Geotechnical Evaluation and Geologic Hazard Assessment

K.I. Jones Elementary School
2001 Winston Drive
Fairfield, California

Mr. Paul Speed
Fairfield Suisun Unified School District
2490 Hilborn Road  |  Fairfield, California 94534

July 28, 2017  |  Project No. 403070001

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Distribution:  (1) Addressee (via e-mail)
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1 INTRODUCTION
In accordance with your authorization, we have performed a geotechnical evaluation and geologic hazards assessment for the proposed improvements at K.I. Jones Elementary School at 2001 Winston Drive in Fairfield, California (Figure 1). This report presents the findings and conclusions from our geologic hazards assessment, and our geotechnical recommendations for improvements at the site.

2 SCOPE OF SERVICES
Our scope of services included the following:

- Review of readily available background materials, including geologic maps, aerial photographs, topographic data, and hazard maps.
- Site reconnaissance to observe the general site conditions and to mark the locations for our subsurface exploration.
- Coordination with Underground Service Alert (USA) to locate underground utilities in the vicinity of our subsurface exploration.
- Subsurface exploration consisting of six (6) hollow stem auger borings. A representative of Ninyo & Moore logged the subsurface conditions exposed in the borings and collected bulk and relatively undisturbed soil samples for laboratory testing.
- Laboratory testing on selected samples to evaluate in-situ soil moisture content and dry density, grain size distribution, Atterberg limits, expansion index, unconfined compressive strength, R-value, and soil corrosivity.
- Compilation and engineering analysis of the field and laboratory data, and the findings from our background review.
- Preparation of this report presenting our findings and conclusions regarding the potential geologic hazards and geotechnical conditions at the project site, and our geotechnical recommendations for proposed improvements.

3 SITE DESCRIPTION
The school campus is located at 2001 Winston Drive in Fairfield, California (Figure 1). The site is a rectangular-shaped parcel that covers approximately 9½ acres and is bordered to the south by Owen Street, to the west by Mankas Neighborhood Park, to the north by residential houses, and to the east by Winston Drive (Figure 2).

The site is developed with multiple low-rise school classroom and administration buildings and several detached modular buildings. The western portion of the site is generally athletic fields. The eastern portion of the site is generally paved for parking and access with minor
The central portion of the site generally consists of the existing buildings and paved play areas with some minor landscaping. Topographic variation at the site is relatively flat. The elevation on the site ranges from approximately 70 feet above mean sea level (MSL) in the northern portion to approximately 63 feet MSL at the southern portion of the site (Google Earth, 2017).

Based on a review of historical aerial photographs, the project site was primarily undeveloped open space ranch land prior to development. Photographs from 1968 indicate the area of the present day school and vicinity was undeveloped at that time. A small drainage channel, located approximately 500 feet to the west of the project site, is visible in the 1968 photographs.

4 PROJECT DESCRIPTION

Based on our review of a preliminary site plan (HMC Architects, 2017), the proposed improvements will consist of four new portable buildings near the north portion of the campus with a combined approximate footprint of 4,200 square feet, two new restroom additions near the central portion of the campus, a new administration and kindergarten classrooms building near the southern portion of the campus with an approximate footprint of 10,000 square feet, and a new parking lot near the southern portion of the campus (Figure 2). We anticipate that building loads will be low to moderate for the proposed structures and that site grading will consist of minor grade changes, about 2 feet or less. Related site improvements will include new underground utilities, hardscape, and landscaping improvements.

5 PREVIOUS SITE EVALUATIONS

As part of our geotechnical evaluation, we reviewed a geotechnical report previously prepared for the site by KC Engineering Company [KC Engineering] (2007). The KC Engineering evaluation included drilling of three exploratory borings in the southern portion of the campus to evaluate subsurface conditions for the proposed removal of existing portables and construction of new classrooms. The locations of the KC Engineering borings are shown on Figure 2 and their boring logs are provided in Appendix C. Their borings were drilled to depths of up to 39 feet and encountered soils described as firm to very stiff lean clay and sandy clay; very stiff fat clay; and medium dense clayey sand. Bedrock was encountered as a depth of about 30 feet in KC Engineering Boring 1 and was described as friable highly weathered mudstone. Fill was encountered in Boring 2 to a depth of about 5 feet. Groundwater was encountered at a depth of 8 feet in Borings 1 and 3. KC Engineering identified the primary geotechnical concern for the site to be the presence of highly expansive soils near the ground surface. Recommendations were provided for a select fill layer or lime-treatment to a depth of 24 inches below slabs-on-grade.
FIELD EXPLORATION AND LABORATORY TESTING

Our field exploration for this study included a site reconnaissance conducted on June 27, 2017, and subsurface exploration on July 5, 2017. The subsurface exploration consisted of six hollow stem auger borings. The approximate locations of the borings are shown on Figure 2.

The hollow stem auger borings were advanced to depths of about 5 to 30 feet below the existing grade. Borings B-1 through B-5 were drilled through existing asphalt pavement that consisted of approximately 2 inches of asphalt concrete overlying 5 to 6 inches of aggregate base. A representative of Ninyo & Moore logged the subsurface conditions exposed in the borings and collected relatively undisturbed and bulk soil samples from the borings. The samples were then transported to our geotechnical laboratory for testing. The borings were backfilled with grout after excavation in accordance with the boring permit. Descriptions of the subsurface materials encountered are presented in the following sections. Detailed logs of the borings are presented in Appendix A.

Laboratory testing of soil samples recovered from the borings included tests to evaluate in-situ soil moisture content and dry density, grain size distribution, Atterberg limits, expansion index, unconfined compressive strength, R-value, and soil corrosivity. The results of the in-situ moisture content and dry density tests are presented on the boring logs in Appendix A. The results of the other laboratory tests are presented in Appendix B.

GEOLeGIC AND SUBSURFACE CONDITIONS

Our findings regarding regional geologic setting, site geology, subsurface stratigraphy, and groundwater conditions at the subject site are provided in the following sections.

7.1 Regional Geologic Setting

The campus is north of Suisun Bay in the Coast Ranges geomorphic province of California. The Coast Ranges are comprised of several mountain ranges and structural valleys formed by tectonic processes commonly found around the Circum-Pacific belt. Basement rocks have been sheared, faulted, metamorphosed, and uplifted, and are separated by thick blankets of Cretaceous and Cenozoic sediments that fill structural valleys and line continental margins. The San Francisco Bay Area has several ranges that trend northwest, parallel to major strike-slip faults such as the San Andreas, Hayward, and Calaveras (Figure 3). Major tectonic activity associated with these and other faults within this regional tectonic framework consists primarily of right-lateral, strike-slip movement.
7.2 Site Geology

Review of available geologic maps and reports indicates that the project area is located on Pleistocene age alluvial fan deposits (Figure 4). According to regional geologic studies by Wiegers et al. (2006) and Graymer et al. (2002), the Pleistocene age alluvial deposits typically consist of poorly sorted, moderately to poorly bedded sand, gravel, silt, and clay, which are relatively denser than the younger Holocene age alluvial deposits in the area. The alluvial deposits are derived from the bedrock formations exposed in the nearby foothills and local mountains. The local bedrock formations are generally part of the Jurassic to Cretaceous age Great Valley Sequence, which consists of interbedded layers of sandstone, siltstone, mudstone, and shale.

7.3 Subsurface Conditions

The following sections provide a generalized description of the geologic units encountered during our subsurface evaluation. More detailed descriptions are presented on the logs in Appendix A. Two cross sections depicting our interpretation of the subsurface conditions are presented as Figures 5 and 6.

7.3.1 Fill

Fill was encountered in the borings from the ground surface to depths of approximately 2 feet. The fill encountered generally consisted of brown, moist, stiff to very stiff lean clay. Borings B-1 through B-5 were drilled through existing asphalt pavement that consisted of approximately 2 inches of asphalt concrete overlying 5 to 6 inches of aggregate base.

7.3.2 Alluvium

Alluvium was encountered in the borings from below to the fill to the depths explored of up to approximately 30 feet. The alluvium, as encountered, generally consisted of firm to hard lean clay and sandy lean clay.

7.4 Groundwater

Groundwater was measured at a depth of about 9½ feet below the existing grade in Boring B-4. Groundwater was encountered at a depth of about 18½ feet below the existing grade in Boring B-5 during drilling. Groundwater was not encountered in the other borings. KC Engineering encountered groundwater at a depth of 8 feet in their Borings 1 and 3. Fluctuations in the groundwater level across the site and over time may occur due to seasonal precipitation, tidal fluctuation, variations in topography or subsurface hydrogeologic conditions, or as a result of changes to nearby irrigation practices or groundwater pumping. In addition, seeps may be encountered at elevations above the observed groundwater levels due to perched groundwater.
conditions, leaking pipes, preferential drainage, or other factors not evident at the time of our exploration.

8 GEOLOGIC HAZARDS AND CONSIDERATIONS
This study considered a number of issues relevant to the proposed construction, including seismic hazards, flood hazards, landsliding and slope stability, naturally occurring asbestos, settlement of compressible soil layers from static loading, unsuitable materials, excavation characteristics, soil corrosivity, and expansive soils. These issues are discussed in the following subsections.

8.1 Seismic Hazards
The seismic hazards considered in this study include the potential for ground rupture due to faulting, seismic ground shaking, liquefaction, dynamic settlement, seismic slope stability, and tsunamis. These potential hazards are discussed in the following subsections.

8.1.1 Historical Seismicity
The site is located in a seismically active region. Figure 3 presents the location of the site relative to the epicenters of historic earthquakes with magnitudes of 5.5 or more from 1800 to 2000. Records of historic ground effects related to seismic activity (e.g. liquefaction, sand boils, lateral spreading, ground cracking) compiled by Knudsen et al. (2000), indicate that no ground effects related to historic seismic activity have been reported for the site vicinity.

8.1.2 Faulting and Ground Surface Rupture
There are numerous recognized faults in northern California. Selected characteristics, as evaluated by the 2007 Working Group on California Earthquake Probabilities (WGCEP, 2008), for recognized and postulated faults (Caltrans, 2017) near the site are presented in Table 1. The fault characteristics in the table are presented in order of decreasing peak ground acceleration (PGA) based on a deterministic seismic hazard analysis utilizing the Chiou & Youngs (2008) and Campbell & Bozorgnia (2008) attenuation relationships.

<table>
<thead>
<tr>
<th>Fault</th>
<th>ID</th>
<th>Type</th>
<th>Max Moment Magnitude</th>
<th>Distance to Site (kilometers)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Great Valley 04b Gordon Valley</td>
<td>104</td>
<td>Reverse</td>
<td>6.7</td>
<td>11.9</td>
</tr>
<tr>
<td>Los Medanos – Roe Island</td>
<td>120</td>
<td>Reverse</td>
<td>6.8</td>
<td>11.0</td>
</tr>
<tr>
<td>Cordelia</td>
<td>107</td>
<td>Strike Slip</td>
<td>6.5</td>
<td>6.9</td>
</tr>
</tbody>
</table>
The site is not located within an Alquist-Priolo Earthquake Fault Zone established by the state geologist (CGS, 2007) to delineate regions of potential ground surface rupture adjacent to active faults. As defined by the California Geological Survey (CGS), active faults are faults that have caused surface displacement within Holocene time, or within approximately the last 11,000 years (CGS, 2007). The closest fault rupture hazard zone is the one associated with the Cordelia Fault, which is located approximately 4¼ miles southwest of the site.

Based on our review of the referenced geologic maps, known active faults are not mapped on the campus and the site is not located within a fault-rupture hazard zone. Therefore, the probability of damage from surface fault rupture is considered to be low.

### 8.1.3 Strong Ground Motion

Based on historic activity, the potential for future strong ground motion at the site is considered significant. Seismic design criteria to address ground shaking are provided in Section 10.2. The peak ground acceleration (PGA) associated with the Maximum Considered Earthquake Geometric Mean (MCE<sub>G</sub>) was calculated in accordance with the American Society of Civil Engineers (ASCE) 7-10 Standard and the 2016 California Building Code (CBC). The MCE<sub>G</sub> peak ground acceleration with adjustment for site class effects (PGA<sub>M</sub>) was calculated as 0.773g using the USGS seismic design tool (USGS, 2017) that yielded a mapped MCE<sub>G</sub> peak ground acceleration of 0.773g for the site and a site coefficient (F<sub>P</sub><sub>PGA</sub>) of 1.000 for Site Class D.
8.1.4 Liquefaction and Strain Softening

The strong vibratory motions generated by earthquakes can trigger a rapid loss of shear strength in saturated, loose, granular soils of low plasticity (liquefaction) or in wet, sensitive, cohesive soils (strain softening). Liquefaction and strain softening can result in a loss of foundation bearing capacity or lateral spreading of sloping or unconfined ground. Liquefaction can also generate sand boils leading to subsidence at the ground surface. Liquefaction (or strain softening) is generally not a concern at depths more than 50 feet below ground surface. The site is in an area where the California Geological Survey has not yet evaluated or established seismic hazard zones for liquefaction. The Association of Bay Area Governments (ABAG) notes that the campus is in an area considered to have a low susceptibility to liquefaction based on regional studies (Knudsen et al., 2000; Witter et al., 2006). Our subsurface exploration indicates the site is generally underlain by stiff to very stiff lean clay which is not susceptible to liquefaction or strain softening behavior. Consequently, liquefaction and liquefaction-related seismic hazards (e.g., loss of foundation bearing capacity, sand boil induced ground subsidence, and lateral spreading) are not design considerations for the project. The potential for dynamic settlement due to seismic ground motion is addressed in the following section of this report.

