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TO WHOM IT MAY CONCERN

We brought compactable soil to refill excavated area at 451 Hegenberger Rd, Oakland, CA from Santa

planned residential Development located at 15900 Simoni Drive, San Jose, .

Kevin Trucking Inc.
Kulwinder Singh Virdee

# REPORT – ADDITIONAL SOIL INVESTIGATION INCLUDING GEOLOGIC & SLOPE STABILITY ANALYSIS

**PLANNED RESIDENTIAL DEVELOPMENT**
**15900 SIMONI DRIVE**
**SAN JOSE, SANTA CLARA COUNTY**
**CALIFORNIA**

GEI
Geotechnical
Engineering
Inc.

# REPORT – ADDITIONAL SOIL INVESTIGATION INCLUDING GEOLOGIC & SLOPE STABILITY ANALYSIS

## PLANNED RESIDENTIAL DEVELOPMENT
## 15900 SIMONI DRIVE
## SAN JOSE, SANTA CLARA COUNTY
## CALIFORNIA

111423B
JULY 31, 2008

GEI Geotechnical Engineering Inc.

CONSULTANTS IN GEOLOGIC & SOIL ENGINEERING

SUITE B-1 38750 PASEO PADRE PARKWAY FREMONT, CA 94536 TELEPHONE 800-791-SOIL 510-791-0100 FAX 510-791-1705

July 31, 2008
**Job No. 111423B**

Mr. Dang Nguyen
REBI
PO Box 32568
San Jose, CA 95125

Dear Mr. Nguyen:

Five copies of "Report-Additional Soil Investigation Including Geologic & Slope Stability Analysis, Planned Residential Development, 15900 Simoni Drive, San Jose, Santa Clara County, California" are herewith submitted. The work was authorized on April 16, 2008.

Based on The results of a detailed subsurface exploration of the home sites, laboratory testing, slope stability analysis, review of site related geologic and seismic literature and a site reconnaissance, it is our professional opinion that the proposed building sites are suitable for construction of the planned residences. Therefore, provided that the recommendations of this report with respect to site preparation, compaction, keyway construction, foundations, retaining walls, slab on grade, drainage (surface and subdrains), paving, inspection and those of the geologic section of this report are properly implemented under the supervision of GEI and continually maintained the building foundations should perform satisfactorily.

It has been a pleasure to serve you on this project. Should you have any questions or require additional information, please do not hesitate to call us.

Very truly yours,
Geotechnical Engineering, Inc.

REGISTERED PROFESSIONAL ENGINEER
ALAN S. BORIS
№ GE 155
Exp. 06/09
GEOTECHNICAL
STATE OF CALIFORNIA

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MICHAEL N. CLARK
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Engineering Geologist

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PLANNED RESIDENTIAL DEVELOPMENT
15900 SIMONI DRIVE
SAN JOSE, SANTA CLARA COUNTY
CALIFORNIA

## INTRODUCTION

### General

In this report, we present the results of the additional soil investigation, geologic and slope evaluation at the referenced site. The work was authorized by you on April 16, 2008, in accordance with the scope of the Geotechnical Engineering, Inc. (GEI) proposal of the same date. **This report presents additional geologic and geotechnical information relevant to the revised location of your planned houses. The initial GEI geologic and soil studies for a previous house location were dated September 15, and February 26, 2004.**

### Geotechnical Background

The sloping vacant site which will contain a new primary residence and a nearby secondary house, is located near the southwest portion of undeveloped property. The project site is accessed from Simoni Drive by paved cul-de-sac. The approximate locations of the planned residences are shown on vicinity map, **plate 1**. A site plan showing the site boundaries, borings and lines of cross-sections is shown on Geologic Map, **Plate 2**.

The proposed project site occupies the lower area of a sloping property uphill from and near the east end of Simoni Drive. Properties developed as private residences are located west of the planned residences. The property which is bordered on the northwest by Greebe Heights Developmen; Bella Montoya Heights and Celeo Lane lie southwest of the site. The property is covered by scattered brush, grasses and small trees.

Based upon geologic maps of the area **(plates 9 & 10)**, the bedrock consists of Cretaceous Berryessa formation (conglomerate & shale). The average gradient of the property is about 2.6 Horizontal : 1 Vertical (H:V). The slope on which the project is located drains to the west.

**Based upon California geologic maps, the property is located within CGS Earthquake Landslide Hazard "blue" area. The property is located outside a CGS Earthquake Fault Zone for the active Hayward fault.**

## Planned Construction

Based upon a site plan dated 2008 provided by Mr. Nguyen, we understand that new 1- and 2-story residences will be constructed. The project site is accessed from Simoni Drive by a paved cul-de-sac. New wood-frame, single-family residences are planned for construction at the southwest portion of the undeveloped property. New concrete retaining walls will also be constructed. The new building site is placed in a portion of the hillside that appears to be relatively stable; just uphill from existing houses. The site slopes southwest down toward existing adjacent residences; The average gradient of the property is about 2.6 Horizontal : 1 Vertical (H:V). The existing grades range from about 750 feet (in the southwest) to almost 1,100 feet.

As measured from the topographic map the site coordinates are:

**Latitude: 37.3856°N      Longitude: 121.8068°W**

## Purpose and Scope

As was outlined in our proposal dated April 16, 2008, the purposes of the investigation at the new residential site were to:

**(A) Provide information on regional geology, including a fault map and a site location map, with cross sections and discussion.**

**(B) Prepare a geologic map of the site (including cross sections),**

(C) Conduct a geologic reconnaissance of the site by a Certified Engineering Geologist to observe and document surface features indicative of possible geologic hazards.

(D) Discussion on expansive soils together with laboratory test results.

(E) Evaluation of historic seismicity, slope stability in accordance with CS Note 48.

(F) California Seismic zone, site class, and significant faults, site seismicity of the vicinity.

(G) Review of current published and unpublished citations for geology, seismology including review of slope maps, and calculating slope stability in the vicinity of the project.

(H) Drilling a total of 3 borings up to 30 feet deep to evaluate subsurface condition at the site and in the area of the planned house and provide appropriate recommendations for grading, compaction, foundations, retaining walls, drainage (surface & subdrain), slab on grade and paving; and

(I) Summarize the results of field explorations, geologic reconnaissance, laboratory testing and recommendations in an additional soil and geologic report.

Other engineering services such as preliminary environmental site assessment, sampling and testing for hazardous/toxic material, chemical / corrosion potential of the on-site soils and ground water, analysis of soil liquefaction potential, etc. were not included in our scope of services.

The purposes of the geologic assessment were to identify and assess potential geologic hazards at or near the site in accordance with the requirements for such studies set forth by the California Department of Conservation, California Geological Survey (CGS) Special Publication 117 (Guidelines for Evaluating and Mitigating Seismic Hazards); CGS Note 44 (Recommended Guidelines for Preparing Engineering Geologic Reports), and CGS Note 48 (Checklist for the review of Engineering Geology and

**Seismology Reports). References reviewed for compilation of this report are listed in the "References" section at the end of this report.**

In order to evaluate the subsurface conditions, a program of field explorations was undertaken including drilling **3 borings using track mounted rig with frequent sampling. A field geologic reconnaissance was also performed by our engineering geologist.** Laboratory testing was carried out on representative samples to provide the basis for engineering analyses. The results of the investigation together with our recommendations are presented as follows:

## GEOLOGY

### Regional Geology

The Alum Rock area of Santa Clara County lies within the Coast Ranges Geomorphic Province, a discontinuous series of northwest-southeast trending mountain ranges, ridges, and intervening valleys characterized by complex folding and faulting. The general geologic framework of the Alum Rock area of Santa Clara County, in and around the project site is illustrated in studies by Wentworth et al.(1999), as well as Helley and Wesling, (1990) and Dibblee (1973). Geologic structures within the Coast Ranges 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 Ranges 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 (Regional Geologic Map, **Plate 8**). Together, these and other faults are referred to as the San Andreas fault system. The general trend (about N40°W) of the faults within this system is responsible for the strong northwest-southeast structural grain of regional geologic and geomorphic features in the Coast Ranges Province (Regional Geologic Map, **Plate 7**).

## Area & Site Geology

Mapping by Wentworth et al., 1999 indicates that the site is underlain by bedrock of Cretaceous age Berryessa Formation Conglomerate (Kbc) and Shale (Kbs). Previous reports (Helley and Wesling 1990; Dibblee, 1973) referred to the same unit as Oakland Conglomerate and Berryessa Formation shale. Our site reconnaissance showed the presence of a shale unit as well as the mapped conglomerate. In this report the shale unit will be referred to as the Berryessa Formation Shale (Kbs) (**Plate 8** Geologic Map). The Berryessa Formation Conglomerate (Kbc) consists of thick, indistinct beds of pebble, cobble, and less common boulder conglomerate interfingered with coarse-grained mica-quartz- lithic wacke (Wentworth et al. 1999). The Berryessa Formation shale (Kbs) consists of mica-bearing siltstone and claystone.

Two Quaternary age landslide units are mapped on the site, (**Plate 2**, Site Plan and Geologic Map) as Older Landslide (Qol) and Younger Landslide (Qyl). These units consist of chaotic mixtures of angular fragments of Berryessa rocks in a debris matrix. The units are distinguished by the differences in the freshness of their geomorphic expression.

## Field Investigation

Our field investigation consisted of a surface reconnaissance together with subsurface geotechnical exploration program. Our Engineering Geologist visited the site on June 14, 2008 to observe and map surface conditions that may relate to geologic hazards.

A total of 3 exploratory borings were drilled to depths between 15.5 to 30 feet below existing ground surface using a track-mounted auger rig. The approximate locations of the soil borings are shown on the Site Plan and Geologic Map **plate 2**. The locations of the borings were estimated in the field by pacing and measuring from the limits of existing site features.

The soils encountered in the borings were visually classified in the field and a log of each boring was recorded. The logs of the borings are presented on **Plates 11 through 13.**

## Surface Conditions

A cut slope extends about 200 feet along a North-South direction in the western corner of the site (**plate 2**). Berryessa shale (Kbs) overlain by colluvium (old landslide debris) is exposed in the cut slope.

## Subsurface Conditions

The near-surface materials encountered in the borings at the site are described as relatively uniform - beneath top soils, upper clay and intermediate silty clay strata are underlain by highly weathered but hard shale and sandstone bedrock. Details concerning the boring logs are shown on **Plates 11 through 13.**

Free ground water was not encountered in the borings to the maximum depth drilled of **30 feet.** The Geologic Cross sections presented on **Plate 4** depict an interpretation of subsurface conditions across the property.

# FAULTING AND SEISMICITY

## Local And Regional Faulting

Our site reconnaissance did not reveal geomorphic features indicating the presence of active faults. The site is not located within a State of California Earthquake Fault Zone and no mapped active or potentially active fault traces are known to transverse the site (**plate 10**).

The Calaveras fault lies about 2.7 miles east of the site. The Hayward fault lies about 7.3 miles northeast of the site, and the San Andreas lies about 29 miles from the site. In addition to the Hayward, Calaveras, and San Andreas faults, there are other active or potentially active faults in the region that could cause severe shaking at the site in the event of an earthquake. **Table 1** lists significant faults within about 50 miles of the site, which are considered by the California Geologic Survey and the UBC to be active or potentially active seismogenic sources, and gives selected seismic parameters. The closest map distances from the site to the surface expression of these faults, presented in **Table 1**, are based on a modified database from Blake (2000). Seismic shaking at the site in the event of earthquakes on the listed faults are based on attenuation curves developed by Campbell & Bozorgnia 1994/1997 for hard rock. The locations of the representative 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 Geologic Map.

**Table 1: Significant Faults within 50 Miles of the Project Site**

| Fault Name | Map Distance to the fault Mi. (km) | Magnitude of Maximum Earthquake (MCE) | Peak Accelerati on (g) | Modified Mercalli Intensity Scale * |
|---|---|---|---|---|
| Calaveras (So. of Calaveras Res) | 2.7 (4.5) | 6.2 | 0.411 | X |
| Hayward (Total Length) | 4.5 (7.3) | 7.1 | 0.428 | X |
| Monte Vista - Shannon | 14.2 (22.8) | 6.8 | 0.158 | VIII |
| Greenville | 16.6 (26.7) | 6.9 | 0.122 | VII |
| San Andreas (1906) | 17.8 (28.7) | 7.9 | 0.215 | VIII |
| San Andreas (Peninsula) | 17.8 (28.7) | 7.1 | 0.129 | VIII |
| San Andreas (Santa Cruz Mtn.) | 18.1 (29.2) | 7.0 | 0.117 | VII |
| Sargent | 18.7 (30.1) | 6.8 | 0.097 | VII |
| Zayante-Vergeles | 22.6 (36.3) | 6.8 | 0.074 | VII |
| Great Valley | 22.9 (36.8) | 6.7 | 0.073 | VII |
| Hayward (North) | 30.0 (48.2) | 6.9 | 0.054 | VI |
| Ortigalita | 30.0 (48.3) | 6.9 | 0.054 | VI |
| San Andreas (Pajaro) | 30.4 (49.0) | 6.8 | 0.048 | VI |
| San Gregorio | 32.2 (51.9) | 7.3 | 0.068 | VI |
| Concord - Green Valley | 36.0 (57.9) | 6.9 | 0.041 | V |
| Monterey Bay - Tularcitos | 36.5 (58.7) | 7.1 | 0.047 | VI |
| Quien Sabe | 40.0 (64.4) | 6.4 | 0.023 | IV |
| Palo Colorado - Sur | 44.7 (71.9) | 7.0 | 0.032 | V |
| Great Valley 5 | 45.6 (73.4) | 6.5 | 0.021 | IV |
| Great Valley 9 | 46.4 (74.6) | 6.6 | 0.022 | IV |
| Rinconada | 49.2 (79.2) | 7.3 | 0.036 | V |

* Modified Mercalli Scale, Plate 9

## 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 the 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) 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 as well and was of an estimated magnitude M6.2.

