GEOTECHNICAL EXPLORATION
PROPOSED CITY OF GOLETA FIRE STATION NO. 10
7952 HOLLISTER AVENUE
GOLETA, CALIFORNIA

Prepared for:

CITY OF GOLETA
Goleta, California 93030-5650

Project No. 11389.001

February 21, 2017
February 21, 2017

Project No. 11389.001

City of Goleta
130 Cremona Drive, Suite B
Goleta, California 93117

Attention: Ms. Claudia Dato, Project Manager

Subject: Geotechnical Exploration
Proposed City of Goleta Fire Station No. 10
7952 Hollister Avenue
Goleta, California

In accordance with our May 5, 2016 proposal authorized on June 21, 2016, Leighton Consulting, Inc. presents the results of our geotechnical exploration for use in entitling and designing the proposed City of Goleta Fire Station No. 10. It will be constructed at the northeast corner of Hollister Avenue and Cathedral Oaks Avenue in western Goleta, California. The purpose of our exploration has been to (1) explore subsurface soil conditions onsite, and (2) provide geotechnical recommendations for design and construction of this proposed fire station.

The site is essentially flat and covered with grasses, shrubs and eucalyptus trees. The northern side of the property has a 35-foot high, slope that descends to the north towards Union Pacific Railroad property at a gradient of approximately 1:1 h:v (horizontal:vertical), and has been subjected to severe erosion. The property boundary between the Fire Station #10 site and the Union Pacific Railroad property is located mid-slope. Site soils consist of a thin layer of undocumented fill and or native soil mantling marine terrace deposits to the depths explored. No groundwater was encountered during site exploration, however we observed runoff ponded in the southeast corner of the site after heavy rainfall.

The principal constraints to site development is the stability and potential for continued severe erosion of the slope at the north site boundary. Leighton has evaluated and
provided design parameters for three options to mitigate slope instability, which include: piles installed at top of slope, in between the property line and the of the slope, and at property line. We have also provided recommendations for a structural setback from the top of the slope. Based on discussions with you, the City’s preferred option is a pile wall at the midslope property line and the placement of fill behind it in order to obtain additional buildable space. to proposed finished grade at the property line. We also present foundation design recommendations for the proposed fire station and other improvements proposed for the project.

More detailed recommendations are presented in this report. If you have any questions regarding, please do not hesitate to contact this office at (805) 654-9257 or 866-LEIGHTON, directly at the phone extensions and/or e-mail addresses listed below. We appreciated being of service.

Respectfully submitted,

LEIGHTON CONSULTING, INC.

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LJD/VPI/GIM/gv

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# 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 Site Location and Project Description</td>
<td>1</td>
</tr>
<tr>
<td>1.2 Purpose and Scope of Exploration</td>
<td>2</td>
</tr>
<tr>
<td>2.0 FINDINGS</td>
<td>4</td>
</tr>
<tr>
<td>2.1 Regional Geologic Setting</td>
<td>4</td>
</tr>
<tr>
<td>2.2 Current Site Surface Condition</td>
<td>4</td>
</tr>
<tr>
<td>2.3 Available Site History</td>
<td>5</td>
</tr>
<tr>
<td>2.4 Previous Geotechnical Subsurface Explorations</td>
<td>5</td>
</tr>
<tr>
<td>2.5 Subsurface Soil Conditions</td>
<td>5</td>
</tr>
<tr>
<td>2.5.1 Expansive Soil</td>
<td>7</td>
</tr>
<tr>
<td>2.5.2 Sulfate and Chloride Content, Resistivity and pH</td>
<td>7</td>
</tr>
<tr>
<td>2.6 Infiltration Testing</td>
<td>8</td>
</tr>
<tr>
<td>2.7 Groundwater</td>
<td>9</td>
</tr>
<tr>
<td>2.8 Flood Hazard</td>
<td>9</td>
</tr>
<tr>
<td>2.9 Faulting and Earthquakes</td>
<td>9</td>
</tr>
<tr>
<td>2.10 Secondary Seismic Hazards</td>
<td>12</td>
</tr>
<tr>
<td>2.10.1 Liquefaction Potential</td>
<td>12</td>
</tr>
<tr>
<td>2.10.2 Seismically-Induced Settlement</td>
<td>13</td>
</tr>
<tr>
<td>2.10.3 Seismically-Induced Landslides</td>
<td>13</td>
</tr>
<tr>
<td>2.10.4 Earthquake-Induced Flooding</td>
<td>13</td>
</tr>
<tr>
<td>2.10.5 Seiches and Tsunamis</td>
<td>14</td>
</tr>
<tr>
<td>2.11 Slope Stability</td>
<td>14</td>
</tr>
<tr>
<td>2.11.1 Shear Strength Parameters</td>
<td>15</td>
</tr>
<tr>
<td>2.11.2 Factor of Safety</td>
<td>16</td>
</tr>
<tr>
<td>2.11.3 Conditions Analyzed</td>
<td>16</td>
</tr>
<tr>
<td>2.11.4 Results of Slope Stability</td>
<td>17</td>
</tr>
<tr>
<td>3.0 CONCLUSIONS AND RECOMMENDATIONS</td>
<td>19</td>
</tr>
<tr>
<td>3.1 Summary of Conclusions and Recommendations</td>
<td>19</td>
</tr>
<tr>
<td>3.2 Plans, Specifications, and Construction Review</td>
<td>19</td>
</tr>
<tr>
<td>3.3 Earthwork</td>
<td>20</td>
</tr>
<tr>
<td>3.3.1 Preparation</td>
<td>20</td>
</tr>
<tr>
<td>3.3.2 Overexcavation and Recompaction</td>
<td>21</td>
</tr>
<tr>
<td>3.3.3 Fill Placement and Compaction</td>
<td>21</td>
</tr>
<tr>
<td>3.3.4 Utility Trench Backfill</td>
<td>22</td>
</tr>
<tr>
<td>3.3.5 Surface Drainage</td>
<td>23</td>
</tr>
</tbody>
</table>
TABLE OF CONTENTS (Cont’d)

<table>
<thead>
<tr>
<th>Section</th>
<th>Page</th>
</tr>
</thead>
<tbody>
<tr>
<td>3.3.6</td>
<td>Construction Observation</td>
</tr>
<tr>
<td>3.4</td>
<td>Slope Mitigation Alternatives</td>
</tr>
<tr>
<td>3.4.1</td>
<td>Design Earth Pressure</td>
</tr>
<tr>
<td>3.4.2</td>
<td>Pile Embedment</td>
</tr>
<tr>
<td>3.4.3</td>
<td>Drilled Soldier Piles</td>
</tr>
<tr>
<td>3.4.4</td>
<td>Backfill</td>
</tr>
<tr>
<td>3.4.5</td>
<td>Drainage</td>
</tr>
<tr>
<td>3.5</td>
<td>Seismic Design Parameters</td>
</tr>
<tr>
<td>3.6</td>
<td>Shallow Foundations</td>
</tr>
<tr>
<td>3.6.1</td>
<td>Allowable Bearing</td>
</tr>
<tr>
<td>3.6.2</td>
<td>Lateral Load Resistance</td>
</tr>
<tr>
<td>3.6.3</td>
<td>Settlement Estimates</td>
</tr>
<tr>
<td>3.7</td>
<td>Foundations for Ancillary Improvements</td>
</tr>
<tr>
<td>3.7.1</td>
<td>Minimum Embedment and Width</td>
</tr>
<tr>
<td>3.7.2</td>
<td>Allowable Bearing Capacity</td>
</tr>
<tr>
<td>3.7.3</td>
<td>Lateral Load Resistance</td>
</tr>
<tr>
<td>3.8</td>
<td>Pier Foundations (Flagpole and Light Poles)</td>
</tr>
<tr>
<td>3.8.1</td>
<td>Downward Pier Capacity</td>
</tr>
<tr>
<td>3.8.2</td>
<td>Lateral Pier Capacity</td>
</tr>
<tr>
<td>3.9</td>
<td>Portland Cement Type and Corrosion Protection</td>
</tr>
<tr>
<td>3.10</td>
<td>Preliminary Pavement Design</td>
</tr>
<tr>
<td>3.10.1</td>
<td>General Pavement Recommendations</td>
</tr>
<tr>
<td>4.0</td>
<td>CONSTRUCTION CONSIDERATIONS</td>
</tr>
<tr>
<td>4.1</td>
<td>Temporary Excavations</td>
</tr>
<tr>
<td>4.2</td>
<td>Drilled Cast-In-Place Concrete Pile/Pier Installation Considerations</td>
</tr>
<tr>
<td>4.3</td>
<td>Geotechnical Review During Construction</td>
</tr>
<tr>
<td>5.0</td>
<td>LIMITATIONS</td>
</tr>
</tbody>
</table>

REFERENCES

TABLES
Table 1 – Select Geotechnical Laboratory Testing Results | 6
Table 2 – Shear Strength Test Results | 16
Table 3 – Conditions Analyzed | 17
Table 4 – Summary of Stability Analyses for Conditions Analyzed | 18
Table 5 – Active Earth Pressures | 26
TABLE OF CONTENTS (Cont’d)

<table>
<thead>
<tr>
<th>Section</th>
<th>Page</th>
</tr>
</thead>
<tbody>
<tr>
<td>Table 6 – 2016 CBC Based Seismic Design Parameters</td>
<td>29</td>
</tr>
<tr>
<td>Table 7 – AC Pavement Section based on R-Value =40</td>
<td>34</td>
</tr>
<tr>
<td>Table 8 – Caltrans Concrete Pavement Section</td>
<td>35</td>
</tr>
</tbody>
</table>

FIGURES AND PLATES

- Figure 1 – Site Location Map
- Figure 2a – Regional Geology Map
- Figure 2b – Legend to Regional Geology Map
- Figure 3 – Regional Fault Map
- Figure 4 – Historic Seismicity Map
- Plate 1 – Geotechnical Exploration Location Map
- Plate 2 – Geotechnical Cross Section A-A’ and B-B’

APPENDICES

- Appendix A – Field Exploration
- Appendix B – Geotechnical Laboratory Testing
- Appendix C – Seismic Design Parameters
- Appendix D – Slope Stability
- Appendix E – Previous Site Exploration Data
- Appendix F – Earthwork and Grading Guide Specifications
- Appendix G – ASFE Important Information About Your Geotechnical Report
1.0 INTRODUCTION

1.1 Site Location and Project Description

The proposed fire station site is located at 7952 Hollister Avenue in western Goleta, as depicted on Figure 1, Site Location Map. This approximately 2-acre site is bounded on the north by an approximately 35-foot-high slope that descends to railroad tracks, on the east by an existing multifamily development, on the south by Hollister Avenue, and on the west by the Cathedral Oaks/Winchester Canyon Road overpass. The descending slope has an overall gradient of approximately 1:1 (horizontal:vertical, h:v), is sparsely vegetated and exhibits severe rill erosion typical of marine terrace deposits along the coast of southern California. To the west, the slope is protected by concrete adjacent to the overpass. The toe of the northern slope is located about 15 to 20 feet from the railroad tracks and no water seepage was observed in the slope. The central and western portion of the site was formerly occupied by a gas station that was demolished in the early 1990s.