8.1.5 Dynamic Settlement

The strong vibratory motion associated with earthquakes can also dynamically compact loose granular soil leading to surficial settlements. Dynamic settlement is not limited to the near surface environment and may occur in both dry and saturated sand and silt. Cohesive soil is not typically susceptible to dynamic settlement.

Based on the results of our subsurface exploration, dynamic settlement is not a design consideration for the project.

8.1.6 Seismic Slope Stability

No significant slopes are present on the site, as such, we do not regard seismic slope stability as a design consideration for this project.

8.1.7 Tsunamis and Seiches

Tsunamis are long wavelength seismic sea waves (long compared to ocean depth) generated by the sudden movements of the ocean floor during submarine earthquakes, landslides, or volcanic activity. The project is not located within a tsunami evacuation area
as shown on the Tsunami Inundation Map for Emergency Planning (State of California, 2009). Potential damage due to a tsunami is not a design consideration for this site.

Seiches are waves generated in a large enclosed body of water. Based on the lack of large enclosed bodies of water inland from the campus, the potential for damage due to seiches is not design a consideration.

8.2 Flood Hazards
Our review of Federal Emergency Management Agency (FEMA) Flood Insurance Rate Maps (FEMA, 2016) indicates that the campus is not located in a special flood hazard zone.

8.3 Landsliding and Slope Stability
The ground surface at the campus is relatively flat. Based on the existing topography, we do not regard slope stability or landsliding of existing slopes as a design consideration for this project.

8.4 Naturally Occurring Asbestos
According to State of California guidelines established by the Department of Toxic Substances and Control (2004 and 2005), a Preliminary Environmental Assessment (PEA) is recommended for school sites that are located within a 10-mile radius of any rock formation that may contain naturally occurring asbestos (NOA). The nearest mapped location of ultramafic rock from which NOA may be found is over 10 miles from the school campus. NOA is not a design consideration for this project.

8.5 Static Settlement
The findings from our subsurface exploration indicate that the campus is generally underlain by firm to hard lean clay and sandy lean clay. Based on the site topography and our project understanding, we anticipate fill placed to raise grades will be relatively minor, on the order of two feet or less. We anticipate that static settlement due to sustained loading will be tolerable for footings designed and constructed in accordance with the recommendations in this report and for minor amounts (two feet or less) of fill placement.

8.6 Unsuitable Materials
Fill materials that were not placed and compacted under the observation of a geotechnical engineer, or fill materials lacking documentation of such observation, are considered undocumented fill. Undocumented fill is unsuitable as a bearing material below foundations due
to the potential for differential settlement resulting from variable support characteristics or the potential inclusion of deleterious materials. Undocumented fill was encountered up to depths of about 2 feet below the ground surface during our subsurface exploration. Subgrade preparation and foundation embedment recommendations are provided to mitigate the undocumented fill concerns.

### 8.7 Corrosive/Deleterious Soil

An evaluation of the corrosivity of the on-site material was conducted to assess the impact to concrete and metals. The corrosion impact was evaluated using the results of limited laboratory testing on samples obtained during our subsurface study. Laboratory testing to quantify pH, resistivity, chloride, and soluble sulfate contents was performed on a sample of the near-surface soil. The results of the corrosivity tests are presented in Appendix B. California Department of Transportation (Caltrans) defines a corrosive environment as an area within 1,000 feet of brackish water or where the soil contains more than 500 parts per million (ppm) of chlorides, sulfates of 0.2 (2,000 ppm) percent or more, or pH of 5.5 or less (Caltrans, 2012). Based on these criteria, the site does not meet the definition of a corrosive environment. Ferrous metal will still undergo corrosion on site, but special mitigation measures are not needed. The criteria used to evaluate the deleterious nature of soil on concrete and recommendations from the American Concrete Institute (ACI) for sulfate exposure classes are presented in Table 2. Based on these criteria, the soil on site is defined as Exposure Class S0.

<table>
<thead>
<tr>
<th>Sulfate Content Percent by Weight</th>
<th>Exposure Class</th>
<th>Maximum Water to Cement Ratio</th>
<th>Minimum 28-day Compressive Strength</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.0 to 0.1</td>
<td>S0</td>
<td>N/A</td>
<td>2,500</td>
</tr>
<tr>
<td>0.1 to 0.2</td>
<td>S1</td>
<td>0.50</td>
<td>4,000</td>
</tr>
<tr>
<td>0.2 to 2.0</td>
<td>S2</td>
<td>0.45</td>
<td>4,500</td>
</tr>
<tr>
<td>&gt; 2.0</td>
<td>S3</td>
<td>0.45</td>
<td>4,500</td>
</tr>
</tbody>
</table>

Reference: American Concrete Institute (ACI) Committee 318 Table 19.3.1.1 and Table 19.3.2.1 (ACI, 2014)

### 8.8 Expansive Soils

Some clay minerals undergo volume changes upon wetting or drying. Unsaturated soils containing those minerals will shrink/swell with the removal/addition of water. The heaving pressures associated with this expansion can damage structures and flatwork. Laboratory
testing was performed on a select sample of the near-surface soil to evaluate the expansion index. The tests were performed in general accordance with the American Society of Testing and Materials (ASTM) Standard D 4829 (Expansion Index). The results of our laboratory testing indicate that the expansion index of the near-surface soil is 60, which is consistent with a medium expansion characteristic. To reduce the potential for differential movement and distress to the proposed improvements due to shrink/swell behavior, recommendations are provided for foundation embedment depths and to create a zone of material with low expansion potential by removing the existing soil and placing fill with low expansion characteristics below building slabs-on-grade, flatwork, and pavement. Alternatively, the on-site soil may be chemically treated by mixing the soil with lime to reduce the expansion characteristic.

9 CONCLUSIONS

Based on our review of the referenced background data, our site field reconnaissance, subsurface evaluation, and laboratory testing, it is our opinion that proposed construction is feasible from a geotechnical standpoint. Geotechnical considerations include the following:

- Our subsurface exploration encountered fill and alluvium. The fill encountered generally consisted of stiff to very stiff lean clay. The alluvium encountered generally consisted of stiff clay and sandy clay.

- Groundwater was encountered at depths of about 9½ feet and 18½ feet below the existing grade in borings B-4 and B-5, respectively. Groundwater was encountered in a previous evaluation by KC Engineering at a depth of 8 feet in their Borings 1 and 3. Variation and fluctuation in groundwater levels should be anticipated as discussed in Section 7.4.

- The site could experience a relatively large degree of ground shaking during a significant earthquake on a nearby fault. Seismic design criteria are presented in Section 10.2.

- Liquefaction, seismic strain softening, dynamic settlement, and other liquefaction-related hazards are not design considerations based on the subsurface conditions encountered.

- Tsunamis, seiches, and ground surface rupture due to faulting are not design considerations based on the location of the project.

- Excavations that remain unsupported are exposed to water, or encounter seepage, fractured rock, or granular soil may be unstable and prone to sloughing. Recommendations for excavation stabilization are provided.

- Excavations in the fill may encounter debris, rubble, oversize material, buried objects, or other potential obstructions.

- The site is not in a flood hazard zone.

- High concentrations of naturally occurring asbestos (NOA) in the natural soils at the site are unlikely based on the nearest mapped location of ultramafic rock from which NOA may be found is over 10 miles from the school campus.
• Static settlement should be tolerable for the proposed improvements provided that the proposed structures are supported on foundations that conform with our recommendations and fill placement to raise grades is less than 2 feet in height.

• Undocumented fill was encountered up to depths of about 2 feet below the ground surface during our subsurface exploration. Subgrade preparation and foundation embedment recommendations are provided to mitigate the undocumented fill concerns.

• Based on the results of our limited soil corrosivity tests during this study and Caltrans corrosion guidelines (2012), the site does not meet the definition of a corrosive environment.

• Expansion Index testing indicates that the near-surface soil on site has a medium expansion characteristic. Recommendations are provided to reduce the potential for expansive soil movement below proposed improvements.

10 RECOMMENDATIONS

The following sections present our geotechnical recommendations for the design and construction of the proposed improvements. The project improvements should be designed and constructed in accordance with these recommendations, applicable codes, and appropriate construction practices.

10.1 Earthwork

The earthwork should be conducted in accordance with the relevant grading ordinances having jurisdiction and the following recommendations. The geotechnical engineer should observe earthwork operations. Evaluations performed by the geotechnical engineer during the course of field operations may result in new recommendations, which could supersede the recommendations in this section.

10.1.1 Pre-Construction Conference

We recommend that a pre-construction conference be held to discuss the grading recommendations presented in the report. The owner and/or their representative, the architect, the engineer, Ninyo & Moore, and the contractor should be in attendance to discuss project schedule and earthwork requirements.

10.1.2 Site Preparation

Site preparation should begin with the removal of vegetation, utility lines, debris and other deleterious materials from areas to be graded. Tree stumps and roots should be removed to such a depth that organic material is generally not present. Clearing and grubbing should extend to the outside of the proposed excavation and fill areas. Rubble and excavated materials that do not meet criteria for use as fill should be disposed of in an appropriate
landfill. Existing utilities to be abandoned should be removed, crushed in place, or backfilled with grout.

Excavations resulting from removal of buried utilities, tree stumps, or obstructions should be backfilled with compacted fill in accordance with the recommendations in the following sections.

### 10.1.3 Subgrade Observations

Prior to placement of fill, erection of forms or placement of reinforcement for foundations, the client should request an evaluation of the exposed subgrade by Ninyo & Moore. Materials that are considered unsuitable shall be excavated under the observation of the geotechnical engineer in accordance with the recommendations in this section or the field recommendations of the geotechnical engineer.

Unsuitable materials include, but may not be limited to dry, loose, soft, wet, expansive, organic, or compressible natural soil; and undocumented or otherwise deleterious fill materials. Unsuitable materials should be removed from trench bottoms and below bearing surfaces to a depth at which suitable foundation subgrade, as evaluated in the field by the geotechnical engineer, is exposed.

### 10.1.4 Remedial Grading for Site Improvements

Laboratory testing indicates that the near-surface soil on site has a medium expansion characteristic. To reduce the potential for differential movement and distress to the proposed improvements due to shrink/swell behavior, a zone of material with low expansion potential should be created by removing the existing soil, as-needed, and placing fill with low expansion characteristics below building slabs-on-grade, flatwork, or pavement. The zone of low expansion fill should consist of select, low-expansion import fill conforming with Section 10.1.5. Alternatively, the on-site soil may be chemically treated by mixing the soil with lime as described in Section 10.1.6 to reduce the expansion characteristic and create the zone of low-expansion material.

The lateral limits of over-excavations or chemical treatment should extend a distance of 5 feet or more beyond the limits of the slab-on-grade and 2 feet or more beyond the limits of the flatwork or pavement. The zone of low expansion material should extend to a depth of 18 inches, or more, below building slabs-on-grade and pavement; and 12 inches below exterior flatwork. The aggregate base or capillary break gravel under building slabs, pavement, or exterior flatwork may be considered as part of the zone of low expansion material. The zone of exclusion/removal or lime treatment should be detailed on the
construction plans to reduce the potential that these recommendations are overlooked during construction bidding.

Undocumented fill was encountered in the borings to a depth of about 2 feet below the existing ground surface. Undocumented fill, where encountered, should be removed from below new building footings. Excavations should be backfilled with controlled low strength material (CLSM) as per Section 10.1.5. Alternatively the footings may be extended to bear on suitable alluvium. The depth of the undocumented fill may vary and extend deeper than observed in the borings. Undocumented fill that can be processed to meet the general criteria in Section 10.1.5 can be re-used as general fill.

10.1.5 Material Recommendations

Materials used during earthwork, grading, and paving operations should comply with the requirements listed in Table 3. Materials should be evaluated by the geotechnical engineer for suitability prior to use. The contractor should notify the geotechnical consultant 72 hours prior to import of materials or use of on-site materials to permit time for sampling, testing, and evaluation of the proposed materials. On-site materials may need to be dried out before re-use as fill. The contractor should be responsible for the consistency of import material brought to the site.

<table>
<thead>
<tr>
<th>Table 3 – Recommended Material Requirements</th>
</tr>
</thead>
<tbody>
<tr>
<td>Material and Use</td>
</tr>
<tr>
<td>General Fill: Import - for uses not otherwise specified</td>
</tr>
<tr>
<td>General Fill: On-Site - for uses not otherwise specified</td>
</tr>
<tr>
<td>Select (Low Expansion) Fill: Import - below building slabs, pavement, or flatwork</td>
</tr>
<tr>
<td>Select (Low Expansion) Fill: On-site - below building slabs, pavement, or flatwork</td>
</tr>
</tbody>
</table>
### Controlled Low Strength Material (CLSM)

<table>
<thead>
<tr>
<th>Material</th>
<th>Import</th>
<th>CSS(^4) Section 19-3.02F</th>
</tr>
</thead>
<tbody>
<tr>
<td>Permeable Aggregate - capillary break gravel</td>
<td>Import</td>
<td>Open-graded, clean, compactable crushed rock or angular gravel; nominal size (\frac{3}{4}) inch or less</td>
</tr>
<tr>
<td>Asphalt Concrete</td>
<td>Import</td>
<td>Type A; CSS(^4) Section 39-2</td>
</tr>
<tr>
<td>Aggregate Base</td>
<td>Import</td>
<td>Class II; CSS(^4) Section 26-1.02</td>
</tr>
<tr>
<td>Pipe/Conduit Bedding and Pipe Zone Material - material below pipe invert to 12 inches above pipe</td>
<td>Import</td>
<td>90 to 100 percent (by mass) should pass No. 4 sieve, and 5 percent or less should pass No. 200 sieve</td>
</tr>
<tr>
<td>Trench Backfill - above bedding material</td>
<td>Import or on-site borrow</td>
<td>As per general fill and excluding rock/lumps retained on 4-inch sieve or 2-inch sieve in top 12 inches</td>
</tr>
</tbody>
</table>

1. In general, fill should not consist of pea-gravel and should be free of rocks or lumps in excess of 6-inches diameter, trash, debris, roots, vegetation or other deleterious material.
2. In general, import fill should be tested or documented to be non-corrosive\(^3\) and free from hazardous materials in concentrations above levels of concern.
3. Non-corrosive as defined by the Corrosion Guidelines (Caltrans, 2012).
4. CSS is California Standard Specifications (Caltrans, 2015).

