

---

**To:** Mr. Drew Bagdasarian  
**Company:** SMCCCD c/o DRB Associates  
**Address:** 220 Montgomery Street, Suite 1830  
San Francisco, California 94104  
**Cc:**  
**Project:** Cañada College Electrical Infrastructure  
Replacement Project  
**Project No.:** 177-1-7

---

Dear Drew,

Please find our enclosed Geotechnical Investigation and Geologic Hazards Evaluation report for your use and information. Thank you again for this opportunity to provide our services for your project.

Please let us know if you have any question or need anything else. I will give you a call next week to follow up.

Sincerely,

**Cornerstone Earth Group**



---

Danh T. Tran, P.E.  
Principal Engineer

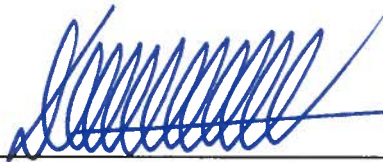
<b>TYPE OF SERVICES</b>	Geotechnical Investigation and Geologic Hazards Evaluation
<b>PROJECT NAME</b>	Canada College Electrical Infrastructure Replacement
<b>LOCATION</b>	4200 Farm Hill Road Redwood City, California
<b>CLIENT</b>	San Mateo County Community College District
<b>PROJECT NUMBER</b>	177-1-7
<b>DATE</b>	April 23, 2010



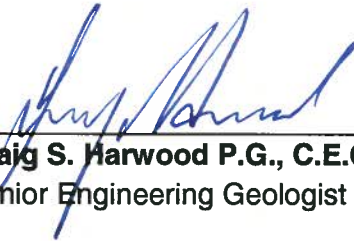
GEOTECHNICAL

<b>Type of Services</b>	<b>Geotechnical Investigation and Geologic Hazards Evaluation</b>
<b>Project Name</b>	<b>Cañada College Electrical Infrastructure Replacement</b>
<b>Location</b>	<b>4200 Farm Hill Road Redwood City, California</b>
<b>Client</b>	<b>San Mateo County Community College District</b>
<b>Client Address</b>	<b>2700 West Hillsdale Boulevard, Building 27 San Mateo, California</b>
<b>Project Number</b>	<b>177-1-7</b>
<b>Date</b>	<b>April 23, 2010</b>

**Prepared by**



**Danh T. Tran, P.E.**  
Principal Engineer  
Geotechnical Project Manager

**Craig S. Harwood P.G., C.E.G.**  
Senior Engineering Geologist




**Scott E. Fitinghoff, P.E., G.E.**  
Principal Engineer  
Quality Assurance Reviewer



**TABLE OF CONTENTS**

**SECTION 1: INTRODUCTION..... 1**

1.1 PROJECT DESCRIPTION ----- 1

1.2 SCOPE OF SERVICES ----- 2

1.3 EXPLORATION PROGRAM----- 2

1.4 LABORATORY TESTING PROGRAM ----- 2

1.5 ENVIRONMENTAL SERVICES----- 2

**SECTION 2: REGIONAL SETTING..... 2**

2.1 GEOLOGIC SETTING ----- 2

2.2 REGIONAL SEISMICITY ----- 3

    Table 1: Proximity of Active Fault within 100-Kilometers..... 4

2.3 HISTORICAL EARTHQUAKES----- 5

**SECTION 3: SITE CONDITIONS..... 5**

3.1 GEOMORPHOLOGY AND RECENT HISTORY----- 5

3.2 SITE RECONNAISSANCE AND SURFACE DESCRIPTION----- 5

3.3 SITE GEOLOGY ----- 6

3.4 SUBSURFACE CONDITIONS ----- 6

    3.4.1 New Building 30..... 6

    3.4.2 Existing Building 3 ..... 7

    3.4.3 Existing Building 16 ..... 7

3.5 PLASTICITY/EXPANSION POTENTIAL ----- 7

3.6 GROUND WATER----- 7

3.7 CORROSION EVALUATION ----- 8

    Table 2: Summary of Corrosion Test Results..... 8

    3.7.1 Preliminary Soil Corrosion Screening..... 8

**SECTION 4: GEOLOGIC HAZARDS ..... 8**

4.1 FAULT RUPTURE----- 9

4.2 HISTORICAL GROUND FAILURES----- 9

4.3 GROUND MOTION HAZARD ANALYSIS ----- 9

    4.3.1 Current Seismic Design Criteria and Requirements ..... 9

    4.3.2 Next Generation Attenuation (NGA) Relations ..... 10

    4.3.3 Non-Next Generation Attenuation (NGA) Relations ..... 10

    4.3.4 Project Specific Attenuation Relations..... 10

4.4 LIQUEFACTION POTENTIAL----- 10

    4.4.1 Background..... 11

    4.4.2 Analysis and Summary of Results..... 11

4.5 LATERAL SPREADING ----- 11

4.6 SEISMIC SETTLEMENT----- 11

4.7 GROUND LURCHING AND LANDSLIDING ----- 12

4.8 FLOODING AND RESERVOIR INUNDATION----- 12



4.9	SEISMICALLY INDUCED WAVES - TSUNAMIS/SEICHES -----	12
4.10	VOLCANIC ERUPTION -----	12
	<b>SECTION 5: CBC SEISMIC DESIGN CRITERIA.....</b>	<b>13</b>
5.1	2007 CBC SITE CLASSIFICATION AND SEISMIC COEFFICIENTS -----	13
	Table 3: 2007 CBC Site Categorization and Site Coefficients .....	13
5.2	SITE SPECIFIC GROUND MOTION HAZARD ANALYSIS -----	14
5.2.1	Probabilistic MCE – Section 21.2.1 .....	14
5.2.2	Deterministic MCE – Section 21.2.2 .....	14
5.2.3	Site Specific MCE – Section 21.2.3 .....	15
5.2.4	Code Design Response Spectrum – Section 11.4.5 .....	15
5.2.5	Design Response Spectrum – Section 21.3 .....	15
5.2.6	Design Response Spectrum and Acceleration Parameters – Section 21.4 .....	15
	<b>SECTION 6: CONCLUSIONS.....</b>	<b>16</b>
6.1	PRIMARY GEOTECHNICAL CONCERNS -----	16
6.1.1	Expansive Weathered Bedrock .....	16
6.1.2	Presence of Undocumented Fill .....	16
6.1.3	Potential for Difficult Excavation .....	17
6.2	PLANS AND SPECIFICATIONS REVIEW -----	17
6.3	CONSTRUCTION OBSERVATION AND TESTING -----	17
	<b>SECTION 7: EARTHWORK .....</b>	<b>17</b>
7.1	SITE DEMOLITION, CLEARING AND PREPARATION -----	17
7.1.1	Site Stripping .....	17
7.1.2	Tree and Shrub Removal .....	18
7.1.3	Demolition of Existing Slabs, Foundations and Pavements .....	18
7.1.4	Abandonment of Existing Utilities .....	18
7.2	REMOVAL OF EXISTING FILLS -----	18
7.3	BEDROCK EXCAVATION -----	19
7.4	TEMPORARY CUT AND FILL SLOPES -----	19
7.5	SUBGRADE PREPARATION -----	20
7.6	MATERIAL FOR FILL -----	20
7.6.1	Re-Use of On-site Soils .....	20
7.6.2	Potential Import Sources .....	20
7.7	COMPACTION REQUIREMENTS -----	20
	Table 4: Compaction Requirements .....	21
7.8	TRENCH BACKFILL -----	22
7.9	SITE DRAINAGE -----	22
	<b>SECTION 8: FOUNDATIONS.....</b>	<b>22</b>
8.1	SUMMARY OF RECOMMENDATIONS -----	22
8.2	SEISMIC DESIGN CRITERIA -----	22
8.3	SHALLOW FOUNDATIONS -----	23
8.3.1	Spread Footings .....	23
8.3.2	Footing Settlement .....	23

8.3.3 Lateral Loading.....	24
8.3.4 Shallow Footing Construction Considerations .....	24
<b>SECTION 9: CONCRETE SLABS AND PEDESTRIAN PAVEMENTS .....</b>	<b>24</b>
9.1 INTERIOR SLABS-ON-GRADE .....	24
9.2 INTERIOR SLABS MOISTURE PROTECTION CONSIDERATIONS .....	25
9.3 PEDESTRIAN CONCRETE FLATWORK.....	26
<b>SECTION 10: VEHICULAR PAVEMENTS.....</b>	<b>26</b>
10.1 ASPHALT CONCRETE .....	26
Table 4: Asphalt Concrete Pavement Recommendations.....	26
Estimated Design R-value = 15 .....	26
10.2 PAVEMENT CUTOFF .....	26
<b>SECTION 11: LIMITATIONS .....</b>	<b>27</b>
<b>SECTION 12: REFERENCES.....</b>	<b>28</b>

FIGURE 1: VICINITY MAP	
FIGURE 2A: SITE PLAN AND GEOLOGIC MAP – BUILDING 30	
FIGURE 2B: SITE PLAN – BUILDINGS 3 AND 16	
FIGURE 3: VICINITY GEOLOGIC MAP	
FIGURE 4: REGIONAL FAULT MAP	
FIGURE 5: HISTORICAL EARTHQUAKES MAP	
FIGURE 6: GEOLOGIC CROSS SECTION A-A'	
FIGURE 7: SLOPE STABILITY DURING EARTHQUAKES	
FIGURE 8: PROBABILISTIC MCE AND MAXIMUM DETERMINISTIC HAZARD ANALYSIS	
FIGURE 9: DETERMINISTIC MCE SITE SPECIFIC RESPONSE SPECTRUM	
FIGURE 10: DETERMINISTIC MCE, 2/3 DETERMINISTIC MCE, AND 80 PERCENT SITE SPECIFIC CODE DESIGN RESPONSE SPECTRUM	
FIGURE 11: PROJECT DESIGN RESPONSE SPECTRUM	
FIGURE 12: PROJECT DESIGN RESPONSE SPECTRUM AND RECOMMENDED ACCELERATION DESIGN VALUES	

APPENDIX A: FIELD INVESTIGATION	
APPENDIX B: LABORATORY TEST PROGRAM	
APPENDIX C: PREVIOUS FIELD INVESTIGATION AND LABORATORY TEST PROGRAM	

<b>Type of Services</b>	<b>Geotechnical Investigation and Geologic Hazards Evaluation</b>
<b>Project Name</b>	<b>Cañada College Electrical Infrastructure Replacement</b>
<b>Location</b>	<b>4200 Farm Hill Boulevard Redwood City, California</b>

## **SECTION 1: INTRODUCTION**

This report has been prepared for the sole use of the San Mateo County Community College District for the proposed Cañada College Electrical Infrastructure Replacement project. The location of the site is shown on the Vicinity Map, Figure 1.

For our use, we have been provided the following set of plans:

- A set of civil plans titled “Cañada College Electrical Infrastructure Project” prepared by BKF Engineers/Surveyors/Planners dated April 2, 2010.
- A geotechnical report titled “Geotechnical Investigation, New Chiller Slab Platform and Central Plant Cooling Towers, Cañada College, Redwood City, California” prepared by Lowney Associates dated February 2, 2005.

### **1.1 PROJECT DESCRIPTION**

Based on information provided to date and our understanding, the project will generally include a “primary service replacement” to the existing campus power system. The new switch gear facility, Building 30), will be located near the college entrance from Farm Hill Road adjacent to an existing service and/or maintenance road. The facility will be approximately 25 feet by 35 feet and architecturally enclosed using CMU walls. Building 30 will have a finished floor of approximately Elevation 602 feet (datum unspecified). A new asphalt concrete access road will be constructed from the Campus Loop Road to the new Building 30. New joint trench lines and “subsurface vaults” will also be constructed from the new switch gear facility to existing manholes located near the tennis courts driveway entrance. In addition, associated equipment and conduits will also be constructed as part of the overall project. New constructions will generally be located northeast of the existing Buildings 3 and 16. Recommendations meeting California Geological Survey (CGS) requirements are not expected for these “non-structural” equipments.

## **1.2 SCOPE OF SERVICES**

Our scope of services was presented in our proposal dated February 25, 2010 (SMCCCD Exhibit 15 dated March 3, 2010), and consisted of field and laboratory programs to evaluate the physical and engineering properties of the subsurface soils, engineering analysis to prepare recommendations for site work and grading, foundations, flatwork, and preparation of this report. Brief descriptions of our exploration and laboratory programs are presented below.

## **1.3 EXPLORATION PROGRAM**

Field exploration consisted of four borings drilled on April 5, 2010 using truck-mounted hollow-stem auger drilling equipment. Borings were drilled to depths of 10 and 30 feet.

The borings were backfilled with cement grout in accordance with local requirements. Approximate boring locations are shown on the Site Plan and Geologic Map, Figure 2A, and Site Plan, Figure 2B. Details regarding our field program are included in Appendix A.

## **1.4 LABORATORY TESTING PROGRAM**

In addition to visual classification of samples, the laboratory program focused on obtaining data for foundation and pavement design. Testing included moisture contents (ASTM D2216), dry densities (ASTM D2937), Plasticity Index (ASTM D4318), and sulfate content (Caltrans 417 (m)). Details regarding our laboratory program are included in Appendix B.

## **1.5 ENVIRONMENTAL SERVICES**

Environmental services were not requested for this project. If environmental concerns are determined to be present during future evaluations, the project environmental consultant should review our geotechnical recommendations for compatibility with the environmental concerns.

## **SECTION 2: REGIONAL SETTING**

### **2.1 GEOLOGIC SETTING**

The San Francisco peninsula is a relatively narrow band of rock at the north end of the Santa Cruz Mountains separating the Pacific Ocean from San Francisco Bay. It represents one mountain range in a series of northwesterly-aligned mountains forming the Coast Ranges geomorphic province of California that stretches from the Oregon border nearly to Point Conception. In the San Francisco Bay area, most of the Coast Ranges have developed on a basement of tectonically mixed Cretaceous- and Jurassic-age (70 to 200 million years old) rocks of the Franciscan Complex. Locally, these basement rocks are capped by younger sedimentary and volcanic rocks. Most of the Coast Ranges are covered by younger surficial deposits that reflect geologic conditions for approximately the last million years.

Lateral and vertical movement on the many splays of the San Andreas Fault system and other secondary faults has produced the dominant northwest-oriented structural and topographic



trend seen throughout the Coast Ranges today. This trend reflects the boundary between two of the Earth's major tectonic plates: the North American plate to the east and the Pacific plate to the west.

The San Andreas Fault system is approximately 40 miles wide in the Bay area and extends from the San Gregorio fault at the coastline to the Coast Ranges-Central Valley blind thrust at the western edge of the Great Central Valley. The San Andreas Fault is the dominant structure in the system, nearly spanning the length of California, and capable of producing the highest magnitude earthquakes. Many other sub-parallel or branch faults within the San Andreas system are equally active and nearly as capable of generating large earthquakes. Right-lateral movement dominates these faults, but an increasingly large amount of thrust faulting resulting from compression across the system is now being identified as well.

Cañada College is located approximately 1½ mile northeast of the San Andreas Fault near the crest of an upland area generally sloping to the southwest. Distances to the San Andreas and other nearby active faults are shown in Table 1.

Geology of the Cañada College campus and vicinity is shown in Figure 3, Vicinity Geologic Map (a partial reproduction of that of Brabb, et al., 2000).

## **2.2 REGIONAL SEISMICITY**

The San Francisco Bay area is recognized by geologists and seismologists as one of the most seismically active regions in the United States. Significant earthquakes occurring in the Bay area are generally associated with crustal movement along well-defined, active fault zones of the San Andreas Fault system. The faults considered capable of generating significant earthquakes are generally associated with the well-defined areas of crustal movement, which trend northwesterly.

The San Andreas Fault generated the great San Francisco earthquake of 1906 and the Loma Prieta earthquake of 1989, and passes about 1½ mile southwest of the school site. Two other nearby active faults are the Monte Vista-Shannon and San Gregorio faults, located about 1 mile southwest and 10 miles northeast of the site, respectively. Table 1 lists all known active faults in order of increasing distance within 100 kilometers (62 miles) of the site. The seismic characteristics of some faults vary along its length so different segments of the same fault could be listed separately in the table.

A regional fault map is presented as Figure 4, illustrating the distances of the site to significant fault zones. Figure 5 shows regional faults with historical earthquake information superimposed.

**Table 1: Proximity of Active Fault within 100-Kilometers**

Fault Name	Distance (miles)	Distance (kilometers)
Monte Vista-Shannon	1	1½
San Andreas (1906)	1⅓	2¼
San Gregorio	10	17
Hayward (Total Length)	18	29
Calaveras	23	38
Hayward (Southeast Extension)	20	33
Sargent	28	45
Zayante-Vergeles	29	59
Concord-Green Valley	34	55
Greenville	36	58
Monterey Bay – Tularcitos	37	60
Rogers Creek	44	71
Great Valley	44	71
Palo Colorado (Sur)	45	72
Point Reyes	46	74
West Napa	49	79
Ortigalita	55	89

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 (2007) forecast a 99.7 percent chance of a magnitude 6.7 or greater earthquake somewhere in California before 2038. Previously, they (Working Group on California Earthquake Probabilities, 2003) 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 the investigation because any major earthquake can cause damage throughout the region. The probability of a magnitude 6.7 or greater earthquake on the peninsula segment of the San Andreas Fault, which is closest to the Cañada College campus, is believed to be 13 percent in that time period. During such an earthquake the danger of fault ground rupture at the site is slight, but strong ground shaking would occur.

This potential was demonstrated when the 1989 Loma Prieta earthquake caused severe damage in Oakland and San Francisco, more than 50 miles from the epicenter. Although earthquakes can cause damage at a considerable distance, shaking will be very intense near the fault rupture. Therefore, earthquakes centered in urbanized areas of the region have the potential to cause much more damage than the 1989 Loma Prieta earthquake.

## **2.3 HISTORICAL EARTHQUAKES**

We reviewed and performed a data search of known historical earthquakes of magnitude 5 or greater within a 100-kilometer radius of the site using available published data from the CDMG computerized earthquake catalog of events through December 1999. Figure 5 shows the epicenters of these magnitude 5 or greater events. We also included data from Townley and Allen (1939) and the U.S. Geological Survey Earthquake Data Base System, giving 200 years of data in the search area. The results of our computer search indicated that about 79 known earthquakes of Richter Magnitude 5 or greater have occurred within 100 kilometers of the site between 1800 and December 1999. Figure 5 shows the epicenters of these magnitude 5 or greater events.

## **SECTION 3: SITE CONDITIONS**

### **3.1 GEOMORPHOLOGY AND RECENT HISTORY**

Based on our observations and review of historical aerial photos, we infer the area for Building 30 was created by cutting a bench into the natural, easterly facing hillside. Subsequent grading to establish the campus Loop Road occurred. The site is located at a sharp “hair-pin” bend in the campus perimeter road which borders it on the north, east and west. At the time of our reconnaissance, the site consisted of a dirt access road with down slopes on the west and east. The slope on the west is a steep, east facing natural slope that extends down to the level terrace. On the terrace a large stockpile of non-engineered fill exists which was probably placed from various construction projects on campus.

Aerial photographs listed in the References show the site vicinity in 1943 and 1973. The site was completely undeveloped in 1943 and existed as grass covered hillside. The surrounding area was largely in agricultural mode with moderate to large parcels. We observed no evidence of landsliding (hummochy ground, scarps, tonal contrasts, etc.) at the site. By 1973 the campus was developed along with the surrounding area. The fill stockpile currently located at the Building 30 site was placed sometime after the 1973 photos were taken.

### **3.2 SITE RECONNAISSANCE AND SURFACE DESCRIPTION**

Generally, the existing Cañada College campus is located north of Farm Hill Boulevard and east of Highway 280 in Redwood City, California.

A reconnaissance of the site and immediate vicinity was performed by our Certified Engineering Geologist on March 26, 2010, for the purpose of observing and recording data and field relations. Our site-specific geologic map for the new Building 30 presents our interpretation of the distribution of geologic materials and mapped features, Site Plan and Geologic Map, Figure 2A.

### **3.3 SITE GEOLOGY**

Several regional scale geologic maps have been published of the area including those by Lajole et al. (1974), Leighton (1976), Brabb and Pampeyan (1983), Wentworth et al. (1985), Pampeyan (1994), Brabb et al. (1998) and Brabb et al. (2000) depict similar geologic units underlying the site. Of these published maps Pampeyan's depiction of the bedrock units is consistent with our site observations (see below). His mapping depicts the general area of the proposed Building 30 as near the fault contact between Franciscan Complex greenstone (fg) on the west, against Serpentinite (sp) on the east.

Pampeyan characterized the greenstone as; "altered basaltic volcanic rocks, including flows, pillow lavas, breccias, tuff breccias, tuffs, and minor related intrusive rocks, in unknown proportions. Some Franciscan chert and limestone bodies are also associated with the unit. The serpentinite is characterized as; soft sheared serpentinite which encloses blocks of hard unshaped serpentinite and ultramafic rocks. A review of available published geologic maps reveals no landslides have been mapped at the site.

A geologic cross-section, Figure 6, illustrates the generalized subsurface conditions within the new Building 30 site location.

### **3.4 SUBSURFACE CONDITIONS**

#### **3.4.1 New Building 30**

At the time of our site reconnaissance and this investigation, the proposed Building 30 project is generally located near the campus entrance from Loop Road on a level terrace. An existing stockpile with a maximum height of about 13 feet extends over much of the terrace. The fill forms a moderate to steep, west facing slope and extends partially into the building pad area. Locally the fill is 1½ to 2 feet high within the building pad area, and substantially thicker just on the east side of the building footprint. This conclusion is confirmed by recent exploratory borings conducted during the current investigation. Further (160 feet) to the east the terrace ends at an easterly facing natural slope which extends down to the campus entrance road. The stockpile has a thick growth of grasses and weeds and sparse large shrubs. The remainder of the terrace is devoid of vegetation.

This cut slope exposes severely weathered greenstone. The greenstone has closely spaced fractures. Outcrops located further to the west show the greenstone containing relict flow structure dipping very steeply (82°) to the west. Locally greenstone is exposed at the ground surface in erosion scars on slopes adjacent to the perimeter access road. Based on observed field conditions, it is apparent that bedrock is very shallow across the site and adjacent slopes (except where the stock pile is located). Although exposures north and east of the site are rare due to ground cover, development and landscaping, subsurface conditions encountered in our exploratory borings suggests the serpentinite underlies the building envelope and the fault may transect the terrace just west of the building envelope.



We noted no physical signs of slope instability (hummocky topography, scarps, etc.) on the slopes located adjacent to the site. This is consistent with our interpretation of stereo aerial photos reviewed as part of our work. Serpentinite bedrock at the borings locations was found to be dark reddish brown to yellow-brown and gray, soft to moderately hard, severely weathered with standard penetration blow counts of 25 to 53 blows per foot. Due to its geologic history this bedrock can vary in texture over short distances from highly sheared and weak to hard.

### **3.4.2 Existing Building 3**

Based on our review of available information, the existing parking lot located northeast of the existing Building 3 was underlain by approximately 2 to 4 inches of asphalt concrete over 2½ to 4 inches of aggregate base. Except for a former excavation area, below the asphalt concrete pavement section, the site was underlain by clayey and sandy soils to depths ranging from approximately 1 foot to 3 feet. The surficial alluvial soils were underlain by highly weathered Franciscan greenstone bedrock to at least 25 feet.