The conceptual project layout showing project elements was provided by KBZ Architects (2016) and consists of an 11,000 square foot facility, centrally located on the site. The project layout was used as a base map for Plate 1, Geotechnical Exploration Location Map. We understand that the following improvements are proposed:

- **Fire Station:** The general outline of the fire station was provided by KBZ Architects, and may be one or two stories. This building will be located centrally on the site and oriented parallel to Hollister Avenue. A concrete access driveway will wrap around the building with fire truck turnarounds located at the back. Parking spaces will be located in the front of the building.

- **Appurtenant Structures:** Trash enclosures and fire hose drying racks will be located at the edge of the driveway and along the slope at the back of the building. We assume masonry screen walls, light poles, and a flagpole will be located onsite, along with landscaping and drainage features.

Basements or substructures are not anticipated. Based on June 2016 conceptual plans from KBZ Architects, and a site survey by Wallace Group dated June 13, 2016, it appears that site grading will consist of minor cuts and fills on
the order of five to seven feet to attain the desired site grades and surface drainage.

1.2 **Purpose and Scope of Exploration**

Purposes of this geotechnical exploration have been to (1) explore subsurface soil conditions onsite, and (2) provide geotechnical recommendations for design and construction of this proposed fire station. We have relied on discussions with you during the exploration phase of this project, and on the preliminary plan prepared provided by the project team. In accordance with our May 5, 2016 proposal (costs revised July 12, 2016), the scope of our exploration has included the following tasks:

- **Background Review:** A background review of readily-available, relevant, public and in-house geotechnical literature was performed. Information was obtained from adjacent projects, including the condominiums to the east of the site and the reconfiguration of the Cathedral Oaks Railroad overpass to the west. Pertinent references are presented at the end of this report text.

- **Subsurface Exploration:** Prior to subsurface exploration, we contacted Underground Service Alert (USA) so they would locate and mark existing registered public underground utilities onsite. Our subsurface exploration consisted of a total of seven hollow stem auger borings drilled to a maximum depth of 55 feet and sampled to a depth of 56.5 feet. Earth material samples were retrieved from our borings for classification and geotechnical laboratory testing. Approximate locations of the borings are shown on Plate 1. A more detailed description of our field exploration is presented in Appendix A, *Field Exploration*.

- **Geotechnical Laboratory Testing:** Physical and engineering properties of sampled subsurface soils were evaluated by visually classifying recovered samples and performing various geotechnical laboratory tests on selected samples at our in-house (Irvine) geotechnical laboratory. Descriptions of these tests and results are presented in Appendix B, “*Geotechnical Laboratory Testing*.”

- **Geotechnical Analyses:** Geotechnical engineering analyses were performed, including a site-specific ground motion study, a seismic settlement...
analysis, and slope stability analyses, to develop geotechnical recommendations and design parameters for this proposed fire station.

- **Report Preparation:** This report was prepared to summarize our findings, and to present our design-specific geotechnical conclusions and recommendations solely for design of Fire Station No. 10 and appurtenant structures.

As mentioned earlier, the site was formerly occupied by a gas station with underground tanks located in the western portion of the site under the proposed driveway area, and the former building footprint is located close to the proposed fire station. The approximate locations of these former structures are shown on Figure 2 of the Holguin Fahan (2012) Site Closure report included in Appendix E and also on Plate 1. Site assessment to evaluate the environmental conditions at the site was not a part of our scope of work.
2.0 FINDINGS

2.1 Regional Geologic Setting

The site lies in the Santa Barbara Coastal Plain, within the western portion of the Transverse Ranges Geomorphic Province, which is characterized by west-trending compressional (thrust and reverse) faults and mountain ranges. The Santa Barbara Coastal Plain (SBCP) is bounded to the north by the Santa Ynez Mountains, by the Santa Barbara Channel and the Pacific Ocean to the south, and narrows on the western boundary to the west of the City of Goleta and on the eastern boundary east of the City of Carpinteria (USGS, 2009).

The SBCP is dominated by irregularly deformed Cretaceous-through Pleistocene-age marine and non-marine sedimentary strata, which form the ridgelines of the Santa Ynez Mountains north of the site. These highly resistant strata record a long history of continental-margin tectonism, and deposits as young as Quaternary age, and have sustained strong deformations that include faulting, folding, and clockwise rotation of crustal blocks. Quaternary deformation (uplift and folding) has exposed less resistant terrestrial and marine sedimentary rocks, which form the coastal hills and mesas that bound the southwestern limits of the City of Santa Barbara to the east of the site.

2.2 Current Site Surface Condition

This approximately 2-acre site was previously occupied by a one-story gas station that was demolished in 1993. Currently the site is vacant and covered with brush, leaves, and eucalyptus trees. The site topography is uneven and there is approximately 4 feet of relief across the site. Generally, the site slopes gently towards the southeast corner. Drainage is to the southeast via sheet flow runoff, but collects in low spots across the site and also appears to flow over the northern slope locally. The site has an average elevation of between 117 feet and 121 feet above mean sea level (MSL), with a 35-foot-high slope at the northern portion of the site which descends to the railroad tracks offsite. The northeast corner of the site slopes gently towards this slope, and runoff drains over the slope in this area resulting in periodic, localized, severe erosion on the slope.
2.3 **Available Site History**

Based on our review of historic aerial photographs and available site documentation, the property was formerly occupied by a gas station from 1969 until 1993. The approximate location of the gas station and associated facilities is depicted in the reports by Holguin and Fahan (2012), and were located on the western portion of the property in the vicinity of LB-7.

2.4 **Previous Geotechnical Subsurface Explorations**

No geotechnical investigations have been performed on the site. However, exploration was performed nearby for the Cathedral Oaks Overpass to the west and the Whimbrel Lane residential development to the east. The City of Goleta provided the Log of Test Borings provided by State of California (1968) for the Hollister Avenue Overpass and the preliminary geotechnical investigation performed by Padre Geotechnical (1999) for the residential development to the east. A subsequent update report by others was not made available, however, plans for a shear pin wall installed along the slope immediately east of the site were provided by Burnett & Young, the shoring engineers for that project (Burnett & Young, 2013). This information is included in Appendix E.

2.5 **Subsurface Soil Conditions**

Based upon our review of pertinent geotechnical literature and our recent subsurface exploration, the site is blanketed by undocumented fill (Afu) as much as 5 feet thick, overlying Pleistocene-age Marine Terrace Deposits (Qmt) to the depths explored. Locations of geotechnical explorations are shown on the Geotechnical Exploration Location Map, Plate 1, and logs of the explorations, LB-1 through LB-7, are included in Appendix A, Field Exploration. Fill and terrace deposits are described in further detail as follows:

- **Undocumented Fill (Afu):** Undocumented fill consisted of brown silty sand with angular gravel as much as 5 feet in thickness across the old gas station site. Fill placement during previous construction and demolition associated with the gas station is not well documented. Therefore, considering past site development, undocumented fill may be as deep as 7 to 10 feet in the area of former tank locations onsite, which, based on the proposed site layout, is under the proposed western driveway area. We are unaware of any
engineered fill placement documentation for this site, so we classify all fill soils on site as undocumented.

- **Marine Terrace Deposits (Qmt):** Below the fill to the depths explored, we sampled Pleistocene-age Marine Terrace Deposits. These deposits consisted of interbeds and lenses of dense silty sand (SM) and sandy silt (ML) with some minor stiff clay layers that were interbedded with three distinct layers of dense to very dense, silty to poorly graded sand (SP). The sand layers were encountered at depths of 10 feet, 25 feet and 50 feet, and ranged from 5 to 10 feet in thickness.

The interpreted site stratigraphy is depicted on Geotechnical Cross-Sections A-A’ and B-B’ (Plate 2).

The geotechnical properties of samples of the site soils that were tested are summarized in the table below, described in the following subsections, and summarized in Appendix B, Geotechnical Laboratory Testing.

**Table 1 – Select Geotechnical Laboratory Testing Results**

<table>
<thead>
<tr>
<th>Material Description</th>
<th>Depth Interval, bgs (feet)</th>
<th>Average Fines* (percent)</th>
<th>Unit Weight $\gamma_{tot}$ (pcf)</th>
<th>Other Index Properties</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Undocumented Artificial Fill (Afu)</strong> silty SAND (SM) to clayey SAND (SC)</td>
<td>0 to 5</td>
<td>30-40</td>
<td>125</td>
<td>Plasticity Index (PI) = 24 Expansion Index (EI) = 6 R-value &gt;40 (64). Maximum Density = 129.5 pcf OMC 8.4%</td>
</tr>
<tr>
<td><strong>Marine Terrace Deposits (Qmt)</strong> coarse grained silty SAND (SM)</td>
<td>10 to 15 25 to 35</td>
<td>25-40</td>
<td>120</td>
<td>See Appendix B for additional test results.</td>
</tr>
<tr>
<td><strong>Marine Terrace Deposits (Qmt)</strong> fine grained sandy, silty CLAY (CL) and SILT (ML)</td>
<td>5 to 10 15 to 25 35 to 50</td>
<td>50-70</td>
<td>120</td>
<td></td>
</tr>
<tr>
<td><strong>Marine Terrace Deposits (Qmt)</strong> well graded SAND (SM)</td>
<td>50 to 56.5</td>
<td>13</td>
<td>120</td>
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</tr>
</tbody>
</table>

*Qualitative average of percent silt and clay passing the No. 200 U.S. Standard Sieve*
2.5.1 **Expansive Soil**

Collapse (moisture sensitivity) potential refers to the potential settlement of a soil under existing stresses upon being wetted. Representative samples from 10 feet depth in Borings LB-5 and LB-6 were tested for swell and collapse potential. Test results suggest that sandy soils at this site within 15 feet of the ground surface may have a low collapse potential of about 1%. Test results of the clay soils at 10 to 15 feet depths indicated the potential for swell of up to 4%, indicating that some clay layers at those depths have the potential to swell moderately. The clay layer tested is on the order of two to five thick feet at depths of 10 feet or more below grade. Therefore, if swelling occurs, the maximum swell anticipated is about two inches but at depths that may not adversely impact the proposed improvements.

