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GEOTECHNICAL REPORTS

FEASIBILITY-LEVEL GEOLOGIC HAZARDS ASSESSMENT ARCADIA PROPERTY SAN JOSE, CALIFORNIA

Prepared For: DAVID J. POWERS & ASSOCIATES
1885 The Alameda, Suite 204
San Jose, California 95126
Attention: Mr. Hesler

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June 7, 2005

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DAVID J. POWERS & ASSOCIATES
1885 The Alameda, Suite 204
San Jose, California 95126

SUBJECT: Feasibility-Level Geologic Hazards Assessment for the Arcadia Property, San Jose, California

Mr. Hesler:

We are pleased to submit our geologic hazards assessment report for the Arcadia Property located in San Jose, California. The accompanying report summarizes the results of our data review, field investigation, and geologic interpretation.

This feasibility-level geologic hazards assessment report describes the geologic setting, faulting, seismicity, geologic site characterization, geologic and seismic hazards, and near-fault issues associated with the site. The primary geologic hazard considerations in this assessment include surface fault rupture, seismic shaking, liquefaction, dynamic compaction, landslides, seismically induced ground failure, erosion, naturally occurring asbestos, and flooding. Conclusions regarding potential impacts of these geologic hazards are provided in the report.

The primary geological issue of concern is the susceptibility for strong ground shaking as a result of future seismic events along one of the Bay Area active earthquake faults. Based on our assessment, it is our opinion that the site is geologically suitable for development provided that the conclusions and recommendations presented herein are adhered to and incorporated in the design and construction of the proposed development. This report recommends that a site-specific design-level geotechnical investigation be undertaken to provide grading, foundation, drainage, and pavement design recommendations. The State of California requires that the school sites and essential service building be specifically evaluated for geologic hazards in addition to this study that covers the entire property. If you have any questions regarding the information or recommendations presented in our report, please contact us at your convenience.

Sincerely,

KLEINFELDER, INC.

Michael Clark, C.E.G. 1264
Senior Engineering Geologist

Mark Swank
Staff Geologist

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FEASIBILITY-LEVEL GEOLOGIC HAZARDS ASSESSMENT ARCADIA PROPERTY SAN JOSE, CALIFORNIA

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Important Information About Your

Geotechnical Engineering Report

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

The following information is provided to help you manage your risks.

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

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

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

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

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

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

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

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

Subsurface Conditions Can Change

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

Most Geotechnical Findings Are Professional Opinions

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

A Report's Recommendations Are *NOT* Final

Don't over-rely on the construction recommendations included in your report. *Those recommendations are not final*, because Geotechnical engineers develop them principally from judgment and opinion. Geotechnical engineers can finalize their recommendations only by observing actual subsurface conditions revealed during construction. *The Geotechnical engineer who developed your report cannot assume responsibility or liability for the report's recommendations if that engineer does not perform construction observation.*

A Geotechnical Engineering Report is Subject to Misinterpretation

Other design team members' misinterpretations of Geotechnical engineering reports has resulted in costly problems. Lower that risk by having your Geotechnical engineer confer with appropriate members of the design team after submitting the report. Also retain your Geotechnical engineer to review pertinent elements of the design team's plans and specifications. Contractors can also misinterpret a Geotechnical engineering report. Reduce that risk by having your Geotechnical engineer participate in pre-bid and pre-construction conferences, and by providing construction observation.

Do Not Redraw the Engineer's Logs

Geotechnical engineers prepare final boring and testing logs based upon their interpretation of field logs and laboratory data. To prevent errors or omissions, the logs included in a Geotechnical engineering report should never be redrawn for inclusion in architectural or other design drawings. Only photographic or electronic reproduction is acceptable, but recognize that separating logs from the report can elevate risk.

Give Contractors a Complete Report and Guidance

Some owners and design professionals mistakenly believe they can make contractors liable for unanticipated subsurface conditions by limiting what they provide for bid preparation. To help prevent costly problems, give contractors the complete Geotechnical engineering report, but preface it with a clearly written letter of transmittal. In that letter, advise contractors that the report was not prepared for purposes of bid development and that the report's accuracy is limited; encourage them to confer with the Geotechnical engineer who prepared the report (a modest fee may be required) and/or to conduct additional study to obtain the specific types of information they need or prefer. A pre-bid conference can also be valuable. Be sure contractors have sufficient time to perform additional study. Only then might you be in a position to give contractors the best information available to you, while requiring them to at least share some of the financial responsibilities and risks. Read these provisions closely. Ask questions. Your Geotechnical engineer should respond fully and frankly.

Geoenvironmental Concerns Are Not Covered

The equipment, techniques, and personnel used to perform a geoenvironmental study differ significantly from those used to perform a Geotechnical study. For that reason, a Geotechnical engineering report does not usually relate any geoenvironmental findings, conclusions, or recommendations; e.g., about the likelihood of encountering underground storage tanks or regulated contaminants. Unanticipated environmental problems have led to numerous project failures. If you have not yet obtained your own geoenvironmental information, ask your Geotechnical consultant for risk management guidance. Do not rely on an environmental report prepared for someone else.

Rely on Your Geotechnical Engineer for Additional Assistance

Membership in ASFE exposes Geotechnical engineers to a wide array of risk management techniques that can be of genuine benefit for everyone involved with a construction project. Confer with your ASFE-member Geotechnical engineer for more information.

ASFE PROFESSIONAL FIRMS PRACTICING IN THE GEOSCIENCES

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FEASIBILITY-LEVEL
GEOLOGIC HAZARDS ASSESSMENT
100 FERGUSON DRIVE
MOUNTAIN VIEW, CALIFORNIA

1 INTRODUCTION

This report presents the results of our feasibility-level geologic hazards assessment for the Arcadia Property in San Jose, California. The location of the project site with respect to surrounding cultural and topographic features is depicted on the Site Vicinity Map, Plate 1.

1.1 PROJECT DESCRIPTION

The details regarding the number, locations, design and type of buildings for the proposed development were not available at the time this report was prepared. For the purpose of this geologic hazards assessment, it is assumed that the project will consist of mixed-use buildings of sizes similar to structures in the vicinity of the project site. In addition to the buildings, other site improvements are anticipated to include new underground utilities, exterior flatwork, paved parking and access driveways, and landscaping.

1.2 SITE DESCRIPTION

The project site is located southwest of the intersection of Quimby Road and Capitol Expressway in San Jose, California (Site Aerial Photo, Plate 2). The 81-acre site consists of seven APN's (670-20-71, 670-24-45, 670-25-27, 670-29-02, 670-29-17, and 670-29-20). Historically, the site has been used for agricultural production, with a large acreage of the field currently fallow and tilled. The areas not tilled are generally covered with high standing weeds consisting of thistles and tall grasses.

The ground surface at the site is essentially flat with a very slight gradient downward toward the north. The highest elevation of about 145 feet above mean sea level is near the southeast corner of the site and the lowest elevation of 137 feet is on the northwest corner of the site. Based on data presented on USGS Quad Map for East San Jose 7.5-minute quadrangle, the site coordinates are:

Latitude: 37.391° North Longitude: -121.809° West

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1.3 PURPOSE AND SCOPE OF SERVICES

The purposes of this geologic assessment are to identify and assess potential geologic hazards at or near the site. Our scope of services included the following:

- Research and review readily available geologic, geotechnical, and seismologic reports, and FEMA publications and maps in our library that pertain to the site and vicinity;
- Conduct a geologic reconnaissance of the site by our engineering geologist to observe and document surface features indicative of possible geologic hazards;
- Conduct a limited subsurface investigation with geotechnical laboratory testing of selected soil to evaluate soil conditions as they may relate to geologic hazards at the site;
- Assess significant faults and site seismicity and conduct an analysis of potential earthquake impact at the site; and
- Evaluate the researched data and prepare this report with conclusions and recommendations regarding possible geologic and seismic hazards affecting the site and the proposed project.

References reviewed for compilation of this report are listed in the "References" section. This investigation excludes the assessment of environmental characteristics, particularly those involving hazardous substances. Environmental services such as chemical analysis of soil and groundwater were not included in our scope of services.

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2 GEOLOGIC SETTING

2.1 REGIONAL GEOLOGY

The San Francisco Bay Area lies within the Coast Range Geomorphic Province, a more or less discontinuous series of northwest trending mountain ranges, ridges, and intervening valleys characterized by complex folding and faulting (Regional Geologic Map, Plate 3). Geologic and geomorphic structures within the San Francisco Bay Area are dominated by tectonic deformation. The San Andreas Fault system is a right-lateral strike-slip series of fault that extends on land from the Gulf of California in Mexico, to Cape Mendocino, on the coast of Humboldt County in northern California. It forms a portion of the boundary between two independent tectonic plates on the surface of the earth. To the west of the San Andreas Fault, the Pacific plate moves north relative to the North American plate, located east of the fault. In the San Francisco Bay Area, movement across this plate boundary is distributed across the San Andreas Fault and a number of other faults including the Hayward, Calaveras, and San Gregorio. Together, these faults are referred to as the San Andreas Fault system. The general trend of the faults within this system is responsible for the strong northwest-southeast structural grain of geologic and geomorphic features in the San Francisco Bay Area.

For most of the length of the San Andreas Fault, basement rock on the east generally consists of a chaotic mixture of highly deformed marine sedimentary, submarine volcanic, and metamorphic rocks of the Franciscan Complex. The Franciscan rocks are generally considered Jurassic and Cretaceous age (about 65 to 205 million years old). Overlying the basement rocks are Cretaceous marine, as well as Tertiary (about 65 to 1.6 million years old) marine and non-marine sedimentary rocks with some continental volcanic rock. These Cretaceous and Tertiary rocks typically have been extensively folded and faulted largely as a result of movement along the San Andreas Fault system over about the last 25 million years.

The project site is located on the broad alluvial-covered plain lying between the Santa Cruz Mountains, forming the backbone of the San Francisco peninsula to the northwest, and the Diablo Range to the east. The inland valleys, as well as the structural depression within which San Francisco Bay is located, are filled with unconsolidated to semi-consolidated deposits of late Pliocene and Quaternary age. Continental deposits (alluvium) consist of unconsolidated to semi-consolidated sand, silt, clay, and gravel, and the bay deposits typically consist of very soft organic rich silt and clay or sand.

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2.2 AREA AND SITE GEOLOGY

The Arcadia Property, at the corner of Quimby Rd. and Capitol Expressway in San Jose, is located in the alluvial plain of the Santa Clara Valley. Mapping by Wentworth et al., (1999) indicates that the site is located on basin deposits of Holocene age (**Qhb**) (less than 11,000 years old), and alluvial fan deposits of older Holocene (**Qhf2**) and Upper Pleistocene (**Qpf**) age (Vicinity Geologic Map I, Plate 4). Knudsen and others (2000) excluded the Upper Pleistocene alluvial fan deposit (**Qpf**) and interpreted it as (**Ql**) (Late Pleistocene to Holocene alluvial fan levee deposits). Furthermore, Knudsen and others (2000) replace **Qhf2** and **Qhb** by **Qhf** (Holocene alluvial fan deposits) and **Qhff** (Holocene alluvial fan deposits, fine-grained facies) within the site, respectively (Vicinity Geologic Map II, Plate 5). For the purposes of this report, and analyses of the on-site deposits, the mapping by Knudsen and others (2000), further interpreted by the California Geological Survey (CGS formally known as the California Department of Mines and Geology), will be used. The soil samples collected for use in the CDMG report (2000), that pertain to the Arcadia property, consist of **Qhf** (primarily lean clay (41%) and silt (19%), as well as sandy silt (17%)), **Qhff** (primarily lean clay (71%) and silt (17%)), and **Ql** (primarily lean clay (50%) and silt (38%)). Thompson Creek and other creeks, which flow from the southern Diablo Range southeast of the site, are interpreted to have deposited these alluvial deposits.

Past regional geologic investigations by others (Cooper Clark & Associates, 1974 and City of San Jose, 1983) project a trace of the Silver Creek fault toward the project site from the south but do not extend the fault trace to the site. Recent subsurface investigations using seismic reflection is interpreted to indicate the fault, if active, impinges on the deepest of Quaternary alluvial sediments (HMM/Bechtel, 2005) in the Santa Clara Valley and is not shown to transect the project site.

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3 SITE INVESTIGATION

3.1 AERIAL PHOTOGRAPH REVIEW

Aerial photographs of the site were reviewed for lineations, tonal character, or other geomorphic features (Plate 2). In our review of the aerial photographs, we did not observe geomorphic features suggestive of faulting on or projecting toward the site nor were other geologic hazards observed.

3.2 FIELD INVESTIGATION

Our field investigation consisted of a surface reconnaissance and a subsurface exploration program. Our exploration program, performed on May 13 and May 25, 2005, included two exploratory soil borings and eight Cone Penetrometer Test (CPT) holes. The borings and CPTs were located in areas spread across the site to give an overall characterization of the subsurface conditions. The borings were drilled to depths approximately 45 feet below existing ground surface (bgs) using a mud rotary drill rig equipped with 4-7/8-inch diameter drag bit. The CPTs ranged in depth up to about 45 feet. The approximate locations of our borings and CPTs are shown on the Site Aerial Photograph, Plate 2.

The soils encountered in our borings were visually classified in the field in general accordance with the Unified Soil Classification System (ASTM D2488) by our engineering staff. The results of our laboratory tests were used to refine the field classifications based on ASTM D2487. A key for classification of the soils is presented on the Boring Log Legend, Plate A-1. The logs of the borings are presented on Plates A-2 through A-3 in Appendix A.

Representative soil samples were obtained from the borings at selected depths by driving a 2-inch inside diameter Modified California sampler. The sampler was driven up to 18 inches into the soil at each sampling interval using a 140-pound hammer falling 30 inches. The number of blows required to drive the sampler the last 12 inches was noted on the boring logs. Samples collected from the borings were returned to our laboratory for further evaluation and testing. The borings were backfilled with cement grout.

John Sarmiento & Associates performed the cone penetrometer tests on -------2005. The tip resistance, side friction, and pore pressure measured by the cone as it was pushed through the soil strata were recorded electronically every 0.05 meters (approximately 2 inches). The CPT data, presented in Appendix A, include the following with respect to depth:

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Qc	Tip Resistance,
Fs	Local Friction,
Rf	Friction Ratio
SPT(N)	Equivalent Standard Penetration N-value
SPT (N')	Corrected Equivalent Standard Penetration N-value
TotVtStr	Total Overburden Stress
PHI	Internal friction angle for granular soils
Su	Undrained Shear Strength for cohesive soils
Soil Behavior type	
Density Range	

3.3 LABORATORY TESTING

Laboratory testing was performed on selected soil samples collected from the borings. Tests performed included moisture content, Atterberg limits, grain-size analysis, and percent fines using a hydrometer. Selected laboratory-test results are presented on the borings.

3.4 SUBSURFACE CONDITIONS

In boring B-1, medium stiff clay was encountered from the ground surface to approximately 35 feet below ground surface. The clayey soils in the top 35 feet are generally medium stiff to stiff. Between 35 feet and 40 feet, a layer of dense to very dense well-graded gravelly sand was encountered. Stiff to very stiff lean silty clay with some sand was located from a depth of 40 feet to the bottom of the boring (46.5 feet). In boring B-2, clayey soils (both lean and fat) were encountered from the surface to the bottom of the boring (46.5 feet), with a thin layer of clayey sand between depths of 25.5 feet and 26.5 feet. These soils correspond to the alluvial deposits of Holocene age in the Area and Site Geology section of this report.

CPT-1, CPT-2, and CPT-4 were located on the western side of the Arcadia property. CPT-3, CPT-5, and CPT-7 were located in a north-south line along the approximate center of the property. CPT-6 and CPT-8 were located on the eastern side of the property. The interpreted soil behavior types from the CPTs suggest the subsurface materials, in general, correlate to those encountered in the borings. The above is a general description of the subsurface soil conditions encountered in our borings and CPTs performed for this investigation. Geologic cross-sections constructed across the site, based on the soils encountered in the borings and graphs created by the CPTs are presented on Plate 6. For a more detailed description of the soil conditions encountered, refer to the boring logs and CPT data sheets in Appendix A.

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3.5 GROUNDWATER

Because mud rotary drilling obscures the first occurrence of groundwater, the first ten feet of the borings were advanced using a 6-inch diameter auger; no groundwater was encountered at this depth. Near soil boring B-1, an environmental boring was drilled to a depth of 10 feet with no groundwater encountered. Groundwater was encountered in boring B-2 at a depth of about 8 feet. CGS, 2000 shows historical highest groundwater level at the site to be less than 10 feet deep in the northeast corner of the property and more than 20 feet deep in the southwest corner of the property (Historical High Groundwater, Plate 7). It should be noted that fluctuations in groundwater level could occur due to variations in rainfall, irrigation, pumping from wells, temperature, and other factors that were not evident at the time of our investigation. If significant variations in the groundwater level are encountered during construction, it may be necessary for Kleinfelder to review the recommendations and provide adjustments as necessary.

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4 FAULTING AND SEISMICITY

4.1 LOCAL AND REGIONAL FAULTING

Our aerial-photographic review and site reconnaissance did not reveal geomorphic features indicating fault activity on or projecting toward the project site. Previous researchers have mapped the location of the Silver Creek fault zone projecting toward the proposed development site. The Silver Creek Fault is mapped as a generally north-northwest trending oblique reverse-slip fault that extends over a distance of up to 70 km, sub-parallel to and west of the Hayward and Calaveras fault zone (Fenton and Hitchcock, 2001). The southern extent of the fault is exposed at the surface near Anderson Reservoir while the northern reach is buried beneath Quaternary sediments of the San Francisco bay plain. The northern reach of the fault extends from southeast San Jose, northwest along the east side of San Francisco Bay to Milpitas or Fremont (Silicon Valley Rapid Transit Project, 2005). Taylor (1956), Robbins (1971), and Lowney/Kaldvecor Associates (1971) place the possible trace of the fault within about 2,000 feet of the site but do not extend it to the site. An additional study conducted by Cooper Clark & Associates discontinues the trace of the fault before it reaches the project site due to lack of data (Plate 8 and 9).

According to the 1997 UBC, the site is located in Seismic Zone 4 as is most of coastal California. Plate 10, Regional Fault Map depicts traces of mapped faults and indicates their potential activities. Table 1 lists active faults and selected seismic parameters that are mapped within about 100 km of the site according to data compiled from the CGS. The parameters presented in Table 2 are based on Blake (2000). The attenuation calculation used is from Campbell and Bozorgnia (1997, revised) for alluvial sites.

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TABLE 1
SIGNIFICANT FAULTS WITHIN ABOUT 50 MILES OF THE PROJECT SITE

Fault Name	Approximate Distance to Fault mi(km)	Magnitude of Maximum Earthquake	Peak Site Acceleration (g)
Hayward (SE Extension)	2.5 (4)	6.5	0.417
Calaveras (So.of Calaveras Res)	6 (9)	6.2	0.286
Hayward (Total Length)	9 (14)	7.1	0.332
Calaveras (No.Of Calaveras Res)	9 (14)	6.8	0.286
Monte Vista - Shannon	10 (16)	6.8	0.310
Sargent	15 (23)	6.8	0.182
San Andreas (1906)	15 (24)	7.9	0.347
Greenville	18 (30)	6.9	0.152
Zayante-Vergeles	19 (30)	6.8	0.139
Great Valley 7	26 (42)	6.7	0.090
Great Valley 6	27 (44)	6.7	0.085
Ortigalita	30 (47)	6.9	0.087
San Gregorio	30 (49)	7.3	0.117
Great Valley 8	31 (51)	6.6	0.065
Monterey Bay - Tularcitos	33 (52)	7.1	0.091
Quien Sabe	37 (59)	6.4	0.042
Concord - Green Valley	40 (65)	6.9	0.058
Palo Colorado - Sur	41 (65)	7.0	0.063
Great Valley 9	45 (72)	6.6	0.040
Rinconada	45 (72)	7.3	0.072

4.2 HISTORICAL SEISMICITY

The project site is located in an area characterized by high seismic activity. A number of large earthquakes have occurred within this area in past years. Some of the significant nearby events include the 1906 (M7.9) "Great" San Francisco earthquake, the 1838 (M7) San Francisco Peninsula earthquake, the 1865 (M6.4) Santa Cruz Mountains earthquake, the 1868 (M6.8) Hayward earthquake, the 1890 (M6.2) Pajaro Gap earthquake, the 1899 (M5.8) and 1984 (M6.1)

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Morgan Hill earthquakes, the 1882 (M5.8) and 1892 (M5.8) Hollister earthquakes, the 1897 (M6.2) Gilroy earthquake, the two 1903 (M5.5) San Jose earthquakes, the 1910 (M5.8) Watsonville earthquake, two 1926 (M6) Monterey Bay earthquakes, and the 1989 (M6.9) Loma Prieta earthquake. A recent study by Toppozada and Borcherdt (1998) indicates an 1836 (M6.8) earthquake, previously attributed to the Hayward fault, occurred in the Monterey Bay area and was of an estimated magnitude M6.2. During the 1989 Loma Prieta earthquake on the San Andreas Fault, several California Strong Motion Instrumentation Program (CSMIP) stations in the area recorded free-field horizontal peak ground accelerations ranging from 0.1 to 0.5 g (Thiel Jr., et al., 1990).

The Working Group On California Earthquake Probabilities (2003) has estimated probabilities of large earthquakes (magnitude 6.7 or greater) in the San Francisco Bay Region for segments of the San Andreas, Hayward, and Calaveras faults. The study estimated that there is a 62 percent probability of a large earthquake in the San Francisco Bay Region in the next 30 years (from 2002). With respect to individual fault segments considered in the study, estimates were provided of 21 percent for the San Francisco Peninsula segment of the San Andreas Fault, 27 percent for the Hayward fault and 11 percent for the Calaveras fault (Plate 11).

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5 CONCLUSIONS - GEOLOGIC HAZARDS

A discussion of specific geologic hazards that could impact the site is included below. The hazards considered include: surface fault rupture; seismic shaking; liquefaction, dynamic compaction; landslides, seismically induced ground failures, flooding, erosion and naturally occurring asbestos.

5.1 SURFACE FAULT RUPTURE

Mapping by others (Cooper Clark & Associates, 1974 and City of San Jose, 1983, Wentworth and Others, 1999, HMM/Bechtel, 2005) does not show active faults crossing the project site. No geomorphic expressions of faulting were observed on the stereoscopic aerial photographs. Additionally, no evidence of active faulting was visible on the site during our site reconnaissance. Therefore, it is our opinion that the potential for fault-related surface rupture at the Arcadia Site is low.

5.2 NEAR-SOURCE SEISMIC SHAKING

For this site, the southeast extension of the Hayward fault should be considered the controlling source for near-fault motions since it is the closest significant fault. Because the Hayward fault is located about 4 km from the site (ICBO, 1998) and is classified as a Seismic Source Type B (Table 16A-U of 1997 UBC) the Near-Source Factors Na and Nv are 1.3 and 1.7, respectively (Tables 16-S and 16-T of the 1997 UBC). The Type A San Andreas Fault, located approximately 15 km from the site, presents the values for both Na and Nv of 1.0. The Na value may be modified in accordance with UBC section 1629.4.2, section 1630.2.3.2 or other sections as determined appropriate by the structural engineer. Alternatively, consideration of near-source factors should be applied to dynamic analyses utilizing site-specific response spectra that account for the types of near source effects observed in the recent Northridge, California and Kobe, Japan earthquakes.

5.3 SITE SOIL PROFILE TYPE

The characteristics of the soils underlying the site are used to evaluate site-specific seismic design criteria. Considering the results of our field investigation and data review, the site generally is underlain by alluvial deposits of Holocene age, which consists primarily of clay layers with blow counts of 15 or less. Based on these findings, it is our opinion that the site is consistent with a Soil Profile Type S_E in accordance with the Table 16A-J of the 2001 CBC. S_E is defined as a soil profile consisting of soft soil with shear wave velocities less than 600 ft/s,

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SPT-N less than 15, or undrained shear strength (Su) less than 1,000 over the upper 100 feet (30 meters). We note that our investigation is preliminary in scope, and that the site soil-profile type should be confirmed for each building site during the design-level geotechnical investigation performed for the project.

5.4 SEISMICALLY INDUCED GROUND FAILURE

5.4.1 Liquefaction

Soil liquefaction is a condition where saturated, granular soils undergo a substantial loss of strength and deformation due to pore pressure increase resulting from cyclic stress application induced by earthquakes. In the process, the soil acquires mobility sufficient to permit both horizontal and vertical movements if the soil mass is not confined. Soils most susceptible to liquefaction are Holocene age, saturated, loose, clean, uniformly graded, and fine-grained sand deposits. As shown on Plate 12, the site is partially located in an area designated to be subject to potential liquefaction (CGS, 2003)

No accounts of liquefaction were recorded at the site during the 1989 Loma Prieta earthquake or the 1906 "Great" San Francisco earthquake (Youd and Hoose, 1978; Tinsley, 1998; and Lawson, 1908). Considering the results of our field investigation and data review, the site is underlain by alluvial deposits of Holocene age to the depths of our soil borings and CPT probes. These soil materials consist predominantly of medium stiff to stiff clay and dense to very dense gravelly sand to depths of about 46 feet. Groundwater was recorded in boring B-2 at a depth of approximately 8 feet. Interpretative Cross Sections depicting subsurface soil layers and groundwater across the site are provided on Plate 6. Based on the above information, we infer that the susceptibility for liquefaction at the site is moderate and should be further evaluated during the site-specific design-level geotechnical investigation. Laboratory testing of the fine material encountered within the granular soil deposits encountered in our boring focused on identifying their relative plasticity for use as an indicator of the soil's susceptibility to liquefaction. Results of our analysis indicate that liquefaction settlement (the primary liquefaction related concern at the site) will be minor to negligible.

Another concern during an earthquake is ground-surface disturbance or ground failure. Ground failure can be in the form of sand boils, ground fissures, ground oscillation such as buckled pavements, curbs, broken pipelines, etc., and lateral-ground displacement. Since the non-liquefiable cover over the liquefiable materials is sufficient to dampen these affects, ground-surface disruption is not likely during a seismic event.

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Lateral spreading is a potential hazard commonly associated with liquefaction where extensional ground cracking and settlement occur as a response to lateral migration of subsurface liquefiable material or sensitive clays. This phenomenon typically occurs adjacent to free faces such as creek channels, harbors and canals. Given that the site is virtually level, it is unlikely that lateral spreading could occur at this site.

5.4.2 Dynamic Compaction

Dynamic compaction or seismic settlement, a form of seismically induced ground failure, can occur as a result of seismic shaking. Such phenomena typically occur in unsaturated, loose granular material or uncompacted fill. Considering the relative density of the subsurface soil, the probability of dynamic compaction occurring in the event of severe seismic shaking at the site is considered low. However, because the site is located in an area associated with high seismic activity, the possibility of shaking-related random ground cracking affecting the site and surrounding areas should not be excluded.

5.4.3 Landslides and Seismically Induced Slope Failures

Because of the relatively level surface topography at the site and adjacent areas, the potential for landsliding to affect the site is considered negligible.

5.5 FLOOD HAZARD

Flood hazards are generally considered from three sources, which include seismically induced waves (tsunami or seiche), dam failure, and long-cycle storm events. The site is located approximately 14 miles from San Francisco at an elevation of about 140 feet above mean sea level. The only historical account of tsunamis impacting the San Francisco Bay area is the "Good Friday" earthquake of 1964 (generated of the coast of Alaska), which caused only very minor damage at Monterey and Moss Landing Harbors (CGS, 1972). Run-up at the Golden Gate Bridge was measured at 7.4 feet from the Good Friday earthquake and generally less further south. Based upon the site's distance to San Francisco Bay, elevation, and the lack of historically damaging tsunamis and seiches, we judge that the potential for a seismically induced wave to impact the site is very low.

The California Office of Emergency Services shows that no reservoirs are located upgradient from the project site that could cause site inundation in the event of catastrophic dam failure. Therefore, the potential of flooding resulting from dam failure inundation is non-existent

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With respect to the 100-year storm events, ESRI/FEMA (Project Impact Information and Awareness Site [http://www.esri.com/hazards] and FEMA Flood Insurance Rate Map, Community-Panel Number 060337-0640 D, August 1982) indicates that the site is not within a 100-year flooding area.

5.6 EROSION

Because the project site is virtually level, the potential for severe erosion at the site is considered to be minimal. Normal erosion-control methods as prescribed in the erosion-control plan to be developed for site development should be implemented during site grading.

5.7 NATURALLY OCCURRING ASBESTOS

The nearest mapped location of ultramafic rock from which naturally occurring asbestos may be sourced is less than 1.5 miles to the south of the project site (Churchill, and Hill, 2000). Based on this information, a possibility of rocks containing asbestos minerals at the site exists. The fragments are assumed to be contained within the alluvial deposits from erosion upgradient.

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6 RECOMMENDATIONS

From a geologic-hazards perspective, it is our opinion that the Arcadia property at the intersection of Quimby Rd. and Capitol Expressway in San Jose site can be developed if the following recommendations are incorporated in the project design, plans, and construction.

- Procedures from 1997 UBC at a minimum should be implemented for a code-equivalent lateral-force design of structures within the project area. Near-Source Factors Na and Nv to be used at the project site are 1.3 and 1.7, respectively.
- Further site-specific design-level geotechnical investigations should be conducted for the proposed development once building design and layout are determined. Additional geologic hazards studies will be required for essential service buildings including fire stations and schools.

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7 LIMITATIONS

This report may be used only by David Powers & Associates and the Arcadia Project design team for the proposed development of the Arcadia property at the intersection of Quimby Rd. and Capitol Expressway in San Jose, and only for the purposes stated, within one year from its issuance. Land use, site conditions (both on site and off site) or other factors may change over time, and additional work may be required with the passage of time. Any party other David Powers & Associates and their project design team who wishes to use this report shall notify Kleinfelder of such intended use. Based on the intended use of the report, Kleinfelder may require that additional work be performed and that an updated report be issued. Non-compliance with any of these requirements by the clients or anyone else will release Kleinfelder from any liability resulting from the use of this report by any unauthorized party.

The conclusions and recommendations in this report are for the proposed development at the intersection of Quimby and Capitol Expressway in San Jose and only for that proposed development as described in the text of this report. The logs of the exploratory borings and CPTs do not provide a warranty as to the conditions that may exist beneath the entire site. The extent and nature of subsurface soil and groundwater variations may not become evident until construction begins. It is possible that variations in soil conditions and depth to groundwater could exist beyond the points of exploration that may require additional studies, consultation, and possible design revisions. If conditions are encountered in the field during construction that differ from those described in this report, our firm should be contacted immediately to provide any necessary revisions to these recommendations.

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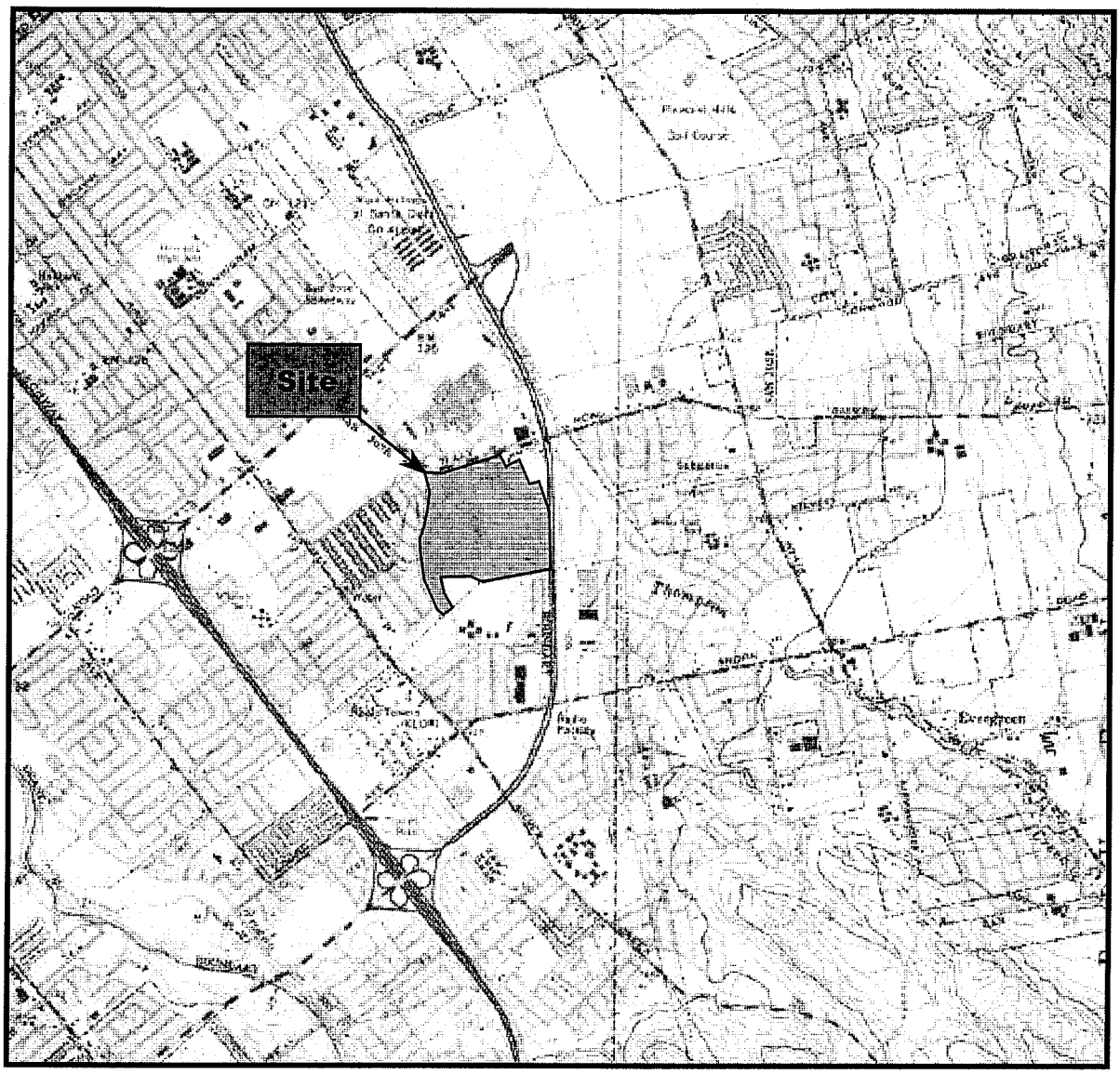
PLATES

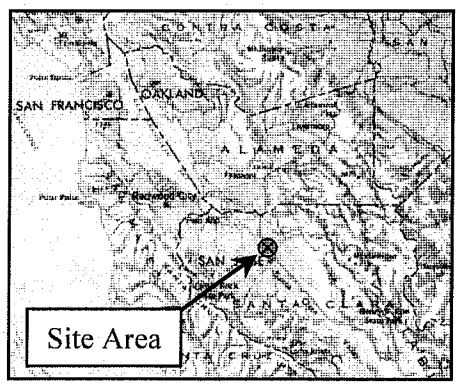


N

Approximate Scale

0 ½ 1 mile

Base: U.S. Geological Survey, East San Jose 7½-minute quadrangle



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Site Vicinity Map
Arcadia
Quimby Rd. & Capitol Expressway
San Jose, California

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PLATE 1

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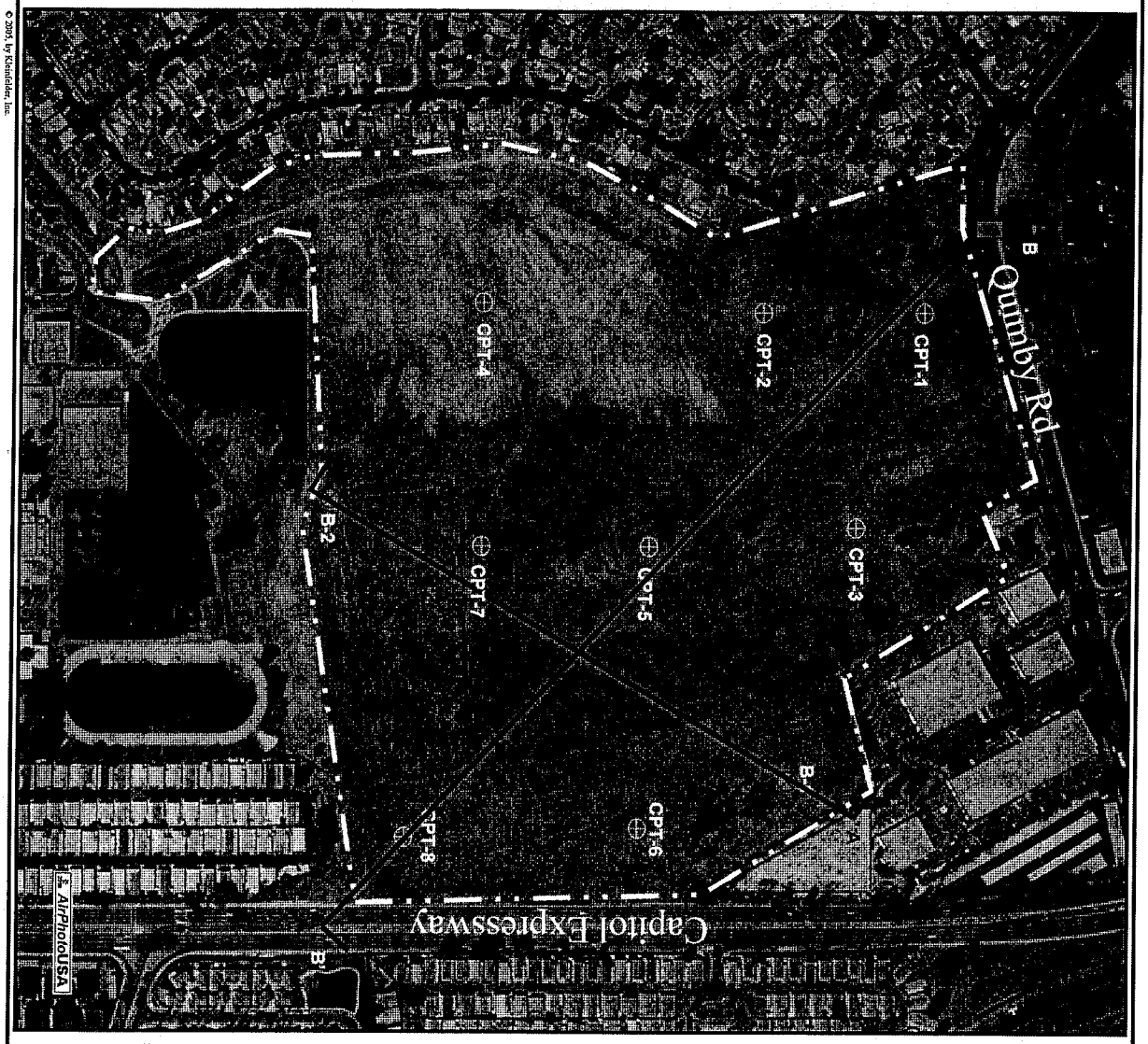

CPT

Soil Boring, Kleinfelder 2005

Property line

N

0 500 1000

Scale feet

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Site Aerial Photograph

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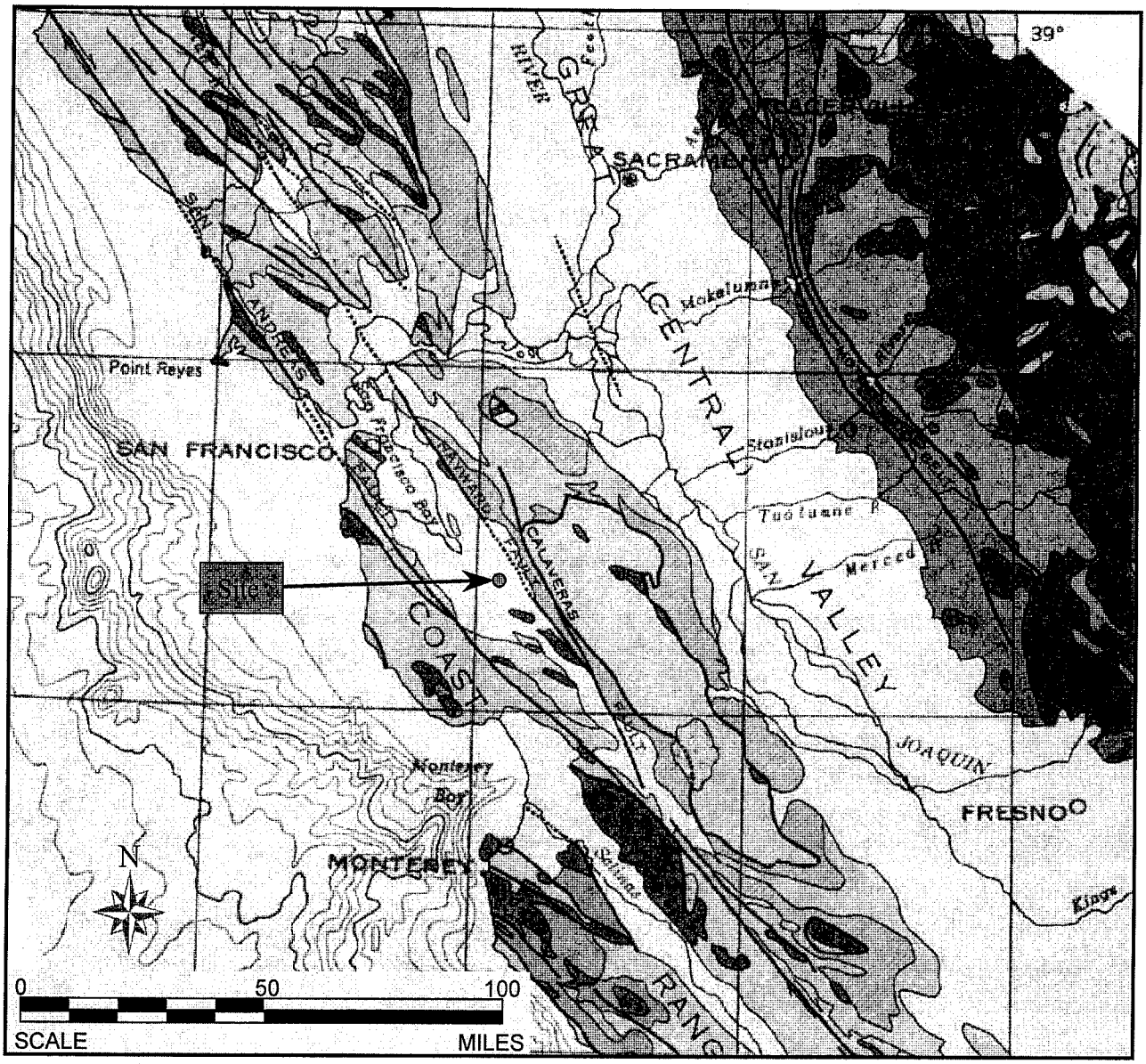
PLATE

2



Map source: California Geological Survey, 2002, Geologic Map of California, California Department of Conservation, used with permission.