### **3.4.3 Existing Building 16**

We performed one boring in the northeast corner of the existing Building 16, (EB-4). Our boring was located in a landscaping area and encountered Franciscan greenstone within ½ foot of the ground surface. The greenstone was deeply weathered, weak, and of low hardness. The greenstone extended to at least 10 feet, the maximum depth of the boring at this location.

## **3.5 PLASTICITY/EXPANSION POTENTIAL**

We performed a Plasticity Index (PI) test on a representative sample of the highly weathered Franciscan serpentinite bedrock at a depth of 3½ feet. The test resulted in a PI of 28 with a liquid limit of 45, indicating a moderate expansion potential to wetting and drying cycles.

Additional recommendations regarding expansion potential are provided in the following sections of this report.

## **3.6 GROUND WATER**

Ground water was not encountered in all borings at depths ranging from approximately 10 to 30 feet during drilling. However, based on our experience with other projects on the Cañada College campus and similar subsurface conditions, during site grading and excavation, a perched ground water condition within larger and deeper excavations can occur and should be expected. Based on our judgment and opinion, perched water would most likely infiltrated through joints and fractures in the bedrock. In addition, perched ground water also may be present in areas located adjacent to irrigated landscaping and downslope from hillsides.

Fluctuations in ground water levels occur due to many factors, including seasonal fluctuations, underground drainage patterns, regional fluctuations, and other factors.

### 3.7 CORROSION EVALUATION

One sample from boring EB-1 at a depth of 4 feet was tested for in-situ resistivity, pH, soluble sulfates, and chlorides. Laboratory test results are summarized in Table 2.

**Table 2: Summary of Corrosion Test Results**

Sample/Test Location Number	Depth (feet)	Soil pH	Minimum Resistivity <sup>1</sup> (ohm-cm)	Chloride (mg/kg) <sup>2</sup>	Sulfate (mg/kg) / % <sup>2</sup>
EB-1	4	7.8	2,434	< 2	< 5 / 0.0005

Note: 1 Resistivity measured at 100% saturation  
 2 mg/kg = milligrams per kilogram = parts per million (ppm)

Many factors can affect the corrosion potential of soil including moisture content, resistivity, permeability, and pH, as well as chloride and sulfate concentration. Typically, soil resistivity, which is a measurement of how easily electrical current flows through a medium (soil and/or water), is the most influential factor. In addition to soil resistivity, chloride and sulfate ion concentrations, and pH also contribute in affecting corrosion potential.

#### 3.7.1 Preliminary Soil Corrosion Screening

Based on laboratory test results summarized in Table 2, near-surface soils may be considered moderately corrosive to buried metallic improvements. Other corrosion parameters (pH, chloride and sulfate ion content) indicate a negligible corrosion potential to buried metallic or concrete structures. The 2007 California Building Code (CBC) no longer provides requirements for concrete exposed to sulfate-containing solutions, but instead references American Concrete Institute (ACI) Code 318-08, Section 4.3. ACI Code 318-08 Table 4.2.1 indicates that concrete exposed to water-soluble sulfate concentrations in soil less than 0.10 percent by weight (< 1,000 parts per million (ppm)) is considered negligible.

Based on the above information, the weathered serpentinite bedrock can be characterized as non-corrosive to buried concrete based the ACI Code 318-08, Section 4.3. Therefore, the concrete mix design does not require modification in accordance with ACI Code 318 requirements. We recommend a corrosion engineer and/or utility consultant be retained to provide additional recommendations, as required, for buried metallic improvements.

## SECTION 4: GEOLOGIC HAZARDS

This section presents our geologic hazards review per requirements of the Division of State Architects (DSA), the Office of Regulatory Services (ORS), and California Geological Survey (CGS), for the planned modernization project, located on the Cañada College campus in Redwood City, California. The site is located approximately at Latitude 37.44855°N and Longitude 122.261279°W. Our comments concerning potential hazards are presented in subsequent sections.

## **4.1 FAULT RUPTURE**

A map showing known faults in the region surrounding Cañada College 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 (CDMG, 1974) and no surface expression of active faulting was seen on aerial photographs or in the field. Brown (1972) does not show any active or probably active faults or zones of potential surface deformation due to fault movement near the site. The closest active fault to the site is the San Andreas that passes approximately  $1\frac{1}{3}$  southwest of the site. As mentioned above, Brabb (1998, 2000) shows a northeast trending fault located at and parallel to Farm Hill Boulevard (also in the area of the proposed Building 30) and another located in the western portion of the campus. Both these faults cut Franciscan bedrock units but do not cut geologically younger units therefore they are not considered active faults according to State of California criteria. The San Andreas Fault zone in the area is located approximately  $2\frac{1}{4}$  kilometers southwest. In our judgment, neither primary nor coseismic fault rupture is anticipated at the new Building 30 site.

## **4.2 HISTORICAL GROUND FAILURES**

Many historical earthquakes have occurred on active faults and fault branches throughout coastal California, but the San Andreas Fault is considered one of the major active faults of the region. It generated significant, damaging earthquakes in 1836 and 1868, as well as the great San Francisco Earthquake of 1906, which had an approximate Richter Magnitude of 8.3, and the Loma Prieta Earthquake of 1989. Lawson (1908, p. 259) reported considerable damage from the 1906 earthquake in the Redwood City area. The intensity of the earthquake in Redwood City was about IX (Rossi-Forel Intensity).

## **4.3 GROUND MOTION HAZARD ANALYSIS**

As required in California Geologic Survey (CGS) Note 48 (Section 19) and in CBC Section 1614A.1.2, for a site located within 10 km of a state-considered active fault, a site specific Ground Motion Hazard Analysis (GMHA) is required and should be performed in accordance with Chapter 21, Section 21.2 of ASCE 7-05. This hazard analysis incorporates results from both probabilistic and deterministic analyses, as discussed in the sections below.

### **4.3.1 Current Seismic Design Criteria and Requirements**

As noted in Division of the State Architect (DSA) Bulletin 09-01 dated January 26, 2009, the use of Next Generation Attenuation (NGA) or newer attenuation relationships (relationships used for the USGS 1997 and the 1997 Probabilistic Seismic Hazard Mapping) are acceptable provided requirements outlined in the Office of Statewide Health Planning and Development (OSHPD) Code Application Notice (CAN) 2-1802A.6.2 are applied. This is for projects where the geologic hazard report is submitted to DSA after February 28, 2009.

### **4.3.2 Next Generation Attenuation (NGA) Relations**

If NGA relations are used, above referenced OSHPD CAN interpretations and provisions should be incorporated. Those interpretations are summarized below.

- No fewer than three NGA relations shall be used.
- Site specific probabilistic analysis using NGA relations shall use the maximum rotated component of the ground motion.
- Site specific deterministic analysis using NGA relations shall use the 84<sup>th</sup> percentile of the maximum rotated component of the ground motion in-lieu of using the 150 percent of the median value.
- Prior attenuation relations established using geomean and NGA relations shall not be intermixed in determining the site specific ground motion, unless approved in advance by DSA.

### **4.3.3 Non-Next Generation Attenuation (NGA) Relations**

As an alternative, we understand non-NGA attenuation relations used to develop Seismic Hazard Maps of the 2007 California Building Code (USGS Project '02 and the 2002 California Probabilistic Seismic Hazard Maps), can also be used for site analysis. Those attenuation relations include:

- Boore et al (1997)
- Sadigh et al (1997)
- Abrahamson and Silva (1997)
- Campbell and Bozorgnia (2003)
- Spudich (1999)

### **4.3.4 Project Specific Attenuation Relations**

For this project, we have performed our site specific ground motion hazard analysis using the NGA attenuation relations referenced above. Additional information and recommendations are provided in Section 5.

## **4.4 LIQUEFACTION POTENTIAL**

The site is not currently mapped within a State-designated Liquefaction Hazard Zone. However, our field and laboratory programs addressed this issue by performing visual classification on sampled materials and performing various tests to further classify the soil properties, as required.



#### **4.4.1 Background**

During strong seismic shaking, cyclically induced stresses can cause increased pore pressures within the soil matrix that can result in liquefaction triggering, soil softening due to shear stress loss, potentially significant ground deformation due to settlement within sandy liquefiable layers as pore pressures dissipate, and/or flow failures in sloping ground or where open faces are present (lateral spreading) (NCEER 1998). Limited field and laboratory data is available regarding ground deformation due to settlement; however, in clean sand layers settlement on the order of 2 to 3 percent of the liquefied layer thickness can occur. Soils most susceptible to liquefaction are loose, non-cohesive soils that are saturated and are interbedded with poor drainage materials, such as sand and silt layers bedded with a cohesive cap.

#### **4.4.2 Analysis and Summary of Results**

Field exploration and local ground water studies indicate that ground water is deep, with historic high ground water greater than 50 feet in the general area except of perched ground water within the shallow bedrock. In addition, borings performed for this investigation have encountered stiff to hard clays and weathered bedrock to the maximum depth explored. Based on guidelines set forth in CGS (formerly CDMG) Special Publication 117 (CGS, 1997), "screening investigations" can be used to determine whether a particular site has "obvious indicators" for potential failure as a result of liquefaction. Three of these indicators include soil type, soil density, and depth to ground water. As discussed above, ground water is expected to be deeper than 50 feet at the site except within rock fissures as a perched condition. In addition, Franciscan greenstone and serpentinite bedrock is located below the surficial fill at a depth of approximately 1½ feet. Therefore, and based on above referenced "indicators", and our engineering judgment, the potential for liquefaction is considered low during seismic shaking.

#### **4.5 LATERAL SPREADING**

Lateral spreading is horizontal/lateral ground movement of relatively flat-lying soil deposits towards a free face such as an excavation, channel, or open body of water; typically lateral spreading is associated with liquefaction of one or more subsurface layers near the bottom of an exposed slope. Since failure tends to propagate as block failures, it is difficult to analyze and estimate where the first tension crack will form.

As recommended above, the potential for liquefaction occurring at the site is considered low. In addition and as discussed, the site is generally underlain by shallow bedrock. Therefore, in our opinion, the potential for lateral spreading to affect the site is low.

#### **4.6 SEISMIC SETTLEMENT**

Loose unsaturated sandy soils can settle during strong seismic shaking. As the soils encountered at the site were predominantly stiff to hard clays and weathered bedrock, in our opinion, the potential for significant differential seismic settlement affecting the proposed improvements is low.

#### **4.7 GROUND LURCHING AND LANDSLIDING**

Our interpretation of the landslide potential at the site is based on our geologic reconnaissance, aerial photograph interpretation, and review of the previous geotechnical investigation reports, research of published geologic maps and reports, and our subsurface exploration. The published maps reviewed show no landslides occurring at the subject site.

The site is located within a hilly area and underlain by bedrock considered not particularly susceptible to landsliding. Although there are localized areas with steeper slopes surrounding the college campus, no active or recent landslides were identified on aerial photographs or during our site reconnaissance. Brabb and others (1972) show various parts of the campus having landslide susceptibility ranging from very low to moderate, depending mostly on slope inclination. In addition, ground water is expected to be deeper than 50 feet below the ground and is not expected to impact slope stability.

Wieczorek and others (1985) have produced a map showing slope stability during earthquakes in San Mateo County based on lithologic, hydrologic, slope, and seismic data. A portion of that map is shown in Figure 7, Slope Stability During Earthquakes. The map shows portions of the campus, generally those with steep to moderately steep slopes (17 degree to 35 degree slope inclination of Mark and others, 1988), as having high susceptibility to seismically induced slope failure, but the building site is shown as having very low susceptibility. The presence of moderately hard to hard greenstone and serpentinite bedrock underlying the site and adjacent hillsides and the lack of past instability in this area indicates there is a low potential for slope instability impacting the substation facility.

#### **4.8 FLOODING AND RESERVOIR INUNDATION**

The Cañada College campus is located on a hilltop so the only surface waters are the result of rain, surface run-off, or import water for irrigation. While these sources are capable of minor localized flooding caused by plugged drains, adequate design and maintenance should reduce this hazard to a minor problem.

#### **4.9 SEISMICALLY INDUCED WAVES - TSUNAMIS/SEICHES**

Due to the topographic characteristics and geographic position of the site, this location is not within a tsunami hazard zone as mapped by San Mateo County (2008). The site is also not located next to any major drainage areas that would be affected by or generate a seismically induced wave. Therefore, this potential hazard is not anticipated at the site.

#### **4.10 VOLCANIC ERUPTION**

The youngest volcanic rocks mapped in the site area are isolated patches shown by Pampeyan (1993) about 4.6 miles to the southeast. These belong to the Page Mill Basalt formation, have been dated at 14.8 million years old, and show no signs of more recent activity. We do not believe a volcanic eruption hazard exists at the new Building 30 site.

## SECTION 5: CBC SEISMIC DESIGN CRITERIA

### 5.1 2007 CBC SITE CLASSIFICATION AND SEISMIC COEFFICIENTS

The first step in the GMHA is to determine the 2007 CBC “Seismic Coefficients,” which are established based on a series of tables and figures addressing different site factors, including the soil profile within the upper 100 feet below grade and mapped spectral acceleration parameters based on distance to the controlling seismic source/fault system. Based on borings performed and review of local geology, the site is underlain by relatively shallow bedrock with typical SPT “N” values greater than 50 blows per foot. Therefore, we have classified the site as Soil Classification B. The mapped spectral acceleration parameters  $S_S$  and  $S_1$  were calculated using the USGS computer program Earthquake Ground Motion Parameters, Version 5.0.9, revision date October 6, 2008, based on the site coordinates presented in Table 3 and the site classification. Table 3 also lists various factors used to determine the seismic coefficients and other parameters.

**Table 3: 2007 CBC Site Categorization and Site Coefficients**

Classification/Coefficient	Design Value
Site Class	B
Site Latitude	37.448550°
Site Longitude	-122.261279°
0.2-second Period Mapped Spectral Acceleration <sup>1</sup> , $S_S$	2.367g
1-second Period Mapped Spectral Acceleration <sup>1</sup> , $S_1$	1.155g
Site Occupancy Category (per Table 1-1, ASCE/SEI 7-05)	III
Seismic Design Category (per Section 11.6, ASCE/SEI 7-05, where $S_1 > 0.75g$ )	E
Short-Period Site Coefficient – $F_a$	1.0
Long-Period Site Coefficient – $F_v$	1.0
0.2-second Period, Maximum Considered Earthquake Spectral Response Acceleration Adjusted for Site Effects - $S_{MS}$	2.367g
1-second Period, Maximum Considered Earthquake Spectral Response Acceleration Adjusted for Site Effects – $S_{M1}$	1.155g
0.2-second Period, Design Earthquake Spectral Response Acceleration – $S_{DS}$ <b>(Note <math>S_{DS}</math> may be revised based on Ground Motion Hazard Analysis below)</b>	1.578g
1-second Period, Design Earthquake Spectral Response Acceleration – $S_{D1}$ <b>(Note <math>S_{DS}</math> may be revised based on Ground Motion Hazard Analysis below)</b>	0.770g
Revised 0.2-second Period, Design Earthquake Spectral Response Acceleration – $S_{DS}$ <b>(Site Specific Ground Motion Hazard Analysis per ASCE 7-05 and Modified as Discussed Below)</b>	1.825g
Revised 1-second Period, Design Earthquake Spectral Response Acceleration – $S_{D1}$ <b>(Site Specific Ground Motion Hazard Analysis per ASCE 7-05 and Modified as Discussed Below)</b>	1.573g

Note: 1 – for Site Class B, 5 percent damped  
 2 – Site Specific GMHA is performed for site located within 10 km of an active fault  
 3 – Site Specific GMHA recommended  $S_{DS}$  and  $S_{D1}$  design earthquake spectral accelerations exceed the 80 percent minimum requirement per ASCE 7-05

## 5.2 SITE SPECIFIC GROUND MOTION HAZARD ANALYSIS

As discussed above, we have performed our ground motion hazard analysis using NGA attenuation relations. Based on our previous discussion with Ms. Anne Rosinski, P.G. with CGS, this is acceptable provided the appropriate analysis procedures are followed in accordance with DSA Bulletin 09-01 summarized above.

As required in California Geologic Survey (CGS) Note 48 (Section 19) and in CBC Section 1614A.1.2, for a site located within 10 kilometers (km) of an active fault, a site specific Ground Motion Hazard Analysis (GMHA) is required and should be performed in accordance with Chapter 21, Section 21.2 through Section 21.4 of ASCE 7-05. We have outlined our procedures for this analysis and methodology below.

### 5.2.1 Probabilistic MCE – Section 21.2.1

To perform the site-specific GMHA, we first estimated the Probabilistic Maximum Considered Earthquake (MCE) spectral response accelerations represented by a 5 percent damped acceleration response spectrum having a 2 percent chance of exceedance in a 50 year period (2,475 year return period). To make these computations, we used EZ-FRISK version 7.37 by Risk Engineering Inc., to evaluate various ground motion levels at the site utilizing NGA relationships by Boore-Atkinson (2008), Campbell-Bozorgnia (2008), and Chiou-Youngs (2007). These attenuation relationships are based on the geometric mean of the maximum rotated component of ground motion and are currently being used for this type of analyses. This approach takes into account the geological slip rate of all active faults and background seismicity within 100 km (62 miles) of the site and the site-specific response characteristics. A mean value (50<sup>th</sup> percentile) of all these attenuation relationships was used to estimate the mean probabilistic response spectrum for the site. The result of our probabilistic analysis is graphically plotted on Figure 8.

### 5.2.2 Deterministic MCE – Section 21.2.2

The next step in our analysis was to evaluate the deterministic MCE response acceleration for a 5 percent damping for various periods using the same attenuation relationships as the probabilistic model. From a deterministic viewpoint, we modeled both the San Andreas ( $M_w$  7.9) and Monte Vista Shannon ( $M_w$  6.8) faults which are located about 2¼ and 1½ kilometers from the site, respectively. These two faults were chosen because they are the major contributors of ground motion to the site based on deaggregation of the probabilistic analysis. Then the largest seismic source with the largest expected moment magnitude ( $M_w$ ) of all deaggregated seismic sources within 100 kilometers of the site (i.e. the San Andreas fault) was determined and plotted on Figure 8.

The 84<sup>th</sup> percentile of the maximum rotated component of the ground motion was used to establish the 5 percent damped response spectrum. The 84<sup>th</sup> percentile ground motion was compared to the deterministic lower limit on the MCE response spectrum per Figure 21.2-1 of ASCE 7-05. The deterministic MCE Ground Motion Response Spectrum is determined by taking the larger spectral acceleration of the 84<sup>th</sup> percentile 5 percent damped response



spectrum but shall not be lower than the deterministic lower limit on MCE response spectrum. This deterministic MCE Ground Motion Response Spectrum is shown on Figure 9 (double black lines).

### 5.2.3 Site Specific MCE – Section 21.2.3

To determine the Site-Specific MCE spectral response acceleration, we took the lesser of the spectral response accelerations from the Probabilistic MCE of Section 21.2.1 (Section 5.2.1 of this report) and the Deterministic MCE of Section 21.2.2 (Section 5.2.2 of this report).

### 5.2.4 Code Design Response Spectrum – Section 11.4.5

The next step was to determine the Design Response Spectrum per the code and section referenced. The design spectral response accelerations,  $S_a$ , were based on the period and determined using appropriate equations (Equations 11.4-5 through 11.4-7). The design response spectrum and  $\frac{2}{3}$  of that design spectrum were determined and are shown in Figure 10.

### 5.2.5 Design Response Spectrum – Section 21.3

The project Design Response Spectrum (Project DRS) is defined as the maximum spectral acceleration values between the following two response spectra:

- 80 percent of the 2007 CBC Site Specific Code Design Response Spectrum (80 percent Code DRS)
- Two-thirds of the Deterministic MCE Site Specific Response Spectrum

The Code DRS was determined per Section 11.4.5 of ASCE 7-05 using the  $S_{DS}$  and  $S_{D1}$  values obtained from the USGS computer program (Java calculator) and presented in Table 2. The 80 percent Code DRS, the two-thirds Deterministic MCE Site Specific Response Spectra, and the Project DRS (double black lines) are plotted graphically on Figure 11.

### 5.2.6 Design Response Spectrum and Acceleration Parameters – Section 21.4

The final Design Response Spectrum is plotted on Figure 12. In accordance with Section 21.4 of ASCE 7-05, we determined design acceleration parameters  $S_{DS}$  and  $S_{D1}$  based on this final Design Response Spectrum. Based on this evaluation, we recommend  $S_{DS}$  be revised to 1.825g and that  $S_{D1}$  be revised to 1.573g. Accordingly,  $S_{MS}$  and  $S_{M1}$  should be revised to 2.738g and 2.360g, respectively. These acceleration parameters are also graphically shown on Figure 12 and meet the criteria outlined in ASCE 7-05 Section 21.4.

## **SECTION 6: CONCLUSIONS**

### **6.1 PRIMARY GEOTECHNICAL CONCERNS**

The proposed electrical infrastructure replacement project may be constructed as planned, provided design and construction is performed in accordance with recommendations presented in this report.

Primary geotechnical concerns are:

- Expansive weathered bedrock (serpentinite)
- Presence of undocumented fill
- Potential for difficult excavation

Brief outlines of our recommendations follow and additional recommendations provided in subsequent sections of this report.

#### **6.1.1 Expansive Weathered Bedrock**

The site is underlain by shallow and highly weathered bedrock. We estimate cuts on the order of 1 to 2 feet into the bedrock will be required for footings. In addition, we expect the building pad finished subgrade will be processed and replaced as engineered fill. Therefore, the bearing material is expected to consist of highly weathered rock that will behave more closely to residual soil. Based on laboratory test results, that residual soil has a moderate expansion potential.

Expansive soils can undergo significant volume change with changes in moisture content. They shrink and harden when dried and expand and soften when wetted. To reduce the potential for damage, the planned structure and slab-on-grade should have sufficient reinforcement and be supported on a layer of non-expansive fill; foundations should be designed to extend below the zone of seasonal moisture fluctuation or to resist uplift forces. In addition, it is important to limit moisture changes in the surficial soils by using positive drainage away from the building as well as limiting landscaping watering. Detailed grading and foundation recommendations addressing this concern are provided in Section 7 and 8.

#### **6.1.2 Presence of Undocumented Fill**

As discussed, undocumented fill was encountered during this investigation in borings EB-1 and EB-2 to a depth of approximately 1½ foot below existing ground surface. As with most past development, undocumented fill is likely present in other areas of the site and therefore should be expected. Construction of proposed improvements on undocumented fill may pose a risk to various structures and surface improvements such as buildings, sidewalks, and at-grade pavement areas because the fill may settle under imposed loads. Undocumented fill should be

completely removed and/or replaced with engineered fill, as required. Additional recommendations addressing this concern are provided in Section 7.2 of this report.

### **6.1.3 Potential for Difficult Excavation**

Our borings encountered highly weathered Franciscan greenstone and serpentinite bedrock of low rock hardness. Based on our understanding, we expect majority of excavations will be limited to minor cuts on the order of 2 to 10 feet. Based on these estimated depths, we anticipate excavations into the underlying bedrock will be required. In our opinion, the surficial fill soils and weathered bedrock should be relatively easy to excavate with heavy-duty excavating equipment (such as large backhoes or excavators). More competent bedrock areas may exist and require breaking with a hoe-ram. Additional recommendations are provided in Section 7.3.

## **6.2 PLANS AND SPECIFICATIONS REVIEW**

We recommend that we be retained to review the geotechnical aspects of the project structural, civil, and landscape plans and specifications, allowing sufficient time to provide the design team with any comments prior to issuing the plans for construction.

