APPENDIX B

Geotechnical Investigation Report
(For Reference Only)
REPORT OF
GEOTECHNICAL INVESTIGATION
PROPOSED EUCALYPTUS BOOSTER STATION
EASTERN MUNICIPAL WATER DISTRICT
MORENO VALLEY, CALIFORNIA

October 28, 2010
October 28, 2011  
Project No. 112644

Mr. Jeff Endersby, P.E.  
Brown and Caldwell  
9665 Chesapeake Drive, Suite 201  
San Diego, California 92123

Subject: Report of Geotechnical Investigation  
Proposed Eucalyptus Booster Station  
Eastern Municipal Water District  
Moreno Valley, California

Dear Mr. Endersby:

Kleinfelder is pleased to present this report summarizing our geotechnical investigation for the subject site. The site is located on the south side of Eucalyptus Avenue approximately 1,300 feet west of Moreno Beach Drive in Moreno Valley, California. The purpose of our investigation was to evaluate the subsurface conditions at the site and develop geotechnical recommendations for project design and construction.

In summary, based on our understanding of the project, the site is suitable for development as planned provided the recommendations presented in the attached report are incorporated into the design and construction. The conclusions and recommendations presented in this report are subject to the limitations presented in Section 7.

We appreciate the opportunity to be of service on this project and look forward to continued work with you. If you have any questions or require additional information, please do not hesitate to contact the undersigned at your convenience.

Respectfully submitted,  
KLEINFELDER WEST, INC.  

Eric W. Noel, PE, GE  
Senior Geotechnical Engineer  

Dale Hamelehle, PG, CEG  
Senior Geologist

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Page ii of iv  
October 28, 2011
# TABLE OF CONTENTS

<table>
<thead>
<tr>
<th>Section</th>
<th>Page</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.0 INTRODUCTION</td>
<td>1</td>
</tr>
<tr>
<td>1.1 General</td>
<td>1</td>
</tr>
<tr>
<td>1.2 Background, Site, and Proposed Project Description</td>
<td>1</td>
</tr>
<tr>
<td>1.3 Purpose and Scope</td>
<td>3</td>
</tr>
<tr>
<td>2.0 REGIONAL GEOLOGY</td>
<td>7</td>
</tr>
<tr>
<td>3.0 LOCAL GEOLOGIC CONDITIONS</td>
<td>8</td>
</tr>
<tr>
<td>3.1 Subsurface Soil Conditions</td>
<td>8</td>
</tr>
<tr>
<td>3.1.1 Undocumented Artificial Fill</td>
<td>8</td>
</tr>
<tr>
<td>3.1.2 Alluvium</td>
<td>9</td>
</tr>
<tr>
<td>3.1.3 Bedrock (Undifferentiated Tonalite)</td>
<td>9</td>
</tr>
<tr>
<td>3.2 Groundwater</td>
<td>9</td>
</tr>
<tr>
<td>3.3 Collapse Potential</td>
<td>10</td>
</tr>
<tr>
<td>3.4 Expansive Soils</td>
<td>11</td>
</tr>
<tr>
<td>3.5 Corrosivity</td>
<td>12</td>
</tr>
<tr>
<td>4.0 GEOLOGIC HAZARDS</td>
<td>14</td>
</tr>
<tr>
<td>4.1 Faulting and Seismicity</td>
<td>14</td>
</tr>
<tr>
<td>4.2 Fault Rupture</td>
<td>15</td>
</tr>
<tr>
<td>4.3 Secondary Seismic Hazards</td>
<td>16</td>
</tr>
<tr>
<td>4.4 Liquefaction and Seismically Induced Settlement</td>
<td>16</td>
</tr>
<tr>
<td>5.0 CONCLUSIONS AND RECOMMENDATIONS</td>
<td>18</td>
</tr>
<tr>
<td>5.1 General</td>
<td>18</td>
</tr>
<tr>
<td>5.2 Seismic Design Considerations</td>
<td>19</td>
</tr>
<tr>
<td>5.2.1 General</td>
<td>19</td>
</tr>
<tr>
<td>5.2.2 Ground Shaking</td>
<td>19</td>
</tr>
<tr>
<td>5.3 Earthwork</td>
<td>20</td>
</tr>
<tr>
<td>5.3.1 General</td>
<td>20</td>
</tr>
<tr>
<td>5.3.2 Construction Observation</td>
<td>20</td>
</tr>
<tr>
<td>5.3.3 Stripping and Grubbing</td>
<td>21</td>
</tr>
<tr>
<td>5.3.4 Remedial Grading</td>
<td>21</td>
</tr>
<tr>
<td>5.3.5 On-site Materials</td>
<td>23</td>
</tr>
<tr>
<td>5.3.6 Import Materials</td>
<td>23</td>
</tr>
<tr>
<td>5.3.7 Compaction Criteria</td>
<td>24</td>
</tr>
<tr>
<td>5.4 Foundation Recommendations</td>
<td>24</td>
</tr>
<tr>
<td>5.4.1 General</td>
<td>24</td>
</tr>
<tr>
<td>5.4.2 Conventional Shallow Foundations</td>
<td>24</td>
</tr>
<tr>
<td>5.4.3 Slabs-On-Grade</td>
<td>26</td>
</tr>
<tr>
<td>5.4.4 Exterior Slabs-On-Grade</td>
<td>27</td>
</tr>
<tr>
<td>5.4.5 Footing Observation</td>
<td>28</td>
</tr>
<tr>
<td>5.4.6 Estimated Settlements</td>
<td>28</td>
</tr>
<tr>
<td>5.4.7 Lateral Resistance and Earth Pressures</td>
<td>29</td>
</tr>
</tbody>
</table>
### TABLE OF CONTENTS (continued)

<table>
<thead>
<tr>
<th>Section</th>
<th>Page</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>5.5</strong> Temporary Excavations</td>
<td>30</td>
</tr>
<tr>
<td>5.5.1 General</td>
<td>30</td>
</tr>
<tr>
<td>5.5.2 Temporary Slopes</td>
<td>31</td>
</tr>
<tr>
<td>5.5.3 Shoring</td>
<td>32</td>
</tr>
<tr>
<td><strong>5.6</strong> Trench Backfill</td>
<td>32</td>
</tr>
<tr>
<td>5.6.1 Overexcavation</td>
<td>32</td>
</tr>
<tr>
<td>5.6.2 Trench Backfill</td>
<td>33</td>
</tr>
<tr>
<td>5.6.3 Compaction Criteria</td>
<td>34</td>
</tr>
<tr>
<td><strong>5.7</strong> Surface and Subgrade Drainage</td>
<td>34</td>
</tr>
<tr>
<td><strong>5.8</strong> Preliminary Pavement Sections</td>
<td>36</td>
</tr>
<tr>
<td><strong>6.0</strong> ADDITIONAL SERVICES</td>
<td>40</td>
</tr>
<tr>
<td>6.1 Plans and Specifications Review</td>
<td>40</td>
</tr>
<tr>
<td>6.2 Construction Observation and Testing</td>
<td>40</td>
</tr>
<tr>
<td><strong>7.0</strong> LIMITATIONS</td>
<td>41</td>
</tr>
<tr>
<td><strong>8.0</strong> REFERENCES</td>
<td>44</td>
</tr>
</tbody>
</table>

### PLATES

- **Plate 1** Site Vicinity Map
- **Plate 2** Boring Location Map

### APPENDICES

- **Appendix A** Field Exploration
- **Appendix B** Laboratory Testing
1.0 INTRODUCTION

1.1 GENERAL

Kleinfelder was retained by Brown and Caldwell to conduct a geotechnical investigation at the site of the proposed Eucalyptus Booster Station for Eastern Municipal Water District (EMWD, the District). Our services were performed in accordance with our authorized contract based on our proposal entitled Proposal to Conduct a Geotechnical Investigation, Eucalyptus Booster Station Final Design, Eastern Municipal Water District, Moreno Valley, California, (DBA10P027R), 2\textsuperscript{nd} revision dated March 18, 2010.

This report presents our recommendations relative to the geotechnical aspects of design and construction for the proposed project. Conclusions and recommendations presented in this report are based on the subsurface conditions encountered at the locations of our field excavations, and the provisions and requirements outlined in the Additional Services and Limitations sections of this report. Recommendations presented in this report should not be extrapolated to other areas or be used for other projects without our prior review. The recommendations presented within are provided in general accordance with the 2007 California Building Code and Unified Facilities Criteria as applicable to the project proposed.

1.2 BACKGROUND, SITE, AND PROPOSED PROJECT DESCRIPTION

The site proposed for development is located on the south side of Eucalyptus Avenue approximately 1,300 feet west of Moreno Beach Drive in Moreno Valley, California (see Plate 1, Site Vicinity Map and Plate 2, Boring Location Map). Development in the vicinity of the site consists of shopping centers located to the east, north, and northwest. Housing communities are located to the southwest and mountains are located to the south.

The Eucalyptus Booster Station (BS) site is located at an approximate latitude of 33.9368°N and longitude of -117.1829°W. Based upon the USGS Topographic Map of the Sunnymead (7.5’) quadrangle (photo-revised 1980), the site has an approximate elevation of 1,764 feet. Access to the site is security-controlled and was coordinated through EMWD representatives.
We understand that the District’s site for the proposed Eucalyptus BS is approximately 70-feet-wide by 140-feet-long, and is adjacent to the District’s 20-foot wide water pipeline easement located along the east side of the site. Based on the Preliminary Design prepared by Brown and Caldwell (November 17, 2010), the Eucalyptus BS project will consist of the following components:

- A split-faced reinforced concrete block building, 27’-4” wide by 65’-4” long, with a pitched roof, asphalt shingle roof decking with skylights over each vertical turbine pump.
- The station will consist of three rooms: one for the mechanical and electrical equipment; pumps, valves and appurtenances, and electrical/motor control center with climate control; and another room with sound attenuation to house an emergency electric generator, and a restroom.
- Eight-foot-wide double doors for equipment maintenance and access to the pump room.
- Two 12-foot wide removable louvers, one along the north and one on the east side of the generator room, for generator access and removal.
- Reinforced masonry split-faced block screen wall around the site (north, south and west walls) blending in with the overall theme and design of the future Beazer Homes development, including a 24-foot wide ornamental style steel drive gate and 4-foot wide walk gate.
- Wrought Iron fencing along the east property boundary.
- Four vertical turbines, constant speed pumps (three duty, one standby) with a design point of 1900 gallons per minute (gpm) and 120 feet total dynamic head (TDH).
- Full-capacity 350-kilowatt (kw) emergency power generator located within a separate sound attenuated room.
- Interior and exterior facility lighting (task, flood, and security).
- 20-inch discharge header, 30-inch suction header and 14-inch bypass.

Appurtenant utility construction is expected to include laterals and tie-ins.

The site is rough graded with moderate low-lying vegetation on the surface. The land surrounding the site consists of roughly rectangular-shaped graded parcels with terraced fill pads that were originally a part of a residential housing tract that has not been constructed and currently appear abandoned. Based on a review of previous
aerial photographs of the area from 1996 to 2009 (Google Earth, September 2010), the site was covered with moderate amounts of vegetation and contained a natural drainage swale that traversed the length of the site from north to south. The drainage swale appeared to bisect (N-S direction) the area proposed for the booster station structure. Previous use of the site prior to association with the residential development appears to be agricultural. The drainage swale has since been backfilled with soil; however, it is unclear the method of backfill or if backfill compaction was performed under engineering control. The fill appears to have been placed periodically throughout the years based upon our photo review.

Our field observation of the soil samples collected identified areas of surficial and subsurface fill that was non-uniform in nature. No records documenting this fill placement could be located by Brown and Caldwell or EMWD which would indicate if proper removals of loose soil material were performed in the drainage swale, how the fill was placed, or how it was compacted. Additionally, within the southern portion of the site, fill has been placed that is approximately 5 to 6 feet higher in elevation than the northern majority of the site. This fill appears to be associated with grading for the adjacent abandoned residential tract. Survey points or property markers were not available during our field investigation, however, based upon hand measurements utilizing the site maps and dimensioning provided, portions of the toes of slopes and fill associated with the residential grading may impinge on the Booster site. Boring B-3 was drilled adjacent to the toe of the residential fill pad slope. Based upon review of past aerial photographs, the project site may have been used as a construction staging area or used to traverse equipment for the residential tract. Additionally, artificial fill (spoil) and small soil stockpiles, possibly associated with the nearby grading, are present on the Booster site and in the vicinity of the toes of the residential grading near the southern portion of the site. Preliminary grading and foundation plans were not available at the time that this report was prepared; however, we anticipate that cuts and fills required to bring the project site to finished grade will be on the order of 5-feet or less, excluding the requirements for remedial grading.

1.3 PURPOSE AND SCOPE

The purpose of this investigation was to evaluate subsurface conditions at the site and provide geotechnical recommendations for design and construction of the proposed development. A description of the scope of services performed is presented below.
Task 1 – Literature Review

We began our investigation by reviewing available geologic and geotechnical literature pertaining to the project site. We reviewed published and unpublished soil and geologic data in our files and as available from appropriate public agencies. This included a review of hazard, fault and geologic maps prepared by the California Geological Survey, the U.S. Geological Survey, the County of Riverside, and other governmental agencies as they relate to the site.

Task 2 – Utility Clearance and Encroachment Permits

Prior to the field exploration, we contacted Underground Service Alert (USA) to identify potential conflicts between the planned boring locations and existing, documented, underground utilities. Locations were adjusted as necessary to avoid conflict with existing utilities. Access to the site was coordinated through Eastern Municipal Water District.

Task 3 – Field Exploration

The subsurface exploration program included advancing three exploratory borings. The borings were drilled to depths ranging from approximately 31 to 50½ feet below existing grades using a truck-mounted hollow-stem auger drill rig. Our typical sampling interval within the borings was approximately 2 feet within the upper approximately 10 feet, and 5 feet at depths below 10 feet to the full depth explored. Samples were recovered from the borings using a Modified California (ring) Sampler or Standard Penetration Test (SPT) sampler. Both ring and SPT samplers were driven using a 140-pound hammer falling for a height of 30 inches. The number of blows necessary to drive either the ring or the SPT samplers was recorded. In addition, representative bulk samples were collected from the drill cuttings.

Each soil sample was observed and described in general accordance with the Unified Soil Classification System (USCS). The depth to groundwater, if any, was measured in the boreholes. Following drilling, sampling, logging, and groundwater measurement, the borings were backfilled with native cuttings. The locations of the borings are shown on Plate 2, Boring Location Map.
A Kleinfelder staff engineer supervised the field operations and logged the borings. Selected bulk, disturbed and drive samples were retrieved, sealed and transported to our laboratory for further evaluation. The boring logs are presented in Appendix A.

**Task 4 – Laboratory Soil Testing**

Laboratory testing was performed on soil samples collected during our field exploration to substantiate field classifications and to assess the physical characteristics of the subsurface soils. Testing consisted of in-situ dry unit weight and moisture content, grain size distribution, laboratory maximum dry unit weight and optimum moisture content, direct shear strength, consolidation/collapse potential, and expansion potential. A summary of the laboratory test program and the test results are presented in Appendix B.

**Task 5 – Geotechnical Analyses and Report Preparation**

We analyzed the field data obtained, performed engineering analyses, and are providing recommended design parameters for earthwork and shallow foundations for the proposed structure as described herein. This report has been prepared by a Registered Civil Engineer and Certified Engineering Geologist. This report includes the following items:

- A description of the proposed project including a site plan showing the approximate boring locations. The proposed boring locations were located in the field by hand measuring devices such as tape or a wheel, based on the control provided.

- A description of the subsurface site conditions encountered during our field investigation including groundwater conditions.

- A description of the site geologic setting and geology-related hazards, including a liquefaction, subsidence, and seismic settlement analysis.

- A discussion of regional geology and site seismicity.

- A description of local and regional active faults, their distances from the site, and their potential for future earthquakes.

- A discussion of other geologic hazards such as ground shaking, landslides, flooding, and tsunamis.
• A discussion of site conditions, including the excavation characteristics and geotechnical suitability of the site for the general type of construction proposed.

• Recommendations for foundation design including parameters for shallow foundations, and subgrade preparation;

• Recommendations for geotechnical seismic design coefficients in accordance with the 2007 California Building Code (CBC);

• Recommendations for lateral load resistance.

• Anticipated total and differential settlements for the recommended foundation system;

• Recommendations for slabs-on-grade, including recommendations for reducing the potential for moisture transmission through interior slabs.

• Recommendations for collapsible or expansive soils (if applicable).

• Recommendations for site preparation, earthwork, and fill compaction specifications;

• Recommendations for imported fill (if required) for use in compacted fills;

• Recommended trench sidewall slope inclinations and geotechnical engineering parameters for design of cantilevered and braced shoring.

• Evaluation and recommendations on the use of excavated materials;

• Special preparation requirements for pipeline subgrade, if required.

• Recommendations for underground utility trench bedding and backfill;

• Preliminary recommendations for de-watering (if applicable);

• A discussion of the corrosion potential of the near-surface soils encountered during our field exploration.