### 10.1.6 Chemical Treatment

The on-site soil may be chemically treated with quicklime to reduce the expansion characteristic of the soil as an alternative to importing select fill. The on-site soil may be chemically treated with quicklime and cement to improve the subgrade support characteristics for pavements and reduce the quantity of import aggregate base material needed for pavements. The quicklime should conform with ASTM standard C977. The cement used for treatment should conform with ASTM standard C150 for Type II/V or Type V cement.

On-site materials containing roots or other organic matter are not suitable for chemical treatment and should be stripped from the area at which the treatment is to be performed. The chemical treatment should be performed by an experienced contractor that specializes in the chemical treatment of soil. The chemical agent should be proportioned and spread with a mechanical spreader and mixed into the soil on a mixing table or in place to produce consistent distribution of the agent within the treated layer. The depth of mixing should not exceed 18 inches per lift or the capacity of the mixer if less. Precautions to reduce the potential for dusting of quicklime or cement, such as scheduling or suspending operations to avoid windy weather, should be taken. Casting or tailgating of the chemical agent should not be permitted. The mixer should be equipped with a rotary cutting/mixing assembly,
grade checker, and an automatic water distribution system. Mixing or spreading operations should not be performed during inclement weather or when the ambient temperature is less than 35 degrees Fahrenheit or during foggy or rainy weather. Adjacent passes of the mixer should overlap by 4 inches or more.

To reduce the expansive soil characteristic, quicklime should be mixed into the soil at a rate of 3 percent or more by dry weight of soil. Mixing and pulverizing should continue until the treated soil does not contain untreated soil clods larger than 1 inch and the quantity of untreated soil clods retained on the No. 4 sieve is less than 40 percent of the dry soil mass. Water should be added as-needed during the mixing process to achieve moisture content above the optimum, as evaluated by ASTM D1557, for the lime-soil mixture. The lime-soil mixture should be re-mixed following a 16-hour mellowing period after the initial mixing. The lime-soil mixture should be compacted within 3 days after initial mixing to achieve 95 percent of the reference density as evaluated by ASTM D1557 on a wet density basis.

To improve the subgrade support characteristics, quicklime should be mixed into the soil as described above. Following the 16-hour mellowing period after the initial mixing, cement should be mixed into the soil at rate of 3 percent or more by dry weight of soil. The moisture content of the soil should not exceed the optimum moisture content of the material, as evaluated by ASTM D1557, when the cement is spread and initially mixed. The subgrade should be mixed and aerated as-needed to reduce the moisture content. If additional water is needed to achieve the optimum moisture, the water should be added during a re-mixing operation after the cement has been initially mixed into the subgrade so as to reduce the potential for the formation of cement balls when water is applied. The cement-treated soil should be compacted within 2 hours of initial mixing to achieve 95 percent of the reference density as evaluated by ASTM D1557 on a wet density basis. Vehicular traffic and heavy construction equipment should not be allowed on the treated material for a 1 hour period after compaction. The cement-treated material should be maintained in a moist condition for a 7-day curing period by routinely sprinkling water, covering the treated material with moist straw, or placing fill over the treated subgrade. Treated subgrade for pavements should be proof-rolled with a loaded water truck to check for yielding conditions. Mitigation of yielding areas by pulverizing and re-mixing with additional stabilizing agent should be anticipated.

10.1.7 Subgrade Preparation

Subgrade in trenches and below slabs, footings, pavements, flatwork, or fill, should be prepared as per the recommendations in Table 4. Prepared subgrade should be maintained
in a moist (but not saturated) condition by the periodic sprinkling of water prior to placement of additional overlying fill or construction of footings and slabs.

<table>
<thead>
<tr>
<th>Subgrade Location</th>
<th>Preparation Recommendations</th>
</tr>
</thead>
</table>
| Below Pavement    | • After clearing and grubbing per Section 10.1.2, check for unsuitable materials as per Section 10.1.3.  
• Perform remedial grading as per Section 10.1.4. Scarify 8 inches then moisture condition and compact as per Section 10.1.8 if in-place lime treatment is not performed.  
• Proof roll compacted subgrade with loaded water truck under the observation of the geotechnical engineer. Mitigate yielding areas in accordance with the recommendations of the engineer.  
• Keep in moist condition by sprinkling water. |
| Below Slabs & Flatwork | • After clearing and grubbing per Section 10.1.2, check for unsuitable materials as per Section 10.1.3.  
• Perform remedial grading as per Section 10.1.4. Scarify 8 inches then moisture condition and compact as per Section 10.1.8 if in-place lime treatment is not performed.  
• Keep in moist condition by sprinkling water. |
| Below Footings | • Check for unsuitable materials and remove as-needed per Sections 10.1.3 and 10.1.4. Replace overexcavated soil with CLSM or extend footing as-needed.  
• Scarify and moisture condition exposed subgrade as-needed to achieve a moisture content 2 points or more above the optimum as evaluated by ASTM D1557 and compact exposed subgrade to 95 percent of the reference density as evaluated by ASTM D1557.  
• Keep in moist condition by sprinkling water. |
| Below Fill | • After clearing and grubbing per Section 10.1.2, check for unsuitable materials as per Section 10.1.3.  
• Scarify 8 inches then moisture condition and compact as per Section 10.1.8.  
• Keep in moist condition by sprinkling water. |
| Utility Trenches | • After clearing per Section 10.1.2.  
• Remove or compact loose/soft material. |

Subgrade that has been permitted to dry out and loosen or develop desiccation cracking, should be scarified, moisture-conditioned, and recompacted as per the requirements above. A thin layer (approximately 3 inches) of lean concrete or controlled low strength material (CLSM) may be poured over prepared subgrade for footings or slabs to maintain the appropriate moisture condition during erections of forms and placement of reinforcing steel.
10.1.8 Fill Placement and Compaction

Fill and backfill should be compacted in horizontal lifts in conformance with the recommendations presented in Table 5. The allowable uncompacted thickness of each lift of fill depends on the type of compaction equipment utilized, but generally should not exceed 8 inches in loose thickness.

### Table 5 – Fill Placement and Compaction Recommendations

<table>
<thead>
<tr>
<th>Fill Type</th>
<th>Location</th>
<th>Compacted Density¹</th>
<th>Moisture Content²</th>
</tr>
</thead>
<tbody>
<tr>
<td>Subgrade</td>
<td>Below Footings, Pavement, and Areas Subject to Vehicular Loading</td>
<td>95 percent</td>
<td>+ 2 percent or above</td>
</tr>
<tr>
<td></td>
<td>In locations not already specified</td>
<td>90 percent</td>
<td>+ 2 percent or above</td>
</tr>
<tr>
<td>Bedding and Pipe Zone Fill</td>
<td>Material below invert to 12 inches above pipe or conduit</td>
<td>90 percent</td>
<td>Near Optimum</td>
</tr>
<tr>
<td>Trench Backfill</td>
<td>Top 18 inches below finish subgrade for parking lot</td>
<td>95 percent</td>
<td>+ 2 percent or above</td>
</tr>
<tr>
<td></td>
<td>In locations not already specified</td>
<td>90 percent</td>
<td>+ 2 percent or above</td>
</tr>
<tr>
<td>Select or General Fill</td>
<td>In locations not already specified</td>
<td>90 percent</td>
<td>Near Optimum</td>
</tr>
<tr>
<td>Lime- or cement treated subgrade</td>
<td>In locations not already specified</td>
<td>90 percent</td>
<td>+ 2 percent or above</td>
</tr>
<tr>
<td>Asphalt Concrete</td>
<td>Pavement section</td>
<td>91 to 97 percent</td>
<td>Not Applicable</td>
</tr>
<tr>
<td>Aggregate Base</td>
<td>Below Parking Lot and Hardscape</td>
<td>95 percent</td>
<td>Near Optimum</td>
</tr>
</tbody>
</table>

Notes:
1. Expressed as percent relative compaction or ratio of field density to reference density (typically on a dry density basis for soil and aggregate and on a wet density basis for asphalt concrete and lime treated subgrade). The reference density of soil, lime-treated subgrade, and aggregate should be evaluated by ASTM D 1557. The reference density of asphalt concrete should be evaluated by ASTM D 2041.
2. Target moisture content at compaction relative to the optimum as evaluated by ASTM D 1557.

Compacted fill should be maintained in a moist (but not saturated) condition by the periodic sprinkling of water prior to placement of additional overlying fill or construction of footings and slabs. Fill that has been permitted to dry out and loosen or develop desiccation cracking, should be scarified, moisture conditioned, and recompacted as per the requirements above.
10.1.9 Excavation Stabilization

Excavations, including foundation and utility excavations, should be stabilized by shoring sidewalls or laying slopes back in accordance with the Excavation Rules and Regulations (29 Code of Federal Regulations [CFR], Part 1926) stipulated by the Occupational Safety and Health Administration (OSHA). Table 6 lists the OSHA material type classifications and corresponding allowable temporary slope layback inclinations for soil deposits that may be encountered on site. Alternatively, a shoring system conforming to the OSHA Excavation Rules and Regulations (29 CFR Part 1926) may be used to stabilize excavation sidewalls during construction. The lateral earth pressures listed in Table 6 may be used to design or select an internally-braced shoring system or trench shield conforming to the OSHA guidelines. Our recommendations for lateral earth pressures and allowable slope gradients are based upon the limited subsurface data provided by our exploratory borings and reflect the influence of the environmental conditions that existed at the time of our exploration. Excavation stability, material classifications, allowable slopes, and shoring pressures should be re-evaluated and revised, as-needed, during construction. Excavations, shoring systems and the surrounding areas should be evaluated daily by a competent person for indications of possible instability or collapse. Dewatering pits or sumps should be used to depress the groundwater level (if encountered) below the bottom of the excavation.

<table>
<thead>
<tr>
<th>Formation</th>
<th>OSHA Classification</th>
<th>Allowable Temporary Slope(^{1,2,3})</th>
<th>Lateral Earth Pressure on Shoring(^4) (psf)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Fill &amp; Alluvium (above groundwater)</td>
<td>Type C</td>
<td>1 ½ h:1v (34°)</td>
<td>80xD + 72</td>
</tr>
</tbody>
</table>

Notes:
1. Allowable slope for excavations less than 20 feet deep. Excavation sidewalls in cohesive soil may be benched to meet the allowable slope criteria (measured from the bottom edge of the excavation). The allowable bench height is 4 feet. The bench at the bottom of the excavation may protrude above the allowable slope criteria.
2. In layered soil, layers shall not be sloped steeper than the layer below.
3. Temporary excavations less than 5 feet deep may be made with vertical side slopes and remain unshored if judged to be stable by a competent person (29 CFR, Part 1926.650).
4. ‘D’ is depth of excavation for excavations up to 20 feet deep. Includes a surface surcharge equivalent to two feet of soil.

The shoring system should be designed or selected by a suitably qualified individual or specialty subcontractor. The shoring parameters presented in this report are preliminary design criteria, and the designer should evaluate the adequacy of these parameters and make appropriate modifications for their design. We recommend that the contractor take appropriate measures to protect workers. OSHA requirements pertaining to worker safety should be observed.
Excavations made in close proximity to existing structures may undermine the foundation of those structures and/or cause soil movement related distress to the existing structures. Stabilization techniques for excavations in close proximity to existing structures will need to account for the additional loads imposed on the shoring system and appropriate setback distances for temporary slopes. The geotechnical engineer should be consulted for additional recommendations if the proposed excavations cross below a plane extending down and away from the foundation bearing surfaces of the adjacent structure at an angle of 2:1 (horizontal to vertical) from the bottom edge of the footing or if the proposed excavation is less than 18 inches from the face of the footing.

The excavation bottoms may become unstable and subject to pumping under heavy equipment loads if the excavation subgrade is exposed to water. The contractor should be prepared to stabilize the bottom of the excavations. In general, unstable bottom conditions may be mitigated by scarifying the subgrade and aerating the soil to achieve a moisture content near the optimum, dewatering to depress groundwater levels below the bottom of the excavation, overexcavating to a suitable depth and replacing the wet material with suitable fill, compacting a layer of crushed rock fill into the subgrade, or using geogrid to stabilize additional fill. Specific recommendations for excavation stabilization will be influenced by the nature of the excavation and the conditions encountered during construction.

10.1.10 Construction Dewatering

Water intrusion into the excavations may occur as a result of groundwater seepage or surface runoff. The contractor should be prepared to take appropriate dewatering measures in the event that water intrudes into the excavations. Sump pits, trenches, or similar measures should be used to depress the water level below the bottom of the excavation. Considerations for construction dewatering should include anticipated drawdown, volume of pumping, potential for settlement, and groundwater discharge. Disposal of groundwater should be performed in accordance with the guidelines of the Regional Water Quality Control Board.

