GEOTECHNICAL ENGINEERING
AND GEOLOGIC HAZARDS STUDY

Willow Cove Elementary School
New Gymnasium Building
1880 Hanlon Way
Pittsburg, California 94565

Prepared for:

Pittsburg Unified School District
2000 Railroad Avenue
Pittsburg, California 94565

Prepared by:

GEOSPHERE CONSULTANTS, INC.
2001 Crow Canyon Road, Suite 210
San Ramon, California 94583
Project No. 91-03477-A
May 11, 2015

Pittsburg Unified School District
2000 Railroad Avenue
Pittsburg, California 94565

Attention: Mr. Michael Camigi

Subject: Geotechnical Engineering and Geologic Hazards Study
Willow Cove Elementary School – New Gymnasium Building
1880 Hanlon Way, Pittsburg, California 94565
Geosphere Project No. 91-03477-A

Dear Mr. Camigi:

In accordance with your authorization, Geosphere Consultants, Inc. (Geosphere) has completed a Geotechnical Engineering and Geologic Hazards Study for the proposed construction of the proposed new gymnasium building at Willow Cove Elementary School in Pittsburg, Contra Costa County, California. This report has been prepared in accordance with the requirements set forth in California Geological Survey Note 48 and the California Building Code, 2013. Transmitted herewith are the results of our findings, conclusions, and recommendations for the design and construction of proposed building foundations, site grading, drainage, and utility trench backfilling. In general, the proposed improvements at the site are considered to be geotechnically and geologically feasible provided the recommendations of this report are implemented in the design and construction of the project.

Should you or members of the design team have questions or need additional information, please contact the undersigned by email: cdare@geosphereinc.net or eswenson@geosphereinc.net or at (925) 314-7180. We greatly appreciate the opportunity to provide our services to the Pittsburg Unified School District and to be involved in the design of this project.

Sincerely,

GEOSPHERE CONSULTANTS, INC.

Corey T. Dare, P.E., G.E.
Principal Geotechnical Engineer

Eric J. Swenson, G.E., C.E.G.
President/Principal Engineering Geologist

Distribution: 2 plus PDF to Addressee (mcamigi@pittsburg.k12.ca.us)
3 plus PDF to Mr. Rup Chand, ATI Architects and Engineers, 3860 Blackhawk Road, #160, Danville, California 94506 (rchand@atiae.com)

CTD/EJS:pmf
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1.0 INTRODUCTION

1.1 Purpose and Scope

The purposes of this study were to evaluate the geologic hazards and the subsurface conditions at the site and prepare geotechnical recommendations for the proposed improvements. We understand that the improvements will include construction of a new one-story gymnasium building, together with hardscape and other site improvements. This study provides recommendations for the design and construction of proposed foundations, and for site grading, drainage, and utility trench backfill. This study was performed in accordance with the scope of work outlined in our proposal dated March 18, 2015.

The scope of this study included the review of pertinent published and unpublished documents related to the site, a field exploration, field and laboratory testing, engineering analysis of the accumulated data, and preparation of this report. The conclusions and recommendations presented in this report are based on the data acquired and analyzed during this study, and on prudent engineering judgment and experience. This study did not include an in-depth assessment of potentially toxic or hazardous materials that may be present on or beneath the site. However, a preliminary screening of onsite soils was conducted in order to assess potential contamination by toxic or hazardous materials, as indicated in our proposal. These test results were provided in a separate report.

1.2 Site Description

The proposed project site is located at 1880 Hanlon Way in Pittsburg, Contra Costa County, California, as shown on the attached Figure 1, Site Vicinity Map. The school site is surrounded by Hanlon Way on the south, a vacant area and existing single family residences facing West Trident Drive on the east, and existing single family residential properties either facing Loftus Road on the west, or facing various residential streets on the north. The general topography of the school site descends gently toward the north to northeast.
As shown on Figure 2, *Site Plan and Site Geology Map*, the project site is currently a vacant, undeveloped, grassy open field area on the eastern side of the existing school property directly east of the main school building complex and north of a paved access driveway. The building site is currently near level with site elevations based on Google Earth to range from +58 along the west side of the proposed building footprint to between +59 and +61 along the east side of the footprint. The average geographic coordinates of the proposed building site used for engineering analysis are 38.0239 degrees north latitude and 121.9258 degrees west longitude.

### 1.3 Proposed Development

We understand that, based on the information provided by the District, the proposed project will consist of the construction of a new single-story gymnasium building and associated site development at the approximate location shown on Figure 2. The building would be between about 82 and 99 feet wide by 165 feet long with an order of 14,400 square feet of building footprint, and contain a multi-purpose room, kitchen, platform, and storage rooms. A site improvement plan of the project was not provided for our review at the time this report was prepared, but site improvements such as new exterior hardscape are anticipated.
2.0 PROCEDURES AND RESULTS

2.1 Literature Review

Pertinent geologic and geotechnical literature pertaining to the site area was reviewed. These included various USGS, CGS, and other government publications, websites and maps, as listed in the References section.

2.2 Field Exploration

A total of three borings were drilled at the site on April 6, 2015 at the approximate locations shown on Figure 2. The borings were drilled to a maximum depth of approximately 50 feet below the existing ground surface within the proposed building footprint using a truck mounted, CME 75 drill rig equipped with six-inch solid flight augers and mud rotary wash drilling as appropriate. A Geosphere Staff Engineer visually classified the materials encountered in the borings according to the Unified Soil Classification System as the borings were advanced. Relatively undisturbed soil samples were recovered at selected intervals using a three-inch outside diameter Modified California split spoon sampler containing six-inch long brass liners. A two-inch outside diameter Standard Penetration Test (SPT) sampler was used to obtain SPT blow counts and obtain disturbed soil samples. The samplers were driven by using a mechanical-trip, 140-pound hammer with an approximate 30-inch fall utilizing N-rods as necessary. Resistance to penetration was recorded as the number of hammer blows required to drive the sampler the final foot of an 18-inch drive. All of the blow counts recorded using Modified California split spoon samplers in the field were converted to equivalent SPT blow counts using appropriate modification factors suggested by Burmister (1948), i.e., a factor of 0.65 assuming an inner diameter of 2.5 inches. Therefore, all blow counts shown on the final boring logs are either directly measured (SPT sampler) or equivalent SPT (MC sampler) blow counts. Bulk samples were obtained in the upper few feet of the borings from the auger cuttings as needed.

The boring logs with descriptions of the various materials encountered in each boring, the penetration resistance values, and some of the laboratory test results are presented in Appendix A. The ground surface elevations indicated on the soil boring logs are approximate (i.e., rounded to the nearest ½ foot) and were estimated from the Google Earth web application. Actual surface elevations at the boring locations could differ. The locations and elevations of the borings should only be considered accurate to the degree implied by the means and methods used to define them.
2.3 Laboratory Testing

Laboratory tests were performed on selected samples to determine some of the physical and engineering properties of the subsurface soils. The results of the laboratory testing are presented on the boring logs, and are included in the appendices. The following soil tests were performed for this study:

Dry Density and Moisture Content (ASTM D2937 and D2216) – In-situ density and moisture tests were conducted to measure the in-place dry density and moisture content of the subsurface materials at the tested sample locations and depths. These properties provide information for evaluating the physical characteristics of the subsurface soil. Test results are presented in Appendix B and also shown on the boring logs.

Atterberg Limits (ASTM D4318 and CT204) - Liquid Limit, Plastic Limit, and Plasticity Index are useful in the classification and characterization of the engineering properties of soil, including evaluating the expansive characteristics of the soil, and determining the soil type according to the USCS. Two tests were performed, and the test results are presented in Appendix B and summarized on the boring logs.

Particle Size Analysis (Wet and Dry Sieve) and Hydrometer (ASTM D422, D1140, and CT202) - Sieve analysis testing is conducted on selected samples to determine the soil particle size distribution. This information is useful for the evaluation of liquefaction potential and characterizing the soil type according to USCS. Two tests were performed, and the results presented in Appendix B.

Unconsolidated-Undrained Compressive Strength Test (ASTM D2850) – An Unconsolidated-Undrained compression test was performed on one relatively undisturbed sample of the cohesive subsurface soils to evaluate the undrained shear strength of these materials. The test was performed on a sample having a diameter of 2.4 inches and height of 5.0 inches. Failure was taken as the peak normal stress minus the confining pressure (i.e., deviator stress). The results of the test are presented in Appendix B and on the boring log at the appropriate sample depth.

Soil Corrosivity - Soil corrosivity testing was performed to determine the effects of constituents in the soil on buried steel and concrete. Water-soluble sulfate testing is required by the CBC and IBC. Test results are presented in Appendix B and discussed in Section 4.3.
3.0 GEOLOGIC AND SEISMIC OVERVIEW

3.1 Geologic Setting

The site is located in the central portion of the northern Coast Ranges geomorphic province of California. The Coast Ranges extend from the Transverse Ranges in southern California to the Oregon border and are comprised of a northwest-trending series of mountain ranges and intervening valleys that reflect the overall structural grain of the province. The ranges consist of a variably thick veneer of Cenozoic volcanic and sedimentary deposits overlying a Mesozoic basement of sedimentary, metamorphic, and basic igneous Franciscan Formation and primarily marine sedimentary rocks of the Great Valley Sequence. East-dipping sedimentary rocks of the Coast Ranges are flanked on the east by sedimentary rocks of the Great Valley geomorphic province (Page, 1966).

More specifically, the site is located east of San Francisco Bay at the base of the northeast flank of, and northwest end of the Diablo Range. The site is mapped by both Dibblee and Minch (2006) and Graymer et al. (2006) as being underlain by late Pleistocene-age alluvial sediments (identified on the maps as Qoa or Qpa, respectively), as shown on Figure 3, Site Vicinity Geologic Map. Bedrock upslope and to the south at the base of the Diablo Range consists of the late Pliocene-age Oro Loma Formation, which generally consists of interbedded pebble conglomerate, sandstone and claystone.

According to the USDA Natural Resources Conservation Service, Web Soil Survey, the site soil is classified as Capay clay. These soils where undisturbed are classified by USDA near the ground surface as consisting of generally moderately expansive clays (CL) of alluvial origin derived from sedimentary rock.