We performed preliminary testing on the shallow soil which may be used for backfill next to, and in contact with the proposed construction foundations and flatwork. A bulk sample from Boring LB-7 was tested for expansion potential. Test results indicated an Expansion Index (EI) of 6. Based on this test result, our field observation of onsite soil and our experience in the area, near-surface onsite soil does not appear to be expansive. Due to the presence of fine-grained soils, pockets of expansive soil may be present at the site.

2.5.2 **Sulfate and Chloride Content, Resistivity and pH**

The near-surface on-site soil was screened for corrosion potential. The test results are as follows:

- Sulfates: 425 ppm
- Chlorides: 345 ppm
- pH: 7.07
- Resistivity: 988 ohm-cm

Water-soluble sulfates in soil can react adversely with concrete. However, concrete in contact with soil containing sulfate concentrations less-than (<) 0.10 percent (1,000 parts-per-million) is considered to have negligible sulfate exposure (see ACI 318-05, Table 4.3.1 as referenced in the 2016 California Building Code - CBC, Section 1904). As summarized, based on ACI criteria, sulfate exposure, although slightly elevated for this near-
marine environment, can be considered “negligible” for soils sampled at this site and tested.

Corrosivity of soils in direct contact with ferrous metals and embedded metals can be evaluated by measuring electrical resistivity, soluble chloride content and pH of soil. In general, soil having a minimum resistivity less-than (<) 3,000 ohm-cm is considered corrosive to ferrous metals. Soil with a chloride content greater-than-or-equal-to (≥) 500 ppm is considered corrosive to embedded metals. Based on the test results, soils tested were considered corrosive to buried ferrous metals. However, the on-site soil is not considered to be corrosive to embedded metals.

2.6 Infiltration Testing

Leighton drilled two hollow-stem auger borings adjacent to borings that were logged and sampled in locations identified by the civil engineer as potential infiltration areas. LB-1 was drilled to a depth of 7 feet (adjacent to LB-2) and LB-3A was drilled to a depth of 25 feet (adjacent to LB-3). LB-3A was drilled to a depth of 25 feet and encountered sand at a depth of about 20 feet. Before the pipe could be installed, the hole collapsed to a depth of about 15 feet. After drilling, slotted 2-inch diameter standpipes were installed and backfilled in accordance with County of Santa Barbara Water Resources Division (2014) requirements for infiltration testing. Potable water was provided by the drilling contractor. After infiltration testing was completed, Leighton attempted to remove the pipes, but could not. Therefore, the pipes were cut off one foot below grade and the borings backfilled with soil cuttings to the ground surface. The results of infiltration testing are included in Appendix A, Field Exploration. County of Santa Barbara and Central Coast Water Board Standards refer to Orange County Technical Guidelines (2011) for calculating Infiltration Rates. The resulting calculated Infiltration Rates are as follows, and do not include a factor of safety of 2:

- LB-1 (depth 7 feet): 0.05 inches/hour
- LB-3A (depth 15 feet): 0.07 inches/hour
2.7 **Groundwater**

The site is located near the western boundary of the Goleta Basin, within the Goleta West Basin. Groundwater in Goleta occurs in the alluvium, the fanglomerate, and the Santa Barbara Formation (CADWR, 2004). Groundwater in the Goleta Basin is generally divided into a shallow zone and a deep zone. The shallow zone includes the recent alluvium, parts of the Upper Pleistocene alluvium, and the upper part of the fanglomerate. The deep zone includes the lower part of the Upper Pleistocene alluvium and the Santa Barbara Formation. Based on the above information, regionally, depth to groundwater in the site vicinity appears to be below 100 ft. bgs, and the groundwater flow direction in the Goleta Basin is generally toward the south.

Based on historical data from Caltrans for the Hollister Avenue overpass (State of California, 1968) and more recent data from the adjacent residential site (Padre Geotechnical, 1999), drilled during historically wet periods, ground water has not been encountered to depths of 70 feet. The state borings were drilled in March 1957 and extended into bedrock to depths of 75 feet. Padre borings extended into terrace deposits to depths of 51.5 feet. Groundwater was not encountered during our July 2016 exploration. Also, groundwater was not observed in nearby borings, advanced up to 70 feet deep, performed in support of the Cathedral Oaks Crossing or for the adjacent development to the east. Seepage was not observed on the slope at the northern site boundary during the field investigation.

2.8 **Flood Hazard**

According to the FEMA Flood Insurance Rate Map (FEMA, 2012), this site is located in an area determined to be “outside the 0.2% annual chance floodplain”, but does not imply the absence of a flood hazard.

2.9 **Faulting and Earthquakes**

The site lies within the Santa Barbara Fold and Fault Belt (SBFFB), a region within the SBCP characterized by folds and partially-buried oblique and reverse faults that transect the coastal plain, and which are expressed geomorphically on the surface as mesas and hills characteristic of the area (USGS, 2009). Active faults are defined as those that have demonstrated surface displacement within Holocene time (i.e. within the last 11,000 years). Potentially active faults are
those that have demonstrated surface displacement in Quaternary time (i.e. the last 1.6 million years).

Our review of available in-house and published literature indicates that there are no currently known active or potentially active faults that traverse or project toward the site, and the site is not located within a currently designated Alquist-Priolo Earthquake Fault Zone (Bryant and Hart, 2007; CGS, 2010).

The closest active fault to the site is a portion of the Santa Ynez Fault, located in the eastern section of the Santa Ynez fault zone approximately 8.5 miles to the northeast.

Potentially active faults mapped within the SBFFB are closer to the site and include the north and south branches of the More Ranch Fault of the Mission Ridge fault system, located approximately 0.4 miles to the south and 1.6 miles to the southeast, respectively, and the Glen Annie Fault, located approximately 1.0 miles to the north (see Figure 3, Regional Fault Map) (USGS, 2009; CGS, 2010). The USGS (2009) Geologic Map of the Santa Barbara Coastal Plain Area names the north and south branches of the More Ranch Fault as “North Branch Western More Ranch Fault” and “South Branch Western More Ranch Fault” (see Figure 2, Regional Geology Map), whereas the CGS (2010) Fault Activity Map of California makes no distinction between the two branches and labels both branches as the “More Ranch Fault” (see http://www.quake.ca.gov/gmaps/FAM/faultactivitymap.html). In Figure 3, Regional Fault Map, the two branches are distinguished by “north branch” and “south branch”. Although it has not been demonstrated that many of the mapped faults in the immediate vicinity are active, this may be due to a lack of subsurface exploration of these faults which may mean some of the faults are active.

The nearest fault outside of the SBFFB is the Pitas Point Fault located approximately 15.7 miles to the southeast. Other nearby active and potentially active faults that could cause strong ground shaking at the site include the Los Alamos Fault, located approximately 25.7 miles to the northwest, the active portion (southern strand) of the Red Mountain Fault within the Red Mountain fault zone, located approximately 26.1 miles to the southeast, the Javon Canyon Fault located approximately 28.9 miles to the southeast, and the Santa Ana section of the Mission Ridge fault system and faults near Oakview and Meiners Oaks, California, located approximately 34.1 miles to the east. The San Andreas Fault
is the largest fault in southern California and is located approximately 44 miles to the northeast (CGS, 2010). Additional faults identified within 100 kilometers (62 miles) of the site are presented in Appendix C (Blake 2000a).

**Historical Seismicity** Figure 4 illustrates the epicentral locations of historic earthquakes in the vicinity of the site. Strong offshore earthquakes located within 0.6 miles of the SBFFB, including a 6.3 magnitude earthquake in 1925, a 5.5 magnitude earthquake in 1941, and a 5.1 magnitude earthquake in 1978. Several other seismic events offshore of, and within, the SBFFB are likely to have occurred along oblique-slip faults that may be continuous with Quaternary-age reverse faults in the area. Faults within and around the SBFFB pose a significant risk for activity and strong ground shaking (USGS, 2009). Additionally, the San Andreas Fault Zone, a 744-mile long fault system located approximately 44 miles to the northeast and outside of the SBFFB, has been responsible for several significant historical seismic events including the 1857 magnitude 7.9 Fort Tejon Earthquake, and can also pose a significant risk for activity and strong ground shaking (SCEDC, 2016).

The computer program EQSEARCH (Blake, 2000b) was used to evaluate past, documented seismic activity near the site. This program performs an automated search of a catalog of historic southern California earthquakes, and computes the distance from a project site to each of the earthquake epicenters within a specified search radius of 62 miles (approximately 100 kilometers). From the computed distances, the program also estimates (using an appropriate attenuation relationship) the peak horizontal ground acceleration that may have occurred at the site due to each earthquake. A database of recorded earthquakes with magnitudes of 4.0 or larger between 1800 and 2016 was used in the analysis. The results of each analysis, including an earthquake epicenter map for events from 1800 to 2016, and a listing of historic earthquakes with an epicentral distance of less than 62 miles from the site, are presented in Appendix C.

