UPDATE OF GEOTECHNICAL ENGINEERING REPORT
HOLLISTER KELLOGG COMMUNITY PARK
GOLETA, CALIFORNIA

PROJECT NO.: 301849-001
APRIL 5, 2018

PREPARED FOR
CITY OF GOLETA
ATTENTION: JOANNE PLUMMER

BY
EARTH SYSTEMS PACIFIC
1731-A WALTER STREET
VENTURA, CALIFORNIA 93003
April 5, 2018

City of Goleta
Attention: JoAnne Plummer
130 Cremona Drive, Suite B
Goleta, CA 93117

Project: Hollister Kellogg Community Park
Goleta, California
Subject: Update of Geotechnical Engineering Report

Introduction
It is proposed to construct a community park (known as Hollister Kellogg Community Park) just north of Hollister Avenue and east of South Kellogg Avenue in Goleta, California. Earth Systems Southern California [now Earth Systems Pacific (Earth Systems)] prepared a Geotechnical Engineering Report (Reference No. 1) in October 2013 addressing the proposed park construction. Current plans by Van Atta Associates (Reference No. 2) indicate the park will include: a multi-use athletic field, a skate plaza, a hand ball court, a pickleball court, a basketball flex court, a bocce ball court, a nature-themed play area, a restroom building, a future storage building, permeable parking, and hardscaping/landscaping.

A representative of this office performed a site visit on March 8, 2018, to observe the existing site conditions in relation to the conditions observed during the preparation of the referenced report. The conditions were similar to those observed in 2013. This letter serves as an update of the referenced report along with the revised recommendations/design values. The referenced Geotechnical Engineering Report is attached to this report.

Revised Seismic Design Values
Although the site is not within a State-designated "fault rupture hazard zone", it is located in an active seismic region where large numbers of earthquakes are recorded each year. Historically, major earthquakes felt in the vicinity of the subject site have originated from faults near or outside the site area. These include the December 21, 1812 "Santa Barbara Region" earthquake, that was presumably centered in the Santa Barbara Channel, the
1857 Fort Tejon earthquake, the 1872 Owens Valley earthquake, and the 1952 Arvin-Teahachapi earthquake.

It is assumed that the 2016 CBC and ASCE 7-10 guidelines will apply for the seismic design parameters. The 2016 CBC includes several seismic design parameters that are influenced by the geographic site location with respect to active and potentially active faults, and with respect to subsurface soil or rock conditions. The seismic design parameters presented herein were determined by the U.S. Seismic Design Maps "risk-targeted" calculator on the USGS website for the jobsite coordinates (34.4369° North Latitude and 119.8197° West Longitude). The calculator adjusts for Soil Site Class D, and for Occupancy (Risk) Category I/II/III.

The calculated 2016 California Building Code (CBC) and ASCE 7-10 seismic parameters typically used for structural design are included in Appendix C and summarized in the table below.

<table>
<thead>
<tr>
<th>Summary of Seismic Parameters (2016 CBC)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Seismic Design Category</td>
</tr>
<tr>
<td>Site Class (Table 20.3-1 of ASCE 7-10 with 2013 update)</td>
</tr>
<tr>
<td>Occupancy (Risk) Category</td>
</tr>
<tr>
<td>Maximum Considered Earthquake (MCE) Ground Motion</td>
</tr>
<tr>
<td>Peak Modified Ground Acceleration – PGA_m</td>
</tr>
<tr>
<td>Spectral Response Acceleration, Short Period – S_s</td>
</tr>
<tr>
<td>Spectral Response Acceleration at 1 sec. – S_1</td>
</tr>
<tr>
<td>Site Coefficient – F_a</td>
</tr>
<tr>
<td>Site Coefficient – F_v</td>
</tr>
<tr>
<td>Site-Modified Spectral Response Acceleration, Short Period – S_MS</td>
</tr>
<tr>
<td>Site-Modified Spectral Response Acceleration at 1 sec. – S_M1</td>
</tr>
<tr>
<td>Design Earthquake Ground Motion</td>
</tr>
<tr>
<td>Short Period Spectral Response – S_DS</td>
</tr>
<tr>
<td>One Second Spectral Response – S_D1</td>
</tr>
</tbody>
</table>

The values presented in the table above are appropriate for a 2 percent probability of exceedance in 50 years. A listing of the calculated 2016 CBC and ASCE 7-10 seismic parameters is attached.

The attached Fault Parameters table lists the significant "active" and "potentially active" faults within a 50-mile (80-kilometer) radius of the subject site. The distance between the site and the nearest portion of each fault is shown, as well as the respective estimated
maximum earthquake magnitudes, and the deterministic mean site peak ground accelerations.

Grading Recommendations
The grading recommendations set forth in the referenced Geotechnical Engineering Report will apply to the proposed construction. Surface draining systems should be designed so that water is not discharged into bearing soils or near the structure. Final site grade could be such that all water is diverted away from the building toward either hardscapes or drain inlets, and is not allowed to pond. In landscape areas adjacent to the building, the 2016 California Building Code (Section 1803.3) requires a minimum gradient of 5% away from the edge of the building foundation for a minimum distance of 10 feet.

Foundation Recommendations
The foundation design recommendations set forth in Reference 1 will apply to the proposed construction. It is our understanding that no retaining walls greater than 6 feet in height will be constructed, therefore, no seismic increment is required for retaining walls that are retaining 6 feet or less measured from the finish grade to top of fill behind the wall.

Conclusions
Based on the reviews of the various data, it appears that the referenced reports, with the exceptions and augmentations provided above, remains applicable to the currently proposed replacement residence. This report shall serve to update the referenced (and attached) reports for a period of one year.

Please call if you have any questions, or if we can be of further service.

Respectfully submitted,

EARTH SYSTEMS PACIFIC

Todd J. Tranby  
Engineering Geologist

Anthony P. Mazzei  
Geotechnical Engineer

Attached: 2016 CBC (ASCE7-10) Seismic Design Parameters  
USGS Summary and Design Maps  
Table 1 - Fault Parameters  
Geotechnical Engineering Report (Reference 1)

Copies: 4 - Client (3 mail, 1 email)  
1 - Earth Systems Office File
2016 California Building Code (CBC) (ASCE 7-10) Seismic Design Parameters

Seismic Design Category | E | Table 1613.5.6 | Table 11.6-2
Site Class | D | Table 1613.5.2 | Table 20.3-1
Latitude: 34.437 N
Longitude: -119.820 W

Maximum Considered Earthquake (MCE) Ground Motion

- Short Period Spectral Response: \( S_S = 2.889 \, g \)
- 1 second Spectral Response: \( S_I = 1.010 \, g \)
- Site Coefficient: \( F_a = 1.00 \)
- Site Coefficient: \( F_v = 1.50 \)
- Site Coefficient: \( S_{MS} = 2.889 \, g = F_a \times S_S \)
- Site Coefficient: \( S_{MI} = 1.515 \, g = F_v \times S_I \)

Design Earthquake Ground Motion

- Short Period Spectral Response: \( S_{DS} = 1.926 \, g = \frac{2}{3} \times S_{MS} \)
- 1 second Spectral Response: \( S_{DI} = 1.010 \, g = \frac{2}{3} \times S_{MI} \)
- \( T_0 = 0.10 \, \text{sec} \)
- \( T_s = 0.52 \, \text{sec} \)
- Site Coefficient: \( S_{DI} / S_{DS} \)
- Site Coefficient: \( S_{MI} / S_{DS} \)
- Site Coefficient: \( F_{PGA} = 1.00 \)

Seismic Importance Factor: \( I = 1.00 \)

Table 11.5-1 Design Earthquake Ground Motion Parameters

<table>
<thead>
<tr>
<th>Period (sec)</th>
<th>Sa (g)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.00</td>
<td>0.770</td>
</tr>
<tr>
<td>0.05</td>
<td>1.321</td>
</tr>
<tr>
<td>0.10</td>
<td>1.926</td>
</tr>
<tr>
<td>0.52</td>
<td>1.926</td>
</tr>
<tr>
<td>0.70</td>
<td>1.443</td>
</tr>
<tr>
<td>0.90</td>
<td>1.122</td>
</tr>
<tr>
<td>1.10</td>
<td>0.918</td>
</tr>
<tr>
<td>1.30</td>
<td>0.777</td>
</tr>
<tr>
<td>1.50</td>
<td>0.673</td>
</tr>
<tr>
<td>1.70</td>
<td>0.594</td>
</tr>
<tr>
<td>1.90</td>
<td>0.532</td>
</tr>
<tr>
<td>2.10</td>
<td>0.481</td>
</tr>
<tr>
<td>2.30</td>
<td>0.439</td>
</tr>
<tr>
<td>2.50</td>
<td>0.404</td>
</tr>
<tr>
<td>2.70</td>
<td>0.374</td>
</tr>
<tr>
<td>2.90</td>
<td>0.348</td>
</tr>
</tbody>
</table>
Design Maps Summary Report

User-Specified Input

Report Title: Hollister Kellogg Park  
Wed March 14, 2018 21:06:22 UTC

Building Code Reference Document: ASCE 7-10 Standard  
(which utilizes USGS hazard data available in 2008)

Site Coordinates: 34.4369°N, 119.8197°W

Site Soil Classification: Site Class D – “Stiff Soil”

Risk Category: I/II/III

USGS-Provided Output

\[
\begin{align*}
S_S &= 2.889 \text{ g} & S_{MS} &= 2.889 \text{ g} & S_{DS} &= 1.926 \text{ g} \\
S_1 &= 1.010 \text{ g} & S_{M1} &= 1.515 \text{ g} & S_{D1} &= 1.010 \text{ g}
\end{align*}
\]

For information on how the SS and S1 values above have been calculated from probabilistic (risk-targeted) and deterministic ground motions in the direction of maximum horizontal response, please return to the application and select the “2009 NEHRP” building code reference document.

For PGA, T_L, C_RS, and C_R1 values, please view the detailed report.

Although this information is a product of the U.S. Geological Survey, we provide no warranty, expressed or implied, as to the accuracy of the data contained therein. This tool is not a substitute for technical subject-matter knowledge.
Design Maps Detailed Report

ASCE 7-10 Standard (34.4369°N, 119.8197°W)

Site Class D – "Stiff Soil", Risk Category I/II/III

Section 11.4.1 — Mapped Acceleration Parameters

Note: Ground motion values provided below are for the direction of maximum horizontal spectral response acceleration. They have been converted from corresponding geometric mean ground motions computed by the USGS by applying factors of 1.1 (to obtain $S_S$) and 1.3 (to obtain $S_1$). Maps in the 2010 ASCE-7 Standard are provided for Site Class B. Adjustments for other Site Classes are made, as needed, in Section 11.4.3.