3.2 Geologic Evolution of the Northern Coast Ranges

The subject site is located within the tectonically active and geologically complex northern Coast Ranges, which have been shaped by continuous deformation resulting from tectonic plate convergence (subduction) beginning in the Jurassic period (about 145 million years ago). Eastward thrusting of the oceanic plate beneath the continental plate resulted in the accretion of materials onto the continental plate. These accreted materials now largely comprise the Coast Ranges. The dominant tectonic structures formed during this time include generally east-dipping thrust and reverse faults.
Beginning in the Cenozoic time period (about 25 to 30 million years ago), the tectonics along the California coast changed to a transpressional regime and right-lateral strike-slip displacements as well as thrusting were superimposed on the earlier structures resulting in the formation of northwest-trending, near-vertical faults comprising the San Andreas Fault System. The northern Coast Ranges were segmented into a series of tectonic blocks separated by major faults including the San Andreas, Hayward, and Calaveras. The project site is situated east of the active Hayward, Calaveras, and San Andreas faults, and no known active faults with Holocene movement (i.e., last 11,000 years) lie within the limits of or in close proximity to the site. The site is not mapped within an Alquist-Priolo Earthquake Fault Zone.

3.3 Regional Faulting and Tectonics

Regional transpression has caused uplift and folding of the bedrock units within the Coast Ranges. This structural deformation occurred during periods of tectonic activity that began in the Pliocene and continues today. The site is located in a seismically active region that has experienced periodic, large magnitude earthquakes during historic times. This seismic activity appears to be largely controlled by displacement between the Pacific and North American crustal plates, separated by the San Andreas Fault zone located approximately 39 miles (62 km) southwest of the site. This plate displacement produced regional strain that is concentrated along major faults of the San Andreas Fault System including the San Andreas, Hayward, and Calaveras Faults in the greater San Francisco Bay area.

The site is located in a seismically active region dominated by major faults of the San Andreas Fault System. Major active faults include the aforementioned San Andreas Fault; the Calaveras Fault, located on the order of 12 miles (19 km) southwest of the site; and the Hayward Fault, located approximately 20.5 miles (33 km) southwest of the site. In addition, closer active or potentially active faults include the Greenville-Marsh Creek Fault system, located at its closest point about 2.5 miles (4 km) southwest of the site; the Concord-Green Valley Fault, located about 6.8 miles (11 km) southwest of the site; and the Mt. Diablo Fault, the northern end of which is located on the order of 10 miles (16 km) southwest of the site. The site location relative to active and potentially active faults in the San Francisco Bay Area is shown on Figure 4, Regional Fault Map. A discussion of these faults, ordered by increasing distance from the site, follows.
3.3.1 Greenville-Marsh Creek Fault

The northwest-trending Greenville-Marsh Creek Fault system lies within the Diablo Range and extends from Arroyo Mocho southeast of Livermore, across the eastern edge of the Livermore Valley and the northeast edge of Mt. Diablo into the Clayton Valley area. On the east side of Mt. Diablo, the fault has been referred to as the Marsh Creek Fault, connecting to the Clayton Fault in the Clayton Valley. The fault zone has been divided (CGS, 2002) into two segments, Greenville-North and Greenville-South. The site is located on the order of 2.5 miles (4 km) northeast of the Clayton Fault. The slip rate of the Greenville-North segment is estimated to be about 2 mm/year and has been assigned a moment magnitude ($M_{max}$) of 6.6 (CGS, 2002). The Working Group on California Earthquake Probabilities (WGCEP2007) has estimated that there is a three percent probability of at least one magnitude 6.7 or greater earthquake before 2037 along the Greenville Fault (USGS, 2008).

3.3.2 Concord - Green Valley Fault

The north to northwest trending Concord Fault extends from the approximate central Walnut Creek and Concord border, northward into the Green Valley Fault. The Green Valley Fault extends northward from Suisun Bay up to just west of Lake Curry, northeast of Napa. The site is located on the order of 6.8 miles (11 km) northeast of the Concord Fault. The slip rate of the Concord Fault is estimated to be about 4 mm/year and has been assigned a moment magnitude ($M_{max}$) of 6.2 (CGS, 2002). The Working Group on California Earthquake Probabilities (WG07) has estimated that there is a three percent probability of at least one magnitude 6.7 or greater earthquake before 2037 occurring on the Concord – Green Valley Fault system (USGS, 2008).

3.3.3 Mt. Diablo Thrust Fault

The northwest trending Mt. Diablo Fault extends from near the Alamo and Walnut Creek border along Highway 680, extending southward to the vicinity of Cottonwood Canyon, in northern Dublin. The Mt. Diablo Fault is not currently mapped by CGS, but fault coordinates have been published. The fault is different from the majority of Bay Area faults in that it is not a strike-slip fault exhibiting horizontal plate motion, but rather a “blind thrust” fault. A blind thrust fault is a compressional fault without apparent surface expression. In the case of the Mt. Diablo Fault, the compression is caused by the differential horizontal motions of the Concord and Greenville faults which abut Mt. Diablo. The fault locations for this blind thrust fault are inferred. The 1994 Northridge earthquake is a recent example of faulting on a blind thrust fault. The site is located on the order of 10 miles (16 km) northeast of the estimated location of the Mt. Diablo Fault. The slip rate of the Mt. Diablo Thrust Fault is
estimated to be about 2 mm/year and has been assigned a moment magnitude \((M_{\text{max}})\) of 6.7 (CGS, 2002). The Working Group on California Earthquake Probabilities (WGCEP2007) has estimated that there is a one percent probability of at least one magnitude 6.7 or greater earthquake on the Mt. Diablo Thrust Fault before 2037 (USGS, 2008).

### 3.3.4 Calaveras Fault

The Calaveras Fault trends northwesterly about 123 km in length from near Hollister, extending to north of the Danville area. The Calaveras Fault has been divided into three segments, the Northern, Central, and Southern segments. The site is located on the order of 12 miles (19 km) northeast of the estimated northern end of northern segment of the Calaveras Fault. The slip rate on the north segment of the Calaveras Fault is estimated to be about 6 mm/year and has been assigned a moment magnitude \((M_{\text{max}})\) of 6.8 (CGS, 2003). The Working Group on California Earthquake Probabilities (WG07) has estimated that there is a seven percent probability of at least one magnitude 6.7 or greater earthquake before 2037 along the Calaveras Fault.

### 3.3.5 Hayward Fault

The Hayward Fault trends northwesterly on the order of 88 km from the Milpitas area to San Pablo Bay. The Hayward Fault has been divided into two main segments, the Northern and Southern segments. The Rodgers Creek Fault, considered as a possible extension of the Hayward Fault, extends northward from beneath San Pablo Bay up to near Healdsburg, where it is aligned with the Healdsburg Fault zone, currently considered to be inactive. The site is located approximately 20.5 miles (33 km) northeast of the northern segment of the Hayward Fault. The slip rate on this segment of the Hayward Fault is estimated to be about 9 mm/year and has been assigned a moment magnitude \((M_{\text{max}})\) of 6.4 (CGS, 2003). The Working Group on California Earthquake Probabilities (WG07) has estimated that there is a 31 percent probability of at least one magnitude 6.7 or greater earthquake before 2037 along the Hayward – Rodgers Creek Fault system.

### 3.3.6 San Andreas Fault

The northwest-trending San Andreas Fault runs along the western coast of California extending on the order of 625 miles from the north near Point Arena to the Salton Sea area in southern California (Jennings, 1994). The fault zone has been divided into 11 segments. The site is located about 39 miles (62 km) northeast of the Peninsula segment. The slip rate on the Peninsula segment of the San Andreas Fault is estimated to be about 17 mm/year and has been assigned a moment magnitude \((M_{\text{max}})\) of 7.1 (CGS, 2003). The Working Group on
California Earthquake Probabilities (WG07) has estimated that there is a 21 percent probability of at least one magnitude 6.7 or greater earthquake before 2037 along the San Andreas Fault.

### 3.4 Historic Seismicity

As discussed above, the San Francisco Bay Area is subject to a high level of seismic activity. Within the period of 1800 to 2000 there were an estimated 20 earthquakes exceeding a Richter magnitude of 6.0 within a 100 mile radius of the site, seven exceeding 6.5, four exceeding 7.0 and one exceeding 7.5. There have been six major Bay Area earthquakes since 1800. Those were in 1836 and 1868 on the Hayward-Rodgers Creek Fault, in 1861 on the Calaveras Fault, and in 1838, 1906, and 1989 on the San Andreas Fault.

The site is reported to have experienced shaking from on the order of 57 earthquakes of magnitude 5.5 or greater during the period of 1800 to 2000, occurring at various distances away from the site. Of those, 17 were greater than Magnitude 6.0, seven exceeded 6.5, four exceeded 7.0 and one was greater than 7.5. The most significant known ground shaking affecting the site has been attributed to the 1868 Hayward earthquake, as well as the Magnitude 6.3 Antioch earthquake that occurred on May 19, 1889, which reportedly caused numerous chimneys to fall, but caused no serious damage (Keeler, 1890).
4.0 SUBSURFACE CONDITIONS

4.1 Subsurface Soil Conditions

The subsurface conditions of the project site are relatively uniform with an upper layer generally consisting of stiff sandy clay to depths on the order of five feet, underlain by generally very stiff to hard silty to sandy clay to the maximum depth explored of 50 feet below the existing ground surface. Soils encountered in Boring B-1 below a depth of about 18 feet consisted of medium dense clayey sand to the maximum depth explored of 25 feet.

Test results of near-surface soil samples recovered in the uppermost three feet of the soil profile collected from Borings B-1 & B-2 indicated measured Liquid Limits of 39 and 45 and corresponding Plasticity Indices of 22 and 29. Based on these results, the near-surface soils are considered to have a medium plasticity and a moderate to high expansion (shrink/swell) potential. Additional details of the soils encountered in the exploratory borings are included in the boring logs presented in Appendix A. A geological cross section through the proposed development area is presented in Figure 5, Schematic Geologic Cross Section A-A’.

4.2 Groundwater

Groundwater was not encountered during our field exploration, including in two borings drilled to depths of up to 25 feet using a solid auger. Boring B-3 was drilled using the rotary wash method to a depth of 50 feet, but groundwater could not be measured in the boring due to the use of drilling fluid.

Groundwater level in the project vicinity is expected to be relatively deep. Research on the California Department of Water Resources’ website groundwater elevation database showed a measurement point nearest to the project site about 1,200 feet downslope and to the north of the site. Recorded groundwater depths at this well (DWR Well No. 02N01W12P001M) between 1969 and 1975 ranged from about 26.5 to 28 feet below the ground surface, corresponding to groundwater elevations of 2 to 3.5 ft. above Mean Sea Level (NGVD29), based on a well surface elevation of +30, about 28 to 30 feet lower than the project site. Additionally, another DWR well (Bodega MW-290) located about 0.93 miles east-southeast of the site showed recorded groundwater depths between October 2011 and March 2015 ranging between about 40 and 44.5 feet below the ground surface, corresponding to groundwater elevations of 3.5 to 8 ft. above Mean Sea Level (NGVD29), based on a well surface elevation of +48, about 10 to 12 feet lower than the project site.
Based on the aforementioned DWR well data we estimate the historical high groundwater level at the project site to be on the order of 50 feet below the existing ground surface, based on an average project site elevation of +60 and an assumed high groundwater elevation on the order of +10. Groundwater levels can vary in response to time of year, variations in seasonal rainfall, tidal influence, well pumping, irrigation, and alterations to site drainage.