10.1.11 Utility Trenches

Trenches constructed for the installation of underground utilities should be stabilized in accordance with our recommendations in Section 10.1.9. Utility trenches should be backfilled with materials that conform to our recommendations in Section 10.1.5. Trench backfill, bedding, and pipe zone fill should be compacted in accordance with Section 10.1.8 of this report. Bedding and pipe zone fill should be shoveled under pipe haunches and
compacted by manual or mechanical, hand-held tampers. Trench backfill should be compacted by mechanical means. Densification of trench backfill by flooding or jetting should not be permitted.

To reduce potential for moisture intrusion into the building envelope, we recommend plugging utility trenches at locations where the trench excavations cross under the building perimeter. The trench plug should be constructed of a compacted, fine-grained, cohesive soil that fills the cross-sectional area of the trench for a distance equivalent to the depth of the excavation. Alternatively, the plug may be constructed of concrete or CLSM.

10.1.12 Rainy Weather Considerations
We recommend that the construction be performed during the period between approximately April 15 and October 15 to avoid the rainy season. In the event that grading is performed during the rainy season, the plans for the project should be supplemented to include a stormwater management plan prepared in accordance with the requirements of the relevant agency having jurisdiction. The plan should include details of measures to protect the subject property and adjoining off-site properties from damage by erosion, flooding or the deposition of mud, debris, or construction-related pollutants, which may originate from the site or result from the grading operation. The protective measures should be installed by the commencement of grading, or prior to the start of the rainy season. The protective measures should be maintained in good working order unless the project drainage system is installed by that date and approval has been granted by the building official to remove the temporary devices.

In addition, construction activities performed during rainy weather may impact the stability of excavation subgrade and exposed ground. Temporary swales should be constructed to divert surface runoff away from excavations and slopes. Steep temporary slopes should be covered with plastic sheeting during significant rains. The geotechnical consultant should be consulted for recommendations to stabilize the site as-needed. A thin layer (approximately 3 inches) of lean concrete or CLSM may be poured over prepared subgrade for footings or slabs to maintain the appropriate moisture condition during erections of forms and placement of reinforcing steel.

10.2 Seismic Design Criteria
Design of the proposed improvements should be performed in accordance with the requirements of governing jurisdictions and applicable building codes. Table 7 presents the
seismic design parameters for the site in accordance with the CBC (2016) guidelines and adjusted MCER spectral response acceleration parameters (USGS, 2017).

### Table 7 – 2016 California Building Code Seismic Design Criteria

<table>
<thead>
<tr>
<th>Seismic Design Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Site Class</td>
<td>D</td>
</tr>
<tr>
<td>Site Coefficient, Fa</td>
<td>1.0</td>
</tr>
<tr>
<td>Site Coefficient, Fv</td>
<td>1.5</td>
</tr>
<tr>
<td>Mapped M\textsubscript{CER} Spectral Response Acceleration at 0.2-second period, S\textsubscript{S}</td>
<td>2.126g</td>
</tr>
<tr>
<td>Mapped M\textsubscript{CER} Spectral Response Acceleration at 1.0-second period, S\textsubscript{1}</td>
<td>0.682g</td>
</tr>
<tr>
<td>Site-Adjusted M\textsubscript{CER} Spectral Acceleration at 0.2-second period, S\textsubscript{MS}</td>
<td>2.126g</td>
</tr>
<tr>
<td>Site-Adjusted M\textsubscript{CER} Spectral Acceleration at 1.0-second period, S\textsubscript{M1}</td>
<td>1.023g</td>
</tr>
<tr>
<td>Design Spectral Response Acceleration at 0.2-second Period, S\textsubscript{DS}</td>
<td>1.417g</td>
</tr>
<tr>
<td>Design Spectral Response Acceleration at 1.0-second Period, S\textsubscript{D1}</td>
<td>0.682g</td>
</tr>
<tr>
<td>Seismic Design Category for Risk Category I, II, or III</td>
<td>D</td>
</tr>
</tbody>
</table>

#### 10.3 Foundation Recommendations

New buildings may be supported on spread footings with slab-on-grade floors. Foundations should be designed in accordance with structural considerations and the following recommendations. In addition, requirements of the appropriate governing jurisdictions and applicable building codes should be considered in design of the structures.

##### 10.3.1 Spread Footings

Footings bearing on alluvium or new engineered fill with subgrade prepared in accordance with the recommendations in Section 10.1.7 may be designed using the criteria listed in Table 8. The geotechnical engineer should observe the footing excavations to evaluate bearing materials and subgrade condition before the exposed subgrade is covered.

### Table 8 – Recommended Bearing Design Parameters for Footings

<table>
<thead>
<tr>
<th>Footing</th>
<th>Sustained Loads</th>
<th>Footing Widths</th>
<th>Bearing Depth$^2$</th>
<th>Allowable Bearing Capacity$^3$</th>
<th>Static Settlement$^4$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Wall Footing</td>
<td>5 kips/foot or less</td>
<td>1½ feet or more</td>
<td>2 feet or more</td>
<td>2,500 psf</td>
<td>1⅛ inches total $^{\frac{3}{4}}$ inch differential over 20 feet</td>
</tr>
</tbody>
</table>
Table 8 – Recommended Bearing Design Parameters for Footings

<table>
<thead>
<tr>
<th>Footing</th>
<th>Sustained Loads</th>
<th>Footing Widths</th>
<th>Bearing Depth</th>
<th>Allowable Bearing Capacity</th>
<th>Static Settlement</th>
</tr>
</thead>
<tbody>
<tr>
<td>Column Footing</td>
<td>40 kips or less</td>
<td>2 feet or more</td>
<td>2 feet or more</td>
<td>3,000 psf</td>
<td>1 inch total</td>
</tr>
<tr>
<td></td>
<td>100 kips</td>
<td>6 feet or more</td>
<td>2 feet or more</td>
<td>3,000 psf</td>
<td>1½ inches total</td>
</tr>
<tr>
<td></td>
<td>180 kips</td>
<td>8 feet or more</td>
<td>2 feet or more</td>
<td>3,000 psf</td>
<td>1¾ inches total</td>
</tr>
</tbody>
</table>

Notes:
1 Assumes square footing shape.
2 Below the adjacent finish grade and the existing grade.
3 Net allowable bearing capacity in pounds per square foot. Listed value includes a Factor of Safety of 3 or more. Allowable bearing capacity may be increased by one-third when considering loads of short duration such as wind or seismic loads.
4 Based on sustained long-term loading conditions. Assumes that if footing width is increased from that shown in table, sustained load is equal to or less than value shown for each case.
5 Designer can interpolate between the values presented in the Table. For example, for a sustained load of 70 kips, footing width should be 4 feet or more, bearing depth 2 feet or more, allowable bearing pressure should be 3,000 psf, and static settlement should be anticipated to be 1¾ inch total.

Footing settlement due to static loads may be further evaluated using a modulus of subgrade reaction. Recommended values for the modulus of subgrade reaction are provided in Table 9. The designer may interpolate between the values in the table for intermediate footing widths.

Table 9 – Footing Modulus of Subgrade Reaction

<table>
<thead>
<tr>
<th>Footing1</th>
<th>1 foot</th>
<th>2 feet</th>
<th>3 feet</th>
<th>4 feet</th>
<th>5 feet</th>
</tr>
</thead>
<tbody>
<tr>
<td>Wall Footing</td>
<td>85 pci</td>
<td>37 pci</td>
<td>23 pci</td>
<td>17 pci</td>
<td>13 pci</td>
</tr>
<tr>
<td>Column Footing2</td>
<td>168 pci</td>
<td>72 pci</td>
<td>44 pci</td>
<td>31 pci</td>
<td>24 pci</td>
</tr>
</tbody>
</table>

Notes:
1 Assumes bearing depth of 24 inches below adjacent finish grade.
2 Assumes square footing shape for columns.
3 Modulus of Subgrade Reaction in units of pounds per cubic inch.

The spread footings should be reinforced with deformed steel bars as detailed by the project structural engineer. Where footings are located adjacent to utility trenches or other
excavations, the footing bearing surfaces should bear below an imaginary plane extending upward from the bottom edge of the adjacent trench/excavation at a 2:1 (horizontal to vertical) angle above the bottom edge of the footing. Footings should be deepened or excavation depths reduced as-needed.

A friction coefficient of 0.30 may be assumed for evaluating frictional resistance to lateral loads. A lateral bearing pressure of 300 psf per foot of depth up to 3,000 psf may be used to evaluate the resistance of footings to lateral loads for level ground conditions. The lateral bearing pressure should be neglected to a depth of 1 foot where the ground adjacent to the foundation is not covered by a slab or pavement. The lateral resistance can be taken as the sum of the frictional resistance and passive resistance. The friction coefficient and passive lateral bearing pressure should be considered ultimate values. The lateral bearing pressure may be increased by one-third when considering loads of short duration such as wind or seismic forces.

The weight of the material above a plane rising up and away from the bottom edges of the footings at 20 degrees off plumb may be considered, along with the weight of the footing and the material over the footing, when evaluating footing resistance to uplift. A unit weight of 115 pounds per cubic foot (pcf) for soil or aggregate and 150 pcf for normal weight concrete may be assumed for this evaluation.

10.3.2 Slabs-on-Grade

Building floor slabs should be designed by the project structural engineer based on the anticipated loading conditions. The slab should be reinforced with deformed steel bars. We recommend that masonry briquettes or plastic chairs be used to aid in the correct placement of slab reinforcement in the upper half of the slab. Refer to Section 10.5 for the recommended concrete cover over reinforcing steel. A vapor retarder is recommended in areas where moisture-sensitive floor coverings or conditioned environments are anticipated. See Section 10.6 for vapor retarding system recommendations. Joints consistent with ACI guidelines (ACI, 2016) may be constructed at periodic intervals to reduce the potential for random cracking of the slab.

10.3.3 Drilled Piers

Drilled piers for minor structures such as fences, light poles, and shade structures embedded 5 to 20 feet below grade may be designed for an allowable side friction of 300 psf to evaluate resistance to downward axial loads and 200 psf per foot depth for upward axial loads. The allowable skin friction includes a factor of safety of 2 for downward loading.
and 3 for upward loading. The allowable side friction may be increased by one-third when considering loads of short duration such as wind or seismic loads. The spacing between adjacent piers should be equivalent to eight pier diameters, or more to mitigate reduction due to group effects.

A lateral bearing pressure of 100 pounds per square foot (psf) per foot depth up to 1,500 psf may be used to evaluate resistance to lateral loads and overturning moments in accordance with Section 1806 of the 2016 CBC. The allowable lateral bearing pressure may be increased by one-third for wind or seismic load combinations and by an additional factor of two for structures that can accommodate ½ inch of lateral deflection of the top of the pier foundation. Drilled pier excavations should be cleaned of loose material prior to pouring concrete. Drilled pier excavations that encounter groundwater or cohesionless soil may be unstable and may need to be stabilized by temporary casing or use of drilling mud. Standing water should be removed from the pier excavation or the concrete should be delivered to the bottom of the excavation, below the water surface, by tremie pipe. Casing should be removed from the excavation as the concrete is placed. Concrete should be placed in the piers in a manner that reduces the potential for segregation of the components.

10.4 Pavements and Flatwork

Recommendations for asphalt pavement, concrete pavement, concrete pavers, and exterior flatwork are presented in the following sections. The design R-value used for evaluate the pavement sections was selected based on R-value testing performed on a sample collected during our subsurface exploration. The pavement subgrade should be observed by the geotechnical engineer during grading to check that the exposed materials are consistent with the findings from our subsurface exploration and the support characteristics assumed for pavement design. Additional R-value testing may be needed, based on these observations, with subsequent revision to the pavement sections. Recommendations for preparation of subgrade are presented in Section 10.1.7.

Pavement sections were evaluated for a range of traffic indexes or loading conditions. The designer may interpolate between the values provided once a traffic index or loading condition has been selected.

10.4.1 Asphalt Pavement

Ninyo & Moore conducted an analysis to evaluate appropriate asphalt pavement structural sections following the methodology presented in the Highway Design Manual (Caltrans,
2016). Alternative sections were evaluated. The pavement sections were designed for a 20-year service life presuming that periodic maintenance, including crack sealing and resurfacing will be performed during the service life of the pavement. Premature deterioration may occur without periodic maintenance. Our recommendations for the pavement sections are presented in Table 10.

Paving operations and base preparation should be observed and tested by Ninyo & Moore. Subgrade enhancement geotextiles, where utilized, should be rolled out flat and tight, without folds or wrinkles, over prepared subgrade in the direction of travel. The geotextile should be pinned to the subgrade with nails and washers or u-shaped sod staples. Adjacent rolls should overlap 12 inches or more. Abutting rolls should overlap in the direction of fill placement to reduce the potential for peeling of the geotextile during fill placement. Aggregate base fill should be pushed over the geotextile into position and compacted. To reduce the potential for displacement of the geotextile or deterioration of the subgrade, construction equipment should not operate on the geotextile with 6 inches of aggregate base cover.

