TYPE OF SERVICES: Geotechnical Investigation and Geologic Hazards Evaluation

PROJECT NAME: Frank McCoppin Elementary School Improvements

LOCATION: 651 6th Avenue
San Francisco, California

CLIENT: San Francisco Unified School District

PROJECT NUMBER: 608-5-1

DATE: January 23, 2015
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<td>Location</td>
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<td>San Francisco, California</td>
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<tr>
<td>Client</td>
<td>San Francisco Unified School District</td>
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<tr>
<td>Client Address</td>
<td>135 Van Ness Avenue - Room 207a</td>
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<td>San Francisco, California</td>
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SECTION 1: INTRODUCTION

This geotechnical investigation and geologic hazards evaluation report was prepared for the sole use of San Francisco Unified School District for the Frank McCoppin Elementary School Improvements project located at 651 6th Avenue in San Francisco, California. The location of the site is shown on the Vicinity Map, Figure 1. The site is located at Latitude 37.776310°N and Longitude -122.464418°W.

For our use, we were provided the following documents:


- A document titled: “Information of Proposed Boring Sampling Location at Frank McCoppin Elementary School,” prepared by Kendall Young Associates, dated December 8, 2014, which described the proposed site improvements.


1.1 PROJECT DESCRIPTION

The site is currently occupied by one and two-story classroom buildings on the eastern half of the campus, a portable classroom building on the western half of the campus, and an asphalt concrete playground.
We understand that the improvements for this site will include: 1) converting the exposed atrium of the Administration/EES Building (southern classroom building) into an interior space, 2) new walkway canopies on the north and west sides of the Administration/EES Building, 3) an elevated egress balcony along the south side of the Academic Building (northern building), 4) a new 14-foot tall fence on the northwestern portion of the campus, and 5) installation of two new exterior partitions, up to 20 feet in height located at both of the stair cases along the east side of the property. New shallow footing and drilled pier foundations are anticipated. Appurtenant parking, utilities, landscaping and other improvements necessary for site development are also planned.

We anticipate this project will involve minor grading; however, cuts and fills on the order of 1 to 2 feet may be required. Cuts on the order of 3 to 5 feet may be required to expose existing footings or build new footings.

1.2 SCOPE OF SERVICES

Our scope of services was presented in our proposal dated December 11, 2014, and consisted of field and laboratory programs to evaluate physical and engineering properties of the subsurface soil, engineering analysis to prepare recommendations for site work and grading, 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

Our field exploration consisted of performing a geologic site reconnaissance and advancing four exploratory borings to depths of 15 to 50 feet. Borings (EB-1 through EB-4) were drilled on December 22, 2014, using truck-mounted hollow stem auger or portable “minute-man,” solid-flight drilling equipment. The borings were backfilled with cement grout in accordance with local requirements; exploration permits were obtained as required by local jurisdictions.

The approximate locations of our exploratory borings are shown on the Site Plan and Geologic Map, Figure 2. Details regarding our field program are included in Appendix A.

1.4 LABORATORY TESTING PROGRAM

In addition to visual classification of samples, the laboratory program focused on obtaining data for foundation design and seismic ground deformation estimates. Testing included moisture contents, dry densities, and one Plasticity Index. One suite of corrosion tests were performed a sample of the surficial soil, consisting of: saturated resistivity, pH, and soluble sulfates and chlorides. Details regarding our laboratory program are included in Appendix B.

1.5 ENVIRONMENTAL SERVICES

We understand that environmental services for the project are being provided by Millennium Consulting Associates. If environmental concerns are present, Millennium Consulting
Associates should review our geotechnical recommendations for compatibility with the environmental concerns.

SECTION 2: REGIONAL SETTING

2.1 GEOLOGIC SETTING

2.1.1 Regional Geologic Setting

The San Francisco Peninsula is a relatively narrow geographic land feature at the north end of the Santa Cruz Mountains. The peninsula has developed on a basement of tectonically mixed Cretaceous- and Jurassic- age (70 to 200 million years old) rocks of the Franciscan Complex. Uplift, erosion and subsequent re-deposition of sedimentary rocks within this province have been driven by the strike-slip movement of the tectonic plates and the associated northeast oriented compressional stress. The surficial geologic deposits in the northwestern portion of the City of San Francisco (see Figure 3), are mapped as unconsolidated late Pleistocene and Holocene deposits which are broken out as; Dune sand (Qd), marine terrace deposits (Qt), Slope Debris and ravine fill (Qsr), the Colma Formation (Qc) and undifferentiated sedimentary deposits. The Plio-Pleistocene Merced Formation underlies these surficial deposits (Bonilla). These units were deposited on the earlier, irregular topographic surface of Franciscan Complex rocks (Bonilla, 1964) or Pleistocene deposits. Artificial fill (af) is present locally within the general area. Underlying these relatively geologically young formations are rocks of the Franciscan Complex of Jurassic or Cretaceous age.

The tectonic regime in the San Francisco Bay region is primarily translational, expressed by mostly right-lateral strike-slip movement along the faults of the San Andreas Fault system, including the nearby Calaveras and Hayward Faults. A small component of compression is active in the region, resulting in continued folding and faulting of the geologic units.

2.2 REGIONAL SEISMICITY

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 that time period. During such an earthquake the danger of fault ground rupture at the sites is slight, but strong ground shaking would occur.

The San Francisco Bay area including the coastal region is recognized by geologists and seismologists as one of the most seismically active regions in the United States. 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 4.7 miles west of the campus. The San Gregorio Fault is located about 8.0 miles from the site.

The 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 State-considered active faults within 100 kilometers (62 miles) of the site. It is noted that the fault distances presented in Table 1 were determined from EZ Frisk and represent the rupture distance and may not be the distance to the surface expression of the fault that is shown on published geological maps and on-line resources such as Google Earth, etc. 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 100-Kilometers**

<table>
<thead>
<tr>
<th>Fault Name</th>
<th>Distance (miles)</th>
<th>Distance (kilometers)</th>
</tr>
</thead>
<tbody>
<tr>
<td>San Andreas</td>
<td>4.7</td>
<td>7.5</td>
</tr>
<tr>
<td>San Gregorio</td>
<td>8.0</td>
<td>12.9</td>
</tr>
<tr>
<td>Hayward – Rodgers Creek</td>
<td>13.4</td>
<td>21.5</td>
</tr>
<tr>
<td>Point Reyes</td>
<td>23.2</td>
<td>37.4</td>
</tr>
<tr>
<td>Mount Diablo</td>
<td>24.7</td>
<td>39.8</td>
</tr>
<tr>
<td>Calaveras</td>
<td>25.0</td>
<td>40.2</td>
</tr>
<tr>
<td>Monte Vista-Shannon</td>
<td>26.1</td>
<td>42.1</td>
</tr>
<tr>
<td>Green Valley</td>
<td>27.1</td>
<td>43.6</td>
</tr>
<tr>
<td>West Napa</td>
<td>29.3</td>
<td>47.2</td>
</tr>
<tr>
<td>Greenville</td>
<td>35.2</td>
<td>56.7</td>
</tr>
<tr>
<td>Great Valley 5, Pittsburg Kirby Hills</td>
<td>38.1</td>
<td>61.3</td>
</tr>
<tr>
<td>Great Valley 4b, Gordon Valley</td>
<td>38.6</td>
<td>62.2</td>
</tr>
<tr>
<td>Great Valley 7</td>
<td>48.1</td>
<td>77.4</td>
</tr>
<tr>
<td>Hunting Creek-Berryessa</td>
<td>48.9</td>
<td>78.8</td>
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<tr>
<td>Great Valley 4a, Trout Creek</td>
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<td>84.0</td>
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<td>Zayante-Vergeles</td>
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<td>87.8</td>
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<tr>
<td>Maacama-Garberville</td>
<td>56.6</td>
<td>91.1</td>
</tr>
<tr>
<td>Monterey Bay-Tularcitos</td>
<td>61.9</td>
<td>99.7</td>
</tr>
<tr>
<td>Great Valley 3, Mysterious Ridge</td>
<td>62.0</td>
<td>99.9</td>
</tr>
</tbody>
</table>

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

We reviewed and performed a data search of known historical earthquakes of magnitude 5 or greater within a 100-kilometer radius of the site using available published data from the California Division of Mines and Geology (CDMG) computerized earthquake catalog of events through December 1999. Figure 5 shows the epicenters of these magnitude 5 or greater events. We also included data from Townley and Allen (1939) and the U.S. Geological Survey Earthquake Data Base System, giving 200 years of data in the search area. The results of our computer search indicated that about 94 known earthquakes of Richter Magnitude 5 or greater have occurred within 100 kilometers of the site between 1800 and December 1999.

Four relatively large magnitude earthquakes have occurred in the region during the above noted time period, including the Loma Prieta Earthquake of 1989 that was centered approximately 40 miles southeast of the site. Figure 5, adopted from Toppozada & Others (2000) illustrates these historical earthquakes.

2.4 MAXIMUM PAST GROUND SHAKING

The 1906 earthquake on the San Andreas Fault was the highest magnitude earthquake recorded in California. Lawson (1908, p. 259) reported considerable damage from the 1906 earthquake in the area, although it should be noted that the western portion of the City of San Francisco where dune fields existed were sparsely to very sparsely developed. “The intensity of the earthquake in area was characterized as “strong” with many buildings partially wrecked and/or were moved upon their foundations, and a majority of the houses had the plaster badly cracked. Ninety-four percent of the chimneys fell, and dishes and similar objects were universally thrown down. Ground cracks and localized subsidence was noted near the edge of the bay (Lawson, 1908). Nason (1980) considers the damage here to be Rossi-Forel intensity VIII. The Loma Prieta earthquake of 1989 was smaller in magnitude (M 6.9) than the 1906 earthquake (M 8.25) but its epicenter was further away from the site than the 1906 quake.

