be influenced by the degree of relative compaction and moisture content of the soil at the time of placement. Mitigation measures, including higher compaction standards and higher moisture content at time of placement, will be implemented to reduce the effects of hydro-consolidation. The test data suggests that the consolidation/swell behavior of compacted soil materials is also influenced by the clay content of the material. The analyses indicate that settlement due to hydro-consolidation of fills placed on the site to the compaction standards recommended in this report will not occur when fills are on the order of 50 to 75 feet thick. For fills between 75 and 100 feet thick, potential net settlement could be on the order of 4.5 inches (0.4 percent of total fill thickness). For fills between 100 and 115 feet thick, net settlement could be on the order of 9 inches (0.6 percent). Rigid foundation systems may be required to accommodate the anticipated settlements. See discussion in section VI.R.

VI.Q. Settlement Potential of Compacted Fills Due to Ground Shaking: Investigative Results

Analysis of the compacted fills under dynamic conditions indicates that seismically induced settlement can occur in the event of a major earthquake. Previous studies have indicated that the susceptibility of fill soils to dynamic consolidation is influenced by the soil type and relative compaction and moisture content of the fill materials at the time of construction. The recommended compation standards presented in this report will reduce the potential settlement due to a seismic event. The amount of settlement predicted by the analysis, based on implementation of the compaction standards recommended in this report, will be approximately 0.3 inches for a 25 foot thick fill; 0.75 inches for a 50 foot thick fill; 1.5inches for a 75 foot thick fill; and 2 inches for a 100 foot fill. These values are based on a nearby magnitude 6.9 earthquake with a peak horizontal ground acceleration of 0.34g. See discussion in the next section.

VI.R. Discussion of Settlement Potential: Investigative Results

The prediction of settlement of either natural deposits, such as exist on the site, or compacted fills to be constructed on the site, for a development of this type is difficult because of all the variables involved. For natural deposits, the compression characteristics vary significantly over a short distance in both the vertical and horizontal directions, such that they cannot be accurately predicted. In addition, with hillside canyon fills, the loading conditions vary laterally as well as vertically, due to variations in thickness of the fills and variations in density of the fills. The settlement calculations represent a general order of magnitude.

The potential settlement of the proposed deep compacted fills on the site would result from three independent mechanisms; consolidation of natural soils; hydro-compaction of the compacted fills; and seismically induced settlement of the compacted fills. Of these three sources of potential settlement, hydro-consolidation is the only one that has a high probability of occurring.

The potential static consolidation of the underlying natural materials will be larger than the settlement anticipated from hydro-consolidation. To mitigate this condition ESCNC is recommending removal of the thicker colluvial, alluvial, and landslide deposits to bedrock and replacement with compacted fill. The 4 to 24 inches of settlement potential would be eliminated by removal of the compressible native soils. The additional 14 to 26 feet of compacted fill would result in a potential additional 1 to 5 inches of settlement due to hydro-consolidation.

Settlement due to hydro-consolidation potentially will occur as a result of consolidation of the deeper fill soils. The effect will be an aerial settlement of the ground surface. While there could be some differential settlement due to variations in fill composition laterally, the primary cause of differential settlement would be differential fill thickness below the site. Buildings, hardscape, streets and utilities will generally settle in unison, relative to the subsurface conditions.

Seismically induced settlement will occur only in the event of a major earthquake on one of the major faults in the Bay Area. The magnitude of settlement predicted by the Stewart et al method (0.3 to 2 inches) is based on a major earthquake on the nearby Calaveras fault, an event with a low probability. The fills on the subject site will have a significant amount of cohesive fill material that will not be as susceptible to seismic consolidation.

Mitigation measures consisting of higher fill compaction standards, over-optimum moisture content and stiffened foundations have been recommended for this project. Studies have shown that these measures will reduce potential consolidations of fills and the resultant surface settlements. The potential settlement values presented in this results section are based on the assumption that these mitigation measures will be implemented during grading.

VI.S. Liquefaction Potential: Investigative Results

The results of the liquefaction analysis indicate that there is a high potential for liquefaction to occur in a loose sand deposit at a depth of 19.5 to 24.5 feet in boring B8 under present conditions. The liquefaction potential is low in the other areas evaluated. Calculations indicate that consolidation of the layer could be on the order of 1.7 inches should liquefaction occur. This area is planned for a fill approximately 50 feet thick. This additional overburden pressure will be sufficient to mitigate the liquefaction potential. The results of the liquefaction analyses are presented in Appendix E.

VI.T. Slope Stability: Investigative Results

Analysis Results -- The computer program PCSTABL6H was used to perform two-dimensional stability analyses on seven sections using the Modified Janbu method with circular failure surfaces. The sections were analyzed for both static and dynamic conditions. For the dynamic condition a pseudo-static earthquake coefficient of 0.2 was used. The results of the stability analyses are summarized in Table 3.

TABLE 3
Stability Analysis Summary
Circular Failure Surfaces
All Cases

Cross Section	Condition	Height	Factor of Safety Static	Factor of Safety Dynamic	Figure No.
E-E'	Fill Slope	152	1.8	1.1	F1/F2
F-F'	Cut Slope	140	3.0	1.8	F3/F4
H-H'	Fill Slope	105	2.1	1.2	F5/F6
O-O'	Cut Slope	158	2.4	1.3	F7/F8
P-P'	Cut Slope	114	2.8	1.7	F9/F10
T-T'	Cut Slope	98	2.2	1.2	F11/F12
Z-Z'	Buttressed Slide	n/a	1.8	1.0	F13/F14

Non-circular surfaces and anisotropic strength parameters were also used on section T-T' where geologic structure was modeled representing the bedding of the bedrock. The results are presented in Table 4.

TABLE 4
Stability Analysis Summary
Non-Circular Failure Surfaces
Section T-T'

Cross Section	Condition	Height	Factor of Safety Static	Factor of Safety Dynamic	Figure No.
T-T' Case 1	Cut Slope	98	2.5	1.4	F15/F16
T-T' Case 2	Cut Slope	98	2.3	1.3	F17/F18
T-T' Case 3	Cut Slope	98	2.3	1.3	F19/F20
T-T' Case 4	Cut Slope	98	2.5	1.4	F21/F22

Section Z-Z' was also evaluated using non-circular potential failure surfaces to evaluate whether the existing landslide buttressed by the proposed fill would fail above the buttress. The calculated factor of safety was 1.9 for the static case and 1.0 for the dynamic case (see Table 5). While the static case met the minimum criteria for a factor of safety of 1.5, the dynamic case did not meet the minimum criteria for a 1.1 factor of safety.