• An appendix, which will include a summary of the field investigation and laboratory testing program.
Regionally, the site is situated within the northern Peninsular Ranges Geomorphic Province of California. The Peninsular Ranges are a northwest-southeast oriented complex of blocks separated by similarly trending faults which extend 125 miles from the Transverse Ranges south to the Mexican border and beyond another 775 miles to the tip of Baja California. The province varies in width from 30 to 100 miles and is bounded on the east by the Colorado Desert and the Gulf of California, and on the west by the Pacific Ocean. The Peninsular Ranges contain Jurassic-age and Cretaceous-age igneous and metamorphic rocks, as well as a thick sequence of marine and non-marine sedimentary rock. The Peninsular Ranges Province is further described by sub-units, which include the Perris Block, the Santa Ana Mountains, and the San Jacinto Mountains. The subject site is situated within the Perris Block, which is characterized as a broad area of intermixed valleys and low mountain ranges situated between the Elsinore and San Jacinto fault zones.
3.0 LOCAL GEOLOGIC CONDITIONS

3.1 SUBSURFACE SOIL CONDITIONS

Three borings were drilled, sampled, logged, and backfilled within the proposed site area. The results of our site investigation indicate that undocumented fill and two geologic units underlie the project site. Undocumented artificial fill soils (fill) were encountered within each boring excavation. The locations of our exploratory borings are shown on Plate 2, Boring Location Map. Descriptions of the materials encountered during our exploration are provided on the boring logs in Appendix A. Generalized descriptions of these units, as observed on the site and as described in the cited literature, are presented below:

3.1.1 Undocumented Artificial Fill

Artificial fill soils were encountered within all three boring excavations at the site. The observed fill varied in thickness from 5 to 7 feet; however, deeper fills may exist in isolated areas not encountered during our investigation. The presence of additional fill soils should be evaluated during grading. The fill soils encountered generally consist of olive brown to yellow brown, medium dense to very dense, silty sand (SM). Borings B-1 and B-2 encountered fill potentially associated with the previously existing drainage swale though the site. Boring B-3 encountered fill associated with the fill pad placed for the nearby residential development and possibly the previously existing drainage swale. A compaction report documenting removals of unsuitable drainage swale or other materials, and the placement of fill within the fill pad and/or drainage swale area was not provided during this investigation. As such, the observed fill and removals (if any) are undocumented. Additional undocumented fill associated with the drainage swale may exist across the site. Non-uniformity between the existing alluvium and undocumented fill is expected to exist across the proposed building pad. To reduce the potential for non-uniform or deleterious fill conditions throughout the building pad, recommends for removal and re-compaction of the upper soils are provided in the conclusions and recommendations section of this report.
3.1.2 Alluvium

Alluvium was observed to underlie the fill at each of the boring excavations. The alluvium in Borings B-1 and B-2 extended to the bedrock located at approximately 25 feet and 35 feet below existing grade, respectively. Boring B-3 encountered alluvium below the artificial fill to the full depth of exploration of 31½ feet. The alluvial soils generally consist of olive brown to yellow brown, sand with varying amounts of silt, clay, and gravel.

3.1.3 Bedrock (Undifferentiated Tonalite)

Granitic bedrock was encountered in Borings B-1 and B-2 at depths of 25 and 35 feet, respectively below the existing ground surface. A review of the referenced regional geology maps has identified nearby exposures of bedrock as undifferentiated Tonalite and based on our borings, we observed the bedrock to be weathered Tonalite.

3.2 GROUNDWATER

Groundwater was not encountered within the excavations performed for this study. Regional groundwater depths in the study area were evaluated based on records available through the Department of Water Resources and the Western Municipal Water District Cooperative Well Measuring Program (Fall 2009).

The depth to groundwater in a well located within approximately 3 miles southeast of the site and at a gage datum of 1,500 feet, was recorded as approximately 102 feet below the ground surface (bgs) in 1941 (Well No. 03S02W07P001S). The depth to groundwater in a well located approximately 1-1/2 miles east of the site, and at a gage datum of 1,774 feet, was recorded to range from approximately 209 to 240 feet below the ground surface (bgs) between 2004 and 2009 (Well No. 03S03W02L002R). The depth to groundwater in a well located approximately 1-3/4 miles south of the site, and at a gage datum of 1,539 feet, was recorded at approximately 100 feet below the ground surface (bgs) in 1952 (Well No. 03S03W15F001S). Groundwater levels in the vicinity of the site are anticipated to be greater than 100 feet below the existing ground surface.
Due to the recorded depths to groundwater being greater than 100 feet below the ground surface, groundwater is not anticipated to be encountered during construction. Fluctuations of the groundwater level, localized zones of perched or seepage water and soil moisture content should be anticipated during and following the rainy season or from irrigation. Irrigation of landscaped areas on and adjacent to the site can also cause a fluctuation of local groundwater levels and perched water conditions can develop.

### 3.3 COLLAPSE POTENTIAL

Collapsible soil deposits generally exist in regions of moisture deficiency. Collapsible soils are generally defined as soils that have potential to suddenly decrease in volume upon increase in moisture content even without increase in external loads. Soils susceptible to collapse include loess, weakly cemented sands and silts where the cementing agent is soluble (e.g. soluble gypsum, halite), valley alluvial deposits within semi-arid to arid climate, and certain granite residual soils.

The degree of collapse of a soil can be defined by the Collapse Potential (CP) value, which is expressed as a percent of collapse of the total sample using the Collapse Potential Test (ASTM Standard Test Method D 5333). Based on the Naval Facilities Engineering Command (NAVFAC) Design Manual 7.1, the severity of collapse potential is commonly evaluated by the following Table 1, Collapse Potential Values.

<table>
<thead>
<tr>
<th>Collapse Potential Value</th>
<th>Severity of Problem</th>
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<tbody>
<tr>
<td>0-1%</td>
<td>No Problem</td>
</tr>
<tr>
<td>1-5%</td>
<td>Moderate Problem</td>
</tr>
<tr>
<td>5-10%</td>
<td>Trouble</td>
</tr>
<tr>
<td>10-20%</td>
<td>Severe Trouble</td>
</tr>
<tr>
<td>&gt; 20%</td>
<td>Very Severe Trouble</td>
</tr>
</tbody>
</table>

Table 1 can be combined with other factors such as the probability of ground wetting to occur on-site and the extent or depth of potential collapsible soil zone to evaluate the potential hazard by collapsible soil at a specific site. A hazard ranking system
associated with collapsible soil as developed by Hunt (1984) is presented in Table 2, Collapsible Soil Hazard Ranking System.

### Table 2
**Collapsible Soil Hazard Ranking System**

<table>
<thead>
<tr>
<th>Degree of Hazard</th>
<th>Definition of Hazard</th>
</tr>
</thead>
<tbody>
<tr>
<td>No Hazard</td>
<td>No hazard exists where the potential collapse magnitudes are non-existent under any condition of ground wetting.</td>
</tr>
<tr>
<td>Low Hazard</td>
<td>Low hazards exist where the potential collapse magnitudes are small (CP values 0-1%) and tolerable or the probability of significant ground wetting is low.</td>
</tr>
<tr>
<td>Moderate Hazard</td>
<td>Moderate hazards exist where the potential collapse magnitudes are undesirable (CP values 1-5%) or the probability of substantial ground wetting is low, or the occurrence of the collapsible unit is limited.</td>
</tr>
<tr>
<td>High Hazard</td>
<td>High hazard exist where potential collapse magnitudes are undesirably high (CP values 5-20%) and the probability of occurrence is high.</td>
</tr>
</tbody>
</table>

The project site is located in a geologic environment where the potential for collapsible soil exists. The results of collapse potential tests performed on 6 selected samples from different depths throughout the project site indicate a range of collapse potential on the order of 0.4 to 1.8 percent for the total soil layer thickness at applied vertical stress of 2,000 psf.

Based on our laboratory test results and geotechnical analyses conducted, it is our opinion that, in general, a low to moderate degree of hazard exists with regard to collapsible soil conditions. Collapse test results of near-surface soils are presented in Appendix B.

To reduce the collapse potential hazard at this site, recommends for removal and re-compaction of the upper soils are provided in the conclusions and recommendations section of this report.

### 3.4 EXPANSIVE SOILS

One near surface soil sample obtained from the upper 5 feet of site soils during our field exploration was tested for expansion potential in accordance with ASTM D 4829. The
results of this test indicate an Expansion Index (EI) of 0 indicating a “very low” Expansion Potential. Soils with an EI between 0 and 20 are considered to have a “very low” expansion potential as defined by ASTM Test Method D 4829. The California Building Code Section 1802.3.2 presents soils with an EI greater than 20 are deemed to be “expansive”. Testing of the final subgrade soils after completion of grading within the building pad and at the footing grade should be performed to further evaluate the expansion potential and confirm or modify the recommendations presented herein.

3.5 CORROSIVITY

Two samples of the near-surface soils at the site were tested for potential to corrosion of concrete and ferrous metals. The tests were conducted in general accordance with the California Standard Test Methods (CTM) to evaluate pH, resistivity, and water-soluble sulfate and chloride content. The test results are presented in Appendix B. These tests should be considered as only an indicator of corrosivity for the samples tested. Other earth materials found on site may be more, less, or of a similar corrosive nature. Although Kleinfelder does not practice corrosion engineering, the corrosion values from the soil tested are normally considered as being severely corrosive to buried metals and as possessing a “negligible” to “moderate” exposure to sulfate attack for concrete as defined in American Concrete Institute (ACI) 318, Section 4.3, as referenced in the 2007 CBC.

Water-soluble sulfates in soil can react adversely with concrete. ACI 318 provides the relationship between corrosivity to concrete and sulfate concentration, presented below in Table 3, Sulfate Content and Corrosion Correlation:

<table>
<thead>
<tr>
<th>Water-Soluble Sulfate in Soil (ppm)</th>
<th>Corrosivity to Concrete</th>
</tr>
</thead>
<tbody>
<tr>
<td>0-150</td>
<td>Negligible</td>
</tr>
<tr>
<td>150 – 1,500</td>
<td>Moderate</td>
</tr>
<tr>
<td>1,500 – 10,000</td>
<td>Severe</td>
</tr>
<tr>
<td>Over 10,000</td>
<td>Very Severe</td>
</tr>
</tbody>
</table>

Test results (presented in Appendix B) indicate sulfate contents ranging from approximately 39 to 156 ppm, indicating “negligible” to “moderate” sulfate exposure.
Soil resistivity is a measure of how easily electrical current flows through soils and is the most influential factor. Based on the findings of studies presented in ASTM STP 1013 titled “Effects of Soil Characteristics on Corrosion” (February, 1989), the approximate relationship between soil resistivity and soil corrosivity was developed as shown below in Table 4, Soil Resistivity and Corrosion Correlation.

<table>
<thead>
<tr>
<th>Soil Resistivity (Ohm-cm)</th>
<th>Corrosivity to Ferrous Metals</th>
</tr>
</thead>
<tbody>
<tr>
<td>0 to 900</td>
<td>Very Severely Corrosive</td>
</tr>
<tr>
<td>900 to 2,300</td>
<td>Severely Corrosive</td>
</tr>
<tr>
<td>2,300 to 5,000</td>
<td>Moderately Corrosive</td>
</tr>
<tr>
<td>5,000 to 10,000</td>
<td>Mildly Corrosive</td>
</tr>
<tr>
<td>10,000 to &gt;100,000</td>
<td>Very Mildly Corrosive</td>
</tr>
</tbody>
</table>

Test results (presented in Appendix B) show pH values ranging from 6.9 to 8.2, chloride contents ranging from 216 to 388 ppm and minimum resistivity ranging from 520 to 890 Ohm-cm. These resistivity results indicate a “Very Severely Corrosive” condition. We recommend that a competent corrosion engineer be retained to evaluate the corrosion potential of the site soils to concrete and buried metals, to recommend further testing as required, and to provide specific corrosion mitigation methods appropriate for the project. At a minimum, corrosion design should be performed in accordance with the California Building Code.
4.0 GEOLOGIC HAZARDS

4.1 FAULTING AND SEISMICITY

The project site is located in the highly seismic southern California region within the influence of several fault systems that are considered to be active or potentially active. An active fault is a fault that has experienced seismic activity during historic time (since roughly 1800) or exhibits evidence of surface displacement during Holocene time (Bryant and Hart, 2007). The definition of “potentially active” varies. A generally accepted definition of “potentially active” is a fault showing evidence of displacement that is older than 11,000 years (Holocene age) and younger than 1.7 million years (Pleistocene age). However, “potentially active” is no longer used as criteria for zoning by the California Geologic Survey (CGS). The terms “sufficiently active” and “well-defined” are now used by the CGS as criteria for zoning faults under the Alquist-Priolo Earthquake Fault Act. A “sufficiently active fault” is a fault that shows evidence of Holocene surface displacement along one or more of its segments and branches, while a “well-defined fault” is a fault whose trace is clearly detectable by a trained geologist as a physical feature at or just below the ground surface. The definition “inactive” generally implies that a fault has not been active since the beginning of the Pleistocene Epoch (older than 1.7 million years old).

We have performed a computer-aided search of the known active and potentially active faults within a 62-mile (100-kilometer) radius of the site and researched available literature to assess the maximum credible earthquakes expected to be generated on each fault. Table 5 summarizes these parameters for 11 out of the 43 known active and potentially active faults or fault segments within the searched radius of the site that in our opinion will have the greatest impact upon the site. Selection of the faults was based on their proximity to the site and their potential to generate strong ground motion on the site. Table 5 was generated using in part the EQFAULT computer program (version 3.00) developed by Blake (2000) as modified using the fault parameters from CGS Open File Report. This table represents deterministic data only and therefore does not identify the probability of reactivation or the on-site effects from earthquakes occurring on any of the other faults in the region.
The closest active faults to the project site are the San Jacinto – San Jacinto Valley, and the San Jacinto – San Bernardino fault zones, with closest distances of approximately 1.7 and 6.3 miles, respectively.

<table>
<thead>
<tr>
<th>Fault Name</th>
<th>Approximate Distance to Site mi. (km)</th>
<th>Magnitude of Maximum Earthquake **</th>
</tr>
</thead>
<tbody>
<tr>
<td>San Jacinto – San Jacinto Valley</td>
<td>1.7 (2.8)</td>
<td>6.9</td>
</tr>
<tr>
<td>San Jacinto – San Bernardino</td>
<td>6.3 (10.2)</td>
<td>6.7</td>
</tr>
<tr>
<td>San Andreas-San Bernardino</td>
<td>12.6 (20.2)</td>
<td>7.5</td>
</tr>
<tr>
<td>San Andreas-All Southern Segments</td>
<td>12.6 (20.2)</td>
<td>8.0</td>
</tr>
<tr>
<td>North Frontal Fault Zone</td>
<td>19.9 (32.1)</td>
<td>7.2</td>
</tr>
<tr>
<td>San Jacinto – Anza</td>
<td>20.4 (32.9)</td>
<td>7.2</td>
</tr>
<tr>
<td>Elsinore – Glen Ivy</td>
<td>21.5 (34.6)</td>
<td>6.8</td>
</tr>
<tr>
<td>Elsinore - Temecula</td>
<td>22.4 (36.0)</td>
<td>6.8</td>
</tr>
<tr>
<td>Cucamonga</td>
<td>22.4 (36.1)</td>
<td>6.9</td>
</tr>
<tr>
<td>Chino- Central Avenue (Elsinore)</td>
<td>23.3 (37.5)</td>
<td>6.7</td>
</tr>
<tr>
<td>Cleghorn</td>
<td>23.7 (38.2)</td>
<td>6.5</td>
</tr>
</tbody>
</table>

** Moment Magnitude is an estimate of an earthquake’s size by utilizing rock rigidity, amount of slip, and area of rupture.

These active and potentially active faults are capable of producing seismic shaking and it is anticipated that the project site will periodically experience ground acceleration as the result of moderate to large magnitude earthquakes.

### 4.2 FAULT RUPTURE

Faults identified by the State as being either active or potentially active are not known to cross the project site. The proposed site is not located within a State of California designated Earthquake Fault Rupture Zone (Bryant and Hart, 2007) and, as such, the potential for surface fault rupture across the site is considered to be “low.” While fault rupture would most likely occur along previously established fault traces, fault rupture could occur at other locations.
4.3 SECONDARY SEISMIC HAZARDS

Secondary seismic hazards related to ground shaking include tsunamis, seiches, and ground deformation. Due to the inland location of the site and lack of a nearby large body of open water the hazards from tsunamis and seiches are considered low. The site is not within the area identified as a dam inundation area for the Pigeon Pass or Lake Perris reservoirs (City of Moreno Valley General Plan, 2006). The site is located down-gradient from an approximate 100 foot diameter tank reservoir. The tank is located south of the site. Due to the distance from the tank to the site (approximately 1700 feet), and the gentle topographic gradient between the tank and the site, it is our opinion that the potential for water reaching the site should a tank failure occur is considered low. As the site is relatively flat and level and not in the near vicinity of free-face slopes, hazards from lateral spreading and landslides are considered low.

Non-tectonic ground deformation consists of surface cracking of the ground with little to no displacement. This type of deformation is not caused by fault rupture. Rather it is generally associated with differential shaking of two or more geologic units with differing physical characteristics. The site possesses relatively consistent geologic materials. The potential for ground deformation is considered “low” under current conditions and proposed grades.

4.4 LIQUEFACTION AND SEISMICALLY INDUCED SETTLEMENT

Liquefaction is the sudden loss of shear strength in a loose, saturated granular soil due to vibratory motions such as those associated with earthquakes. Seismically induced soil liquefaction generally occurs in loose, saturated, cohesionless soil when pore pressures within the soil increase during ground shaking. The increase in pore pressure transforms the soil from a solid to a semi-liquid state. These soils typically lose a portion or all of their shear strength and regain strength sometime after shaking stops.