In terms of measured seismic shaking, the **1989** Loma Prieta earthquake has provided relevant seismogenic information. California Strong Motion Instrumentation Program (CSMIP) stations in the Santa Cruz area recorded free-field horizontal peak ground accelerations of 0.47g and 0.54g (Thiel, 1990) during the Loma Prieta event.

## Site Soil Profile Type

The characteristics of the soils underlying the site were used to evaluate site specific seismic design criteria. Based on the results of our field investigation and data review, the site is underlain by weathered sandstone. From a seismic-shaking perspective, sandstone materials will control the soil `characteristics. Considering the above evaluation of the site, the soil profile may be classified as **Soil Class Type C (soft rock)** in accordance with the Table 16A-J of the 2001 UBC. Soil Profile Type $S_C$ is defined as soft rock with shear wave velocities between 1.200 feet/sec and 2,500 feet/sec, SPT-N greater than 50.

| Mapped Spectral Acceleration Values | Period (seconds) | Sa (g) |
|---|---|---|
| Ss, Site Class C | .2 | 1.01 Minimum |
| S1, Site Class C | 1.0 | 0.52 Minimum |

| Soil Profile Type | Description | Near Source Factor $N_a$ | Near Source Factor $N_v$ |
|---|---|---|---|
| $S_C$ | Soft Rock | 1.5 | 2.0 |

## GEOLOGIC CONCLUSIONS

### General

A discussion of specific geologic hazards that could impact the site is provided below. The hazards considered include: surface fault rupture; seismic shaking; liquefaction, dynamic compaction; landsliding and earth flows, seismically induced ground failures, expansive soils, soil creep, and flooding.

Hillside development is by its nature higher risk than flatland development. These additional risks should be considered and weighed by the property owner and developer prior to development. We understand that the critical geologic concerns at the site include the possible presence of landslide(s).

### Surface Fault Rupture

The site is not within an Alquist-Priolo Earthquake Fault Zone (Earthquake Fault Zone Map, **Plate 10**) and, based upon the reviewed geologic and seismologic reports and maps, no known active or potentially active faults cross or project toward the site. Additionally, during our site reconnaissance no evidence of active faulting was observed on or projecting toward the site. **Therefore, it is our professional opinion that the potential for fault-related surface rupture at the site is very low.**

## SEISMIC SHAKING

### Near Fault Issues

In recent years, many 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 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.

**Due to potential near-fault motion resulting from the Calaveras, Hayward and other nearby faults, near-source effects should be considered in the structural design of buildings in the project area. Structures with strength discontinuities, soft stories, plan irregularities, discontinuous shear walls and ductile-moment frames are particularly vulnerable to these type of motions and should either be avoided or properly designed and constructed.**

The site is located about 4.5 miles from a Type A fault (Hayward) and about 2.7 miles from a Type B fault (Calaveras).

## SEISMICALLY INDUCED GROUND FAILURE

### 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 induced by earthquakes. In the process, if the soil mass is not confined the soil acquires mobility sufficient to permit both horizontal and vertical movements. Soils most susceptible to liquefaction are Holocene age, saturated, loose, clean, uniformly graded, and fine-grain sand deposits.

No accounts of liquefaction were recorded at the site during the 1989 Loma Prieta earthquake or the 1906 "Great" San Francisco earthquake (Holzer, 1998 and Lawson, 1908). A review of the boring logs indicates that the depth to bedrock over the site is approximately 12 feet. Near surface materials above bedrock consist of clay and silty clay, which should not be subject to liquefaction. The absence of ground water in the soil overlying bedrock observed in the borings further suggests that the property will not be subject to liquefaction. **We conclude therefore, that liquefaction is not considered a geologic hazard at this site.**

## 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 proximity of bedrock to the ground surface, dynamic compaction is not considered a hazard to the project site.**

# SLOPE STABILITY ASSESSMENT

## General

The California Geological Survey (CGS) Seismic Hazard Zone Report for the Hayward quadrangle (CGS 2003) (**Plate 3**) indicates that the slopes may be subject to landsliding in the event of a large earthquake on nearby faults. During the field reconnaissance by our engineering geologist, this slope was examined for geomorphic features that would indicate the presence or absence of downslope movement. Subtle features indicative of an older landslide were observed. **No ground cracks suggesting recent landsliding were observed at the**

**site. Shallow soil creep indicated by bent and leaning trees was observed on the slopes east of the existing residences.**

## Slope Stability Evaluation

Special Publication 117 (CGS, 1997) outlines a procedure for screening specific sites with respect to slope stability. The publication states:

> *"The purpose of screening investigations for sites within zones of required investigation is to evaluate the severity of potential seismic hazards, or to screen out sites included in these zones that have a low potential for seismic hazards. If a screening investigation can clearly demonstrate the absence of seismic hazards at a project site, and if the lead agency technical reviewer concurs with this finding, the screening investigation will satisfy the site-investigation report requirement and no further investigation will be required.*
>
> *The simplest approach to a dynamic slope stability calculation is the pseudo-static analysis, in which the earthquake load is simulated by an "equivalent" static horizontal acceleration acting on the mass of the landslide, in a limit-equilibrium analysis (Nash, 1987; Janbu, 1973; Bromhead, 1986; Chowdhury, 1978; Morgenstern and Sangrey, 1978; Hunt, 1984b; Duncan, 1996). The pseudo-static approach has certain limitations (Cotecchia, 1987; Kramer, 1996), but this methodology is considered to be generally conservative, and is the one most often used in current practice".*

**Plate 2**, Site plan shows the transect A-A' used for our slope stability evaluations. **Plate 4**, Cross Section depicts the slope profile along the line A-A' that was used in the slope analysis. The slope evaluations were performed as much as possible along a line perpendicular to the contours of the property and through the planned building location.

## Model Description

To assess potential failure mechanisms of the slope at the proposed building site, we performed a slope-stability evaluation using GSLOPE. GSLOPE & GSTABL7 are computer programs that are used to perform two-dimensional, limit equilibrium analyses to compute the factor of safety for slopes. The programs can be used to search for most critical surfaces or the factor of safety may be calculated for a specified surface. Based on the soil and rock types present at the site, a circular failure evaluation was used in our model. The slope configuration at the site used in the stability evaluation was derived from the **Cross Section, Plate 4.**

## Ground-Water Conditions

Ground water was not encountered in the borings near the proposed residential sites to a depth of 30 feet. For a conservative analysis of the slope, **we assumed groundwater to be about 20 feet deep near the west property line and generally to follow to about 140 feet below ground surface near the crest of the ridge at the eastern portion of the site.**

## Rock Strength Parameters

Model strength parameters for the conglomerate and shale were derived from laboratory test data for soils. Because the greatest concern in stability analyses would be a failure within the rock and because soils layers encountered were relatively thin, soils were not considered in the initial stability analyses. The rock strength parameters that were used in our slope-stability model are presented in the table below:

| | Unit weight (pcf) | Cohesion ($lb/ft^2$) | $\phi$ (deg.) |
|---|---|---|---|
| Kbc, Conglomerate | 130 | 1500 | 45 |
| Kbs, Shale | 125 | 1300 | 39 |

## Soil Strength Parameters

Model strength parameters for the colluvium and shale were derived from laboratory test data for soils. The depths of soils in stability analyses were based upon soil conditions in boring 11. The soil strength parameters that were used in our slope-stability model are presented in the table below:

| | Unit weight (pcf) | Cohesion ($lb/ft^2$) | $\phi$ (deg.) |
|---|---|---|---|
| Clay colluvium (0-8') | 120 | 300 | 32 |
| Silty clay (8-12') | 125 | 500 | 35 |
| Soft rock (12-26') | 130 | 900 | 39 |
| Nard rock (>26') | 135 | 1500 | 45 |

## Seismic Coefficient

The Screening Procedure described in Southern California Earthquake Center (SCEC, 2002) was also used to evaluate potential landslide failure. The procedures for analyzing landslide by the SCEC (2002) were used to input a horizontal seismic coefficient for our analysis. The SCEC method uses the following relationship to determine the seismic coefficient ($k_{eq}$):

$$k_{eq} = f_{eq} \times (MHA_r / g)$$

where ($MHA_r$ / $g$) is the maximum horizontal acceleration at the site for a rock-site condition, and $f_{eq}$ is a factor related to the seismicity based on the magnitude of the controlling fault and distance from the site.

- From CGS (2004), ($MHA_r$ / $g$) ≈ 0.68
- From SCEC (2002), $f_{eq}$ = 0.5 (for distance to the seismic source less than 10 km, and M = 7)

$k_{eq} = f_{eq} \times (MHA_r / g)$ = 0.5 x 0.687 = 0.34g

## Results Of The Stability Analysis Of The Middle Portion Of The Slope

The results of our stability evaluation for the slope passing through the entire parcel is graphically shown on **Plates 5 & 6**, Slope Stability with Pseudo-static Load, for lower and upper slopes, respectively. The most critical failure surface was evaluated with a pseudo-static seismic coefficient of 0.34g acting in the horizontal direction out of the slope. As shown on **Plates 5 & 6**. The failure surface shown on **plate 5** (lower portion of the slope, has a Factor of Safety (FOS) of **1.73 ***). **This indicates that a probable lower FOS may exist higher on the slope. Plate 6 shows that the failure surface for a higher portion of the slope has a FOS of 1.27*.**

**Although the 1.27* FOS in the higher portion of the slope is at the upper limit of the solution grid pattern, it indicates that slope movement in the upper portion of the slope, if it should occur at all, would be of considerable distance away from the building sites to impact the residences.**

**The results of GEI soil analyses of the slope behind the retaining wall are presented on plate 16. A factor of safety of 1.88* (FOS) in the mid-portion of the slope indicates that slope movement, would be relatively unlikely and would not impact the residences.**

**No evidence of landsliding or slope movement was observed on the portion of the lot proposed to be developed during our site visit, in our aerial photographic analysis, nor during our subsurface explorations. Our stability evaluation of the proposed building site incorporated the following parameters.**

---

*** These FOS values include earthquake forces**

1. Conservative rock-strength parameters.
2. Slope configuration from site topography.
3. Conservative ground water conditions; and
4. A seismic coefficient of .34 g.

The result of the rock analyses give factor of safety of 1.7* for pseudo-static conditions in the vicinity of the planned residences, and 1.27* some distance away and in the upper reaches of the slope. In our professional opinion, considering the conservative parameters used in the stability model, results of this evaluation indicate this screening procedure gives an adequate margin of safety under pseudo-static conditions.

**Soil Creep**

Geotechnical reconnaissance of the site indicates that soil creep is present on the slopes adjacent to the subject site. Soil creep should be considered during design and construction of retaining structures and foundations.

**Flood Hazard**

Flood hazards are generally considered from three sources, which include seismically induced waves (tsunami or seiche), reservoir failure, and long-cycle storm events. The site is located at an elevation of at least 750 feet above MSLD. Based on the site's elevation, we judge that the potential for a seismically induced wave to impact the site is negligible.

With respect to reservoir failure flooding, there are **no large reservoirs or tanks** upgradient of the project site. In our opinion, the site is not likely to be inundated by flooding from an upstream dam break.

---

* These FOS values include earthquake forces

With respect to storm-food events, FEMA (1984) indicates that the site is not within an area affected by a 100-year flood or 500-year flood.

### Erosion

Given the steepness of the slopes in the area and the observation that bedrock is highly weathered and relatively soft, **we recommend that the plans for the proposed development of the property include that all surface water be collected in closed pipes and directed to the cul-de-sac and not be discharged on the slopes.**

### Asbestos Containing Minerals

No serpentinite or other rocks that ordinarily contain asbestos minerals occur on the site. **Therefore, we do not consider asbestos to be a hazard at this site.**

## SOIL ENGINEERING

The results of the field and laboratory testing are described herein. The recommendations for site preparation, compaction, drainage, and foundations are presented below.

### Field Explorations

As stated, subsurface conditions at the site were explored on April 30, 2008 by drilling and sampling a total of 3 borings at the locations shown on **plate 2**. Due to sloping site, the borings were drilled using track mounted drilling equipment to depths of from 15.5 to 30 feet. All 3 borings extended through upper soil strata and into hard rock. The boring logs are presented on **plates 11 through 13**. The overburden soils are classified according to the Unified Soil Classification System.

