Geotechnical Investigation and Geologic Hazard Evaluation

John Yehall Chin Elementary School
San Francisco Unified School District
350 Broadway Street
San Francisco, California

Prepared for:
San Francisco Unified School District
San Francisco, California

Prepared by:
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November 2014

Project No. OD14170930
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Project OD14170930

Mr. Sajeev Madhavan
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Subject: Geotechnical Investigation and Geologic Hazard Evaluation

John Yehall Chin Elementary School
San Francisco Unified School District
350 Broadway Street
San Francisco, California 94114

Dear Mr. Madhavan:

AMEC Environmental & Infrastructure, Inc. (AMEC) is pleased to submit this geotechnical investigation and geologic hazard evaluation report to support the evaluation and design of improvements to the John Yehall Chin Elementary School. This report was developed in accordance with our Master Services Agreement with the San Francisco Unified School District, dated March 19, 2012 (Number 01473) revised on February 19, 2013 and Contract Modification No. (3), dated October 15, 2014.

Our investigation included compiling and reviewing existing data, performing a field investigation, performing geotechnical laboratory tests, performing engineering evaluations and analyses, assessing geologic hazards, developing geotechnical recommendations, and preparing this report.

If you have any questions about this report, please call any of the undersigned. It has been a pleasure working with you and we look forward to working with you on other future phases of the project.

Sincerely yours,

AMEC Environment & Infrastructure, Inc.

Joseph C. de Larios, PE, GE
Associate Engineer

Donald L. Wells, CEG
Senior Associate Engineering Geologist

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Principal Engineer

Enclosure
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John Yehall Chin Elementary School
San Francisco Unified School District
350 Broadway Street
San Francisco, California

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This report was prepared by the staff of AMEC under the supervision of the Engineer and/or Geologist whose seals and signatures appear hereon.

The findings, recommendations, specifications, or professional opinions are presented within the limits described by the client, in accordance with generally accepted professional engineering and geologic practice. No warranty is expressed or implied.

Donald L. Wells, CEG
Senior Associate Engineering Geologist

Joseph C. de Larios, PE, GE
Associate Engineer
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1.0 INTRODUCTION AND PURPOSE

This report presents the results of the geotechnical investigation and geologic hazard evaluation that AMEC Environment & Infrastructure, Inc. (AMEC) performed to support the design and construction of proposed improvements to the John Yehall Chin Elementary School (JYCES), San Francisco Unified School District (SFUSD). JYCES is located at 350 Broadway Street in San Francisco, California (Figure 1).

The primary objective of this geotechnical investigation and geologic hazard evaluation is to 1) assess the potential for seismic (earthquake-induced) hazards that could affect the project, and 2) develop geotechnical recommendations to support design and construction of proposed improvements to the main building. The seismic hazards addressed in this study are: (1) surface fault rupture, (2) liquefaction, (3) seismically-induced settlement, (4) seismically-induced landsliding, (5) seismically-induced inundation, and (6) soil swelling or shrinkage potential, and are discussed in Section 6. Geotechnical recommendations are discussed in Section 7.

This report was prepared in general accordance with the applicable requirements of 2013 California Administrative Code (CAC) Title 24, Part 1, Chapter 4 (California Building Standards Commission [CBSC], 2013b), and 2013 California Building Code (CBC) Title 24, Part 2 (CBSC, 2013a), for construction or alterations to public school buildings. The report also was prepared to provide relevant information specified on California Geological Survey Note 48, “Checklist for the Review of Engineering Geology and Seismology Reports for California Public Schools, Hospitals, and Essential Services Buildings.” We understand the report will be submitted to the Division of the State Architect (DSA), and their reviewer, the California Geological Survey (CGS).

2.0 EXISTING CONDITIONS AND PROJECT DESCRIPTION

JYCES is located at 350 Broadway Street, at the base of Telegraph Hill, in San Francisco, California (Figure 1). The site coordinates are latitude 37.7986° N and longitude 122.4031° W. The western portion of the site includes the Middle and Upper Yards (playgrounds), while the Main School Building occupies much of the eastern and central portions of the site (Figure 2).
The Main School Building is a three-story steel frame structure with brick masonry infill walls. The building was constructed in about 1913, and has been renovated several times for various use functions. The building is supported on a shallow foundation system that includes shallow spread footings (under building columns), which are typically connected to adjacent spread footings by retaining walls that also appear to function as continuous footings. Drawings showing the configuration of the existing footings are presented in Appendix C (drawings selected from the September 1913 construction drawings set).

Overall, the site topography slopes gently down from north to south, and west to east. It appears that the site has been terraced by cutting into the sloping terrain at the southeast base of Telegraph Hill. Retaining structures of various heights support the relatively level areas of the terraces. An access road/narrow parking area extends north from Broadway Street between the playgrounds and the Main School Building (Figure 2). A narrow lower yard that is elevated above the street, is located on the east side of the Main Building. It appears that some fill is present along the eastern portion of the site between the building and the east property line, where this lower yard is elevated above Broadway Street.

AMEC was not provided with a site topographic map. Based on review of publicly available large scale topographic maps, and our site reconnaissance, it appears that there is a difference of about 30 feet between the ground surface elevation on the sidewalk at the southeast corner of the site and the northwestern corner of the uppermost yard area. For the purpose of estimating the elevations, AMEC has assumed a project datum based on a ground surface elevation of 57 feet at the building entrance adjacent to the exterior exit staircase (near the southwest corner of the building).

The project team has not yet identified specific improvements for the building. However, it is our understanding that information on the subsurface conditions and the available bearing capacity of the existing foundations, and recommendations regarding modification of existing footings/construction of new shallow foundation elements is required for analysis and design of improvements for the building.

To provide information on subsurface conditions and samples for laboratory testing, a geologic and geotechnical investigation of the site was conducted on October 13, 2014 with two borings advanced in the west parking lot (between the upper playground and Main School building). The results of our site exploration, laboratory testing, geotechnical analysis, seismic hazards analyses, and recommendations for design and construction of improvements to the foundations are described in the following sections of this report.
3.0 SITE INVESTIGATION

The site investigation that was performed for the proposed improvements consisted of the following tasks:

- Site Reconnaissance
- Data Review
- Field Exploration
- Geotechnical Laboratory Testing

The site investigation methods are described in more detail below.

3.1 SITE RECONNAISSANCE

An AMEC representative visited the site on October 7, 2014. The purpose of the site reconnaissance was to review the site conditions and to identify visible signs of potential distress in the existing building and adjacent paved/terraced areas that may be related to foundation and subsurface conditions. In addition, sites for the proposed field exploration program, including consideration of equipment access restrictions were evaluated. Two proposed boring locations were marked in the western parking lot, on the west side of the Main School building. Underground Service Alert service was then notified of the proposed work and exploration locations.

On October 10, 2014, the proposed boring locations were screened for the presence of buried utilities by AMEC’s utility-locating subcontractor, Subtronic Corporation of Martinez, California.

3.2 DATA REVIEW

AMEC compiled and reviewed published sources of information and data to evaluate geologic and geotechnical conditions at the project site. These sources included geologic and fault maps prepared by the United States Geological Survey (USGS) and the California Geological Survey (CGS), and imagery available in Google Earth. Citations for the published sources are compiled in the References section (Section 9) at the end of this report.

3.3 FIELD EXPLORATION

On Monday, October 13, 2014, two borings, designated AB-1 and AB-2, were drilled to explore and sample the subsurface conditions at the site. Borings AB-1 and AB-2 were drilled to depths of about 9.5 and 17.5 feet below the ground surface (bgs), respectively, at the locations shown on Figure 2. The borings were advanced by HEW Drilling Company of Palo Alto, California with a hand auger and/or a truck-mounted drilling rig equipped with solid-stem flight augers. Boring AB-1 was advanced using only a hand auger. Bulk bag samples were collected from the hand auger bucket. Boring AB-2 was sampled at five foot intervals using Modified-California (Mod-Cal) and Standard Penetration (SPT) samplers driven with a 140-pound
hammer as described in Appendix A. Field logs were prepared describing the materials observed in the drilling spoil and samples.

Upon completion of drilling, borings were backfilled with neat cement grout in accordance with the exploration permit issued by the San Francisco Department of Public Health. Spoil generated during drilling (e.g., soil cuttings) was collected into a single 55-gallon drum. A composite sample of the drilling spoil was analyzed in the laboratory of Curtis & Tompkins, Ltd., of Berkeley, California. Analytical tests included total lead and total petroleum hydrocarbons. Based on the results of the analytical testing, the drum contents are judged to be “non-hazardous.” The drum is scheduled to be removed from the site, and the contents will be properly disposed by Advanced Environmental Services of Baker City, Oregon.