The largest historical earthquake within the 62-mile radius of the site was the 1857 magnitude 7.9 Fort Tejon Earthquake that occurred on the San Andreas Fault approximately 60.3 miles to the northeast. It is estimated to have produced a horizontal ground acceleration of 0.13g at the site. The earthquake event to have produced the highest estimated horizontal ground acceleration, 0.25g, at the site was a magnitude 5.7 earthquake that occurred approximately 4.9 miles
east-southeast of the site near the More Ranch fault in 1862. Other significant historical earthquakes within southern California include the 1952 magnitude 7.7 Arvin-Tehachapi Earthquake that occurred approximately 63.9 miles to the northeast, the 1971 magnitude 6.6 San Fernando Earthquake that occurred approximately 86.3 miles to the east, and the 1994 magnitude 6.7 Northridge Earthquake that occurred approximately 79.6 miles to the east-southeast (SCEDC, 2016).

2.10 Secondary Seismic Hazards

The Goleta Quadrangle has not yet been evaluated and mapped by the State of California Seismic Hazard Mapping Program. However, the County of Santa Barbara has characterized seismic hazards in the region in planning documents (County of Santa Barbara, 2015).

2.10.1 Liquefaction Potential

Liquefaction is the loss of soil strength due to a buildup of excess pore-water pressure during strong and long-duration ground shaking. Liquefaction is associated primarily with loose (low density), saturated, relatively uniform fine- to medium-grained, clean cohesionless soils. As shaking action of an earthquake progresses, soil granules are rearranged and the soil densifies within a short period. This rapid densification of soil results in a buildup of pore-water pressure. When the pore-water pressure approaches the total overburden pressure, soil shear strength reduces abruptly and temporarily behaves similar to a fluid. For liquefaction to occur, there must be:

(1) Loose granular soils,
(2) Shallow groundwater, and
(3) Strong, long-duration ground shaking

If all above conditions occur or exist simultaneously, then liquidation may occur. If one is missing, then liquefaction will not occur. Effects of liquefaction can include sand boils, settlement and bearing capacity failures below structures. Based on blow counts, and current and historic groundwater conditions, the potential for liquefaction at the site is considered low.
Based on a review of the County of Santa Barbara’s Seismic Safety & Safety Element (County of Santa Barbara, Planning and Development, 2015) and the County of Santa Barbara’s 2016 Multi-Jurisdictional Hazard Mitigation Plan (see Section 5 of: https://www.countyofsbc.org/ceo/oem/2016HMP.sbc), the site appears to have a low potential for liquefaction potential.

2.10.2 **Seismically-Induced Settlement**

During a strong seismic event, seismically-induced settlement can occur within loose to moderately dense, dry or saturated granular soil. Settlement caused by ground shaking is often non-uniformly distributed, which can result in differential settlement. Because the shallow surface soils will be over-excavated and recompacted, the potential for dry sand seismic settlement is expected to be low to moderate. Based on SPT blow counts from Boring LB-2, Leighton analyzed dry sand seismic settlement using the program LIQUEFY-PRO. The stratigraphy is relatively uniform across the site, and Boring LB-2 was judged to be representative of site conditions. As tabulated, we estimate that seismically-induced settlement due to dry sand settlement is on the order of 1.5 to 2 inches. Differential settlement may be assumed to be one-half of the total settlement over a horizontal distance of 40 feet.

2.10.3 **Seismically-Induced Landslides**

As shown on Plate 1, the site is bounded on the north by an approximately 35-foot-high descending slope that has a gradient of about 1H:1V and which is locally steeper. Based on the results of our slope stability analyses, the descending slope is grossly stable with respect to pseudostatic conditions based on seismic screening procedures (CGS, 2008). A discussion of our findings and conclusions is presented in the section 2.11 Slope Stability.

2.10.4 **Earthquake-Induced Flooding**

Earthquake-induced flooding can result from failure of up-gradient dams or other water-retaining structures resulting from earthquakes. There are no significant retained bodies of water located up-gradient from the site. Therefore, the site is not considered susceptible to earthquake-induced flooding.
2.10.5 **Seiches and Tsunamis**

Seiches are large waves generated in enclosed bodies of water in response to ground shaking. No bodies of water wherein seiches may occur are proximal to the site. Tsunamis are waves generated in large bodies of water by fault displacement or major ground movement. According to the California Geological Survey (CGS, 2009), although relatively close to the Pacific Ocean, this site (roughly 120 feet above mean sea level) is not within an area currently considered susceptible to tsunami hazard.

2.11 **Slope Stability**

A topographic survey of the site was performed by Wallace Group in June 2016 and is part of the base map for our Geotechnical Exploration Map. Site layout plans by KBZ Architects (2016) were also used. The topographic survey shows the location and elevation of the toe and top of slope, which were utilized to establish general slope geometries for our stability analyses by utilizing the locations of the tops and toes of the slope. The survey does not provide detail of gradient changes within the slope or the slope gradient. Geotechnical Cross-Sections A-A′ and B-B′ extend through the site perpendicular to the northern slope and depict the slope geometry assumed for our analyses.

Detailed topography was not available for the lower portion of the slope beyond the City’s property line, except of the location and elevation of the toe of the slope; therefore, this portion of the slope along each cross section line was estimated simply by joining the closest known points. The slope may have a steeper or gentler gradient than shown due to the uneven erosion and differences in soils type exposed. Based on observations in early January 2017, it appears that on the eastern half of the slope, surficial erosion due to recent rains has created a talus of soil on the lower portion of the slope, and created a steeper more vertical slope section on the upper portion of the slope. The western half of the slope is more vegetated and the slope gradient from toe to crest is more even and regular.

Geotechnical Cross Sections A-A′ and B-B′ were analyzed for gross stability in accordance with current standard practice. Cross section locations were chosen based on representative soil profiles and critical locations with respect to slope height, gradient and assumed subsurface conditions. The approximate locations of
the analyzed geotechnical cross-sections are presented on the Geotechnical Map (Plate 1). Geotechnical Cross-Section A-A’ (Plate 2) extends through the center of the proposed Fire Station structure and depicts the slope geometry assumed for our analyses. Geotechnical Cross-Section B-B’ (Plate 2) extends through the western side of the site, where portions of the slope have eroded and retreated, and depicts the slope geometry assumed for our analyses.

The slopes were analyzed using the computer software SLIDE (RocScience, 2015) which performs 2D Limit Equilibrium slope stability analyses capable of analyzing circular and non-circular slip surfaces by a number of analysis methods. For this project, stability analysis was performed using Bishop’s (simplified) method and Spencer’s method, run simultaneously for comparison, for circular failure slip surfaces to evaluate the effect of layered sediments on the stability of the slope. Pseudostatic screening analysis was performed using the SP117A Guidelines for Evaluating and Mitigating Seismic Hazards in California" (CGS, 2008). These guidelines provide methods of screening seismic stability for slopes, which utilize coefficients of horizontal acceleration ($k_h$) based on predicting ground deformation 5cm (two inches) and 15cm (six inches). Factors of Safety and the results of the stability analyses are presented below. A more detailed explanation, of the material parameters and our calculations is presented in Appendix D.

2.11.1 Shear Strength Parameters

Shear strength parameters were derived from laboratory testing performed on samples recovered during our subsurface exploration, in-situ testing and previous reports. Shear strength parameters are summarized in Table 2. Ultimate and peak strengths of the soil were used to analyze the static and pseudo-static stability of the slopes, respectively.
Table 2 – Shear Strength Test Results

<table>
<thead>
<tr>
<th>Layer</th>
<th>Description</th>
<th>Unit Weight $\gamma_{TOT}$ (pcf)</th>
<th>Ultimate Shear Strength</th>
<th>Peak Shear Strength</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>$\gamma_{TOT}$ (pcf)</td>
<td>Cohesion (psf)</td>
<td>$\phi$</td>
</tr>
<tr>
<td>Fill</td>
<td>0 to 5 ft Remolded, 90%</td>
<td>125</td>
<td>0</td>
<td>34</td>
</tr>
<tr>
<td>1</td>
<td>Mixed fine grained, SM-ML</td>
<td>120</td>
<td>132</td>
<td>33</td>
</tr>
<tr>
<td>2</td>
<td>Silty Sand, SM</td>
<td>120</td>
<td>50</td>
<td>33</td>
</tr>
<tr>
<td>3</td>
<td>Silty Sand, SC-SM</td>
<td>120</td>
<td>50</td>
<td>39</td>
</tr>
<tr>
<td>4</td>
<td>Clay and Silt, CL-ML</td>
<td>120</td>
<td>247</td>
<td>25</td>
</tr>
<tr>
<td>5</td>
<td>Well-graded Sand, SW</td>
<td>120</td>
<td>0</td>
<td>36</td>
</tr>
</tbody>
</table>

2.11.2 Factor of Safety

Analysis of the static factor of safety is straightforward, and the standard threshold is $FS \geq 1.5$. Pseudo-static analysis of slopes is subject to applicable guidelines which provide methods and associated acceptable factors of safety. The Kho is then applied as a seismic load on the modeled slope to generate a factor of safety. The calculated factors of safety generated are compared to minimum factors of safety in order to assess the potential for failure for the slope configurations as modeled. The following minimum factors of safety (FS) were considered reasonable or acceptable minimum parameters:

- Static Analysis: minimum $FS = 1.5$
- Pseudo-Static (Seismic) Analysis with a seismic coefficient of $K_h = 0.211$: minimum $FS = 1.0$, with up to 15cm of displacement.

2.11.3 Conditions Analyzed

The existing slope was analyzed to assess whether mitigation of slope stability was needed. The existing slope was calculated to not meet minimum required static factors of safety. Therefore, various mitigation options were evaluated and analyzed based on a cursory assessment of constructability and magnitude of seismic loading. In addition to setback
from the existing unmodified slope, three mitigation options were evaluated:

- piles at the top of the slope,
- piles in between the property line and the top of the slope with a reconstructed slope behind it, and
- piles at the property line extended to proposed finished grade, with backfill behind it to create additional level space.

Each of these options is summarized in the following table.