The field program was directed by our soil engineer and engineering geologist, who maintained continuous logs of the materials encountered, obtained frequent samples, and recorded blow count data.

## Laboratory Testing

A series of laboratory tests were performed to evaluate the pertinent physical properties of the materials encountered. The laboratory-testing program included **moisture content, dry density, Atterberg limits, direct shear and compaction tests.**

The moisture-density results and Atterberg limits (Liquid Limit = LL, Plastic Limit = PL, Plasticity Index = PI) are presented on **plates 11 through 13.** The direct shear test data are summarized on **plate 14.** The results of the compaction test are presented on **plate 15.**

The laboratory testing was performed in accordance with the procedures of the American Society for Testing and Materials.

# SITE CONDITIONS

## Subsurface Conditions

Based upon our review of the boring logs, examination of the samples, experience in the vicinity, and laboratory test data, the subsurface conditions appear to be relatively uniform. **Beneath top soils, clay and silty clay (colluvium) are underlain by shale and sandstone rock.**

Beneath about 6 inches of top soils, generally about 7 feet of very plastic (highly expansive) upper clay colluvium was encountered in all test borings. **The upper medium stiff clay stratum, which is very plastic (Plasticity Index of 29), would be highly expansive, swelling with increasing moisture contents.**

From 7 to 12 feet deep, moderately plastic (medium expansive) intermediate silty clay soils (colluvium) was encountered in borings 11 & 13. **The stiff**

**intermediate silty clay stratum, which is moderately plastic (Plasticity Index of 20), would be moderately expansive, swelling somewhat with increasing moisture contents.**

Below from 7 to 12 feet deep, moderately hard shale and sandstone was encountered in all borings. The underlying rock grades harder with increasing depth; hard rock was generally encountered below from 13 to 28 feet deep. As stated, the borings generally extended to slow drilling and were terminated in hard rock.

**Detailed descriptions of the materials encountered are presented on the boring logs, plates 11 through 13.**

A profiles and interpretative cross-section line A – A' is shown on the Subsurface Cross Section (**Plate 4**).

As stated, free ground water was not encountered in the test borings; the maximum depth explored was **30 feet** below ground surface.

**Seismicity And Structural Design of Buildings**

In accordance with recent geologic maps of the vicinity, the property is located **outside** a State of California Special Studies Zone **(active Hayward fault)**. Because of the proximity of the site to the **active Hayward fault**, and other active Bay Area faults, depending upon the intensity and magnitude of earthquakes, the residences will probably experience "**very strong**" shaking during the project life. Therefore, it is recommended that the houses be **appropriately reinforced by a structural engineer and at least in accordance with the applicable Seismic Code to resist earthquakes.** This however, does not guarantee or insure that the residences will not sustain structural damage in the event of future earthquakes. Some residences constructed under the provisions of recent building codes suffered significant damage during the October, 1989 earthquake on the San Andreas fault.

## RECOMMENDATIONS

Based upon the results of the field explorations, geologic evaluation, laboratory testing, and engineering analyses, it is our professional opinion that merely from a geotechnical standpoint the site would be suitable for development of the planned residences.

- **Because of sloping site and presence of existing upper colluvial deposits, GEI recommends supporting the new residences and any retaining walls using relatively deep pier and grade beam foundations.**
- **Conventional footings are not recommended for use at the planned retaining wall site.**
- **Slope (landslide) repairs (to stabilize the hillside) should consist of a keyway with subdrains which should be extended sufficiently into underlying competent rock. All grading should be controlled and approved by GEI to ensure proper construction.**

Recommendations for site preparation, slope repair, excavating, compaction, drainage, foundations, slabs, retaining walls, paving, etc. are presented herein.

## SITE PREPARATION

### Stripping

All existing vegetation and any debris, etc. should be removed from the area of the new residences prior to construction. At the time of our field explorations, we estimated that a stripping depth of approximately 6 inches would be appropriate. The actual stripping depth should be determined in the field by the soil engineer at the time of construction.

## Note of Caution

**Prior to any excavations, any existing nearby utility lines and any other buried structures should be clearly marked for safety and in order to avoid any mishap.**

## Excavations

Any **temporary** excavations (less than four feet deep) that are constructed **during the dry season** may be constructed using vertical slopes. Temporary cuts or any excavation deeper than four feet should be sloped back at 0.5 (horizontal) : 1 (vertical) or be properly **shored** for safety.

Any permanent cut and fill slopes should be relatively gentle. Provided the drainage and grading recommendations presented herein are properly implemented, **we recommend using a slope ratio of 2 (horizontal): 1 (vertical) for all permanent cuts and properly keyed in & compacted fill slopes.**

**The grading contractor should be alerted to possibility of encountering hard rock and difficult digging, so that he could plan appropriate contingencies.**

**All excavations for future utility trenches beneath slab and paving areas should be properly backfilled in accordance with the compaction criteria of this report.**

## Fill Materials

Because the near surface soils are very expansive, they would be relatively lifficult to use for engineered fill beneath building areas. Any required mported fill materials used in backfilling should consist of non-expansive soils ith a Plasticity Index of less than 15 percent. The project soil engineer should pprove the suitability of any imported fill materials prior to placing fill.

Keyway Construction

After removing any fill and all colluvial soils up to 12 feet deep, the exposed subgrade in these fill areas should be benched so that all fill can be placed and compacted in horizontal lifts, properly keyed at least 5 feet into the underlying competent rock, sloping about one percent into the hillside, and properly compacted.

The compacted fill should be stepped into the undisturbed natural materials, benches or "notches" should be excavated at least 5 feet into rock at approximately 10 feet vertical intervals (maximum interval of 15 feet), and compacted fill placed in the keyway and notches.

A properly constructed subdrain consisting of 4-inch perforated pipe wrapped in ¾-inch drain rock & filter fabric or 4 inch EZflow should be installed in all keyways.

Subgrade Preparation Beneath Garage & Paving – After removal of top soils, the garage excavations and all paved areas should be scarified at least **18 inches deep**, moisture conditioned to about 2 percent above optimum moisture content, and properly compacted. The subgrade beneath paving areas should be similarly prepared.

All grading, drainage and compaction work should be performed under the supervision of GEI to enable proper construction.

Compaction Criteria

All required fill and backfill in fill areas should be placed in uniform lifts not exceeding 8 inches in loose thickness, conditioned to approximately 2 percent above optimum moisture, and compacted to a dry density of **at least 90 percent** of the maximum dry density determined using the American Society for Testing and Materials Designation: D1557-78 (modified Proctor) method. The upper

**8 inches** of fill as well as all required imported fill and base materials beneath paved areas should be compacted to **at least 95 percent** relative compaction.

## Drainage

We recommend that final grades be selected so that a gentle slope is provided to divert all surface water away from the residences, slabs and paving. The surface water runoff including all downspouts should be **securely** connected to closed pipes leading to the base of the slopes in the ravines southwest of the development area. **At no time should water be allowed to pond adjacent to foundations, slabs and paving.**

**Surface Drain** – **Concrete lined** surface drains or v-ditches should be constructed upslope (east) of the new residences to minimize runoff from flowing into the crawl space and over slopes.

# FOUNDATIONS

## Drilled Piers

As stated, because of the sloping site and presence of colluvium, the planned residences and any retaining walls should be supported on properly designed and constructed drilled concrete pier and grade beam foundations. The piers which should be properly designed and reinforced by your structural engineer should be carried **at least 18 to 22 feet** below existing grade (at least **10 feet** into underlying competent rock). **The drilling contractor should be alerted to the probability of encountering hard rock and/ water seepage so that he could plan appropriate contingencies.** The piers, which should be **24-inches** in diameter, should be carried to appropriate depths depending upon structural loads, spacing, and pier diameter as can be determined by your structural engineer. The structural engineer is also responsible for determining the amount, size and location of the reinforcing. **GEI recommends using an allowable friction of 1,000 pounds per square foot for design of piers in underlying competent rock. The friction of existing soils**

**should be ignored.** The recommended unit friction may be increased by one-third for resistance to wind and earthquake loads. **An allowable friction of 600 pounds per square foot may be used against uplift.**

**Because of the proximity of the site to the nearby active Hayward fault, all piers should be tied together with grade beams to act as a unit in resisting lateral loads (UBC 1997, Section 1807.2). The steel should be bent into the grade beams to achieve transfer of moment stresses. We recommend that all exterior grade beams extend at least 8 inches below the final exterior rough grade.**

We recommend that all exterior grade beams extend at least **8 inches below the final exterior rough grade, and be underlain by at least 2 inches of card board void forms. To this end, all exterior grade beams should be designed in such a way to span unsupported between piers.**

## Group Action

We anticipate that pier spacing will exceed three pier diameters. Because of the relatively wide pier spacing, there will be no reduction in pier capacity due to group action. In the event that pier spacing less than recommended herein is considered, we should be contacted in order to provide appropriate reduction in pier capacity.

## Estimated Settlements - Residences & Compacted Fill

The estimated total settlements of foundations designed as recommended herein are expected to be moderate. We anticipate that the maximum total and differential settlements of the residences constructed according to the recommendations presented herein will be on the order of 0.75 and 0.375 inch, respectively.

Provided the grading and compaction in keyways, structural and paving areas have been performed in accordance with the recommendations of this report, it is fessional opinion that the amount of settlement of properly compacted fill be relatively small and on the order of 0.5 inch.

## al Loads

Lateral forces resulting from wind, seismic and active earth pressures may be ed by passive earth pressure and by friction between foundation concrete and supporting subgrade. An allowable coefficient of friction of **0.3** may be used the concrete and subgrade. For design purposes, the passive pressures of ent rock may be taken as equal to the pressure developed by a fluid having ty of **450** pounds per cubic foot. **The passive pressure of existing lls should be ignored.** The passive pressure may be **doubled** for use with a formula. A combination of both friction and passive pressure may be used ided that one of the values is reduced by 50 percent.

## lls Below Grade

Depending upon the slope of the backfill materials, the criteria for design f walls below grade should include the following to resist active earth essures.

| Slope of Backfill Materials | Active Earth Pressure *(Unrestrained Walls))* | Active Earth Pressure *(Restrained Walls)* |
|---|---|---|
| Level Backfill (up to 5:1 slope) | 35 | 45 |
| 3:1 (horizontal: vertical) slope | 45 | 55 |
| 2:1 (horizontal: vertical) slope | 65 | 75 |

Where surcharge loads may act above walls below grade, an **additional pressure equal to one-third to one-half of the maximum anticipated surcharge load should be applied to the surface behind unrestrained walls and restrained walls, respectively.** All walls below grade should be properly waterproofed and provisions for positive drainage (weep holes, subdrain, Miradrain, Amerdrain, Sure-drain V, EZflow, etc) should be provided, as appropriate.

In the **subdrain** trench, which should be at least one foot wide, a continuous minimum 4 inch diameter perforated plastic pipe "bedded" on a minimum of **6** inches of drain rock wrapped in filter fabric, Hydraway Drain 2000 water collection system, EZflow, etc. should be placed at the bottom of the trench. The balance of the trench may be backfilled using ¾ inch drain rock material up to within approximately 12 inches of final grade.

**Paving Design**

The subgrade beneath paving areas should be properly scarified and compacted at least 12 inches deep and properly compacted. Based upon the soil classification and using an appropriate R-value (based upon experience), a traffic index of 5.0 and using Cal Trans procedure 301, the recommended paving design section for the planned paved cul-de-sac would be as tabulated below:

**Asphalt Paving Design**
(Light / Medium Traffic)

| Asphaltic Concrete (Inches) | Good Quality Class II* Aggregate Base (Inches) | Total Paving Thickness (Inches) |
|---|---|---|
| 3 | 9.5 | 12.5 |

- Not recycled materials

**Slabs On Grade**

**Because of the sloping ground, GEI recommends using structurally supported raised wood floor in all living areas to avoid problem with wet carpet and other damages.**

We recommend that all **garage slabs on grade** should be properly reinforced by **No. 4 rebar 12 inches on center**, running both ways by your structural engineer and be underlain by a layer of granular base. The base materials should consist of clean, **free draining** crushed rock or drain rock. After the subgrade has been prepared in accordance with the site preparation recommendations of this report, at least **10 inches** of drain rock or properly compacted crushed rock should be placed beneath any slabs. The sand and gravel should be covered with **Vapor Block 10 membrane, (Raven 800-635-3456), Moistop Under Slab (800-773-4777) or Stego Wrap (949-493-5460)** to act as a vapor barrier in order to prevent condensation beneath interior garage slabs. The membrane should be covered, in turn, with at least **2 inches of sand** for protection during construction. The subgrade should be **moistened overnight** before placing impermeable membrane and pouring concrete.

The garage slabs should not be connected to foundations and should be allowed to float.