TABLE 5
Stability Analysis Summary
Non-Circular Failure Surfaces
Section Z-Z'

Cross Section	Condition	Height	Factor of Safety Static	Factor of Safety Dynamic	Figure No.
Z-Z'	Buttressed Slide	n/a	1.9	1.0	F23/F24
Z-Z'	Modified Buttress	n/a	2.3	1.1	F25/F26

To satisfy the minimum factor of safety under seismic conditions a buttress fill approximately 31 feet high will be required. The top of this buttress would be located approximately 45 feet south of the property line, with a 2.75 to 1 slope down to the current design pad level .The buttress would also be required to extend off site to the north. The calculated factors of safety were 2.3 for the static case and 1.1 for the dynamic case, thereby meeting the minimum requirements. The results of these additional studies are presented in Table 5.

The buttress design was also reanalyzed for circular failure surfaces with a resulting calculated factors of safety of 2.1 and 1.1, for static and dynamic cased, respectively (See Table 6).

TABLE 6
Stability Analysis Summary
Circular Failure Surfaces
Section Z-Z'

Cross Section	Condition	Height	Factor of Safety Static	Dynamic	Figure No.
Z-Z'	Buttressed Slide	n/a	2.1	1.1	F23/F24

The analyses indicated that the cut slopes modeled meet the required minimum standards of 1.5 for the static case and 1.1 for the dynamic case. For the fill slopes the critical failure surfaces pass below the fill, into the underlying weaker colluvial and alluvial soils, exiting near the toe of the slope. For the buttressed landslide in the northwest corner of the site, additional buttress fill will be required to achieve the required minimum factor of safety for the dynamic case. The computer printouts of the stability analysis results are presented in Appendix F.

VI.U. Rippability: Investigative Results

Ten borings were drilled in cut areas. The planned depth of the borings was 10 to 15 feet below the depth of the proposed cuts. As a general rule, if the formation can be drilled with a 6-inch solid flight auger with a rock bit it can be excavated. Three of the borings met refusal short of the design depth (B4, B7 and B11). These borings were located on the mid-central ridge, a spur ridge on the east side of the central ridge, and the southern portion of the western ridge respectively, and were 32, 34 and 30 feet short of design depth. Boring B10, also located on the western ridge, hit refusal ten foot short of the total depth for design. Boring B4 was drilled with an 8-inch hollow stem auger, which does not correlate well with rippability. However, borings B7, B10, and B11 were drilled with 5-inch solid stem auger, which, in our experience, correlates well with rippability. It is anticipated that most of the materials in the proposed areas will be rippable by conventional equipment such as a D-9 bulldozer with a heavy duty single tooth ripper. However, boulders should be anticipated in the deep cuts and likely (but not guaranteed) that those areas can be ripped by a D-11 bulldozer down to the depths of drill refusal (60 to 80 feet). Below that depth ripping is anticipated to be very difficult and significant oversize material will be generated. Percussion bit "boulder buster" equipment or other rock splitting techniques may be necessary to reduce the size of the boulders in these areas.

VII. SUMMARY OF GEOTECHNICAL ENGINEERING CONCLUSIONS

The following conclusions are drawn from the data acquired and evaluated during our preliminary geotechnical engineering study for the proposed mixed use development on the 290-acre Faria Preserve in San Ramon, California.

VII.A. Site Suitability

Based on the findings of the field and laboratory studies and our engineering analysis of the collected data, it is concluded that the site is suitable from a geotechnical engineering standpoint for the proposed development as shown in the Vesting Tentative Map, provided the recommendations presented in this report and subsequent geotechnical studies are implemented.

VII.B. Soil and Bedrock Conditions

The site is characterized as consisting of sandstone ridges with colluvial covered flanks and sediment filled valleys. The valleys and their flanks are generally underlain by sedimentary mudstone and claystone bedrock materials. The cuttings from the test borings indicated that the materials generated from the cut areas would vary from fat clays to gravelly sands. Large quantities of hard bedrock sandstone with a silty sand matrix will be generated in the deeper cuts. It is anticipated that the soils and the highly weathered bedrock materials can be excavated with conventional grading equipment. Below 35 feet some oversize material will be encountered. Below approximately 75 to 85 feet in the cut areas excavation will be very difficult and a significant amount of oversize rock should be anticipated, requiring more specialized excavation equipment.

VII.C. Groundwater

Groundwater levels varied from 11 to 25 feet in the some of the landslide masses and under the valley floor, and 34 to 75 feet in areas designated for deep cuts. Groundwater will likely be encountered during sub-excavation of the surficial soil deposits in the drainages and in keyways placed for valley fills. Subdrains will collect and control this groundwater following construction of the engineered fills, but there may be a need for pumping or other dewatering measures during grading. It is anticipated that groundwater will daylight on some of the deeper cut slopes. Mitigation measures include finger drains, hydraugers, and gallery drains.

VII.D. Expansive Soil

The site soils and much of the bedrock (except for the sandstone and conglomerate) are moderately to highly expansive and will require mitigation. In general this will consist of the placement of the expansive soils and bedrock materials in the deeper portions of the engineered fills. Slope faces and near surface fill materials should be derived from the less expansive sandstone cuts.

VII.E. Seismicity

The seismic history of the San Francisco Bay region indicates that it is probable that the site will be shaken by at least one major earthquake with a Richter magnitude comparable to that experienced during the 1989 Loma Prieta earthquake, and most likely by a number of earthquakes of lesser magnitude. While the U.S. Geological Survey has foregone attempts to predict the occurrence and magnitude of future earthquakes, the Working Group on California Earthquake Probabilities (2003) has estimated that there is a 62% probability that one or more major earthquakes (M_W 6.7+) will occur in the Bay Area by the year 2032.

VII.F. Ground Shaking

Moderate to severe ground shaking can be expected during the life of the project. Bedrock is shallow in the ridge areas and the site has a low characteristic site period. Seismically induced ground failure by lurch cracking and lateral spreading, is considered moderate along the banks of the drainage swales in the undeveloped areas of the site under pre-development conditions. The potential for seismically induced landsliding in the undeveloped areas adjacent to the drainages, in the deeper colluvial areas of the valley slopes, and in areas of existing unmitigated landslides is moderate to high under pre-development conditions. Site development will include grading and related measures to eliminate risks associated with ground failures.

VII.G. Landslides

The landslides and debris flows that have been identified in the geologic study will require mitigation where they are located in or near areas to be developed. According to the Vesting Tentative Map grading plan, the landslides will either be at least partially buttressed or encapsulated by the planned valley fills. The landslides in the deeper fill areas will be removed to reduce potential settlements. Landslide areas along the edges of the valley fills and in shallow cut or natural portions of the development will be removed and replaced. On the west side of the development in Neighborhood A, catchment basins and berms are recommended to contain potential debris flows that may occur on the steep upslope adjacent to the development.