From **Figure 22-1**

$S_S = 2.889$ g

From **Figure 22-2**

$S_1 = 1.010$ g

Section 11.4.2 — Site Class

The authority having jurisdiction (not the USGS), site-specific geotechnical data, and/or the default has classified the site as Site Class D, based on the site soil properties in accordance with Chapter 20.

<table>
<thead>
<tr>
<th>Site Class</th>
<th>$v_s$</th>
<th>$\bar{N}$ or $\bar{N}_{ch}$</th>
<th>$\bar{s}_u$</th>
</tr>
</thead>
<tbody>
<tr>
<td>A. Hard Rock</td>
<td>&gt;5,000 ft/s</td>
<td>N/A</td>
<td>N/A</td>
</tr>
<tr>
<td>B. Rock</td>
<td>2,500 to 5,000 ft/s</td>
<td>N/A</td>
<td>N/A</td>
</tr>
<tr>
<td>C. Very dense soil and soft rock</td>
<td>1,200 to 2,500 ft/s</td>
<td>&gt;50</td>
<td>&gt;2,000 psf</td>
</tr>
<tr>
<td>D. Stiff Soil</td>
<td>600 to 1,200 ft/s</td>
<td>15 to 50</td>
<td>1,000 to 2,000 psf</td>
</tr>
<tr>
<td>E. Soft clay soil</td>
<td>&lt;600 ft/s</td>
<td>&lt;15</td>
<td>&lt;1,000 psf</td>
</tr>
</tbody>
</table>

Any profile with more than 10 ft of soil having the characteristics:
- Plasticity index $PI > 20$,
- Moisture content $w \geq 40\%$,
- Undrained shear strength $\bar{s}_u < 500$ psf

F. Soils requiring site response analysis in accordance with Section 21.1

See Section 20.3.1

For SI: 1ft/s = 0.3048 m/s 1lb/ft² = 0.0479 kN/m²
Section 11.4.3 — Site Coefficients and Risk–Targeted Maximum Considered Earthquake (MCE\textsubscript{R}) Spectral Response Acceleration Parameters

Table 11.4–1: Site Coefficient F\textsubscript{a}

<table>
<thead>
<tr>
<th>Site Class</th>
<th>Mapped MCE\textsubscript{R} Spectral Response Acceleration Parameter at Short Period</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>S\textsubscript{s} \leq 0.25</td>
</tr>
<tr>
<td>A</td>
<td>0.8</td>
</tr>
<tr>
<td>B</td>
<td>1.0</td>
</tr>
<tr>
<td>C</td>
<td>1.2</td>
</tr>
<tr>
<td>D</td>
<td>1.6</td>
</tr>
<tr>
<td>E</td>
<td>2.5</td>
</tr>
<tr>
<td>F</td>
<td>See Section 11.4.7 of ASCE 7</td>
</tr>
</tbody>
</table>

Note: Use straight–line interpolation for intermediate values of S\textsubscript{s}

For Site Class = D and S\textsubscript{s} = 2.889 g, F\textsubscript{a} = 1.000

Table 11.4–2: Site Coefficient F\textsubscript{v}

<table>
<thead>
<tr>
<th>Site Class</th>
<th>Mapped MCE\textsubscript{R} Spectral Response Acceleration Parameter at 1–s Period</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>S\textsubscript{1} \leq 0.10</td>
</tr>
<tr>
<td>A</td>
<td>0.8</td>
</tr>
<tr>
<td>B</td>
<td>1.0</td>
</tr>
<tr>
<td>C</td>
<td>1.7</td>
</tr>
<tr>
<td>D</td>
<td>2.4</td>
</tr>
<tr>
<td>E</td>
<td>3.5</td>
</tr>
<tr>
<td>F</td>
<td>See Section 11.4.7 of ASCE 7</td>
</tr>
</tbody>
</table>

Note: Use straight–line interpolation for intermediate values of S\textsubscript{1}

For Site Class = D and S\textsubscript{1} = 1.010 g, F\textsubscript{v} = 1.500
**Equation (11.4–1):**

\[ S_{MS} = F_a S_s = 1.000 \times 2.889 = 2.889 \text{ g} \]

**Equation (11.4–2):**

\[ S_{M1} = F_v S_1 = 1.500 \times 1.010 = 1.515 \text{ g} \]

**Section 11.4.4 — Design Spectral Acceleration Parameters**

**Equation (11.4–3):**

\[ S_{DS} = \frac{2}{3} S_{MS} = \frac{2}{3} \times 2.889 = 1.926 \text{ g} \]

**Equation (11.4–4):**

\[ S_{D1} = \frac{2}{3} S_{M1} = \frac{2}{3} \times 1.515 = 1.010 \text{ g} \]

**Section 11.4.5 — Design Response Spectrum**

From **Figure 22-12**

\[ T_L = 8 \text{ seconds} \]

**Figure 11.4–1: Design Response Spectrum**

\[
\begin{align*}
T < T_0 : S_a &= S_{DS} (0.4 + 0.6 T / T_0) \\
T_0 \leq T \leq T_0 : S_a &= S_{DS} \\
T_0 < T \leq T_L : S_a &= S_{D1} / T \\
T > T_L : S_a &= S_{D1} T_L / T^2
\end{align*}
\]
Section 11.4.6 — Risk-Targeted Maximum Considered Earthquake (MCE$_R$) Response Spectrum

The MCE$_R$ Response Spectrum is determined by multiplying the design response spectrum above by 1.5.
Section 11.8.3 — Additional Geotechnical Investigation Report Requirements for Seismic Design Categories D through F

From Figure 22-7\(^4\)

Equation (11.8–1):

\[ \text{PGA}_M = F_{\text{PGA}} \times \text{PGA} = 1.000 \times 1.151 = 1.151 \text{ g} \]

Table 11.8–1: Site Coefficient \(F_{\text{PGA}}\)

<table>
<thead>
<tr>
<th>Site Class</th>
<th>Mapped MCE Geometric Mean Peak Ground Acceleration, PGA</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>PGA ≤ 0.10</td>
</tr>
<tr>
<td>A</td>
<td>0.8</td>
</tr>
<tr>
<td>B</td>
<td>1.0</td>
</tr>
<tr>
<td>C</td>
<td>1.2</td>
</tr>
<tr>
<td>D</td>
<td>1.6</td>
</tr>
<tr>
<td>E</td>
<td>2.5</td>
</tr>
<tr>
<td>F</td>
<td>See Section 11.4.7 of ASCE 7</td>
</tr>
</tbody>
</table>

Note: Use straight–line interpolation for intermediate values of PGA

For Site Class = D and PGA = 1.151 g, \(F_{\text{PGA}} = 1.000\)

Section 21.2.1.1 — Method 1 (from Chapter 21 – Site-Specific Ground Motion Procedures for Seismic Design)

From Figure 22-17\(^5\)

\(C_{\text{RS}} = 0.875\)

From Figure 22-18\(^6\)

\(C_{R_1} = 0.880\)
Section 11.6 — Seismic Design Category

Table 11.6-1 Seismic Design Category Based on Short Period Response Acceleration Parameter

<table>
<thead>
<tr>
<th>VALUE OF S_{ds}</th>
<th>RISK CATEGORY</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>I or II</td>
</tr>
<tr>
<td>S_{ds} &lt; 0.167g</td>
<td>A</td>
</tr>
<tr>
<td>0.167g ≤ S_{ds} &lt; 0.33g</td>
<td>B</td>
</tr>
<tr>
<td>0.33g ≤ S_{ds} &lt; 0.50g</td>
<td>C</td>
</tr>
<tr>
<td>0.50g ≤ S_{ds}</td>
<td>D</td>
</tr>
</tbody>
</table>

For Risk Category = I and S_{ds} = 1.926 g, Seismic Design Category = D

Table 11.6-2 Seismic Design Category Based on 1-S Period Response Acceleration Parameter

<table>
<thead>
<tr>
<th>VALUE OF S_{d1}</th>
<th>RISK CATEGORY</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>I or II</td>
</tr>
<tr>
<td>S_{d1} &lt; 0.067g</td>
<td>A</td>
</tr>
<tr>
<td>0.067g ≤ S_{d1} &lt; 0.133g</td>
<td>B</td>
</tr>
<tr>
<td>0.133g ≤ S_{d1} &lt; 0.20g</td>
<td>C</td>
</tr>
<tr>
<td>0.20g ≤ S_{d1}</td>
<td>D</td>
</tr>
</tbody>
</table>

For Risk Category = I and S_{d1} = 1.010 g, Seismic Design Category = D

Note: When S_{s} is greater than or equal to 0.75g, the Seismic Design Category is E for buildings in Risk Categories I, II, and III, and F for those in Risk Category IV, irrespective of the above.

Seismic Design Category ≡ “the more severe design category in accordance with Table 11.6-1 or 11.6-2” = E

Note: See Section 11.6 for alternative approaches to calculating Seismic Design Category.