3.4 Geotechnical Laboratory Testing

The soil samples obtained from the borings were delivered to AMEC’s Oakland, California warehouse for further examination. Selected samples were transported to Cooper Testing Labs of Palo Alto, California for geotechnical laboratory testing, and tests were performed on selected soil samples to evaluate their physical characteristics and engineering properties. Samples were tested for dry density, moisture content, Atterberg limits, and percentage of fines. The laboratory testing program is described, and graphic presentations of test results are presented in Appendix B.

Boring logs were prepared based on the field logs, subsequent examination of the samples obtained from the borings, and laboratory test results. The laboratory testing results are noted on the boring logs, which are presented in Appendix A. Results of moisture content, dry density, Atterberg limits, and percent fines are also indicated at the corresponding sample locations on the boring logs in Appendix A.

In addition, a suite of analytical tests to assess the corrosivity potential of the near-surface soils were performed on a soil sample from Boring AB-1. The results of the tests, which were performed using standard California Department of Transportation (Caltrans) California Test Methods, are presented in Appendix B.

4.0 Geologic, Tectonic, and Seismic Setting

The regional geologic and tectonic setting and the seismic setting are described in the following sections.

4.1 Regional Geologic Setting

The City of San Francisco comprises a series of rolling hills and valleys bounded by the Pacific Ocean to the west, San Francisco Bay to the east, and the entrance of the bay (the Golden Gate) to the north. The city lies along the north end of the peninsula that forms the southwest
margin of the large basin that comprises the San Francisco Bay area. The basin is filled with Quaternary alluvial, fluvial, and estuarine deposits overlying older basement rocks. The oldest rocks in the San Francisco Bay area belong to the Franciscan Complex of Jurassic to Cretaceous age (200 to 66 million years old). These rocks form the basement complex beneath the basin and are exposed along the margins of the basin. The Franciscan Complex consists primarily of highly deformed graywacke (sandstone), siltstone, and claystone (shale), with minor amounts of greenstone, chert, metavolcanics, limestone, and conglomerate. Extensive shearing along ancient faults is characteristic of the Franciscan Complex, and rock types of widely variable physical properties commonly are juxtaposed (Bailey et al., 1964). The hills of San Francisco are composed of Franciscan Complex sandstone, siltstone, claystone, and serpentinite, while the valleys are filled with late Pleistocene-Holocene sediments (less than 120,000 years old) overlying the Franciscan Complex rocks (Schlocker, 1974; Blake et al., 2000; Figure 4)).

Locally, Pleistocene aged (2.6 million [Ma] to 10,000 years ago) slope debris and ravine fill (colluvium), along with undifferentiated sedimentary deposits, constitute the near-surface site soils as shown on Figure 4. These deposits contain varying amounts of clay, and typically range from clayey sands to lean clays. Bedrock beneath the Pleistocene deposits consists of the Cretaceous age Franciscan Complex (~145 to 66 Ma). In the vicinity of the site, materials of the Franciscan Complex have been mapped as massive sandstone, and sandstone and shale.

4.2 **TECTONIC SETTING AND REGIONAL FAULTS**

Seismicity in the San Francisco Bay Area is related to activity on the San Andreas fault system. The entire region is located within a very broad belt of right-lateral shear marking the plate boundary zone between the North American and Pacific Plates, and it is dominated by strike-slip faulting. Numerous major active faults near San Francisco that may be sources of large earthquakes include the San Andreas, Hayward-Rodgers Creek, and San Gregorio fault zones (Figure 5). The San Andreas fault zone trends north-northwest along the San Francisco Peninsula, extends off shore at Daly City and west of the Golden Gate at the entrance to San Francisco Bay, and onshore between Bolinas and Tomalas Bays, bounding the Point Reyes Peninsula. The closest approach of the San Andreas fault zone is approximately 8.3 miles (13.4 km) west of the site. The Hayward fault zone, which also trends north-northwest, is located approximately 9.7 miles (15.6 km) east of the site. The San Gregorio fault zone, located offshore of the San Francisco Peninsula and trending north-northwest to intersect the San Andreas near Bolinas Bay, is about 12 miles (19 km) west of the site. Other active faults located farther from San Francisco that may be sources for future earthquakes include the Rodgers Creek, Concord-Green Valley, Calaveras, and Mt. Diablo faults zones (Figure 5). The
distance, direction, characteristic (maximum) earthquake, and slip rate for the major active faults located near the site are listed in Table 1.

4.3 Historical Seismicity

During the past 200 years, numerous damaging earthquakes have occurred along the San Andreas and other faults near San Francisco. The first reported earthquake to have affected San Francisco had a magnitude of approximately 5.5 (estimated from descriptions of felt intensities), occurring on the Peninsula segment of the San Andreas fault zone in 1808. The Peninsula and Santa Cruz Mountains segments of the San Andreas fault zone extend from approximately San Juan Bautista to offshore of the Golden Gate; the North Coast segment extends northward to Cape Mendocino. The first significant earthquake reported to have affected San Francisco had a magnitude in the range of 7.0 to 7.5, and occurred on the Peninsula segment of the San Andreas fault in 1838 (Toppozada and Borchardt, 1998). A series of smaller but damaging earthquakes between 1850 and 1865 affected San Francisco, with the 1865 shock centered near the Santa Cruz Mountains being the most severe (Townley and Allen, 1939). The 1868 Hayward earthquake ruptured the southern Hayward fault, had an estimated magnitude of 6.9, and damaged or destroyed numerous buildings in Hayward, San Leandro, and San Francisco (Lawson, 1908). The 1838 and 1868 earthquakes resulted in damage in San Francisco corresponding to Modified Mercalli Intensity (MMI) VIII (partial damage to buildings, walls). A magnitude 6.2 (estimated from felt intensities) earthquake in 1898 was centered near Mare Island, damaged buildings in Sonoma County, and was felt in San Francisco with MMI VII effects (Toppozada et al., 1981; Ellsworth, 1990).

During the moment magnitude (Mw) 7.9 1906 San Francisco earthquake, the North Coast, Peninsula, and Santa Cruz Mountains segments of the San Andreas fault zone ruptured over a distance of about 296 miles (474 km) from Shelter Cove near Cape Mendocino southward to near San Juan Bautista. Maximum lateral displacements of 15 to 20 feet (4.6 to 6.1 meters) occurred north of the Golden Gate at Olema in Marin County (Lawson, 1908). Landslides, liquefaction, and ground settlement occurred throughout the Bay Area and in the vicinity of the surface rupture as a result of this earthquake. The ground shaking in San Francisco during the 1906 earthquake is characterized as MMI IX (major damage to and collapse of structures; Lawson, 1908; Boatwright and Bundock, 2005). The 1906 earthquake and resulting fire destroyed a significant portion of San Francisco. The most severe shaking and damage occurred in low-lying areas along former creek basins and in areas of fill along the waterfront. Ground failure effects, including liquefaction, lateral spreading, and settlement occurred in association with the buried creeks and filled areas, such as the reclaimed areas of Mission Bay.
More recent earthquakes in the region that were felt in San Francisco include: the 1957 Daly City earthquake on the San Andreas fault zone (ML 5.3)\(^1\); the two Santa Rosa earthquakes of 1969 on the Healdsburg-Rodgers Creek fault zone (ML 5.6 and 5.7); the Coyote Lake and Morgan Hill earthquakes of 1979 and 1984 on the Calaveras fault zone (ML 5.9 and 6.2, respectively); the 1980 Livermore earthquake on the Greenville fault zone (ML 5.8); the 1989 MW 7.0 Loma Prieta earthquake on the San Andreas fault zone or a parallel subsidiary fault; the 1999 ML 5.0 earthquake near Bolinas on the San Andreas fault zone; the October 30, 2007 MW 5.4 Alum Rock earthquake near Fremont on the Calaveras fault zone; and the MW 6.0 August 28, 2014 South Napa earthquake on the West Napa fault zone. The 1989 Loma Prieta earthquake caused significant damage to structures in the City of Santa Cruz, the Santa Cruz Mountains, and in more distant areas of fill and soft soils such as the Marina district of San Francisco. Little damage occurred to structures founded on rock or stiff alluvium in San Francisco or other Bay Area communities as a result of the more recent earthquakes, with the exception of the recent South Napa earthquake. The South Napa earthquake resulted in significant structural damage to un-reinforced masonry buildings and some wood frame buildings in the City of Napa, and significant non-structural damage to residential and commercial facilities across the Napa Valley. Surface fault rupture occurred along several traces of the West Napa fault over a distance of about 7 to 9 miles (12 to 15 km), resulting in damage to residential structures and roads traversed by the rupture (Bray et al., 2014).