## Inspection

**Grading and foundation plans and any other geotechnically related plans should be reviewed and approved in writing by GEI to ensure conformance to the recommendations of this report prior to issuance of building permit.**

**All earthwork, subgrade preparation, foundation construction, compaction in structural areas and drainage should be observed, controlled and approved by GEI in writing to enable proper construction. It is the responsibility of the owner and/or his agents to implement the recommendations in this report. GEI cannot be held responsible for compliance with design recommendations for grading, foundations and drainage (surface & subdrains) controlled and approved by others.**

## Limitations

The recommendations made in this report are based on the assumption that the subsurface conditions do not deviate appreciably from those disclosed in the borings. This report does not reflect any variations, which may occur between these borings. The nature and extent of variations between the borings may not become evident until the course of construction. If during construction subsurface conditions different from those encountered in the borings are observed or appear to be present, we should be advised at once so that we can review these conditions and make appropriate changes to our recommendations. To this end, some contingency fund is recommended to accommodate these required additional expenditures to attain a properly constructed project. This report is therefore not to be construed as a guarantee or warranty, nor is it intended for the purpose of establishing a value, nor as an opinion as to the advisability of construction. No reliance on this report shall be made by anyone other than the client's name, which appears on the cover letter to this report.

The conclusions and opinions presented herein were prepared in accordance with generally accepted engineering principles and practices at the time of the investigation. This warranty is in lieu of all other warranties either expressed or implied. In the event that recommendations are made by others, these are not the responsibility of Geotechnical Engineering, Inc., unless we have been given the opportunity to review and concur in writing.

The findings of this report are valid as of the present date. However, changes in the conditions of a property can occur with the passage of time, whether they are due to natural processes or to the works of man, on this and adjacent properties. In addition, changes in applicable or appropriate standards occur, whether they result from legislation or the broadening of knowledge. Accordingly, the findings of this report may be invalidated, wholly or partially, by changes outside our control. This report should therefore be reviewed after a period of one year in the light of changes on the site, future planned construction, and the then current applicable codes.

This report has been prepared in order to assist in the project design. In the event of changes in the planned new residences and / or locations, the conclusions and recommendations shall not be considered valid unless we have been given an opportunity to review and approve or modify this report in writing.

Very truly yours,
Geotechnical Engineering, Inc.

Taghi Manbeian, Ph.D., P.E.
President

Alan S. Boris, GE 155, CE 15796
Geotechnical Engineer

Michael Clark, CEG 1264
Engineering Geologist

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Base Map: U.S. Geological Survey, 7½ min.
Calaveras Reservoir quadrangle

**GEI** Geotechnical Engineering Inc.

CONSULTANTS IN GEOLOGIC & SOIL ENGINEERING

PROJECT NO. GEI037-E

**Site Vicinity Map**

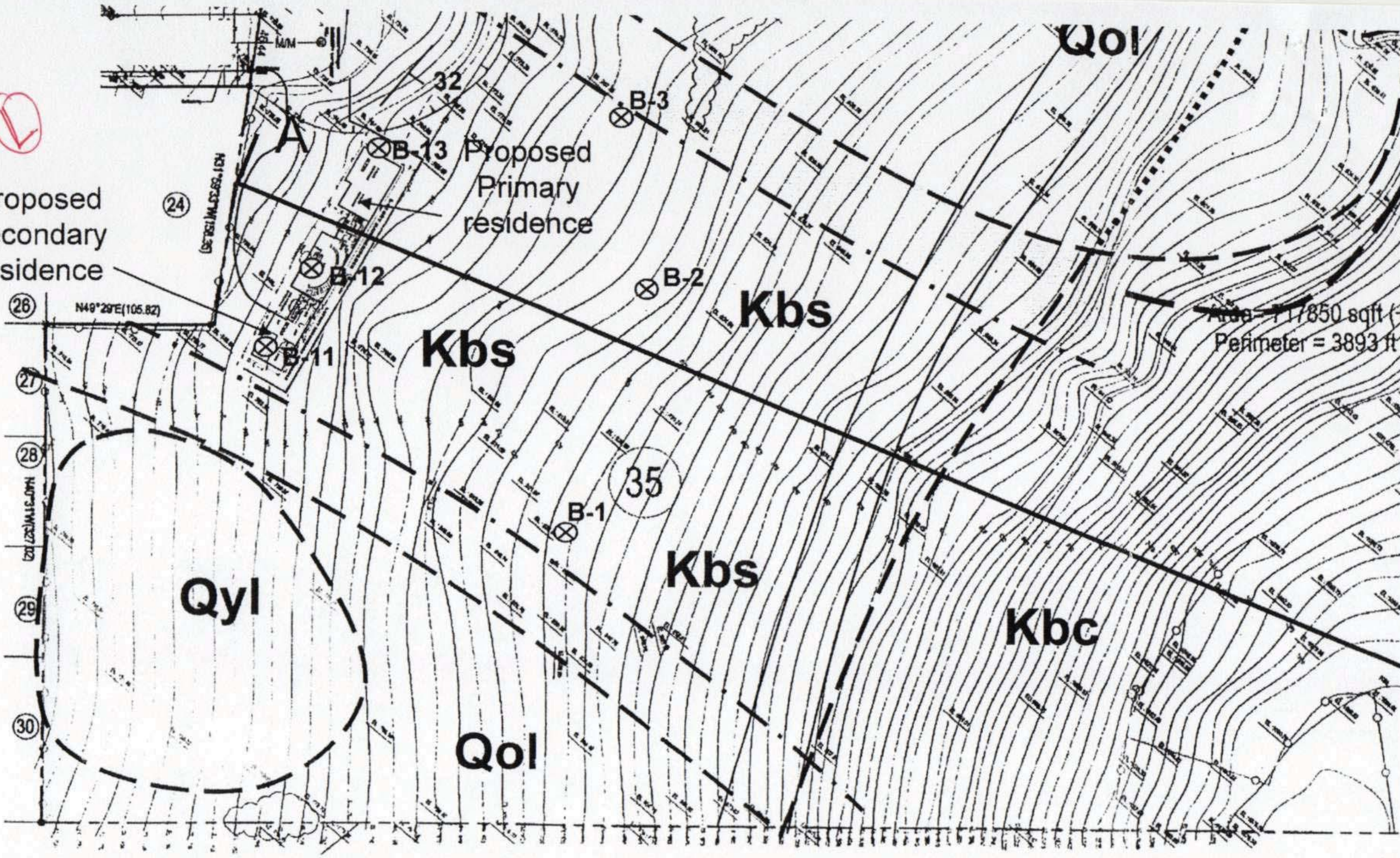
**Planned Single-Family Residence**

East of 15880 Simoni Drive, San Jose, CA

**PLATE**

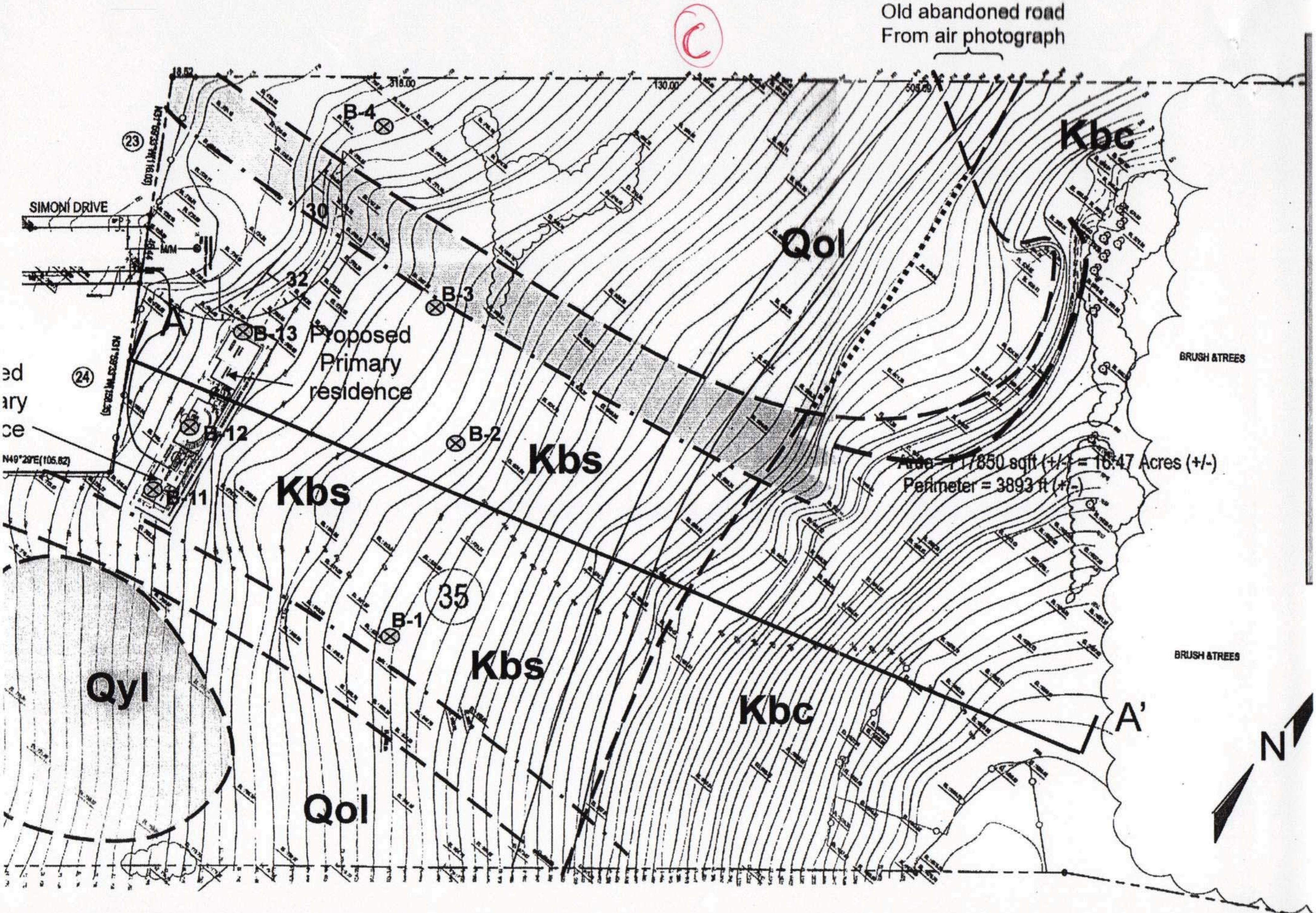
1

Plate Compiled by: AMC   Reviewed by: MNC   Original Plate in Color




Base: Modified from CT Cistran Group Inc.; Topographic Map APN# 612-10-004, not dated
Proposed residence location from client

| | | |
|---|---|---|
| PROJECT NO. GEI037-E | GEI Geotechnical Engineering Inc. | CONSULTANTS IN GEOLOGIC & SOIL ENGINEERING |
| Plate Compiled by: AMC | | |
| Reviewed by: MNC | | |
| Date: 08/28/04 | | |