VII.H. Slope Stability

Analysis of the proposed cut and fill slopes indicates that these slopes can be cut to a stable configuration as shown on the preliminary grading plan (Figure 4). Additional buttress fill will be required at the toe of the large deep seated landslide in the northwestern portion of the site.

VII.I. Settlement

Settlement due to static consolidation of the natural soils deposits in the valleys has been calculated to have an unmitigated maximum potential of 4 to 24 inches. Hydro-consolidation settlement is possible in the deeper fill areas with fills in excess of 50 to 75 feet. In these areas, settlements on the order of 0.4 to 0.7 percent of the total fill thickness are possible. Potential settlement of the fills from ground shaking has been calculated to be on the order of 0.3 to 2 inches depending on the fill thickness.

The settlement of the native soil deposits will be mitigated by removing these soils and replacing them with compacted fill. Utilizing such mitigation, the potential settlement in that zone of the fill can be reduced to 1 to 5 inches, resulting from hydro-consolidation. Mitigation measures to reduce hydro-consolidation and seismically induced settlement, include higher compaction effort, higher moisture content at the time of placement, contour grading of the underlying ground surface, and stiffened foundations to accommodate the anticipated settlements, have been incorporated into the recommendations in this report. The magnitude of settlements discussed above assume that these measures have been implemented during construction.

For design purposes, the accumulative effects of hydro-consolidation and dynamic consolidation should be considered. For fill thickness up to 50 feet, anticipated settlements will be negligible. For fills 75 feet to 100+ feet thick, dynamic consolidation and hydro-consolidation will both contribute. The following settlement factors are recommended for design of surface drainage and gravity utilities. The surface settlements are expressed as a percentage of the total thickness of the underlying fill. For fills 50 to 75 feet thick, use 0.15 percent; for fill 75 to 100 feet thick, use 0.5 percent; and for fills greater than 100 feet thick use 1 percent

VII.J. Liquefaction Potential

The liquefaction analysis indicated that the majority of the soil deposits on the floor of the valleys are clayey in nature and not susceptible to liquefaction. Potentially liquefiable soil was only found in boring B8 at the north end of the central drainage. The potentially liquefiable layer is between 19.5 and 24.5 feet below the existing ground surface. Absent mitigation, should this layer liquefy, the potential consolidation of this layer would be approximately 1.7 inches. This area will receive approximately 50 feet of engineered fill. This additional overburden pressure will eliminate the potential for this layer to liquefy in a seismic event. It is ESCNC's conclusion that liquefaction will not be a factor in the performance of the development.

VII.K. Rippability

It is anticipated that the bedrock materials on the site will be rippable to a depth of approximately 75 feet. Borings B4, B7, B10 and B11 encountered drilling refusal in bedrock at approximately that depth. The drilling refusal indicates that larger excavation equipment may not be able to remove the material from within these cuts to design level or the cut material may be too large to effectively incorporate into the fills. Heavier duty mechanical means may be required to effectively remove the lower cut areas and produce useable borrow material for incorporation into the fills.

VII.L. Foundations

Site conditions will vary significantly across the site. In cut areas the predominant surface condition will be the exposure of weathered to hard sandstone and conglomerate with good foundation bearing characteristics. Some mudstone and siltstone bedrock may be exposed. The swell potential of these expansive bedrock materials can be mitigated by removal of the near surface material and replacement with non- or low- expansive fill. In the cut areas it is anticipated that conventional concrete slab-on-grade foundations, conventional strip foundations with wood frame floors or post-tensioned slabs can be constructed.

In the valley areas, fills up to 110 feet thick will be constructed. The primary issue of fill areas in regard to foundations is the potential for differential settlement due to variable thickness of fill under the building units. Over the canyon flanks and at many of the cut/fill transition areas differential fill thickness on the order of 30 to 40 feet in a horizontal distance of 50 feet is not uncommon. The differential fill conditions can be mitigated to some degree by contour grading. However, in such steep terrain, even contour grading can be limiting. Therefore, structures to be constructed in these areas will have to be supported on stiffened foundation systems that can withstand differential settlement. Such a system would be a waffle slab foundation, a thick mat post-tensioned foundation with stiffener ribs, or a stiffened foundation with underpinning piers.

VIII. RECOMMENDATIONS

The following conclusions and recommendations are based on a review of the data obtained for the geological and geotechnical engineering study described in this report, and on geologic and geotechnical information obtained from previous studies of the area. These recommendations are for the use of the client and the project civil engineer, in preparing conceptual grading plans for the planned development. The following recommendations are intended to minimize the effects of the geologic and geotechnical concerns identified at the site. It is the opinion of ESCNC that conventional geotechnical engineering techniques may be utilized to mitigate potential problems such as slope instability, erosion, expansive soils, and seepage within the areas of proposed development.

VIII.A. General

1. The recommendations of this report are for the general grading of the proposed 290-acre Faria Preserve development as shown on the accompanying Figure 4. As the design details of the development are refined, it is anticipated that additional soil engineering studies and reports will be required for culvert crossings, the water tanks, retention basins, debris flow diversion berms, stabilization buttresses, and other improvements that will require geotechnical engineering input. In addition, supplemental foundation design studies will be required for each of the development areas as final plans are developed.

2. Site grading should be observed by a representative from ESCNC and tested, as necessary, to determine general compliance with the following recommendations. As used in this report, the term Geotechnical Engineer refers to a representative of ESCNC. In addition, it is recommended that a registered Geotechnical Engineer and Certified Engineering Geologist from ESCNC observe conditions exposed by the grading, record significant geologic features and/or changes that may be exposed and make supplemental recommendations when necessary.

3. Due to the requirements for special handling of some on-site materials, it is recommended that the aspects of mass grading be thoroughly covered in a pre-construction conference with representatives of the owner, grading contractor, Civil Engineer, and ESCNC.

4. Structures proposed for this site should be designed by a design professional familiar with the design techniques appropriate for structures that will experience strong seismic shaking and potential large differential settlements. The design factors recommended by the Uniform Building Code typically represent only minimum guidelines, and should be reviewed by the design engineer for their applicability to this project.

VIII.B. Site Development and Grading

5. Site clearing, preparation of fill areas, placement of subdrains, placement of fill and other grading operations at the site must be conducted in accordance with the following itemized recommendations and as recommended by the Geotechnical Engineer in the field. The work associated with site grading must be performed under the full-time observation of representatives of ESCNC.