EXPLANATION
of units in SF Bay Area

- Cenozoic nonmarine
- Cenozoic marine
- Late Mesozoic shelf and slope
- Late Mesozoic of the Franciscan Formation
- Fault, dotted where concealed, arrows indicate direction of movement
- Mesozoic Granitic rocks
- Mesozoic Ultramafic rocks

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Regional Geologic Map
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PLATE
3

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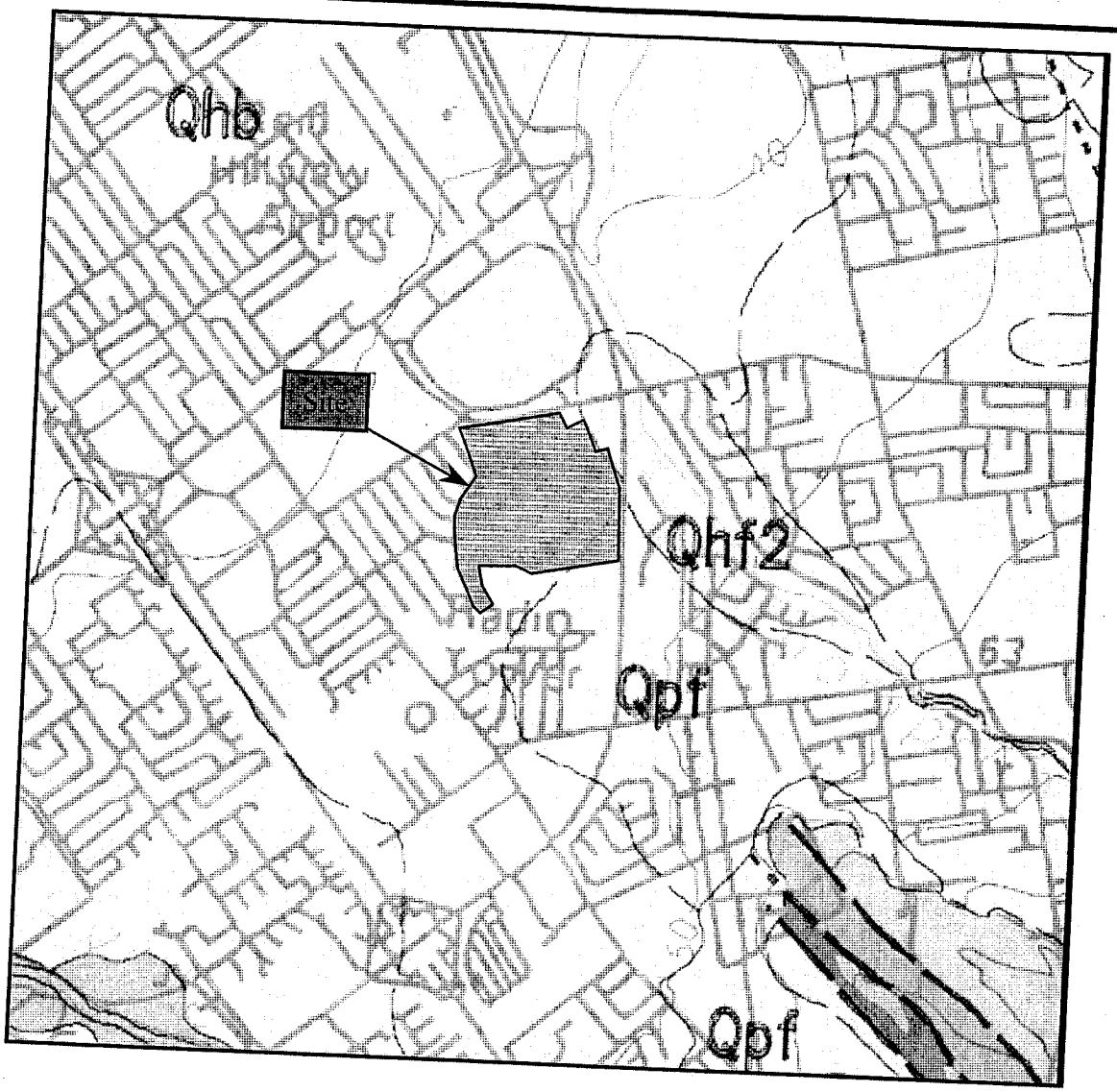

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scale

Original in Color

Base: Wentworth, C.M., Blake, M.C., McLaughlin, R.J., and Graymer, R.W., 1999, Preliminary geologic map of the San Jose 30 X 60-minute quadrangle

N

EXPLANATION

Qhb	Basin deposits (Holocene)
Qhf2	Alluvial fan deposits (Older Holocene)
Qpf	Alluvial fan deposits (Upper Pleistocene)

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Vicinity Geologic Map I
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PLATE 4

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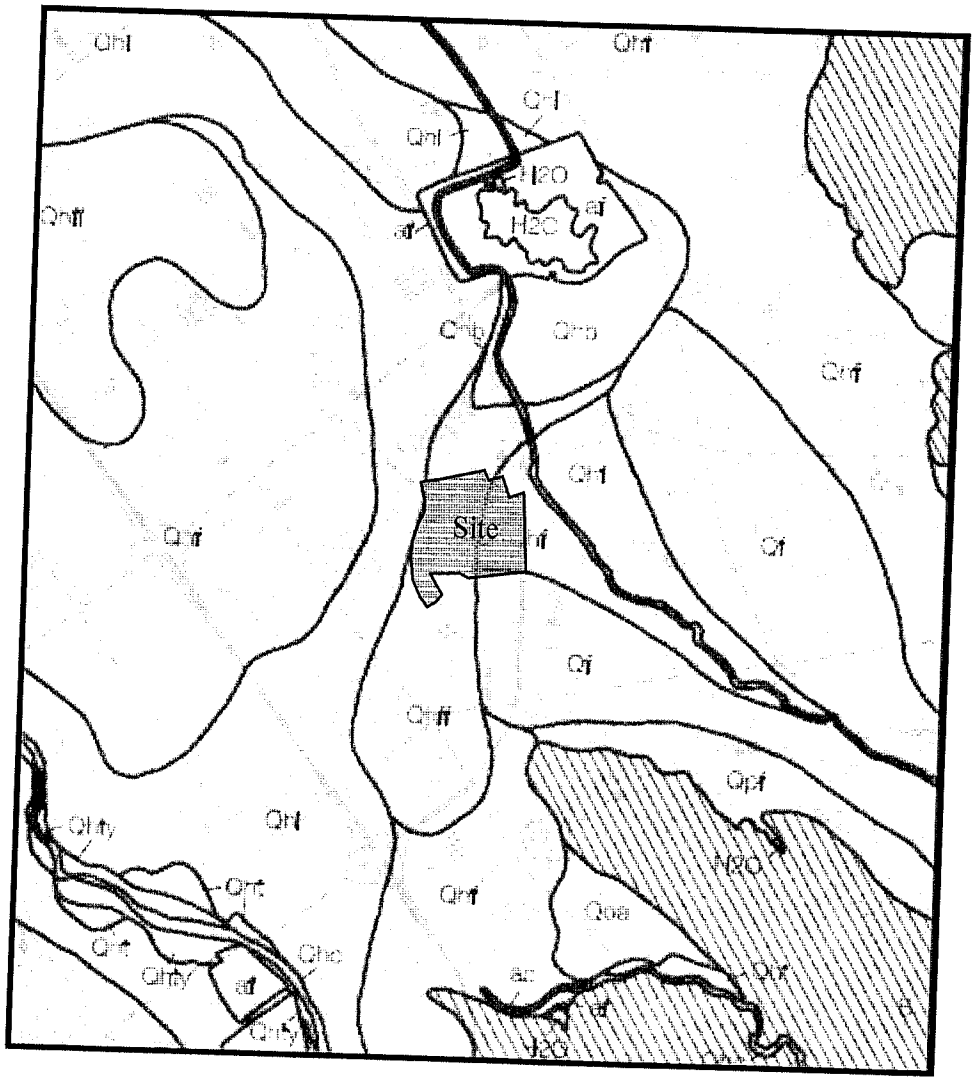

Modified from Knudsen and others (2000)

ONE MILE

SCALE

N

Pre–Quaternary bedrock

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Vicinity Geologic Map II
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PLATE
5

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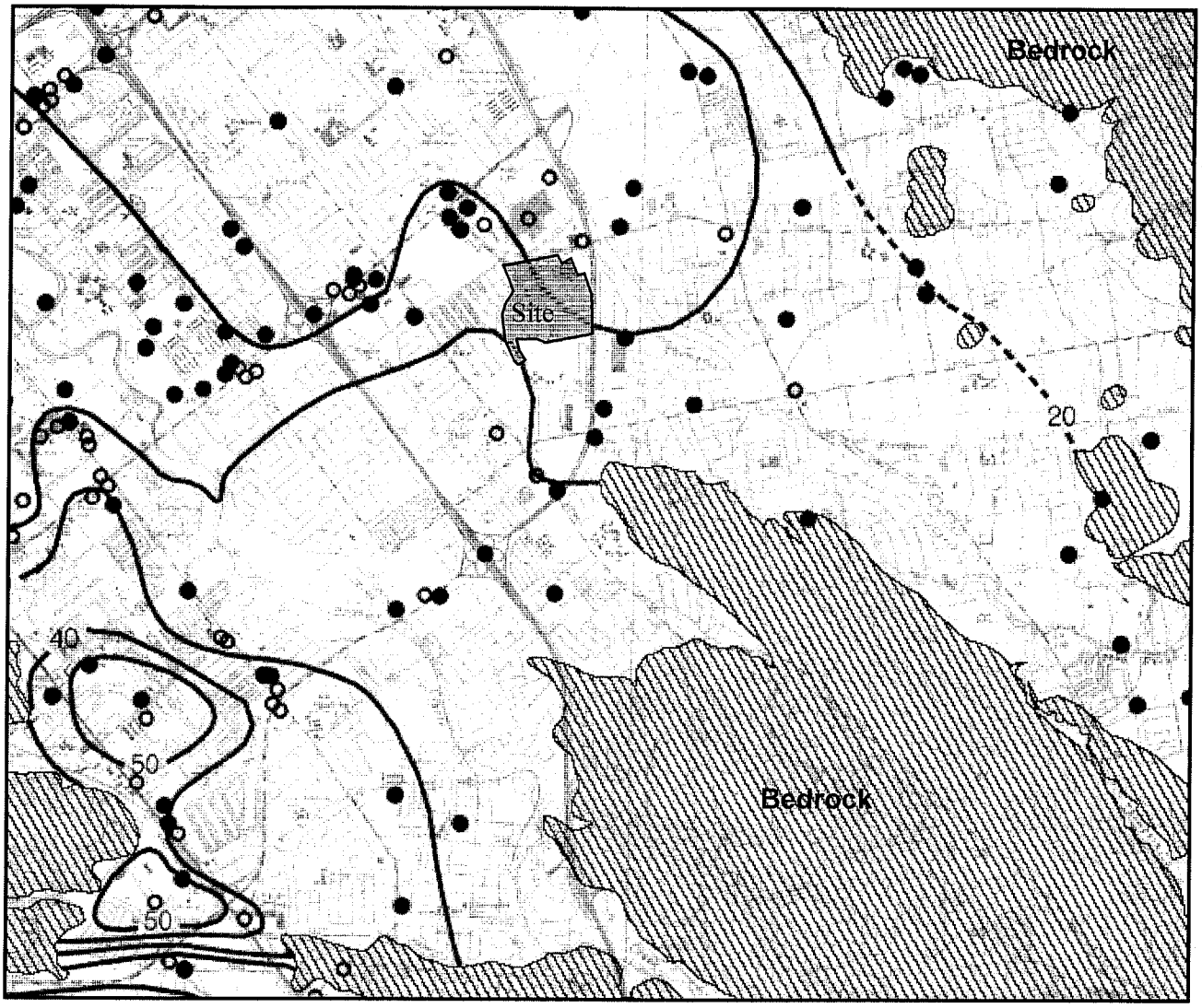

ONE MILE

SCALE

N

Source: California Geological Survey (CGS), 2000, Seismic Hazard Evaluation of the San Jose East 7.5-minute Quadrangle, Santa Clara County, California; Open-file report 2000-010

Water depth contour intervals in feet below ground surface. Circles are well data points

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Historical High Groundwater
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PLATE 7

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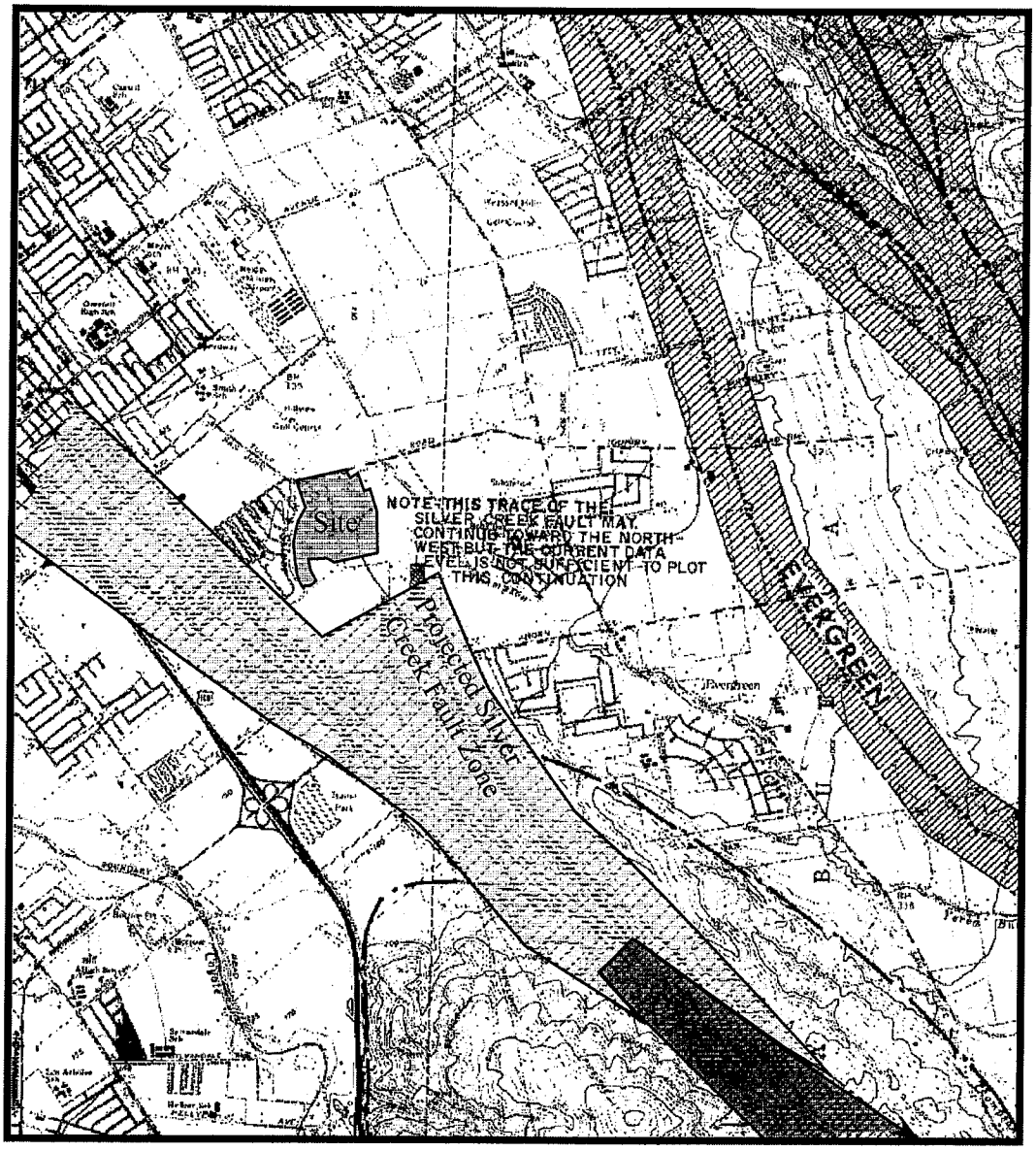

Scale

0 2000 ft. 4000 ft.

N

Source: Cooper, Clark & Associates
San Jose Geotechnical Investigation (1974)

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Silver Creek Fault Location
Arcadia
Quimby Rd. & Capitol Expressway
San Jose, California

PROJECT NO.: 56815

PLATE 8

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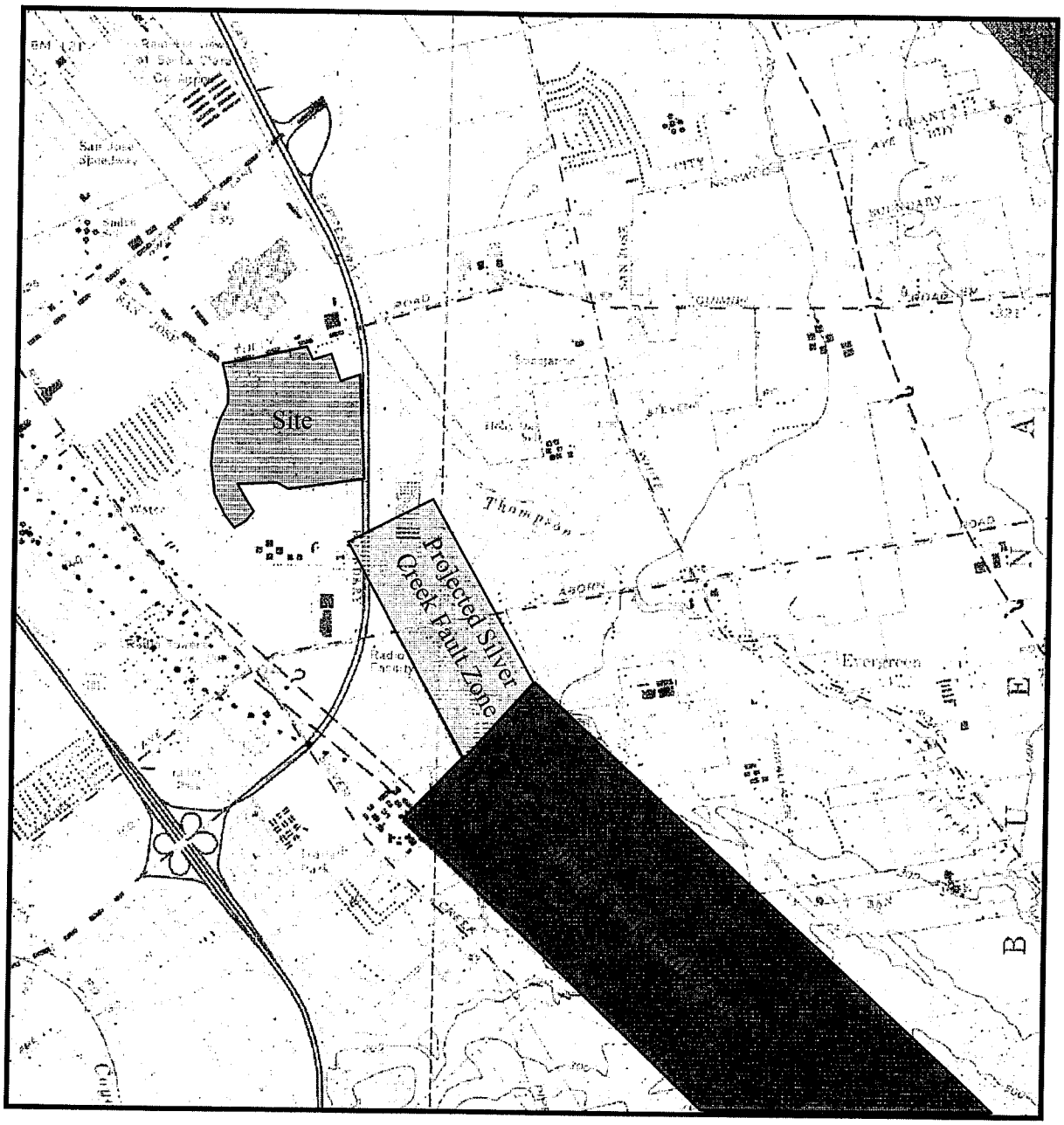

Source: City of San Jose, 1983

N

No scale

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Silver Creek Fault Projection
Arcadia
Quimby Rd. & Capitol Expressway
San Jose, California

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PLATE
9

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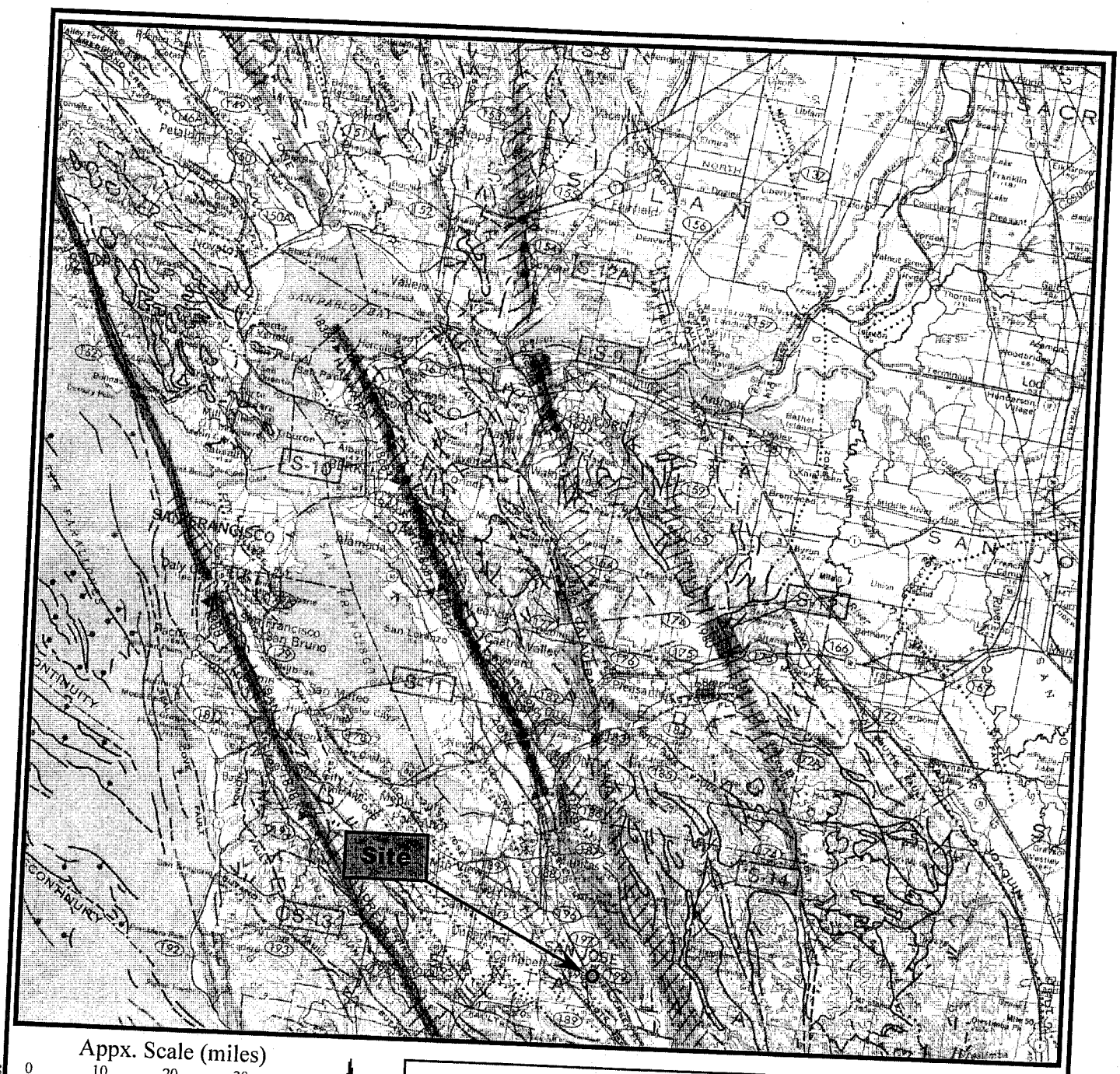

Appx. Scale (miles)

0 10 20 30 40

Reference:

Jennings, C.W., 1994, Fault activity map of California and adjacent area, with locations and ages or Recent volcanic eruptions; California geologic data map series, map no. 6, Department of Conservation, Division of Mines and Geology

EXPLANATION

- Fault with historic movement
- Fault with movement within the last 10,000 years
- Faults with movement older than 10,000 years or undifferentiated

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Regional Fault Map
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PLATE 10

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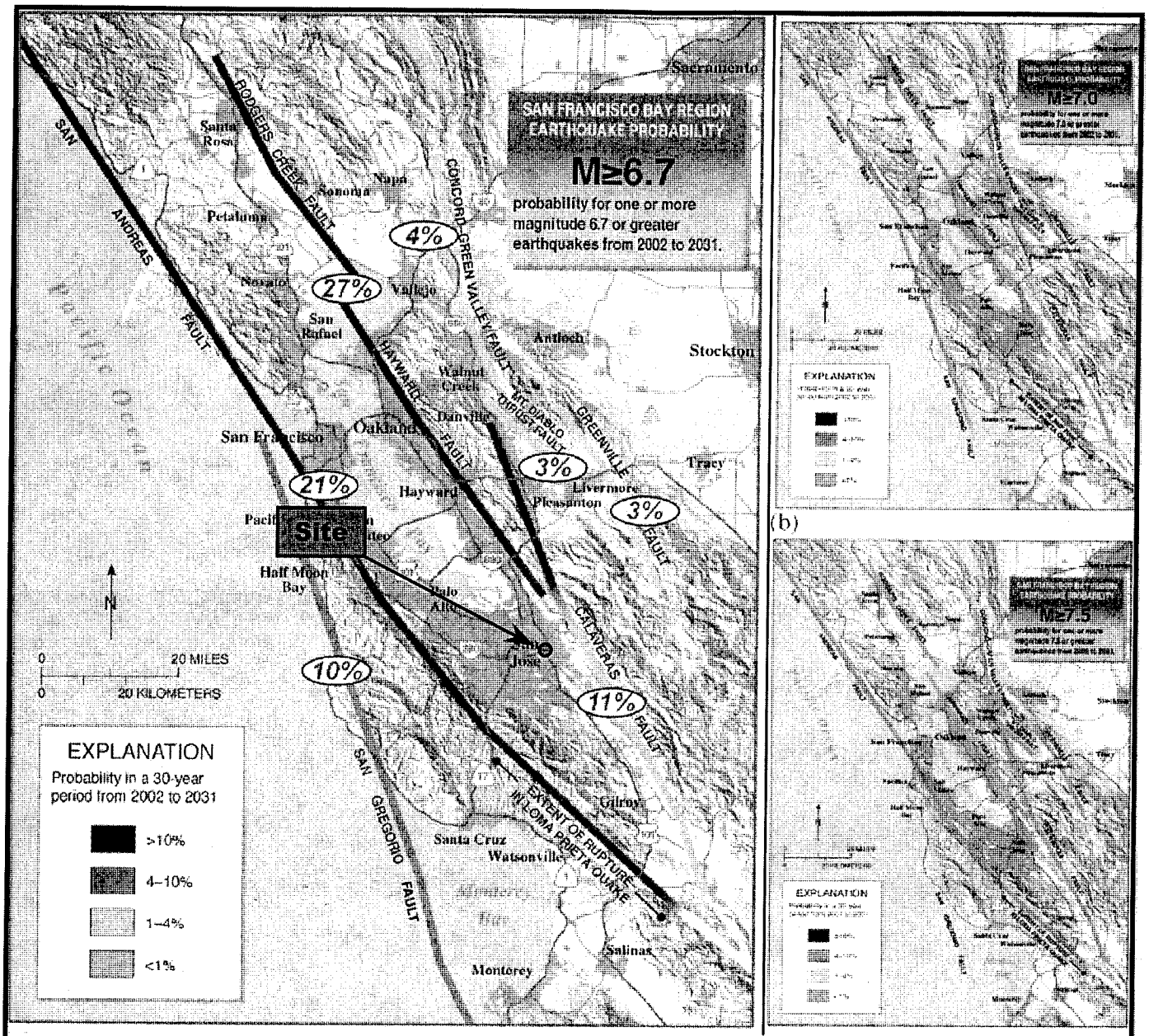

URL: http://geopubs.wr.usgs.gov/open-file/of03-214/

KLEINFELDER 2011 N. Capitol Avenue San Jose, California 95132 Ph. (408) 586-7611 Fax. (408) 586-7688	**Earthquake Probability Map** Arcadia Quimby Rd. & Capitol Expressway San Jose, California	**PLATE** **11**
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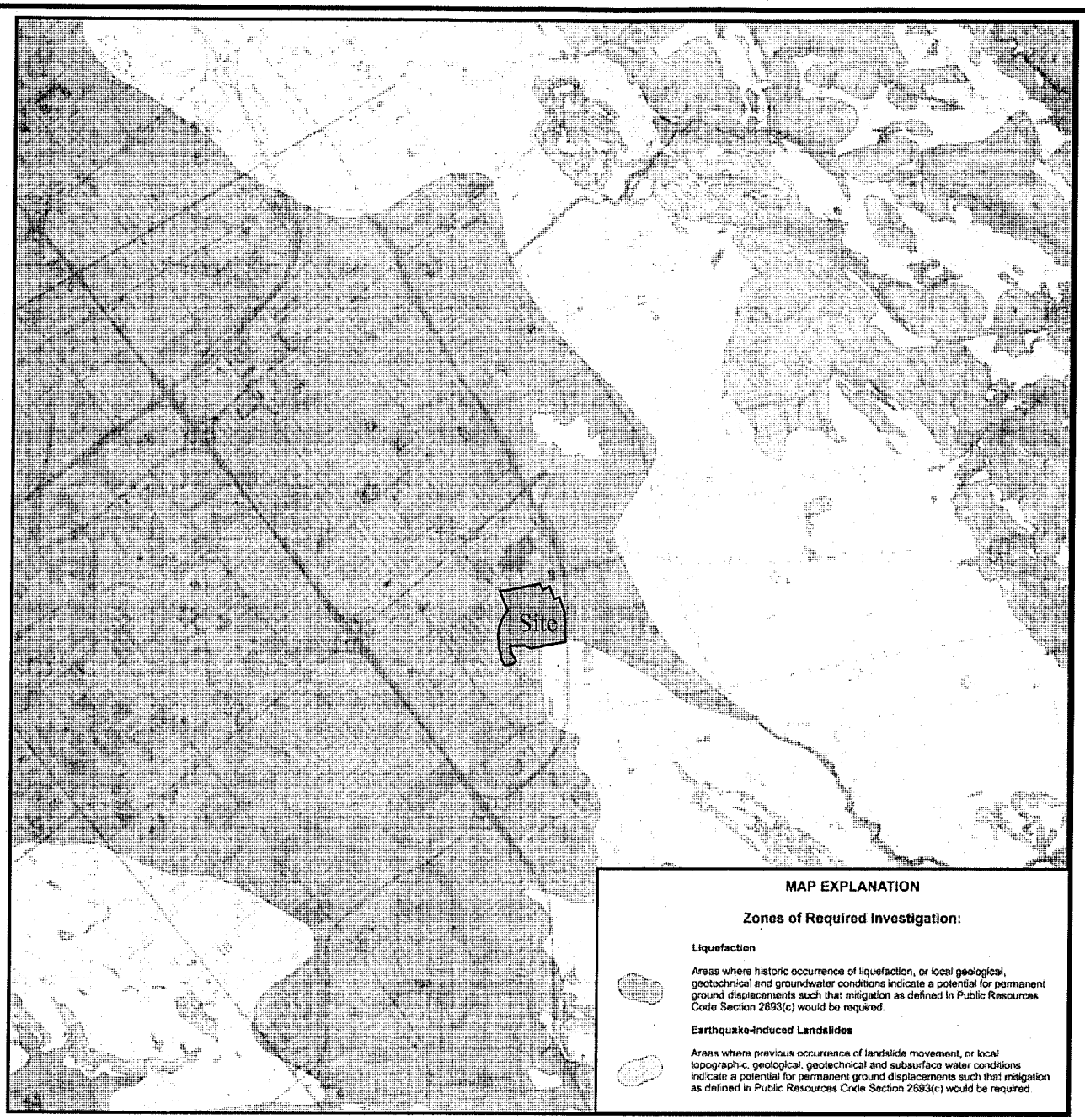

Source: State of California Seismic Hazard Zones
San Jose East Quad (2001)

Scale

0 1 mile

N

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Areas of Potential Liquefaction
Arcadia
Quimby Rd. & Capitol Expressway
San Jose, California

PROJECT NO.: 56815

PLATE
12

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APPENDIX A

UNIFIED SOIL CLASSIFICATION SYSTEM

MAJOR DIVISIONS		LTR	ID	DESCRIPTION
COARSE GRAINED SOILS	GRAVEL AND GRAVELLY	GW		Well-graded gravels or gravel with sand, little or no fines.
		GP		Poorly-graded gravels or gravel with sand, little or no fines.
		GM		Silty gravels, silty gravel with sand mixture.
		GC		Clayey gravels, clayey gravel with sand mixture
	SAND AND SANDY	SW		Well-graded sands or gravelly sands, little or no fines.
		SP		Poorly-graded sands or gravelly sands, little or no fines.
		SM		Silty sand.
		SC		Clayey sand.

MAJOR DIVISIONS		LTR	ID	DESCRIPTION
FINE GRAINED SOILS	SILTS AND CLAYS	ML		Inorganic silts and very fine sands, rock flour or clayey silts with slight plasticity.
		CL		Inorganic lean clays of low to medium plasticity, gravelly clays, sandy clays, silty clays.
		OL		Organic silts and organic silt-clays of low plasticity.
	SILTS AND CLAYS	MH		Inorganic elastic silts, micaceous or diatomaceous or silty soils.
		CH		Inorganic fat clays (high plasticity).
		OH		Organic clays of medium high to high plasticity.
HIGHLY ORGANIC SOILS		Pt		Peat and other highly organic soils.

Standard Penetration Split Spoon Sampler 2.0 inch, 1.4 inch I.D.

Modified California Sampler 2.5 inch O.D., 2.0 inch I.D.

Bulk Sample

California Sampler, 3.0 inch O.D., 2.5 inch I.D.

Shelby Tube 3.0 inch O.D.

0800, 5/31 Approximate water level first observed in boring. Time recorded in reference to a 24 hour clock.

0800, 5/31 Approximate water level observed in boring following drilling

PEN Pocket Pentrometer reading, in tsf

TV:Su Torvane shear strength, in ksf

LL	LIQUID LIMIT	TX	TRIAXIAL SHEAR
PI	PLASTICITY INDEX	CONSOL	CONSOLIDATION
%-#200	SIEVE ANALYSIS (#200 SCREEN)	R-Value	RESISTANCE VALUE
DS	DIRECT SHEAR	SE	SAND EQUIVALENT
C	COHESION (PSF)	EI	EXPANSION EQUIVLANT
PHI	FRCITION ANGLE	FS	FREE SWELL (U.S.B.R.)

Notes: Blow counts represent the number of blows a 140-pound hammer falling 30 inches required to drive a sampler through the last 12 inches of an 18 inch penetration, unless otherwise noted.

The lines separating strata on the logs represent approximate boundaries only. The actual transition may be gradual. No warranty is provided as to the continuity of soil strata between borings. Logs represent the soil section observed at the boring location on the date of drilling only.

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PROJECT NO. 56815

BORING LOG LEGEND

ARCADIA PROPERTY
Quimby Road and Capitol Expy.
San Jose, CA

PLATE

A-1

6/6/2005 10:20:57 AM

Date Completed:	5/13/05	*Sampler:*	Modified California Sampler 2.5 inch O.D., 2.0 inch I.D.
Logged By:	M.Swank		
Total Depth:	46.5 ft	*Method:*	Rotary Wash
Hammer Wt:	140 lbs., 30" drop		

Depth, ft	FIELD: Sample	FIELD: Blows/ft	LABORATORY: Dry Density pcf	LABORATORY: Moisture Content %	LABORATORY: Compress. Strength tsf	LABORATORY: Other Tests	Pen, tsf	DESCRIPTION (*Surface Elevation:* Estimated feet (Above MSL))
		8	88	24			2.0	**LEAN CLAY (CL)**- dark brown, moist, stiff, trace of organics
5		18	82	27		LL=51; PI=27	3.0	**FAT CLAY WITH SAND (CH)**- mottled dark gray, moist, very stiff, fine to coarse sand
10		15					1.5	-olive gray, stiff
15		15					1.5	-mottled olive gray
20		12					2.5	-brown, very stiff
25		18				Passing -#200=39%		**CLAYEY SAND (SC)**- dark grayish brown, moist, medium dense, some fine gravel
30								**FAT CLAY WITH SAND (CH)**- dark yellowish brown, moist, stiff, fine to coarse sand

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PROJECT NO. 56815

LOG OF BORING NO. B-1
ARCADIA PROPERTY
Quimby Road and Capitol Expy.
San Jose, CA

PLATE **A-2**

U:\GINT\PROJECT\56815.GPJ

Depth, ft	FIELD: Sample	FIELD: Blows/ft	LABORATORY: Dry Density pcf	LABORATORY: Moisture Content %	LABORATORY: Compress. Strength tsf	LABORATORY: Other Tests	Pen, tsf	DESCRIPTION
								(Continued from previous plate)
		15					1.5	
35		80				Passing -#200=18%		**WELL GRADED SAND WITH CLAY AND GRAVEL (SW-SC)**- dark yellowish brown, moist, very dense, fine to coarse sand, fine to coarse gravel
40		31				Passing -#200=84%	4.5	**SANDY LEAN CLAY (CL)**- light olive brown, moist, hard, some fine to coarse sand
45		12					2.5	-very stiff, fine to coarse sand
								Bottom of Boring No Groundwater encountered Boring backfilled with grout
50								
55								
60								

KLEINFELDER

PROJECT NO. 56815

LOG OF BORING NO. B-1
ARCADIA PROPERTY
Quimby Road and Capitol Expy.
San Jose, CA

PLATE
A-2
(cont'd)

U:\GINT\PROJECTS\56815.GPJ

Date Completed:	5/13/05	*Sampler:*	Modified California Sampler 2.5 inch O.D., 2.0 inch I.D.
Logged By:	M.Swank		
Total Depth:	46.5 ft	*Method:*	Rotary Wash
Hammer Wt:	140 lbs., 30" drop		

Depth,ft	FIELD Sample	FIELD Blows/ft	LABORATORY Dry Density pcf	LABORATORY Moisture Content %	LABORATORY Compress. Strength tsf	LABORATORY Other Tests	Pen, tsf	DESCRIPTION (*Surface Elevation:* Estimated feet (Above MSL))
		20				Passing -#200=96%	1.2	**LEAN CLAY (CL)**- dark gray, dry to moist, stiff, trace of fine grained sand, high plasticity
5		5	101	28		LL=42; PI=18		**SILT (ML)**- olive brown, moist, stiff
10		15	100	22		LL=37; PI=17	1.5	**FAT CLAY (CH)**- light olive brown, dry to moist, stiff
15		9	90	29			0.8	-wet, medium stiff
20		17					2.0	-stiff
25		24					2.0	
								POORLY GRADED SAND WITH CLAY (CL)- light olive brown, moist, medium dense
								FAT CLAY (CH)- dark gray, moist, stiff
30								

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PROJECT NO. 56815

LOG OF BORING NO. B-2
ARCADIA PROPERTY
Quimby Road and Capitol Expy.
San Jose, CA

PLATE
A-3

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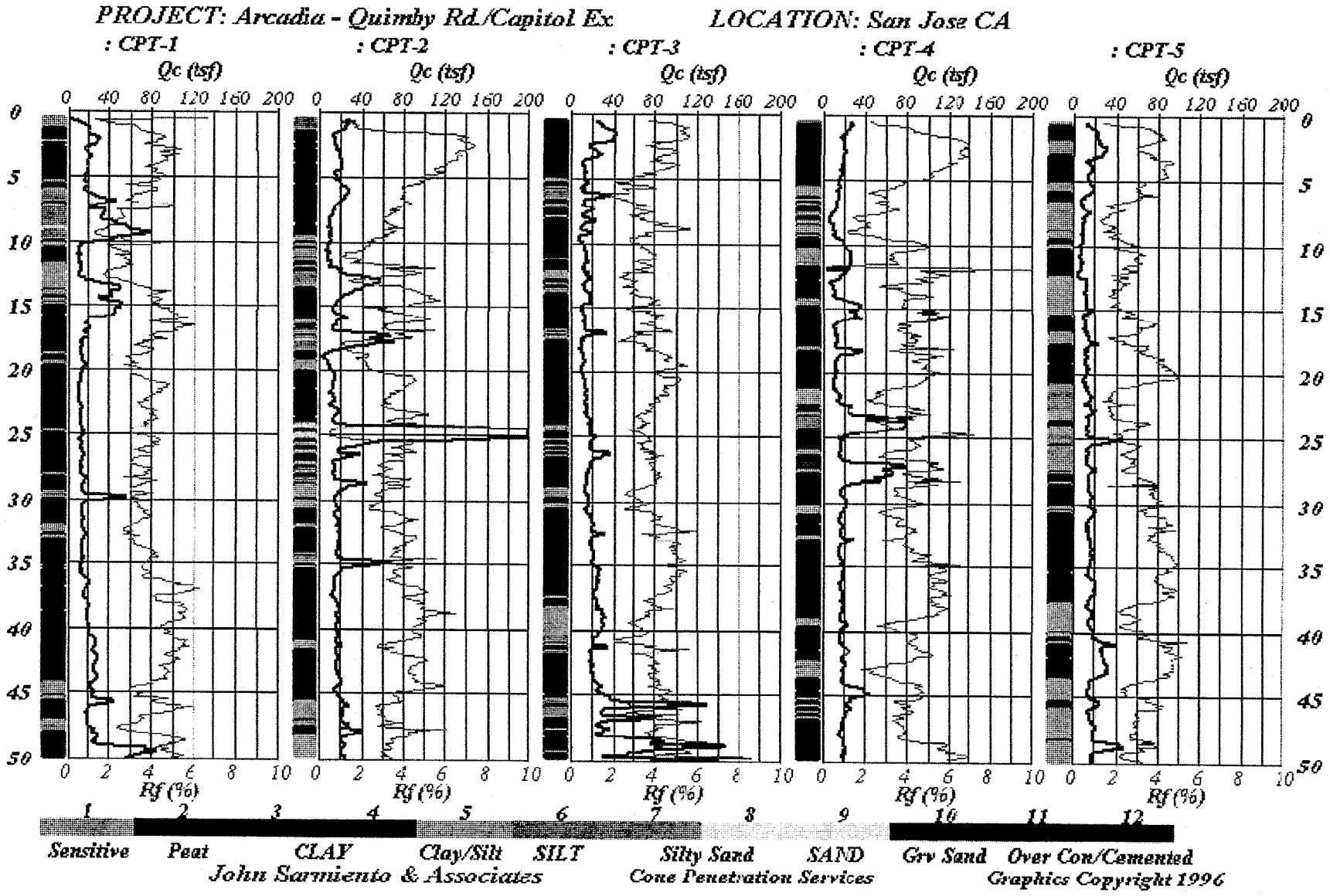

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San Jose, California

PROJECT NO.: 56815

PLATE

A-4

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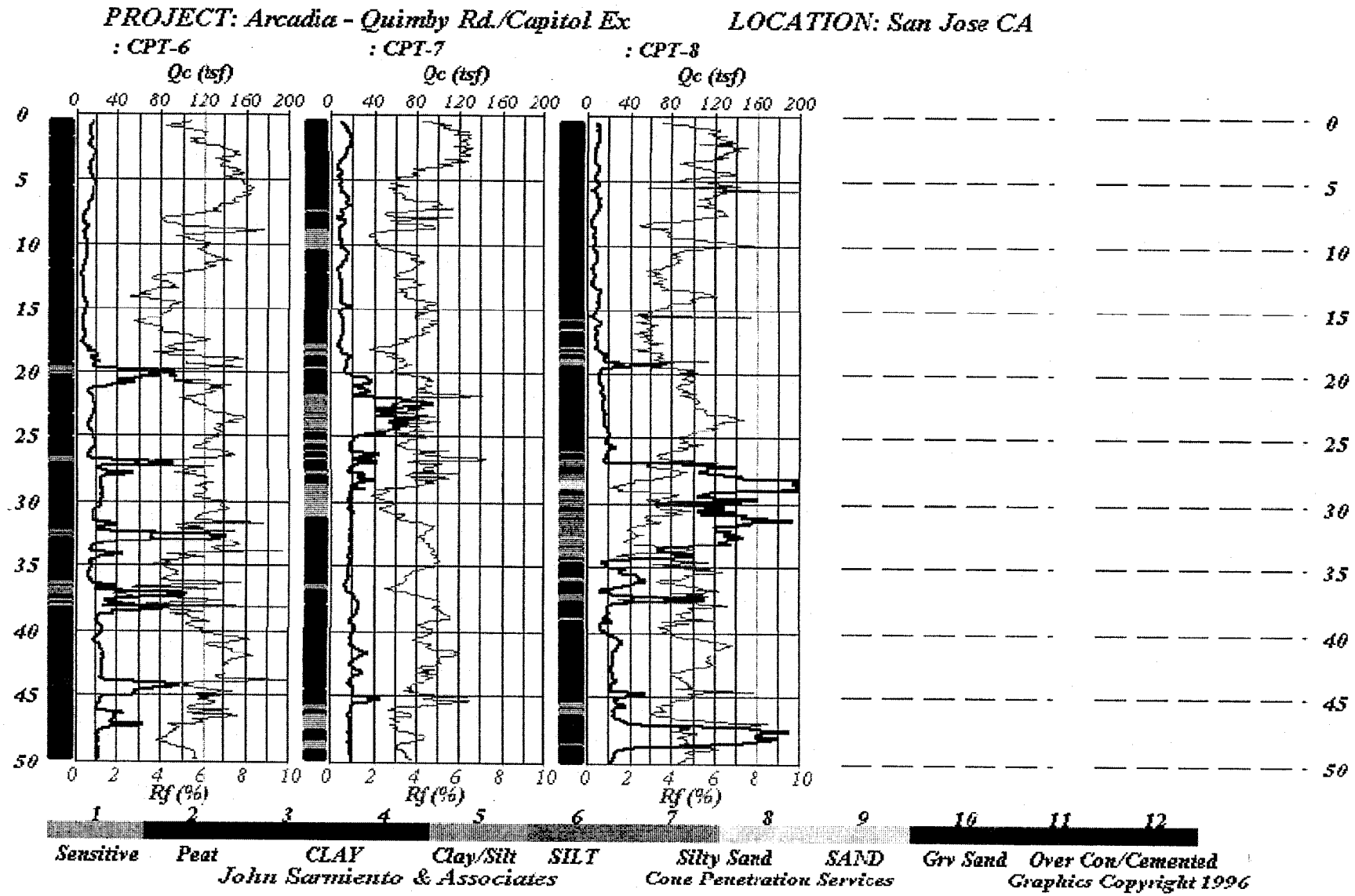

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PROJECT NO.: 56815

PLATE

A-4

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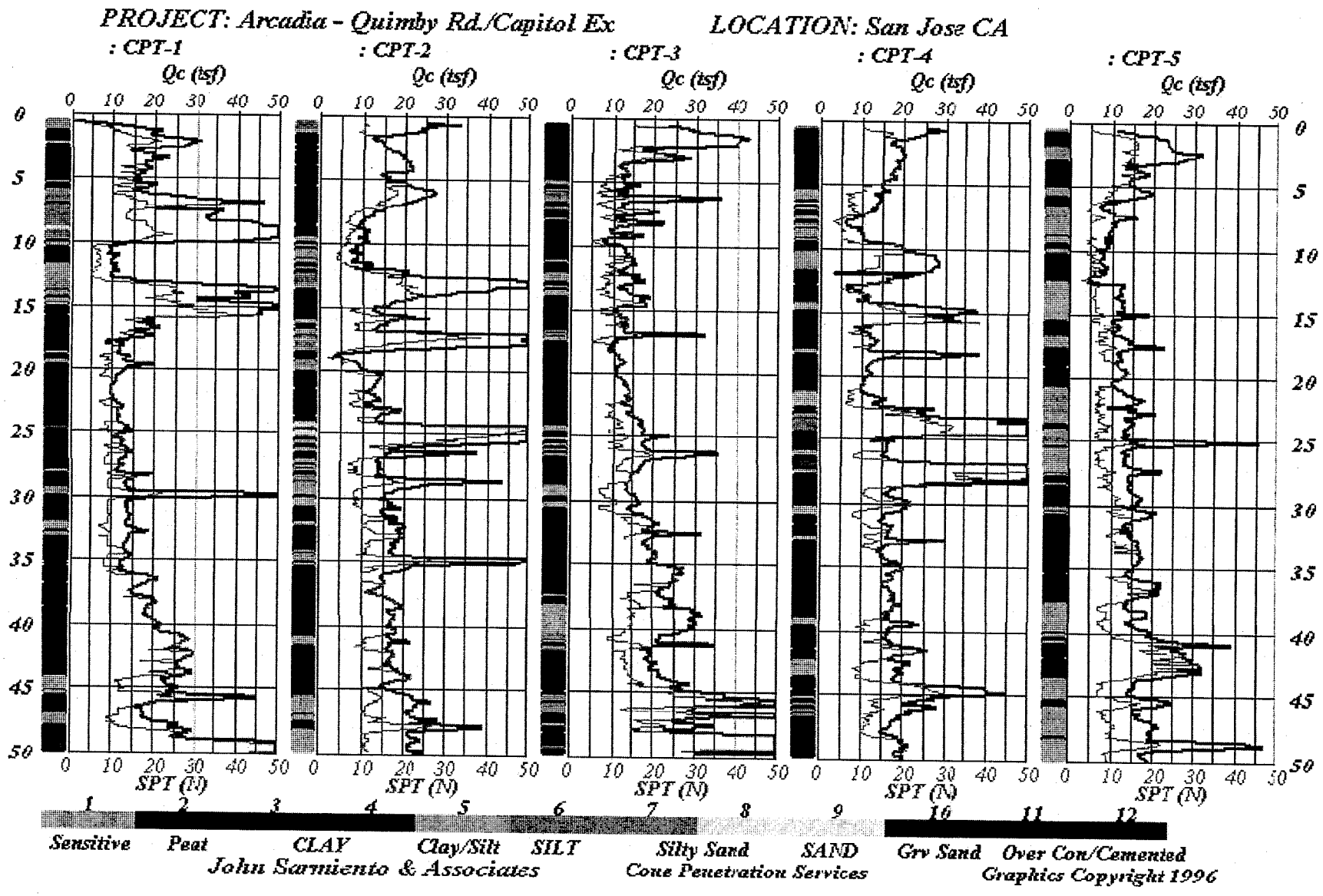

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CPT (SPT)
Arcadia
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PLATE
A-5

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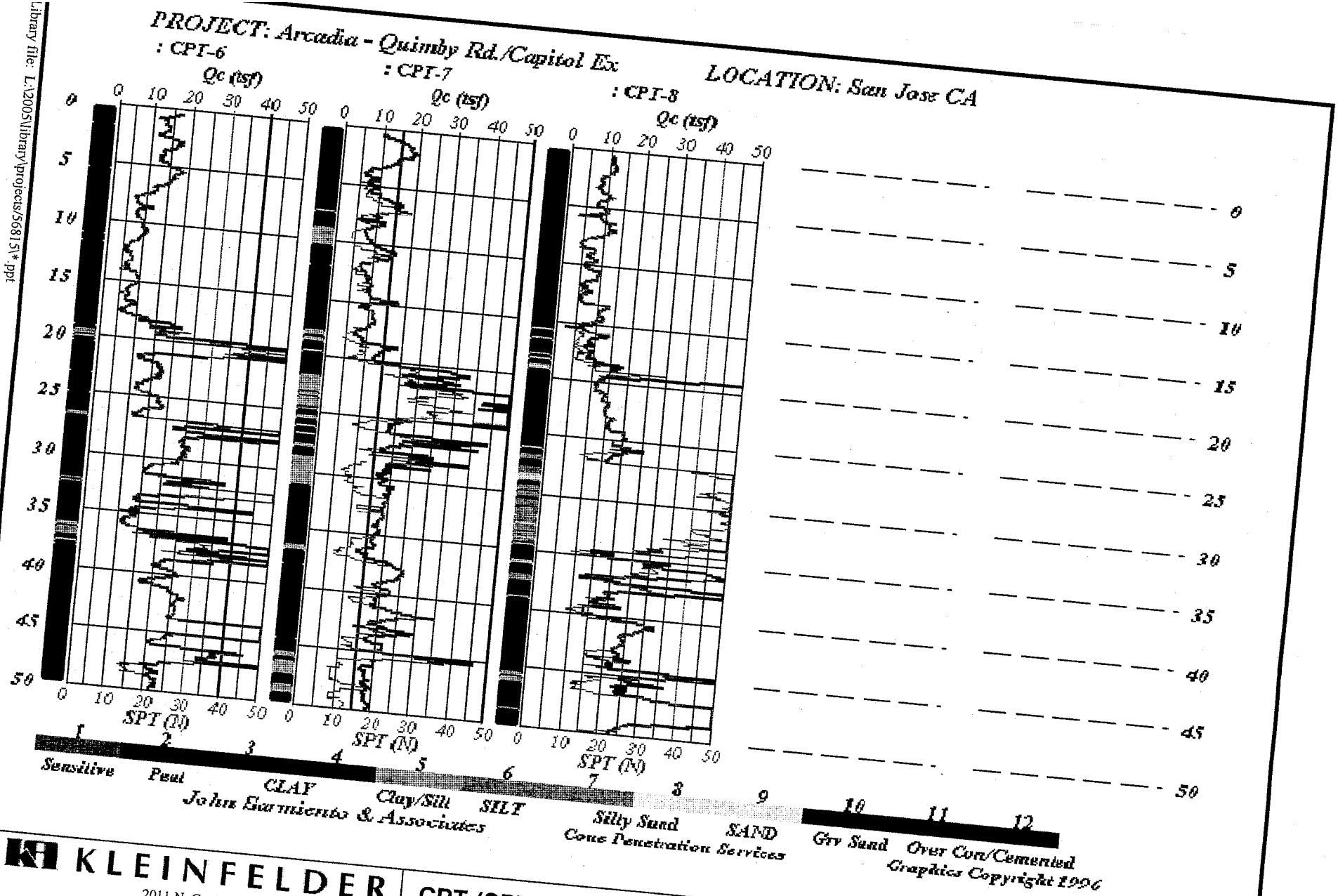

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Arcadia
Quimby Rd. & Capitol Expressway
San Jose, California

PROJECT NO.: 56815

PLATE
A-5
(cont'd)

GEOTECHNICAL INVESTIGATION
On

Pleasant Hills Golf Course
San Jose, California

For
KB HOME

By

TERRASEARCH, Inc.