: Modified from CT Cistran Group
Topographic Map APN# 612-10-004,

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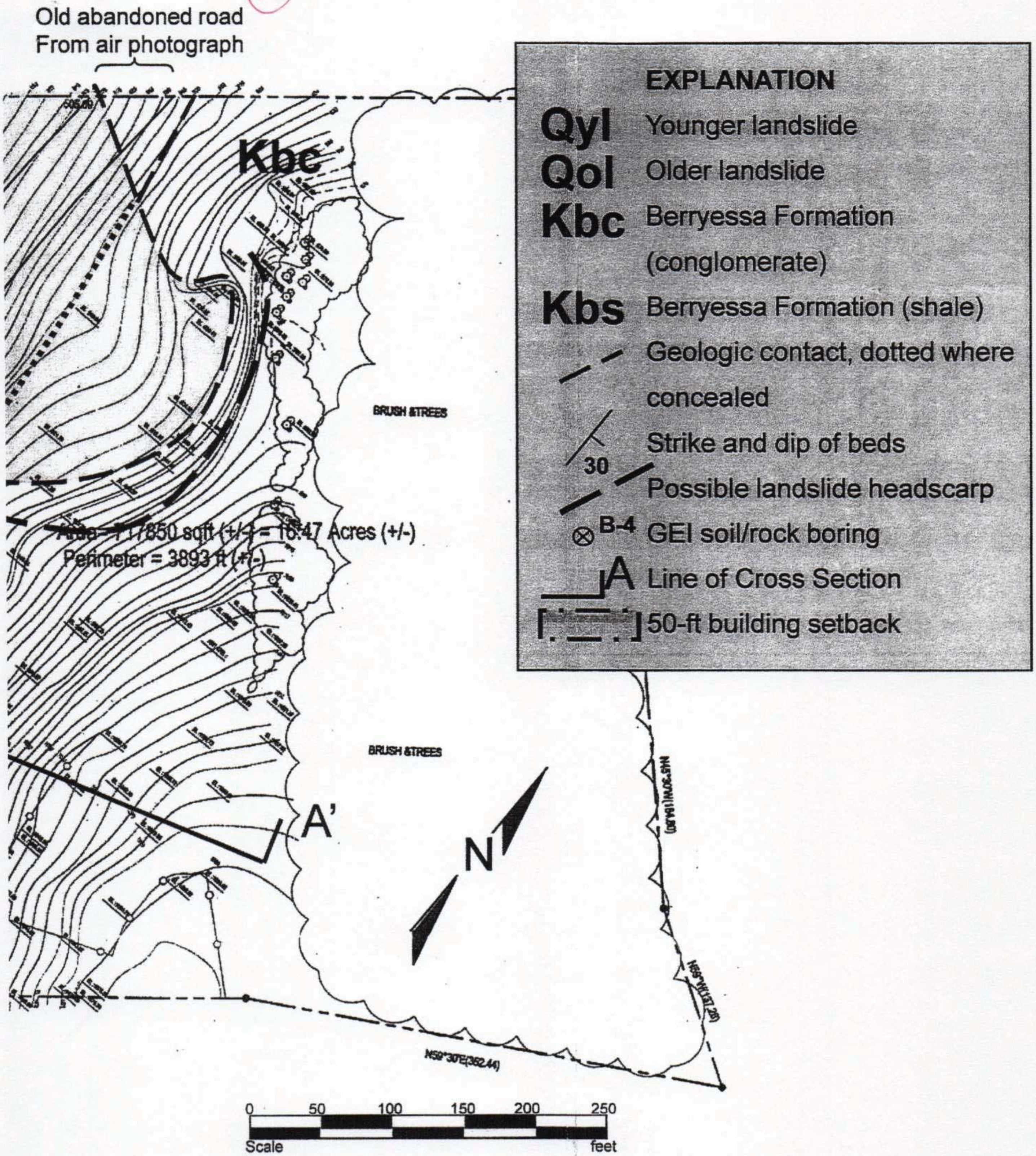
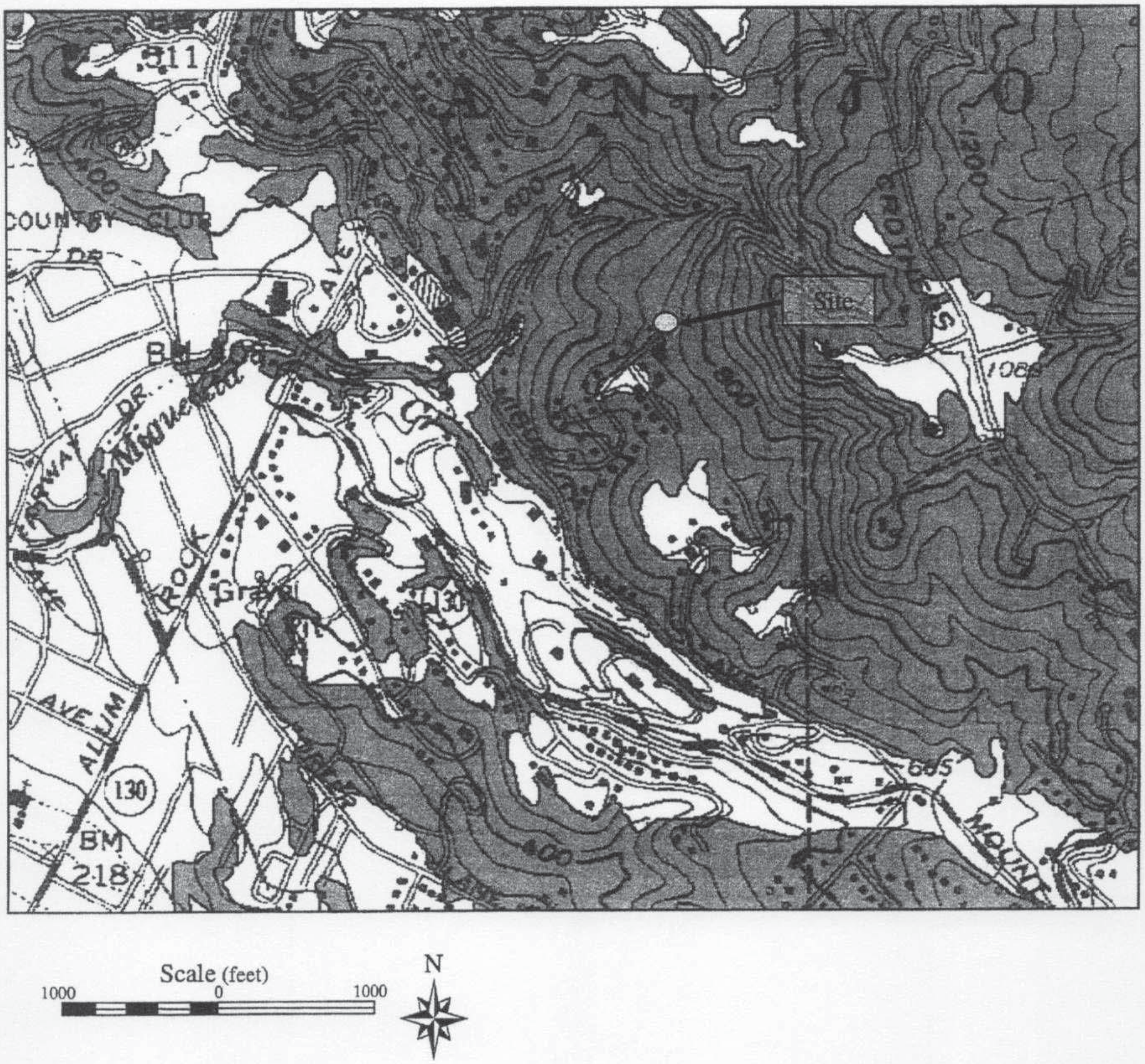



## Site Plan and Geologic Map

**Planned Single-Family Residence**
East of 15880 Simoni Drive, San Jose, CA

**PLATE**

**2**

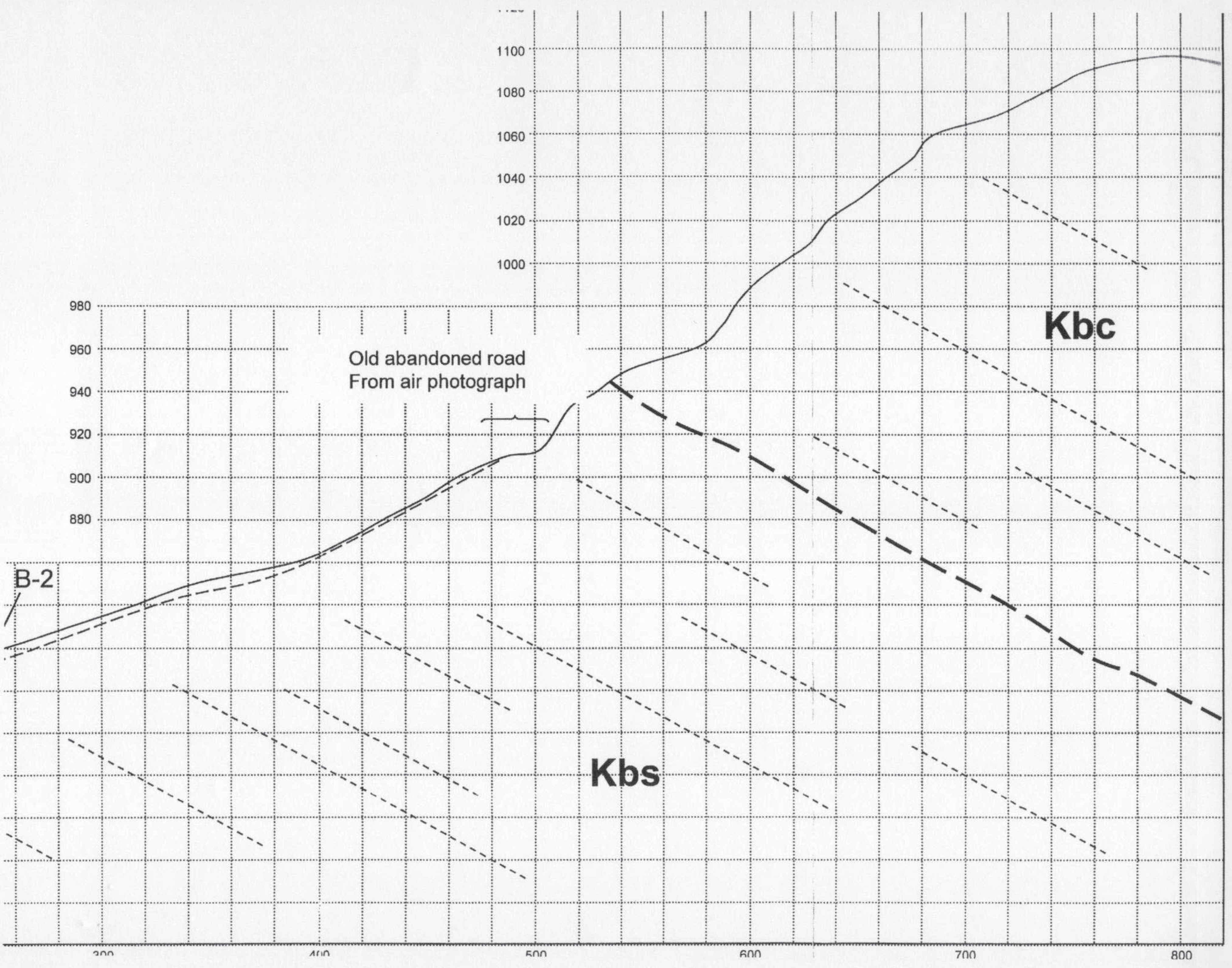



Area of potential seismically induced landslide movement

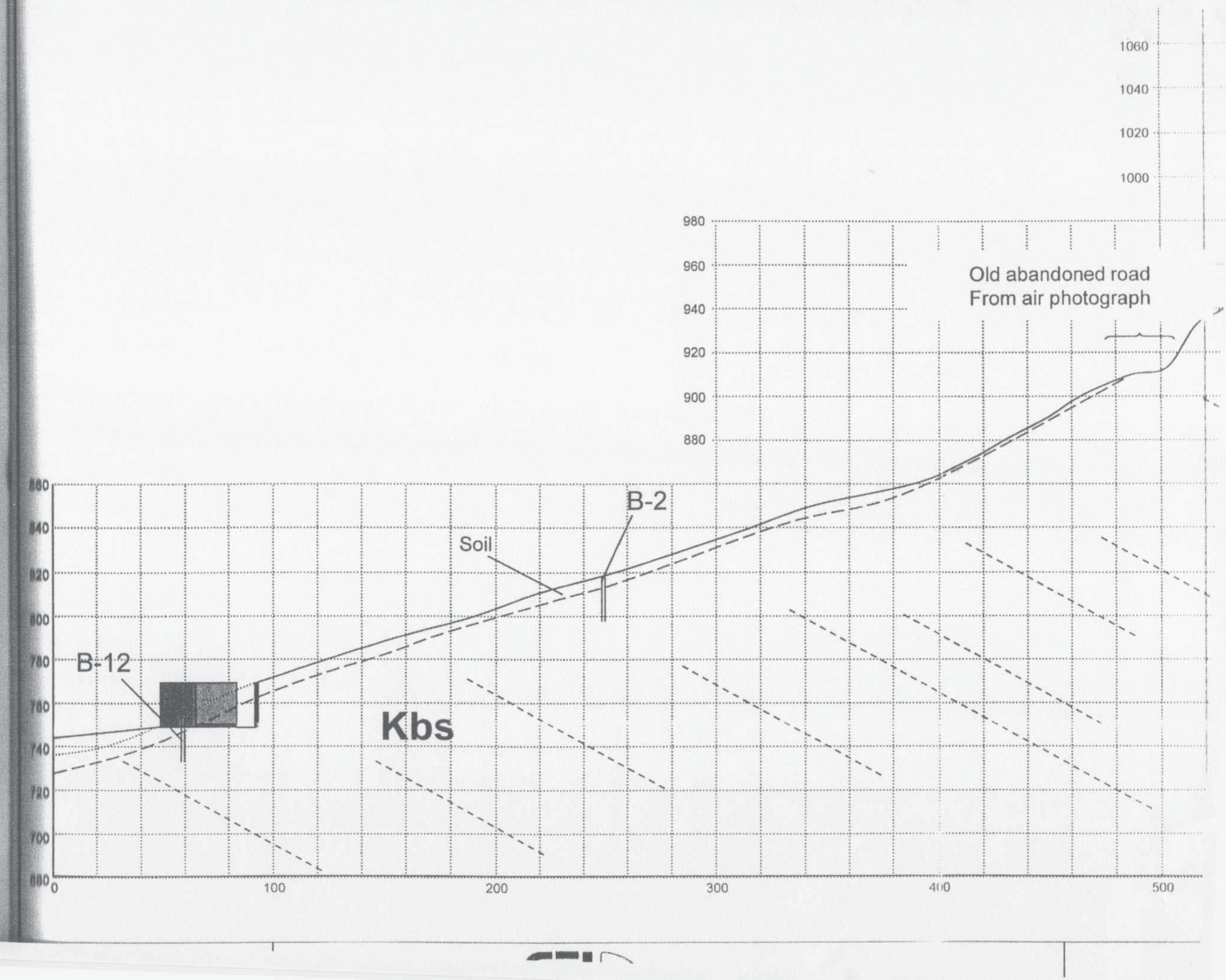
Base: California Geological Survey, Seismic Hazard Map, Calaveras Reservoir, Official Map Released: October 17, 2001

| | | |
|---|---|---|
| GEI Geotechnical Engineering Inc. CONSULTANTS IN GEOLOGIC & SOIL ENGINEERING | **Seismic Hazard Map** | **PLATE** |
| PROJECT NO. GEI037-E | **Planned Single-Family Residence**<br>East of 15880 Simoni Drive, San Jose, CA | 3 |