4.3 Corrosion Testing

A representative sample collected from the upper two feet of the soil profile at Boring B-2 was tested to measure sulfate content, chloride content, redox potential, pH, resistivity, and presence of sulfides. Test results are included in Appendix B and are summarized on the following table.

Table 1: Summary of Corrosion Test Results

<table>
<thead>
<tr>
<th>Soil Description</th>
<th>Sample Depth (feet)</th>
<th>Sulfate (mg/kg)</th>
<th>Chloride (mg/kg)</th>
<th>Redox (mV)</th>
<th>Resistivity (ohm-cm)</th>
<th>Sulfide</th>
<th>pH</th>
</tr>
</thead>
<tbody>
<tr>
<td>Dark Gray Brown Sandy CLAY (CL)</td>
<td>1-2</td>
<td>71</td>
<td>34</td>
<td>550</td>
<td>962</td>
<td>Negative</td>
<td>7.6</td>
</tr>
</tbody>
</table>

Water-soluble sulfate can affect the concrete mix design for concrete in contact with the ground, such as shallow foundations, piles, piers, and concrete slabs. Section 4.3 in American Concrete Institute (ACI) 318, as referenced by the CBC, provides the following evaluation criteria:

Table 2: Sulfate Evaluation Criteria

<table>
<thead>
<tr>
<th>Sulfate Exposure</th>
<th>Water-Soluble Sulfate in Soil, Percentage by Weight or (mg/kg)</th>
<th>Sulfate in Water, ppm</th>
<th>Cement Type</th>
<th>Max. Water Cementitious Ratio by Weight</th>
<th>Min. Unconfined Compressive Strength, psi</th>
</tr>
</thead>
<tbody>
<tr>
<td>Negligible</td>
<td>0.00-0.10 (0-1,000)</td>
<td>0-150</td>
<td>NA</td>
<td>NA</td>
<td>NA</td>
</tr>
<tr>
<td>Moderate</td>
<td>0.10-0.20 (1,000-2,000)</td>
<td>150-1,500</td>
<td>II, IP (MS), IS (MS)</td>
<td>0.50</td>
<td>4,000</td>
</tr>
<tr>
<td>Severe</td>
<td>0.20-2.00 (2,000-20,000)</td>
<td>1,500-10,000</td>
<td>V</td>
<td>0.45</td>
<td>4,500</td>
</tr>
<tr>
<td>Very Severe</td>
<td>Over 2.00 (20,000)</td>
<td>Over 10,000</td>
<td>V plus pozzolan</td>
<td>0.45</td>
<td>4,500</td>
</tr>
</tbody>
</table>
The water-soluble sulfate content was measured to be about 71 mg/kg (ppm) or 0.0071% by dry weight in the soil sample, suggesting the site soil should have negligible impact on buried concrete structures at the site. However, it should be pointed out that the water-soluble sulfate concentrations can vary due to the addition of fertilizer, irrigation, and other possible development activities.

Table 4.4.1 in ACI 318 suggests use of mitigation measures to protect reinforcing steel from corrosion where chloride ion contents are above 0.06% by dry weight. The chloride content was measured to be about 34 mg/kg (ppm) or 0.0034% by dry weight in the soil sample. Therefore, the test result for chloride content does not suggest a corrosion hazard for mortar-coated steel and reinforced concrete structures due to high concentration of chloride.

In addition to sulfate and chloride contents described above, pH, oxidation reduction potential (Redox), and resistivity values were measured in the soil sample. For cast and ductile iron pipes, an evaluation was based on the 10-Point scaling method developed by the Cast Iron Pipe Research Association (CIPRA) and as detailed in Appendix A of the American Water Works Association (AWWA) publication C-105, and shown on Table 3.

<table>
<thead>
<tr>
<th>Soil Characteristics</th>
<th>Points</th>
<th>Soil Characteristics</th>
<th>Points</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Resistivity, ohm-cm, based on single probe or water-saturated soil box.</strong></td>
<td></td>
<td><strong>Redox Potential, mV</strong></td>
<td></td>
</tr>
<tr>
<td>&lt;700</td>
<td>10</td>
<td>&gt;+100</td>
<td>0</td>
</tr>
<tr>
<td>700-1,000</td>
<td>8</td>
<td>+50 to +100</td>
<td>3.5</td>
</tr>
<tr>
<td>1,000-1,200</td>
<td>5</td>
<td>0 to 50</td>
<td>4</td>
</tr>
<tr>
<td>1,200-1,500</td>
<td>2</td>
<td>Negative</td>
<td>5</td>
</tr>
<tr>
<td>1,500-2,000</td>
<td>1</td>
<td>Sulfides</td>
<td></td>
</tr>
<tr>
<td>&gt;2,000</td>
<td>0</td>
<td>Positive</td>
<td>3.5</td>
</tr>
<tr>
<td><strong>PH</strong></td>
<td></td>
<td>Trace</td>
<td>2</td>
</tr>
<tr>
<td>0-2</td>
<td>5</td>
<td>Negative</td>
<td>0</td>
</tr>
<tr>
<td>2-4</td>
<td>3</td>
<td><strong>Moisture</strong></td>
<td></td>
</tr>
<tr>
<td>4-6.5</td>
<td>0</td>
<td>Poor drainage, continuously wet</td>
<td>2</td>
</tr>
<tr>
<td>6.5-7.5</td>
<td>0</td>
<td>Fair drainage, generally moist</td>
<td>1</td>
</tr>
<tr>
<td>7.5-8.5</td>
<td>0</td>
<td>Good drainage, generally dry</td>
<td>0</td>
</tr>
<tr>
<td>&gt;8.5</td>
<td>5</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Assuming fair site drainage, the tested soil sample had a total score of 9 points, indicating a non-corrosive rating. When total points on the AWWA corrosivity scale are at least 10, the soil is classified as corrosive to cast and ductile iron pipe, and use of cathodic corrosion protection is often recommended.
These results are preliminary, and provide information only on the specific soil sampled and tested. Other soil at the site may be more or less corrosive. Providing a complete assessment of the corrosion potential of the site soils are not within our scope of work. For specific long-term corrosion control design recommendations, we recommend that a California-registered professional corrosion engineer evaluate the corrosion potential of the soil environment on buried concrete structures, steel pipe coated with cement-mortar, and ferrous metals.
5.0 GEOLOGIC HAZARDS

5.1 Seismic Induced Hazards

Seismic hazards resulting from the effects of an earthquake generally include ground shaking, liquefaction, lateral spreading, dynamic settlement, fault ground rupture and fault creep, dam inundation, and tsunamis and seiches. The site is not necessarily impacted by all of these potential seismic hazards. Nonetheless, potential seismic hazards are discussed and evaluated in the following sections in relation to the planned construction.

5.1.1 Ground Shaking

The site may experience moderate to strong ground shaking from a major earthquake originating from one or more of the close or major Bay Area faults such as the Concord (approximately 6.8 miles from the site) or Hayward (approximately 20.5 miles from the site) faults. Moderate shaking may also be generated at the site by the Greenville-Marsh Creek Fault (approximately 2.5 miles from the site), Mt. Diablo Thrust Fault (approximately 10 miles from the site), Calaveras Fault (approximately 12 miles from the site), or San Andreas Fault (approximately 39 miles from the site).

5.1.2 Liquefaction Induced Phenomena

Research and historical data indicate that soil liquefaction generally occurs in saturated, loose granular soil (primarily fine to medium-grained, clean sand deposits) during or after strong seismic ground shaking and is typified by a loss of shear strength in the affected soil layer, thereby causing the soil to flow as a liquid. However, because of the higher inter-granular pressure of the soil at greater depths, the potential for liquefaction is generally limited to the upper 40 feet of the soil. Potential hazards associated with soil liquefaction below or near a structure include loss of foundation support, lateral spreading, sand boils, and areal and differential settlement.

Lateral spreading is lateral ground movement, with some vertical component, as a result of liquefaction. The soil literally rides on top of the liquefied layer. Lateral spreading can occur on relatively flat sites with slopes less than two percent under certain circumstances. Lateral spreading can cause ground cracking and settlement.

The site is mapped by the USGS and William Lettis & Associates in corporation with the CGS as being located within a zone of low liquefaction potential (see Figure 6, Liquefaction Susceptibility Map). The site was generally underlain within the upper 50 feet explored by stiff to hard, medium plasticity sandy to silty clay (CL) ranging to
medium dense clayey sands (SC) with measured fines content on the order of 40 percent. The historical high groundwater is estimated to be on the order of 50 feet. Based on these soil characteristics and the assumed historic high groundwater depth, we anticipate the liquefaction potential of the site subsurface soils to be very low to nil. Due to the very stiff nature of the subsurface soils, the lack of liquefaction-susceptible subsurface soils, and the lack of a nearby free slope face, the potential for lateral spreading is considered to be very low to nil.

5.1.3 Dynamic Compaction (Settlement)

Dynamic compaction is a phenomenon where loose, relatively clean sandy soil located above the water table is densified from vibratory loading, typically from seismic shaking or vibratory equipment. The site soils consist primarily of sandy clay and clayey sand of relatively high fines content (i.e., over 40 percent fines) and stiff/medium dense to hard/dense consistency. Therefore, in our opinion, the site soils are not susceptible to dynamic compaction phenomena.

5.1.4 Fault Ground Rupture and Fault Creep

The State of California adopted the Alquist-Priolo Earthquake Fault Zone Act of 1972 (Chapter 7.5, Division 2, Sections 2621 – 2630, California Public Resources Code), which regulates development near active faults for the purpose of preventing surface fault rupture hazards to structures for human occupancy. In accordance with the Alquist-Priolo (A-P) Act, the California Geological Survey established boundary zones, or Earthquake Fault Zone surrounding faults or fault segments judged to be sufficiently active, well-defined, and mapped for some distance. Structures for human occupancy within designated Earthquake Fault Zone boundaries are not permitted unless surface fault rupture and fault creep hazards are adequately addressed in a site-specific evaluation of the development site.

The site is not currently mapped or within a designated Earthquake Fault Zone as defined by the State (Hart and Bryant, 1997). The closest Earthquake Fault Zones are associated with the Concord/Green Valley Fault, about 6.8 miles southwest of the site. Since the site is not within or near an A-P Earthquake Fault Zone, the potential for fault ground rupture and fault creep hazards are judged to be very low.