SECTION 3: SITE CONDITIONS

3.1 RECENT HISTORY AND SURFACE CONDITIONS

Historic maps available through the David Rumsay Historical Map collection show the area of the site at various points in time. The 1905 map of the area indicates that the streets in the site area had already been established. Aerial photos were reviewed spanning a period from 1938 to 1973. As of 1938 the area of the school site very extensively developed with residential and commercial sites (see Figure 6). The school campus had a large rectangular building within the west portion and blacktop covering the remainder of the eastern portion of the site. A modular classroom building was later installed near the southwest corner of the campus at a later date. All of the streets within the area had already been established. The subsequent photos show the site and adjacent areas essentially unchanged.

The site encompasses an area that slopes gently to the north (3 to 5 degrees) within the Richmond District, just north of Golden Gate Park. The site is completely developed with two
large classroom buildings, and the remainder has asphalt backdrop and miscellaneous flatwork and minimal landscaping. The site was apparently developed by creating a cut and fill terrace between 6th Avenue and 7th Avenue. The cut was made within the southwest corner of the site and fill was placed across the remainder of the site in order to create a flat terrace. Due to the northerly slope of the adjacent streets on the west and east sides, the retaining walls gradually increase from south to north and achieve a maximum height of approximately 7.5 feet at the northeast property corner. Accordingly this is where the fill achieves its maximum thickness. The site characteristics and proposed improvements are shown on Figure 2.

The existing structures consist of two wood-frame buildings (classroom and Administration buildings) and one modular building. The Administration and modular buildings are one story and the classroom building is two stories in height. Based on the referenced plans, the existing concrete building is supported on shallow foundations consisting of perimeter strip and isolated column footings that were designed with a soil bearing pressure of 3,000 pounds per square foot (psf). The plans indicate that the footings extend at least 24 inches below existing grade.

### 3.2 SITE GEOLOGY AND SUBSURFACE CONDITIONS

The geologic map by Schlocker (1958, 1974) shows the area of the site is an area where Dune sand (“Qd”) overlies the older geologic units in the area. Dune sand covers the older geologic units throughout the northern half of San Francisco due to the prevailing westerly winds which have, over time swept the sand from Ocean Beach and Baker Beach over the city. The dune sand can be as much as 100 feet or more in thickness and in fact Schlocker and Bonilla (1961) shows the depth to Franciscan bedrock in the immediate area at an estimated or projected depth of 100 feet. The Dune sand is characterized as “Well sorted fine–grained sand, gray and loose in most places, grayish orange to reddish brown and firm in a few places” (Bonilla, 1998). The age of the unit extends into the Pleistocene. Knudsen et al., (2000) provide additional interpretation regarding the dune sand: “This unit includes active dunes along with recently stabilized dunes in coastal environments. Dune sand typically is very well sorted fine to medium sand. This unit is mapped in only a few places, typically near beaches, where Holocene age for much of the deposit is likely. Large latest Pleistocene dune fields like the Antioch-Oakley dunes, the Merritt Sand, and most of the dunes covering the northern San Francisco Peninsula, which are mapped as latest Pleistocene to Holocene dune sand (Qds), likely contain areas of unmapped Holocene dune sand. Typical soils developed on this unit (Qhds) are inceptisols.” Knudsen et al. (2000) describe the Dune sand as “Latest Pleistocene to Holocene Dune Sand which they characterize as very well sorted fine to medium grained eolian sand”. Although Schlocker and Bonilla (1961) show the Dune sand underlain by the Colma Formation within the Richmond District, the Colma is shown on the map by Schlocker (1974) as pinching out going from northeast to southwest within the Richmond District. We did not encounter evidence of the Colma Formation within our explorations. The Colma Formation likely underlies the Dune sand at a somewhat greater depth that the maximum depth achieved (50 feet) during our explorations.

Due to the extensive development at and adjacent to the site, subsurface geologic units are not exposed in the immediate area. Our exploration confirms the presence of accumulations of up to several feet of manmade fill overlying the Dune sand geologic unit (Qd) at the site. We have
adopted the nomenclature of Schlocker (1958, 1974) in assigning geologic unit names for our characterization of the site.

All of our explorations encountered undocumented fill overlying Dune sand at depth. The undocumented fill varied in thickness from 2½ feet to approximately 7½ feet thick. On our subsurface cross sections we have projected the assumed fill base between our explorations. The fill was generally loose to medium dense although locally dense as well and consisted of poorly graded sand with gravel or clayey sand with gravel. The location of the cut/fill contact was inferred from the topographic characteristics of the site, as well as the results of our subsurface exploration. The dune sand unit consisted of poorly graded fine to medium grained sand. The uncorrected field blow counts recorded during our subsurface exploration generally indicated that the Dune sand was loose to dense. Sample return of the fill and underlying Qd unit was approximately 80 to 90%.

The aerial distribution of earth materials at the site are mapped on Figure 2, Site Plan and Geologic Map. Our geologic cross sections A-A’ and B-B’ were generated from the site geologic map as well as some of the exploratory boring data (Figures 7 and 8).

3.3 GROUND WATER

Ground water was encountered was not encountered in our borings to a maximum depth of 50 feet. All measurements were taken at the time of drilling and may not represent the stabilized levels, which can be higher than the initial levels encountered.

Relatively low laboratory moisture contents indicate that the stabilized ground water level is greater than 50 feet, the maximum depth explored.

The Seismic Hazard Zone Report for San Francisco indicates that the historic high ground water depth is on the order of 50 feet at the site (CDMG, 2000). Fluctuations in ground water levels occur due to many factors including seasonal fluctuation, underground drainage patterns, regional fluctuations, and other factors.

3.4 CORROSION SCREENING

We tested one sample collected at a depth of 3½ feet for resistivity, pH, soluble sulfates, and chlorides. Laboratory test results are presented in Table 2.

<table>
<thead>
<tr>
<th>Boring</th>
<th>Depth (feet)</th>
<th>Soil pH¹</th>
<th>Resistivity² (ohm-cm)</th>
<th>Chloride³ (mg/kg)</th>
<th>Sulfate⁴,⁵ (mg/kg)</th>
</tr>
</thead>
<tbody>
<tr>
<td>EB-4</td>
<td>3½</td>
<td>8.3</td>
<td>41,864</td>
<td>&lt; 2</td>
<td>10</td>
</tr>
</tbody>
</table>

Notes: ¹ASTM G51  
²ASTM G57 - 100% saturation  
³ASTM D3427/Cal 422 Modified  
⁴ASTM D3427/Cal 417 Modified  
⁵1 mg/kg = 0.0001 % by dry weight
Many factors can affect the corrosion potential of soil including moisture content, resistivity, permeability, and pH, as well as chloride and sulfate concentration. Typically, soil resistivity, which is a measurement of how easily electrical current flows through a medium (soil and/or water), is the most influential factor. In addition to soil resistivity, chloride and sulfate ion concentrations, and pH also contribute in affecting corrosion potential.

Based on the laboratory test results summarized in Table 2, the soils are considered moderately very severely corrosive to buried metallic improvements (Palmer, 1989). Other corrosion parameters (pH and chloride content) do not indicate a significant contribution to corrosion potential to buried metallic structures. In accordance with the 2013 CBC, Chapter 19, Section 1904A.2:

> Concrete mixtures shall conform to the most restrictive maximum water-cementitious materials ratios, maximum cementitious admixtures, minimum air-entrained and minimum specified concrete compressive strength requirements of ACI 318 based on the exposure classes assigned in Section 1904A.1.

We recommend the structural engineer and a corrosion engineer be retained to confirm the information provided and for additional recommendations, as required.

**SECTION 4: GEOLOGIC HAZARDS**

This section presents the results of our Geologic Hazards review, following the requirements of the Division of State Architects (DSA), the Office of Regulatory Services (ORS), and the California Geological Survey (CGS). Our Certified Engineering Geologist performed a reconnaissance of the site on Friday January 9, 2014.

### 4.1 FAULT RUPTURE

Although there are several significant faults located within 100 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. In addition to the known active faults in the region, a number of potentially active faults exist in the area including; the Serra Fault, the Foothill Thrust Fault, the Pilarcitos Fault, and others. More locally, some older faults in the immediate area include;

The Serra Fault is concealed beneath young surficial deposits and is probably not seismogenic (capable of generating an earthquake) however, it is subject to (co-seismic) displacements during large earthquake events on the San Andreas Fault. More locally Schlocker (1958,1974) shows the “City College Fault” as well as the Fort point-Hunters Point Shear Zone trending through the Sunset and Richmond Districts with a northwesterly trend. The near-point of City College fault is approximately 1.75 miles southwest of the site, whereas the Fort Point-Hunters Point Shear Zone is located ½ miles to the northeast of the site. Both of these fault cut Franciscan rocks, where exposed, but otherwise are both shown as concealed beneath young
surficial deposits and is probably not seismogenic (capable of generating an earthquake) however, these fault may be subject to (co-seismic) displacements during large earthquake events on the San Andreas Fault (Marlow, 1994).