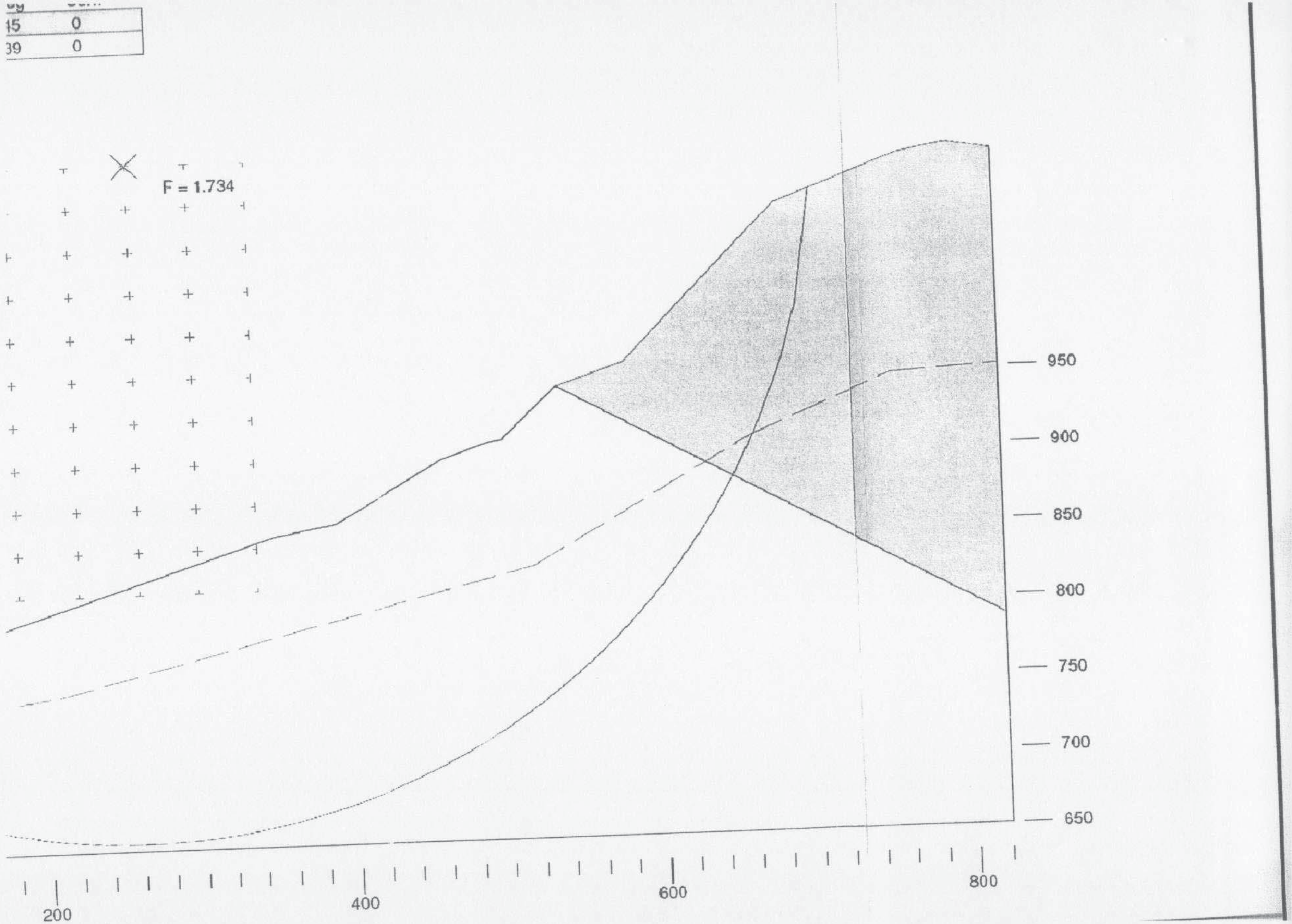
Plate Compiled by: AMC Reviewed by: MNC Original Plate in Color



1100
1080
1060
1040
1020
1000
980
960
940
920
900
880
Old abandoned road
From air photograph
Kbc
B-2
Kbs
300
400
500
600
700
800

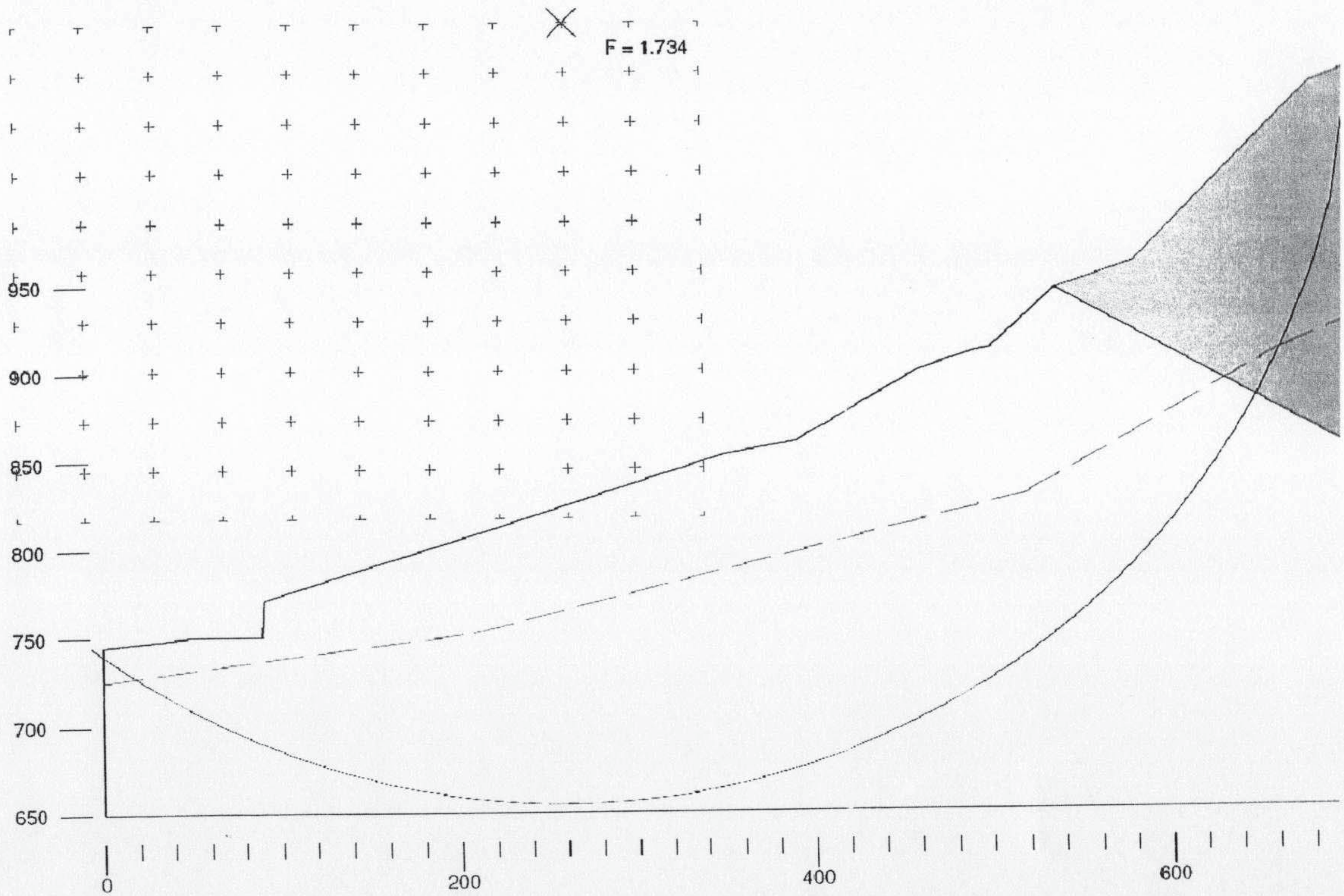


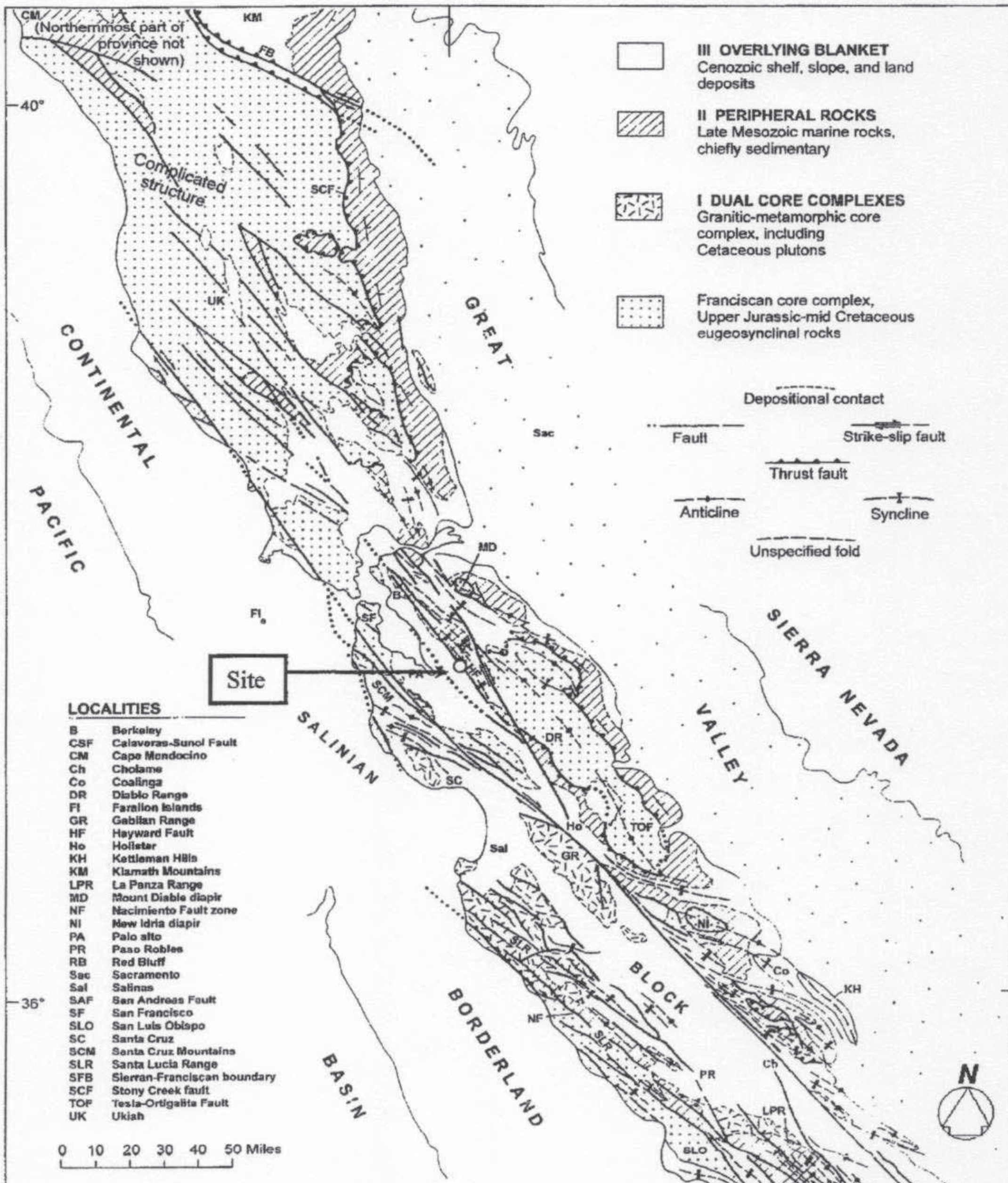
Old abandoned road
From air photograph
B-2
Soil
B-12
Kbs
1060
1040
1020
1000
980
960
940
920
900
880
860
840
820
800
780
760
740
720
700
680
0
100
200
300
400
500


15 0
39 0
F = 1.734
950
900
850
800
750
700
650
200
400
600
800

| | | | | |
|---|---|---|---|---|
| Tbc Berryessa conglo | 130 | 1500 | 45 | 0 |
| Tbs Berryessa shale | 125 | 1300 | 39 | 0 |