Project No. 9831.G
10 December 2004

Project No. 9831.G
10 December 2004

Mr. Joe Sorte
KB Home, South Bay Inc.
6700 Koll Center Parkway
Pleasanton, CA 94566

SUBJECT: Proposed Residential Development
Pleasant Hills Golf Course Site
2050 South White Road
San Jose, California
GEOTECHNICAL INVESTIGATION

Dear Mr. Sorte:

In accordance with your authorization, *TERRASEARCH, Inc.*, has investigated the geotechnical conditions at the subject site located off South White Road in San Jose, California.

The accompanying report presents our conclusions and recommendations based on our investigation. Our findings indicate that the site is suitable, from a geotechnical standpoint, for the proposed development provided the recommendations of this report are carefully followed and are incorporated into the project plans and specifications.

Should you have any questions relating to the contents of this report or should you require additional information, please do not hesitate to contact our office at your convenience.

Very truly yours,
TERRASEARCH, Inc.,

George Makdissy, P.E.
Vice President

Robert Pollak, P.E.
Senior Engineer

Copies: 6 to KB Homes

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GEOTECHNICAL INVESTIGATION

Purpose and Scope

The purpose of our investigation for the proposed residential development to be located at the site of the *Pleasant Hills Golf Course,* in San Jose, California, was to determine the surface and subsurface soil conditions at the subject site. Based on the results of the investigation, criteria were established for grading the site, and to establish geotechnical recommendations for the proposed development. The enclosed geotechnical recommendations are based on our evaluation and investigation and on an Aerial Base Sheet by HMH Engineers, dated 12 February 2003.

Our investigation included the following:

a) Review of pertinent published geotechnical literature on the site;
b) Surface reconnaissance by the Soil Engineer;
c) Drilling and sampling of the subsurface soils at 20 locations;
d) Laboratory testing of selected soil samples;
e) Engineering analysis of the data and formulation of conclusions and recommendations;
f) Preparation of this written report.

Details of our field and laboratory field investigation are presented in Appendices A and B.

Proposed Development

The proposed development is understood to consist of single family wood-frame, detached homes and of townhouse condominiums on the subject site. The project will also include installation of underground public utilities and interior public streets. Based on the tentative site plan provided, and due to the nature of the site, it is anticipated that grading operations will consist of relatively minor grading to achieve final design grades and construct the building pads.

Site Location and Description

The irregularly shaped, relatively flat site is approximately 115 acres in areal extent and until September, 2004 was used as a privately owned, public golf course. The site is bounded by residential developments on the northwest, east and south and by Lake Cunningham Park across South White Road to the southwest. Current site access is from South White Road. The location of the site is shown on the Vicinity Map, Figure 1 of Appendix A.

Structures on site include a clubhouse, maintenance facility, and other minor structures. Numerous large, established trees line each side of the fairways and site perimeter. A pond is located southwest of the clubhouse, and a depression that was presumably excavated as a second pond is located in the northeast portion of the site. At the time of our investigation, the balance of the site was occupied by grass fairways, sand traps, an asphalt paved parking lot, and paved and unpaved access roads.

This site description is based on a reconnaissance by the Soil Engineer, the referenced aerial base sheet, and our conversations with the owner. The aerial base sheet is the basis for our "Site Plan," Figure 2 of Appendix A.

Sub-surface Conditions

The sub-surface soil conditions as encountered in our 20 test borings indicate that the near surface sub-grade soils and ground water conditions are variable across the site. The near surface soils and groundwater elevations generally follow northwest-southeast trending isolines. Highly plastic, critically expansive soil and very high, near surface groundwater occur along South White Road, gradationally changing to highly expansive soil with groundwater at 8 to 10 feet through the middle of the site. Moderately to highly expansive clays with relatively deep groundwater were encountered along the north and northeast portions of the site. Fill soil has been placed in some locations to create greens, sand traps, and tee-off locations. It may be assumed that substantial amounts of soft, saturated clay underlie the pond, and possibly the excavation located at the northeast portion of the site.

Potentially liquefiable soil types were encountered in Borings 1, 3, 17, and 19. Analysis indicates a potential for minor liquefaction induced settlements near South White Road (see below under Liquefaction Potential).

Groundwater was encountered along South White Road at depths as shallow as 3½ feet below the ground surface. No ground water was encountered at the northeast portion of the site to depths of up to 25 feet below the ground surface. Borings in intermediate areas encountered ground water at intermediate depths.

Seismic Considerations

Because of its proximity to the San Andreas Fault system, Santa Clara County is considered to be one of the most seismically active regions in the United States. Since historic records have been kept in the region, major earthquakes have been recorded on the San Andreas and Hayward Faults.

The Southeast Extension of the Hayward Fault, a type B fault, located approximately 2½ kilometers (3.1 miles) to the northeast, is the most likely fault to affect the site with potentially destructive strong ground motions, however, the San Andreas Fault, located 25½ kilometers to the southwest, and the main branch of the Hayward Fault, located 12 kilometers to the north (both type A faults) may also affect the site with strong ground motions.

Seismic Hazards

Seismic hazards can be divided into two broad classifications; 1) Primary hazards such as seismic shaking and damage produced directly from fault surface ruptures, and 2) Secondary hazards produced by seismic shaking including landslides, lurching, floods, subsidence, and liquefaction.

Primary Hazards

The project site is not within the boundaries of the Alquist-Priolo Special Studies Zone and no faults are known to lie within the site. The likelihood of a surface fault rupture occurring on this site is considered low. Based on historical evidence however, it is likely that at least one significant earthquake will produce strong ground motions at this site during the design life of the proposed structure. Structural considerations for construction on this site should include the seismic design parameters listed under UBC Seismic Design Criteria below.

Secondary Hazards

The distance of the subject site from rivers and other large bodies of water makes secondary earthquake hazards from, flooding (from tsunamis, seiches, and damn failures) highly improbable.

Liquefaction Potential

Liquefaction describes the phenomenon wherein soils lose their supportive strength and become prone to rapid settlement and loss of bearing capacity. Liquefaction occurs during earthquake conditions in saturated, relatively loose, granular soils located near the ground surface.

The data used for evaluating liquefaction potential of the subsurface soils consisted of: the penetration resistance encountered during soil sampling, the soil type, sieve analysis, and the relative density of the materials.

For each instance of potentially liquefiable soil material (granular), a corrected blow-count was obtained based on recommendations contained in *Recent Advances in Soil Liquefaction Engineering: A Unified and Consistent Framework,* from the 26th Annual ASCE Los Angeles Geotechnical Spring Seminar, April 30, 2003.

Boring	Depth of Stratum	σ_v' (atmos.)	N	C_N	C_R	C_B	$N_{1,60}$
1	43'	1.64	36	.78	.985	1.15	32
3	5'	0.30	46	1.82	.75	1.15	72
17	34'	1.31	44	.87	.97	1.15	43
17	44'	1.65	31	.78	.985	1.15	28
19	16'	0.53	50	1.37	.85	1.15	67

N = blow count from field penetration test; σ_v' = effective stress at stratum of interest; C_N = correction factor for overburden; C_R = correction factor for rod length; C_B = correction factor for bore hole diameter

Potentially liquefiable soil materials that exhibit corrected blow counts ($N_{1,60}$) greater than 30 are not known to liquefy. Based on the data obtained and our analysis, the potential for liquefaction was precluded for all strata of interest with the exception of Boring 17 at 44 feet below the ground surface.

A peak ground acceleration value of 0.56g was used in our analysis. This value was based on the *California Geological Survey Probabilistic Seismic Hazards Mapping Ground Motion Study* based on a latitude of 37.340464° and a longitude of -121.804697° for 10% exceedance for "Soft Rock Conditions".

Cyclic Stress Ratio (CSRE) = 0.65 x a_{max}/g x total stress/effective stress x rd[1].
[1]stress reduction factor

B-17

Stratum	Depths (feet)	Soil Type	%<#200 Sieve	Blow Counts $(N_1)_{60}$	CSRE	CSR[1]	f.s.	liquefy	Settlement (%)	Settlement (total) inches
1	38 –48	SM	9.4	28	0.54	0.50	<1	y	0.9	1.08

1. From Seed et. Al. 1984

Conclusions:

1. Based on our analysis, there is a potential for liquefaction induced settlements on this site in the event of a major seismic event.
2. Prudent structural and civil design will anticipate total settlements on this site up to 1.08 inches with differential settlements of 0.70 inches.
3. Due to the relatively flat site conditions and the depth to the potentially liquefiable material, lateral spreading is considered very unlikely on this site.

Seismic Conclusions

The most significant seismic hazards are those of shaking. Additionally, some liquefaction induced settlements should be anticipated, particularly in those areas adjacent to South White Road. A prudent design will anticipate up to 1.08 inches of settlement during a significant seismic event. Any structural design should incorporate the current state of practice for seismic loads listed in UBC Seismic Design Criteria and the potential for seismically induced settlements as described above.

UBC Seismic Design Criteria

The 1997 Uniform Building Code, Chapter 16, Division IV Earthquake Design requires near-source factors to be used for sites in Seismic Zone 4 that are within certain distances of critical faults. In 1998, the International Conference of Building Officials (ICBO) published a map folio to be used in scaling distances to the critical faults. According to this map folio, the site is located approximately 2½ km from a Type B fault, the southeast extension of the Hayward Fault, and approximately 12 km from the main trace of the Hayward Fault, a Type A fault.

Based on Tables 16-R, S and T of the 1997 Uniform Building Code and the data presented in this report, a summary of the earthquake design criteria for use in the design of the proposed structures is as follows:

Seismic Zone	=	4
Soil Profile Type	=	S_D
Near Source Factor, N_a	=	1.25
Near Source Factor, N_v	=	1.55

DISCUSSIONS, CONCLUSIONS, AND RECOMMENDATIONS

General

1. From a geotechnical perspective, the site is suitable for construction of the proposed development provided the recommendations presented in this report are incorporated into the project plans and specifications.

2. The proposed residential structures may be supported on a structural mat or post tensioned slab foundation system. Specific recommendations for both are provided in the "foundation" section herein.

3. The most significant geotechnical factors affecting the site are the presence of high groundwater over the southern and western portions of the site, the presence of highly to critically expansive clay soils, and the potential for liquefaction induced settlements.

Demolition

4. Prior to any grading, demolition of the site should be completed. This should include the Demolition of any structures and trees designated by the owner for removal. Demolition should include the complete removal of all subsurface structures, if any, and any concrete, tanks, pipe inlets, foundations, debris and trash, which may be encountered. It is recommended that any known underground structures be located on the grading plans so that proper removal may be carried out. *TERRASEARCH, Inc.*, should intermittently observe demolition and should be notified a minimum of two working days in advance prior to any demolition and/or grading operation to properly coordinate the work with the contractor(s).

5. Excavations resulting from demolition operations should be properly backfilled with engineered fill under the observation of the Soil Engineer. Should the location of any localized excavation be found to underlie any structure, backfill should be compacted to a minimum relative compaction of 95%, or the excavation should be widened to extend 5 feet beyond the footprint of the structure and backfilled to the specifications for engineered fill as recommended in the following "grading" section.

6. The grading is expected to consist primarily of cut and fill operations to achieve design grades and to construct the building pads, and the anticipated backfilling of the excavated area in the northeast portion of the site. It is our understanding that the existing "lake" will remain. Grading requirements presented herein are an integral part of the grading specifications presented in Appendix C of this report and should be considered as such.

7. Grading activities during the rainy season will be hampered by excessive moisture. Grading activities may be performed during the rainy season, however, achieving proper compaction may be difficult due to excessive moisture; and delays may occur. Grading performed during the dry months will minimize the occurrence of the above problems.

High Ground Water

8. Our exploratory borings encountered high groundwater in the southwest and south portions of the site. High groundwater can interfere with grading operations including soil excavation, compaction and trenching operations. Additional, non-geotechnical problems can be associated with high groundwater including site storm drainage system problems, landscaping difficulties, and perennial soft, wet areas. To reduce these problems, consideration should be given to using fill soil to elevate the affected areas.

Grading & Existing Fill Soil

9. It is anticipated that the minor amounts of fill soil used to construct sand traps, tee-off areas, etc., will be removed during grading operations and construction of the building pads, The observed fill was shallow, well placed, and need not be removed, however, if any loose or soft areas are encountered during the grading operations, they must be removed and replaced as engineered fill. Materials generated from loose/soft soils may be used as engineered fill with the approval of the Soil Engineer provided they do not contain debris

10. The requirement and degree of stripping will depend on site conditions at the commencement of grading operations and may depend on the time of the year grading operations are conducted and on the site preparations performed prior to the commencement of grading. If excessive vegetation is present at the time of grading, the surface of the site in areas containing that vegetation should be stripped to remove the vegetation and/or other deleterious material. The need and degree of stripping will be determined in the field by the Soil Engineer at the time the grading operations commence. For bidding purposes it may be assumed that 2 inches of stripping will be required for those areas currently containing grassy vegetation.

11. Following removal of any loose and/or soft soil, the top 8 inches of exposed ground for fill areas should be scarified and compacted to a minimum degree of relative compaction of 90% at a moisture content above optimum as determined by ASTM D1557-98 Laboratory Test Procedure. After recompacting the subgrade, the site may be brought to the desired finished grades by placing engineered fill in lifts not to exceed 8 inches in uncompacted thickness and compacted to the relative compaction requirements in accordance with the aforementioned test procedure. For any areas that will receive in excess of 8 feet of fill, all fill placed below 8 feet must be compacted to a minimum relative compaction of 95% as determined by ASTM D1557-98 Laboratory Test Procedure All soils encountered during our investigation would be suitable for use as engineered fill when placed and compacted at the recommended moisture content.

12. Should select import material be required to establish the proper grading for the proposed development, the import material should be approved by the Soil Engineer before it is brought to the site and should meet the following requirements:

a. Have an R-Value of not less than 25;
b. Have a Plasticity Index not higher than 12;
c. Not more than 15% passing the No. 200 sieve;
d. No rocks larger than 6 inches in maximum size.

13. Import material meeting the requirements stated above should be compacted to the requirements stated above. In addition, import should be placed in such a way as to provide uniformity beneath all structural areas. No ponding of storm water is to be permitted on cut or fill pads during prolonged periods of inclement weather.

14. Should any building encompass a cut/fill pad, the cut area should be over-excavated to provide a minimum of two feet of uniform fill below the foundation. Over-excavation is necessary to minimize the effects of differential movement.

Foundations

15. The proposed structures should be supported on either structural mat slab or post-tensioned slab foundation systems.

16. Structural mats and post tensioned slabs may be designed based for expansive soil conditions as described below. Additionally, structural mats and post tensioned slabs should be designed to accommodate differential settlements of up to 0.7 inch due to liquefaction induced settlements. Differential settlements are measured from the corner of the slab to the center.

17. Post-tensioned slabs and structural mat slabs should have a minimum thickness of 10 inches.

Structural Mat

18. Structural mats may be designed using the method presented in the 1997 Uniform Building Code, Chapter 18, Division III, Section 1815, Design of Slab-on-Grade Foundations.

19. Based on the above, it is recommended that the structural mat foundation be designed based on an effective Plasticity Index of 35.

Post-Tensioned Slab Foundation

20. Post-tensioned slabs should be a minimum 10 inches in thickness and designed using the following criteria which is based on the design method of the 1997 Uniform Building Code, Chapter 18, Division III, Sections 1816 and 1817, Design of Post-Tensioned Slabs on Ground:

Plasticity Index	=	35
Allowable Bearing Capacity	=	2,500 p.s.f.
Depth to Constant Moisture	=	5 feet
Percent Passing #200	=	70%
Edge Moisture Variation Distance:		
Edge Lift	=	3.0 feet
Center Lift	=	5.0 feet
Differential Swell:		
Y_m (Edge Lift)	=	0.53 inches
Y_m (Center Lift)	=	1.70 inches

General Slab Construction Requirements

21. Slabs may be constructed at pad grade. The perimeter of the slab should be thickened to bear on the prepared building pad and to confine the sand. A 10 mil Visqueen-type membrane should be placed between the prepared subgrade and the slab to provide an effective vapor retarder, and to minimize moisture condensation under the floor covering. The vapor retarder membrane shall be lapped adequately to provide a continuous vapor retarding barrier under the entire slab. Care must be taken to assure that the membrane does not become torn and entangled with the reinforcing. A minimum of two inches of wetted sand should be placed over the vapor retarder to act as a cushion to protect the membrane and to facilitate curing of the concrete. During winter construction, sand may become saturated due to rainy weather prior to pouring. Saturated sand is not desirable because there exists a high probability of creating sand pockets within the slab section during the concrete pour. As an alternative, a sand-fine gravel mixture that is stable under saturated conditions may be used. However, the material must be approved by the Soil Engineer prior to use. Alternatively, the sand may be omitted and the concrete placed directly on the Visqueen, provided the concrete mix used has a maximum water/cement ratio of 0.45, and two layers of 10 mil Visqueen or one layer of 20 mil Visqueen vapor barrier is used.

22. The slabs should be adequately reinforced as determined by the project structural engineer. The reinforcement shall be placed in the center of the slab unless otherwise designated by the design engineer.

23. All flatwork slabs should be poured structurally independent of the foundations. A 30-pound felt strip, expansive joint material, or other positive separator should be provided around the edge of all floating slabs to prevent bond to the structure's foundation.

24. To reduce the effects of expansive soil, moisture conditioning of pads containing expansive soil is recommended. The upper 12 to 18 inches of soil sub-grade for those pads containing expansive soil should be saturated until a moisture equilibrium is achieved prior to pouring concrete. The Soil Engineer should observe and verify the pad sub-grade soil saturation before the slabs are poured. Typically, 12 inches of penetration with a thin metal probe may indicate sufficient moisture conditioning. The need for moisture conditioning should be determined by the Soil Engineer on a pad by pad basis during grading operations.

Pavement Design

25. After underground facilities have been placed in the areas to receive pavement and removal of excess material has been completed, the upper 6 inches of the subgrade soil should be scarified, moisture conditioned and compacted to a minimum relative compaction of 95% at a moisture content above optimum in accordance with the grading recommendations specified in this report.

26. All aggregate base material placed subsequently should also be compacted to a minimum relative compaction of 95% based on the ASTM D1557-98 Test Procedure. The construction of the pavement in the traffic areas should conform to the requirements set forth by the latest Standard Specifications of the Department of Transportation of the State of California and/or City of San Jose, Department of Public Works.

27. Since grading is anticipated to consist of cuts and fills, it is difficult to determine what type of soils will comprise the street subgrade in order to perform R-Value testing. Additionally, soil conditions effecting R-values are variable across the site and generally improve with distance from South White Road. However, for design purposes, R-Values ranging from 6, near South White Road, to 16 at the northeast portion of the site can be assumed. The recommended design thicknesses presented in Table 1 were calculated in accordance with the methods presented in Topic 608 of the California Department of Transportation Highway Design Manual" based on an **R-value of 6** and presents the worst likely case. During grading operations, representative samples of actual subgrade soil should be collected and tested to determine the actual R-Value's so that a final design may be obtained.

TABLE 2
Recommended Asphalt Concrete Pavement Sections

Design Traffic Index	Asphalt Concrete Type B (inches)	Aggregate Base Class II[1] (inches)
4.5	2.5	9.5
5.0	3.0	10.0
5.5	3.0	12.0

Notes:
(1) Minimum R-Value = 78
(2) R-Value = Resistance Value
(3) All layers in compacted thickness to Cal-Trans Standard Specifications

28. Where planters are planned within or adjacent to a pavement area, provisions should be made to prevent irrigation water from entering the pavement sub-grade. Foliage requiring minimal irrigation be considered. Water entering the pavement section at sub-grade level could cause softening of this zone and subsequently pavement failure will occur. It is recommended that landscape islands adjacent to pavement be equipped with a sub-drain system that discharges to a location approved by the project Civil Engineer.

29. Concrete driveway slabs on expansive clay soil are susceptible to damage, particularly if careful site drainage isn't established and maintained by the owner. Use of crushed rock to cushion the slab may introduce more water into the sub-grade, thus exacerbating the problem. Driveway slab performance can be improved with increased thickness of the concrete slab, increased reinforcement as directed by the project structural engineer, improved soil sub-grade materials, improved surface drainage, or sub-drain trenches placed adjacent to the driveways. The Soil Engineer should review the final grading plan to determine the appropriate measures for specific field conditions.

Retaining Walls

30. Any retaining walls that are to be incorporated into the development should be designed to resist lateral pressures exerted from a media having an equivalent fluid weight as follows.

TABLE 1 - Recommended Lateral Pressures for Retaining Wall Design

Gradient of Back Slope	Equivalent Fluid Weight (p.c.f.)		Coefficient of Friction
	Unrestrained Condition (Active)	Passive Resistance	
Horizontal	50	275	0.30
2:1	70	275	0.30

In addition, restrained retaining walls should be designed to resist an additional uniform pressure of 100 psf over the entire height of the wall.

31. Active conditions occur when the top of the wall is free to move outward. At-rest conditions apply when the top of the wall is restrained from any movement. It should be noted that the effects of any surcharge or compaction loads behind the walls must be accounted for in the design of the walls.

32. The above criteria are based on fully drained conditions. If drained conditions are not possible, then the hydrostatic pressure must be included in the design of the wall. A linear distribution of hydrostatic pressure of 63 p.c.f. should be adopted.

33. In order to achieve fully-drained conditions, a drainage filter blanket should be placed behind the wall. The blanket should be a minimum of 12 inches thick and should extend the full height of the wall to within 12 inches of the surface. If the excavated area behind the wall exceeds 12 inches, the entire excavated space behind the 12-inch blanket should consist of compacted engineered fill or blanket material. The drainage blanket material may consist of either granular crushed rock and drain pipe fully encapsulated in geotextile filter fabric or Class II permeable material that meets CalTrans Specification, Section 68, with drainage pipe, and optional fabric. A 4-inch perforated drain pipe should be installed in the bottom of the drainage blanket and should be underlain by at least 4 inches of filter type material. A 12-inch cap of native soil material should be placed over the drainage blanket.

34. The drainage blanket for any retaining wall that is part of a residence should extend to at least 8 inches below the bottom of the slab.

35. Piping with adequate gradient shall be provided to discharge water that collects behind the walls to an adequately controlled discharge system away from the structure foundation.

36. As an alternate to the 12-inch drainage blanket, a prefabricated strip drain (such as Miradrain) may be used between the wall and retained soil. In this case, the wall must be designed to resist an additional lateral hydrostatic pressure of 63 p.c.f. equivalent fluid weight.

37. Incidental retaining walls may be founded on a spread footing foundation. Spread footings should have a minimum depth of 18 inches below the lowest adjacent pad grade (i.e. trenching depth). Design bearing pressures for footings should not exceed 2,500 p.s.f. due to dead plus sustained live loads and 3,200 p.s.f. due to all loads which include wind or seismic. Retaining walls that are part of a residence may be founded directly on the structural mat or post-tensioned slab.

38. The above spread footing foundation design criteria apply when the front face of the footing or keyway is at least 10 feet horizontally away from the surface of a slope at the foundation level. Design criteria for footings closer than 10 feet will be provided for the specific situation.

Utility Trenches

39. Utility trench operations in the south and southwest portions of the site may be hampered by high groundwater (see High Ground Water; item no. 8).

40. Applicable safety standards require that trenches in excess of 5 feet must be properly shored or that the walls of the trench slope back to provide safety for installation of lines. If trench wall sloping is performed, the inclination should vary with the soil type. The underground contractor should request an opinion from the Soil Engineer as to the type of soil and the resulting inclination.

41. With respect to state-of-the-art construction or local requirements, utility lines are generally bedded with granular materials. These materials can convey surface or subsurface water beneath the structures. It is, therefore, recommended that all utility trenches which possess the potential to transport water be sealed with a compacted impervious cohesive soil material or lean concrete where the trench enters/exits the building perimeter. This impervious seal should extend a minimum of 2 feet away from the building perimeter and must be observed and approved by the Soil Engineer..

42. Utility trenches extending underneath all traffic areas must be backfilled with native or approved import material and compacted to a minimum relative compaction of 90% at a moisture content above optimum to within 6 inches of the subgrade. The upper 6 inches should be compacted to a minimum of 95% relative compaction in accordance with Laboratory Test Procedure ASTM D1557-98. Backfilling and compaction of these trenches must meet the requirements set forth by the City of San Jose, Department of Public Works. Utility trenches within landscape areas may be compacted to a relative compaction of 85%.

General Construction Requirements

43. Liberal lot slopes and drainage must be provided by the project Civil Engineer to remove all storm water from the pads and to prevent storm and/or irrigation water from seeping beneath the structures. Should surface water be allowed to seep under the structures, foundation movement resulting in structural damage will occur. All compacted, finished grades should be sloped at a minimum 2% gradient away from the exterior foundation for a distance of 3 feet.

Should the recommended surface drainage be altered by the property owner, then a subdrain system should be constructed around the perimeter of the residence. Specific recommendations for sub-drain construction will be provided upon request.

44. Roof gutters and downspouts are recommended to carry storm water away from the structures and graded areas and, thus, reduce the possibility of soil saturation adjacent to the foundations.

45. Flower beds or planters are not recommended adjacent to the building foundations because of the possibility of irrigation water affecting the foundations or slabs. Should planters be constructed, foliage requiring little irrigation should be planted. Planters adjacent to the buildings should be equipped with drainage inlets that discharge to a location approved by the project Civil Engineer. It is preferred that irrigation adjacent to the building foundations consist of a drip system. Sprinkler systems may be used; however, it is preferred that sprinkler heads do not water closer than 3 feet from the building foundations. If sprinklers are used within 3 feet, then excessive watering should not be allowed; and good surface drainage in the planter area must be provided. In any case, it is recommended that area surface drains be incorporated into the landscaping to discharge any excessive irrigation or rainwater that may accumulate in the planter area. These surface drains must be constructed in a manner that easy flow of surface water runoff is allowed into the pipe inlets.

Project Review and Construction Monitoring

46. All grading and foundation plans for the development must be reviewed by the Soil Engineer prior to contract bidding or submitted to governmental agencies so that plans are reconciled with soil conditions and sufficient time is allowed for suitable mitigative measures to be incorporated into the final grading specifications.

47. *TERRASEARCH, Inc.*, should be notified at least two working days prior to site clearing, grading, and/or foundation operations on the property. This will give the Soil Engineer ample time to discuss the problems that may be encountered in the field and coordinate the work with the contractor.

48. Field observation and testing during the grading and/or foundation operations must be provided by representatives of *TERRASEARCH, Inc.*, to enable them to form an opinion regarding the adequacy of the site preparation, the acceptability of fill materials, and the extent to which the earthwork construction and the degree of compaction comply with the specification requirements. Any work related to the grading and/or foundation operations performed without the full knowledge and under the direct observation of the Soil Engineer will render the recommendations of this report invalid. This does not imply full-time observation. The degree of observation and frequency of testing services would depend on the construction methods and schedule, and the item of work. Please refer to "Guidelines For Required Services" for an outline of our involvement during project development.

49. Should another geotechnical consultant be engaged to perform project review and/or construction monitoring, then *TERRASEARCH, Inc.*, must receive a letter of indemnification releasing us of any responsibility on the project.

GUIDELINES FOR REQUIRED SERVICES

The following list of services are the services required and must be provided by *TERRASEARCH, Inc.*, during the project development. These services are presented in check list format as a convenience to those entrusted with their implementation.

The items listed are included in the body of the report in detail. This list is intended only as an outline of the required services and does not replace specific recommendations and, therefore, must be used with reference to the total report. The degree of observation and frequency of testing services would depend on the construction methods and schedule, and the item of work.

The importance of careful adherence to the report recommendations cannot be overemphasized. It should be noted, however, that this report is issued with the understanding that each step of the project development will be performed under the direct observation of *TERRASEARCH, Inc.*

The use of this report by others presumes that they have verified all information and assume full responsibility for the total project.

	Item Description	Required	Not Required	Not Anticipated
1.	Provide foundation design parameters	X		
2.	Review grading plans and specifications	X		
3.	Review foundation plans and specifications	X		
4.	Observe and provide recommendations regarding demolition	X		
5.	Observe and provide recommendations regarding site stripping	X		
6.	Observe and provide recommendations on moisture conditioning, removal, and/or precompaction of unsuitable existing soils	X		
7.	Observe and provide recommendations on the installation of subdrain facilities			X
8.	Observe and provide testing services on fill areas and/or imported fill materials	X		
9.	Review as-graded plans and provide additional foundation recommendations, if necessary	X		
10.	Observe and provide compaction tests on sanitary sewers, storm drain, water lines and PG&E trenches (if not done by city)	X		
11.	Observe foundation excavations and provide supplemental recommendations, if necessary prior to placing concrete	X		
12.	Observe and provide moisture conditioning recommendations for foundation areas prior to placing concrete	X		
13.	Provide design parameters for retaining walls	X		
14.	Provide observations and recommendations for keyway excavations and cutslopes during grading			X
15.	Excavate and recompact all geologic trenches and/or test pits		X	
16.	Observe installation of subdrains behind retaining walls	X		

LIMITATIONS AND UNIFORMITY OF CONDITIONS

1. It should be noted that it is the responsibility of the owner or his representative to notify *TERRASEARCH, Inc.*, in writing, a minimum of two working days before any clearing, grading, or foundation excavations can commence at the site.

2. The recommendations of this report are based upon the assumption that the soil conditions do not deviate from those disclosed in the borings and from a reconnaissance of the site. Should any variations or undesirable conditions be encountered during the development of the site, *TERRASEARCH, Inc.*, will provide supplemental recommendations as dictated by the field conditions.

3. This report is issued with the understanding that it is the responsibility of the owner, or his representative, to ensure that the information and recommendations contained herein are brought to the attention of the Architect and Engineer for the project and incorporated into the plans and that the necessary steps are taken to see that the Contractor and Subcontractors carry out such recommendations in the field.

4. At the present date, the findings of this report are valid for the property investigated. With the passage of time, significant changes in the conditions of a property can occur due to natural processes or works of man on this or adjacent properties. In addition, legislation or the broadening of knowledge may result in changes in applicable standards. Changes outside of our control may render this report invalid, wholly or partially. Therefore, this report should not be considered valid after a period of two (2) years without our review, nor should it be used, or is it applicable, for any properties other than those investigated.

5. Not withstanding, all the foregoing applicable codes must be adhered to at all times.

APPENDIX A

Field Investigation

Vicinity Map

Site Plan

Logs of Test Borings

Boring Log Legend and Notes

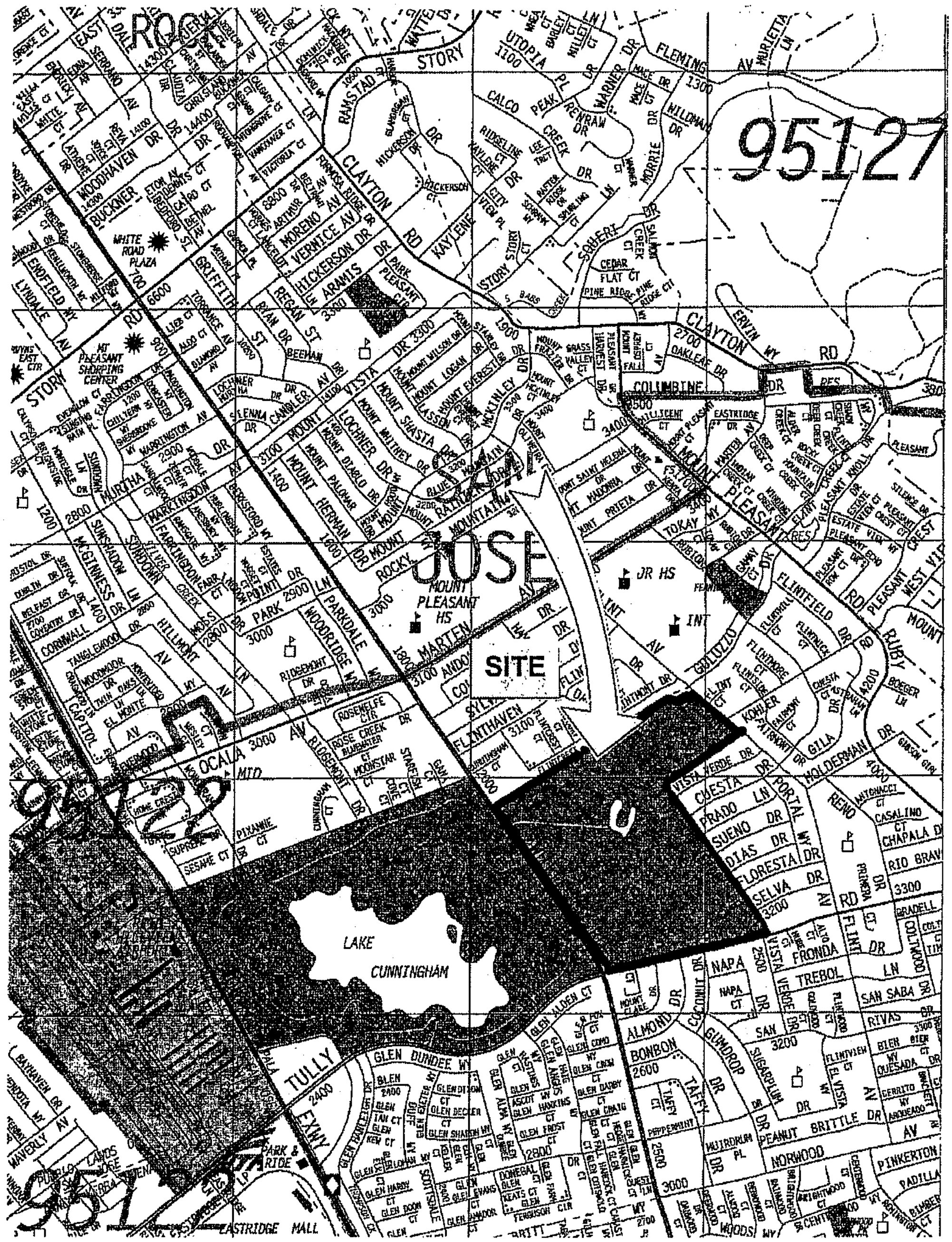
FIELD INVESTIGATION

The field investigation was performed on 4 and 5 November 2004, and included a reconnaissance of the site and the drilling of 20 exploratory borings at the approximate locations shown on Figure 2, "Site Plan."

The borings were drilled to a maximum depth of 50 feet below the existing ground surface. The drilling was performed with a Mobile B53 truck mounted drilling equipment using power-driven, 8-inch diameter, hollow stem augers. Visual classifications were made from auger cuttings and the samples in the field. As the drilling proceeded, relatively undisturbed core samples were obtained by means of a 2.5 inch O.D. Modified California split-tube sampler containing 2 inch O.D. brass liners. The sampler was advanced into the soils at various depths under the impact of a 140-pound hammer having a free fall of 30 inches. The number of blows required to advance the sampler 12 inches into the soil, after seating the sampler 6 inches, were adjusted to the standard penetration resistance (N-Value). The final N-value in boring 1 was taken with an SPT sampler.

The samples were sealed and returned to our laboratory for testing. Classifications made in the field were verified in the laboratory after further examination and testing.

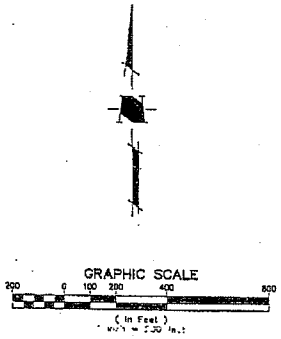
The stratification of the soils, descriptions, location of undisturbed soil samples and standard penetration resistance are shown on the respective "Logs of Test Borings" contained within this appendix.



BASE: Thomas Guide

1 inch =1900 feet

Project No. 9831.G	December 2004	LOCATION MAP Pleasant Hills Golf Course Site San Jose, California	FIGURE NO. 1
TERRASEARCH, inc.			



B - 1 Boring Location (Typical 20)

Base Map by HMH Engineers

Project No. 9831.G	December 2004	**Pleasant Hills Golf Course**	FIGURE 2
TERRASEARCH, inc. San Jose, California (408) 362-4920		South White Road San Jose, California	

GEOLOGIC HAZARDS ASSESSMENT AND FAULT INVESTIGATION EVERGREEN I AND II FOWLER AND YERBA BUENA ROADS SAN JOSE, CALIFORNIA

Kleinfelder Project No.: 49205

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GEOLOGIC HAZARDS ASSESSMENT AND
FAULT INVESTIGATION
EVERGREEN I AND II
FOWLER AND YERBA BUENA ROADS
SAN JOSE, CALIFORNIA

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GEOLOGIC HAZARDS ASSESSMENT AND FAULT INVESTIGATION EVERGREEN I AND II FOWLER AND YERBA BUENA ROADS SAN JOSE, CALIFORNIA

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GEOLOGIC HAZARDS ASSESSMENT AND
FAULT INVESTIGATION
EVERGREEN I AND II
FOWLER AND YERBA BUENA ROADS
SAN JOSE, CALIFORNIA

1.0 INTRODUCTION

This report presents the results of Kleinfelder's geologic hazards assessment and fault investigation for the Evergreen I and II properties located in southeastern San Jose, California. As shown on the Site Vicinity Map, Plate 1 and the Site Aerial Photograph, Plate 2, the project consists of two properties totaling approximately 200-acres. Evergreen I occupies 90.584 acres that lay adjacent to and north of Fowler Road (Site Plan and Geologic Map, Evergreen I, Plate 3). Evergreen II is approximately 115 acres +/- located southeast of the intersection of Yerba Buena and Fowler Roads (Site Plan and Geologic Map, Evergreen II, Plate 4).

1.1 PROPOSED DEVELOPMENT

At the time of this investigation the type of development of the site has not been determined. The design and layout of site structures, utilities and other infrastructure have not been completed. We understand that two debris basins are proposed to be constructed along the east margins of Evergreen I and II but that their final design and placement have not been determined.

The Site Plan indicates that the property ranges in elevation from about 485 feet above mean sea level where the south fork of Fowler Creek meets Yerba Buena Road to about 555 feet where Fowler Creek enters Evergreen II from the east. The USGS 7.5 Minute San Jose East Quadrangle (USGS, 1980) indicates the approximate center of the site is located, using the North American Datum of 1983 (NAD83), at coordinates:

Latitude: 37.31386° N Longitude: -121.75694° W.

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1.2 SITE DESCRIPTION

Both Evergreen properties consist of relatively flat terrain that slopes down gently from east to west at a gradient of about 1% to 5%. East of the properties the ground surface rises more steeply to northwest-trending ridge crests. The properties are on the margins of the Santa Clara Valley, which lies to the southwest.

Evergreen I is bounded on the north by Aborn Road, on the west by Fowler Creek Park and a residential development, on the south by Fowler Road, and on the east by grazing lands of the Walls Ranch. The property is fenced by six-foot high chain link and barbed wire. A few oak trees are scattered along the channel of Fowler Creek. There are no structures on site. Utility lines consisting of underground water lines and aboveground power lines cross the site which currently is pasture for cattle grazing. An active water well is located about 150 feet north of Fowler Road and about 175 feet east of the City of San Jose water tank. The City's water tank is a partially underground, metal roofed tank enclosed in a chain-link fence at the northwest corner of Fowler and Yerba Buena Roads.

Evergreen II is bounded on the north by Fowler Road, on the west by Yerba Buena Road, on the south by property of Investment Development Services (IDS) and the Behring Diagnostic Campus (formerly Syntex Evergreen) and on the east by additional property of Berg & Berg Enterprises. Several unimproved and paved roads cross the property. The road that enters the property from Yerba Buena Road opposite Altia Avenue (Site Aerial Photograph) accesses the City of San Jose Municipal Water Services Evergreen Zone ¾ Water Tank by way of a concrete bridge over the south fork of the Fowler Creek. Between this access road and the south property line, a residential structure and fenced yard that appears to be used for storage of large equipment is located adjacent to Yerba Buena Road. Another occupied residence is located in the north-central portion of Evergreen II and is accessed by an unpaved road from Fowler Road.

At the time of our site visits, Evergreen II was used for grazing. The surface of the property was covered with grasses and scattered brush. Trees of an abandoned orchard along with native oak and brush are located in the northeast corner of the property where the south fork of Fowler Creek flows onto the property. A large pile of soil and rock occupies an area in the southeast portion of the site (Site Plan and Geologic Map, Evergreen II). We understand that the

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stockpiled soil was generated during grading and development of the City of San Jose Municipal Water Services Evergreen Zone ¾ Water Tank located on a narrow ridge east of the Evergreen II property.

Two intermediate stream channels cross the Evergreen I and II properties. Fowler Creek flows from a canyon mouth east of the Evergreen I property line and forms a channel that dissects the property from near the barn on the Walls Ranch property to near the southwest corner of the project site. The south fork of Fowler Creek emerges onto Evergreen II near the northeast corner of the property and disappears into nearby sediment deposits. The channel reemerges to the south, is crossed by a bridge that accesses the City of San Jose Municipal Water Services Evergreen Zone ¾ Water Tank and forms a broad drainage surface that is intercepted by a man-made channel that drains normal to Yerba Buena Road and appears to dissipate prior to reaching the roadway.

1.3 BACKGROUND

Several previous geologic hazard and geotechnical investigations have been conducted for the Evergreen I and II properties. These reports address the sites by several names assigned by previous prospective developers. Reports provided for our review for the Evergreen I property north of Fowler Road include:

Letter prepared by: City of San Jose, February 8, 1996, Preliminary Geologic Review, Proposed Industrial Complex – Former Exxon Site (APN 659-02-077 & 660-33-1, 4, 5, 6, 9, & 11) (PDC 98-5-35) Future Murillo Avenue South of Aborn Road, Project No. 3-6434.

Report prepared by: Friar Associates Incorporated, October 8, 1999, Supplemental Phase I Data, 90-Acre Commercial Site, Fowler Road at Aborn Road, San Jose, California

Report prepared by: Friar Associates Incorporated, October 8, 1999b, Phase II Geologic Investigation, 90-Acre Commercial Site, Fowler Road at Aborn Road, San Jose, California

Report prepared by: Friar Associates Incorporated, September 1999, Preliminary Geologic Investigation, 90-Acre Commercial Site, Fowler Road at Aborn Road, San Jose, California

Report prepared by: Friar Associates Incorporated, December 1999, Geologic Investigation, 90-Acre Commercial Site, Fowler Road at Aborn Road, San Jose, California

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Reports and letters provided for our review for the Evergreen II property south of Fowler Road include:

Letter Report prepared by: Louis A. Richardson, June 7, 1996, Results of Preliminary Geophysical Survey for Proposed SVSP Complex at Future Murillo Avenue South of Aborn Road, San Jose, CA

Letter Report prepared by: Louis A. Richardson, March 27, 1998, Interim Report of Fault Investigation for 107-Acre Parcel at Proposed SVSP Complex, on East Side of Yerba Buena Road Extension, San Jose, CA.

Letter Report prepared by: United Soil Engineering, Inc, May 1, 1998, Proposed Development, Evergreen Industrial Park, Fowler Road Properties, San Jose, California, Geologic Report Review

Letter prepared by: Orchard Properties, May 19, 1998, Orchard Evergreen Technology Park/Pacific Rim Science Park, Geologic Hazard Clearance

Report prepared by: United Soils Engineering, Inc., June 1998, Proposed Orchard Evergreen Technology Park / Pacific Rim Science Park, Southeast Corner of Yerba Buena & Fowler Roads, San Jose, California, Preliminary Geotechnical Investigation.

Letter prepared by: ouis A. Richardson, July 23, 1998, Orchard Evergreen Technology Park/Pacific Rim Science Park, Geologic Hazard Clearance

Report prepared by: Louis A. Richardson, Consulting Engineering Geologist, November 20, 1998, Geologic Hazard Investigation Report, Orchard Evergreen Technology Park / Pacific Rim Science Park, San Jose, California

Letter Report prepared by: Louis A. Richardson, February 25, 1999, Building Exclusion Zone, Orchard Evergreen Technology Park/Pacific Rim Science Park, San Jose, CA

Letter prepared by: City of San Jose, March 2, 1999, Certification of Geologic Clearance, Proposed Pacific Rim Science Park – Former Exxon Site (APN 660-33-1, 4, 5, 6, 9, & 11) (PDC 98-5-35) Yerba Buena Road South of Fowler Road, Project No. 3-6434.

Report prepared by: Earth Systems Consultants, January 2000, Preliminary Geologic Feasibility Study, Proposed Debris Basin, Pacific Rim Science Park Southeast Corner of Yerba Buena and Fowler Roads, San Jose, California

Report prepared by: NorCal Engineering, June 15, 2000, Geotechnical Engineering Investigation, Pacific Rim Science Park Development, Yerba Buena Road and Fowler Road, San Jose, California; Appendix C is a copy of Louis A. Richardson, Consulting Engineering Geologist, November 20, 1998, Geologic Hazard Investigation Report, Orchard Evergreen Technology Park / Pacific Rim Science Park, San Jose, California

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Reports provided for our review for both Evergreen I and II properties or related properties include:

Report prepared by: Applied Soil Mechanics, Inc. October 17, 1980, Geologic Investigation, Proposed Evergreen Industrial Site, San Jose, Santa Clara County, California, Appendix A is a copy of Geoconsultants, Inc., September 3, 1980, Fault Location Investigation, Evergreen Area – Aborn Road, Santa Clara County, California.

Letter Report prepared by: Louis A. Richardson, June 7, 1996, Results of Preliminary Geophysical Survey for Proposed SVSP Complex at Future Murillo Avenue, South of Aborn Road, San Jose, CA.

Report prepared by: Evergreen Specific Plan Property Owners and the City of San Jose Municipal Water System Division, September 6, 2000, Final Supplement to the Evergreen Zone ¾ Reservoir.

In addition to the above documents, we reviewed readily available published maps prepared by the U.S. Geological Survey and the California Geological Survey (CGS, formally known as the California Department of Mines and Geology). We discussed the project with the City of San Jose geologist, Mr. Michael Shimamoto, and met with geologic investigators working on adjacent properties.