With the exception of the 1989 Loma Prieta earthquake, ground shaking experienced in the JYCES vicinity from recent historic earthquakes has been of generally imperceptible to light amplitude and produced effects observed in San Francisco that may be categorized as MMI I through V (MMI VI corresponds to the lowest intensity level with which some damage (slight) is associated, although fragile contents may be broken at MMI V\(^2\)). The previously-mentioned Loma Prieta earthquake ruptured on or near the southern portion of the Peninsula segment and the Santa Cruz Mountains segment of the San Andreas fault zone approximately 45 miles (70 km) south of San Francisco and produced MMI VII effects in the vicinity of the JYCES (McNutt and Toppozada, 1990).

Several strong ground motion records for the Loma Prieta earthquake were obtained at ground sites near JYCES as shown in the table below, with a peak ground acceleration (PGA) of 0.08 g for the closest station (on Franciscan rock on Stockton Street on the northwest side of

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\(^1\) ML – Local or Richter magnitude; MW – Moment magnitude.

\(^2\) MMI V is described as follows: “Felt outdoors; direction estimated. Sleepers wakened. Liquids disturbed, some spilled. Small unstable objects displaced or upset. Doors swing, close, open. Shutters, pictures move. Pendulum clocks stop, start, change rate.”
Telegraph Hill), and a maximum PGA of 0.21 g at a site in the Presidio (on serpentine). Most stations on rock sites in San Francisco had a PGA in the range of 0.06 to 0.11 g\(^3\).

<table>
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<th>Peak Ground Acceleration (g, direction)</th>
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<td>Telegraph Hill, San Francisco (58133)</td>
<td>Franciscan sandstone, shale</td>
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<td></td>
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<td>Serpentine rock</td>
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<td>Franciscan rock</td>
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<td>0.06 E-W</td>
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<tr>
<td></td>
<td></td>
<td>6-story school, Parnassus, San Francisco (58479)</td>
<td>Franciscan rock</td>
<td>58</td>
<td>0.09 N-S</td>
</tr>
</tbody>
</table>

The U.S. Geological Survey (USGS) Working Group on California Earthquake Probabilities (WGCEP, 2008) estimated a 63 percent probability that at least one major earthquake (moment magnitude \(M_W \geq 6.7\)) will occur in the San Francisco Bay Area before 2037\(^4\). That study indicates there is a high probability that ground motions stronger than those recorded during the 1989 Loma Prieta earthquake will occur at the school site during the next 30 years.

### 5.0 SITE CONDITIONS

This section describes both surface and subsurface characteristics of the site. The subsurface conditions presented below are based on the results of the field exploration conducted at the site by AMEC at the boring locations shown on Figure 2. Logs of these subsurface explorations are provided in Appendix A.

#### 5.1 SURFACE CONDITIONS

John Yehall Chin Elementary School is located in an urban setting, on an artificially terraced site in gently sloping terrain. The site is covered by the existing school building and by asphalt paving on the upper, middle, and lower play yards and the lower parking lot (Figure 3). Most

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\(^3\) Sources of data include: California Strong Motion Instrumentation Program data included in Earthquake Spectra May 1990, Supplement to vol. 6; COSMOS website (URL: http://db.cosmos-eq.org/scripts/default.plx); and U.S. National Center for Engineering Strong Motion Data (URL: http://www.strongmotioncenter.org/).

surface drainage at the site is directed to storm drains within paved areas. No springs, seeps, surfi
cicial mineral deposits, or other evidence of groundwater surfacing were observed during our site reconnaissance. Rock outcrops or native soil exposures were not observed at the site.

Retaining walls of varying heights have been constructed around perimeter of the site and at several locations within the property to provide relatively level areas. In the middle of the site, an approximately 3-foot tall retaining wall separates the upper play yard from the parking lot and school building. No evidence of significant settlement or settlement effects was observed in the school structure or any of the adjacent retaining structures at the time of AMEC’s site reconnaissance and field investigation.

5.2 Subsurface Conditions

In Boring AB-1, very stiff to hard sandy clays were encountered in about the upper 9 feet, overlying hard sandy clay (weathered bedrock) to the bottom of the boring at 9½ feet bgs. In Boring AB-2, very stiff to hard sandy clays were encountered in the upper 10½ feet, overlying hard sandy clay (weathered bedrock). At about 15 feet bgs, more indurated bedrock was encountered (to the maximum depth explored of about 17¾ feet). Given the proximity of the site to Telegraph Hill, these upper 10 feet of materials may be colluvium (slope wash) shed from the nearby topographic high point, with a very thin veneer (less than 2 feet) of fill derived from the native colluvium.

Mapping by Blake et al. (2000) indicates that the site is underlain by Pleistocene colluvium (slope wash and ravine fill) and Franciscan bedrock. Based on boring logs from our geotechnical exploration (Appendix A), the site is predominantly underlain by sandy clay and weathered bedrock to the explored depths, which is in accord with the regional geologic mapping (Figure 4).

5.3 Groundwater Conditions

During our exploration, free groundwater was not observed in either boring. This is consistent with mapping of the highest historical groundwater level by the CGS (2000), which shows that the site is underlain by bedrock and that groundwater is deeper than 10 feet bgs.

Although fluctuations in the groundwater level may occur due to variations in rainfall, temperature, and other factors, the available information shows that the highest groundwater levels are at or below the top of bedrock at the site.

6.0 Seismic-Geologic Hazards

This section provides an assessment of the earthquake-related geologic and geotechnical hazards for the site, including the potential for surface fault rupture; liquefaction; seismically induced settlements; seismically induced landsliding; inundation due to tsunami, seiche, or
seismically induced failure of water-retention facilities; and soil swelling or shrinkage potential. The hazard evaluation methodology involves one or two steps. First, the potential for occurrence of each type of geologic hazard is assessed. If there is potential for a hazard to occur, the second step is to assess whether the hazard will impact the proposed project improvements. For this evaluation, a significant hazard is defined as one that results in structural damage and threatens life-safety in an earthquake.

6.1 SURFACE FAULT RUPTURE

Earthquakes generally are caused by a sudden slip or displacement along a zone of weakness, termed a fault, in the Earth's crust. Surface fault rupture, which is a manifestation of the fault displacement at the ground surface, usually is associated with moderate- to large-magnitude earthquakes (magnitudes of about 6 or larger) occurring on active faults having mapped traces or zones at the ground surface. The amount of surface fault displacement can be as much as 20 feet or more, depending on the earthquake magnitude and other factors. The displacements associated with surface fault rupture can have devastating effects on structures and lifelines situated astride the zone of rupture.

No active or potentially active faults have been identified in the immediate vicinity of the site according to maps published by the California Geological Survey (Jennings and Bryant, 2010; Bryant and Hart, 2007; Figure 5). The nearest known active fault is the San Andreas fault zone, which is located approximately 8.3 miles (13.4 km) west of the site (Figure 4). The San Gregorio and Hayward fault zones are located approximately 11.7 and 9.7 miles (18.8 and 15.6 km), respectively, from of the site (Figure 4). Therefore, we conclude that the potential for surface fault rupture at the site is very low.

6.2 LIQUEFACTION

Liquefaction is a soil behavior phenomenon in which a soil located below the groundwater surface loses a substantial amount of strength due to strong earthquake ground shaking. Some types of soil tend to compact during earthquake shaking, inducing excess pore water pressure in the saturated soil, which, in turn, causes a reduction in strength of the soil. Recently deposited (i.e., geologically young) and relatively loose natural soils, and uncompacted or poorly compacted fills are potentially susceptible to liquefaction. Loose sands are particularly susceptible. Loose silts and gravel also have potential for liquefaction. Dense natural soils and well-compacted fills have low susceptibility to liquefaction. Clayey soils and bedrock generally are not susceptible to liquefaction.

The site is not within a mapped liquefaction hazard zone according to CGS (2000) or an area where the soils are mapped as being susceptible to liquefaction (Knudsen et al., 2000; Figure 6). As discussed in Section 5.2, the soils encountered at the site are dominantly clayey, contain varying amounts of fine to medium sand, and overlie weathered bedrock at depths of
about 9 to 10½ feet bgs. The soils were generally very stiff to hard, and groundwater was not encountered at either boring. Very stiff to hard clayey soils and bedrock are not susceptible to liquefaction, even if saturated. Based on this information, we conclude that the potential for liquefaction is very low and that there is no significant hazard associated with liquefaction at the site.