A regional fault map showing known faults in the region surrounding the subject site is presented in Figure 4. 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.

4.2 HISTORICAL OCCURENCES OF GROUND FAILURE

Youd, et al. (1978) and Knudsen, et al. (2000) report an incident of ground cracking near the intersection of Park Presidio Boulevard and Fulton Street during the 1906 earthquake, approximately 0.4 miles southwest of the site (Location #226). This incident was originally reported in Lawson, et al. (1908) and was associated with failure of the Fulton Street embankment fill:

“On Fulton Street, between Twelfth and Thirteenth Avenues, there was much slumping of the street-filling down into the Park adjacent; and exactly the same sort of damage occurred on H Street, between Ninth and Fourteenth Avenues…They were especially susceptible to damage from earthquake shock, being particularly loose earth embankments.”

There were no reported occurrences of ground failure in the site vicinity during the 1989 Loma Prieta Earthquake (Seed et al, 1990, Tinsley et al, 1998).

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 sand or silt. The site is not located with a state Seismic Hazard Zone for liquefaction (CDMG, 2000). As discussed in Section 3.3, free ground water was not encountered in our borings to a depth of 50 feet and the historic high ground water level in the area is mapped at a depth of approximately 50 feet. Based on the depth to ground water and the relatively dense state of the soil below a depth of 30 to 40 feet, it is our opinion that the potential for liquefaction at the site is very low.

4.4 LATERAL SPREADING

Lateral spreading (sometimes called ground or earth lurching) typically occurs as a form of horizontal displacement of relatively flat-lying material toward an open face such as an excavation, channel, or body of water. Generally in soil, this movement is due to failure along a weak plane and may be associated with liquefaction. As cracks develop within the weakened material, blocks of soil displace laterally towards the open face, such as a creek. Cracking and lateral movement may gradually propagate away from the face as blocks continue to break free.
Due to the lack of potentially-liquefiable soils and lack of a free face, the potential for lateral spreading is low.

4.5 SEISMIC SETTLEMENT/UNSATURATED SAND SETTLEMENT

Loose unsaturated sandy soils can settle during strong seismic shaking resulting in settlement of the ground surface and building foundations. Seismic compression of unsaturated sand occurs due to rearrangement of soil particles during shaking and compression of the void space. The magnitude of volumetric compression of unsaturated sand is largely a function of seismic loading (effective shear strain and number of cycles) and the state of the soil (relative density and degree of saturation).

Our borings encountered loose to medium dense sands to depths of up to 30 feet. To evaluate the potential magnitude of seismically-induced settlement at the site, we estimated the potential in-situ void ratio and relative density as well as employed the Pradell (1998) method for estimating seismic settlement based on SPT blow counts. We estimated the potential magnitude of volumetric compression based on the available void space.

Our analyses indicate that the site could experience volumetric compression up to 2 percent for loose layers during strong ground shaking. Our estimates of seismically-induced settlement ranged from approximately 1/3-inch (EB-1 and EB-2) to approximately 1½ inches (EB-3 and EB-4). Design recommendations are presented in the "Foundations" section of this report.

4.6 LANDSLIDING

The site is in an area that is substantially flat or nearly so and the nearest significant terrain is located over a mile to the east and also to the southeast. The published regional scale geologic maps covering the region do not show any landslides in the immediate area (Schlocker; 1958, 1974; Brabb and Olsen, 1986, California Geological Survey, 2000) and this is consistent with the results of our interpretation of stereo aerial photos covering the area of the site, as well as our site reconnaissance. The state of California (CGS, 2000) shows the site is not located within a seismically induced landslide hazard mapping for covering the San Francisco Quadrangle.

4.7 TSUNAMI/SEICHE

The County of San Francisco has published a Coastal Tsunami Inundation Map (CGS, 2009) as part of their hazard mitigation plan which indicating estimated wave run-up heights based on a worst case scenario (the Alaskan earthquake of 1964). The potential tsunami inundation area is limited to several blocks from the city’s western shoreline. The site is situated at an elevation of approximately 200 feet and is not within a mapped tsunami hazard zone. The potential for tsunami inundation at the site is low.

4.8 FLOODING AND RESERVOIR INUNDATION

The County of San Francisco has published a Coastal Tsunami Inundation Map (CGS, 2009) as part of their hazard mitigation plan which indicating estimated wave run-up heights based on a
worst case scenario (the Alaskan earthquake of 1964). The site is situated at an elevation of approximately 200 to 202 feet and is not within a mapped tsunami hazard zone. The site is also not located next to any major drainage areas that would be affected by or generate a seismically induced wave. The area of the site has not been mapped through the FEMA Flood Hazards Mapping program. Although there are no watersheds or water courses in the vicinity of the site, the site is located slightly upslope of several small ponds within Golden Gate Park, located ¼ mile to the south and southwest. Therefore, these potential hazards are not anticipated at the site.

4.9 VOLCANIC ERUPTION

The site is located over 200 miles hundred miles from the nearest potentially or historically active volcano (at Mt. Lassen Park). We believe the volcanic eruption hazard for the school site is very low.

4.10 NATURALLY OCCURRING ASBESTOS

Greenstone may contain ultra-mafic rocks such as serpentine that contain Naturally Occurring Asbestos (NOA). Serpentine or greenstone bedrock or other ultra-mafic rocks were not observed at the site during our site reconnaissance. Serpentinite is mapped approximately 1/3 miles east and Greenstone was mapped about 1.5 miles southeast of the site. However it is unlikely that asbestos bearing detritus would have traveled such a distance over variable terrain. Therefore, NOA is not anticipated to be present at the site based on the site geology.

SECTION 5: CBC SEISMIC DESIGN CRITERIA

We developed site-specific seismic design parameters in accordance with Chapters 16A and 18A of the 2013 California Building Code (CBC) and Chapters 11 and 21 of ASCE 7-10.

5.1 SITE CLASSIFICATION

As discussed in Section 3, our borings encountered loose to dense dune sand below the surficial fills.

We reviewed several geologic and shear wave velocity logs performed by the USGS within approximately three miles of the site and within the same geologic unit (Gibbs et al, 1977, USGS Open-File Report 77-850).

Additionally, Wills, et al. (2000) map the site as Site Class D. Schlockner (1974) maps the depth to bedrock in the site vicinity to be approximately 100 feet.

Based on the conditions encountered in our explorations, SPT N-values, and available geologic data, the site may be considered Site Class D. We estimated that the average shear wave velocity for the top 100 feet (30 meters) of the soil profile to be approximately 280 m/s, which was used in our hazard calculations.
Table 4: Shear Wave Velocity Measurements with the Site Vicinity

<table>
<thead>
<tr>
<th>Boring Designation</th>
<th>Distance from Site (miles)</th>
<th>$V_{ss0}$ (m/s)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Chain of Lakes</td>
<td>2.1 W</td>
<td>276</td>
</tr>
<tr>
<td>Windmill</td>
<td>2.5 W</td>
<td>285</td>
</tr>
<tr>
<td>Quintara</td>
<td>2.7 SW</td>
<td>360</td>
</tr>
</tbody>
</table>

5.2 CODE-BASED SEISMIC DESIGN PARAMETERS

Code-based spectral acceleration parameters were determined based on mapped acceleration response parameters adjusted for the specific site conditions. Mapped Risk-Adjusted Maximum Considered Earthquake (MCE$_R$) spectral acceleration parameters ($S_S$ and $S_1$) were calculated using the U. S. Seismic Design Maps on-line hazard calculator (USGS, 2013).

The mapped acceleration parameters were adjusted for local site conditions based on the average soils conditions for the upper 100 feet (30 meters) of the soil profile. MCE$_R$ spectral response acceleration parameters adjusted for site effects ($S_{MS}$ and $S_{M1}$) and design spectral response acceleration parameters ($S_{DS}$ and $S_{D1}$) are presented in Table 5.

In accordance with CBC Section 1613A.3.5, Risk Category I, II, or III structures with mapped spectral response acceleration parameter at the 1-second period ($S_1$) greater than 0.75, are assigned Seismic Design Category E. In accordance with CBC 1616A.1.3, Seismic Design Category E structures require a site-specific ground motion hazard analysis. Site-specific seismic design parameters are presented in Table 8, Section 5.4. The values in Table 5 should not be used for design. Values are provided for determination of Seismic Design Category and comparison with minimum code requirements in our site-specific ground motion hazard analysis.
Table 5: 2013 CBC Site Categorization and Site Coefficients

<table>
<thead>
<tr>
<th>Classification/Coefficient</th>
<th>Design Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Site Class</td>
<td>D</td>
</tr>
<tr>
<td>Site Latitude</td>
<td>37.776310°</td>
</tr>
<tr>
<td>Site Longitude</td>
<td>-122.464418°</td>
</tr>
<tr>
<td>Risk Category</td>
<td>I, II, or III</td>
</tr>
<tr>
<td>Seismic Design Category</td>
<td>E</td>
</tr>
<tr>
<td>Short Period Mapped Spectral Acceleration – SS</td>
<td>1.658g</td>
</tr>
<tr>
<td>1-second Period Mapped Spectral Acceleration – S1</td>
<td>0.763g</td>
</tr>
<tr>
<td>Short-Period Site Coefficient – Fa</td>
<td>1.0</td>
</tr>
<tr>
<td>Long-Period Site Coefficient – Fv</td>
<td>1.5</td>
</tr>
<tr>
<td>Short Period MCE Spectral Response Acceleration Adjusted for Site Effects – SMS</td>
<td>1.658g</td>
</tr>
<tr>
<td>1-second Period MCE Spectral Response Acceleration Adjusted for Site Effects – SM1</td>
<td>1.145g</td>
</tr>
<tr>
<td>Short Period, Design Earthquake Spectral Response Acceleration – SDS</td>
<td>1.105g</td>
</tr>
<tr>
<td>1-second Period, Design Earthquake Spectral Response Acceleration – SD1</td>
<td>0.763g</td>
</tr>
<tr>
<td>Long-Period Transition – TL</td>
<td>12 seconds</td>
</tr>
<tr>
<td>Site Coefficient F_{PGA}</td>
<td>1.0</td>
</tr>
<tr>
<td>PGA_{M} Equation 11.8-1</td>
<td>0.661g</td>
</tr>
</tbody>
</table>

5.3 SITE-SPECIFIC SEISMIC HAZARD ANALYSIS

We performed a site-specific hazard analysis in accordance with ASCE 7-10 Chapter 21.2 and 2013 CBC Section 1803A.6. Our analyses were performed using the computer program EZ-Frisk, version 7.62 (Risk Engineering, 2012) and the 2008 USGS fault model (Petersen, et al., 2008).