Groundwater was not encountered in the borings excavated for this investigation, to the maximum depth explored of approximately 50½ feet below existing grade. Our research indicates that the groundwater levels in the vicinity of the site are anticipated to be greater than 100 feet below the existing ground surface. Due to the anticipated depth to groundwater, it is our opinion that under a preliminary, screening level
liquefaction analysis, the site has a remote potential for liquefaction. The site is located in a “low” liquefaction potential zone as identified on the Riverside County Geographic Information System.

During a strong seismic event, seismically induced settlement can occur within loose to moderately dense, unsaturated granular soils. Settlement caused by ground shaking is often non-uniformly distributed, which can result in differential settlement. Based upon methods presented by Tokimatsu and Seed (1987), the potential for seismically induced dry settlement of soils above the groundwater table was calculated to be on the order of approximately 1/4 inch. Differential settlement is typically equivalent to one-half to two-thirds of the total seismically induced settlements.

The total seismically induced settlement is exclusive and independent of any static settlement that may occur from static loads. The potential for total and differential static settlement is addressed in a later Section of this report.
5.0 CONCLUSIONS AND RECOMMENDATIONS

5.1 GENERAL

Based on our field exploration, laboratory testing, and geotechnical analyses conducted for this study, it is our opinion that the site is suitable, from a geotechnical standpoint, for construction of the new improvements as planned, provided the recommendations presented in this report are incorporated into project design and construction.

A geotechnical constraint for development of this site, as identified by our investigation, is the existence of artificial fill soils throughout the site which are not considered suitable for the support of building structures. The existing artificial fill soils are not considered suitable for structure support due to the potential for inadequate removal of unsuitable soils during the backfill of the drainage swale, the undocumented nature of compaction control and documentation during backfill, and the non-uniformity of earth units (alluvium and fill) which would be present across the proposed building pad. Based on laboratory test results and our geotechnical analyses conducted, it is our opinion that, in general, a low to moderate level of hazard exists with regard to collapsible soil conditions. To reduce the undocumented fill and collapse potential hazard at this site, we recommend removal and re-compaction of the upper soils. The recommendations presented are intended to reduce the magnitude and severity of differential settlement distress to the proposed structure, such that the estimated ground settlement presented within can be accommodated in structural design.

The site has potentially corrosive soils that can be generally classified as “Very Severely” corrosive. As Kleinfelder does not practice corrosion engineering, we recommend that an engineer qualified to practice corrosion engineering evaluate the corrosion potential of the site soils to concrete and buried metals, to recommend further testing as required, and to provide specific corrosion mitigation methods appropriate for the project.

Geotechnical engineering recommendations addressing remedial grading and foundations are presented in the following sections of this report. The following opinions, conclusions, and recommendations are based on the properties of the materials encountered in our exploratory excavations, and the results of our laboratory
testing program. The recommendations presented in this report may change pending a review of final grading plans (if found different than presented within) and foundation plans. Recommendations presented in this report should not be extrapolated to other areas or be used for other projects (beyond those expressly identified within) without our prior review and comment.

5.2 SEISMIC DESIGN CONSIDERATIONS

5.2.1 General

The site is located in the seismically active southern California region within the influence of several fault systems that are considered to be active or potentially active. These active and potentially active faults are capable of producing potentially damaging seismic shaking at the site. It is anticipated that the project site will periodically experience strong ground acceleration as the result of moderate to large magnitude earthquakes.

5.2.2 Ground Shaking

The project site, like all Southern California, is a seismically active area and is likely to experience ground shaking as a result of earthquakes on nearby or more distant faults. The San Jacinto fault zone dominates the seismicity of the area. The San Jacinto-San Jacinto Valley fault zone (CDMG, 1998) is located approximately 1.7 miles (2.8 kilometers) east of the project site.

We understand that the proposed structures will be designed in accordance with the requirements of the latest 2007 edition of the California Building Code (CBC). It should be noted that the seismic provision of the 2007 CBC are based on and refer to (for more requirements) “Minimum Design Loads for Buildings and Other Structures, ASCE Standard 7” (referred to herein as “ASCE 7”).

The following table provides the seismic design parameters based on the 2007 CBC. According to the 2007 CBC, every structure, and portion thereof, including non-structural components that are permanently attached to structures and their supports and attachments, shall be designed and constructed to resist the effects of earthquake motions in accordance with ASCE 7-05, excluding Chapter 14 and Appendix 11A. The
seismic design category for a structure may be determined in accordance with Section 1613 of the 2007 CBC or ASCE 7-05. Based upon the soils and bedrock encountered and anticipated at depths approaching 100 feet, we classify the site as Site Class C. The 2007 CBC Seismic Design Parameters are summarized in Table 6, 2007 CBC Seismic Design Parameters.

Table 6  
2007 CBC Seismic Design Parameters*

| Site Class |  
|------------|---|
| C          |   |
| S_s (Figure 1613.5(3)) (g) | 1.792 |
| S_t (Figure 1613.5(4)) (g) | 0.668 |
| F_a (Table 1613.5.3(1)) | 1.0 |
| F_v (Table 1613.5.3(2)) | 1.3 |
| S_{MS} (Equation 16-37) (g) | 1.792 |
| S_{M1} (Equation 16-38) (g) | 0.869 |
| S_{DS} (Equation 16-39) (g) | 1.195 |
| S_{D1} (Equation 16-40) (g) | 0.579 |

*USGS Java Ground Motion Parameter Calculator 5.0.9a

5.3 EARTHWORK

5.3.1 General

All site preparation and earthwork operations should be performed in accordance with applicable codes, safety regulations, and other local, state, or federal specifications. All references to maximum unit weight should be established in accordance with the current ASTM Standard Test Method D 1557 and may supersede references cited herein.

5.3.2 Construction Observation

The recommendations presented in this report are based on our understanding of the proposed project and on our evaluation of the data collected. The interpolated subsurface conditions should be evaluated in the field during construction. Final project
drawings and specifications should be reviewed by the project geotechnical consultant prior to the commencement of construction. The project geotechnical consultant should observe the foundation preparation, grading, and backfilling operations. Compacted fill and backfill soils should be tested for specified compaction by the geotechnical consultant.

5.3.3 Stripping and Grubbing

Prior to site grading, existing structures and foundations (if any), surficial soil and debris stockpiles, vegetation (including the entire removal of tree root-balls, if any), landscaping, topsoil, existing asphalt concrete, Portland cement concrete, oversized materials, wet and/or soft soil, and deleterious matter should be removed and disposed of outside the project limits. Deeper stripping or grubbing may be required where concentrations of organics or other deleterious materials are encountered during site grading. Stripped topsoil (less any debris) may be stockpiled and reused for landscape purposes; however, this material should not be incorporated into any engineered backfill. Undocumented fill and other soil (less any debris and deleterious matter) may be stockpiled and reused for engineered backfill. Concrete, brick, organic matter, and other debris within end-dumped stockpiles may not be used as fill materials.

The demolition or relocation of underground structures and/or utilities, if encountered, should be accomplished under the observation of our representative in order to identify areas requiring over-excavation and compaction during grading operations. Any existing utilities that extend beyond the limits of the proposed development and are to be abandoned in place should be appropriately capped to prevent migration of soil and/or water. All debris or structures that may be encountered during the stripping operations, including concrete, asphalt concrete, wood, steel, piping, etc. should be separated and disposed of off-site. All demolition of underground structures should be accomplished under the observation of our representative in order to identify areas requiring over-excavation and compaction during grading observation.

5.3.4 Remedial Grading

Subsequent to stripping and grubbing operations, areas to receive fill should be stripped of all topsoil, and any loose or soft earth materials until a uniform, firm subgrade is exposed, as evaluated by the geotechnical engineer or geologist.
To reduce the potential effects of static settlement and/or collapse induced differential settlement, we recommend that, after stripping and grubbing, existing undocumented artificial fill soils across the building pad footprint should be removed until native alluvium is exposed (as evaluated by the project geotechnical engineer or geologist) or overexcavated to a minimum depth of 7-feet below the existing grade, whichever is deeper. Within the southern portion of the site, if a portion of the proposed building pad is to extend into the existing fill pad or slopes associated with the nearby abandoned residential development, the existing fill pad or slope should be removed. Overexcavation should then be performed as above such that the area is then overexcavated a minimum 7-feet below the resulting grade. Slopes for the residential fill pad may require reconfiguration to remain stable.

Additionally, the building pad and retaining/block wall areas should be overexcavated such that there is a minimum of 3 feet of engineered, compacted fill that exists beneath the footing at the completion of grading. The bottom of the overexcavation for the building pad should extend laterally a distance of 7-feet beyond the building's perimeters. Within retaining/block wall areas, the bottom of the overexcavation should extend laterally a distance of 1-foot on each side of the retaining wall footing. Remedial grading within pavement areas should be based upon the minimum requirements outlined in Section 5.8.

All over-excavations should extend to a depth where the project geologist or his representative has deemed the exposed soils as being suitable for receiving compacted fill. The over-excavation depths may be modified once final grading plans are prepared and reviewed by Kleinfelder. The removal and stripping operations should expose a firm subgrade that is free of significant voids. Additional removals may be required as a result of observation and/or testing of the exposed subgrade subsequent to the required over-excavation. Voids resulting from the removal of existing utility lines or structures should be completely backfilled with compacted fill soil. A representative from our firm should be present during all site clearing, demolition, and grading operations to monitor site conditions; substantiate proper use of materials; evaluate compaction operations; and verify that the recommendations contained herein are met.

Prior to the placement of compacted fill, and after site preparation and remedial grading have been performed, processing of the approved excavation bottom should be performed by scarifying to a minimum depth of 8 inches, moisture conditioning to near optimum moisture content, and compacting to a minimum of 90 percent of the maximum
dry unit weight, based on ASTM D 1557 (minimum 95 percent within the upper 12 inches of subgrade soils beneath pavement sections). Compaction efforts should also include proof-rolling of the exposed overexcavation bottom with heavy compaction-specific equipment (i.e. drum rollers, rubber-tired rollers, etc.). Fill slopes, if any, should be constructed at inclinations no steeper than 2:1 (H:V). Fill slopes taller than 10 feet should be evaluated by the project soils engineer prior to construction.

Since the introduction of water into the soils can initiate collapse at other areas of the site, proper measures should be taken to reduce the potential for future pipe leakage and/or landscape irrigation infiltration into the soils within the areas of the proposed improvements. It is very important that storm water runoff be directed from the roof and other improvements to appropriate storm water collection systems. Diligent efforts should be made to provide an efficient means of transport of runoff water to collection devices, preventing runoff water from infiltrating soils within the vicinity of the structure.

5.3.5 On-site Materials

Borings were advanced with moderate effort within the existing on-site soils. Conventional earth moving equipment is expected to be capable of performing the excavations required.

The on-site soils encountered during our investigation, excluding organics, debris, and/or other deleterious materials, are considered suitable for use as engineered fill. When adequately compacted at appropriate moisture content, these materials can be expected to possess suitable bearing and settlement characteristics for the proposed project.

5.3.6 Import Materials

Import fill soils should be free from deleterious material and debris, and have a general gradation similar to the on-site materials. In general, well-graded mixtures of gravel, sand, and silt, are acceptable for use as engineered fill. Import materials should have a “very low” expansion potential, i.e. have an expansion index of less than 20 as defined by ASTM D 4829. All imported fill should be compacted to the general recommendations provided for engineered fill.
5.3.7 Compaction Criteria

All fill soils, either native, imported, or blended mixes required to bring the site to final grade should be placed as compacted fill. All soil intended for compacted fill should be uniformly moisture-conditioned to near optimum moisture content, placed in horizontal lifts less than 8 inches in loose thickness, and compacted to at least 90 percent relative compaction based on ASTM D 1557. The upper 12 inches of fill within pavement areas should be compacted to a minimum of 95 percent relative compaction (ASTM D 1557). Additional fill lifts should not be placed if the previous lift did not meet the required dry unit weight or if soil conditions are not stable. Careful moisture control should be implemented by the contractor to reduce the potential for pumping conditions in soils above the optimum moisture content.

A schedule of observation and testing should be developed by the geotechnical engineer based upon the proposed excavation method chosen during grading. The 2007 CBC requires that all backfill greater than 12 inches in total thickness be documented by the project geotechnical engineer and tested in accordance with the frequency as identified in Chapter 17, Structural Tests and Special Inspection.

5.4 FOUNDATION RECOMMENDATIONS

5.4.1 General

Based upon the data collected during this study, and from a geotechnical engineering standpoint, it is our opinion that the building foundations may be designed using conventional spread and continuous strip footings with slab-on-grade floors, provided the recommendations presented in this report are incorporated into the design and construction of the project. Foundations should be designed in accordance with the 2007 CBC, as supplemented within. Recommendations for the design of foundations and slabs-on-grade are presented in the following sections.

5.4.2 Conventional Shallow Foundations

The proposed building and retaining/block walls can be designed with conventional continuous strip and isolated spread footings. Footings should have a minimum width of 18 inches and a minimum embedment depth of 18 inches below the bottom of floor
slabs (and any vapor barrier or capillary break) or the lowest adjacent grade (grade within 3 feet laterally), whichever is deeper. Foundations should bear upon a minimum of 3 feet of engineered, compacted fill prepared as recommended in Section 5.3. Foundations constructed in accordance with these recommendations may be designed using an allowable soil bearing pressure of 2,500 pounds per square foot (psf). The allowable bearing pressure provided above is a net value; therefore, the weight of the foundation (which extends below grade) may be neglected when computing dead loads. The allowable bearing pressure may be increased by 1/3 for short-term loading due to wind or seismic forces. The above allowable bearing pressures consider a Factor of Safety of 3 in relation to the ultimate bearing pressure. Footings should be designed and reinforced by the structural engineer for the specific loading, settlement, or expansive soil conditions.

An average modulus of subgrade reaction, k, of 150 pounds per cubic inch (pci) can be used to design footings founded upon compacted fill. Stepped foundations should be designed in accordance with the 2007 CBC.

Soils removed during any footing or utility trench excavation should not be spread or placed over compacted finished grade soils unless subsequently compacted to at least 90 percent of the maximum dry unit weight, as evaluated by ASTM D 1557, and near optimum moisture content.

Footings may experience an overall loss in bearing capacity or an increased potential to settle where located in close proximity to existing or future utility trenches. Furthermore, stresses imposed by the footings on the utility lines may cause cracking, collapse, and/or a loss of serviceability. To reduce this risk, footings should extend below a 1:1 plane projected upward from the closest bottom corner of the trench. Where piping enters or is in close proximity to the structure (as identified above), it should be sleeved in accordance with the manufacturers recommendations to reduce the stresses on the piping imposed by foundation (footing) loads, designed to resist footing loads, or placed at depths where footing loads are insignificant.

At the completion of grading, representative samples of the materials at footing grade should be evaluated for expansion potential and corrosivity.
Special precautions must be taken during the placement and curing of all concrete. Excessive slump (high water-cement ratio) of the concrete and/or improper curing procedures used during either hot or cold weather conditions could lead to excessive shrinkage, or cracking. High water-cement ratio and/or improper curing also greatly increase the water vapor permeability of concrete. We recommend that all concrete selection, placement and curing operations be performed in accordance with the American Concrete Institute (ACI) Manual.

5.4.3 Slabs-On-Grade

Concrete slabs-on-grade may be used for the proposed structure. Slabs-on-grade should be placed on compacted engineered fill that is uniform in composition and placed on overexcavated and compacted soil as recommended within. To reduce the potential for soil moisture migrating upwards toward the slab, the designer may consider a capillary break, such as compacted crushed stone, beneath interior concrete slabs. Concrete floor slabs should be reinforced as required by the structural engineer. The structural engineer should design all slabs for any specific loading, settlement, or expansive soil conditions. At the completion of grading, representative samples of the materials at pad grade should be evaluated for expansion potential to verify or modify these recommendations.

An average modulus of subgrade reaction, k, of 150 pounds per cubic inch (pci) can be used to design slab-on-grade founded upon compacted fill.

Within areas where moisture sensitive flooring will be placed, we recommend the placement of an impermeable membrane such as visqueen, or equivalent, to act as a vapor barrier to reduce the potential for upward migration of water vapor through the slab. The vapor barrier should have a thickness of at least ten mils. To promote uniform curing of the slab and provide protection of the membrane during construction, clean, fine-to-medium-grained sand, 2 inches thick, should be placed on top of and below the membrane, for a 4-inch thick sand blanket, prior to the placement of concrete. This sand should be moistened immediately prior to concrete placement.

Clean sand should conform to the specifications for concrete sand in the latest editions of the Standard Specifications for Public Works Construction (Green Book). Generally, the gradation range of clean sand should consist of 100% passing the No. 4 sieve to
less than 5% passing the No. 200 sieve. In general, the on-site soils do not appear to meet this requirement. All areas adjacent to the building, including planters, should be designed to drain away from the structure to avoid an accumulation of water beneath the slab.

Concrete should not be placed if sand overlying the vapor barrier has been allowed to become wet (due to precipitation or excessive moistening) or if standing water is present above the membrane. Excessive water beneath interior floor slabs could result in significant vapor transmission through the slab, adversely affecting moisture-sensitive floor coverings.

