Appendix D Geotechnical Reports

CORNERSTONE EARTH GROUP

Type of Services	**Geotechnical and Geologic Investigation**
Project Name	**Crystal Springs Uplands Middle School**
Location	**Davis Drive** **Belmont, California**
Client	**Crystal Springs Uplands School**
Client Address	**400 Uplands Drive** **Hillsborough, CA 94010**
Project Number	**471-1-1**
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TABLE OF CONTENTS

SECTION 1: INTRODUCTION 1
1.1 Project Description 1
1.2 Scope of Services 1
1.3 Exploration Program 2
1.4 Laboratory Testing Program 2
1.5 Corrosion Evaluation 2
1.6 Environmental Services 2
SECTION 2: REGIONAL GEOLOGIC SETTING 2
2.1 Regional Seismicity 3
Table 1: Approximate Fault Distances within 25-Kilometers 4
SECTION 3: SITE CONDITIONS 4
3.1 Site Reconnaissance and Surface Description 4
3.2 Geomorphology, Recent History, Surface Conditions, and Previous Grading 5
3.3 Site geology and Subsurface Geotechnical Conditions 6
3.3.1 Plasticity/Expansion Potential 7
3.3.2 In-Situ Moisture Contents 7
3.4 Ground Water 7
SECTION 4: GEOLOGIC HAZARDS 7
4.1 Fault Rupture 7
4.2 Estimated Ground Shaking 8
4.3 Liquefaction Potential 8
4.4 Lateral Spreading 8
4.5 Seismic Settlement/Unsaturated Sand Shaking 8
4.6 Landsliding 8
4.7 SEISMICALLY INDUCED WAVES-Tsunami/seiche 9
4.8 Volcanic Eruption 9
SECTION 5: SLOPE STABILITY ANALYSES 10
5.1 General 10
5.2 Method of Analysis 10

5.3 Soil Properties ---------- 11
Table 3: Soil Parameters 11
5.4 Static Analysis ---------- 11
5.5 Seismic Analysis ---------- 12
5.6 Slope Stability Results ---------- 12
SECTION 6: CONCLUSIONS 12
6.1 Summary ---------- 12
6.1.1 Presence of Previously Placed Undocumented Fill 13
6.1.2 Presence of Moderately Expansive Soils 13
6.1.3 Variable Consistency or Hardness of Spread Footing Bearing Materials 13
6.1.4 Slope Instability 13
6.2 Plans and Specifications Review ---------- 13
6.3 Construction Observation and Testing ---------- 14
SECTION 7: EARTHWORK 14
7.1 Site Demolition, Clearing and Preparation ---------- 14
7.1.1 Site Stripping 14
7.1.2 Tree and Shrub Removal 14
7.1.3 Demolition of Existing Pavements, Slabs, Foundations and Irrigation System Piping and Components 14
7.1.4 Abandonment of Existing Utilities 15
7.2 Removal of Existing Fills ---------- 15
7.3 Temporary Cut and Fill Slopes ---------- 15
7.4 Construction Dewatering ---------- 16
7.5 Subgrade Preparation ---------- 16
7.6 Subgrade Stabilization Measures ---------- 16
7.6.1 Scarification and Drying 16
7.6.2 Removal and Replacement 16
7.6.3 Chemical Treatment 17
7.7 Material for Fill ---------- 17
7.7.1 Re-Use of On-site Soils 17
7.7.2 Re-Use of On-Site Site Improvements 17
7.7.3 Potential Import Sources 17
7.8 Compaction Requirements ---------- 18
Table 4: Compaction Requirements 18
7.8.1 Construction Moisture Conditioning 19
7.9 Trench Backfill ---------- 19
7.10 Site Drainage ---------- 20
7.11 Landscape Considerations ---------- 20

SECTION 8: FOUNDATIONS 20
8.1 Summary of Recommendations 20
8.2 Seismic Design Criteria 21
Table 5: CBC Site Categorization and Site Coefficients 21
8.3 Shallow Foundations 21
8.3.1 Spread Footings 21
Table 6: Estimated Top or Beginning Elevation of Weathered Sandstone to Support Spread Footings 22
8.3.2 Footing Settlement 22
Table 7: Assumed Structural Loading 23
8.3.3 Lateral Loading 23
8.3.4 Spread Footing Construction Considerations 23
SECTION 9: CONCRETE SLABS AND PEDESTRIAN PAVEMENTS 24
9.1 Interior Slabs-on-Grade 24
9.2 Interior Slabs Moisture Protection Considerations 24
9.3 Exterior Flatwork 25
9.4 Pedestrian Pavers 26
SECTION 10: VEHICULAR PAVEMENTS 26
10.1 Asphalt Concrete 26
Table 8: Asphalt Concrete Pavement Recommendations, Design R-value = 10 26
10.2 Portland Cement Concrete 27
Table 9: PCC Pavement Recommendations, Design R-value = 10 27
SECTION 11: RETAINING WALLS 27
11.1 Static Lateral Earth Pressures 27
Table 10: Recommended Lateral Earth Pressures 28
11.2 Seismic Lateral Earth Pressures 28
11.3 Wall Drainage 28
11.4 WALL Backfill 29
11.5 WALL Foundations 29
SECTION 12: SWIMMING POOLS 29
12.1 Earth Pressures 29
12.2 Swimming Pool Decks 30
12.3 Pool Sub-drainage 30
SECTION 13: LIMITATIONS 30
SECTION 14: REFERENCES 31

FIGURE 1: VICINITY MAP
FIGURE 2: SITE PLAN
FIGURE 3: CROSS SECTION A-A'
FIGURE 4: REGIONAL FAULT MAP
FIGURE 5: VICINITY GEOLOGIC MAP

APPENDIX A: FIELD INVESTIGATION
APPENDIX B: LABORATORY TEST PROGRAM
APPENDIX C: SITE CORROSIVITY EVALUATION
APPENDIX D: SLOPE STABILITY RESULTS

Type of Services	**Geotechnical and Geologic Investigation**
Project Name	**Crystal Springs Uplands Middle School**
Location	**Davis Drive** **Belmont, California**

SECTION 1: INTRODUCTION

This geotechnical and geologic investigation report was prepared for the sole use of Crystal Springs Uplands School for the planned Crystal Springs Uplands Middle School that will be located on the west side of Davis Drive in Belmont, California. The location of the approximately 2-acre site is shown on the Vicinity Map, Figure1.

For our use, we were provided with the ALTA/ACSM Land Title Survey prepared by B&H Surveying, Inc. and dated February, 2011, as well as an architectural conceptual drawing of the proposed school campus dated May 9, 2011.

1.1 PROJECT DESCRIPTION

The proposed project consists of the demolition of the existing commercial buildings and ancillary site developments presently located on the site and the construction of new school structures up to 3 stories in height, and on the order of 60,000 square feet. Planned structures include a Classroom building, an Art/Café building, a new Gymnasium/Locker facility and a pool. The gymnasium may be lowered a half-level below existing grade. Appurtenant parking, utilities, landscaping, a recreational sportsfield, and other improvements necessary for site development are also planned.

We anticipate that the structures will be of either steel- or wood-frame construction. Structural loads were not available at the time of the preparation of this report; however, we expect structural loads to be relatively light and within typical ranges for the planned type of building. We expect that only minor site grading will be needed for the relatively level site. The site is bounded by commercial development to the north, and east, open space to the south, and a school to the west.

1.2 SCOPE OF SERVICES

Our scope of services was presented in our proposal dated May 17, 2011 and consisted of field and laboratory programs to evaluate physical and engineering properties of the subsurface soils, engineering analysis to prepare recommendations for site work and grading, engineering

setbacks, building foundations, flatwork, retaining walls, and pavements, and preparation of this report. Brief descriptions of our exploration and laboratory programs are presented below.

1.3 EXPLORATION PROGRAM

Field exploration consisted of nine borings drilled on July 22, 2011 with truck-mounted hollow-stem auger drilling equipment and two test pits excavated and logged on August 23, 2011. The borings were drilled to depths of 5½ to 20 feet. The approximate locations of our exploratory borings and test pits are shown on the Site Plan, Figure 2. Details regarding our field program are included in Appendix A.

The borings were backfilled with cement grout in accordance with local requirements. The test pits were backfilled with the excavated soils and compacted with the weight of the backhoe bucket. Exploration permits were obtained prior to conducting the field investigation, as required by local jurisdictions.

1.4 LABORATORY TESTING PROGRAM

In addition to visual classification of samples, the laboratory program focused on obtaining data for foundation design and seismic ground deformation estimates. Testing included moisture contents, dry densities, a Plasticity Index test, triaxial testing, and corrosivity testing. Details regarding our laboratory program are included in Appendix B.

1.5 CORROSION EVALUATION

We performed two field resistivity measurements in the field, and submitted three (3) samples from our borings from depths from about 2 to 3 feet were chemically analyzed for corrosivity, including saturated resistivity, pH, soluble sulfates and chlorides. JDH Corrosion Consultants prepared a soil corrosion evaluation letter based on the laboratory data, which is attached to this report in Appendix C. In general, due to the low levels of water-soluble sulfates found in these soils, there is no special requirement for sulfate resistant concrete to be used at this site. The soils at the project site are generally considered to be “moderately corrosive” to ductile/cast iron, steel and dielectric coated steel based on the saturated resistivity measurements. For more information on the evaluation, please refer to the letter in Appendix C.

1.6 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 GEOLOGIC SETTING

The subject site is located on a flat-topped ridge within the San Francisco Peninsula segment of the Santa Cruz Mountains. The San Francisco Peninsula is a relatively narrow band of terrain, about 5 to 20 miles wide, at the north end of the Santa Cruz Mountains separating the Pacific

Ocean from San Francisco Bay. The San Francisco Peninsula is one mountain range in a series of northwesterly-aligned mountains forming the Coast Ranges Geomorphic Province of California that stretches from the Oregon border to nearly Point Conception. In the San Francisco Bay area, most of the Coast Ranges developed on a basement of tectonically mixed Cretaceous- and Jurassic-age (70- to 200-million years old) rocks of the Franciscan Complex. Locally, in some areas but not at the site, younger sedimentary and volcanic rocks cap the basement rocks. Still younger surficial deposits that reflect geologic conditions of the last million years or so cover most of the Coast Ranges.

Movement on the many minor faults (splay faults) associated with the San Andreas Fault system 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 is part of a system that extends from the San Gregorio Fault near the coastline to the Coast Ranges-Central Valley blind thrust at the western edge of the Great Central Valley as shown on the Regional Fault Map, Figure 4. 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 subparallel or branch faults within the San Andreas system are equally active and nearly as capable of generating large earthquakes. Right-lateral movement dominates on these faults but an increasingly large amount of thrust faulting resulting from compression across the system is now being identified also.

2.1 REGIONAL SEISMICITY

Geologists and seismologists recognize the San Francisco Bay Area as one of the most seismically active regions in the United State, including the coastal region. Significant earthquakes occurring in the area are generally associated with crustal movement along well-defined, active fault zones of the San Andreas Fault system (Figure 4). The San Andreas Fault generated the great San Francisco earthquake of 1906 and the Loma Prieta earthquake of 1989 and passes about 1.7 miles west of the site. The potentially active Monte Vista-Shannon fault is located 6.1 miles south of the site. The San Gregorio fault is located about 9.6 miles east of the site.

The Working Group on California Earthquake Probabilities (2007) developed estimates of earthquake probabilities in the San Francisco Bay area for the period from 2002 to 2031. Their findings suggest the probability of a Magnitude 6.7 or greater earthquake occurring during this time period in the San Francisco Bay region is 62 percent. The probability of a Magnitude 6.7 or greater earthquake on the San Francisco peninsula segment of the San Andreas Fault, that is the controlling ground motion fault for the site, is believed to be 11 percent in this time period. During such an earthquake the danger of fault ground rupture at the site is slight, but strong ground shaking would likely occur.

Faults considered capable of generating significant earthquakes are generally associated with the well-defined areas of crustal movement, which trend northwesterly. Table 1 presents the presently state recognized active faults within 25 kilometers (15½ 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.

Table 1: Approximate Fault Distances within 25-Kilometers

Fault Name	Distance (miles)	Distance (kilometers)
San Andreas (1906)	1.7	2.7
Monte Vista-Shannon	6.1	9.8
San Gregorio	9.6	15.5

A regional fault map is presented as Figure 4, illustrating the relative distances of the site to significant fault zones.

SECTION 3: SITE CONDITIONS

3.1 SITE RECONNAISSANCE AND SURFACE DESCRIPTION

Our Certified Engineering Geologist performed a reconnaissance of the site and immediate vicinity on July 22, 2011 for the purpose of observing site conditions. As already noted, the project site is located on foothills of the San Francisco Peninsula Range at the north end of the Santa Cruz Mountains. The site consists of two parcels totaling approximately 5.42 acres or 236,140 square feet. A one- and two-story commercial building, a concrete tilt-up warehouse building, and surrounding parking and landscaped areas are presently located on the site as shown on the Site Plan and Geologic Map, Figure 2. The site is bounded by Davis Drive and an adjacent commercial building site to the east, open downward sloping ground to the south and west, and an adjacent commercial building site to the north.

Figure 2 shows the topography of the area, existing structures, distribution of cut and fill slopes and geologic features and materials and the locations of our nine exploratory borings and two test pits. The aerial distribution of soil and bedrock materials at the site are also mapped on Figure 2. Geologic Cross Section A-A' was generated from the site geologic map as well as some of the exploratory boring and test pit data (Figure 3). The developed portion of the property is extensively landscaped and also hardscaped with asphalt and/or concrete walkways. The parking lot and access drive showed no obvious signs of distress.

An approximately 2.4:1 (horizontal to vertical) cut slope separates the site from the Ralston Middle School site to the west and exposes blocky Franciscan greenstone throughout. A fill slope is located south of this area, which was placed over a slope depression or drainage in order to create more parking spaces. The approximately 2.3:1 to 2.6:1 fill and natural slopes located on the west side of the parking lot and south of the buildings have a moderate to locally thick growth of brush and canopy of trees. Runoff from the parking lot is collected at catch basins and delivered to the slope located west of the site. The natural slope located south of the site is largely unaltered except for a vehicle width hiking trail located about 100 feet down slope of the site.

A small wet-ground spring area was noted on the south facing natural slope. A subtle slope depression, which we interpreted to be a possible older slump, occurs at this same slope position. These features are shown on the Site Plan and Geologic Map, Figure 2. No landslide debris was noted below this subtle scarp.

3.2 GEOMORPHOLOGY, RECENT HISTORY, SURFACE CONDITIONS, AND PREVIOUS GRADING

The site is nearly topographically flat with locally low areas that were created to facilitate drainage to catch basins. Ground surface elevations mostly range from about 300 to 305 feet NGVD (National Geodetic Vertical Datum) over the site development area. The site is bordered by natural down slopes on the south and a steep, east facing cut slope separating the subject site from the Ralston Middle School located to the west. The site was created primarily by slightly cutting into a ridge top and redistributing the cuttings around the site to fill locally low areas, facilitate drainage and achieve the design grades. The greatest amount of cut occurred at the approximately 30-foot-high, 2.4:1 (horizontal distance to vertical height ratio) cut slope along the west property line.

The ground surface is flat for some distance on the north and east sides of the site. Surface runoff (sheet wash) has deposited surficial materials (interlayered colluvium and alluvium) over the older geologic formations (principally Franciscan Complex bedrock) on slopes adjacent to the south of the site. As a result of the initial grading of the site, surficial man-made fills locally have been placed over the underlying geologic formations. The location of cut/fill contacts were inferred from the topographic characteristics of the site. Because development of the site required cutting and redistributing the excavated materials to create the design grades, the naturally occurring soils and bedrock were primarily observed on the perimeter slopes on the southeast, south, southwest and west sides of the site.

Aerial photographs listed in the References, Section 13.0, show the site as it looked at six times during the period from 1946 to 1974. The earliest photos (1946) show the site and surrounding ridgetop areas are totally undeveloped and exist as open grass-covered fields. The 1956 photos show several dirt roads criss-crossing the ridgetop in the area, including one trending through the subject site, but no structures exist and no roads appear paved. The forest cover on the sloping portions of the site appear even and uninterrupted. By 1968, Davis Drive has been paved and the subject site contains two buildings and a small parking lot. The adjacent Ralston School has been developed with buildings and playfields. The slope at the west of the subject site appears to be primarily a cut slope with perhaps a relatively thin fill at the top. Landscaping exists around the main building complex. By 1973 additional paved access drives and a parking lot was constructed. According to Allison Knapp Wollam Consulting, an environmental consultant, the building at 8 Davis Drive was constructed sometime in 1964 and the building at 10 Davis Drive was constructed in 1979. Google Earth Images reviewed covering a period from 1993 through 2011 revealed no changes at the site in the subsequent years of aerial photo coverage.

3.3 SITE GEOLOGY AND SUBSURFACE GEOTECHNICAL CONDITIONS

Below the ground surface in unpaved areas, and below the existing pavement in the paved areas of the site, our explorations generally encountered a sequence of:

- previously placed fill, ranging in thickness from none to about 3 feet and mostly described as a medium dense, reddish brown, clayey sand with gravel, over,

- completely weathered sandstone bedrock (residual soil), varying from about ½ to 3 feet thick and mostly described as a dense to very dense, reddish brown, clayey sand with gravel, over,

- weathered to highly weathered sandstone bedrock – described as low to moderately hard (rock hardness scale) sandstone

Exposures of subsurface materials occur sparsely at the ground surface. Franciscan sandstone underlies most of the site. Due to the extensive development, landscaping and surficial soils overlying the sandstone, they are not exposed at the ground surface. The cut slope that separates the site from the Ralston Middle School along and near the west property line exposes blocky Franciscan greenstone throughout. Where exposed, the greenstone forms hard blocky outcrops. It is moderately to severely weathered and of a very hard condition (i.e., produces a ringing sound when struck with a hammer). The contact between sandstone (upslope) and greenstone (downslope) has been projected through the southerly facing natural slope as shown on Figure 2, (Site Geologic Map). Pampeyan (1994) shows bedding at the roadcut for Ralston Drive (within the sandstone) as oriented with a 40 degree dip to the southwest. However our Test Pit TP-2 encountered sandstone with bedding dipping very gently 4° to the southeast. The tectonic evolution of the Franciscan complex is such that structure is typically highly variable over distance.

A pavement section was encountered at Borings EB-1, EB-2 and EB-4 through EB-9. The pavement section consisted of 3 to 3½ inches of asphalt over 3 to 6 inches of aggregate base. Previously placed, and generally medium dense fill varying in thickness from about 3½ to 5½ feet was encountered in Borings EB-2, EB-3 and EB-4. At our test pits, TP-1 and TP-2, colluvium comprised of silty sand, with variable amounts of gravel underlies the previously placed fill. Our exploratory test pits revealed colluvial soils overlying sandstone at those locations. Where encountered, the sandstone is severely weathered, soft to moderately hard (rock hardness), fractured (with close spacing), and is massive in structure. The upper few to several feet of the sandstone is deeply weathered so the bonds between crystals are broken and the material is largely friable and can be disaggregated with small amounts of finger pressure. At Test Pit TP-2 the weathering within the sandstone retains its original structure, which is thin bedded to thick bedded. Sample return on the sandstone and shale was generally good (80 to 90%) and produced uncorrected blow counts of about 41 blows per foot (bpf) to practical refusal (100 bpf or greater) with 100 bpf, or greater, being the more common value encountered.