6. In areas to be graded, trees, brush and debris must be removed and the resulting depressions properly backfilled in accordance with the following recommendations. Surface vegetation, topsoil and local deposits of soft or wet soils must be removed from areas to be graded. Surface vegetation, topsoil and other "unsuitable" materials may be stockpiled for possible later use in landscaping fills or, if possible, conditioned for use in other fills, with the approval of the Geotechnical Engineer. It is anticipated that, in general, the vegetative stripping will involve the upper few inches of topsoil from most areas of the site. In the swales, draws or spring areas, landslide deposits, colluvial soils or soft, wet soils may extend to depths of several feet. Deeper over-excavation should be anticipated in these areas. The actual depth of required stripping and over-excavation should be determined by the Geotechnical Engineer in the field during grading operations. If the backfill in the exploratory trenches and pits is not removed during the course of grading, these materials should be excavated and recompacted in accordance

with the field recommendations of the Geotechnical Engineer. The approximate locations of exploratory trenches are shown on Figure 3.

7. Due to the compressibility of the colluvial, alluvial soils and the landslide deposits, it is recommended that these deposits be overexcavated to bedrock to reduce the settlement potential. The subexcavated materials may be reused as engineered fill. Many of these soils will be wet and will require aeration or mixing with dryer soils before being compacted. Not used (deleted)

8. Undocumented fills should be removed and the underlying natural ground recompacted before placement of fill. Channels and gullies to be filled should be cleaned of organics and loose or wet soil before the placement of fill. Some of these channels will receive subdrains before fill operations commence. Not used (deleted)

9. The native soil and rock materials, with the exception of the organically contaminated surface soil, may be used for compacted structural fill. Imported soil proposed for structural fill should be approved by the Geotechnical Engineer. The use of cohesive soil materials will be restricted in constructing fill slopes. Refer to paragraph 14 for details.

10. Areas to receive fill, pavement sections or foundations, should be scarified to a minimum depth of 8 inches, moisture conditioned as necessary and recompacted to the specified compaction requirements. Cut pads should also be scarified, moisture conditioned and recompacted, if deemed necessary by the Geotechnical Engineer. Not used (deleted)

11. Fills placed on sloping ground (steeper than 10 to 1, horizontal to vertical) must be initiated on a base key wide enough for proper compaction or as recommended by the Geotechnical Engineer. Keys should be constructed at locations and depths as required by the Geotechnical Engineer during grading operations. As a general rule base keyways should be a minimum of 25 feet wide or one-third the height of the slope, which ever is narrower. These base keyways should be sloped a minimum of 2 percent downward, in an upslope direction, and Not used (deleted)

be founded in firm materials approved by the Geotechnical Engineer. The keyways should extend at least 5 feet into bedrock or stiff soil as determined by the Geotechnical Engineer. Anticipated keyway depths are 10 to 25 feet, depending upon the location of the key. The base of the keys should be scarified and recompacted, as specified for areas to receive compacted fill. A subdrain should be placed at the back of the keyway after recompaction.

12. Fills should be placed in thin lifts, moisture conditioned as necessary to a moisture content above optimum when compacted. Due to the presence of springs and shallow groundwater, some of the material excavated during grading may be too wet to compact and will require aeration or blending with drier material before the proper compaction can be achieved. As the fills increase in depth, they should be continuously benched into the firm natural slopes to provide a bond between the fill and the undisturbed natural ground. The benching should be carried out such that at least the upper 5 feet of all surface soil is removed and recompacted.

Not used (deleted)

13. Constructed cut slopes should be no steeper than 3 to 1 (horizontal to vertical) without the written approval of the Geotechnical Engineer. Cut slopes should be observed by the Geotechnical Engineer and Engineering Geologist to ascertain the need for stabilizing buttress grading.

Not used (deleted)

14. Slope stability analyses of the proposed fill slopes indicate that the claystone, mudstone and siltstone materials of the Orinda and Briones formation on the site would result in a marginal factor of safety if placed near slope faces. Therefore, fill slope faces and keyways should be constructed with the more granular sandstone and conglomerate materials, as determined by the Geotechnical Engineer. No significant amounts of clayey soils should be placed within 300 feet (horizontally) of the face of constructed fill slopes. The borrow site should be approved by the Geotechnical Engineer prior to the placement of fill in these slope areas.

Not used (deleted)

15. Fill slopes should be overbuilt horizontally approximately 1 foot and trimmed back to a firm surface. Track walking is not considered a suitable means of compacting loose material on the surface of the slopes. Not used (deleted)

16. The Uniform Building Code requires drainage benches for cut and fill slopes steeper than 3 to 1 (horizontal to vertical). The proposed slopes on the site will be constructed to a 3 to 1 gradient and drainage benches will not be required by the code. It has been ESCNC's experience that drainage control on 3 to 1 slopes is necessary to reduce erosion. Constructed slopes higher than 35 feet should be constructed with mid-slope benches, where feasible, to control surface drainage and debris. Not used (deleted) The drainage benches should be a minimum of 6 feet in width and should not be spaced further than 30 feet apart (vertically). Each drainage bench should contain a paved drainage swale to pick up and dispose of surface runoff. An alternative to benches and paved swales would be properly constructed and maintained concrete "J" ditches at similar vertical intervals.

17. Cut portions of cut/fill transition building pads will require over-excavation and rebuilding with compacted fill to provide a uniform support for the proposed structures. For fills up to 4 feet deep the depth of over-excavation should be equal to the thickness of the fill. Not used (deleted) For fills up to 16 feet the depth of over-excavation should be 4 feet. For fills greater than 16 feet the depth of over-excavation should be one half the thickness of the fill placed on the "fill" portion of the lot, up to a maximum of 15 feet, or as recommended by the Geotechnical Engineer. The horizontal limits of over-excavation should extend beyond the perimeter building lines a distance equal to the depth of over-excavation, or to a minimum distance of 5 feet, which ever is greater (See Figure 15).

18. When constructing compound (cut and fill) slopes, the cut portion of the proposed slope should be over-excavated from the toe of the slope to a depth of 18 inches below the pad grade Not used (deleted) of the lower lot, and for a horizontal distance of at least 12 feet into the cut slope face. It may be necessary to cut back further than 12 feet, depending on the type of compaction equipment used

to recompact the slope. The back of the over-excavated cut surface should slope back at the same inclination as the proposed finish slope surface. The over-excavated area should then be reconstructed as a buttress to complete the slope to its final design configuration. This remedial grading should be accomplished prior to placement of the normal fill for the upper lot and upper portion of the design slope.