The City of San Jose issued a Preliminary Geologic Review letter dated February 8, 1996 referencing the Evergreen I and II project sites that concluded that the geologic reports provided to the City at that time were outdated and incomplete with respect to possible presence of geologic hazards at the sites. Since the issuance of the City's 1996 review letter at least six reports referencing geologic conditions for the Evergreen I site have been prepared. During this period at least ten reports referencing geologic conditions at the Evergreen II project site have been prepared. Based on the above reports for the Evergreen II property, the City of San Jose issued a geologic clearance letter on March 2, 1999 to allow development of the Evergreen II site to proceed. The City of San Jose, March 2, 1999 letter established a fifty-foot building setback from the east ends of trenches logged by Louis A. Richardson (February 25, 1999). According to the City's letter, "No buildings or structures shall be constructed" . . . in the setback zone . . . "without additional review and approval by the City Geologist." The City's letter also states that "Any changes to the project geologic consultant of record or project design, location, or concept must be reviewed and approved by the City's Engineering Geologist.

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Significant changes or additions will require a new Geologic Hazard Clearance." This report is prepared to provide geologic hazard evaluation to allow the City to issue a Certificate of Geologic Hazard Clearance for both Evergreen I and II properties.

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2.0 PURPOSE AND SCOPE

The purposes of this Geologic Hazards Assessment and Fault Investigation of The Evergreen I and II properties were to evaluate the geologic and seismic setting of the sites, and to explore the sites for evidence of active faulting and other geologic hazards. Our field subsurface exploration was limited to the east margins of the properties to investigate the possible presence or absence of fault expressions to and beyond the east property lines.

The scope of this Geologic Hazards Assessment and Fault Investigation consisted of:

- reviewing previous on-site fault and geologic studies provided to us by Berg & Berg Enterprises;
- researching readily available published and unpublished geologic reports and maps;
- analyzing selected aerial photographs ;
- geologic mapping of the site;
- subsurface exploration by means of trench logging;
- geologic interpretation of the data collected; and
- preparation of this written report.

This investigation addresses regional and site geology, seismicity, faults, and fault rupture and other geologic hazards. This report generally follows guidelines for engineering geologic studies including those published by the Association of Engineering Geologists and the CGS. More specifically, the geologic assessment follows the requirements of the Certificate of Geologic Hazard Clearance, and the State's 1996 revision of the *Guidelines for Evaluating the Hazard of Surface Fault Rupture,* which is included in the Division of Mines and Geology's Special Publication 42.

As shown on Plate 5, the City of San Jose designates the eastern margin of the two properties to be in a Seismic Hazard Zone according to the City's Fault Hazard map (1983). The proposed scope and the results of our study were discussed with Mr. Michael Shimamoto, geologist for the Department of Public Works, City of San Jose. The City's letter dated February 8, 1996 to

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Pacific Rim Financial Corporation describes its response to a previous clearance application and outlines the scope of work required for additional geologic investigation. Approval of the Geologic Hazard Clearance by the City is, in part, contingent upon the satisfactory compliance and implementation of geologic hazard mitigation as recommended in the project geologic and geotechnical reports. We understand that this report is to be submitted for application for a Geologic Hazard Clearance for the Evergreen I and II properties. An integral part of our study was to critically review work by previous investigators particularly with respect to remote-sensed and subsurface fault exploration. The methods used and accuracy with which data were presented in these previous reports was evaluated by comparison to current geologic-investigative techniques. We have drawn conclusions with respect to the applicability of the data provided in the older reports to our evaluation. Data from these previous investigations are used in this investigation where they are considered valid and where appropriate to our study.

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3.0 GEOLOGIC SETTING

The property consists of slightly sloping land situated in the southeastern part of the Santa Clara Valley at the south end of the San Francisco Bay. The area is within the Coast Ranges Geomorphic Province, which is characterized by northwesterly trending ridges and valleys. Geomorphic features lie parallel to subparallel to the Coast Ranges structural grain that is aligned with several major faults. The general geologic framework of the central Pacific Coast margin of California is illustrated in studies by Page (1966), California Geological Survey (2002) included as the Regional Geologic Map (Plate 8), as well as in studies by Jennings and Strand (1958) and others.

Geologic structures within the Coast Range Province are generally controlled by a major tectonic transform plate boundary. This right-lateral strike-slip fault system extends from the Gulf of California, in Mexico, to Cape Mendocino, off the coast of Humboldt County in northern California and forms a portion of the boundary between two global tectonic plates. In this portion of the Coast Range Province, the Pacific Plate moves north relative to the North American Plate, which is located east of the transform boundary. Deformation along this plate boundary is distributed across a wide fault zone, which includes the San Andreas, Hayward, Calaveras, and San Gregorio faults. Together, these and other faults are referred to as the San Andreas fault system. The general trend (about N40W) of the faults within this system is responsible for the strong northwest-southeast structural grain of most geologic and geomorphic features in the Coast Ranges Province.

Santa Clara Valley is flanked on the west by the Santa Cruz Mountains and on the east by the Diablo Range, which consists of older Franciscan and related rocks and overlying sedimentary rocks ranging in age from the Cretaceous through Tertiary time. As shown on the Vicinity Geologic Map (Plate 9), the predominate geologic units at the Evergreen sites are alluvial fan deposits (Qpf) of Upper Pleistocene Age (about 1.6 million years to 11,000 years before present) that consist of tan to reddish brown gravel that are clast supported. The clasts are typically cobble sized with clayey and sandy matrix that in places align along crude bedding.

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3.1 FAULTING AND SEISMICITY

The San Francisco Bay Region is considered to be one of the most seismically active regions in the United States, and is dominated by the San Andreas fault system. Periodic earthquakes have occurred throughout the Bay Region in historic time, several of which had magnitudes of 6 to 8 on the Richter scale. The largest and most destructive earthquakes to affect Santa Clara County are the 1868 earthquake, which was centered on the Hayward fault, and the 1906 earthquake that occurred on the San Andreas fault. Considerable damage also occurred in the Santa Clara Valley from the 1989 Loma Prieta earthquake centered on the San Andreas fault in the nearby Santa Cruz Mountains. A minor earthquake was felt in the nearby Coyote – Morgan Hill area in January 1999, with no reports of significant damage. The earthquake was centered on the nearby Calaveras fault.

Some of the significant nearby events include the 1906 (M7.9) "Great" San Francisco earthquake, the 1838 (M7) San Francisco Peninsula earthquake, the 1865 (M6.4) Santa Cruz Mountains earthquake, the 1868 (M6.8) Hayward earthquake, the 1890 (M6.2) Pajaro Gap earthquake, the 1899 (M5.8) and 1984 (M6.1) Morgan Hill earthquakes, the 1882 (M5.8) and 1892 (M5.8) Hollister earthquakes, the 1897 (M6.2) Gilroy earthquake, the two 1903 (M5.5) San Jose earthquakes, the 1910 (M5.8) Watsonville earthquake, two 1926 (M6) Monterey Bay earthquakes, and the 1989 (M6.9) Loma Prieta earthquake. A recent study by Toppozada and Borcherdt (1998) indicates an 1836 (M6.8) earthquake, previously attributed to the Hayward fault, occurred in the Monterey Bay area and was of an estimated magnitude M6.2. In terms of measured seismic shaking, the 1989 Loma Prieta earthquake has provided relevant information of seismic conditions in the vicinity of the project site.

As shown on the Regional Geologic Map, Plate 8, the major active faults in the vicinity of the project sites are the Hayward and Calaveras faults, both recognized as sources of past earthquakes. Also present is the Quimby fault, mapped by Dibblee (1972) along the east property lines of Evergreen I and II. This fault is considered by the State of California to not be an active source of ground rupture and by the UBC (2001) to not be an active seismic shaking source but is included on the City of San Jose Fault Hazard Map as a potential fault hazard. The Evergreen fault is mapped about 1000 feet southwest of the Evergreen I site and is considered by

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the State of California to be an active source of ground rupture (California Earthquake Fault Zones, Plate 6) but not an active seismic shaking source by the UBC (2001) and is included on the City of San Jose Fault Hazard Map. A portion of the 1983 City of San Jose Special Study Zones Map is shown on Plate 5. The San Jose Special Study Zones Map indicates that the eastern portions of Evergreen I and II are within the City's "Special Studies Zones." With regard to the City's Fault Hazard Map, comprehensive geologic studies are required for sites within the Special Studies Zones (such as for the Quimby, Silver Creek, Calaveras, San Andreas faults).

Table 1 lists significant faults, which are considered by the Uniform Building Code (UBC) to be active or potentially active seismogenic sources and gives selected seismic parameters. The closest distance from the site to these faults and associated parameters presented in Table 1 are based on data derived from Blake (2000) based on attenuation factors from Bozorgnia Campbell Niazi (1999). The locations of the faults presented on Table 1 and other active and potentially active faults in the area with respect to the subject site are shown on the Regional Fault Map, Plate 10.

Table 1 Significant faults within about 50 miles of the project site

Fault Name	Approximate Distance to Fault mi(km)	Magnitude of Maximum Earthquake	Peak Site Acceleration (g)	Modified Mercalli[1]
Hayward (Se Extension)	0.3 (0.5)	6.4	0.461	X
Calaveras (So.Of Calaveras Res)	3 (5)	6.2	0.326	IX
Hayward (Total Length)	10 (16)	7.1	0.251	IX
Hayward (South)	10 (16)	6.9	0.225	IX
Calaveras (No.Of Calaveras Res)	10 (16)	6.8	0.212	VIII
Monte Vista - Shannon	10 (17)	6.8	0.282	IX
Sargent	15 (24)	6.8	0.140	VIII
Greenville	16 (26)	6.9	0.143	VIII
San Andreas (1906)	16 (26)	7.9	0.259	IX
San Andreas (Santa Cruz Mtn.)	16 (26)	7.0	0.149	VIII

[1] Modified Mercalli Intensity Scale is defined on Plate 11.

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Table 1 Significant faults within about 50 miles of the project site

Fault Name	Approximate Distance to Fault mi(km)	Magnitude of Maximum Earthquake	Peak Site Acceleration (g)	Modified Mercalli[1]
San Andreas (Peninsula)	17 (27)	7.1	0.155	VIII
Zayante-Vergeles	20 (32)	6.8	0.108	VII
Great Valley 7	25 (39)	6.7	0.114	VII
San Andreas (Pajaro)	25 (41)	6.8	0.084	VII
Ortigalita	27 (43)	6.9	0.086	VII
Great Valley 6	27 (43)	6.7	0.104	VII
Great Valley 8	29 (46)	6.6	0.091	VII
San Gregorio	33 (53)	7.3	0.090	VII
Monterey Bay - Tularcitos	34 (54)	7.1	0.107	VII
Quien Sabe	35 (56)	6.4	0.047	VI
Hayward (North)	36 (57)	6.9	0.063	VI
San Andreas (Creeping)	37 (59)	6.5	0.047	VI
Concord - Green Valley	42 (67)	6.9	0.054	VI
Palo Colorado - Sur	42 (67)	7.0	0.057	VI
Great Valley 9	42 (67)	6.6	0.061	VI
Rinconada	44 (71)	7.3	0.066	VI

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4.0 INVESTIGATIVE METHODS

4.1 RESEARCH AND REVIEW

Our engineering geologist reviewed the geologic and geotechnical investigative reports provided by Berg & Berg Enterprises for the project sites. Comparisons were made between the logs provided from previous trench reports with information gathered during our subsurface investigation. It was found that the material and stratigraphic conditions described in the previous reports matched well with our findings. Where previous reports interpreted the subsurface data to show the absence of active faulting, data from our trenches showed consistently similar results. We have also examined magnetometer traces taken across the project sites and have reevaluated their interpretations. Our findings are generally consistent with previously reported findings by these and other investigators.

In the course of this investigation, we have also researched published reports, maps, and other technical documents, listed in the References section of this report. The basic geology of the site is shown on geologic quadrangle maps by Dibblee, 1972 and Wentworth and others (1999). Pertinent geologic hazard and fault information are in reports by Rogers and Williams, 1974, Borcherdt, Gibbs, and Lajoie, 1990, Brown and Lee, 1971, Brown, 1970, Bryant, 1981, City of San Jose maps, 1974, and Jennings, 1994. We have shown the geologic conditions in the vicinity of the project site on the Vicinity Geologic Map (Plate 9).

Kleinfelder personnel met with Engeo's project geologist to discuss their findings during recent fault trenching on property southeast of and adjacent to the Evergreen II property. The Site Plan and Geologic Map for Evergreen II shows the approximate location of Engeo's north-most trenches ET-8 and ET-9. Kleinfelder personnel also met with Mr. Mike Shimamoto of the City of San Jose to reviewed and discuss geologic-investigation reports for properties north and south of the Evergreen sites.

We have reviewed the findings by Earth Systems Consultants (2000) regarding their geologic feasibility study for a proposed debris basin to be located near the east property line of Evergreen II. Mr. Sadek Derrega, the engineering geologist who prepared the report for Earth Systems Consultants, is now a Certified Engineering Geologist with Kleinfelder. The scope and

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conclusions of the Earth Systems Consultants' report were discussed with Mr. Derrega. The report concluded that the primary geologic hazards that could affect the debris-basin site are the potential for surface rupture during an earthquake on the Quimby fault; very strong ground shaking during future earthquakes in the region; and the potential for reactivation of the old large landslides mapped upslope of the proposed structure (Vicinity Geologic Map). Earth Systems Consultants further concluded that the proposed debris basin is not a critical structure because it will not be designed to dam, retain or detain water runoff. Accordingly, trenching in the area of the proposed debris basin to locate the Quimby fault trace would not be needed. Finally, Earth Systems Consultants concluded that a design-level soils-engineering study be conducted to evaluate the site's subsurface materials and groundwater and to provide recommendations for site grading, drainage, and embankment design. A subsurface exploration program including test pits and continuously sampled deep borings should be implemented to evaluate the presence and activity of the mapped landslide (Site Plan and Geologic Map, Evergreen II). Special attention and evaluation should be paid to the proposed grading (cutting) of soil/rock material near or at the base (toe) of the mapped landslide deposit.

4.2 AERIAL PHOTOGRAPHIC ANALYSIS

HMH provided aerial photographs of the site for analysis and our evaluation of landforms and as an aid in geologic interpretation. Aerial photographs on file at the U.S. Geological Survey in Menlo Park were also reviewed. Our photographic analysis revealed lineation or tonal variations that may be fault related. Primary among these is the frontal escarpment along the east margin of the two properties. An aerial photograph of the project sites showing the probable fault-controlled frontal escarpment along the east margins of the sites is provided on Plate 2.

4.3 FIELD GEOLOGIC MAPPING

Geologic mapping of the sites was performed by our senior engineering geologist using a 1" = 100' scale topographic base map (Plates 3 and 4). Most rock exposures are limited to the occasional outcrops on the hill slopes east of the properties. Our geologist also mapped rock exposures on adjacent properties to the east and south for verification of local geologic conditions as they project onto the site, which were then confirmed by the on-site subsurface

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investigation. Fault features including springs were noted just east of Evergreen I and II, which correspond to tonal lineation observed on aerial photographs. Some topographic irregularities at Evergreen II in the southeast corner of the property are associated with fill placement and differential weathering of the various types of soil and rock. The results of the mapping and surface geologic interpretation are presented on the Site Plans and Geologic Maps (Plates 3 and 4).

The field mapping also included the location of colluvial deposits, earth flows, and landslides which predominately are located off-site and upslope of the properties (Site Geologic Maps). Comparisons were made with the State of California Seismic Hazard CGS Earthquake Fault Zones (CGS, 2001, California Seismic Hazard Zones Map, Plate 7) to evaluate slope stability conditions. The seismic hazard zone map shows that the areas considered by the state that may be subject to landsliding are located east of the Evergreen I and II property lines. Our field mapping did not show the presence of active deep-seated, bedrock landslides or areas of slope instability on the slopes adjacent to the properties. The Seismic Hazard Zones Map, Plate 7, shows a small thumb-shaped area of potential landslide south of Fowler Creek extending onto the Evergreen I property. Because there was no evidence observed during our site reconnaissance that this "thumb" area would be subject to possible slope instability, we have inferred that this area does not pose a slope stability hazard to the proposed development.

Some isolated areas of colluvium that may have been deposited as earth flows were identified during our field mapping and logging of exploratory trenches. During the design-level soils-engineering investigation these areas and others thought to be underlain by the colluvial deposits should be further investigated and recommendations prepared that describe the engineering remediation needed.

4.4 FIELD EXPLORATION

Prior to the fieldwork portion of this phase of the geologic investigation, our field personnel marked the site and notified Underground Service Alert for its member utilities to establish the locations of underground utilities. Cruz Brothers Locators, of Scotts Valley, California, was also engaged to locate underground utilities and other structures prior to excavating at the site.

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An Environmental Clearance permit was applied for through the City of San Jose and approved with restrictions which were implemented during site excavation. Before excavation of the trenches, Basin Research Associates of San Leandro, California conducted an archaeological screening of the site to explore for the possible presence of archaeological sites or artifacts. Two areas were identified as shown on the Site Aerial Photograph, Plate 2. During our trench excavation in the vicinity of these two areas, a representative of Basin Research Associates was on site to observe for the possible presence of objects of archeological significance. It is our understanding that none were found.

Field exploration of the sites for the fault evaluation was performed from September 27, 2004 through October 6, 2004, and consisted of field geologic mapping and excavating and logging of six exploratory trenches totaling about 1,325 linear feet, the locations of which are shown on the Site Plans and Geologic Maps, Plates 3 and 4. The trenches were excavated with a track-mounted excavator and were logged by our engineering geologist. Kleinfelder personnel shored the trenches where they were deeper than five feet for safe entry. One trench wall was hand picked to allow for geologic mapping and observation. The other trench wall was picked clean where needed to verify localized geologic features. Logs of the trenches are presented on Plates 13 and 14.

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5.0 SITE GEOLOGY

In general, the geology of the site and adjacent areas is complex and consists of uplifted terrain underlain by units of the Berryessa Formation including Oakland Conglomerate. This complexity is partly the result of past tectonic folding and faulting, and partly due to past deposition of valley sediments at higher elevations than present, and subsequent differential weathering of the variably consolidated materials.

5.1 SURFICIAL DEPOSITS

Streams debouching from adjacent highlands minimally incise the planner surface of Evergreen I and II. The streams flow across a soil-mantled surface of indurated Upper Pleistocene-age conglomerate/alluvial-fan deposits. Locally, adjacent to the upper reaches of the streams and to a lesser degree on the Pleistocene plain, Holocene alluvium is deposited in narrow, shallow fans. Differentiation of various ages of the fan deposits is primarily subjective and is not included in this report except where age of fault offset is needed to determine time since last seismic activity. Our interpretation of the subsurface geologic conditions is illustrated on the Geologic Cross Sections A – A' and B – B', Plate 15.

Holocene colluvial deposits were noted to lie in swales east of Evergreen II. Colluvial aprons extend onto Evergreen II near the south fork of Fowler Creek, downslope from the water tank located east of Evergreen II, as well as in the trough near exploratory trench KA-T-1.

A landslide is mapped in the northeast corner of the Evergreen II property where the south fork of Fowler Creek enters the property. The landslide appears to have displaced the creek channel to the south suggesting recent activity.

5.2 BEDROCK

Consolidated bedrock of the Berryessa Formation underlies the site at depth beneath the Pleistocene alluvium (Wentworth and others, 1999). Wentworth and others (1999) map distinctive units of the Berryessa formation which include interbedded layers of massive, indistinctly bedded, coarse- to fine-grained, mica-quartz-lithic wacke and mica-bearing siltstone and claystone, and thick, indistinct beds of pebble, cobble, and less common boulder

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conglomerate interfingered with coarse-grained mica-quartz-lithic wacke. Clasts include silicic to intermediate volcanic rocks, black chert and argillite, quartz, mica schist, semi-gneissic meta-andesite, granodiorite and granite, black hornfels, and rip-up clasts of mudstone and lithic wacke.

Berryessa bedrock does not crop out on the Evergreen properties and was not exposed in our exploratory trenches. The bedrock formation is inferred to be greater than 40-feet deep near the center of the property based on borings by Applied Soil Mechanics (1980).

5.3 FAULTS

The Evergreen area of the San Francisco bay area is a region of complex, interlacing faults along the western front of the Diablo Range near the south end of the Hayward fault. According to the 1983 City of San Jose Special Studies Zones Map (Plate 5), the eastern portions of the project sites are within a potential fault hazard zone. The City's zone is based on the possible presence of the Quimby fault located on the hillside to the east of the project sites. The Quimby fault is not within a State Earthquake Fault Zone (Plate 6), since it does not meet the California Division of Mines and Geology criteria of "sufficiently active" or "well-defined" (Hart and Bryant, 1999). However, the City's 1983 map shows the Quimby designated as a "special studies zone," and requires a fault investigation similar to that for development in a State mandated special studies zone. The Quimby fault shown on Dibblee's geologic map of the San Jose East Quadrangle (1972) indicates the fault lies at the base of the steep slope just east of the Evergreen I and II properties. This fault trend is included on the 1974 Regional Geologic Map by Cooper-Clark & Associates prepared for the City of San Jose as a planning document for San Jose's Sphere of Influence.

To evaluate fault-investigative work by previous investigators on the Evergreen I and II properties, we compared the description of subsurface materials and stratigraphic relationships recorded graphically in trench logs with the findings from the exploratory trenches we logged for this project. Where data and interpretations were consistent, we included the previous interpretations in our analysis. An example of compatible data would be Applied Soil Mechanics (1980) trench TR-1 and Kleinfelder's trench KA-T-6. Our trench, located east of the

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Wall's Ranch access road (Site Plan and Geologic Map, Evergreen I), contained silty clay residual soil over claystone with three-inch diameter gravel and cobble clasts (claystone is, in this case, silty clay inferred to be greater than 11,000 years old) and the Applied Soil Mechanics' trench west of Wall's Ranch road revealed sandy silt residual soil over silty clay with increasing gravel content with depth (would be classified as claystone if described as older than 11,000 years). Both Kleinfelder's and Applied Soil Mechanics' trenches show a general slope of the uninterrupted stratigraphic layers west of Station 0+45 in KA-T-6 that is down to the west. Given the similarity in materials described in the Kleinfelder and Applied Soil Mechanics trenches, we interpreted the trenches to show the absence of faulting between Station 0+45 of KA-T-6 and the west end of TR-1. The short gap between the two trenches where Wall's Ranch road is located is shadowed by trench ET-1 (Friar Associates, 1999) which also demonstrates similar subsurface materials and conditions

Earth Science Associates (ESA 1980 and 1992) conducted geologic investigations for the property south of Evergreen II for the Syntex Corporation. ESA placed its exploratory trench TR-1 in a general northeast to east trend in an attempt to locate the Dibblee trace of the Quimby fault mapped along Syntex's east property line. Trench TR-1 crossed the mapped location of the Quimby fault without revealing evidence of faulted strata. Trench TR-1, however, did encounter a fault trace, with an attitude of N25°W 15°E, about 800 feet west of the Syntex east property line. Further investigation confirmed the fault's location in Earth Science Associates' trenches TR-4 and TR-6 (Earth Science Associates, 1980). Later, two exploratory trenches were excavated and logged to evaluate the trace of the fault to the north of Trench TR-1 (ESA, 1992). Trench TR-8, located about 330 feet northwest of Trench TR-1 did not contain evidence of the fault. Trench TR-9, located about 100 feet north of TR-4 and about 200 feet north of Trench TR-1 where the fault was identified, also did not reveal evidence of faulting. As a result of the 1980 and 1993 investigations by Earth Science Associates, a trace of a fault is mapped south of, and trending toward, the Evergreen II property. However, because the fault trace is not continuous, the building setback zone established by Earth Science Associates does not extend to the Evergreen II property, but ends about 400 feet south of the subject property.

In his November 20, 1998 Geologic Hazard Investigation Report for the currently named Evergreen II project site, Louis A. Richardson shows the location of three magnetometer lines

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and six exploratory trenches. Magnetometer lines ML-3 and ML-4 extend beyond the existing property lines of Evergreen II onto property presently owned by IDS. Richardson placed Trenches T2 (Site Plan and Geologic Map, Evergreen II) to intercept the projection of the fault features observes by ESA south of the Evergreen property. No evidence of faulting was observed by Richardson in Trench T-2. Trench T-3 was placed to investigate a magnetic anomaly observed in magnetometer line ML-3. Again no evidence of faulting was found in the subsurface. Comparison of Richardson's descriptive trench logs correlate well with materials observed in trenches logged for this investigation and therefore the Richardson trench logs are used as extensions of this study. One exception is Richardson's designation of his units C and C1 in his trench T1 as Oakland conglomerate, which is of Cretaceous age (older than 88 million years). Our work at the site suggests this material, which matches with other units in our trenches, is considered to be Pleistocene in age based on degree of induration, natural compaction and correlation with mapped units by other investigators including Wentworth and others (1999).

Richardson's magnetometer lines ML-3 and ML-4 and trenches T-2 and T-3 do not show the presence of faulting in the southwest portion of the site he investigated which now is the IDS property. Richardson further investigated the Evergreen II site by excavating and logging exploratory trenches T-1, T-4, T-5 and T-6 in the east portion of the property (Site Plan and Geologic Map, Evergreen II). The absence of fault features in these trenches allowed Richardson (1999) to establish a building-exclusion zone that is setback 50 feet "from the easterly property line of this site."

Richardson's November 20, 1998 report reviewed and discussed other geologic hazards at the Evergreen II site including landsliding, soil liquefaction and flooding. Our review of geologic conditions at the Evergreen II site is consistent with the conclusions presented by Richardson. Based on Richardson's findings regarding geologic hazards for the Evergreen II property, the City of San Jose issued a Certificate of Geologic Hazard Clearance for the Evergreen II property that included the currently separate IDS property at the southwest corner of the property and showed Richardson's building-exclusion zone.

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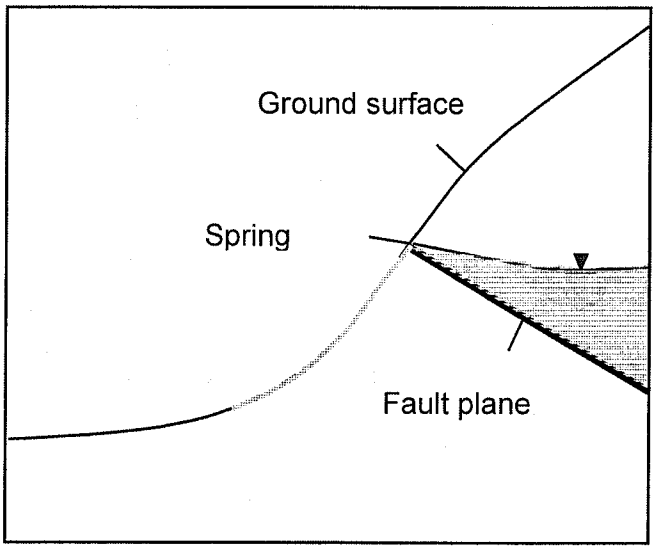
Engeo personnel shared with Kleinfelder their findings regarding fault trenching south of the Evergreen II property. Their trenches ET-8 and ET-9 (Site Plan and Geologic Map, Evergreen II) showed the absence of faulting. Trenches Engeo excavated and logged south of ET-9 showed evidence of a fault trending northeast away from Evergreen II property.

Field mapping carried out during this investigation revealed springs on the slope east of Evergreen I. Springs can be associated with fault traces where the fault plane acts as a groundwater barrier and causes groundwater to flow to the surface as shown on the diagram below.



Diagrammatic cross section through a spring

Based on the absence of fault features in all but one exploratory trench excavated at the Evergreen I and II properties and the alignment of the fault exposed in trench KA-T-6 with the springs east of Evergreen I (Site Plan and Geologic Map, Evergreen I), we have shown a relocated trace of the Quimby fault east of Evergreen I. This trace of the Quimby fault aligns with the linear valley that Fowler Road ascends east of Evergreen II. The linear escarpment along the east margin of Evergreen I appears to be fault controlled but the fault appears to be

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somewhat higher on the slope than previously mapped by Dibblee (1972) and Wentworth and others (1999).

5.4 GROUNDWATER

Free water was not encountered in any of the six trenches excavated for this investigation. Groundwater was not encountered in borings by NorCal (2000) to the depths of 40 feet. Only boring BH-11 by Applied Soil Mechanics encountered groundwater at a depth of 38½ feet (Cross Section B – B', Plate 15).

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Several previous fault studies by various investigators were for the purpose of verifying and evaluating suspected faults that were originally identified by Dibblee (1972). The Kleinfelder trenching assisted in interpretation of geologic conditions and confirmed both the absence and presence of faults in and adjacent to the planned development. The locations of trenches completed for this investigation as well as test-pit, trench, soil-boring, and magnetometer-traverse locations for previous investigations are shown on the Site Plans and Geologic Maps, Plates 3 and 4. Permission was acquired by Berg & Berg to locate Trenches KA-T-4 and KA-T-6 partially or wholly onto the Walls Ranch property to extend our investigation at least 50 feet east of the Evergreen I property line.

Graphic illustrations of the trench logs and faults identified in the Kleinfelder trenches are depicted on the Exploratory Trench Logs, Plates 13 and 14. The trenches were laid out generally from northeast to southwest normal to regional faulting and folding to maximize exposure of faults. The trenches on the Berg & Berg Enterprises property were loosely backfilled by replacing the excavated soil with little or no compactive effort. During site grading, Kleinfelder's trenches as well as previous subsurface exploration points should be excavated and recompacted.

6.1 TRENCHES

Trench KA-T-1 extended about 425 feet through the erosion saddle in the southeast corner of Evergreen II. The purpose of this trench was to extend the subsurface fault investigation to and past the east property line of Evergreen II. Below a residual topsoil of about four-inches thick, indurated clast-supported conglomerate consisting of subangular gravel and cobbles was encountered from Station 0+00 to about Station 3+40. East of this station to the northeast end of the trench, red claystone with gravel was encountered. The contact between the conglomerate and underlying claystone is undulatory, in places gradational, without a sharp border and appears to be depositional. The earth materials observed below the residual soil are described as conglomerate and claystone instead of a soil based on the degree of induration and partial lithfication. The inferred age of the material based on its texture is considered to be Upper

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Pleistocene as shown on mapping by Wentworth and others (1999). There were no features observed in trench KA-T-1 that were considered to be associated with fault offset.

Trench KA-T-2 was located about midway along the east property line of Evergreen II . The trench was placed to extend, by about 75 feet, the previous subsurface investigations that stopped at the fence line (Plate 4). Beneath an approximately half-foot thick layer of residual soil and colluvium, uninterrupted beds of claystone were encountered. No evidence of fault displacement of subsurface materials was observed in Trench KA-T-2.

Trench KA-T-3 was about 130 feet long and was located north of the south fork of Fowler Creek east of previous trench locations to extend the fault investigation beyond the proposed development area. Beneath a colluvial wedge that thickens upslope probably originating from down slope earth flows, uninterrupted beds of claystone similar to that observed in Trench KA-T-2 was observed. No evidence of fault displacement of Pleistocene sediments or soil was observed in Trench KA-T-3.

Trenches KA-T-4 and KA-T-5 were placed to shadow the width of the City of San Jose's special studies zone across Evergreen I. The two trenches were laid out to overlap across the waterline extending from the well near the south property line to the Walls Ranch house. The total length of the trenches was about 655 feet with about 5-feet overlap. Upper Pleistocene age deposits of claystone, sandstone and conglomerate were exposed the entire length of the trench below a mantle of silty sand residual soil and Holocene alluvium. Considerable quantities of trash consisting of glass, wood, metal, plastic, cloth and other debris was encountered to depth of about five feet at various locations in Trench KA-T-4. No evidence of fault displacement was observed in Trenches KA-T-4 and KA-T-5.

Trench KA-T-6 was located on the hillside northeast of the Evergreen II entirely on Wells Ranch property. Offset and distorted Upper Pleistocene Age claystone and residual soil inferred to be of Holocene age were encountered east of Station 0+45. Based on the displacement of Holocene deposits the faults east of Station 0+45 are considered to be active. A subtle geomorphic bench was also observed at this location. No evidence of fault displacement was observed west of Station 0+45 in Trench KA-T-6.

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6.2 CROSS SECTIONS

Cross Section A - A' and B – B' present a geologic interpretation of subsurface conditions at the Evergreen I and II sites. Based on borehole data from Applied Soil Mechanics (1990) and NorCal (2000), depth to Berryessa Formation bedrock beneath the Pleistocene alluvium at Evergreen I and II is greater than 40 feet. Regional mapping by Dibblee (1972) and Wentworth and others (1999) suggest that bedding dips back into the slopes east of the project sites. The Cross Sections are shown on Plate 15, Cross Sections A-A' and B - B'.

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7.0 CONCLUSIONS

7.1 EVALUATION OF SITE GEOLOGIC HAZARDS

7.1.1 Site Soil Profile Type

The characteristics of the soils underlying the site are used to evaluate site-specific seismic design criteria. Based on the results of our field investigation and data review, the site is underlain by alluvial deposits of Pleistocene Age to a depth of 40 feet or greater which overlies Cretaceous conglomerate and shale. Considering the above evaluation of the site, the soil profile is classified as Soil Profile Type S_C in accordance with the Table 16A-J of the 1997 UBC. Soil Profile Type S_C is defined as very dense soil or soft rock with a shear wave velocity between 1,200 and 2,500 feet per second, SPT_n blow count greater than 50 and undrained shear strength greater than 2,000 pounds per square inch.

7.1.2 Seismic Shaking

A recent publication prepared by the USGS regarding earthquake probabilities in the Bay Area (Working Group on California Earthquake Probabilities, 2003) concludes that there is a 62 percent chance that one of the major faults within the Bay Area will experience a major (M6.7+) earthquake during the period of 2002-2031. As has been demonstrated by the 1989 (M6.9) Loma Prieta earthquake, the 1994 (M6.7) Northridge earthquake, and the 1995 (M6.9) Kobe earthquake, earthquakes of this magnitude range can cause severe ground shaking and significant damage to modern urban environments. The intensity of shaking at the project site will depend on the distance to the earthquake epicenter, the magnitude, and the response characteristics of the foundation materials. Some structural damage from stronger shaking can be expected, and structures should be designed in accordance with local building design requirements. Specific criteria with respect to our seismic risk analysis will be provided when building type and layout are determined.

In recent years, many modern structures located near a seismic source have been severely damaged or have collapsed. The severe damage or collapse is attributed to near-fault motions that are characterized by energetic unidirectional velocity pulses (Singh 1984, 1985). These

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motions are particularly damaging because of the impulse duration sustained during a near-source earthquake. A structural system that yields during a long-duration pulse (impulse loading) may experience very large permanent deformation or may collapse. The extent of these actions depends on the strength and natural period of the structure and the structure articulation, as well as the amplitude, duration, and shape of the pulse. Near-fault-pulse type motions can be particularly damaging because they can accumulate inelastic deformations in one direction. Consideration of these impulse motions in near-fault conditions should be properly evaluated.

Procedures from the 1997 UBC should be implemented for a code-equivalent lateral-force design of structures within the project area. The Near-Source Factors N_a and N_v in the code proposal are incorporated into the seismic coefficients C_a and C_v which are both used to determine the total design lateral force or shear at the base of the structure. The values of these factors depend on the distance of the structure from the fault and the fault type. These factors can be obtained from Tables 16A-Q through 16A-T of 1997 UBC.

The site is located within 10 km of a Type B fault (Hayward fault, southeast extension) according to the fault parameter criteria presented in Table 16A-U of the 1997 UBC. As such, Near-Source Factors N_a and N_v will be 1.3 and 1.6, respectively, per Tables 16A-Q through 16A-T of 1997 UBC. This value of N_a may be modified in accordance with UBC Sections 1629.4.2 or 1630.2.3.2 or other sections as determined appropriate by the structural engineer. Alternatively, consideration may be given to dynamic analyses utilizing site-specific response spectra that more accurately account for the types of near source effects observed in the recent Northridge, California and Kobe, Japan earthquakes.

7.1.3 Faulting and Surface Rupture

The Quimby fault as mapped by Dibblee (1972) is shown in this investigation to not be located at the base of the slope along the east margin of the Evergreen I and II project sites. Fault investigation work by Engeo on the property south of Evergreen II indicates that the fault trace veers to the east several hundred feet south of the Evergreen II property. Louis Richardson's (1998) trenches T-1, T-4 and our trenches KA-T1, KA-T2 and KA-T-3 (on the Evergreen II property) and Applied Soil Mechanics' (1980) trenches TR-1, TR-2 and our trenches KA-T-4,

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and KA-T-5 (on the Evergreen I property) show the absence of fault offset any younger than Upper Pleistocene between the west margin of the City of San Jose's special studies zone and the east ends of the trenches. Trench KA-T-6 shows the presence of fault features east of Station 0+45. The combined length and overlap of trenches excavated for this and previous fault investigations shadow the entire City of San Jose Special Studies Zone and demonstrate the absence of faulting on the Evergreen I and II properties.

No geomorphic features suggesting the possible presence of fault traces were observed in our aerial photographic review or during our site reconnaissance on or projecting toward the Evergreen sites. Considering the evidence presented here, we conclude that the likelihood of a fault causing ground rupture at the sites is low. However, there still may be deformation of the surface in the event of movement at depth on nearby faults. The possibility of shaking related random ground cracking and area deformation affecting the sites and surrounding areas cannot be precluded.

Trenches excavated for this investigation extend at least 50 feet east of the Evergreen I and II property lines (Site Plans and Geologic Maps, Plates 3 and 4). Based on this information, a revised building-exclusion zone is established along the east property lines of Evergreen I and II, Plates 3 and 4. No structures intended for human occupancy should be constructed east of this boundary unless further geologic investigations are carried out and approved by the City of San Jose.

7.1.4 Landslides and Slope Stability

In general, landslides caused by earthquakes and instability (weak soil, overly steep slope, excess weight, or high groundwater) may occur wherever existing or proposed slopes are not strong enough to support themselves (California Seismic Hazard Zones Map, Plate 7). However, because the Evergreen I and II properties are generally flatter than 5%, landslides are not expected to occur on site. An exception is the area where the south fork of Fowler Creek flows onto Evergreen II. Landslide deposits are mapped in the upper reaches of the stream in this corner of the property (Site Plan and Geologic Map). The landslide does not appear active but could be reactivated by grading or other changes to the topography. The mapped landslide and its potential runout, if reactivated, is entirely within the building-exclusion zone established

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from our fault investigation and should not pose a negative impact to the proposed development outside of the exclusion zone.

The slopes east of the project sites are considerable steeper than on the Evergreen properties. Rare outcrops on the hillsides east of the Evergreen properties indicate that the slopes are supported by Cretaceous age, lithified, well indurated conglomerate and shale whose beds dip back into the slope. No active landslides were observed on the slopes east of the project sites. Our site mapping indicated that there are isolated areas of colluviual deposits and debris flows, which do not appear to be deep-seated, bedrock landslides. These colluvial areas should be addressed in the design-level soils-engineering report once site layout is defined. With appropriate engineering mitigation these areas of colluvial deposits should not pose as threat to the proposed development.

7.1.5 Other Geologic Hazards

Considering the presence of dense to hard clay matrix in the Pleistocene conglomerate observed in trenches for this investigation and stiff clayey silt (NorCal, 2000) and stiff to hard soils (Applied Soil Mechanics, 1980) over the remainder of the Evergreen I and II properties, liquefaction, seismic settlement, and differential compaction are considered to have minimal, if any, impact at the site. The high plastic nature of the clay matrix observed in the Pleistocene alluvium generally would indicate that expansive soils may be present at the site, however, our field observations did not note the occurrence of desiccation cracks usually associated with expansive soils. Expansive soils should, however, be tested for during the design-level soils-engineering investigation.

Flood hazards are generally considered from three sources, which include seismically induced waves (tsunami or seiche), reservoir failure, and long-cycle storm events. Because the site is removed from the coast at an elevation generally greater than 500 feet, the likelihood of tsunami impacting the site is negligible. No dams or open reservoirs are located at higher elevations above the site that would be significantly close to the site to cause damage in the event of a break. Therefore, inundation from these sources is also considered negligible. Two large capacity water tanks are near the project sites. The City of San Jose water tank located northwest of the intersection of Yerba Buena and Fowler Roads is mostly belowground and may

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leak but probably would not cause catastrophic flooding if ruptured. The City of San Jose Municipal Water Services Evergreen Zone ¾ Water Tank (Site Aerial Photograph) is located about 200 feet higher than the Evergreen II property and, if ruptured, could inundate a portion of the west part of the property with water. With respect to the 100-year storm events, ESRI/FEMA (Project Impact Information and Awareness Site [http://www.esri.com/hazards]) indicate that the site is not within a 100-year flooding area.

7.1.6 Proposed Debris Basins

Two debris basins are proposed to be constructed at the project sites, one near the point where Fowler Creek enters Evergreen I and the other where the south fork of Fowler Creek enters Evergreen II. The proposed debris basin on Evergreen II (Site Plan and Geologic Map, Evergreen I, Plate 3) has been preliminarily investigated by Earth Systems Consultants (2000) who concluded that fault trenching would not be required at the site of the basin because it will not be a critical structure. Earth Systems Consultants further concluded that the basin would be subject to strong earthquake shaking, possible ground rupture in the event of movement on the Quimby fault, and possibly impacted in the event of movement of the landslide mapped upslope from the basin site. Earth Systems Consultants recommended in their report that a design-level geotechnical investigation be conducted at the basin site to evaluate the site's subsurface materials and groundwater and to provide recommendations for site grading, drainage, and embankment design. Our field observations, mapping at the site and nearby properties, and discussions with Mr. Derraga, author of the Earth Systems Consultants report, have led us to concur with the findings of Earth Systems Consultants.

The debris basin proposed to be constructed on Evergreen I (Site Plan and Geologic Map, Evergreen I, Plate 3) is similar to the basin proposed on Evergreen II and would be subject to similar conditions. However, the basin site on Evergreen I is not within or adjacent to colluvial or landslide deposits.

7.1.7 IDS Property

The IDS property, located adjacent to the southwest corner of Evergreen II expresses similar geologic conditions as the large, western portion of Evergreen II. Magnetometer lines and trenches by Richardson (1996, 1998) that, on review by Kleinfelder's geologist, appear to be

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appropriately interpreted, show the absence of faulting on the property. Our aerial photographic review and field mapping did not reveal geomorphic features on the site that may be associated with active faulting. Given the findings of previous reliable geologic studies and our limited investigation of the IDS property, in our opinion, the IDS property has no geologic conditions that should adversely affect development if the recommendations contained in this and the sight-specific, design-level soils-engineering investigation are implemented in the design and development of the site.

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8.0 RECOMMENDATIONS

From a geologic-hazards perspective, it is our opinion that the Evergreen I and II properties can be developed as proposed if the following recommendations and the details provided in the text of this report are incorporated in the project design, plans, and construction.

- Based on the limits of subsurface trench exploration, a building-exclusion zone is recommended 50 feet west of the east ends of the exploratory trenches KA-T-1, KA-T-2, KA-T-3, and KA-T-5 and 50 feet east of Station 0+45 of Trench KA-T-6 (Plates 3 and 4). No structures intended for human occupancy should be constructed east of this boundary unless further geologic investigations are carried out and approved by the City of San Jose.
- Procedures from the 1997 UBC, at a minimum, should be implemented for a code-equivalent lateral-force design of structures within the project area. Near-Source Factors N_a and N_v to be used at the project site are 1.3 and 1.6, respectively.
- A design-level soils-engineering report describing mitigation measures for potential expansive soils, colluvial and earth-flow deposits and other soil and foundation related conditions should be completed for the Evergreen I and II project sites. During the soils-engineering investigation, areas underlain by fill, colluvial and earth-flow deposits and other soft-soil areas should be further investigated and recommendations prepared that describe the engineering remediation needed.
- The soils-engineering report for the proposed debris basins on Evergreen I and II should include descriptions of the site's subsurface materials and groundwater and provide recommendations for foundations, site grading, drainage, and embankment design for the debris basins as well as remediation of colluvial and landslide material where present. Other general recommendations contained in Earth Systems Associate (2000) report pertaining to the debris basin should be incorporated in the design, construction and maintenance of both proposed debris basins on Evergreen I and II.

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- During site grading, Kleinfelder's trenches as well as previous subsurface exploration points and any other discovered areas of fill or loose soil should be excavated and recompacted as prescribed in the design-level soils-engineering report.
- The project civil engineer should evaluate potential flooding and provide design recommendations for the unlikely event of catastrophic failure of the water tank east of the Evergreen II property.

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9.0 LIMITATIONS

The information in this report is based on our field observations, exploratory excavations, review and evaluation of reports and maps available to us including previous fault investigations, and our knowledge of geologic conditions in the area. Our evaluation of fault conditions on the subject site is also based on our interpretation and projection of conditions observed in the exploration trenches. It is possible that geologic conditions could vary between the trenches. The accuracy of the information presented in this report should not be implied beyond the limitations of the methods described. We have prepared this report in substantial accordance with the generally accepted engineering geologic procedures and guidelines as they exist today. No warranty is expressed or implied including no warranty that the City of San Jose will accept this report as complete and acceptable submission for issuance of a Certificate of Geologic Clearance.

It should be noted that fluctuations in the groundwater level may occur due to variations in rainfall, temperature, irrigation, pumping from wells and possibly other factors that were not evident at the time of our investigation. If significant variations in the groundwater level are encountered during construction, it may be necessary for Kleinfelder to review the recommendations presented herein and provide adjustments as necessary.

This report may be used only by the client and only for the purposes stated, within a reasonable time from its issuance. Land use, site conditions (both on- and off-site) or other factors may change over time, and additional work may be required. Based on the intended use of the report, Kleinfelder may require that additional work be performed and that an updated report be issued. Non-compliance with any of these requirements by the client or anyone else, unless specifically agreed to in advance by Kleinfelder in writing will release Kleinfelder from any liability resulting from the use of this report by any unauthorized party.