**Table 3 – Conditions Analyzed**

<table>
<thead>
<tr>
<th>Case</th>
<th>Description</th>
</tr>
</thead>
</table>
| **Mitigation Option 1:**  
Piles at top of slope, no repair of erosion on City owned portion of slope. |  
- Cross-section A-A’ (critical). Piles installed at top of slope, from El. 122. to El. 85  
- Long term condition, erosion of entire slope to angle of repose (2.5:1 h:v). |
| **Mitigation Option 2A and 2B –**  
Piles in between the top of slope and the property line, and a trimmed or reconstructed upper slope at 2:1 h:v |  
- Cross-section A-A’ (critical).  
- Piles installed to min El. 84:  
  **2A - Property Line, at El. 111**, long term condition, possible UPRR removal toe of slope (i.e. removal of slope support).  
  **2B - Intermediate location on slope at El. 117**, long term condition, slope between property line and toe of slope erodes to angle of repose (2.5:1 h:v ). |
| **Mitigation Option 3 –**  
Piles at property line, extended to retain backfill to Finished Grade |  
- Cross-section A-A’ (critical)  
- Piles installed property line and extend to Finished Grade at approximate El. 122.  
- Long term condition, possible UPRR removal toe of slope (i.e. removal of slope support). |

**2.11.4 Results of Slope Stability**

As modeled, the long-term static stability analysis of the northern descending slope below the site yielded calculated static factors of safety (FOS) of 1.27 at Section A-A’ and 1.43 at Section B-B’. This is below the code minimum required FS of 1.5 and therefore mitigation is required. When analyzed for pseudo static conditions under SCEC SP117A Guidelines, the slope meets the minimum screening criteria of 1.0 with a displacement of up to 15cm. Table 4 summarizes the results of the analysis.
Table 4 – Summary of Stability Analyses for Conditions Analyzed

<table>
<thead>
<tr>
<th>Option/Geotechnical Cross Section</th>
<th>Condition Evaluated</th>
<th>Factor of Safety</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Static</td>
</tr>
<tr>
<td>Section A-A' – Circular</td>
<td>Existing, Long term</td>
<td>1.27</td>
</tr>
<tr>
<td>Section B-B’ – Circular</td>
<td>Existing, Long term</td>
<td>1.42</td>
</tr>
<tr>
<td>Section A-A’ - Circular</td>
<td>Short Term, Construction loads (minimum FS 1.25)</td>
<td>1.39</td>
</tr>
<tr>
<td>Option 1– Section A-A’ Piles at top of slope</td>
<td>Long Term, Stabilization Minimum embedment El 84</td>
<td>1.79</td>
</tr>
<tr>
<td>Option 2A and 2B Section A-A’ Piles at property line, no wall</td>
<td>Long Term, Stabilization Minimum embedment El 84</td>
<td>1.53</td>
</tr>
<tr>
<td>Option 3 Section A-A’ Piles at property line, extend to FG El122</td>
<td>Long Term, Stabilization Minimum embedment El 84</td>
<td>1.59</td>
</tr>
</tbody>
</table>

Because the existing slope without modification met the screening criteria for seismically-induced landslides, Newmark analyses for estimating slope deformation during strong seismic shaking was not required or performed. Under the seismic loading criteria selected, the slope meets the minimum screening factor of safety with 15cm deformation and in front of the FS=1.1 line. The structures should be set back from this line, or designed to accommodate displacement up to 15 cm. At Cross-Section A-A’ this setback distance is 16 feet, and at Cross-Section B-B’ this setback distance is 10 feet.
3.0 CONCLUSIONS AND RECOMMENDATIONS

3.1 Summary of Conclusions and Recommendations

This proposed Fire Station No. 10 site is not within a currently designated Alquist-Priolo Earthquake Fault Zone. However, as is the case for most of southern California, strong ground shaking is expected to occur at this site, and these buildings should be designed to resist ground shaking.

The soils at the site consist of undocumented fill and or native soils encountered to depths up to approximately 5 feet below existing grade; possibly deeper locally. Existing undocumented fill should not be used to support new shallow foundations without excavation, replacement in lifts on undisturbed native soils, and proper compaction throughout, as described below.

Based on local historical and recent subsurface exploration data, (State of California 1968, Padre 1999), ground water has not been encountered to depths of 70 feet. On site soils consist of a thin layer of undocumented fill mantling young marine terrace deposits to the depths explored. Due to the potential for ground shaking, we have performed a seismic settlement evaluation based on Boring LB-2 which has SPT blow counts at 2½-foot intervals. Based on our analyses, dry sand settlement has been predicted to occur at this site with a potential for surficial settlement in the range of 1.5 to 2 inches resulting from a strong ground shaking induced by a regional earthquake.

A conventional shallow spread-footings may be utilized to support the structure. Foundations should be designed to tolerate anticipated settlements on the order of 2 inches and differential settlement on-the-order of an inch over 40 feet.

Specific geotechnical design recommendations are provided in the following sections for use in designing this fire station.

3.2 Plans, Specifications, and Construction Review

Because subsurface conditions will vary due to previous construction and demolition on site, we anticipate that while the conditions are expected to be relatively uniform across the site at depth, shallow soil conditions may vary considerably between and beyond our borings (particularly in the area of the old gas station). Based on this and to check that these report recommendations have
been properly implemented, we recommend that a California licensed Geotechnical Engineer be retained to:

1) Review final civil and structural plans and specifications,
2) Observe and document existing fill removal excavations, and
3) Observe and test all structural and ancillary/utility backfill.

In addition, our assumed and/or actual geotechnical conditions can be greatly affected by construction processes and seasonal weather changes. In addition, our conclusions and recommendations in this report have been based on assumed subsurface conditions using a limited number of exploration locations. For these reasons, our geotechnical recommendations are contingent upon a reputable California licensed Geotechnical Engineer providing geotechnical observation and testing services when actual subsurface conditions become known within excavations across all portions of proposed improvement areas during construction.

3.3 Earthwork

All earthwork should be performed in accordance with the Earthwork and Grading Guide Specifications presented in Appendix F, unless specifically revised or amended below or by our future review of project design documents. Site-specific earthwork recommendations are presented in the following subsections:

3.3.1 Preparation

Prior to grading, the site should be cleared of vegetation, trash, significant organic material and debris. Any underground obstructions onsite should be removed (e.g. abandoned utilities, monitoring wells, etc.). Resulting cavities should be properly backfilled and compacted. Efforts should be made to locate any existing utilities. Those utilities should be removed or rerouted where interfering with proposed construction and resulting cavities should be properly backfilled and compacted. In addition, any uncontrolled artificial fill, if encountered, should be excavated from proposed areas of improvements.
3.3.2 Overexcavation and Recompaction

All undocumented fill at the site should be removed within all proposed structural and flatwork areas. Thereafter, and to reduce the potential for adverse differential settlement of proposed structures, we recommend that onsite soil below the proposed buildings be over-excavated and recompressed such that at least 3 feet of compacted fill results below the bottom of proposed foundations. Overexcavation and recompression should extend a minimum horizontal distance of 3 feet from the exterior portions of shallow foundation perimeters, or the thickness of fill that underlies footings, whichever is greater. These preceding recommendations are valid for buildings up to two stories in height.

Areas outside overexcavation limits for buildings planned for asphalt or concrete pavement, flatwork, and/or areas to receive fill should be over-excavated to a minimum depth of 2 feet below proposed pavement subgrade, whichever is lower. Local conditions may require deeper overexcavation (such as areas of former USTs backfilled with non-engineered fill); such areas should be evaluated by the Geotechnical Engineer of Record during grading. If encountered under building footprints, all undocumented fill should be excavated and recompressed.

After completion of the undocumented fill removal, overexcavation, and prior to fill placement, exposed surfaces should be scarified to a minimum depth of 6 inches and moisture-conditioned. After moisture conditioning, exposed surfaces at the bottom of excavations should be moisture conditioned to approximately 2% above optimum moisture and recompressed to a minimum 90 percent relative compaction, relative to the ASTM D 1557 laboratory maximum density.

3.3.3 Fill Placement and Compaction

Encountered onsite sandy silt materials are generally suitable for use as compacted structural fill, provided that proposed fill soils are free of significant organic material, debris and oversized rock (greater-than 6 inches in greatest dimension). Soil to be placed as fill, whether onsite or import material, should be observed and reviewed by the California licensed Geotechnical Engineer of Record; and tested if deemed necessary. Clay, organic and/or contaminated soils should not be
imported to the site. Soils near the surface at our exploration locations were at or near Optimum Moisture content when samples from these locations were tested by us. However, some moisture conditioning of earth materials may be required to achieve adequate compaction.

All fill soil should be placed in thin, loose lifts, sufficiently and uniformly moisture-conditioned to approximately 2% above optimum moisture, and compacted to a minimum of 90 percent relative compaction within proposed building footprints, and 95 percent relative compaction underneath paved areas. Aggregate base should also be compacted to a minimum of 95 percent relative compaction. In all cases for this project, relative compaction should be measured using the ASTM D 1557 modified Proctor laboratory maximum-density test-method.

3.3.4 Utility Trench Backfill

Utility trenches should be backfilled with compacted fill in accordance with Sections 306-1.2 and 306-1.3 of the Standard Specifications for Public Works Construction, (“Greenbook”), current Edition. Otherwise, or as an option, the pipe bedding zone can be backfilled with Controlled Low Strength Material (CLSM) consisting of at least one-and-one-half (1.5) sack of Portland cement per cubic-yard of sand, and conforming to Section 201-6 of the current edition of Greenbook and City of Goleta standards. Specifically, prior to backfilling trenches, pipes should be bedded in and covered with either CLSM or a uniform, granular material that has a Sand Equivalent (SE) of 30 or greater, and a gradation meeting requirements of the pipe manufacturer. Onsite soils are predominantly unsuitable for the pipe zone. Bedding should be placed to 1-foot (0.3 m) over the top of the conduit, and densified by jetting in areas of granular soils, if allowed by the permitting agency. Jetting of the bedding around the conduits should be observed by the Geotechnical Engineer of Record during pipe installation. CLSM should not be jetted.