Although vapor barrier systems are currently the industry standard, this system may not be completely effective in preventing floor slab moisture problems. These systems typically will not necessarily ensure that floor slab moisture transmission rates will meet floor-covering manufacturer standards or that indoor humidity levels will be appropriate to inhibit mold growth. In many cases, floor moisture problems are the result of either improper curing of floor slabs or improper application of flooring adhesives.

Our evaluation has not included services to address the influence of moisture vapor transmission through building floor slabs. Slab and flooring system design experts should be retained to provide design recommendations consistent with the maximum allowable moisture transmission rate as it may affect flooring performance and indoor humidity levels.

Special precautions must be taken during the placement and curing of all concrete slabs. Excessive slump (high water-cement ratio) of the concrete and/or improper curing procedures used during either hot or cold weather conditions could lead to excessive shrinkage, cracking, or curling of the slabs. High water-cement ratio and/or improper curing also greatly increase the water vapor permeability of concrete. We recommend that all concrete placement and curing operations be performed in accordance with the American Concrete Institute (ACI) Manual.

5.4.4 Exterior Slabs-On-Grade

Concrete slabs-on-grade may be used for proposed lightly loaded flatwork, i.e. sidewalks. Slabs-on-grade should be placed on at least 6 inches of recompacted, engineered fill that is uniform in composition and thickness and prepared as outlined in
Section 5.3.7. Subgrade soils within slab areas should be in a stable, non-pumping condition at the time of concrete placement.

5.4.5 Footing Observation

Prior to placing steel or concrete, footing excavations should be cleaned of all debris, loose or soft soil, and water. All footing excavations should be observed by an engineer or geologist from our firm just prior to placing steel or concrete to verify that the recommendations contained herein are implemented during construction.

5.4.6 Estimated Settlements

Total static settlement of the foundation will vary depending on the plan dimensions of the foundation and the actual load supported. Actual footing dimensions and structural loads were not available for the preparation of this report, however, based upon the foundation dimensions presented within, the assumed maximum bearing pressures provided, and assuming the site is prepared as recommended within this report, it is our opinion that a total and differential static settlement between similarly loaded adjacent isolated columns or over a distance of 50 feet for wall footings should not exceed 1-inch and 1/2-inch, respectively. It may be considered that the building structure will settle independently of piping or pump cans. For the purpose of the estimation of the magnitude of differential settlement, the designer may consider that piping is fixed in relation to the building settlement presented above.

Additionally, for design purposes of the current structure proposed and evaluated, and assuming the site is prepared at recommended within, we recommend an additional potential differential settlement be added to the above static settlements. We recommend a seismically induced dry differential settlement total of 1/4 inch be added to the above static differential settlement and be applied between adjacent isolated columns or over a distance of 50 feet for wall footings.

At a minimum, foundations should be designed by the structural engineer for the specific static and seismic settlement conditions presented. Due to the granular nature of the site soils, the total static settlement is expected to occur during and shortly after construction.
5.4.7 Lateral Resistance and Earth Pressures

- Retaining walls should be designed for an active soil pressure equivalent to a fluid density of 45 pcf. The active lateral earth pressures are for horizontal (level) backfills using the on-site native soils on walls that are free to rotate at least 0.1 percent of the wall height. Walls, which are restrained against movement or rotation at the top, should be designed for an at-rest equivalent fluid pressure of 65 pcf. The lateral earth pressure values for level backfill are provided for walls backfilled with drainage materials and existing on-site soils.

- Resistance to lateral loads (including those due to wind or seismic forces) may be provided by frictional resistance between the bottom of concrete foundations and the underlying soil, and by passive soil pressure against the foundations. An allowable coefficient of friction of 0.35 may be used between cast-in-place concrete foundations and slabs and the underlying soil. Allowable passive pressure may be taken as equivalent to the pressure exerted by a fluid weighing 350 pounds per cubic foot (pcf). Vertical uplift resistance may consider a soil unit weight of 120 pounds per cubic foot.

- Passive resistance for thrust blocks bearing against firm natural soil or properly compacted backfill can be calculated using an equivalent fluid pressure of 350 pcf. The maximum passive resistance should not exceed 2,000 psf.

- Friction and soil pressure values (resistance) presented above are considered to have a factor of safety of 1.5.

- Construction employing poles or posts (i.e. lamp posts) may utilize design methods presented in Section 1805.7 of the CBC for Sand, silty sand, clayey sand, silty gravel and clayey gravel (SW, SP, SM, SC, GM, and GC) material class.

In addition to the active or at rest soil pressure, the proposed structures may be designed to include forces from dynamic (seismic) earth pressure. Dynamic earth pressures should be estimated by the structural engineer using methods such as the Mononobe-Okabe method (Mononobe and Matsuo, 1929), Seed and Whitman (1970), or other suitable technique. The soils engineer should be consulted on a case-by-case basis to provide design recommendations for dynamic earth forces (if required). Generally, retaining walls less than 12 feet in height do not require the inclusion of dynamic forces in design.

The recommended values do not include compaction or truck-induced wall pressures. Care must be taken during the compaction operation not to overstress the wall. Heavy construction equipment should be maintained a distance of at least 3 feet away from the
walls while the backfill soils are placed. Hand-operated compaction equipment should be used to compact the backfill soils within a 3-foot-wide zone adjacent to the walls.

The recommended lateral earth pressures assume that drainage is provided behind the walls to prevent the buildup of hydrostatic pressures. Walls should be provided with drains to reduce the potential for the buildup of hydrostatic pressure. Drains may consist of a 2-foot-wide zone of permeable material located immediately behind the wall extending to within 1 foot of the ground surface. Perforated Schedule 40 PVC pipe should be installed at the base of the drain and sloped to discharge to a suitable collection facility. Commercially available drainage panels could be used as an alternative. The product manufacturer's recommendations should be followed in the installation of a drainage panel.

The upper 1-foot of soil should not be considered when calculating passive pressure unless confined by overlying asphalt concrete pavement or Portland cement concrete slab. The soils pressures presented have considered onsite fill soils. Testing or observation should be performed during grading by the soils engineer or his representative to confirm or revise the presented values.

The passive resistance of the subsurface soils will diminish or be non-existent if trench sidewalls slough, cave, or are overwidened during or following excavations. If this condition is encountered, our firm should be notified to review the condition and provide remedial recommendations, if warranted.

5.5 TEMPORARY EXCAVATIONS

5.5.1 General

All excavations must comply with applicable local, state, and federal safety regulations including the current OSHA Excavation and Trench Safety Standards. Construction site safety generally is the sole responsibility of the Contractor, who shall also be solely responsible for the means, methods, and sequencing of construction operations. We are providing the information below solely as a service to our client. Under no circumstances should the information provided be interpreted to mean that Kleinfelder is assuming responsibility for construction site safety or the Contractor's activities; such responsibility is not being implied and should not be inferred. The contractor should
make his own determination of soils types and allowable inclinations based upon the conditions encountered during excavation.

Based on the materials encountered within our exploratory borings, we anticipate that the soil deposits should be excavatable using conventional trenching equipment such as an excavator or backhoe. Loose, cohesionless soils in the form of running sands may be encountered during excavation. This condition could result in instability and caving of excavation sidewall materials. Shoring of trench walls or alternate methods of trench stability should be incorporated into the project planning.

The contractor should carefully review the boring logs in this report, and perform their own assessment of potential construction difficulties. Installation construction methods should be selected accordingly, and the associated costs should be included in the bid submittal. We recommend that the contractor’s actual method of construction be evaluated by the geotechnical and civil engineer prior to construction to verify that the installation method is consistent with the design assumptions.

Many existing underground utilities, including other pipelines and other infrastructure installations, are likely present in the site area. Fill soils associated with these improvements may exist that may require special attention during construction to avoid trench wall collapse, undermining, and damage to existing facilities. Shoring of trench walls or alternate methods of trench stability should be incorporated into the project planning.

We recommend that all individuals utilizing this report review the boring logs presented in Appendix A for greater detail. Subsurface conditions will vary between and beyond the points explored and may differ from the general conditions presented above. If soil conditions are encountered during construction which differ from those described, we should be notified immediately in order that a review may be made. Supplemental recommendations and construction techniques may be required.

5.5.2 Temporary Slopes

Near-surface soils encountered during our field investigation consisted predominantly of loose sands with varying amounts of silt and clay. In our opinion, these soils would be considered a Type “C” soil with regard to the OSHA regulations (1989). For this soil type, OSHA requires a maximum slope inclination of 1.5:1 (horizontal:vertical) or flatter
for excavations 20 feet or less in depth. Steeper cut slopes may be utilized for excavations less than 5 feet deep, depending on the strength, moisture content, and homogeneity of the soils as observed during construction. Loose or running soils may require flatter slope inclinations.

### 5.5.3 Shoring

Shoring may be required where space or other restrictions do not allow a sloped excavation. Braced excavations should be designed to resist a uniform horizontal soil pressure of 30H (in pounds per square foot, psf) where “H” is the excavation depth in feet. The values provided above assume a level ground surface adjacent to the top of the shoring.

Fifty percent of an areal surcharge placed adjacent to the shoring may be assumed to act as a uniform horizontal pressure against the shoring. Special cases such as combinations of slopes and shoring or other surcharge loads (not specified above) may require an increase in the design values recommended above. These conditions should be evaluated by the project geotechnical engineer on a case-by-case basis. The contractor should be responsible for the structural design and safety of all temporary shoring systems.

Cantilevered shoring must extend to a sufficient depth below the excavation bottom to provide the required lateral resistance. We recommend required embedment depths be determined using methods for evaluating sheet pile walls and based on the principles of force and moment equilibrium. For this method, the allowable passive pressure against shoring, which extends below the level of excavation may be assumed to be equivalent to a fluid weighing 300 pcf. Additionally, we recommend a factor of safety of at least 1.2 be applied to the calculated embedment depth and that passive pressure be limited to 2,000 psf.

### 5.6 TRENCH BACKFILL

#### 5.6.1 Overexcavation

Trench excavation operations must expose a firm subgrade that is free of significant voids, loose, soft, or wet soil, oversize material, organics, or other deleterious material.
The subgrade soils exposed at the bottom of each excavation for proposed piping should be observed by a representative from our firm prior to the placement of any pipeline or fill. If unsuitable conditions are encountered, additional removal and replacement of excavation bottom soils and/or other remediation techniques may be required to provide a stable excavation bottom to uniformly support the pipe. Where loose/soft or wet soils are exposed at the pipe invert elevation, we recommend trenches be overexcavated to a depth of at least 8 inches below the bottom of the pipe invert section to allow for adequate bedding material. The trench bottom should be stabilized in accordance with EMWD Standard Specifications. Additionally, prior to the placement of bedding material or stabilization rock, a layer of geogrid such as Tensar BX1100 or direct equivalent should be placed on the bottom of the excavation for the full trench width. Geogrid should be overlapped a minimum of 3 feet. Geogrid should be placed in accordance with the manufacturer's recommendations. A representative from our firm should be present during all excavation and fill placement operations to observe the materials uncovered during excavation, substantiate the proper use of materials, and verify or modify the recommendations presented herein.

5.6.2 Trench Backfill

Pipe zone backfill (i.e., material beneath (bedding) and in the immediate vicinity of the pipe extending to 12 inches above the pipe crown) should consist of sand or similar granular material having a minimum sand equivalent value of 30 and conform to EMWD Standard Specifications for backfill and placement. The native soils do not meet these criteria. Trench zone backfill (i.e., material placed between the pipe zone backfill and finished subgrade) may consist of native or import soil, which meets the requirements for engineered fill provided.

In general, well-graded mixtures of gravel, sand, and silt, with small quantities of cobbles (less than 3 inches maximum dimension), rock fragments, and/or clay are acceptable for use as trench zone backfill. Import materials, if required, should have a very low expansion potential, i.e. have an expansion index of less than 20. All import fill soils should be free from deleterious material and debris.

If import material is used for pipe zone backfill, we recommend it consist of well-graded fine to medium grained sand. In general, poorly graded coarse-grained sand and/or open-graded gravel should not be used for pipe or trench zone backfill due to the potential for soil migration into the relatively large void spaces present in this type of
material and water seepage along trenches backfilled with coarse-grained sand and/or gravel. Recommendations provided above for pipe zone backfill are minimum requirements only. More stringent material specifications may be required to fulfill local building requirements and/or bedding requirements for specific types of pipes. We recommend the project civil engineer develop these material specifications based on planned pipe types, bedding conditions, and other factors beyond the scope of this study. During excavation of the native soils, “oversize” rock material is any material with maximum dimension greater than 3 inches.

5.6.3 Compaction Criteria

All fill soils, native, imported, or blended soil mixes required to bring the site to final grade should be placed as compacted fill. All backfill (compacted fill) should be moisture conditioned to near optimum moisture content and placed in horizontal lifts less than 6 inches in loose thickness and compacted to at least 90 percent of the maximum dry unit weight based on ASTM Test Method D 1557 or as approved by the project geotechnical engineer based upon site conditions. Beneath pavement sections, the upper 12 inches of trench backfill should be compacted to a minimum 95 percent relative compaction (ASTM D 1557). The pipeline or additional fill lifts should not be placed if soil conditions are not stable or if the previous lift did not meet the required minimum dry unit weight. Backfill materials should be brought up at substantially the same rate on both sides of the pipe. Reduction of the lift thickness may be necessary to achieve the above recommended compaction. Mechanical compaction is recommended; ponding or jetting is not recommended.

5.7 SURFACE AND SUBGRADE DRAINAGE

Foundation, slab, and slope performance depends greatly on how well runoff water drains from the site. Positive drainage should be provided to direct the flow of water away from structures and slopes. Adequate provisions should be made to control and limit moisture changes in the subgrade beneath footings and slabs. It is recommended that all pad drainage and roof runoff be collected and drained away from the building pads to proper drain outlets.

Measures should be taken to reduce the potential for future pipe leakage and/or landscape irrigation infiltration into the soils within the areas of the proposed structures.
Surface water should not be allowed to flow against or toward foundation elements. Outlets for gutter systems should discharge water either into storm drains or onto paved surfaces that quickly remove water from the area. Where outlets must discharge into landscape areas, the outlets should have splash blocks or other means of directing the water away from the foundation. Landscape borders should not act as trap for water within landscape areas.

To reduce the potential for water infiltration into structure areas, the use of onsite planters should minimized. Any planters should be placed a minimum 10 feet away from any structure areas and maintenance personnel should be instructed to limit irrigation to the minimum actually necessary to properly sustain landscaping plants. Potential sources of water such as water pipes, drains, and the like should be frequently examined for signs of leakage or damage. Any such leakage or damage should be promptly repaired.

Positive drainage or other appropriate erosion control techniques should be provided by the civil designer to avoid erosion of slopes (if any). Adequate provisions should be made to control and limit the flow of runoff water across the site. Uncontrolled runoff water should not be allowed to flow freely from the site over slope faces.

All runoff water should be controlled, collected, and drained away from the slopes and into proper drain outlets. Control methods may include may curbing, slope facing (such as reinforced Portland Cement concrete or shotcrete, ribbon gutters, 'V' ditches, or other suitable containment and redirection devices. Additionally, appropriate erosion control techniques should be provided by the civil designer to avoid erosion in the vicinity of the site and runoff water discharge areas.

Proper slope protection and maintenance should help reduce erosion and improve the stability of constructed slopes.

- Surface runoff should be directed away from the top of graded slopes and adjacent areas.

- Recommendations for slope planting should be provided by a qualified landscape architect. We strongly recommend that erosion control measures, such as deep-rooted plants, erosion control blankets or fabrics, sprayed tackifiers, or some combination of these, be utilized.
• Slope plantings and irrigation systems should be maintained. It is recommended that all project landscaping be provided with automatic sprinkler shutoffs in order to help prevent over-saturation of slope faces and surficial slope instability. Leaks in any irrigation system should be fixed without delay.

• Slopes should be periodically inspected for evidence of cracking, erosion, or rodent infestation. Any problems should be repaired immediately.

• Slopes should be constructed using typical slope drainage systems in accordance with CBC (2007) including mid-slope drainage swales and brow ditches.

The drainage pattern should be established at the time of final grading and maintained throughout the life of the project. Structural performance is dependent on many drainage-related factors such as landscaping, irrigation, lateral drainage patterns and other improvements.

5.8 PRELIMINARY PAVEMENT SECTIONS

The appropriate pavement design section depends primarily on the shear strength of the subgrade soil exposed after grading and anticipated traffic over the useful life of the pavement. R-value testing should be performed during grading to verify and/or modify the preliminary pavement sections presented within this report. Pavement designs assume that heavy construction traffic will not be allowed on finished pavement sections.

Pavement sections presented in the Table 6 are based on an estimated design R-value of 50, current Caltrans design procedures, and EMWD minimum sections. Pavement sections provided above are based on the soil conditions encountered during our field investigation and our assumptions regarding final site grades. We recommend representative subgrade samples be obtained during grading and R-value tests performed. Should the results of these tests indicate a significant difference, the design pavement sections provided above may need to be revised.
Table 6
Preliminary Flexible Pavement Sections

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<tr>
<th>Traffic Index</th>
<th>Asphalt Concrete Thickness (in.)</th>
<th>Aggregate Base Thickness (in.)</th>
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<td>6 or less</td>
<td>4*</td>
<td>6*</td>
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*Section governed by EMWD minimum section

Rigid pavements, i.e. Portland cement concrete (PCC) pavements, are recommended in areas that will be subject to relatively high static wheel loads and/or heavy vehicle loading/unloading and turning areas (i.e. truck/bus lanes). Based on the Guide for Construction of Concrete Parking Lots, ACI 330R-92, and the assumptions outlined below, PCC sections may have a minimum thickness of 4-1/2-inches and be underlain by compacted native soils; however, based upon EMWD guidelines, the minimum PCC pavement section shall consist of 6 inches of reinforced PCC concrete overlying 6 inches of aggregate base (Caltrans Class 2). As such, we recommend using the minimum EMWD specification (6-inches of PCC over 6 inches of aggregate base). In pavement areas, the upper 12-inches of the subgrade soils should be compacted to a minimum 95 percent of the maximum dry unit weight at near optimum moisture content. A modulus of subgrade reaction of 100 pounds per cubic inch may be used for driveway design.