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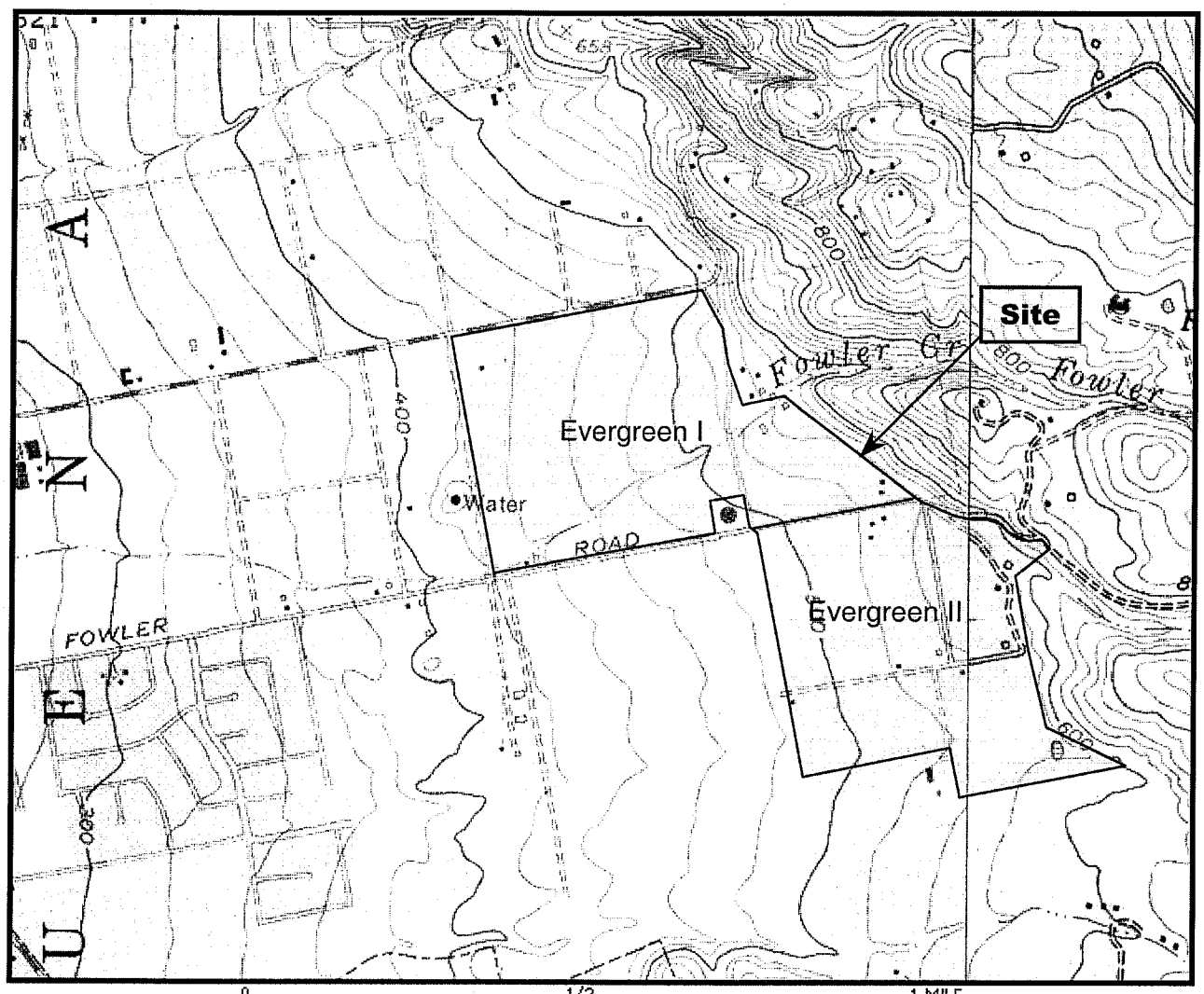

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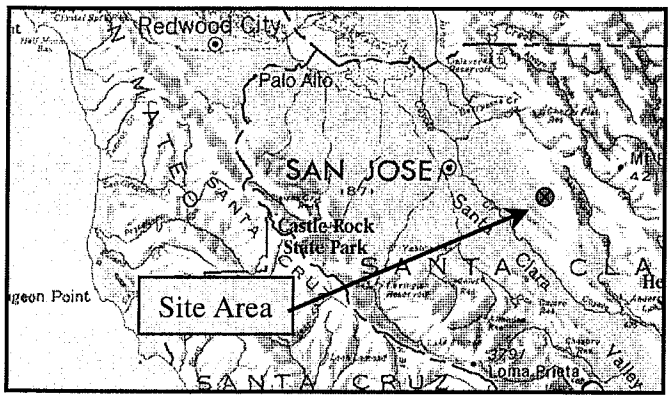

Base: U.S. Geological Survey,San Jose East and Lick Observatory 7½-minute quadrangles (printed 1980 and 1970, respectively)

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Site Vicinity Map

Evergreen I and II
Fowler and Yerba Buena Roads
San Jose, California

PROJECT NO.: 49205/flt

PLATE 1

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Aerial Photograph provided by Berg & Berg and HMH

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Areas of archeological significance

N

0 0.5 1 1.5 2

Scale thousand feet

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Site Aerial Photograph

Evergreen I and II
Fowler and Yerba Buena Roads
San Jose, California

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PLATE 2

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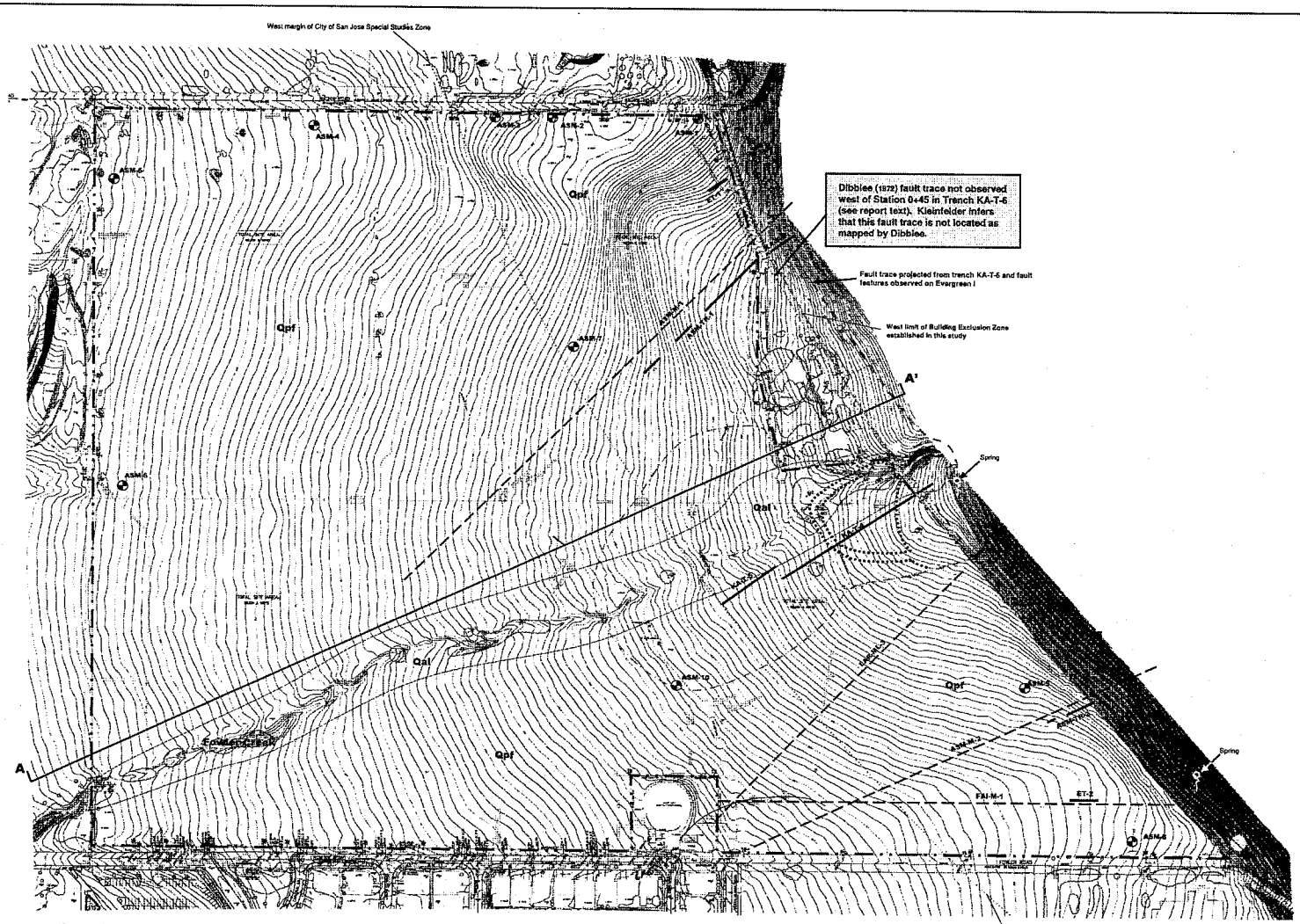

SCALE 0 100 200 300 400 500 FEET

N

EXPLANATION OF SYMBOLS

ET/FAI	Fries Associates (October 8, 1999, October 8, 1999b, September 1999, December 1999)
ASM	Applied Soil Mechanics (1960)
LAR	Louis A. Richardson (1996)
KA	Kleinfelder (2004)

Soil Boring
Test Pit / Trench
Magnetometer Traverse
City of San Jose Seismic Hazard Zone
Building Exclusion Zone (this study)
Spring
Property Line
Fault (Dibblee, 1972)
Geologic Contact
Fault trace (this study)
Geologic Cross Section
Proposed Debris Basin
Trash area

NOTE: Points of surface and subsurface investigation are approximately located

EXPLANATION OF GEOLOGIC UNITS

af	Artificial fill
Qal	Alluvial deposits (Holocene)
Qpf	Alluvial fan deposits (Pleistocene)
Kcg	Conglomerate (Cretaceous)

Original Date

No.	REVISION	BY	DATE

SITE PLAN AND GEOLOGIC MAP
EVERGREEN I

EVERGREEN DEVELOPMENT
SAN JOSE, CALIFORNIA

KLEINFELDER
Bay Area Fax (510) 351-5271
Ph. (510) 351-2000 / Toll Free (888) 5-KLEIN (171)

DATE: 10-13-04
SCALE: 1" = 100'
PLATE: 3 of sheets

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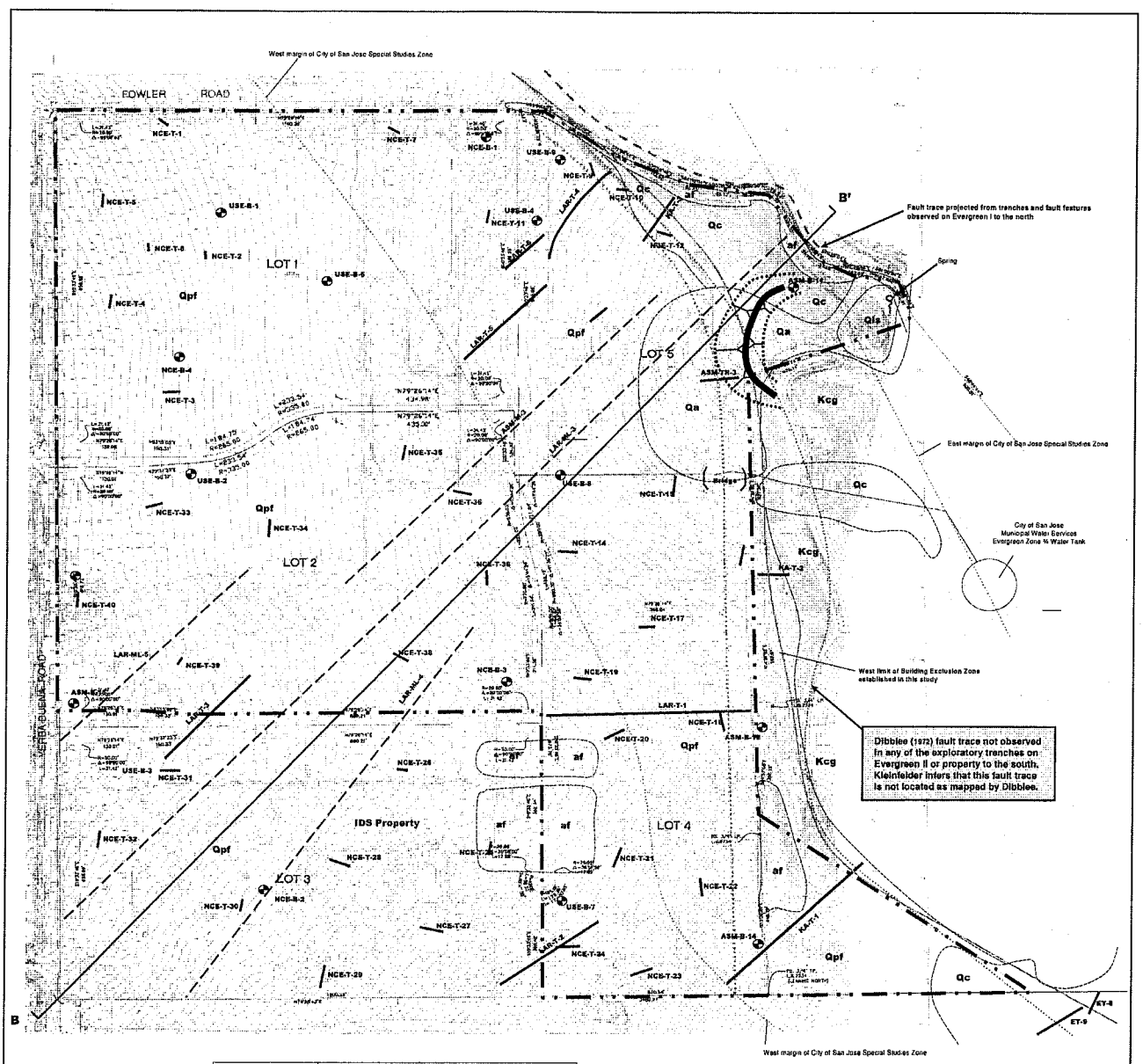

EXPLANATION OF SYMBOLS

NC	NorCal Engineering (2000)
ASM	Applied Soil Mechanics (1990)
USE	United Soil Engineering (1988)
LAR	Lewis A. Richardson (1996, 1999)
KA	Kleinfelder (2004)

Soil Boring
Test Pit / Trench
Magnetometer Traverse
City of San Jose Seismic Hazard Zone
Building Exclusion Zone (this study)
Property Line
Fault (Dibblee, 1972)
Geologic Contact
Fault trace (this study)
Proposed Debris Basin
Geologic Cross Section
Spring

NOTE: Points of surface and subsurface investigation are approximately located

EXPLANATION OF GEOLOGIC UNITS

af	Artificial fill
Qal	Alluvial deposits (Holocene)
Qc	Colluvium / earth flow (Holocene)
Qls	Landslide deposits (Holocene)
Qpf	Alluvial fan deposits (Pleistocene)
Kcg	Conglomerate (Cretaceous)

SCALE 0 100 200 300 400 500 FEET

N

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PROJ. NO. 49226 | DATE PLOTTED 11/10/04 | FILE LOC. | ORIGINAL SCALE REPRESENT FOR REDUCED PLANS
DRAWN BY: | CHECKED BY: MNC | DESIGNED BY: MNC

SITE PLAN AND GEOLOGIC MAP
EVERGREEN II

EVERGREEN DEVELOPMENT
SAN JOSE, CALIFORNIA

No.	REVISION	BY	DATE
△			
△			
△			
△			

DATE: 10-13-04
SCALE: 1" = 100'
PLATE: 4 of sheets

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Evergreen fault zone

City of San Jose Special Studies Zone

Hayward fault zone

N

0 1 2 3 4

Scale thousand feet

Map source: City of San Jose, 1983, Fault Hazard Maps, San Jose East and Lick Observatory 7½-minute quadrangles

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City of San Jose Special Studies Zone

Evergreen I and II
Fowler and Yerba Buena Roads
San Jose, California

PROJECT NO.: 49205/flt

PLATE

5

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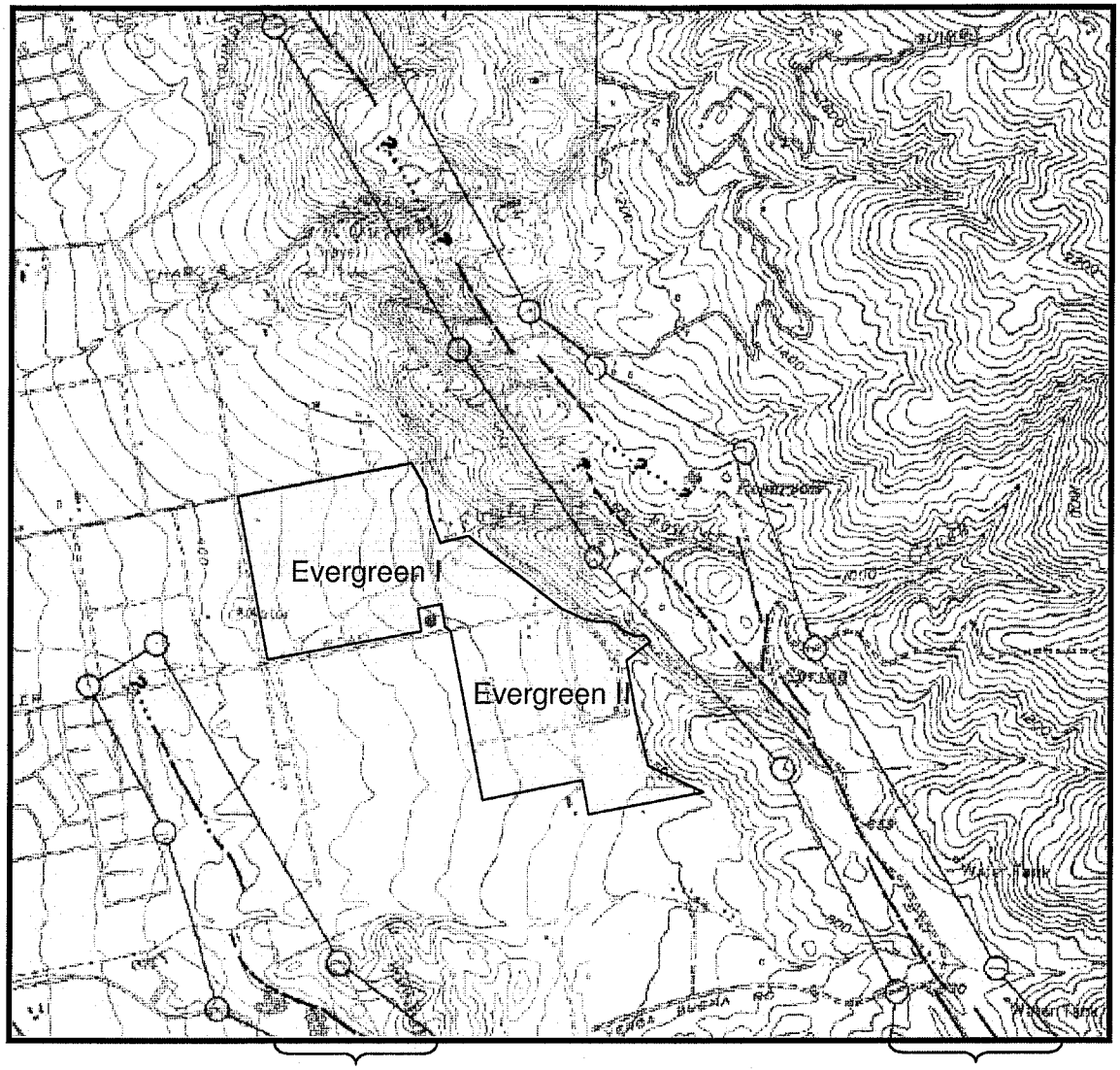

Evergreen fault zone

Hayward fault zone

Map source: California Geologic Survey, 2000, Digital Images of Official Maps or Alquist-Priolo Earthquake Fault Zones of California

Original in Color

Appx. Scale (feet)

0 1000 2000

N

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California Earthquake Fault Zones

Evergreen I and II
Fowler and Yerba Buena Roads
San Jose, California

PROJECT NO.: 49205/flt

PLATE

6

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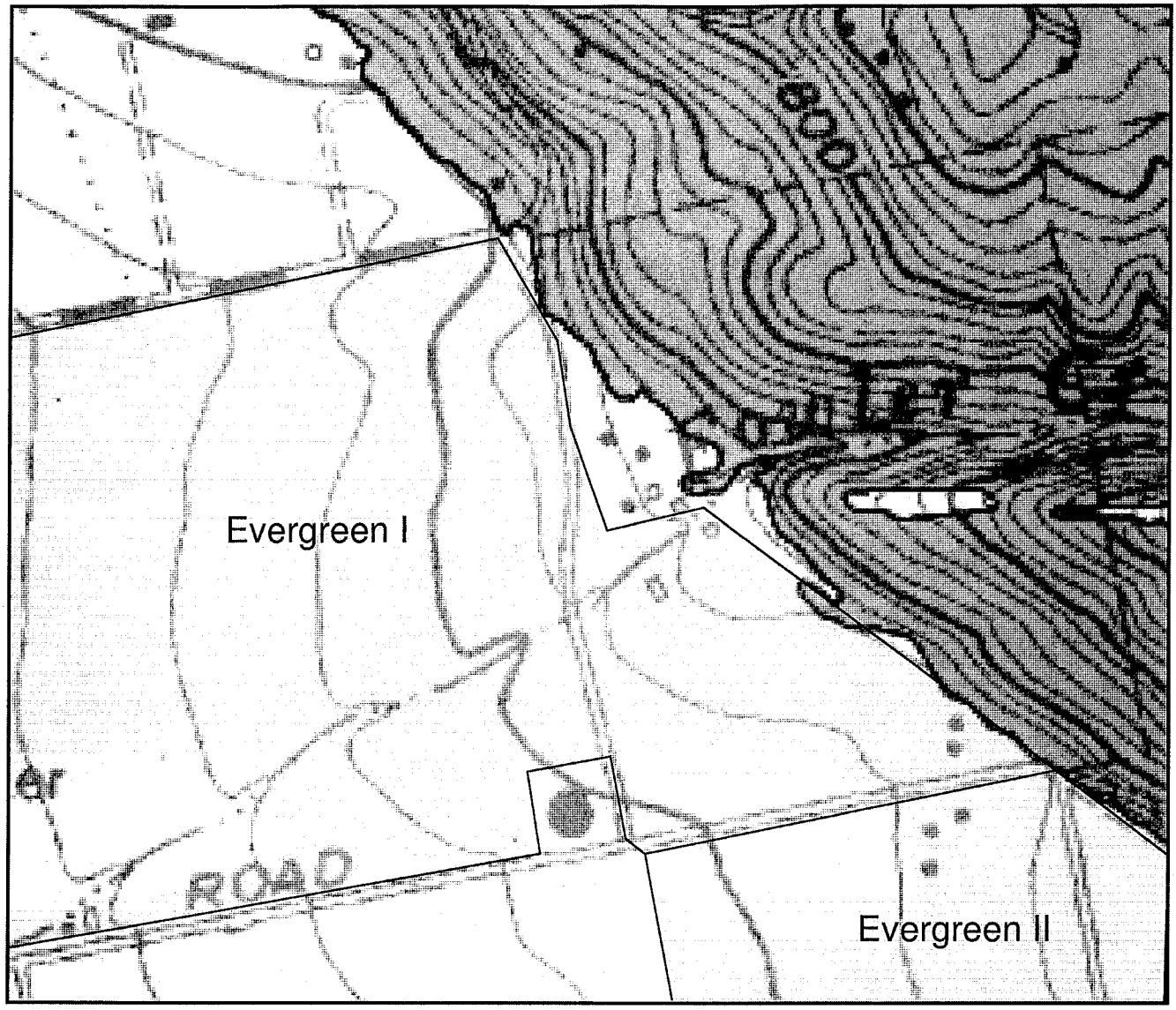

Note: The Seismic Hazard Zones Map for the Lick Observatory quadrangle to the east of this sheet is planned to be produced by the California Geologic Survey but is not yet available.

Appx. Scale
(feet)
0 500 1000

N

Map source: Seismic Hazard Zone Report, Seismic Hazard Zone Report for the San Jose East 7.5-minute Quadrangle, Santa Clara County, California 2001

Areas where previous occurrence of landslide movement, or local topographic, geological, geotechnical and subsurface water conditions indicate a potential for permanent ground displacements such that mitigation as defined in Public Resources Code Section 2693(c) would be required.

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California Seismic Hazard Zones Map

Evergreen I and II
Fowler and Yerba Buena Roads
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PLATE

7

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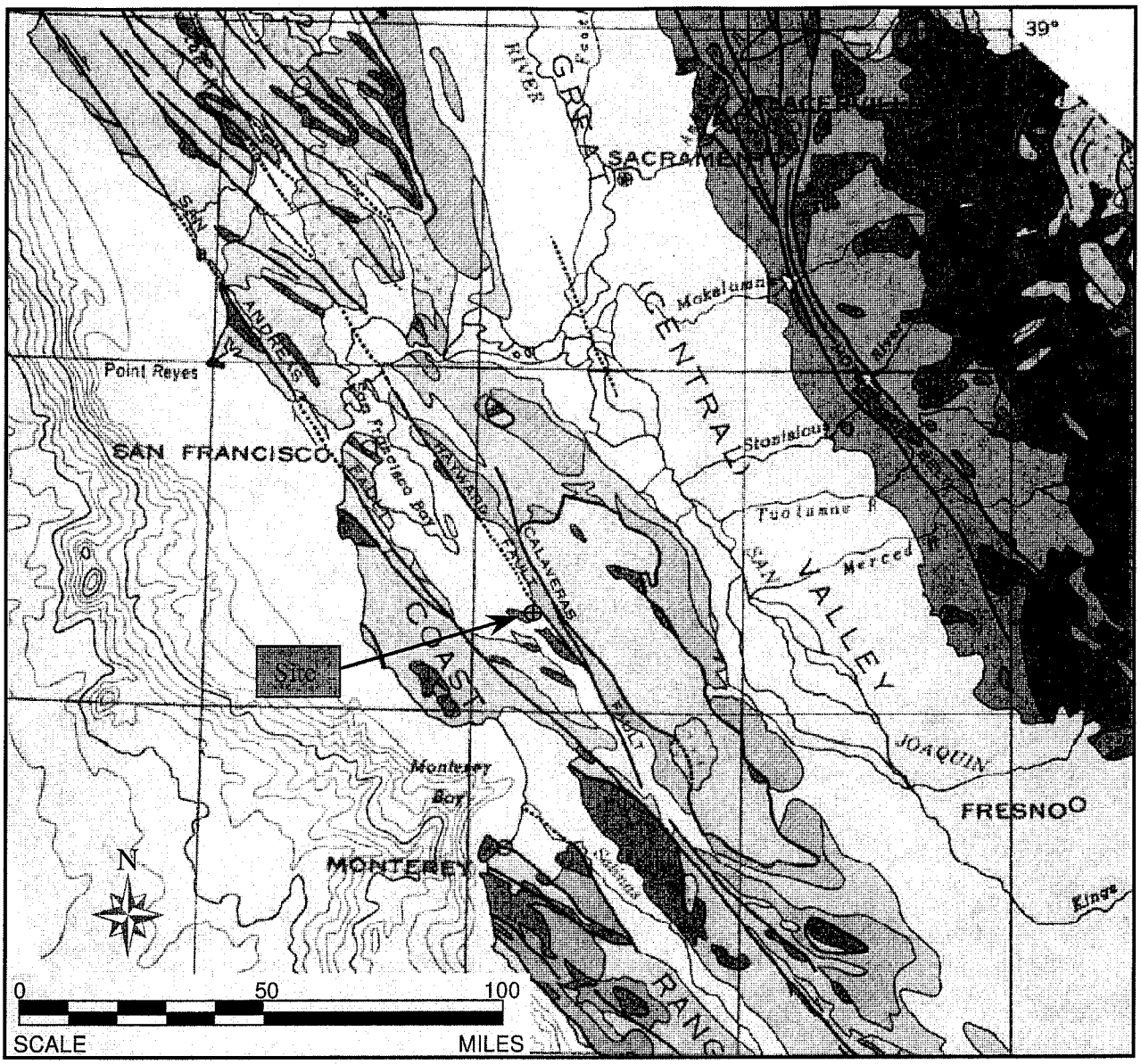

Map source: California Geological Survey, 2002, Geologic Map of California, California Department of Conservation used with permission.

Original in Color

EXPLANATION
of units in SF Bay Area

- Cenozoic nonmarine
- Cenozoic marine
- Late Mesozoic shelf and slope
- Late Mesozoic of the Franciscan Formation
- Fault, dotted where concealed, arrows indicate direction of movement
- Mesozoic Granitic rocks
- Mesozoic Ultramafic rocks

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Regional Geologic Map

Evergreen I and II
Fowler and Yerba Buena Roads
San Jose, California

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PLATE

8

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Appx. Scale (miles)

0 10 20 30 40

Reference:

Jennings, C.W., 1994, Fault activity map of California and adjacent area, with locations and ages or Recent volcanic eruptions; California geologic data map series, map no. 6, Department of Conservation, Division of Mines and Geology

EXPLANATION

- Fault with historic movement
- Fault with movement within the last 10,000 years
- Faults with movement older than 10,000 years or undifferentiated

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Regional Fault Map

Evergreen I and II
Fowler and Yerba Buena Roads
San Jose, California

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PLATE 9

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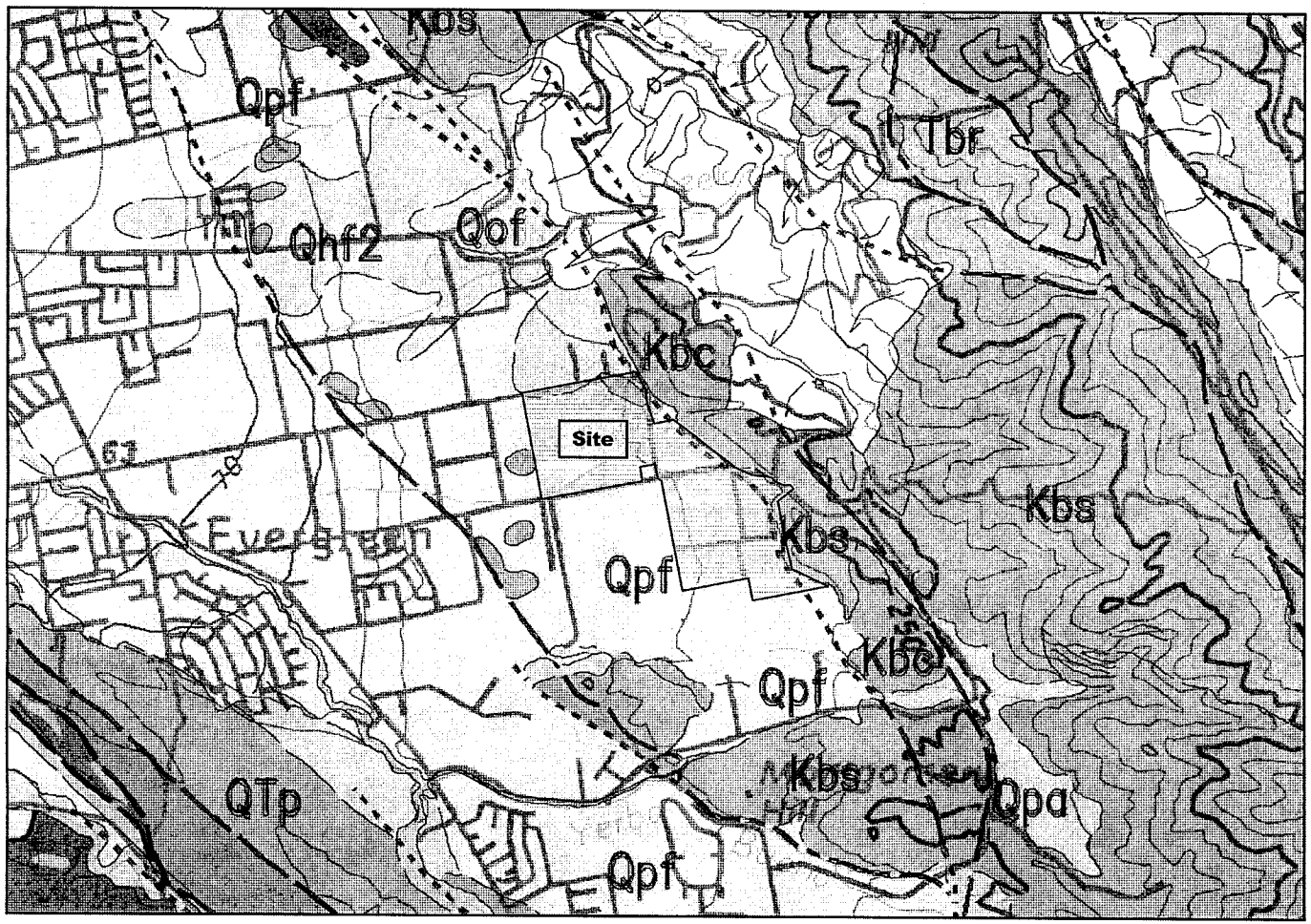

EXPLANATION

Qpf Alluvial fan deposits (Pleistocene)
Tbr Briones Sandstone (Tertiary)
Kbs Berryessa Formation shale (Cretaceous)
Kbc Berryessa Formation conglomerate (Cretaceous)
Landslide showing direction of movement
Fault

0 1 2 3 4
Scale thousand feet

N

Map source: Wentworth, C.M., Blake, M.C., McLaughlin, R.J., and Graymer, R.W., 1999, Preliminary geologic map of the San Jose 30 X 60-minute quadrangle, California: a digital database: U.S. Geological Survey, Open-File Report OF-98-795, scale 1:100000.

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Vicinity Geologic Map
Evergreen I and II
Fowler and Yerba Buena Roads
San Jose, California

PLATE
10

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The Modified Mercalli Intensity Scale

The effect of an earthquake on the Earth's surface is called the intensity. The intensity scale consists of a series of certain key responses such as people awakening, movement of furniture, damage to chimneys, and finally - total destruction. Although numerous intensity scales have been developed over the last several hundred years to evaluate the effects of earthquakes, the one currently used in the United States is the Modified Mercalli (MM) Intensity Scale. It was developed in 1931 by the American seismologists Harry Wood and Frank Neumann. This scale, composed of 12 increasing levels of intensity that range from imperceptible shaking to catastrophic destruction, is designated by Roman numerals. It does not have a mathematical basis; instead it is an arbitrary ranking based on observed effects. The Modified Mercalli Intensity value assigned to a specific site after an earthquake has a more meaningful measure of severity to the nonscientist than the magnitude because intensity refers to the effects actually experienced at that place. After the occurrence of widely-felt earthquakes, the Geological Survey mails questionnaires to postmasters in the disturbed area requesting the information so that intensity values can be assigned. The results of this postal canvass and information furnished by other sources are used to assign an intensity within the felt area. The maximum observed intensity generally occurs near the epicenter.

The lower numbers of the intensity scale generally deal with the manner in which the earthquake is felt by people. The higher numbers of the scale are based on observed structural damage. Structural engineers usually contribute information for assigning intensity values of VIII or above.

The following is an abbreviated description of the 12 levels of Modified Mercalli intensity.

I. Not felt except by a very few under especially favorable conditions.

II. Felt only by a few persons at rest, especially on upper floors of buildings.

III. Felt quite noticeably by persons indoors, especially on upper floors of buildings. Many people do not recognize it as an earthquake. Standing motor cars may rock slightly. Vibrations similar to the passing of a truck. Duration estimated.

IV. Felt indoors by many, outdoors by few during the day. At night, some awakened. Dishes, windows, doors disturbed; walls make cracking sound. Sensation like heavy truck striking building. Standing motor cars rocked noticeably.

V. Felt by nearly everyone; many awakened. Some dishes, windows broken. Unstable objects overturned. Pendulum clocks may stop.

VI. Felt by all, many frightened. Some heavy furniture moved; a few instances of fallen plaster. Damage slight.

VII. Damage negligible in buildings of good design and construction; slight to moderate in well-built ordinary structures; considerable damage in poorly built or badly designed structures; some chimneys broken.

VIII. Damage slight in specially designed structures; considerable damage in ordinary substantial buildings with partial collapse. Damage great in poorly built structures. Fall of chimneys, factory stacks, columns, monuments, walls. Heavy furniture overturned.

IX. Damage considerable in specially designed structures; well-designed frame structures thrown out of plumb. Damage great in substantial buildings, with partial collapse. Buildings shifted off foundations.

X. Some well-built wooden structures destroyed; most masonry and frame structures destroyed with foundations. Rails bent.

XI. Few, if any (masonry) structures remain standing. Bridges destroyed. Rails bent greatly.

XII. Damage total. Lines of sight and level are distorted. Objects thrown into the air.

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Modified Mercalli Intensity Scale

Evergreen I and II
Fowler and Yerba Buena Roads
San Jose, California

PROJECT NO.: 49205/flt

PLATE 11

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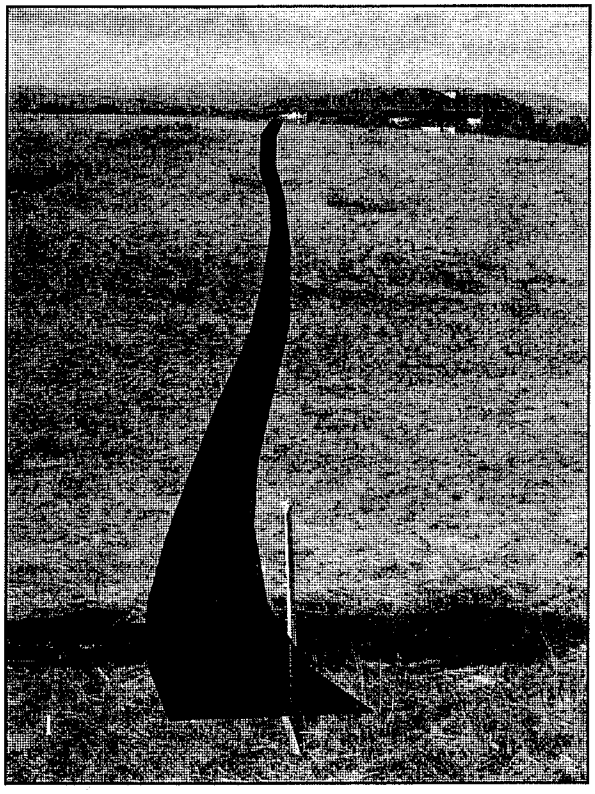

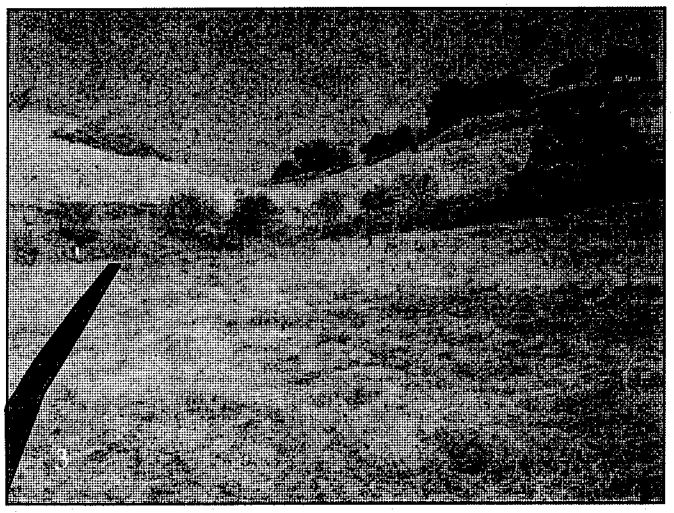

1 Evergreen II - Approximate location of Trench KA-T-1, looking southwest from east end of trench.

2 Evergreen II - Approximate location of Trench KA-T-2, looking southeast from above trench location.

3 Evergreen II - Approximate location of Trench KA-T-3, looking northeast from near west end of trench.

4 Evergreen I – Approximate location of Trench KA-T-4, looking northeast from near the west end of the trench.

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Site Photographs

Evergreen I and II
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San Jose, California

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PLATE 12

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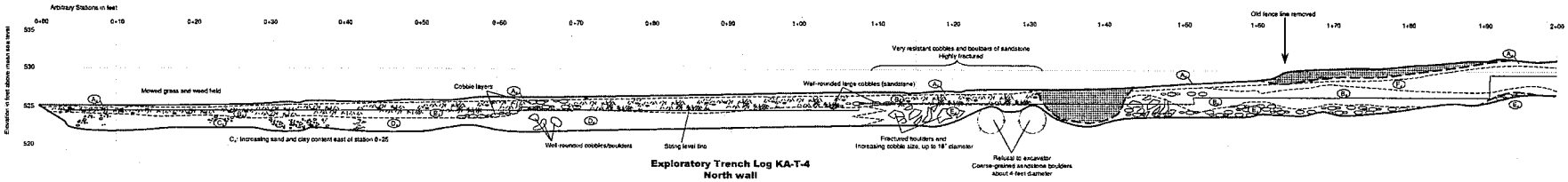
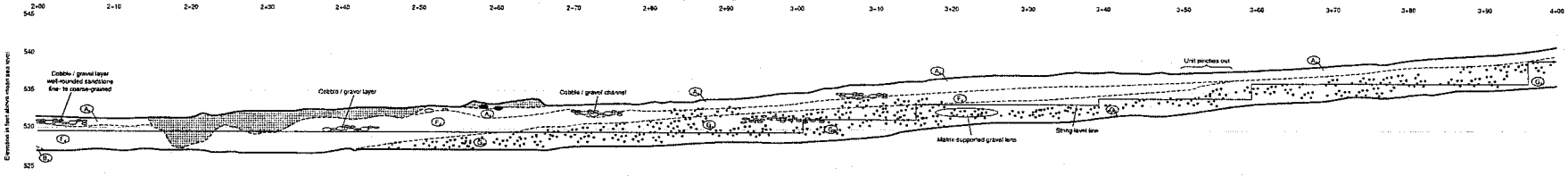
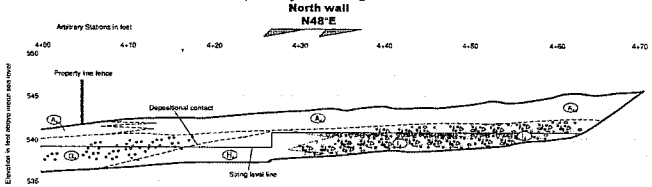
Exploratory Trench Log KA-T-4
North wall
N48°E



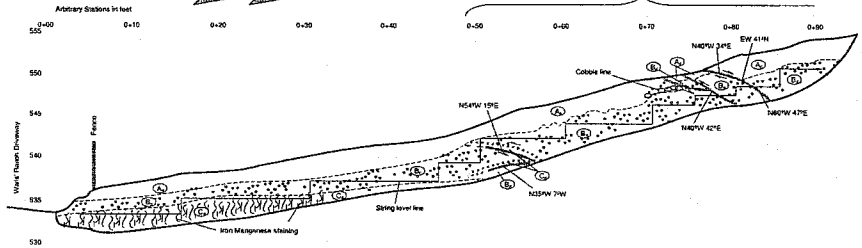
Exploratory Trench Log KA-T-4
North wall
N48°E



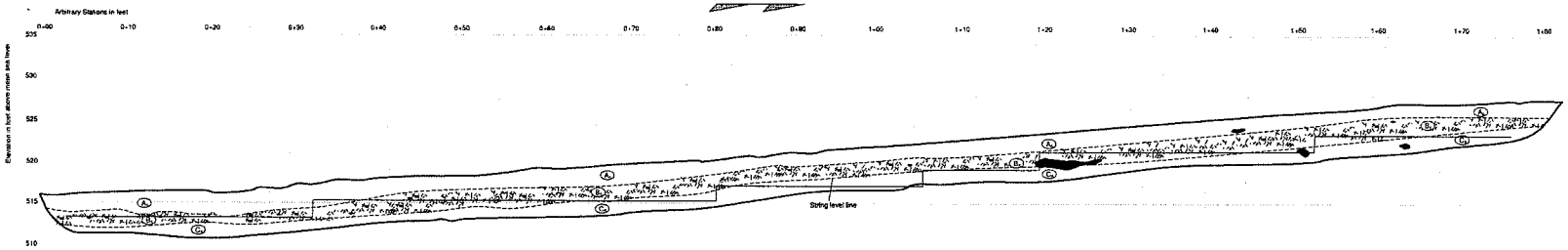
Exploratory Trench Log KA-T-4
North wall
N48°E



Exploratory Trench Log KA-T-6
North wall
N64°E



Exploratory Trench Log KA-T-5
North wall
N46°E



UNIT	DESCRIPTION TRENCH KA-T-4 and KA-T-5
A, A, A	Silty sand (SM), light tan to gray, dry, dense, bioturbated throughout with in-filled animal burrows and worm tubes, small minor gravel of subrounded and fractured sandstone, residual soil. East of Station 4+10 soil is probably stream and flood deposits along Fowler Creek.
B, B	Sandy clay with gravel (CH), medium red brown, dry, hard, minor gravel is subrounded to angular and fractured quartzite, chert, rare rounded cobbles up to 6-inches maximum diameter.
C	Conglomerate (Qpf), red-brown, moderate to highly weathered, clasts of sandstone are subrounded, up to approximately 4-inches maximum diameter and randomly fractured, matrix is sandy clay, hard, cobble to cobble contact.
D, E	Claystone (Qpf), medium to dark red-brown, moderately weathered, no visable fractures or joints, contains abundant gravel and cobbles, cobbles are matrix supported.
E	Conglomerate (Qpf), moderate to highly weathered, clasts of sandstone are subrounded, up to approximately 18-inches maximum diameter and randomly fractured, matrix is sandy clay, red-brown, hard, cobble to cobble contact.
F	Clayey sandstone (Qpf), moderate to highly weathered, clasts of sandstone are subrounded, up to approximately 8-inches maximum diameter and randomly fractured.
G	Claystone (Qpf), medium to dark red-brown, moderately weathered, no visable fractures or joints, contains abundant gravel and cobbles, cobbles are matrix supported.
H	Sandstone (Qpf), gray brown, highly weathered, friable, non-fractured, with gravel and gravelly sand (SP), gray brown, dry, loose, weakly cemented, fine to medium-grained sand, gravel is derived from Merryvale Formation.
I	Conglomerate (Qpf), dark brown, highly weathered, friable and weak, clasts of sandstone are subrounded, up to approximately 4-inches maximum diameter and randomly fractured, matrix is sandy clay, red-brown, hard, cobble to cobble contact.

UNIT	DESCRIPTION TRENCH KA-T-6
A	Silty clay (CL), dark brown, dry, hard, moderately cemented (residual soil)
B	Claystone (Qpf), light brown, moderately weathered, weak, dry, chert gravel (1" to 3" diameter) angular.
C	Claystone (Qpf), red brown, moderately weathered, no fracturing, moderately strong to weak, contains numerous manganese stringers and staining.

Krotovina, in-filled animal burrow.

Fill: Sandy clay with bottles, glass, metal, plastic, cloth, boards, wire, and other trash.