3.3.1 Plasticity/Expansion Potential

We performed one Plasticity Index (PI) tests on a representative shallow existing-fill soil sample obtained at a depth of 1½ feet in Boring EB-4. The PI test result was used to evaluate expansion potential of the shallow soils. The PI test results indicate a PI of 21 indicating low to moderate expansion potential to wetting and drying cycles.

3.3.2 In-Situ Moisture Contents

Laboratory testing indicate that moisture contents within the upper 10 feet range from about 7 to 17 percent in the previously placed fill and 5 to 14 percent in the sandstone bedrock underlying the fill.

3.4 GROUND WATER

Ground water was not encountered in the borings or test pits. As already noted, we observed a small spring area on the southerly facing, natural slope located down slope of the developed portion of the site. This spring may occur as a result of the hard greenstone, which is overlain by the generally more permeable and fractured sandstone. This groundwater is likely a localized condition rather than being indicative of a laterally extensive groundwater table.

Fluctuations in ground water levels can occur due to many factors including rain fall, irrigation, surface water and runoffs, and other factors not in evidence at the time our measurements were made.

SECTION 4: GEOLOGIC HAZARDS

4.1 FAULT RUPTURE

Although there are several significant faults located within 25 kilometers of the site no active or potentially active faults are mapped transecting the site. Accordingly, the site is not located within a currently designated Alquist-Priolo Earthquake Fault Zone (known formerly as a Special Studies Zone) (CDMG, 1974). We observed no geomorphic or tonal evidence in the aerial photos that would suggest the presence of a fault surface trace transecting the site. Pampeyan (1994) shows three discontinuous faults cutting Franciscan age rocks in the area with one surface trace located approximately 1,360 feet to the east, another trace 1,960 feet southwest of the main building and another surface trace located approximately 600 feet south of the site. These faults are not considered active and therefore would not impact the subject site.

A regional fault map showing known faults in the region surrounding the subject site is presented in Figure 5. It is our conclusion that there is a low potential for the occurrence of fault surface rupture (primary or coseismic) to occur at the subject site.

As discussed above several significant faults are located within 25 kilometers of the site. The site is not located within a State-designated Alquist-Priolo Earthquake Fault Zone, or a Santa Clara County Fault Hazard Zone. As shown in Figure 3, no known surface expression of fault

traces have been recognized to cross the site; therefore, fault rupture hazard is not a significant geologic hazard at the site.

4.2 ESTIMATED GROUND SHAKING

Moderate to severe (design-level) earthquakes can cause strong ground shaking, which is the case for most sites within the Bay Area. A peak ground acceleration (PGA) was estimated for analysis using a value equal to $S_{DS}/2.5$ as allowed in the California Building Code. For our liquefaction and slope stability analysis, we used a PGA of 0.55g.

4.3 LIQUEFACTION POTENTIAL

Liquefaction results from loss of strength during cyclic loading, such as imposed by earthquakes. Soils most susceptible to liquefaction are Quaternary-aged, clean, loose, saturated, uniformly graded, fine-grained sands or silts. San Mateo County shows the site located in an area with very low potential for liquefaction. No evidence of liquefaction or associated ground failures was identified in the area following the 1989 Loma Prieta earthquake (Tinsley et.al., 1998). The site is also not currently mapped within a State-designated Liquefaction Hazard Zone. Because the building areas at the site are underlain by medium dense to dense soils and bedrock, it is our engineering judgment that the potential for liquefaction of these materials as a result of seismic shaking is low. This is consistent with the geological mapping in the area.

4.4 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 the exposed slope. As discussed, the site is underlain by medium dense to dense thin soils underlain by dense bedrock; therefore, it is our opinion that the potential for lateral spreading to affect the site is low.

4.5 SEISMIC SETTLEMENT/UNSATURATED SAND SHAKING

Loose, unsaturated sandy soils can settle during strong seismic shaking. As the soils encountered at the site were predominantly stiff to very stiff clayey sands with gravel underlain by bedrock, in our opinion, the potential for significant differential seismic settlement affecting the proposed improvements during seismic shaking is low.

4.6 LANDSLIDING

The published maps reviewed indicate no landslides mapped at or immediately upslope of the site (Brabb and Pampeyan, 1972; Pampeyan 1994; County of San Mateo Planning Department, 1997, Brabb et al., 1998). The County Landslide Map (based on Wentworth et al., 1997) shows the site in an area characterized as containing few landslides. The site is located in an area interpreted in published maps as having a moderate susceptibility for landsliding during a major

earthquake (Weiczorek et al., 1985). The steep southerly facing natural slope located on the south of the site is located in a mapped “potential debris flow source area” (County of San Mateo Planning Department, 1997). It should be noted these published interpretive maps are based on generalized characterization and are applicable for planning purposes but do not necessarily include site specific subsurface data. We did observe a subtle lobate shaped slope depression on the slope located about 125 feet south of the slope crest. This feature may be an old slump scar but does not appear associated with any landforms that would suggest an accumulation of landslide debris.

We identified a relatively thin accumulation of undocumented fill along the slope crest of the southerly facing slope could be potential debris flow source material. A debris flow from that source, however, would move downslope in a southerly direction and away from the subject site in an area that is part of an undeveloped County Park (Water Dog Park). The cutslope at the west property line is underlain essentially at the ground surface by hard greenstone bedrock of a stable configuration and condition. In our opinion, the potential for landsliding impacting the site appears low provided the recommendations for engineering setbacks are implemented. Further discussion of landsliding and our analysis is in the Slope Stability section of this report.

4.7 SEISMICALLY INDUCED WAVES-TSUNAMI/SEICHE

The terms tsunami or seiche are described as ocean waves or similar waves usually created by undersea fault movement or by a coastal or submerged landslide. Tsunamis may be generated at great distance from shore (far field events) or nearby (near field events). Waves are formed, as the displaced water moves to regain equilibrium, and radiates across the open water, similar to ripples from a rock being thrown into a pond. When the waveform reaches the coastline, it quickly raises the water level, with water velocities as high as 15 to 20 knots. The water mass, as well as vessels, vehicles, or other objects in its path create tremendous forces as they impact coastal structures.

The site is situated on a ridge top at an elevation of approximately 301 feet above mean sea level. 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. The topographic position and elevation are such that this potential hazard is not anticipated at the site.

4.8 VOLCANIC ERUPTION

The youngest volcanic rocks mapped in the site area are isolated patches shown by Brabb et al. (2000) about 9 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 building site.

SECTION 5: SLOPE STABILITY ANALYSES

5.1 GENERAL

The proposed Classroom and Art/Café Buildings are planned near the natural slope at the south end of the site. Slope stability analyses were performed to evaluate the stability of the southern slope. As part of our analyses, we developed a representative geologic cross-section of the southern slope at the west end of the proposed Classroom structure where the topography extended the furthest downslope and provided the most information. This condition was, in our opinion, representative of the slope and subsurface conditions. Our cross section was based on our site reconnaissance, mapped site conditions, regional geologic information, subsurface conditions encountered in our borings and test pits, and site grades on the project topo prepared by B&H Surveying, Inc. Factors of safety computed via slope stability analyses are primarily dependent on the geometric model used and the strength parameters of the various components of the cross section.

Based on our test pits, relatively flatly bedded sandstone mantled massive bedrock, and further discontinuities could not be recorded. For this reason, kinematic analysis to examine modes of potential failure could not be performed. In an effort to document slope stability, we retained bulk samples to remold for strength testing and rotational failure modes. We judge that this approach will be conservative as the remolding process will remove most of the natural structure and cementation of the sandstone.

A brief discussion of our slope stability methodology is presented below.

5.2 METHOD OF ANALYSIS

The stability of a slope is influenced by many factors including but not limited to the geologic structure and composition, inclination, height of slope, ground water, and climatic factors such as rainfall and irrigation. In geotechnical engineering, “stability” is expressed as a ratio of resisting moments and forces divided by driving moments and forces termed the factor of safety. Factors of safety can be calculated for static and seismic loading conditions. In performing the slope stability analysis, we followed the guidelines set forth by CGS in Special Publication 117A (2008).

The stability of the southern slope was evaluated using the computer program GSTABL7, and circular modes of failure. Input parameters for the analyses include slope geometry, soil layers or zones, total and saturated unit weights and strength parameters, as well as ground water conditions. Our soil layers, soil properties, and ground water assumptions were interpreted based on our review of information, site reconnaissance, and our experience. We modeled a perched ground water condition above the less permeable greenstone mapped at the site, within the sandstone.

In evaluating the stability of slopes under seismic conditions, GSTABL7 uses a "pseudo-static" method of analysis. The pseudo-static method models the effects of transient or pulsating earthquake loading on a potential slide mass by using an “equivalent” static horizontal

acceleration acting on the mass of the potential landslide in a limit-equilibrium analysis. The ground motion parameter used in a pseudo-static analysis is referred to as the seismic coefficient "k". CGS (2008) has published recommendations for the selection of the "k" value in a publication titled, "Guidelines for Evaluation and Mitigation of Seismic Hazards in California, SP 117A." The site is located adjacent to the San Andreas Fault and strong ground shaking can be expected during a seismic event near the site. In accordance with the CGS Guidelines, we have performed our pseudo-static analysis using simplified design procedures in accordance with Stewart and others (2003) to develop a "screen analysis procedure," based on a pseudo-static approach that accounts for the anticipated seismicity at the site and allows for different levels of acceptable displacements. For a magnitude M7 earthquake, and limiting displacements to a 15 cm threshold, we obtained a "k" value of 0.38 for our analysis.

Based on current procedures recommended in SP 117A, the minimum allowable factor of safety with respect to slope stability is 1.5 for static conditions. Slopes that have a pseudo-static factor of safety greater than 1.0 using a seismic coefficient derived from the screening analysis procedure of Stewart and others (2003), can be considered stable.

5.3 SOIL PROPERTIES

Our slope stability analyses utilized three different soil units, which included the surficial undocumented fills, the weathered sandstone bedrock, and the underlying greenstone bedrock.

The soil parameters used in our slope stability analyses are shown in Table 1 below, and on the individual stability analysis sections presented in Appendix C. The strength parameters were selected based on laboratory test data, our experience, and our engineering judgment.

Table 3: Soil Parameters

Soil Description	Saturated Unit Weight (pcf)	Unit Strength (Static/Seismic)	
		Cohesion (psf)	Phi (degrees)
Undocumented Fill	128	50	30
Bedrock (Sandstone)	133	700	22
Bedrock (Greenstone)	141	700	36

5.4 STATIC ANALYSIS

Using the subsurface profile depicted on Cross Section A-A' (Figure 3), and topography from the most recent available survey data (B&H Surveying, Inc., 2011), we performed static slope stability analysis using the limit equilibrium computer program GSTABL7. The objective of our analysis was to evaluate the potential failure of the southern slope adjacent to the planned structures, and whether the slope would yield factors of safety greater than 1.5 under static conditions. The Modified Bishop method of analyses was used in our static slope stability analyses. In these methods, the computer program was allowed to search for the circular failure surface corresponding to the lowest factor of safety for the pertinent slope.

Our analyses indicate that factors of safety with respect to slope stability are greater than 1.5, implying stability under static conditions.

Copies of the stability output, also illustrating the soil parameters, subsurface geometry, and low factor-of-safety surfaces, are attached in Appendix C.

5.5 SEISMIC ANALYSIS

We also performed seismic slope stability analysis for the proposed cross sections. As previously discussed, a factor of safety greater than 1.0 would imply relative stability and limited deformation under seismic conditions. Our analyses indicate that factor of safety values with respect to seismic slope stability are greater than 1.0, implying relative stability.

Copies of the seismic analysis stability output is attached in Appendix C.

5.6 SLOPE STABILITY RESULTS

The southern slope, in close proximity to the proposed structures, appears stable under both static and seismic conditions. In our opinion, because of the variability of materials present, the presence of undocumented fill at the top of slope, the proximity of the San Andreas Fault and probability of strong shaking, and the potential for minor slope instability near the crest of the slope during strong shaking, we recommend a minimum engineering setback of 15 feet from the top of slope.

In addition, in our opinion, removing the undocumented fill and replacing it with engineered fill in the area of the proposed structures, and within the engineering setback zone, would reduce the potential for minor landsliding at the top of slope resulting from strong seismic shaking. Recommendations are presented in subsequent report sections.

SECTION 6: CONCLUSIONS

6.1 SUMMARY

From a geotechnical viewpoint, the project is feasible provided the concerns listed below are addressed in the project design. Descriptions of each concern with brief outlines of our recommendations follow the listed concerns.

- Presence of Previously Placed Undocumented Fill
- Presence of Moderately Expansive Soils
- Variable Consistency or Hardness of Spread Footing Bearing Materials
- Potential for Slope Instability

6.1.1 Presence of Previously Placed Undocumented Fill

Our borings and test pits encountered previously placed fill to estimated depths up to about 5 feet, mostly around the southern and southwestern perimeter areas of the site. In our judgment, undocumented fills may have variable and unreliable strength and compressibility properties and should not be used for structural support. All previously placed fill should be excavated, reworked and moisture conditioned and recompacted in accordance with our recommendations discussed in following sections of this report.

6.1.2 Presence of Moderately Expansive Soils

As discussed, moderately expansive surficial soils were encountered in the surficial soils that blanket the site. Expansive soils can undergo significant volume change with changes in moisture content. They shrink and harden when dried and expand and soften when wetted. If structures are underlain by expansive soils it is important that foundation systems be capable of tolerating or resisting any potentially damaging soil movements. In addition, it is important to limit moisture changes in the surficial soils by using positive drainage away from buildings as well as limiting landscaping watering. Grading and foundation recommendations addressing this concern are presented in Sections 6.10 and 6.11 of this report.

6.1.3 Variable Consistency or Hardness of Spread Footing Bearing Materials

As discussed further in following sections of this report, the consistency or hardness of the weathered sandstone soils and bedrock at depths of ½ foot to about 3 feet are expected to range from very dense or hard soil consistency to moderately hard to hard rock. For this reason, we may recommend over-excavation and replacement with well compacted aggregate base or approved equivalent at some footing locations at the time of construction to provide either a cushion beneath footings on moderately hard to hard bedrock or to extend footings 1 or 2 feet deeper where lower consistency soils may occur.

6.1.4 Slope Instability

As discussed in Section 5, we performed a slope stability evaluation for the project. The results of our analysis indicated that the stability of the slope has an adequate safety factor for static and seismic conditions. However, we recommend an engineering setback from the top of slope of 15 feet. Minor seepage may occur on the face of the slope below the development. This seepage may cause minor, localized slumps in the thin surficial soils. We judge that the minor seepage and possible localized slumps have a low potential to affect the proposed development.

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 the small-diameter borings performed during this investigation, we also recommend that a Cornerstone Earth Group 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 on Cornerstone Earth Group (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 site should be stripped of all surface vegetation, and surface and subsurface improvements within the proposed development area. Surface vegetation and topsoil should be stripped to a sufficient depth to remove all material greater than 3 percent organic content by weight. Based on our site observations, surficial stripping should extend about 2 to 3 inches below existing grade in vegetated areas.

A detailed discussion of removal of existing fills is provided later in this report. Demolition of existing improvements is discussed in detail below.

7.1.2 Tree and Shrub Removal

Trees and shrubs designated for removal should have the root balls and any roots greater than ½-inch diameter removed completely. Mature trees are estimated to have root balls extending to depths of 2 to 4 feet, depending on the tree size. Significant root zones are anticipated to extend to the diameter of the tree canopy. Grade depressions resulting from root ball 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 Pavements, Slabs, Foundations and Irrigation System Piping and Components

All pavements, slabs, miscellaneous foundations, irrigation system piping and components should be completely removed from within planned building areas. A discussion of recycling existing improvements is provided later in this report.

7.1.4 Abandonment of Existing Utilities

All utility piping and components should be completely removed from within planned building areas. For any utility line to be considered acceptable to remain within building areas, the utility line must be completely backfilled with grout or sand-cement slurry (sand slurry is not acceptable), the ends outside the building 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 building areas unless provided written confirmation from both the owner and the geotechnical engineer.

Utilities extending beyond the building area 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

All previously placed fills should be completely removed from within building areas and to a lateral distance of at least 5 feet beyond the building footprint or to a lateral distance equal to fill depth below the perimeter footing, whichever is greater. We estimate that the amount of previously placed fill to be removed will vary from a ½ foot or less near the central portion of the site to about 3 to 4 feet thick average along the southern and southwestern perimeter of the site. Provided the excavated fills meet the “Material for Fill” requirements below, the fills may be reused when backfilling the excavations. Based on review of the samples collected from our borings and test pits, our preliminary judgment is that the existing fill may be reused for fill. If materials are encountered that do not meet the requirements, such as debris, wood, trash, those materials should screened out of the remaining material and be removed from the site. Backfill of excavations should be placed in lifts and compacted in accordance with the “Compaction” section below.

7.3 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. On a preliminary basis, the upper 12 feet at the site may be classified as OSHA Site B material, except where there is undocumented fill material mapped on Figure 2, which should be assumed to be Site C type material.

Excavations performed during site demolition and fill removal, if required, should be sloped at 3:1 (horizontal:vertical) within the upper 5 feet below building subgrade. OSHA classification determinations at the time of excavation may allow steeper inclinations in the underlying bedrock.

7.4 CONSTRUCTION DEWATERING

Ground water levels are expected to be more than about 10 feet below the bottom of spread footing excavations. Therefore, construction dewatering is not expected to be required for the construction other than for localized ponding or accumulation of water in shallow excavations during wet weather periods.

7.5 SUBGRADE PREPARATION

After site clearing, demolition, removal of previously placed fills in building areas, removal of existing fills in new pavement, flatwork and other improvement areas, 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.

7.6 SUBGRADE STABILIZATION MEASURES

Soil subgrade and fill materials, especially soils with high fines contents such as clays and silty soils, can become unstable due to high moisture content, whether from high in-situ moisture contents or from winter rains. As the moisture content increases over the laboratory optimum, it becomes more likely the materials will be subject to softening and yielding (pumping) from construction loading or become unworkable during placement and compaction.

There are several methods to address potential unstable soil conditions and facilitate fill placement and trench backfill. Some of the methods are briefly discussed below. Implementation of the appropriate stabilization measures should be evaluated on a case-by-case basis according to the project construction goals and the particular site conditions.

7.6.1 Scarification and Drying

The subgrade may be scarified to a depth of 6 to 8 inches and allowed to dry to near optimum conditions, if sufficient dry weather is anticipated to allow sufficient drying. More than one round of scarification may be needed to break up the soil clods.

7.6.2 Removal and Replacement

As an alternative to scarification, the contractor may choose to over-excavate the unstable soils and replace them with dry on-site or import materials. A Cornerstone representative should be present to provide recommendations regarding the appropriate depth of over-excavation,

whether a geosynthethic (stabilization fabric or geogrid) is recommended, and what materials are recommended for backfill.