19. As an alternative to paragraph 17, where a fill slope is to be placed above a cut slope, the key under the toe of the fill should be at least 15 feet wide and must be observed by the Geotechnical Engineer. The purpose of the key is to provide a suitable foundation for the fill. In addition, a terrace at least 10 feet wide should be established in the natural ground between the toe of the fill and the top of the cut slope. Refer to Figure 16 for schematic details of compound slope construction. Not used (deleted)

20. It is anticipated that the majority of the rock materials can be handled with conventional heavy duty grading equipment. Test borings B4, B7, B10, and B11 met drill refusal at depths of between 60 and 80 feet, approximately 10 to 34 feet short of design cut depths. These hard rock masses in the deeper cut areas may require heavy duty ripping and mechanical rock splitting. Not used (deleted)

21. Rock pieces larger than 6 inches are not recommended within the fills, except where approved by the Geotechnical Engineer. Rock pieces larger than 6 inches should not be placed in fill areas within 15 feet of finish grade. Not used (deleted)

22. The harder sandstone masses will produce large cobbles and boulder size pieces larger than 6 inches in diameter. These large rock pieces should either be stockpiled for use as rip-rap at the numerous subdrain discharge points, check dams along the recreated stream channel, or disposed of in a deep fill location to be determined by the Geotechnical Engineer. Not used (deleted)

23. The harder rock in some of the deep cut areas may make utility trench excavation difficult to achieve. It may be desirable to over-excavate roadway areas during mass grading and Not used (deleted)

replace the rock with more excavatable compacted fill to reduce the difficulty of subsequent utility line construction.

24. Scarified and recompacted natural ground should be compacted to at least 90 percent relative compaction. In general, fills comprised of clay soils should be compacted to at least 95 percent relative compaction when placed more than 40 feet below finish grade and 93 percent between 10 and 40 feet below finish grade. Clay soils should not be placed within 5 feet of finish building pad grades if possible. Clay soils between 5 and 10 feet below finish pad grade should be compacted to between 87 and 92 percent. Sandstone and conglomerate soils should be compacted to at least 95 percent regardless of their depth of placement. Soils should be compacted while at a moisture content that is a minimum 3 percentage points over the optimum values for clay soils and 2 percentage points for granular soils, in order to reduce the swell potential of expansive soils and hydro-compaction of the fill embankments.

VIII.C. Subsurface Drainage

25. Subsurface drainage will be required beneath all sidehill fills, fills that cover existing drainage swales, channels, or valleys, and in spring areas. Tributary drains will be required in areas where excess moisture is encountered or may be anticipate in the future. Adjustments in the locations, extent and total quantities of subdrains are expected to be made by the Geotechnical Engineer in the field during grading operations. The exact location of the subdrains will be determined in the field by the Geotechnical Engineer during construction to assure that maximum efficiency will be gained from the use of the system. After construction has begun, it may be necessary to provide additional subsurface drains based on conditions revealed in the field.

26. Perforated plastic drain pipes, covered with import filter material, must be placed in each draw or canyon where fill will be placed in general conformance with Figure 17. Filter rock for subdrains should be Class 2 permeable material as specified in Section 68-1.025 of the

Caltrans Standard Specification, or as otherwise approved by the Geotechnical Engineer. It is anticipated that these subdrain trenches will be between 5 and 10 feet in depth.

27. Subdrain pipe should be perforated plastic drain pipe, Type SDR 23.5 or equivalent as approved by the Geotechnical Engineer when installed at depths over 30 feet below finished grade and Type SDR 35 when installed less than 30 feet below finished grade. Laterals up to 50 feet in length should be at least 4 inches in diameter unless directed otherwise by the Geotechnical Engineer. Laterals over 50 feet in length should be at least 6 inches in diameter. Main subdrain lines should be either 8 inches or 12 inches in diameter as directed by the Geotechnical Engineer.

28. Trenches of underdrains should be excavated to a width equal to the outside diameter of the pipe plus 1 foot. The bottom of the trench should then be covered full width by 2 inches, minimum, of specified filter material and the drain pipe laid with the perforations at the bottom. The sections should be joined together with suitable couplers. Drain pipes should be installed with a minimum slope of 1 percent or greater.

29. Base keyways supporting fill embankments should be drained, where deemed necessary, with a subdrain installed in the heel of the keyway. Keyway subdrains are to consist of perforated pipe surrounded by filter rock and connected to the main subdrain line or a suitable outfall point beyond the toe of the completed fill. Keyway heel drains may be banked, trenched or a combination. Banked drains should have a base width of at least 2 feet and a minimum height against the bank of 5 feet. See Figure 17 for schematic subdrain details.

30. The subdrains should be connected directly, at their lower ends, to either a storm drain system or to an approved discharge facility where the pipe daylights near the toe of the fill slopes. Where subdrain pipes discharge at daylight points, the pipe should be fitted with an internal screen to prevent the intrusion of rodents and the discharge point protected with a

dissipater system to retard discharge flows and prevent erosion or disturbance of the outer end of the pipe.

31. Due to the large elevation differentials across the development and the anticipated use of granular material as backfill, it is recommended that the underground utility trenches (joint trench) be properly drained to prevent the accumulation of irrigation and rain water in the granular backfill. This can be accomplished by providing periodic filter rock filled drainage connections between the utility lines and the storm drain system.

Not used (detail)

VIII.D. Surface Site Drainage and Slope Protection

32. Surface drainage control must be provided throughout the completed project to protect the future stability of roadways and slopes. The site should be graded to provide positive removal of surface water away from the tops of grades slopes, and away from areas of identified landsliding. Surface drainage should be properly intercepted and discharged into appropriately designed facilities to avoid uncontrolled flow.

Not used (detail)

33. Drainage swales and lined interceptor ditches should be provided on or above constructed slopes to divert surface runoff water away from the top edges of slopes and into the general storm drain system. Lots should be graded with positive slopes and drainage swales to provide for a rapid and positive removal of rain and irrigation runoff.

Not used (detail)

VIII.E. Diversion Berm Construction

34. The steep slopes along the west side of Neighborhood A are prone to sloughing and debris flows. It is recommended that a diversion berm be constructed along the edge of the subdivision to collect landslide materials that may come off the slope in the event of a slope failure. The berm should be at least 15 feet high. The channel width at the bottom of the basin should be 10 feet minimum. The crest of the berm should be at least 8 feet wide. The side slopes of the berm may be constructed at a gradient of 2 to 1 (horizontal to vertical). The basin bottom should slope at a 2 percent gradient to a proper discharge point.

