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SUBJECT: Geotechnical Engineering Investigation Report  
Contra Costa College District Headquarters  
500 Court Street, Martinez, California

Dear Mr. Roach:

Kleinfelder is pleased to present this geotechnical engineering investigation report for the seismic retrofit of the Contra Costa College Headquarters in Martinez, California.

The purpose of our geotechnical engineering investigation was to explore and characterize the subsurface conditions and provide seismic and geotechnical recommendations to be used for project design and construction. This report contains site-specific response spectra per the standard ASCE 41-13. In addition, recommendations regarding modulus of subgrade reaction and ultimate bearing pressures are also provided.

If you have any questions regarding the information or recommendations presented in our report, please contact us at your convenience at (925) 484-1700.

Sincerely,

KLEINFELDER, INC.

Zia Zafir, PhD, PE, GE  
Senior Principal Engineer  

Don Adams, PE  
Project Manager
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1 INTRODUCTION

This report presents the results of our geotechnical engineering investigation performed for the planned seismic retrofit of the Contra Costa College District Headquarters located at 500 Court Street in Martinez, California. The approximate location of the facility is shown on the Site Location Map (Figure 1), and the approximate limit of the existing building is shown on the Boring Location Map (Figure 2).

We understand that the District plans to seismically upgrade the existing headquarter building. The existing headquarters building is a six-story reinforced concrete mid-rise with plan dimensions of 88 feet by 88 feet. It is located at the northwest corner of the intersection of Court Street and Escobar Street. It was constructed in the early 1970’s, and is supported on a ribbed, reinforced concrete mat foundation. The ground floor elevation is +12’ (Contra Costa County Datum). Structural loads are not known at this time.

If the project differs from that presented above, we should be contacted to review the applicability and potential modifications to our scope of services.

1.1 GENERAL SITE DESCRIPTIONS

The building sits on a lot which is relatively flat and is surrounded by a paved parking lot.

According to the U.S. Geological Survey (USGS, 1993) 7½-Minute Vine Hill Topographic Quadrangle map, the existing ground elevation at the subject site ranges between about 7 and 9 feet above mean sea level. The coordinates at the center of the existing building are approximately:

- Latitude: 38.0190° N
- Longitude: 122.1353° W

1.2 PREVIOUS INVESTIGATIONS

A preliminary soil investigation was conducted by Harding, Miller, Lawson & Associates in 1969, and the results were presented in a report titled “Report, Preliminary Soil Investigation,”
Educational Center Building 543-68, Contra Costa Junior College District, Martinez, California,” dated September 3, 1969. Two borings, Borings 1 and 2, were drilled to depths of about 50 feet using a rotary wash.

A soil investigation was conducted by Harding, Miller, Lawson & Associates in 1970, and the results were presented in a report titled “Soil Investigation, District Education Center Building, Contra Costa Junior College District, Martinez, California,” dated December 10, 1970. Two borings, Borings 3 and 4, were drilled to depths of about 50 feet using a hollow stem augers.

Of these four borings, Borings 2, 3, and 4 are located near the south, west, and north corners of the subject building, respectively. Boring 1 was drilled on Howard Street, further away from the building on the north side.

According to the 1970 report, at the building location, the then existing ground surface ranged in elevation from about +19’ in the east corner, to about +12’ in the west corner. Below the upper three to six feet of old fill, the borings encountered alluvial deposits of clays and silts interbedded with discontinuous sand and gravel layers. The soil layers appear to dip gently to the northwest. The water level measured at the time of exploration was eight feet below the ground surface in Borings 2 and 3 (Boring 2 was converted to an observation well).

1.3 PURPOSE AND SCOPE OF SERVICES

The purpose of our geotechnical investigation was to explore and evaluate the subsurface conditions at the site in order to develop recommendations related to the geotechnical aspects of seismic retrofit. The proposed scope of our services was outlined in our Proposal (LOCALMKT.WEOH/PLE17P63109) dated July 24, 2017. Our services as presented in this report include the following:

- A site reconnaissance to observe the surface conditions
- A field investigation that consisted of drilling one boring to explore the subsurface conditions
- Laboratory testing of selected soil samples obtained during the field investigation to evaluate relevant physical and engineering parameters of the subsurface soils
• Evaluation of the field and laboratory data obtained and performing engineering analyses
to develop our recommendations

• Preparation of this report which includes:

  o Site Location Map, and Boring Location Map showing the approximate test boring
    location;

  o Description of the project;

  o Boring log and laboratory test results;

  o Discussion of general site subsurface conditions, as encountered in our test
    borings;

  o Discussion of site seismicity;

  o Site-specific response spectra for BSE-1N and BSE-2N per ASCE 41-13;

  o Discussion of liquefaction analysis and settlement potential and magnitude;

  o Ultimate bearing pressure for seismic loading; and

  o Recommendations for modulus for subgrade reaction for existing mat foundation.