7.6.3 Chemical Treatment

Where the unstable area exceeds about 5,000 to 10,000 square feet and/or site winterization is desired, chemical treatment with quicklime (CaO), kiln-dust, or cement may be more cost-effective than removal and replacement. Recommended chemical treatment depths will typically range from 12 to 18 inches depending on the magnitude of the instability.

7.7 MATERIAL FOR FILL

7.7.1 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.

7.7.2 Re-Use of On-Site Site Improvements

We anticipate that a potentially useful quantity of asphalt concrete (AC) grindings and aggregate base (AB) may be generated from demolition of the existing asphalt-paved area of the site. If the AC grindings are mixed with the underlying AB to meet Class 2 AB specifications, they may be reused within the new pavement and flatwork structural sections outside planned building areas. Mixed AC/AB grindings may not be reused within building areas. Laboratory testing will be required to confirm the grindings meet project specifications. The asphalt paving grinding operation may leave significant oversize chunks not meeting Class 2 AB gradation requirements but may meet Caltrans subbase requirements. Depending on the quantities of oversized material, the grindings may still be used within the pavement structural section; however, the pavement design will need to be modified to account for the difference, typically resulting in the addition of about 1 inch to the structural section.

7.7.3 Potential Import Sources

Imported and non-expansive material should be inorganic with a Plasticity Index (PI) of 15 or less, and should not contain recycled asphalt concrete where it will be used within the building areas. 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.

7.8 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 the table below. In general, clayey soils should be compacted with sheepsfoot equipment and sandy/gravelly soils with vibratory 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 recommendations 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[1] Compaction (percent)	Moisture[2] Content (percent)
General Fill (within upper 5 feet)	On-Site Expansive Soils	87 – 92	>3
	Low Expansion Soils	90	>1
Trench Backfill	On-Site Expansive Soils	87 – 92	>3
Trench Backfill	Low Expansion Soils	90	>1
Trench Backfill (upper 6 inches of 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	Optimum

Table 4 Continues.

Table 4 Continued.

Flatwork Subgrade	On-Site Expansive Soils	87 - 92	>3
Flatwork Subgrade	Low Expansion Soils	90	>1
Flatwork Aggregate Base	Class 2 Aggregate Base[3]	90	Optimum
Pavement Subgrade	On-Site Expansive Soils	87 - 92	>3
Pavement Subgrade	Low Expansion Soils	95	>1
Pavement Aggregate Base	Class 2 Aggregate Base[3]	95	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 light-weight compaction or walls should be braced

7.8.1 Construction Moisture Conditioning

Expansive soils can undergo significant volume change when dried then wetted. The contractor should keep all exposed expansive soil subgrade (and also trench excavation side walls) moist until protected by overlying improvements (or trenches are backfilled). If expansive soils are allowed to dry out significantly, re-moisture conditioning may require several days of re-wetting (flooding is not recommended), or deep scarification, moisture conditioning, and re-compaction.

7.9 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 (⅜-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.

On expansive soils sites it is desirable to reduce the potential for water migration into building and pavement areas through the granular shading materials. We recommend that a plug of low-permeability clay soil, sand-cement slurry, or lean concrete be placed within trenches just outside where the trenches pass into building and pavement areas.

We recommend that the project civil engineer or land surveyor be retained to survey in place solid pipes, cleanouts and sub-drainage lines, and create an as-built plan. This plan will be of use for any future maintenance or repair work.

7.10 SITE DRAINAGE

Ponding should not be allowed adjacent to building foundations, slabs-on-grade, or pavements. Hardscape surfaces should slope at least 1 to 2 percent towards suitable discharge facilities; landscape areas should slope at least 2 to 3 percent. Roof runoff should be directed away from building areas.

7.11 LANDSCAPE CONSIDERATIONS

Since the near-surface soils are moderately expansive, we recommend greatly reducing the amount of surface water infiltrating these soils near foundations and exterior slabs-on-grade. This can typically be achieved by:

- Using drip irrigation,
- Avoiding open planting within 3 feet of the building perimeter or near the top of existing slopes,
- Regulating the amount of water distributed to lawns or planter areas by using irrigation timers, and
- Selecting landscaping that requires little or no watering, especially near foundations.

We recommend that the landscape architect consider these items when developing landscaping plans.

SECTION 8: FOUNDATIONS

8.1 SUMMARY OF RECOMMENDATIONS

In our opinion, the proposed structures may be supported on shallow foundations provided the recommendations in the "Earthwork" section and this section are followed.

8.2 SEISMIC DESIGN CRITERIA

We understand that the project structural design will be based on the 2010 California Building Code (CBC), which provides criteria for the seismic design of buildings in Chapter 16. The "Seismic Coefficients" used to design buildings are established based on a series of tables and figures addressing different site factors, including the soil profile in the upper 100 feet below grade and mapped spectral acceleration parameters based on distance to the controlling seismic source/fault system. Based on our borings and review of local geology, the site is underlain by shallow bedrock. However, without more extensive testing of shear wave velocities of the bedrock, we have classified the site as Soil Classification C. The mapped spectral acceleration parameters S_S and S_1 were calculated using the USGS computer program *Earthquake Ground Motion Parameters*, Version 5.1.0, revision date February 10, 2011, based on the site coordinates presented below and the site classification. The table below lists the various factors used to determine the seismic coefficients and other parameters.

Table 5: CBC Site Categorization and Site Coefficients

Classification/Coefficient	Design Value
Site Class	C
Site Latitude	37.510594°
Site Longitude	-121.308987°
0.2-second Period Mapped Spectral Acceleration[1], S_S	2.072g
1-second Period Mapped Spectral Acceleration[1], S_1	1.127g
Short-Period Site Coefficient – Fa	1.0
Long-Period Site Coefficient – Fv	1.3
0.2-second Period, Maximum Considered Earthquake Spectral Response Acceleration Adjusted for Site Effects - S_{MS}	2.072g
1-second Period, Maximum Considered Earthquake Spectral Response Acceleration Adjusted for Site Effects – S_{M1}	1.465g
0.2-second Period, Design Earthquake Spectral Response Acceleration – S_{DS}	1.381g
1-second Period, Design Earthquake Spectral Response Acceleration – S_{D1}	0.977g

[1]For Site Class B, 5 percent damped.

8.3 SHALLOW FOUNDATIONS

8.3.1 Spread Footings

Spread footings should bear on very dense or stronger undisturbed weathered sandstone, be at least 18 inches wide, and extend at least 18 inches below the lowest adjacent grade. Lowest adjacent grade is defined as the deeper of the following: 1) bottom of the adjacent interior slab-on-grade, or 2) finished exterior grade, excluding landscaping topsoil.

Spread footings should bear at or below the estimated beginning of very dense or stronger weathered sandstone elevations shown in Table 4 below. We should observe the spread

footing excavations at the time of construction and may recommend over-excavation at some footing locations and replacement with compacted aggregate base, either to provide a more compressible cushion where the exposed sandstone is harder than adjacent footings or to extend footings into very dense or stronger weathered sandstone. For bidding purposes, an average of 24 inches of embedment for shallow footings would be a reasonable assumption, in our opinion.

Table 6: Estimated Top or Beginning Elevation of Weathered Sandstone to Support Spread Footings

Boring or Test Pit Location	Approximate Ground Surface Elevation[1.]	Estimated Top or Beginning of Weathered Sandstone
EB-1	301	300
EB-2	301	297
EB-3	301	296
EB-4	300	296
EB-5	303	302
EB-6	300	299
EB-7	302	301
EB-8	304	303
EB-9	303	302
TP-1	302	300
TP-2	301	297

Notes:
1. Elevations estimated at boring and test pit locations from Alta/ACSM Land Title Survey prepared by B&H Surveying, dated February 2011.

Footings constructed to the above dimensions and in accordance with the “Earthwork” recommendations of this report are capable of supporting maximum allowable bearing pressures 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. These 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 for the portion of the footing extending below grade (typically, the full footing depth). Top and bottom mats of reinforcing steel should be included in continuous footings to help span irregularities and differential settlement.

8.3.2 Footing Settlement

Structural loads were not provided to us at the time this report was prepared; therefore, we assumed the typical loading in the following table.

Table 7: Assumed Structural Loading

Foundation Area	Range of Assumed Loads
Interior Isolated Column Footing	75 to 150 kips
Exterior Isolated Column Footing	75 to 150 kips
Perimeter Strip Footing	3 to 5 kips per lineal foot

Based on the above loading and the allowable bearing pressures presented above, we estimate that total static footing settlements will be on the order of ½-inch, with about ¼-inch of post-construction static differential settlement between adjacent foundation elements. In addition we estimate that differential seismic movement will be on the order of less than ½-inch, resulting in estimated total post-construction static plus seismic differential footing movements of less than ¾-inch to 1-inches between foundation elements, assumed to be on the order of 30 feet. As our footing loads were assumed, we recommend we be retained to review the final footing layout and loading, and verify the settlement estimates above.

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 0.5 applied to the footing dead load, and an ultimate passive pressure based on an equivalent fluid pressure of 500 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 passive pressure capacity.

8.3.4 Spread 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. As previously discussed, we may recommend over-excavation and replacement with compacted aggregate base at some footing elevations, depending on the bearing soils exposed in the footing excavations. 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 the Plasticity Index (PI) of the surficial soils ranges up to 21, the proposed slabs-on-grade should be supported on at least 6 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 moisture-sensitive floor coverings are planned, the recommendations in the "Interior Slabs Moisture Protection Considerations" section below may be incorporated in the project design if desired. If significant time elapses between initial subgrade preparation and slab-on-grade [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 1 percent over the optimum moisture content.

The structural engineer should determine the appropriate slab reinforcement for the loading requirements and considering the expansion potential of the underlying soils. Consideration should be given to limiting the control joint spacing to a maximum of about 2 feet in each direction for each inch of unreinforced concrete thickness, and at greater distances for reinforced slabs.

9.2 INTERIOR SLABS MOISTURE PROTECTION CONSIDERATIONS

The following general guidelines for concrete slab-on-grade construction where floor coverings are planned are presented for the consideration by the developer, 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 EXTERIOR FLATWORK

Exterior slabs-on-grade, such as pedestrian walkways, patios, driveways, and sidewalks, may experience seasonal movement due to the native expansive soils; therefore, some cracking or vertical movement of conventional slabs should be anticipated where imported fill is not planned in flatwork areas. There are several alternatives for mitigating the impacts of expansive soils beneath concrete flatwork. We are providing recommendations to reduce distress to concrete flatwork that includes moisture conditioning the subgrade soils, using non-expansive fill, and providing adequate construction and control joints to control cracks that do occur. It should be noted that minor slab movement or localized cracking and/or distress could still occur.

- The minimum recommendation for concrete flatwork constructed on moderately expansive soils is to properly prepare the clayey soils prior to placing concrete. This is typically achieved by scarifying, moisture conditioning, and re-compacting the subgrade soil. Subgrade soil should be moisture conditioned to at least 3 percent over the laboratory optimum and compacted using moderate compaction effort to a relative compaction of 87 to 92 percent (ASTM Test Method D1557). Since the near surface soils may have been previously compacted and tested, the subgrade soils could possibly be moisture conditioned by gradually wetting the soil, depending on the time of year slab construction occurs. This should not include flooding or excessively watering the soil, which would likely result in a soft, unstable subgrade condition, and possible delays in the construction while waiting for the soil to dry out. In general, the subgrade should be relatively firm and non-yielding prior to construction.

- Concrete flatwork, excluding pavements that would be subject to wheel loads, should be at least 4 inches thick and underlain by at least 6 inches of Class II aggregate base to meet the non-expansive fill requirement. Non-expansive fill should be compacted to at least 90 percent relative compaction. Flatwork that will be subject to heavier or frequent vehicular loading should be designed in accordance with the recommendations in the "Vehicular Pavements" section below.

- We recommend a maximum control joint spacing of about 2 feet in each direction for each inch of concrete thickness and a construction joint spacing of 10 to 12 feet. Construction joints that abut the foundations or garage slabs should include a felt strip, or approved equivalent, that extends the full depth of the exterior slab. This will help to reduce the potential for permanent vertical offset between the slabs due to friction between the concrete edges. We recommend that exterior slabs be isolated from adjacent foundations.

At the owner's option, if desired to reduce the potential for vertical offset or widening of concrete cracks, consideration should be given to using reinforcing steel, such as No. 3 rebar spaced at 18 inches on center each direction.

9.4 PEDESTRIAN PAVERS

Concrete unit pavers subject to pedestrian loading only should be at least 60 mm thick and supported on at least 8 inches of Class 2 aggregate base overlying subgrade prepared in accordance with the "Earthwork" recommendations of this report. A maximum 1-inch-thick layer of sand may be used as a leveling/setting bed over the aggregate base. We should be retained to provide a design for pavers that will be subject to heavier or frequent vehicular loading.

SECTION 10: VEHICULAR PAVEMENTS

10.1 ASPHALT CONCRETE

The following asphalt concrete pavement recommendations tabulated below are generally based on the Procedure 608 of the Caltrans Highway Design Manual, estimated traffic indices for various pavement-loading conditions, and on a design R-value of 10 The design R-value was chosen based on the results of the laboratory testing performed on a surficial sample collected from the proposed pavement area and engineering judgment considering the variable surface conditions.

Table 8: Asphalt Concrete Pavement Recommendations, Design R-value = 10

Design Traffic Index (TI)	Asphalt Concrete (inches)	Class 2 Aggregate Base* (inches)	Total Pavement Section Thickness (inches)
4.0	2½	7	9½
4.5	2½	8½	11
5.0	3	10	13
5.5	3	11	14
6.0	3½	12½	16
6.5	4	12½	16½

*Caltrans Class 2 aggregate base; minimum R-value of 78

Frequently, the full asphalt concrete section is not constructed prior to construction traffic loading. This can result in significant loss of asphalt concrete layer life, rutting, or other pavement failures. To improve the pavement life and reduce the potential for pavement distress through construction, we recommend the full design asphalt concrete section be constructed prior to construction traffic loading. Alternatively, a higher traffic index may be chosen for the areas where construction traffic will be use the pavements.

Asphalt concrete pavements constructed on expansive subgrade where the adjacent areas will not be irrigated for several months after the pavements are constructed may experience longitudinal cracking parallel to the pavement edge. These cracks typically form within a few feet of the pavement edge and are due to seasonal wetting and drying of the adjacent soil. The cracking may also occur during construction where the adjacent grade is allowed to significantly dry during the summer, pulling moisture out of the pavement subgrade. Any cracks that form should be sealed with bituminous sealant prior to the start of winter rains. One alternative to reduce the potential for this type of cracking is to install a moisture barrier at least 24 inches deep behind the pavement curb.

10.2 PORTLAND CEMENT CONCRETE

The exterior Portland Cement Concrete (PCC) pavement recommendations tabulated below are based on methods presented in the Portland Cement Association (PCA) design manual (PCA, 1984). Recommendations for garage slabs-on-grade were provided in the "Concrete Slabs and Pedestrian Pavements" section above. We have provided a few pavement alternatives as an anticipated Average Daily Truck Traffic (ADTT) was not provided. An allowable ADTT should be chosen that is greater than what is expected for the development.

Table 9: PCC Pavement Recommendations, Design R-value = 10

Allowable ADTT	Minimum PCC Thickness (inches)
13	5½
130	6

The PCC thicknesses above are based on a concrete compressive strength of at least 3,500 psi, supporting the PCC on at least 6 inches of Class 2 aggregate base compacted as recommended in the "Earthwork" section, and laterally restraining the PCC with curbs or concrete shoulders. 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 unreinforced concrete thickness.

SECTION 11: RETAINING WALLS

11.1 STATIC LATERAL EARTH PRESSURES

The structural design of any site retaining wall should include resistance to lateral earth pressures that develop from the soil behind the wall, any undrained water pressure, and surcharge loads acting behind the wall. Below grade basement walls should be design for the restrained condition. Provided a drainage system is constructed behind the wall to prevent the build-up of hydrostatic pressures as discussed in the section below, we recommend that the walls with level backfill be designed for the following pressures:

Table 10: Recommended Lateral Earth Pressures

Wall Condition	Lateral Earth Pressure*	Additional Surcharge Loads
Unrestrained – Cantilever Wall	45 pcf	⅓ of vertical loads at top of wall
Restrained – Braced Wall	45 pcf + 8H** psf	½ of vertical loads at top of wall

* Lateral earth pressures are based on an equivalent fluid pressure for level backfill conditions
** H is the distance in feet between the bottom of footing and top of retained soil

If adequate drainage cannot be provided behind the wall, an additional equivalent fluid pressure of 40 pcf should be added to the values above for both restrained and unrestrained walls for the portion of the wall that will not have drainage. Damp proofing or waterproofing of the walls may be considered where moisture penetration and/or efflorescence are not desired.

11.2 SEISMIC LATERAL EARTH PRESSURES

Section 1802A.2.7 of the 2007 and 2010 CBC states that lateral pressures from earthquakes should be considered in the design of basements and retaining walls. We reviewed the seismic earth pressures for the proposed basement using procedures generally based on the Mononobe-Okabe method and updated provisional recommendations for design of building basement walls (Lew et al., SEAOC 2010). Although the proposed basement walls will reportedly be less than 10 feet, in our opinion, it is prudent to check the walls against the seismic increment because of the proximity of local faults.

As recommended in Lew et al., seismic earth increments should be added to the active earth pressure. Because active earth pressures plus a seismic increment exceed the total force from the recommended at-rest earth pressures, we recommend that below grade basement walls be checked for seismic conditions using an equivalent fluid pressure of 71 pcf (i.e. triangular distribution).

At this time, we are not aware of any exterior landscape retaining walls for the project. However, minor landscaping walls (i.e. walls 4 feet or less in height) may be proposed. In our opinion, design of these walls for seismic lateral earth pressures in addition to static earth pressures recommended above is not warranted.

11.3 WALL DRAINAGE

Adequate drainage should be provided by a subdrain system behind all walls. This system should consist of a 4-inch minimum diameter perforated pipe placed near the base of the wall (perforations placed downward). The pipe should be bedded and backfilled with Class 2 Permeable Material per Caltrans Standard Specifications, latest edition. The permeable backfill should extend at least 12 inches out from the wall and to within 2 feet of outside finished grade. Alternatively, ½-inch to ¾-inch crushed rock may be used in place of the Class 2 Permeable Material provided the crushed rock and pipe are enclosed in filter fabric, such as Mirafi 140N or approved equivalent. The upper 2 feet of wall backfill should consist of compacted on-site soil. The subdrain outlet should be connected to a free-draining outlet or sump.

Miradrain, Geotech Drainage Panels, or equivalent drainage matting can be used for wall drainage as an alternative to the Class 2 Permeable Material or drain rock backfill. Horizontal strip drains connecting to the vertical drainage matting may be used in lieu of the perforated pipe and crushed rock section. The vertical drainage panel should be connected to the perforated pipe or horizontal drainage strip at the base of the wall, or to some other closed or through-wall system such as the TotalDrain system from AmerDrain. Sections of horizontal drainage strips should be connected with either the manufacturer's connector pieces or by pulling back the filter fabric, overlapping the panel dimples, and replacing the filter fabric over the connection. At corners, a corner guard, corner connection insert, or a section of crushed rock covered with filter fabric must be used to maintain the drainage path.