## **6.3 CONSTRUCTION OBSERVATION AND TESTING**

As site conditions may vary significantly between small-diameter and widely spaced borings performed during this investigation, we also recommend that a Cornerstone representative be present to provide geotechnical observation and testing during earthwork and foundation construction. This will allow us to form an opinion and prepare a letter at the end of construction regarding contractor compliance with project plans and specifications, and with the recommendations in our report. We will also be allowed to evaluate any conditions differing from those encountered during our investigation, and provide supplemental recommendations as necessary. For these reasons, the recommendations in this report are contingent of Cornerstone providing observation and testing during construction. Contractors should provide at least a 48-hour notice when scheduling our field personnel.

# **SECTION 7: EARTHWORK**

## **7.1 SITE DEMOLITION, CLEARING AND PREPARATION**

### **7.1.1 Site Stripping**

The proposed building and other site improvement areas should be stripped of all surface vegetation, and any existing surface and subsurface improvements. Demolition of existing improvements is discussed in detail below. Based on our observations, the majority of the new Building 30 footprint is currently covered by an existing soil stockpile. Remaining areas near the Building 3 and Building 16 are covered with asphalt concrete pavement. Where applicable, surface vegetation and topsoil should be stripped to a sufficient depth to remove all material greater than 3 percent organic content by weight.

### **7.1.2 Tree and Shrub Removal**

Where applicable, trees and shrubs designated for removal should have rootballs and any roots larger than ½-inch diameter removed completely. Grade depressions resulting from rootball removal should be cleaned of loose material and backfilled in accordance with the recommendations in the “Compaction” section of this report.

### **7.1.3 Demolition of Existing Slabs, Foundations and Pavements**

All slabs, foundations, and pavements scheduled for demolition should be completely removed from within planned building and improvement areas. Slabs, foundations, and pavements that extend into planned flatwork or pavement areas may be left in place provided there is at least 2 feet of engineered fill overlying the remaining materials and they are determined not to conflict with new utilities.

### **7.1.4 Abandonment of Existing Utilities**

If applicable, all utilities should be completely removed from within planned improvement areas. For any utility line to be considered acceptable to remain within an improvement area, the utility line must be completely backfilled with grout or sand-cement slurry (sand slurry is not acceptable), the ends outside the improvement area capped with concrete, and the trench fills either removed and replaced as engineered fill with the trench side slopes flattened to at least 1:1, or the trench fills are determined not to be a risk to the structure. The assessment of the level of risk posed by the particular utility line will determine whether the utility may be abandoned in place or needs to be completely removed. The contractor should assume that all utilities will be removed from within improvement areas unless provided written confirmation from both the owner and the geotechnical engineer.

Utilities extending beyond the improvement areas may be abandoned in place provided the ends are plugged with concrete, they do not conflict with planned improvements, and that the trench fills do not pose significant risk to the planned surface improvements.

The risks associated with abandoning utilities in place include the potential for future differential settlement of existing trench fills, and/or partial collapse and potential ground loss into utility lines that are not completely filled with grout. In general, the risk is relatively low for single utility lines less than 4 inches in diameter, and increases with increasing pipe diameter.

## **7.2 REMOVAL OF EXISTING FILLS**

Fill removal should extend a lateral distance of at least 5 feet beyond building footprints or to a lateral distance equal to fill depth below the perimeter footing, whichever is greater. Fills extending into planned pavement and flatwork areas may be left in place provided they are determined to be a low risk for future differential settlement and that the upper 12 inches of fill below pavement subgrade is re-worked and compacted. Areas where fill thickness is relatively shallow (one foot or less), the fill can be rework in-place. Deeper fill areas, if encountered,

should be entirely removed and replaced as recommended herein. We recommend earthwork contractors include a unit price for fill removal and compaction of undocumented fill. Side slopes of fill removals in building and pavement areas should be sloped at inclinations of 1:1 (horizontal to vertical) or flatter to minimize abrupt variations in fill thickness. Provided fills meet the “Material for Fill” requirements below, fills may be reused when backfilling excavations. Materials that do not meet these requirements, such as debris, wood, and trash should be screened out of the remaining material and not be reused. Backfill of excavations should be placed in lifts and compacted in accordance with the “Compaction” section below.

### **7.3 BEDROCK EXCAVATION**

As discussed, the site is generally underlain by highly weathered Franciscan greenstone and serpentinite bedrock of low hardness. Borings performed for this and previous investigations encountered the bedrock materials from the ground surface to approximately 3 feet below the ground surface except in the former underground tank location (EB-1, Lowney Associates, 2005). Based on the expected finished subgrade elevation for Building 30 and associated conduit trenches, we anticipate excavations into the underlying bedrock will be required.

In general, the upper 5 to 10 feet of the underlying bedrock can be excavated using conventional trench excavating equipment (heavy duty backhoes and excavators). However, localized areas of more competent rocks may exist and therefore should be expected. This is especially true for the greenstone rock encountered in the vicinity of Buildings 3 and 16. If encountered, these areas with stronger rock may require the use of hoe-rams to excavate or a different excavation technique. Additional handling and separation of oversized rock or materials may also be required prior to reusing for backfill. Cuts deeper than approximately 20 feet may encounter harder rock that might not be rippable with conventional excavation equipment. The contractor should be made aware of the difficult excavation condition below this depth and other excavation methods should be planned for.

### **7.4 TEMPORARY CUT AND FILL SLOPES**

The contractor is responsible for maintaining all temporary slopes and providing temporary shoring where required. Temporary shoring, bracing, and cuts/fills should be performed in accordance with the strictest government safety standards.

Excavations performed during site demolition and fill removal should be sloped at 3:1 (horizontal:vertical) within the upper 5 feet below building subgrade. Excavations extending more than 5 feet below building subgrade and excavations in pavement and flatwork areas should be slope at a 1:1 inclination unless the OSHA soil classification indicates that slope should not exceed 1.5:1.



## **7.5 SUBGRADE PREPARATION**

After site clearing and demolition is complete, and prior to backfilling any excavations resulting from fill removal or demolition, the excavation subgrade and subgrade within areas to receive additional site fills, slabs-on-grade and/or pavements should be scarified to a depth of 6 inches, moisture conditioned, and compacted in accordance with the “Compaction” section below. Subgrade consisting of weathered bedrock does not require scarification or pre-compaction.

## **7.6 MATERIAL FOR FILL**

### **7.6.1 Re-Use of On-site Soils**

On-site soils with an organic content less than 3 percent by weight may be reused as general fill. General fill should not have lumps, clods or cobble pieces larger than 6 inches in diameter; 85 percent of the fill should be smaller than 2½ inches in diameter. Minor amounts of oversize material (smaller than 12 inches in diameter) may be allowed provided the oversized pieces are not allowed to nest together and the compaction method will allow for loosely placed lifts not exceeding 12 inches.

### **7.6.2 Potential Import Sources**

Imported and non-expansive material should be inorganic with a Plasticity Index (PI) of 15 or less. To prevent significant caving during trenching or foundation construction, imported material should have sufficient fines. Samples of potential import sources should be delivered to our office at least 10 days prior to the desired import start date. Information regarding the import source should be provided, such as any site geotechnical reports. If the material will be derived from an excavation rather than a stockpile, potholes will likely be required to collect samples from throughout the depth of the planned cut that will be imported. At a minimum, laboratory testing will include PI tests. Material data sheets for select fill materials (Class 2 aggregate base, ¾-inch crushed rock, quarry fines, etc.) listing current laboratory testing data (not older than 6 months from the import date) may be provided for our review without providing a sample. If current data is not available, specification testing will need to be completed prior to approval.

Environmental and soil corrosion characterization should also be considered by the project team prior to acceptance. Suitable environmental laboratory data to the planned import quantity should be provided to the project environmental consultant; additional laboratory testing may be required based on the project environmental consultant’s review. The potential import source should also not be more corrosive than the on-site soils, based on pH, saturated resistivity, and soluble sulfate and chloride testing.

## **7.7 COMPACTION REQUIREMENTS**

All fills, and subgrade areas where fill, slabs-on-grade, and pavements are planned, should be placed in loose lifts 8 inches thick or less and compacted in accordance with ASTM D1557 (latest version) requirements as shown in Table 4. In general, clayey soils should be

compacted with sheepsfoot equipment and sandy/gravelly soils with vibratory smooth drum equipment; open-graded materials such as crushed rock should be placed in lifts no thicker than 18 inches consolidated in place with vibratory equipment. Each lift of fill and all subgrade should be firm and unyielding under construction equipment loading in addition to meeting the compaction requirements to be approved. The contractor (with input from a Cornerstone representative) should evaluate the in-situ moisture conditions, as the use of vibratory equipment on soils with high moistures can cause unstable conditions. General guidelines for soil stabilization are provided in the “Subgrade Stabilization Measures” section of this report. Where the soil’s PI is 20 or greater, the expansive soil criteria should be used.

**Table 4: Compaction Requirements**

Description	Material Description	Minimum Relative <sup>1</sup> Compaction (percent)	Moisture <sup>2</sup> Content (percent)
General Fill	On-Site Expansive Soils	87 – 92	>3
(within upper 5 feet)	On-Site Low Expansion Soils	90	>1
General Fill	On-Site Expansive Soils	95	>3
(below a depth of 5 feet)	On-Site Low Expansion Soils	95	>1
Wall Backfill	Without Surface Improvements	90	>1
Wall Backfill	With Surface Improvements	95 <sup>4</sup>	>1
Trench Backfill	On-Site Expansive Soils	87 – 92	>3
Trench Backfill	On-Site Low Expansion Soils	90	>1
Trench Backfill (upper 6 inches of pavement subgrade)	On-Site Low Expansion Soils	95	>1
Crushed Rock Fill	¾-inch Clean Crushed Rock	Consolidate In-Place	NA
Non-Expansive Fill	Imported Non-Expansive Fill	90	Near Optimum
Flatwork Subgrade	On-Site Expansive Soils	87 - 92	>3
Flatwork Subgrade	On-Site Low Expansion Soils	90	>1
Flatwork Aggregate Base	Class 2 Aggregate Base <sup>3</sup>	90	Near Optimum
Pavement Subgrade	On-Site Expansive Soils	87 - 92	>3
Pavement Subgrade	On-Site Low Expansion Soils	95	>1
Pavement Aggregate Base	Class 2 Aggregate Base <sup>3</sup>	95	Near Optimum
Asphalt Concrete	Asphalt Concrete	95 (Marshall)	NA

1 – Relative compaction based on maximum density determined by ASTM D1557 (latest version)

2 – Moisture content based on optimum moisture content determined by ASTM D1557 (latest version)

3 – Class 2 aggregate base shall conform to Caltrans Standard Specifications, latest edition, except that the relative compaction should be determined by ASTM D1557 (latest version)

4 – Using lightweight compaction or walls should be braced

## 7.8 TRENCH BACKFILL

Utility lines constructed within public right-of-way should be trenched, bedded and shaded, and backfilled in accordance with the local or governing jurisdictional requirements. Utility lines in private improvement areas should be constructed in accordance with the following requirements unless superseded by other governing requirements.

All utility lines should be bedded and shaded to at least 6 inches over the top of the lines with crushed rock (¾-inch-diameter or greater) or well-graded sand and gravel materials conforming to the pipe manufacturer's requirements. Open-graded shading materials should be consolidated in place with vibratory equipment and well-graded materials should be compacted to at least 90 percent relative compaction with vibratory equipment prior to placing subsequent backfill materials.

General backfill over shading materials may consist of on-site native materials provided they meet the requirements in the "Material for Fill" section, and are moisture conditioned and compacted in accordance with the requirements in the "Compaction" section.

Where utility lines will cross perpendicular to strip footings, the footing should be deepened to encase the utility line, providing sleeves or flexible cushions to protect the pipes from anticipated foundation settlement, or the utility lines should be backfilled to the bottom of footing with sand-cement slurry or lean concrete. Where utility lines will parallel footings and will extend below the "foundation plane of influence," an imaginary 1:1 plane projected down from the bottom edge of the footing, either the footing will need to be deepened so that the pipe is above the foundation plane of influence or the utility trench will need to be backfilled with sand-cement slurry or lean concrete within the influence zone. Sand-cement slurry used within foundation influence zones should have a minimum compressive strength of 75 psi.

## 7.9 SITE DRAINAGE

Surface water runoff should not be allowed to pond adjacent to building foundations, slabs-on-grade, or pavements. New hardscape surfaces should slope at least 2 percent towards suitable discharge facilities; landscape areas should slope at least 3 percent.

# SECTION 8: FOUNDATIONS

## 8.1 SUMMARY OF RECOMMENDATIONS

In our opinion, the proposed Building 30 may be supported on shallow foundations provided the recommendations in the "Earthwork" section and the sections below are followed.

## 8.2 SEISMIC DESIGN CRITERIA

The 2007 California Building Code (CBC) "Seismic Coefficients," the recommended Project Design Response Spectrum, and recommended revisions to the CBC Seismic Coefficients are

presented in Section 5, “CBC Seismic Design Criteria,” of this report. That section including Table 3 should be referred to for applicable seismic design criteria and values.

### **8.3 SHALLOW FOUNDATIONS**

In our opinion, the new Building 30 may be supported on spread footings. Detailed foundation recommendations are provided in subsequent sections of this report.

#### **8.3.1 Spread Footings**

Foundations should consist of conventional continuous perimeter strip and isolated interior, shallow spread footings. Spread footings should bear entirely on natural competent rock or engineered fill as recommended herein. Footings should not be allowed to span both engineered fill and weathered rock or soil and rock. Footings should extend at least 18 inches below the lowest adjacent finished grade. Lowest adjacent grade is defined as the deeper of the following:

- bottom of the adjacent interior slab-on-grade, or
- finished exterior grade, excluding landscaping topsoil.

Based on the planned finished floor at Elevation 602 feet, footings are expected to bear in the weathered rock based on an 18-inch footing depth to approximately Elevation 600.5 feet. As previously discussed, undocumented fill is expected to extend to approximately 1½ feet below the existing ground surface or equivalent to approximately Elevation 601 feet.

Footings bear entirely on competent rock and constructed in accordance with above recommendations would be capable of supporting a maximum allowable bearing pressure of 4,000 psf for dead loads, 6,000 psf for combined dead plus live loads, and 8,000 psf for all loads including wind and seismic. If footings bear entirely on engineered fill instead of competent rock, these allowable bearing pressures need to be reduced to 2,000psf for dead loads, 3,000 psf for combined dead plus live loads, and 4,000 psf for all loads including wind and seismic. Above pressures are based on factors of safety of 3.0, 2.0, and 1.5 applied to the ultimate bearing pressure for dead, dead plus live, and all loads, respectively. These pressures are net values; the weight of the footing may be neglected.

#### **8.3.2 Footing Settlement**

For our settlement estimates, we have used an estimated column load of approximately 50 kips and a perimeter strip footing load of less than 1 kip per linear foot. Based on these structural loads, we estimate footings will undergo total settlements of ½ inch with post-construction differential settlement of less than approximately ½ inch between adjacent foundation elements under static loads.

### 8.3.3 Lateral Loading

Lateral loads may be resisted by friction between the bottom of footing and the supporting subgrade, and also by passive pressures generated against footing sidewalls. An ultimate frictional resistance of 0.45 applied to the footing dead load, and an ultimate passive pressure based on an equivalent fluid pressure of 450 pcf may be used in design. The structural engineer should apply an appropriate factor of safety (such as 1.5) to the ultimate values above. Where footings are adjacent to landscape areas without hardscape, the upper 12 inches of soil should be neglected when determining lateral passive pressure resistance.

### 8.3.4 Shallow Footing Construction Considerations

Where utility lines will cross perpendicular to strip footings, the footing should be deepened to encase the utility line, providing sleeves or flexible cushions to protect the pipes from anticipated foundation settlement, or the utility lines should be backfilled to the bottom of footing with sand-cement slurry or lean concrete. Where utility lines will parallel footings and will extend below the “foundation plane of influence,” an imaginary 1:1 plane projected down from the bottom edge of the footing, either the footing will need to be deepened so that the pipe is above the foundation plane of influence or the utility trench will need to be backfilled with sand-cement slurry or lean concrete within the influence zone. Sand-cement slurry used within foundation influence zones should have a minimum compressive strength of 75 psi.

Footing excavations should be filled as soon as possible or be kept moist until concrete placement by regular sprinkling to prevent desiccation. A Cornerstone representative should observe all footing excavations prior to placing reinforcing steel and concrete. If there is a significant schedule delay between our initial observation and concrete placement, we may need to re-observe the excavations.

## SECTION 9: CONCRETE SLABS AND PEDESTRIAN PAVEMENTS

### 9.1 INTERIOR SLABS-ON-GRADE

As discussed previously, the new Building 30 planned finished floor will be at approximately Elevation 602 feet. As a result, the slab-on-grade floor is expected to bear near the soil and bedrock interface. Appropriate under-slab construction and requirements for soil and rock conditions are provided herein.

If at least 12 inches of low expansive engineered fill material is used to build the pad, the proposed slab-on-grade may be supported directly on subgrade prepared in accordance with the recommendations in the “Earthwork” section of this report. However, if the slab-on-grade will bear on the underlying weathered serpentinite rock, the proposed slab-on-grade should be supported on at least 12 inches of non-expansive fill (NEF) to reduce the potential for slab damage due to soil heave. The NEF layer should be constructed over subgrade prepared in accordance with the recommendations in the “Earthwork” section of this report. If significant time elapses between initial subgrade preparation and NEF construction, the subgrade should be proof-rolled to confirm subgrade stability, and if the soil has been allowed to dry out, the

subgrade should be re-moisture conditioned to at least 3 percent over the optimum moisture content.

## **9.2 INTERIOR SLABS MOISTURE PROTECTION CONSIDERATIONS**

If moisture-sensitive floor coverings are planned, the recommendations below may be incorporated in the project design if desired. If significant time elapses between initial subgrade preparation and slab-on-grade construction, the subgrade should be proof-rolled to confirm subgrade stability, and if the soil has been allowed to dry out, the subgrade should be re-moisture conditioned to near optimum moisture content.

The structural engineer should determine the appropriate slab reinforcement for the loading requirements and considering the expansion potential of the underlying materials, as required. Consideration should be given to limiting the control joint spacing to a maximum of about 2 feet in each direction for each inch of concrete thickness.

The following general guidelines for concrete slab-on-grade construction where floor coverings are planned are presented for the consideration by the District, design team, and contractor. These guidelines are based on information obtained from a variety of sources, including the American Concrete Institute (ACI) and are intended to reduce the potential for moisture-related problems causing floor covering failures, and may be supplemented as necessary based on project-specific requirements. The application of these guidelines or not will not affect the geotechnical aspects of the slab-on-grade performance.

- Place a minimum 10-mil vapor retarder conforming to ASTM E 1745, Class C requirements or better directly below the concrete slab; the vapor retarder should extend to the slab edges and be sealed at all seams and penetrations in accordance with manufacturer's recommendations and ASTM E 1643 requirements. A 4-inch-thick capillary break, consisting of ½- to ¾-inch crushed rock with less than 5 percent passing the No. 200 sieve, should be placed below the vapor retarder and consolidated in place with vibratory equipment. The capillary break rock may be considered as the upper 4 inches of the non-expansive fill previously recommended.
- The concrete water:cement ratio should be 0.45 or less. Mid-range plasticizers may be used to increase concrete workability and facilitate pumping and placement.
- Water should not be added after initial batching unless the slump is less than specified and/or the resulting water:cement ratio will not exceed 0.45.
- Where floor coverings are planned, all concrete surfaces should be properly cured.
- Water vapor emission levels and concrete pH should be determined in accordance with ASTM F1869-98 and F710-98 requirements and evaluated against the floor covering manufacturer's requirements prior to installation.



### 9.3 PEDESTRIAN CONCRETE FLATWORK

Exterior concrete flatwork subject to pedestrian and/or occasional light pick up loading should be at least 4 inches thick and supported on at least 6 inches of Class 2 aggregate base. The subgrade should be prepared in accordance with the “Earthwork” recommendations of this report. To help reduce the potential for uncontrolled shrinkage cracking, adequate expansion and control joints should be included. Consideration should be given to limiting the control joint spacing to a maximum of about 2 feet in each direction for each inch of concrete thickness.

## SECTION 10: VEHICULAR PAVEMENTS

### 10.1 ASPHALT CONCRETE

The asphalt concrete pavement recommendations presented in Table 4 are based on Procedure 608 of the Caltrans Highway Design Manual and a design R-value of 15. This design R-value was chosen based on R-value tests previously performed, experience with similar soil types, and our engineering judgment considering the variable soil conditions. We recommend the project civil engineer determine the appropriate TI and pavement section alternatives based on anticipated loading conditions.

**Table 4: Asphalt Concrete Pavement Recommendations**  
**Estimated Design R-value = 15**

Design Traffic Index (TI)	Asphalt Concrete (inches)	Class 2 Aggregate Base (inches)	Total Pavement Section Thickness (inches)
4.0	2.5	6.0	8.5
4.5	2.5	7.5	10.0
5.0	3.0	8.0	11.0
5.5	3.0	10.0	13.0
6.0	3.5	10.5	14.0
6.5	4.0	11.5	16.0
7.0	4.0	15.0	19.0

Note: 1 Caltrans Class 2 aggregate base; minimum R-value of 78

### 10.2 PAVEMENT CUTOFF

Surface water penetration into the pavement section can significantly reduce the pavement life, due to the high clay content of near-surface soils. While quantifying the life reduction is difficult, a normal 20-year pavement design could be reduced to less than 10 years; therefore, increased long-term maintenance may be required.

It would be beneficial to include a pavement cut-off, such as deepened curbs, redwood-headers, or “Deep-Root Moisture Barriers” that are keyed at least 6 inches into the pavement subgrade. This will help limit the additional long-term maintenance.

## **SECTION 11: LIMITATIONS**

This report, an instrument of professional service, has been prepared for the sole use of the San Mateo County Community College District and their representatives specifically to support the design of the Cañada College Electrical Infrastructure Replacement projects in Redwood City, California. The opinions, conclusions, and recommendations presented in this report have been formulated in accordance with accepted geotechnical engineering practices that exist in Northern California at the time this report was prepared. No warranty, expressed or implied, is made or should be inferred.

Recommendations in this report are based upon the soil and ground water conditions encountered during our subsurface exploration. If variations or unsuitable conditions are encountered during construction, Cornerstone must be contacted to provide supplemental recommendations, as needed.

The San Mateo County Community College District may have provided Cornerstone with plans, reports and other documents prepared by others. The San Mateo County Community College District understands that Cornerstone reviewed and relied on the information presented in these documents and cannot be responsible for their accuracy.

Cornerstone prepared this report with the understanding that it is the responsibility of the owner or his representatives to see that the recommendations contained in this report are presented to other members of the design team and incorporated into the project plans and specifications, and that appropriate actions are taken to implement the geotechnical recommendations during construction.

Conclusions and recommendations presented in this report are valid as of the present time for the development as currently planned. Changes in the condition of the property or adjacent properties may occur with the passage of time, whether by natural processes or the acts of other persons. In addition, changes in applicable or appropriate standards may occur through legislation or the broadening of knowledge. Therefore, the conclusions and recommendations presented in this report may be invalidated, wholly or in part, by changes beyond Cornerstone's control. This report should be reviewed by Cornerstone after a period of three (3) years has elapsed from the date of this report. In addition, if the current project design is changed, then Cornerstone must review the proposed changes and provide supplemental recommendations, as needed.

An electronic transmission of this report may also have been issued. While Cornerstone has taken precautions to produce a complete and secure electronic transmission, please check the electronic transmission against the hard copy version for conformity.