Rigid Pavement Section Assumptions

- Traffic Category: A-1 (ADTT = 1)
- Modulus of Rupture, Mr: 575 psi
- Compressive Strength, fc: 3500 psi
- Modulus of Subgrade Reaction, k: 100 psi

Should the actual traffic category or traffic indices vary from those assumed and listed above, these sections should be modified.
The recommended preliminary pavement sections are contingent on the following recommendations being implemented during construction:

- Prior to site grading, existing structures and foundations (if any), undocumented fill, vegetation (including the entire removal of tree root-balls, if any), landscaping, topsoil, existing asphalt concrete, Portland cement concrete, oversized materials, wet and/or soft soil, and deleterious matter should be removed and disposed of outside the project limits. Deeper stripping or grubbing may be required where concentrations of organics or other deleterious materials are encountered during site grading.

- Subsequent to stripping and grubbing operations, areas to receive fill should be stripped of loose or soft earth materials until a firm subgrade is exposed, as evaluated by the geotechnical engineer, geologist, or their representative.

- The upper 12 inches of subgrade soil below pavement sections should scarified, moisture conditioned to near optimum moisture content, and compacted to a minimum of 95 percent relative compaction (ASTM D 1557). Fill to bring the site to grade should be placed in accordance with earthwork recommendations given in Section 5.3 of this report. This compacted subgrade thickness is in addition to the asphalt concrete and base course pavement sections.

- Subgrade soils and aggregate base should be in a stable, non-pumping condition at the time of placement and compaction.

- All concrete curbs separating pavement from landscaped areas should extend at least 6 inches into the subgrade soils to reduce the potential for movement of moisture into the aggregate base layer (this reduces the risk of pavement failures due to subsurface water originating from landscaped areas).

- PCC concrete pavements should be constructed with transverse joints at maximum spacing of 15 feet. A thickeened edge should be used where possible and, as a minimum, where concrete pavements abut asphalt pavements. The thickened edge should be 1.2 times the thickness of the pavement (7-1/2 inches for a 6-inch pavement), and should taper back to the pavement thickness over a horizontal distance on the order of 3 feet.

- All longitudinal or transverse control joints should be constructed by hand forming or placing a pre-molded filler such as "zip strips." Expansion joints should be used to isolate fixed objects abutting or within the PCC pavement area. The expansion joint should extend the full depth of the pavement. Joints should run continuously and extend through integral curbs and thickened edges. We recommend that joint layout be adjusted to coincide with the corners of objects and structures. In addition, the following is recommended for concrete pavements:
1. Slope pavement at least 2 percent to provide drainage;

2. Provide rough surface texture for traction;

3. Cure PCC concrete with curing compound or keep continuously moist for a minimum of seven days;

4. Keep all traffic off concrete until compressive strength exceeds 2,000 pounds per square inch; and

5. Give consideration to using slip dowels on 24-inch centers to strengthen control and construction joints.

- At a minimum, asphalt and PCC concrete paving, aggregate base materials, and placement methods should conform to the Caltrans Standard Specifications, (latest edition) and EMWD specifications.

- Aggregate base materials should meet the requirements for Class 2 Aggregate Base in Section 26 of the latest edition of the Caltrans Standard Specifications and should be compacted to at least 95 percent relative compaction (ASTM D 1557).

- All asphalt concrete should be compacted to at least 95 percent relative compaction relative to the maximum wet density.

- Within the structural pavement section areas, positive drainage (both surface and subsurface) should be provided. In no instance should water be allowed to pond on the pavement. Roadway performance depends greatly on how well runoff water drains from the site. This drainage should be maintained both during construction and over the entire life of the project.

- Proper methods, such as hot-sealing, should be employed to limit water infiltration into the pavement base course and/or subgrade at construction joints between existing and reconstructed asphalt concrete sections.

To reduce the potential for detrimental settlement, excess soil material, and/or fill material removed during any footing or utility trench excavation, should not be spread or placed over compacted finished grade soils unless subsequently compacted to at least 95 percent of the maximum dry unit weight, as evaluated by ASTM D 1557 test procedure, at near optimum moisture content, if placed under areas designated for pavement.
6.0 ADDITIONAL SERVICES

6.1 PLANS AND SPECIFICATIONS REVIEW

This report was prepared based on the assumption that Kleinfelder would be retained to provide a program of additional services including final plan review and construction monitoring and testing during the final design and construction phases to evaluate conformance with the recommendations presented in this report. If we are not accorded the privilege of performing this review, we can assume no responsibility for the suitability or misinterpretation of our recommendations. Maintaining Kleinfelder as the geotechnical engineering consultant from beginning to end of this project will help provide continuity of services. The recommended services include, but are not necessarily limited to, the following services which can be performed for additional fee in accordance with our agreed upon fee schedule:

- Consultation as required during the final design stages of the project.
- Review of final foundation design and/or grading plans.

6.2 CONSTRUCTION OBSERVATION AND TESTING

It is also recommended that Kleinfelder, Inc. be retained to provide observation and testing services during site earthwork and construction of foundations and structures (Special Inspection). This will allow us the opportunity to compare actual subsurface soil conditions with those encountered during the field exploration and, if necessary, to provide supplemental recommendations, if warranted due to unanticipated subsurface conditions. The recommended services include, but are not necessarily limited to, the following services which can be performed for additional fee in accordance with our agreed upon fee schedule:

- Observation and testing during site preparation, placement of engineered fill, foundation excavation, and backfill of utility trenches.
- Consultation as required during construction.
- Materials testing, observation, and Special Inspection services (concrete, masonry, steel, rebar, etc.) during construction.
7.0 LIMITATIONS

Recommendations contained in this report are based on our field observations and subsurface explorations, laboratory tests, and our present knowledge of the proposed expansion construction. It is possible that soil conditions could vary between or beyond the points explored. If soil conditions are encountered during construction, which differ from those described herein, we should be notified immediately in order that a review may be made and any supplemental recommendations provided. If the scope of the proposed construction changes from that described in this report, the conclusions and recommendations contained in this report are not considered valid unless the changes are reviewed, and the conclusions of this report are modified or approved in writing, by Kleinfelder. We have not reviewed the final plans for the project. The work performed was based on project information provided by Brown and Caldwell. If Brown and Caldwell or EMWD do not retain Kleinfelder to review any plans and specifications, including any revisions or modifications to the plans and specifications, Kleinfelder assumes no responsibility for the suitability of our recommendations. In addition, if there are any changes in the field to the plans and specifications, Brown and Caldwell or EMWD must obtain written approval from Kleinfelder’s engineer that such changes do not affect our recommendations. Failure to do so will vitiate Kleinfelder’s recommendations.

References to elevations and locations provided within this report were based upon general information provided for our use. Kleinfelder, Inc. did not provide surveying services and, therefore an opinion regarding the accuracy of the surface location or elevations with respect to the approved plans and current site surveying is not provided.

Our service includes providing information obtained through laboratory testing regarding corrosivity of onsite soils. Kleinfelder does not practice corrosion engineering and, therefore, detailed analysis of corrosion test results is not included. We recommend a qualified corrosion engineer be retained to review the test results and design protective systems that may be required.

Our evaluation of subsurface conditions at the site has considered subgrade soil and groundwater conditions present at the time of our investigation. The influence(s) of post-construction changes to these conditions such as introduction of water into the subsurface will likely influence future performance of the proposed project. Whereas
our scope of services addresses present groundwater conditions; future irrigation, broken water pipelines, etc. may adversely influence the project and should be addressed and mitigated, as necessary.

Other standards or documents referenced in any given standard cited in this report, or otherwise relied upon by the authors of this report, are only mentioned in the given standard; they are not incorporated into it or "included by reference", as the latter term is used relative to contracts or other matters of law.

This work was performed in a manner consistent with that level of care and skill ordinarily exercised by other members of Kleinfelder’s profession practicing in the same locality, under similar conditions and at the date the services are provided. Our conclusions, opinions and recommendations are based on a limited number of observations and data. It is possible that conditions could vary between or beyond the data evaluated. Kleinfelder makes no representation, guarantee or warranty, express or implied, regarding the services, communication (oral or written), report, opinion, or instrument of service provided. The recommendations provided in this report are based on the assumption that Kleinfelder will be retained to provide a program of tests and observations during the construction phase in order to evaluate compliance with our recommendations and to evaluate the site conditions exposed. Information and recommendations presented in this report should not be extrapolated to other areas or be used for other projects without our prior review and response. The Client has the responsibility to see that all parties to the project, including the architect, civil designer, structural engineer, governing agency, etc., are made aware of this letter in its entirety and in order to verify that the recommendations are appropriate for the project currently proposed. Additionally, this report should be incorporated by reference into the contract and specification documents.

This report may be used only by the Brown and Caldwell or EMWD and the registered design professional in responsible charge and only for the purposes stated for this specific engagement within a reasonable time from its issuance, but in no event later than two (2) years from the date of the report. Land use, site conditions (both on site and off site) or other factors may change over time, and additional work may be required with the passage of time. Any party other than the client who wishes to use this report shall notify Kleinfelder of such intended use. Based on the intended use of the report, Kleinfelder may require that additional work be performed and that an updated report be issued. Non-compliance with any of these requirements by the client
or anyone else will release Kleinfelder from any liability resulting from the use of this report by any unauthorized party.

This report, and any future addenda or reports regarding this site, may be made available to bidders to supply them with only the data contained in the report regarding subsurface conditions and laboratory test results at the point and time noted. Bidders may not rely on interpretations, opinion, recommendations, or conclusions contained in the report. Because of the limited nature of any subsurface study, the contractor may encounter conditions during construction which differ from those presented in this report. In such event, the contractor should promptly notify the owner so that Kleinfelder’s geotechnical engineer can be contacted to confirm those conditions. We recommend the contractor describe the nature and extent of the differing conditions in writing and that the construction contract include provisions for dealing with differing conditions. Contingency funds should be reserved for potential problems during earthwork and foundation construction. Furthermore, the contractor should be prepared to handle contamination conditions encountered at this site, which may affect the excavation, removal, or disposal of soil; dewatering of excavations; and health and safety of workers.

The scope of our geotechnical services did not include any environmental site assessment for the presence or absence of hazardous/toxic materials. Kleinfelder will assume no responsibility or liability whatsoever for any claim, damage, or injury which results from pre-existing hazardous materials being encountered or present on the project site, or from the discovery of such hazardous materials.
8.0 REFERENCES

American Society of Civil Engineers (ASCE7-05), 2006, Minimum Design Loads for Buildings and Other Structures.


California Department of Water Resources (DWR), 2010, website: http://well.water.ca.gov/

California Division of Mines and Geology, 2004, Preliminary Digital Geologic Map of the Santa Ana 30’x60’ Quadrangle, Southern California, compiled by D.M. Morton.

California Division of Mines and Geology, 2001 Geology Map of the Sunnymead 7.5’ Quadrangle, Riverside County, California, Open-File Report 01-450, compiled by D.M. Morton and J.C. Matti.

California Division of Mines and Geology, 1974, Special Studies Zones, Sunnymead Quadrangle, dated July 1, 1974.


City of Moreno Valley, 2006, General Plan (July 2006).


Eastern Municipal Water District, 2010, Potable Water Booster Pumping Station Submittal and Design Guidelines, Final March 1, 2010


FEMA, 2009, FEMA Digital Q3 Flood Data, ESRI/FEMA Project Impact Hazard Web Site, Environmental Systems Research Institute, Inc.


Jennings, Charles W., 1994, Fault Activity Map of California and Adjacent Areas, California Division of Mines and Geology, Geologic Data Map No. 6.


Western Municipal Water District, 2009, Cooperative Well Measuring Program, Covering the Upper Santa Ana River Watershed, San Jacinto Watershed, and Santa Margarita Watershed, Fall 2009.

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PROPOSED EUCALYPTUS BOOSTER STATION FOR EASTERN MUNICIPAL WATER DISTRICTMORENO VALLEY, CALIFORNIA

SITE VICINITY MAP

FILE NAME: 112644p1.dwg

PROJECT NO. 112644
DRAWN: 9/2010
DRAWN BY: DMF
CHECKED BY: KP

www.kleinfelder.com
Field Exploration
APPENDIX A
FIELD EXPLORATION

The subsurface exploration program for the proposed project consisted of excavating and logging a total of 3 hollow-stem auger borings. The borings were drilled with a mobile B-61 truck-mounted drill rig equipped with 6-inch hollow stem augers provided by Cal Pac Drilling and Well Developing, of Calimesa, California. Approximate boring locations are shown on Plate 2, Boring Location Map.

The logs of the borings are presented as Plates A-2 through A-4. An explanation to the logs is presented on Plate A-1. The logs of borings describe the earth materials encountered, samples obtained, and show field and laboratory tests performed. The logs also show the boring number, drilling date, boring elevation and the name of the logger and drilling subcontractor. A Kleinfelder staff engineer logged the borings using methods outlined in the Unified Soil Classification System (USCS) and general procedures established in ASTM D 2488. The boundaries between soil types shown on the logs are approximate because the transition between different soil layers may be gradual. Bulk and drive samples of representative earth materials were obtained from the borings at maximum intervals of approximately 5 feet.

In-place soil samples were obtained at the test boring locations using a Standard Penetration (SPT) or California-type Sampler driven a total of 18-inches (or until practical refusal) into the undisturbed soil at the bottom of the boring. The soil sampled by the SPT (2-inch O.D., 1.5 inches I.D.) or California-type sampler (3-inch O.D., 2.4 inches I.D.) was returned to our laboratory for testing. The samplers and associated rod (threaded) were driven using a 140-pound automatic hammer falling 30 inches. The total number of hammer blows required to drive the sampler the final 12 inches is termed the blow count and is recorded on the Logs of Borings. The blow count values on the boring logs are presented as field values and have not been corrected for the effects such as overburden pressure, sampler size, hammer efficiency, etc. Bulk samples of the surface soils were retrieved directly from the auger blades. All borings were backfilled using the soil from the excavation and capped as appropriate.
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**NOTES ON FIELD INVESTIGATION**

1. **SAMPLE** = Graphical representation of sample type as shown below.
   - Split Spoon = Standard Penetration Test Sample (SPT)
   - Drive Sample = California Sample (Cal)
   - Bulk Sample = Obtained by collecting cuttings in a plastic bag
   - Tube Sample = Shelby/Pitcher Tube Sample

2. **SAMPLE NO.** = Sample Number
3. **BLOWS/FT** = Number of blows required to advance sampler 1 foot (unless a lesser distance is specified).
4. **GRAPHIC LOG** = Standard symbols for soil and rock types, as shown on plate A-1b.
5. **GEOTECHNICAL DESCRIPTION**
   - Soil = Soil classifications are based on the United Soil Classification System per ASTM D-2967, and designations include consistency, moisture, color and other modifiers. Field descriptions have been modified to reflect results of laboratory analyses where deemed appropriate.
   - Rock = Rock classifications generally include a rock type, color, moisture, mineral constituents, degree of weathering, alteration, and the mechanical properties of the rock. Fabric, lineations, bedding spacing, foliations, and degree of cementation are also presented where appropriate.
   - Description of soil origin or rock formation is placed in brackets at the beginning of the description where applicable, for example, Residual Soil
6. **DRY DENSITY, MOISTURE CONTENT** = As estimated by laboratory or field testing.
7. **ADDITIONAL TESTS** = (Indicates sample tested for properties other than the above):
   - MAX = Maximum Dry Density
   - SG = Specific Gravity
   - GC = Grain Size Distribution
   - SE = Sand Equivalent
   - EI = Expansion Index
   - CHEM = Sulfate and Chloride Content, pH, Resistivity
   - PM = Permeability
   - UU = Undrained Unconfined Triaxial
   - PP = Pocket Penetrometer
   - WA = Wash Analysis
   - AL = Atterberg Limits
   - RV = R-Value
   - CN = Consolidation
   - CU = Consolidation Undrained Triaxial
   - CD = Consolidated Drained Triaxial
   - CP = Collapse Potential
   - UC = Unconfined Compression
   - T = Torvane
8. **ATTITUDES** = Orientation of rock discontinuity observed in bucket auger boring or rock core, expressed in strike/dip and dip angle, respectively, preceded by a one-letter symbol denoting nature of discontinuity as shown below.
   - B = Bedding Plane
   - J = Jointing
   - C = Contact
   - F = Fault
   - S = Shear