Drainage panels should terminate 18 to 24 inches from final exterior grade. The Miradrain panel filter fabric should be extended over the top of and behind the panel to protect it from intrusion of the adjacent soil.

11.4 WALL BACKFILL

Where surface improvements will be located over the retaining wall backfill, backfill placed behind the walls should be compacted to at least 95 percent relative compaction using light compaction equipment. Where no surface improvements are planned, backfill should be compacted to at least 90 percent. If heavy compaction equipment is used, the walls should be temporarily braced.

11.5 WALL FOUNDATIONS

Retaining walls may be supported on a continuous spread footing designed in accordance with the recommendations presented in the "Foundations" section of this report.

SECTION 12: SWIMMING POOLS

12.1 EARTH PRESSURES

The swimming pool should be designed to resist at-rest earth pressures due to adjacent native or engineered backfill materials, hydrostatic pressures, as well as surcharge loads. Pool walls should be designed to resist an equivalent fluid pressure of 85 pcf (at-rest plus hydrostatic pressures) in addition to one-half of any surcharge load applied at the surface. Because of the shallow bedrock conditions at the site, we anticipate that pools will likely be able to be constructed with vertical cut slopes. Because some fills may be located in the upper 1 to 4 feet in areas of the site, bracing, stabilization or sloping of the upper fill materials may be required. Contractors should be made aware of the difficult excavation conditions below a depth of about 5 feet as the bedrock becomes fresher with depth.

12.2 SWIMMING POOL DECKS

Concrete flatwork and/or pavers around swimming pools should be at least 4 inches thick and supported on 8 inches of non-expansive fill overlying subgrade prepared in accordance with the "Earthwork" recommendations of this report. The upper 4 inches of the non-expansive fill should consist of Class 2 aggregate base.

Proper surface drainage should be provided to divert surface water to closed pipe storm drainage facilities. Flexible bituminous caulking or equivalent should be applied between the pool and deck and deck expansion joints to reduce surface water penetration into the native expansive soils.

12.3 POOL SUB-DRAINAGE

The pool should have pressure relief valves incorporated into the pool bottom to relieve pressure buildup and potential heave during pool draining for maintenance. Consideration should be given to placing at least 4 inches of Caltrans Class 1 Permeable Material below the pool bottom to allow for pressure relief across the pool. Alternatively, ¾-inch clean, crushed rock may be placed provided a layer of filter fabric is placed beneath the crushed rock.

SECTION 13: LIMITATIONS

This report, an instrument of professional service, has been prepared for the sole use of Crystal Springs Uplands School specifically to support the design of the Crystal Springs Uplands Middle School project in Belmont, 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.

Crystal Springs Uplands School may have provided Cornerstone with plans, reports and other documents prepared by others. Crystal Springs Uplands School 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.

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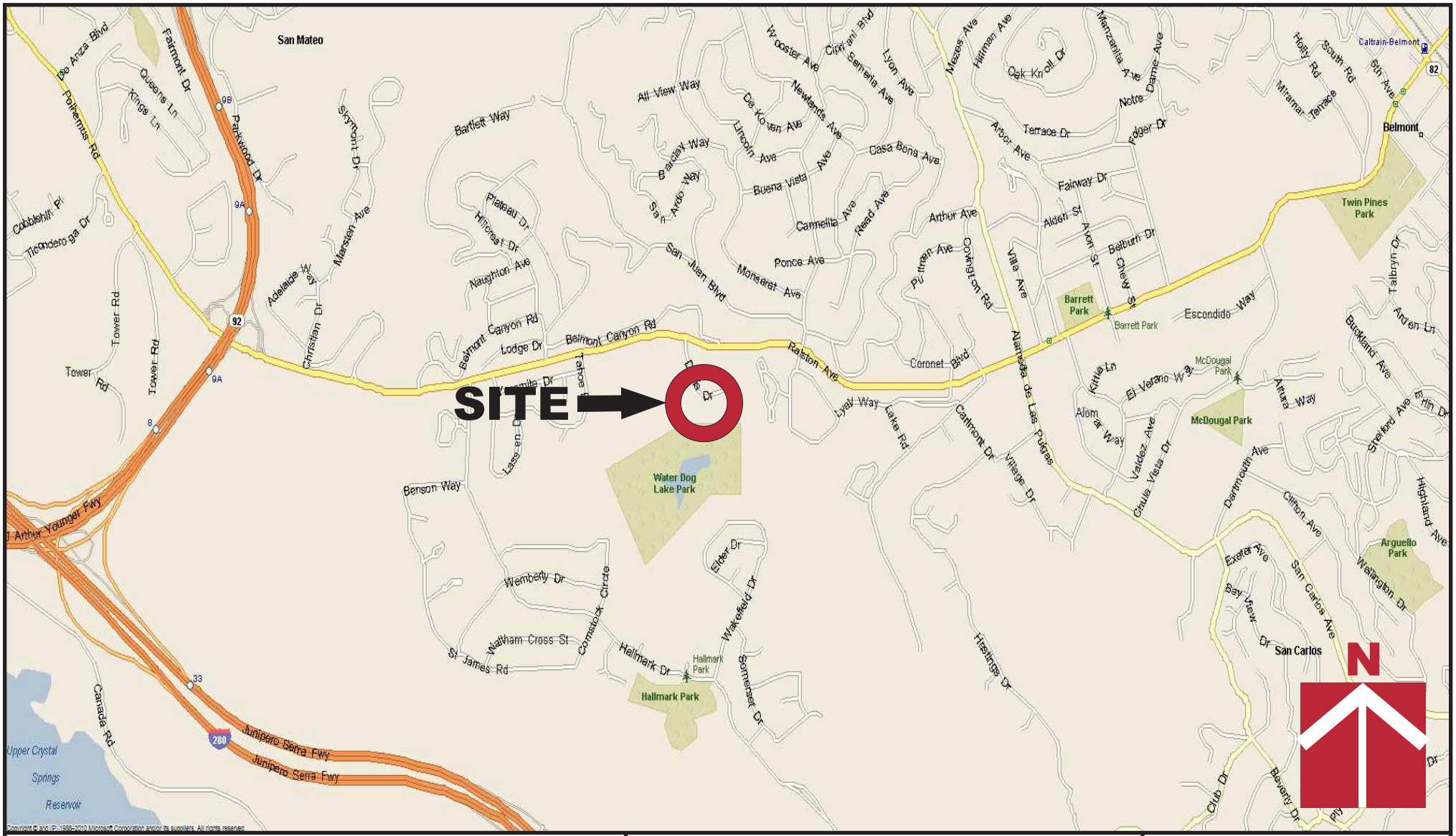
Aerial Photographs:

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

Date	Flight	Frames	Scale	Type
July 29, 1946	GS-CP-2	186, 187	1:23,600	black & white
June 9, 1956	DDB-3R	31, 32	1:20,000	black & white
April 20, 1968	GS-VBZJ-2	78, 79	1:30,000	black & white
May 9, 1973	3567-1	39	1:12,000	black & white
June 25, 1974	Area 9	15, 16	1:20,000	Natural Color



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CORNERSTONE EARTH GROUP

Vicinity Map

Crystal Springs Middle School
Belmont, CA

Project Number	471-1-1
Figure Number	Figure 1
Date	September 2011
Drawn By	RRN

PROPERTY DESCRIPTION:

PARCEL 1 AS SHOWN ON THAT CERTAIN PARCEL MAP FILED FOR RECORD ON DECEMBER 27, 1977, IN BOOK 40 OF PARCEL MAPS AT PAGE 10, SAN MATEO COUNTY.

PARCEL 2 AS SHOWN ON THAT CERTAIN PARCEL MAP FILED FOR RECORD ON DECEMBER 27, 1977, IN BOOK 40 OF PARCEL MAPS AT PAGE 10, SAN MATEO COUNTY.

PRELIMINARY TITLE REPORT:

NORTH AMERICAN TITLE COMPANY,
ORDER NO. 50905-1062634-11
ESCROW OFFICER: CATHERINE LEARY
TITLE OFFICER: JIM WHITE

PROPERTY OWNER:

THOMSON LEARNING, INC.

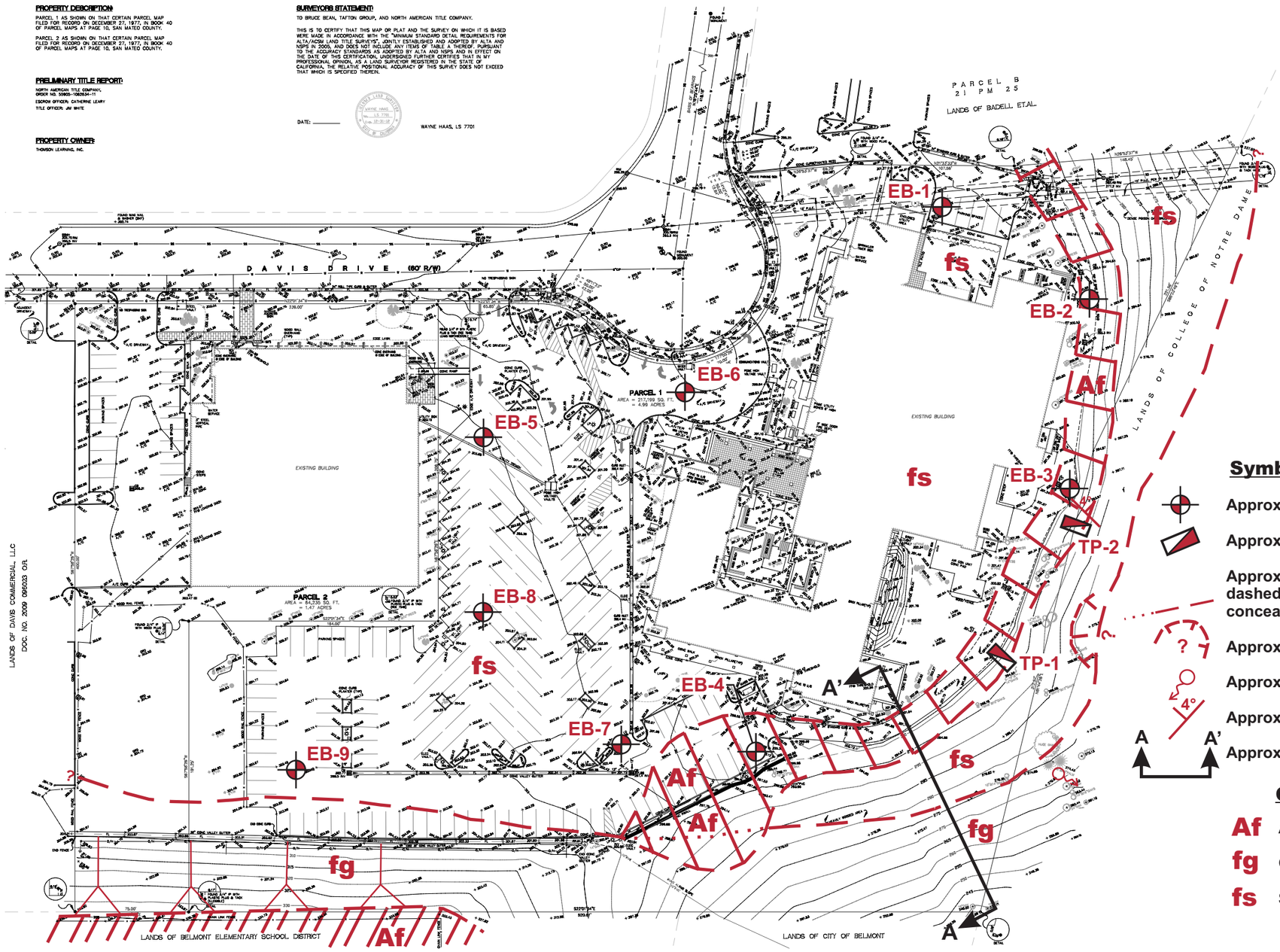
SURVEYOR'S STATEMENT:

TO BRUCE BEAN, TAFTON GROUP, AND NORTH AMERICAN TITLE COMPANY.

THIS IS TO CERTIFY THAT THIS MAP OR PLAT AND THE SURVEY ON WHICH IT IS BASED WERE MADE IN ACCORDANCE WITH THE "MINIMUM STANDARD DETAIL REQUIREMENTS FOR ALTA/ACSM LAND TITLE SURVEYS", JOINTLY ESTABLISHED AND ADOPTED BY ALTA AND NSPS IN 2005, AND DOES NOT INCLUDE ANY ITEMS OF TABLE A THEREOF. PURSUANT TO THE ACCURACY STANDARDS AS ADOPTED BY ALTA AND NSPS AND IN EFFECT ON THE DATE THIS CERTIFICATION, UNDERSIGNED FURTHER CERTIFIES THAT IN MY PROFESSIONAL OPINION, AS A LAND SURVEYOR REGISTERED IN THE STATE OF CALIFORNIA, THE RELATIVE POSITIONAL ACCURACY OF THIS SURVEY DOES NOT EXCEED THAT WHICH IS SPECIFIED THEREIN.

DATE: _____

WAYNE HAAS, LS 7701



Symbols

- Approximate location of exploratory boring
- Approximate location of exploratory test pit
- Approximate location of geologic contact, dashed where approximate, dotted where concealed
- Approximate location of possible landslide scar
- Approximate location of spring
- Approximate location of bedding orientation
- Approximate location of cross section

Geologic Units

- **Af** Artificial fill
- **fg** Greenstone of the Franiciscan complex
- **fs** Sandstone of the Franiciscan complex

0 75 140

APPROXIMATE SCALE (FEET)

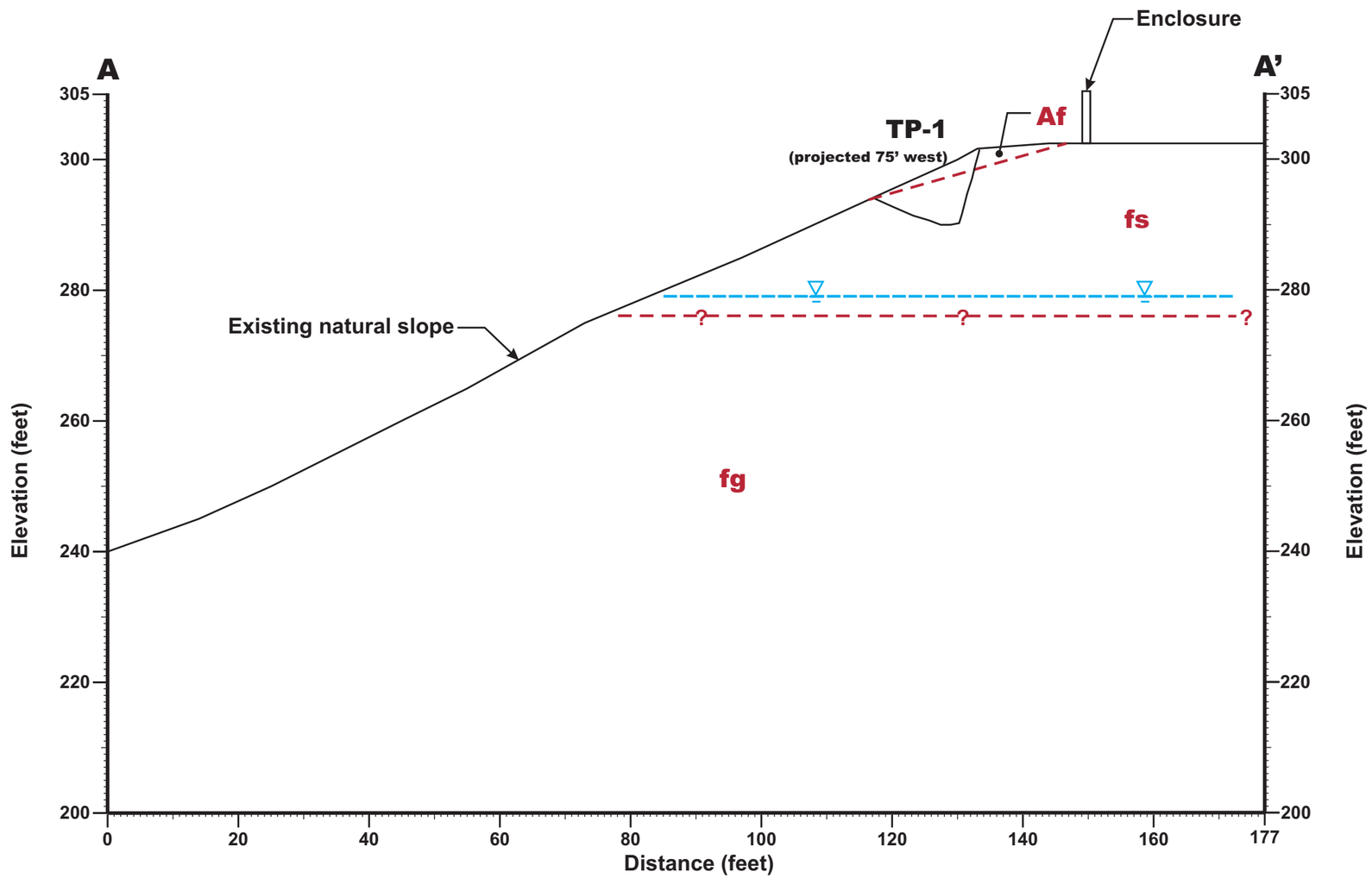
Base by B & H Surveying, Inc., " ALTA/ACSM Land Title Survey," dated February 2011

Project Number	471-1-1
Figure Number	Figure 2
Date	September 2011
Drawn By	RRN

Site Plan and Geologic Map

Crystal Springs Middle School
Belmont, CA

CORNERSTONE EARTH GROUP



Section A-A'

(View Looking Northwest)
1"=20' Horizontal
and Vertical

Explanation

Geologic Units

Af	Artifical fill (Historic)
fg	Greenstone, Franciscan Complex (Cretaceous and Jurassic)
fs	Sandstone, Franciscan Complex (Cretaceous and Jurassic)
- - -	Approximate geologic contact

Symbols

Inferred seasonally perched ground water

Notes:
1) Topographical information provided by B & H Surveying, Inc., dated February 2011.
2) The subsurface profile is conceptual and is based on limited subsurface data obtained from widely spaced borings. Actual subsurface conditions may vary significantly between borings.
3) See Figure 2 for location of cross section.