Recommendations provided in this report are based on the assumption that Cornerstone will be retained to provide observation and testing services during construction to confirm that conditions are similar to that assumed for design, and to form an opinion as to whether the work has been performed in accordance with the project plans and specifications. If we are not retained for these services, Cornerstone cannot assume any responsibility for any potential

claims that may arise during or after construction as a result of misuse or misinterpretation of Cornerstone's report by others. Furthermore, Cornerstone will cease to be the Geotechnical-Engineer-of-Record if we are not retained for these services.

## **SECTION 12: REFERENCES**

Bortugno, E.J., McJunkin, R.D., and Wagner, D.L., 1991, Map showing recentcy of faulting, San Francisco-San Jose quadrangle, California: California Division of Mines and Geology Regional Geologic Map Series, Map 5A, Sheet 5, scale 1: 250,000.

Brabb, E.E., and Olson, J.A., 1986, Map showing faults and earthquake epicenters in San Mateo County, California: U.S. Geological Survey Miscellaneous Investigations Series Map I-1257-F, scale 1:62,500.

Brabb, E.E., and Pampeyan, E.H., 1983, Geologic map of San Mateo County, California: U.S. Geological Survey Miscellaneous Investigations Series Map I-1257-A, scale 1:62,500.

Brabb, E.E., Pampeyan, E.H., and Bonilla, M.G., 1972, Landslide susceptibility in San Mateo County, California: U.S. Geological Survey Basic Data Contribution 43, scale 1:62,500.

California Building Code, 2007, Structural Engineering Design Provisions, Vol. 2.

California Department of Conservation Division of Mines and Geology, 1998, Maps of Known Active Fault Near-Source Zones in California and Adjacent Portions of Nevada, International Conference of Building Officials, February, 1998.

California Division of Mines and Geology (1997), "Guidelines for Evaluating and Mitigating Seismic Hazards in California, Special Publication 117A, March.

Cornerstone Earth Group, 2008, Geotechnical Investigation and Geologic Evaluation, Cañada College Building 5/6 Renovations, Redwood City, California, unpublished report number 177-1-3 for San Mateo County Community College District dated March 20, 2008.

Cornerstone Earth Group, 2008, Geotechnical Investigation, Cañada College Faculty Housing, 4200 Farm Hill Boulevard, Redwood City, California, unpublished report number 259-1-1 for Thompson Dorfman Partners, LLC dated July 31, 2008.

Dames & Moore, 1959, Soil and Foundation Investigation, Proposed College Heights Campus near Crystal Springs Road and Parrott Drive, San Mateo County, California, unpublished report for San Mateo Junior College District, dated May 27, 1959

Knudsen, K.L., Sowers, J.M., Witter, R.C., Wentworth, C.M., and Helley, E.J., 2000, Preliminary maps of Quaternary deposits and liquefaction susceptibility, nine-county San Francisco Bay region, California: a digital database: U.S. Geological Survey Open-File Report 00-444, scale 1:275,000.

Leighton and Associates Geotechnical Engineers, 1976, Geotechnical Hazards Synthesis Map for San Mateo County, sheet 1 of 5, scale 1: 24,000.

Lowney Associates, 2005, Geotechnical Investigation, New Chiller Slab Platform and Central Plant Cooling Towers, Cañada College, Redwood City, California, unpublished report number 1620-25 for San Mateo County Community College District dated February 2, 2005.

Mark, R.K., Newman, E.B., and Brabb, E.E., 1988, Slope map of San Mateo County, California: U.S. Geological Survey Miscellaneous Investigations Series Map I-1257-J, scale 1:62,500.

Nason, R.D., 1980, Damage in San Mateo County, California, in the earthquake of 18 April 1906: U.S. Geological Survey Open File Report 80-176, 49 p.

Pampeyan, E.H., 1993, Geologic map of the Palo Alto and part of the Redwood Point 7 1/2' quadrangles, San Mateo and Santa Clara counties, California: U.S. Geological Survey Miscellaneous Investigations Series Map I-2371, scale 1:24,000.

Pampeyan, E.H., 1994, Geologic map of the Montara Mountain and San Mateo 7.5-minute quadrangles, San Mateo County, California: U.S. Geological Survey Miscellaneous Investigation Series Map I-2390, scale 1:24,000.

San Mateo County, 2008, Hazards Mitigation Maps: on-line site at [http://www.co.sanmateo.ca.us/smc/departments/home/0,5557771\\_5558929\\_436489912,00.html](http://www.co.sanmateo.ca.us/smc/departments/home/0,5557771_5558929_436489912,00.html)

Ritter, J.R., and Dupre, W.R., 1972, Map Showing Areas of Potential Inundation by Tsunamis in the San Francisco Bay Region, California: San Francisco Bay Region Environment and Resources Planning Study, USGS Basic Data Contribution 52, Misc. Field Studies Map MF-480.

State of California Department of Transportation, 1990, Highway Design Manual, Fifth Edition, July 1, 1990.

Topozada, T. et. al., 2000, Epicenters of and Areas Damaged by M>5 California Earthquakes, 1800-1999, pages 861 – 878.

Townley, S.D. and M.W. Allen, 1939, Descriptive Catalog of Earthquakes of the Pacific Coast of the United States, 1769 to 1928: Bulletin of the Seismological Society of America, Vol. 29, No. 1, pp. 1247-1255.

USGS, 2007, Earthquake Ground Motion Parameters, Version 5.0.9, revision date October 6, 2008 - A Computer Program for determining mapped ground motion parameters for use with IBC 2006 available at <http://earthquake.usgs.gov/research/hazmaps/design/index.php>.  
Working Group on California Earthquake Probabilities, 2003, Probabilities of Large Earthquakes in the San Francisco Bay Region, California: U.S.G.S. Fact Sheet 039-03.

Working Group on California Earthquake Probabilities, 2008, Uniform California Earthquake Rupture Forecast, Version 2 (UCERF), U.S. Geological Survey Open File Report 2007-1437 (CGS Special Report 203; SCEC Contribution #1138).

Youd, T.L. and C.T. Garris, 1995, Liquefaction-Induced Ground-Surface Disruption: Journal of Geotechnical Engineering, Vol. 121, No. 11, pp. 805 - 809.

Youd, T.L. and Idriss, I.M., et al, 1997, Proceedings of the NCEER Workshop on Evaluation of Liquefaction Resistance of Soils: National Center for Earthquake Engineering Research, Technical Report NCEER - 97-0022, January 5, 6, 1996.

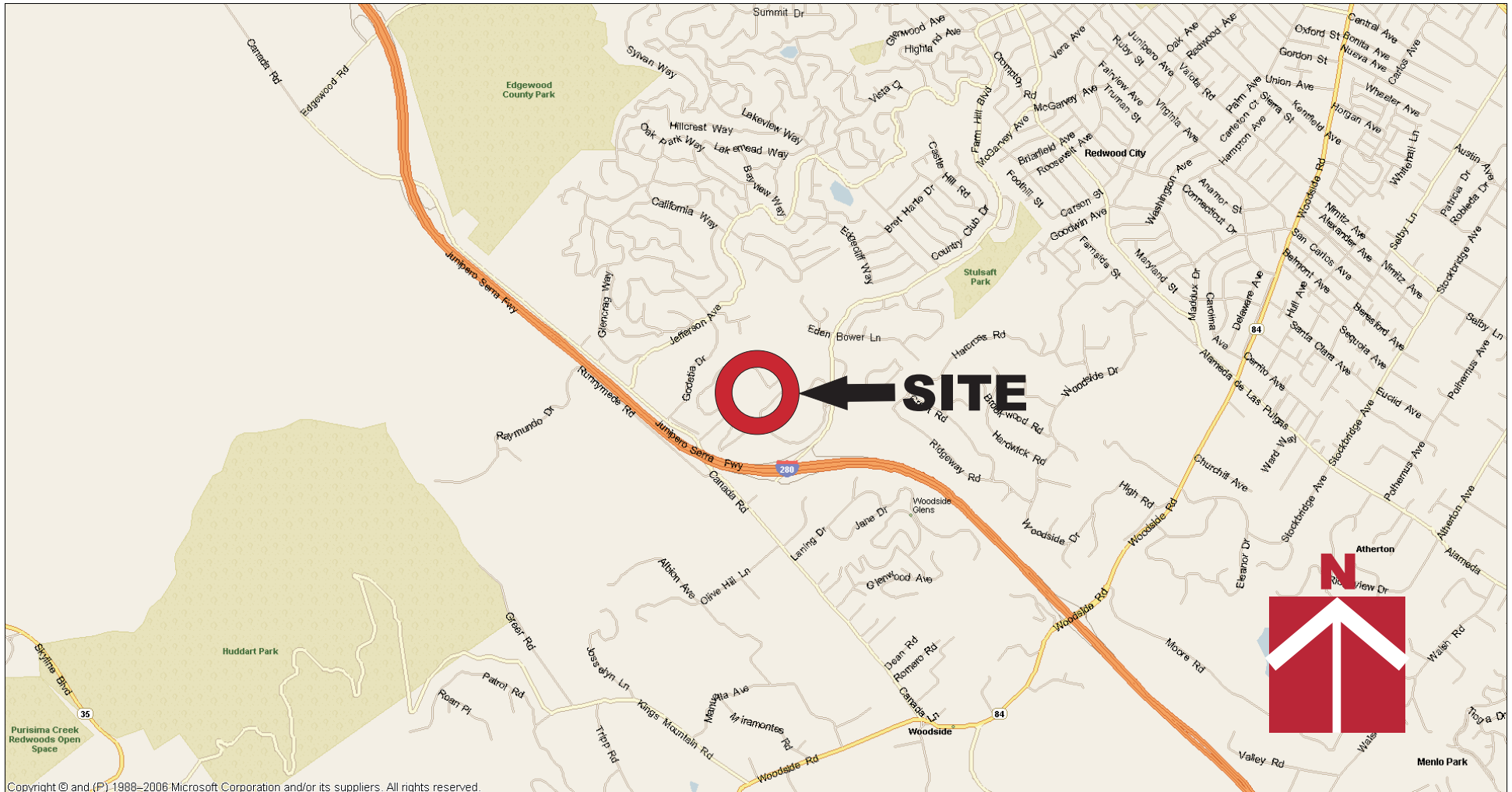
Youd et al., 2001, "Liquefaction Resistance of Soils: Summary Report from the 1996 NCEER and 1998 NCEER/NSF Workshops on Evaluation of Liquefaction Resistance of Soils," ASCE Journal of Geotechnical and Geoenvironmental Engineering, Vo. 127, No. 10, October, 2001.

Youd, T.L. and Hoose, S.N., 1978, Historic Ground Failures in Northern California Triggered by Earthquakes, United States Geologic Survey Professional Paper 993.

**AERIAL PHOTOS REVIEWED AT U.S. GEOLOGICAL SURVEY, MENLO PARK, CA:**

Geomorphic features on the following aerial photographs were interpreted at the U.S. Geological Survey in Menlo Park as part of this investigation:

<b>Date</b>	<b>Flight</b>	<b>Frames</b>	<b>Scale</b>	<b>Type</b>
October 05, 1943	DDB	1B-100, -101	1:20,000	black & white
June 7, 1973	3567	4-078, 079	1:12,000	black & white



Copyright © and (P) 1988–2006 Microsoft Corporation and/or its suppliers. All rights reserved.



**CORNERSTONE**  
**EARTH GROUP**

**Vicinity Map**

**Cañada College Electrical  
Infrastructure Replacement  
Redwood City, CA**

Project Number

177-1-7

Figure Number

Figure 1

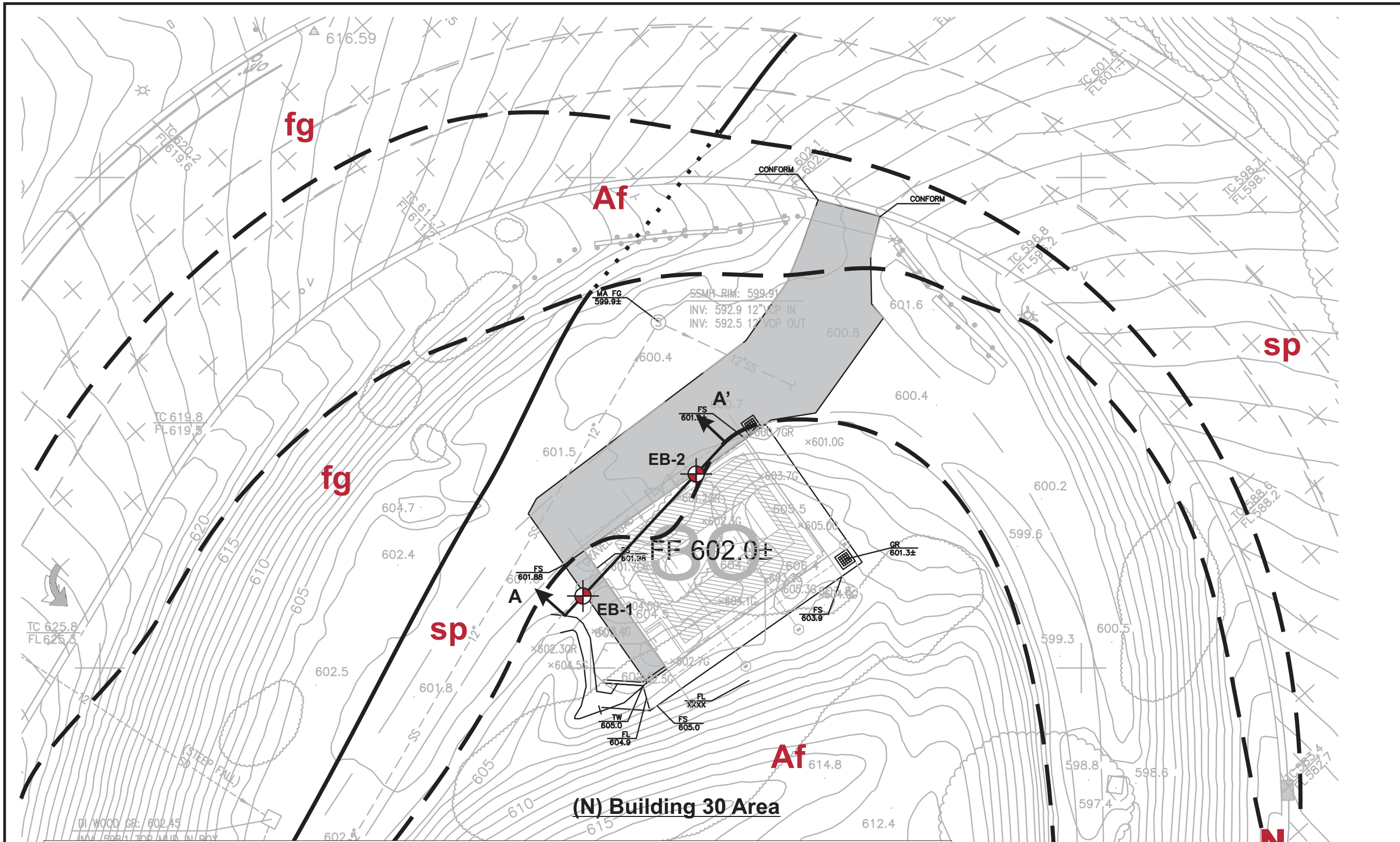
Date

April 2010

Drawn By

MGV

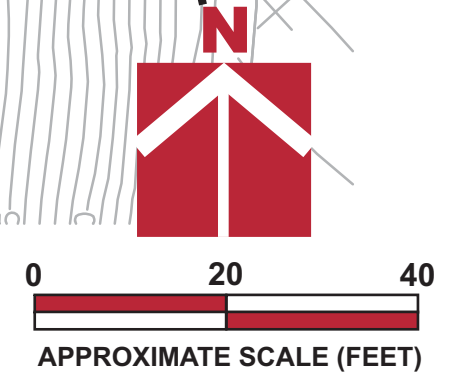




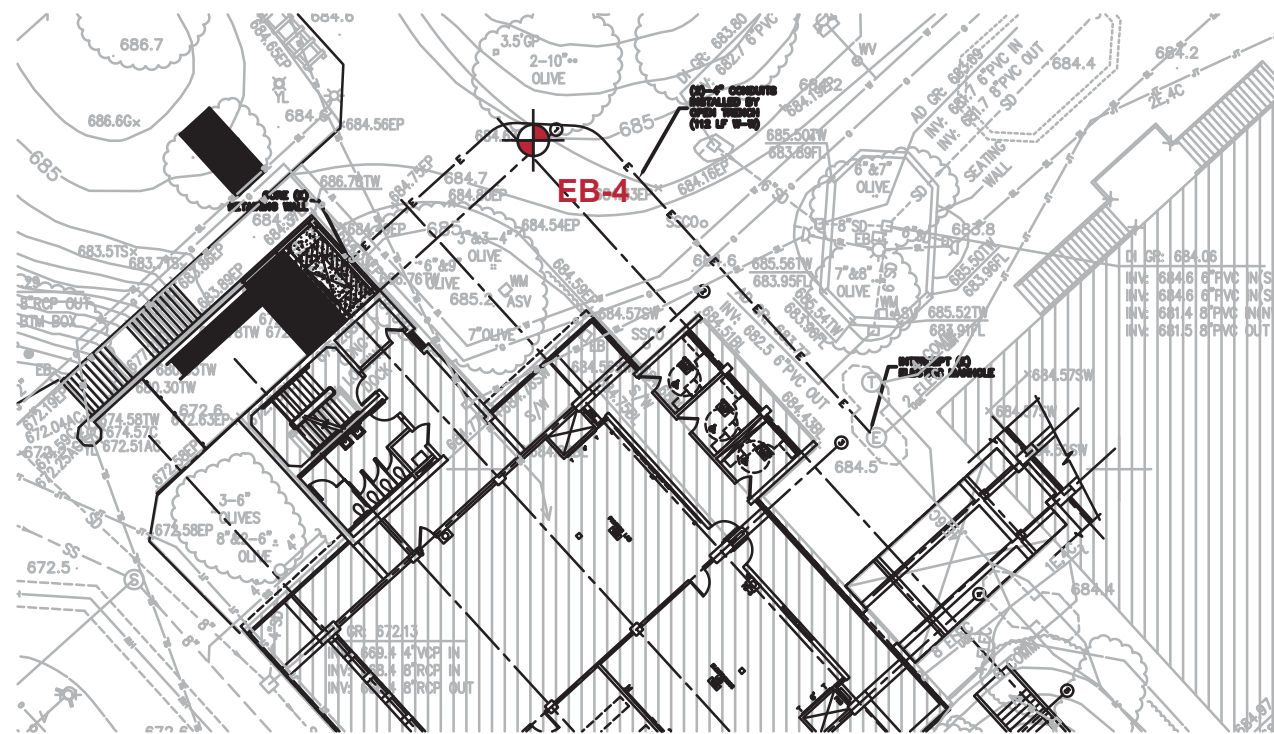
Site Plan and Geologic Map  
**Cañada College Electrical  
 Infrastructure Replacement**  
**Redwood City, CA**

**CORNERSTONE  
 EARTH GROUP**

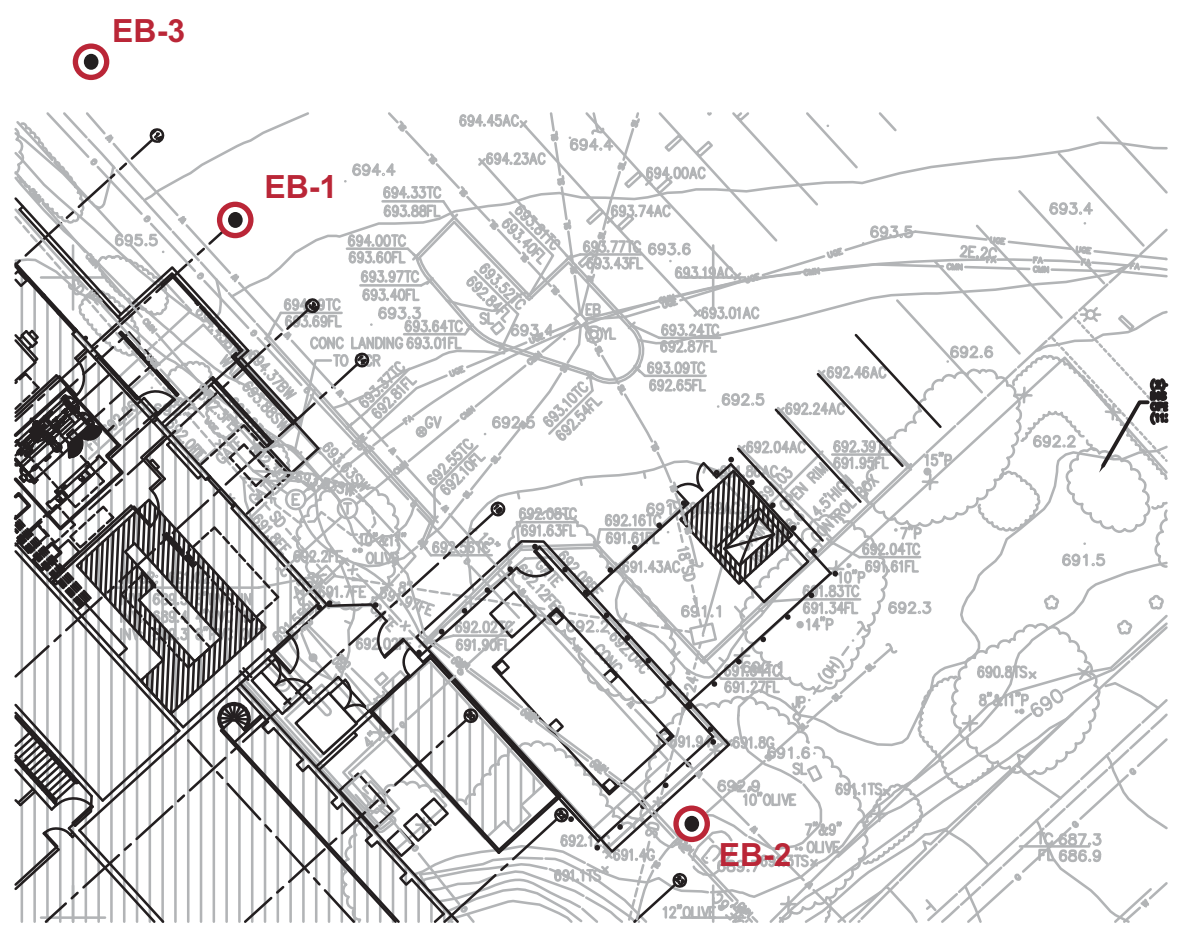
Map Symbols/Units		Legend	
	Geologic contact; dashed where approximately located.	Af	Artificial fill
	Postulated (inactive) fault	fg	Franciscan Complex Greenstone
		sp	Serpentinite
			Approximate Location of Exploratory Boring by Cornerstone Earth Group, April 2010