Our evaluation also specifically excluded the assessment of environmental spills and hazardous
substances at the site.

We understand that the subject building is the District headquarters and not a school building.
Therefore, we were instructed that the building does not fall under the jurisdiction of the California
Division of State Architect (DSA) and California Geological Survey (CGS) Note 48.
2.1 FIELD INVESTIGATION

2.1.1 Pre-Field Activities

Prior to the start of the field investigation, Underground Service Alert (USA) was contacted to locate utilities in the vicinity of the boring location. We also subcontracted the services of a private utility locator who identified and marked underground utilities in the vicinity of our boring location. As required by local ordinance, a drilling permit was obtained from the Contra Costa County Environmental Health Division.

2.1.2 Exploratory Borings

We drilled one test boring on the northeast side of the existing building on August 30, 2017 to a depth of approximately 50½ feet. The approximate location of the boring is shown on Figure 2. The boring was drilled by Gregg Drilling & Testing, Inc., of Martinez, California, using a truck-mounted drill rig with 6-inch outside-diameter hollow-stem augers to a depth of 7 ½ feet and switching to mud rotary methods beyond that depth to the full depth of 50 ½ feet. The boring location was located in the field by measuring from existing landmarks. Horizontal coordinates and elevations of the borings were not surveyed.

A Kleinfelder professional maintained logs of the borings, visually classified the soils/bedrock encountered and obtained relatively undisturbed and bulk samples of the subsurface materials. Soil classifications made in the field from samples and auger cuttings were in accordance with American Society for Testing and Materials (ASTM) Method D 2488. These classifications were re-evaluated in the laboratory after further examination and testing in accordance with ASTM D 2487. Sample classifications, blow counts recorded during sampling, and other related information were recorded on the boring logs. The blow counts listed on the boring logs have not been corrected for the effects of overburden pressure, rod length, sampler size, or hammer efficiency. Correction factors were applied to the raw blow counts to estimate the sample apparent density noted on the boring logs and for engineering analyses. After the borings were completed, they were backfilled with cement grout and patched with asphalt at the surface, where applicable. Excess drill cuttings were spread in landscape areas on site.
Keys to the soil descriptions and symbols used on the boring logs are presented on Figures A-1 and A-2 in Appendix A. Log of the boring is presented on Figures A-3.

2.2 SAMPLING PROCEDURES

Soil samples were collected from the boring at depth intervals of approximately 5 feet. Samples were collected from the borings at selected depths by driving either a 2.5-inch inside-diameter (I.D.) California sampler or a 1.4-inch I.D. Standard Penetration Test (SPT) sampler 18 inches (unless otherwise noted) into undisturbed soil. The samplers were driven using a 140-pound automatic hammer free-falling a distance of about 30 inches. Blow counts were recorded at 6-inch intervals for each sample attempt and are reported on the log.

The SPT sampler did not contain liners, but had space for them. The 2.5-inch I.D. California sampler contained stainless steel liners. The California sampler was in general conformance with ASTM D 3550. The SPT sampler was in general conformance with ASTM D 1586.

Soil/bedrock samples obtained from the borings were packaged and sealed in the field to reduce moisture loss and disturbance. Following drilling, the samples were returned to our laboratory for further examination and testing.