5.1.5 Tsunamis and Seiches

Tsunamis are long-period sea waves generated by seafloor movements from submarine earthquakes or volcanic
eruptions that rapidly displace large volumes of water. Coastal communities along the Pacific Ocean are particularly susceptible to such phenomena. The California Emergency Management Agency tsunami inundation maps do not include any mapped potential tsunami inundation zones east of the Benicia-Martinez Bridge. In addition, the site is situated at about El. +60. Therefore, the potential for tsunami inundation at the site is considered to be nil.

Earthquake-induced waves generated within enclosed bodies of water are called seiches. Such waves may overtop dam embankments or extreme cases, cause dam failure, and in either case result in downstream inundation. The site is not within the downstream drainage area of any body of water. Therefore, the site is not considered to be susceptible to seiches.

5.2 Other Hazards

Potential geologic hazards other than those caused by a seismic event generally include ground failure and subsidence, consolidation settlement, landslides under static loading conditions, expansive and collapsible soils, flooding, naturally occurring asbestos (NOA) and soil erosion. These are discussed and evaluated in the following sections.

5.2.1 Ground Cracking and Subsidence

Withdrawal of groundwater and other fluids (i.e. petroleum and the extraction of natural gas) from beneath the surface has been linked to large-scale land subsidence and associated cracking on the ground surface. Other causes for ground cracking and subsidence include the oxidation and resultant compaction of peat beds, the decline of groundwater levels and consequent compaction of aquifers, hydro-compaction and subsequent settlement of alluvial deposits above the water table from irrigation, or a combination of any of these causes. Determining the impacts from subsidence on the project is beyond the scope of this study however, subsidence generally impacts a region, and should not produce excessive differential settlement in a single location. Local and regional locations prone to subsidence generally subside equally over time.

5.2.2 Consolidation Settlement

Consolidation is the densification of soil into a more dense arrangement from additional loading, such as from new fills or foundation loads. Consolidation of clayey soils is usually a long-term process, whereby the water is squeezed out of the soil matrix with time. Sandy soils consolidate relatively rapidly with an introduction of a
load. Consolidation of soft and loose soil layers and lenses can cause settlement of the ground surface or buildings. Based on stiff to hard subsurface site soils, anticipated lack of consistently saturated soils below the proposed building footprint, and anticipated relatively light building loads, the near-surface soil is considered to have a low potential to consolidate to an extent to have significant impact on the performance of the building.

5.2.3 Landslides

Landslides can occur under a variety of loading conditions, including both static and seismic, but involve sloping ground. As shown on Figure 7, Existing Landslide Map, the site is classified as “flatland.” The site and immediate vicinity is relatively flat and does not exhibit landslide features as determined by our site reconnaissance and literature review. Therefore, the site is not susceptible to landsliding.

5.2.4 Expansive and Collapsible Soils

Visual observation and laboratory testing of selective samples of the near-surface soils (upper less than five-feet deep) indicated a medium plasticity, and the near-surface soils are considered to be generally moderately to highly expansive. Hence, we recommend moderate mitigation measures be considered. We recommend at least the upper 18 inches of the building pad be prepared with non-expansive soils as recommended in the Building Pad Preparation/General Grading Sections. If unanticipated, very highly expansive soils are encountered during grading construction, Geosphere should be given an opportunity to reevaluate the soil conditions and an appropriate mitigation technique will be provided.

The subsurface deposits encountered during the drilling program generally consisted of stiff to hard sandy clays and medium dense clayey sands at depth. Collapsible soils typically consist of loose fine sandy and silty soils that have been laid down by the action of flowing water, usually in alluvial fan deposits. Terrace deposits and fluvial deposits can also contain collapsible soil deposits. The soil particles are usually bound together with a mineral precipitate. The loose structure is maintained in the soil until a load is imposed on the soil and water is introduced. The water breaks down the inter-particle bonds and the newly imposed loading densifies the soil. The soil samples retrieved during this study did not have the characteristics of collapsible soil and the site is not in an area known for such soils. The potential for collapsible soils underlying the site is therefore considered to be low.
5.2.5 Flooding

The site is located in an area of minimal flooding hazard. FIRM (2009) has mapped the site vicinity as in an urbanized area outside the 0.2% annual chance floodplain, as shown on attached Figure 8, *Flood Hazard Map*. Based on this mapping, the site should be considered to have a very low hazard potential for seasonal flooding. The flood hazard of the site should also be confirmed by the project Civil Engineer, and a flood specialist should be contacted if a more in-depth flooding analysis is desired.

5.2.6 Soil Erosion

Present construction techniques and agency requirements have provisions to limit soil erosion and resultant siltation during construction. These measures will reduce the potential for soil erosion at the site during the various construction phases. Long-term erosion at the site will be reduced by designing landscaping and hardscape areas such as parking lots and walkways with appropriate surface drainage facilities.

5.2.7 Other Geologic Hazards

Due to the site’s location and geology, subsurface soil conditions, groundwater levels and land use factors, the site is not subject to the potential geologic hazards of loss of mineral resources, naturally occurring asbestos (NOA), volcanism, cyclic softening of soils or loss of unique geologic features.
6.0 CONCLUSIONS AND RECOMMENDATIONS

The following conclusions and recommendations are based upon the analysis of the information gathered during the course of this study and our understanding of the proposed improvements.

6.1 Conclusions

The site is considered suitable from a geotechnical and geologic perspective for the proposed improvements provided the recommendations of this report are incorporated into the design and implemented during construction. The predominant geotechnical and geological issues that will need to be addressed at this site are summarized below.

Seismic Considerations – The site is located within a seismically active region and should be designed to account for earthquake ground motions, as described in this report.

Expansive Soils - The near-surface native soils at the site appear to have a moderate to high expansion potential. As a result, interior slabs-on-grade should be steel reinforced to resist expansion pressures as well as be supported on a minimum 18-inch thickness of select, non-expansive fill. Moisture conditioning of the fill and upper processed cut surfaces should also be performed, and import fill should be non-expansive.

Winter Construction – In the event grading occurs in the winter rainy season, appropriate erosion control measures will be required and weatherproofing of the building pad and foundation excavations should be considered. Winter rains will also impact foundation excavations and underground utilities.

Other potential geotechnical considerations, including those that should not significantly impact the project are explained below.

Liquefaction and Dynamic Settlements – We do not anticipate any significant liquefaction or other dynamic settlements to occur at the project site.

Groundwater – Groundwater at the site is anticipated to be sufficiently deep so as not be problematic with placement and/or construction of the anticipated structures and most utility trenches.

Utility Connections – As a general suggestion, where utility damage during a design seismic event may be an issue, the Structural Engineer may wish to consider using utility connections at building perimeters designed for
up to one inch of potential movement in any direction where the utility enters the buildings. This flexibility would help accommodate potential differential movement during a seismic event.

### 6.2 Seismic Parameters

The site is located within a seismically active region and should be designed to account for earthquake ground motions as described in this report. For seismic analysis of the proposed site in accordance with the seismic provisions of the 2013 California Building Code (CBC), we recommend the following:

<table>
<thead>
<tr>
<th>Table 4: Seismic Design Parameters Based on 2013 CBC (per ASCE 7-10)</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Item</strong></td>
</tr>
<tr>
<td>Site Class</td>
</tr>
<tr>
<td>Mapped Spectral Response Accelerations</td>
</tr>
<tr>
<td>Short Period, SS</td>
</tr>
<tr>
<td>1-second Period, $S_1$</td>
</tr>
<tr>
<td>Site Coefficient, $F_a$</td>
</tr>
<tr>
<td>Site Coefficient, $F_v$</td>
</tr>
<tr>
<td>MCE ($S_{M1}$)</td>
</tr>
<tr>
<td>MCE ($S_{M2}$)</td>
</tr>
<tr>
<td>Design Spectral Response Acceleration</td>
</tr>
<tr>
<td>Short Period, $S_D$</td>
</tr>
<tr>
<td>1-second Period, $S_{D1}$</td>
</tr>
<tr>
<td>Peak Ground Acceleration ($PGA_M$)</td>
</tr>
</tbody>
</table>

Note: the Seismic Design Category is D for buildings in all Risk Categories I, II, III, and IV.

### 6.3 Site Grading

#### 6.3.1 Site Preparation and Demolition

Site grading should be performed in accordance with these recommendations. A pre-construction conference should be held at the jobsite with representatives from the owner, general contractor, grading contractor, and Geosphere prior to starting the stripping and demolition operations at the site.

Minor site grading will be required to prepare the site for the proposed improvements. The site should be cleared of vegetation, organic topsoil, debris, and other deleterious materials within the proposed development
area. The grading contractor should be aware that there is a potential for encountering buried objects and underground utilities at the site.

Buried objects and debris should be removed from the site. The resulting excavations should be backfilled with properly compacted fill or other material approved by the Geotechnical Engineer. Soil from the over-excavation areas can be stockpiled and utilized for fill at the site if the material is non-expansive and meets the requirements of backfill materials.

Existing underground utilities to be abandoned at the site should be properly grouted closed, or removed as needed. If the utilities are removed, the excavations should be backfilled with properly compacted fill or other material approved by the Geotechnical Engineer.

6.3.2 Project Compaction Recommendations

The following table summarizes the recommended minimum compaction requirements for this project. Not all soils, aggregates, and scenarios listed below may be applicable for this project. Specific grading recommendations will be discussed individually within applicable sections of this report.

<table>
<thead>
<tr>
<th>Description</th>
<th>Percent Relative Compaction</th>
<th>Minimum Percent Above/below Optimum Moisture Content</th>
</tr>
</thead>
<tbody>
<tr>
<td>Building Pads, Onsite Soil –scarified or used as fill</td>
<td>90</td>
<td>+ 3</td>
</tr>
<tr>
<td>Building Pads, Subgrade (upper 6 inches)</td>
<td>90</td>
<td>+ 3</td>
</tr>
<tr>
<td>Building Pads, Import Select Fill</td>
<td>90</td>
<td>+ 2</td>
</tr>
<tr>
<td>Concrete Hardscape, Subgrade Soil (upper 6 inches)</td>
<td>90</td>
<td>+ 3</td>
</tr>
<tr>
<td>Concrete Hardscape, Subgrade Soil (upper 6 inches) subject to vehicular traffic</td>
<td>95</td>
<td>+ 3</td>
</tr>
<tr>
<td>Concrete Hardscape, Class 2 Baserock</td>
<td>90</td>
<td>+ 2</td>
</tr>
<tr>
<td>Concrete Hardscape, Class 2 Baserock subject to vehicular traffic</td>
<td>95</td>
<td>+ 2</td>
</tr>
<tr>
<td>Landscape Areas</td>
<td>85</td>
<td>+ 3</td>
</tr>
<tr>
<td>Underground Utility Backfill, Onsite or Import</td>
<td>90</td>
<td>+ 3</td>
</tr>
</tbody>
</table>

6.3.3 General Grading

Imported soil should be non-expansive, having a Plasticity Index of 15 or less, an R-Value greater than 40, and contain sufficient fines so the soil can bind together. Imported materials should be free of organic materials and
debris, and should not contain rocks or lumps greater than three inches in maximum size. Import fill materials should be approved by the Geotechnical Engineer prior to use onsite.