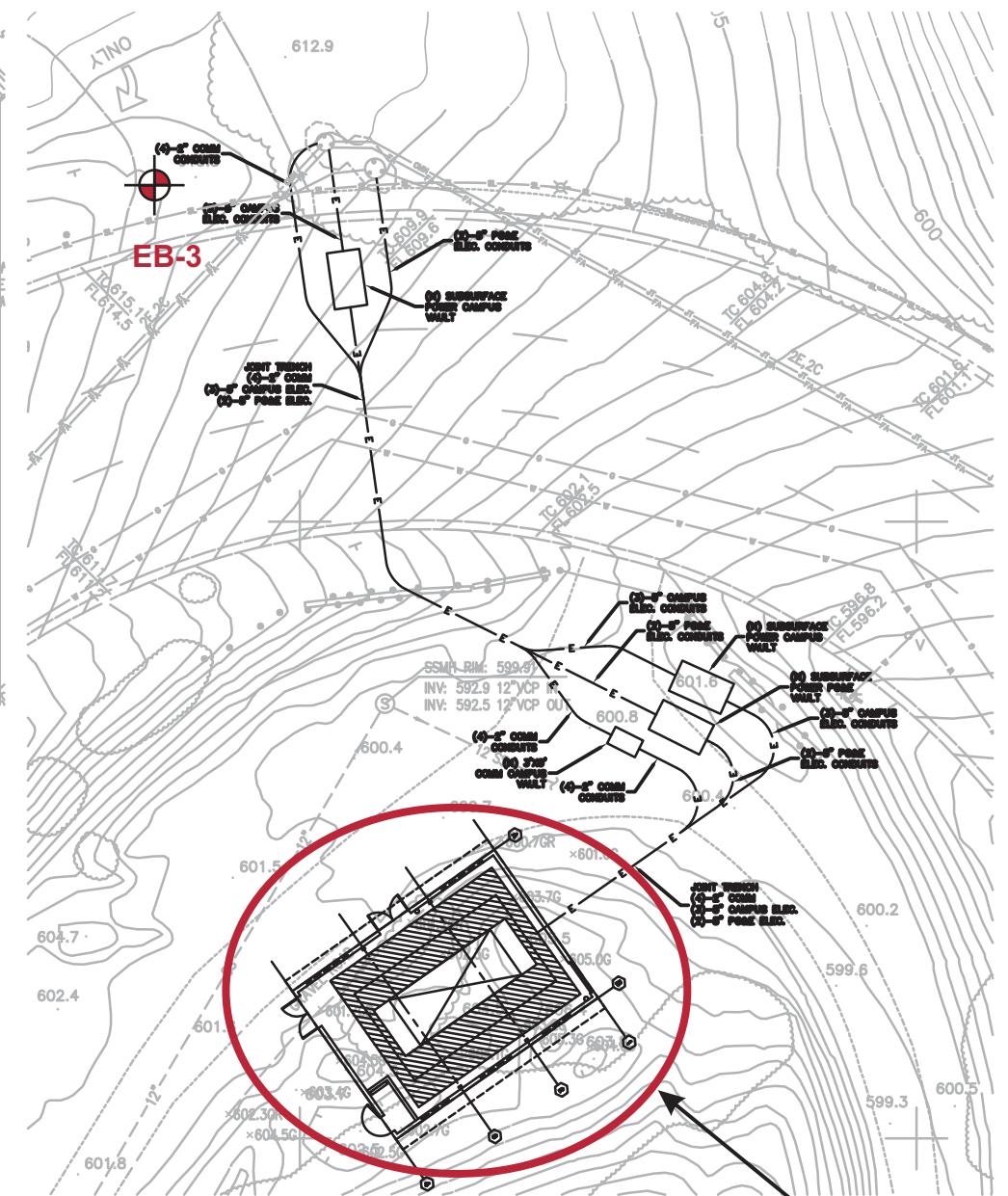




**BUILDING 16 AREA**





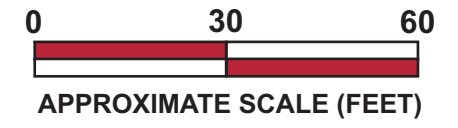
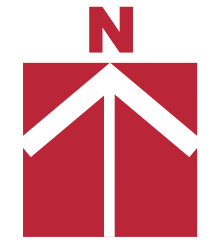
**BUILDING 3 AREA**



**(N) BUILDING 30 AREA**

Refer to Figure 2A for Details

-  Approximate Location of Exploratory Boring by Cornerstone Earth Group, April 2010
-  Approximate Location of Exploratory Boring by Lowney Associates, February 2005



Project Number  
177-1-7

Figure Number  
Figure 2B

Date  
April 2010

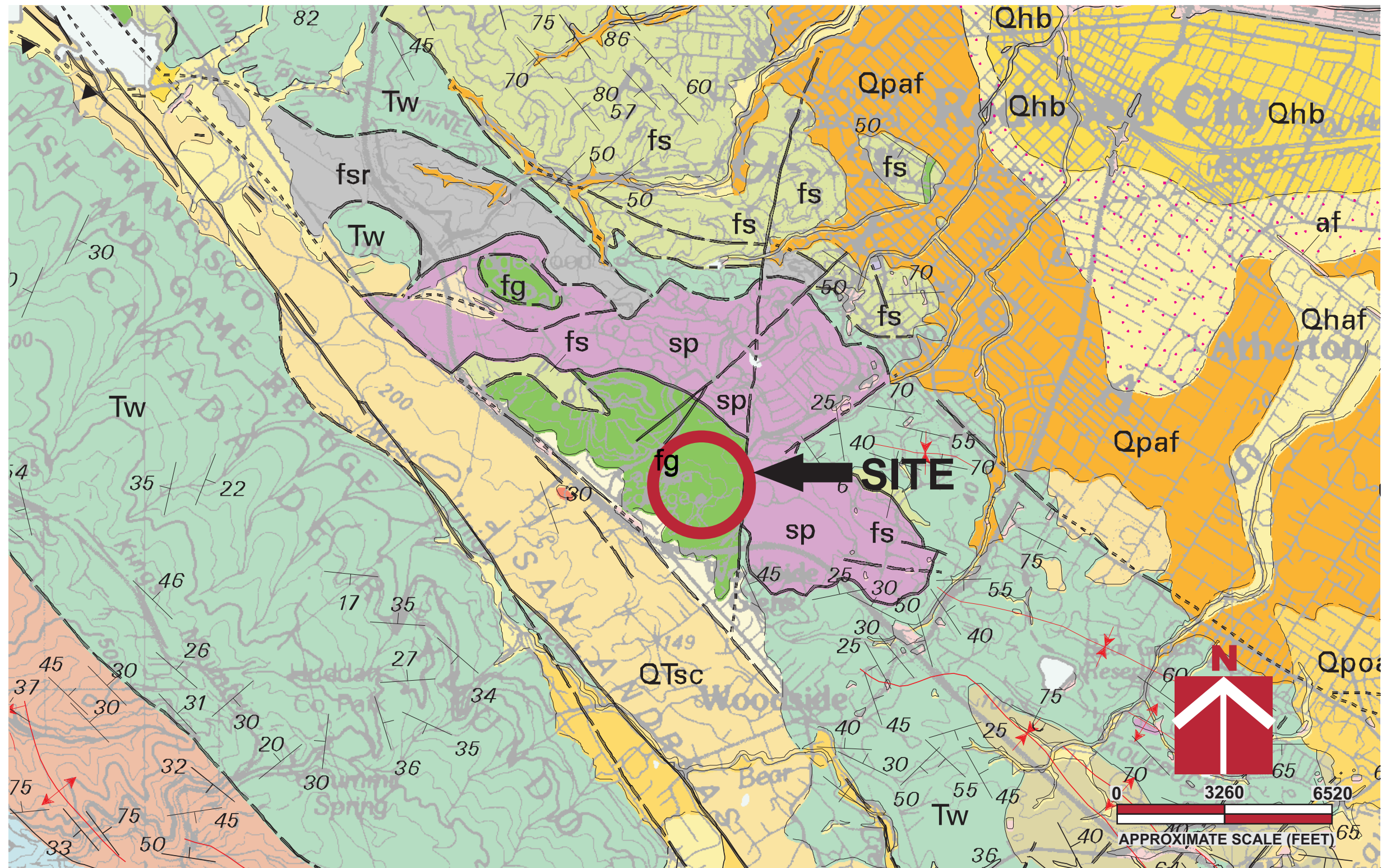
Drawn By  
MGV

Site Plan

**Cañada College Electrical  
Infrastructure Replacement  
Redwood City, CA**








**Local Geologic Units**

- QTsc** Santa Clara Formation (lower Pleistocene and upper Pliocene)
- Tw** Whiskey Hill Formation (middle and lower Eocene)
- fg** Franciscan Complex Greenstone (Cretaceous and Jurassic)
- fs** Franciscan Complex Sandstone (Cretaceous and Jurassic)
- sp** Serpentinite (Cretaceous and/or Jurassic)

**Geologic Symbols**

- — — — — Geologic contact; dashed where approximately located
- - - - - Fault; dashed where approximately located; short dashes where inferred
-  strike and dip of bedding

Note: Geologic map from E.E. Brabb, R.W. Graymer, and D.L. Jones (2000).

Project Number  
177-1-7

Figure Number  
Figure 3

Date  
April 2010

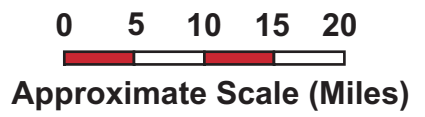
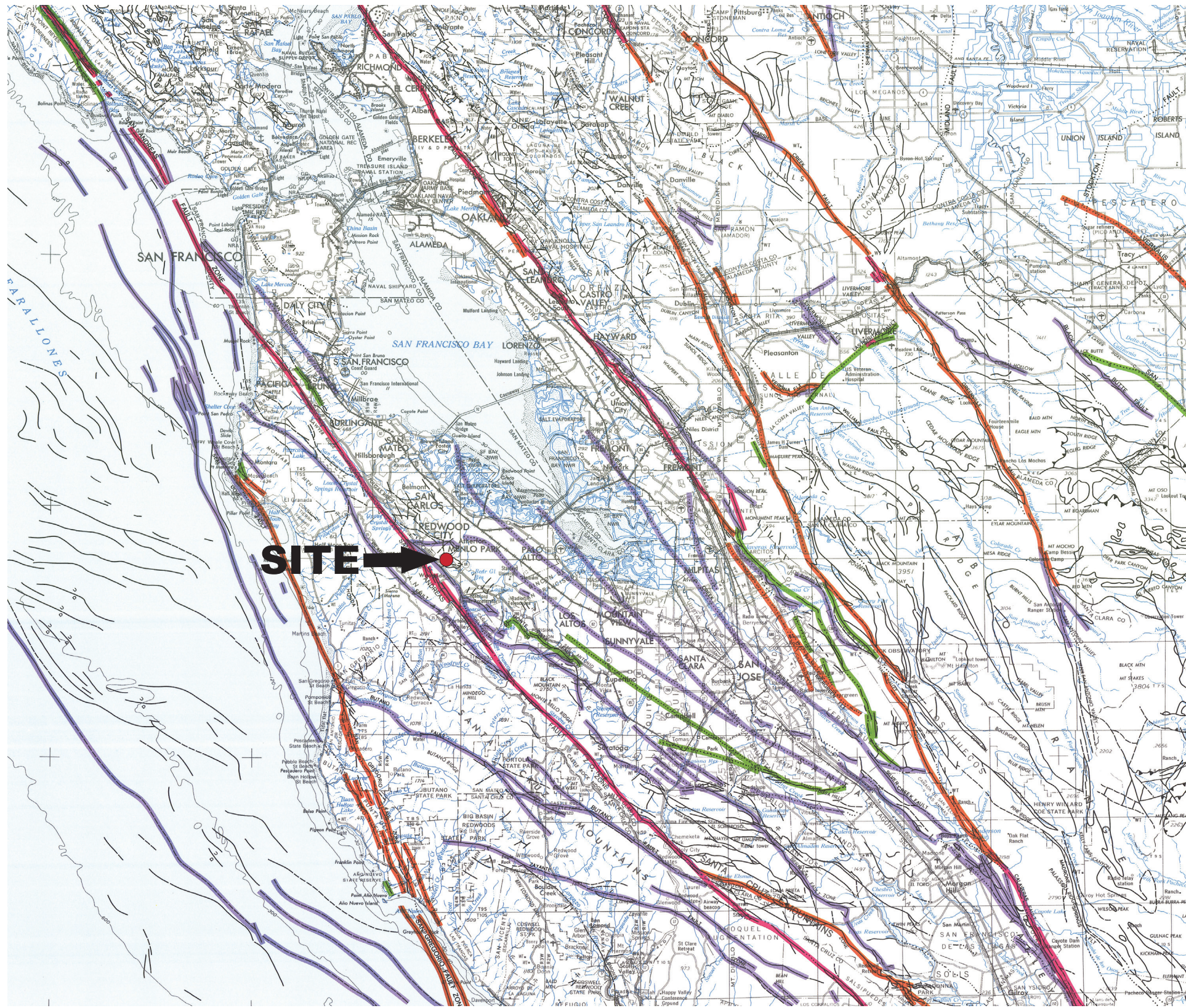
Drawn By  
MGV

Vicinity Geologic Map

**Cañada College Electrical  
Infrastructure Replacement  
Redwood City, CA**







Geologic Time Scale	Years Before Present (Approx.)	Fault Symbol	Recency of Movement on Land Offshore <sup>1</sup>	DESCRIPTION
Quaternary	Holocene/Historic			Displacement during historic time (e.g. San Andreas fault 1906). Includes areas of known fault creep.
	Late Quaternary			Displacement during Holocene time. <sup>2</sup>
Early Quaternary	Pleistocene			Faults showing evidence of displacement during late Quaternary time. <sup>3,4</sup>
				Quaternary (undifferentiated) faults – most faults in this category show evidence of displacement during the last 2,000,000 years; possible exceptions are faults which displace rocks of undifferentiated Pliocene age.
Pre-Quaternary	Pliocene			
	Miocene			Fault showing evidence of no displacement during Quaternary time or faults without recognized Quaternary displacement.

Base Map: "Map showing recency of faulting, San Francisco - San Jose Quadrangle California," by Bortugno, et.al. (California Division of Mines and Geology) dated 1991.

Project Number  
177-1-7

Figure Number  
Figure 4

Date  
April 2010

Drawn By  
MGV

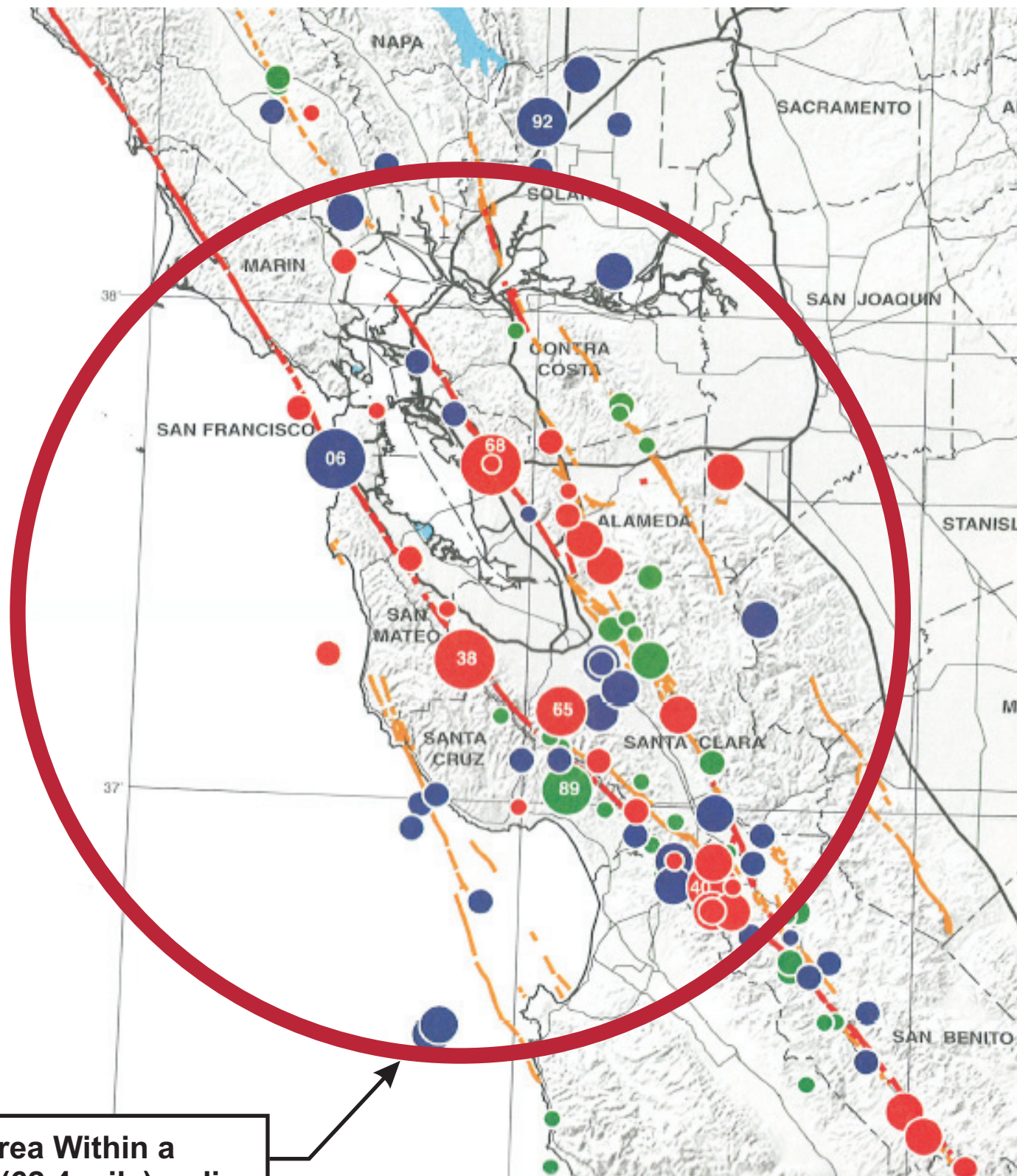
Regional Fault Map

**Cañada College Electrical Infrastructure Replacement Redwood City, CA**

**CORNERSTONE**  
**EARTH GROUP**

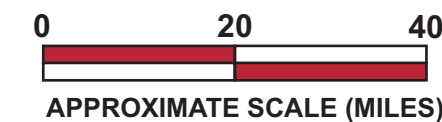
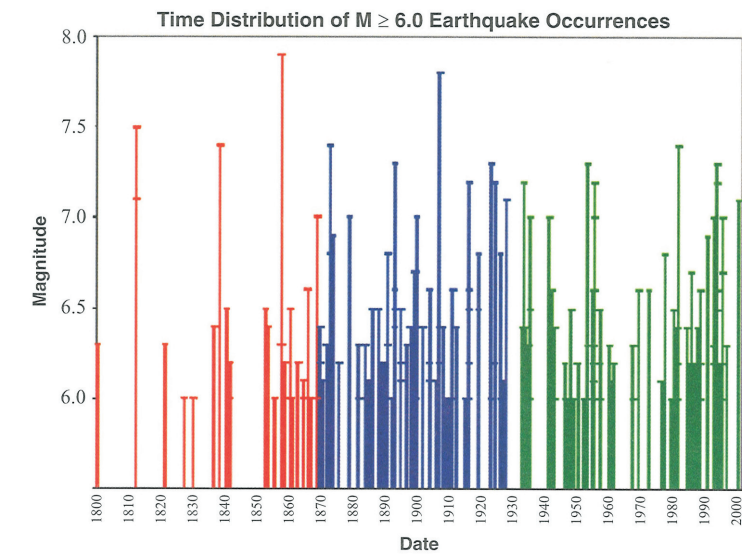
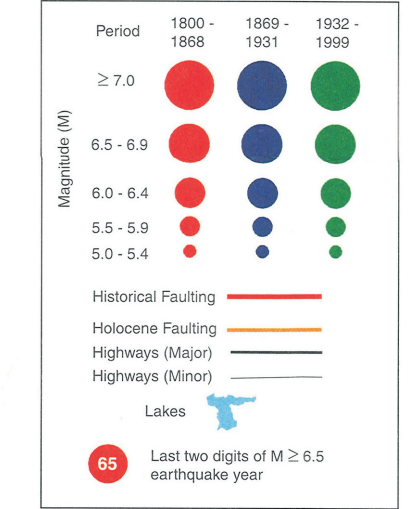






Area Within a 100km (62.4 mile) radius

**EPICENTER MAP LEGEND**



From: T. Toppozada & Others (2000)