References

1. Figure 22-1: https://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/2010_ASCE-7_Figure_22-1.pdf
2. Figure 22-2: https://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/2010_ASCE-7_Figure_22-2.pdf
3. Figure 22-12: https://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/2010_ASCE-7_Figure_22-12.pdf
4. Figure 22-7: https://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/2010_ASCE-7_Figure_22-7.pdf
5. Figure 22-17: https://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/2010_ASCE-7_Figure_22-17.pdf
6. Figure 22-18: https://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/2010_ASCE-7_Figure_22-18.pdf
<table>
<thead>
<tr>
<th>Fault Section Name</th>
<th>Distance (miles)</th>
<th>Avg Dip Angle (deg.)</th>
<th>Avg Dip Direction (deg.)</th>
<th>Avg Rake (deg.)</th>
<th>Trace Length (km)</th>
<th>Fault Type</th>
<th>Mean Mag</th>
<th>Mean Return Interval (years)</th>
<th>Mean Slip Rate (mm/yr)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Mission Ridge-Arroyo Parida-Santa Ana</td>
<td>0.3</td>
<td>0.6</td>
<td>70</td>
<td>176</td>
<td>90</td>
<td>69</td>
<td>B</td>
<td>6.8</td>
<td>0.4</td>
</tr>
<tr>
<td>Red Mountain</td>
<td>4.2</td>
<td>6.8</td>
<td>56</td>
<td>2</td>
<td>90</td>
<td>101</td>
<td>B</td>
<td>7.4</td>
<td>2</td>
</tr>
<tr>
<td>North Channel</td>
<td>6.7</td>
<td>10.7</td>
<td>26</td>
<td>10</td>
<td>90</td>
<td>51</td>
<td>B</td>
<td>6.7</td>
<td>1</td>
</tr>
<tr>
<td>Pitas Point (Upper)</td>
<td>7.2</td>
<td>11.5</td>
<td>42</td>
<td>15</td>
<td>90</td>
<td>35</td>
<td>B</td>
<td>6.8</td>
<td>1</td>
</tr>
<tr>
<td>Santa Ynez (West)</td>
<td>7.4</td>
<td>11.9</td>
<td>70</td>
<td>182</td>
<td>0</td>
<td>63</td>
<td>B</td>
<td>6.9</td>
<td>2</td>
</tr>
<tr>
<td>Oak Ridge (Offshore), west extension</td>
<td>9.0</td>
<td>14.5</td>
<td>67</td>
<td>195</td>
<td>na</td>
<td>28</td>
<td>B'</td>
<td>6.1</td>
<td></td>
</tr>
<tr>
<td>Santa Ynez (East)</td>
<td>11.3</td>
<td>18.2</td>
<td>70</td>
<td>172</td>
<td>0</td>
<td>68</td>
<td>B</td>
<td>7.2</td>
<td>2</td>
</tr>
<tr>
<td>Pitas Point (Lower, West)</td>
<td>11.5</td>
<td>18.5</td>
<td>13</td>
<td>3</td>
<td>90</td>
<td>35</td>
<td>B</td>
<td>7.2</td>
<td>2.5</td>
</tr>
<tr>
<td>Channel Islands Western Deep Ramp</td>
<td>11.6</td>
<td>18.7</td>
<td>21</td>
<td>204</td>
<td>90</td>
<td>62</td>
<td>B'</td>
<td>7.3</td>
<td></td>
</tr>
<tr>
<td>Oak Ridge (Offshore)</td>
<td>15.1</td>
<td>24.3</td>
<td>32</td>
<td>180</td>
<td>90</td>
<td>38</td>
<td>B</td>
<td>6.9</td>
<td>3</td>
</tr>
<tr>
<td>Ventura-Pitas Point</td>
<td>16.1</td>
<td>25.9</td>
<td>64</td>
<td>353</td>
<td>60</td>
<td>44</td>
<td>B</td>
<td>6.9</td>
<td>1</td>
</tr>
<tr>
<td>Pitas Point (Lower)-Montalvo</td>
<td>18.6</td>
<td>29.9</td>
<td>16</td>
<td>359</td>
<td>90</td>
<td>30</td>
<td>B</td>
<td>7.3</td>
<td>2.5</td>
</tr>
<tr>
<td>Los Alamos-West Baseline</td>
<td>19.4</td>
<td>31.1</td>
<td>30</td>
<td>211</td>
<td>90</td>
<td>28</td>
<td>B</td>
<td>6.8</td>
<td>0.7</td>
</tr>
<tr>
<td>Big Pine (West)</td>
<td>19.6</td>
<td>31.5</td>
<td>50</td>
<td>2</td>
<td>na</td>
<td>18</td>
<td>B'</td>
<td>6.5</td>
<td></td>
</tr>
<tr>
<td>Nacimiento</td>
<td>21.1</td>
<td>33.9</td>
<td>66</td>
<td>40</td>
<td>na</td>
<td>113</td>
<td>B</td>
<td>7.1</td>
<td></td>
</tr>
<tr>
<td>Santa Cruz Island</td>
<td>27.2</td>
<td>43.8</td>
<td>90</td>
<td>188</td>
<td>30</td>
<td>69</td>
<td>B</td>
<td>7.1</td>
<td>1</td>
</tr>
<tr>
<td>Pine Mtn</td>
<td>27.3</td>
<td>44.0</td>
<td>45</td>
<td>5</td>
<td>na</td>
<td>62</td>
<td>B'</td>
<td>7.3</td>
<td></td>
</tr>
<tr>
<td>Lions Head</td>
<td>29.7</td>
<td>47.8</td>
<td>75</td>
<td>29</td>
<td>90</td>
<td>41</td>
<td>B</td>
<td>6.7</td>
<td>0.02</td>
</tr>
<tr>
<td>Santa Rosa Island</td>
<td>29.8</td>
<td>48.0</td>
<td>90</td>
<td>1</td>
<td>30</td>
<td>58</td>
<td>B</td>
<td>6.8</td>
<td>1</td>
</tr>
<tr>
<td>Sisar</td>
<td>30.4</td>
<td>48.9</td>
<td>29</td>
<td>168</td>
<td>na</td>
<td>20</td>
<td>B'</td>
<td>7.0</td>
<td></td>
</tr>
<tr>
<td>Big Pine (Central)</td>
<td>30.8</td>
<td>49.6</td>
<td>76</td>
<td>167</td>
<td>na</td>
<td>23</td>
<td>B'</td>
<td>6.3</td>
<td></td>
</tr>
<tr>
<td>South Cuyama</td>
<td>31.6</td>
<td>50.8</td>
<td>33</td>
<td>210</td>
<td>na</td>
<td>48</td>
<td>B</td>
<td>6.8</td>
<td></td>
</tr>
<tr>
<td>Channel Islands Thrust</td>
<td>32.1</td>
<td>51.7</td>
<td>20</td>
<td>354</td>
<td>90</td>
<td>59</td>
<td>B</td>
<td>7.3</td>
<td>1.5</td>
</tr>
<tr>
<td>Santa Cruz Catalina Ridge</td>
<td>36.5</td>
<td>58.8</td>
<td>90</td>
<td>38</td>
<td>na</td>
<td>137</td>
<td>B'</td>
<td>7.3</td>
<td></td>
</tr>
<tr>
<td>San Luis Range (So Margin)</td>
<td>37.1</td>
<td>59.7</td>
<td>45</td>
<td>37</td>
<td>90</td>
<td>64</td>
<td>B</td>
<td>7.1</td>
<td>0.2</td>
</tr>
<tr>
<td>Oak Ridge (Onshore)</td>
<td>37.4</td>
<td>60.2</td>
<td>65</td>
<td>159</td>
<td>90</td>
<td>49</td>
<td>B</td>
<td>7.2</td>
<td>4</td>
</tr>
<tr>
<td>Morales (West)</td>
<td>37.6</td>
<td>60.4</td>
<td>32</td>
<td>49</td>
<td>na</td>
<td>28</td>
<td>B'</td>
<td>6.8</td>
<td></td>
</tr>
<tr>
<td>San Cayetano</td>
<td>37.6</td>
<td>60.6</td>
<td>42</td>
<td>3</td>
<td>90</td>
<td>42</td>
<td>B</td>
<td>7.2</td>
<td>6</td>
</tr>
<tr>
<td>Morales (East)</td>
<td>38.3</td>
<td>61.6</td>
<td>32</td>
<td>14</td>
<td>na</td>
<td>18</td>
<td>B'</td>
<td>6.6</td>
<td></td>
</tr>
<tr>
<td>Malibu Coast (Extension), alt 1</td>
<td>38.8</td>
<td>62.4</td>
<td>74</td>
<td>4</td>
<td>30</td>
<td>35</td>
<td>B'</td>
<td>6.5</td>
<td></td>
</tr>
<tr>
<td>Malibu Coast (Extension), alt 2</td>
<td>38.8</td>
<td>62.4</td>
<td>74</td>
<td>4</td>
<td>30</td>
<td>35</td>
<td>B'</td>
<td>6.9</td>
<td></td>
</tr>
<tr>
<td>Big Pine (East)</td>
<td>39.6</td>
<td>63.7</td>
<td>73</td>
<td>338</td>
<td>na</td>
<td>23</td>
<td>B'</td>
<td>6.6</td>
<td></td>
</tr>
<tr>
<td>Casmalia (Orcutt Frontal)</td>
<td>41.0</td>
<td>66.0</td>
<td>75</td>
<td>206</td>
<td>90</td>
<td>29</td>
<td>B</td>
<td>6.6</td>
<td>0.25</td>
</tr>
<tr>
<td>San Andreas (Big Bend)</td>
<td>42.0</td>
<td>67.6</td>
<td>90</td>
<td>198</td>
<td>180</td>
<td>50</td>
<td>A</td>
<td>7.8</td>
<td>108</td>
</tr>
<tr>
<td>San Andreas (Carrizo) rev</td>
<td>42.8</td>
<td>68.9</td>
<td>90</td>
<td>224</td>
<td>180</td>
<td>59</td>
<td>A</td>
<td>7.8</td>
<td>106</td>
</tr>
<tr>
<td>Simi-Santa Rosa</td>
<td>43.8</td>
<td>70.4</td>
<td>60</td>
<td>346</td>
<td>30</td>
<td>39</td>
<td>B</td>
<td>6.8</td>
<td>1</td>
</tr>
<tr>
<td>Pleito</td>
<td>45.8</td>
<td>73.7</td>
<td>46</td>
<td>181</td>
<td>90</td>
<td>44</td>
<td>B</td>
<td>7.1</td>
<td>2</td>
</tr>
<tr>
<td>Anacapa-Dume, alt 1</td>
<td>48.2</td>
<td>77.6</td>
<td>45</td>
<td>354</td>
<td>60</td>
<td>51</td>
<td>B</td>
<td>7.2</td>
<td>3</td>
</tr>
<tr>
<td>Anacapa-Dume, alt 2</td>
<td>48.2</td>
<td>77.6</td>
<td>41</td>
<td>352</td>
<td>60</td>
<td>65</td>
<td>B</td>
<td>7.2</td>
<td>3</td>
</tr>
<tr>
<td>San Juan</td>
<td>49.7</td>
<td>80.0</td>
<td>90</td>
<td>243</td>
<td>180</td>
<td>68</td>
<td>B</td>
<td>7.1</td>
<td>1</td>
</tr>
</tbody>
</table>

Reference: USGS OFR 2007-1437 (CGS SP 203) Based on Site Coordinates of 34.4369 Latitude, -119.8197 Longitude

Mean Magnitude for Type A Faults based on 0.1 weight for unsegmented section, 0.9 weight for segmented model (weighted by probability of each scenario with section listed as given on Table 3 of Appendix G in OFR 2007-1437). Mean magnitude is average of Ellsworths-B and Hanks & Bakun moment area relationship.
GEOTECHNICAL ENGINEERING REPORT
FOR
PROPOSED HOLLISTER KELLOGG COMMUNITY PARK
HOLLISTER AVENUE AND KELLOGG ROAD
GOLETA, CALIFORNIA

VT-24840-01
OCTOBER 17, 2013

PREPARED FOR
VAN ATTA ASSOCIATES, INC.