Project Number 471-1-1
Figure Number Figure 3
Date September 2011
Drawn By RRN

Geological Cross Section A-A'

Crystal Springs Middle School
Belmont, CA

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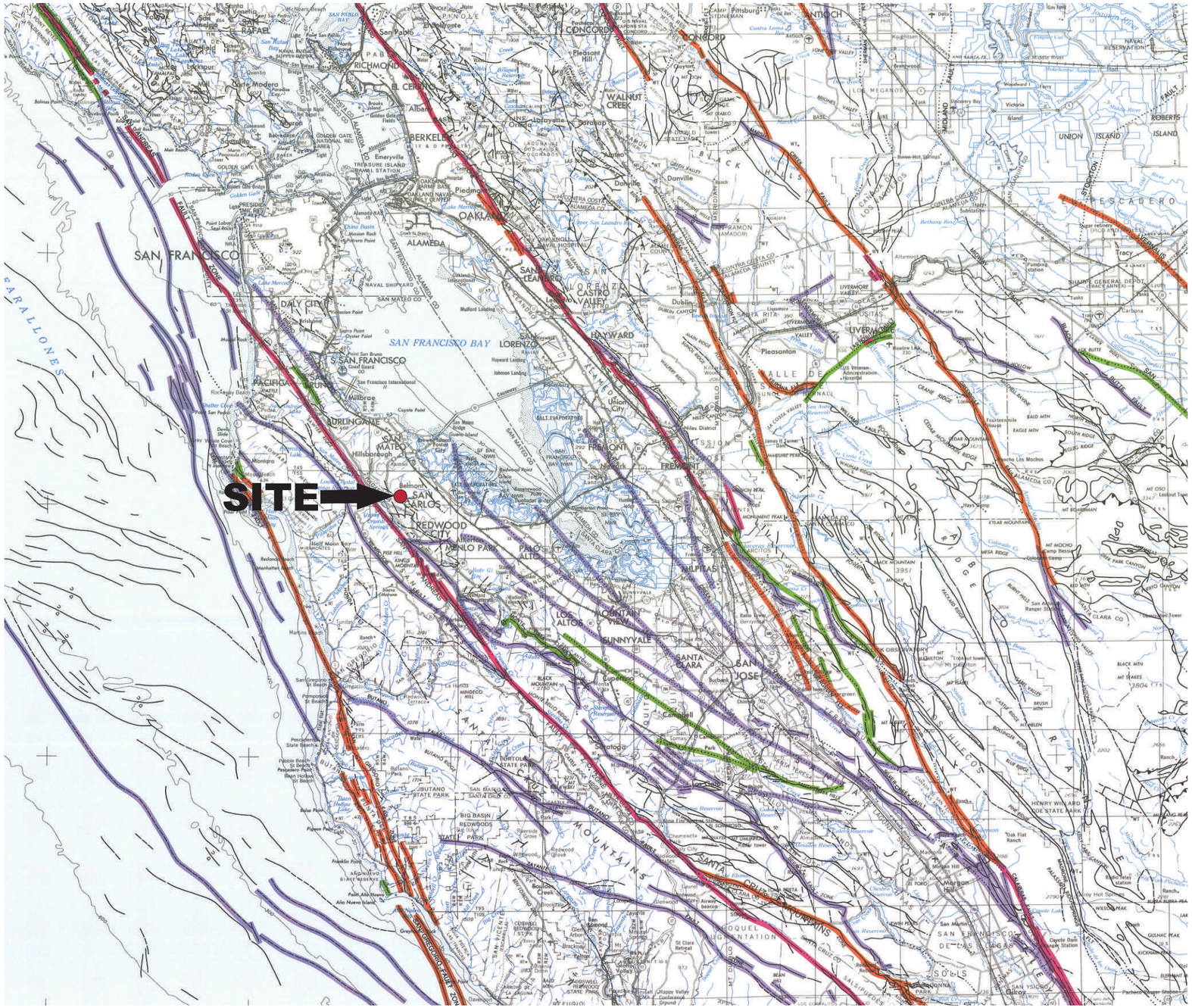


Geologic Time Scale			Years Before Present (Approx.)	Fault Symbol	Recency of Movement on Land Offshore[1]	DESCRIPTION
Quaternary	Late Quaternary	Historic				Displacement during historic time (e.g. San Andreas fault 1906). Includes areas of known fault creep.
		Holocene	200			Displacement during Holocene time.[2]
			10,000			Faults showing evidence of displacement during late Quaternary time.[3, 4]
	Early Quaternary	Pleistocene	700,000			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 Plio-Pleistocene age.
			2,000,000			
Pre-Quaternary	Pliocene		5,000,000			Fault showing evidence of no displacement during Quaternary time or faults without recognized Quaternary displacement.
	Miocene					

N

0 10 20

APPROXIMATE SCALE (MILES)

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 471-1-1

Figure Number Figure 4

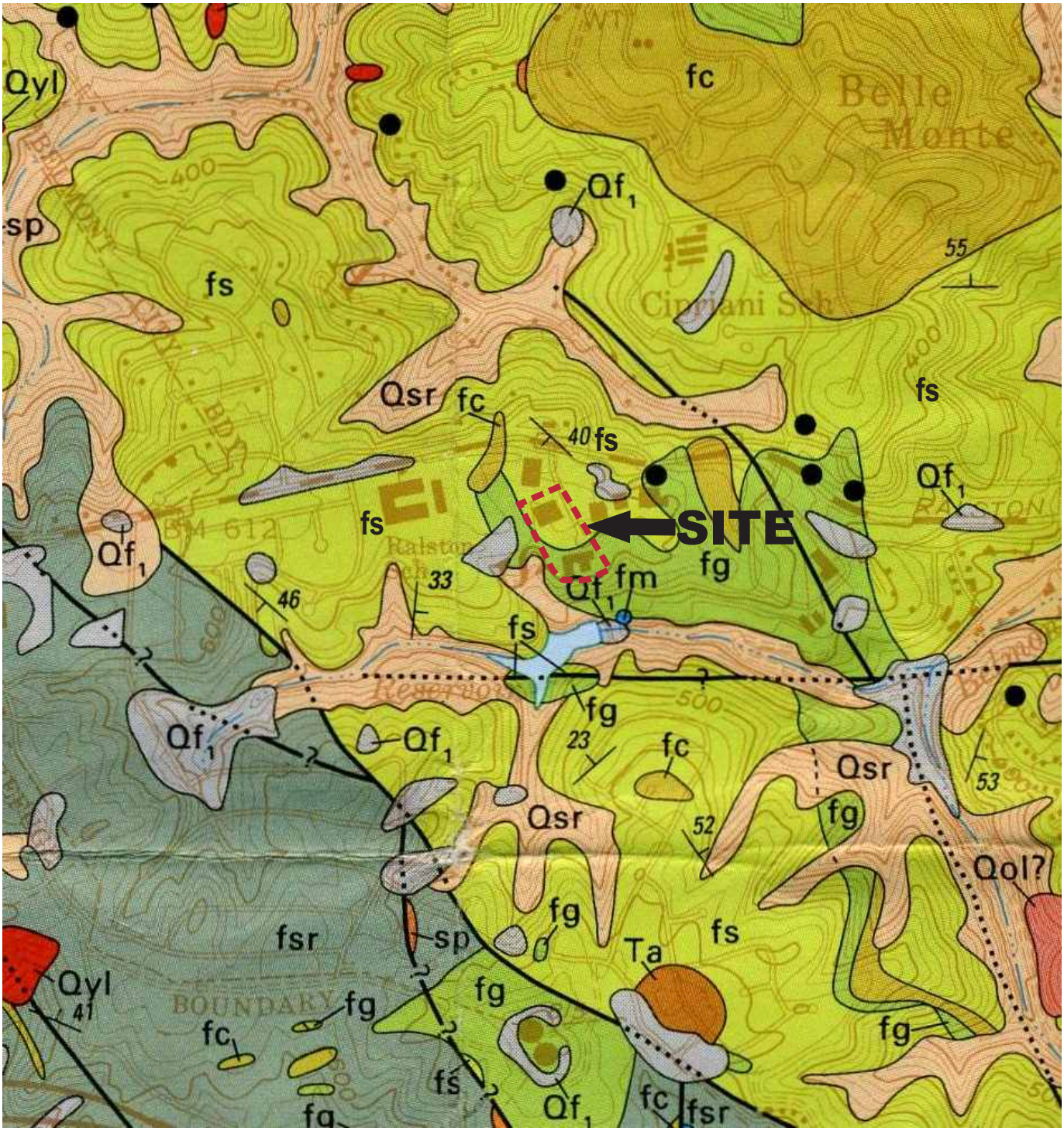
Date September 2011

Drawn By RRN

Regional Fault Map

Crystal Springs Middle School
Belmont, CA

CORNERSTONE EARTH GROUP



Geologic Units

Qf	Artificial Fill (Holocene)
Qsr	Geologically recent (Holocene) slope wash, ravine fill and colluvium
fg	Greenstone of the Franciscan Complex (Cretaceous or Jurassic)
fc	Chert of the Franciscan Complex (Cretaceous or Jurassic)
fm	Metamorphic rock of the Franciscan Complex (Cretaceous or Jurassic)
fs	Sandstone of the Franciscan Complex (Cretaceous or Jurassic)

Explanation

Contact- dotted where concealed, queried where uncertain

Fault- dashed where approximate, dotted where concealed, queried where uncertain

N

0 1000 2000

APPROXIMATE SCALE (FEET)

CORNERSTONE EARTH GROUP

Vicinity Geologic Map

Crystal Springs Middle School
Belmont, CA

Project Number	471-1-1
Figure Number	Figure 5
Date	September 2011
Drawn By	RRN

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 for the borings and a backhoe for the test pit excavations. Nine 8-inch-diameter exploratory borings were drilled on July 22, 2011 to depths of 5 to 20 feet. Two backhoe test pits were excavated to a depth of 12 feet on July 30, 2011. The approximate locations of exploratory borings and test pits are shown on the Site Plan, Figure 2. 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 and Test Pit Logs, as well as a key to the classification of the soil and bedrock, are included as part of this appendix.

Boring and test pit locations and elevations were approximated using existing site boundaries and other site features as references. Boring and Test Pit elevations were not determined by Cornerstone, but were approximated based on available topographic information. The locations and elevations of the borings and test pits should be considered accurate only to the degree implied by the method used.

Representative soil samples were obtained from the borings and test pits 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. The results of these tests are presented on the individual boring logs at the appropriate sample depths.

Attached boring and test 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 and test pit 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-2487-10)

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
		ORGANIC	LL (oven dried)/LL (not dried)<0.75	OL	ORGANIC CLAY OR SILT
	SILTS AND CLAYS LIQUID LIMIT>50	INORGANIC	PI PLOTS >"A" LINE	CH	FAT CLAY
			PI PLOTS <"A" LINE	MH	ELASTIC SILT
		ORGANIC	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

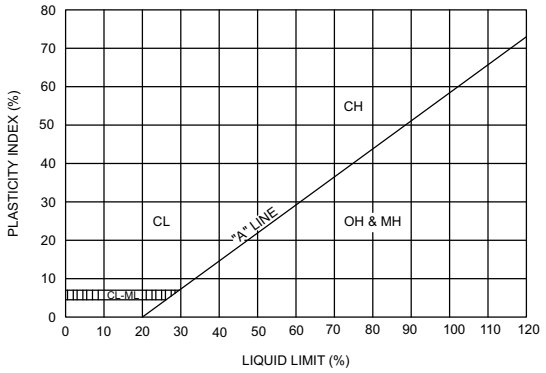
SAMPLER TYPES

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

CORNERSTONE EARTH GROUP

LEGEND TO SOIL DESCRIPTIONS

Figure Number A-1

HARDNESS

Soft – Reserved for plastic material alone.
Low hardness – Can be gouged deeply or carved easily with a knife blade.
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.
Hard – Can be scratched with difficulty: scratch produces little powder and is often faintly visible.
Very hard – Cannot be scratched with knife blade: leaves a metallic streak.

STRENGTH

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

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.
Moderate – Slight change or partial decomposition of minerals: little disintegration: cementation little to unaffected. Moderate to occasionally intense discoloration. Moderately coated fractures.
Little – No megascopic decomposition of minerals: little or no effect on normal cementation. Slight and intermittent, or localized discoloration. Few stains or fracture surfaces.
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

BORING NUMBER EB-1

CORNERSTONE EARTH GROUP

PROJECT NAME Crystal Springs Middle School
PROJECT NUMBER 471-1-1
PROJECT LOCATION Belmont, CA

DATE STARTED 7/22/11 **DATE COMPLETED** 7/22/11
GROUND ELEVATION **BORING DEPTH** 13.5 ft.
DRILLING CONTRACTOR Exploration Geoservices, Inc.
LATITUDE **LONGITUDE**
DRILLING METHOD Mobile B-53, 8 inch Hollow-Stem Auger
GROUND WATER LEVELS:
LOGGED BY CSH
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
	0		3 inches asphalt concrete over 5½ inches aggregate base						
			Clayey Sand (SC) [Fill] loose, moist, yellowish brown, fine to mediium sand, some fine to coarse subangular to angular gravel	50 5"	MC-1		5		
			Sandstone [fs] low hardness, moderately strong, moderate weathering, brown	50 6"	SPT				
	5		weak	41	SPT				
			moderately hard	50 6"	SPT				
	10		weak	44	SPT				
			Bottom of Boring at 13.5 feet.						
	15								
	20								

UNDRAINED SHEAR STRENGTH, ksf
○ HAND PENETROMETER
△ TORVANE
● UNCONFINED COMPRESSION
▲ UNCONSOLIDATED-UNDRAINED TRIAXIAL
1.0 2.0 3.0 4.0

CORNERSTONE EARTH GROUP2 - CORNERSTONE.GDT - 8/31/11 14:03 - P:\DRAFTING\GINT FILES\471-1-1 CRYSTAL SPRINGS MS.GPJ

BORING NUMBER EB-2

CORNERSTONE EARTH GROUP

PROJECT NAME Crystal Springs Middle School
PROJECT NUMBER 471-1-1
PROJECT LOCATION Belmont, CA

DATE STARTED 7/22/11 **DATE COMPLETED** 7/22/11
GROUND ELEVATION **BORING DEPTH** 13.9 ft.
DRILLING CONTRACTOR Exploration Geoservices, Inc.
LATITUDE **LONGITUDE**
DRILLING METHOD Mobile B-53, 8 inch Hollow-Stem Auger
GROUND WATER LEVELS:
LOGGED BY CSH
AT TIME OF DRILLING Not Encountered
NOTES
AT END OF DRILLING Not Encountered

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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)
	0		5 inches aggregate base							
			Clayey Sand with Gravel (SC) medium dense, moist, yellowish brown, fine to mediium sand, some fine to coarse subangular to angular gravel	23	MC-2	104	7			
				31	MC-4	110	10			
	5		**Sandstone [fs]** low hardness, weak, moderate to deep weathering, light yellowish brown	65	MC-6	119	10			
				50 / 5.5"	MC					
	10		moderately hard							
				50 / 5"	SPT					
			Bottom of Boring at 13.9 feet.							
	15									
	20									

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BORING NUMBER EB-3

CORNERSTONE EARTH GROUP

PROJECT NAME Crystal Springs Middle School
PROJECT NUMBER 471-1-1
PROJECT LOCATION Belmont, CA
DATE STARTED 7/22/11 **DATE COMPLETED** 7/22/11
GROUND ELEVATION **BORING DEPTH** 20 ft.
DRILLING CONTRACTOR Exploration Geoservices, Inc.
LATITUDE **LONGITUDE**
DRILLING METHOD Mobile B-53, 8 inch Hollow-Stem Auger
GROUND WATER LEVELS:
LOGGED BY CSH
AT TIME OF DRILLING Not Encountered
NOTES
AT END OF DRILLING Not Encountered

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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)
	0		**Clayey Sand with Gravel (SC) [Fill]** medium dense, moist, yellowish brown, fine to mediium sand, some fine to coarse subangular to sunbrounded gravel	29	MC-2	103	8			
				31	MC					
	5		**Sandstone [fs]** low hardness, weak, deep weathering, yellowish brownand gray, intensely fractured	56	MC-6	111	11			
				50 / 5.5"	MC					
	10									
				47	SPT					
	15									
				50 / 6"	SPT					
	20		Bottom of Boring at 20.0 feet.							

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BORING NUMBER EB-4

CORNERSTONE EARTH GROUP

PROJECT NAME Crystal Springs Middle School
PROJECT NUMBER 471-1-1
PROJECT LOCATION Belmont, CA

DATE STARTED 7/22/11 **DATE COMPLETED** 7/22/11
GROUND ELEVATION **BORING DEPTH** 14.5 ft.
DRILLING CONTRACTOR Exploration Geoservices, Inc.
LATITUDE **LONGITUDE**
DRILLING METHOD Mobile B-53, 8 inch Hollow-Stem Auger
GROUND WATER LEVELS:
LOGGED BY CSH
AT TIME OF DRILLING Not Encountered
NOTES
AT END OF DRILLING Not Encountered

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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
	0		3 inches asphalt concrete over 5 inches aggregate base							
			Sandy Lean Clay (CL) [Fill] hard, moist, brown, fine to mediium sand, some fine to coarse subangular to angular gravel, moderate plasticity Liquid Limit = 41, Plastic Limit = 20	36	MC-1	96	17	21		
			Sandstone [fs] moderately hard, weak, deep weathering, yellowish brown	50 / 5"	SPT-4	126	6			
	5			50 / 5"	SPT					
			hard, moderately strong, gray	50 / 3.5"	SPT					
	10									
			yellowish broan and gray mottled	50 / 5"	SPT					
	15		Bottom of Boring at 14.5 feet.							
	20									

CORNERSTONE EARTH GROUP2 - CORNERSTONE.GDT - 8/31/11 14:03 - P:\DRAFTING\GINT FILES\471-1-1 CRYSTAL SPRINGS MS.GPJ

BORING NUMBER EB-5

CORNERSTONE EARTH GROUP

PROJECT NAME Crystal Springs Middle School
PROJECT NUMBER 471-1-1
PROJECT LOCATION Belmont, CA

DATE STARTED 7/22/11 **DATE COMPLETED** 7/22/11
GROUND ELEVATION **BORING DEPTH** 11.5 ft.
DRILLING CONTRACTOR Exploration Geoservices, Inc.
LATITUDE **LONGITUDE**
DRILLING METHOD Mobile B-53, 8 inch Hollow-Stem Auger
GROUND WATER LEVELS:
LOGGED BY CSH
AT TIME OF DRILLING Not Encountered
NOTES
AT END OF DRILLING Not Encountered

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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
	0		3 inches asphalt concrete over 6 inches aggregate base						
			Clayey Sand with Gravel (SC) [Fill] loose, moist, brown to dark brown mottled, fine to mediium sand, some fine to coarse subangular to angular gravel	50 / 5.5"	MC-2	129	7		
			Sandstone [fs] moderately hard, moderately strong, moderate weathering, gray and reddish brown mottled, intensely fractured	50 / 5"	MC-4	112	14		
	5			50 / 4"	MC				
				50 / 4"	SPT				
	10		dark gray	50 / 6"	SPT				
			Bottom of Boring at 11.5 feet.						
	15								
	20								

UNDRAINED SHEAR STRENGTH, ksf
○ HAND PENETROMETER
△ TORVANE
● UNCONFINED COMPRESSION
▲ UNCONSOLIDATED-UNDRAINED TRIAXIAL
1.0 2.0 3.0 4.0

CORNERSTONE EARTH GROUP2 - CORNERSTONE.GDT - 8/31/11 14:03 - P:\DRAFTING\GINT FILES\471-1-1 CRYSTAL SPRINGS MS.GPJ

BORING NUMBER EB-6

CORNERSTONE EARTH GROUP

PROJECT NAME Crystal Springs Middle School
PROJECT NUMBER 471-1-1
PROJECT LOCATION Belmont, CA