Final grading should be designed to provide positive drainage away from structures. Soil areas within 10 feet of proposed structures should slope at a minimum of five percent away from the buildings. Roof leaders and downspouts, if any, should discharge onto paved surfaces sloping away from the structure or into a closed pipe system channeled away from the structure to an approved collector or outfall.

6.3.4 Building Pad Grading/Preparation

After cutting subgrade soil to the required depth, the building pad subgrade soil should be scarified to a depth of at least eight inches, moisture conditioned to at least three percent over optimum moisture, and compacted to the project compaction requirements listed on Table 5 as determined based on ASTM D1557 (Modified Proctor). If loose or soft soil is encountered, these soils should be removed to expose firm soil and backfilled with engineered fill. The upper 18 inches of the building pad should be constructed with select non-expansive engineered fill. Onsite soils consist of generally stiff, moderately to highly expansive clay within the upper approximately five feet and are not suitable for reuse as select, non-expansive material. Engineered fill should be placed in maximum eight-inch thick, pre-compacted lifts. The fill should be moisture conditioned and thoroughly mixed during placement to provide uniformity in each layer, and subsequently compacted per the requirements listed in Table 5.

6.3.5 Grading Hardscape Areas

Areas to receive concrete hardscape should be scarified to a depth of eight inches below existing grade or final subgrade, whichever is lower. Scarified areas should be moisture conditioned and compacted per the recommendations presented in Table 5. Where required, engineered fill should be placed and compacted to reach design subgrade elevation.

6.3.6 Site Winterization and Unstable Subgrade Conditions

If grading occurs in the winter rainy season, unstable and unworkable subgrade conditions may be present and compaction of on-site soils may not be feasible. These conditions may be remedied using appropriate soil admixtures, such as lime or other admixtures. More detailed recommendations can be provided during construction. Stabilizing subgrade in small, isolated areas can be accomplished with the approval of the
Geotechnical Engineer by over-excavating one foot, placing Tensar BX1100 or TriAx TX-140 geogrid or equivalent geogrid on the soil, and then placing 12 inches of Class 2 baserock on the geogrid. The upper six inches of the baserock should be compacted to at least 90 percent relative compaction. Alternatively, a non-woven stabilization geotextile such as Mirafi 500X overlain by a minimum 18 inches of baserock may be substituted for geogrid and baserock.

6.4 Utility Trench Construction

Utility trenches may be backfilled with approved native soil above the utility bedding and shading materials compacted to the recommended compaction presented in Table 5. If rocks larger than four inches in maximum size are encountered, these should be removed from the fill prior to placement in the utility trenches. Utility bedding and shading compaction requirements should be in conformance with the requirements of the local agencies having jurisdiction and as recommended by the pipe manufacturers. Jetting of trench backfill is not recommended.

Pea gravel, rod mill (pea gravel with sand), or other similar self-compacting material should NOT be utilized as trench backfill at the site. This material may act as a conduit for subsurface moisture migration. Utility trenches ideally should be completely sealed/cut off with concrete, clayey soil, sand-cement slurry, or controlled density fill (CDF) where the utility enters the building under the perimeter foundation. This would reduce the potential for migration of water beneath the building through the shading material in the utility trench.

If rain is expected and the trench will remain open, the bottom of the trench may be lined with one to two inches of gravel. This would provide a working surface in the trench bottom. The trench bottom may have to be sloped to a low point to pump the water out of the trench.

6.5 Foundation Recommendations

6.5.1 Shallow Foundations

The proposed building can be supported on conventional continuous and/or isolated spread footings bearing on undisturbed firm/very stiff native soil or properly compacted fill. Footings should be founded a minimum of 24 inches below lowest adjacent finished grade. Continuous footings should have a minimum width of at least 18 inches, and isolated column footings should have a minimum width of at least 24 inches. In addition, footings located adjacent to other footings or utility trenches should bear below an imaginary 1.5:1 (horizontal to
vertical) plane projected upward from the bottom edge of the adjacent footings or utility trenches. Footing reinforcement should be determined by the project Structural Engineer.

Footings should be designed for the following allowable bearing pressures, assuming design Factors of Safety of 3.0, 2.0 and 1.5 for dead loads, dead plus live loads and total loads, respectively, from the calculated ultimate bearing pressure.

Table 6: Allowable Bearing Pressures for Spread Footings

<table>
<thead>
<tr>
<th>Load Condition</th>
<th>Allowable Bearing Pressure (psf)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Dead Load</td>
<td>2,000</td>
</tr>
<tr>
<td>Dead plus Live Loads</td>
<td>3,000</td>
</tr>
<tr>
<td>Total Loads (including wind or seismic)</td>
<td>4,000</td>
</tr>
</tbody>
</table>

If site preparation and foundation observation services are conducted as outlined in the Geotechnical Study report, vertical static settlement is expected to be less than one inch for footings bearing within the materials described in the report and designed to the aforementioned allowable bearing pressures. Differential settlement across the structure is not expected to exceed about ½ this value within a 30 foot span.

Geosphere personnel should be retained to observe and confirm that footing excavations prior to formwork and reinforcing steel placement bear in soils suitable for the recommended maximum design bearing pressure. If unsuitable soil is present, the excavation should be deepened until suitable supporting material is encountered. The over excavation should be backfilled using engineered soil or lean concrete (or a sand-cement slurry mix acceptable to the Geotechnical Engineer) up to the bottom of the footing concrete.

Footing excavations should have firm bottoms and be free from excessive slough prior to concrete or reinforcing steel placement. Care should also be taken to prevent excessive wetting or drying of the bearing materials during construction. Extremely wet or dry or any loose or disturbed material in the bottom of the footing excavations should be removed prior to placing concrete. If construction occurs during the winter months, a thin layer of concrete (sometimes referred to as a rat slab) could be placed at the bottom of the footing excavations. This will protect the bearing soil and facilitate removal of water and slough if rainwater fills the excavations.
6.5.2 Lateral Resistance

Shallow foundations can resist lateral loads with a combination of bottom friction and passive resistance. An allowable coefficient of friction of 0.35 between the base of the foundation elements and underlying material is recommended. In addition, an ultimate passive resistance equal to an equivalent fluid weighing 350 pounds per cubic foot (pcf) acting against the foundation may be used for lateral load resistance against the sides of footings perpendicular to the direction of loading where the footing is poured neat against undisturbed material. The top foot of passive resistance at foundations not adjacent to pavement or hardscape should be neglected. In order to fully mobilize this passive resistance, a lateral footing deflection on the order of one to two percent of the embedment of the footing is required. If it is desired to limit the amount of lateral deflection to mobilize the passive resistance, a proportional safety factor should be applied. The friction between the bottom of a slab-on-grade floor and the underlying soil should not be utilized to resist lateral forces.

6.6 Concrete Slabs-on-Grade

6.6.1 Interior Concrete Slabs

Non-structural concrete slab-on-grade floors should be a minimum of five inches in thickness. The actual required slab thickness and reinforcement should be determined by the project Structural Engineer, but as a minimum, reinforcement for interior floor slabs should consist of No. 4 bars spaced at 18-inch intervals each way. In addition, the floor slab should be underlain by a minimum 18-inch thickness of select, non-expansive fill. The non-expansive fill should extend to a minimum of five feet beyond the building envelope or the edge of any perimeter hardscape, whichever is greater. The floor slab should also be tied to the perimeter footings.

Care should be taken to maintain the minimum recommended moisture content in the subgrade until floor slabs and/or engineered fills are constructed. Positive drainage should also be developed away from the building to prevent water from ponding along the perimeter and affecting future floor slab performance. We recommend a positive cutoff in utility trenches at the structure/building lines to reduce the potential for water migrating through the utility trench backfill to areas under the building.

Slab-on-grade concrete floors with moisture sensitive floor coverings should be underlain by a moisture retarder system constructed between the slab and subgrade. Such a system could consist of four inches of free-draining gravel, such as 3/4-inch, clean, crushed, uniformly graded gravel with less than three percent passing No. 200 sieve, or equivalent, overlain by a relatively impermeable vapor retarder placed between the subgrade soil and
the slab. The vapor retarder should be at least 10-mil thick and should conform to the requirements for ASTM E 1745 Class C Underslab Vapor Retarders (e.g., Griffolyn Type 65, Griffolyn Vapor Guard, Moistop Ultra C, or equivalent). If additional protection is desired by the owner, a higher quality vapor barrier conforming to the requirements of ASTM E 1745 Class A, with a water vapor transmission rate less than or equal to 0.006 gr/ft²/hr (i.e., 0.012 perms) per ASTM E 96 (e.g., 15-mil thick “Stego Wrap Class A”), or to Class B (Griffolyn Type 85, Moistop Ultra B, or equivalent) may be used in place of a Class C retarder.

The vapor retarder or barrier should be placed directly under the slab. A capillary rock layer or rock cushion is not required if a Class A barrier is used beneath the floor slab, and a sand layer is not required over the vapor retarder from a geotechnical standpoint. If sand on top of the vapor retarder is required by the design structural engineer, we suggest the thickness be minimized to less than one inch. If construction occurs in the winter months, water may pond within the sand layer since the vapor retarder may prevent the vertical percolation of rainwater. Sand and crushed rock layers may be considered to comprise part of the thickness of the recommended non-expansive fill underlying the interior slab.

ASTM E1643 should be utilized as a guideline for the installation of the vapor retarder. During construction, all penetrations (e.g., pipes and conduits,) overlap seams, and punctures should be completely sealed using a waterproof tape or mastic applied in accordance with the vapor retarder manufacturer’s specifications. The vapor retarder or barrier should extend to the perimeter cutoff beam or footing.

6.6.2 Exterior Concrete Flatwork

Exterior concrete flatwork with pedestrian traffic should be at least four inches thick. If an underlying baserock layer is used, the baserock layer should be at least four inches thick and should be cut off from direct moisture transmission from directly adjacent landscape areas by use of a concrete cutoff or a deep header board extending at least two inches below the base of the aggregate base layer. In any case, due to expansive subgrade conditions, we recommend that at least the upper 12-inches of flatwork subgrade be moisture conditioned to a minimum of three to five percent over optimum moisture. This moisture should be verified within 24 hours of placing baserock over the subgrade and again prior to concrete pouring.