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**EXPLANATION OF LOGS**

**PLATE**
A-1a
### Unified Soil Classification System (ASTM D-2487)

<table>
<thead>
<tr>
<th>PRIMARY DIVISIONS</th>
<th>GROUP SYMBOLS</th>
<th>SECONDARY DIVISIONS</th>
</tr>
</thead>
<tbody>
<tr>
<td>COARSE GRAINED SOILS</td>
<td>CLEAN GRAVELS (LESS THAN 5% FINES)</td>
<td>GW</td>
</tr>
<tr>
<td></td>
<td>CLEAN SANDS (LESS THAN 5% FINES)</td>
<td>SW</td>
</tr>
<tr>
<td></td>
<td>SANDS WITH FINES</td>
<td>SM</td>
</tr>
<tr>
<td></td>
<td>GRAVEL WITH FINES</td>
<td>GM</td>
</tr>
<tr>
<td></td>
<td>CLAYEY GRAVELS, GRAVEL-SAND-CLAY MIXTURES</td>
<td>GC</td>
</tr>
<tr>
<td></td>
<td>POORLY GRADED GRAVELS OR GRAVEL-SAND MIXTURES, LITTLE OR NO FINES</td>
<td>GP</td>
</tr>
<tr>
<td>FINE GRAINED SOILS</td>
<td>INORGANIC SILTS, VERY FINE SANDS, ROCK FLOWER, SILTY OR CLAYEY FINE SANDS</td>
<td>ML</td>
</tr>
<tr>
<td></td>
<td>INORGANIC CLAYS OF LOW TO MEDIUM PLASTICITY, GRAVELLY CLAYS, SANDY CLAYS, SILTY CLAYS, LEAN CLAYS</td>
<td>CL</td>
</tr>
<tr>
<td></td>
<td>ORGANIC SILTS AND ORGANIC SILT-CLAYS OF LOW PLASTICITY</td>
<td>OL</td>
</tr>
<tr>
<td></td>
<td>INORGANIC CLAYS OF HIGH PLASTICITY, FAT CLAYS</td>
<td>CH</td>
</tr>
<tr>
<td></td>
<td>ORGANIC CLAYS OF MEDIUM TO HIGH PLASTICITY, ORGANIC SILTS</td>
<td>OH</td>
</tr>
<tr>
<td>HIGHLY ORGANIC SOILS</td>
<td>PT</td>
<td>PEAT, MUCK AND OTHER HIGHLY ORGANIC SOILS</td>
</tr>
</tbody>
</table>

### Consistency Criteria Based on Field Tests

<table>
<thead>
<tr>
<th>RELATIVE DENSITY - COARSE - GRAIN SOIL</th>
<th>CONSISTENCY - FINE - GRAIN SOIL</th>
<th>TORVANE</th>
<th>POCKET ** PENETROMETER</th>
</tr>
</thead>
<tbody>
<tr>
<td>RELATIVE DENSITY</td>
<td>SPT * (# blows/ft)</td>
<td>RELATIVE DENSITY (%)</td>
<td>CONSISTENCY</td>
</tr>
<tr>
<td>Very Loose</td>
<td>&lt;4</td>
<td>0 - 15</td>
<td>Very Soft</td>
</tr>
<tr>
<td>Loose</td>
<td>4 - 10</td>
<td>15 - 35</td>
<td>Soft</td>
</tr>
<tr>
<td>Medium Dense</td>
<td>10 - 30</td>
<td>35 - 65</td>
<td>Medium Stiff</td>
</tr>
<tr>
<td>Dense</td>
<td>30 - 50</td>
<td>65 - 85</td>
<td>Stiff</td>
</tr>
<tr>
<td>Very Dense</td>
<td>&gt;50</td>
<td>85 - 100</td>
<td>Very Stiff</td>
</tr>
<tr>
<td>Hard</td>
<td>&gt;30</td>
<td>&gt;2.0</td>
<td>** UNCONFINED COMPRESSION STRENGTH IN TONS/SQFT, READ FROM POCKET PENETROMETER</td>
</tr>
</tbody>
</table>

### Moisur Content

<table>
<thead>
<tr>
<th>DESCRIPTION</th>
<th>FIELD TEST</th>
</tr>
</thead>
<tbody>
<tr>
<td>Dry</td>
<td>Absence of moisture, dusty, dry to the touch</td>
</tr>
<tr>
<td>Moist</td>
<td>Damp but no visible water</td>
</tr>
<tr>
<td>Wet</td>
<td>Visible free water, usually soil is below water table</td>
</tr>
</tbody>
</table>

### CEMENTATION

<table>
<thead>
<tr>
<th>DESCRIPTION</th>
<th>FIELD TEST</th>
</tr>
</thead>
<tbody>
<tr>
<td>Weekly</td>
<td>Crumbles or breaks with handling or slight finger pressure</td>
</tr>
<tr>
<td>Moderately</td>
<td>Crumbles or breaks with considerable finger pressure</td>
</tr>
<tr>
<td>Strongly</td>
<td>Will not crumble or break with finger pressure</td>
</tr>
</tbody>
</table>

---

**Explanation of Logs**

**PLATE A-1b**

---

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**GEOTECHNICAL DESCRIPTION AND CLASSIFICATION**

<table>
<thead>
<tr>
<th>Sample Number</th>
<th>Blows per Foot</th>
<th>Sample Type</th>
<th>Graphic Log</th>
<th>USCS Description</th>
<th>Water Content (%</th>
<th>Note</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>26</td>
<td>SM</td>
<td></td>
<td>Artificial Fill:</td>
<td>120.0</td>
<td></td>
</tr>
<tr>
<td>2</td>
<td>26</td>
<td>SM</td>
<td></td>
<td>Silty Sand (SM):</td>
<td>122.0</td>
<td></td>
</tr>
<tr>
<td>3</td>
<td>22</td>
<td>ML</td>
<td></td>
<td>Alluvium:</td>
<td>117.0</td>
<td></td>
</tr>
<tr>
<td>4</td>
<td>22</td>
<td>SM</td>
<td></td>
<td>Artificial Fill:</td>
<td>120.0</td>
<td></td>
</tr>
<tr>
<td>5</td>
<td>18</td>
<td>SM</td>
<td></td>
<td>Silty Sand (SM):</td>
<td>122.0</td>
<td></td>
</tr>
<tr>
<td>6</td>
<td>18</td>
<td>SM</td>
<td></td>
<td>Silty Sand (SM):</td>
<td>122.0</td>
<td></td>
</tr>
<tr>
<td>7</td>
<td>32</td>
<td></td>
<td></td>
<td>Alluvium:</td>
<td>117.0</td>
<td></td>
</tr>
<tr>
<td>8</td>
<td>26</td>
<td></td>
<td></td>
<td>Bedrock:</td>
<td>1764 feet</td>
<td></td>
</tr>
<tr>
<td>9</td>
<td>26</td>
<td>SP-SM</td>
<td></td>
<td>Granitic:</td>
<td>1764 feet</td>
<td></td>
</tr>
</tbody>
</table>

**Proposed Eucalyptus Booster Station**
Moreno Valley, California

**Legend to Logs on Plate A-1**

Note: The boundaries between soil and/or rock types shown on the logs are approximate as the transition between different soil layers may be gradual.
**GEOTECHNICAL DESCRIPTION AND CLASSIFICATION**

<table>
<thead>
<tr>
<th>Elevation (approx)</th>
<th>Sample Number</th>
<th>Blows per Foot</th>
<th>Sample Type</th>
<th>Graphic Log</th>
<th>USCS Description</th>
<th>Dry Unit Weight (pcf)</th>
<th>Moisture Content (%)</th>
<th>Additional Tests &amp; Remarks</th>
</tr>
</thead>
<tbody>
<tr>
<td>1840</td>
<td>1</td>
<td></td>
<td>SM</td>
<td>Drawn</td>
<td>Artificial Fill:</td>
<td>124.0</td>
<td>5.2</td>
<td>CP</td>
</tr>
<tr>
<td>1850</td>
<td>2</td>
<td>31</td>
<td>SM</td>
<td>Drawn</td>
<td>Silty Sand (SM):</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Olive yellow to yellow brown, moist, medium dense, fine to medium grained sand, some coarse grained sand, trace fine gravel</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>--brown, increase in fine gravel content</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1860</td>
<td>3</td>
<td>23</td>
<td>SM</td>
<td>Drawn</td>
<td>Alluvium:</td>
<td>117.0</td>
<td>11.3</td>
<td>DS</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Silty Sand (SM):</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Olive yellow, moist, medium dense, fine to medium grained sand, trace fine gravel up to 1/2 inch, no calcium carbonate, fine grained sand at approximately 6 feet</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>--brown, decrease in sand grain size, increase in silt content, no gravel</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>--increase in silt content, trace fine gravel to 1/2 inch, with a thin layer of sandy silt at approximately 16-1/2 feet</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>--increase in silt content</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1870</td>
<td>4</td>
<td>20</td>
<td>SM-ML</td>
<td>Drawn</td>
<td>Silty Sand to Sandy Silt (SM/ML):</td>
<td>113.0</td>
<td>10.7</td>
<td>CP</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Brown, moist, hard to medium dense, fine grained sand, trace medium grained sand</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>--color change to olive yellow in soil cuttings</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1880</td>
<td>5</td>
<td>47</td>
<td>SP-SM</td>
<td>Drawn</td>
<td>Poorly Graded Sand with Silt (SP-SM):</td>
<td>113.0</td>
<td>10.7</td>
<td>CP</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Olive yellow, moist, dense, fine to medium grained sand, some clay pockets at approximately 30 feet, weakly to moderately cemented</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

**Note:** The boundaries between soil and/or rock types shown on the logs are approximate as the transition between different soil layers may be gradual.
### Bedrock:

**Granitic:** Excavates as Poorly Graded Sand with Silt (SP-SM): Olive yellow to light olive, dry, fine grained sand, moderately to highly weathered, weak

--mottled black, white, and orange brown, slightly weathered, weak to moderately strong, fine to medium grained

--fine to medium grained, trace coarse grained, moderately weathered

--no sample recovery

Boring terminated at approximately 50-1/2 feet

No refusal

No groundwater observed

Hole backfilled with cuttings
**Proposed Eucalyptus Booster Station**  
**Moreno Valley, California**  

**GEOTECHNICAL DESCRIPTION AND CLASSIFICATION**

<table>
<thead>
<tr>
<th>Depth (feet)</th>
<th>Blows per Foot</th>
<th>Sample Type</th>
<th>Graphic Log</th>
<th>USCS Description</th>
<th>Dry Unit Weight (pcf)</th>
<th>Moisture Content (%)</th>
<th>Additional Tests &amp; Remarks</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>81</td>
<td>SM</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>2</td>
<td>62</td>
<td>CL</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>3</td>
<td>47</td>
<td>SM</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>4</td>
<td>11</td>
<td>CL</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>5</td>
<td>27</td>
<td>SM</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>6</td>
<td>40</td>
<td>CL</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>7</td>
<td>32</td>
<td>SM</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>8</td>
<td>21</td>
<td>SM</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

**Artificial Fill:**  
- **Silty Sand (SM):** Olive yellow, moist, very dense, fine grained sand, some medium to coarse grained sand, trace fine gravel

**Alluvium:**  
- **Sandy Lean Clay (CL):** Olive, moist, firm to hard, fine grained sand, medium plasticity

- **Silty Sand (SM):** Olive brown to yellow brown, moist, dense at approximately 8 feet, fine grained sand, trace medium to coarse grained sand, medium dense at approximately 10 feet

- **Sandy Lean Clay (CL):** Olive yellow, moist, hard, fine grained sand, with silt

**Clayey Sand to Silty Sand (SC/SM):** Olive brown, moist, dense, fine grained sand, trace medium to coarse grained sand

- **Silty Sand (SM):** Olive, moist, medium dense, fine to medium grained sand, some coarse grained sand, trace fine gravel in cuttings to 1/2 inch, trace clay

---

- Olive brown, increase in medium grained sand, decrease in clay content

- Boring terminated at approximately 31-1/2 feet
- No refusal, No groundwater observed
- Hole backfilled with cuttings

---

**Note:** The boundaries between soil and/or rock types shown on the logs are approximate as the transition between different soil layers may be gradual.
Laboratory Testing
APPENDIX B
LABORATORY TESTING

Laboratory tests were performed on drive and bulk soil samples to estimate engineering characteristics of the various earth materials encountered. Testing was performed in general accordance with procedures outlined in the American Society for Testing and Materials, or other accepted procedures.

LABORATORY MOISTURE AND DENSITY DETERMINATIONS

Natural moisture content and dry density tests were performed on selected soil samples collected. Moisture content was evaluated in general accordance with ASTM Test Method D 2216; dry unit weight was evaluated using procedures similar to ASTM Test Method D 2937. The results are presented on the Logs of Borings and are summarized in Table B-1, Moisture Content and Unit Weight.

<table>
<thead>
<tr>
<th>Table B-1</th>
<th>Moisture Content and Unit Weight</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Boring</strong></td>
<td><strong>Depth (ft)</strong></td>
</tr>
<tr>
<td>B-1</td>
<td>2</td>
</tr>
<tr>
<td>B-1</td>
<td>5</td>
</tr>
<tr>
<td>B-1</td>
<td>7</td>
</tr>
<tr>
<td>B-2</td>
<td>2</td>
</tr>
<tr>
<td>B-2</td>
<td>5</td>
</tr>
<tr>
<td>B-2</td>
<td>7</td>
</tr>
<tr>
<td>B-3</td>
<td>2</td>
</tr>
<tr>
<td>B-3</td>
<td>5</td>
</tr>
<tr>
<td>B-3</td>
<td>7</td>
</tr>
</tbody>
</table>

SIEVE ANALYSIS

Sieve analyses were performed on two samples of the materials encountered at the site to evaluate the grain size distribution characteristics of the soils and to aid in their classification. The tests were performed in general accordance with ASTM Test Method D 422. The test results are presented as Plate B-1, Grain Size Distribution.
DIRECT SHEAR

Direct shear testing was performed on two soil samples to determine the soil shear strength and cohesion values in accordance with ASTM Standard Test Method D 3080. Samples were soaked to near saturation. The test results are presented on Plates B-2 and B-3.

COLLAPSE POTENTIAL TEST

Collapse potential testing was performed on six relatively undisturbed samples in accordance with ASTM Standard Test Method D 2435 and D 5333. The test results are presented on Plates B-4 through B-9, Collapse Potential Test.

MAXIMUM DENSITY/OPTIMUM MOISTURE TEST

One maximum density/optimum moisture test was performed on a select bulk sample of the on-site soils to determine compaction characteristics. The test was performed in accordance with ASTM Standard Test Method D 1557. The test result is presented in Table B-2, Maximum Density / Optimum Moisture Test Results.

<table>
<thead>
<tr>
<th>Boring</th>
<th>Depth (ft)</th>
<th>Maximum Density (pcf)</th>
<th>Optimum Moisture (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>B-1</td>
<td>0-5</td>
<td>135</td>
<td>7.5</td>
</tr>
</tbody>
</table>
EXPANSION INDEX

Expansion index testing was performed on one sample of the near-surface soils to evaluate their expansion characteristics. The tests were performed in accordance with ASTM Standard Test Method D 4829. The test result is presented in Table B-3, Expansion Index Test Results and may be compared to the table presented below to qualitatively evaluate the expansion potential of the near-surface site soils.

<table>
<thead>
<tr>
<th>Expansion Index, EI</th>
<th>Potential Expansion</th>
</tr>
</thead>
<tbody>
<tr>
<td>0 – 20</td>
<td>Very Low</td>
</tr>
<tr>
<td>21 – 50</td>
<td>Low</td>
</tr>
<tr>
<td>51 – 90</td>
<td>Medium</td>
</tr>
<tr>
<td>91 – 130</td>
<td>High</td>
</tr>
<tr>
<td>&gt; 130</td>
<td>Very High</td>
</tr>
</tbody>
</table>

Table B-3
Expansion Index Test Results

<table>
<thead>
<tr>
<th>Boring</th>
<th>Depth (ft)</th>
<th>Expansion Index (EI)</th>
<th>Expansion Potential</th>
</tr>
</thead>
<tbody>
<tr>
<td>B-3</td>
<td>0-5</td>
<td>0</td>
<td>Very Low</td>
</tr>
</tbody>
</table>

CORROSIVITY TESTS

A series of chemical tests were performed on two selected samples of the near-surface soils to estimate pH, resistivity, sulfate and chloride contents. Test results may be used by a qualified corrosion engineer to evaluate the general corrosion potential with respect to construction materials. The results are presented in Table B-4, Corrosion Test Results.