2.3 GEOTECHNICAL LABORATORY TESTING

Laboratory tests were performed on selected soil samples to evaluate their physical characteristics and engineering properties. The laboratory testing program included unit weight and moisture content, Atterberg limits, unconsolidated-undrained triaxial, and sieve analysis (percentage passing the No. 200 sieve) tests. Most of the laboratory test results are presented on the boring log. A summary of geotechnical laboratory tests is presented on Figure B-1. The results of the Atterberg Limits, unconsolidated-undrained triaxial tests are presented graphically on Figures B-2 through B-5 in Appendix B.
3 SURFACE AND SUBSURFACE CONDITIONS

3.1 SURFACE CONDITIONS

The existing headquarters building is a six-story reinforced concrete mid-rise with plan dimensions of 88 feet by 88 feet. It is located at the northwest corner of the intersection of Court Street and Escobar Street. It was constructed in the early 1970's, and is supported on a ribbed, reinforced concrete mat foundation. Surrounding area is a paved parking lot.

3.2 SUBSURFACE CONDITIONS

The subsurface conditions described herein are based on the soil and groundwater conditions encountered during the current and previous geotechnical investigations for this site. Currently the site is a paved parking lot with about 3 inches of asphalt concrete overlying 9 inches of aggregate base. The aggregate base is underlain by about 1 foot of fill overlying silty sand with clay and fat clay to a depth of about 5½ feet. The fat clay layer is underlain by alternating layers of soft to medium stiff sandy lean clay and lean clay with sand to a depth of about 25 feet. The sandy lean clay layer is underlain by stiff to very stiff fat clay to a depth of 30 feet overlying dense sandy silt layer to a depth of about 47 feet. Clayey sandstone was encountered at a depth of about 47 feet. The sandstone was mostly decomposed.

Groundwater was observed at a depth of about 7½ feet in the boring. It should be noted that groundwater levels can fluctuate depending on factors such as seasonal rainfall and construction activities on this or adjacent properties, and may rise several feet during a normal rainy season.

The above is a general description of soil/bedrock and groundwater conditions encountered in the borings from this investigation and our experience at the campus. More detailed descriptions of the subsurface conditions encountered are presented in the Boring Log on Figure A-3 in Appendix A.

Soil/bedrock and groundwater conditions can deviate from those conditions encountered at the boring location. If significant variations in the subsurface conditions are encountered during construction, Kleinfelder should be notified immediately, and it may be necessary for us to review the recommendations presented herein and recommend adjustments as necessary.
This section presents the results of our site-specific seismic hazard analysis per ASCE 41-13 (ASCE 2013) for the Contra Costa College Headquarters in Martinez, California. The subsurface soil conditions used in this study were obtained from our current geotechnical investigations at the project site and from previous two investigations performed in 1969 and 1970.

The purpose of this seismic hazard analysis is to develop site-specific ground motion criteria in terms of peak ground accelerations and response spectral accelerations for the subject site by using a seismic source model (proximity to active faults, major historical earthquakes, and regional seismicity) and subsurface soil conditions at the site. The response spectrum is a graphical representation relating the maximum response of a single degree of freedom, elastic damped oscillator with different fundamental periods to dynamic loads. Site-specific spectrum for any given return period represents earthquake ground motions consistent with the seismic source model and the local site response. Specifically, our scope of services includes the following:

- Literature review of available geologic and seismic setting of the area and developing a site-specific seismic source model.
- Estimating the average shear wave velocity in the upper 100 feet ($V_{s30}$) of the site based on the results of the field explorations.
- Classification of the site per Chapter 20 of ASCE 7-10.
- Performing site-specific probabilistic and deterministic seismic hazard analyses (PSHA and DSHA) to obtain spectral accelerations for 2% probability of exceedance in 50 years and for 84$^{th}$ percentile deterministic per Chapter 21 of ASCE 7-10.
- Developing site-specific response spectra for the BSE-1N and the BSE-2N per Chapter 2 of ASCE 41-13 for damping value of 5%.
- Developing site-specific ground motion parameters ($S_{Xs}$ and $S_{X1}$) per ASCE 41-13.
4.1 REGIONAL FAULTING

According to Hart and Bryant (1997), the site is not located within an Alquist-Priolo Earthquake Fault Zone. Closest fault to the site is the Green Valley fault with a rupture distance of about 6.2 km. Other major faults located close to the site are, the Hayward-Rodgers Creek fault at about 18 km, the West Napa fault at about 19 km, the Green Valley Connected fault at about 31 km, the Mount Diablo Thrust at about 20 km, the Calaveras fault at about 25 km, and the Northern San Andreas fault at about 46 km. A seismic event on any of these faults could cause significant ground shaking at the site. Figure 3 shows the known faults within 100 km from the site. However, only independent seismogenic sources have been labeled. All the other faults have been included in the background seismic sources.