Our analysis utilized the mean ground motions predicted by five of the Next Generation Attenuation (NGA) relationships: Boore-Atkinson (2008), Campbell-Bozorgnia (2008), Chiou-Youngs (2007), and Abrahamson-Silva (2007). Our analysis used the FEMA P-750 (2009) method for calculating the maximum rotated component of ground motions, which is based on Huang et al. (2008).

5.3.1 Deterministic MCE_{R}

We performed deterministic seismic hazard analyses in accordance with ASCE 7-10 Section 21.2.2. The deterministic MCE_{R} acceleration response spectrum is defined as the largest 84\textsuperscript{th} percentile ground motion in the direction of maximum horizontal response for each period for characteristic earthquakes on all known active faults within the region. Our analysis
considered all known active faults within 100 kilometers of the site. As shown in Table 1, the site is located close to four major fault sources. The largest deterministic ground motion for all periods resulted from a $M_w$ 8.05 earthquake on the San Andreas Fault.

The 84th percentile ground motion in the direction of maximum horizontal response for this event is presented on Figure 10 (green line). Spectral ordinates are tabulated in Table 6, Column 3.

ASCE 7-10 specifies that the deterministic MCE$_R$ shall not be less than the Deterministic Lower Limit MCE response spectrum (ASCE 7-10 Figure 21.2-1). The Deterministic Lower Limit spectrum is presented on Figure 10 (blue line). Spectral ordinates are tabulated in Table 6, Column 4.

The deterministic MCE spectrum was calculated by taking the greater of Table 6, Columns 3 and 4. Spectral ordinates are tabulated in Table 6, Column 5. The deterministic MCE is presented graphically on Figure 10 (dashed black line).

### 5.3.2 Probabilistic MCE$_R$

We performed a probabilistic seismic hazard analysis (PSHA) in accordance with ASCE 7-10 Section 21.2.1. The probabilistic MCE acceleration response spectrum is defined as the 5 percent damped acceleration response spectrum having a 2 percent probability of exceedance in a 50 year period (2,475-year return period). Our PSHA considered all known active faults within 200 kilometers of the site as well as a gridded seismic source modeled by the USGS (2008). The probabilistic MCE spectrum was multiplied by Risk Coefficients (C$_R$) to determine the probabilistic MCE$_R$. We used Risk Coefficients (C$_{RS}$ and C$_{R1}$) of 0.995 and 0.945, respectively, based on ASCE 7-10 Section 21.2.1.1 - Method 1 and the USGS on-line calculator.

The resulting probabilistic MCE$_R$ is presented on Figure 11 (red line). Spectral ordinates are tabulated in Table 6, Column 6.

### 5.3.3 Site-Specific MCE$_R$

The site-specific MCE is defined by ASCE 7-10 Section 21.2.3 as the lesser of the deterministic and probabilistic MCE’s at each period. The site-specific MCE spectrum was calculated by taking the lesser of the deterministic MCE (Table 6, Column 5, MCE, Figure 11, blue line) and the probabilistic MCE (Table 6, Column 6, Figure 11, red line). Spectral ordinates for the site-specific MCE are tabulated in Table 6, Column 7 and shown graphically on Figure 11 (dashed black line).
Table 6: Development of Site-Specific MCE Spectrum

<table>
<thead>
<tr>
<th>Period (seconds)</th>
<th>CBC General Spectrum (g)</th>
<th>Largest 84th Percentile Deterministic (g)</th>
<th>Deterministic Lower Limit (g)</th>
<th>Deterministic MCE (g)</th>
<th>Probabilistic MCE (g)</th>
<th>Site-Specific MCE (g)</th>
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</thead>
<tbody>
<tr>
<td>0</td>
<td>0.442</td>
<td>0.776</td>
<td>0.600</td>
<td>0.776</td>
<td>1.035</td>
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<tr>
<td>0.05</td>
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<td>0.975</td>
<td>0.975</td>
<td>1.181</td>
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<td>2.115</td>
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</tbody>
</table>

5.3.4 Design Response Spectrum

The site-specific Design Response Spectrum (DRS) is defined in ASCE 7-10 Section 21.3 as \( \frac{2}{3} \) of the site-specific MCE, but not less than 80% of the general design response spectrum. Spectral accelerations corresponding to \( \frac{2}{3} \) of the MCE are tabulated in Table 7, Column 2. Ordinates corresponding to 80% of the general Site Class D response spectrum are tabulated in Table 7, Column 3. Ordinates of the site-specific DRS are tabulated in Table 7, Column 4. Development of the site-specific DRS is presented graphically on Figure 12.
Table 7: Development of Site-Specific Design Response Spectrum

<table>
<thead>
<tr>
<th>Period (seconds)</th>
<th>2/3 Site-Specific MCE (g)</th>
<th>80% CBC General Spectrum (g)</th>
<th>Design Response Spectrum (g)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>0.517</td>
<td>0.354</td>
<td>0.517</td>
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<td>0.650</td>
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<td>1.015</td>
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<td>0.30</td>
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<td>1.087</td>
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<td>0.40</td>
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<td>1.098</td>
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<td>1.077</td>
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<td>0.884</td>
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<td>0.814</td>
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<td>0.305</td>
<td>0.632</td>
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<td>0.203</td>
<td>0.471</td>
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<td>4.00</td>
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<td>0.153</td>
<td>0.362</td>
</tr>
<tr>
<td>5.00</td>
<td>0.288</td>
<td>0.122</td>
<td>0.288</td>
</tr>
</tbody>
</table>

5.4 DESIGN ACCELERATION PARAMETERS

Site-specific design acceleration parameters ($S_{DS}$ and $S_{D1}$) were determined in accordance with Section 21.4 of ASCE 7-10. $S_{DS}$ is defined as the design spectral acceleration at a period of 0.2 seconds, but not less than 90% of the spectral acceleration at any period greater than 0.2 seconds. $S_{D1}$ is defined as the greater of the design spectral acceleration at a period of 1 second or two times the spectral acceleration at a period of 2 seconds.

Site-specific MCE spectral response acceleration parameters ($S_{MS}$ and $S_{M1}$) are calculated as 1.5 times the $S_{DS}$ and $S_{D1}$ values, respectively, but not less than 80% of the code-based values presented in Table 5. Site-specific design acceleration parameters are summarized in Table 8.

When using the Equivalent Lateral Force Procedure, ASCE 7-10 Section 21.4 allows using the spectral acceleration at any period (T) in lieu of $S_{D1}/T$ in Eq. 12.8-3. The site-specific spectral acceleration at any period may be calculated by interpolation of the spectral ordinates in Table 7, Column 4.
Table 8: Site-Specific Design Acceleration Parameters

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>S_{DS}</td>
<td>1.015</td>
</tr>
<tr>
<td>S_{D1}</td>
<td>1.264</td>
</tr>
<tr>
<td>S_{MS}</td>
<td>1.523</td>
</tr>
<tr>
<td>S_{M1}</td>
<td>1.896</td>
</tr>
</tbody>
</table>

5.5 MCE\textsubscript{G} PEAK GROUND ACCELERATION

We calculated the MCE Geometric Mean Peak Ground Acceleration (MCE\textsubscript{G}) in accordance with ASCE 7-10 Section 21.5. The MCE\textsubscript{G} is calculated as the lesser of probabilistic and deterministic geometric mean PGA. The probabilistic geometric mean PGA is 0.94g. The deterministic MCE\textsubscript{G} is considered the greater of the largest 84\textsuperscript{th} percentile deterministic geometric mean PGA (0.71g) or one-half of the tabulated F\textsubscript{PGA} value from ASCE 7-10 Table 11.8.1. For the site, F\textsubscript{PGA} is 1.0 and one half of the F\textsubscript{PGA} is 0.5g; therefore, the deterministic MCE\textsubscript{G} is 0.71g. Additionally, the MCE\textsubscript{G} may not be less than 80\% of the mapped PGA\textsubscript{M} determined from ASCE -10 Equation 11.8-1. The PGA\textsubscript{M} for the site is 0.66g; 80\% of PGA\textsubscript{M} is 0.53g. Therefore, the MCE\textsubscript{G} for the site may be considered 0.71g.