BY
EARTH SYSTEMS
SOUTHERN CALIFORNIA
1731-A WALTER STREET
VENTURA, CALIFORNIA
October 17, 2013

Attention: Guillermo Gonzalez
Van Atta Associates, Inc.
235 Palm Avenue
Santa Barbara, CA 93101

Project: Proposed Hollister Kellogg Community Park
Goleta, California
Subject: Geotechnical Engineering Report

As authorized, we have performed a geotechnical engineering study for proposed Hollister Kellogg Community Park to be located north of Hollister Avenue and east of Kellogg Road in the City of Goleta, California. The accompanying Geotechnical Engineering Report presents the results of our research, as well as our conclusions and recommendations pertaining to geotechnical aspects of project design.

We have appreciated the opportunity to be of service to you on this project. Please call if you have any questions, or if we can be of further service.

Respectfully submitted,

EARTH SYSTEMS SOUTHERN CALIFORNIA

Reviewed and Approved

Anthony P. Mazzei
Geotechnical Engineer

Richard M. Beard
Geotechnical Engineer

Copies: 4 - Client
1 - Project File
# TABLE OF CONTENTS

INTRODUCTION ........................................................................................................... 1  
  Project Description ................................................................................................. 1  
  Purpose and Scope of Work ..................................................................................... 1  
  Site Setting .............................................................................................................. 2  
REGIONAL GEOLOGY .................................................................................................. 2  
SUBSURFACE CONDITIONS ......................................................................................... 3  
GEOLOGIC HAZARDS .................................................................................................. 4  
  Ground Shaking ....................................................................................................... 4  
  Fault Rupture .......................................................................................................... 5  
  Landsliding and Rock Fall ......................................................................................... 6  
  Flooding .................................................................................................................... 6  
  Liquefaction, Cyclic Softening and Lateral Spreading ............................................. 6  
  Seismic-Induced Settlement of Dry Sands ............................................................... 9  
DISCUSSIONS AND CONCLUSIONS .......................................................................... 10  
RECOMMENDATIONS .................................................................................................. 11  
  Grading .................................................................................................................... 11  
    Pre-Grading Considerations ................................................................................... 11  
    Rough Grading / Areas of Development ............................................................... 11  
  Excavations .............................................................................................................. 14  
  Utility Trenches ...................................................................................................... 15  
  Conventional Foundations ....................................................................................... 15  
  Slab Foundations .................................................................................................... 17  
  Slab-on-Grade Construction .................................................................................... 19  
  Retaining Walls ...................................................................................................... 20  
  Pavements ................................................................................................................. 22  
  Storm Water Infiltration Design Parameters ......................................................... 23  
ADDITIONAL SERVICES .............................................................................................. 24  
LIMITATIONS AND UNIFORMITY OF CONDITIONS ................................................ 25  
REFERENCES .............................................................................................................. 27
Table of Contents (Continued)

APPENDIX A
Field Study
Vicinity Map
Regional Geology Map (Dibblee)
Site Plan with Boring and Test Hole Locations
Boring Logs
Symbols Commonly Used on Boring Logs
Unified Soil Classification

APPENDIX B
Laboratory Testing
Tabulated Test Results
Individual Test Results
Soil Chemistry Results
Minimum Foundation Design Table

APPENDIX C
2010 California Building Code (CBC) & ASCE 7-05 Seismic Parameters
Table 1 Fault Parameters
Results of Liquefaction Analysis
Results of Lateral Spreading Analysis

APPENDIX D
Percolation Test Data
INTRODUCTION

A. Project Description
This report presents results of a Geotechnical Engineering study performed for proposed Hollister Kellogg Community Park to be located north of Hollister Avenue and east of South Kellogg Road in the City of Goleta, California. It is our understanding that the proposed improvements will include a skate plaza, a restroom building, shade structures, a parking lot with permeable pavement, basketball and handball courts, play areas, picnic areas, walkways, and a multi-use field. It is assumed that the proposed restroom building will be a one-story pre-fabricated building supported on a slab foundation. Structural considerations for building column loads of up to 10 kips with maximum wall loads of 1.0 kip per lineal foot were used as a basis for the recommendations of this report. If actual loads vary significantly from these assumed loads, Earth Systems Southern California should be notified since reevaluation of the recommendations contained in this report may be required.

Since grading plans were not currently available, proposed cuts and fills are unknown at this time. However, as site topography is relatively level, cuts and fills during the majority of the site grading work are anticipated to be minimal (2 feet or less) to provide a level foundation pad with positive site drainage. Excavations for underground utilities and any portion of the proposed skate plaza below grade are not anticipated to exceed 7 feet below final site grade.

B. Purpose and Scope of Work
The purpose of the geotechnical study that led to this report was to analyze the soil conditions of the site with respect to the proposed improvements. These conditions include surface and subsurface soil types, expansion potential, settlement potential, bearing capacity, the presence or absence of subsurface water, infiltration characteristics, and liquefaction potential. The scope of work included:

1. Reconnaissance of the site.
2. Excavating, sampling, and logging of three (3) exploratory soil borings to study soil and groundwater conditions. In addition; excavating, logging and constructing three (3) infiltration test holes.
3. Performing percolation tests to evaluate the infiltration characteristics of the near-surface soils.
4. Laboratory testing of soil samples obtained from the subsurface exploration to determine their physical and engineering properties.
5. Analyzing the geotechnical data obtained.
6. Preparing this report.

Contained in this report are:

1. Descriptions and results of field and laboratory tests that were performed.
2. Discussions pertaining to the local soil and groundwater conditions.
3. Conclusions and recommendations pertaining to site grading and structural design.

C. Site Setting
The proposed park site is located north of Hollister Avenue and east of South Kellogg Road in the City of Goleta, California (see Vicinity Map in Appendix A). A portion of the site is currently occupied by an existing staging/storage yard for City of Goleta’s contractor construction equipment and supplies. The remainder of the site is vacant land with small grass and native trees lining the eastern bank of a seasonal creek that lies to the east. Former building foundations are present near the southern and central portion of the property. The subject site is relatively flat with gentle slope gradients to the east toward the creek and is bounded by South Kellogg Avenue to the west, existing residential lots to the north, and a car dealership to the south.

REGIONAL GEOLOGY

The site lies in the alluviated Goleta Valley, south of the Santa Barbara foothills, in the western portion of the Transverse Ranges geologic province. Numerous east-west trending folds and reverse faults indicative of active north-south transpressional tectonics characterize the region. The ongoing regional compression produces the east-west trending faults which deform early Pleistocene to Tertiary aged marine and non-marine sedimentary bedrock units. The ongoing regional compression has locally resulted in the east-west trending faults. Dibblee maps the More Ranch segment of the Mission Ridge fault system approximately 0.9 mile south of the site. The fault is shown on the attached Regional Geology Map by Dibblee in Appendix A. The site does not lie within any State-mapped hazard zones for fault rupture or landslides.
faults or landslides were observed to be documented in the general area of the proposed construction site.

**SUBSURFACE CONDITIONS**

Based on our borings, artificial fill material was encountered in Boring B-2 and Percolation Test Hole T-1. The artificial fill extended to a depth of approximately 1 foot below existing site grade in Boring B-1, and to a depth of approximately 3 feet in Percolation Test Hole T-1. The artificial fill was underlain by native alluvial soils. With the exception of Boring B-1, the near-surface native alluvial soils encountered beneath the artificial fill and at the ground surface in the remaining borings typically consisted of loose, silty sand to depths ranging from approximately 4 to 10 feet below existing site grades. In Boring B-1, the upper 4 feet consisted of stiff, sandy silt. The near-surface sandy silts and silty sands were underlain by interbedded and discontinuous strata of loose to medium dense, poorly- to well-graded sands; and medium stiff to stiff, clayey to sandy silts to the maximum depths explored. In Boring B-1, a stratum of soft, low plasticity silty clay was encountered between the depths of 17.5 and 22.5 feet below the existing site grade. More detailed descriptions of the encountered subsurface soil conditions are included in the boring logs located in Appendix A.

Testing indicates that the near-surface soils at the site lie in the "low'' expansion range on the attached Minimum Foundation Design Table in Appendix B.

A sample of near-surface soils was tested for pH, resistivity, soluble sulfates and soluble chlorides. Testing indicates that anticipated bearing soils lie within the "negligible" sulfate exposure range of ACI 318-08, Table 4.2.1 (referenced in Chapter 19, Section 1904A.3 of the 2010 California Building Code). Hence, special concrete designs do not appear necessary to combat sulfate attack. A soil resistivity measurement of 8,800 Ohms-cm indicates that the soil is "moderately corrosive" to ferrous metals according to a table in the Los Angeles County Manual for Preparation of Geotechnical Reports (2010). The pH of the near-surface soils was measured to be 7.2. The test results provided in Appendix B should be provided to the project designers for their interpretations pertaining to the corrosivity or reactivity of various construction materials (such as concrete and piping) with the soils.

Perched groundwater was initially encountered at a depth of 16 feet in Boring B-1. Following drilling activities, a stabilized static groundwater level was measured at 20 feet below the existing site grade. Groundwater was encountered at a depth of approximately 16.2 feet below
the existing site grade in Boring B-2, and at a depth of approximately 21 feet below the existing site grade in Boring B-3. It should also be noted that fluctuations in the groundwater levels and soil moisture conditions do occur due to change in seasons, variations in rainfall, irrigation practices, construction impacts, and other factors.

Based on our exploratory borings, the near-surface soils across the site are generally silty sands to sandy silts that have a moderate traffic support capacity when recompacted and used as pavement subgrade. A sample of the near-surface soil was collected from Boring B-3 for R-value testing. The test on the near-surface soil yielded an R-value of 37.

GEOLOGIC HAZARDS

Some common geologic hazards which could impact a site in the Southern California region include ground shaking from regional earthquakes, faulting (or other ground deformation), tsunami, seiche, and landslides.

Ground Shaking
The site is located in Southern California which is within an active seismic area where large numbers of earthquakes are recorded each year. Historically, major earthquakes felt in the vicinity of the subject site have originated from faults outside the area. These include the 1812 Santa Barbara Channel earthquake, 1857 Fort Tejon earthquake, the 1872 Owens Valley earthquake, and the 1952 Arvin-Tehachapi earthquake. The June 25, 1925 Santa Barbara earthquake had an estimated magnitude of 6.8 and is not considered a "major" earthquake. However, it did cause widespread damage around Santa Barbara because of poor construction techniques.