– berm Geo-4

– Not used (detail)

VIII.F. Building Clearances From Ascending Slopes

35. Building clearances from adjacent ascending slopes (upslope conditions) should conform to Section 1806.4.2 and Figure 18-I-1 of the 1997 Uniform Building Code (UBC). Not used (detail)

VIII.G. Foundation Setbacks From Descending Slope Surfaces

36. Foundation setbacks from descending slope surfaces should conform to Section 1806.4.3 and Figure 18-I-1 of the 1997 UBC. Where the adjacent descending (downslope condition) slope exceeds a height of 15 feet, the footings should be set back or the footings deepened such that a minimum horizontal distance equal to one-third the slope height, up to a maximum setback of 40 feet, is maintained between the outside bottom edges of the footings and the slope face. Where the adjacent descending slope is less than 15 feet high, a minimum horizontal clearance of 7 feet should be maintained between the outside bottom edges of the footing and the slope face. This can be accomplished with such foundation elements as deep strip footings or drilled piers. Not used (detail)

IX. LIMITATIONS AND UNIFORMITY OF CONDITIONS

1. The recommendations of this report are based upon the assumption that the soil conditions do not deviate from those disclosed in the subsurface explorations. If any variations or undesirable conditions are encountered during construction, Earth Systems Consultants Northern California should be notified so that supplemental recommendations can be given.

2. This report is issued with the understanding that it is the responsibility of the owner or his representatives to see that the information and recommendations contained herein are called to the attention of the other members of the design team (engineer and architect) for the project and are incorporated into the plans, and that the necessary steps are taken to see that the contractors and subcontractors carry out such recommendations.

3. The findings of this report are valid as of the present date. However, changes in the conditions of a property can occur with the passage of time, whether due to natural processes or to the works of man, on this or adjacent properties. In addition, changes in applicable or appropriate standards may occur, whether they result from legislation or the broadening of knowledge. Accordingly, the findings of this report may be invalidated, wholly or in part, by changes outside of our control. Therefore this report is subject to review by Earth Systems Consultants Northern California after a period of three (3) years has elapsed from date of issuance of this report.

4. The body of the report specifically recommends that Earth Systems Consultants Northern California be provided the opportunity for a general review of the structure plans and specifications for this project, and that Earth Systems Consultants Northern California be retained to provide observation and testing services during construction. The validity of the recommendations of this report assumes that Earth Systems Consultants Northern California will be retained to provide these services.

5. This report was prepared upon your request for our services, and in accordance with currently accepted geotechnical engineering practice. No warranty based on the contents of this report is intended, and none shall be inferred from the statements or opinions expressed herein.

6. The scope of our services did not include any determination of soil corrosion potential nor environmental assessment or investigation for the presence or absence of wetlands or hazardous or toxic materials in the soil, surface water, groundwater or air, on or below or around this site. Any statements in this report or on the soil boring (test pit) logs regarding odors noted or unusual or suspicious items or condition observed, are strictly for the information of our client.

REFERENCES CITED

Association of Bay Area Governments, 1995, ON SHAKY GROUND – CITY MAPS, CITIES OF SOUTHEAST SAN JOSE, Publication Number P925002EQK-SC-9

Berlogar Geotechnical Consultants, 1991, GEOTECHNICAL INVESTIGATION FARIA PROPERTY, Contra Costa County

Blake, T., 2000, Computation of 1997 Uniform Building Code Seismic Design Parameters, UBCSeis, Version 1.03

Blake, T.F., 1989-2000, EQFAULT, V 3.0, A Computer Program for the Deterministic Prediction of Peak Horizontal Acceleration from Digitized California Faults.

Campbell and Bozorgnia, 1994, ESTIMATION OF PEAK HORIZONTAL GROUND ACCELERATION FROM DIGITIZED CALIFORNIA FAULTS, from EQFAULT computer program.

CDMG, 1982, SPECIAL STUDIES ZONE MAP- DIABLO QUADRANGLE in CGS CD 2000.

Crane, R.C., 1988, Diablo Quadrangle, from 1988 NCGS Field Trip Guide to the Geology of the San Ramon Valley and Environs.

Dibblee, T. W., 1980, PRELIMINARY GEOLOGIC MAP OF THE DIABLO QUADRANGLE, ALAMEDA AND CONTRA COSTA COUNTIES, CALIFORNIA; U.S.G.S. Open File No. 80-546.

ENGEO, 1978, PARCEL OFF OLD CROW CANYON ROAD, SAN RAMON, CALIFORNIA; AP 879 report for FER, dated October 25.

ENGEO, 1983, ADDENDUM TO AP REPORT- PARCEL OFF OLD CROW CANYON ROAD, SAN RAMON, CALIFORNIA; AP 879 report for FER, dated March 15.

Graymer, R.W., Jones, D.L., and Brabb, E.E., 1994, PRELIMINARY GEOLOGIC MAP EMPHASIZING BEDROCK FORMATIONS IN CONTRA COSTA COUNTY, CALIFORNIA: Derived from Digital Database; U.S.G.S., Open File Report No. 94-622.

Hall, Timothy N., Andrei M. Sarna-Wojcicki, and William R. Dupre, 1974, FAULTS AND THEIR POTENTIAL HAZARDS IN SANTA CRUZ COUNTY, CALIFORNIA, USGS Map MF-626, Scale 1:62,500

Hart, E. W., 1981a, CALAVERAS, PLEASANTON AND SHERBURNE HILLS FAULTS, DIABLO QUADRANGLE: CDMG Fault Evaluation Report FER-110

Hart, E.W., 1981b, EVIDENCE FOR RECENT FAULTING, CALAVERAS AND PLEASANTON FAULTS, DIABLO AND DUBLIN QUADRANGLES, CDMG Open File Report 81-9 SF, Sheet 1 of 2.

Hart, E.W., 1984, EVIDENCE OF SURFACE FAULTING ASSOCIATED WITH MORGAN HILL EARTHQUAKE 0F APRIL 24, 1984, in The 1984 Morgan Hill Earthquake, California Division of Mines and Geology Special Publication 68, 271 p.

Hart, E.W., 1997, FAULT RUPTURE HAZARD ZONES IN CALIFORNIA, California Division of Mines and Geology Special Publication 42, 24 p.

Hartzell, S.H., Carver, D.L., and King, K.W., 1994, INITIAL INVESTIGATION OF SITE AND TOPOGRAPHIC EFFECTS AT ROBINWOOD RIDGE, CALIFORNIA, Bulletin of the Seismological Society of America, Vol. 84, No. 5, pp. 1336-1349, October 1994.

Lawson, A. C., 1908, THE CALIFORNIA EARTHQUAKE OF APRIL 18, 1906 - REPORT OF THE STATE EARTHQUAKE COMMISSION, two volumes, Carnegie Institute of Washington.