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.

- Potential for seismic-induced dry sand settlement
- Presence of undocumented fill
- Construction of temporary slopes in sands

6.1.1 Potential for Seismic-Induced Dry Sand Settlement

As discussed, our borings encountered loose to medium dense sand to a depth of up to approximately 30 feet. Our analysis indicates that seismically-induced dry sand settlement of less than 1/3-inch (EB-1 and EB-2) to ½ to 1½ inches in EB-3 and EB-4. For the improvements in the vicinity of EB-1 and EB-2, we would recommend they be supported on a grade-beam type foundation system to reduce the potential for differential settlement. Detailed foundation recommendations are presented in the “Foundations” section.
6.1.2 Presence of Undocumented Fill

As previously discussed, our borings encountered undocumented fill to a depth of approximately 2½ feet at EB-1 and up to about 7½ feet in EB-2, EB-3, and EB-4. The fill is undocumented because compaction test records are not known to exist at the time this report was prepared. Based on our review of the boring logs, the fill in the area of the new foundations (EB-1 and EB-2) appears to have been compacted to a denser state than the underlying native soils. Based on our review of the foundation plans for the existing buildings, it appears that they were constructed on native soils and undocumented fill. No observed settlement of the improvements on the existing fill was observed or reported. New foundation elements should be founded on undisturbed soils. For the new flatwork and pavement improvements, the upper 8 inches of the soil subgrade should be recompacted. Recommendations are provided in the “Earthwork” section to address this concern.

6.1.3 Construction of Temporary Slopes in Sands

Depending on the proximity of the new improvements to the existing footings, underpinning of existing foundations may be needed. We recommend that underpinning be performed in segments and the excavation be continuously lagged with lagging installed as the excavation proceeds downward. Recommendations for underpinning are presented in the “Earthwork” section of this report.

The soil encountered during our subsurface exploration consists of fine to medium sand ranging from loose to medium dense. The sandy soil is likely to not stand vertical when excavated (i.e. foundation, utility trench, and drilled pier excavations). The contractor will need to address this issue. We recommend that consideration be given to installing Stay-Form®, or similar, on all excavations including shallow footings/grade beams and trenches to reduce the potential for sidewall collapse. Recommendations addressing this concern are presented in the “Earthwork” section of this report.

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

SECTION 7: EARTHWORK

7.1 SITE DEMOLITION, CLEARING AND PREPARATION

7.1.1 Site Stripping

Areas that will be redeveloped should be stripped of all surface vegetation, and surface and subsurface improvements within the proposed development area. Demolition of existing improvements is discussed in detail below. A detailed discussion of removal of existing fills is provided later in this report. Surface vegetation and topsoil should be stripped to a sufficient depth to remove all material greater than 3 percent organic content by weight.

7.1.2 Tree and Shrub Removal

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 Slabs, Foundations and Pavements

All slabs, foundations, and pavements 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 utilities 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 remove all abandoned utilities 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 TEMPORARY CUT 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 native dune sand, as well existing undocumented fills constructed of sand, should be classified as primarily as OSHA Type C materials. A competent person should determine that actual soil classification during construction and be responsible for implementing and maintaining safe excavation slope inclination and/or shoring at the site during construction. For cuts of 10 feet or less, the side slopes should be 1.5:1 (H:V) or flatter per OSHA requirements or supported by shoring. No unsupported cuts should be made. Use of Stay-Form® or continuous lagging which is installed as the excavations progress down and should be implemented by the contractor to support excavations. Recommended soil parameters for temporary shoring are provided in the “Temporary Shoring” section of this report.

7.3 SUBGRADE PREPARATION

After site clearing and demolition is complete, and prior to backfilling any excavations resulting from fill removal or demolition, the excavation subgrade and subgrade within areas to receive additional site fills, slabs-on-grade and/or pavements should be scarified to a depth of 6 inches, moisture conditioned, and compacted in accordance with the “Compaction” section below.

Due to the sandy soil likely to be encountered at the subgrade elevation, we suggest recommend that subgrade compaction and proof rolling be performed within 24 hours of capillary break layer or slab-on-grade construction.

7.4 MATERIAL FOR FILL

7.4.1 Re-Use of On-site Soil

On-site soil 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.

7.4.2 Potential Import Sources

Imported and non-expansive material should be inorganic with a Plasticity Index (PI) of 15 or less, and 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 for review and acceptance or rejection based on these criteria. Information regarding the import source shall 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 shall 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 by a qualified laboratory 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 shall be provided to the owner; additional laboratory testing may be required based on the owner’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.

7.4.3 Controlled Low-Strength Material

Controlled Low-Strength Material (CLSM) may be used as engineered fill. As with all engineered fill, CLSM should be placed on subgrade soils prepared in accordance with Section 7.5. CLSM should have a minimum 28-day unconfined compressive strength of 50 to 100 pounds per square inch (psi). Unconfined compression testing should be performed in accordance with ASTM D4832. CLSM should be placed and tested in accordance with DSA IR 18-1.

7.5 BELOW-GRADE EXCAVATIONS

Below-grade excavations may be constructed with temporary slopes in accordance with the “Temporary Cut Slopes” section above if space allows. Alternatively, temporary shoring may support the planned cuts up to 10 feet. We have provided geotechnical parameters for shoring design in the section below. The choice of shoring method should be left to the contractor’s judgment based on experience, economic considerations and adjacent improvements such as utilities, pavements, and foundation loads. Temporary shoring should support adjacent improvements without distress and should be the contractor’s responsibility. A pre-condition survey including photographs and installation of monitoring points for existing site improvements should be included in the contractor’s scope. We should be provided the opportunity to review the geotechnical parameters of the shoring design prior to implementation; the project structural engineer should be consulted regarding support of adjacent structures.

7.5.1 Temporary Shoring

Based on the site conditions encountered during our investigation, the cuts may be supported by soldier beams and tie-backs, braced excavations, soil nailing, or potentially other methods.
Where shoring will extend more than about 10 feet vertically, restrained shoring will most likely be required to limit detrimental lateral deflections and settlement behind the shoring. In addition to soil earth pressures, the shoring system will need to support adjacent loads such as construction vehicles and incidental loading, existing structure foundation loads, and street loading. We recommend that heavy construction loads (cranes and concrete and watering trucks, etc.) and material stockpiles be kept at least 15 feet behind the shoring. Where this loading cannot be set back, the shoring will need to be designed to support the loading. The shoring designer employed by the contractor should provide for timely and uniform mobilization of soil pressures that will not result in excessive lateral deflections. Minimum suggested geotechnical parameters for shoring design are provided in the table below.

**Table 9: Suggested Temporary Shoring Design Parameters**

<table>
<thead>
<tr>
<th>Design Parameter</th>
<th>Design Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Minimum Lateral Wall Surcharge (upper 5 feet)</td>
<td>120 psf</td>
</tr>
<tr>
<td>Cantilever Wall – Triangular Earth Pressure</td>
<td>35 pcf</td>
</tr>
<tr>
<td>Restrained Wall – Uniform Earth Pressure</td>
<td>25H*</td>
</tr>
<tr>
<td>Passive Pressure – Starting at 2 feet below the bottom of the excavation</td>
<td>400 pcf up to 2,000 psf maximum uniform pressure</td>
</tr>
</tbody>
</table>

* H equals the height of the excavation; passive pressures are assumed to act over twice the soldier pile diameter

The restrained earth pressure may also be distributed as described in Figure 24 of the *FHWA Circular No. 4 – Ground Anchors and Anchored Systems* (with the hinge points at ¼H and ¾H) provided the total pressure is established from the uniform pressure above.

If shotcrete lagging is used for the shoring facing, the permanent retaining wall drainage materials, as discussed in the “Wall Drainage” section of this report, will need to be installed during temporary shoring construction. At a minimum, 2-foot-wide vertical panels should be placed between soil nails or tiebacks that are spaced at 6-foot centers. For 8-foot centers, 4-foot-wide vertical panels should be provided. A horizontal strip drain connecting the vertical panels should be provided, or pass-through connections should be included for each vertical panel.

We performed our borings with hollow-stem auger drilling equipment and as such were not able to evaluate the potential for caving soil, which can create difficult conditions during soldier beam, tie-back, or soil nail installation; caving soil can also be problematic during excavation and lagging placement. The contractor is responsible for evaluating excavation difficulties prior to construction. Where relatively clean sands (especially encountered below ground water) or difficult drilling or cobble conditions were encountered during our exploration, pilot holes performed by the contractor may be desired to further evaluate these conditions prior to the finalization of the shoring budget. Based on the conditions observed, the drilling contractor should anticipate the use of casing to stabilize the sidewalls.
In addition to anticipated deflection of the shoring system, other factors such as voids created by soil sloughing, and erosion of granular layers due to perched water conditions can create adverse ground subsidence and deflections. The contractor should attempt to cut the excavation as close to neat lines as possible; where voids are created they should be backfilled as soon as possible with sand, gravel, or grout to reduce the impact of sloughing and caving soil on adjacent improvements.

As previously mentioned, we recommend that a monitoring program be developed and implemented to evaluate the effects of the shoring on adjacent improvements. All sensitive improvements should be located and monitored for horizontal and vertical deflections and distress cracking based on a pre-construction survey. For multi-level excavations, the installation of inclinometers at critical areas may be desired for more detailed deflection monitoring. The monitoring frequency should be established and agree to by the project team prior to start of shoring construction.