No.	REVISION	BY	DATE

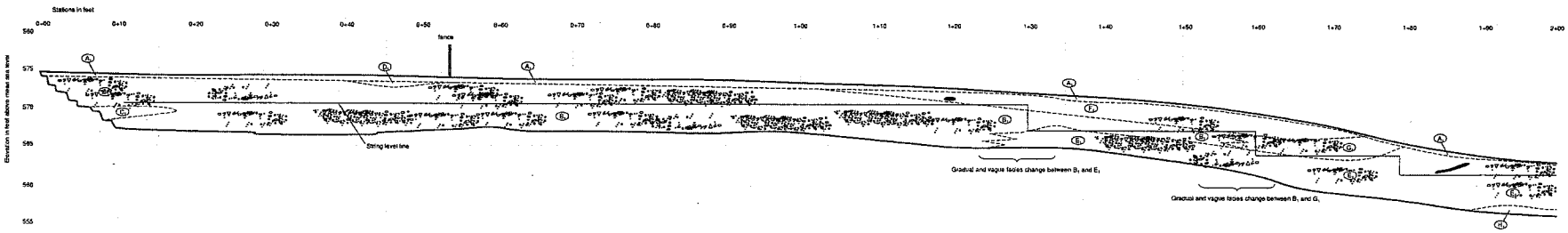
EXPLORATORY TRENCH LOGS KA-T-4, KA-T-5, KA-T-6
EVERGREEN I

EVERGREEN DEVELOPMENT
SAN JOSE, CALIFORNIA

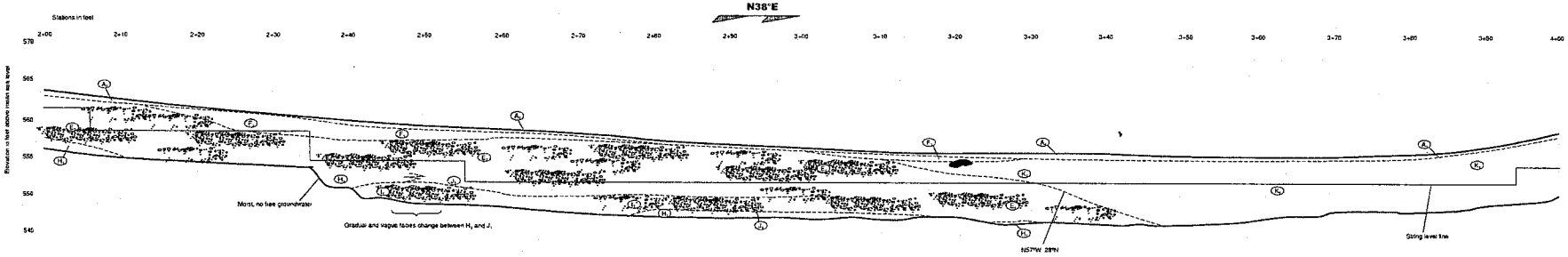
KLEINFELDER

DATE: 11-11-04
SCALE: As shown
PLATE: 13 of sheets

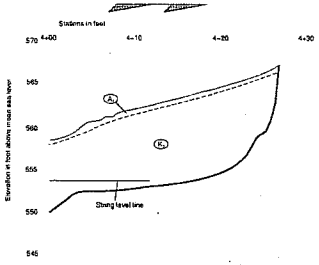
Exploratory Trench Log KA-T-1
North wall
N38°E



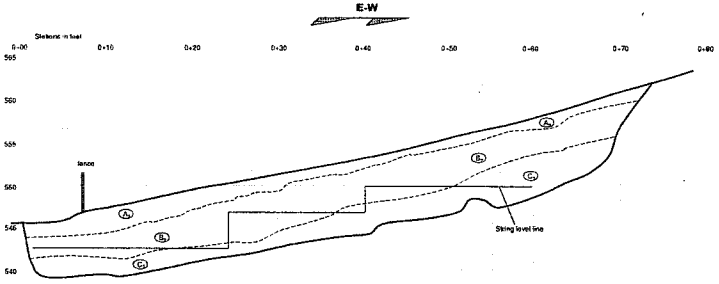
Exploratory Trench Log KA-T-1
North wall
N38°E



Exploratory Trench Log KA-T-1
North wall
N38°E

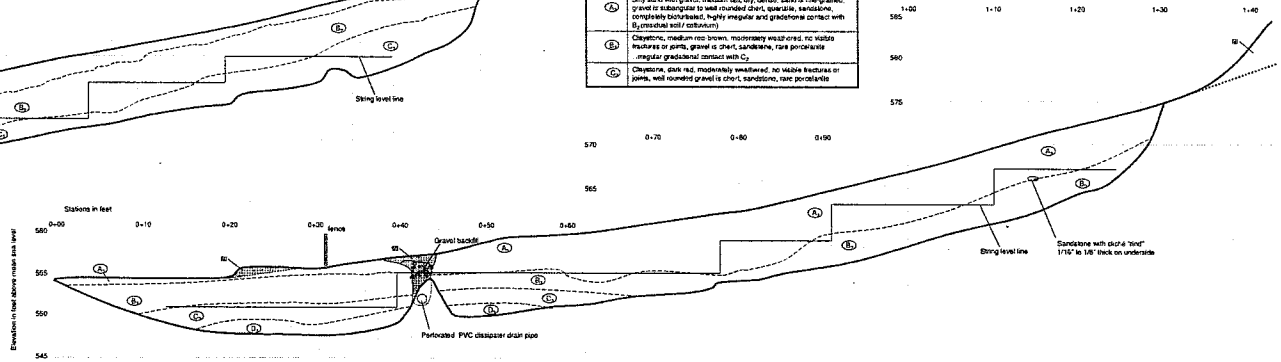


Exploratory Trench Log KA-T-2
North wall
E-W



UNIT	DESCRIPTION TRENCH KA-T-2
A_2	Silty sand with gravel, medium tan, dry, dense, sand is fine-grained, gravel is subangular to well rounded chert, quartzite, sandstone, completely bioturbated, highly irregular and gradational contact with B_2 (residual soil / colluvium)
B_2	Claystone, medium red-brown, moderately weathered, no visible fractures or joints, gravel is chert, sandstone, rare porcelanite ..irregular gradational contact with C_2
C_2	Claystone, dark red, moderately weathered, no visible fractures or joints, well rounded gravel is chert, sandstone, rare porcelanite

Exploratory Trench Log KA-T-3
North wall
N34°E



UNIT	DESCRIPTION TRENCH KA-T-1
A_1	Sandy silt with minor gravel (ML), medium gray-brown, dry, stiff to very stiff, gravel is well rounded clasts up to 1½" diameter (residual soil)
B_1	Conglomerate with matrix of clayey sand (Qpf), highly weathered, no fractures, matrix is dark red brown, dry to slightly moist, very stiff to hard, well indurated, gravel is mixed size up to 1" diameter, well rounded, clay skins on gravel increases with depth, at bottom of unit clay skins are well developed.
C_1	Claystone (Qpf), medium red-brown with minor vertical manganese stains, slightly moist, stiff to very stiff.
D_1	Fat clay with gravel (CH), medium red-brown, dry, stiff to hard, well developed soil peds.
E_1	Conglomerate with matrix of clayey sand (Qpf), highly weathered, no fractures, matrix is light brown to tan, dry to slightly moist, dense to very dense, well indurated, gravel a mixed size up to 1" diameter, well rounded, clay skins on gravel increases with depth, at bottom of unit clay skins are well developed.
F_1	Lean clay with sand, medium tan to brown, dry, stiff, abundant in-filled animal burrows and vertical worm tubes, thoroughly bioturbated (Colluvium, earth flow)
G_1	Conglomerate with matrix of clayey sand (Qpf), highly weathered, no fractures, matrix is medium red brown, dry to slightly moist, very stiff to hard, well indurated, gravel is mixed size up to 1" diameter, angular to sub angular chert and quartzite and rounded sandstone and shale.
H_1	Claystone (Qpf), medium brown, slightly moist, stiff to very stiff
L	Conglomerate with matrix of clayey sand (Qpf), highly weathered, no fractures, matrix is mottled red brown and gray, moist, loose to very loose, sand is medium to coarse-grained, gravel is mixed size up to 2½" diameter, angular to sub angular chert and quartzite.
J_1	Claystone with minor gravel and sand (Qpf), red-brown, slightly moist, stiff to very stiff, sand is fine-grained, slightly moist.
K_1	Claystone with cobbles and gravel (Qpf), red-brown, hard to very hard, highly weathered, no visible fractures or joints, cobbles and gravel are well rounded with fresh surfaces.

UNIT	DESCRIPTION TRENCH KA-T-3
A_3	Silty sand with gravel, medium gray, dry, loose to dense, sand is fine-grained, gravel up to 1-inch diameter, subangular to rounded chert, fine-grained sandstone. East of about Station 0+40 unit is completed bioturbated, mixture of in-filled animal burrows and worm tubes, high organic content, rotted roots and grass (earth flow)
B_3	Claystone, medium brown to medium red-brown, dry, hard, moderately weathered, no visible fractures or joints, gravel is angular to moderately rounded chert and sandstone.
C_3	Claystone, red-brown, slightly moist, moderately weathered, no visible fractures or joints, gravel is rounded clasts of sandstone, chert, quartzite, sandy matrix a fine- to coarse-grained.
D_3	Claystone, red-brown to tan, slightly moist, moderately weathered, no visible fractures or joints, gravel is rounded clasts of sandstone, chert, quartzite, fat clay matrix.

Krotovina: In-filled animal burrows.

Fill: Clayey sand, loose to dense.

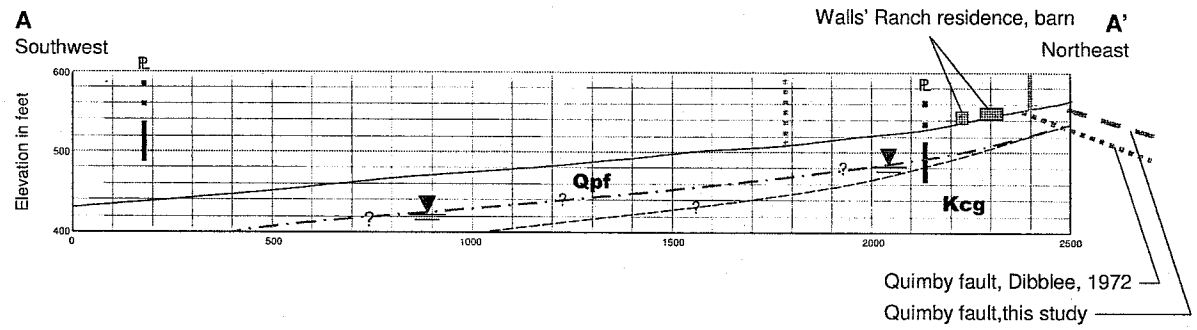
No.	REVISION	BY	DATE

EXPLORATORY TRENCH LOGS KA-T-1, KA-T-2, KA-T-3
EVERGREEN II

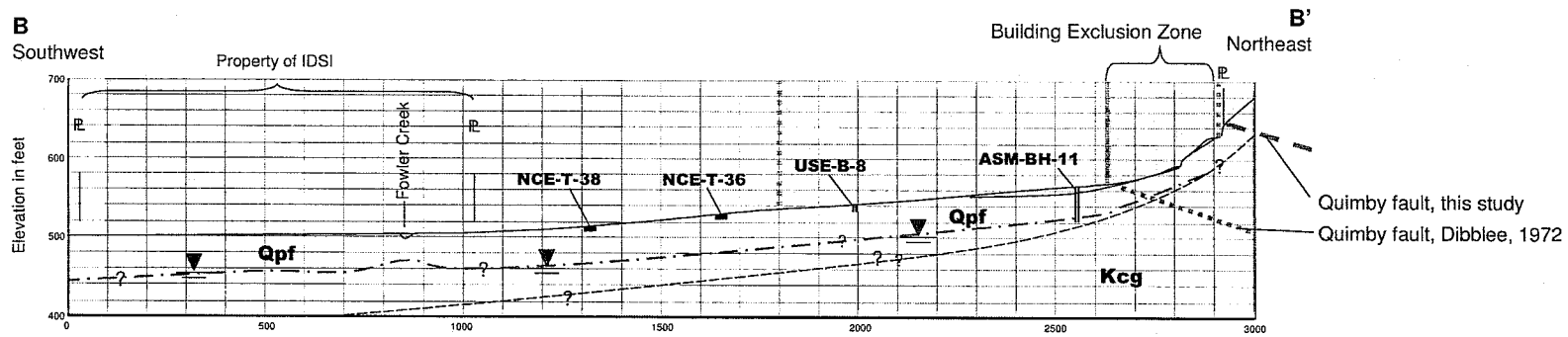
EVERGREEN DEVELOPMENT
SAN JOSE, CALIFORNIA

KLEINFELDER

DATE: 11-11-04
SCALE: As shown
PLATE: 14
of sheets



Qpf	Alluvial fan deposits (Pleistocene)
Kcg	Conglomerate (Cretaceous)
▼	Groundwater table
	City of San Jose Seismic Hazard Zone
	Building Exclusion Zone (this study)



Vertical exaggeration = 2X horizontal
Lines of sections are shown on Plates 3 and 4

PROJECT NO.: 49205/flt

Compiled by: P. Holland	Date: 10/15/04
Reviewed by: M. Clark	Revision date:

KLEINFELDER
1362 Ridder Park Drive
San José, California 95131
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Geologic Cross Sections A – A' and B – B'

Evergreen I and II
Fowler and Yerba Buena Roads
San Jose, California

PLATE
15

Library file: L:\2004\library\projects\Evergreen*.ppt

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GEOTECHNICAL EXPLORATION AND SUPPLEMENTAL FAULT STUDY

EVERGREEN VIEW

SAN JOSE, CALIFORNIA

SUBMITTED

TO

WESTERN PACIFIC HOUSING

PLEASANTON, CALIFORNIA

PREPARED

BY

ENGEO INCORPORATED

PROJECT NO. 5056.3.500.01

JULY 19, 2004

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ENGEO
INCORPORATED

GEOTECHNICAL
ENVIRONMENTAL
WATER RESOURCES
CONSTRUCTION SERVICES

Project No.
5056.3.500.001

July 19, 2004

Mr. Terry Pries
Western Pacific Housing
6658 Owens Drive
Pleasanton, CA 94588

Subject: Evergreen View
San Jose, California

**GEOTECHNICAL EXPLORATION
AND SUPPLEMENTAL FAULT STUDY**

Dear Mr. Pries:

With your authorization, we conducted a supplemental geotechnical exploration for the proposed Evergreen View project located in San Jose, California.

The accompanying report contains our exploration data and conclusions, and provides site grading, drainage, and foundation recommendations for development of the subject site. It is our opinion that the proposed development is feasible from a geotechnical standpoint, provided the recommendations included in this report are followed.

We are pleased to have been of service to you on this project and will be glad to consult further with you and your design team.

Very truly yours,

ENGEO INCORPORATED

Micah B. Silvey

Matthew R. Harrell
mbs/mrh/jf:gex

Reviewed by:

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THESE APPENDICES ARE ON file WITH the City of SAN JOSE AND ARE AVAILABLE for Review UPON REQUEST.

CONTACT: JOHN BATY
PLANNING Dept.
(408) 535-7894

INTRODUCTION

Purpose and Scope

The purpose of this supplemental geotechnical report was to expand upon the geotechnical information gathered from prior explorations, further assess geologic hazards at the site and in the project area; confirm the suitability of the site for the proposed development; and provide recommendations regarding treatment of geotechnical constraints, site grading and drainage, foundation design, and preliminary pavement sections for the proposed residential development and appurtenant streets.

The scope of our work included a review of readily available literature and geologic maps for the project area; drilling nine exploratory boreholes with collection of subsurface samples; excavating eight test pits and nine trenches; laboratory testing of subsurface materials collected from the boreholes and trenches; analysis of the gathered geotechnical data; and preparation of this report summarizing our findings and recommendations for site development.

This report was prepared for the exclusive use of Western Pacific Housing and its design team consultants. In the event that any changes are made in the character, design, or layout of the development, the conclusions and recommendations contained in this report should be reviewed by ENGEO Incorporated to determine whether modifications to the report are necessary. This document may not be reproduced in whole or in part by any means whatsoever, nor may it be quoted or excerpted without the express written consent of ENGEO Incorporated.

Site Location and Description

The site is located northeast of the intersection between Yerba Buena Road and Old Yerba Buena Road, in San Jose California (Figure 1). The site is approximately 121 acres in size and is

roughly L shaped (Figure 2). Evergreen Creek runs from the southeast corner of the property, through the property, and then along the north side of the western portion of the site.

The topography within the subject area slopes towards the west, with knolls and ridgelines situated at the western and eastern portions of the site, respectively. Site elevations range from about 517 feet above msl at the southwest boundary to about 700 feet above mean sea level (msl) along the east boundary. Terrain beyond the eastern project boundary steepens and is native open space land.

The site has been partially graded under the testing and observation of ENGEO to establish rough grade for roadways associated with the previously planned Technology Park. The site is currently vegetated with native grasses, shrubs, and sparse trees. Portions of the site were formerly an orchard and some of the trees remain on site.

Proposed Development

Based upon conceptual information provided for our use, it appears that the site will be developed as a mixed-use residential development, including single- and multi-family homes. We anticipate up to 3-story structures of wood-framed construction; therefore, the building loads are expected to be light to moderate.

Previous Studies

ENGEO Incorporated conducted a preliminary geotechnical exploration in 2000, which included the drilling of eight exploratory borings and subsequent laboratory testing. This report addressed potential seismic hazards and other geotechnical issues related to the subject site, and gave preliminary recommendations regarding grading and drainage and foundation design for the

proposed commercial structures. The boring logs and laboratory testing data from our 2000 study are presented in Appendix D.

Earth Sciences Associates (ESA) conducted a geotechnical feasibility study in 1980, which included excavating and logging seven exploratory trenches and drilling five mud rotary borings. One trench (TR-7) and one boring (RD-2) are located off site to the northwest as shown on Figure 2. The findings of the ESA exploration are discussed in more detail in subsequent sections of this report; however, based on their findings, ESA provided recommendations for a building setback zones for the Quimby fault along the northeastern portion of the site and for a second shear zone (Fault A, Figure 2) that was encountered in the northern portion of the site. The boring logs, trench logs, and laboratory testing data from this study are presented in Appendix E.

A supplemental study was performed by ESA in 1992 to further define the limits of the shear zone (Fault A, Figure 2) identified in their 1980 study. This study consisted of two additional trenches. Based on this exploration, the approximate location of the north end of Fault A was characterized and a 100-foot-wide building set back zone was recommended by ESA for Fault A. A building setback of 100 feet from the trace of the Quimby fault as mapped by Dibblee (1972) was also recommended by ESA. ESA also gave recommendations for a planning restriction zone in the northern portion of the site pending further exploration.

Previous fault exploration was performed by Parikh Consultants (2002) in the southeastern portion of the site in conjunction with studies for a proposed water tank. The location of the exploratory trench is shown on Figure 2. A fault feature (Fault B) was encountered, and Parikh Consultants recommended a setback from the fault be established for the planned water tank.

A regional study was conducted by William Lettis & Associates, Inc. (WLA) to evaluate the near-surface geometry and late Quaternary surficial deformation related to reverse faulting,

specifically the Evergreen, Quimby, and Silver Creek faults, in eastern Santa Clara Valley. Through the evaluation of offset stream terrace deposits at the Evergreen and Quimby faults, the study suggests that the Evergreen and Quimby faults are likely secondary reverse faults associated with the Calaveras Fault system and not a primary seismic source, with evidence of displacement most likely during the late Pleistocene to early Holocene. The shallow angle of the faults projects to an intersection with the Calaveras Fault system at a depth of 2.5 to 3 miles and suggests that any rupture will likely be in conjunction with, or triggered by, an earthquake on the Calaveras Fault system.

GEOLOGY AND SEISMICITY

Regional Geology

The subject property is located in the Santa Clara Valley within the Coast Ranges Province. The geologic units mapped in this area consist of Quaternary-age upper Pleistocene alluvial fan deposits (Qpf) and Cretaceous-age conglomerate (Kbc) and interbedded sandstone and shale (Kbs) of the Berryessa Formation at the western and eastern limits (Wentworth, 1999). Bedrock structure in the area generally strikes to the northwest and dips at an inclination of about 20 to 40 degrees to the northeast to east. The site geology is similarly mapped by Dibblee (1972). The geologic setting of the site is shown on the attached Regional Geologic Map, Figure 3.

Faulting and Seismicity

The site is not located within a State of California Earthquake Fault Hazard Zone (1982). The northeast portion of the property is located within the City of San Jose Special Study Zone for the Quimby fault, Figure 4. In addition, a number of active faults capable of causing strong ground shaking at the site trend through the surrounding region.

A recent study completed by William Lettis & Associates (2002), for the U.S. Geological Survey suggests that the Quimby Fault is likely a secondary structure associated with the Calaveras Fault System and not a potential primary source. Any potential fault rupture is expected to be in conjunction with or triggered by larger earthquakes on the Calaveras Fault System. The closest known active fault[1] to the property is the southeast extension of the Hayward fault mapped approximately 1,500 feet to the east. Other active faults include the Calaveras Fault located

[1] Faults are classified into several types according to their activity status. Active or Holocene faults are those that have had surface displacement within the last 11,000 years. Potentially-active or Quaternary faults are defined by the State Geologist as those faults that have had surface displacement during the last 2 to 3 million years. Inactive faults are faults within no history of Quaternary movement.

about 2.5 miles east of the site; and the San Andreas Fault located approximately 15 miles southwest of the site. The potentially active Evergreen Fault lies just west of the site (WLA, 2002).

The San Andreas Fault represents an active crustal plate boundary that is expected to produce the maximum probable earthquake for the region. Other active faults of coastal California are lesser-order features of the same stress-strain system. A Regional Faulting and Seismicity Map is presented on Figure 5 that shows the approximate location of major active faults and significant historic earthquakes with respect to the site.

FIELD EXPLORATION

The field exploration for this study was conducted in April 2004 and included drilling nine exploratory borings and excavating eight test pits and nine trenches. The logs of borings, test pits, and trenches are presented in Appendix A. The approximate locations of the borings, test pits, and trenches are presented on Figure 2. The locations of borings and test pits were determined by pacing from existing features and should be considered accurate only to the degree implied by the method used. The exploratory trench locations were subsequently as-built surveyed by HMH, Incorporated.

Test Borings

The test borings were drilled using a truck-mounted drill rig equipped with 4-inch-diameter solid flight augers, a manual-trip hydraulic hammer, and drill rods to keep the hammer near the ground surface. The borings ranged in depth between 6½ and 21 feet below ground surface, depending upon location. An ENGEO engineer logged the boreholes in the field and collected soil samples using a 3-inch O.D. California-type split-spoon sampler fitted with 6-inch-long brass liners or a 2-inch O.D. Standard Penetration Test (SPT) split-spoon sampler.

The penetration of the split-spoon samplers into the subsurface materials was field recorded as the number of blows needed to drive the sampler 18 inches in 6-inch increments using the 140-pound hammer with a 30-inch drop. The report boring logs represent the actual field blow counts for the last one foot of penetration and have not been subjected to conversion factors to achieve representative SPT results.

The field logs for the borings were used to develop the report boring logs, which are located in Appendix A. The boring logs depict subsurface conditions within the borings for the date of site

activities; however, subsurface conditions may vary with time. The boreholes were backfilled on the day of field exploration activities.

Test Pits and Trenches

Eight exploratory test pits and nine trenches were excavated at the site using a rubber-tired tractor-mounted backhoe. The approximate locations of the test pits and trenches are shown on Figure 2. The test pits and trenches were located by pacing and estimating distances from features shown on the topographic base map. The logs were then used to develop the test pit and trench logs presented in this report (Appendix A).

The test pits and trenches were backfilled with nominal compactive effort. Test pits and trenches within the development area that are not completely removed by design cuts will require overexcavation and recompaction during site grading.

Laboratory Testing

Following the field exploration, the collected soil samples were reexamined in our laboratory to confirm field classifications. Representative soil samples recovered from the borings and test pits were tested for the following physical characteristics:

Characteristic	Test Method	Location of Results Within this Report
Natural Unit Weight and Moisture Content	ASTM D-2216	Boring Logs, Appendix A
Atterberg Limits	ASTM D-4318	Appendix B
Gradation	ASTM D-422	Appendix B
Direct Shear	ASTM D-3080	Appendix B
Compaction Curve	ASTM D-1557	Appendix B

The laboratory test results are shown on the boring logs (Appendix A), with individual test results presented in Appendix B. Several bulk and miscellaneous liner samples were collected from within the exploratory trenches and test pits, and were subsequently subjected to laboratory testing as shown in Appendix B. These samples were collected from the following locations:

Sample Name	Sample Location
Bulk 1	Test Pit TP-1
Bulk 6	Test Pit TP-4
Bulk A	Exploratory Trench ET-6 (colluvium)
Bulk B	Exploratory Trench ET-6 (alluvium)
L2T2	Exploratory Trench ET-2 (5-foot depth)

Subsurface Stratigraphy and Bedrock

Artificial Fill. Areas where existing fills were observed are identified as "Qaf1 or Qaf2" on the Geologic Map, Figure 2. Qaf1 refers to undocumented fill and Qaf2 refers to engineered fill, observed and tested by ENGEO in 2001. A large area of existing undocumented fill is situated at the southwest portion of the site, off the eastern and southern base of the knoll. It appears as though the undocumented fill materials may range in thickness up to possibly 10 feet, according to the borings in the vicinity and from a comparison of older and current topographic maps. The undocumented fill was placed within a low-lying drainage swale depression to create a relatively level area. The material below the undocumented fill is a dark gray critically expansive clay, down to a depth of about 14 feet. Atterberg Limits testing of the soil resulted in a Plasticity Index (PI) of 61, indicating that it is critically expansive when subjected to fluctuations in moisture content. Clay pipe shards were encountered at a depth of approximately 12 feet in Test Pit 4, within the dark gray colluvial material, indicating that it is most likely undocumented fill as well.

No records pertaining to the placement of the undocumented fills could be found, and they are therefore considered to be non-engineered. Non-engineered fills can be highly variable and potentially compressible.

Numerous other fills have been placed adjacent to the roadway alignments that were previously graded. The majority of these fills (Qaf2) were observed and tested by ENGEO in 2001 and are considered engineered.

Colluvial Deposits. Colluvium "Qc" has been mapped in a swale in the northeastern portion of the site. Colluvium is material that is transported predominantly by erosion and sheet wash and deposited in low lying areas. These deposits were found to consist of dark brown silty clay with lesser amounts of sand and gravel. Based on the findings of the exploratory borings and exposures of the colluvium during exploration, the maximum colluvium depth appears to be approximately 15 feet below the ground surface. Atterberg Limits testing of colluvial soils resulted in Plasticity Indices (PI) of 10 to 29. The colluvial materials are therefore considered moderately to highly expansive when subjected to fluctuations in moisture content.

Pleistocene Alluvial Fan Deposits. Upper Pleistocene-age alluvium "Qpf" has been mapped over most of the site. Alluvial deposits consist of moderately graded, unconsolidated clay, silt, sand, and gravel with varying amounts of weathered bedrock fragments. Based on the findings of the exploratory borings and exposures of the alluvium during exploration, the alluvium extends down to the maximum depths explored, approximately 21 feet. The near-surface layers of these deposits appear stiff to very stiff or dense to very dense, and with greater depth, these deposits increase in stiffness and density.

Landslide Deposits. Regional landslide mapping by Nilsen (1972) shows two landslide areas on the east side of the site. One of the mapped landslides is located on the northeast side of Evergreen Creek and is several hundred feet from the area of planned development, due to the proposed fault

setback recommendations discussed below. Therefore, this mapped landslide does not appear to be a potential hazard for the planned development.

A second mapped landslide is located on the flank of the drainage course that extends upslope of Trench ET-7. It appears that slope movement of this mapped landslide area could contribute debris to the drainage course that enters the subject site in the vicinity of Trench ET-7.

ESA (1980) identified a landslide near the center of the east property boundary as a shallow slump-type failure that predominately involves soil with some highly weathered bedrock material. ESA based the location of the landslide on the local geomorphology and no exploration was conducted in the vicinity of the slide at the time of their site investigation. Test Pit TP-8 was excavated within the landslide area mapped by ESA and no features indicative of landsliding were encountered.

Bedrock. Bedrock at the site consists of the Cretaceous age conglomerate (Kbc) and interbedded sandstone and shale (Kbs) of the Berryessa formation. Conglomerate was encountered during our exploration in the west end of Trench ET-1 and in Test Pits TP-1 and TP-3. The conglomerate was friable to weak and contained cobbles up to about 12 inches in diameter. The backhoe used for our exploration was able to excavate the conglomerate but with some degree of effort. Test Pit TP-2 encountered dark brown shale generally striking to the northwest and dipping at an inclination of 18 degrees to the east.

Groundwater Conditions

Groundwater was not encountered within the borings, test pits, or trenches during the field exploration, with the exception of groundwater seepage observed within Test Pit TP-6 at a depth of approximately 11 feet. Fluctuations in groundwater levels occur seasonally and over a period of years because of variations in precipitation, temperature, irrigation, and other factors. Future irrigation may cause an overall rise in groundwater levels.

FAULT EXPLORATION

Interpretation of Aerial Photographs

Black and white stereo-paired aerial photographs were used for the purpose of observing natural landforms on the site. Our aerial photograph interpretation was used in conjunction with field mapping, subsurface exploration, and literature research to develop the "Geologic Map" presented as Figure 2. These photographs were of high resolution and good contrast so that geomorphic features could be studied. Aerial photographs were used in this exploration to aid in interpreting the relationships between landforms and the underlying bedrock, soil, and geologic structures; to discern recent changes which may have occurred at the site; and to observe the presence, character, and activity of slope failures on or adjacent to the site. The photographs were also studied for the presence of terrain features characteristic of fault zones, such as linear discontinuities in rock or soil, offset water courses, linear scarps, topographic lows, or breaks in slope.

A review of the aerial photographs revealed a break in slope parallel to the east boundary of the property in the vicinity of the Quimby fault mapped by Dibblee (1972) and a similarly located, unnamed fault by Wentworth (1999). Also, a faint linear discontinuity is visible on the saddle near the center of the east boundary (north of Evergreen Creek). Evergreen Creek is offset right-laterally in this vicinity suggesting a translational component for the Quimby fault. Geomorphic details in the vicinity of Faults "A" and "B" such as tonal lineations are mostly disrupted by previous agricultural use. Fault "A" is located along a slight break in slope west of the Quimby fault, but the features become obscure with the change to shallow slope and orchard trees in the agricultural areas.

Previous Fault Exploration

As noted above, fault segments have been mapped crossing the site by Dibblee (1972), Wentworth (1999), ESA (1980, 1992), and the State of California. The approximate locations of the previously mapped faults are shown on Figure 2.

Exploration by ESA (1980) included excavation of seven trenches on the site. An additional two trenches were excavated by ESA (1992) to supplement the original exploration. The locations of these trenches are shown on Figure 2. Extensive geologic studies for the Evergreen Site by ESA (1980, 1992) found a fault trace that has not experienced movement for at least 10,000 to 20,000 years. Discussion regarding the findings of the ESA trenches on site is as follows:

ESA Trench TR-1. A feature interpreted as Fault A was encountered in ESA Trench TR-1 striking N25^{o}W dipping 15°E. The fault feature was mapped at a clay seam shear that displaced an ancient soil horizon (paleosol) roughly 3 feet. The inclination of the shear plane shallows to nearly horizontal at it approached the ground surface, and the shear plane appears to terminate within a paleosol at about 4 to 5 feet below the ground surface. Trench TR-1 also crossed the Quimby fault, as mapped by Dibblee (1972), but no features indicative of faulting were found by ESA in that location.

ESA Trench TR-4. A feature interpreted as Fault A was encountered in ESA Trench TR-4 with a similar orientation to TR-1. A gravelly lens within the alluvial fan material was offset an apparent distance of 0.8 foot. Paleosol development is not as well defined as in TR-1.

ESA Trench TR-6. A feature interpreted as Fault A was encountered in ESA Trench TR-6 with a similar orientation to TR-1. A shear plane defined by a slightly discolored zone of weaker, wet to saturated clay and sand was observed without offset relationships. However, apparent warping of a sandy lens was truncated by the shear feature.

ESA Trench TR-9. Trench TR-9 was excavated across the northerly projection of the fault encountered in TR-4. ESA found no features indicative of faulting in Trench TR-9.

The ESA report (1980) concluded that the last period of activity for the observed Fault A trace was during the last 10,000 to 20,000 years during the early Holocene to late Pleistocene.

As noted above, previous fault exploration was performed by Parikh Consultants (2002) in the southeastern portion of the site in conjunction with studies for a proposed water tank. The location of the exploratory trench is shown on Figure 2. A fault feature (Fault B) was encountered and Parikh Consultants recommended a setback from the fault be established for the planned water tank. The fault feature is described in their logs as a slickensided surface striking N35W. The feature dipped nearly vertical at the base of the trench (15 feet deep) and flattened to a dip of about 45 degrees toward the east.

Current Fault Exploration

To evaluate the previously mapped fault traces on the site, nine exploratory trenches were excavated. The trenching program involved approximately 1,150 lineal feet of exploratory trenching. The trenches were cleaned with picking tools and logged by an engineering geologist from ENGEO. Logs of the trenches are included in Appendix A. The conditions encountered in each of the trenches are summarized below.

Trench ET-1. Trench ET-1 was excavated starting near the east property line, trending west across a topographic saddle where a fault trace was previously mapped by Dibblee (1972) and ESA (1980, 1992). Trench ET-1 encountered a feature interpreted as the Quimby fault with an orientation of N16°W 17°E. The fault feature was mapped as a clay seam with slickensides at Station 1+25 displacing a red to red-orange gravelly clay unit (Qpf) over conglomerate (Kbc). The fault feature

flattened to nearly horizontal and terminated in an overlying red-brown silty clay unit at Station 1+38 approximately 4 to 5 feet below the existing ground surface.

Trench ET-2. Trench ET-2 was excavated starting near the east property line, trending west across a slope where a fault trace was previously mapped by Dibblee (1972) and ESA (1980, 1992). Trench ET-2 encountered a feature interpreted as the Quimby fault with an orientation of N14°W 14°E. The fault feature was mapped as a clay seam with slickensides at Station 0+08 surrounded by a zone of carbonate particles. The fault feature shallows out to horizontal and fades out below an overlying olive to yellow brown clay unit at Station 0+37 approximately 3 to 4 feet below the existing ground surface. The red-brown clay unit within the fault feature displayed no distinct offset although the overlying olive to yellowish brown clay unit was observed to have minor apparent warping. A second fault feature was mapped with an orientation of N10°W 30°E at Station 0+00 that shallows out to horizontal and dies out below the overlying olive to yellow brown clay unit at Station 0+11, 5 feet below the existing ground surface.

Trench ET-3. Trench ET-3 was excavated 50 feet north of Trench ET-1 to confirm the orientation of the fault feature encountered in Trench ET-1. Trench ET-3 encountered a feature interpreted as a fault with an orientation of N33°W 42°E. The fault feature was mapped as a clay seam with slickensides at Station 0+32, within a distinct olive silty clay unit with sand, shallowing out to nearly horizontal and terminating at Station 0+41 into an alluvial fan deposit of clayey sands with gravels. Striations were observed in the dip direction on the slickensided surface. No distinct evidence of offset was observed in the overlying silty clay unit although a clayey gravel unit was observed to have apparent warping.

Trench ET-4. Trench ET-4 was excavated in the southeast portion of the property across the projection of Fault B that was previously found in trenching by Parikh Consultants (2002). Soil deposits consistently 4 to 5 feet thick were encountered over alluvial fan deposits consisting of clayey sands and gravels. Silty clay lenses were observed within the alluvial fan deposits. The

alluvial deposits were well stratified and appeared to be Pleistocene fan deposits (Qpf) as mapped by Wentworth (1999). No features indicative of faulting were observed.

Based on these findings, Fault B appears to terminate somewhere between the trench by Parikh Consultants (2002) and Trench ET-4 or more likely, the fault may curve to the east and intersect Fault A or the Quimby fault.

Trench ET-5. Trench ET-5 was excavated across a previously graded area near the northwest corner of the site adjacent to the Syntex property. The purpose of this trench was to explore an area where studies by ESA suggested that a fault to the northwest may project onto the subject site. Fill placed during grading was observed to be 1 to 2 feet thick from Station 0+00 to 0+92. Deposits of sandy clays and clayey sands were observed with gravel lenses that appear to be Pleistocene fan deposits as mapped by Wentworth (1999). No features indicative of faulting were encountered.

Trench ET-6. Trench ET-6 was excavated near the east property line, trending west across a slope 50 feet north of Trench ET-2. Trench ET-6 encountered a feature interpreted as a fault with an orientation of N28°W 32°E. The fault feature was mapped as a clay seam with slickensides at Station 0+24, within a distinct olive silty clay unit, bounded by a carbonate zone, overlying a clayey sand with gravel unit. The fault feature flattened to nearly horizontal at Station 0+35 and terminated in an overlying reddish brown silty clay unit approximately 3 feet below the existing ground surface. The overlying unit displays an apparent warping. At the base of the excavation, the clay seam faded into a clayey sand unit with gravel that displayed a poor developed parting at Station 0+23, 9 to 10 feet below the existing ground surface.

Trench ET-7. Trench ET-7 was excavated about 30 feet north of ESA Trench TR-1 (1980) starting near the east property line and trending west across a slope. Two soil units were observed to be relatively undisturbed across the trench, with an olive brown silty clay unit overlying a reddish brown silty clay unit 5 to 6 feet below the existing ground surface. Overlying colluvium was observed to be

2 to 3 feet thick. No features indicative of faulting were encountered suggesting that fault features observed to the south of Trench ET-7 trend toward the east, off of the subject property.

Trench ET-8. Trench ET-8 was excavated near the northeastern corner of the property, trending southwest across a slope. Several soil units were observed in the excavation, but no evidence of displacement was noted within the units. Zones of carbonate nodules were observed at unit interfaces at 4, 7 and 9 feet below the existing ground surface. A clayey sand unit with gravel was observed 10 to 11 feet below the existing ground surface. The overlying colluvium was between 3 and 4 feet thick. A zone of carbonate particles was observed in the colluvium from Station 0+00 to 0+11 at depths between 2 and 4 feet below the existing ground surface. No features indicative of faulting were observed.

Trench ET-9. Trench ET-9 was excavated near the northeast corner of the project, overlapping Trench ET-8, trending southwest across a slope. Several very dark brown to dark brown silty clay units overlying alluvial fan deposits were observed in the excavation, but no evidence of displacement was noted within the units. Near the location of an existing swale feature at Station 0+93, an erosional discontinuity was observed with a reddish brown silty clay unit with sand onlapping/overlying the alluvial fan deposits. The silty clay with sand exhibited properties of vertical parting that appeared to be desiccation cracks. No features indicative of faulting were observed.

Dr. Glen Borchardt, with Soil Tectonics, was retained to evaluate the relative age of the soil units exposed in Trench ET-1. The pedochronological report prepared by Dr. Borchardt is included as Appendix C to this report. Based on analysis of soil stratigraphic features, Dr. Borchardt concluded that the oldest soil unit exposed in soil Trench ET-1 that did not exhibit fault displacement is at least 40,000 years old.

Based on the fault features encountered in Trenches ET-1, ET-2, ET-3, and ET-6, the Quimby fault appears to be a shallow, east dipping fault. Striations within the observed shears suggest that the sense of movement is a slightly oblique reverse fault.

DISCUSSION AND CONCLUSIONS

Seismic Hazards

Potential seismic hazards resulting from a nearby moderate to major earthquake can generally be classified as primary and secondary. The primary effect is ground rupture, also called surface faulting. The common secondary seismic hazards include ground shaking, ground lurching, soil liquefaction, lateral spreading and seismically-induced landsliding.

Based on topographic and lithologic data, risk from earthquake-induced lurch cracking, regional subsidence/uplift, tsunamis, or seiches is considered low at the site.

Ground Rupture. The property is not located within a State of California Earthquake Fault Hazard Zone. The site is within a Special Studies Zone designated by the City of San Jose (Figure 4) and indications of potentially active faulting were found during our exploration of the site. Based on the previous study by ESA (1980) and the findings of our exploratory trenching, the Quimby fault and the faults that have been designated Faults A and B appear to be potentially active (defined as having experienced movement within the last 2 million years) and may experience sympathetic movement in response to movement on other active faults in the region. The likelihood of sympathetic movement along these faults is considered likely since the faults appear to intersect known active faults.

To reduce the potential for adverse impacts from ground rupture associated with sympathetic faulting, we recommend that all structures intended for human occupancy be located in conformance with the setbacks shown on Figure 2 and as described below:

1. A minimum structural setback of 40 feet from the surveyed location of the Quimby fault, and where the fault appears to extend off the site to the east, a minimum of 40 feet from the eastern property line.

2. A minimum structural setback of 50 feet (previously recommended by ESA, and not further studied, from the surveyed location of Fault A.

3. A minimum structural setback of 50 feet west of the surveyed location of Fault B in the Parikh Consultants exploratory trench and 25 feet west of the east end of ENGEO Trench ET-4.

4. No structures should be planned on the east side of the mapped location of Fault B unless additional fault exploration is performed to further characterize faulting in that portion of the site.

The recommended setback distance of 40 feet from the Quimby fault is based on the surveyed locations of the fault at numerous trench locations that were excavated in conjunction with the current study, whereas the 50-foot setback from Fault A is based on findings of prior consultants. The location of the faults should be confirmed during grading operations by geologic mapping in the areas of proposed cuts. Since the mapped faults dip to the east, the fault locations could shift to the east if significant cuts are made. If significant fills are made, the faults may project to the graded ground level to the west of the currently mapped fault locations. ENGEO should be given the opportunity to review the grading plans and adjust the fault setback zones if needed.

In the event that one or more of the faults on the site experiences displacement, improvements such as roadways and underground utilities that cross the faults could be damaged and would need repair. We recommend that pressurized pipelines that cross the faults be designed with shut-off valves adjacent to the fault zone.

Ground Shaking. An earthquake of moderate to high magnitude generated within the San Francisco Bay Region, similar to those which have occurred in the past, could cause considerable ground shaking at the site. To mitigate the shaking effects, all structures should be designed

using sound engineering judgment and the latest Uniform Building Code (UBC) or California Building Codes (CBC) requirements as a minimum.

Seismic design provisions of current building codes generally prescribe minimum lateral forces, applied statically to the structure, combined with the gravity forces of dead and live loads. The code-prescribed lateral forces are generally substantially smaller than the expected peak forces that would be associated with a major earthquake. Therefore, structures should be able to: (1) resist minor earthquakes without damage, (2) resist moderate earthquakes without structural damage but with some nonstructural damage, and (3) resist major earthquakes without collapse but with some structural as well as nonstructural damage. Conformance to the current building code recommendations does not constitute any kind of guarantee that significant structural damage would not occur in the event of a maximum magnitude earthquake; however, it is reasonable to expect that a well-designed and well-constructed structure will not collapse or cause loss of life in a major earthquake (SEAOC, 1996).

Lurching. Ground lurching is a result of the rolling motion imparted to the ground surface during energy released by an earthquake. Such rolling motion can cause ground cracks to form in weaker soil materials. Site preparation and grading in accordance with the recommendations provided in this report are intended to reduce the potential for lurch cracking to a low level.

Liquefaction. Liquefaction is a phenomenon in which saturated cohesionless soils are subject to a temporary but essentially total loss of shear strength because of pore pressure build up under the cyclic shear stresses associated with earthquakes. Mapping by the Association of Bay Area Governments (April 2003) indicates that the susceptibility to liquefaction at the site is low to very low. Based on the medium dense to very dense nature of the granular materials encountered at the site, and the fact that groundwater was not encountered during the course of our exploration, it is our opinion that the potential for liquefaction at the site is low.

Densification Due to Earthquake Shaking. Densification of sandy soils above and below groundwater levels can result in settlement/densification during an earthquake. Due to the relatively high density of the granular deposits encountered, it is our opinion that the potential for earthquake induced densification is low.

Lateral Spreading. Lateral spreading is a failure within weaker soil materials, typically due to liquefaction, which causes the soil mass to move toward an open channel, or down a gentle slope. As described above, the site soils have a low susceptibility to liquefaction; therefore, the potential for liquefaction-induced lateral spreading is considered low.

As is common practice, a setback from the creek is advisable to allow for meander or erosion of the banks. We recommend a minimum setback of 25 feet from top of the Evergreen Creek Banks. If that setback distance is maintained during the planning and construction of the proposed structures and improvements, it is our opinion that minor bank movement will have negligible impact on planned site improvements.

Slope Stability

Steep slopes on the flanks of the drainage course that extends upslope of Trench ET-7 appear to be subject to erosion and landslide movement. It appears that erosion and slope movement in this drainage course could contribute debris to the drainage course that enters the subject site in the vicinity of Trench ET-7. To reduce the potential for adverse impacts to the planned improvements from debris or debris laden runoff, the project design should include measures to contain or divert runoff from this swale. Measures such as a debris basin or debris deflection berm may be appropriate. We should be given the opportunity to consult with the project civil engineer regarding the design of these measures.

Cut slopes along the eastern edge of the property may expose colluvial soil deposits and/or sheared bedrock materials associated with the Quimby fault. All cut slopes should be reviewed and approved by an Engineering Geologist during grading operations to confirm that adverse geologic conditions or long-term performance issues are not present. If features are observed that could adversely affect slope stability, supplemental recommendations will be provided to improve long term slope performance.

To evaluate the stability of the existing and proposed slopes on the eastern edge of the property, drained direct shear testing was performed on an in-situ alluvial sample and a remolded colluvium sample of site materials to determine strength parameters for use in future effective stress slope stability analyses. It should be noted that the two samples were collected from lower strength zones of alluvium and colluvium encountered during our exploration.

The tests were performed at a very slow rate of strain; 0.006 mm/min, which assures that pore pressures are not developed and the drained condition is modeled correctly. These tests are performed to determine the effective peak and fully-softened parameters (cohesion-c' and friction angle-Ø'). The effective peak strength parameters (c_p' and $Ø_p'$) correspond to the highest values attained during the test. These parameters are appropriate to use for intact soil and rock. The effective fully-softened strength parameters (c_s' and $Ø_s'$) correspond to a condition in which the soil has been strained beyond the peak strength and weakened as a result of the "softening." The effective softened strength parameters are appropriate to use where a changed slope condition may result in strain softening, such as in a free-standing cut slope. Effective residual strength parameters (c_r' and $Ø_r'$) correspond to the values obtained when samples are sheared with large displacements and are appropriate for modeling existing slide planes and slide debris.

The colluvium sample was remolded to 92 percent compaction at a moisture content of 3 percentage points over optimum to model engineered fill. The effective peak parameters are appropriate to use in an engineered fill situation. Laboratory test results of direct shear strength

are presented in Appendix B. A summary of the obtained shear strength parameters are presented below.

Sample Name/Material (depth below grade)	Soil Strength Parameters					
	Peak		Softened		Residual	
	Friction Angle	Cohesion (psf)	Friction Angle	Cohesion (psf)	Friction Angle	Cohesion (psf)
L2T2 Alluvium (10 feet)	26	580	25	80	14	0
Bulk A/Colluvium (5 feet) - Remolded	31	40	26	0	-	-

Slope stability analysis for the proposed cut, cut-fill, and fill slopes should be performed as part of the 40-scale grading plan review and preparation of the corrective grading plan set. The slopes will be analyzed for both long-term static and short-term seismic conditions. Depending upon the results of the analyses, minor slope gradient modifications and incorporation of slope reinforcing materials, such as geogrid, may be warranted to create a stable condition.

Soil Creep

Soils on steeper natural slopes are subject to soil creep. Soil creep is the slow downslope movement of soil that occurs with the annual cycle of wetting and drying under the influence of gravity. The potential for adverse impacts from soil creep can be minimized by benching through surficial soils during fill placement as recommended in this report. If foundations for homes and fences are situated on natural sloping terrain, they should be designed to reduce the potential for adverse impacts from soil creep.

Debris Benches

For graded areas that abut open space slopes, debris benches are recommended at the toes of the uphill slopes to act as a buffer between the open hillside areas and the development. This will allow minor erosion, sloughing or soil creep to take place above the benches, which should not adversely impact the development areas located at the toes of the slopes. A concrete ditch should be constructed on the outboard side of the debris bench to collect and transport storm water from the natural areas above. Figure 6 provides a conceptual cross-section for the debris bench. Sizing of the bench width and associated subdrained keyway should be based upon the upslope terrain and geologic conditions.

The debris benches will require periodic maintenance, consisting of the removal and disposal of accumulated slope debris. Proper access should be provided for the equipment that may be required for removal of the loose soil or debris from the benches. A maintenance program should be developed to observe and maintain the graded condition of the debris bench.

Existing Fill

The southwest portion of the site contains undocumented fill within a drainage swale. Figure 2 depicts the approximate location of the fill material. As identified by Boring B-9, the fill is composed of dark brown sandy silty clay up to 10 feet thick. Below this layer is a critically expansive dark gray clay layer extending down to a depth of 14 feet. Test Pit TP-4 encountered some clay pipe shards at a depth of 12 feet, within the dark gray clay layer, indicating that it is also likely undocumented fill. The undocumented fill should be removed to achieve a firm base, moisture conditioned, and recompacted as engineered fill. Prior to reuse as fill, any debris should be removed. Existing fill that does not comply with project specifications should be removed. This applies to the dark gray critically expansive clay discussed above. Engineered fill has been previously placed along the shoulders of the existing road cuts. This fill was tested

and observed by ENGEO in 2001 and is shown on Figure 2 as Qaf2. Further recommendations regarding removal and replacement of the fill material are provided in subsequent sections.

Expansive Soils

Expansive soils are susceptible to shrink and swell resulting from variations in moisture content. Expansive soils and bedrock may cause heaving and cracking of slabs-on-grade, pavements, and foundations. Building damage due to volume changes associated with expansive soils may be reduced by the following measures: (1) selectively placing the more expansive materials in the lower portions of the deeper fill areas (generally at depths below 10 feet from finished grades), or placing the expansive materials outside the limits of the proposed residential structures and site improvements (such as in landscape areas); (2) performing proper moisture conditioning and compaction of fill materials within specified ranges to reduce their swell potential; and (3) supporting houses upon structurally reinforced mats and/or post-tensioned mats designed to resist the deflections associated with expansion/compression-related movements.

Bedrock Rippability and Suitability

Based on field observations during drilling and test pit excavation at the subject site, it is our opinion that the site bedrock should be rippable with conventional heavy construction equipment (such as a Caterpillar D-9 or larger). Localized cemented lenses or beds may be encountered where cuts are planned in the conglomerate bedrock that may require considerable ripping effort and generate oversize material (greater than six inches in diameter). Backhoes may experience difficulty excavating in some of the less weathered conglomerate; however, we anticipate that large excavators should be capable of trenching the materials on site without significant difficulty.

Conclusions

Based on the exploration and laboratory test results, and review of prior geotechnical studies, it is our opinion that the site remains feasible for the proposed development from a geotechnical standpoint. The recommendations included in this report, along with other sound engineering practices, should be incorporated in the design and construction of the project.

RECOMMENDATIONS

Grading

Grading and site development plans should be coordinated with the Engineering Geologist and the Geotechnical Engineer to modify the plans for mitigation of soil and geologic hazards during the planning process. The final 40-scale grading plans for the project site should be reviewed by the Geotechnical Engineer prior to construction in order to develop a corrective grading plan.