Above the pipe zone, trenches can be backfilled with excavated onsite soils free of debris, organic and oversized material greater-than 3-inches in largest dimension. As an option, the whole trench can be backfilled with one-sack CLSM in the same manner as presented above as an option for the pipe bedding zone. Oversized rock (cobbles and/or boulders) should either be removed from the alignment, or pulverized for use in backfill
above the pipe zone. Gravel larger than ¾-inch in diameter should be mixed with at least 80-percent soil, by volume which passes the No. 4 sieve. Native soil backfill over the pipe bedding zone should be placed in thin lifts, moisture conditioned, as necessary, and mechanically compacted using a minimum standard of 90 percent relative compaction, relative to the ASTM D 1557 laboratory maximum dry density. Backfill above the pipe zone should not be jetted. In any case, backfill above the pipe zone (bedding) should be observed and tested under the supervision of a California licensed Geotechnical Engineer.

3.3.5 **Surface Drainage**

Surface drainage should be designed to direct water away from foundations and the top of the northern slope toward approved drainage devices. Irrigation of landscaping (if any) should be controlled to maintain, as much as possible, a consistent moisture content sufficient to provide healthy plant growth without over-watering. Water should not be allowed to flow uncontrolled to flood foundation soils. Care should be taken by the civil engineer to ensure the final project grades direct surface drainage away from the slope.

**Landscaping irrigation should not be installed adjacent to the northern slope** and surface drainage should be directed away from the slope. Portions of the slope that are not improved will be especially subject to erosion, and any water that enters into the subsurface near or immediately adjacent to the slope may cause seepage.

Field testing indicated low infiltration rates, when compared to the sandy nature of the marine terrace deposits onsite. Based on the depth and the soils encountered in the borings, it appears that the tests were affected by shallow clayey soils identified in the boring logs at depths of between 10 to 15 feet. Sandy soils are present below this depth, as observed in LB-3A, which collapsed from a depth of 25 feet to 15 feet before the standpipe casing could be installed. If onsite infiltration systems are to be designed, they should be set well away from the slope and deeper infiltration testing should be performed to determine the infiltration rate of the sandy soils below 15 feet.
3.3.6 **Construction Observation**

All grading and earthwork should be observed and testing under the direction of a licensed California Geotechnical Engineer to check that the site is properly prepared, the selected fill materials are satisfactory, and that placement and compaction of the fills has been performed in accordance with our recommendations and the project specifications. Sufficient notification to the Geotechnical Engineer of Record prior to earthwork is essential. Project plans and specifications should incorporate all recommendations contained in the text of this report.

Variations in site conditions are possible and may be encountered during construction. To confirm correlation between subsurface data obtained during our subsurface explorations and actual subsurface conditions encountered during construction, and to observe conformance with the plans and specifications, it is essential that a reputable California licensed Geotechnical Engineer be retained to perform continuous or intermittent review during earthwork, excavation and foundation construction phases.

3.4 **Slope Mitigation Alternatives**

At this time, the existing slope that extends across and parallel to the north site margin has a long term Factor of Safety (FS) of less than 1.5, and requires mitigation. Based on discussions with the project team and the City, the City requested evaluation of the option to install piles along the northern property line and extend it above grade. This has been included as Mitigation Option 3 in this report, and preliminary design parameters are provided following. However, there are design and constructability issues with this alternative that must be addressed with a pile installation contractor and the structural engineer before final design parameters can be provided. Leighton has evaluated two other options, Option 2A and 2B, that also use piles and a reconstructed slope to mitigate slope stability and slope erosion. We provide those recommendations herein.

**Design Considerations:** During project meetings and engineering evaluation, Leighton concluded the following:

- UPRR has stated that it reserves the right to modify the slope within its right of way at any time.
Because of the loose sandy nature of the onsite soil, any exposed slope will need to be protected from erosion.

Based on stability analyses for seismic conditions, the area adjacent to the slope will be subject to seismic deformation on the order of 15cm. If a lower threshold of displacement is preferred, higher seismic design loads will be required.

Piles placed between the top of slope and the property line will need to retain the entire height of the slope that is subject to seismic displacement to finished grade (depending on option selected), from approximately elevation 84 feet (minimum five-foot pile embedment into competent materials) to design grade.

Based on the above design considerations, following are slope mitigation alternatives;

- **Option 1**: Piles at top of existing slope, no mitigation of the descending slope. This is the simplest mitigation for site stability, but without repair of the existing eroded slope, may not be acceptable to the City.

- **Option 2A and 2B**: Piles installed at either the property line (El. 111) with a reconstructed 2:1 h:v slope behind it (Option 2A) or piles installed midslope (El. 117) with a reconstructed 2:1 h:v slope behind, utilizing soil amendments or erosion control in order to improve its long term surficial performance. This option addresses stability and the eroded slope condition.

- **Option 3, City preferred alternative**: Piles installed at the property line (El. 111) and extended to finished grade (estimated at El. 122). This option is feasible, but due to existing topography, this option requires special consideration due to constructability issues. Extending the piles from the current elevation of the property line to the finished grade level will require special construction methods and structural details. Input will be needed from a pile installer. Design loads will require specialized design-specific analysis once a final configuration has been selected.

Recommendations following are provided for mitigation Options 2 and 3.

**3.4.1 Design Earth Pressure**

The analysis assumed continuous 24-inch diameter piles utilizing a mass concrete shear strength derived from a concrete compressive strength of
3,600 psi. The actual shear strength of the pile should be higher if the shear strength of the steel reinforcement (not known at this time) is included in the calculations. The recommended earth pressure parameters designing the piles are as follows:

Table 5 – Active Earth Pressures

<table>
<thead>
<tr>
<th>Options</th>
<th>$K_a$, Active</th>
<th>$K_{eq}$, (kh=0.211)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Option 2A Piles along property line</td>
<td>0.239</td>
<td>0.503</td>
</tr>
<tr>
<td>Option 2B – Piles at mid-slope between top of slope and property line</td>
<td>0.227</td>
<td>0.127</td>
</tr>
<tr>
<td>Option 3 - Pile Wall at Property Line</td>
<td>0.234</td>
<td>0.1</td>
</tr>
</tbody>
</table>

A passive coefficient $K_p$ of 3.5 may be used to calculate the passive resistance in front of the piles. The value should be reduced by one-half if the piles are embedded in sloping ground. A unit weight of 120 pounds per cubic foot (pcf) may be used in calculating the equivalent fluid pressure. A uniform surcharge load of 100 psf should be included in calculating the lateral earth pressure.

3.4.2 Pile Embedment

In order to determine the minimum embedment depth for piles, static and pseudo-static global stability analyses were conducted evaluating the final conditions. Both options assume 24-inch diameter piles along the slope. Based on the analyses, global stability issues requires a minimum pile embedment depth of 5 feet below the lowest adjacent Railroad grade at toe of slope (approximately El. 89), whether the piles are installed near the property line or at the top of slope. Minimum embedment depth is based on stability analyses for final conditions assumed; actual pile embedment will need to be determined based on design by the project structural engineer.

3.4.3 Drilled Soldier Piles

Drilled reinforced piles or soldier piles should be designed by a licensed structural engineer. Due to the sandy, slightly cemented nature of the soils, and potential for future erosion, a continuous line of piles should be installed along the slope for global stabilization. Piles will need to return
into the slope at both western and eastern limits of the stabilized slope. Piles should be spaced horizontally at a minimum of three pile diameters center-to-center, and the space between may be bridged with smaller piles, depending on final design. Groundwater is not expected to be encountered above a depth of 70 feet based on previous explorations in the vicinity (Padre, 1999 and Caltrans, 1968). Pile reinforcement should be designed to allow workers to lower a concrete pump hose down through the reinforcement cage, for proper concrete placement.

Drilling for pile installation must be monitored by Leighton Consulting, Inc. to confirm that piles are properly constructed. Insufficient cleaning of the caisson or soldier pile excavation bottoms and improper placement of concrete may greatly reduce supporting capacity. All proposed construction methods should be reviewed by Leighton Consulting, Inc. prior to the start of construction.

3.4.4 Backfill

Retaining structures planned at the site should be backfilled with granular, non-expansive soil (Expansion Index less than 20). Based on our tests, the use of onsite soils can be considered. Backfill should be compacted to at least 90 percent of the maximum dry density obtained by ASTM Test Method D 1557. Relatively light equipment should be used for backfilling behind retaining structures.

3.4.5 Drainage

All walls should be constructed with a backdrain. The backdrain should be sloped at a minimum of two percent toward an approved non-erosive outlet. The walls should also be waterproofed or at least damp-proofed, depending upon the degree of moisture protection desired. Surface drainage should be designed to direct water away from the wall and toward approved drainage devices.

3.5 Seismic Design Parameters

Moderate to strong ground shaking due to seismic activity is expected at the site during the life span of the project. A site-specific ground motion analysis was performed in accordance with the 2016 California Building Code (CBC) following
the procedures of ASCE 7-10 Publication, Section 21.2, as presented in *Seismic Design Parameters*, Appendix C.

The deterministic and probabilistic seismic hazard analysis was performed using the computer program EZ-FRISK (Risk Engineering, 2011) to estimate peak horizontal ground acceleration (PHGA) that could occur at the site, and to develop design response spectra. Various probabilistic density functions were used in this analysis to assess uncertainty inherent in these calculations with respect to magnitude, distance and ground motion. An averaging of the following next-generation attenuation relationships (NGAs) was used with equal weights to calculate site-specific PHGA and spectra:

- Abrahamson et al. (2014),
- Boore et al. (2014),
- Campbell and Bozorgnia (2014), and
- Chiou and Youngs (2014).