The above recommendations are for the use of the design team; the contractor in conjunction with input from the shoring designer should perform additional subsurface exploration they deem necessary to design the chosen shoring system. A California-licensed civil or structural engineer employed by the general contractor must design and be in responsible charge of the temporary shoring design. The contractor is responsible for means and methods of construction, as well as site safety.

7.6 COMPACCTION 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, sandy/gravelly soil should be compacted with vibratory equipment and open-graded material such as crushed rock should be placed in lifts not thicker than 18 inches and 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.
Table 10: Compaction Requirements

<table>
<thead>
<tr>
<th>Description</th>
<th>Material Description</th>
<th>Minimum Relative(^1) Compaction (percent)</th>
<th>Moisture(^2) Content (percent)</th>
</tr>
</thead>
<tbody>
<tr>
<td>General Fill</td>
<td>On-Site Soils</td>
<td>90</td>
<td>&gt;1</td>
</tr>
<tr>
<td>Trench Backfill</td>
<td>On-Site Soils</td>
<td>90</td>
<td>&gt;1</td>
</tr>
<tr>
<td>Trench Backfill (upper 6 inches of pavement subgrade)</td>
<td>On-Site Soils</td>
<td>95</td>
<td>&gt;1</td>
</tr>
<tr>
<td>Crushed Rock Fill</td>
<td>(\frac{3}{4})-inch Clean Crushed Rock</td>
<td>Consolidate In-Place</td>
<td>NA</td>
</tr>
<tr>
<td>Flatwork Subgrade</td>
<td>On-Site Soils</td>
<td>90</td>
<td>&gt;1</td>
</tr>
<tr>
<td>Flatwork Aggregate Base</td>
<td>Class 2 Aggregate Base(^3)</td>
<td>90</td>
<td>Optimum</td>
</tr>
<tr>
<td>Pavement Subgrade</td>
<td>On-Site Soils</td>
<td>95</td>
<td>&gt;1</td>
</tr>
<tr>
<td>Pavement Aggregate Base</td>
<td>Class 2 Aggregate Base(^3)</td>
<td>95</td>
<td>Optimum</td>
</tr>
<tr>
<td>Asphalt Concrete</td>
<td>Asphalt Concrete</td>
<td>95</td>
<td>NA</td>
</tr>
</tbody>
</table>

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

7.7 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 (\(\frac{3}{4}\)-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.
concrete within the influence zone. Sand-cement slurry used within foundation influence zones should have a minimum compressive strength of 75 psi.

7.8 UNDERPINNING EXISTING FOOTINGS

Excavations for new foundations may potentially extend below the influence zone for existing building foundations, undermining the foundation. On a preliminary basis, to address this issue, the footings should be underpinned in a segmented manner. The segments for underpinning from a geotechnical perspective should be about 4 feet wide, at 8 foot spacing from center, and excavated at alternate locations along the existing footing width. Underpinning should be constructed in an alternating segmented manner to avoid undermining the entirety of the existing footing all at once. Underpinning excavations should be continuously lagged from the top down to avoid caving of the native soil. A Cornerstone representative should be onsite while this operation is performed to verify the spacing, depths, and bearing materials.

7.9 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.

SECTION 8: FOUNDATIONS

8.1 SUMMARY OF RECOMMENDATIONS

In our opinion, the proposed structural improvements may be supported on shallow foundations provided the recommendations in the “Earthwork” section and the sections below are followed and the structure can withstand the total and differential settlement estimates.

Seismic design criteria in accordance with the 2013 California Building Code are presented in Section 5.

8.2 GRADE BEAM FOUNDATIONS

Continuous grade beam footings should bear on natural, undisturbed soil or engineered fill, be at least 18 inches wide, and extend at least 24 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 existing footing, or 2) finished exterior grade, excluding landscaping topsoil.

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 2,000 psf for dead loads, 3,000 psf for combined dead plus live loads, and 4,000 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.

8.2.2 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 11: Assumed Structural Loading

<table>
<thead>
<tr>
<th>Foundation Area</th>
<th>Range of Assumed Loads</th>
</tr>
</thead>
<tbody>
<tr>
<td>Grade Beams</td>
<td>1 to 3 kips per lineal foot</td>
</tr>
</tbody>
</table>

Based on the above loading and the allowable bearing pressures presented above, we estimate that the total static footing settlement will be on the order of 1/3-inch, with less than ¼-inch of post-construction differential settlement between adjacent foundation elements. In addition, we estimate that differential seismic movement will be on the order of less than ¼-inch in the vicinity of EB-1 and EB-2 and up to 2/3 to 1-inch in the vicinity of EB-3 and EB-4 between adjacent foundation elements or over a horizontal distance of 50 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.

Additionally, we should work with the structural engineer to develop modulus of subgrade reaction for their SAFE analysis, if needed.

8.2.3 Lateral Loading

Lateral loads may be resisted by friction between the bottom of footing and the supporting subgrade, and also by passive pressures generated against footing sidewalls. An ultimate frictional resistance of 0.45 applied to the footing dead load, and an ultimate passive pressure based on an equivalent fluid pressure of 400 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.

8.2.4 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.

Sloughing of the sandy soil is considered likely; therefore we recommend that Stay-Form® or similar be placed within the footing excavations as they are excavated during construction of the foundation elements. Footing excavations should be filled as soon as possible or be kept moist until concrete placement to further reduce the potential for sloughing. The contractor should anticipate side wall over break or caving and additional concrete. A Cornerstone representative should observe all footing excavations prior to placing reinforcing steel and concrete. If there is a significant schedule delay between our initial observation and concrete placement, we may need to re-observe the excavations.

SECTION 9: CONCRETE SLABS AND PEDESTRIAN PAVEMENTS

9.1 INTERIOR SLABS-ON-GRADE

The proposed slabs-on-grade may be supported directly on 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 construction, the subgrade should be proof-rolled to confirm subgrade stability, and if the soil has been allowed to dry out, the subgrade should be re-moisture conditioned to near optimum moisture content.

The structural engineer should determine the appropriate slab reinforcement for the loading requirements. Consideration should be given to limiting the control joint spacing to a maximum of about 2 feet in each direction for each inch of concrete thickness.

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 owner, 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 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 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 concrete flatwork subject to pedestrian and/or occasional light pick up loading should be at least 4 inches thick and supported directly upon subgrade prepared in accordance with the “Earthwork” recommendations of this report. 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. To help reduce the potential for uncontrolled shrinkage cracking, adequate expansion and control joints should be included. Consideration should be given to limiting the control joint spacing to a maximum of about 2 feet in each direction for each inch of concrete thickness. Flatwork should be isolated from adjacent foundations or retaining walls except where limited sections of structural slabs are included to help span irregularities in retaining wall backfill at the transitions between at-grade and on-structure flatwork.

SECTION 10: VEHICULAR PAVEMENTS

10.1 ASPHALT CONCRETE

The following asphalt concrete pavement recommendations tabulated below are 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 20. The design R-value was chosen based on engineering judgment considering the variable surface conditions.
Table 11: Asphalt Concrete Pavement Recommendations

<table>
<thead>
<tr>
<th>Design Traffic Index (TI)</th>
<th>Asphalt Concrete (inches)</th>
<th>Class 2 Aggregate Base* (inches)</th>
<th>Total Pavement Section Thickness (inches)</th>
</tr>
</thead>
<tbody>
<tr>
<td>4.0</td>
<td>2.5</td>
<td>5.5</td>
<td>8.0</td>
</tr>
<tr>
<td>4.5</td>
<td>2.5</td>
<td>7.0</td>
<td>9.5</td>
</tr>
<tr>
<td>5.0</td>
<td>3.0</td>
<td>7.0</td>
<td>10.0</td>
</tr>
<tr>
<td>5.5</td>
<td>3.0</td>
<td>9.0</td>
<td>12.0</td>
</tr>
<tr>
<td>6.0</td>
<td>3.5</td>
<td>9.5</td>
<td>13.0</td>
</tr>
<tr>
<td>6.5</td>
<td>4.0</td>
<td>10.5</td>
<td>14.5</td>
</tr>
</tbody>
</table>

*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 use the pavements.

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 12: PCC Pavement Recommendations

<table>
<thead>
<tr>
<th>Allowable ADTT</th>
<th>Minimum PCC Thickness (inches)</th>
</tr>
</thead>
<tbody>
<tr>
<td>13</td>
<td>5.5</td>
</tr>
<tr>
<td>130</td>
<td>6.0</td>
</tr>
</tbody>
</table>

The PCC thicknesses above are based on a concrete compressive strength of at least 3,500 psi, supporting the PCC on at least 4 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 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. 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 13: Recommended Lateral Earth Pressures

<table>
<thead>
<tr>
<th>Wall Condition</th>
<th>Lateral Earth Pressure*</th>
<th>Additional Surcharge Loads</th>
</tr>
</thead>
<tbody>
<tr>
<td>Unrestrained – Cantilever Wall</td>
<td>45 pcf</td>
<td>½ of vertical loads at top of wall</td>
</tr>
<tr>
<td>Restrained – Braced Wall</td>
<td>45 pcf + 8H** psf</td>
<td>½ of vertical loads at top of wall</td>
</tr>
</tbody>
</table>

* 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

The 2013 CBC states that lateral pressures from earthquakes should be considered in the design of basements and retaining walls. At this time, we are not aware of any new retaining walls for the project and have not provided seismic earth pressures with this report. Seismic earth pressures can be provided at a later time, if requested by the project design team. Seismic earth pressures are not required to design minor landscape retaining walls.