ENGEO should be notified at least 48 hours prior to grading in order to coordinate our schedule with the grading contractor. Grading operations should meet the requirements of the Guide Contract Specifications included in Appendix F and should be observed and tested by ENGEO's field representatives.

After the grading operations commence, geologic observations of cut and subexcavation areas should be made by the Engineering Geologist. This is beneficial if revised geologic recommendations need to be incorporated into updated grading plans as grading proceeds.

Ponding of storm water, except within sediment detention basins, particularly during work stoppage for rainy weather, should not be permitted. Before the grading is halted by rain, positive slopes should be provided to carry the surface runoff to storm drainage structures in a controlled manner to prevent erosion damage.

Selection of Materials. With the exception of debris (wood, brick, asphalt, concrete, metal, etc.), organically contaminated materials (soil which contains more than 3 percent organics), trees or their root balls, and the critically expansive colluvium material, the site soils are suitable for use as engineered fill.

Subject to approval by the Landscape Architect, organically contaminated soil may be stockpiled in approved areas, and located outside of the grading limits for future placement within landscape areas. All other materials and debris, including any trees with their root balls, should be removed from the project site. As described below, the critically expansive colluvium material may be reused in non-structural fill areas.

Oversized soil or rock materials (those exceeding two-thirds of the lift thickness or 6 inches in dimension, whichever is less) should be removed from the fill and broken down to meet this requirement or otherwise off-hauled.

The Geotechnical Engineer should be informed when import materials are planned for the site. Import materials should be submitted and approved by the Geotechnical Engineer prior to delivery at the site and should conform to the requirements provided in the Guide Contract Specifications (Appendix F).

Demolition and Stripping. Site development will commence with stripping seasonal grasses and the removal of existing degradable materials (trees, root balls, and wood debris) as well other potential debris. Tree roots should be removed to a depth of at least 3 feet below original grade. These operations should be conducted under the observation of a representative from ENGEO.

Areas to receive fill and areas that will serve as a source of borrow should be stripped of existing vegetation. Topsoil is estimated to be from 3 to 6 inches in thickness depending on location. Actual depths of stripping and removal of tree roots will be determined by the Geotechnical Engineer's qualified representative in the field during grading.

Existing Fills. Existing undocumented fill (Qafl), approximately located on Figure 2, should be removed to expose native materials. According to exploratory borings and test pits, the depth of undocumented fills could be up to 14 feet deep within the swale feature.

The upper zone of silty clay (up to 10 feet thick) can be reused as engineered fill if deemed suitable and placed in accordance with the Fill Placement section of this report and under the observation and testing of a representative from ENGEO. The critically expansive dark gray clay within the lower zone of fill should be stockpiled and off-hauled or used in non-engineered fill applications.

The existing engineered fills (Qaf2) may be left in place, depending on the nature of the proposed grading. This decision will be made by an ENGEO representative once the grading plans have been reviewed and the fill material has been exposed and evaluated.

Loose or Compressible Surface Soils. Once removal of unsuitable debris is completed, the site should be observed for its suitability to receive engineered fill materials or as foundation soils by wheel rolling the site with heavy construction equipment. If unsuitable soil materials are observed, these soils should be subexcavated as necessary to encounter firm native materials. If no yielding is observed, then the exposed surface should be scarified, moisture conditioned, and compacted. The requirements for backfill materials and placement procedures are the same as those for engineered fill, as described in subsequent sections. The actual depth for reworking should be determined by a qualified ENGEO field representative at the time of grading.

It is important that the exploratory fault trenches and test pits excavated during ENGEO's and previous fault studies be located and the loosely backfilled trench spoils be removed and replaced as engineered fill under the testing and observation of ENGEO. This will not be necessary where trenches are located outside of grading limits.

No loose or uncontrolled backfilling of depressions resulting from demolition, stripping, removal of undocumented fill material, swale cleanout, or removal of tree root balls should be permitted.

Toe Keyways. After stripping, site grading should begin with construction of keyways placed at the toe of all fill slopes. The keyway should have a minimum width of 18 feet and should extend at least 5 feet into firm material on the downhill side. Typical keyway subdrains and minimum keyway sizes are shown in Figures 7 and 8, respectively. Keyway sizes and locations will be approximately shown on the grading plans during our 40-scale review.

All fills should be adequately keyed/benched into firm material during fill slope construction. Unless otherwise recommended by the Geotechnical Engineer at the time of grading, such benches should be placed at vertical height intervals of not less than 5 feet. The actual depth of the keyways and subsequent benching will be determined by the Geotechnical Engineer in the field during grading.

Subsurface Drainage Facilities. Subsurface drainage systems should be installed in all keyways and in swales or natural drainage ways which are to be filled. The approximate locations of the recommended subdrains should be shown on the final 40-scale grading plans.

Drainage courses which are to be filled should be provided with adequate subsurface drainage prior to placement of any fill. Swales should be cleaned of soft or compressible material to a firm soil or rock base. A subdrain should then be installed through the center of the subexcavation (Figure 7). Desiccated, cracked surface clays and slumping soils located along the swale areas should be removed, and the slopes should be benched prior to the placement of fill. Actual limits of subexcavation should be determined in the field at the time of grading by the Geotechnical Engineer.

Additional subdrains should be added where seepage or wet conditions are encountered during excavation. Subdrain systems should consist of a minimum 6-inch-diameter perforated pipe encased in an 18-inch minimum thickness of Caltrans Class 2 Permeable Material or coarse rock wrapped in geotextile filter fabric. The subdrain pipe and drainage blanket should meet the

requirements contained in Section 2.05, Part I of the Guide Contract Specifications with their general placement guidelines presented on Figure 7. As an alternative, prefabricated geocomposite subdrain systems are also available in lieu of round subdrain pipe encapsulated in a granular medium. Approved alternative subdrain system products may be provided at your request.

Discharge from the subdrains will generally be low but in some instances may be continuous. Subdrains should outlet into the storm drain system or other approved outlets; their locations should be surveyed by the Civil Engineer and documented on an as-built subdrain plan for future maintenance.

Not all sources of seepage were uncovered during our field work because of the intermittent nature of some of these conditions and their dependence on long-term climatic conditions. Furthermore, new sources of seepage may be created by a combination of changed topography, irrigation patterns and potential utility leakage. Since uncontrolled water flows are one of the major causes of detrimental soil movements, it is of utmost importance that the Geotechnical Engineer be advised of any seepage conditions so that remedial action may be initiated, if necessary. All subdrain connections and tie-ins to storm drain inlets should be observed and approved by the Geotechnical Engineer.

Graded Slopes. We recommend the following slope gradient guidelines for the proposed cut and fill slopes:

Maximum Slope Gradient (horizontal:vertical)	Cut Slope Height (feet)	Fill Slope Height (feet)
2:1	10 or less	20 or less
3:1	Greater than 10	Greater than 20

Where slopes higher or steeper than those recommended above are desired, supplemental slope stabilization techniques such as slope rebuilding or the incorporation of geogrid reinforcing materials may be required. Geogrid designs can be provided by ENGEO once the final grading plan has been prepared.

In addition, as determined by field observations at the time of grading and during our 40-scale plan review, some cut slopes that satisfy the above guidelines may be unstable for long-term performance and may be recommended for reconstruction as engineered fill slopes. Cut-fill transition slopes will also be recommended for reconstruction as engineered fill slopes.

All cut slopes and subexcavations should be reviewed and approved by an Engineering Geologist during grading operations to confirm that adverse geologic conditions or long-term performance issues are not present.

Cut Lots and Cut-Fill Transition Lots. Some residential lots in this project will likely be entirely in cut or traversed by a cut-fill transition. We recommend that the upper 24 inches of the flat building pads situated in cut or cut-fill be made uniform. This also applies to lots with a fill thickness of less than 24 inches. This can be accomplished by removing the upper 12 inches and reprocessing the next 12 inches of soil in place. If a highly variable material is encountered, these recommendations should be revisited.

Differential Fill Thickness. Differential building movements, although not seriously damaging, may become apparent for large differential fill thicknesses. Therefore, we recommend that the differential fill thickness under individual buildings be less than 10 feet. Local subexcavation of material and replacement by engineered fill will be necessary to achieve this requirement.

Fill Placement. Once a suitable firm base is achieved, the exposed non-yielding surface should be scarified to a depth of 10 inches, moisture conditioned, and recompacted to provide adequate

bonding with the initial lift of fill. All fills should be placed in thin lifts, with the lift thickness not to exceed 10 inches or the depth of penetration of the compaction equipment used, whichever is less.

The following compaction control requirements should be applied to general fill areas:

Test Procedures:	ASTM D-1557.
Required Moisture Content:	Not less than 3 percentage points above optimum moisture content.
Minimum Relative Compaction:	Not less than 90 percent.

The following compaction control requirements should be applied in keyways and in fills that are greater than 40 feet below finished grade.

Test Procedures:	ASTM D-1557.
Required Moisture Content:	Not less than 2 percentage points above optimum moisture content.
Minimum Relative Compaction:	Not less than 95 percent.

Since excessive compaction of expansive materials may produce an undesirable environment for expansion in the zone of significant seasonal moisture variation, special requirements for placement of expansive soils are necessary within the upper foundation zone of building areas. For this condition, it is important to compact the soils such that their swell potential is reduced to acceptable values. The following specifications for moisture and compaction of expansive soils (PI greater than 15) within the upper 2 feet from pad grade are proposed.

Test Procedure:	ASTM D-1557.
Required Moisture Content:	At least 4 percentage points above optimum moisture content.
Relative Compaction:	Not less than 88 percent and not more than 93 percent.

As noted above, it is important that all site preparation, including demolition and stripping, be performed under the observation of the Geotechnical Engineer, or their qualified field representative, and should be carried out according to the requirements contained herein and within the Guide Contract Specifications in Appendix F.

The final grading plans should be submitted to the Geotechnical Engineer for review.

Building Code Seismic Information

In accordance with the Uniform Building Code (1997), the site is located within Seismic Zone 4. As shown in Figure 16-2 and Table 16-1, this indicates a seismic zone factor (Z) of 0.40. Based on site conditions, the soil profile can be classified as S_D, stiff soil, as defined in Table 16-J. In reference to Tables 16-Q and 16-R, seismic coefficients, C_a of $0.44N_a$ and C_V of $0.64N_V$ can be used, while in reference to Tables 16-S and 16-T, near source factors, N_a of 1.3 and N_V of 1.6 can be assumed, based on seismic input from the Hayward fault (southeast extension).

Foundations

The major consideration in foundation design at the site is the swell characteristic of the potential foundation soils. The effect of expansive soils can be reduced by the choice of a proper foundation system. In order to reduce the effects of the potentially expansive soils, the

foundations should be sufficiently stiff to move as rigid units with minimum differential movements.

Provided that all building pads are prepared in accordance with the recommendations provided herein, it is our opinion that a post-tensioned or conventionally-reinforced floating mat foundation system would be most appropriate due to the moderate to high expansion potential in the upper site soils.

Based upon the existing soil conditions, we recommend using the following soil criteria for design of post-tensioned mat foundations:

Center Lift Condition:

Edge Moisture Variation Distance, $e_m = 5$ feet
Differential Soil Movement, $y_m = 2.3$ inches

Edge Lift Condition:

Edge Moisture Variation Distance, $e_m = 4$ feet
Differential Soil Movement, $y_m = 1.1$ inches

The above parameters are applicable for the design methodology provided in the 1996 (Second Edition) Post-Tensioning Institute, "Design and Construction of Post-Tensioned Slabs-On-Ground" manual. If other procedures are utilized for design, the parameters provided above should be reviewed for suitability.

The post-tensioned mats should be designed to impose a maximum allowable bearing pressure of 1,000 psf for dead-plus-live loads. This value may be increased by one-third when considering total loads including wind or seismic. It is important that the Structural Engineer design the superstructure to be compatible with the deflections that will be imposed by the foundation.

A minimum mat thickness of 10 inches is recommended. The perimeter should be thickened to at least 12 inches, and the minimum backfill height against the foundation at the perimeter should be 6 inches.

A wafflemat foundation or conventionally-reinforced mat foundation system would also be applicable for the proposed construction. ENGEO can provide additional design information and recommendations if such systems are desired.

The final foundation plans should be reviewed by the Geotechnical Engineer when they become available to verify conformance with these design criteria, and if desired, the actual foundation materials could be sampled once finished pads are achieved and tested to determine if the parameters presented herein remain applicable

Slab Subgrade Treatment

Prior to mat foundation construction, pad moisture conditioning should be performed to regain a moisture content of at least 4 percentage points above optimum within the surficial materials prior to placing the concrete. The subgrade should not be allowed to dry prior to concrete placement. ENGEO should review and approve the moisture conditioning operation.

Corrosive Soils

An evaluation of possible corrosion impacts to site improvements has not been conducted on the site soils as a part of this study. The primary purpose for sulfate (corrosion) testing is to determine if sulfate-resistant concrete is needed for foundation construction, based on the criteria presented in Table 19-A-4 of the 1997 UBC. It is recommended that chemical tests be conducted on the subgrade soils after grading of the pads is completed, but prior to building and utility construction.

In lieu of performing additional chemical testing to assess the corrosion potential, concrete foundations could be designed considering a "severe" corrosion potential as defined in the 1997 UBC. This includes using Type V cement or equivalent with a maximum water/cement ratio of 0.45 and a minimum concrete compressive strength of 4,500 pounds per square inch (psi).

Secondary Slab-on-Grade Construction

The following guidelines apply to secondary slabs such as exterior patio slabs, walkways, driveways, steps, and porch slabs (if not incorporated into the mat foundation). These secondary slabs-on-grade should be constructed structurally independent of the foundation system. Steel should not be used to tie porch slabs, exterior patio slabs, driveways, walkways, or steps to adjacent foundations. This allows slab movement to occur with minimum distress to the foundations. Where slab-on-grade construction is anticipated, care must be exercised in attaining a near-saturation condition of the subgrade soil immediately prior to concrete placement.

Conventional slabs-on-grade should have a minimum thickness of 4 inches. A 4-inch-thick layer of clean crushed rock or gravel should be placed under slabs-on-grade to act as a capillary break in accordance with commonly-accepted practice.

Slabs-on-grade should be designed specifically for their intended use and loading requirements, and provided with frequent joints in accordance with ACI 302.1R-89 specifications. Some concrete cracking should be expected in the future due to concrete shrinkage and soil shrinking/swelling. Therefore, slab reinforcement is recommended to minimize the amount of cracking. Although the Structural Engineer should design the actual slab reinforcement, for the given soil conditions, we recommend that slabs-on-grade be reinforced with No. 3 bars spaced 16 inches on-center each way as a minimum. It has been our experience that welded wire mesh has not performed well for control of cracking.

Some additional recommendations that can be considered to reduce the cracking and distress that is common in construction of slabs on expansive soils follow.

- Use frequent control joints. This will reduce unsightly cracks by causing some cracking to form at the control joints.
- Provide additional diagonal reinforcing bars at slab corners.
- Thicken free edges and extend 6 inches into subgrade soils. This will reduce infiltration of water into the subgrade soils.
- Increase the slab thickness and/or add reinforcement.
- Provide subdrains at the edges of slabs where they abut planted areas.

Retaining Walls

Unrestrained drained retaining walls less than 10 feet in vertical height and constructed with a level foreground may be designed for active lateral fluid pressures determined as follows:

Backfill Slope Condition	Active Pressure (pcf)
Level	50
3:1	60
2:1	70

Passive pressures acting on foundations and keyways may be assumed as 250 pcf provided that the area in front of the retaining wall is level for a distance of at least 10 feet or three times the depth of foundation and keyway, whichever is greater. Whenever possible, walls should be located at the toe of slopes, rather than at the top of slopes to create level terrain in front of the wall. The Geotechnical Engineer should be consulted on design values where surcharge loads, such as from automobiles, are expected or where a downhill slope exists below a proposed wall.

For retaining walls supported on a footing foundation, it is recommended that the footings be designed using an allowable bearing pressure of 2,500 psf in native firm materials or fill. The friction factor for sliding resistance may be assumed as 0.35. Footings should be embedded at least 24 inches below lowest adjacent grade. Appropriate safety factors against overturning and sliding should be incorporated into the design calculations.

All retaining walls should be provided with drainage facilities to prevent the buildup of hydrostatic pressures behind the walls. Wall drainage should consist of a 4-inch-diameter perforated pipe encapsulated in either Class 2 permeable material or free-draining gravel surrounded by a minimum 6-ounce unit weight, synthetic filter fabric (Part I of Guide Contract Specification, Section 2.05B). As an alternative, prefabricated synthetic wall drain panels can be used if pre-approved by the Geotechnical Engineer. The thickness of the granular drain blanket should be at least 12 inches and should extend to about one foot below finished grade. The upper one foot of wall backfill should consist of compacted on-site soils. Collector pipes should be directed to an outlet approved by the Civil Engineer. All materials used for wall drainage should meet the minimum requirements provided in Part I of the Guide Contract Specifications.

All backfill should be placed in accordance with the recommendations provided above for engineered fill. Light equipment should be used during backfill compaction to minimize possible overstressing of the walls. The foundation details and structural calculations for the walls should be submitted for review.

Preliminary Pavement Design

For the given fine-grained surface soils, we have assumed a Resistance value (R-value) of 10 for the subgrade in calculating the pavement section. For the proposed access streets, Traffic Indices of 5 through 8 were assumed.

The following preliminary pavement sections have been determined for Traffic Indices of 5 through 8 and the assumed R-value of 10 according to methods contained in Topic 608.4 of Highway Design Manual by Caltrans and requirements of City of San Jose.

Traffic Index	AC (inches)	AB (inches)
5	4.2*	6.5
6	4.2*	10.0
7	4.2*	14.0
8	5.0	16.0

AC – Asphalt Concrete (*City minimum thickness)
AB – Caltrans Class 2 Aggregate Base (R – 78 minimum)

The above pavement section is provided for estimating only. The actual subgrade material should be tested for R-value. The Traffic Index should be confirmed by the Civil Engineer and the City of San Jose.

Pavement materials and construction should conform to the specifications and requirements of the Standard Specifications by the Division of Highways, Department of Public Works, State of California, city requirements and the following minimum requirements.

- All granular pavement subgrades should be scarified to a depth of 12 inches below finished subgrade elevation, moisture conditioned to at least 2 percentage points above optimum, and compacted to at least 95 percent relative compaction and in accordance with city requirements (ASTM Methods). Subgrades consisting of predominantly clayey soil, defined in this case as having a PI greater than 15, should be scarified to a depth of 12 inches below finished subgrade elevation, moisture conditioned to at least 3 percentage points above optimum, and compacted to at least 92 percent relative compaction.

- Subgrade soils should be in a stable, non-pumping condition at the time aggregate baserock materials are placed and compacted.

- Adequate provisions must be made such that the subgrade soils and aggregate baserock materials are not allowed to become saturated.

- Aggregate baserock materials should meet current Caltrans specifications for Class 2 aggregate baserock and should be compacted to at least 95 percent of maximum dry density at a moisture content of at least optimum (ASTM Methods).

- Asphalt paving materials should meet current Caltrans specifications for asphalt concrete and should be compacted to at least 95 percent of maximum wet density (Caltrans Methods) unless otherwise noted by the City.

- All concrete curbs separating pavement and irrigated landscaped areas should extend into the subgrade and below the bottom of adjacent aggregate base rock materials. Median and edge drains could be considered to help prevent infiltration of water under pavement areas.

Drainage Requirements

The lots must be positively graded at all times to provide for rapid removal of surface water runoff away from the foundation systems, and to prevent ponding of water under foundations or seepage toward the foundation systems at any time during or after construction. Ponded water will cause undesirable soil swell and loss of strength. As a minimum requirement, finished grades should have slopes of at least 3 percent within 5 feet, as applicable, from the exterior walls and at right angles to allow surface water to drain positively away from the structures. For paved areas, the slope gradient can be reduced to 2 percent.

All lots should be drained individually. All roof storm water should be collected and directed to downspouts. Since the site soils are not well-suited for natural percolation due the high fines content, storm water from roof downspouts should not be allowed to discharge directly onto the ground surface in close proximity to the foundation system. Rather, storm water from roof downspouts should be directed to a solid pipe that discharges into the street or to an outlet approved by the Civil Engineer.

If, at any time, adequate drainage away from the foundation cannot be achieved, then additional measures to hinder saturation of foundation soils must be provided. This may be accomplished by installing a perimeter subdrain system.

Requirements for Landscaping Irrigation

Vegetation should not be planted immediately adjacent to structures. If planting adjacent to a structure is desired, we recommend using plants that require very little moisture with drip irrigation systems. Irrigation of landscaped areas should be strictly limited to that necessary to sustain vegetation as excessive irrigation and ponding could result in saturating, weakening, and possible swelling of foundation soils. The Landscape Architect and prospective owners should be informed of the surface drainage requirements included in this report.

Sprinkler systems should not be installed where they may cause ponding or saturation of foundation soils within 3 feet of the walls or under structures.

Erosion Control

Based on our observations at the site and the results of laboratory testing conducted for site soils, the soils at the site may be erodible. We estimate that erosion control measures in the form of seeding, planting and possibly erosion control fabric will be necessary on all exposed cut and fill slopes in order to mitigate the hazard of erosion during winter rains. The tops of fill and cut slopes should be graded in such a way as to prevent water from flowing freely down the slopes.

Erosion of graded slopes could be significant in areas where slopes are not properly vegetated or erosion control measures are not properly installed. In addition to vegetated cover, viable erosion mitigation measures may include concrete or asphalt-lined drainage facilities. The

purpose of the drainage facilities is to intercept and divert the surface water runoff from the slopes.

In the design of slopes, consideration should be given to surface drainage and the potential for slope degradation by erosion. Common practice has been to provide drainage benches at regular intervals on graded slopes that are steeper than 3:1 (horizontal:vertical) and higher than 30 feet for control of surface drainage. Typical requirements are included in Section 7012 of the Uniform Building Code (UBC). It is our opinion that with proper erosion protection, drainage ditches are not necessary on 3:1 (horizontal to vertical) or flatter slopes.

Because the existing bedrock and alluvium is relatively nutrient-poor, it may be difficult for vegetation to become properly established, resulting in a potential for slope erosion. Revegetation of graded slopes can be aided by retaining the organic-rich strippings and spreading these materials in a thin layer (less than about 6 inches) trackwalked on the graded slopes having inclinations of 3:1 (horizontal:vertical) or flatter prior to the winter rains and following rough grading. When utilizing this method, it is sometimes possible to avoid hydroseeding. All landscaped slopes should be maintained in a vegetated state after project completion. The use of drought-tolerant vegetation requiring drip irrigation not more frequently than once a month during summer is recommended. No pressurized irrigation lines should be placed on or above graded slopes.

Utilities

It is recommended that utility trench backfilling be done under the observation of a Geotechnical Engineer. Ideally, pipe zone backfill (i.e. material beneath and immediately surrounding the pipe) should consist of native material less than ¾ inch in maximum dimension compacted in accordance with recommendations provided above for engineered fill. Trench

zone backfill (i.e. material placed between the pipe zone backfill and the ground surface) should also consist of native soil compacted in accordance with recommendations for engineered fill.

If required by local agencies, where import material is used for pipe zone backfill, we recommend it consist of quarry fines, fine- to medium-grained sand, or a well-graded mixture of sand and gravel and that this material not be used within 2 feet of finish subgrades. This material should be compacted to at least 90 percent relative compaction at a moisture content of not less than optimum.

In general, uniformly graded gravel should not be used for pipe or trench zone backfill due to the potential for migration of: (1) soil into the relatively large void spaces present in this type of material, and (2) water along trenches backfilled with this type of material. If uniformly graded gravel is used, we recommend that it be encapsulated in 6-ounce filter fabric. Providing outlet locations into manholes or catch basins for water collected in granular import pipe zone trench backfill should also be considered.

All utility trenches entering buildings and paved areas should be provided with an impervious seal where the trenches pass under the building perimeter or curb lines. The impervious plug should extend at least 3 feet to either side of the crossing and should be placed below, around, and above the utility pipe such that it is entirely in contact with the trench walls and pipe. This is to prevent surface water percolation into the import sand or gravel pipe zone backfill under foundations and pavements where such water would remain trapped in a perched condition, allowing clays to develop their full expansion potential.

Care should be exercised where utility trenches are located beside foundation areas. Utility trenches constructed parallel to foundations should be located entirely above a plane extending down from the lower edge of the footing at an angle of 45 degrees. Utility companies and Landscape Architects should be made aware of this information.

Utility trenches in areas to be paved should be constructed in accordance with City of San Jose requirements or approved alternatives. Compaction of backfill by jetting should not be allowed at this site. If there appears to be a conflict between City or other agency requirements and the recommendations contained in this report, this should be brought to the Owner's attention for resolution prior to submitting bids.

Excavation Safety

All excavations including utility trenches should be properly excavated, and shored as applicable, to create a stable and safe condition. It is the responsibility of the Contractor to provide such stable, safe trench and construction slope conditions and to follow OSHA safety requirements. Since excavation procedures may be very dangerous, it is also the responsibility of the Contractor to provide a trained "competent person" as defined by OSHA to supervise all excavation operations, ensure that all personnel are working in safe conditions, and have thorough knowledge of OSHA excavation safety requirements.

LIMITATIONS AND UNIFORMITY OF CONDITIONS

This report is issued with the understanding that it is the responsibility of the owner to transmit the information and recommendations of this report to developers, contractors, buyers, architects, engineers, and designers for the project so that the necessary steps can be taken by the contractors and subcontractors to carry out such recommendations in the field. The conclusions and recommendations contained in this report are solely professional opinions.

The professional staff of ENGEO Incorporated strives to perform its services in a proper and professional manner with reasonable care and competence but is not infallible. There are risks of earth movement and property damages inherent in land development. We are unable to eliminate all risks or provide insurance; therefore, we are unable to guarantee or warrant the results of our work.

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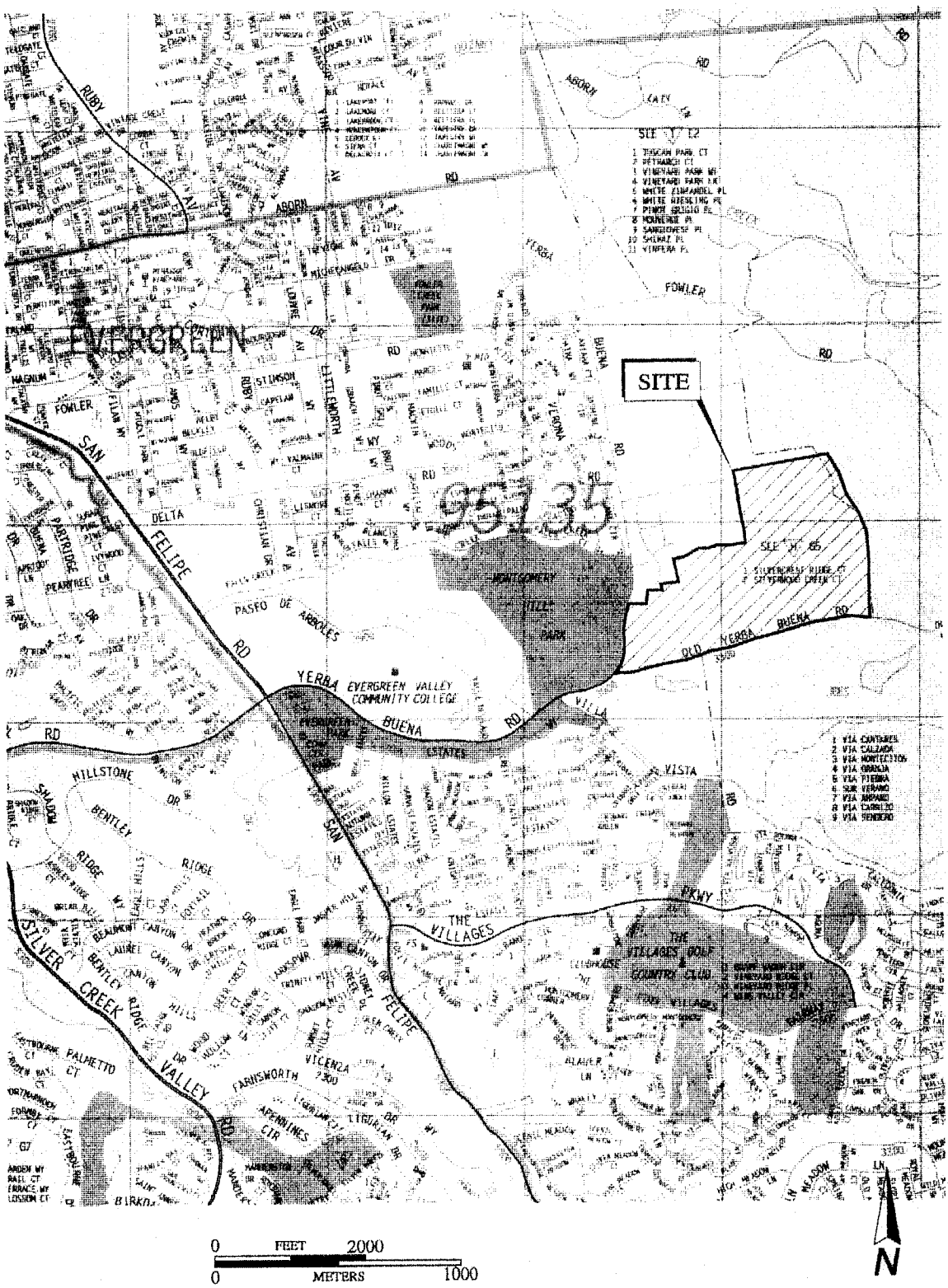

BASE MAP SOURCE: THOMAS BROTHERS

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SITE VICINITY MAP
EVERGREEN VIEW
SAN JOSE, CALIFORNIA

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FIGURE NO. 1

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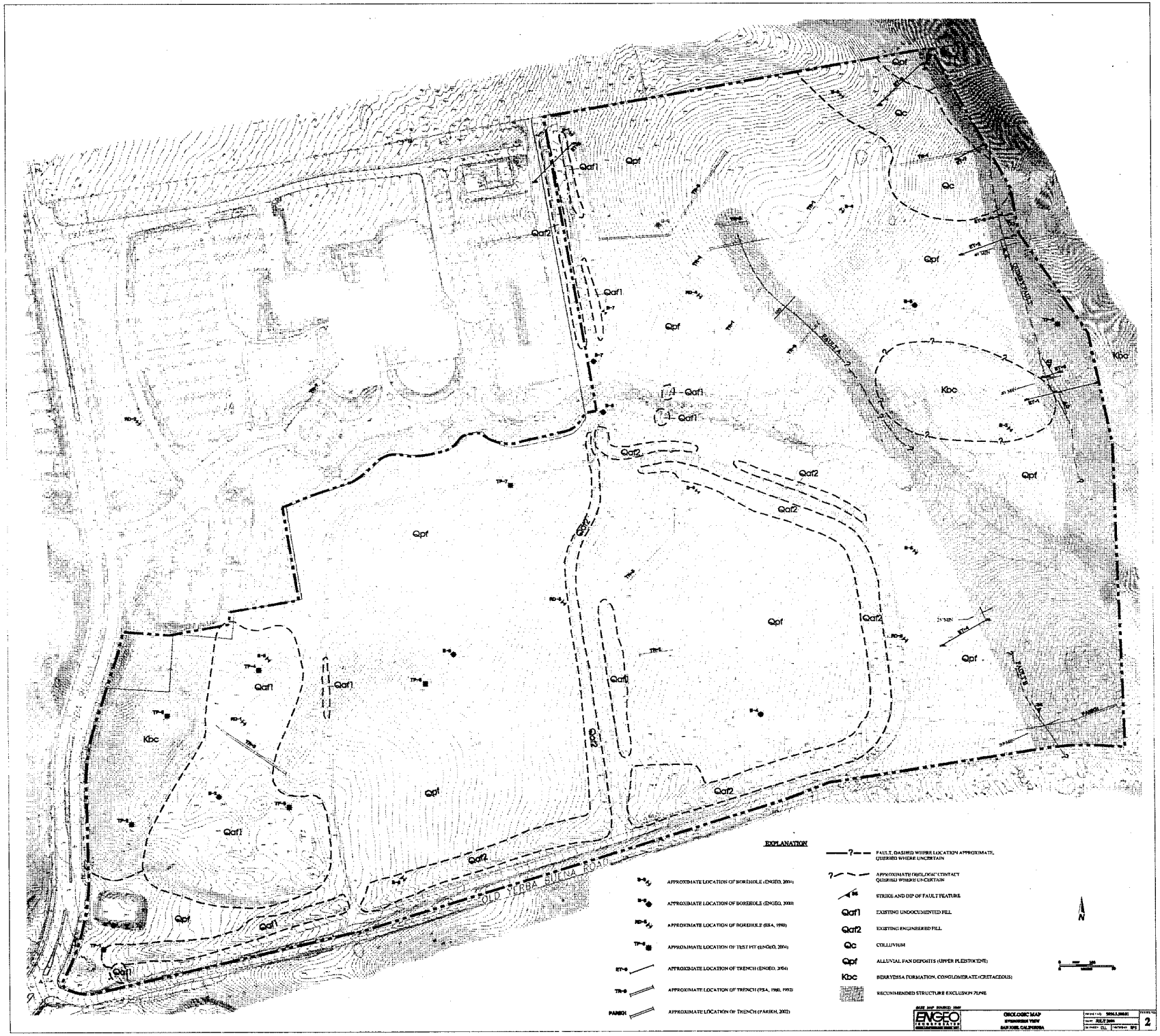
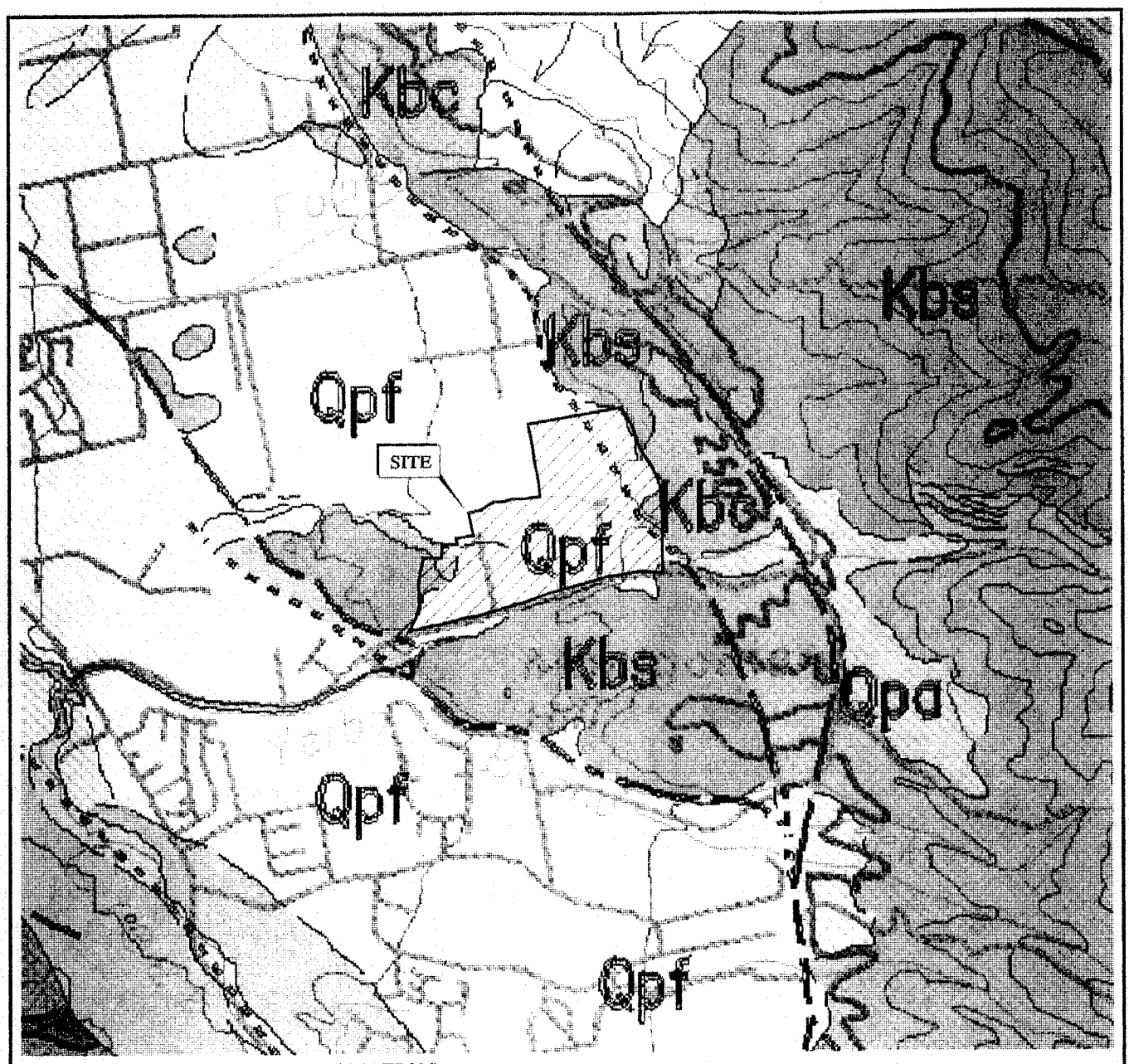

Qpf
Qc
Qaf1
Qaf2
Kbc
OLD YERBA BUENA ROAD
EXPLANATION
APPROXIMATE LOCATION OF BOREHOLE (ENGEO, 2004)
APPROXIMATE LOCATION OF BOREHOLE (ENGEO, 2003)
APPROXIMATE LOCATION OF BOREHOLE (EKA, 1990)
APPROXIMATE LOCATION OF TEST PIT (ENGEO, 2004)
APPROXIMATE LOCATION OF TRENCH (ENGEO, 2004)
APPROXIMATE LOCATION OF TRENCH (PSA, 1988, 1992)
PARISH
APPROXIMATE LOCATION OF TRENCH (PARISH, 2002)
FAULT, DASHED WHERE LOCATION APPROXIMATE, QUERIED WHERE UNCERTAIN
APPROXIMATE GEOLOGIC CONTACT QUERIED WHERE UNCERTAIN
STRIKE AND DIP OF FAULT FEATURE
Qaf1 EXISTING UNDOCUMENTED FILL
Qaf2 EXISTING ENGINEERED FILL
Qc COLLUVIUM
Qpf ALLUVIAL FAN DEPOSITS (UPPER PLEISTOCENE)
Kbc BERRYESSA FORMATION, CONGLOMERATE (CRETACEOUS)
RECOMMENDED STRUCTURE EXCLUSION ZONE
N
ENGEO INCORPORATED
GEOLOGIC MAP
SAN JOSE, CALIFORNIA
2



EXPLANATION

Qpa ALLUVIUM (UPPER PLEISTOCENE)

Qpf ALLUVIAL FAN DEPOSITS (UPPER PLEISTOCENE)

Kbs BARRYESSA FORMATION (CRETACEOUS) SANDSTONE AND MUDSTONE

Kbc BARRYESSA FORMATION (CRETACEOUS) CONGLOMERATE

APPROXIMATE LOCATION OF FAULT DASHED WHERE CONCEALED

N

0 FEET 2000

0 METERS 1000

BASE MAP SOURCE: WENTWORTH, 1999

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REGIONAL GEOLOGIC MAP
EVERGREEN VIEW
SAN JOSE, CALIFORNIA

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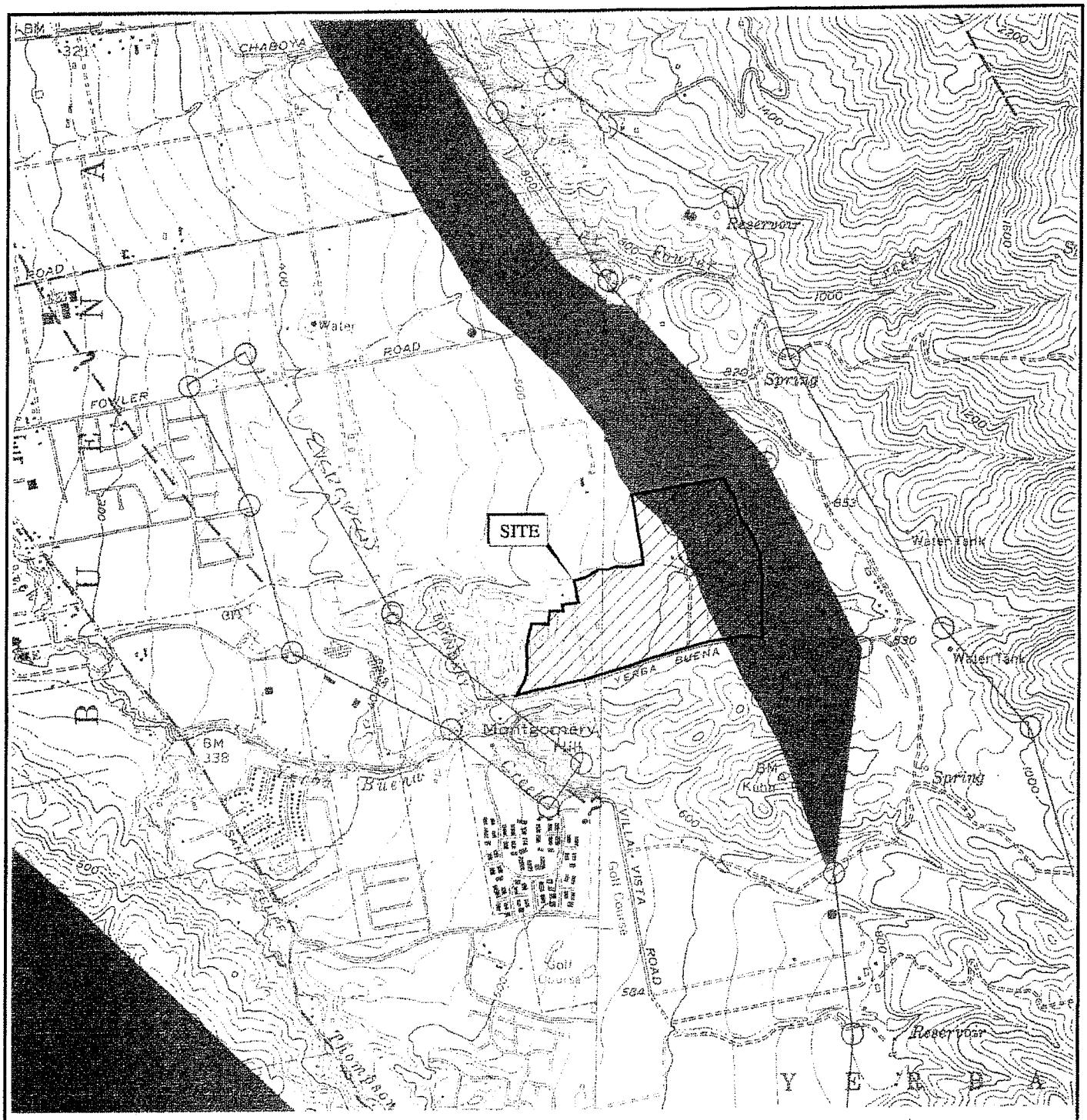

EXPLANATION

ALQUIST-PRIOLO SPECIAL STUDIES ZONE

CITY OF SAN JOSE SPECIAL STUDIES ZONE

REPORTED FAULTS QUERIED WHERE IN DOUBT

N

0 FEET 2000

0 METERS 1000

BASE MAP SOURCE: USGS, 1983

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FAULT HAZARD MAP

EVERGREEN VIEW

SAN JOSE, CALIFORNIA

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FIGURE NO. 4

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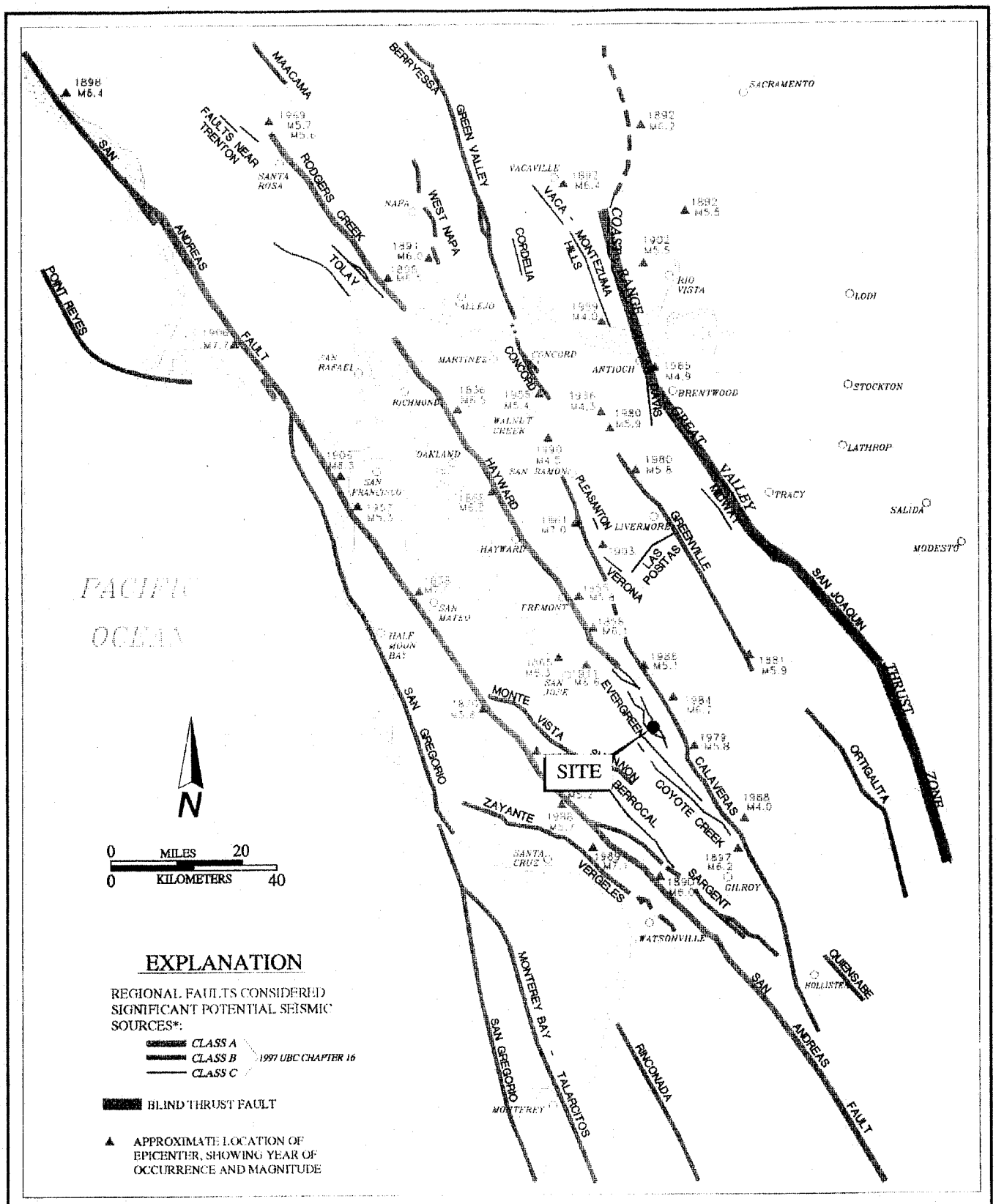

EXPLANATION

REGIONAL FAULTS CONSIDERED SIGNIFICANT POTENTIAL SEISMIC SOURCES*:

- CLASS A
- CLASS B
- CLASS C

1997 UBC CHAPTER 16

BLIND THRUST FAULT

▲ APPROXIMATE LOCATION OF EPICENTER, SHOWING YEAR OF OCCURRENCE AND MAGNITUDE

*BASED ON USGS OPEN FILE 96-706

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REGIONAL FAULTING AND SEISMICITY
EVERGREEN VIEW
SAN JOSE, CALIFORNIA