6.3 **Seismically-Induced Settlement**

Strong earthquake ground shaking can cause densification of loose soil, resulting in settlement. Seismic densification of loose soil above the groundwater table occurs suddenly, during ground shaking. Seismic densification of loose soil below the groundwater table may occur following liquefaction, occurring with time as excess pore water pressures in the soil dissipate after ground shaking ceases. If densification does not occur uniformly over an area, the resulting differential settlement can damage to structures supported on the loose soil.

The subsurface data available for the site (as described in Section 5.2), indicate the upper 10 feet of soils consist mainly of very stiff to hard sandy clay materials, overlying weathered bedrock. We did not observe signs of settlement adjacent to the existing structures at the time of AMEC’s reconnaissance and site investigation.

The available foundation plans (Appendix C) indicate that embedment for the existing spread footings ranges from about 4 to as much as about 12 feet below the adjacent exterior ground surface. Based on the embedment depth shown on the foundation plans, it appears that the existing footings are founded on hard colluvial soils or on weathered bedrock. Therefore, it is our opinion that the potential for significant seismically-induced settlement is low. The magnitude of future potential seismically-induced settlement, if any, is not expected to be large enough to impact design or performance of the proposed project improvements.

6.4 **Seismically-Induced Landsliding**

Earthquake ground shaking can reduce the stability of a slope and cause sliding or failure of the soil or rock materials composing the slope. During ground shaking, seismic inertia forces are induced within the slope, increasing the loads that the slope materials must sustain to resist landsliding (or rockfalls). If the forces tending to cause landsliding exceed the strength of the materials resisting landsliding, a temporary instability is created that is manifested by lateral or downslope displacement of the slope materials. In some cases, strong ground shaking can also reduce the strength of the soil or rock materials, reducing their ability to resist the forces that cause landsliding.

The site is not mapped within an area (zone) with potential for earthquake-induced landsliding (CGS, 2000; Figure 6). As discussed in Section 3, the site appears to have been terraced by cutting into sloping terrain. Although the mapping by CGS (2000) shown on Figure 6 indicates...
there is potential for slope instability in areas along Telegraph Hill north and west of the site, the adjacent properties north and west of the site are largely covered by buildings and supported by retaining structures, such that there are no slopes in the vicinity of the site that could fail onto, or project toward the site. There also are no slopes in the site vicinity that could fail in a “flowing” manner and approach the school. Therefore, it is our opinion that the potential for seismically-induced landslides at the site is very low, and that there is no significant seismically-induced landslide or slope instability hazard for the existing building at the site.

6.5 Seismically-Induced Inundation

Seismically-induced inundation can result in a variety of phenomena, including tsunami waves, seiche waves, or flooding resulting from seismically-induced failure of water-retention facilities. Tsunamis are either ocean waves generated by vertical seafloor displacements associated with large offshore earthquakes or waves in any body of water that result from rapid landsliding into the water or rapid landsliding of slopes covered by the body of water. Seiches are waves associated with the oscillating surface of an enclosed or partly enclosed body of water caused by interaction of the water body with arriving seismic waves. Flooding and acute erosion may result from seismically-induced failure of levees or water-retention facilities such as dams, reservoirs, or tanks upstream or upslope of a site. The degree of flooding is associated with the volume of water released, the proximity of the source to the site, topography of the site, and other factors.

The site is located at elevations of roughly 50 feet to 80 feet (assumed project datum). The site is located approximately 6.4 miles from the Pacific Coast to the west and 0.3 miles from the San Francisco Bay to the east. The site is not mapped as a potential tsunami inundation area by the State of California (2009), is not located adjacent to or downstream of, water-retention facilities or drainage channels, and is not located within a FEMA 100-year floodplain. Therefore, it is our opinion that the potential for seismically-induced inundation is very low, and that there is no significant hazard associated with seismically-induced inundation at the site.

6.6 Soil Swelling or Shrinkage Potential

The subsurface soils consist predominantly of very stiff to hard, sandy clays. An Atterberg Limits test on a clayey soil sample taken from about 1 to 2 feet bgs in Boring AB-1 indicates a Plasticity Index (PI) of 15, which is considered to be on the borderline between “low” and “moderate” expansion potential. It is our judgment that the hazard to seismic retrofit elements associated with potential swelling or shrinkage of these soils is low. If facilities sensitive to shrink/swell behavior are planned, mitigation measures may be warranted.
7.0 GEOTECHNICAL RECOMMENDATIONS

This section discusses geotechnical recommendations for design and construction, including considerations for new foundations and minor earthwork. As described in Section 5, the building site is underlain by slope debris (slope wash) and ravine fill, consisting primarily of very stiff to hard, sandy clays and medium dense to dense, clayey sands, and weathered bedrock. Groundwater is estimated to be 15 feet or deeper bgs.

The key geotechnical considerations for design and construction are discussed in the following sections.

7.1 FOUNDATION RECOMMENDATIONS

The following recommendations may be used for evaluation of the existing foundations or for the design of new shallow foundations (spread and continuous footings).

Materials interpreted as fill were encountered in the uppermost portion of each boring drilled for this project. Based on our observation of the drilling and visual examination of the drill cuttings, it is our interpretation that native materials were encountered at about two feet below the ground surface (bgs) in both borings. The transition from colluvial materials to weathered bedrock was observed at about 9 to 10 feet bgs. As noted in Section 6.3, based on our review of the foundation drawing for the original structure, it is our judgment that the existing building foundations are embedded in native materials consisting of hard sandy clays and possibly weathered bedrock.

The estimated “ultimate,” net bearing capacity on undisturbed native soils or weathered bedrock can be taken as 15,000 pounds per square foot (psf). A factor of safety of 3.0 should be used to determine the allowable bearing stress for dead loads; a factor of safety of 2.0 should be used to determine the allowable bearing stress for dead plus long term live loads; and for total loads, including wind and seismic, a factor of safety of 1.5 should be used. Using the recommended factors of safety, footings bearing on undisturbed native soils or weathered bedrock may be designed using the following allowable, net bearing capacities:

- Dead Load 5,000 psf
- Dead plus long-term live load 7,500 psf
- Total loads including seismic 10,000 psf

These bearing capacities are net values; therefore, the weight of the foundations may be neglected for design purposes. These bearing pressure values assume that the shallow foundations (spread and continuous footings) are a minimum of 1½ feet wide and are embedded a minimum of 2 feet into native materials.
The unit modulus of subgrade reaction \( (K_w) \) for shallow foundations can be calculated using Equation 8-11 in Method 3 of American Society of Civil Engineers/Structural Engineers Institute (ASCE/SEI) Standard 41-13 (ASCE, 2014). For the purposes of calculating the modulus of subgrade reaction using Equation 8-11 in ASCE/SEI 41-13, we recommend using the following input parameters:

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Small Strain Loading</th>
</tr>
</thead>
<tbody>
<tr>
<td>( G ), best estimate (psf)</td>
<td>7,500,000</td>
</tr>
<tr>
<td>( G ), lower bound (psf)</td>
<td>3,750,000</td>
</tr>
<tr>
<td>( G ), upper bound (psf)</td>
<td>15,000,000</td>
</tr>
<tr>
<td>( \nu ) (Poisson's ratio)</td>
<td>0.4</td>
</tr>
<tr>
<td>( B ), maximum (feet)</td>
<td>20</td>
</tr>
</tbody>
</table>

Appropriate conversion relationships should be applied to the unit subgrade modulus to account for the tributary area for each spring in the structural model.

New footings located adjacent to existing footings or utility trenches should have their bearing surfaces situated below an imaginary surface projected 1½\(H\):1V (horizontal-to-vertical) upward from the bottom of the adjacent footing or utility trench.

### 7.2 Resistance to Lateral Loads

Mobilized passive resistance of adjacent soil acting against the vertical faces of foundation elements is a function of the height of the foundation element, the deflection of that element, the location of the groundwater table, and the properties of the adjacent subsurface materials. The recommendations in this section assume that foundation elements are embedded in native materials (i.e., hard sandy clays and weathered bedrock).

For static design, passive resistance should be neglected. For seismic design, the passive soil resistance against the footing faces may be added to the frictional resistance on the base of the footings to determine the total lateral capacity of the foundation system. Lateral resistance due to base friction can be determined by using an ultimate coefficient of sliding resistance of 0.40.