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 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.

As discussed previously, consideration should be given to the transitions from on-grade to on-structure. Providing subslabs or other methods for reducing differential movement of flatwork or pavements across this transition should be included in the project design.

11.5 FOUNDATIONS

Retaining walls may be supported on spread footings or drilled piers designed in accordance with the recommendations presented in the “Foundations” section of this report.

SECTION 12: LIMITATIONS

This report, an instrument of professional service, has been prepared for the sole use of San Francisco Unified School District specifically to support the design of the Frank McCoppin Elementary School Improvements project at 651 6th Avenue in San Francisco, 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.
San Francisco Unified School District may have provided Cornerstone with plans, reports and other documents prepared by others. San Francisco Unified School District understands that Cornerstone reviewed and relied on the information presented in these documents and cannot be responsible for their accuracy.

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

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

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

Recommendations provided in this report are based on the assumption that Cornerstone will be retained to provide observation and testing services during construction to confirm that conditions are similar to that assumed for design, and to form an opinion as to whether the work has been performed in accordance with the project plans and specifications. If we are not retained for these services, Cornerstone cannot assume any responsibility for any potential claims that may arise during or after construction as a result of misuse or misinterpretation of Cornerstone’s report by others. Furthermore, Cornerstone will cease to be the Geotechnical-Engineer-of-Record if we are not retained for these services.

SECTION 13: REFERENCES


Bonilla, M.G., 1971, Preliminary geologic map of the San Francisco South Quadrangle and part of the Hunter's Point Quadrangle, California: U.S. Geological Survey Miscellaneous Field Studies Map MF-311, 2 sheets, scale 1:24,000.


California Division of Mines and Geology, 2000, State of California Seismic Hazard Zone Report for the City and County of San Francisco, SHZR 043.


Schlocker, J., 1974, Geology of the San Francisco North Quadrangle, California, USGS Professional Paper 782.


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

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

<table>
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<tr>
<th>Date</th>
<th>Flight</th>
<th>Frames</th>
<th>Scale</th>
<th>Type</th>
</tr>
</thead>
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<td>August, 1938</td>
<td>Harrison Ryker</td>
<td>5852 - 49, 50</td>
<td>1:20,000</td>
<td>vertical black &amp; white</td>
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<td>October 11, 1943</td>
<td>DDB-2B</td>
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<td>1:20,000</td>
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<tr>
<td>October 5, 1946</td>
<td>GS-CP-2</td>
<td>83, 84</td>
<td>1:20,000</td>
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<td>32</td>
<td>1:73, 821</td>
<td>vertical black &amp; white</td>
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<td>September 9, 1956</td>
<td>GS-VLX-1</td>
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<td>1:23,600</td>
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<td>March 26, 1986</td>
<td>CDBW-APU-C</td>
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</table>
Vicinity Map

McCoppin Elementary School
San Francisco, CA
Geologic Units

Qaf  Artificial fill (Holocene)

Qd  Dune sand (Holocene)

Explanation

Contact- dashed where approximate, dotted where concealed

Vicinity Geologic Map
McCoppin Elementary School
San Francisco, CA

Project Number: 608-5-1
Figure Number: Figure 4
Date: January 2015
Area Within a 100km (62.4 mile) radius

From: T. Toppozada & Others (2000)
Section A-A'
(View Looking Southwest)
1"=20' H:V

Explanation

Geologic Units

Af  Man-made fill
Qd  Quaternary dune sand

Symbols

● Approximate location of exploratory boring (EB)
--- Approximate geologic contact

Notes:
1) Surficial fills associated with existing pavements, landscaping or utilities are not shown.
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.
**Explanation**

**Geologic Units**

- **Af**  Man-made fill
- **Qd**  Quaternary dune sand

**Symbols**

- *Approximate location of exploratory boring (EB)*
- *Approximate geologic contact*

**Section B-B’**

(View Looking North)

1’=20’ H:V

**Notes:**

1) Surficial fills associated with existing pavements, landscaping or utilities are not shown.
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.
Explanation

Liquefaction
Areas where historic occurrence of liquefaction, or local geological, geotechnical and groundwater conditions indicate a potential for permanent ground displacements such that mitigation as defined in Public Resources Code Section 2863(c) would be required.

Earthquake-Induced Landslides
Areas where previous occurrence of landslide movement, or local topographic, geological, geotechnical and subsurface water conditions indicate a potential for permanent ground displacements such that mitigation as defined in Public Resources Code Section 2863(c) would be required.

Base by State of California, Seismic Hazard Zones.
The Deterministic Maximum Considered Earthquake (MCEₕ) is defined as the greater of the following at all periods:

- The largest 84th percentile ground motion from a characteristic earthquake on all known active faults, or
- The Deterministic Lower Limit MCE Spectrum – ASCE 7–10, Figure 21.2–1.

Spectral ordinates are presented in Table 6.
The Site-Specific Maximum Considered Earthquake is defined as the lesser of the following at all periods:

- Deterministic $MCE_R$ – developed on Figure 10, or
- Probabilistic $MCE_R$ – defined as 2,475-year ground motion.

Spectral ordinates are presented in Table 6.
The Site-Specific Design Response Spectrum is defined as the greater of the following at all periods:

- 2/3 of the Site-Specific MCE — developed on Figure 11, or
- 80% of the CBC General Spectrum.

Spectral ordinates are presented in Table 7.
APPENDIX A: FIELD INVESTIGATION

The field investigation consisted of a surface reconnaissance and a subsurface exploration program using truck-mounted hollow-stem auger and portable “minute-man,” solid-flight auger drilling equipment. Four 4-inch diameter exploratory borings were drilled on December 22, 2014, to depths of 15 to 50 feet. The approximate locations of exploratory borings are shown on the Site Plan and Geologic Map, 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, as well as a key to the classification of the soil and bedrock, are included as part of this appendix.

Boring locations were approximated using existing site boundaries, a hand held GPS unit, and other site features as references. Boring elevations were determined based on interpolation of plan contours. The locations and elevations of the borings should be considered accurate only to the degree implied by the method used.

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

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

**DESCRIPTION**

- **Poorly Graded Sand (SP) [Fill]**
  - medium dense, moist, brown, fine sand, some fine to coarse subrounded to subangular gravel
  - NP = Non-plastic
- **Poorly Graded Sand (SP)**
  - medium dense, moist, brown, fine sand
  - loose
- **Dense**
  - Bottom of Boring at 15.0 feet.

**GROUNDED WATER LEVELS:**
- **AT TIME OF DRILLING** Not Encountered
- **AT END OF DRILLING** Not Encountered

**PROJECT INFORMATION**
- **PROJECT NAME**: McCoppin Elementary School Improvements
- **PROJECT NUMBER**: 608-5-1
- **PROJECT LOCATION**: 651 6th Ave. San Francisco, CA
- **GROUND ELEVATION**: 201.5 FT +/-
- **BORING DEPTH**: 15 ft.
- **LATITUDE**: 37.776175°
- **LONGITUDE**: -122.464250°

**DRILLING INFORMATION**
- **DATE STARTED**: 12/22/14
- **DATE COMPLETED**: 12/22/14
- **DRILLING METHOD**: Minuteman, 4 inch Solid Flight Auger
- **DRILLING CONTRACTOR**: Exploration Geoservices, Inc.

**GROUND WATER LEVELS:**
- **DATE STARTED**: 12/22/14
- **DATE COMPLETED**: 12/22/14

**BORING NUMBER EB-1**

**ELEVATION (ft)**

<table>
<thead>
<tr>
<th>SYMBOL</th>
<th>N-Value (uncorrected) blows per foot</th>
<th>SAMPLES</th>
<th>TYPE AND NUMBER</th>
<th>DRY UNIT WEIGHT</th>
<th>NATURAL MOISTURE CONTENT</th>
<th>PLASTICITY INDEX %</th>
<th>UNCONSOLIDATED-UNDRAINED TRIAXIAL</th>
</tr>
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<td>0</td>
<td></td>
<td></td>
<td></td>
<td></td>
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<td>20</td>
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</table>

**UNCONFINED COMPRESSION**

<table>
<thead>
<tr>
<th>SYMBOL</th>
<th>N-Value (uncorrected) blows per foot</th>
<th>SAMPLES</th>
<th>TYPE AND NUMBER</th>
<th>DRY UNIT WEIGHT</th>
<th>NATURAL MOISTURE CONTENT</th>
<th>PLASTICITY INDEX %</th>
<th>UNCONSOLIDATED-UNDRAINED TRIAXIAL</th>
</tr>
</thead>
</table>
**Description**

**Poorly Graded Sand with Silt (SP-SM) [Fill]**
- medium dense to dense, moist, brown to dark brown, fine to medium sand, some fine to coarse subangular gravel

**Silty Sand (SM) [Fill]**
- medium dense to dense, moist, brown to reddish brown, fine to medium sand, some fine to coarse subangular gravel

**Poorly Graded Sand (SP)**
- medium dense, moist, brown, fine sand

**Notes**

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**Ground Water Levels:**
- **Not Encountered** at time of drilling
- **Not Encountered** at end of drilling
**BORING NUMBER EB-3**