Project Number 177-1-7

Figure Number Figure 5

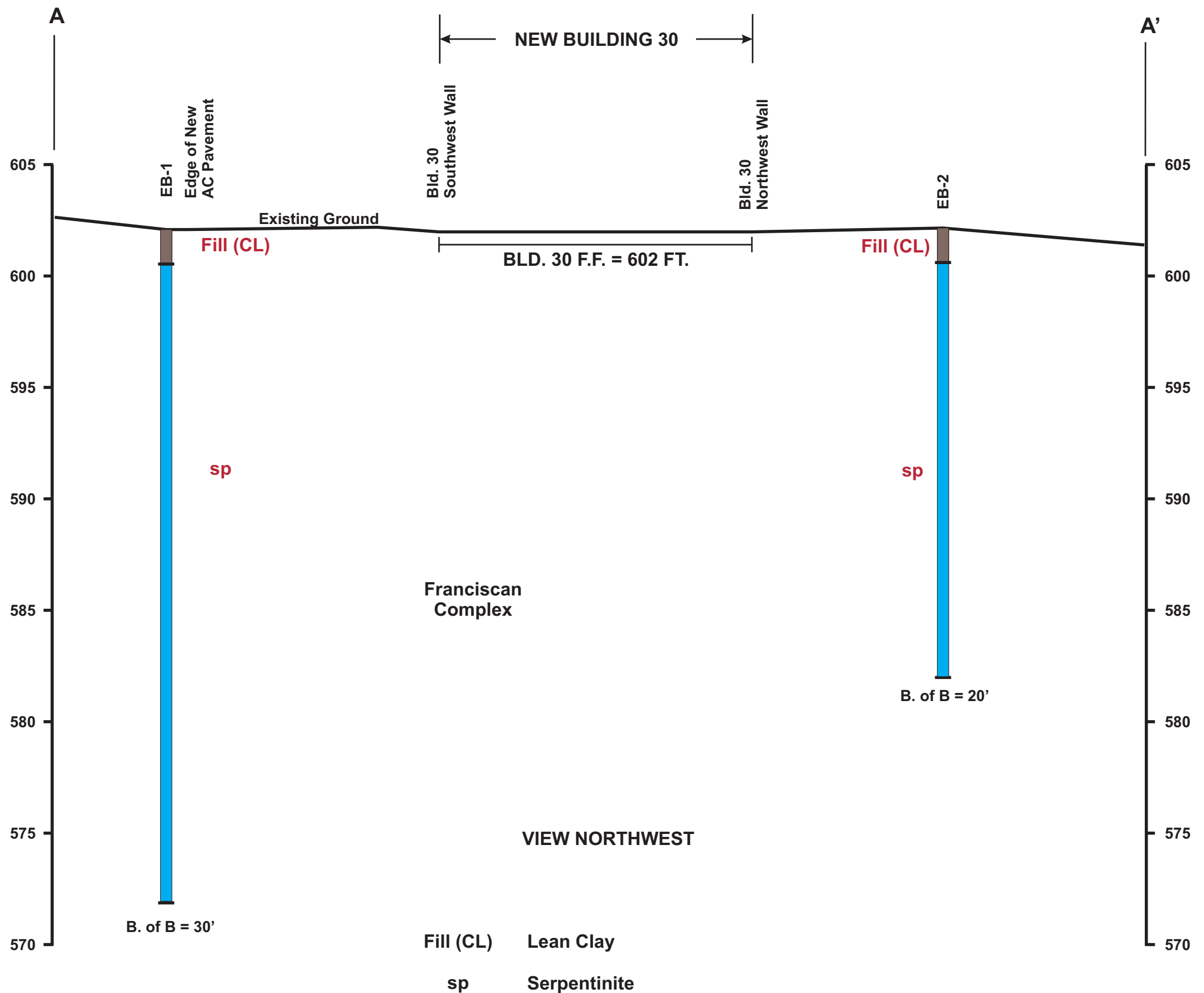
Date April 2010 Drawn By MGW

Historical Earthquake Map

Cañada College Electrical Infrastructure Replacement Redwood City, CA


**CORNERSTONE**  
**EARTH GROUP**



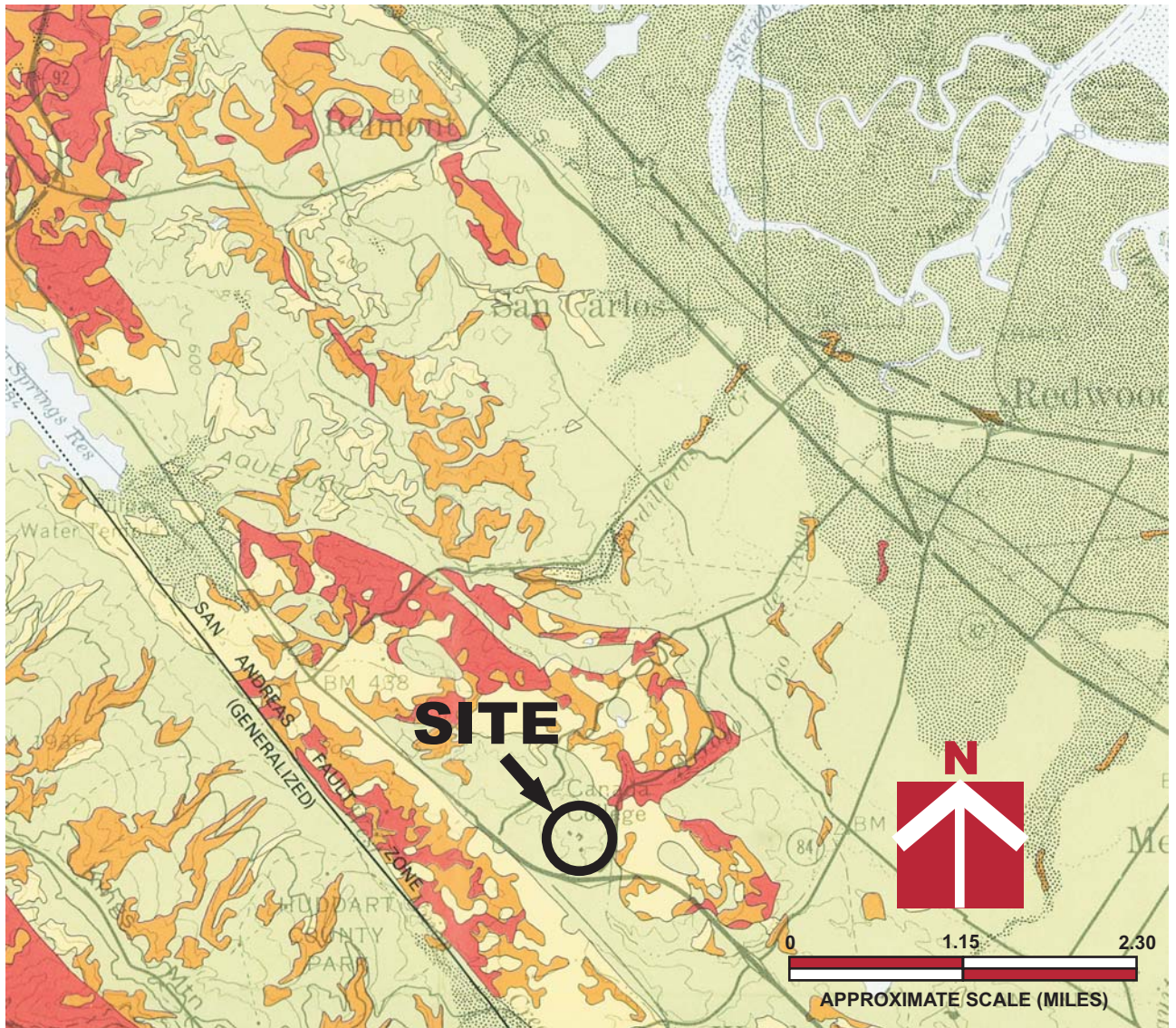


See Figure 2A for location of cross-section.

SCALE: 1" = 5'  
 Vertical & Horizontal

	<b>Geologic Cross Section A-A'</b>	
	Project Number 177-1-7	Figure Number Figure 6
Date April 2010		Drawn By MGV





**EXPLANATION**

- Susceptibility and percentage of area likely to fail in a major earthquake
- High* ----- less than or equal to 25%
  - Moderate* ----- approximately 15%
  - Low* ----- approximately 5%
  - Very low* ----- less than 3%
- AREA OF LOW, MODERATE, OR HIGH LIQUEFACTION SUSCEPTIBILITY—Data from T. L. Youd and J. B. Perkins (1985)
- FAULT ZONE—Approximately located. Data modified from Brabb and Pampeyan (1983). Specific location generalized, therefore should not be used for evaluation of fault-rupture hazard

From: Wieczorek & others (1985)



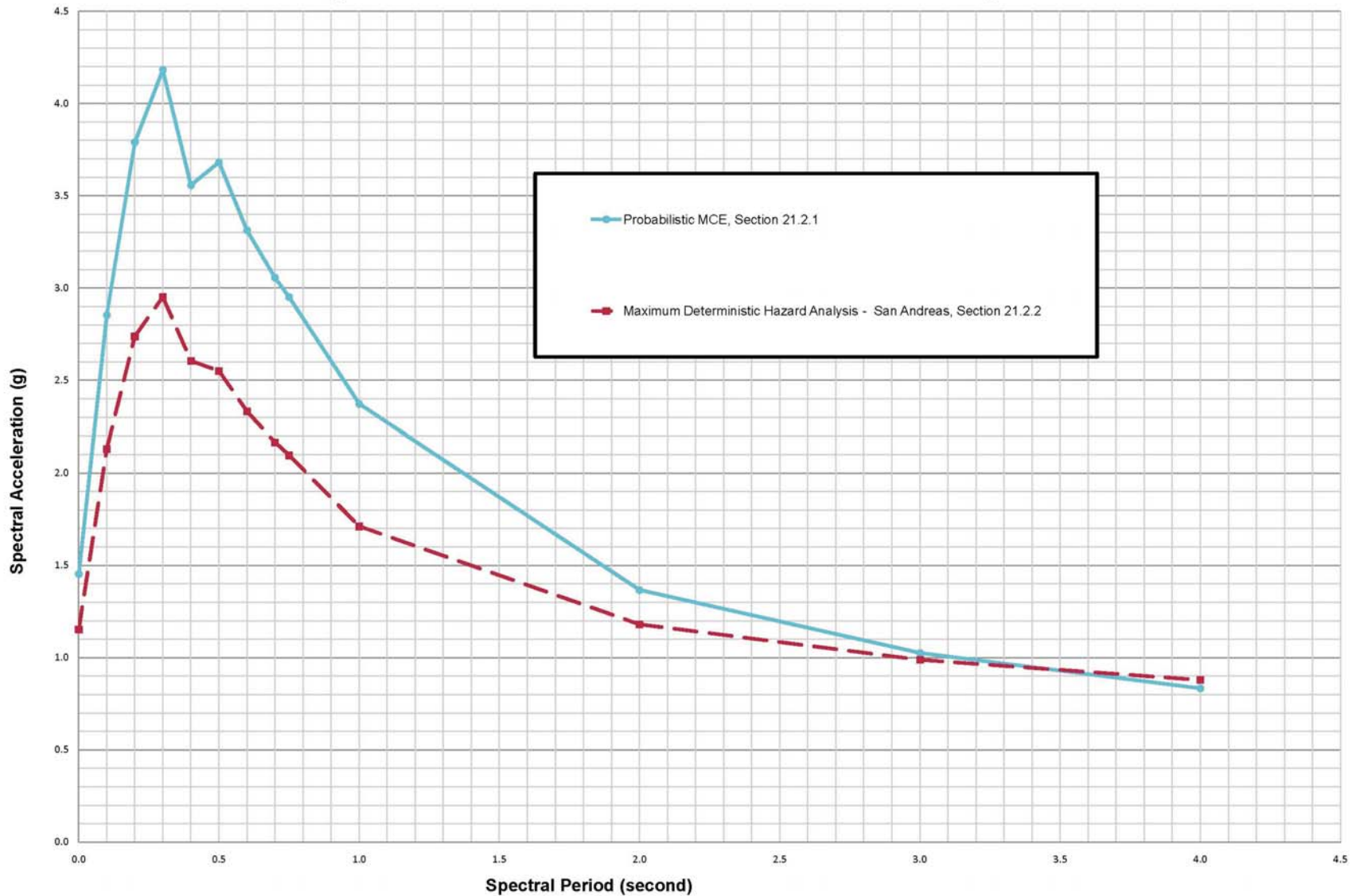
**Slope Stability During Earthquakes**  
**Cañada College Electrical Infrastructure Replacement**  
**Redwood City, CA**

Project Number  
 177-1-7

Figure Number  
 Figure 7

Date  
 April 2010

Drawn By  
 MGW



Probabilistic MCE and Maximum Deterministic Hazard Analysis

Cañada College Electrical  
Infrastructure Replacement  
Redwood City, CA

Project Number

177-1-7

Figure Number

Figure 8

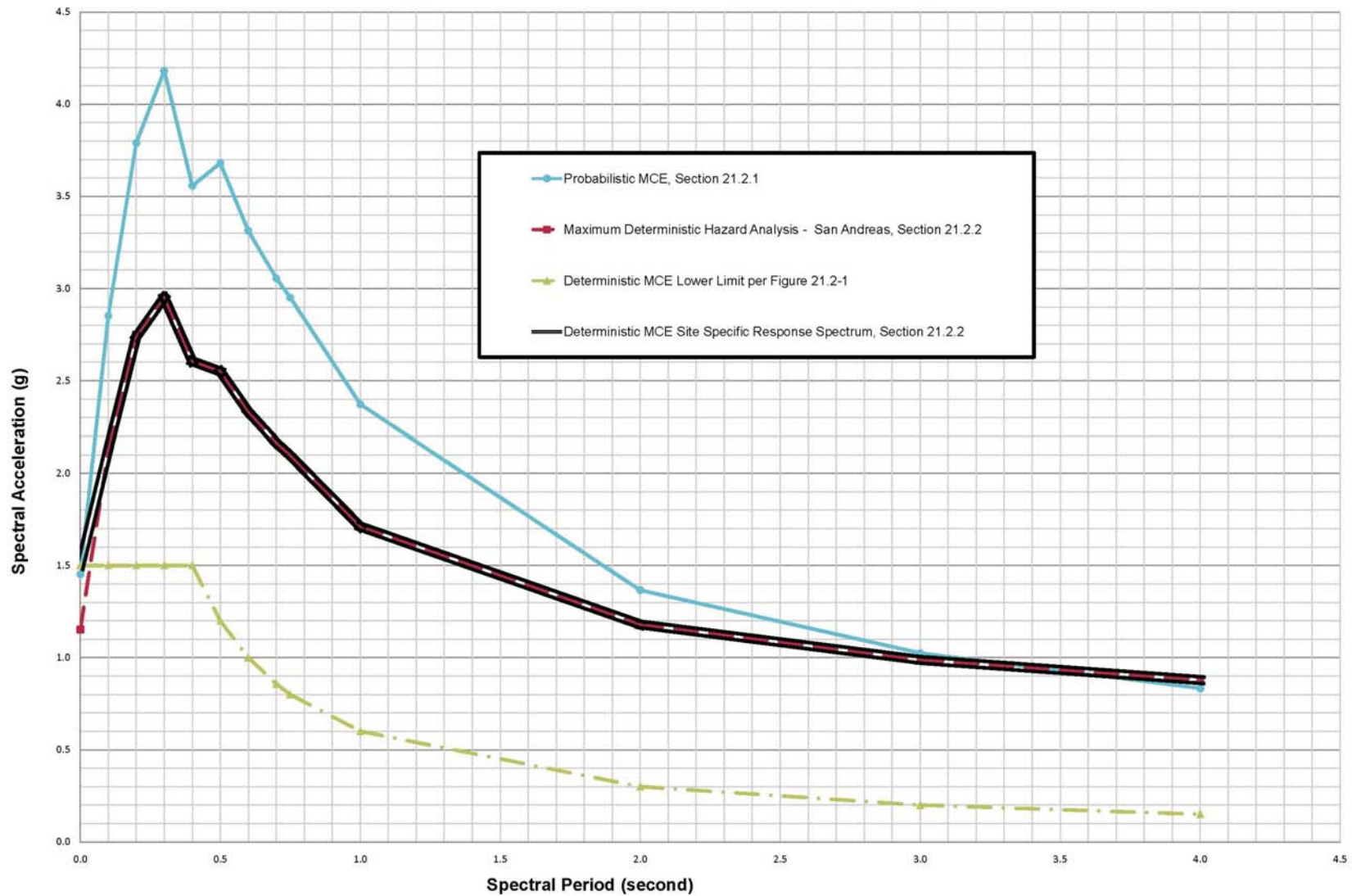
Date

April 2010

Drawn By

FLL





Deterministic MCE Site Specific Response Spectrum

Cañada College Electrical  
Infrastructure Replacement  
Redwood City, CA

Project Number

177-1-7

Figure Number

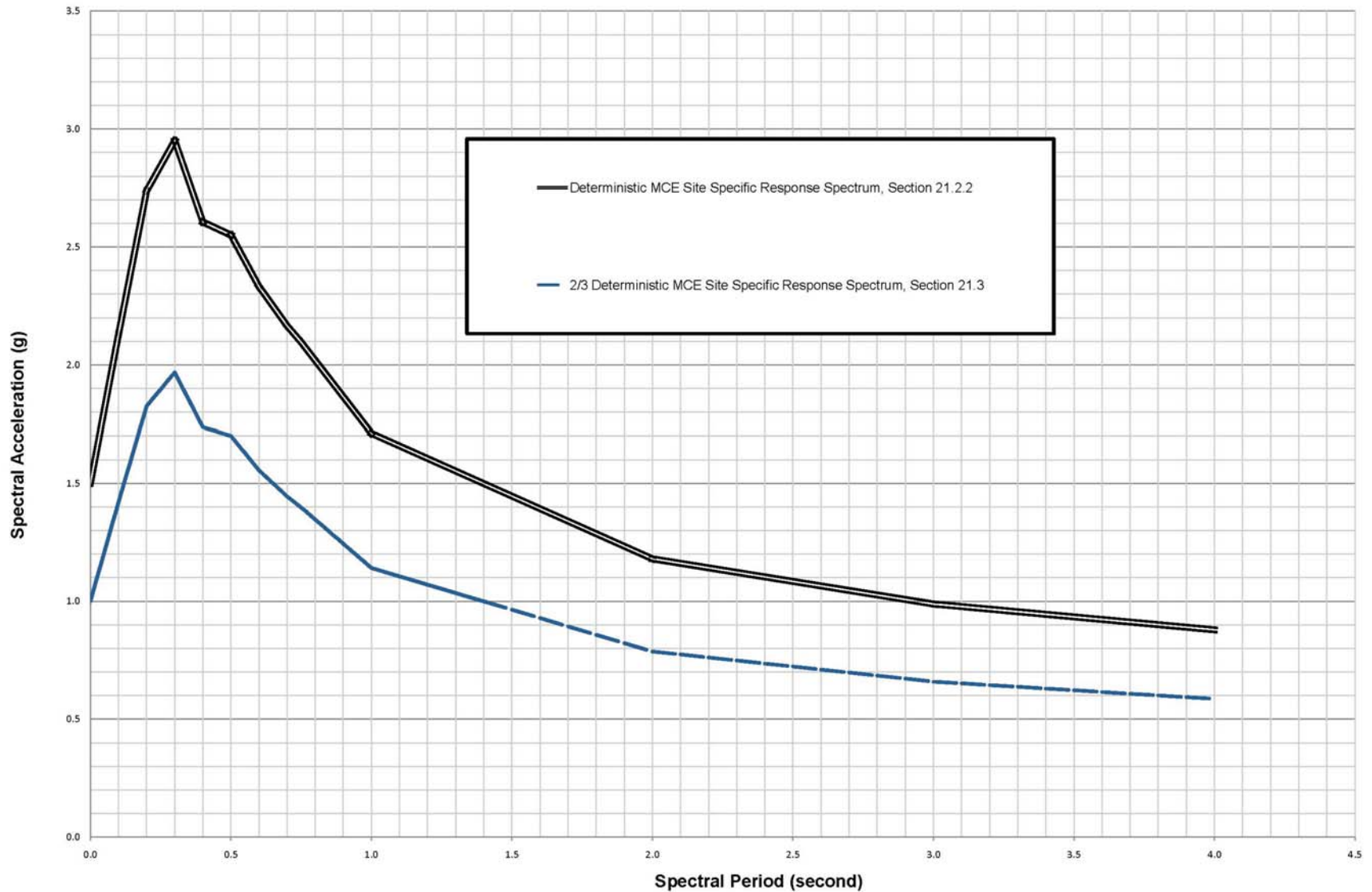
Figure 9

Date

April 2010

Drawn By

FLL



2/3 Deterministic MCE and 80-Percent Site Specific Code DRS

Cañada College Electrical  
Infrastructure Replacement  
Redwood City, CA

Project Number

177-1-7

Figure Number

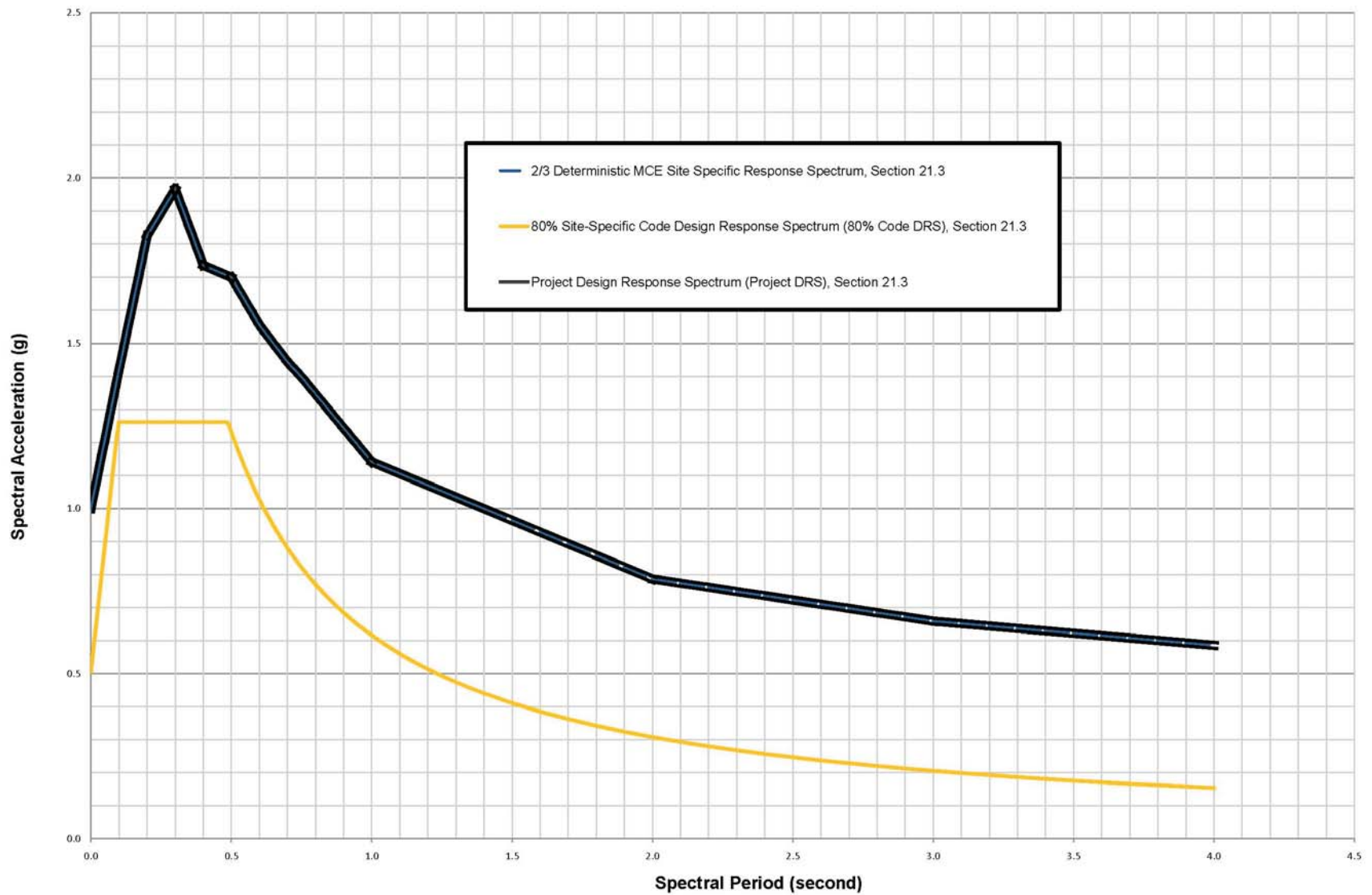
Figure 10

Date

April 2010

Drawn By

FLL



**Project Design Response Spectrum**

**Cañada College Electrical  
Infrastructure Replacement  
Redwood City, CA**

Project Number

177-1-7

Figure Number

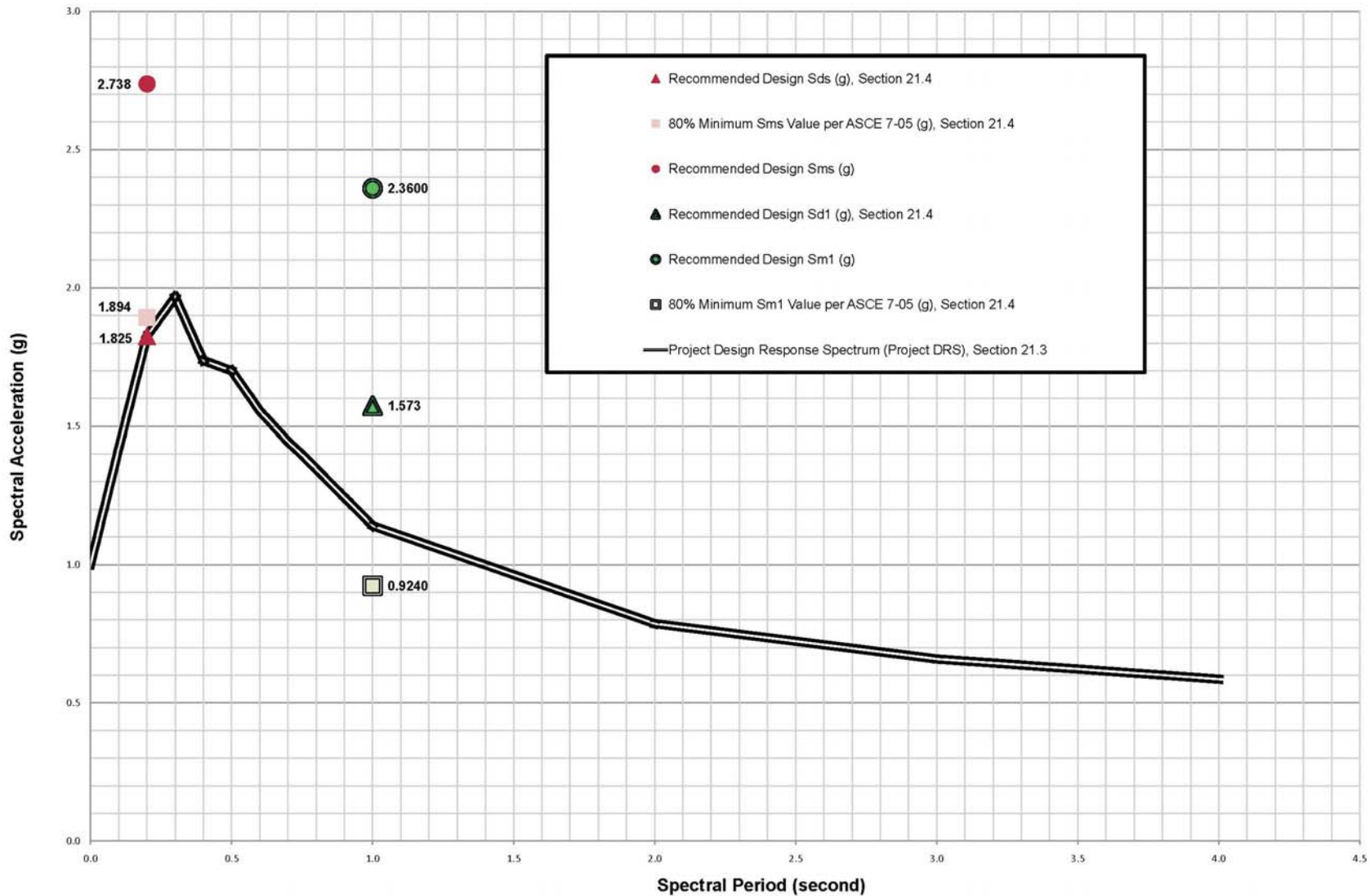
Figure 11

Date

April 2010

Drawn By

FLL



Project Design Response Spectrum and  
Recommended Acceleration Design Parameters

Cañada College Electrical  
Infrastructure Replacement  
Redwood City, CA

Project Number

177-1-7

Figure Number

Figure 12

Date

April 2010

Drawn By

FLL



## APPENDIX A: FIELD INVESTIGATION

The field investigation consisted of a surface reconnaissance and a subsurface exploration program using truck-mounted, hollow-stem auger drilling equipment. Four 8-inch-diameter exploratory borings were drilled on April 5, 2010 to depths of 10 to 30 feet. The approximate locations of exploratory borings are shown on the Site Plan and Geologic Map, Figure 2A and Site Plan, Figure 2B. The soils encountered were continuously logged in the field by our representative and described in accordance with the Unified Soil Classification System (ASTM D2488). Boring logs, as well as a key to the classification of the soil, are included as part of this appendix.
