For foundation elements embedded in native materials (i.e., hard sandy clays and weathered bedrock), the “ultimate” passive soil resistance against the faces of the footings will be developed under lateral translation. For the purpose of design, the “ultimate” passive pressure can be assumed to be mobilized at a lateral displacement of approximately 2 percent of the footing face depth [where the footing face depth is measured from the top of subgrade to the bottom of footing]). This assumes that the footing excavation sidewalls remain stable during construction and that the foundation concrete is poured neat against the soil (i.e., no formwork).
We recommend using an ultimate passive soil resistance of 900 pounds per cubic foot, expressed as equivalent fluid unit weight. The passive resistance of the upper 12 inches of soil should be neglected in locations that are not covered by surface pavement. Ultimate passive pressure should be capped at a maximum value of 4,000 psf.

The normalized passive pressure mobilization values in the table below assume that maximum passive pressure is mobilized at a horizontal deflection equal to 2 percent of the height of the corresponding foundation element. Because the magnitude of actual mobilized passive pressure will vary as a function of the height of the corresponding foundation elements, the displacement compatibility of passive pressures between foundation elements should be considered in the structural model.

At deflections less than 2 percent of the footing face depth, the following table may be used:

<table>
<thead>
<tr>
<th>( \frac{y}{y_{\text{ult}}} )</th>
<th>( \frac{p}{p_{\text{ult}}} )</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.00</td>
<td>0.00</td>
</tr>
<tr>
<td>0.01</td>
<td>0.20</td>
</tr>
<tr>
<td>0.02</td>
<td>0.43</td>
</tr>
<tr>
<td>0.04</td>
<td>0.56</td>
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<tr>
<td>0.07</td>
<td>0.73</td>
</tr>
<tr>
<td>0.15</td>
<td>0.85</td>
</tr>
<tr>
<td>0.37</td>
<td>0.95</td>
</tr>
<tr>
<td>0.74</td>
<td>0.99</td>
</tr>
<tr>
<td>1.00</td>
<td>1.00</td>
</tr>
<tr>
<td>&gt;1</td>
<td>1.00</td>
</tr>
</tbody>
</table>

Where:

\[ y = \text{lateral deflection of pile cap}, \]
\[ y_{\text{ult}} = \text{lateral deflection at ultimate load}, \]
\[ p = \text{load on pile cap}; \text{ and} \]
\[ p_{\text{ult}} = \text{ultimate passive soil resistance} \]

The above values for lateral resistance do not contain a safety factor (i.e., the base friction and passive resistance values provided above are ultimate values). A factor of safety of 2.0 is recommended to determine the allowable lateral resistance for dead plus long term live loads. For total loads, including wind and seismic, a factor of safety of 1.5 should be used.
7.3 Retaining Walls

The recommendations below are intended for new, cast-in-place retaining structures within and adjacent to the existing building. Assessment of the conditions behind existing, site perimeter retaining walls was not included in the scope of AMEC’s study.

7.3.1 New Retaining Walls

Unrestrained, cantilever walls should be designed to resist an active equivalent fluid pressure of 35 pcf. Restrained, braced walls should be designed to resist an at-rest equivalent fluid pressure of 50 pcf. These values assume level backfill and that the retaining wall backfill will be capped with an impervious cover (i.e., asphalt or concrete) to inhibit water infiltration and prevent the buildup of hydrostatic pressures.

For unrestrained, cantilever walls over 6 feet in net retained height, seismically induced earth pressure can be estimated using a seismic increment of 20H psf where H is the height of the wall in feet. For restrained, braced walls over 6 feet in net retained height, seismically induced earth pressure can be estimated using a seismic increment of 10H psf. The pressure induced by this additional force can be approximated by a uniform distribution along the height of the wall. The seismic earth pressure is in addition to the active or at-rest lateral earth pressure provided above.

Passive pressure acting against the vertical faces of retaining wall foundations can develop lateral load resistance in proportion to the magnitude of the deflection. Refer to Section 7.2 for additional discussions regarding passive resistance.

7.4 Seismic Design Parameters

The seismic design for this project will be in accordance with 2013 CBC (CBSC, 2013a), which incorporates by reference the seismic design procedures of ASCE/SEI Standard 7-10 (ASCE/SEI, 2010). Seismic design parameters utilized by the 2013 CBC and ASCE/SEI 7-10 for new construction correspond to two levels of ground motion, the Maximum Considered Earthquake (MCE) and the Design Level. The intensity and characteristics of these ground motions are based on the location of the site relative to potential sources of earthquakes in the site region and on subsurface conditions at the site. Based on the expected ground motion intensity and site class designation, site coefficients are defined to account for site response effect in establishing the MCE and DE seismic design parameters appropriate to the site.

7.4.1 Site Classification for Seismic Design

Based on our review of local geologic information and logs of borings performed on the project site, we note that the subsurface conditions at the site consist of up to about 10 feet of very stiff to hard clayey and sandy soils overlying weathered Franciscan sandstone bedrock. In the
absence of measured shear wave velocity data at the site to confirm the shear wave velocity of the bedrock, we recommend using a Site Class C, which corresponds to very dense soil and soft rock (ASCE/SEI 7-10, Chapter 20), for characterizing potential earthquake ground shaking conditions and seismic design considerations.

### 7.4.2 MCE and Design Level Seismic Parameters

In accordance with the 2013 CBC (1613A), the following seismic design parameters may be used for the project. The values of \( S_S \), \( S_1 \), \( F_a \), and \( F_v \) used in development of the site-adjusted MCE spectral parameters \( S_{MS} \) and \( S_{M1} \) are listed below. The values of \( S_S \) and \( S_1 \) were obtained from the USGS online tool, U.S. Seismic Design Maps (http://earthquake.usgs.gov/hazards/designmaps/usdesign.php). The values of \( F_a \) and \( F_v \) are for Site Class C. The spectral acceleration parameters \( S_{DS} \) and \( S_{D1} \) are equal to two-thirds of \( S_{MS} \) and \( S_{M1} \), respectively.

For use with the equivalent lateral force procedure, the mapped value of \( S_1 \) from ASCE/SEI 7-10 and the 2013 CBC is 0.600g, and the mapped long period transition period (\( T_L \)) is equal to 12 seconds.

The project is identified as Risk Category III (2013 CBC 1604A.5), and for the mapped \( S_1 \) value of 0.600g, the Seismic Design Category is D (2013 CBC 1613A.3.5).

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Site Classification</td>
<td>C</td>
</tr>
<tr>
<td>Mapped MCE Spectral Acceleration for Short Periods, ( S_s )</td>
<td>1.500g</td>
</tr>
<tr>
<td>Mapped MCE Spectral Acceleration for Period of 1.0 second, ( S_1 )</td>
<td>0.600g</td>
</tr>
<tr>
<td>Site Coefficient, ( F_a )</td>
<td>1.00</td>
</tr>
<tr>
<td>Site Coefficient, ( F_v )</td>
<td>1.30</td>
</tr>
<tr>
<td>Adjusted MCE Spectral Acceleration for Short Periods, ( S_{MS} )</td>
<td>1.500g</td>
</tr>
<tr>
<td>Adjusted MCE Spectral Acceleration for Period of 1.0 Second, ( S_{M1} )</td>
<td>0.780g</td>
</tr>
<tr>
<td>Design Spectral Acceleration for Short Periods, ( S_{DS} )</td>
<td>1.000g</td>
</tr>
<tr>
<td>Design Spectral Acceleration for Period of 1.0 Second, ( S_{D1} )</td>
<td>0.520g</td>
</tr>
<tr>
<td>Long-Period Transition, ( T_L ) (seconds)</td>
<td>12</td>
</tr>
<tr>
<td>Risk Category</td>
<td>III</td>
</tr>
<tr>
<td>Seismic Design Category</td>
<td>D</td>
</tr>
</tbody>
</table>
7.4.3 Seismic Parameters for Liquefaction, Settlement, and Seismic Earth Pressures

The ground motion required by the 2013 CBC for assessment of liquefaction and seismically-induced settlement is the geomean MCE peak ground acceleration (PGA_{M}). The PGA_{M} from the 2013 CBC and ASCE/SEI 7-10 is 0.508g.

7.5 EARTHWORK

Earthwork may be necessary for improvements to the existing building foundation and retaining walls or for construction of new shallow foundations and retaining walls. Because the nature of specific improvements is not yet known, we provide the following recommendations for shallow footing foundations for potential modifications to existing foundation elements and construction of new foundation elements (including retaining walls).

The work areas should initially be cleared of any existing foundation elements, slabs, pavements, retaining walls, and debris, and these materials should be removed from the site. The work areas should then be stripped to sufficient depth to remove any surface vegetation and/or weeds that may be present and these materials should also be removed from the site.