Nilsen, T., 1975, PRELIMINARY PHOTOINTERPRETATION MAP OF LANDSLIDE AND OTHER SURFICIAL DEPOSITS, DIABLO 7.5 MINUTE QUADRANGLE, ALAMEDA AND CONTRA COSTA COUNTIES, CALIFORNIA; USGS Map 75-277-14

Plafker, G., and Galloway, J.P., 1989, LESSONS LEARNED FROM THE LOMA PRIETA, CALIFORNIA, EARTHQUAKE OF OCTOBER 17, 1989, U.S. Geological Survey Circular 1045, p. 48

Ploessel, M.R., and Slossen, J.E., 1974, REPEATABLE HIGH GROUND ACCELERATIONS FROM EARTHQUAKES, California Geology, v. 27, no. 9., p. 195-199.

Rogers, J.D. and Halliday, J.M, 1992, EXPLORING THE CALAVERAS-LAS TRAMPAS FAULT JUNCTION IN THE DANVILLE-SAN RAMON AREA in Borchardt, G. and others, eds. Proceedings of the Second Conference on Earthquake Hazards in the Eastern San Francisco Bay Area, CDMG Spec. Pub. 113, p. 261-270.

Steinbrugge, Karl V., et al, 1987, EARTHQUAKE PLANNING SCENARIO FOR A MAGNITUDE 7.5 EARTHQUAKE ON THE HAYWARD FAULT IN THE SAN FRANCISCO BAY AREA, California Division of Mines and Geology, Special Publication 78.

Stewart, Jonathan P., Whang, Daniel H., Moyneur, Matthew, and Duku, Pendo, 2004, SEISMIC COMPRESSION OF AS-COMPACTED FILL SOILS WITH VARIABLE LEVELS OF FINES CONTENT AND FINES PLASTICITY, Earthquake Damage Assessment and Repair Project, CUREE Publication EDA-05.

Stover, Carl W., 1984, INTENSITY DISTRIBUTION AND ISOSEISMAL MAP FOR THE MORGAN HILL, CALIFORNIA, EARTHQUAKE OF APRIL 24, 1984, in Bennett, J.M., and Sherburne, R.W., eds., "The 1984 Morgan Hill, California Earthquake", California Division of Mines and Geology, 1984, Special Publication 68.

Toppozada, T.R., Real, C.R., and Parke, D.L., 1981, PREPARATION OF ISOSEISMAL MAPS AND SUMMARIES OF REPORTED EFFECTS FOR PRE-1900 CALIFORNIA EARTHQUAKES, California Division of Mines and Geology, Open File Report 81-11 SAC.

United States Department of Agriculture (USDA), issued September 1977, SOIL SURVEY OF CONTRA COSTA COUNTY, CALIFORNIA

Treadwell & Rollo, 2004, PEER REVIEW REPORT " Geologic Hazards and Preliminary Geotechnical Engineering Investigation" Lands of Faria, San Ramon, California; prepared for EDAW, dated June 29.

Wagner, J.R., 1978, LATE CENOZOIC HISTORY OF THE COAST RANGES EAST OF SAN FRANCISCO BAY- Geologic Map of the Dublin 7.5 minute Quadrangle; Ph. D. thesis U.C. Berkeley.

Wagner, Bortugno and McJunkin, 1991,GEOLOGIC MAP OF THE SAN FRANCISCO- SAN JOSE QUADRANGLE: 1:250,000; California Division of Mines and Geology Regional Geologic Map Series.

Wahler Associates, 1987, PRELIMINARY SOILS & GEOLOGIC INVESTIGATION FARIA RANCH, Development- San Ramon, Contra Costa County, California

Working Group on California Earthquake Probabilities, 2003, PROBABILITIES OF LARGE EARTHQUAKES IN THE SAN FRANCISCO BAY REGION, CALIFORNIA, U.S. Geological Survey Digital Publication. Fact Sheet 039-03, by; A, J. Michael, S.L. Ross, R.W. Simpson, M.L. Zoback, D.P. Swartz, M.L. Blanpied, and Working Group 2002.