DATE STARTED 7/22/11 **DATE COMPLETED** 7/22/11
GROUND ELEVATION **BORING DEPTH** 2 ft.
DRILLING CONTRACTOR Exploration Geoservices, Inc.
LATITUDE **LONGITUDE**
DRILLING METHOD Hand Auger
GROUND WATER LEVELS:
LOGGED BY CSH
AT TIME OF DRILLING Not Encountered
NOTES
AT END OF DRILLING Not Encountered

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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
	0		3 inches asphalt concrete over 3½ inches aggregate base							
			Sandstone [fs] moderately hard, weak, moderate to deep weathering, yellowish brown, intensely fractured	50 / 5"	MC-1	116	6			
			Bottom of Boring at 2.0 feet.							
	5									
	10									
	15									
	20									

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BORING NUMBER EB-7

CORNERSTONE EARTH GROUP

PROJECT NAME Crystal Springs Middle School
PROJECT NUMBER 471-1-1
PROJECT LOCATION Belmont, CA

DATE STARTED 7/22/11 **DATE COMPLETED** 7/22/11
GROUND ELEVATION **BORING DEPTH** 4.5 ft.
DRILLING CONTRACTOR Exploration Geoservices, Inc.
LATITUDE **LONGITUDE**
DRILLING METHOD Mobile B-53, 8 inch Hollow-Stem Auger
GROUND WATER LEVELS:
LOGGED BY CSH
AT TIME OF DRILLING Not Encountered
NOTES
AT END OF DRILLING Not Encountered

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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
	0		3 inches asphalt concrete over 3½ inches aggregate base							
			Clayey Sand with Gravel (SC) [Fill] dense, moist, reddish brown and gray mottled, fine to mediium sand, some fine to coarse subangular to angular gravel	50 / 5"	MC-1	114	4			
			Sandstone [fs] moderately hard, moderately strong, moderate weathering, reddish brown mottled, intensely fractured	50 / 2"	SPT					
				50 / 3"	SPT					
	5		Bottom of Boring at 4.5 feet.							
	10									
	15									
	20									

CORNERSTONE EARTH GROUP2 - CORNERSTONE.GDT - 8/31/11 14:03 - P:\DRAFTING\GINT FILES\471-1-1 CRYSTAL SPRINGS MS.GPJ

BORING NUMBER EB-8

CORNERSTONE EARTH GROUP

PROJECT NAME Crystal Springs Middle School
PROJECT NUMBER 471-1-1
PROJECT LOCATION Belmont, CA

DATE STARTED 7/22/11 **DATE COMPLETED** 7/22/11
GROUND ELEVATION **BORING DEPTH** 5 ft.
DRILLING CONTRACTOR Exploration Geoservices, Inc.
LATITUDE **LONGITUDE**
DRILLING METHOD Mobile B-53, 8 inch Hollow-Stem Auger
GROUND WATER LEVELS:
LOGGED BY CSH
AT TIME OF DRILLING Not Encountered
NOTES
AT END OF DRILLING Not Encountered

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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
	0		3 inches asphalt concrete over 5½ inches aggregate base							
			Clayey Sand with Gravel (SC) [Fill] medium dense, moist, dark reddish brown, fine to mediium sand, some fine to coarse subangular to angular gravel	51	MC-2	118	8			
			Clayey Sand with Gravel (SC) medium dense, moist, dark reddish brown, fine to mediium sand, some fine to coarse subangular to angular gravel	24	SPT					
	5		Bottom of Boring at 5.0 feet.							
	10									
	15									
	20									

CORNERSTONE EARTH GROUP2 - CORNERSTONE.GDT - 8/31/11 14:03 - P:\DRAFTING\GINT FILES\471-1-1 CRYSTAL SPRINGS MS.GPJ

BORING NUMBER EB-9

CORNERSTONE EARTH GROUP

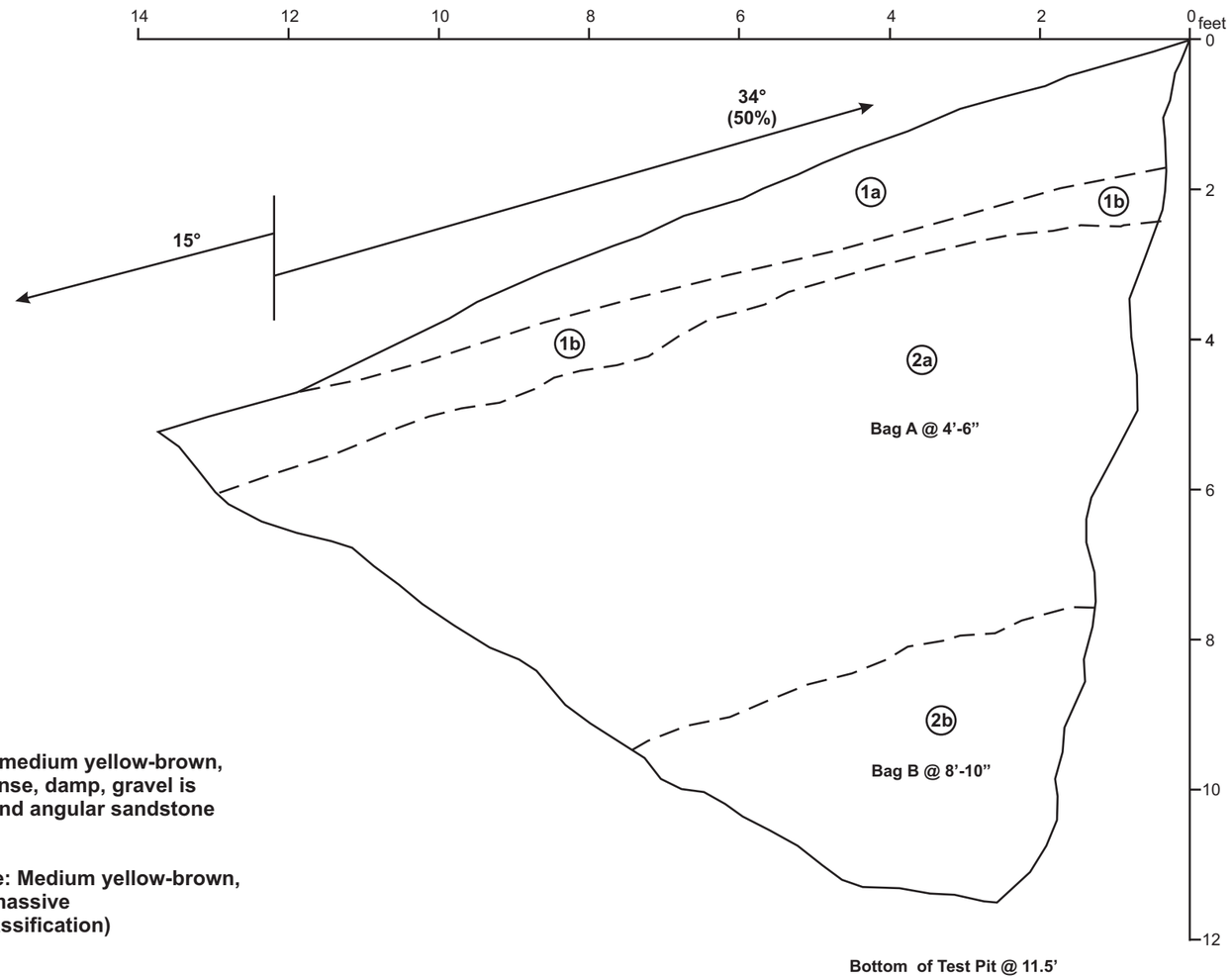
PROJECT NAME Crystal Springs Middle School
PROJECT NUMBER 471-1-1
PROJECT LOCATION Belmont, CA

DATE STARTED 7/22/11 **DATE COMPLETED** 7/22/11
GROUND ELEVATION **BORING DEPTH** 5 ft.
DRILLING CONTRACTOR Exploration Geoservices, Inc.
LATITUDE **LONGITUDE**
DRILLING METHOD Mobile B-53, 8 inch Hollow-Stem Auger
GROUND WATER LEVELS:
LOGGED BY CSH
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
	0		3 inches asphalt concrete over 5½ inches aggregate base							
			Clayey Sand with Gravel (SC) [Fill] dense, moist, dark reddish brown and gray, fine to mediium sand, some fine to coarse subangular to angular gravel	50 5"	MC-2	125	5			
			Clayey Sand with Gravel (SC) medium dense, moist, dark reddish brown and gray mottled, fine to mediium sand, some fine to coarse subangular to angular gravel	46	SPT					
	5		Bottom of Boring at 5.0 feet.							
	10									
	15									
	20									

CORNERSTONE EARTH GROUP2 - CORNERSTONE.GDT - 8/31/11 14:03 - P:\DRAFTING\GINT FILES\471-1-1 CRYSTAL SPRINGS MS.GPJ



1a Gravelly silty sand: medium yellow-brown, loose to medium dense, damp, gravel is medium to course and angular sandstone [Fill]

1b Bedrock: Sandstone: Medium yellow-brown, soft disintegrated, massive (Hard as a USCS classification) [A hard, silty sand]

2a Bedrock: Sandstone: Light yellow-brown, damp, medium hard, severely weathered,

2b Silty sandstone with gravel: Medium yellow-brown, damp, stiff, gravel is fine to medium sub-angular sandstone, [Colluvium]

TP-1

Scale: 1"=2'

N 10° E

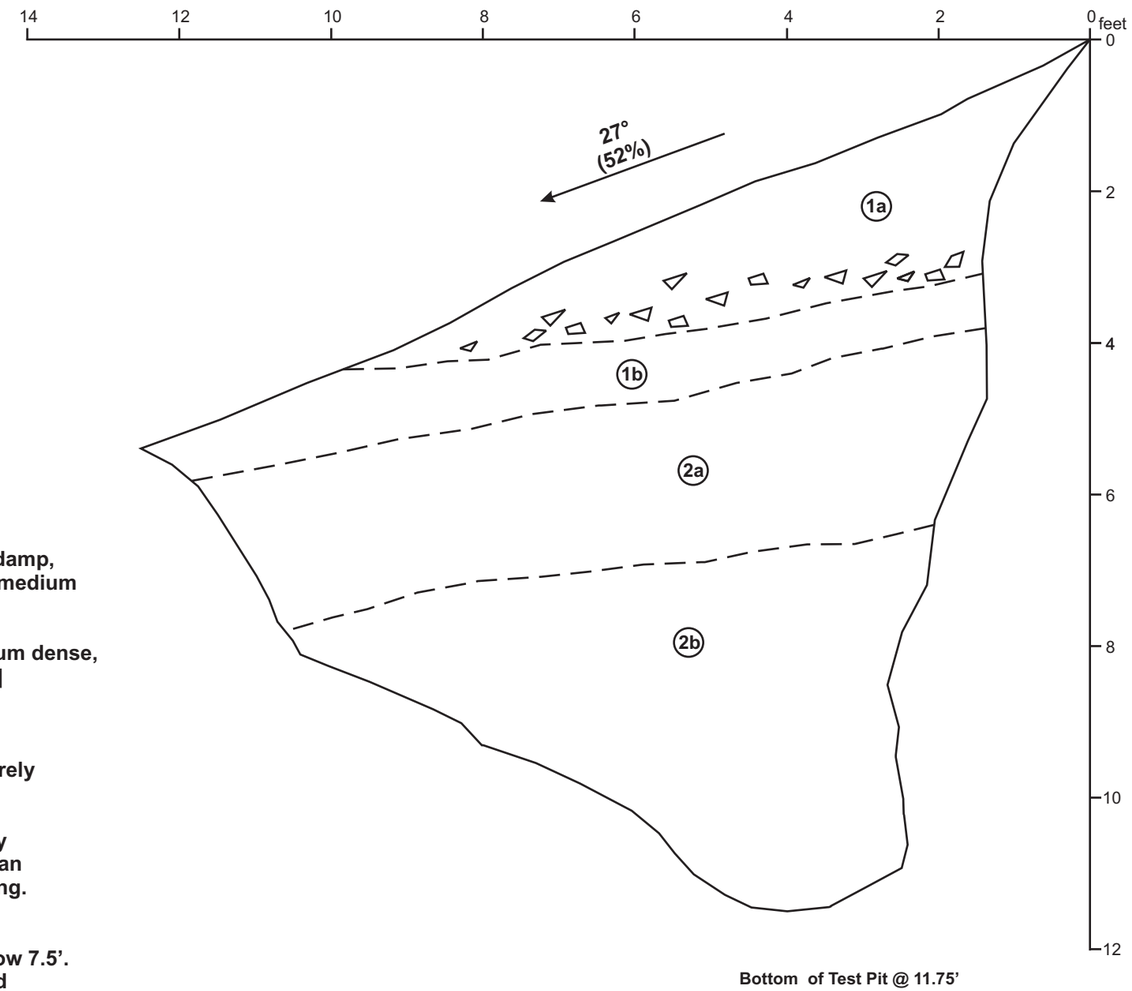
Test Pit Log: TP-1

Crystal Springs Middle School
Belmont, CA

CORNERSTONE EARTH GROUP

Figure Number
Figure A-1

Project Number
471-1-1
August 2011



1a Silty sand with gravel: Medium yellow-brown, damp, loose to medium dense, gravel is sub-angular medium to course sandstone [Fill]

1b Silty sand: Medium yellow-brown, damp, medium dense, fine gravel, sub-angular sandstone [Colluvium]

2a Sandstone: Medium yellow-brown, damp, severely weathered to disintegrated, fine gromed week

2b Sandstone: Light yellow-brown, damp, severely weathered, fine gromed week to mod strong than bedding, discontinuous frectres less than 3' long.

- Hard excavating below 7.5'.
- Becoming more massive and mod strong below 7.5'. Fractures are highly vorable in orientation and discontinuous.

TP-2

Scale: 1"=2'

N 2° E

Test Pit Log: TP-2

Crystal Springs Middle School
Belmont, CA

CORNERSTONE EARTH GROUP

Figure Number
Figure A-2

Project Number
471-1-1
August 2011

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.

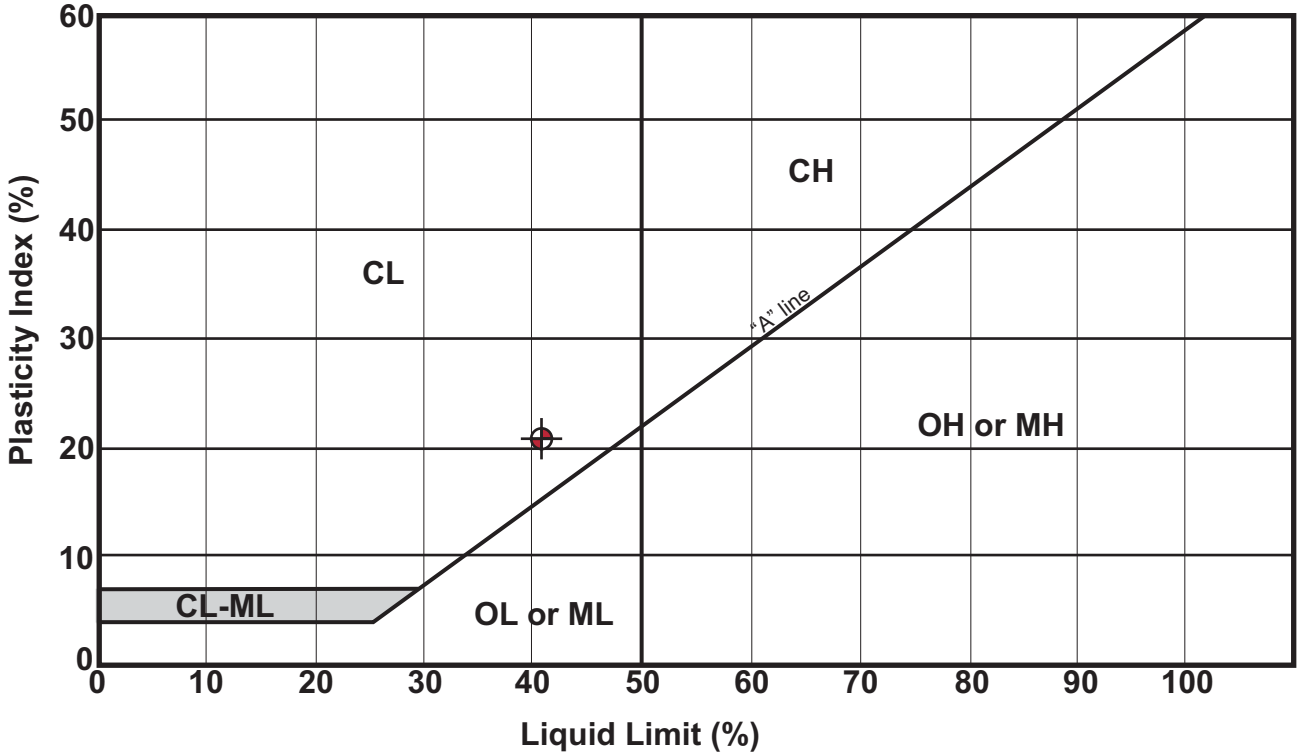
Moisture Content: The natural water content was determined (ASTM D2216) on 14 samples of the materials recovered from the borings. These water contents are recorded on the boring logs at the appropriate sample depths.

Dry Densities: In place dry density determinations (ASTM D2937) were performed on 13 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.

Plasticity Index: One Plasticity Index determination (ASTM D4318) was performed on a sample of the subsurface soil to measure the range of water contents over which this material exhibits 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 this test are shown on the boring log at the appropriate sample depth.

Corrosion Testing: Corrosion screening tests were performed on two samples, including Resistivity, Chloride, Sulfate and pH testing. Results of these tests are shown on the data sheet in this appendix.