After the work areas are cleared and stripped, the excavation for the new foundations can be made. Temporary excavations should be designed, planned, constructed, and maintained by the Contractor and should conform to current CAL-OSHA requirements for worker safety and all state and/or federal safety regulations and requirements. For planning purposes, we recommend that temporary cuts at the site be made such that they do not exceed a maximum inclination of 1\(\frac{3}{4}\)H:1V (horizontal-to-vertical). It may be possible that temporary cut slopes may need to be cut flatter based on the actual conditions found at the time of construction. Existing footings, utilities, or other improvements situated above an imaginary surface extending upward at a 1\(\frac{1}{2}\)H:1V (horizontal-to-vertical) slope from bottom nearest edge of the excavation should be underpinned. The contractor should be responsible for the evaluation and design of any underpinning and ground support measures.

Native, on-site soils having an organic content of less than 3 percent by volume are suitable for use as fill. Any fill placed at the site should not contain rocks or lumps greater than 4 inches in greatest dimension, and no more than 15 percent of the fill particles should be larger than 2\(\frac{1}{2}\) inches in greatest dimension. Any imported fill material used at the site should be non-expansive with a plasticity index of 15 or less. Import fill should also have more than 5, and less than 50 percent passing the No. 200 sieve, and no more than 15 percent of the fill particles should be larger than 2\(\frac{1}{2}\) inches in greatest dimension.

Fill and backfill should be moisture conditioned to between 0 and 3 percent above optimum moisture content and compacted to at least 90 percent relative compaction (as determined by
Fill and backfill should be placed on a firm, unyielding surface in horizontal lifts that do not exceed 8 inches before being compacted.

Subgrade soils within areas to receive fill and below exterior slabs-on-grades (i.e., sidewalks, walkways, etc.) subject to should be scarified to a minimum depth of 8 inches, moisture conditioned to between 1 and 3 percent above optimum moisture content, and recompacted to at least 90 percent relative compaction (as determined by ASTM Test Method D1557-12).

Subgrade soils below pavements, driveways, or any other flatwork/slabs/pavers subject to vehicle loads should be scarified to a minimum depth of 8 inches, moisture conditioned to between 1 and 3 percent above optimum moisture content, and recompacted to at least 95 percent relative compaction (as determined by ASTM Test Method D1557-12).

Aggregate base materials that may be used for reconstructing pavements or below exterior slabs-on-grades should meet the requirements in the 2010 Caltrans Standard Specifications, Section 26 for Class 2 Aggregate Base (¾-inch maximum particle size). Aggregate base should be uniformly moisture conditioned to a moisture content of 1 to 3 percent above optimum and compacted to at least 95 percent relative compaction as determined by ASTM Test Method D1557-12. The material should be placed in horizontal lifts that do not exceed 8 inches before being compacted.

7.6 *CORROSIVE REACTIVE GEOCHEMISTRY*

As discussed above and in Appendix B, a suite of analytical tests was performed on a sample of near-surface soil to assess corrosion potential of foundation elements embedded in site soils. Tests were performed using standard California Department of Transportation (Caltrans) California Test Methods. Caltrans defines corrosion potential in terms of the resistivity, pH, and soluble salt content.

For structural elements, Caltrans considers a site to be corrosive if a soil sample has a chloride concentration of 500 ppm or greater, a sulfate concentration of 2000 ppm or greater, and/or a pH of 5.5 or less (Caltrans, 2012). Based on the analytical test results presented in Appendix B, and following these Caltrans guidelines, the site soils are not classified to be corrosive to structure foundations. Per Table 4.2.1 in the American Concrete Institute (ACI) Building Code Requirements for Structural Concrete, ACI 318-11 (ACI, 2011), the analytical test results also indicate that the site soils fall into Sulfate Exposure Class S0.

8.0 *BASIS OF RECOMMENDATIONS*

The evaluations made in this report are based on the assumption that soil conditions at the site do not deviate appreciably from those described herein, and are disclosed in the exploratory borings. In the performance of our professional services, AMEC, its employees, and its agents comply with the standards of care and skill ordinarily exercised by members of
our profession practicing in the same or similar localities. No warranty, either express or implied, is made or intended in connection with the work performed by us, or by the proposal for consulting or other services or by the furnishing of oral or written reports or findings.

We are responsible for the evaluations contained in this report, which are based on data related only to the specific project and location discussed herein. In the event conclusions based on these data are made by others, such conclusions are not our responsibility unless we have been given an opportunity to review and concur in writing with such conclusions.

9.0 REFERENCES

American Concrete Institute (ACI), 2011, Building Code Requirements for Structural Concrete (ACI 318-11) and Commentary, August.

American Society of Civil Engineers/Structural Engineers Institute (ASCE/SEI), 2010, Minimum Design Loads for Buildings and Other Structures, ASCE/SEI 7-10: American Society of Civil Engineers (ASCE), Reston, Virginia.


California Department of Transportation (Caltrans), 2012, Corrosion Guidelines, Version 2.0, Division of Engineering Services, Materials Engineering and Testing Services, Corrosion and Structural Concrete, Field Investigation Branch, November.

California Geological Survey, 2000, Seismic Hazard Zone Map and Report for the City and County of San Francisco, California, CGS Seismic Hazard Zone Report 043.


Jennings, C.W., 1994, Fault activity of California and adjacent areas with locations and ages of recent volcanic eruptions: California Division of Mines and Geology Geologic Data Map Series, Map No. 6, scale 1:750,000.


### TABLE 1

**SOURCE PARAMETERS FOR ACTIVE AND POTENTIALLY ACTIVE FAULTS**

John Yehall Chin Elementary School  
San Francisco Unified School District  
San Francisco, California

<table>
<thead>
<tr>
<th>Fault</th>
<th>Distance from Site (mi)</th>
<th>Direction from Site</th>
<th>Magnitude (M$_W$)</th>
<th>Slip Rate (mm/yr)</th>
</tr>
</thead>
<tbody>
<tr>
<td>San Andreas</td>
<td>8.3</td>
<td>West</td>
<td>7.5 - 8.0</td>
<td>17 - 24</td>
</tr>
<tr>
<td>San Gregorio</td>
<td>11.7</td>
<td>West</td>
<td>6.9 - 7.4</td>
<td>7.0</td>
</tr>
<tr>
<td>Hayward</td>
<td>9.7</td>
<td>East</td>
<td>6.5 - 7.0</td>
<td>9.0</td>
</tr>
<tr>
<td>Rodgers Creek</td>
<td>20.1</td>
<td>Northeast</td>
<td>7.0</td>
<td>9.0</td>
</tr>
<tr>
<td>Mount Diablo Thrust</td>
<td>20.6</td>
<td>East</td>
<td>6.6</td>
<td>2.0</td>
</tr>
<tr>
<td>Calaveras</td>
<td>21.1</td>
<td>Southeast</td>
<td>5.8 - 6.9</td>
<td>15.0</td>
</tr>
<tr>
<td>Monte Vista-Shannon</td>
<td>26.2</td>
<td>Southeast</td>
<td>6.4</td>
<td>0.4</td>
</tr>
<tr>
<td>Green Valley</td>
<td>42.2</td>
<td>Northeast</td>
<td>6.0</td>
<td>5.0</td>
</tr>
<tr>
<td>Greenville</td>
<td>31.7</td>
<td>East</td>
<td>6.7</td>
<td>2.0</td>
</tr>
</tbody>
</table>

Notes  
1. Distances represent closest approach of fault to site.  
2. Magnitude and slip rate from California Geological Survey Seismic Hazard Model for California as reported in WGCEP (2008).
Base map from USGS 7.5' San Francisco North, California topographic quadrangle.
SITE LAYOUT AND BORING LOCATION PLAN
John Yehall Chin Elementary School
San Francisco Unified School District
San Francisco, California

Approximate boring location
Base map provided by SFUSD

By: DO  Date: 10/29/2014  Project No. OD14170930

Figure 2
A) West side of John Yehall Chin Elementary School viewed from the south east corner of the play yard.

B) South side of John Yehall Chin Elementary School viewed from the street.
Approximate site perimeter

Explanation
Quaternary
- Qaf: Artificial fill
- Qd: Dune sand
- Qsr: Slope debris and ravine fill
- Qu: Undifferentiated surficial deposits
- Qc: Colma Formation
Cretaceous
- Kfss: Massive sandstone
- Kfsh: Thin-bedded sandstone and shale

Base map from U.S.G.S. 7.5' San Francisco North, California topographic quadrangle.