This site, like all other sites in the general area, can be affected by moderate to major earthquakes centered on faults in southern California. An estimate of the seismic shaking that the proposed improvements could experience was made by a calculation (dividing the $S_{50}$ seismic design value by 2.5) as recommended in the 2010 California Building Code. This calculation results in an estimated peak horizontal ground acceleration of about 0.46-g.

It is assumed that the 2010 California Building Code will be applicable to the proposed project. This building code utilizes several seismic design parameters that are primarily influenced by the geographic site location with respect to active and potentially active faults, and with respect to subsurface soil or rock conditions. The seismic design parameters presented herein
were calculated by determining the jobsite coordinates (Latitude 34.4365° North and Longitude 119.8200° West), entering them into the U.S.G.S. Ground Motion Parameter Calculator for Site Class B soils, then entering the calculated "short period" and "one second" spectral responses into a spreadsheet that adjusts for the actual site class of soils (Site Class D). For the Maximum Credible Earthquake, the Short Period Spectral Response ($S_S$) was found to be 1.713 g, and the 1 Second Spectral Response ($S_D$) was found to be 0.668 g. For design purposes, the Short Period Spectral Response ($S_{SP}$) was found to be 1.142 g, and the 1 Second Spectral Response ($S_{D1}$) was found to be 0.668 g. These values are appropriate for a 10 percent probability of exceedance in 50 years. (A listing of the calculated 2010 CBC and ASCE 7-05 Seismic Parameters is presented in Appendix C of this report).

The Fault Parameters table in Appendix C lists the significant "active" and "potentially active" faults within a 63-mile (100-kilometer) radius of the subject site. The distance between the site and the nearest portion of each fault is shown, as well as the respective estimated maximum earthquake magnitudes, and the deterministic mean site peak ground accelerations.

Fault Rupture

A fault is a break in the earth’s crust upon which movement has occurred in the recent geologic past and future movement is expected. As previously mentioned in the Regional Geology section of this report, the east-west trending More Ranch segment of the Mission Ridge fault system mapped by Dibblee and the USGS is located approximately 0.9 mile south of the site. A summary of nearby active faults is presented in the Appendix B under Table 1 Fault Parameters.

The site does not lie within a State of California designated active fault hazard zone. The activity of faults is classified by the State of California based on the Alquist-Priolo Earthquake Fault Zoning Act (1972). An active fault has had surface rupture with Holocene time (the past 11,000 years). A potentially active fault shows evidence of surface displacement during Quaternary time (last 1.6 million years). An inactive fault has no evidence of movement within the Quaternary time.

Because there are no fault traces mapped across or within approximately 0.5 mile of the site, the potential for fault rupture hazard at the proposed construction is considered low.
Landsliding and Rock Fall

The subject site is relatively flat, and not close in proximity to any formations that are prone to landslides and/or rock falls. As a result, it appears that the hazards posed by landsliding and rock fall are considered nil for the subject site.

Flooding

Earthquake-induced flooding types include tsunamis, seiches, and reservoir failure. Due to the inland location of the site, hazards from tsunamis and seiches are considered extremely unlikely. There are no large water reservoirs up gradient that appear to have the capacity to flood the site in the event of an instant failure; therefore, earthquake-induced flooding does not appear to be a potential hazard.

Liquefaction, Cyclic Softening and Lateral Spreading

The site is not located in a State of California or County of Santa Barbara special study zone for liquefaction related settlement.

Earthquake-induced cyclic loading can be the cause of several significant phenomena, including liquefaction in fine sands and silty sands. Liquefaction results in a loss of strength and can cause structures to settle or even overturn if it occurs in the bearing zone. Cyclic softening in clays during earthquakes has resulted in buildings experiencing foundation failure and ground surface deformation similar to that resultant from liquefaction. If liquefaction or cyclic softening occurs beneath sloping ground, a phenomenon known as lateral spreading can occur. Liquefaction and cyclic softening is typically limited to the upper 50 feet of the subsurface soils. There are a number of conditions that need to be satisfied for liquefaction or cyclic softening to occur. Of primary importance is that groundwater, perched or otherwise, usually must be within the upper 50 feet of soils.

Earthquake-induced vibrations can be the cause of several significant phenomena, including liquefaction in fine sands and silty sands. Liquefaction results in a loss of strength and can cause structures to settle or even overturn if it occurs in the bearing zone. Liquefaction is typically limited to the upper 50 feet of soils underlying a site.

Fine sands and silty sands that are poorly graded and lie below the groundwater table are the soils most susceptible to liquefaction. Soils that have $I_c$ values greater than 2.6, soils with plasticity indices (PI) greater than 7, sufficiently dense soils, and/or soils located above the groundwater table are not generally susceptible to liquefaction.
An examination of the conditions existing at the site, in relation to the criteria listed above, indicates the following:

1. Perched groundwater was initially encountered at a depth of 16 feet in Boring B-1. Following drilling activities, a stabilized static groundwater level was measured at 20 feet below the existing site grade. Groundwater was encountered at a depth of approximately 16.2 feet below the existing site grade in Boring B-2, and at a depth of approximately 21 feet below the existing site grade in Boring B-3. Historic depth to water was analyzed using the Department of Water Resources data for the two closest nearby wells. Well 04N28W009K002S is located approximately 1,550 feet northwest of the site and is approximately 19 feet higher in elevation. Well 04N28W009J002S is located approximately 2,170 feet northeast of the site and is approximately 30 feet higher in elevation. Based on records between June 2005 and December 2010, historical depths to groundwater from the ground surface at Well 04N28W009K002S have been as deep as 46.6 feet during December 2009 and as shallow as 29.8 feet in December 2006. Based on records between January 2005 and December 2010, historical depths to groundwater from the ground surface at Well 04N28W009J002S have been as deep as 58.9 feet during December 2009 and as shallow as 41.8 feet in December 2006. Based on the elevation difference between the site and two wells used, historic high groundwater beneath the site is anticipated to be between 10.8 to 11.8 feet below the ground surface. As a conservative measure, a historic high groundwater level of 10 feet was used in the liquefaction analysis.

2. Much of the soil profile consists of loose to medium dense alluvial sands and low plasticity silts.

3. Plasticity index values (PI's) were determined from samples taken from select depth intervals for the fine-grained soils encountered in Boring B-1. Test results indicate that the sample collected at approximately 20 feet exhibits a plasticity index that is greater than 7 and is expected to exhibit clay-like behavior during earthquake cyclic loading. PI's of 3 and 5 were measured for the samples taken at 30 and 41 feet, respectively. These soils may be susceptible to liquefaction during earthquake cyclic loading.

4. Standard penetration tests conducted in the borings indicate that soils within the tested depth are in a variably dense state.

Based on the above, a cyclic mobility analysis was undertaken to analyze the liquefaction potentials of the various soil layers. The analysis was performed using a proprietary
spreadsheet that implemented several equations in general accordance with the methods proposed by Youd and Idriss during NCEER proceedings in 1996 and 1998 (in "Cyclic Liquefaction and Its Evaluation Based on the SPT and CPT"). In the analysis, the design earthquake was considered to be a 7.2 magnitude event, and a site acceleration of 0.46 g was assumed (based on the site-specific $S_{0s}$ value divided by 2.5).

The analysis for soils in the deeper Boring B-1 indicated that approximately 29 feet of the upper 50 feet may be expected to liquefy in the event of a significant nearby earthquake during a period of high groundwater.

The volumetric strain for the potentially liquefiable zones was estimated using a chart derived by Tokimatsu and Seed (in "Proposed Relationship between Cyclic Stress Ratio and Volumetric Strain for Clean Sands", 1987). For the potentially liquefiable zones identified within Boring B-1, the volumetric strain was found to be approximately 5.4 inches.

Based on interpretation of a chart derived by Ishihara (National Academy Press, 1985), the thickness of non-liquefiable soils at the surface is not sufficiently thick to prevent the potentially liquefiable layer below from generating "ground" damage. (The depth to the first zone of potential liquefaction is about 12.5 feet, and the thickness of the first zone of potentially liquefiable soils is about 5 feet.) However, the presence of a 2.5-foot thick pad of reinforced soil beneath the proposed improvements (discussed on Page 10 of this report) should mitigate ground damage, from sand boils and ground cracks.

There is a potential for differential areal settlement suggested by the findings. As mentioned previously, the total liquefaction-related settlement could potentially range up to about 5.4 inches. According to SCEC (1999), up to about half of the total settlement could be realized as differential settlement. As a result, differential settlement could range up to about 2.7 inches at the ground surface.

Based on the above, it is the opinion of this firm that a potential for liquefaction exists at this site. Results of the liquefaction analysis are included in Appendix C of this report. Mitigation should include designing for the diminished lateral capacities and seismically-induced settlements related to liquefaction that may be experienced during seismic events.
Potential ground deformations may arise from cyclic softening for soils with a sensitivity greater than 8. Tests performed on the sample of clay encountered between the depths of 17.5 and 22.5 feet in Boring B-1 had a liquidity index of about 0.65 and a sensitivity of about 4. Hence, strength loss and post-liquefaction consolidation of the clay soil is not thought to be significant concerns. Low plasticity silts were encountered between the depths of 27.5 and 32.5 feet and 41 and 43 feet in Boring B-1. Tests performed on the sample collected at 30 feet had a liquidity index of about 0.17 and a sensitivity of about 1. Hence, strength loss and post-liquefaction consolidation of the silt soil is not thought to be significant concerns.

Due to the close proximity of the proposed park site to the creek that borders the east side of the site; "free face" lateral spreading (Youd, Hansen, Bartlett, 2002) was evaluated for the restroom facility. The SPT \( (N_1)_{60} \) blow count value measured within the potentially liquefiable zone between 12.5 and 17.5 feet in Boring B-1 is less than 15. The thickness of this potentially liquefiable layer is about 5 feet, or about 1.5 meters. The potential lateral spreading was analyzed in accordance with procedures developed by Youd, Hansen and Bartlett (2002). In the analysis, it was assumed that free face ratio (i.e., height of free face/distance to the free face) was 4 percent, the average fines content of the soil in zones with SPT \( (N_1)_{60} \) less than 15 was assumed to be 5 percent, and the mean grain of the soil in zones with SPT \( (N_1)_{60} \) less than 15 was 0.5 millimeters. Based on these assumptions, the cumulative displacement was calculated to be about 5 feet if this zone were to liquefy. (Calculations are included within Appendix C of this report.) Mitigation for this phenomenon is discussed in the Rough Grading Recommendations section of this report.