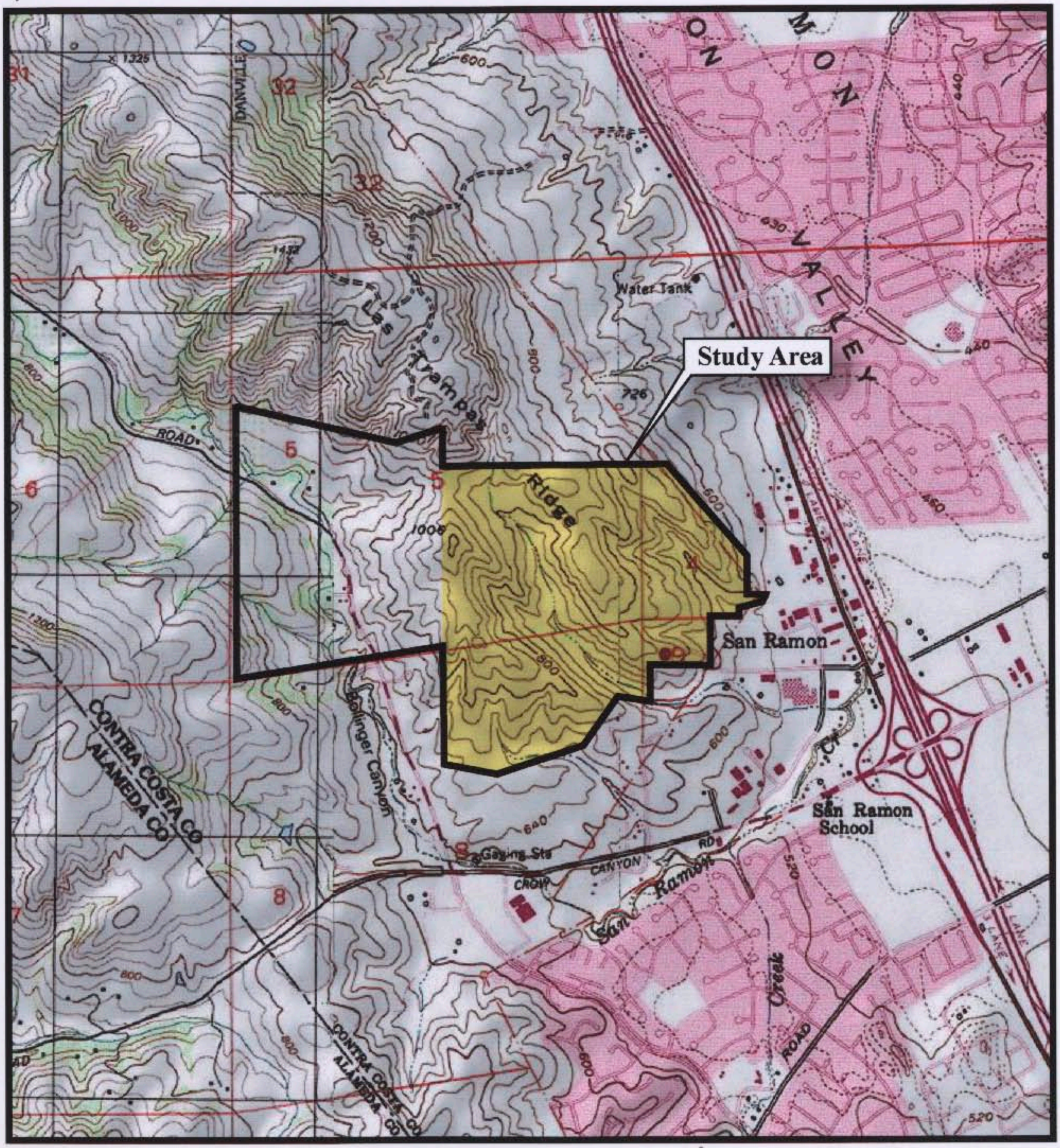
AERIAL PHOTOGRAPHS

Date	Type	Scale	Reference	Source
3/7/58	B/W	1:36,000	SF Area 3-61	Pacific Resources
7/2/59	B/W	1:12,000	AV-334 22 24, 25	Pacific Aerial
12/17/71	B/W	1:12,000	AV-101-01-04, 05	Pacific Aerial
11/14/94	B/W	1:12,000	CC AV 4625 22 30, 31	Pacific Aerial
6/27/02	B/W	1:12,000	CC AV 8202 21 28, 29	Pacific Aerial

FIGURES

File No. FRG-3379-03
October 2004

TN MN
15°



N

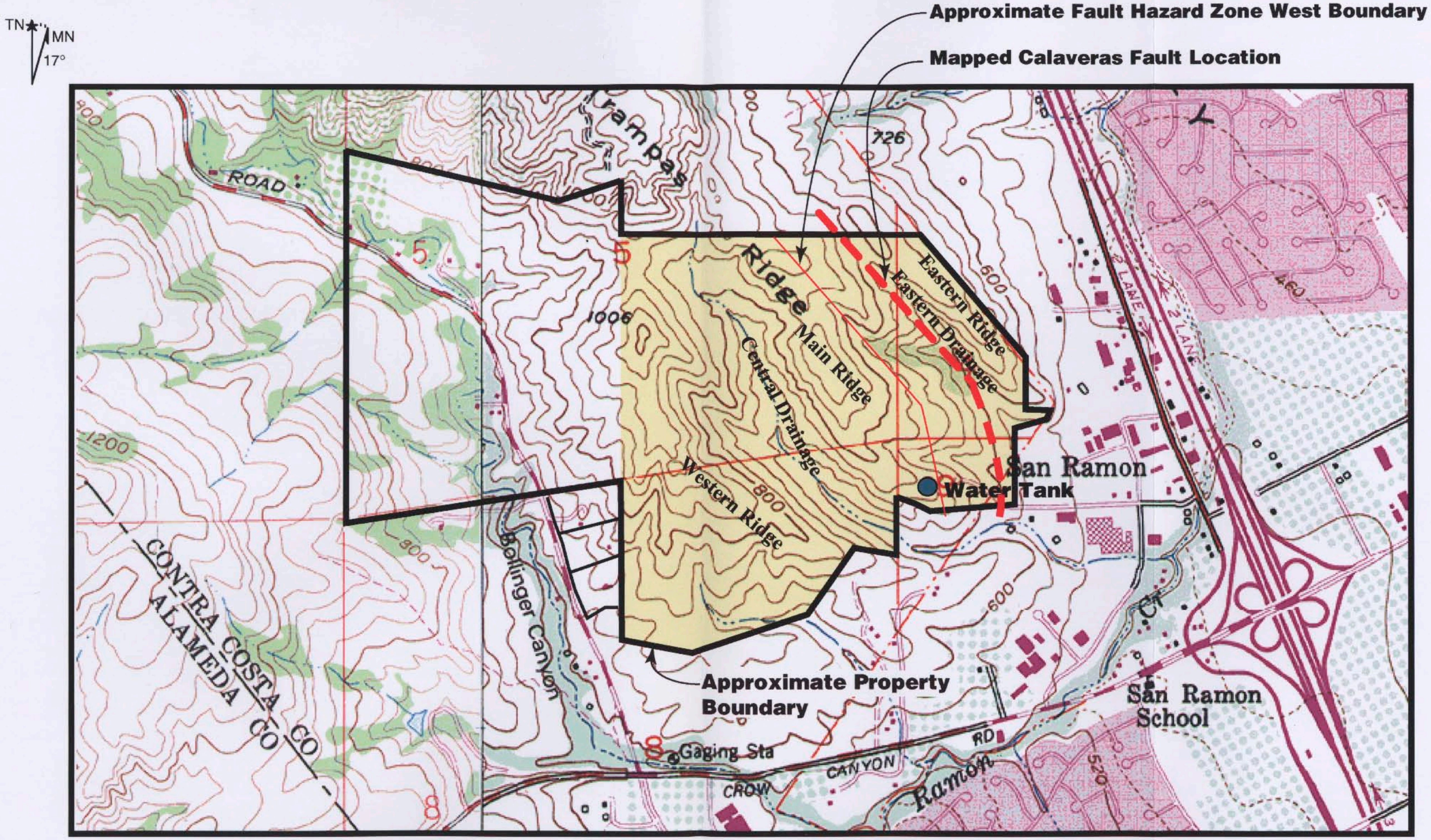
Approximate Scale 1: 24,000

1 0 1 Mile

1 0 1 Kilometer

Base: U.S.G.S. 7.5 minute Diablo, (1980) Quadrangle
Map created with Topo!® © 2003 National Geographic (www.nationalgeographic.cm/topo)

Earth Systems Consultants **Northern California**	Lands of Faria San Ramon, California	**STUDY AREA LOCATION** **Figure 1**



Study Area (This Report)

0 1/2 1

Approximate Scale in Miles

Base U.S.G.S. 7.5 Minute Las Trampas & Diablo Quad Topographic Map
Printed from TOPO!™ ©1995 Wildflower Productions
(415) 282-9112 TopoSF@aol.com

Earth Systems Consultants Northern California	Lands of Faria San Ramon, California	**STUDY AREA**
		Date: October 2004 File No: FRG-3379-03 — Figure 2