The flatwork should be reinforced to reduce potential tripping hazards, but welded wire mesh should not be utilized. The flatwork should be doweled into the building foundation adjacent to doorways and into curbs to prevent possible tripping hazards. We also recommend that control joints be designed and constructed in
accordance with American Concrete Institute (ACI) recommendations. In general, this would require control joints on a maximum spacing of approximately 10-feet by 10-feet, with a closer spacing depending on the shape of the concrete slab.

If possible, the building should have a concrete apron around the building to reduce the potential for surface water percolating down through the soil adjacent to the building. Landscaping adjacent to the building should be avoided if possible. Typically, landscape areas directly adjacent to buildings are surrounded by concrete flatwork, which can prevent irrigation and rainwater from flowing away from the building. The ponded water trapped by the surrounding flatwork could percolate down or possibly laterally through perimeter foundation construction joints or pipe sleeves and travel beneath the floor slab. Drainage can be provided in these landscape areas, but the top of the drains should be located below the building pad subgrade elevation.

6.7 Site Retaining Walls

6.7.1 Lateral Earth Pressures

The following recommended lateral earth design pressures are based on the assumption that on-site soils will be used as wall backfill. For a level backfill condition, unrestrained walls (i.e., walls that are free to deflect or rotate) should be designed to resist an equivalent fluid pressure of 40 pounds per cubic foot. Restrained walls for a level backfill condition should be designed to resist an equivalent fluid pressure of 40 pounds per cubic foot, plus an additional uniform lateral pressure of 8H pounds per square foot, where H = height of backfill above the top of the wall footing, in feet. A seismic increment is not required for site walls retaining less than six feet (higher walls are not anticipated at this site).

Walls with inclined backfill should be designed for an additional equivalent fluid pressure of one pound per cubic foot for every two degrees of slope inclination from horizontal. Walls subjected to surcharge loads should be designed for an additional uniform lateral pressure equal to 0.33 times the anticipated surcharge load for unrestrained walls, and 0.50 times the anticipated surcharge load for restrained walls.

In addition, an ultimate passive resistance equal to an equivalent fluid weighing 350 pounds per cubic foot (pcf) acting against the foundation may be used for lateral load resistance against the sides of the footing perpendicular to the direction of loading where the footing is poured neat against undisturbed material (i.e., native soils or engineered fills). The top foot of passive resistance at foundations not adjacent to and confined
by pavement, interior floor slab, or hardscape should be neglected. In order to fully mobilize this passive resistance, a lateral footing deflection on the order of one to two percent of the embedment of the footing is required. If it is desired to limit the amount of lateral deflection to mobilize the passive resistance, a proportional safety factor should be applied.

The lateral earth pressures herein do not include any factor of safety and are not applicable for submerged soils/hydrostatic loading. Additional recommendations may be necessary if submerged conditions are to be included in the design.

6.7.2 Retaining Wall Foundations

Retaining and below-grade walls may be founded on spread footing foundations. Assuming a minimum 24-inch footing embedment below lowest adjacent grade, retaining wall footings may be designed using an allowable dead plus live load bearing capacity of 3,000 pounds per square foot (psf), with a one-third increase for additional transient seismic loads. An allowable coefficient of friction between the bottom of footing and underlying material of 0.35 may be used.

6.7.3 Retaining Wall Drainage

The aforementioned recommended lateral pressures assume that walls are fully back drained to prevent the build-up of hydrostatic pressures. To reduce the potential for hydrostatic loading on retaining and below-grade walls due to possible seasonal subsurface groundwater seepage, a subsurface drain system may be considered for construction behind below-grade walls. Alternatively, below-grade walls can be designed to accommodate an additional hydrostatic pressure increment.

If used, the drain system should consist of a minimum 12-inch width of free-draining granular soils containing less than five percent fines (by weight) passing a No. 200 sieve placed adjacent to the wall. The free-draining granular material should be graded to prevent the intrusion of fines (e.g., a Caltrans Class 2 permeable material) or encapsulated in a suitable filter fabric. A drainage system consisting of either weep holes or perforated drain lines (placed near the base of the wall) should be used to intercept and discharge water which would tend to saturate the backfill. An impervious soil should be used in the upper layer of backfill to reduce the potential for water infiltration. As an alternative, a prefabricated drainage structure such as a geocomposite drain (e.g., MiraDRAIN 6000) may be used as a substitute for the granular backfill adjacent to the wall.
The retaining wall drainage system should be sloped to discharge by gravity to an adjacent storm drain system or other appropriate facility.

6.7.4 Retaining Wall Backfill Compaction

Retaining wall backfill less than five feet deep should be compacted to at least 90 percent relative compaction using light compaction equipment. Backfill greater than a depth of five feet should be compacted to at least 95 percent relative compaction. If heavy compaction equipment is used, the walls should be appropriately designed to withstand loads exerted by the heavy equipment, and/or temporarily braced. Over compaction or surcharge from heavy equipment too close to the wall may cause excessive lateral earth pressures which could result in excessive outward wall movement.

6.8 Plan Review

It is recommended that Geosphere be provided the opportunity to review the foundation, grading, pavement, and drainage plans prior to construction. The purpose of this review is to assess the general compliance of the plans with the recommendations provided in this report and the incorporation of these recommendations into the project plans and specifications.

6.9 Observation and Testing During Construction

It is recommended that Geosphere be retained to provide observation and testing services during site preparation, site grading, utility construction, foundation excavation, pavement section preparation and to observe final site drainage. This is to observe compliance with the design concepts, specifications and recommendations, and to allow for possible changes in the event that subsurface conditions differ from those anticipated prior to the start of construction.

6.10 Validity of Report

This report is valid for three years after publication. If construction begins after this time period, Geosphere should be contacted to confirm that the site conditions have not changed significantly. If the proposed development differs considerably from that described above, Geosphere should be notified to determine if additional recommendations are required. Additionally, if Geosphere is not involved during the geotechnical aspects of construction, this report may become wholly or in part invalid since Geosphere’s geotechnical personnel need to verify that the subsurface conditions anticipated preparing this report are similar to the
subsurface conditions revealed during construction. Geosphere’s involvement should include foundation, grading, and pavement plan review; observation of foundation excavations; grading observation and testing; testing of utility trench backfill and retaining wall backfill as applicable to the project; and subgrade preparation in hardscape areas.
7.0 LIMITATIONS AND UNIFORMITY OF CONDITIONS

The recommendations of this report are based upon the soil and conditions encountered in the borings. If variations or undesirable conditions are encountered during construction, Geosphere should be contacted so that supplemental recommendations may be provided.

This report is issued with the understanding that it is the responsibility of the owner or his representatives to see that the information and recommendations contained herein are called to the attention of the other members of the design team and incorporated into the plans and specifications, and that the necessary steps are taken to see that the recommendations are implemented during construction.

The findings and recommendations presented in this report are valid as of the present time for the development as currently proposed. However, changes in the conditions 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. Accordingly the findings and recommendations presented in this report may be invalidated, wholly or in part, by changes outside our control. Therefore, this report is subject to review by Geosphere after a period of three (3) years has elapsed from the date of issuance of this report. In addition, if the currently proposed design scheme as noted in this report is altered, Geosphere should be provided the opportunity to review the changed design and provide supplemental recommendations as needed.

Recommendations are presented in this report which specifically request that Geosphere be provided the opportunity to review the project plans prior to construction and that we be retained to provide observation and testing services during construction. The validity of the recommendations of this report assumes that Geosphere will be retained to provide these services.

This report was prepared upon your request for our services, and in accordance with currently accepted geotechnical engineering practice. No warranty based on the contents of this report is intended, and none shall be inferred from the statements or opinions expressed herein.

The scope of our services for this report did not include an environmental assessment or investigation for the presence or absence of wetlands or hazardous or toxic materials in the soil, surface water, groundwater or air,
on, below or around this site. Any statements within this report or on the attached figures, logs or records regarding odors noted or other items or conditions observed are for the information of our client only.
8.0 REFERENCES


Association of Bay Area Governments (ABAG), Liquefaction Susceptibility maps, based on mapping by William Lettis and Associates and USGS by Witter et al. (2006), website: http://www.abag.ca.gov

Association of Bay Area Governments (ABAG) website, Landslide Maps, www.abag.ca.gov

California Building Code, 2013, California Code of Regulations, Title 24, Chapter 16.

California Department of Transportation (Caltrans), California Standard Specifications, 2010.

California Department of Transportation (Caltrans), Highway Design Manual, 2008.

California Department of Water Resources website, Planning and Local Assistance, Groundwater Data, http://wdl.water.ca.gov


Publications may have been used as general reference and not specifically cited in the report text.
FIGURES

Figure 1 – Site Vicinity Map
Figure 2 – Site Plan and Site Geologic Map
Figure 3 – Site Vicinity Geologic Map
Figure 4 – Regional Fault Map
Figure 5 – Schematic Geologic Cross Section A-A’
Figure 6 – Liquefaction Susceptibility Map
Figure 7 – Existing Landslide Map
Figure 8 – Flood Hazard Map
Figure 1: Site Vicinity Map

Willow Cove Elementary School
1880 Hanlon Way
Pittsburg, California

Source: Honker Bay, CA 7.5 x 7.5 minute quadrangle topographic map, USGS (2015)

Site Vicinity Map
91-03477-A
May 2015

Geesyphere Consultants, Inc
Figure 3

Source: United States Geological Survey, Geologic Map of Alameda County, USGS Scientific Map Investigations 2918

Willow Cove Elementary School
1880 Hanlon Way
Pittsburg, California

Af - Artificial Fill
Qpa - Alluvium, Pleistocene
Qha - Alluvium, Holocene

Site Vicinity Geologic Map
91-03477-A
May 2015

Figure 3
Figure 4: Regional Fault Map

Base Map Reference: California Geological Survey - 2010 Fault Activity Map of California

Site: Willow Cove Elementary School
1880 Hanlon Way
Pittsburg, California

Site Number: 0D\\n201591-03477-A

Fault Activity:
- Displacement during historical time
- Faults showing evidence of displacement during historical time
- Indicated Quaternary landform faults in this category show evidence of displacement during historical time. May be a consequence of Pleistocene or Holocene fault displacement near fault/landform age.
- Faults without recognized Pleistocene or Holocene fault age or evidence of displacement during Quaternary time.