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FIGURE NO. 5

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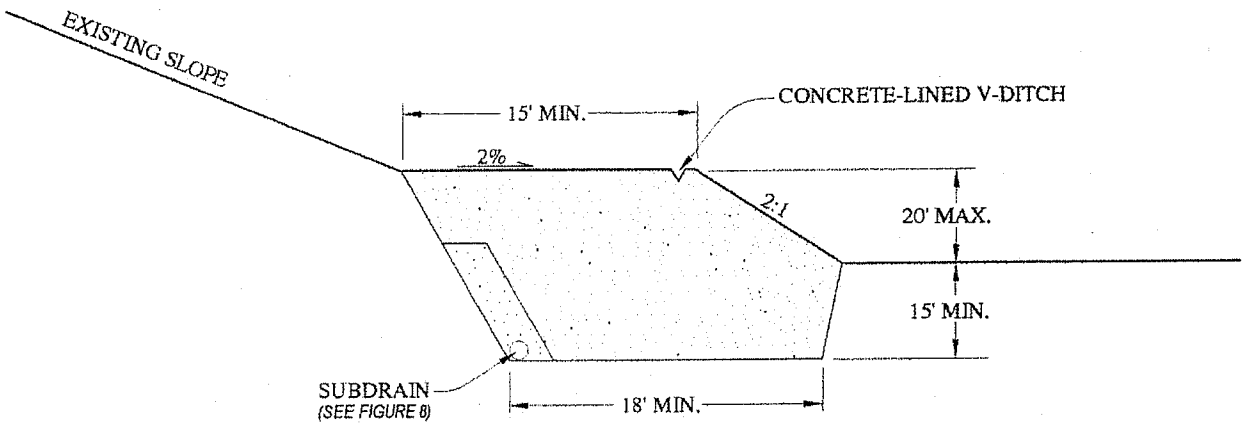

ACTUAL SIZE AND DEPTH OF KEYWAY TO BE DETERMINED IN THE FIELD BY THE GEOTECHNICAL ENGINEER

NO SCALE

ENGEO INCORPORATED
EXCELLENT SERVICE SINCE 1971

TYPICAL DEBRIS BENCH BUTRESS DETAIL
EVERGREEN VIEW
SAN JOSE, CALIFORNIA

PROJECT NO.: 5056.3.500.01	FIGURE NO. 6
DATE: JULY 2004	
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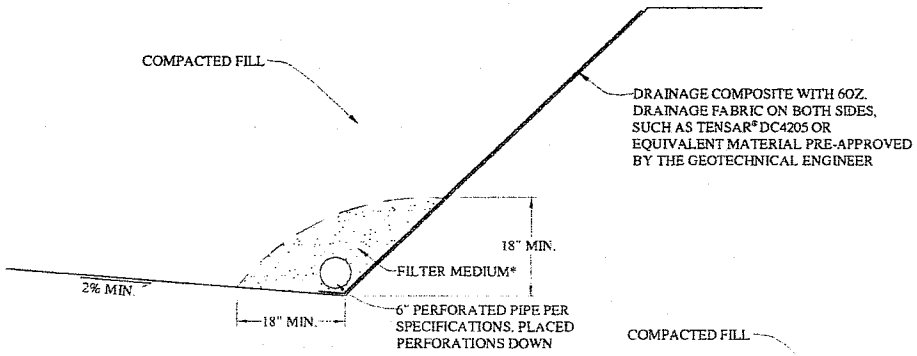
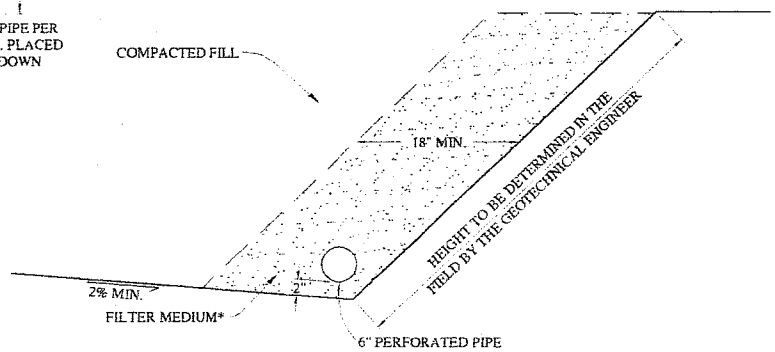
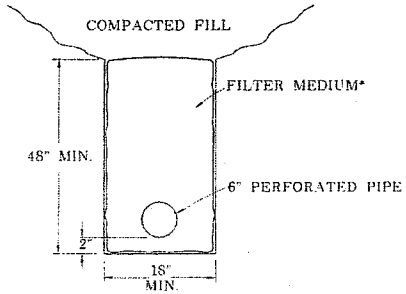


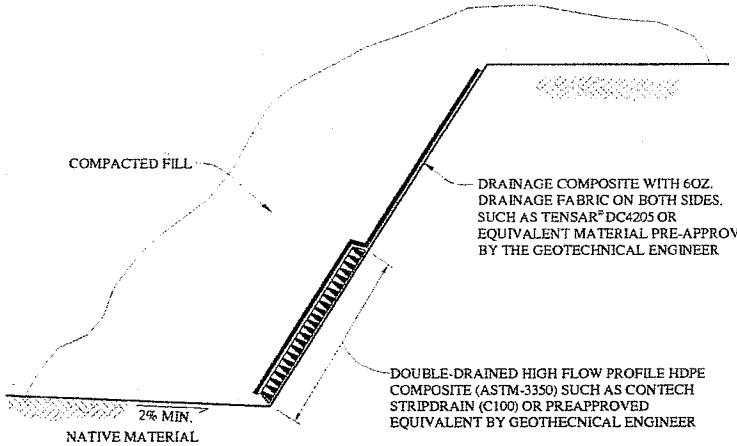
KEYWAY SUBDRAIN - OPTION 1



KEYWAY SUBDRAIN - OPTION 2



SWALE SUBDRAIN



ALTERNATE KEYWAY SUBDRAIN - OPTION 3

(FOR DEPTHS LESS THAN 30 FEET)

FILTER MEDIUM

ALTERNATIVE A - CLASS 2 PERMEABLE MATERIAL

MATERIAL SHALL CONSIST OF CLEAN, COARSE SAND AND GRAVEL OR CRUSHED STONE, CONFORMING TO THE FOLLOWING GRADING REQUIREMENTS:

SIEVE SIZE	% PASSING SIEVE
1"	100
3/4"	90-100
3/8"	40-100
#4	25-40
#8	18-33
#30	5-15
#50	0-7
#200	0-3

ALTERNATIVE B - CLEAN CRUSHED ROCK OR GRAVEL WRAPPED IN FILTER FABRIC

ALL FILTER FABRIC SHALL MEET THE FOLLOWING MINIMUM AVERAGE ROLL VALUES UNLESS OTHERWISE SPECIFIED BY ENGEO:

GRAB STRENGTH (ASTM D-4632)	180 lbs
MASS PER UNIT AREA (ASTM D-4751)	6 oz/yd²
APPARENT OPENING SIZE (ASTM D-4751)	70-100 U.S. STD. SIEVE
FLOW RATE (ASTM D-4491)	80 gal/min/ft
PUNCTURE STRENGTH (ASTM D-4833)	80 lbs

NOTES:

1. *ALL PIPE JOINTS SHALL BE GLUED*
2. *ALL PERFORATED PIPE PLACED PERFORATIONS DOWN*
3. *1% FALL (MINIMUM) ON ALL TRENCHES AND DRAIN LINES*

NO SCALE

ENGEO INCORPORATED EXCELLENT SERVICE SINCE 1971	TYPICAL SUBDRAIN DETAILS EVERGREEN VIEW SAN JOSE, CALIFORNIA	PROJECT NO.: 5056.3.500.01 DATE: JULY 2004 DRAWN BY: CLL CHECKED BY:	FIGURE NO. 7

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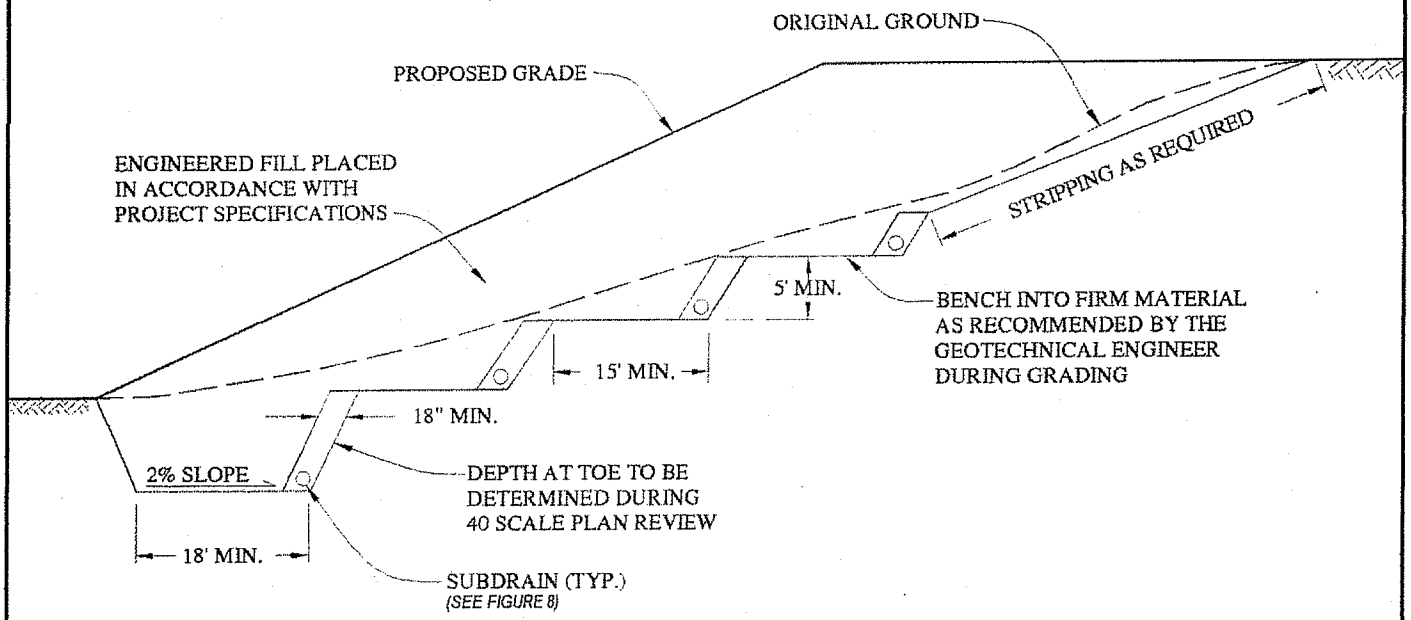

NO SCALE

ENGEO INCORPORATED
EXCELLENT SERVICE SINCE 1971

TYPICAL KEYWAY SECTION
EVERGREEN VIEW
SAN JOSE, CALIFORNIA

PROJECT NO.: 5056.3.500.01	FIGURE NO. 8
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LOWNEY ASSOCIATES
Environmental/Geotechnical/Engineering Services

Mountain View
Fairfield
Oakland
San Ramon
Fullerton

January 12, 2005
2100-1

Mr. Michael Hill
Vice Chancellor -Administrative Services
SAN JOSE/EVERGREEN COMMUNITY COLLEGE DISTRICT
4750 San Felipe Road
San Jose, California 95135

RE: GEOTECHNICAL AND GEOLOGIC FEASIBILITY STUDY EVERGREEN VALLEY COLLEGE LAND DEVELOPMENT SAN JOSE, CALIFORNIA

Dear Mr. Hill:

We are pleased to present the results of our geotechnical and geologic feasibility study for the above referenced project. Our report includes a description of the geotechnical, geologic, and seismic aspects of the site along with our conclusions and recommendations for conceptual planning and preparation of the project Environmental Impact Report (EIR).

We refer you to the text of the report for detailed recommendations. To help us continue to add value to your projects please visit the feedback section on our web site at www.lowney.com/feedback. Your opinion is important to us. If you have any questions concerning our findings, please call and we will be glad to discuss them with you.

Very truly yours,

LOWNEY ASSOCIATES

Jerrold A. Hanson, P.E.
Senior Project Engineer

JAH:LA:BRW:JRD:jcm

Copies: Addressee (1)
HMH Engineers (1)
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LOWNEY ASSOCIATES
Environmental/Geotechnical/Engineering Services

Geotechnical and Geologic Feasibility Study

Evergreen Valley College Land Development
San Jose, California

Report No. 2100-1 has been prepared for:

San Jose/Evergreen Community College District
4750 San Felipe Road, San Jose, California 95135

January 12, 2005

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FIGURE 1 — VICINITY MAP

FIGURE 2 — SITE PLAN

FIGURE 3 — VICINITY GEOLOGIC MAP

FIGURE 4 — REGIONAL FAULT MAP

GEOTECHNICAL AND GEOLOGIC FEASIBILITY STUDY
EVERGREEN VALLEY COLLEGE LAND DEVELOPMENT
SAN JOSE, CALIFORNIA

1.0 INTRODUCTION

In this report we present the results of our geotechnical and geologic hazards review for the Evergreen Valley College mixed-use development to be located in San Jose, California. The location of the site is shown on the Vicinity Map, Figure 1. The purpose of our investigation was to review published and unpublished material regarding the geotechnical, geologic, and seismic aspects of the site; and to evaluate potential geologic hazards at the site, provide our geotechnial conclusions and recommendations for conceptual planning of the development; and preparation of the project Environmental Impact Report (EIR).

1.1 Project Description

We understand that San Jose/Evergreen Community College District is considering developing four parcels totaling approximately 27 acres at the western end of the existing campus (Figure 2). The development would be mixed-use plan with uses for commercial, residential, and student/staff housing, along with other possible uses that fit with the college or community's objectives.

The four parcels include both developed and undeveloped acreage consisting of the following:

- 10-acre Yerba Buena Road parcel that is the largest of the four parcels and is currently undeveloped,
- 6.7-acre District Offices parcel along San Felipe Road that is currently occupied by an existing office building,
- 5.6-acre Criminal Justice Training Center currently occupied by two buildings, and
- 4.8-acre Northeast Site that was previously proposed for future expansion.

The final development plan will determine the actual parcel use, with possible changes to the existing parcel configurations (dividing or combining). The development will also include the installation of underground utilities, and roadway construction to service the commercial, and residential land use.

1.2 Scope of Services

Our scope of services was presented in detail in our agreement with you dated September 22, 2004. To accomplish this work, we provided the following services:

- Review of available geologic and seismic hazard maps, information in our files for the surrounding areas, and geotechnical and geologic hazard investigations from the adjacent site by others.
- Review of aerial photographs at Pacific Aerial Survey/USGS for geomorphic evidence of recent faulting, past site usage, old fills, slope instability, or other site issues.
- Geologic site reconnaissance to evaluate evidence of geologically recent faulting drainage patterns, springs, and other related features indicative of potential geologic hazards.
- Preparation of this report to summarize our findings and to present our conclusions and recommendations.

We also provided services to evaluate the environmental aspects of this project. A summary of our Phase I evaluation is presented in another report, along with our conclusions and recommendations.

2.0 GEOLOGIC SETTING

The site is situated on the distal end of alluvial fan deposits between a northwest-trending bedrock ridge that rises above the relatively flat alluvial plain of the Santa Clara Valley and the East Bay Hills. The Santa Clara Valley, a northwest-southeast trending valley within the Coast Ranges Geomorphic Province, is within the San Francisco Bay Block, which is bounded to the east by the Hayward and Calaveras Faults, and to the west by the San Andreas Fault. The eastern edge of the valley is bounded by active and potentially active faults, such as the Calaveras and Hayward Faults.

The ridge that extends to the southwest of the site is underlain by Pleistocene and Pliocene sedimentary rocks (designated as Packwood gravels by Wentworth and others, 1999), sedimentary rocks of the Lower Cretaceous and Upper Jurassic Knoxville formation, as well as Jurassic serpentinite and Lower Tertiary and Cretaceous Franciscan melange. The East Bay Hills to the northeast are underlain mainly by sandstone and mudstone of the Cretaceous Berryessa formation. Several generally northwest oriented sub-parallel faults traverse these rocks.

The site is underlain with younger Pleistocene alluvial deposits generally consisting of weakly consolidated clay, silt, sand, and gravel (Helley and others, 1994). No faults are mapped through the site. The closest fault, zoned as active, is the Evergreen Fault, approximately 600 feet to the northeast. Figure 3 depicts the geology of the site area.

3.0 SEISMICITY

3.1 Seismic Setting

The San Francisco Bay Area is one of the most seismically active regions in the United States. The project site is located in a California Building Code (CBC) Seismic Zone 4, as is the entire Bay Area. Large magnitude earthquakes have impacted the Bay Area in recent history. The significant earthquakes that occur in the Bay Area are generally

associated with crustal movement along well-defined, active fault zones of the San Andreas Fault system, which regionally trend in a northwesterly direction.

3.2 Regional Active Faults

The Santa Clara valley lies between two major right-lateral, strike-slip faults: the San Andreas Fault Zone to the west and the Hayward Fault Zone to the east. The San Andreas Fault passes about 15 miles southwest of the site. The Southeast Extension of the Hayward Fault is located approximately 1¼ miles to the northeast of the site. Three other major active faults in the area are the Calaveras Fault, located approximately 3½ miles northeast of the site, Monte Vista-Shannon Fault, located approximately 7 miles to the southwest, and the Hayward Fault, located approximately 10 miles northeast (Figure 4).

3.3 Local Faults

The Evergreen Fault traverses the eastern side of the Evergreen Valley College campus and appears to be a splay of the Hayward Fault. The Evergreen Fault is a northwest-striking, northeast-dipping reverse or reverse-oblique fault that has Cretaceous thrust shales and sandstones against and over late Pleistocene and early Quaternary alluvial deposits.

The State of California has designated the Evergreen Fault as an active feature and established an Earthquake Fault Zone pursuant to the Alquist–Priolo Earthquake Fault Zone Act. Woodward-Clyde Consultants (WCC) performed several fault investigations to locate the Evergreen Fault on the Evergreen Valley College campus (1994). Based on the WCC investigation, the Evergreen Fault is located 600 feet east of the eastern most part of the proposed development site. The proposed development site is not located within a City of San Jose Special Studies Zone or Potential Hazard Zone or a Santa Clara County Fault Hazard Zone.

3.4 Historic and Future Earthquakes

Although research on earthquake prediction has greatly increased in recent years, seismologists cannot predict when or where an earthquake will occur. The U.S. Geological Survey's Working Group on California Earthquake Probabilities (2003), referred to as WG03, determined there is a 62 percent chance of at least one magnitude 6.7 or greater earthquake striking the San Francisco Bay region between 2003 and 2032.

This result is an important outcome of WG03's work, because any major earthquake can cause damage throughout the region. This potential was demonstrated when the 1989 Loma Prieta earthquake caused severe damage in Oakland and San Francisco, more than 50 miles from the fault rupture. Although earthquakes can cause damage at a considerable distance, shaking will be very intense near the fault rupture. Therefore, earthquakes located in urbanized areas of the region have the potential to cause much more damage than the 1989 Loma Prieta earthquake.

The Probabilistic Seismic Hazard Analysis (PSHA) performed by the United States Geological Survey, estimates a peak horizontal ground acceleration of 0.61g at the site with a 10 percent probability of exceedance in 50 years (USGS OFR 02-420).

We performed a computer search of known historical earthquakes of Richter Magnitude 5 or greater within a 100 kilometer radius of the site using an abbreviated version of the California Division of Mines and Geology (CDMG) computerized earthquake catalog of events through December 1998. We also included data from Townley and Allen (1939) and the U.S. Geological Survey Earthquake Data Base System, giving 198 years of data in the search area. The results of our computer search indicate that 95 known earthquakes of Richter Magnitude 5 or greater have occurred within 100 kilometers of the site between 1800 and December 1998.

Four earthquakes of Richter Magnitude 7 or greater have occurred in the region during the above noted time period, including the Loma Prieta Earthquake of 1989, centered about 19 miles from the site. Based on attenuation methods of Campbell and Bozorgnia (1994), the maximum historic site acceleration that may have been experienced at the site is computed at approximately 0.54g, resulting from the Richter Magnitude 8.25, 1906 San Francisco earthquake located approximately 49 miles from the site.

The computer-generated acceleration values and probabilities should only be considered reasonable best estimates. All of the influences affecting attenuation and occurrence rates are not yet known; furthermore, there are uncertainties in every parameter used to obtain such results. At the present time there is no test available to verify the validity of the acceleration and probability data. Therefore, significant deviations from the indicated values are possible due to geotechnical and geological uncertainties and other specific site conditions.

4.0 SITE CONDITIONS

4.1 Surface

The site consists of four parcels located west of the existing campus on the corner of San Felipe Road and Yerba Buena Road in San Jose, California. The largest (10 acres) and smallest (4.8-acres) are currently undeveloped and do not have any existing structures. The other two parcels (6.7-acres and 5.6 acres) are currently developed with office buildings and appurtenant parking.

Based on U.S. Geological Survey (USGS) topographic map, the site's elevation is approximately Elevation 358 feet above mean sea level along the eastern boundary to Elevation 302 feet above mean sea level in the western portion of the site. Topography in the vicinity of the site slopes gently to the southwest towards Thompson Creek.

4.2 Subsurface

Based on our review of available geologic maps and the WCC investigations (Woodward-Clyde, 1994, and URS Greiner Woodward Clyde, 1999) on the adjacent Evergreen Valley College, the soil beneath the site generally consists of stiff to very stiff lean clays with varying amounts of sand and gravel, and interbedded medium dense to very dense clayey sands.

4.3 Ground Water

Ground water was not encountered in borings drilled on the eastern portion of the campus for the Gullo Student Center (Woodward Clyde, 1995) and the proposed Nursing/Biology Addition (URS Greiner Woodward Clyde, 1999) to the maximum depths explored, 25 and 31½ feet, respectively. Information for nearby sites available on the Santa Clara Valley Water District (SCVWD) web site, indicate that ground water is approximately 35 to 40 feet beneath the site. Fluctuations in the level of the ground water may occur because of variations in rainfall, underground drainage patterns, and other factors not evident at the time of drilling.

5.0 GEOLOGIC HAZARDS

A brief qualitative evaluation of geologic hazards was made during this investigation. Our comments concerning these hazards are presented below.

5.1 Fault Rupture Hazard

A Regional Fault Map illustrating known active faults relative to the site is presented in Figure 4. The site is not located within a currently designated Alquist-Priolo Earthquake Fault Zone (known formerly as a Special Studies Zone) or a City of San Jose Potential Hazard Zone or a Santa Clara County Fault Rupture Hazard Zone. As discussed on Section 3.3, no known surface expression of active faults is believed to cross the site; therefore, fault rupture through the site is not anticipated.

5.2 Ground Shaking

Strong ground shaking can be expected at the site during moderate to severe earthquakes in the general region. This is common to virtually all developments in the San Francisco Bay Area. Potential levels of ground shaking at the site are discussed in the "Seismicity" section of this report. Site improvements should be designed in accordance with the seismic provisions of the 2001 California Building Code (CBC).

5.3 Liquefaction

Soil liquefaction results from loss of strength during cyclic loading, such as imposed by earthquakes. During cyclic ground shaking, such as during earthquakes, cyclically induced stresses may cause increased pore water pressures within the soil matrix, resulting in liquefaction that results in a decrease in shear strength that may lead to large vertical deformation beneath foundations or sloping ground (NCEER/NSF, 1998) Liquefied soil can also settle (compact) as pore pressures dissipate following an earthquake. Soils most susceptible to liquefaction are loose to moderately dense, saturated non-cohesive soils with poor drainage, such as sands and silts with interbedded or capping layers of relatively low permeability soil.

The site is not located within a State of California Seismic Hazard Zone for liquefaction or a Santa Clara County Geologic Hazard Zone for liquefaction. It is underlain by alluvial fan deposits considered to have a low potential for liquefaction (Knudsen, 2000 and CGS, 2000). In addition, the granular soils encountered in the borings on the adjacent property were generally medium dense to very dense and contained a significant amount of fine-grained material. Based on this information, the potential for liquefaction at the site appears to be low.

5.4 Seismically-Induced Settlements

Soils most susceptible to seismically-induced settlement are loose to medium sands with small amounts of fines. Because the subsurface soils encountered in borings on the adjacent property were stiff to very stiff clays and medium dense to very dense sands with significant fines contents, we judge the probability of significant seismically-induced settlement impacting site improvements to be low.

5.5 Lateral Spreading

Lateral spreading typically occurs as a form of horizontal displacement of relatively flat-lying alluvial material toward an open or "free" face such as an open body of water, channel, or excavation. In soils this movement is generally due to failure along a weak plane, and may often be associated with liquefaction. Yerba Buena Creek is located within 100 feet of the southern boundary of the site; however, lateral spreading is not anticipated due to the apparent absence of a potentially liquefiable layer. Therefore, the probability of lateral spreading occurring at the site during a seismic event is most likely low.

5.6 Landsliding

The site is not located within a State of California Seismic Hazard Zone for earthquake-induced landslides or a Santa Clara County Landslide Hazard Zone. Furthermore, the site improvements are to be located on a broad, gently sloping plane with a very low potential for deep-seated lateral instability. Therefore, the potential for landsliding at the site is remote.

5.7 Flooding

As shown on the August 17, 1998, Federal Emergency Management Agency (FEMA) "Flood Insurance Rate Map" (FIRM), this site is within Zone D, described as "Areas of undetermined, but possible, flood hazards." The site is not located within a currently designated Santa Clara County Dike Failure Hazard Zone.

6.0 CONCLUSIONS AND DEVELOPMENT CONSIDERATIONS

From a geotechnical engineering viewpoint, the site is suitable for the proposed mixed-use development. Design-specific geotechnical recommendations will need to be prepared for site development when a development plan becomes available. We recommend that a final geotechnical investigation be performed once conceptual grading plans have been finalized. The purpose of a design-level investigation would be to further evaluate the geotechnical characteristics of the site, collect and test samples of the subsurface materials, and develop detailed geotechnical recommendations for project design. The preliminary foundation recommendations that follow are intended to allow conceptual planning and preparation of the project Environmental Impact Report (EIR).

7.0 PRELIMINARY FOUNDATION RECOMMENDATIONS

Proposed structures can likely be supported on either conventional shallow footings or mat foundations.

7.1 Footings

The proposed buildings may be supported on conventional continuous and/or isolated spread footings bearing on natural, undisturbed soil or compacted fill. On a preliminary basis, footings should have the minimum footing dimensions shown in Table 1 below. Footing depths are taken from lowest adjacent finished grade, considered as the bottom of interior slab-on-grade or the finished exterior grade, excluding landscape topsoil, whichever is lower.

Table 1. Minimum Footing Dimensions

Number of Stories	Minimum Footing Width (inches)	Minimum Footing Depth (inches)
1	12	18
2	15	18
3	24	24

Footings constructed in accordance with the above recommendations would be capable of supporting maximum allowable bearing pressures on the order of 2,000 pounds per square foot (psf) for dead loads, 3,000 psf for combined dead and live loads, and 4,000 psf for all loads including wind or seismic. These allowable bearing pressures are based upon factors of safety of 3.0, 2.0, and 1.5 for dead, dead plus live, and seismic loads, respectively.

7.2 Slabs-on-Grade with Footings

Slab-on-grade floors may be used in conjunction with shallow foundations. The proposed slab-on-grade floors may be supported directly on non-expansive compacted natural soil or fill. Before slab construction, the subgrade surface should be proof-rolled to provide a smooth, firm surface for slab support. Slab reinforcing should be provided in accordance with the anticipated use and loading of the slab.

7.3 Mat Foundations

As an alternative to shallow footings, proposed structures may be supported on either conventionally reinforced or post-tensioned mats bearing on native soils and/or compacted fill. Mat foundations would be capable of supporting average allowable bearing pressures on the order of 1,000 pounds per square foot (psf) for dead plus live loads, with maximum localized bearing pressures of approximately 3,000 psf at column or wall loads. Allowable bearing pressures can be increased by one-third for all loads including wind or seismic. All mats should be designed with a thickened edge at least 12 inches wide and 12 inches thick.

8.0 LIMITATIONS

This report has been prepared for the sole use of the San Jose/Evergreen Community College District, specifically for conceptual planning and preparation of the project Environmental Impact Report (EIR) of the mixed-use development on the western portion of the Evergreen Community College campus in San Jose, California. The opinions, conclusions, and recommendations presented in this report have been formulated in accordance with accepted geotechnical engineering practices that exist in the San Francisco Bay Area at the time this report was written. No other warranty, expressed or implied, is made or should be inferred.

The opinions, conclusions and recommendations contained in this report are based upon the information obtained from our investigation, which includes data from visual observations during our site reconnaissance, and review of other geotechnical data provided to us, along with local experience and engineering judgment. We are not responsible for the data presented by others.

As already noted, a design-level investigation should be performed to further evaluate the geotechnical and geologic characteristics of the site. The geotechnical aspects of the final plans and specifications should be reviewed for conformance with the recommendations in a design-level geotechnical report.

Lowney Associates cannot assume any responsibility for any potential claims that may arise during or after construction as a result of misuse or misinterpretation of Lowney Associates' report by others. Furthermore, Lowney Associates will cease to be the Geotechnical-Engineer-of-Record if we are not retained for these services and/or at the time another consultant is retained for follow up service to this report.

The opinions presented in this report are valid as of the present date for the property evaluated. Changes in the condition of the property will likely occur with the passage of time due to natural processes and/or the works of man. In addition, changes in applicable standards of practice can occur as a result of legislation and/or the broadening of knowledge. Furthermore, geotechnical issues may arise that were not apparent at the time of our investigation. Accordingly, the opinions presented in this report may be invalidated, wholly or partially, by changes outside of our control. Therefore, this report is subject to review and should not be relied upon after a period of three years, nor should it be used, or is it applicable, for any other properties.

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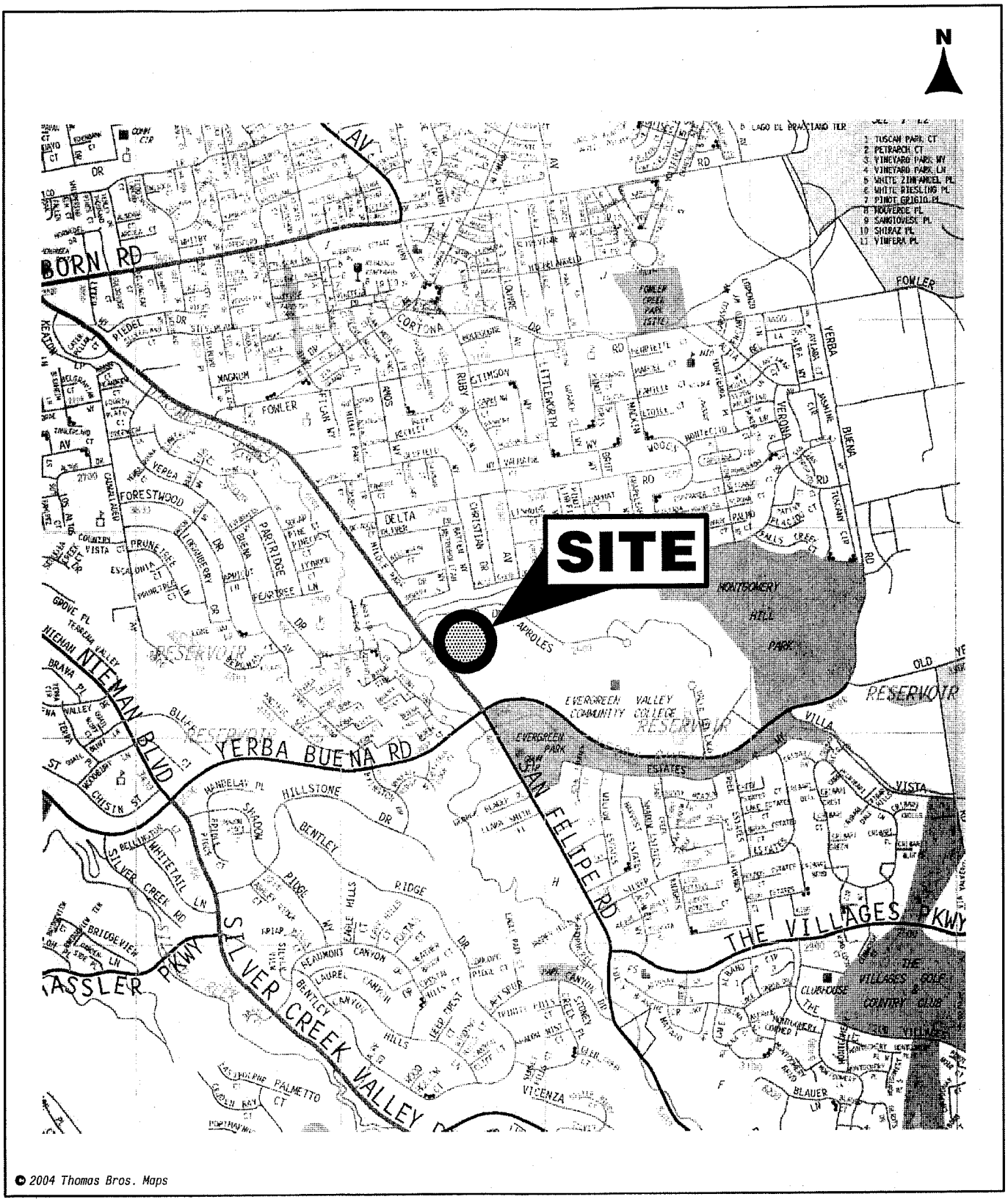
Youd et al., 2001, *Liquefaction Resistance of Soils:* Summary Report from the 1996 NCEER and 1998 NCEER/NSF Workshops on Evaluation of Liquefaction Resistance of Soils, ASCE Journal of Geotechnical and Geoenvironmental Engineering, Vo. 127, No. 10, October, 2001.

9.2 Aerial Photographs

Aerial photographs, dated 1939, 1956, 1982, and 1993 were reviewed to observe historic surficial features on the site.

* * * * * * * * * * * * *



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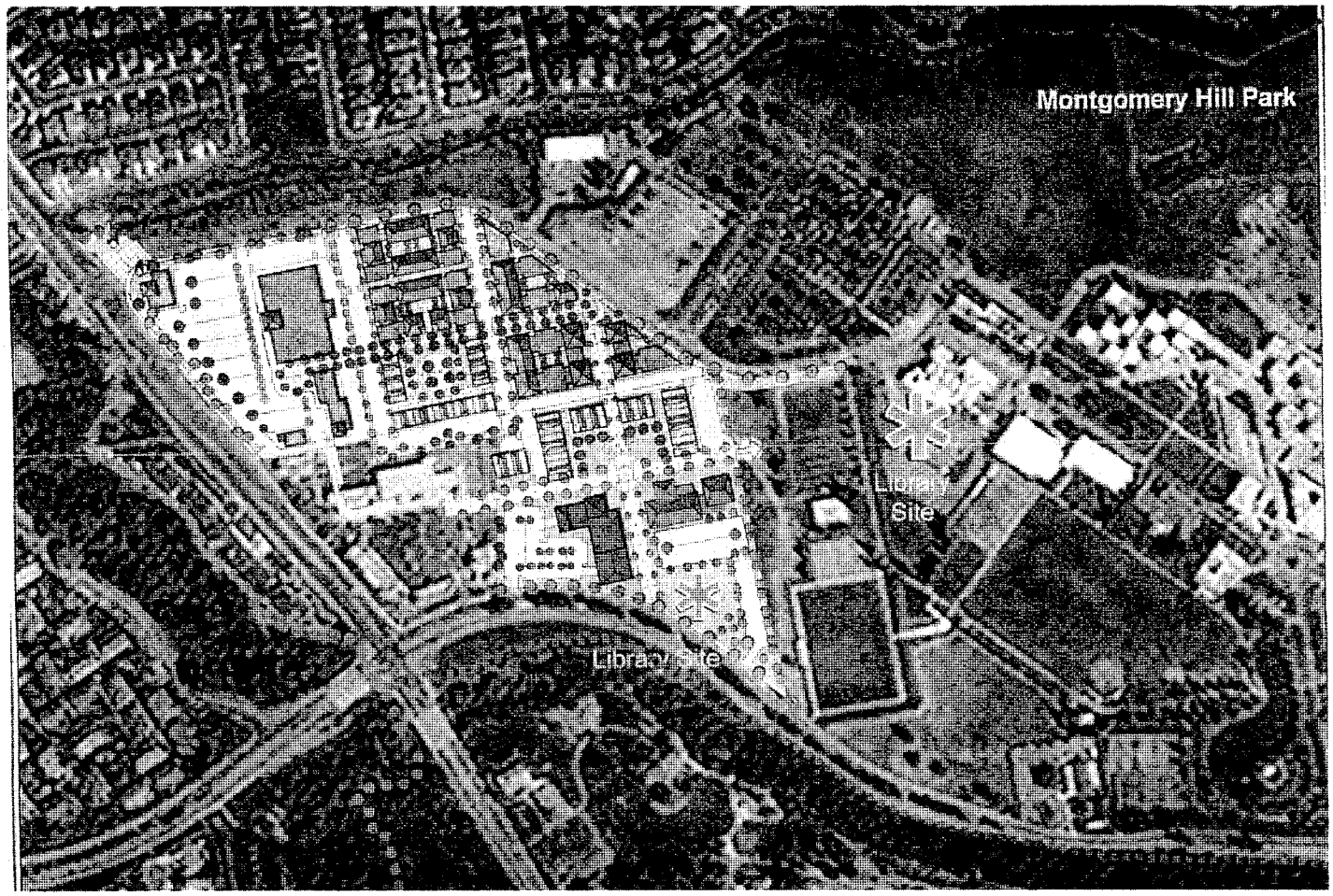
12/04*EB

VICINITY MAP

EVERGREEN COMMUNITY COLLEGE FEASIBILITY REVIEW
San Jose, California

LOWNEY ASSOCIATES
Environmental/Geotechnical/Engineering Services

FIGURE 1
2100-1



N

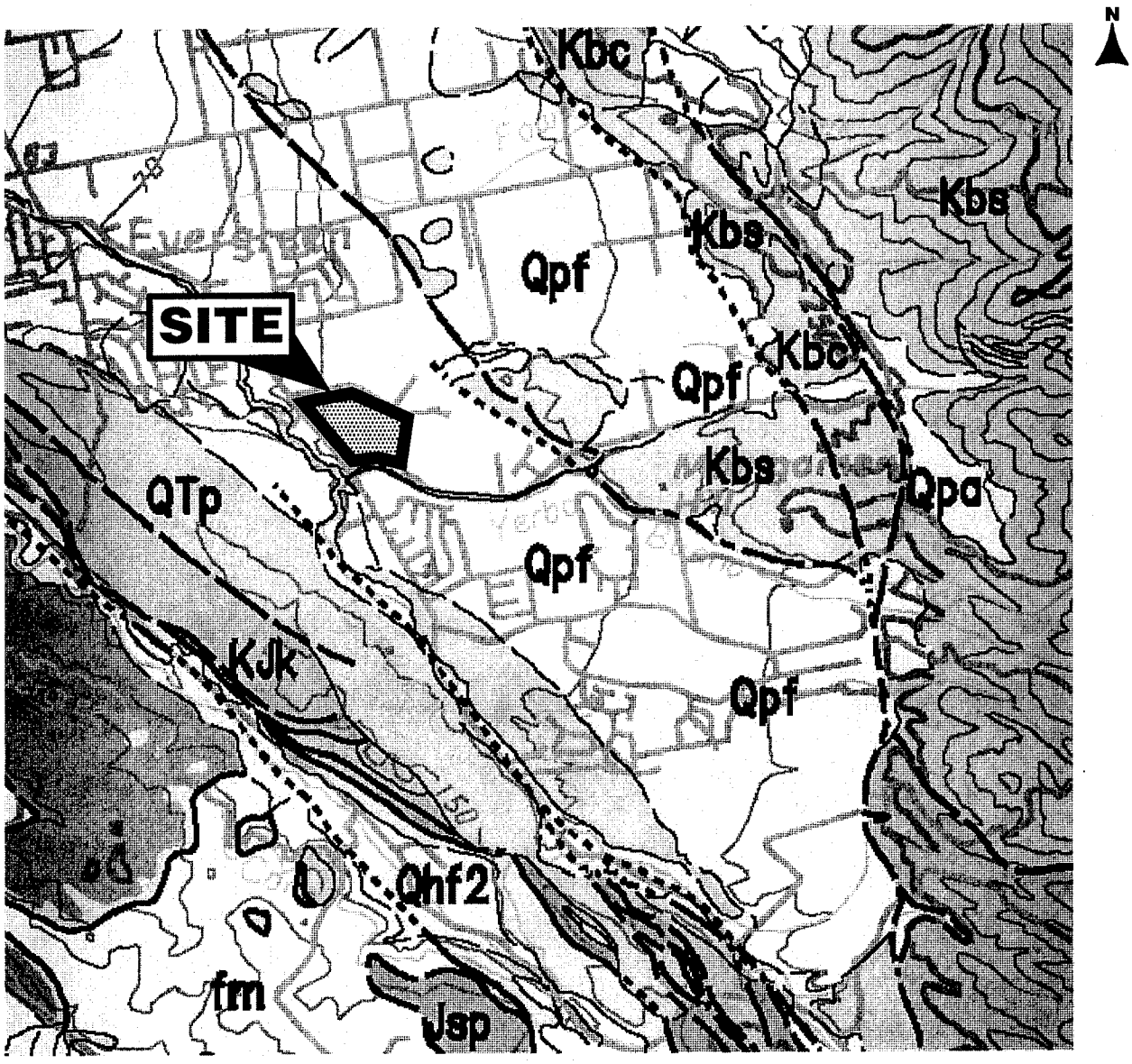
SITE PLAN
EVERGREEN COMMUNITY COLLEGE
FEASIBILITY REVIEW
San Jose, California

Base by Unknown.

LOWNEY ASSOCIATES
Environmental/Geotechnical/Engineering Services

FIGURE 2
2100-1

1/05*EB



EXPLANATION

Qhf2	Alluvial Fan Deposit (Holocene)	Kbs	Berryessa Formation, Sandstone and Mudstone (Cretaceous)
Qpf	Alluvial Fan Deposit (Upper Pleistocene)	Kbc	Berryessa Formation, Conglomerate (Cretaceous)
QTp	Packwood Gravels (Pleistocene? & Pliocene)	KJc	Knoxville (Lower Cretaceous and Upper Jurrassic)
		Jsp	Serpentinite (Jurassic)

——— - - - ···? Contact; dashed where approximately located, dotted where concealed

——— - - - ···? Fault; dashed where approximately located, dotted where concealed, queried where location uncertain

0 3,000 ft.

From: Wentworth and others (1999)

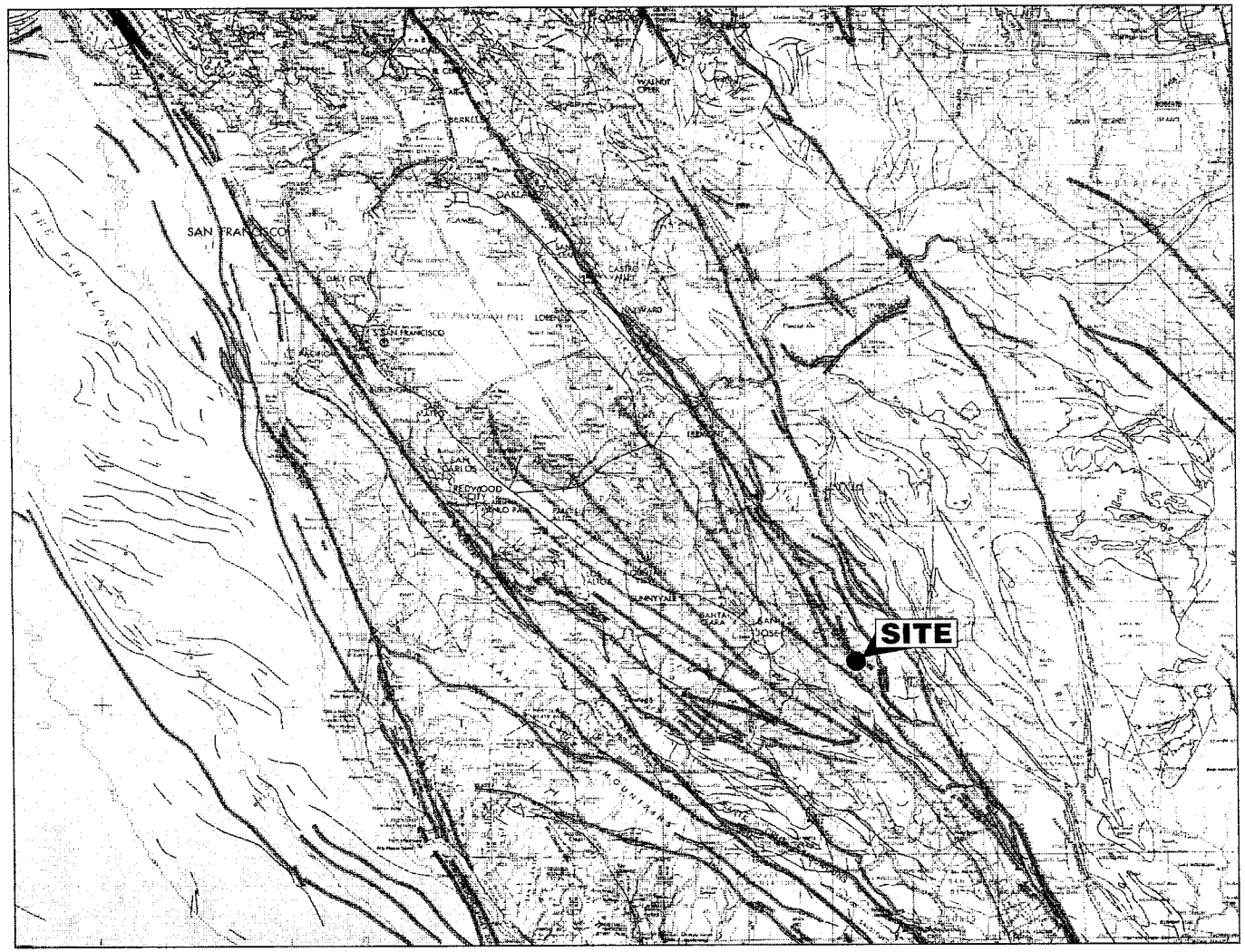
12/04*EB

VICINITY GEOLOGIC MAP

EVERGREEN COMMUNITY COLLEGE FEASIBILITY REVIEW
San Jose, California

LOWNEY ASSOCIATES
Environmental/Geotechnical/Engineering Services

FIGURE 3
2100-1



Note: Some faults highlighted in purple are not considered active (Holocene Movement) by the State of California.

Geologic Time Scale			Years Before Present Approx.	Fault Symbol	Recency of Movement on Land Offshore	DESCRIPTION
Quaternary	Late Quaternary	Historic				Displacement during historic time (e.g. San Andreas [illegible])
		Holocene	200			Displacement during Holocene time
			10,000			[illegible]
	Early Quaternary	Pleistocene	[illegible]			[illegible]
Pre-Quaternary	Pliocene		[illegible]			[illegible]
	Miocene		[illegible]			[illegible]

Base map is a composite of part the San Francisco 1:250,000 scale map (reference code 37 122-A1-TF-250-00, 1980) and the San Jose 1:250,000 scale map (reference code 37 120-A1-TF-250-00, 1969). For cartographic details, refer to these maps. Bathymetric information is not intended for navigational purposes.

Transverse Mercator Projection 10,000-meter Universal Transverse Mercator grid, zone 10.

Minor corrections and additions to culture by California Division of Mines and Geology 1987.

From: Bortugno & others (1991)

0 10 20 Miles

Scale 1:500,000

REGIONAL FAULT MAP
EVERGREEN COMMUNITY COLLEGE
FEASIBILITY REVIEW
San Jose, California

LOWNEY ASSOCIATES
Environmental/Geotechnical/Engineering Services

FIGURE 4
2100-1

Division of Mines and Geology, James F. Davis, State Geologist

6/03'EB