Table 10 – Asphalt Concrete Pavement Structural Sections

<table>
<thead>
<tr>
<th>Design R-Value</th>
<th>Traffic Index</th>
<th>Alternative 1</th>
<th>Alternative 2</th>
<th>Alternative 3</th>
</tr>
</thead>
<tbody>
<tr>
<td>5</td>
<td>5</td>
<td>3 inches AC 12 inches AB</td>
<td>3 inches AC 12 inches AB SEG</td>
<td>3 inches AC 6 inches AB 12 inches TS</td>
</tr>
<tr>
<td>5</td>
<td>7.5</td>
<td>4½ inches AC 17 inches AB</td>
<td>4½ inches AC 13 inches AB SEG</td>
<td>4½ inches AC 10 inches AB 12 inches TS</td>
</tr>
<tr>
<td>5</td>
<td>10.5</td>
<td>6½ inches AC 25 inches AB</td>
<td>6½ inches AC 19 inches AB SEG</td>
<td>6½ inches AC 14 inches AB 12 inches TS</td>
</tr>
</tbody>
</table>

Notes:
2 AB is Class II Aggregate Base complying with Caltrans Standard Specification 26-1.02 (2015).
3 SEG is subgrade enhancement geotextile such as Mirafi 600X.
4 TS is chemically treated subgrade consistent with the recommendations in Section 10.1.6.

Aggregate base for pavement should be placed in lifts of no more than 8 inches in loose thickness, moisture-conditioned as-needed to approximately 2 percentage points above the optimum moisture content, and compacted to 95 percent or more of the reference density as evaluated by ASTM D1557. Asphalt concrete should be placed and compacted in
accordance with Section 39 of the Caltrans Standard Specification (2015) to not less than 91 percent and not more than 97 percent of the reference density as evaluated by ASTM D2041 on a wet density basis. Pavements should be sloped so that runoff is diverted to an appropriate collector (concrete gutter, swale, or area drain) to reduce the potential for ponding of water on the pavement. Concentration of runoff over asphalt pavement should be discouraged.

10.4.2 Concrete Pavement

Concrete pavement sections based on methodologies developed by the Portland Cement Associate (PCA) are presented in Table 11 for a 20-year design period with appropriate periodic maintenance.

<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>233,000 Annual Vehicles including: 52 annual 48-kip garbage trucks 57 annual 38-kip, 40-foot buses 1 annual 75-kip emergency vehicle</td>
<td>9</td>
<td>20 years</td>
<td>50 pci</td>
<td>7 inches PCC[3] 12 inches AB[4] or 6 inches AB on 12 inches TS[5]</td>
</tr>
<tr>
<td>230,000 Annual Vehicles including: 52 annual 48-kip garbage trucks 3,100 annual 35-kip, 40-foot buses 1 annual 75-kip emergency vehicle</td>
<td>9</td>
<td>20 years</td>
<td>50 pci</td>
<td>8 inches PCC[3] 12 inches AB[4] or 6 inches AB on 12 inches TS[5]</td>
</tr>
<tr>
<td>181,000 Annual Vehicles including: 52 annual 48-kip garbage trucks 6,200 annual 35-kip, 40-foot buses 1 annual 75-kip emergency vehicle</td>
<td>9</td>
<td>20 years</td>
<td>50 pci</td>
<td>8½ inches PCC[3] 12 inches AB[4] or 6 inches AB on 12 inches TS[5]</td>
</tr>
</tbody>
</table>

Notes:
1 Assumes 24-ton garbage truck with 12-kip single and 36-kip tandem axles; 38-kip bus with 13- and 25-kip single axles; and a 75-kip ladder truck with 23-kip single and 52 kip tandem axles.
2 Modulus of Subgrade Reaction in pounds per cubic inch (pci).
3 PCC is Portland Cement Concrete complying with Caltrans Standard Specification Section 90 (2015).
4 AB is Class II Aggregate Base complying with Caltrans Standard Specification Section 26 (2015).
5 TS is chemically treated subgrade consistent with the recommendations in Section 10.1.6.

The recommended sections presume that the concrete will have a 28-day flexural strength of 600 psi or an equivalent compressive strength of 5,000 psi at 28 days. Aggregate base for pavement should be placed in lifts of no more than 8 inches in loose thickness, moisture-conditioned as-needed to approximately 2 percentage points above the optimum moisture content, and compacted to 95 percent of the reference density as evaluated by ASTM D1557.
Appropriate jointing of concrete pavement can reduce the potential for crack development between joints. Joints should be laid out in a consistent square pattern. Contraction, construction, and isolation joints should be detailed and constructed in accordance with the guidelines of the ACI Committee 302 (ACI, 2015). Contraction joints formed by premolded inserts, grooving plastic concrete, or saw-cutting at initial hardening, should extend to a depth equivalent to 25 percent of the slab thickness and 1 inch or more for thin slabs. Contraction joints should be reinforced with smooth dowels placed across the joint at mid-slab height. Construction joints subject to traffic loading should be reinforced with smooth dowels as for contraction joints. Construction joints within the middle third of the typical joint spacing pattern should be reinforced with tiebars. Recommendations for contraction joint spacing, dowel dimensions, dowel spacing, tiebar dimensions, and tiebar spacing are provided in Table 12. Isolation joints should consist of full-depth premolded joint filler placed where the pavement abuts structures or other fixed objects. At isolation joints where the edge of the pavement will be subjected to traffic loading, the thickness of the slab should be increased by 20 percent at the edge of the pavement with a 40:1 taper (horizontal to vertical) to the nominal slab thickness.

<table>
<thead>
<tr>
<th>Slab Thickness</th>
<th>Contraction Joint Spacing</th>
<th>Dowels</th>
<th>Tiebars at 10 feet to free edge</th>
<th>Tiebars at 25 feet to free edge</th>
<th>Distributed Steel</th>
</tr>
</thead>
<tbody>
<tr>
<td>7 inches</td>
<td>14 feet or less</td>
<td>⅞ x 14 at 12 inches</td>
<td>½ x 24 at 30 inches</td>
<td>½ x 24 at 18 inches</td>
<td>#4 at 18 inches both ways</td>
</tr>
<tr>
<td>8 inches</td>
<td>16 feet or less</td>
<td>1 x 14 at 12 inches</td>
<td>½ x 24 at 30 inches</td>
<td>½ x 24 at 16 inches</td>
<td>#5 at 18 inches both ways</td>
</tr>
<tr>
<td>8½ inches</td>
<td>17 feet or less</td>
<td>1¼ x 18 at 12 inches</td>
<td>½ x 24 at 30 inches</td>
<td>¾ x 30 at 26 inches</td>
<td>#5 at 18 inches both ways</td>
</tr>
</tbody>
</table>

Notes:
1 Dowels and Tiebars specified in nominal diameter x length at spacing along joint in inches. The designer may interpolate between the values provide for an intermediate distance to the free edge of pavement.

The recommended sections presume that the pavement is laterally restrained by curbs, structures, driveway aprons, or other pavements. The thickness of the recommended concrete sections should be increased by 1 inch for pavements that are unrestrained and joints that are parallel and adjacent to pavement edges should be reinforced with deformed steel tiebars, instead of dowels, placed across the joint at mid-slab height as recommended in Table 12.
Distributed reinforcing steel consisting of deformed steel bars may be placed to reduce the potential for differential slab movement, should cracking occur between joints. The spacing between contraction joints may be increased where distributed reinforcing steel is utilized. Pavements reinforced with distributed steel consistent with the recommendations in Table 12, may be designed for a contraction joint spacing of up to 70 feet. Masonry briquettes or plastic chairs should be used to maintain the position of the reinforcement in the upper half of the slab with 1½ inches of concrete cover over the steel. The distributed steel should be terminated about 6 inches from contraction or isolation joints.

The pavement surface and subgrade should be sloped to provide positive drainage toward suitable drainage devices. To reduce the potential for subsurface water intrusion into the subgrade and base layer, curbs or similar cutoff devices should be provided and joints should include a formed or sawcut reservoir for placement of foam backer rod and recessed, self-leveling silicone sealant. Periodic maintenance of the pavement should include sealing cracks that develop and replacement of joint sealant as-needed.

10.4.3 Exterior Flatwork

Concrete walkways and other exterior flatwork not subject to vehicular loading should be 4 inches thick (or more) over 6 inches of aggregate base. Appropriate jointing of concrete flatwork can encourage cracks to form at joints, reducing the potential for crack development between joints. Joints should be laid out in a square pattern at consistent intervals. Contraction and construction should be detailed and constructed in accordance with the guidelines of ACI Committee 302 (ACI, 2016). The lateral spacing between contraction joints should be 8 feet or less for a 4-inch thick slab.

Distributed reinforcing steel may be utilized to reduce the potential for differential slab movement, should cracking occur between joints. The distributed reinforcing steel should be terminated about 6 inches from contraction joints and should consist of No. 3 deformed bars at 18 inches on center, both ways. Slabs reinforced with distributed steel should be 5 inches thick (or more). To reduce the potential for differential slab movement across joints, the distributed steel may be extended through the joints. This improvement will be balanced by a reduction in the functionality of the contraction joint to encourage crack formation at joints. Masonry briquettes or plastic chairs should be used to maintain the position of the reinforcement in the upper half of the slab with 1½ inches of cover over the steel.
10.5 Concrete
Laboratory testing indicated that the concentration of sulfate and corresponding potential for sulfate attack on concrete is negligible for the soil tested. However, due to the variability in the on-site soil and the potential future use of reclaimed water at the site, we recommend that Type II/V or Type V cement be used for concrete structures in contact with soil. In addition, we recommend a water-to-cement ratio of no more than 0.45. A 3-inch thick, or thicker, concrete cover should be maintained over reinforcing steel where concrete is in contact with soil in accordance with recommendations of ACI Committee 318 (ACI, 2014).

10.6 Moisture Vapor Retarder
The migration of moisture through slabs underlying enclosed spaces or overlain by moisture sensitive floor coverings should be discouraged by providing a moisture vapor retarding system between the subgrade soil and the bottom of slabs. We recommend that the moisture vapor retarding system consist of a 4-inch-thick capillary break, overlain by a 15-mil-thick plastic membrane. The capillary break should be constructed of clean, compacted, open-graded crushed rock or angular gravel of ¾-inch nominal size. To reduce the potential for slab curling and cracking, an appropriate concrete mix with low shrinkage characteristics and a low water-to-cementitious-materials ratio should be specified. In addition, the concrete should be delivered and placed in accordance with ASTM C94 with attention to concrete temperature and elapsed time from batching to placement, and the slab should be cured in accordance with the ACI Manual of Concrete Practice (ACI, 2016), as appropriate. The plastic membrane should conform to the requirements in the latest version of ASTM Standard E 1745 for a Class A membrane. The bottom of the moisture barrier system should be higher in elevation than the exterior grade, if possible. Positive drainage should be established and maintained adjacent to foundations and flatwork.

Where the exterior grade is at a higher elevation than the moisture vapor retarding system (including the capillary break layer), consideration should be given to constructing a subdrain around the foundation perimeter. The subdrain should consist of ¾-inch crushed rock wrapped in filter fabric (Mirafi 140N, or equivalent). The subdrain should be capped by a pavement or 12 inches of native soil and drained by a perforated pipe (Schedule 40 polyvinyl chloride pipe, or similar). The pipe should be sloped at 1 percent or more to discharge at an appropriate outlet away from the foundation. The pipe should be located below the bottom elevation of the moisture vapor retarding system but above a plane extending down and away from the bottom edge of the foundation at a 2:1 (horizontal to vertical) gradient.
10.7 Surface Drainage and Site Maintenance

Surface drainage on the site should generally be provided so that water is diverted away from structures and is not permitted to pond. Positive drainage should be established adjacent to structures to divert surface water to an appropriate collector (graded swale, v-ditch, or area drain) with a suitable outlet. Drainage gradients should be 2 percent or more a distance of 5 feet or more from the structure for impervious surfaces and 5 percent or more a distance of 10 feet or more from the structure for pervious surfaces. Slope, pad, and roof drainage (from adjacent structures) should be collected and diverted to suitable discharge areas away from structures or other slopes by non-erodible devices (e.g., gutters, downspouts, concrete swales, etc.). Graded swales, v-ditches, or curb and gutter should be provided at the site perimeter to restrict flow of surface water onto and off of the site. Slopes should be vegetated or otherwise armored to reduce potential for erosion of soil. Drainage structures should be periodically cleaned out and repaired, as-needed, to maintain appropriate site drainage patterns.

Landscaping adjacent to foundations should include vegetation with low-water demands and irrigation should limited to that which is needed to sustain the plants. Trees should be restricted from the areas adjacent to foundations a distance equivalent to the canopy radius of the mature tree. Bioretention areas should not be located within a distance of 20 feet from structure foundations.

Care should be taken by the contractor during grading to preserve any berms, drainage terraces, interceptor swales or other drainage devices on or adjacent to the project area. Drainage patterns established at the time of grading should be maintained for the life of the project. The property owner and maintenance personnel should be made aware that altering drainage patterns might be detrimental to wall performance.

10.8 Review of Construction Plans

The recommendations provided in this report are based on preliminary design information for the proposed construction. We recommend that a copy of the plans be provided to Ninyo & Moore for review before bidding to check the interpretation of our recommendations and that the designed improvements are consistent with our assumptions. It should be noted that, upon review of these documents, some recommendations presented in this report might be revised or modified to meet the project requirements.

10.9 Construction Observation and Testing

The recommendations provided in this report are based on subsurface conditions encountered in relatively widely spaced exploratory borings. During construction, the geotechnical engineer
or his representative in the field should be allowed to check the exposed subsurface conditions. During construction, the geotechnical engineer or his representative should be allowed to:

- Observe preparation and compaction of subgrade.
- Observe mitigation of unsuitable materials by excavation or chemical treatment.
- Check and test imported materials prior to use as fill.
- Observe placement and compaction of fill, aggregate base, and asphalt concrete.
- Perform field density tests to evaluate fill and subgrade compaction.
- Observe foundation excavations for bearing materials and cleaning prior to placement of reinforcing steel and concrete.
- Observe placement of reinforcing steel in footings and slabs.
- Observe condition of water vapor retarding system prior to concrete placement.