**Seismic-Induced Settlement of Dry Sands**

Sands tend to settle and densify when subjected to earthquake shaking. The amount of settlement is a function of relative density, cyclic shear strain magnitude, and the number of strain cycles. A procedure to evaluate this type of settlement was developed by Seed and Silver (1972) and later modified by Pyke, et al (1975). Tokimatsu and Seed (1987) presented a simplified procedure that has been reduced to a series of equations by Pradel (1998).

Proposed construction at this site will include a minimum 2.5-foot thickness of reinforced soil pad prepared underlying the proposed structural improvements as recommended in Section A of this report. The near-surface sandy soils underlying the 2.5-foot thickness of reinforced soil pad are predominantly silty sands and have moisture contents ranging from approximately 7 to 14 percent. Based on these data, it appears unlikely that there is a potential for seismic-induced settlement of dry sands that will adversely affect the structures.
DISCUSSIONS AND CONCLUSIONS

The site is suitable for the proposed improvements from a Geotechnical Engineering standpoint provided that some sort of mitigation is carried out for the estimated differential ground subsidence in order to mitigate the threat of structural damage posed by potentially liquefiable deposits.

Methods may consist of structural mitigation measures or ground improvement. For habitable buildings, structural mitigation is commonly acceptable where total seismic-induced settlement of 4 inches or less is predicted, whereas ground modification may be required where the predicted total earthquake-induced settlement exceeds 4 inches. Methods to mitigate the earthquake-related ground movement hazard may include ground improvement (compaction grouting, in-situ soil mixing, stone columns, etc.) to reduce the susceptibility of the soil to liquefaction, or structural measures such as ridged mat foundations or deep pile foundations that extend down to firm soil below the liquefiable soil. Detailed recommendations for deep ground improvement or deep foundations can be provided if requested.

If the foundations for the proposed structures can be designed to accommodate the anticipated total and differential ground settlements and localized loss of ground support and ground improvement of the underlying liquefiable soils is not desired, Earth Systems Southern California recommends that the structures be supported by a relatively rigid foundation system such as a structural mat slab. To minimize the propagation of earthquake-induced ground damage up to the structure foundation and to minimize differential settlement, Earth Systems Southern California recommends a pad of compacted soil reinforced with "geogrid" beneath the structural mat slab. Recent studies indicate that use of relatively stiff foundation systems and slabs and use of reinforced soil beneath structures can be effective in minimizing the hazards associated with earthquake-related ground disturbance (Bray, 2001). The use of the recommended geogrid reinforced pad beneath the proposed restroom facility will help to reduce the differential settlements should a design level earthquake occur, but it will not eliminate or completely mitigate them.
RECOMMENDATIONS

Based upon field exploration, laboratory testing, interpretation of the data, and past experience, the following recommendations should be incorporated into site preparation, design, and construction of the proposed improvements. These conclusions and recommendations shall be considered preliminary until validated by observations made and in-situ testing conducted during construction.

A. Grading

1. Pre-Grading Considerations
   a. Plans and specifications should be provided to Earth Systems Southern California prior to grading. Plans should include the grading plans, foundation plans, and foundation details.
   b. Roof draining systems for the proposed restroom facility and shade structures should be designed so that water is not discharged into bearing soils or near the structures. Final site grade could be such that all water is diverted away from the structures toward either hardscapes or drain inlets, and is not allowed to pond. In landscape areas adjacent to the residence the 2010 California Building Code (Section 1803.3) requires a minimum gradient of 5 percent away from the edge of the residence foundation for a minimum distance of 10 feet.
   c. It is recommended that Earth Systems Southern California be retained to provide Geotechnical Engineering services during site development and grading, and foundation construction phases of the work to observe compliance with the design concepts, specifications and recommendations, and to allow design changes in the event that subsurface conditions differ from those anticipated prior to the start of construction.

2. Rough Grading / Areas of Development
   a. Grading at a minimum should conform to Appendix J of the 2010 California Building Code (CBC) and with the recommendations of the Geotechnical Engineer of Record during construction. Where the recommendations of this report and the cited section of Title 24 are in conflict, the Owner should request clarification from the Geotechnical Engineer.
b. All vegetation, loose fill, overly dry soil and overly wet soil, trash, pavements, abandoned underground utilities, and other debris should be removed from the proposed grading areas. All stripping and debris should be removed from the site in order to preclude their incorporation in site fill or remedial excavation backfill. Depressions resulting from such removals should have debris and loose soils removed and filled with suitable soils placed as recommended below.

c. Although not encountered at the time of our investigation, any underground tanks or other similar substructures present within the limits of construction should be removed in their entirety including any liquids or sediment remaining at the bottom of the pits or tanks. Any brick, concrete or steel lining should be completely removed. The void resulting from removal of the pits or tanks should be backfilled with suitable soils placed as recommended below.

d. To minimize the propagation of liquefaction-induced ground damage to the proposed restroom facility, shade structures and skate plaza and to minimize differential settlement, Earth Systems Southern California recommends the following:

- Native soils beneath the proposed improvements should be excavated a minimum of 2 feet below the bottom of the foundations. Remedial excavations should be performed to a distance of at least 5 feet laterally beyond the outside edge of the foundation element. The base of the remedial excavation should be a relatively level elevation. Structural plans and details should be checked carefully during grading to establish the actual bottom of foundation elevations in the field.

- The bottom of the remedial excavation should be scarified to a depth of 6 inches, uniformly moisture conditioned to near optimum moisture content, and compacted to achieve a relative compaction of between 90 percent of the ASTM D1557 maximum dry density. **Compaction of the prepared subgrade should be verified by testing.**

- The excavated soils may be reused to backfill the remedial excavations provided they are processed to remove any deleterious materials and debris, and are properly moisture conditioned and compacted. During replacement of the excavated soils in the remedial excavations, and recompaction of the scarified soils, the soils should be moisture conditioned to near optimum moisture content and be uniformly compacted to achieve a relative compaction of between 90 percent of the ASTM D1557 maximum dry density using mechanical compaction equipment. To aid in the compaction operation, fill should be placed
in lifts not exceeding 6 inches compacted thickness. **Compaction should be verified by testing.** Additional fill lifts should not be placed if the previous lift did not meet the required relative compaction or if soil conditions are not stable.

- To minimize the propagation of liquefaction-induced ground damage up to the structure and to minimize differential settlement, the fill may be reinforced with "geogrids" to create a pad of reinforced soil beneath the proposed improvement. The reinforcing material should consist of geogrid (Tensar TriAx TX140 or equivalent). We recommend that a layer of geogrid should be placed on the prepared subgrade at the bottom of the overexcavation and in the engineered fill at a depth of 1 foot below the bottom of the footing. The geogrid should be installed in accordance with the manufacturer's recommendations. Special care should be exercised to place any underground utilities above the geogrid where practical or to splice geogrid reinforcement over utility trenches if it is necessary to place utilities below the geogrid.

e. Following clearing operations, we recommend that the upper 12 inches in areas to support pavement, concrete flatwork, and hardcourt be removed and replaced with engineered fill. The zone of overexcavation should extend laterally at least 5 feet beyond the perimeter of the proposed improvements. If soft or yielding soils are exposed by this processing, excavation should continue until stiff, non-yielding soils are encountered. The depth and extent of required overexcavations should be approved in the field by the Geotechnical Engineer or his representative prior to placement of fill or improvements.

f. After overexcavation has been achieved, the exposed subgrade should be scarified to a depth of 6 inches; uniformly moisture conditioned to near optimum moisture content, and compacted to achieve a relative compaction of between 90 percent of the ASTM D1557 maximum dry density. **Field density tests should be taken to verify compaction of the prepared subgrade in these areas.**

g. Engineered fill should be placed in a series of horizontal layers not exceeding 8 inches in loose thickness, uniformly moisture-conditioned to near optimum moisture content, and compacted to achieve a minimum relative compaction of 90 percent of the ASTM D1557 maximum dry density. Additional fill lifts should not be placed if the previous lift did not meet the required relative compaction or if soil conditions are not stable. Discing, tilling, and/or blending may be required to uniformly moisture-condition soils used for engineered fill. **The upper 12 inches**
of pavement subgrades should be compacted to achieve a minimum relative compaction of 95 percent of the ASTM D1557 maximum dry density.

h. The Geotechnical Engineer’s representative should review the site grading prior to scarification of the bottom of the remedial excavations. Local variations in soil conditions may warrant increasing the depth of remedial excavation. Any deeper areas of loose soils should be removed and be replaced as compacted, engineered fill.

i. Backfill around or adjacent to confined areas (i.e. interior utility trench excavations, etc.) may be performed with a lean sand/cement slurry (maximum 28-day compressive strength of 200 psi) or "flowable fill" material (a mixture of sand/cement/fly ash). The fluidity and lift placement thickness of any such material should be controlled in order to prevent "floating" of any "submerged" structure. Alternatively gravel mat be used, subject to approval by the Geotechnical Engineer and the City official.

j. Final site grades should be designed and constructed so that all water is diverted away from and not allowed to pond on or near the proposed improvements. Drainage devices should be constructed to divert drainage from the project site.

k. It is recommended that Earth Systems Southern California be retained to provide geotechnical engineering services during the grading, excavation, and foundation phases of development. This continuity of services will allow for the geotechnical review of the design concepts and specifications relative to the recommendations of this report and will more readily allow for design changes in the event that subsurface conditions differ from those currently anticipated.

B. **Excavations**

1. Excavations at the site will typically encounter silty to poorly-graded sands and silts. These materials should be easily excavated with conventional earthmoving equipment.

2. Temporary unshored, unsurcharged, open excavations that are free of seeps and less than 10 feet deep in the **drained soils** may be cut at least 1.5H:1V (horizontal to vertical) or flatter provided the adjacent ground is not subject to surcharge loading. If excavations dry out, sloughing will occur. No excavation should be made within a 1:1 line projected downward from the outside edge at the base of any existing footing or slab.

3. During the time excavations are open, no heavy grading equipment or other surcharge loads (i.e. excavation spoils) should be allowed within a horizontal distance
from the top of any slope equal to the depth of the excavation (both distances measured from the top of the excavation slope).

4. Adequate measures should be taken to protect any structural foundations, pavements, or utilities adjacent to any excavations.