The design response spectrum shown on Figure D-1 is derived from a comparison of probabilistic Maximum Considered Earthquake (MCE) and the 84th percentile of the deterministic MCE. In accordance with the 2016 CBC, peak ground accelerations are estimated based on earthquake ground motion having a 2 percent probability of exceedance in 50 years (ASCE, 2013). The seismic coefficients for the General Procedure were calculated utilizing an interactive program on current United States Geological Survey (USGS) website using ASCE 7-10 reference. The site-specific seismic coefficients are presented in Table 1 below.
Table 6 – 2016 CBC Based Seismic Design Parameters

<table>
<thead>
<tr>
<th>Categorization/Coefficients</th>
<th>Code-Based (1) (2)</th>
<th>Site-Specific (2) (3)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Site Longitude (decimal degrees) West</td>
<td>-119.90555° W</td>
<td>-</td>
</tr>
<tr>
<td>Site Latitude (decimal degrees) North</td>
<td>34.43136° N</td>
<td>-</td>
</tr>
<tr>
<td>Site Class</td>
<td>D</td>
<td>D</td>
</tr>
<tr>
<td>Mapped Spectral Response Acceleration at 0.2s Period, $S_s$</td>
<td>2.891</td>
<td>-</td>
</tr>
<tr>
<td>Mapped Spectral Response Acceleration at 1s Period, $S_t$</td>
<td>1.030</td>
<td>-</td>
</tr>
<tr>
<td>Short Period Site Coefficient at 0.2s Period, $F_a$</td>
<td>1.0</td>
<td>-</td>
</tr>
<tr>
<td>Long Period Site Coefficient at 1s Period, $F_v$</td>
<td>1.5</td>
<td>-</td>
</tr>
<tr>
<td>Adjusted Spectral Response Acceleration at 0.2s Period, $S_{MS}$</td>
<td>2.891</td>
<td>2.891</td>
</tr>
<tr>
<td>Adjusted Spectral Response Acceleration at 1s Period, $S_{M1}$</td>
<td>1.545</td>
<td>1.545</td>
</tr>
<tr>
<td>Design Spectral Response Acceleration at 0.2s Period, $S_{DS}$</td>
<td>1.928</td>
<td>1.767</td>
</tr>
<tr>
<td>Design Spectral Response Acceleration at 1s Period, $S_{D1}$</td>
<td>1.030</td>
<td>1.446</td>
</tr>
</tbody>
</table>

1. All were derived from the USGS web page: [http://earthquake.usgs.gov/designmaps/us/application.php](http://earthquake.usgs.gov/designmaps/us/application.php)
2. All coefficients in units of g (spectral acceleration)
3. See Appendix C for details of the site-specific evaluation.

Based on our borings, the building will be underlain by moderately dense silty sand and sandy silt. Therefore, in accordance with the 2016 CBC, this site should be classified as a Class D site. The results of this analysis also indicate that the Peak Ground Acceleration (PGA_M) for this site is 1.186g based on the USGS General Procedure. The summary reports are included in Appendix C.

### 3.6 Shallow Foundations

Based on our exploration and our experience in the region, and discussions with the design team, shallow foundations may be utilized for the Fire Station main building.

#### 3.6.1 Allowable Bearing

Footings should extend at least 18-inches beneath the lowest adjacent finish grade. At these depths, footings should be founded in engineered fill compacted to a minimum 90% relative density and may be designed for a maximum allowable bearing pressure of 2,500 psf. The allowable pressures may be increased by one-third when considering loads of short
duration such as wind or seismic forces. The minimum recommended width of footings is 12 inches for continuous footings and 18 inches for square or round footings. Footings should be designed in accordance with the structural engineer’s requirements and have a minimum reinforcement of four No. 5 reinforcing bars (two top and two bottom).

This allowable bearing value may be increased by 400 psf per foot increase in depth or width to a maximum allowable bearing pressure of 4,000 psf. These allowable bearing pressures are for total dead load and sustained live loads, and can be increased by one-third for short duration seismic and wind loads. Slab reinforcement should be designed by the structural engineer.

3.6.2 Lateral Load Resistance

Soil resistance available to withstand lateral loads on a shallow foundation is a function of frictional resistance along the base of foundations and passive resistance that may develop as the edge of the mat is forced horizontally into soil. Frictional resistance between the base of the foundation and the subgrade soil may be computed using an allowable coefficient of friction of 0.30 (30% of vertical dead plus sustained loads). Passive resistance may be computed using an equivalent fluid pressure of 400 pounds-per-square-foot per foot embedment (pcf), assuming there is constant contact between the foundation and compacted fill. These allowable passive pressure and coefficient of friction values may be increased by one-third when considering loads of short duration, such as those imposed by wind and seismic forces. These design parameters have also already been reduced by a factor-of-safety of 1.5.

3.6.3 Settlement Estimates

Static settlement of the fire station structures will depend on the loads imposed. However, based on the assumption that single-story structures are proposed, recommendations for foundation design and site preparation, we preliminarily estimate static post-construction settlement of less than 1/2 inch (total) and 1/4-inch (differential) over 40 feet. We can provide refined settlement estimates when we are provided structural loads.
Total dynamically-induced settlement for the site was calculated to be on the order of 1.5 to 2 inches due to dry sand settlement. Differential settlement may be taken at half of total settlement, but is expected to be on the order of an inch over a 40-foot horizontal run at this site. These settlement estimates should be reevaluated by us when foundation plans and actual loads for the proposed structures become available.

3.7 Foundations for Ancillary Improvements

Trash enclosures, masonry screen walls and hose drying racks can all be founded on conventional spread footings bearing on at least 2 feet of properly compacted fill over native soils, as described in Section 3.4 of this report. Specific recommendations for ancillary structures are presented in the following subsections:

3.7.1 Minimum Embedment and Width

Spread footings for trash enclosures and screen walls should have a minimum embedment of 24 inches, or 12-inches for other lightly loaded foundations, all with a minimum width of 12-inches.

3.7.2 Allowable Bearing Capacity

An allowable bearing capacity of 2,500 pounds-per-square-foot (psf) may be used, based on the minimum embedment depths and width, above. This allowable bearing value may be increased by 400 psf per foot increase in embedment-depth or width to a maximum allowable bearing pressure of 4,000 psf. These allowable bearing pressures are for total dead load and sustained live loads, and can be increased by one-third when considering short-duration wind or seismic loads. Footing reinforcement should be designed by the structural engineer, but continuous horizontal reinforcement of wall foundations is recommended to span across variations in subgrade support which may result in localized differential settlement.

3.7.3 Lateral Load Resistance

Frictional resistance between the base of footings and compacted subgrade soils may be computed using a coefficient of friction of 0.30. The passive resistance may be computed using an equivalent fluid
pressure of 400 pounds-per-cubic-foot (pcf), assuming there is constant contact between the footing and properly compacted backfill. This passive resistance is reduced compared to what is recommended for mat foundations, due to diminished lateral capacity in areas of less overexcavation and recompaction. These friction and passive values have already been reduced by a factor-of-safety of 1.5, and can be increased by one-third when considering short-duration wind loads. For spread footings and slabs-on-grade bearing on properly compacted fill over undisturbed native soils, full friction and passive resistance can be combined to resist lateral loads; although some lateral displacement is required to mobilize full passive resistance.

3.8 Pier Foundations (Flagpole and Light Poles)

Although specified in the design, we assume that lightpoles and flagpole are likely to be included in site improvements. These can be supported on cast-in-drilled-hole (CIDH) piles (or piers) on site, depending on the structure height, vertical and lateral loads. For shallow drilled cast-in-place concrete friction piles/shafts, the following design recommendations can be used:

3.8.1 Downward Pier Capacity

Flag and light poles can be supported on drilled cast-in-place concrete pier foundations, if caving sands are controlled by temporary casing or other effective means that do not reduce or eliminate skin friction. Friction parameters presented in this section are based on the assumption that drilling mud will not be used to install these piers. An allowable skin friction of 100 pounds-per-square-foot (psf)/foot can be used for concrete piers cast directly against undisturbed native alluvium and/or properly compacted fill, a minimum of 6 feet deep when discounting the top one foot of penetration. End bearing should not be used due to the caving potential and likelihood of loose sands at the bottom of piers. This allowable skin-friction value can be increased by one-third for wind loading (should not be increased for seismic loading). Piers should have a minimum center-to-center spacing of at least three-pier-diameters on center. Otherwise, a group action reduction in capacity will be required for piers spaced closer than three-pier-diameters.
3.8.2 **Lateral Pier Capacity**

Resistance to lateral loads during short-duration wind and/or seismic ground shaking may be developed by passive soil pressures acting on the side of piers cast against undisturbed soil or properly compacted fill. In accordance with Section 1806A.3.4 of the 2016 California Building Code (CBC); specifically, a passive equivalent fluid pressure of 400 pounds-per-square-foot per foot of embedment (pcf) acting against an isolated pier can be used, not to exceed total passive pressure of 3,000 pounds-per-square-foot (psf). This can be increased by a factor of two to 900 pounds-per-square-foot per foot of embedment (pcf) if designed to accommodate one-inch of deflection. This allowable passive pressure can be increased by one-third for wind loading in combination with static loading (should not be increased for seismic loading). This design allowable passive resistance is based on the assumption that piers penetrate either undisturbed native alluvium or new fill compacted to at least 90-percent of the ASTM D 1557 laboratory maximum density, and is also based on the assumption that ½-inch of lateral deflection at the ground surface is allowable.

3.9 **Portland Cement Type and Corrosion Protection**

Based on results of our laboratory testing (soluble sulfate of ≤425 ppm), concrete structures in contact with onsite soil will have "negligible" exposure to water-soluble sulfates in tested site soil. Therefore, in accordance with ACI 318-05, Table 4.3.1 as referenced in the 2016 California Building Code (CBC) Section 1904, there are no special requirements for concrete in contact with shallow site soils we tested provided the concrete will not be exposed to irrigation water. Import fill soils should be tested for corrosivity and sulfate attack before import to the sites. The site soil is also not corrosive to embedded metals (soluble chloride and pH).

Based on our laboratory test results, tested soils exhibited soil resistivity's as low as 988 ohm-centimeters. Based on generally-accepted resistivity correlations, it appears that corrosion potential for onsite soils may be characterized as "very severely corrosive" for ferrous metals in contact with these soils. Therefore, based on these results, ferrous pipe buried in moist to wet site earth materials
should be avoided by using high-density polyethylene (HDPE), polyvinyl chloride (PVC) and/or other non-ferrous coatings or pipe when possible. Ferrous pipe can also be protected by polyethylene bags, tap or coatings, di-electric fittings or other means to separate the pipe from on-site soils.

A corrosion engineer should be consulted for mitigation measures for corrosive soils against buried metals.