**DATE STARTED** 12/22/14  
**DATE COMPLETED** 12/22/14

**DRILLING CONTRACTOR** Exploration Geoservices, Inc.

**DRILLING METHOD** Minuteman, 4 inch Solid Flight Auger

**LOGGED BY** MAA

**NOTES**

---

**DESCRIPTION**

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<th>Depth (ft)</th>
<th>Symbol</th>
<th>Description</th>
<th>N-Value (uncorrected) blows per foot</th>
<th>Samples</th>
<th>Type and Number</th>
<th>Laboratory PCF</th>
<th>Natural Moisture Content</th>
<th>Plasticity Index</th>
<th>Undrained Shear Strength, ksf</th>
<th>Torvane</th>
<th>Hand Penetrometer N-Value (uncorrected) blows per foot</th>
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<td>197.5</td>
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<td></td>
<td>Clayey Sand (SC) [Fill]</td>
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<td>SPT-3</td>
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<tr>
<td>196.0</td>
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<td>Poorly Graded Sand (SP) [Fill]</td>
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<td>193.5</td>
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<td>Poorly Graded Sand (SP) [Fill]</td>
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<td></td>
<td>20</td>
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<td>Poorly Graded Sand (SP)</td>
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<td>SPT-7</td>
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<td></td>
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</tr>
<tr>
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<td>25</td>
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<td></td>
<td>40</td>
<td>SPT-8</td>
<td>4</td>
<td></td>
<td></td>
<td></td>
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<td></td>
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<td>41</td>
<td>SPT-9</td>
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<td>SPT-10</td>
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<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

- Bottom of Boring at 30.0 feet.

---

**PROJECT NAME** McCoppin Elementary School Improvements

**PROJECT NUMBER** 608-5-1

**PROJECT LOCATION** 651 6th Ave. San Francisco, CA

**GROUND ELEVATION** 201 FT +/-  
**BORING DEPTH** 30 ft.

**LATITUDE** 37.776521°  
**LONGITUDE** -122-464880°

**GROUND WATER LEVELS:**

- **AT TIME OF DRILLING** Not Encountered
- **AT END OF DRILLING** Not Encountered

---

**NOTES**

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**PROJECT NAME**  McCoppin Elementary School Improvements  
**PROJECT NUMBER**  608-5-1  
**PROJECT LOCATION**  651 6th Ave. San Francisco, CA  
**GROUND ELEVATION**  201 FT +/-  
**BORING DEPTH**  50 ft.  
**LATITUDE**  37.776533°  
**LONGITUDE**  -122.464539°  

**GROUND WATER LEVELS:**  
\(\checkmark\) **AT TIME OF DRILLING** Not Encountered  
\(\checkmark\) **AT END OF DRILLING** Not Encountered  

**NOTES**  
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<table>
<thead>
<tr>
<th>ELEVATION (ft)</th>
<th>DEPTH (ft)</th>
<th>SYMBOL</th>
<th>5½ inches Portland cement concrete</th>
</tr>
</thead>
<tbody>
<tr>
<td>201.0</td>
<td>0</td>
<td></td>
<td>Silty Sand (SM) [Fill] medium dense, moist, brown to reddish brown, fine to medium sand, some fine to coarse subangular gravel</td>
</tr>
<tr>
<td>198.0</td>
<td>3</td>
<td></td>
<td>Poorly Graded Sand with Silt (SP-SM) [Fill] loose, moist, brown to dark brown, fine to medium sand, some fine to coarse subangular gravel</td>
</tr>
<tr>
<td>193.5</td>
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<td>Poorly Graded Sand (SP) medium dense, moist, brown, fine sand</td>
</tr>
<tr>
<td>173.0</td>
<td>25</td>
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<td>loose</td>
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</tbody>
</table>

**DESCRIPTION**

**UNCONFINED COMPRESSION**

<table>
<thead>
<tr>
<th>UNCONSOLIDATED-UNDRAINED TRIAXIAL</th>
<th>MC-2</th>
<th>119</th>
<th>10</th>
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<tr>
<td>MC-4</td>
<td>27</td>
<td>112</td>
<td>5</td>
</tr>
<tr>
<td>MC-6</td>
<td>12</td>
<td>SPT-7</td>
<td>4</td>
</tr>
<tr>
<td>SPT-8</td>
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<td>3</td>
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</tr>
<tr>
<td>SPT-9</td>
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<td>7</td>
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</tr>
<tr>
<td>SPT-10</td>
<td>13</td>
<td>3</td>
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</tr>
</tbody>
</table>

**HAND PENETROMETER**

<table>
<thead>
<tr>
<th>N-Value (uncorrected) blows per foot</th>
<th>SYMBOL</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.0</td>
<td>2.0</td>
</tr>
</tbody>
</table>

**PROJECT NAME**  McCoppin Elementary School Improvements  
**PROJECT NUMBER**  608-5-1  
**PROJECT LOCATION**  651 6th Ave. San Francisco, CA  
**GROUND ELEVATION**  201 FT +/-  
**BORING DEPTH**  50 ft.  
**LATITUDE**  37.776533°  
**LONGITUDE**  -122.464539°  

**GROUND WATER LEVELS:**  
\(\checkmark\) **AT TIME OF DRILLING** Not Encountered  
\(\checkmark\) **AT END OF DRILLING** Not Encountered  

**NOTES**  
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<table>
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<tr>
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<th>SYMBOL</th>
<th>5½ inches Portland cement concrete</th>
</tr>
</thead>
<tbody>
<tr>
<td>201.0</td>
<td>0</td>
<td></td>
<td>Silty Sand (SM) [Fill] medium dense, moist, brown to reddish brown, fine to medium sand, some fine to coarse subangular gravel</td>
</tr>
<tr>
<td>198.0</td>
<td>3</td>
<td></td>
<td>Poorly Graded Sand with Silt (SP-SM) [Fill] loose, moist, brown to dark brown, fine to medium sand, some fine to coarse subangular gravel</td>
</tr>
<tr>
<td>193.5</td>
<td>5</td>
<td></td>
<td>Poorly Graded Sand (SP) medium dense, moist, brown, fine sand</td>
</tr>
<tr>
<td>173.0</td>
<td>25</td>
<td></td>
<td>loose</td>
</tr>
</tbody>
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**DESCRIPTION**

**UNCONFINED COMPRESSION**

<table>
<thead>
<tr>
<th>UNCONSOLIDATED-UNDRAINED TRIAXIAL</th>
<th>MC-2</th>
<th>119</th>
<th>10</th>
</tr>
</thead>
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<td>27</td>
<td>112</td>
<td>5</td>
</tr>
<tr>
<td>MC-6</td>
<td>12</td>
<td>SPT-7</td>
<td>4</td>
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<tr>
<td>SPT-8</td>
<td>20</td>
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<td>SPT-9</td>
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</tbody>
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**HAND PENETROMETER**

<table>
<thead>
<tr>
<th>N-Value (uncorrected) blows per foot</th>
<th>SYMBOL</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.0</td>
<td>2.0</td>
</tr>
</tbody>
</table>

**PROJECT NAME**  McCoppin Elementary School Improvements  
**PROJECT NUMBER**  608-5-1  
**PROJECT LOCATION**  651 6th Ave. San Francisco, CA  
**GROUND ELEVATION**  201 FT +/-  
**BORING DEPTH**  50 ft.  
**LATITUDE**  37.776533°  
**LONGITUDE**  -122.464539°  

**GROUND WATER LEVELS:**  
\(\checkmark\) **AT TIME OF DRILLING** Not Encountered  
\(\checkmark\) **AT END OF DRILLING** Not Encountered  

**NOTES**  
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<table>
<thead>
<tr>
<th>ELEVATION (ft)</th>
<th>SYMBOL</th>
<th>DESCRIPTION</th>
</tr>
</thead>
<tbody>
<tr>
<td>173.0</td>
<td></td>
<td>Poorly Graded Sand (SP)</td>
</tr>
<tr>
<td></td>
<td></td>
<td>medium dense, moist, brown, fine sand</td>
</tr>
<tr>
<td>30</td>
<td></td>
<td>very dense</td>
</tr>
<tr>
<td>40</td>
<td></td>
<td>dense</td>
</tr>
<tr>
<td>50</td>
<td></td>
<td>Bottom of Boring at 50.0 feet.</td>
</tr>
</tbody>
</table>

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.
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 27 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 seven (7) 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 test (ASTM D4318) was performed on a sample of the surficial soils 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. The test indicates that the soils are non-plastic. Test result is shown on the boring logs at the appropriate sample depth.

**Corrosion:** One soluble sulfate determination (ASTM D4327), one resistivity test (ASTM G57), one chloride determination (ASTM D4327), and one pH determination (ASTM G51) were performed on a sample of the subsurface soil. Results of these tests are attached in this appendix.
<table>
<thead>
<tr>
<th>Symbol</th>
<th>Boring No.</th>
<th>Depth (ft)</th>
<th>Natural Water Content (%)</th>
<th>Liquid Limit (%)</th>
<th>Plastic Limit (%)</th>
<th>Plasticity Index</th>
<th>Passing No. 200 (%)</th>
<th>Group Name (USCS - ASTM D2487)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>EB-1</td>
<td>2.0</td>
<td>7</td>
<td>determined non-plastic</td>
<td>4</td>
<td>Poorly Graded Sand (SP) [Fill]</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Sample prepared in accordance with ASTM D421