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-4	1.5	17	41	20	21	—	Sandy Lean Clay (CL) [Fill]

CORNERSTONE EARTH GROUP

Plasticity Index Testing Summary

Crystal Springs Middle School
Belmont, CA

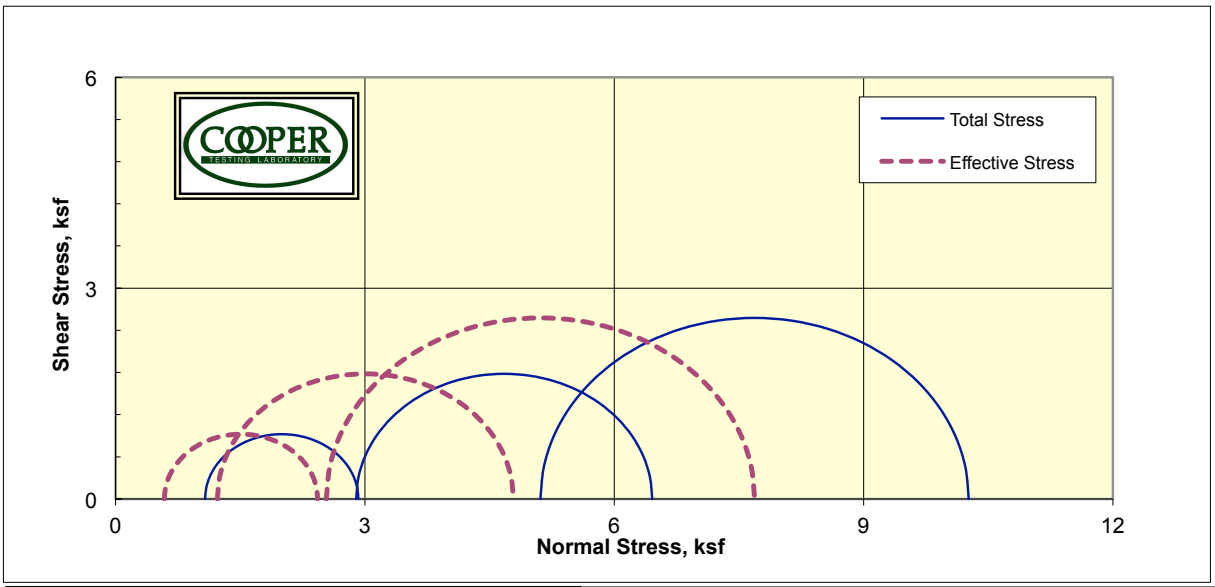
Project Number: 471-1-1

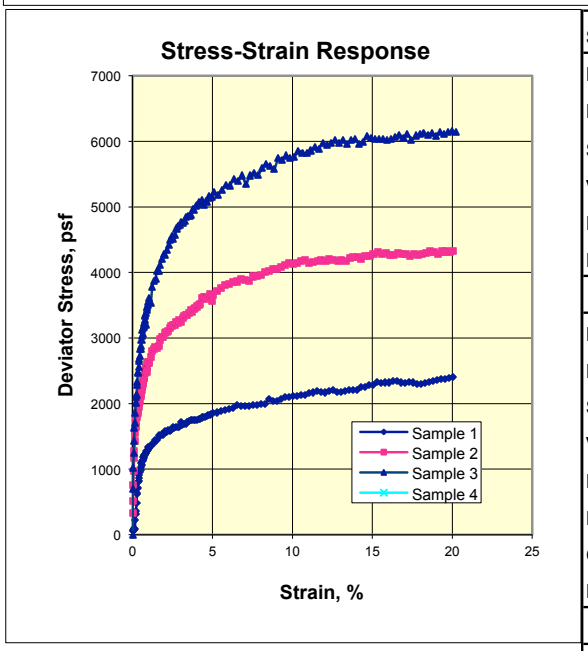
Figure Number: Figure B1

Date: September 2011

Drawn By: FLL

Triaxial Consolidated Undrained with Pore Pressure
ASTM D4767





Sample:	1	2	3	4
MC, %	10.1	10.1	10.1	
DD, pcf	112.1	112.5	112.5	
Sat. %	50.6	51.1	51.1	
Void Ratio	0.559	0.553	0.553	
Diameter in	2.37	2.37	2.37	
Height, in	5.08	5.07	5.08	
	Final			
MC, %	23.6	22.1	20.8	
DD, pcf	105.2	107.9	110.4	
Sat. %	100.0	100.0	100.0	
Void Ratio	0.660	0.620	0.583	
Diameter, in	2.45	2.42	2.40	
Height, in	5.07	5.06	5.03	
Cell, psi	106.9	120.8	134.7	
BP, psi	99.4	100.7	99.2	
	Effective Stresses At:			
Strain, %	5.0	5.0	5.0	
Deviator ksf	1.844	3.562	5.153	
Excess PP	0.490	1.670	2.576	
Sigma 1	2.431	4.786	7.689	
Sigma 3	0.587	1.225	2.536	
P, ksf	1.509	3.006	5.113	
Q, ksf	0.922	1.781	2.577	
Stress Ratio	4.144	3.909	3.032	
Rate in/min	0.0005	0.0005	0.0005	

Job No.: 640-331 **Date:** 9/12/11

Client: Cornerstone Earth Group BY:DC

Project: Crystal Springs Middle School - 471-1-1

Sample 1)	Bulk	Brown Clayey SAND (Weathered Rock)
Sample 2)	Bulk	Brown Clayey SAND (Weathered Rock)
Sample 3)	Bulk	Brown Clayey SAND (Weathered Rock)

Each specimen rebounded from the remolded height so the actual density is slightly lower than the target density. *Possible non linear strength envelope. A linear best fit may overstate the apparent cohesion. Engineering judgement is recommended.

Total C		**ksf**
Total phi		**degrees**
Eff. C		**ksf**
Eff. Phi		**degrees**

Corrosivity Test Summary

CTL # 640-327 **Date:** 8/31/11 **Tested By:** PJ **Checked:** PJ

Client: Cornerstone Earth Group **Project:** Crystal Springs Middle School **Proj. No:** 471-1-1

Remarks:

Sample Location or ID			Resistivity @ 15.5 °C (Ohm-cm)			Chloride	Sulfate-(water soluble)		pH	ORP	Sulfide	Moisture	Soil Visual Description
Boring	Sample, No.	Depth, ft.	As Rec.	Minimum	Saturated	mg/kg	mg/kg	%		(Redox)	Qualitative	%	
						Dry Wt.	Dry Wt.	Dry Wt.		mv	by Lead	At Test	
			ASTM G57	Cal 643	ASTM G57	Cal 422-mod.	Cal 417-mod.	Cal 417-mod.	ASTM G51	SM 2580B	Acetate Paper	ASTM D2216	
EB-2	3	3.5	-	-	2,898	<2	<5	<0.0005	6.4	-	-	11.5	Brown Sandy CLAY
EB-3	3	3.5	-	-	4,835	6	<5	<0.0005	6.5	-	-	8.9	Brown Sandy CLAY
EB-4	3	3.5	-	-	5,771	4	<5	<0.0005	6.5	-	-	5.8	Brown Clayey GRAVEL w/ Sand

APPENDIX C: SITE CORROSIVITY EVALUATION

September 20, 2011

Cornerstone Earth Group
1259 Oakmead Parkway
Sunnyvale, California 94085

Attention: **C. Barry Butler, P.E., G.E.**
Principal Engineer

Subject: **Site Corrosivity Evaluation**
Crystal Springs Middle School
Project No. 147-1-1

Dear Barry,

In accordance with your request, we have reviewed the laboratory soils data for the above referenced project site. Our evaluation of these results and our corresponding recommendations for corrosion control for the above referenced project foundations and buried site utilities are presented herein for your consideration.

Soil Testing & Analysis

Soil Chemical Analysis

Three (3) soil samples from the project site were chemically analyzed for corrosivity by ***Cooper Testing Laboratories***. Each sample was analyzed for chloride and sulfate concentration, pH, resistivity at 100% saturation and moisture percentage. The test results are presented in Cooper Testing Laboratories *Corrosivity Test Summary* dated 8/31/2011. The results of the chemical analysis were as follows:

Soil Laboratory Analysis

Chemical Analysis	Range of Results	Corrosion Classification*
Chlorides	<2 – 6 mg/kg	Non-corrosive*
Sulfates	<5 mg/kg	Non-corrosive**
pH	6.4 – 6.5	Mildly--corrosive*
Moisture (%)	5.8 – 11.5	Not-applicable
Resistivity at 100% Saturation	2,898 – 5,771 ohm-cm	**Moderately Corrosive ***

* With respect to bare steel or ductile iron.
** With respect to mortar coated steel

Discussion

Reinforced Concrete Foundations

Due to the low levels of water-soluble sulfates found in these soils, there is no special requirement for sulfate resistant concrete to be used at this site. The type of cement used should be in accordance with California Building Code (CBC) for soils which have less than 0.10 percent by weight of water soluble sulfate (SO_4) in soil and the minimum depth of cover for the reinforcing steel should be as specified in CBC as well.

Underground Metallic Pipelines

The soils at the project site are generally considered to be "moderately corrosive" to ductile/cast iron, steel and dielectric coated steel based on the saturated resistivity measurements. Therefore, special requirements for corrosion control are required for buried metallic utilities at this site depending upon the critical nature of the piping. Pressure piping systems such as domestic and fire water should be provided with appropriate coating systems and cathodic protection, where warranted. In addition, all underground pipelines should be electrically isolated from above grade structures, reinforced concrete structures and copper lines in order to avoid potential galvanic corrosion problems.

LIMITATIONS

The conclusions and recommendations contained in this report are based on the information and assumptions referenced herein. All services provided herein were performed by persons who are experienced and skilled in providing these types of services and in accordance with the standards of workmanship in this profession. No other warrantees or guarantees, expressed or implied, is provided.

We thank you for the opportunity to be of service to ***Cornerstone Earth Group*** on this project and trust that you find the enclosed information satisfactory. If you have any questions, or if we can be of any additional assistance, please feel free to contact us at (925) 927-6630.

Respectfully submitted,

J. Darby Howard, Jr.

J. Darby Howard, Jr., P.E.
JDH Corrosion Consultants, Inc.
Principal

Brendon Hurley

Brendon Hurley
JDH Corrosion Consultants, Inc.
Field Technician

File: 11117

APPENDIX D: SLOPE STABILITY RESULTS

Crystal Springs School - Static Belmont, Californina

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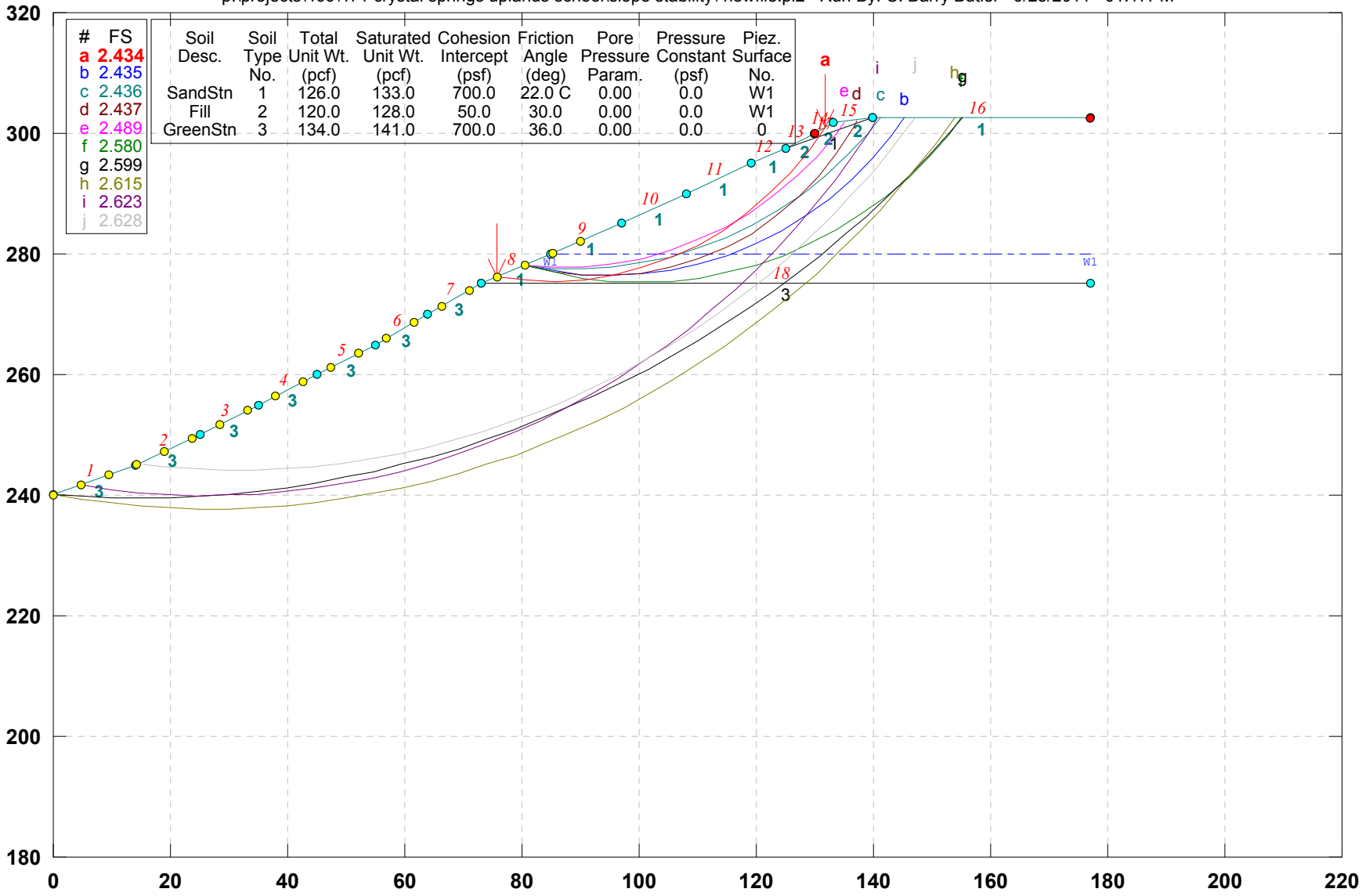

GSTABL7 v.2 FSmin=2.434

Safety Factors Are Calculated By The Modified Bishop Method

GSTABL7

Crystal Springs School - Seismic Belmont, Californina

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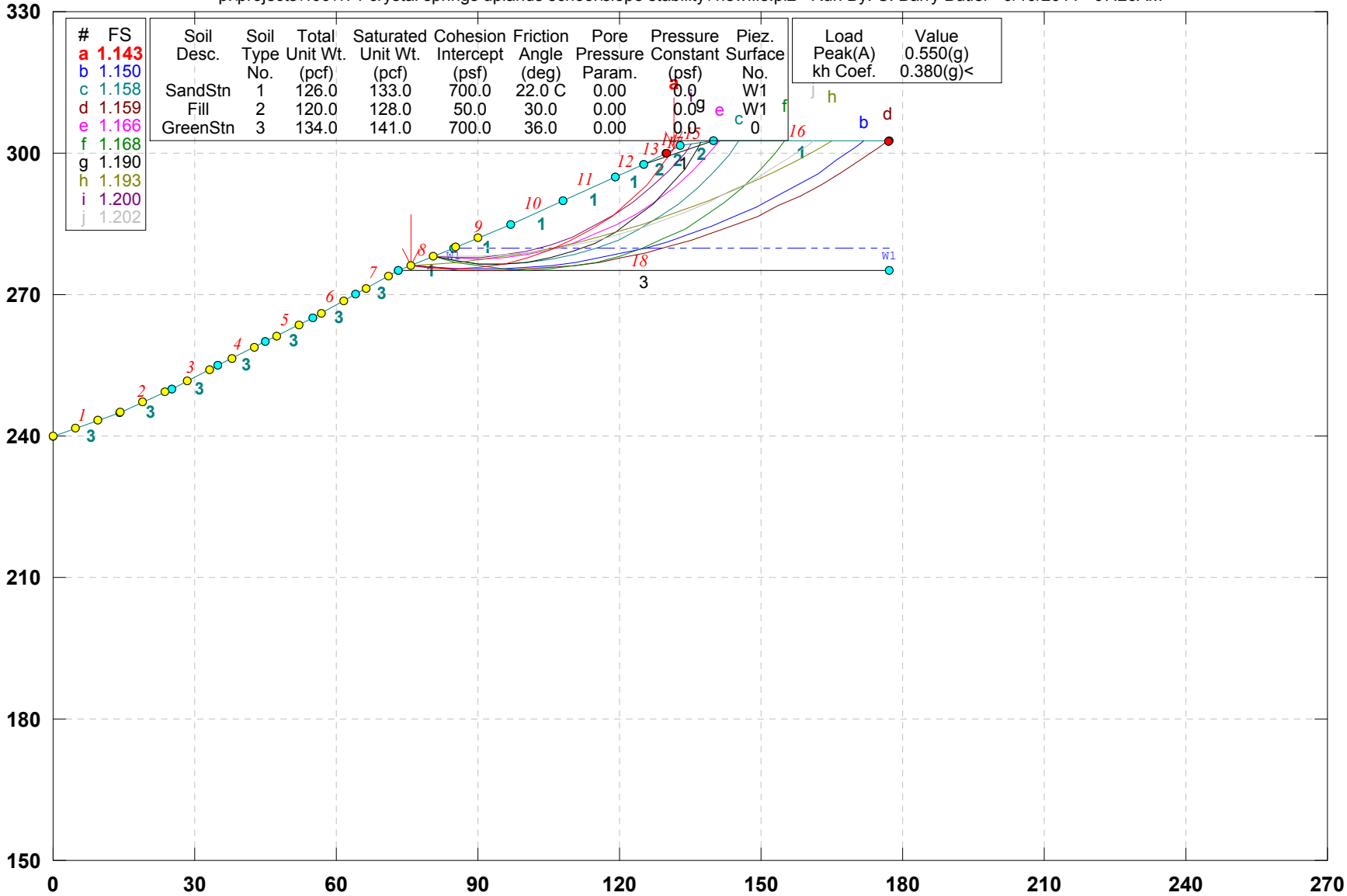

GSTABL7 v.2 FSmin=1.143

Safety Factors Are Calculated By The Modified Bishop Method

GSTABL7

COTTON, SHIRES AND ASSOCIATES, INC.
CONSULTING ENGINEERS AND GEOLOGISTS

November 14, 2011
B5071

TO: Damon DiDonato
Senior Planner
CITY OF BELMONT
1070 Sixth Avenue, #200
Belmont, California 94002

SUBJECT: **Geotechnical Peer Review**
RE: Crystal Springs School
6, 8, and 10 Davis Drive

At your request, we have completed a geotechnical peer review of the application for the proposed site development using:

- Geotechnical and Geologic Investigation, Crystal Springs Uplands Middle School (report) prepared by Cornerstone Group, dated September 21, 2011; and

- Conceptual Plans (19 sheets, various scales) prepared by Boora, dated September 2011.

In addition, we have reviewed pertinent technical documents from our office files, reviewed air photos, and completed a recent site reconnaissance.

DISCUSSION

Based on our review of the referenced documents, we understand that the applicant proposes to construct a new middle school campus on an existing building pad that currently contains two large buildings. Relatively minor additional grading will be necessary to develop most of the site; however, more significant grading may be needed for the proposed amphitheater and the top of the adjacent slope. Access to property will be via Davis Drive at the east end of the subject property.

SITE CONDITIONS

The subject site is characterized by a flat to gentle cut and fill pad flanked by steep to very steep (37 to 67 percent inclination) natural slopes on the south and southwest sides of the property. The upper portions of these natural slopes are capped with fill. On the western side of the building pad, a very steep (51 to 70 percent inclination) cut slope exposes greenstone of the Franciscan Complex. At the top of the cut slope, a steep to very steep (51 percent inclination) fill slope is exposed near the property boundary. Near the southwest corner of the property, an existing fill prism appears to overlie a colluvium-covered slope in the vicinity of the proposed amphitheater.

The slope at south side of the property has several colluvium-filled swales. These swales are primarily located on the adjacent open space property to the south.

They appear to be potential source areas for debris flows that would flow away from the subject property.

Drainage at the site is generally characterized by sheetflow toward various existing drop inlets on the building pad. On the slope at the south side of the property, flow is directed toward the downslope open space area.