The recommendations provided in this report assume that Ninyo & Moore will be retained as the geotechnical consultant during the construction phase of the project. If another geotechnical consultant is selected, we request that the selected consultant provide a letter to the architect and the owner (with a copy to Ninyo & Moore) indicating that they fully understand Ninyo & Moore’s recommendations, and that they are in full agreement with the recommendations contained in this report.

11 LIMITATIONS

The field evaluation, laboratory testing, and geotechnical analyses presented in this geotechnical report have been conducted in general accordance with current practice and the standard of care exercised by geotechnical consultants performing similar tasks in the project area. No warranty, expressed or implied, is made regarding the conclusions, recommendations, and opinions presented in this report. There is no evaluation detailed enough to reveal every subsurface condition. Variations may exist and conditions not observed or described in this report may be encountered during construction. Uncertainties relative to subsurface conditions can be reduced through additional subsurface exploration. Additional subsurface evaluation will be performed upon request. Please also note that our evaluation was limited to assessment of the geotechnical aspects of the project, and did not include evaluation of structural issues, environmental concerns, or the presence of hazardous materials.

This document is intended to be used only in its entirety. No portion of the document, by itself, is designed to completely represent any aspect of the project described herein. Ninyo & Moore
should be contacted if the reader requires additional information or has questions regarding the content, interpretations presented, or completeness of this document.

This report is intended for design purposes only. It does not provide sufficient data to prepare an accurate bid by contractors. It is suggested that the bidders and their geotechnical consultant perform an independent evaluation of the subsurface conditions in the project areas. The independent evaluations may include, but not be limited to, review of other geotechnical reports prepared for the adjacent areas, site reconnaissance, and additional exploration and laboratory testing.

Our conclusions, recommendations, and opinions are based on an analysis of the observed site conditions. If geotechnical conditions different from those described in this report are encountered, our office should be notified and additional recommendations, if warranted, will be provided upon request. It should be understood that the conditions of a site could change with time as a result of natural processes or the activities of man at the subject site or nearby sites. In addition, changes to the applicable laws, regulations, codes, and standards of practice may occur due to government action or the broadening of knowledge. The findings of this report may, therefore, be invalidated over time, in part or in whole, by changes over which Ninyo & Moore has no control.

This report is intended exclusively for use by the client. Any use or reuse of the findings, conclusions, and/or recommendations of this report by parties other than the client is undertaken at said parties’ sole risk.
12 REFERENCES

American Concrete Institute, 2014, Building Code Requirements for Structural Concrete and Commentary, 318-14.

American Concrete Institute, 2015, Guide to Concrete Floor and Slab Construction, 302.1R-15.

American Concrete Institute, 2016, ACI Manual of Concrete Practice.


Association of Bay Area Governments (ABAG), 2017, Resilience Program Website: http://gis.abag.ca.gov/website/Hazards/

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California Department of Transportation (Caltrans), 2015, Standard Specifications: dated May.

California Department of Transportation (Caltrans), 2012, Corrosion Guidelines, Version 2.0, Division of Engineering Services, Materials Engineering and Testing Services, Corrosion Technology Branch: dated November.


Graymer, R.W., Jones, D.L., and Brabb, E.E., 2002, Geologic Map and Map Database of Northeastern San Francisco Bay Region, Most of Solano County and Parts of Napa,


FIGURES
SITE LOCATION

K.I. JONES ELEMENTARY SCHOOL
2001 WINSTON DRIVE, FAIRFIELD, CALIFORNIA

BUILDINGS
NEW PORTABLE ADDITION
NEW RESTROOM ADDITION
NEW RESTROOM ADDITION
NEW ADMIN/KINDERGARDEN CLASSROOM BUILDINGS
NEW PARKING LOT

LEGEND
B-6  NINYO & MOORE BORING; TD=TOTAL DEPTH IN FEET
KC-3  ENGINEERING BORING
B  CROSS SECTION


BORING LOCATIONS
K.I. JONES ELEMENTARY SCHOOL
2001 WINSTON DRIVE, FAIRFIELD, CALIFORNIA

FIGURE 2

403070001  |  7/17

LEGEND

<table>
<thead>
<tr>
<th>Code</th>
<th>Description</th>
<th>Color</th>
</tr>
</thead>
<tbody>
<tr>
<td>Qa</td>
<td>ALLUVIUM (QUATERNARY)</td>
<td></td>
</tr>
<tr>
<td>Qpf</td>
<td>ALLUVIAL FAN DEPOSITS (QUATERNARY)</td>
<td></td>
</tr>
<tr>
<td>Ks</td>
<td>SITES FORMATION (LATE CRETACEOUS)</td>
<td></td>
</tr>
<tr>
<td>Kjgv</td>
<td>SANDSTONE AND SHALE (EARLY CRETACEOUS AND LATE JURASSIC)</td>
<td></td>
</tr>
</tbody>
</table>

GEOLOGIC CONTACT: (DASHED WHERE APPROXIMATE)

NOTE: DIMENSIONS, DIRECTIONS AND LOCATIONS ARE APPROXIMATE.

FIGURE 4

REGIONAL GEOLOGY

K.I.JONES ELEMENTARY SCHOOL

2001 WINSTON DRIVE, FAIRFIELD, CALIFORNIA

Ninio & Moore
Geotechnical & Environmental Sciences Consultants
NOTE: DIMENSIONS, DIRECTIONS AND LOCATIONS ARE APPROXIMATE.

CROSS SECTION A-A'

K.I. JONES ELEMENTARY SCHOOL
2001 WINSTON DRIVE, FAIRFIELD, CALIFORNIA
APPENDIX A

Boring Logs
APPENDIX A

BORING LOGS

Field Procedure for the Collection of Disturbed Samples
Disturbed soil samples were obtained in the field using the following methods.

**Bulk Samples**
Bulk samples of representative earth materials were obtained from the exploratory borings. The samples were bagged and transported to the laboratory for testing.

**The Standard Penetration Test (SPT) Sampler**
Disturbed drive samples of earth materials were obtained by means of a Standard Penetration Test sampler. The sampler is composed of a split barrel with an external diameter of 2 inches and an unlined internal diameter of 1-3/8 inches. The sampler was driven into the ground 12 to 18 inches with a 140-pound hammer free-falling from a height of 30 inches in general accordance with ASTM D 1586. The blow counts were recorded for every 6 inches of penetration; the blow counts reported on the logs are those for the last 12 inches of penetration. Soil samples were observed and removed from the sampler, bagged, sealed and transported to the laboratory for testing.

Field Procedure for the Collection of Relatively Undisturbed Samples
Relatively undisturbed soil samples were obtained in the field using the following methods.

**The Modified Split-Barrel Drive Sampler**
The sampler, with an external diameter of 3.0 inches, was lined with 6-inch long, thin brass liners with inside diameters of approximately 2.4 inches. The sample barrel was driven into the ground with the weight of a hammer in general accordance with ASTM D 3550. The driving weight was permitted to fall freely. The approximate length of the fall, the weight of the hammer, and the number of blows per foot of driving are presented on the boring log as an index to the relative resistance of the materials sampled. The samples were removed from the sample barrel in the brass liners, sealed, and transported to the laboratory for testing.
<table>
<thead>
<tr>
<th>DEPTH (feet)</th>
<th>Bulk SAMPLES</th>
<th>MOISTURE (%)</th>
<th>DRY DENSITY (PCF)</th>
<th>SYMBOL</th>
<th>CLASSIFICATION U.S.C.S.</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td></td>
<td></td>
<td></td>
<td>Bulk sample.</td>
<td></td>
</tr>
<tr>
<td>5</td>
<td></td>
<td></td>
<td></td>
<td>Modified split-barrel drive sampler.</td>
<td></td>
</tr>
<tr>
<td>XX/XX</td>
<td></td>
<td></td>
<td></td>
<td>2-inch inner diameter split-barrel drive sampler.</td>
<td></td>
</tr>
<tr>
<td>10</td>
<td></td>
<td></td>
<td></td>
<td>No recovery with modified split-barrel drive sampler, or 2-inch inner diameter split-barrel drive sampler.</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Sample retained by others.</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Standard Penetration Test (SPT).</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>No recovery with a SPT.</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Shelby tube sample. Distance pushed in inches/length of sample recovered in inches.</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>No recovery with Shelby tube sampler.</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Continuous Push Sample.</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Seepage.</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Groundwater encountered during drilling.</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Groundwater measured after drilling.</td>
<td></td>
</tr>
</tbody>
</table>

**MAJOR MATERIAL TYPE (SOIL):**

- Solid line denotes unit change.
- Dashed line denotes material change.

**Attitudes:** Strike/Dip
- b: Bedding
- c: Contact
- j: Joint
- f: Fracture
- F: Fault
- cs: Clay Seam
- s: Shear
- bss: Basal Slide Surface
- sf: Shear Fracture
- sz: Shear Zone
- sbs: Shear Bedding Surface

The total depth line is a solid line that is drawn at the bottom of the boring.

**BORING LOG EXPLANATION SHEET**

**BORING LOG**

Explanation of Boring Log Symbols

<table>
<thead>
<tr>
<th>PROJECT NO.</th>
<th>DATE</th>
<th>FIGURE</th>
</tr>
</thead>
</table>

<table>
<thead>
<tr>
<th>Ninio &amp; Moore</th>
<th>BORING LOG</th>
<th>Explanation of Boring Log Symbols</th>
<th>PROJECT NO.</th>
<th>DATE</th>
<th>FIGURE</th>
</tr>
</thead>
</table>
# Soil Classification Chart per ASTM D 2488

## Coarse-Grained Soils

### PRIMARY DIVISIONS

- GRAVEL
  - more than 50% of coarse fraction retained on No. 4 sieve
- SAND
  - 50% or more of coarse fraction passes No. 4 sieve

### SECONDARY DIVISIONS

#### GROUP SYMBOL

#### GROUP NAME
- CLEAN GRAVEL less than 5% fines
- CLEAN SAND less than 5% fines
- SAND with FINES more than 12% fines
- GRAVEL with DUAL CLASSIFICATIONS 5% to 12% fines
- GRAVEL with FINES more than 12% fines
- SAND with DUAL CLASSIFICATIONS 5% to 12% fines
- SAND with FINES more than 12% fines

### GRAIN SIZE

<table>
<thead>
<tr>
<th>DESCRIPTION</th>
<th>SIEVE SIZE</th>
<th>GRAIN SIZE</th>
<th>APPROXIMATE SIZE</th>
</tr>
</thead>
<tbody>
<tr>
<td>Boulders</td>
<td>&gt; 12&quot;</td>
<td>&gt; 12&quot;</td>
<td>Larger than basketball-sized</td>
</tr>
<tr>
<td>Cobbles</td>
<td>3 - 12&quot;</td>
<td>3 - 12&quot;</td>
<td>Fist-sized to basketball-sized</td>
</tr>
</tbody>
</table>

### GRAVEL

- **Fine**
  - passing #200
  - < 0.0029"
  - Flour-sized and smaller

### SAND

- **Clean**: less than 5% fines
  - SW
  - well-graded SAND
  - SP
  - poorly graded SAND
  - SM
  - silty SAND
  - SC
  - clayey SAND
  - SC-SM
  - silty, clayey SAND
- **Silt and Clay Liquid Limit**: less than 50%
  - INORGANIC
    - CL
    - lean CLAY
    - ML
    - SILT
    - CL-ML
    - silty CLAY
  - ORGANIC
    - OL (PI > 4)
    - organic CLAY
    - OL (PI < 4)
    - organic SILT
- **Silt and Clay Liquid Limit**: 50% or more
  - INORGANIC
    - CH
    - fat CLAY
    - MH
    - elastic SILT
  - ORGANIC
    - OH
      - (plots on or above "A"-line)
      - organic CLAY
    - OH
      - (plots below "A"-line)
      - organic SILT

### Finely Organic Soils

- PT
- Peat

### Plasticity Chart

- **Plasticity Index (PI)**
- **Liquid Limit (LL)**
  - 0 - 100%
  - 0 - 70%
  - 0 - 50%
  - 0 - 30%

### Apparent Density - Coarse-Grained Soil

<table>
<thead>
<tr>
<th>APPARENT DENSITY</th>
<th>SPORLING CABLE OR CATHEAD</th>
<th>AUTOMATIC TRIP HAMMER</th>
</tr>
</thead>
<tbody>
<tr>
<td>Very Loose</td>
<td>≤ 4</td>
<td>≤ 8</td>
</tr>
<tr>
<td>Loose</td>
<td>5 - 10</td>
<td>9 - 21</td>
</tr>
<tr>
<td>Medium Dense</td>
<td>11 - 30</td>
<td>22 - 63</td>
</tr>
<tr>
<td>Dense</td>
<td>31 - 50</td>
<td>64 - 105</td>
</tr>
<tr>
<td>Very Dense</td>
<td>&gt; 50</td>
<td>&gt; 105</td>
</tr>
</tbody>
</table>