5. All open cuts should be in compliance with applicable Occupational Safety Health Administration (OSHA) regulations (California Construction Safety Orders, Title 8) and should be monitored for evidence of incipient instability. Standard construction techniques should be sufficient for temporary site excavations. Project safety is the responsibility of the Contractor and the Owner. Earth Systems Southern California will not be responsible for project safety.

C. Utility Trenches

1. Utility trench backfill should be governed by the provisions of this report relating to minimum compaction standards. In general, on-site service lines may be backfilled with native soils compacted to achieve a minimum relative compaction of 90 percent of the ASTM D1557 maximum dry density. Backfill of offsite service lines will be subject to the specifications of the jurisdictional agency or this report, whichever is greater.

2. Compaction criteria for trench backfill above the bedding zone may be decreased to 85 percent relative compaction in landscape areas at least 5 feet beyond structural improvements, except in areas overlain by pavements, sidewalks, or other hardscapes. In landscape areas overlain by pavements, sidewalks, or other hardscapes, we recommend that the trench backfill be compacted to a minimum of 90 percent relative compaction to within 1 foot of the finished subgrade surface. The upper 1 foot should be compacted to 95 percent relative compaction in areas to receive asphalt concrete pavement.

3. Backfill operations should be observed and tested by the Geotechnical Engineer to monitor compliance with these recommendations.

4. Jetting should not be utilized for compaction in utility trenches.

D. Conventional Foundations

It is recommended that any building or structure constructed on this site be designed to at least the minimum standards of the 2010 edition of the California Building Code (CBC). Relevant seismic design parameters per the CBC are presented above.
1. To minimize the propagation of liquefaction-induced settlement damage to the structure foundation and to minimize differential settlement, we recommend that conventional shallow spread foundations for the proposed shade structures and retaining walls should be supported by a minimum 2.5-foot thickness of reinforced soil prepared as recommended in Section A of this report. [6 inches of compacted prepared subgrade and 2 feet of reinforced soil].

2. Excavations for foundations should be cleaned of all loose or unsuitable soils and debris prior to placement of concrete. Soil generated from the foundation excavations should not be placed below the floor slab unless properly moisture conditioned and compacted, and only after the area to receive fill has been properly prepared and approved.

3. Continuous and isolated spread foundations for the proposed structures founded in the recommended compacted soil pad may be proportioned for the following values:
   a. **Design Values:** An allowable "net" bearing capacity of 1,750 pounds per square foot (psf) can be utilized for dead and sustained live loads. This value includes a minimum safety factor of three, and may be increased by one-third when transient loads (such as wind and seismic forces) are included.
   b. Continuous spread foundations should be embedded a **minimum** of 15 inches below adjacent grade and be a **minimum** of 12 inches in width. Isolated interior spread foundations should be embedded a **minimum** of 12 inches below adjacent grade and be a **minimum** of 12 inches in width. Actual depth, width, and reinforcement requirements for continuous foundations should be determined by the project Structural Engineer.
   c. The allowable bearing capacity for conventional shallow spread foundations may be increased by 200 psf for each additional 12 inches of foundation depth. The allowable bearing capacity should not exceed 3,000 psf to keep estimated settlements within allowable limits. It should be noted that if a reinforced soil pad beneath the foundations is used to minimize the propagation of liquefaction-induced settlement damage to the structure foundation and to minimize differential settlement, deepening the foundation will require additional removal and replacement in order to provide the recommended 2.5-foot thick pad.

4. Resistance to lateral loading may be provided by friction acting along the foundation base. A coefficient of friction of 0.35 may be used for concrete foundations on
compacted engineered fill and may be used with dead loads. This value includes a safety factor of 1.5.

5. Additional resistance to lateral loading may be provided by passive earth pressure acting against the sides of foundations or grade beams. Passive pressure may be taken as 350 Z psf, where Z = Depth (in feet) into compacted engineered fill. In passive pressure calculations, the upper 1 foot of soil should be subtracted from the depth, Z, unless confined by pavement or slab. The maximum passive pressure used for design should not exceed 4,000 psf. The resisting pressure provided is an ultimate value. An appropriate factor of safety should be used for design calculations (minimum of 1.5 recommended). Frictional and passive resistance to lateral forces may be combined without further reduction.

E. Slab Foundations

Earth Systems Southern California recommends that a relatively rigid foundation system such as a structural slab be used for the proposed restroom facility to minimize the propagation of liquefaction-induced ground damage up to the structure foundation and to minimize differential settlement.

1. The slab foundation for the proposed restroom facility should be supported by a minimum 2.5-foot thickness of reinforced soil prepared as recommended in Section A of this report. [6 inches of compacted prepared subgrade and 2 feet of reinforced soil].

2. An allowable "net" bearing capacity of 1,000 pounds per square foot (psf), for loads distributed over the full footprint of the foundation, may be utilized for dead and sustained live loads for design of the mat foundation. An allowable "net" bearing capacity of 1,750 psf may be used for thickened edges or other concentrated load areas. These values include a safety factor of at least 3.0 may be increased by one-third when considering transient loads such as earthquake or wind forces.

3. A modulus of subgrade reaction ("k_1" value) of 100 pounds per square inch per inch (psi/in for standard 30-inch square bearing plate) may be used provided the foundation subgrade (i.e., reinforced soil pad) is prepared as recommended in Section A of this report. If gravel or crushed rock is used to construct the full thickness of the reinforced pad beneath the slab foundation, a modulus of subgrade reaction of 150 psi/in may be used. The value k_1 reflects a 1-square foot area and must be
appropriately corrected for a loaded rectangular area of width B and length, m x B, using the formula: \( k_s = (k_1)(m+0.5)/1.5m \).

Where:

\[ k_1 = \text{coefficient of subgrade reaction for 1 foot square plate (psi/in)} \]

\[ B = \text{width beneath column or bearing wall, in feet, where stresses are imposed on the ground} \]

A value of B should be assumed to estimate the \( k_s \) value in the initial structural analysis. Then, the calculated B value (from the initial structural analysis) should be used to re-calculate the \( k_s \) value. Additional structural analyses (iterations) should be made using re-calculated \( k_s \) values in the same manner, as appropriate, until the B value calculated from the structural analysis is consistent with the B value used to calculate \( k_s \).

4. The actual depth, width, and reinforcement requirements for the slab foundation should be specified by the project Structural Engineer.

5. Resistance to lateral loading may be provided by friction acting along the slab foundation base. A coefficient of friction of 0.35 may be used for concrete foundations on compacted engineered fill and may be used with dead loads. If gravel or crushed rock is used to construct the reinforced pad beneath the mat, a coefficient of friction of 0.45 may be used. These values include a safety factor of 1.5.

6. Additional resistance to lateral loading may be provided by passive earth pressure acting against the sides of foundations or grade beams. An equivalent fluid weight (EFW) of 350 Z psf may be used for passive pressure, where Z = Depth (in feet) into the reinforced pad below the finished ground elevation. In passive pressure calculations, the upper 1 foot of soil should be subtracted from the depth, Z, unless confined by pavement or slab. The resisting pressure provided is an ultimate value. An appropriate factor of safety should be used for design calculations (minimum of 1.5 recommended).

7. The subgrade for the slab foundation should be cleaned of all loose or unsuitable soils and debris prior to placement of concrete. Soil generated from the foundation excavations should not be placed below the mat slab unless properly moisture conditioned and compacted.
8. In addition to the seismically induced settlement previously discussed the slab foundation will undergo elastic and consolidation settlements. Based on a slab width of 20 feet and an allowable bearing pressure of 1,000 psf, the estimated total elastic settlement under static conditions (not including earthquake) would be on the order of 1.5 inches. Should actual loading conditions result in the contact pressure used in our calculation; the estimated total elastic settlement will be less. (Example: 5000 psf will result in an estimated total elastic settlement of 0.8 inch.) The estimated total consolidation settlement for the clay stratum between the depths of approximately 17.5 and 22.5 feet in Boring B-1 would be on the order of 0.5 inch. Differential settlement should be less than one-half the total settlement. The use of the recommended geogrid reinforced pad beneath the proposed restroom facility will help to reduce the differential settlements, but it will not eliminate or completely mitigate them.

F. Slab-on-Grade Construction

1. Concrete slab-on-grade construction at existing site grade should be supported by compacted soils prepared as recommended in Section A of this report.

2. A minimum of 4 inches of compacted granular material (sand or gravel) should be placed over the finished compacted subgrade prior to placing concrete. This granular material should be moisture conditioned to near optimum moisture content and uniformly compacted using mechanical compaction equipment.

3. Where dampness of floor slabs is to be minimized, the slabs should be constructed on a minimum 4-inch-thick layer of capillary break material covered with a high quality vapor retarder. The capillary break material should be free-draining, clean gravel or rock such as No. 4 by 3/4-inch pea gravel or permeable aggregate complying with Caltrans Standard Specifications, Section 68, Class 1, Type B. A 2-inch-thick protective cover (blotter) of clean sand should be placed over the vapor retarder. The vapor retarder should have a minimum thickness of 15 mils, a permeance as tested before and after mandatory conditioning (ASTM E 1745, Section 7.1.2 – 7.1.5) of less than 0.01 perms [grains/(ft² x hr x inHg)], and comply with the ASTM E 1745 Class A requirements. Vapor retarders having these properties are commonly referred to as "vapor barriers". The designer of record may omit the blotter at their discretion when a concrete with a water-cement ratio of 0.45 or less is specified. The vapor retarder should be constructed in accordance with ASTM E 1643-09 using material which meets ASTM E 1745.
4. Reinforcement of slab-on-grade construction is contingent upon the Structural Engineer's recommendations and the expansion potential of the supporting soils. An expansion index test was performed on a bulk soil sample of the near-surface soils in accordance with ASTM D 4829. The sample yielded an Expansion Index value of 23, which falls within the "low" expansion potential range. This material will most likely be used for engineered fill beneath any slab-on-grade construction at existing site grade, and possibly for construction of the reinforced soil pad beneath the foundations. Based on re-handling of the excavated material onsite, the Expansion Index could vary at the finished subgrade elevation from that tested. Additional testing may be necessary upon completion of site grading operations to reevaluate the Expansion Index of the supporting soils.

5. Actual slab designs should be provided by the Structural Engineer, but the reinforcement and slab thicknesses he recommends should not be less than the criteria set forth in Minimum Foundation Design Table for the appropriate expansion range.

6. Exterior concrete slabs (i.e., sidewalks, hardcourts, etc.) should be constructed over 4 inches of compacted granular material (sand or gravel) over 12 inches of engineered fill as recommended in Section A of this report. The final design exterior slab thickness and reinforcement should be provided by the Project Structural Engineer.