4.2 SEISMIC SOURCE MODEL

Our probabilistic seismic source model is based on the seismic source model used in developing the 2008 update of the United States National Seismic Hazard Maps by California Geological Survey (CGS) and US Geological Survey (Petersen et al. 2008). Table 1 lists these individual fault segments and their seismic parameters. The various combinations of fault segments and different rupture scenarios are accounted for in the logic tree in our seismic source model per Petersen et al. (2008). However, Table 1 only presents the scenario of rupturing all the segments. The maximum earthquake magnitudes presented in this table are based on the moment magnitude scale developed by Hanks and Kanamori (1979). CGS has assigned weights of 0.67 and 0.33 to Characteristics and G-R models, respectively, for all the faults listed in Table 1 except for the Hayward-Rodgers Creek and N. San Andreas faults. For the Hayward-Rodgers Creek and the N. San Andreas faults, Characteristic model was assigned 1.0 weight. We have used the same approach in our analyses. We have used faults within 200 km of the site in our analyses but only faults within 100 km are listed in Table 1.

According to Petersen et al. (2008), characterizations of the Hayward-Rodgers Creek, the N. San Andreas, and the Calaveras faults are based on the following fault rupture segments and fault rupture scenarios:

- The Hayward-Rodgers Creek fault has been characterized by three segments and six rupture scenarios plus a floating earthquake. The three segments are the Rodgers Creek fault (RC), the Hayward North (HN), and the Hayward South (HS).
• The N. San Andreas fault has been characterized by four segments and nine rupture scenarios, plus a floating earthquake. The four segments are Santa Cruz Mountains (SAS), North Coast (SAN), Peninsula (SAP), and Offshore (SAO).

• The Calaveras fault includes three segments and six rupture scenarios, plus a floating earthquake. The three segments are southern (CS), central (CC), and northern (CN).

We have used all of the rupture scenarios for these faults as used by Petersen et al. (2008).

**TABLE 1: SIGNIFICANT FAULTS IN THE SEISMIC SOURCE MODEL**

<table>
<thead>
<tr>
<th>Fault Name</th>
<th>Closest Distance* (km)</th>
<th>Fault Length (km)</th>
<th>Magnitude of Characteristic Earthquake **</th>
<th>Slip Rate (mm/yr)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Green Valley Connected</td>
<td>6.2</td>
<td>56</td>
<td>6.80</td>
<td>4.7</td>
</tr>
<tr>
<td>Hayward-Rodgers Creek</td>
<td>18</td>
<td>150</td>
<td>7.33</td>
<td>9.0</td>
</tr>
<tr>
<td>West Napa</td>
<td>19</td>
<td>30</td>
<td>6.70</td>
<td>1.0</td>
</tr>
<tr>
<td>Mount Diablo Thrust</td>
<td>20</td>
<td>25</td>
<td>6.70</td>
<td>2.0</td>
</tr>
<tr>
<td>Calaveras</td>
<td>25</td>
<td>123</td>
<td>7.03</td>
<td>6-15</td>
</tr>
<tr>
<td>Great Valley 5, Pittsburg Kirby Hills</td>
<td>26</td>
<td>32</td>
<td>6.70</td>
<td>1.0</td>
</tr>
<tr>
<td>Great Valley 4b, Gordon Valley</td>
<td>28</td>
<td>28</td>
<td>6.80</td>
<td>1.3</td>
</tr>
<tr>
<td>Greenville Connected</td>
<td>31</td>
<td>51</td>
<td>7.00</td>
<td>2.0</td>
</tr>
<tr>
<td>Northern San Andreas</td>
<td>46</td>
<td>473</td>
<td>8.05</td>
<td>17-24</td>
</tr>
<tr>
<td>Hunting Creek-Berryessa</td>
<td>49</td>
<td>60</td>
<td>7.10</td>
<td>6.0</td>
</tr>
<tr>
<td>San Gregorio - Connected</td>
<td>51</td>
<td>176</td>
<td>7.50</td>
<td>5.5</td>
</tr>
<tr>
<td>Great Valley 4a, Trout Creek</td>
<td>53</td>
<td>19</td>
<td>6.60</td>
<td>1.3</td>
</tr>
<tr>
<td>Point Reyes</td>
<td>60</td>
<td>47</td>
<td>6.90</td>
<td>0.4</td>
</tr>
<tr>
<td>Great Valley 7</td>
<td>62</td>
<td>45</td>
<td>6.90</td>
<td>1.5</td>
</tr>
<tr>
<td>Monte Vista-Shannon</td>
<td>65</td>
<td>45</td>
<td>6.50</td>
<td>0.4</td>
</tr>
<tr>
<td>Great Valley 3, Mysterious Ridge</td>
<td>71</td>
<td>55</td>
<td>7.10</td>
<td>1.3</td>
</tr>
<tr>
<td>Maacama-Garberville</td>
<td>79</td>
<td>221</td>
<td>7.40</td>
<td>9.0</td>
</tr>
<tr>
<td>Collayomi</td>
<td>97</td>
<td>28</td>
<td>6.70</td>
<td>0.6</td>
</tr>
</tbody>
</table>