3.10 Preliminary Pavement Design

Based on testing of 1 bulk soil sample on site, the R-value of the subgrade section is greater than R=40, which was used in evaluating pavement sections. We understand that both asphaltic concrete pavement (AC) and Portland Cement Concrete (PCC) will be utilized in the project. We have provided pavement sections for both based on Traffic Indices of TI=5 through 12. Appropriate Traffic Index (TI) data should be verified by the project civil engineer or traffic engineering consultant and design R-value of subgrade soils will need to be verified after completion of rough grading to finalize pavement design. The values for AC pavement sections are based on the current Caltrans Highway Design Manual.

Table 7 – AC Pavement Section based on R-Value =40

<table>
<thead>
<tr>
<th>Traffic Condition</th>
<th>Traffic Index (TI)*</th>
<th>Pavement Section Thickness (inches)**</th>
</tr>
</thead>
<tbody>
<tr>
<td>Arterial - Heavy truck traffic</td>
<td>12.0</td>
<td>AC 7.5 Aggregate Base 13.5</td>
</tr>
<tr>
<td>Arterial - Heavy truck traffic</td>
<td>11.0</td>
<td>7.0</td>
</tr>
<tr>
<td>Arterial - Heavy truck traffic</td>
<td>10.0</td>
<td>6.5</td>
</tr>
<tr>
<td>Arterial - Heavy truck traffic</td>
<td>9.0</td>
<td>5.5</td>
</tr>
<tr>
<td>Local Industrial and Major Collector</td>
<td>8.0</td>
<td>4.5</td>
</tr>
<tr>
<td>Local Commercial and Minor Collector</td>
<td>7.0</td>
<td>4.0</td>
</tr>
<tr>
<td>Local surface streets</td>
<td>6.0</td>
<td>3.5</td>
</tr>
<tr>
<td>Local surface streets</td>
<td>5.0</td>
<td>3.0</td>
</tr>
</tbody>
</table>

*TI range provided by Civil Engineer to include possible Hollister Avenue improvement

**This does not include City of Goleta minimum pavement section thickness.

Based on design procedures outlined in the current Caltrans Highway Design Manual (Table 623.1D), an R-value of at least (≥) 40 for subgrade soils (based on fill material in the upper 5 feet from LB-7 and anticipated import soil variations)
and assumed R-value of 78 for aggregate base, jointed plain concrete pavement (JPCP) sections may consist of the following for the Traffic Index (TI) indicated:

**Table 8 – Caltrans Concrete Pavement Section**

<table>
<thead>
<tr>
<th>Traffic Condition</th>
<th>Traffic Index (TI)</th>
<th>Pavement Section Thickness (inches)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>≤9.0</td>
<td>JPCP</td>
</tr>
<tr>
<td>Heavy truck traffic</td>
<td></td>
<td>9</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Aggregate Base</td>
</tr>
<tr>
<td></td>
<td></td>
<td>6</td>
</tr>
</tbody>
</table>

Caltrans JPCP is **not** reinforced other than dowels/tie-bars at joints. PCA-designed concrete pavement should be adequately reinforced to prevent shrinkage cracking (minimal welded-wire-fabric or equivalent) and have a minimum 28-day flexural strength of 550 pounds-per-square-inch (psi). We recommend that crack-control joints be spaced no more than 12 feet on center each way. If sawcuts are used, they should be a minimum depth of ¼ of the slab thickness and made within 24 hours of concrete placement. We recommend that jointed sections be as nearly square as possible (in plan view).

**3.10.1 General Pavement Recommendations**

Prior to placement of aggregate base, the subgrade should be scarified to a depth of 6 inches and large size rocks (greater than 3 inches) should be removed or broken up. The subgrade should be properly moisture conditioned (±2% of optimum) and compacted to a minimum relative compaction of 95 percent of the laboratory dry density (ASTM Test Method D 1557) and non-yielding. Similarly, aggregate base should be properly moisture conditioned and compacted to a minimum relative compaction of 95 percent and non-yielding under typical construction equipment wheel loads.

Adequate drainage (both surface and subsurface) should be provided such that subgrade soils and aggregate base materials are not allowed to become saturated. All pavement construction should be performed in accordance with the Caltrans **Standard Specifications** (current). Recommended structural pavement materials should conform to the specified provisions in the Caltrans **Standard Specifications** (2010) including grading and quality requirements, shown below:
- **Portland Cement Concrete (PCC)** pavement should conform to Section 40 of the *Standard Specifications*. PCC pavement materials (pavement, structures, minor concrete) should conform to Section 90 of the *Standard Specifications*.

- **Asphaltic Concrete (AC)** pavement should conform to City Standards, Caltrans Standard Specifications, or the Greenbook (current).

- **Class 2 Aggregate Base (AB)** should conform to Section 26 of the *Standard Specifications*. 
4.0 CONSTRUCTION CONSIDERATIONS

4.1 Temporary Excavations

The contractor is responsible for all temporary excavations and trenches excavated at the site and is responsible for design of temporary shoring. Shoring, bracing and benching should be performed by the contractor in accordance with the current edition of the California Construction Safety Orders (see: http://www.dir.ca.gov/title8/sb4a6.html). Existing fill soils conform to OSHA soil Type C. Therefore, if workers are to enter unshored excavations, then temporary cut slopes should be cut no steeper than 1½:1 (horizontal:vertical), for a height no-greater-than 20 feet (California Construction Safety Orders, Appendix B to Section 1541.1, Table B-1). Surcharge loads should not be permitted within a horizontal distance equal to either the height of excavation or 5 feet, whichever is greater, measured from the top of the excavation, unless the excavation is shored or shielded appropriately as described in the following section.

During construction, soil conditions should be regularly evaluated to verify that conditions are as anticipated. The contractor is responsible for providing the "competent person" required by OSHA standards to evaluate soil conditions. Soil types will vary, but Type C soils can be expected at shallow depths (in fill and alluvium). Close coordination between the contractor’s competent person and a California licensed Geotechnical Engineer should be maintained to facilitate construction while providing safe excavations. Spoil piles from excavation(s) and construction equipment should be kept away from the sides of cuts. Surcharge loads should not be permitted within a horizontal distance equal to the height of cut or 5 feet, whichever is greater, measured from the top of the cut, without specific review/analysis by a California-licensed Geotechnical Engineer.

4.2 Drilled Cast-In-Place Concrete Pile/Pier Installation Considerations

All pile or pier installation should be observed by the Geotechnical Engineer of Record in accordance with Section/Table 1705A.8/1705.8 of the 2016 California Building Code (CBC). Drilled cast-in-place concrete piles should be installed in general accordance with Section 205-3.3.2 of the current Standard Specifications for Public Works Construction (Green Book). The Geotechnical Engineer of Record should observe and document all pile or pier drilling, and verify that
anticipated properly compacted new fill and/or undisturbed native sands are encountered in drilled shafts to the depths specified by the Structural Engineer.

Our borings were drilled with hollow-stem-augers, and when drilling Boring LB-3A we encountered caving sands at depths ranging from 15 to 25 feet. When drilling deeper than approximately 15 feet, temporary casing will probably be required to control drilled shaft caving in fill soils and cohesionless native sands and gravel, prior and during concrete placement.

Concrete should be placed by pump or tremie to within 6-feet of the deposited surface, to reduce concrete segregation and to reduce the potential for shaft wall caving. Casing must be withdrawn as concrete is placed, with no-less-than a vertical foot of concrete within the casing measured from the bottom of the casing, at any time until concrete has been placed up to the top-of-pile. This is to keep a head of concrete in the shaft at all times to reduce caving.

4.3 Geotechnical Review During Construction

If and when plans and specifications are revised, then the California licensed Geotechnical Engineer of record should review these documents to evaluate proposed changes on geotechnical recommendations and design parameters. Our conclusions and recommendations presented in this report should be reviewed and verified by the California licensed Geotechnical Engineer of Record during site construction, and revised accordingly, if exposed geotechnical conditions vary from our preliminary findings and interpretations. Recommendations presented in this report are only valid if a reputable California licensed Geotechnical Engineer verifies site conditions during construction. Geotechnical observation and testing should be provided by a reputable California licensed Geotechnical Engineer during all earthwork, and/or when any unusual geotechnical conditions are encountered.
5.0 LIMITATIONS

This report was based in part on data obtained from a limited number of observations, site visits, soil excavations, samples and tests. Such information is, by necessity, incomplete. The nature of many sites is such that differing soil or geologic conditions can be present within small distances and under varying climatic conditions. Changes in subsurface conditions can and do occur over time. Therefore, our findings, conclusions and recommendations presented in this report are based on the assumption that a reputable California licensed Geotechnical Engineer will provide geotechnical observation and testing during construction.

Environmental services were not included as part of this study. This report was prepared for the sole use of the City of Goleta and their design team for use in designing proposed Fire Station No. 10 in accordance with generally accepted geotechnical engineering practices at this time in Southern California. Please refer to ASFE’s Important Information About Your Geotechnical Engineering Report presented in Appendix G of this report.
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APPENDIX A

FIELD EXPLORATION

Prior to subsurface exploration, proposed subsurface exploration locations were cleared by Underground Service Alert (USA). Our subsurface exploration consisted of 7 geotechnical exploratory borings (LB-1 through LB-7) drilled to 55 feet and sampled to 56.6 feet and two infiltration test holes at locations approximately depicted on Plate 1 Geotechnical Exploration Map. Borings and infiltration testing were performed on July 23 and July 24, 2016.

Soils encountered and sampled from our borings were continuously logged in the field by our representative and described in accordance with the Unified Soil Classification System (ASTM D 2488). Both relatively undisturbed California ring-lined soil samples and Standard Penetration Test (SPT) soil samples were obtained at selected depth intervals within the hollow-stem-auger borings, driven with a 140-pound hammer falling 30-inches for both samplers. Blow counts to drive the sampler three 6-inch increments are listed on the boring logs. Shallow bulk soil samples were also collected from our borings. All soil samples were transported to our Irvine geotechnical laboratory for evaluation and appropriate geotechnical testing.

Boring logs are included as part of this appendix. These logs and related information depict subsurface condition only at the location indicated and at the particular date designated on the log. Subsurface conditions at other locations may differ from conditions occurring at each boring location. Passage of time may result in altered subsurface conditions due to possible environmental changes. In addition, any stratification lines on logs represent approximate boundaries between soil types and these transitions may be gradual.