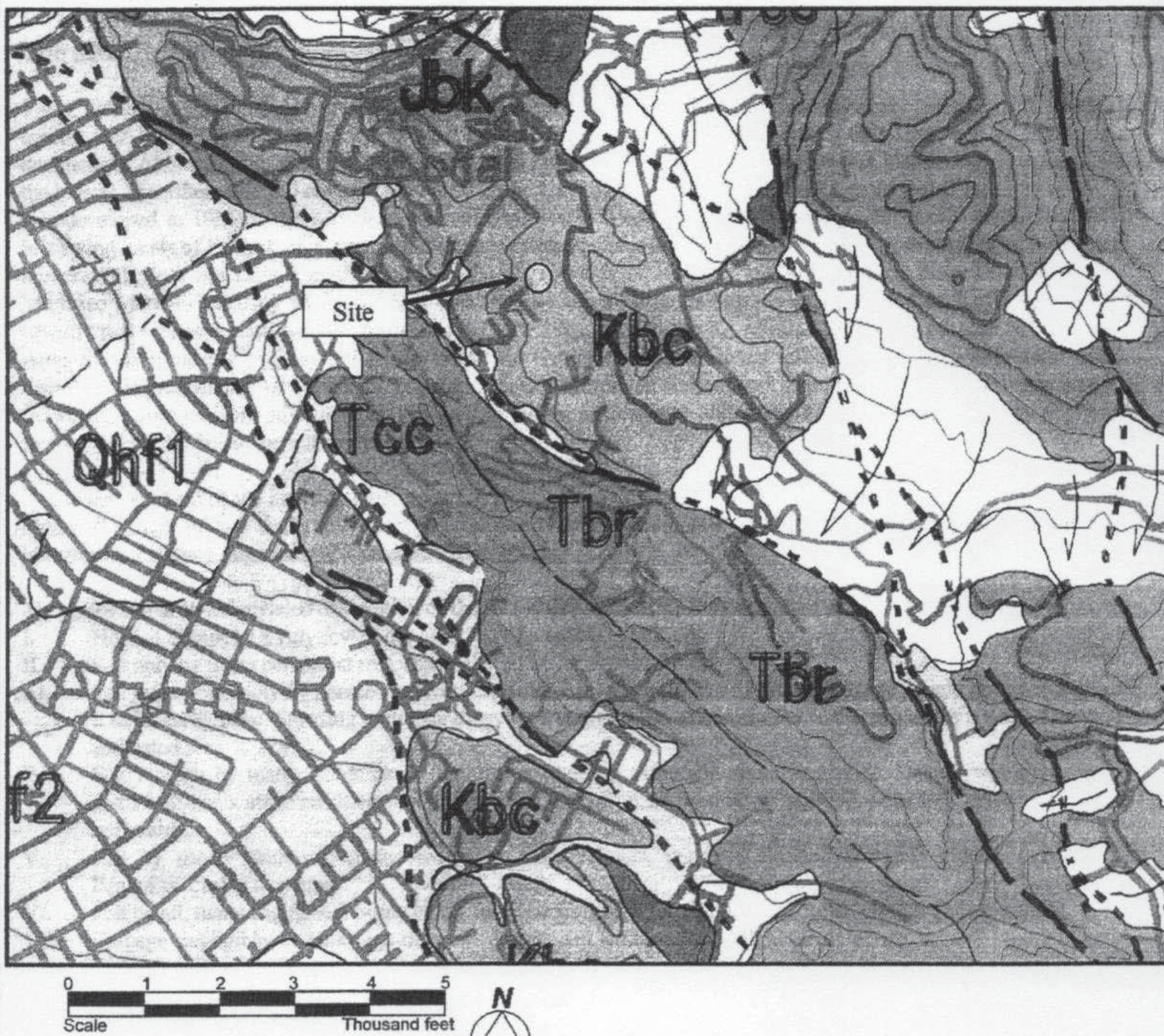
Seismic coefficient = 0.34





Compiled by Ben M. Page, 1965.

| | | |
|---|---|---|
| GEI Geotechnical Engineering Inc. — CONSULTANTS IN GEOLOGIC & SOIL ENGINEERING | **Regional Geologic Map** | **PLATE** |
| PROJECT NO. GEI037-A | **Planned Single-Family Residence**<br>East of 15880 Simoni Drive, San Jose, CA | 7 |




EXPLANATION OF NEAR-BY GEOLOGIC UNITS

| | |
|---|---|
| Kbc | Berryessa Formation (Cretaceous) |
| Tbr | Breonis Formation |

fault (dotted where concealed

Base Map: McLaughlin, R.J., Helley, E.J., 2001, Santa Teresa Hills and Southwestern Part of Morgan Hill Quadrangles

GEI Geotechnical Engineering Inc.

CONSULTANTS IN GEOLOGIC & SOIL ENGINEERING

PROJECT NO. GEI037-A

**Geologic Map**

**Planned Single-Family Residence**

East of 15880 Simoni Drive, San Jose, CA

**PLATE**

**8**

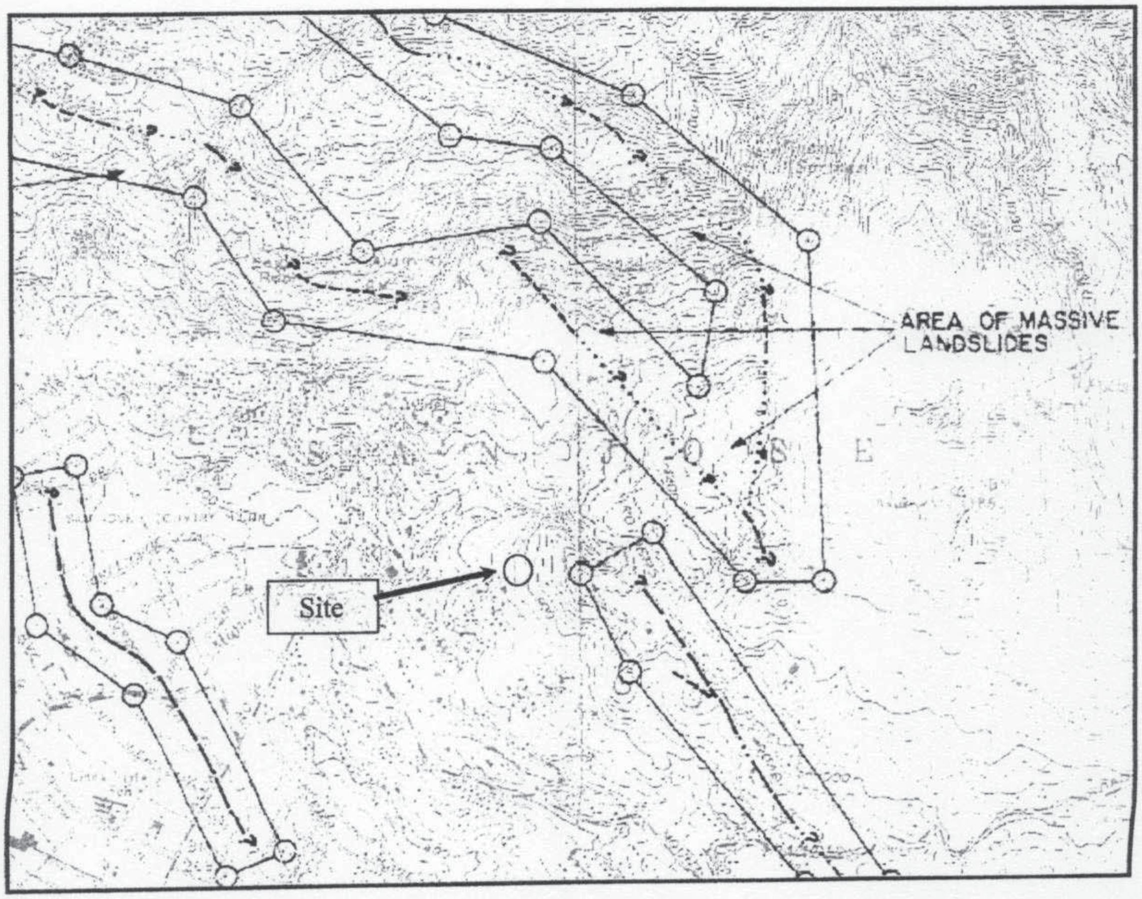
## The Modified Mercalli Intensity Scale

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.




0 1 2 3 4 5
Scale Thousand feet
N

Explanation
1868 - Trace of 1868 earthquake
C - Area of observed creep
- Boundaries of Alquist-Priolo zone

Source: DMG Staff, 2000, Digital images of Official Maps of Alquist-Priolo Earthquake Fault Zones of California, central coast region: California Division of Mines and Geology, CD 2000-004, Calaveras Reservoir, Official Map Released: January 1, 1982, scale 1:24000.

| | | |
|---|---|---|
| GEI Geotechnical Engineering Inc. CONSULTANTS IN GEOLOGIC & SOIL ENGINEERING | **Earthquake Fault Zones Map** | **PLATE** |
| PROJECT NO. GEI037-A | **Planned Single-Family Residence** East of 15880 Simoni Drive, San Jose, CA | 10 |

# BORING 11

1" Diameter Percusion Hole

Drilled 4/30/08

Simoni Drive

| Depth (Ft.) | Test Data | USCS | Description |
|---|---|---|---|
| 0 | | CH | 6" Top Soil |
| | 17% - 87 (17)* | | Brown, clay with few sand (colluvium), medium stiff |
| 6 | | | Stiff @ 5' |
| | 15% - 99 (30)<br>LL=37% PL=17% PI=20 | CL | Mottled brown, sandy clay with some gravel (colluvium), stiff<br>More course gravel @ 10' |
| 12 | 5% - 108 (26) | | Yellow brown, shale & sandstone highly weathered, stiff to very stiff |
| 18 | 5% - 122 (77) | | Medium weathered & very stiff @ 16' |
| 24 | 13% - (N=35) | | Slightly weathered with hard layers |
| | | | Standard Penetration Test (N= Blows per foot) |
| 30 | 13% - (N=53) | | Hard<br>Terminated @ 30' |

*140-lbs. weight falling 30-ins.

Note: Free ground water not encountered

Scale: 0 3 6 9 Feet

5% - 122 (77) Undisturbed Sample

Blows per Foot

Field Dry Density (p.c.f.)

Natural Moisture Content (%)

LOG OF BORING

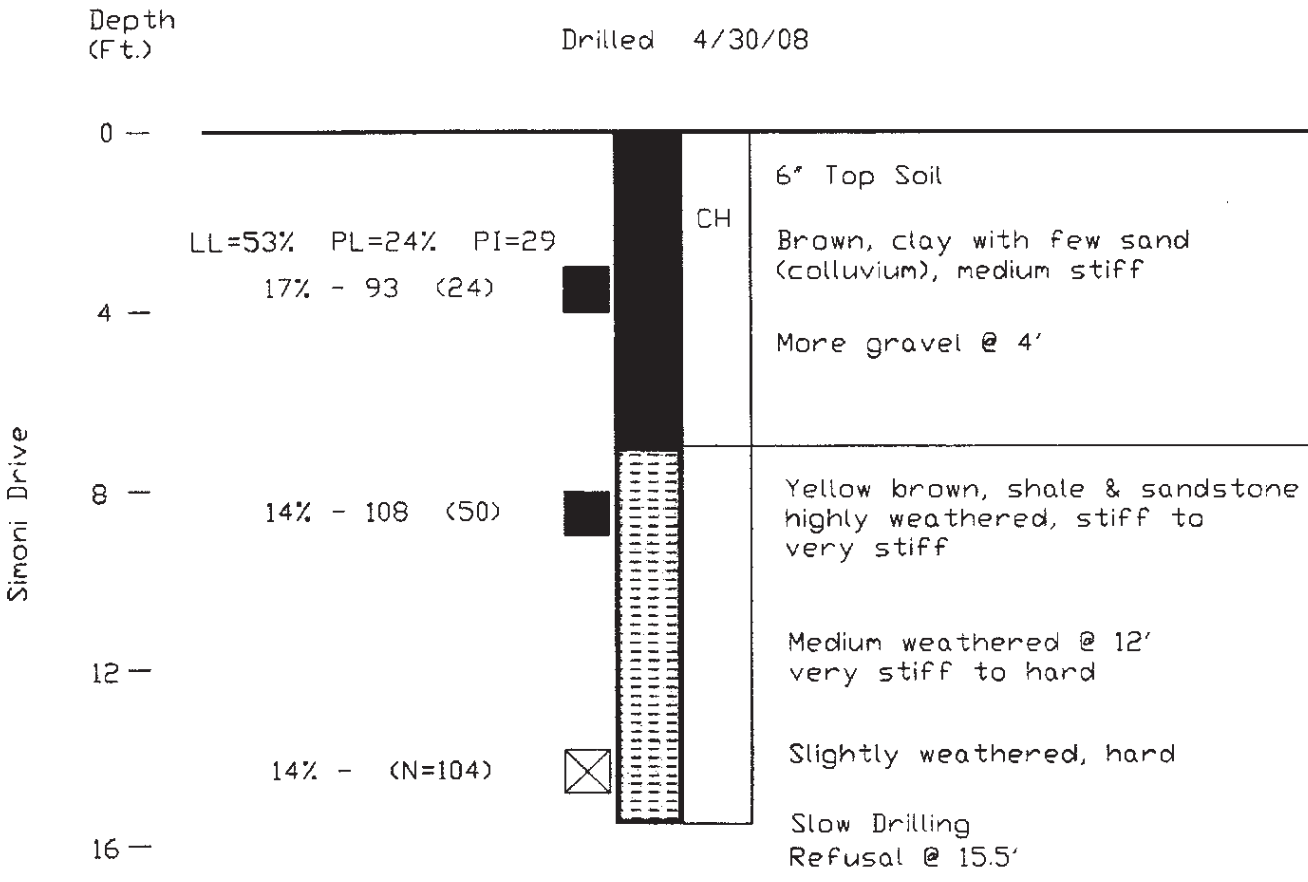
GEOTECHNICAL ENGINEERING, INC.