* Closest distance to potential rupture
** Moment magnitude: An estimate of an earthquake’s magnitude based on the seismic moment
4.3 MAGNITUDE-FREQUENCY DISTRIBUTION

The earthquake probabilities for the faults and their segments were developed using a magnitude-frequency relationship derived from the seismicity catalogs and the fault activity based on their slip rates. In general, there are two models based on magnitude-frequency relationships. In the first, earthquake recurrence is modeled by a truncated form of the Gutenberg-Richter (G-R) (Gutenberg and Richter, 1956) magnitude-frequency relation given by:

\[ \log(N) = a - bM \]

where \( N(M) \) is the cumulative number of earthquakes of magnitude "M" or greater per year, and "a" and "b" are constants based on recurrence analyses. The relation is truncated at the maximum earthquake. In the G-R model, it is assumed that seismicity along a given fault or fault zones satisfies the above equation. This model generally implies that seismic events of all sizes occur continually on a fault during the interval between the occurrences of the maximum expected events along the fault zone.

The second model, generally referred to as a Characteristic model (Schwartz and Coppersmith, 1984), implies that the time between maximum size earthquakes along particular fault zones or fault segments is generally quiescent except for foreshocks, aftershocks, or low level background activity.

We have used the Peterson et al. (2008) approach in our analyses, which used both the G-R and the Characteristic models. A b-value of 0.8 is used for all the faults. The most likely a-values were estimated for each seismic source based on the recurrence rates of earthquakes and events per year associated with that seismic source as reported by Petersen et al. (2008).

4.4 HISTORICAL SEISMICITY

The project site is located in an area characterized by high seismic activity. A number of large earthquakes have occurred within this area in the past years. Some of the significant nearby events include the 1868 (M6.8) Hayward earthquake, the 2014 (M6.0) South Napa earthquake, the 1906 (M7.9) “Great” San Francisco earthquake, the 1838 (M7) San Francisco Peninsula earthquake, the 1865 (M6.4) Santa Cruz Mountains earthquake, the two 1903 (M5.5) San Jose earthquakes, and the 1989 (M6.9) Loma Prieta earthquake. A study by Toppozada and Borcherdt (1998) indicates an 1836 (M6.8) earthquake, previously attributed to the Hayward fault, occurred
in the Monterey Bay area and was of an estimated magnitude M6.2. During the South Napa earthquake, CSMIP stations in the area recorded free-field horizontal peak ground accelerations of less than 0.1g. At the Benicia-Martinez Bridge south station, a PGA of 0.029g was recorded. At the VA Medical Center in Martinez, a PGA of 0.075g was recorded. Epicenters of significant earthquakes (M>4.0) within the vicinity of the site are shown on Figure E-1.

4.5 BACKGROUND SEISMICITY

In addition to the individual seismogenic sources, we also allow for background seismicity that accounts for random earthquakes between M 5 and 7 based on the methodology described by Frankel et al. (1996). Using the seismic source model used by CGS/USGS, some of the local faults in the area are not included in our analyses as independent seismogenic sources. However, their seismicity has been included by allowing for background seismicity in our model. The a-values are calculated using the method described in Weichert (1980). The hazard may then be calculated using this a-value, a b-value of 0.9, minimum magnitude of 5, maximum magnitude of 7, and applying an exponential distribution as described by Hermann (1977).