Geologic Time Scale:
- Pre-Quaternary
- Early Quaternary
- Late Quaternary
- Last Interglacial
- Holocene
- Late Pleistocene
- Middle Pleistocene
- Early Pleistocene
- Middle Miocene
- Late Miocene
- Early Miocene
- Late Eocene
- Early Eocene
- middle Eocene
- Early Eocene
- Early Cretaceous
- Late Cretaceous
- Early Cretaceous
- Late Jurassic
- Early Jurassic
- Late Triassic
- Early Triassic
- Late Permian
- Early Permian
- Late Carboniferous
- Early Carboniferous
- Late Devonian
- Early Devonian
- Late Silurian
- Early Silurian
- Late Ordovician
- Early Ordovician
- Early Cambrian
- Late Cambrian
- Early Cambrian
- Late Ediacaran
- Early Ediacaran
- Early Precambrian
- Late Precambrian
- Early Precambrian
- Late Proterozoic
- Early Proterozoic
- Late Archean
- Early Archean
- Late Hadean
- Early Hadean

Fault Symbol:
- Fault line
- Fault block
- Fault plane
- Fault scarp
- Fault displacement
- Fault slip
- Fault strike
- Fault dip
- Fault direction
- Fault movement

Fault Activity:
- Fault slip on faults
- Fault displacement on faults
- Fault slip on fault segments
- Fault displacement on fault segments
- Fault slip on fault planes
- Fault displacement on fault planes
- Fault slip on fault scarp
- Fault displacement on fault scarp
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- Fault displacement on fault movement
- Fault slip on fault line
This map is intended for planning only and is not intended to be site specific. Rather, it depicts the general risk within neighborhoods and the relative risk from community to community.
Zone X - Areas determined to be outside the 0.2% annual chance floodplain.
APPENDIX A

FIELD EXPLORATION
Key to Boring Log Symbols
Boring Logs
### Key to Exploratory Boring Logs

**Material Types**
- **Coarse Grained Soils**
  - Greater than 50% Coarse Fraction
  - Retained on No. 200 Sieve
  - Assumed as Gravel

**Criteria for Assigning Soil Group Names**
- Clean Gravels: Cu<4 and 1<5Cc<3
- Gravels with Fines: Cu<4 and/or [Cc<1 or Cc>3]
- Sands: Cu<6 and/or [Cc<1 or Cc>3]
- Silts and Clays: PI or [Cc<1 or Cc>3]

**Soil Group Names**
- **Group Symbol**
  - GM: Silty Gravel
  - GC: Clayey Gravel
  - GM: Silty Sand
  - SC: Clayey Sand
  - SW: Well-Graded Sand
  - SP: Poorly-Graded Sand
  - OL: Organic Silt
  - CH: Clay
  - ML: Silt
  - HL: Organic Clay

**General Notes**
1. The boring locations were determined by pacing, sighting and/or measuring from site features. Locations are approximate. Elevations of borings (if included) were determined by interpolation between plan contours or from another source that will be identified in the report or on the project site plan. The location and elevation of borings should be considered accurate only to the degree implied by the method used.
2. The stratification lines represent the approximate boundary between soil types. The transition may be gradual.
3. Water level readings in the drill holes were recorded at time and under conditions stated on the boring logs. This data has been reviewed and interpretations have been made in the text of this report. However, it must be noted that fluctuations in the level of the groundwater may occur due to variations in rainfall, tides, temperature and other factors at the time measurements were made.
4. The boring logs and attached data should only be used in accordance with the report.
<table>
<thead>
<tr>
<th>DEPTH (ft)</th>
<th>GRAPHIC LOG</th>
<th>MATERIAL DESCRIPTION</th>
<th>SAMPLE TYPE NUMBER</th>
<th>RECOVERY % (ROD)</th>
<th>SPT BLOW COUNTS (N VALUE)</th>
<th>POCKET PEN. (ft)</th>
<th>DRY UNIT WT. (pcf)</th>
<th>MOISTURE CONTENT (%)</th>
<th>LIQUID LIMIT</th>
<th>PLASTIC LIMIT</th>
<th>PLASTICITY INDEX</th>
<th>FINES CONTENT (%)</th>
<th>ATTERBERG LIMITS</th>
<th>PLASTICITY INDEX</th>
<th>FINES CONTENT (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td></td>
<td>6&quot; TOPSOIL (CL) SANDY CLAY: Dark brown, moist, stiff, w/ traces of fine sand.</td>
<td>MC 1-1</td>
<td>6-5-6 (11)</td>
<td>&gt;4.5</td>
<td></td>
<td></td>
<td>39</td>
<td>17</td>
<td>22</td>
<td>59</td>
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<tr>
<td>5</td>
<td></td>
<td>(CL) SILTY CLAY: Light brown, moist, hard, w/ increase in silt content. w/ tan veins.</td>
<td>MC 1-2</td>
<td>14-25-39 (64)</td>
<td>&gt;4.5</td>
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<tr>
<td>15</td>
<td></td>
<td>becomes very stiff sand lens @ 14.5-14.8'</td>
<td>MC 1-4</td>
<td>5-9-11 (20)</td>
<td>1.0 &amp; &gt;4.5</td>
<td></td>
<td>95</td>
<td>104</td>
<td>8</td>
<td>23</td>
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<tr>
<td>20</td>
<td></td>
<td>(SC) CLAYEY SAND: Olive brown, moist, medium dense</td>
<td>SPT 1-5</td>
<td>7-7-10 (17)</td>
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</tbody>
</table>

Bottom of borehole at 25.0 feet.
## Ground Description

The boring log shows the following layers:

1. **6" Topsoil**
   - Dark brown, moist, stiff.
   - SPT Blow Count: 5-6-6
   - Recovery: (12)

2. **Sandy Clay**
   - Light brown, moist, stiff, w/ fine sand
   - MC 2-1: 4-5-7
   - MC 2-2: 4.5
   - MC 2-3: 5-9-27
   - MC 2-4: 8-28-27

3. **Silty Clay**
   - Becomes very stiff.
   - SPT 2-5: 6-6-10

The bottom of the borehole is at 20.0 feet.

## Water Levels

- **Groundwater was not encountered.**

## Notes

- *Graphic Log*
- *Material Description*
- *Sample Type Number*
- *Recovery % (RQD)*
- *SPT Blow Counts (N Value)*
- *Pocket Pen. (tsf)*
- *Dry Unit Wt. (lb)*
- *Moisture Content (%)*
- *Liquid Limit (%)*
- *Plastic Limit (%)*
- *Atterberg Limits*
- *Plasticity Index (%)*
- *Fines Content (%)*

---

**Ground Elevation**: 60 ft

**Hole Size**: 6 inches

**Drilling Method**: Solid Flight

**Groundwater Levels**: At time of drilling ---

--- Groundwater was not encountered.

--- Groundwater was not encountered.

**Drilling Contractor**: Geo-Ex Subsurface Exploration

---

**Logged By**: NAA

**Checked By**: CTD

**Date Started**: 4/6/15

**Completed**: 4/6/15

---

**Project Number**: 91-03477-A

**Project Name**: Willow Cove Elementary School

**Project Location**: 1880 Hannan Way, Pittsburg, CA, 94565

**Project Location**: 1880 Hannan Way, Pittsburg, CA, 94565

**Client**: Pittsburg Unified School District

**Address**: 2001 Crow Canyon Road

**San Ramon, CA**
<table>
<thead>
<tr>
<th>DEPTH (ft)</th>
<th>GRAPHIC LOG</th>
<th>MATERIAL DESCRIPTION</th>
<th>SPT BLOW COUNTS (N VALUE)</th>
<th>RECOVERY % (RQD) (%)</th>
<th>POCKET PEN. (tsf)</th>
<th>DRY UNIT WT. (lb)</th>
<th>MOISTURE CONTENT (%)</th>
<th>LIQUID LIMIT (%)</th>
<th>PLASTIC LIMIT (%)</th>
<th>PLASTICITY INDEX</th>
<th>FINES CONTENT (%)</th>
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<tbody>
<tr>
<td>0</td>
<td></td>
<td>6&quot; TOPSOIL (CL)</td>
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<td>5</td>
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<td>SANDY CLAY: Dark-to-yellow brown, moist, stiff.</td>
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<td>becomes dark brown; UU = 3.4 ksf</td>
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<td>becomes light brown; hard</td>
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<td>w/ tan veins.</td>
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<td>25</td>
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<td>becomes olive brown; very stiff.</td>
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(Continued Next Page)
## SANDY CLAY:
Dark-to-yellow brown, moist, stiff. (continued)

<table>
<thead>
<tr>
<th>Depth (ft)</th>
<th>Material Description</th>
<th>SPT BLOW COUNTS (N VALUE)</th>
<th>RECOVERY %</th>
<th>POCKET PEN. (tsf)</th>
<th>DRY UNIT WT. (pcf)</th>
<th>MOISTURE CONTENT (%)</th>
<th>PLASTIC LIMIT</th>
<th>PLASTICITY INDEX</th>
<th>FINES CONTENT (%)</th>
<th>ATTERBERG LIMITS</th>
</tr>
</thead>
<tbody>
<tr>
<td>25</td>
<td>Becomes hard.</td>
<td>SPT 3-7</td>
<td>7-11-13</td>
<td>(24)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>30</td>
<td></td>
<td>SPT 3-8</td>
<td>8-11-16</td>
<td>(27)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>20</td>
</tr>
<tr>
<td>35</td>
<td></td>
<td>SPT 3-9</td>
<td>12-18-21</td>
<td>(39)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>40</td>
<td>Becomes very stiff.</td>
<td>SPT 3-10</td>
<td>8-12-14</td>
<td>(26)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>24</td>
</tr>
<tr>
<td>45</td>
<td></td>
<td>SPT 3-11</td>
<td>9-12-16</td>
<td>(28)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>50</td>
<td></td>
<td>SPT 3-12</td>
<td>8-11-13</td>
<td>(24)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Bottom of borehole at 50.0 feet.
APPENDIX B

LABORATORY TEST RESULTS
Moisture-Density-Porosity Report (2)
Liquid and Plastic Limits Test Report
Particle Size Distribution Report (Sieve Analysis)
Unconsolidated-Undrained Triaxial Test
Corrosivity Tests Summary
Moisture-Density-Porosity Report
Cooper Testing Labs, Inc. (ASTM D7263b)

<table>
<thead>
<tr>
<th>CTL Job No: 724-094a</th>
<th>Project No. 91-03477-A</th>
<th>By: RU</th>
</tr>
</thead>
<tbody>
<tr>
<td>Client: Geosphere Consultants</td>
<td>Date: 04/15/15</td>
<td></td>
</tr>
<tr>
<td>Project Name: Willow Cove ES</td>
<td>Remarks:</td>
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</tbody>
</table>