<table>
<thead>
<tr>
<th>Boring</th>
<th>Depth (ft)</th>
<th>pH</th>
<th>Sulfate (ppm)</th>
<th>Chloride (ppm)</th>
<th>Resistivity (ohm-cm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>B-2</td>
<td>0-5</td>
<td>8.2</td>
<td>39</td>
<td>216</td>
<td>520</td>
</tr>
<tr>
<td>B-2</td>
<td>10</td>
<td>6.9</td>
<td>156</td>
<td>388</td>
<td>890</td>
</tr>
</tbody>
</table>
Proposed Eucalyptus Booster Station
For Eastern Municipal Water District
Moreno Valley, California
GRAIN SIZE DISTRIBUTION
Sample remolded to 90 percent of laboratory maximum dry density based on ASTM D1557
Performed in general accordance with ASTM D 3080

<table>
<thead>
<tr>
<th>SYMBOL</th>
<th>BORING NO.</th>
<th>SAMPLE NO.</th>
<th>DEPTH (ft)</th>
<th>COHESION (psf)</th>
<th>FRICTION ANGLE (deg)</th>
<th>SOIL CLASSIFICATION</th>
<th>USCS TOTAL SAMPLE</th>
</tr>
</thead>
<tbody>
<tr>
<td>PEAK</td>
<td>B-1</td>
<td>1</td>
<td>0-5</td>
<td>0</td>
<td>38</td>
<td>Silty Sand</td>
<td>SM</td>
</tr>
<tr>
<td>ULTIMATE</td>
<td>B-1</td>
<td>1</td>
<td>0-5</td>
<td>0</td>
<td>38</td>
<td>Silty Sand</td>
<td>SM</td>
</tr>
</tbody>
</table>

INITIAL MOISTURE(%): 7.5  
Initial Dry Density (PCF): 121  
Final Moisture(%): 13.6

<table>
<thead>
<tr>
<th>Normal Stress (psf)</th>
<th>Peak Stress (psf)</th>
<th>Ultimate Stress (psf)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1216</td>
<td>980</td>
<td>884</td>
</tr>
<tr>
<td>2528</td>
<td>1862</td>
<td>1799</td>
</tr>
<tr>
<td>4658</td>
<td>3692</td>
<td>3566</td>
</tr>
</tbody>
</table>

Proposed Eucalyptus Booster Station
Eastern Municipal Water District
Moreno Valley, California
REMOLED DIRECT SHEAR TEST
INITIAL MOISTURE(%): 11.3
INITIAL DRY DENSIY (PCF): 117
FINAL MOISTURE(%): 18.4

Performed in general accordance with ASTM D 3080
SAMPLE IDENTIFICATION

<table>
<thead>
<tr>
<th>BORING NO.</th>
<th>SAMPLE NO.</th>
<th>DEPTH (ft)</th>
<th>COLLAPSE POTENTIAL (%)</th>
<th>SOIL CLASSIFICATION</th>
<th>USCS TOTAL SAMPLE</th>
</tr>
</thead>
<tbody>
<tr>
<td>B-1</td>
<td>4</td>
<td>7</td>
<td>0.7</td>
<td>Sandy Silt</td>
<td>ML</td>
</tr>
</tbody>
</table>

INITIAL MOISTURE (%): 8.8
INITIAL DRY DENSITY (PCF): 117
FINAL MOISTURE(%): 19.8

Testing performed in general accordance with ASTM D5333-03
<table>
<thead>
<tr>
<th>SAMPLE IDENTIFICATION</th>
<th>COLLAPSE POTENTIAL (%)</th>
<th>SOIL CLASSIFICATION</th>
<th>USCS TOTAL SAMPLE</th>
</tr>
</thead>
<tbody>
<tr>
<td>BORING NO.</td>
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<tr>
<td>B-2</td>
<td>2</td>
<td>2</td>
<td>1.8</td>
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</tbody>
</table>

INITIAL MOISTURE (%): 5.2
INITIAL DRY DENSITY (PCF): 124
FINAL MOISTURE(%): 14.5

Testing performed in general accordance with ASTM D5333-03

PROJECT NO. 112644

Proposed Eucalyptus Booster Station
For Eastern Municipal Water District
Moreno Valley, California
COLLAPSE POTENTIAL TEST

PLATE B-5
**SAMPLE IDENTIFICATION**

<table>
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<tr>
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<th>COLLAPSE POTENTIAL (%)</th>
<th>SOIL CLASSIFICATION</th>
<th>USCS TOTAL SAMPLE</th>
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</thead>
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<td>4</td>
<td>7</td>
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</table>

**INITIAL MOISTURE (%)**: 10.7  
**INITIAL DRY DENSITY (PCF)**: 113  
**FINAL MOISTURE (%)**: 20.1

Testing performed in general accordance with ASTM D5333-03

Proposed Eucalyptus Booster Station  
For Eastern Municipal Water District  
Moreno Valley, California  
COLLAPSE POTENTIAL TEST
<table>
<thead>
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<th>SAMPLE NO</th>
<th>DEPTH (ft)</th>
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<th>SOIL CLASSIFICATION</th>
<th>USCS TOTAL SAMPLE</th>
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</thead>
<tbody>
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<td>B-3</td>
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<td>SM</td>
<td></td>
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</table>

INITIAL MOISTURE (%): 3.5
INITIAL DRY DENSITY (PCF): 135
FINAL MOISTURE(%): 10.7

Testing performed in general accordance with ASTM D5333-03

Proposed Eucalyptus Booster Station
For Eastern Municipal Water District
Moreno Valley, California
COLLAPSE POTENTIAL TEST

PLATE

B-7
Testing performed in general accordance with ASTM D5333-03

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<th>SOIL CLASSIFICATION</th>
<th>USCS TOTAL SAMPLE</th>
</tr>
</thead>
<tbody>
<tr>
<td>BORING NO.</td>
<td>SAMPLE NO.</td>
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</tr>
<tr>
<td>B-3</td>
<td>3</td>
<td>5</td>
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INITIAL MOISTURE (%): 5.6
INITIAL DRY DENSITY (PCF): 137
FINAL MOISTURE(%): 9.5
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<th>SOIL CLASSIFICATION</th>
<th>USCS TOTAL SAMPLE</th>
</tr>
</thead>
<tbody>
<tr>
<td>BORING NO.</td>
<td>SAMPLE NO.</td>
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<tr>
<td>B-3</td>
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<td>0.9</td>
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</table>

INITIAL MOISTURE (%): 7.5
INITIAL DRY DENSITY (PCF): 136
FINAL MOISTURE (%): 10.1

Testing performed in general accordance with ASTM D5333-03

Proposed Eucalyptus Booster Station
For Eastern Municipal Water District
Moreno Valley, California
COLLAPSE POTENTIAL TEST
PLATE B-9

PROJECT NO. 112644
December 15, 2017
Kleinfelder Project No.: 20182806.001A

Mr. Victor Tsai, PE
Brown and Caldwell
9665 Chesapeake Drive, Suite 201
San Diego, California 92123

SUBJECT: Limited Geotechnical Evaluation
Proposed Eucalyptus Booster Station
Eucalyptus Avenue West of Moreno Beach Drive
Moreno Valley, California

REFERENCES:


Survey Exhibit for EMWD’s Eucalyptus Booster Pump Station, prepared by Cozad & Fox, Inc., dated November 6, 2017.

Dear Mr. Tsai:

Kleinfelder has performed a limited geotechnical evaluation of the proposed reconfiguration of the proposed Eucalyptus Booster Station project. We have reviewed the above referenced report and project plans in order to evaluate the potential design impacts, if any, from the reconfiguration of the project. The location of the project site is presented on Figure 1, Site Vicinity Map. Figure 2, Boring Location Map, shows the previous project parcels and the new parcel obtained for the project. This letter provides updated recommendations for design and construction of this project due to the current building code requirements and the project reconfiguration.

BACKGROUND

We previously performed a geotechnical site investigation for the proposed booster station which consisted of three exploratory borings drilled between 31 and 50 ½ feet below ground surface (Kleinfelder, 2011). The locations of the previous borings are presented in Figure 2, Boring Location Map. However, we understand that the project was put on hold by the owner, Eastern Municipal Water District (EMWD). Additionally, a recent land exchange between EMWD and the adjacent residential developer will result in the reconfiguration of the proposed booster station. Figure 2 shows EMWD’s newly acquired parcel in relation to the original parcel for the proposed booster station. To confirm that soil conditions in the newly acquired parcel are consistent with our previous investigation, Kleinfelder has performed this limited geotechnical evaluation including a field exploration (Appendix A), laboratory testing (Appendix B), and the preparation of this letter report.
PROJECT DESCRIPTION

The project site is located on the south side of Eucalyptus Avenue approximately 1,300 feet west of Moreno Beach Drive in Moreno Valley, California (Figure 1). We understand the project will consist of a booster pump station comprised of one or more of the following: a reinforced concrete block building, reinforced masonry walls around portions of the site, pumps, emergency generator, and various appurtenance utility construction necessary for the station.

We have reviewed the grading and utility provided by Brown and Caldwell (2014) which were based on the original EMWD parcel, which does not include the newly acquired parcel as shown in Figure 2. We understand that the proposed booster station may not be constructed for some time and new plans showing the reconfiguration of the facility will be completed at a later date. Kleinfelder should be contacted once updated plans have been completed to see if the recommendations provided in this letter report and our referenced report remain valid. Although unlikely, depending upon the final project configurations, additional fieldwork or engineering analysis may be required.

SCOPE OF WORK

The scope of our services consisted of a field reconnaissance, limited subsurface exploration and sampling, geotechnical laboratory testing, engineering evaluation and analysis, and preparation of this letter report. More specifically, Kleinfelder performed the following.

- A field reconnaissance to observe and document the condition of the site and to mark field exploration locations;
- A field exploration consisting of one hollow-stem-auger Boring (B-4) advanced to a depth of approximately 41½ feet below the existing grade. The approximate location of the boring is presented on Figure 2. A Kleinfelder staff engineer supervised the field operations and logged the boring. Selected soil samples were retrieved, sealed and transported to Kleinfelder’s laboratory for further evaluation. Descriptions of the soils encountered are presented below. Appendix A presents a description of the field exploration program, exploration log and a legend of terms and symbols used on the log;
- In-situ moisture and dry density, grain size distribution, and corrosivity testing was performed on selected bulk and relatively undisturbed soil samples to evaluate the geotechnical engineering properties. The test results are presented in Appendix B; and
- Preparation of this letter report with maps and graphics summarizing the data collected and presenting our updated findings, conclusions, and recommendations.

SITE CONDITIONS

Based on our recent exploration, as well as a review of our referenced report, the subsurface conditions at the site generally consists of artificial fill underlain by alluvial deposits. The alluvial deposits are underlain by granitic bedrock at depth. The artificial fill encountered in our borings is anticipated to be a result of grading operations for the adjacent residential development. Prior to develop of the Booster Station, documentation of the fill placement should be provided and reviewed by Kleinfelder. If documentation is not available or is not satisfactory, then the artificial fill soils should be considered undocumented and not suitable for support of shallow foundations. The artificial fill generally consist of silty sands with trace amounts of fine gravel that were encountered at approximately 5 to 7 feet below ground surface (bgs) in our borings. Deeper fill
soils may be encountered in areas between our borings. The fill soils are underlain by alluvial soils which generally consist of dense silty and poorly graded sands and stiff sandy silts and very stiff sandy lean clays. The alluvial soils generally become denser with depth. Granitic bedrock was encountered beneath the alluvial soils from 23 to 31 feet bgs.

Groundwater was not encountered during our previous or recent field explorations. However, it should be noted that the groundwater level can fluctuate depending on factors such as seasonal rainfall, groundwater withdrawal, construction activities on this or adjacent properties.

CONCLUSIONS AND RECOMMENDATIONS

General

Based on the results of our limited geotechnical evaluation consisting of field exploration, laboratory testing, and engineering analysis, it is our opinion that the site is suitable, from a geotechnical standpoint, for construction of the proposed booster station provided the recommendations in this report and our previous 2011 report are incorporated into the project design and construction. Based on our supplemental investigation and our current understanding of the project, the recommendations in our previous 2011 report remain applicable to the current project, except as updated herein. Kleinfelder should be given the opportunity to review updated plans, once available, to evaluate if additional fieldwork or modifications to our recommendations are needed. Our referenced 2011 report should be used in conjunction with this supplemental letter report.

2016 CBC Seismic Design Parameters

Based on information obtained from the investigation, published geologic literature and maps, and on our interpretation of the 2016 CBC criteria, it is our opinion that the project site may be classified as Site Class C, Very Dense Soil and Soft Rock, according to Section 1613.3.2 of 2016 CBC and Table 20.3-1 of ASCE/SEI 7-10 (2010). Approximate coordinates for the site are noted below.

| Latitude: 33.9369°N |
| Longitude: -117.1831°W |

The Risk-Targeted Maximum Considered Earthquake (MCE) mapped spectral accelerations for 0.2 seconds and 1 second periods (S₂ and S₁) were estimated using Section 1613.3 of the 2016 CBC and the U.S. Geological Survey (USGS) web based application (available at http://geohazards.usgs.gov/designmaps/us/application.php). The mapped acceleration values and associated soil amplification factors (Fₚ and Fᵥ) based on the 2016 CBC and corresponding site modified spectral accelerations (Sₘₛ and Sₘ₁) and design spectral accelerations (Sₖₛ and Sₖ₁) are presented in the following table.
### 2016 CBC Seismic Design Parameters

<table>
<thead>
<tr>
<th>DESIGN PARAMETER</th>
<th>RECOMMENDED VALUE</th>
</tr>
</thead>
<tbody>
<tr>
<td>Site Class</td>
<td>C</td>
</tr>
<tr>
<td>$S_s$ (g)</td>
<td>2.103</td>
</tr>
<tr>
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<tr>
<td>$F_a$</td>
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<tr>
<td>$F_v$</td>
<td>1.3</td>
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<tr>
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<td>$S_{DS}$ (g)</td>
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<td>$PGA_m$ (g)</td>
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**CLOSING**

Unless specifically superseded in this letter, the recommendations in the above-referenced October 28, 2011, geotechnical report remain applicable. This document is intended to provide site specific recommendations. It cannot be considered an independent document, as it does not contain adequate background information. This document is directed only to the personnel with detailed knowledge of the subject project. The conclusions and recommendations presented in this supplemental letter were prepared under the conditions and limitations presented in our October 28, 2011, geotechnical report.

This work was performed in a manner consistent with that level of care and skill ordinarily exercised by other members of Kleinfelder’s profession practicing in the same locality, under similar conditions, and at the date the services are provided. Our conclusions, opinions, and recommendations are based on a limited number of observations and data. It is possible that conditions could vary between or beyond the data evaluated. Kleinfelder makes no other representation, guarantee, or warranty, express or implied, regarding the services, communication (oral or written), report, opinion, or instrument of service provided.
We appreciate the opportunity to provide geotechnical engineering services to you on this project. If you have any questions regarding this report or if we can be of further service, please do not hesitate to contact Eric Noel, Kleinfelder’s Principal Geotechnical Engineer, at 951.453.4976.

Sincerely,

KLEINFELDER, INC.

Russell P. Ferryman, EIT
Staff Engineer

Eric W. Noel, PE, GE
Principal Geotechnical Engineer

Attachments: Figure 1 – Site Vicinity Map
Figure 2 – Boring Location Map
Appendix A – Field Exploration
Appendix B – Laboratory Testing
The information included on this graphic representation has been compiled from a variety of sources and is subject to change without notice. Kleinfelder makes no representations or warranties, express or implied, as to accuracy, completeness, timeliness, or rights to the use of such information. This document is not intended for use as a land survey product nor is it designed or intended as a construction design document. The use or misuse of the information contained on this graphic representation is at the sole risk of the party using or misusing the information.
REFERENCE: BASE MAP PROVIDED BY COZAD & FOX, INC., DATED 11/06/2017

APPROXIMATE SOIL BORING LOCATION (KLEINFELDER, 2011)
B-3

APPROXIMATE SOIL BORING LOCATION (KLEINFELDER, 2017)
B-4

EXISTING PARCEL
OWNED BY EMWD.

PARCEL EMWD HAD
TO GIVE UP AS PART
OF THE LAND
EXCHANGE.

PARCEL EMWD
RECENTLY ACQUIRED VIA
LAND EXCHANGE.

DIAMOND BAR, CA
PROPOSED EUCALYPTUS BOOSTER STATION
MORENO VALLEY, CALIFORNIA

EXPLANATION

FILE NAME:
20183805_F2.dwg

PROJECT NO.
20183805

DRAWN BY:
DMF

CHECKED BY:
RF

PROPOSED EUCALYPTUS AVENUE WEST OF MORENO BEACH DRIVE
MORENO VALLEY, CALIFORNIA

www.kleinfelder.com

REFERENCE: BASE MAP PROVIDED BY COZAD & FOX, INC., DATED 11/06/2017
APPENDIX A
FIELD EXPLORATION

Our field exploration included a site reconnaissance and subsurface exploration. During the site reconnaissance phase, the proposed boring site was visited to evaluate exploration equipment, accessibility, and general site conditions.

The subsurface exploration program for the project consisted of one hollow stem auger (HSA) boring drilled to approximately 41 ½ feet below ground surface (bgs). The boring was performed using a B-61 Mobile truck-mounted drill rig equipped with 8-inch-diameter augers by California Pacific Drilling of Calimesa, California. The drill rig was equipped with an automatic hammer system to drive the samplers. Prior to drilling in accordance with State law, the Underground Service Alert (USA) of northern California was called to notify utilities of our intent to dig. The boring was hand augered to an approximate depth of 5 feet prior to drilling.

In-place soil samples were obtained from the hollow-stem auger borings using either a Standard Penetration (SPT) or modified California-type sampler (MCS) driven a total of 18-inches (or until practical refusal) into the undisturbed soil at the bottom of the boring. The soil sampled by the SPT (2-inch O.D., 1.5 inches I.D.) or modified California-type sampler (3-inch O.D., 2.4 inches I.D.) were delivered to our lab in Ontario, California for laboratory testing. The samplers were driven using a 140-pound automatic hammer free-falling 30 inches. The total number of hammer blows required to drive the sampler each 6-inch interval was recorded on the Log of Borings. Where the sample was driven less than 12 inches, the number of blows to drive the sample for each 6-inch segment, or portion thereof, is shown on the logs. For example, 50/4" indicates 50 blows to drive the sampler 4 inches to refusal. Bulk samples of the soils were retrieved directly from the auger cuttings. The boring was backfilled with soil cuttings and periodically compacted using the drill rig’s downhole hammer.