### Consistency - Fine-Grained Soil

<table>
<thead>
<tr>
<th>CONSISTENCY</th>
<th>SPT (blows/foot)</th>
<th>MODIFIED SPLIT BARREL (blows/foot)</th>
<th>SPT (blows/foot)</th>
<th>MODIFIED SPLIT BARREL (blows/foot)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Very Soft</td>
<td>&lt; 2</td>
<td>&lt; 3</td>
<td>&lt; 1</td>
<td>&lt; 2</td>
</tr>
<tr>
<td>Soft</td>
<td>2 - 4</td>
<td>3 - 5</td>
<td>1 - 3</td>
<td>2 - 3</td>
</tr>
<tr>
<td>Firm</td>
<td>5 - 8</td>
<td>6 - 10</td>
<td>4 - 5</td>
<td>4 - 6</td>
</tr>
<tr>
<td>Stiff</td>
<td>9 - 15</td>
<td>11 - 20</td>
<td>6 - 10</td>
<td>7 - 13</td>
</tr>
<tr>
<td>Very Stiff</td>
<td>16 - 30</td>
<td>21 - 39</td>
<td>11 - 20</td>
<td>14 - 26</td>
</tr>
<tr>
<td>Hard</td>
<td>&gt; 30</td>
<td>&gt; 39</td>
<td>&gt; 20</td>
<td>&gt; 26</td>
</tr>
</tbody>
</table>

---

**USCS Method of Soil Classification**

Explanation of USCS Method of Soil Classification

<table>
<thead>
<tr>
<th>PROJECT NO.</th>
<th>DATE</th>
<th>FIGURE</th>
</tr>
</thead>
</table>
---
## Geotechnical Data

<table>
<thead>
<tr>
<th>Depth (ft)</th>
<th>Bulk Samples</th>
<th>Driven Błows/foot</th>
<th>Moisture (%)</th>
<th>Dry Density (pcf)</th>
<th>Classification U.S.C.S.</th>
<th>Description/Interpretation</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>CL</td>
<td>Asphalt Concrete: Approximately 2 inches thick.</td>
</tr>
<tr>
<td>37</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>CL</td>
<td>Aggregate Base: Approximately 6 inches thick.</td>
</tr>
<tr>
<td>15</td>
<td></td>
<td>21.6</td>
<td>101.5</td>
<td></td>
<td>CL</td>
<td>Fill: Dark Brown, moist, stiff, sandy lean CLAY.</td>
</tr>
<tr>
<td>10</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Alluvium: Dark Brown, moist, stiff, sandy lean CLAY.</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Yellow brown; very stiff.</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Light brown; stiff.</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Total Depth = 10 ft. Backfilled with cement grout.</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Notes: Groundwater, though not encountered at the time of drilling, may rise to a higher level due to seasonal variations in precipitation and several other factors as discussed in the report.</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>The ground elevation shown above is an estimation only. It is based on our interpretations of published maps and other documents reviewed for the purposes of this evaluations. It is not sufficiently accurate for preparing construction bids and design documents.</td>
</tr>
</tbody>
</table>
ASPHALT CONCRETE: Approximately 2 inches thick.
AGGREGATE BASE: Approximately 5 inches thick.
FILL: Dark Brown, moist, stiff, lean CLAY.
ALLUVIUM: Dark Brown, moist, stiff, lean CLAY.

Yellow Brown; trace sand; very stiff.

Light brown; stiff.

Yellow brown; very stiff.
Total depth = 20 ft.
Backfilled with cement grout.

Notes:
Groundwater, though not encountered at the time of drilling, may rise to a higher level due to seasonal variations in precipitation and several other factors as discussed in the report.

The ground elevation shown above is an estimation only. It is based on our interpretations of published maps and other documents reviewed for the purposes of this evaluations. It is not sufficiently accurate for preparing construction bids and design documents.
### Structural Layers

<table>
<thead>
<tr>
<th>Description</th>
<th>Interpretation</th>
</tr>
</thead>
<tbody>
<tr>
<td>ASPHALT CONCRETE</td>
<td>Approximately 2 inches thick.</td>
</tr>
<tr>
<td>AGGREGATE BASE</td>
<td>Approximately 5 inches thick.</td>
</tr>
<tr>
<td>FILL</td>
<td>Dark brown, moist, stiff, lean CLAY.</td>
</tr>
<tr>
<td>ALLUVIUM</td>
<td>Dark brown, moist, stiff, lean CLAY.</td>
</tr>
</tbody>
</table>

### Groundwater and Notes
- Groundwater, though not encountered at the time of drilling, may rise to a higher level due to seasonal variations in precipitation and several other factors as discussed in the report.
- The ground elevation shown above is an estimation only. It is based on our interpretations of published maps and other documents reviewed for the purposes of this evaluations. It is not sufficiently accurate for preparing construction bids and design documents.

### Table of Logs

<table>
<thead>
<tr>
<th>Depth (feet)</th>
<th>Bulk Samples</th>
<th>Driven Blows/foot</th>
<th>Moisture (%)</th>
<th>Dry Density (PCF)</th>
<th>Symbol</th>
<th>Classification</th>
<th>U.S.C.S.</th>
<th>Description/Interpretation</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>CL</td>
<td></td>
<td></td>
<td>ASPHALT CONCRETE: Approximately 2 inches thick.</td>
</tr>
<tr>
<td>5</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>CL</td>
<td></td>
<td></td>
<td>AGGREGATE BASE: Approximately 5 inches thick.</td>
</tr>
<tr>
<td>10</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>CL</td>
<td></td>
<td></td>
<td>FILL: Dark brown, moist, stiff, lean CLAY.</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>ALLUVIUM: Dark brown, moist, stiff, lean CLAY.</td>
</tr>
<tr>
<td>15</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Yellow brown; hard.</td>
</tr>
<tr>
<td>20</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Total depth = 10 ft. Backfilled with cement grout.</td>
</tr>
</tbody>
</table>

### Additional Information
- DATE DRILLED: 7/5/2017  
- BORING NO.: B-3  
- GROUND ELEVATION: 66' ± (MSL)  
- SHEET: 1 OF 1  
- METHOD OF DRILLING: 8" HAS, B-53 Truck Mounted, Exploration Geo, 3" HA top 5'  
- DRIVE WEIGHT: 140 LBS (wireline)  
- DROP: 30 INCHES  
- SAMPLED BY: GL  
- LOGGED BY: GL  
- REVIEWED BY: TPS  

K.I. JONES ELEMENTARY SCHOOL  
2001 WINSTON DRIVE, FAIRFIELD, CALIFORNIA  
403070001
### DESCRIPTION/INTERPRETATION

<table>
<thead>
<tr>
<th>DEPTH (feet)</th>
<th>SAMPLES</th>
<th>BLOWS/FOOT</th>
<th>MOISTURE (%)</th>
<th>DRY DENSITY (pcf)</th>
<th>CLASSIFICATION U.S.C.S.</th>
<th>DATE DRILLED</th>
<th>BORING NO.</th>
<th>GROUND ELEVATION</th>
<th>METHOD OF DRILLING</th>
<th>DRIVE WEIGHT</th>
<th>DROP</th>
<th>SAMPLED BY</th>
<th>LOGGED BY</th>
<th>REVIEWED BY</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>CL</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>7/5/2017</td>
<td>B-4</td>
<td>65' ± (MSL)</td>
<td>8&quot; HAS, B-53 Truck Mounted, Exploration Geo, 3&quot; HA top 5'</td>
<td>140 LBS (wireline)</td>
<td>30 INCHES</td>
<td>GL</td>
<td>GL</td>
<td>TPS</td>
</tr>
<tr>
<td>5</td>
<td>CL</td>
<td></td>
<td></td>
<td></td>
<td></td>
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<td></td>
<td></td>
<td></td>
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<td></td>
</tr>
<tr>
<td>10</td>
<td></td>
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<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>15</td>
<td>27</td>
<td>19.5</td>
<td>104.0</td>
<td></td>
<td>CL</td>
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<td></td>
</tr>
<tr>
<td>20</td>
<td>58</td>
<td>19.3</td>
<td>107.6</td>
<td></td>
<td>CL</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
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<tr>
<td>25</td>
<td>61</td>
<td></td>
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<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

**ASPHALT CONCRETE:**
Approximately 2 inches thick.

**AGGREGATE BASE:**
Approximately 5 inches thick.

**FILL:**
Dark Brown, moist, stiff, lean CLAY.

**ALLUVIUM:**
Dark Brown, moist, stiff, lean CLAY.

**DESCRIPTION/INTERPRETATION**

- Yellowish brown.
- Very stiff.
  - @ 9.5: Groundwater measured approximately 10 minutes after drilling.
- Reddish yellow brown; trace SAND.
- Yellowish brown.
  - @ 18.5': Groundwater encountered during drilling.
<table>
<thead>
<tr>
<th>DEPTH (feet)</th>
<th>SAMPLES</th>
<th>BLOWS/FOOT</th>
<th>MOISTURE (%)</th>
<th>DRY DENSITY (pcf)</th>
<th>CLASSIFICATION U.C.S.</th>
</tr>
</thead>
<tbody>
<tr>
<td>20</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>25</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>30</td>
<td></td>
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<td>35</td>
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<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>40</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Total Depth = 20 ft.

Backfilled with cement grout.

Groundwater first encountered at 18.5 ft. during drilling. Groundwater measured at 9.5 ft. after drilling.

Notes:
The ground elevation shown above is an estimation only. It is based on our interpretations of published maps and other documents reviewed for the purposes of this evaluation.

Groundwater may rise to a level higher than that measured in borehole due to relatively slow rate of seepage in clay and several other factors as discussed in the report.
**DESCRIPTION/INTERPRETATION**

- **CL**
  - ASPHALT CONCRETE:
    - Approximately 2 inches thick.
  - AGGREGATE BASE:
    - Approximately 5 inches thick.
  - FILL:
    - Dark brown, moist, stiff, lean CLAY.
  - ALLUVIUM:
    - Dark brown, moist, stiff, lean CLAY.
<table>
<thead>
<tr>
<th>DEPTH (feet)</th>
<th>SAMPLES</th>
<th>BLOWS/FOOT</th>
<th>MOISTURE (%)</th>
<th>DRY DENSITY (PCF)</th>
<th>SYMBOL</th>
<th>CLASSIFICATION U.S.C.S.</th>
<th>DESCRIPTION/INTERPRETATION</th>
</tr>
</thead>
<tbody>
<tr>
<td>20</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>ALLUVIUM: (Continued)</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Olive brown, moist, very stiff, sandy lean CLAY.</td>
<td></td>
</tr>
<tr>
<td>25</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Olive gray.</td>
</tr>
<tr>
<td>30</td>
<td>29</td>
<td>55</td>
<td>21.1</td>
<td>104.0</td>
<td></td>
<td>Total Depth = 30 ft.</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Groundwater first encountered at 18.5 ft.</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Backfilled with cement grout.</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Notes: The ground elevation shown above is an estimation only. It is based on our interpretations of published maps and other documents reviewed for the purposes of this evaluation. Groundwater may rise to a level higher than that measured in borehole due to relatively slow rate of seepage in clay and several other factors as discussed in the report.</td>
<td></td>
</tr>
<tr>
<td>DEPTH (feet)</td>
<td>SAMPLES</td>
<td>BLOWS/FOOT</td>
<td>MOISTURE (%)</td>
<td>DRY DENSITY (PCF)</td>
<td>CLASSIFICATION</td>
<td>U.S.C.S.</td>
<td>DESCRIPTION/INTERPRETATION</td>
</tr>
<tr>
<td>-------------</td>
<td>----------</td>
<td>------------</td>
<td>--------------</td>
<td>------------------</td>
<td>----------------</td>
<td>----------</td>
<td>----------------------------</td>
</tr>
<tr>
<td>0</td>
<td>CL</td>
<td></td>
<td></td>
<td></td>
<td>CLAY.</td>
<td></td>
<td>FILL: Dark brown, moist, very stiff, lean CLAY.</td>
</tr>
<tr>
<td>5</td>
<td>CL</td>
<td></td>
<td></td>
<td></td>
<td>CLAY.</td>
<td></td>
<td>ALLUVIUM: Dark brown, moist, very stiff, lean CLAY.</td>
</tr>
<tr>
<td>10</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Total Depth = 5 ft. Backfilled with soil.</td>
</tr>
<tr>
<td>15</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Notes: Groundwater, though not encountered at the time of drilling, may rise to a higher level due to seasonal variations in precipitation and several other factors as discussed in the report.</td>
</tr>
<tr>
<td>20</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>The ground elevation shown above is an estimation only. It is based on our interpretations of published maps and other documents reviewed for the purposes of this evaluations. It is not sufficiently accurate for preparing construction bids and design documents.</td>
</tr>
</tbody>
</table>

DATE DRILLED: 7/05/2017  
BORING NO.: B-6  
GROUND ELEVATION: 63' ± (MSL)  
METHOD OF DRILLING: 3" HA top 5'  
DRIVE WEIGHT: N/A  
DROP: N/A  
SAMPLED BY: GL  
LOGGED BY: GL  
REVIEWED BY: TPS
APPENDIX B

Laboratory Testing
APPENDIX B
LABORATORY TESTING

Classification
Soils were visually and texturally classified in accordance with the Unified Soil Classification System (USCS) in general accordance with ASTM D 2488-00. Soil classifications are indicated on the logs of the exploratory borings in Appendix A.

Moisture Content
The moisture content of samples obtained from the exploratory borings was evaluated in accordance with ASTM D 2216. The test results are presented on the boring logs in Appendix A.

In Place Density Tests
The dry density of relatively undisturbed samples obtained from the exploratory borings was evaluated in general accordance with ASTM D 2937. The test results are presented on the logs of the exploratory borings in Appendix A.