Boring locations were approximated using existing site boundaries, a portable GPS unit, and other site features as references. Boring elevations were not determined. The locations of the borings should be considered accurate only to the degree implied by the method used.















Representative soil samples were obtained from the borings at selected depths. All samples were returned to our laboratory for evaluation and appropriate testing. The standard penetration resistance blow counts were obtained by dropping a 140-pound hammer through a 30-inch free fall. The 2-inch O.D. split-spoon sampler was driven 18 inches and the number of blows was recorded for each 6 inches of penetration (ASTM D1586). 2.5-inch I.D. samples were obtained using a Modified California Sampler driven into the soil with the 140-pound hammer previously described. Unless otherwise indicated, the blows per foot recorded on the boring log represent the accumulated number of blows required to drive the last 12 inches. The various samplers are denoted at the appropriate depth on the boring logs.

Field tests included an evaluation of the unconfined compressive strength of the soil samples using a pocket penetrometer device. Results of these tests are presented on the individual boring logs at the appropriate sample depths.







Attached boring logs and related information depict subsurface conditions at the locations indicated and on the date designated on the logs. Subsurface conditions at other locations may differ from conditions occurring at these boring locations. The passage of time may result in altered subsurface conditions due to environmental changes. In addition, any stratification lines on the logs represent the approximate boundary between soil types and the transition may be gradual.

# UNIFIED SOIL CLASSIFICATION (ASTM D-2487-98)


MATERIAL TYPES	CRITERIA FOR ASSIGNING SOIL GROUP NAMES			GROUP SYMBOL	SOIL GROUP NAMES & LEGEND	
COARSE-GRAINED SOILS >50% RETAINED ON NO. 200 SIEVE	GRAVELS  >50% OF COARSE FRACTION RETAINED ON NO 4. SIEVE	CLEAN GRAVELS <5% FINES	$Cu > 4$ AND $1 < Cc < 3$	GW	WELL-GRADED GRAVEL	
			$Cu > 4$ AND $1 > Cc > 3$	GP	POORLY-GRADED GRAVEL	
		GRAVELS WITH FINES >12% FINES	FINES CLASSIFY AS ML OR CL	GM	SILTY GRAVEL	
			FINES CLASSIFY AS CL OR CH	GC	CLAYEY GRAVEL	
	SANDS  >50% OF COARSE FRACTION PASSES ON NO 4. SIEVE	CLEAN SANDS <5% FINES	$Cu > 6$ AND $1 < Cc < 3$	SW	WELL-GRADED SAND	
			$Cu > 6$ AND $1 > Cc > 3$	SP	POORLY-GRADED SAND	
		SANDS AND FINES >12% FINES	FINES CLASSIFY AS ML OR CL	SM	SILTY SAND	
			FINES CLASSIFY AS CL OR CH	SC	CLAYEY SAND	
FINE-GRAINED SOILS >50% PASSES NO. 200 SIEVE	SILTS AND CLAYS  LIQUID LIMIT < 50	INORGANIC	$PI > 7$ AND PLOTS > "A" LINE	CL	LEAN CLAY	
			$PI > 4$ AND PLOTS < "A" LINE	ML	SILT	
	SILTS AND CLAYS  LIQUID LIMIT > 50	INORGANIC	LL (oven dried)/LL (not dried) < 0.75	OL	ORGANIC CLAY OR SILT	
			PI PLOTS > "A" LINE	CH	FAT CLAY	
			PI PLOTS < "A" LINE	MH	ELASTIC SILT	
			LL (oven dried)/LL (not dried) < 0.75	OH	ORGANIC CLAY OR SILT	
HIGHLY ORGANIC SOILS		PRIMARILY ORGANIC MATTER, DARK IN COLOR, AND ORGANIC ODOR		PT	PEAT	

OTHER MATERIAL SYMBOLS	
	Poorly-Graded Sand with Clay
	Clayey Sand
	Sandy Silt
	Artificial/Undocumented Fill
	Poorly-Graded Gravelly Sand
	Topsoil
	Well-Graded Gravel with Clay
	Well-Graded Gravel with Silt
	Sand
	Silt
	Well Graded Gravelly Sand
	Gravelly Silt
	Asphalt
	Boulders and Cobble

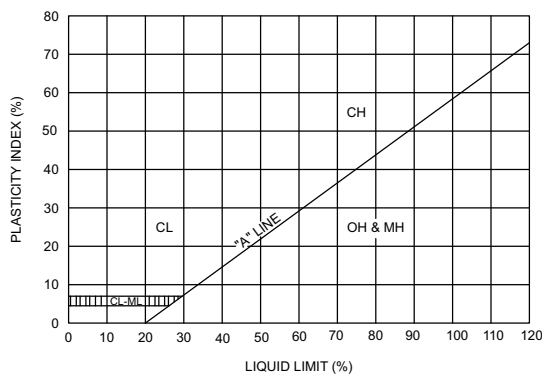
### SAMPLER TYPES

	SPT		Shelby Tube
	Modified California (2.5" I.D.)		No Recovery
	Rock Core		Grab Sample

### ADDITIONAL TESTS

CA - CHEMICAL ANALYSIS (CORROSIVITY)	PI - PLASTICITY INDEX
CD - CONSOLIDATED DRAINED TRIAXIAL	SW - SWELL TEST
CN - CONSOLIDATION	TC - CYCLIC TRIAXIAL
CU - CONSOLIDATED UNDRAINED TRIAXIAL	TV - TORVANE SHEAR
DS - DIRECT SHEAR	UC - UNCONFINED COMPRESSION
PP - POCKET PENETROMETER (TSF)	(1.5) - (WITH SHEAR STRENGTH IN KSF)
(3.0) - (WITH SHEAR STRENGTH IN KSF)	-
RV - R-VALUE	UU - UNCONSOLIDATED UNDRAINED TRIAXIAL
SA - SIEVE ANALYSIS: % PASSING #200 SIEVE	
	- WATER LEVEL

### PLASTICITY CHART



### PENETRATION RESISTANCE (RECORDED AS BLOWS / FOOT)

SAND & GRAVEL		SILT & CLAY		
RELATIVE DENSITY	BLOWS/FOOT*	CONSISTENCY	BLOWS/FOOT*	STRENGTH** (KSF)
VERY LOOSE	0 - 4	VERY SOFT	0 - 2	0 - 0.25
LOOSE	4 - 10	SOFT	2 - 4	0.25 - 0.5
MEDIUM DENSE	10 - 30	MEDIUM STIFF	4 - 8	0.5 - 1.0
DENSE	30 - 50	STIFF	8 - 15	1.0 - 2.0
VERY DENSE	OVER 50	VERY STIFF	15 - 30	2.0 - 4.0
		HARD	OVER 30	OVER 4.0

\* NUMBER OF BLOWS OF 140 LB HAMMER FALLING 30 INCHES TO DRIVE A 2 INCH O.D. (1-3/8 INCH I.D.) SPLIT-BARREL SAMPLER THE LAST 12 INCHES OF AN 18-INCH DRIVE (ASTM-1586 STANDARD PENETRATION TEST).

\*\* UNDRAINED SHEAR STRENGTH IN KIPS/SQ. FT. AS DETERMINED BY LABORATORY TESTING OR APPROXIMATED BY THE STANDARD PENETRATION TEST, POCKET PENETROMETER, TORVANE, OR VISUAL OBSERVATION.

## HARDNESS

1. **Soft** – Reserved for plastic material alone.
2. **Low hardness** – Can be gouged deeply or carved easily with a knife blade.
3. **Moderately hard** – Can be readily scratched by a knife blade: scratch leaves a heavy trace of dust and is readily visible after the powder has been blown away.
4. **Hard** – Can be scratched with difficulty: scratch produces little powder and is often faintly visible.
5. **Very hard** – Cannot be scratched with knife blade: leaves a metallic streak.

## STRENGTH

1. **Plastic** or very low strength.
2. **Friable** – Crumbles easily by rubbing with fingers.
3. **Weak** – An unfractured specimen of such material will crumble under light hammer blows.
4. **Moderately strong** – Specimen will withstand a few heavy hammer blows before breaking.
5. **Strong** – Specimen will withstand a few heavy ringing blows and will yield with difficulty only dust and small flying fragments.
6. **Very strong** – Specimen will resist heavy ringing hammer blows and will yield with difficulty only dust and small flying fragments.

**WEATHERING** – The physical and chemical disintegration and decomposition of rocks and minerals by natural processes such as oxidation, reduction, hydration, solution, carbonation, and freezing and thawing.

- D. **Deep** – Moderate to complete mineral decomposition: extensive disintegration: deep and thorough discoloration: many fractures, all extensively coated or filled with oxides, carbonates and/or clay or silt.
- M. **Moderate** – Slight change or partial decomposition of minerals: little disintegration: cementation little to unaffected. Moderate to occasionally intense discoloration. Moderately coated fractures.
- L. **Little** – No megascopic decomposition of minerals: little or no effect on normal cementation. Slight and intermittent, or localized discoloration. Few stains or fracture surfaces.
- F. **Fresh** – Unaffected by weathering agents. No disintegration or discoloration. Fractures usually less numerous than joints.

## FRACTURING

Intensity	Size of Pieces in Feet
Very little fractured	Greater than 4.0
Occasionally fractured	1.0 to 4.0
Moderately fractured	0.5 to 1.0
Closely fractured	0.1 to 0.5
Intensely fractured	0.05 to 0.1
Crushed	Less than 0.05

## BEDDING OF SEDIMENTARY ROCKS

Splitting Property	Thickness	Stratification
Massive	Greater than 4.0 feet	very thick-bedded
Blocky	2.0 to 4.0 feet	thick-bedded
Slabby	0.2 to 2.0 feet	thin-bedded
Flaggy	0.05 to 0.2 feet	very thin-bedded
Shaly or Platy	0.01 to 0.05 feet	laminated
Papery	less than 0.01 feet	thinly laminated



DATE STARTED 4/5/10 DATE COMPLETED 4/5/10  
 DRILLING CONTRACTOR Exploration Geoservices, Inc.  
 DRILLING METHOD Mobile B-40, 8 inch Hollow-Stem Auger  
 LOGGED BY DTT  
 NOTES \_\_\_\_\_

PROJECT NAME Canada College Electrical Replacement  
 PROJECT NUMBER 177-1-7  
 PROJECT LOCATION Redwood City, CA  
 GROUND ELEVATION \_\_\_\_\_ BORING DEPTH 30 ft.  
 LATITUDE \_\_\_\_\_ LONGITUDE \_\_\_\_\_  
 GROUND WATER LEVELS:  
 ▽ AT TIME OF DRILLING Not Encountered  
 ▼ AT END OF DRILLING Not Encountered

This log is a part of a report by Cornerstone Earth Group, and should not be used as a stand-alone document. This description applies only to the location of the exploration at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with time. The description presented is a simplification of actual conditions encountered. Transitions between soil types may be gradual.

ELEVATION (ft)	DEPTH (ft)	SYMBOL	DESCRIPTION	N-Value (uncorrected) blows per foot	SAMPLES TYPE AND NUMBER	DRY UNIT WEIGHT PCF	NATURAL MOISTURE CONTENT, %	PLASTICITY INDEX, %	PERCENT PASSING NO. 200 SIEVE	UNDRAINED SHEAR STRENGTH, ksf
	0		<b>Lean Clay with Sand (CL) [Fill]</b> stiff, moist, brown to yellow brown, fine to medium sand, some gravel, moderate plasticity							○ HAND PENETROMETER
	0		<b>Serpentinite [sp]</b> low hardness, weak to friable, deep weathering, brown to very light reddish brown, thick bedded, high plasticity							△ TORVANE
	5									● UNCONFINED COMPRESSION
	5									▲ UNCONSOLIDATED-UNDRAINED TRIAXIAL
	5									1.0 2.0 3.0 4.0 5.0
	22			22	MC-1	89	21			
	29			29	MC-3	93	29			
	52			52	MC-5	102	20			
	25			25	SPT					
	15		less weathering, gray mottles	26	SPT					
	20		olive gray with light brown mottles	43	SPT					
	25		moderate weathering, color to green-gray	46	SPT					
	30		color to dark gray	50	SPT					
	30		Bottom of Boring at 30.0 feet.							
	35									

CORNERSTONE EARTH GROUP - CORNERSTONE.GDT - 4/15/10 12:54 - P:\DRAFTING\GINT FILES\177-1-7 CANADA COLLEGE ELECTRICAL.GPJ

PROJECT NAME Canada College Electrical Replacement

 PROJECT NUMBER 177-1-7

 PROJECT LOCATION Redwood City, CA

 DATE STARTED 4/5/10 DATE COMPLETED 4/5/10

 GROUND ELEVATION \_\_\_\_\_ BORING DEPTH 20 ft.

 DRILLING CONTRACTOR Exploration Geoservices, Inc.

LATITUDE \_\_\_\_\_ LONGITUDE \_\_\_\_\_

 DRILLING METHOD Mobile B-40, 8 inch Hollow-Stem Auger
**GROUND WATER LEVELS:**

 LOGGED BY DTT

 ▽ AT TIME OF DRILLING Not Encountered

NOTES \_\_\_\_\_

 ▼ AT END OF DRILLING Not Encountered

This log is a part of a report by Cornerstone Earth Group, and should not be used as a stand-alone document. This description applies only to the location of the exploration at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with time. The description presented is a simplification of actual conditions encountered. Transitions between soil types may be gradual.

ELEVATION (ft)	DEPTH (ft)	SYMBOL	DESCRIPTION	N-Value (uncorrected) blows per foot	SAMPLES TYPE AND NUMBER	DRY UNIT WEIGHT PCF	NATURAL MOISTURE CONTENT, %	PLASTICITY INDEX, %	PERCENT PASSING NO. 200 SIEVE	UNDRAINED SHEAR STRENGTH, ksf									
										○ HAND PENETROMETER △ TORVANE ● UNCONFINED COMPRESSION ▲ UNCONSOLIDATED-UNDRAINED TRIAXIAL 1.0 2.0 3.0 4.0 5.0									
	0		<b>Sandy Lean Clay (CL) [Fill]</b> stiff, moist, brown to yellow brown, fine sand, trace gravel, moderate plasticity																
			<b>Serpentinite [sp]</b> soft, weak to friable, deep weathering, reddish brown to yellow brown with gray mottles, thick bedded Liquid Limit = 45, Plastic Limit = 17	29	MC-2	114	12												
				31	MC-4	115	17	28											
	5			44	MC-5	115	9												
				27	SPT														
	10																		
			olive brown with very light brown mottles	44	SPT														
	15																		
			color to dark reddish brown and gray mottled	53	SPT														
	20		Bottom of Boring at 20.0 feet.																
	25																		
	30																		
	35																		



DATE STARTED 4/5/10 DATE COMPLETED 4/5/10  
 DRILLING CONTRACTOR Exploration Geoservices, Inc.  
 DRILLING METHOD Mobile B-40, 8 inch Hollow-Stem Auger  
 LOGGED BY DTT  
 NOTES \_\_\_\_\_

PROJECT NAME Canada College Electrical Replacement  
 PROJECT NUMBER 177-1-7  
 PROJECT LOCATION Redwood City, CA  
 GROUND ELEVATION \_\_\_\_\_ BORING DEPTH 10 ft.  
 LATITUDE \_\_\_\_\_ LONGITUDE \_\_\_\_\_  
 GROUND WATER LEVELS:  
 ▽ AT TIME OF DRILLING Not Encountered  
 ▼ AT END OF DRILLING Not Encountered

This log is a part of a report by Cornerstone Earth Group, and should not be used as a stand-alone document. This description applies only to the location of the exploration at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with time. The description presented is a simplification of actual conditions encountered. Transitions between soil types may be gradual.

ELEVATION (ft)	DEPTH (ft)	SYMBOL	DESCRIPTION	N-Value (uncorrected) blows per foot	SAMPLES TYPE AND NUMBER	DRY UNIT WEIGHT PCF	NATURAL MOISTURE CONTENT, %	PLASTICITY INDEX, %	PERCENT PASSING NO. 200 SIEVE	UNDRAINED SHEAR STRENGTH, ksf				
										○ HAND PENETROMETER △ TORVANE ● UNCONFINED COMPRESSION ▲ UNCONSOLIDATED-UNDRAINED TRIAXIAL 1.0 2.0 3.0 4.0 5.0				
	0		<b>Gravelly Lean Clay with Sand (CL) [Fill]</b> hard, moist, reddish brown, fine subangular gravel, fine to medium sand, moderate plasticity	50	MC-1	89	18							
	6"													
	61		<b>Serpentinite [sp]</b> low hardness, weak to friable, deep weathering, very light brown, thick bedded, high plasticity		SPT									
	5													
	29		color to brown, olive, and black mottled		SPT									
	26				SPT									
	10		Bottom of Boring at 10.0 feet.											
	15													
	20													
	25													
	30													
	35													





DATE STARTED 4/5/10 DATE COMPLETED 4/5/10  
 DRILLING CONTRACTOR Exploration Geoservices, Inc.  
 DRILLING METHOD Mobile B-40, 8 inch Hollow-Stem Auger  
 LOGGED BY DTT  
 NOTES \_\_\_\_\_

PROJECT NAME Canada College Electrical Replacement  
 PROJECT NUMBER 177-1-7  
 PROJECT LOCATION Redwood City, CA  
 GROUND ELEVATION \_\_\_\_\_ BORING DEPTH 8.9 ft.  
 LATITUDE \_\_\_\_\_ LONGITUDE \_\_\_\_\_  
 GROUND WATER LEVELS:  
 ▽ AT TIME OF DRILLING Not Encountered  
 ▼ AT END OF DRILLING Not Encountered

This log is a part of a report by Cornerstone Earth Group, and should not be used as a stand-alone document. This description applies only to the location of the exploration at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with time. The description presented is a simplification of actual conditions encountered. Transitions between soil types may be gradual.

ELEVATION (ft)	DEPTH (ft)	SYMBOL	DESCRIPTION	N-Value (uncorrected) blows per foot	SAMPLES TYPE AND NUMBER	DRY UNIT WEIGHT PCF	NATURAL MOISTURE CONTENT, %	PLASTICITY INDEX, %	PERCENT PASSING NO. 200 SIEVE	UNDRAINED SHEAR STRENGTH, ksf
	0		<b>Franciscan Complex Greenstone [fg]</b> low hardness, weak to moderately strong, deep weathering, reddish brown, thick bedded	50 6"		124	8			
	5			50 5"						
			moderate weathering, dark reddish brown	50 3"						
	10		Bottom of Boring at 8.9 feet.	50 5"						
	15									
	20									
	25									
	30									
	35									

- HAND PENETROMETER
- △ TORVANE
- UNCONFINED COMPRESSION
- ▲ UNCONSOLIDATED-UNDRAINED TRIAXIAL

## APPENDIX B: LABORATORY TEST PROGRAM

The laboratory-testing program was performed to evaluate the physical and mechanical properties of the soils retrieved from the site to aid in verifying soil classification.

### Moisture Content

The natural water content was determined (ASTM D2216) on eight samples of the materials recovered from the borings. These water contents are recorded on the boring logs at the appropriate sample depths.

### Dry Densities

In place dry density determinations (ASTM D2937) were performed on eight samples to measure the unit weight of the subsurface soils. Results of these tests are shown on the boring logs at the appropriate sample depths.