The subject property is underlain, at depth, by sandstone and greenstone bedrock of the Franciscan Complex, as shown on published geologic maps and confirmed in borings by the Project Geotechnical Consultant. Greenstone of the Franciscan Complex is exposed locally in the cut slope on the west side of the property. The active San Andreas fault is located approximately 1.9 miles (3.1 km) to the southwest of the property.

CONCLUSIONS AND RECOMMENDED ACTION

The proposed structures on the existing building pad are constrained by the proximity of a very steep slope, undocumented old fill at the top of the slope, potential soil creep, and anticipated very strong seismic ground shaking. In the area of the existing building pad, the referenced report addresses these concerns by providing recommendations for setback from the slope as well as removal of existing fill and replacement as engineered fill.

The area of the proposed amphitheater is constrained by potentially unstable colluvium and undocumented fill. The proposed amphitheater site was not specifically addressed in the referenced report, and it appears likely that the consultant was not aware of the proposed amphitheater structure at the time that the report was prepared. The consultant should review the project development plans and provide supplemental recommendations to address the stability of the proposed amphitheater site. Additional subsurface exploration may be necessary to evaluate the thickness of existing fill and colluvium in this area.

In the referenced report, the consultant provides recommendations for compaction of on-site soils that range from 87% to 92% relative compaction. Typically, the standard of practice in the City calls for at least 90% relative compaction. If the consultant believes that 90% relative compaction cannot be achieved with the on-site soils, then suitable import fill should be considered. In addition, the consultant should provide recommendations for keying and benching of new fill material into bedrock.

Detailed construction/civil drawings should be prepared for the project. Prior to submittal to the City, the geotechnical consultant should review the detailed project drawings to confirm that their recommendations have been properly incorporated into the project design. Specifically, the consultant should confirm that their recommendations for a set-back from the top of slope and removal and replacement of potentially weak fill and colluvium have been incorporated in the plans.

We recommend that the Project Geotechnical Consultant and Project Civil Engineering Consultant address the following items prior to geotechnical approval of the pending applications:

1. **Supplemental Geotechnical Evaluations** – The Project Geotechnical Consultant should evaluate the following items and provide necessary additional information:

   - The consultant should re-evaluate their recommendations for compaction of on-site fill. If the consultant concludes that 90% relative compaction cannot be achieved or is unsuitable for on-site soils, then appropriate import material should be considered.

   - The consultant should provide recommendations for keying and benching of new fill material into bedrock.

   - The Consultant should review plans for the proposed amphitheater structure and characterize the geotechnical conditions at the amphitheater site. Additional subsurface investigation should be performed, as necessary, to evaluate the thickness of fill and colluvium in the area of the proposed amphitheater. Geotechnical design criteria should be formulated for the amphitheater grading, drainage, and foundations.

   The results of the supplemental geotechnical evaluations should be summarized in a report with appropriate illustrations to be submitted to the City for review by the City Geotechnical Consultant prior to project geotechnical approval.

2. **Project Construction Plans and Geotechnical Plan Review** – After completion of the supplemental geotechnical evaluations, construction drawings should be prepared for the project. A licensed Civil Engineer should prepare a grading and drainage plan for the project. Prior to submittal to the City, the Project Geotechnical Consultant should review and approve all geotechnical aspects of the project building and grading plans (i.e., site preparation and grading, building set-backs, site drainage improvements and design parameters for foundations, retaining walls and driveway) to ensure that their recommendations have been properly incorporated.

   Appropriate construction/civil plans should be submitted to the City, for review by the City Geotechnical Consultant prior to issuance of building permits. The results of the geotechnical plan review should be summarized by the geotechnical consultant in a letter and submitted to the City for review prior to issuance of building permits.

3. **Geotechnical Field Inspection** - The geotechnical consultant should inspect, test (as needed), and approve all geotechnical aspects of the project construction. The inspections should include, but not necessarily be limited to: site preparation and grading, site surface and subsurface drainage improvements, and excavations for foundations and retaining walls prior to the placement of steel and concrete.

The results of these inspections and the as-built conditions of the project should be described by the geotechnical consultant in a letter and submitted to the City Engineer for review prior to final (granting of occupancy) project approval.

LIMITATIONS

This geotechnical peer review has been performed to provide technical advice to assist the City in its discretionary permit decisions. Our services have been limited to review of the documents previously identified, review of aerial photographs, and a visual review of the property. Our opinions and conclusions are made in accordance with generally accepted principles and practices of the geotechnical profession. This warranty is in lieu of all other warranties, either expressed or implied.

Respectfully submitted,

COTTON, SHIRES AND ASSOCIATES, INC.
CITY GEOTECHNICAL CONSULTANT

Ted Sayre
Principal Engineering Geologist
CEG 1795

David T. Schrier
Principal Geotechnical Engineer
GE 2334

TS:DTS:PJ:st

CORNERSTONE
EARTH GROUP

Date:	January 4, 2012
Project No.:	471-1-1
Prepared For:	Mr. Laird McCulloch **CRYSTAL SPRINGS UPLANDS SCHOOL** 400 Uplands Drive Hillsborough, California 94010
Re:	Geotechnical Response to Supplemental Geotechnical Peer Review Comments Crystal Springs Uplands Middle School Davis Drive Belmont, California

Dear Mr. McCulloch:

As requested, we reviewed the supplemental geotechnical comments received from the City of Belmont in a letter titled, "Geotechnical Peer Review, Crystal Springs School, 6, 8 and 10 Davis Drive," prepared by Cotton, Shires and Associates, Inc., dated December 20, 2011. As you know, we previously performed a geotechnical investigation for the project and presented our findings in our report titled, "Geotechnical and Geologic Investigation, Crystal Springs Uplands Middle School, Davis Drive, Belmont, California," dated September 21, 2011. We also prepared a previous response to comments letter dated December 13, 2011. In this letter we address the comment received in the above referenced letter.

Response to Comments

As requested, we have reviewed the comment received from the City of Belmont, and have prepared a response to the comment. For ease of review, we have italicized the pertinent comments from the letter below, and provide our response below each comment.

Comment: *In the referenced letter, the Consultant contends that supplemental recommendations for keying and benching of fill into bedrock are unnecessary, because the amphitheater is no longer included in the project, improvements will be setback from the slopes, and the existing building pad is level. In response, we wish to point out that much of the existing fill is found at the top of the slope on the southern side of the building pad. As shown in Cross Section A-A' (Figure 3) from the consultant's geotechnical report (dated September 21, 2011), the existing fill forms a fill slope at the outboard edge of the building pad. Given that this slope is clearly steeper than 6:1 and that removal and replacement of the existing fill is planned as part of project grading, recommendations for keying and benching of fill into bedrock appear to be necessary for the project. In addition, northwest of Cross Section A-A', a somewhat higher fill slope would be*

anticipated where the existing fill appears to extend farther down the slope and may have been placed over potentially unstable colluvium. Whether improvements are planned for the outboard edge of the building pad or not, removal of potentially unstable old fill and replacement with engineered fill that is keyed and benched into stable bedrock is typically performed on grading projects within the City.

Response: Removal of all existing fills is not planned as part of the project, only in areas of planned improvements, which only occur on the level portions of the site. A small fill wedge, idealized in our Geologic Cross Section A-A', is located along the southern edge of the level portion of the site. Based on our two test pits cut through the fill material, and logging by our Certified Engineering Geologist, the fill wedge is only about 2 to 3 feet in thickness. The fill does broaden somewhat towards the west parking area, but also appears to be only about 3 feet thick in our Boring EB-4 performed in that area. Our borings indicate this material is medium dense to dense and/or hard. In addition, the fill has been in place for likely 30 years or more, and we observed no signs of creep or instability of these materials during our site reconnaissance or our aerial photo review. In addition, in part to mitigate for any potential instability at the crest of the slope, we established an engineering setback as mitigation for the structures and significant improvements.

No earthwork on or immediately adjacent to the slopes is planned as cutting keyways for 2 to 3 feet of fill would be impractical, would require significant amounts of earthwork, would be detrimental to the existing mature trees/roots and vegetation along the area, and would not provide any additional mitigation for the planned development.

Fill removal and replacement is planned in areas of improvements on the flat portion of the site, if encountered, and the planned structure foundations will extend to bedrock. In our opinion, these mitigations combined with the recommended engineering setbacks, should provide sufficient mitigation for the planned school development. Because the fill is performing well, relatively thin, and will have no affect on significant improvements, we see no reason to disturb the slope area.

Closure

If the development plans change, or if unforeseen conditions are encountered during construction that might be detrimental to the performance of improvements at the site, we will provide supplemental recommendations at that time.

We hope that our responses adequately address the City of Belmont's comments at this time. Our response has been prepared for the sole use of Crystal Springs Uplands School in accordance with generally accepted geotechnical engineering principles and practices in the San Francisco Bay Area at this time. No warranties are either expressed or implied.

CORNERSTONE
EARTH GROUP

Should you have any questions, or if we may be of further service, please contact us at your convenience.

Sincerely,

Cornerstone Earth Group, Inc.

C. Barry Butler, P.E., G.E.
Principal Engineer

CBB:SEF

Copies: Addressee (1 by email)
Luce, Forward, Hamilton & Scripps LLP (1 by email)
Attn: Ms. Jennifer Renk

COTTON, SHIRES AND ASSOCIATES, INC.
CONSULTING ENGINEERS AND GEOLOGISTS

January 12, 2012
B5071B

TO: Damon DiDonato
Senior Planner
CITY OF BELMONT
1070 Sixth Avenue, #200
Belmont, California 94002

SUBJECT: **Second Supplemental Geotechnical Peer Review**
RE: Crystal Springs School
6, 8, and 10 Davis Drive

At your request, we have completed a second supplemental geotechnical peer review of the application for the proposed site development using:

- Geotechnical Response to Geotechnical Peer Review Comments, Crystal Springs Uplands Middle School (letter) prepared by Cornerstone Group, dated January 4, 2012.

In addition, we have reviewed pertinent technical documents from our office files.

DISCUSSION

In our previous geotechnical peer review letter, we requested that the Project Geotechnical Consultant provide recommendations for keying and benching of fill into bedrock. We further requested that construction drawings, including a grading and drainage plan, be prepared for the project. In addition, we requested that the consultant review all geotechnical aspects of the project building and grading plans and verify that their recommendations have been incorporated into the construction plans. The referenced letter was prepared in response to our previous peer review comments.

CONCLUSIONS AND RECOMMENDED ACTION

In the referenced response letter, the Project Geotechnical Consultant indicates that removal of all existing fill is not planned for the project. Apparently, the consultant intends to remove existing fill only in the level portion of the site where improvements are planned, and those improvements will be set back from the top of the adjacent descending slope. The consultant further indicates that they "observed no signs of creep or instability" of the existing fill. During our site reconnaissance, we did observe local linear ground cracks at the outboard edge of the pavement at the top of the descending slope. Based upon our field observations, we conclude that some, local creep of the existing fill has occurred on the descending slope, and this creep has caused local distress at the outboard edge of the existing pavement at the top of the descending slope. Consequently, if new pavement is planned near the top of the descending slope, the applicant should anticipate and be willing to accept local pavement distress. Alternatively, consideration should be given to removal of existing fill and replacement with engineered fill that is keyed and benched into stable bedrock.

With the above understanding, we recommend geotechnical approval of the project permit application with the following conditions:

1. **Project Construction Plans and Geotechnical Plan Review** – Construction drawings should be prepared for the project. A licensed Civil Engineer should also prepare a grading and drainage plan for the project. Prior to submittal to the City, the Project Geotechnical Consultant should review and approve all geotechnical aspects of the project building and grading plans (i.e., site preparation and grading, building set-backs, site drainage improvements and design parameters for foundations, retaining walls and driveways) to ensure that their recommendations have been properly incorporated.

   Appropriate construction/civil plans should be submitted to the City. The results of the geotechnical plan review should be summarized by the geotechnical consultant in a letter and submitted to the City for review prior to issuance of building permits.

2. **Geotechnical Field Inspection** - The geotechnical consultant should inspect, test (as needed), and approve all geotechnical aspects of the project construction. The inspections should include, but not necessarily be limited to: site preparation and grading, site surface and subsurface drainage improvements, and excavations for foundations and retaining walls prior to the placement of steel and concrete.

   The results of these inspections and the as-built conditions of the project should be described by the geotechnical consultant in a letter and submitted to the City Engineer for review prior to final (granting of occupancy) project approval.

LIMITATIONS

This geotechnical peer review has been performed to provide technical advice to assist the City in its discretionary permit decisions. Our services have been limited to review of the documents previously identified, review of aerial photographs, and a visual review of the property. Our opinions and conclusions are made in accordance with generally accepted principles and practices of the geotechnical profession. This warranty is in lieu of all other warranties, either expressed or implied.

Respectfully submitted,

COTTON, SHIRES AND ASSOCIATES, INC.
CITY GEOTECHNICAL CONSULTANT

Ted Sayre
Principal Engineering Geologist
CEG 1795

David T. Schrier
Principal Geotechnical Engineer
GE 2334

TS:DTS:PJ:st

COTTON, SHIRES AND ASSOCIATES, INC.
CONSULTING ENGINEERS AND GEOLOGISTS

February 5, 2014
B5071C

TO: Damon DiDonato
Senior Planner
CITY OF BELMONT
1070 Sixth Avenue, #200
Belmont, California 94002

SUBJECT: **Geotechnical Peer Review**
RE: Crystal Springs School
6, 8, and 10 Davis Drive

At your request, we have completed a geotechnical peer review of the application for the proposed site development using:

- Geotechnical and Geologic Investigation, Crystal Springs Uplands Middle School (report) prepared by Cornerstone Earth Group, dated September 21, 2011;
- Architectural Plans (13 sheets, various scales) prepared by Boora, dated January 2012;
- Landscape Plans (8 sheets, various scales) prepared by Andrea Cochran Landscape Architecture, dated January 10, 2014;
- Civil Plans (4 sheets, various scales) prepared by AMS Associates, dated October 2011;
- Topographic Map (1 sheet, 20 scale) prepared by B&H Surveying, dated February 2011;
- Revised Project Description for Updated Mitigated Negative Declaration, Crystal Springs School (written description) prepared by unknown author, undated.

In addition, we have reviewed pertinent technical documents from our office files.

DISCUSSION

The proposed project consists of demolishing two existing structures and constructing three new school buildings and associated improvements at a site located north of Water Dog Lake. A similar project was originally submitted to the City in 2011, and the current project design appears to mirror the previous submittal. We completed geotechnical peer review of the previous submittal in 2012. In our most recent peer review letter, we recommended geotechnical approval of the project permit application with the understanding that if new pavement areas were planned near the top of the descending slope, the applicant should anticipate and be willing to accept local

pavement distress. We also noted that consideration should be given to removal of existing fill and replacement with engineered fill that is keyed and benched into stable bedrock at the top of the descending slope. At that time, we also recommended that construction plans be prepared for the project. With the recent re-submittal, the applicant has provided construction plans.

CONCLUSIONS AND RECOMMENDED ACTION

The re-submitted project appears to be substantially similar, from a geotechnical perspective, to the project that we peer reviewed during 2011 to 2012. We recommend that the Project Geotechnical Consultant review the referenced plans and project description and confirm that the project is substantially unchanged and that their original recommendations still apply. The Consultant should confirm that their recommendations conform to recent code updates (particularly the 2013 CBC seismic code) and provide supplemental recommendations, as necessary, to conform to applicable building code updates.

We recommend geotechnical approval of the project permit application with the following conditions:

1. **Geotechnical Plan Review and Updated Recommendations** – The Project Geotechnical Consultant should review and approve all geotechnical aspects of the project building and grading plans (i.e., site preparation and grading, building set-backs, site drainage improvements and design parameters for foundations, retaining walls and driveways) to ensure that their recommendations have been properly incorporated.

   The Consultant should confirm that the current project is substantially unchanged from the configuration that they originally investigated and that their original recommendations still apply. The Consultant should make supplemental geotechnical design recommendations, as needed, to comply with all applicable building code updates, particularly the 2013 CBC.

   The results of the geotechnical plan review and updated geotechnical design recommendations should be summarized by the Project Geotechnical Consultant in a letter and submitted to the City for review prior to issuance of building permits.

2. **Geotechnical Construction Inspections** – The Project Geotechnical Consultant should inspect, test (as needed), and approve all geotechnical aspects of project construction. The inspections should include, but not necessarily be limited to: site preparation and grading, site surface and subsurface drainage improvements, and excavations for foundations and retaining walls prior to the placement of steel and concrete.

   The results of these inspections and the as-built conditions of the project should be described by the geotechnical consultant in a letter and submitted to the City Engineer for review prior to final (granting of occupancy) project approval.

LIMITATIONS

This geotechnical peer review has been performed to provide technical advice to assist the City in its discretionary permit decisions. Our services have been limited to review of the documents previously identified and a visual review of the property. Our opinions and conclusions are made in accordance with generally accepted principles and practices of the geotechnical profession. This warranty is in lieu of all other warranties, either expressed or implied.

Respectfully submitted,

COTTON, SHIRES AND ASSOCIATES, INC.
CITY GEOTECHNICAL CONSULTANT

Ted Sayre
Principal Engineering Geologist
CEG 1795

David T. Schrier
Principal Geotechnical Engineer
GE 2334

TS:DTS:PJ:st

CORNERSTONE
EARTH GROUP

Date:	June 2, 2014
Project No.:	471-1-1
Prepared For:	Mr. Laird McCulloch **CRYSTAL SPRINGS UPLANDS SCHOOL** 400 Uplands Drive Hillsborough, CA 94010
Re:	Geotechnical Review and Update Crystal Springs Uplands Middle School Davis Drive Belmont, California

Dear Mr. McCulloch:

At your request, this letter is to provide confirmation that our previous geotechnical report remains valid for design. As you know, Cornerstone Earth Group has previously issued a report titled "Geotechnical and Geologic Investigation, Crystal Springs Uplands Middle School, Davis Drive, Belmont, California," dated September 21, 2011.

GEOTECHNICAL REPORT UPDATE

We understand that the project will be resumed this year, and that the development plan has not changed from the previous plan. In addition, we performed a cursory site reconnaissance of the site, and found no significant changes in the site conditions. Therefore, based on our site reconnaissance, the current development plans, and review of our prior report, in our opinion, the recommendations in our September 21, 2011 report remain applicable and can be used for project design. Once design development does resume, we should be retained to update the Seismic Design Criteria in our prior report to 2013 CBC requirements, and to address any modifications to the development. In addition, to complete our role on the project as the Geotechnical Engineer-of-Record, we should be retained to perform the following services:

- Review the project grading and foundation plans for conformance with the recommendations in the geotechnical report.
- Perform geotechnical testing and observation services during construction

CLOSURE

Recommendations presented in this letter have been prepared for the sole use of Crystal Springs Uplands School, specifically for the retail store project in San Jose, California. Our professional opinions and recommendations are prepared in accordance with generally accepted geotechnical engineering principles and practices at this time and location.

No warranties are expressed or implied. If you have any questions or need any additional information from us, please call and we will be glad to discuss them with you.

Sincerely,

CORNERSTONE EARTH GROUP, INC.

C. Barry Butler, P.E., G.E.
Senior Principal Engineer

CBB

Copies: Addressee (by email)