7. Based on Minimum Foundation Design Table for the "low" expansion range, subgrade soils for all any concrete flatwork should be moisture conditioned to 3 percent over the optimum moisture content to a depth of at least 18 inches within 24 hours prior to placement of concrete. Measures should be taken to maintain the required moisture content until concrete is placed. Actual depths of pre-moistening will be dependent upon the actual Expansion Index of the subgrade soils to be determined following completion of site grading operations.

G. Retaining Walls

1. The sidewalks of the skate plaza may act as retaining walls for portions that extend below the finished grade. Active earth pressures may be used for design of unrestrained retaining walls where the top of the wall is free to translate or rotate. To develop active earth pressures, the walls should be capable of deflecting by at least 0.004H (where H is the height of the wall). At-rest earth pressures should be used for design of retaining walls where the wall top is restrained such that the deflections required for development of active soil pressures cannot occur or are undesirable.
2. Cantilever walls retaining native soils and/or engineered fill may be designed for active or at-rest lateral earth pressures for various backfill slopes using the following equivalent fluid unit weights. The lateral earth pressures presented in the table below assume the wall backfill is drained (no hydrostatic forces acting on the wall) and no traffic or other surcharge loads are applied within a distance of one-half the wall height.

### EQUIVALENT FLUID UNIT WEIGHT (pcf)

<table>
<thead>
<tr>
<th>Backfill Slope</th>
<th>Active Conditions</th>
<th>At-Rest Conditions</th>
</tr>
</thead>
<tbody>
<tr>
<td>Level</td>
<td>41</td>
<td>62</td>
</tr>
<tr>
<td>3H:1V</td>
<td>47</td>
<td>76</td>
</tr>
<tr>
<td>2H:1V</td>
<td>51</td>
<td>83</td>
</tr>
</tbody>
</table>

3. The lateral earth pressures should be applied to a plane extending vertically upward from the base of the heel of the retaining wall to the ground surface. Lateral pressures for backfill slopes other than those given above can be estimated by interpolation.

4. The lateral earth pressure to be resisted by retaining walls should be increased to allow for surcharge loads. The surcharge considered should include the loads from any structures or vehicle traffic within a distance approximately equal to the height of the retaining wall.

5. In addition to the active or at-rest and surcharge lateral soil pressures, retaining walls should be designed to resist additional seismic earth pressures due to earthquake loading. The additional seismic pressure increment may be calculated using an equivalent fluid weight of 2 pcf for active conditions and 15 pcf for at-rest conditions. The determined forces will have approximately triangular pressure distributions with their centroids of pressure located 0.33 H above the base of the retaining walls (Al Atik and Sitar, 2010).

6. Retaining walls for the skate plaza, if applicable, should be supported by continuous spread foundations bearing on a reinforced soil pad as recommended in Section A of this report. Retaining wall foundations should be designed and constructed per Section D above.

7. The pressures recommended above were based on the assumption that the on-site soils will be used for wall backfill and will be compacted to approximately 90 percent...
of maximum dry density. The use of select granular fill may reduce the recommended
driving earth pressure.

8. Backfill immediately behind any retaining structure should be a free-draining granular
material. Comments on the characteristics of import soils will be given by the
geotechnical consultant after the material is on the project, either in place, or
stockpiled in adequate quantities to complete the project.

9. Backfill behind retaining walls should be with soils that have been properly moisture
conditioned to approximately optimum moisture content and uniformly compacted
to at least 90 percent of maximum dry density as determined by ASTM D 1557 test
procedures using mechanical compaction equipment. To aid in the compaction
operation, retaining wall backfill should be placed in lifts not exceeding six inches
compacted thickness.

10. Should be performed by hand operated compaction equipment. This is intended to
reduce potential "locked-in" lateral pressures caused by compaction with heavy
grading equipment.

11. The final grade should be such that all water is diverted away from the retaining wall's
foundation or backfill.

H. Pavements

1. Based on our exploratory borings, the near-surface soils across the site are generally
silty sands to sandy silts that have a moderate traffic support capacity when
recompacted and used as pavement subgrade. A sample of the near-surface soil was
collected from Boring B-3 for R-value testing. The test on the near-surface soil yielded
an R-value of 37.

2. The upper 12 inches below the finished subgrade elevation should be compacted to
achieve a minimum relative compaction of 95 percent of the ASTM D1557 maximum
dry density.

3. All trench backfill for culverts, utilities and pipes underlying paved areas should be
properly placed and compacted to at least 90 percent relative compaction (ASTM
D1557) within 1 foot of finished subgrade elevation. The upper 12 inches of trench
backfill should be compacted to at least 95 percent relative compaction.

4. The subgrade soils should be in a stable, non-pumping condition at the time the
aggregate base material is placed and compacted.

5. Aggregate base materials should conform to the specifications stated in the 2012
edition of the "Greenbook" and be compacted as engineered fill to at least 95 percent
relative compaction.
6. Adequate drainage (both surface and subsurface) should be provided such that the subgrade soils and aggregate base materials are not allowed to become continuously wet.

7. All concrete curbs separating pavement and landscaped areas should extend at least 2 inches into the subgrade and below the bottom of the adjacent aggregate base to provide a barrier against lateral migration of landscape water or runoff into the pavement section.

8. Periodic maintenance should be performed to repair degraded areas and seal cracks with appropriate filler.

9. Due to grading operations, the actual pavement subgrade materials may vary significantly from those tested for this study. If this is the case, representative subgrade samples should be obtained and additional R-value tests performed. If the results of these tests vary significantly, the pavement sections will need to be revised.

I. Storm Water Infiltration Design Parameters

1. Three areas are proposed for possible subsurface infiltration within the project boundaries. Infiltration tests were performed at the east and west ends of the proposed multi-use field and in the southwest corner of the site just north of the proposed permeable parking lot. Those areas were tested for infiltration capacity in accordance with US Bureau of Reclamation guidelines that were specified in their Administrative Manual dated June 1, 2011. (Percolation test data are presented in Appendix D of this report.)

According to the same guidelines, the percolation rates measured in the field are converted to design infiltration rate using the following formulas:

Infiltration Rate (in inches per hour) = Percolation Rate/Reduction Factor

Reduction Factor = [(2\(d_1\) - \(\Delta d\) / Diameter] + 1,
Where \(d_1\) = Initial Water Depth (in inches),
\(\Delta d\) = Water drop of final reading (in inches), and
Diameter = Diameter of boring (in inches).
Based on the testing of Percolation Test Hole T-1, located at the east end of the multi-use field, the design infiltration rate at a depth of approximately 8 feet is:

\[ 1.01 \text{ in./hr.} = 4.27 \times 10^{-2} \text{ cm./sec.} \]

Based on the testing of Percolation Test Hole T-2, located at the west end of the multi-use field, the recommended design infiltration rate at a depth of approximately 8 feet is:

\[ 11.77 \text{ in./hr.} = 0.50 \text{ cm./sec.} \]

Based on the testing of Percolation Test Hole T-3, the recommended design infiltration rate at a depth of approximately 8 feet is:

\[ 1.96 \text{ in./hr.} = 8.29 \times 10^{-2} \text{ cm./sec.} \]

Designers commonly apply a factor-of-safety to the infiltration rate to account for future disposal bed siltation.

**ADDITIONAL SERVICES**

This report is based on the assumption that an adequate program of monitoring and testing will be performed by Earth Systems Southern California during construction to check compliance with the recommendations given in this report. The recommended tests and observations include, but are not necessarily limited to the following:

1. Review of the building and grading plans during the design phase of the project.
2. Observation and testing during site preparation, grading, placing of engineered fill, and foundation construction.
3. Consultation as required during construction.
LIMITATIONS AND UNIFORMITY OF CONDITIONS

The analysis and recommendations submitted in this report are based in part upon the data obtained from the test pits excavated on the site. The nature and extent of variations between and beyond the test pits may not become evident until construction. If variations then appear evident, it will be necessary to reevaluate the recommendations of this report.

The scope of our services did not include any environmental assessment or investigation for the presence or absence of wetlands, hazardous or toxic materials in the soil, surface water, groundwater or air, on, below, or around this site. Any statements in this report or on the soil test pit logs regarding odors noted, unusual or suspicious items or conditions observed, are strictly for the information of our client.

Findings of this report are valid as of this date; however, changes in conditions of a property can occur with passage of time whether they are due to natural processes or works of man on this or adjacent properties. In addition, changes in applicable or appropriate standards may occur whether they result from legislation or broadening of knowledge. Accordingly, findings of this report may be invalidated wholly or partially by changes outside our control. Therefore, this report is subject to review and should not be relied upon after a period of 1 year.

In the event that any changes in the nature, design, or location of the proposed improvements, the conclusions and recommendations contained in this report shall not be considered valid unless the changes are reviewed and conclusions of this report modified or verified in writing.

This report is issued with the understanding that it is the responsibility of the Owner, or of his representative to insure that the information and recommendations contained herein are called to the attention of the Architect and Engineers for the project and incorporated into the plan and that the necessary steps are taken to see that the Contractor and Subcontractors carry out such recommendations in the field.

As the Geotechnical Engineers for this project, Earth Systems Southern California strives to provide our services in accordance with the generally accepted geotechnical engineering practices in this community at this time. No warranty or guarantee is expressed or implied. This report was prepared for the exclusive use of the homeowner and their authorized agents.
It is recommended that Earth Systems Southern California be provided the opportunity for a general review of final design and specifications in order that earthwork and foundation recommendations may be properly interpreted and implemented in the design and specifications. If Earth Systems Southern California is not accorded the privilege of making this recommended review, we can assume no responsibility for misinterpretation of our recommendations.
REFERENCES


CDMG, 1972 (Revised 1994), Fault Rupture Hazard Zones In California, Special Publication 42.


CDMG, 1998, Maps of Known Active Fault Near-Source Zones in California and Adjacent Portions of Nevada.

CDMG, 1999, Map Sheet 48, Seismic Shaking Hazard Map of California.

CDMG Seismic Hazards Mapping Act, 1990.

Dibblee, Thomas W., Jr., 1986, Geologic Map of the Santa Barbara Quadrangle, Santa Barbara County, California.


Jennings, J. E., and Knight, K., 1956, Recent Experiences with the Consolidation Test as a Means of Identifying Conditions of Heaving or Collapse of Foundations on Partially Saturated Soils, Transactions, South African Institution of Civil Engineers, August.


USGS, S. A. Minor and Others, Preliminary Geology Map of the Santa Barbara Coastal Plain Area, 2009.