PLATE 11

# BORING 12

1" Diameter Percusion Hole

Drilled 4/30/08




Note: Free ground water not encountered

# LOG OF BORING


GEOTECHNICAL ENGINEERING, INC.

PLATE 12

# BORING 13

1" Diameter Percusion Hole

Drilled 4/30/08

Depth (Ft.)

Simoni Drive

| Depth (Ft.) | Sample | Soil type | Description |
|---|---|---|---|
| 0 | | CH | 6" Top Soil |
| | 20% - 100 (33) | | Brown, clay with few sand (colluvium), medium stiff |
| 5 | | | Stiff @ 5' |
| | 19% - 106 (30) | CL | Mottled brown, sandy clay with some gravel (colluvium), stiff |
| 10 | | | More course gravel @ 10' |
| | 14% - 113 (60) | | Yellow brown, shale & sandstone medium to highly weathered stiff to very stiff |
| 15 | | | |
| | 11% - (N=68) | | Slightly weathered, hard<br>Slow drilling<br>Refusal @ 18.5' |
| 20 | | | |

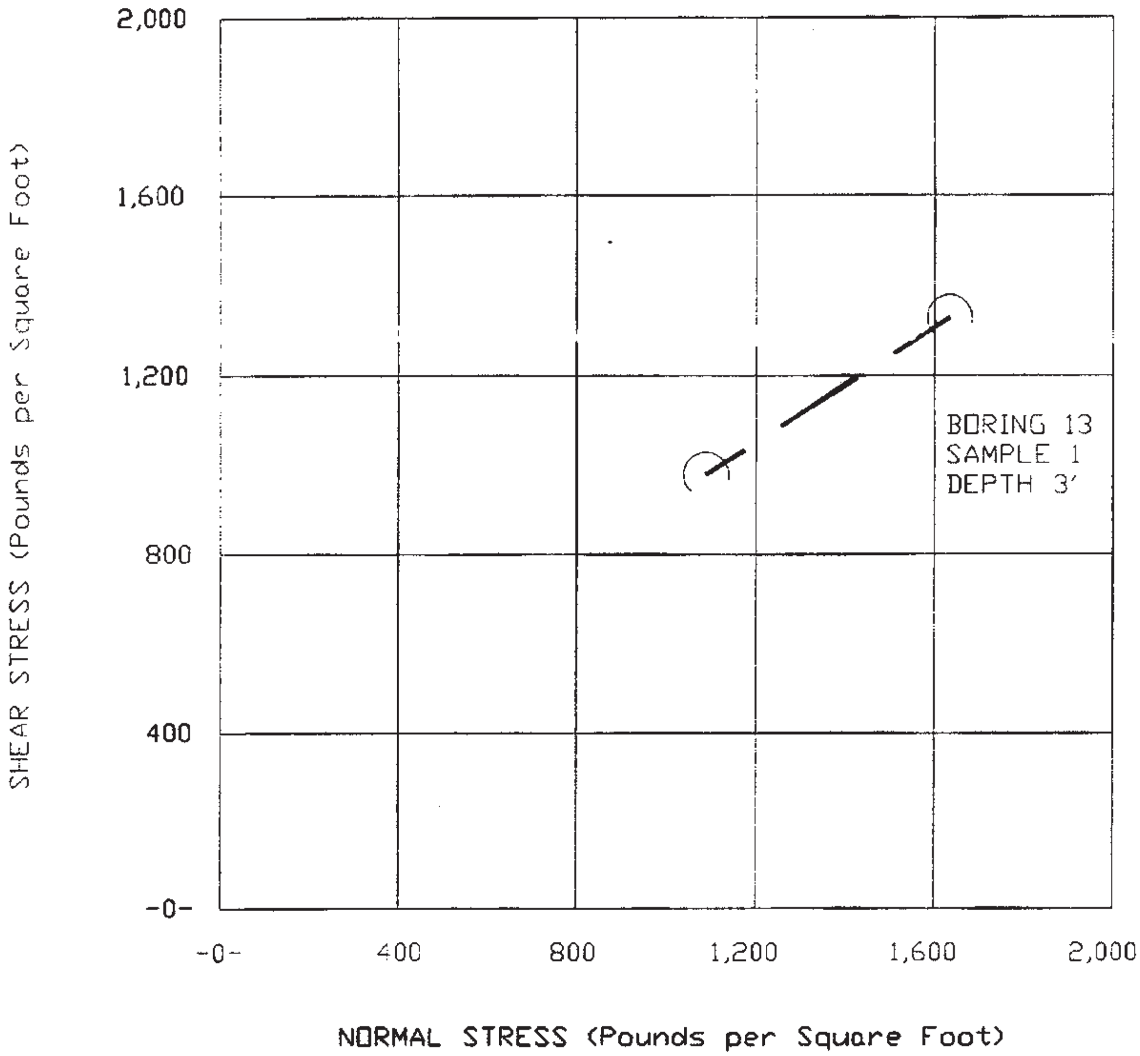
Note: Free ground water not encountered

# LOG OF BORING

Scale 0 2.5 5 7.5 Feet

GEOTECHNICAL ENGINEERING, INC.

PLATE 13




# WET SHEAR TEST DATA

(Saturated Direct Shear)

GEOTECHNICAL ENGINEERING, INC.

PLATE 14

BORING 13

SAMPLE 1

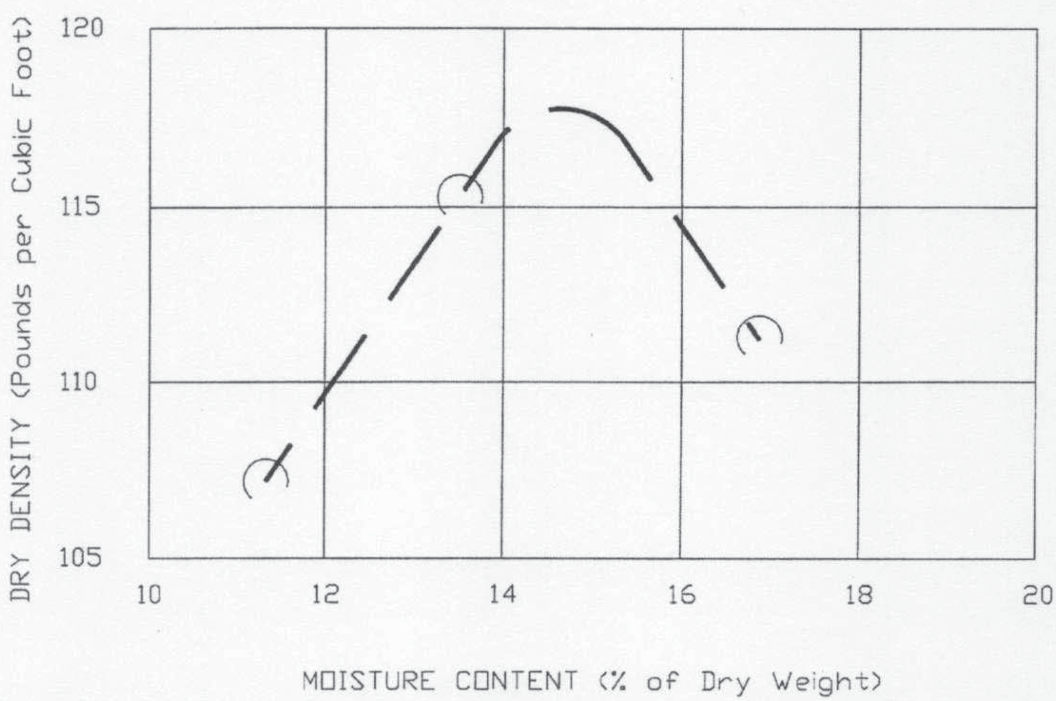
DEPTH 2'

MAXIMUM DRY DENSITY = 118 Pounds Per Cubic Foot

OPTIMUM MOISTURE CONTENT = 14.5 percent

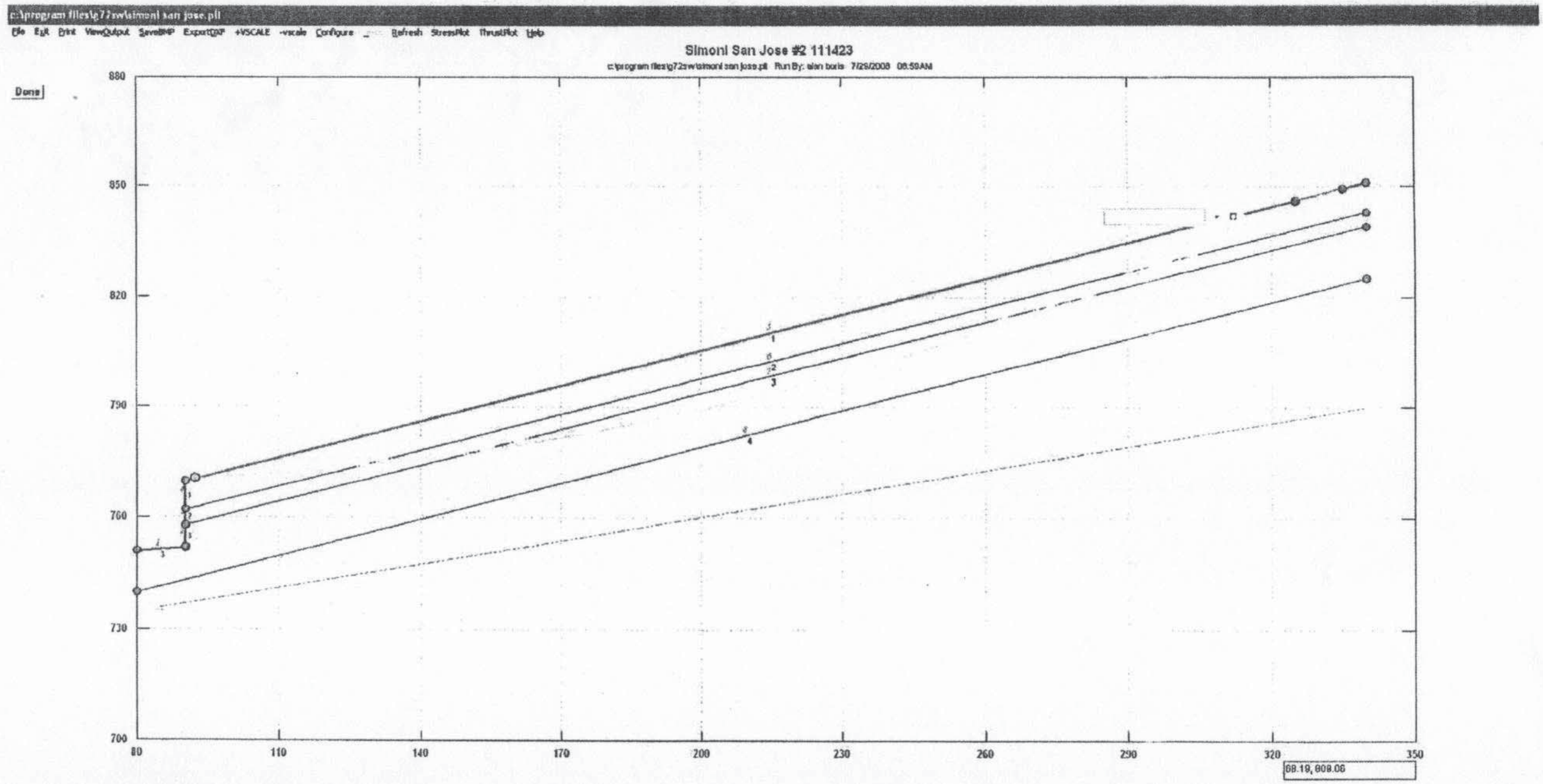
AMERICAN SOCIETY FOR TESTING & MATERIALS DESIGNATION: D:1557-78

(Modified Proctor Compaction Method)




# COMPACTION TEST DATA

GEOTECHNICAL ENGINEERING, INC.




GSTABL7

PLATE 16