From Blake et al., 2000
Figure 5

Regional fault activity and historic seismicity map

John Yehall Chin Elementary School
San Francisco Unified School District
San Francisco, California

By: DLW  Date: 11/03/2014  Project No. OD14170930

Explanation

Fault recency classification
- Historic fault rupture
- Faults that displace Holocene (~11 ka) or latest Pleistocene (~20 ka) deposits or geomorphic surfaces.
- Faults that displace late Quaternary (~780 ka) deposits or geomorphic surfaces.
- Quaternary faults (1.8 ma).

Fault traces on land shown as solid where well located, dashed where uncertain or inferred, dotted where buried. Data from Jennings (1994) and WGCEP (2008).

Independent Seismicity Catalog (M) (1800 - 2005)
- 2.5
- 3.0
- 4.0
- 5.0
- 6.0
- 7.0
- 8.0

Kilometers
1:500,000
Base map from state of California Seismic Hazards Zone Map for the City and County of San Francisco, CDMG open-file report 2000-2009, U.S.G.S. 7.5’ San Francisco North, California topographic quadrangle. Shaded relief from U.S.G.S. DEM.

Explanation

- Liquefaction hazard zone
- Landslide hazard zone

Zones show areas of required investigation for hazards.

Approximate site perimeter

Hazard Zones from CGS, 2000.
APPENDIX A

Field Exploration
APPENDIX A

FIELD EXPLORATION
John Yehall Chin Elementary School
San Francisco Unified School District
San Francisco, California

On October 13, 2014, two borings (AB-1 and AB-2) were drilled to investigate and sample the subsurface materials in the vicinity of the school building. The approximate locations of the borings are shown on Figure 2.

Both borings were drilled by HEW Drilling of Palo Alto, California. Boring AB-1 was advanced to 9¼ feet below the ground surface using a hand auger (about 4 inches in diameter). Boring AB-2 was advanced to 17½ feet bgs, using a truck mounted CME-75 drill rig equipped with 6-inch-diameter solid-stem flight augers, where the rig encountered practical refusal (reportedly, the drive chain for the drilling system broke). Mr. Shaun Cordes, staff geologist, observed the drilling operations and prepared the field boring logs.

Soils encountered in the borings were sampled by collecting grab (bag) samples from the soil cuttings or using drive samplers. The types of samplers used are described on the Boring Log Explanation, Figure A-1. Drive samplers were driven into the soil with a 140-pound hammer falling 30 inches. The CME-75 was equipped with an automatic trip hammer. The number of hammer blows needed to drive the sampler through the final 12 inches of the 18-inch drive was recorded. This number (or blows per foot) is given at the corresponding sample location on the boring logs (see Figures A-2 and A-3).

In several cases, driving refusal was encountered (as determined by field personnel. In these cases, the sampler was not driven the full 18 inches. The number of blows over the distance the sampler was driven (e.g., 95/10”) is given at the corresponding sample location on the boring logs. When the Modified California sampler was used with liners, the soil sample liners were capped and sealed to preserve the in situ water content.

Preliminary soil classifications were made visually in the field in general accordance with ASTM D2488. Soil colors were described using the Munsell Soil Color Chart. Soil classifications were verified by further examination in our warehouse and by laboratory test results.
It should be noted that the boring logs show changes in the subsurface stratigraphy that are based on observations made by our field geologist and the drill rig operators during drilling. The contacts/transitions between the various soils were sometimes based on changes in the soils cuttings and changes in the drilling operations (e.g., chatter of the drill rig). The final boring logs, developed from laboratory test data and conditions recorded on the field logs, are presented in this appendix.

Free groundwater was not observed in either boring. Upon completion of drilling, each boring was grouted (to the ground surface) with neat cement grout in accordance with the permit issued by the County of San Francisco Department of Public Health.

Spoil generated during drilling (e.g., soil cuttings) was placed into a single 55-gallon drum. A composite sample of the drilling spoil was transported under chain-of-custody protocols to the laboratory of Curtis & Tompkins, Ltd., in Berkeley, California. Analytical tests included total lead and total petroleum hydrocarbons (test results were forwarded to SFUSD under separate cover). Based on the results of the analytical testing, the drum contents are judged to be “non-hazardous.” The drum is scheduled to be removed from the site, and the contents properly disposed, by Advanced Environmental Services, Inc., of Baker City, Oregon.
**Boring Log Explanation**

### Material Description

- **Modified California drive sampler, 3-inch outside diameter, 2 1/2-inch inside diameter (with liners)**
- **Standard penetration split-spoon drive sample, 2-inch outside diameter, 1 3/8-inch inside diameter (without liners)**
- **Bulk bag sample collected from soil cuttings**
- **Blow count for last 12 inches of sample, or as noted**
- **Blow count for entire drive, total drive less than 6 inches**

### NOTES:
1. The stratification lines shown on the boring logs represent the approximate boundaries between material types. The actual transitions between materials may be gradual.
2. These logs of the test borings and related information depict subsurface conditions only at the specific locations and at the particular time the boring was made.
3. Soil conditions at other locations may differ from conditions occurring at these locations. Also, the passage of time may result in changes in the soil and groundwater conditions at these locations.
4. Soil and rock colors from Munsell Soil Color Charts.

### Laboratory Tests

- **LL** = Liquid limit, **PI** = Plastic index
- **Grain size distribution**
- **Fines content (percentage of soil passing No. 200 sieve)**
- **Corrosion test**

<table>
<thead>
<tr>
<th>Sample No.</th>
<th>Material Description</th>
<th>LABORATORY TESTS</th>
</tr>
</thead>
<tbody>
<tr>
<td>23</td>
<td></td>
<td></td>
</tr>
<tr>
<td>45</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

**Figure A-1**
Log of Boring No. AB-1

PROJECT: John Yehall Chin Elementary School
350 Broadway Street
San Francisco, CA 94133

BOUGING LOCATION: See Site Layout and Boring Location Plan, Figure 2

DRILLING CONTRACTOR: HEW Drilling Company

DRILLING EQUIPMENT: Hand auger

DRILLING METHOD: Hand auger

DATE STARTED: 10/13/2014
DATE FINISHED: 10/13/2014

TOTAL DEPTH (feet): 9.5
MEASURING POINT: Ground surface

DEPTH TO FREE WATER FIRST ENCOUNTERED: Not encountered

DEPTH TO FREE WATER AT COMPLETION: No free water observed

HAMMER WEIGHT: N/A
HAMMER DROP: N/A
LOGGED BY: S. Cordes

ELEVATION AND DATUM: 60 feet (assumed project datum)

DATE STARTED: 10/13/2014
DATE FINISHED: 10/13/2014

PROJECT: John Yehall Chin Elementary School
350 Broadway Street
San Francisco, CA 94133

Log of Boring No. AB-1

SAMPLES

DEPTH (feet) | SAMPLE No. | Blasted off | MATERIAL DESCRIPTION |
---|---|---|---|
1 | S1 | | PAVEMENT
4 inches of asphalt conrete, over 3 inches of SAND (SP), over 2 inches of portland cement concrete|
2 | S2 | | SANDY CLAY (CL)
moist, light yellowish brown (2.5Y 6/3), mottled yellowish brown (10YR 5/4), medium toughness, medium plasticity, fine to medium sand, trace black charcoal(?) or manganese-oxide nodules throughout, clayey peds approximately 1 inch diameter, trace 0.5 inch brick fragments [FILL]|
3 | S3 | | SANDY CLAY (CL)
moist, light yellowish brown (2.5Y 6/3), mottled yellowish brown (10YR 5/4), medium toughness, medium plasticity, fine to medium sand, trace black charcoal(?) or manganese-oxide nodules throughout, clayey peds approximately 1 inch diameter [COLLUVIUM?]|
4 | S4 | | (8 feet) Becomes strong brown (7.5YR 5/6), peds are friable weathered bedrock(?)
5 | S5 | | SANDY CLAY (CL)
light yellowish brown (2.5Y 6/3) sandy matrix with gray (10YR 6/1) friable granular sandstone(?), less clay than previously observed [WEATHERED BEDROCK]
Boring terminated at 9.5 feet and backfilled with cement grout.