Gradation Analysis
A gradation analysis test was performed on selected representative soil samples in general accordance with ASTM D 422. The grain size distribution curve is shown on Figures B-1, B-2, and B-3. The test results were utilized in evaluating the soil classification in accordance with the Unified Soil Classification System (USCS).

Atterberg Limits
Tests were performed on selected representative fine-grained soil samples to evaluate the liquid limit, plastic limit, and plasticity index in general accordance with ASTM D 4318. These test results were utilized to evaluate the soil classification in accordance with the USCS. The test results and classifications are shown on Figure B-4.

Expansion Index Test
The expansion index of a selected material was evaluated in general accordance with ASTM D 4829. The specimen was molded under a specified compactive energy at approximately 50 percent saturation (plus or minus 1 percent). The prepared 1 inch thick by 4 inch diameter specimen was loaded with a surcharge of 144 pounds per square foot and inundated with tap water. Readings of volumetric swell were made for a period of 24 hours. The test results are presented on Figure B-5.

Unconfined Compression Test
Unconfined compression tests were performed on relatively undisturbed samples in general accordance with ASTM D 2166. The test results are shown on Figures B-6 and B-7.

R Value
The resistance value, or R value, for site soils was evaluated in general accordance with California Test (CT) 301. Samples were prepared and evaluated for exudation pressure and expansion pressure. The equilibrium R-value is reported as the lesser or more conservative of the two calculated results. The test results are shown on Figure B-8.

Soil Corrosivity Tests
Soil pH, and resistivity tests were performed on a representative sample in general accordance with California Test (CT) 643. The soluble sulfate and chloride contents of the selected sample were evaluated in general accordance with CT 417 and CT 422, respectively. The test results are presented on Figure B-9.
PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 422
FIGURE B-2
GRADATION TEST RESULTS
K.I. JONES ELEMENTARY SCHOOL
2001 WINSTON DRIVE, FAIRFIELD, CALIFORNIA
403070001 | 7/17

Coarse      Fine     Coarse    Medium                   SILT CLAY
3"      2" 1-1/2" 1"  3/4"     3/8"    4    10 30 50

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 422
PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 422
<table>
<thead>
<tr>
<th>SYMBOL</th>
<th>LOCATION</th>
<th>DEPTH (ft)</th>
<th>LIQUID LIMIT</th>
<th>PLASTIC LIMIT</th>
<th>PLASTICITY INDEX</th>
<th>USCS CLASSIFICATION</th>
<th>USCS</th>
</tr>
</thead>
<tbody>
<tr>
<td>•</td>
<td>B-1</td>
<td>0.0-5.0</td>
<td>48</td>
<td>12</td>
<td>36</td>
<td>CL</td>
<td>CL</td>
</tr>
<tr>
<td>■</td>
<td>B-3</td>
<td>0.0-5.0</td>
<td>38</td>
<td>12</td>
<td>26</td>
<td>CL</td>
<td>CL</td>
</tr>
<tr>
<td>◆</td>
<td>B-5</td>
<td>13.5-14.0</td>
<td>36</td>
<td>10</td>
<td>26</td>
<td>CL</td>
<td>CL</td>
</tr>
</tbody>
</table>

NP - INDICATES NON-PLASTIC

FIGURE B-4

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 4318
<table>
<thead>
<tr>
<th>SAMPLE LOCATION</th>
<th>SAMPLE DEPTH (ft)</th>
<th>INITIAL MOISTURE (percent)</th>
<th>COMPACTED DRY DENSITY (pcf)</th>
<th>FINAL MOISTURE (percent)</th>
<th>VOLUMETRIC SWELL (in)</th>
<th>EXPANSION INDEX</th>
<th>POTENTIAL EXPANSION</th>
</tr>
</thead>
<tbody>
<tr>
<td>B-4</td>
<td>0.0-5.0</td>
<td>11.7</td>
<td>102.7</td>
<td>24.4</td>
<td>0.061</td>
<td>60</td>
<td>Medium</td>
</tr>
</tbody>
</table>

PERFORMED IN GENERAL ACCORDANCE WITH

- [ ] UBC STANDARD 18-2
- [x] ASTM D 4829

FIGURE B-5

EXPANSION INDEX TEST RESULTS

K.I. JONES ELEMENTARY SCHOOL
2001 WINSTON DRIVE, FAIRFIELD, CALIFORNIA

403070001 | 7/17
PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 2166

<table>
<thead>
<tr>
<th>SYMBOL</th>
<th>DESCRIPTION</th>
<th>SOIL TYPE</th>
<th>SAMPLE LOCATION</th>
<th>SAMPLE DEPTH (ft.)</th>
<th>MOISTURE CONTENT w. (%)</th>
<th>DRY DENSITY $\gamma_d$ (pcf)</th>
<th>STRAIN RATE (%/min.)</th>
<th>UNDRAINED SHEAR STR $s_u$ (ksf)</th>
</tr>
</thead>
<tbody>
<tr>
<td>◆</td>
<td>Yellow brown lean CLAY (CL)</td>
<td>CL</td>
<td>B-2</td>
<td>6.0-6.5</td>
<td>17.2</td>
<td>114.9</td>
<td>1.04</td>
<td>3.42</td>
</tr>
</tbody>
</table>

**FIGURE B-6**

UNCONFINED COMPRESSION RESULTS

K.I. JONES ELEMENTARY SCHOOL
2001 WINSTON DRIVE, FAIRFIELD, CALIFORNIA

403070001 | 7/17

6 and 7_UNCONFINED COMPRESSION's B-2 & B-5
Light brown lean CLAY (CL) | CL | B-5 | 6.0-6.5 | 19.3 | 109.9 | 1.05 | 2.89

**PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 2166**
<table>
<thead>
<tr>
<th>SAMPLE LOCATION</th>
<th>SAMPLE DEPTH (ft)</th>
<th>SOIL TYPE</th>
<th>R-VALUE</th>
</tr>
</thead>
<tbody>
<tr>
<td>B-6</td>
<td>0.0-5.0</td>
<td>CLAY</td>
<td>6</td>
</tr>
</tbody>
</table>

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 2844/CT 301
<table>
<thead>
<tr>
<th>SAMPLE LOCATION</th>
<th>SAMPLE DEPTH (ft)</th>
<th>pH ^1</th>
<th>RESISTIVITY ^1</th>
<th>SULFATE CONTENT ^2</th>
<th>CHLORIDE CONTENT ^3 (ppm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>B-5</td>
<td>0.0-5.0</td>
<td>8.0</td>
<td>960</td>
<td>120</td>
<td>0.012</td>
</tr>
</tbody>
</table>

^1 PERFORMED IN GENERAL ACCORDANCE WITH CALIFORNIA TEST METHOD 643
^2 PERFORMED IN GENERAL ACCORDANCE WITH CALIFORNIA TEST METHOD 417
^3 PERFORMED IN GENERAL ACCORDANCE WITH CALIFORNIA TEST METHOD 422
APPENDIX C

KC Engineering Company Boring Logs
# Log of Test Boring

**Boring No.: 1**

**Project:** KI Jones Elementary School  
**Client:** Fairfield-Suisun Unified School District  
**Location:** 2001 Winston Drive, Fairfield, CA  
**Driller:** Britton Exploration Inc.  
**Drill Rig:** CME-55  
**Depth to Water:** Initial $\frac{\pi}{\pi}$: 8'

<table>
<thead>
<tr>
<th>Depth</th>
<th>Soil Classification</th>
<th>Converted SPT Blow Count (Blows/ft.)</th>
<th>Density (pcf)</th>
<th>Moisture Content (% Per cent)</th>
<th>Additional Tests and Remarks (LL, PI, UCC, etc., Gradation)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>Dark Brown CLAY; moist, stiff</td>
<td>CH</td>
<td>11</td>
<td>103.1</td>
<td>23.2</td>
</tr>
<tr>
<td>1-1</td>
<td>Light Brown Sandy CLAY; wet, firm</td>
<td>CL</td>
<td>6</td>
<td>96.4</td>
<td>25.0</td>
</tr>
</tbody>
</table>
| 1-2   | Yellowish Brown Clayey SAND; wet, medium dense | SC | 12 | 11 | 96.4 | 25.0 | % Gravel=3.0  
% Sand=58.8  
% < 200=38.2 |
| 1-3   | As Above | CL | | | | |
| 1-4   | Light Brown Sandy CLAY; wet, stiff | | | | | |

This information pertains only to this boring and is not necessarily indicative of the whole site.

**Project No.:** VV2359  
**Date:** 12-2-06  
**Elevation:** NA  
**Logged By:** PGT  
**Boring Diameter:** 6"  
**Final $\frac{\pi}{\pi}$: Final After: hrs.**
LOG OF TEST BORING
BORING NO.: 1

PROJECT: KI Jones Elementary School
CLIENT: Fairfield-Suisun Unified School District
LOCATION: 2001 Winston Drive, Fairfield, CA
DRILLER: Britton Exploration Inc.
DRILL RIG: CME-55
DEPTH TO WATER: INITIAL § : 8'

PROJECT NO.: VV2359
DATE: 12-2-06
ELEVATION: NA
LOGGED BY: PGT
BORING DIAMETER: 6"
FINAL § : AFTER: hrs.

GEOTECHNICAL DESCRIPTION AND CLASSIFICATION

DEPTH SAMPLE NO. SAMPLER GRAPHIC LOG
1-5
1-6
30
35
40
45
50

SOIL CLASSIFICATION
As Above; wet, stiff
Grey & Brown MUDSTONE; wet, friable, highly weathered
Boring Terminated @ 39'4"
Groundwater Encountered @ 8'

CONVERTED SPT BLOW COUNT (BLOW/FT.)
9

DRY DENSITY (PCF)
13

MOISTURE CONTENT (PERCENT)
100+

ADDITIONAL TESTS AND REMARKS
(LL, PI, UCC, s&c, Gradation)

This information pertains only to this boring and is not necessarily indicative of the whole site.

KC ENGINEERING CO. Figure 3
# LOG OF TEST BORING

**BORING NO.: 2**

<table>
<thead>
<tr>
<th>DEPTH</th>
<th>SAMPLE NO.</th>
<th>SAMPLER</th>
<th>GRAPHIC LOG</th>
<th>GEOTECHNICAL DESCRIPTION AND CLASSIFICATION</th>
<th>SOIL CLASSIFICATION</th>
<th>CONVERTED SPT BLOW COUNT (BLOWS/FIT.)</th>
<th>DRY DENSITY (PCF)</th>
<th>MOISTURE CONTENT (PERCENT)</th>
<th>ADDITIONAL TESTS AND REMARKS (LL, PI, UCC, a&amp;c, Gradation)</th>
</tr>
</thead>
</table>
| 0     |            |         |             | Reddish Brown Clayey SAND; slightly moist, medium dense to dense (FILL) | SC                 | 30                                    | 111.1             | 11.3                        | %Gravel=2.0  
%Sand=60.6  
%<200=37.4                                             |
| 2-1   |            |         |             | Reddish Brown Sandy CLAY; moist, very stiff | CL                 | 21                                    | 106.3             | 20.4                        |                                                          |
| 2-2   |            |         |             | As Above; moist, very stiff |                    | 19                                    | 110.2             | 20.2                        |                                                          |
| 2-3   |            |         |             | Boring Terminated @ 15'  
Dry At Time Of Drilling |                    |                        |                  |                             |                                                          |

This information pertains only to this boring and is not necessarily indicative of the whole site.
# LOG OF TEST BORING

**BORING NO.: 3**

**PROJECT:** KI Jones Elementary School  
**CLIENT:** Fairfield-Suisun Unified School District  
**LOCATION:** 2001 Winston Drive, Fairfield, CA  
**DRILLER:** Britton Exploration Inc.  
**DRILL RIG:** CME-55  
**DEPTH TO WATER:** INITIAL \( \psi \) : 8'  
**DATE:** 12-2-06  
**ELEVATION:** NA  
**LOGGED BY:** PGT  
**BORING DIAMETER:** 6"  
**FINAL \( \psi \) :** AFTER: hrs.

<table>
<thead>
<tr>
<th>DEPTH</th>
<th>SAMPLE NO.</th>
<th>SAMPLER</th>
<th>GRAPHIC LOG</th>
<th>GEOTECHNICAL DESCRIPTION AND CLASSIFICATION</th>
<th>SOIL CLASSIFICATION</th>
<th>CONVERTED SPT BLOW COUNT (BLOWS/FT.)</th>
<th>DRY DENSITY (PCF)</th>
<th>MOISTURE CONTENT (PERCENT)</th>
<th>ADDITIONAL TESTS AND REMARKS (LL, PL, UC, etc., Gradation)</th>
</tr>
</thead>
</table>
| 0     |            |         |             | Dark Brown CLAY; moist, very stiff             | CH                  | 19                                     | 107.2            | 20.0                       | LL=66%  
|       |            |         |             |                                               |                     |                                        |                  |                            | PI=43                                           |
| 3-1   |            |         |             | Light Brown Sandy CLAY; wet, very stiff        | CL                  | 16                                     | 110.8            | 20.2                       |                                 |
| 3-2   |            |         |             |                                               |                     |                                        |                  |                            |                                                 |
| 3-3   |            |         |             | Boring Terminated @ 15'  
Groundwater Encountered @ 8' |                     | 18                                     | 108.3            | 20.8                       |                                                 |

This information pertains only to this boring and is not necessarily indicative of the whole site.

KC ENGINEERING CO.  
**Figure 5**