### Atterberg Limits

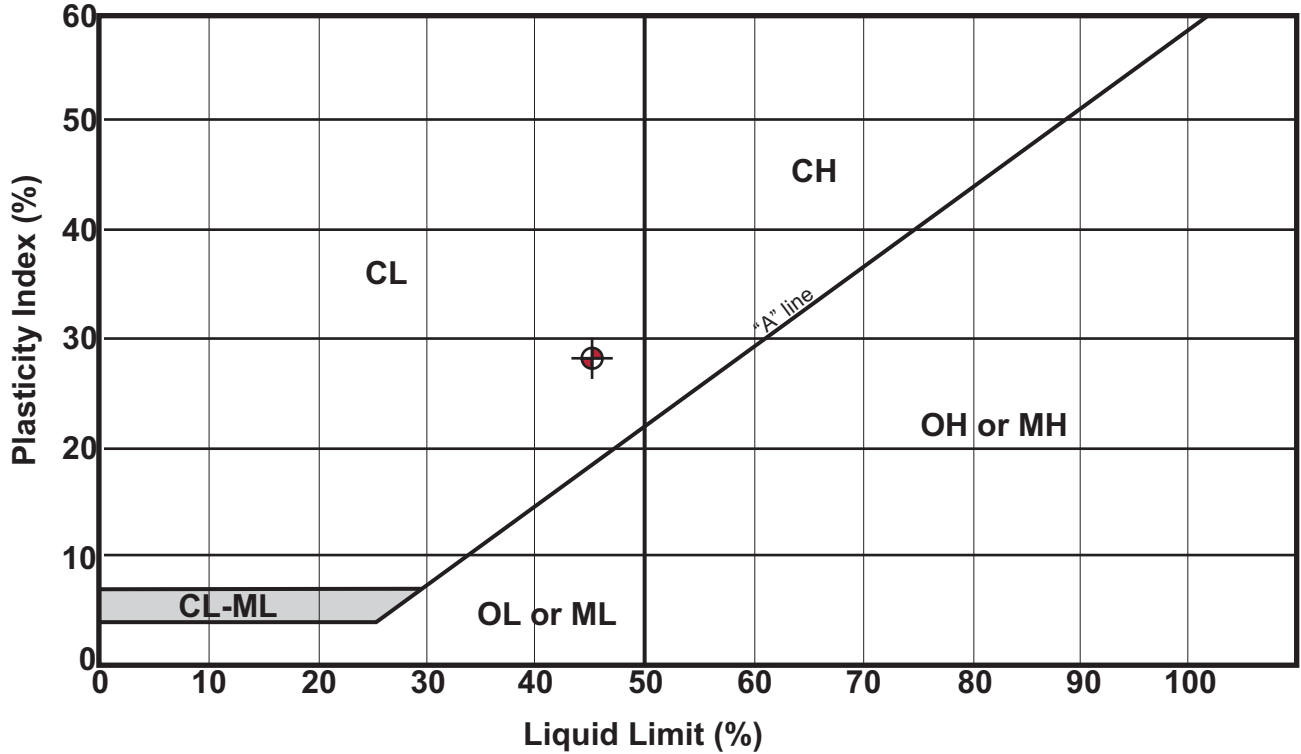
One Atterberg Limit tests (ASTM D4318) was performed on a sample of the weathered Franciscan serpentinite to measure the range of water contents over which this material exhibits plasticity. The Plasticity Index was used to classify the material in accordance with the Unified Soil Classification System and to evaluate the expansion potential. Results of this test indicate that the serpentinite exhibits a moderate plasticity and expansion potential. Results of this test is summarized in Table B-1 below and shown graphically on Figure B-1.

**Table B-1. Summary of Atterberg Limit Tests**

Boring/Sample No.	Sample Depth (feet)	Liquid Limit (percent)	Plastic Limit (percent)	Plasticity Index (percent)	USCS Symbol
EB-2/2	3½	45	17	28	NA

Note: NA – not applicable

### Plasticity Index (ASTM D4318) Testing Summary



Symbol	Boring No.	Depth (ft)	Natural Water Content (%)	Liquid Limit (%)	Plastic Limit (%)	Plasticity Index	Passing No. 200 (%)	Group Name (USCS - ASTM D2487)
⊕	EB-2	3.5	17	45	17	28		Serpentinite [sp]

**APPENDIX C: PREVIOUS FIELD INVESTIGATION AND LABORATORY PROGRAM**

**Lowney Associates – February 2, 2005**

**APPENDIX A**  
**FIELD INVESTIGATION**

The field investigation consisted of a surface reconnaissance and a subsurface exploration program using portable minuteman, continuous-flight and truck-mounted, hollow-stem auger drilling equipment. One 4-inch-diameter exploratory boring was drilled on December 16, 2004 to a depth of 12½ feet using portable drilling equipment. Two 8-inch-diameter exploratory borings were drilled on December 17 and 23, 2004 to a maximum depth of 25 feet using truck-mounted hollow-stem auger drilling equipment. The approximate locations of the exploratory borings are shown on the Site Plan, Figure 2. The soils and rock encountered were logged in the field by our representative and described in accordance with the Unified Soil Classification System (ASTM D2488). The log of the borings, as well as a key to the classification of the soil and rock, are included as part of this appendix.

The locations of the borings were approximately measured by pacing from existing structures and site boundaries. Elevations of the borings were determined by interpolation from plan contours. The locations and elevations of the borings should be considered accurate only to the degree implied by the method used.

Representative soil and rock samples were obtained from the borings at selected depths. All samples were returned to our laboratory for evaluation and appropriate testing. Penetration resistance blow counts were obtained by dropping a 140-pound hammer 30 inches. Modified California 2.5-inch I.D. samples and Standard Penetration Test (SPT) 2-inch O.D. samples were obtained by driving the samplers 18 inches and recording the number of hammer blows for each 6 inches of penetration. The sampler was driven 18 inches or to 50 blows for 6 inches or less of penetration, whichever occurred first, and the number of blows was recorded for each 6 inches or less of penetration (ASTM D1586). Unless otherwise indicated, the blows per foot recorded on the boring logs represent the accumulated number of blows required to drive the last 12 inches or the inches of penetration noted. The sampler type is denoted at the appropriate depth on the boring logs and symbolized as shown on Figure A-1.

The attached boring logs and related information depict subsurface conditions at the location indicated and on the date designated on the logs. Subsurface conditions at other locations may differ from conditions occurring at these boring locations. The passage of time may result in altered subsurface conditions due to environmental changes. In addition, any stratification lines on the log represent the approximate boundary between soil or rock types and the transition may be gradual.

\* \* \* \* \*

PRIMARY DIVISIONS			SOIL TYPE	SECONDARY DIVISIONS
COARSE GRAINED SOILS MORE THAN HALF OF MATERIAL IS LARGER THAN NO. 200 SIEVE SIZE	GRAVELS MORE THAN HALF OF COARSE FRACTION IS LARGER THAN NO. 4 SIEVE	CLEAN GRAVELS (Less than 5% Fines)	GW	Well graded gravels, gravel-sand mixtures, little or no fines
			GP	Poorly graded gravels or gravel-sand mixtures, little or no fines
		GRAVEL WITH FINES	GM	Silty gravels, gravel-sand-silt mixtures, plastic fines
			GC	Clayey gravels, gravel-sand-clay mixtures, plastic fines
	SANDS MORE THAN HALF OF COARSE FRACTION IS SMALLER THAN NO. 4 SIEVE	CLEAN SANDS (Less than 5% Fines)	SW	Well graded sands, gravelly sands, little or no fines
			SP	Poorly graded sands or gravelly sands, little or no fines
		SANDS WITH FINES	SM	Silty sands, sand-silt-mixtures, non-plastic fines
			SC	Clayey sands, sand-clay mixtures, plastic fines
FINE GRAINED SOILS MORE THAN HALF OF MATERIAL IS SMALLER THAN NO. 200 SIEVE SIZE	SILTS AND CLAYS LIQUID LIMIT IS LESS THAN 50 %		ML	Inorganic silts and very fine sands, rock flour, silty or clayey fine sands or clayey silts with slight plasticity
			CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays
			OL	Organic silts and organic silty clays of low plasticity
	SILTS AND CLAYS LIQUID LIMIT IS GREATER THAN 50 %		MH	Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts
			CH	Inorganic clays of high plasticity, fat clays
			OH	Organic clays of medium to high plasticity, organic silts
HIGHLY ORGANIC SOILS			PT	Peat and other highly organic soils

### DEFINITION OF TERMS

U.S. STANDARD SIEVE SIZE				CLEAR SQUARE SIEVE OPENINGS			
200	40	10	4	3/4"	3"	12"	
SILTS AND CLAY	SAND			GRAVEL		COBBLES	BOULDERS
	FINE	MEDIUM	COARSE	FINE	COARSE		
0.08	0.4	2	5	19	76mm		

### GRAIN SIZES

	TERZAGHI SPLIT SPOON STANDARD PENETRATION		MODIFIED CALIFORNIA		D&M UNDERWATER SAMPLER		SHELBY TUBE		NO RECOVERY
---	---	---	---------------------	---	------------------------	---	-------------	---	-------------

### SAMPLERS

SAND AND GRAVEL	BLOWS/FOOT*
VERY LOOSE	0-4
LOOSE	4-10
MEDIUM DENSE	10-30
DENSE	30-50
VERY DENSE	OVER 50

### RELATIVE DENSITY

SILTS AND CLAYS	STRENGTH+	BLOWS/FOOT*
VERY SOFT	0-1/4	0-2
SOFT	1/4-1/2	2-4
MEDIUM STIFF	1/2-1	4-8
STIFF	1-2	8-16
VERY STIFF	2-4	16-32
HARD	OVER 4	OVER 32

### CONSISTENCY

\*Number of blows of 140 pound hammer falling 30 inches to drive a 2-inch O.D. (1-3/8 inch I.D.) split spoon (ASTM D-1586).  
 +Unconfined compressive strength in tons/sq.ft. as determined by laboratory testing or approximated by the standard penetration test (ASTM D-1586), pocket penetrometer, torvane, or visual observation.

## KEY TO EXPLORATORY BORING LOGS

Unified Soil Classification System (ASTM D-2487)



### WEATHERING

<b>FRESH</b>	Rock fresh, crystals bright, few joints may show slight staining. Rock rings under hammer if crystalline.	<b>MODERATELY SEVERE</b>	All rock except quartz, discolored or stained. In granitoid rocks, all feldspars dull and discolored and majority show kaolinization. Rock shows severe loss of strength and can be excavated with geologist's pick. Rock goes "clunk" when struck.
<b>VERY SLIGHT</b>	Rock generally fresh, joints stained, some joints may show thin clay coatings, crystals in broken face show bright. Rock rings under hammer if crystalline.	<b>SEVERE</b>	All rock except quartz discolored or stained. Rock "fabric" clear and evident, but reduced in strength to strong soil. In granitoid rocks, all feldspars kaolinized to some extent. Some fragments of strong rock usually left.
<b>SLIGHT</b>	Rock generally fresh, joints stained, and discoloration extends into rock up to 1 inch. Joints may contain clay. In granitoid rocks some occasional feldspar crystals are dull and discolored. Crystalline rocks ring under hammer.	<b>VERY SEVERE</b>	All rock except quartz discolored and stained. Rock "fabric" discernible, but mass effectively reduced to "soil" with only fragments of strong rock remaining.
<b>MODERATE</b>	Significant portions of rock show discoloration and weathering effects. In granitoid rocks, most feldspars are dull and discolored; some are clayey. Rock has dull sound under hammer and shows significant loss of strength as compared with fresh rock.	<b>COMPLETE</b>	Rock reduced to "soil". Rock "fabric" not discernible or discernible only in small scattered locations. Quartz may be present as dikes or stringers.

### HARDNESS

<b>VERY HARD</b>	Cannot be scratched with knife or sharp pick. Breaking of hand specimens requires several hard blows of geologist's pick.	<b>MEDIUM</b>	Can be grooved or gouged 1/16 inch deep by firm pressure on knife or pick point. Can be excavated in small chips to pieces about 1 inch maximum size by hard blows of the point of a geologist's pick.
<b>HARD</b>	Can be scratched with knife or pick only with difficulty. Hard blow of hammer required to detach hand specimen.	<b>SOFT</b>	Can be gauged or grooved readily with knife or pick point. Can be excavated in chips to pieces several inches in size by moderate blows of a pick point. Small thin pieces can be broken by finger pressure.
<b>MODERATELY HARD</b>	Can be scratched with knife or pick. Gouges or grooves to 1/4 inch deep can be excavated by hard blow or point of a geologist's pick. Hard specimen can be detached by moderate blow.	<b>VERY SOFT</b>	Can be carved with knife. Can be excavated readily with point of pick. Pieces 1 inch or more in thickness can be broken with finger pressure. Can be scratched readily by fingernail.

### JOINT BEDDING AND FOLIATION SPACING IN ROCK\*

Spacing	Joints	Bedding and Foliation
Less than 2 in.	Very close	Very thin
2 in. to 1 ft.	Close	Thin
1 ft. to 3 ft.	Moderately close	Medium
3 ft. to 10 ft.	Wide	Thick
More than 10 ft.	Very Wide	Very thick

### ROCK QUALITY DESIGNATOR (RQD)\*\*

RQD, as a percentage	Diagnostic description
Exceeding 90	Excellent
90-75	Good
75-50	Fair
50-25	Poor
Less than 25	Very poor

\*Joint spacing refers to the distance normal to the plane of the joints of a single system or "set" of joints that are parallel to each other or nearby so. The spacing of each "set" should be described, if possible to establish.

\*\*RQD should always be given as a percentage. Diagnostic description is intended primarily for evaluating problems with tunnels or excavation in rock. RQD = 100 (lengths of core in pieces 4 in. and longer/length of run)(1 in. = 25.4 mm; 1 ft. = 0.305 m)

## KEY TO BEDROCK DESCRIPTIONS

# EXPLORATORY BORING: EB-1

Sheet 1 of 1

DRILL RIG: MOBILE B-61  
 BORING TYPE: 8 INCH HOLLOW-STEM AUGER  
 LOGGED BY: MQL  
 START DATE: 12-17-04      FINISH DATE: 12-17-04

PROJECT NO: 1620-25  
 PROJECT: CANADA COLLEGE CHILLER  
 LOCATION: REDWOOD CITY, CA  
 COMPLETION DEPTH: 16.5 FT.

This log is a part of a report by Lowney Associates, and should not be used as a stand-alone document. This description applies only to the location of the exploration at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with time. The description presented is a simplification of actual conditions encountered. Transitions between soil types may be gradual.

ELEVATION (FT)	DEPTH (FT)	SOIL LEGEND	MATERIAL DESCRIPTION AND REMARKS	SOIL TYPE	PENETRATION RESISTANCE (BLOWS/FT.)	SAMPLER	MOISTURE CONTENT (%)	DRY DENSITY (PCF)	PERCENT PASSING NO. 200 SIEVE	Undrained Shear Strength (ksf)							
										○ Pocket Penetrometer	△ Torvane	● Unconfined Compression	▲ U-U Triaxial Compression				
										1.0	2.0	3.0	4.0				
694.5	0		SURFACE ELEVATION: 695 FT. (+/-)														
694.2	0.2		4 inches asphalt concrete over 4 inches aggregate base														
693.8	0.4		<b>SANDY LEAN CLAY (CL) [FILL]*</b> stiff, moist, reddish brown, fine sand, some fine gravel, low to moderate plasticity	CL, FILL	8	X	11	108									
692.8	1.4		<b>CLAYEY SAND WITH GRAVEL (SC) [FILL]*</b> loose, moist, reddish brown, fine to coarse sand, fine gravel	SC, FILL	9	X											
	5				13	X	13	117									
687.0	10		<b>POORLY GRADED SAND WITH SILT (SP-SM) [FILL]*</b> loose, moist, gray and brown, fine to medium sand, fine gravel	SP-SM, FILL	8	X	4	107									
	15		becomes wet		11	X	18	98									
678.0	16.5		encountered very hard material (concrete like) Bottom of Boring at 16½ feet (refusal)			○											
	20		*Appears to be backfill for previous underground tank removal in the boring area.														
	25																
	30																

**GROUND WATER OBSERVATIONS:**

▽ : FREE GROUND WATER MEASURED DURING DRILLING AT 13.5 FEET

LA CORP.GDT-1/14/05 MW-FLL

# EXPLORATORY BORING: EB-2

Sheet 1 of 1

DRILL RIG: MINUTEMAN

BORING TYPE: 4 INCH FLIGHT AUGER

LOGGED BY: MQL

START DATE: 12-16-04      FINISH DATE: 12-16-04

PROJECT NO: 1620-25

PROJECT: CANADA COLLEGE CHILLER

LOCATION: REDWOOD CITY, CA

COMPLETION DEPTH: 12.5 FT.

This log is a part of a report by Lowney Associates, and should not be used as a stand-alone document. This description applies only to the location of the exploration at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with time. The description presented is a simplification of actual conditions encountered. Transitions between soil types may be gradual.

ELEVATION (FT)	DEPTH (FT)	SOIL LEGEND	MATERIAL DESCRIPTION AND REMARKS	SOIL TYPE	PENETRATION RESISTANCE (BLOWS/FT.)	SAMPLER	MOISTURE CONTENT (%)	DRY DENSITY (PCF)	PERCENT PASSING NO. 200 SIEVE	Undrained Shear Strength (ksf)
692.0	0		SURFACE ELEVATION: 692 FT. (+/-)							
691.3	0.7		<b>FAT CLAY WITH SAND (CH)</b> stiff, moist, brown, fine sand, moderate to high plasticity Plasticity Index = 27, Liquid Limit = 51	CH	48	7	122			
	1.4		<b>GREENSTONE [Franciscan Formation]</b> very soft to soft, severely weathered, highly fractured, brown to reddish brown		50					
	5.0		greenish brown	fg	71	8				
	10.0				50/5"	9				
679.5	12.5		Bottom of Boring at 12½ feet		67	6				

- Undrained Shear Strength (ksf)
- Pocket Penetrometer
  - △ Torvane
  - Unconfined Compression
  - ▲ U-U Triaxial Compression

GROUND WATER OBSERVATIONS:  
NO FREE GROUND WATER ENCOUNTERED

LA CORP.GDT 2/2/05 MV-FLL

# EXPLORATORY BORING: EB-3

Sheet 1 of 1

DRILL RIG: MOBILE B-61  
 BORING TYPE: 8 INCH HOLLOW-STEM AUGER  
 LOGGED BY: BMM  
 START DATE: 12-23-04      FINISH DATE: 12-23-04

PROJECT NO: 1620-25  
 PROJECT: CANADA COLLEGE CHILLER  
 LOCATION: REDWOOD CITY, CA  
 COMPLETION DEPTH: 25.0 FT.

This log is a part of a report by Lowney Associates, and should not be used as a stand-alone document. This description applies only to the location of the exploration at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with time. The description presented is a simplification of actual conditions encountered. Transitions between soil types may be gradual.

ELEVATION (FT)	DEPTH (FT)	SOIL LEGEND	MATERIAL DESCRIPTION AND REMARKS	SOIL TYPE	PENETRATION RESISTANCE (BLOWS/FT.)	SAMPLER	MOISTURE CONTENT (%)	DRY DENSITY (PCF)	PERCENT PASSING NO. 200 SIEVE	Undrained Shear Strength (ksf)
696.5	0		SURFACE ELEVATION: 697 FT. (+/-)							
696.3	0		2 inches asphalt concrete over 2½ inches aggregate base							
696.0	0		<b>CLAYEY SAND (SC)</b> very dense, moist, brown to reddish brown mottled, fine to medium sand, fine gravel Plasticity Index = 13, Liquid Limit = 34	SC	50/5"	X	14	119		
693.5	5		<b>GREENSTONE [Franciscan Formation]</b> soft, very severely weathered, highly fractured, brown to reddish brown		50/5"	X	11			
	10		hard, slightly weathered, highly fractured, dark green		50/5"	X	12			
	15			fg	50/1"	X				
	20				50/1"	X				
671.5	25		Bottom of Boring at 25 feet			X				
	30									

- Undrained Shear Strength (ksf)
- Pocket Penetrometer
  - △ Torvane
  - Unconfined Compression
  - ▲ U-U Triaxial Compression
- 1.0    2.0    3.0    4.0

GROUND WATER OBSERVATIONS:  
 NO FREE GROUND WATER ENCOUNTERED

LA CORP.GDT 1/12/05 MV\* FLL

**APPENDIX B**  
**LABORATORY PROGRAM**

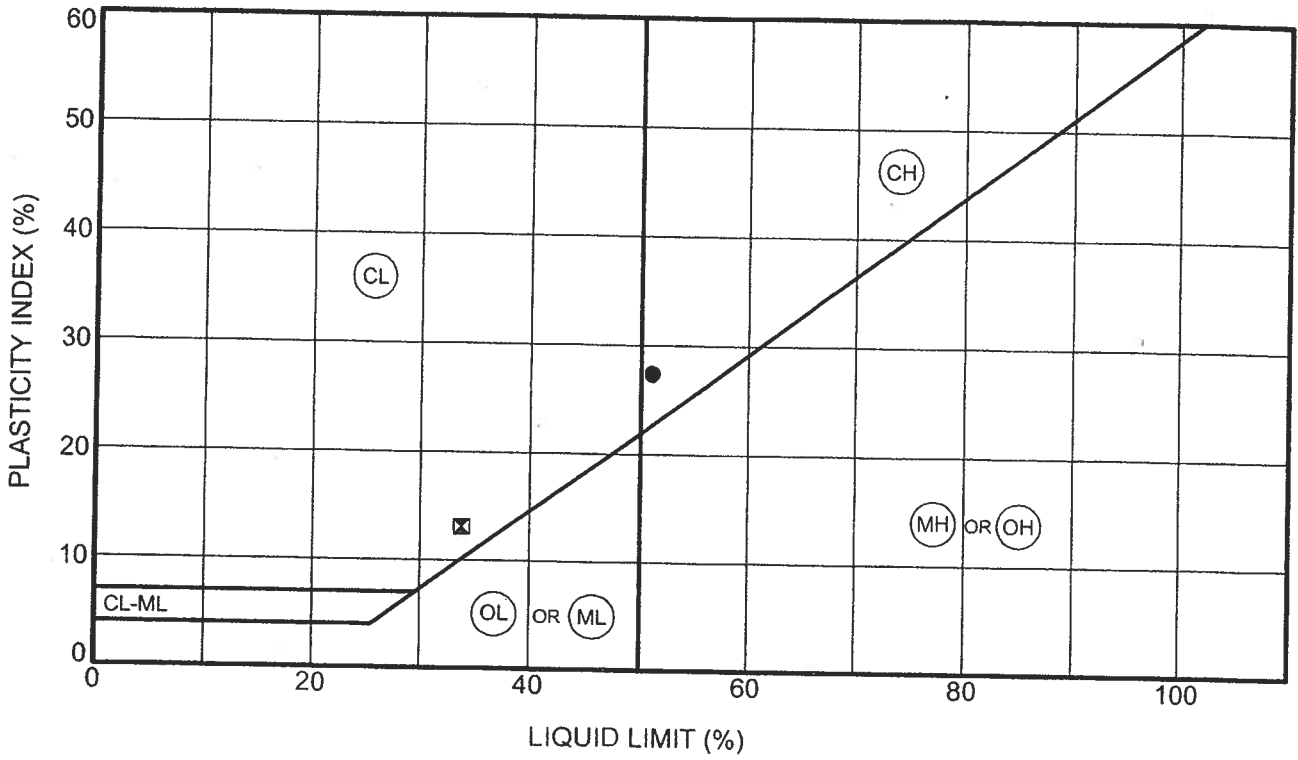
The laboratory testing program was performed to evaluate the physical and mechanical properties of the soils and rock underlying the site and to aid in verifying soil classification.

**Moisture Content:** The natural water content was determined (ASTM D2216) on 11 samples of the materials recovered from the borings. These water contents are recorded on the boring logs at the appropriate sample depths.

**Dry Densities:** In place dry density determinations (ASTM D2937) were performed on six samples to measure the unit weight of the subsurface soils and bedrock. Results of these tests are shown on the boring logs at the appropriate sample depths.

**Plasticity Index:** Plasticity Index determinations (ASTM D4318) were performed on two samples of the subsurface soils to measure the range of water contents over which these materials exhibit plasticity. The Plasticity Index was used to classify the soil in accordance with the Unified Soil Classification System and to evaluate the soil expansion potential. Results of these tests are presented on Figure B-1 and on the logs of the borings at the appropriate sample depths.

\* \* \* \* \*



Symbol	Boring No.	Depth (ft.)	Natural Water Content (%)	Liquid Limit (%)	Plastic Limit (%)	Plasticity Index (%)	Passing No. 200 Sieve	Unified Soil Classification Description
●	EB-2	0.3		51	24	27		FAT CLAY WITH SAND (CH)
⊠	EB-3	2.0	14	34	21	13		CLAYEY SAND (SC)

LA CORP.GDT\_2/2/05 MV\* FLL

**FIGURE B-1**