LABORATORY TESTS

<table>
<thead>
<tr>
<th>DEPTH (feet)</th>
<th>SAMPLE No.</th>
<th>Blasted off</th>
<th>LABORATORY TESTS</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>S1</td>
<td></td>
<td>Moisture Content (%) 20&lt;br&gt; Dry Density (pcf) LL=32&lt;br&gt; PI=15&lt;br&gt; CORR</td>
</tr>
</tbody>
</table>

GT-1 (12/03)

AMEC

Figure A-2
**Log of Boring No. AB-2**

**PROJECT:** John Yehall Chin Elementary School  
350 Broadway Street  
San Francisco, CA 94133

**BORING LOCATION:** See Site Layout and Boring Location Plan, Figure 2

**DRILLING CONTRACTOR:** HEW Drilling Company

**DRILLING EQUIPMENT:** CME-75

**DRILLING METHOD:** 6-inch diameter solid-stem flight auger

**SAMPLING METHOD:** MC, SPT (see figure A-1)

**HAMMER WEIGHT:** 140 pounds  
**HAMMER DROP:** 30 inches  
**LOGGED BY:** S. Cordes

**DATE STARTED:** 10/13/2014  
**DATE FINISHED:** 10/13/2014

**TOTAL DEPTH (feet):** 17.5  
**MEASURING POINT:** Ground surface

<table>
<thead>
<tr>
<th>DEPTH (feet)</th>
<th>SAMPLES</th>
<th>MATERIAL DESCRIPTION</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Sample No.</td>
<td>Sample Code</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Bored soil</td>
</tr>
<tr>
<td>1</td>
<td>S1</td>
<td>PAVEMENT</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Includes asphalt concrete</td>
</tr>
<tr>
<td>2</td>
<td></td>
<td>SANDY CLAY (CL)</td>
</tr>
<tr>
<td></td>
<td></td>
<td>light yellowish brown (2.5Y 6/3), moist, mottled</td>
</tr>
<tr>
<td></td>
<td></td>
<td>yellowish brown (10YR 5/4), medium toughness,</td>
</tr>
<tr>
<td></td>
<td></td>
<td>medium plasticity, approximately 10 - 15% fine to</td>
</tr>
<tr>
<td></td>
<td></td>
<td>medium sand</td>
</tr>
<tr>
<td>3</td>
<td>S1</td>
<td>SANDY CLAY (CL)</td>
</tr>
<tr>
<td></td>
<td></td>
<td>light yellowish brown (2.5Y 6/3), moist, mottled</td>
</tr>
<tr>
<td></td>
<td></td>
<td>yellowish brown (10YR 5/4), medium toughness,</td>
</tr>
<tr>
<td></td>
<td></td>
<td>medium plasticity, approximately 10 - 15% fine to</td>
</tr>
<tr>
<td></td>
<td></td>
<td>medium sand</td>
</tr>
<tr>
<td>4</td>
<td></td>
<td>[FILL?]</td>
</tr>
<tr>
<td>5</td>
<td></td>
<td>CLAY (CL)</td>
</tr>
<tr>
<td></td>
<td></td>
<td>very stiff, yellowish brown (10YR 5/6), moist, medium</td>
</tr>
<tr>
<td></td>
<td></td>
<td>toughness, medium to high plasticity</td>
</tr>
<tr>
<td>6</td>
<td>S1</td>
<td>[COLLUVIUM]</td>
</tr>
<tr>
<td>7</td>
<td></td>
<td>SANDY CLAY (CL)</td>
</tr>
<tr>
<td></td>
<td></td>
<td>very stiff, light yellowish brown (2.5Y 6/4), fine to</td>
</tr>
<tr>
<td></td>
<td></td>
<td>medium grained weathered sandstone matrix, red</td>
</tr>
<tr>
<td></td>
<td></td>
<td>(2.5Y 6/4) iron-oxide staining throughout, trace friable</td>
</tr>
<tr>
<td></td>
<td></td>
<td>gravel clasts less than 0.5 inch diameter, &lt;10% clay</td>
</tr>
<tr>
<td></td>
<td></td>
<td>present [WEATHERED BEDROCK]</td>
</tr>
<tr>
<td>8</td>
<td>S1</td>
<td>(15 feet) Becomes more indurated, vertical quartz vein</td>
</tr>
<tr>
<td>9</td>
<td></td>
<td>less than 2 millimeter diameter near top of sample,</td>
</tr>
<tr>
<td></td>
<td></td>
<td>linear iron-oxide staining along filled/healed joints(?)</td>
</tr>
</tbody>
</table>

**LABORATORY TESTS**

<table>
<thead>
<tr>
<th>Depth (feet)</th>
<th>Moisture Content (%)</th>
<th>Dry Density (pcf)</th>
</tr>
</thead>
<tbody>
<tr>
<td>6</td>
<td>14</td>
<td>120</td>
</tr>
</tbody>
</table>

**DATE:** 10/13/2014  
**LOCATION:** 350 Broadway Street  
**SAN FRANCISCO, CA 94133**

**AMEC**

**Project No. OD14170930**  
**Figure A-3**
Bottom of boring at 17.8 feet due to drill rig refusal. Backfilled with cement grout.
APPENDIX B

Geotechnical Laboratory Testing
APPENDIX B

GEOTECHNICAL LABORATORY TESTING
John Yehall Chin Elementary School
San Francisco Unified School District
San Francisco, California

INTRODUCTION
Laboratory tests were performed on selected samples of soil to assess their engineering properties and physical characteristics. Testing was performed at Cooper Testing Laboratories located in Palo Alto, California. The following tests were performed:

- Moisture content and dry unit weight
- Atterberg limits (liquid and plastic limits)
- Percent passing the No. 200 sieve (wash analysis)
- Corrosivity potential

Test procedures are described below. Results are provided on the boring logs in Appendix A and in tables and figures in this appendix. The figures contained in this appendix are organized by laboratory test type and presented in the order of the test descriptions, below.

MOISTURE CONTENT AND DRY UNIT WEIGHT
Measurements of the moisture content and dry unit weight were performed on a sample selected from Boring AB-2. This test was conducted in general accordance with ASTM Standard Test Method D 2216 and US Army Corps of Engineers EM 1110-2-1906. Results of the moisture content and dry unit weight measurements are presented at the corresponding sample location on the boring log included in Appendix A.

ATTERBERG LIMITS
An Atterberg limit test was performed to evaluate the plasticity of the soil fines and aid classification. The test was performed in general accordance with ASTM Standard Test Method D 4318. Results of the test is presented on a figure in this appendix and indicated at the corresponding sample location on the boring log in Appendix A.
PERCENT PASSING THE NO. 200 SIEVE

A Number 200 sieve wash analysis test was performed on a selected soil sample to assist in classification. This test was performed in general accordance with ASTM Standard Test Method D 1140. The percent (by weight) of the portion of the sample finer than the No. 200 sieve obtained from the test is indicated at the corresponding sample location on the boring log included in Appendix A.

CORROSIVITY POTENTIAL

A suite of tests were performed on a selected soil sample to assess the corrosivity potential of the near-surface soils. Tests were performed using standard California Department of Transportation (Caltrans) California Test Methods (CTM). The following tests were performed:

<table>
<thead>
<tr>
<th>Analyte</th>
<th>CTM Designation</th>
</tr>
</thead>
<tbody>
<tr>
<td>Minimum Resistivity</td>
<td>Cal 643</td>
</tr>
<tr>
<td>Chloride Concentration</td>
<td>Cal 422-mod.</td>
</tr>
<tr>
<td>Sulfate Concentration</td>
<td>Cal 417-mod.</td>
</tr>
<tr>
<td>pH</td>
<td>Cal 643</td>
</tr>
</tbody>
</table>

The results of these tests are presented in a table in this appendix.
**LIQUID AND PLASTIC LIMITS TEST REPORT**

Dashed line indicates the approximate upper limit boundary for natural soils.

---

**MATERIAL DESCRIPTION**

<table>
<thead>
<tr>
<th>LL</th>
<th>PL</th>
<th>PI</th>
<th>%&lt;#40</th>
<th>%&lt;#200</th>
<th>USCS</th>
</tr>
</thead>
<tbody>
<tr>
<td>32</td>
<td>17</td>
<td>15</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Yellowish Brown Sandy Lean CLAY

---

**Project No.** 109-749  **Client:** AMEC
**Project:** John Y. Chin Elementary School - OD14170930
**Source:** AB-1  **Sample No.:** S1  **Elev./Depth:** 1-3’

---

**Remarks:**

---
<table>
<thead>
<tr>
<th>Sample Location or ID</th>
<th>Resistivity @ 15.5 °C (Ohm-cm)</th>
<th>Chloride mg/kg</th>
<th>Sulfate mg/kg</th>
<th>pH %</th>
<th>ORP (Redox) mv</th>
<th>Moisture At Test %</th>
<th>Soil Visual Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>AB-1</td>
<td>S1&amp;S2</td>
<td>1-3</td>
<td>-</td>
<td>2775</td>
<td>-</td>
<td>7</td>
<td>77</td>
</tr>
</tbody>
</table>
APPENDIX C

Original Foundation Drawings for the Structure