MCS samples were obtained within the upper 10 feet bgs. SPT and MCS samples were obtained at approximate 5-foot intervals starting at a depth of approximately 15 feet until the target depth was achieved.

A legend to the logs is presented as Figures A-1, and A-2. The logs of borings are presented as Figures A-3 and A-4. The log describes the earth materials encountered, indicates the locations of the samples obtained, and shows field and laboratory tests performed. The excavations were logged by an engineer from Kleinfelder using methods outlined in the Unified Soil Classification
System (USCS) and general procedures established in ASTM D2488. The boundaries between soil types shown on the logs are approximate because the transition between different soil layers may be gradual. Bulk and in-place samples of representative earth materials were obtained from the borings.

The boring was located using hand equipment such as a wheel tape, measuring tape, and pacing. Figure 2 shows the approximate boring location.
The report and graphics key are an integral part of these logs. All data and interpretations in this log are subject to the explanations and limitations stated in the report. Lines separating strata on the logs represent approximate boundaries only. Actual transitions may be gradual or differ from those shown. No warranty is provided as to the continuity of soil or rock conditions between individual sample locations. Logs represent general soil or rock conditions observed at the point of exploration on the date indicated. In general, Unified Soil Classification System designations presented on the logs were based on visual classification in the field and were modified where appropriate based on gradation and index property testing. Fine grained soils that plot within the hatched area on the Plasticity Chart, and coarse grained soils with between 5% and 12% passing the No. 200 sieve require dual USCS symbols, ... GP-GC, GC-GM, SW-SM, SP-SC, SC-SM. If sampler is not able to be driven at least 6 inches then 50/X indicates number of blows required to drive the identified sampler X inches with a 140 pound hammer falling 30 inches.

ABBREVIATIONS
WOH - Weight of Hammer
WOR - Weight of Rod

GROUND WATER GRAPHICS
- WATER LEVEL (level where first observed)
- WATER LEVEL (level after exploration completion)
- WATER LEVEL (additional levels after exploration)
- OBSERVED SEEPAGE

NOTES
- The report and graphics key are an integral part of these logs. All data and interpretations in this log are subject to the explanations and limitations stated in the report.
- Lines separating strata on the logs represent approximate boundaries only. Actual transitions may be gradual or differ from those shown.
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- If sampler is not able to be driven at least 6 inches then 50/X indicates number of blows required to drive the identified sampler X inches with a 140 pound hammer falling 30 inches.

FINE GRAINED SOILS (More than half of material is smaller than the #200 sieve)
- ML - INORGANIC SILTS AND VERY FINE SILTS
- CL - INORGANIC CLAY-LIKE SILTS OF LOW PLASTICITY
- OL - ORGANIC SILTS & ORGANIC SILTY CLAYS
- OH - ORGANIC CLAYS & ORGANIC SILTS OF MEDIUM-TO-HIGH PLASTICITY

COARSE GRAINED SOILS (More than half of coarse fraction is smaller than the #200 sieve)
- GW - WELL-GRADED GRAVELS, GRAVEL-SAND MIXTURES WITH LITTLE OR NO FINES
- GW-GM - WELL-GRADED GRAVELS, GRAVEL-SAND MIXTURES WITH LITTLE CLAY FINES
- GW-GC - WELL-GRADED GRAVELS, GRAVEL-SAND MIXTURES WITH LITTLE CLAY FINES
- GP - POORLY GRADED GRAVELS, GRAVEL-SAND MIXTURES WITH LITTLE OR NO FINES
- GP-GM - POORLY GRADED GRAVELS, GRAVEL-SAND MIXTURES WITH LITTLE CLAY FINES
- GP-GC - POORLY GRADED GRAVELS, GRAVEL-SAND MIXTURES WITH LITTLE CLAY FINES

SANDS (More than half of coarse fraction is smaller than the #4 sieve)
- SW - WELL-GRADED SANDS, SAND-GRAVEL MIXTURES WITH LITTLE OR NO FINES
- SW-SC - POORLY GRADED SANDS, SAND-GRAVEL MIXTURES WITH LITTLE CLAY FINES
- SW-SM - WELL-GRADED SANDS, SAND-GRAVEL MIXTURES WITH LITTLE CLAY FINES
- SP - POORLY GRADED SANDS, SAND-GRAVEL MIXTURES WITH LITTLE OR NO FINES
- SP-SC - POORLY GRADED SANDS, SAND-GRAVEL MIXTURES WITH LITTLE CLAY FINES
- SM - SILTY SANDS, SAND-GRAVEL-SILT MIXTURES
- SC - CLAYEY SANDS, SAND-GRAVEL-CLAY MIXTURES
- SC-SM - CLAYEY SANDS, SAND-SILT-CLAY MIXTURES

SANDS WITH >12% FINES (More than half of coarse fraction is larger than the #200 sieve)
- Cu 4 and/or 1 Cc 3 - Cu 4 and/or 1 Cc 3

COARSE GRAINED SOILS (More than half of coarse fraction is smaller than the #4 sieve)
- Cu 4 and/or 1 Cc 3 - Cu 4 and/or 1 Cc 3

SANDS WITH >12% FINES (More than half of coarse fraction is larger than the #200 sieve)
- Cu 4 and/or 1 Cc 3 - Cu 4 and/or 1 Cc 3

SANDS WITH >12% FINES (More than half of coarse fraction is larger than the #4 sieve)
- Cu 4 and/or 1 Cc 3 - Cu 4 and/or 1 Cc 3

GROUNDFREEZE GRAPHICS
- WATER LEVEL (level where first observed)
- WATER LEVEL (level after exploration completion)
- WATER LEVEL (additional levels after exploration)
- OBSERVED SEEPAGE

NOTES
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ABBREVIATIONS
WOH - Weight of Hammer
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- OBSERVED SEEPAGE

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ABBREVIATIONS
WOH - Weight of Hammer
WOR - Weight of Rod
# Soil Description Key

## Secondary Constituent

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<th>Term of Use</th>
<th>Amount</th>
<th>Secondary Constituent is Fine Grained</th>
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<tr>
<td>Trace</td>
<td>&lt;5%</td>
<td></td>
<td></td>
</tr>
<tr>
<td>With</td>
<td>&gt;5 to &lt;15%</td>
<td></td>
<td>&gt;15 to &lt;30%</td>
</tr>
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<td>30%</td>
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## Moisture Content

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<th>Pocket Pen (tsf)</th>
<th>Unconfined Compressive Strength (Q)&lt;sub&gt;u&lt;/sub&gt; (psf)</th>
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<td>Very Soft</td>
<td>&lt;2</td>
<td>PP &lt; 0.25</td>
<td>&lt;500</td>
</tr>
<tr>
<td>Soft</td>
<td>2 - 4</td>
<td>0.25 &lt; PP &lt; 0.5</td>
<td>500 - 1000</td>
</tr>
<tr>
<td>Medium Stiff</td>
<td>4 - 8</td>
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<td>1000 - 2000</td>
</tr>
<tr>
<td>Stiff</td>
<td>8 - 15</td>
<td>1 &lt; PP &lt; 2</td>
<td>2000 - 4000</td>
</tr>
<tr>
<td>Very Stiff</td>
<td>15 - 30</td>
<td>2 &lt; PP &lt; 4</td>
<td>4000 - 8000</td>
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<td>Hard</td>
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<td>4 &lt; PP</td>
<td>&gt;8000</td>
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## Cemenation

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<th>Field Test</th>
</tr>
</thead>
<tbody>
<tr>
<td>Thumb will penetrate more than 1 inch (25 mm). Exerts between fingers when squeezed.</td>
<td>Weakly Crumbles or breaks with handling or slight finger pressure</td>
</tr>
<tr>
<td>Thumb will penetrate soil about 1 inch (25 mm). Remolded by light finger pressure.</td>
<td>Moderately Crumbles or breaks with considerable finger pressure</td>
</tr>
<tr>
<td>Thumb will not crumble or break with finger pressure</td>
<td>Strongly Will not crumble or break with finger pressure</td>
</tr>
</tbody>
</table>

## Reaction with Hydrochloric Acid

<table>
<thead>
<tr>
<th>Description</th>
<th>Field Test</th>
</tr>
</thead>
<tbody>
<tr>
<td>None</td>
<td>No visible reaction</td>
</tr>
<tr>
<td>Weak</td>
<td>Some reaction, with bubbles forming slowly</td>
</tr>
<tr>
<td>Strong</td>
<td>Violent reaction, with bubbles forming immediately</td>
</tr>
</tbody>
</table>

## Plasticity

<table>
<thead>
<tr>
<th>Description</th>
<th>LL</th>
<th>Field Test</th>
</tr>
</thead>
<tbody>
<tr>
<td>Non-plastic</td>
<td>NP</td>
<td>A 1/8-in. (3 mm.) thread cannot be rolled at any water content.</td>
</tr>
<tr>
<td>Low (L)</td>
<td>&lt; 30</td>
<td>The thread can barely be rolled and the lump or thread cannot be formed when drier than the plastic limit.</td>
</tr>
<tr>
<td>Medium (M)</td>
<td>30 - 50</td>
<td>The thread is easy to roll and not much time is required to reach the plastic limit. The thread cannot be rolled after reaching the plastic limit. The lump or thread crumbles when drier than the plastic limit.</td>
</tr>
<tr>
<td>High (H)</td>
<td>&gt; 50</td>
<td>The thread can be rolled several times after reaching the plastic limit. The lump or thread can be formed without crumbling when drier than the plastic limit.</td>
</tr>
</tbody>
</table>

## Angularity

<table>
<thead>
<tr>
<th>Description</th>
<th>Criteria</th>
</tr>
</thead>
<tbody>
<tr>
<td>Angular</td>
<td>Particles have sharp edges and relatively plane sides with unpolished surfaces.</td>
</tr>
<tr>
<td>Subangular</td>
<td>Particles are similar to angular description but have rounded edges.</td>
</tr>
<tr>
<td>Subrounded</td>
<td>Particles have nearly plane sides but have well-rounded corners and edges.</td>
</tr>
<tr>
<td>Rounded</td>
<td>Particles have smoothly curved sides and no edges.</td>
</tr>
</tbody>
</table>
Artificial Fill: Silty SAND (SM): fine to medium-grained, yellowish brown, dry, medium dense, trace fine sub-angular gravel up to 1-inch in diameter, moist, decrease in gravel size, increase in gravel amount to fine sub-angular to sub-rounded gravel up to 1/4-inch in diameter.

Alluvium: Silty SAND (SM): fine to medium-grained, brown, moist, dense, trace fine sub-angular to sub-rounded gravel up to 1/4-inch in diameter, trace off-white carbonate, very dense, decreased gravel, color change to reddish yellow, very dense, increased silt.

Poorly graded SAND (SP): fine to coarse-grained, yellowish brown, moist, dense, trace clay/silt, trace off-white carbonate.

Bedrock: GRANITE: reddish brown to light brownish gray and pink, moist, very dense, excavates as Silty SAND, fine to coarse-grained.
Bedrock: GRANITE
reddish brown to light brownish gray and pink, moist, very dense, excavates as silty SAND, fine to coarse-grained at 35 feet - color change to primarily light brownish gray
moist, very dense, color change to white and gray, some pink

The boring was terminated at approximately 41.5 ft. below ground surface. The boring was backfilled with auger cuttings on December 08, 2017.

GROUNDWATER LEVEL INFORMATION:
Groundwater was not observed during drilling or after completion.

GENERAL NOTES:
APPENDIX B
LABORATORY RESULTS

Laboratory tests were performed on drive and bulk soil samples to estimate engineering characteristics of the various earth materials encountered. Testing was performed by our Kleinfelder lab in Ontario, California, with the exception of corrosion testing which was performed by AP Testing and Engineering Inc., of Pomona, California. The test results are presented herein.

IN-SITU MOISTURE CONTENT AND DRY UNIT WEIGHT

Moisture content and dry density tests were performed on relatively undisturbed soil samples. The moisture content and dry density tests were performed in accordance with ASTM Test Designations D2216 and D2937. The moisture and dry density test results are presented adjacent to the appropriate samples on the boring log.

GRAIN SIZE DISTRIBUTION

To provide information on the particle size of the soils encountered at the site, a sieve analysis was performed on select soil samples. The results of the sieve analysis, performed in accordance with ASTM Test Designation D6913 are presented adjacent to the appropriate samples on the boring logs and the test results are presented in Appendix B.

CORROSIVITY TESTS

A sample of the near surface soils collected during our field was tested for corrosion potential. Corrosivity testing was performed by AP Engineering Inc., of Pomona, California. The samples were tested for pH, electrical resistivity, and chloride and sulfate contents in accordance with ASTM test methods. The locations of the samples selected for testing are presented adjacent to the sample on the boring log and the test results are presented summarized in Appendix B.
### Soil Classification

<table>
<thead>
<tr>
<th>SYMBOL</th>
<th>BORING NO.</th>
<th>SAMPLE NO.</th>
<th>DEPTH (ft)</th>
<th>GRAVEL</th>
<th>SAND</th>
<th>FINES</th>
<th>LL</th>
<th>PL</th>
<th>PI</th>
<th>SOIL CLASSIFICATION</th>
</tr>
</thead>
<tbody>
<tr>
<td><img src="image" alt="Symbol" /></td>
<td>B-4</td>
<td>1</td>
<td>0-5'</td>
<td>0.9</td>
<td>63.2</td>
<td>35.9</td>
<td>N/A</td>
<td>N/A</td>
<td>N/A</td>
<td>Silty Sand (SM)</td>
</tr>
<tr>
<td><img src="image" alt="Symbol" /></td>
<td>B-4</td>
<td>4</td>
<td>10'</td>
<td>5.1</td>
<td>63.6</td>
<td>31.3</td>
<td>N/A</td>
<td>N/A</td>
<td>N/A</td>
<td>Silty Sand (SM)</td>
</tr>
</tbody>
</table>

#### Atterberg Limits

- **LL ( Liquidity Limit)**: N/A
- **PL ( Plastic Limit)**: N/A
- **PI ( Plastic Index)**: N/A

#### Grain Size Distribution

<table>
<thead>
<tr>
<th>Grain Size in Millimeters</th>
<th>Cumulative % Passing</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.100</td>
<td>100</td>
</tr>
<tr>
<td>0.010</td>
<td>90</td>
</tr>
<tr>
<td>0.001</td>
<td>80</td>
</tr>
<tr>
<td>0.0001</td>
<td>70</td>
</tr>
<tr>
<td>0.00001</td>
<td>60</td>
</tr>
<tr>
<td>0.000001</td>
<td>50</td>
</tr>
<tr>
<td>0.0000001</td>
<td>40</td>
</tr>
<tr>
<td>0.00000001</td>
<td>30</td>
</tr>
<tr>
<td>0.000000001</td>
<td>20</td>
</tr>
<tr>
<td>0.0000000001</td>
<td>10</td>
</tr>
<tr>
<td>0.00000000001</td>
<td>10</td>
</tr>
<tr>
<td>0.000000000001</td>
<td>0.0001</td>
</tr>
<tr>
<td>0.0000000000001</td>
<td>0.010</td>
</tr>
<tr>
<td>0.00000000000001</td>
<td>0.100</td>
</tr>
<tr>
<td>0.000000000000001</td>
<td>1.000</td>
</tr>
<tr>
<td>0.0000000000000001</td>
<td>10.000</td>
</tr>
<tr>
<td>0.00000000000000001</td>
<td>100.000</td>
</tr>
</tbody>
</table>

#### Sample Identification

- **SAMPLE IDENTIFICATION**: B-4, 1, 0-5' for Silty Sand (SM)
- **SAMPLE IDENTIFICATION**: B-4, 4, 10' for Silty Sand (SM)

#### Test Details

- **PN.**: 20182806.001A
- **TESTED BY**: C. Massa
- **DATE**: 12/11/2017
- **CHECKED BY**: J. Diaz

#### Project Information

- **PROPOSED EUCALYPTUS BOOSTER STATION**: EUCALYPTUS AVE WEST of MORENO BEACH DR, MORENO VALLEY, CALIFORNIA

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KLEINFELDER - 620 Magnolia Avenue, Building G | Ontario, California 91762 | PH: (909) 657-1716 | FAX: (909) 988-0185 | www.kleinfelder.com
# CORROSION TEST RESULTS

**Client Name:** Kleinfelder  
**AP Job No.:** 17-1222  
**Project Name:** EMWD BPS Reactivation  
**Date:** 12/13/17  
**Project No.:** 20182806.001A

<table>
<thead>
<tr>
<th>Boring No.</th>
<th>Sample No.</th>
<th>Depth (feet)</th>
<th>Soil Type</th>
<th>Minimum Resistivity (ohm-cm)</th>
<th>pH</th>
<th>Sulfate Content (ppm)</th>
<th>Chloride Content (ppm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>B-4</td>
<td>1</td>
<td>0-5</td>
<td>SM</td>
<td>3886</td>
<td>7.7</td>
<td>45</td>
<td>36</td>
</tr>
</tbody>
</table>

**NOTES:**  
Resistivity Test and pH: California Test Method 643  
Sulfate Content: California Test Method 417  
Chloride Content: California Test Method 422  
ND = Not Detectable  
NA = Not Sufficient Sample  
NR = Not Requested