**Boring:**

<table>
<thead>
<tr>
<th>Sample</th>
<th>Depth, ft</th>
<th>Visual Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>Top</td>
<td>14.5-15</td>
<td>Light Olive Brown SAND</td>
</tr>
<tr>
<td>Bottom</td>
<td>14.5-15</td>
<td>Yellowish Brown CLAY w/ Sand</td>
</tr>
<tr>
<td>1.5</td>
<td></td>
<td>Dark Grayish Brown Lean CLAY</td>
</tr>
<tr>
<td>4.5-5</td>
<td></td>
<td>Dark Olive Brown CLAY w/ Sand</td>
</tr>
<tr>
<td>9-9.5</td>
<td></td>
<td>Yellowish Brown CLAY w/ Sand</td>
</tr>
<tr>
<td>18.5-20</td>
<td></td>
<td>Light Olive Brown CLAY</td>
</tr>
<tr>
<td>3-3.5</td>
<td></td>
<td>Sandy CLAY</td>
</tr>
<tr>
<td>21-22.5</td>
<td></td>
<td>Sandy CLAY</td>
</tr>
</tbody>
</table>

**Table: Actual G_s, Assumed G_s, Moisture, %, Wet Unit wt, pcf, Dry Unit wt, pcf, Dry Bulk Dens, (g/cc), Saturation, %, Total Porosity, %, Volumetric Water Cont, Θ_w,% & Volumetric Air Cont., Θ_a,%:**

<table>
<thead>
<tr>
<th>Series</th>
<th>Actual G_s</th>
<th>Assumed G_s</th>
<th>Moisture, %</th>
<th>Wet Unit wt, pcf</th>
<th>Dry Unit wt, pcf</th>
<th>Dry Bulk Dens, (g/cc)</th>
<th>Saturation, %</th>
<th>Total Porosity, %</th>
<th>Volumetric Water Cont, Θ_w, %</th>
<th>Volumetric Air Cont., Θ_a, %</th>
<th>Void Ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>2.70</td>
<td>2.70</td>
<td>7.5</td>
<td>102.6</td>
<td>95.4</td>
<td>1.53</td>
<td>26.3</td>
<td>43.4</td>
<td>11.4</td>
<td>32.0</td>
<td>0.77</td>
</tr>
<tr>
<td>2</td>
<td>2.70</td>
<td>2.70</td>
<td>22.9</td>
<td>127.4</td>
<td>103.7</td>
<td>1.66</td>
<td>98.5</td>
<td>38.5</td>
<td>38.0</td>
<td>0.6</td>
<td>0.63</td>
</tr>
<tr>
<td>3</td>
<td>2.70</td>
<td>2.70</td>
<td>17.6</td>
<td>128.5</td>
<td>109.3</td>
<td>1.75</td>
<td>87.4</td>
<td>35.2</td>
<td>30.8</td>
<td>4.4</td>
<td>0.54</td>
</tr>
<tr>
<td>4</td>
<td>2.70</td>
<td>2.70</td>
<td>21.8</td>
<td>128.5</td>
<td>105.5</td>
<td>1.69</td>
<td>98.2</td>
<td>37.4</td>
<td>36.8</td>
<td>0.7</td>
<td>0.60</td>
</tr>
<tr>
<td>5</td>
<td>2.70</td>
<td>2.70</td>
<td>22.8</td>
<td>127.4</td>
<td>103.8</td>
<td>1.66</td>
<td>98.4</td>
<td>38.5</td>
<td>37.8</td>
<td>0.6</td>
<td>0.63</td>
</tr>
<tr>
<td>6</td>
<td>19.5</td>
<td>115.5</td>
<td>99.8</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>7</td>
<td>15.8</td>
<td>99.8</td>
<td>40.8</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>8</td>
<td>26.9</td>
<td>99.8</td>
<td>25.2</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

**Note:** All reported parameters are from the as-received sample condition unless otherwise noted. If an assumed specific gravity (Gs) was used then the saturation, porosities, and void ratio should be considered approximate.

**Diagram:**

- **Moisture-Density**
  - The Zero Air-Voids curves represent the dry density at 100% saturation for each value of specific gravity.
  - Series 1, 2, 3, 4, 5, 6, 7, 8 are plotted on the graph.
  - The graph shows the relationship between Moisture Content, %, and Density, pcf.
### Boring:
- **Boring:** B3-8, B3-10
- **Sample:** 31-32.5, 41-42.5

### Visual Description:
- **Light Olive Brown Sandy CLAY**
- **Light Olive Brown Sandy CLAY**

### Actual $G_s$ | Assumed $G_s$
---|---
20.2 | 23.6

### Wet Unit wt, pcf

### Dry Unit wt, pcf

### Dry Bulk Dens., (g/cc)

### Saturation, %

### Total Porosity, %

### Volumetric Water Cont., $\Theta_w$, %

### Volumetric Air Cont., $\Theta_a$, %

### Void Ratio

### Moisture-Density-Porosity Report

---

**Moisture-Density-Porosity Report**

**Cooper Testing Labs, Inc. (ASTM D7263b)**

---

**ctl Job No:** 724-094b  
**Project No:** 91-03477-A  
**Client:** Geosphere Consultants  
**Project Name:** Willow Cove ES  
**Date:** 04/15/15  
**By:** RU

### Zero Air-Voids Curves, Specific Gravity

The Zero Air-Voids curves represent the dry density at 100% saturation for each value of specific gravity.
LIQUID AND PLASTIC LIMITS TEST REPORT

Dashed line indicates the approximate upper limit boundary for natural soils

![Graph showing liquid and plastic limits](image)

<table>
<thead>
<tr>
<th>MATERIAL DESCRIPTION</th>
<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>Yellowish Brown Sandy Lean CLAY</td>
<td>39</td>
<td>17</td>
<td>22</td>
<td>95.1</td>
<td>58.6</td>
<td>CL</td>
</tr>
<tr>
<td>Dark Grayish Brown Sandy Lean CLAY</td>
<td>45</td>
<td>16</td>
<td>29</td>
<td></td>
<td></td>
<td></td>
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</table>

**Project No.**: 724-094  **Client**: Geosphere Consultants

**Project**: Willow Cove ES - 91-03477-A

**Remarks**:
- Source: B1-1
- Source: B2-1

**Elev./Depth**: 2.5-3.0'

**Elev./Depth**: 1.5'

LIQUID AND PLASTIC LIMITS TEST REPORT

COOPER TESTING LABORATORY

Figure
### Particle Size Distribution Report

#### Soil Description
- **Source:** B1-1
  - Elev./Depth: 2.5-3.0’

- **Source:** B1-5
  - Elev./Depth: 18.5-20’

#### Coefficients
- **C_c**
- **C_u**

#### Remarks

#### Sieve Size Distribution

<table>
<thead>
<tr>
<th>Sieve Size</th>
<th>Percent Finer</th>
</tr>
</thead>
<tbody>
<tr>
<td>% Cobble</td>
<td>0.2</td>
</tr>
<tr>
<td>% Gravel</td>
<td>41.2</td>
</tr>
<tr>
<td>% Sand</td>
<td>58.6</td>
</tr>
<tr>
<td>% Silt</td>
<td>17</td>
</tr>
<tr>
<td>% Clay</td>
<td>39</td>
</tr>
</tbody>
</table>

#### Grain Size - mm

<table>
<thead>
<tr>
<th>Grain Size</th>
<th>Percent Finer</th>
</tr>
</thead>
<tbody>
<tr>
<td>% Cobble</td>
<td>0.2</td>
</tr>
<tr>
<td>% Gravel</td>
<td>41.2</td>
</tr>
<tr>
<td>% Sand</td>
<td>58.6</td>
</tr>
<tr>
<td>% Silt</td>
<td>17</td>
</tr>
<tr>
<td>% Clay</td>
<td>39</td>
</tr>
</tbody>
</table>

#### Remarks

- **Source:** B1-1
- Elev./Depth: 2.5-3.0
- **Source:** B1-5
- Elev./Depth: 18.5-20’
Sample Data

<table>
<thead>
<tr>
<th></th>
<th>1</th>
<th>2</th>
<th>3</th>
<th>4</th>
</tr>
</thead>
<tbody>
<tr>
<td>Moisture %</td>
<td>18.7</td>
<td></td>
<td></td>
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</tr>
<tr>
<td>Dry Den,pcf</td>
<td>112.5</td>
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<tr>
<td>Void Ratio</td>
<td>0.525</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Saturation %</td>
<td>97.8</td>
<td></td>
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<tr>
<td>Height in</td>
<td>5.06</td>
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<tr>
<td>Diameter in</td>
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<tr>
<td>Cell psi</td>
<td>4.0</td>
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</tr>
<tr>
<td>Strain %</td>
<td>15.00</td>
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<tr>
<td>Deviator, ksf</td>
<td>6.889</td>
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<tr>
<td>Rate %/min</td>
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<tr>
<td>in/min</td>
<td>0.050</td>
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<td></td>
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<tr>
<td>Boring</td>
<td>B-3-2</td>
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<tr>
<td>Depth ft</td>
<td>5-5.5</td>
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</tbody>
</table>

Visual Soil Description

<p>| | |</p>
<table>
<thead>
<tr>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Sample #</td>
<td></td>
</tr>
<tr>
<td>1</td>
<td>Dark Olive Brown CLAY w/ Sand</td>
</tr>
<tr>
<td>2</td>
<td></td>
</tr>
<tr>
<td>3</td>
<td></td>
</tr>
<tr>
<td>4</td>
<td></td>
</tr>
</tbody>
</table>

Remarks:

Note: Strengths are picked at the peak deviator stress or 15% strain which ever occurs first per ASTM D2850.
## Corrosivity Tests Summary

<table>
<thead>
<tr>
<th>Sample Location or ID</th>
<th>Resistivity @ 15.5 °C (Ohm-cm)</th>
<th>Chloride</th>
<th>Sulfate</th>
<th>pH</th>
<th>ORP</th>
<th>Sulfide</th>
<th>Moisture</th>
<th>Soil Visual Description</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>As Rec.</td>
<td>Min</td>
<td>Sat.</td>
<td>mg/kg</td>
<td>mg/kg</td>
<td>%</td>
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<tr>
<td></td>
<td></td>
<td>Dry Wt.</td>
<td>Dry Wt.</td>
<td>Dry Wt.</td>
<td></td>
<td></td>
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</tr>
<tr>
<td>B-2</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>962</td>
<td>34</td>
<td>0.0071</td>
<td>7.6</td>
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</tbody>
</table>

**Remarks:**


**Corrosivity Tests Summary**

- **Chloride:** mg/kg
- **pH:**
- **ORP (Redox):**
- **Sulfide:**
- **Moisture:**
- **Soil Visual Description:**

**B-2**

- **Resistivity @ 15.5 °C (Ohm-cm):** 962
- **Chloride:** 34 mg/kg
- **Sulfate:** 71 mg/kg
- **pH:** 7.6
- **ORP:**
- **Sulfide:**
- **Moisture:**
- **Soil Visual Description:** Dark Brown Sandy CLAY