4.6 SEISMIC HAZARD ANALYSIS

Based on the results of the field explorations for this project and using appropriate correlations between penetration resistance and shear wave velocity (Vs) and/or undrained shear strength and Vs, the site is estimated to have average shear wave velocity in the upper 100 feet ($V_{S30}$) of about 820 feet/sec (250 m/s), thus making this site as Site Class D (i.e., Stiff soil) based on Table 20.3-1 of ASCE 7-10. We used Caltrans procedure in estimating $V_{S30}$ for this site (Caltrans, 2012).

According to Section 2.4.1.1 of ASCE 41-13, BSE-2N is consistent with MCE$_R$ of ASCE 7-10 and according to Section 2.4.1.2 of ASCE 41-13, BSE-1N is taken as two-thirds of BSE-2N. According to ASCE 7-10, the MCE$_R$ is defined as the lesser of: (1) 2 percent probability of being exceeded in 50 years (return period of about 2,475 years) adjusted for risk factors and for the maximum direction; and (2) greater of 84th percentile (median + 1 standard deviation) deterministic values (adjusted for the maximum direction) from the controlling fault and deterministic lower limit (DLL) of Figure 21.2-1 of ASCE 7-10. The DE is defined as two-thirds of the MCE$_R$. In addition, for site-specific response spectra, procedures provided in Chapter 21 of ASCE 7-10 should be used and the design spectral accelerations at any period from site-specific analyses should not be less than the 80 percent of the code spectrum based on $S_{DS}$ and $S_{D1}$ values from Chapter 11, ASCE 7-10.
Both probabilistic and deterministic seismic hazard analyses were used to estimate the spectral accelerations for the MCE.<sub>R</sub>. These analyses involve the selection of appropriate predictive relationships to estimate the ground motion parameters, and, through probabilistic and deterministic methods, determination of peak and spectral accelerations.

4.7 GROUND MOTION PREDICTION EQUATIONS (GMPE)

Site-specific ground motions can be influenced by the styles of faulting, magnitudes of the earthquakes, and local soil conditions. The GMPEs used to estimate ground motion from an earthquake source need to consider these effects. Many GMPEs have been developed to estimate the variation of peak ground acceleration with earthquake magnitude and distance from the site to the source of an earthquake.

We have used Boore and Atkinson (2008), Campbell and Bozorgnia (2008), and Chiou and Youngs (2008) NGA-West 1 GMPEs, as these three were used in developing 2008 USGS National Seismic Hazard Maps. All of these GMPEs use an estimate of the average shear wave velocity in the upper 100 feet (V<sub>S30</sub>) of the soil profile in the analysis. Based on the results of our field investigation, a V<sub>S30</sub> of 250 m/s was used in the analyses. Some of these GMPEs also require inputs for depth in meters to a layer with V<sub>s</sub> value of 1,000 m/s (Z<sub>1.0</sub>) and depth in km to the layer with V<sub>s</sub> value of 2,500 m/s (Z<sub>2.5</sub>) to account for deep soil basin effects. Since the site is not located in any known deep soil basin, we used the default (minimum) values in our analysis. Spectral acceleration values were obtained by averaging the individual hazard results. These GMPEs provide mean values of ground motions associated with magnitude, distance, site soil conditions, and mechanism of faulting. The uncertainty in the predicted ground motion is taken into consideration by including a magnitude dependent standard error in the probabilistic analysis.

4.8 PROBABILISTIC SEISMIC HAZARD ANALYSIS

A probabilistic seismic hazard analysis (PSHA) procedure was used to estimate the peak and spectral ground motions corresponding to 2 percent probability of exceedance in 50 years. The PSHA approach is based on the earthquake characteristics and its causative fault. These characteristics include such items as magnitude of the earthquake, distance from the site to the causative fault, and the length and activity of the fault. The effects of site soil conditions and mechanism of faulting are accounted for in the GMPE(s) used for the site.
The theory behind seismic risk analysis has been developed over many years (Cornell, 1968, 1971; Merz and Cornell, 1973), and is based on the "total probability theorem" and on the assumption that earthquakes are events that are independent of time and space from one another. According to this approach, the probability of exceeding PE(Z) at a given level of ground motion, Z, at the site within a specified time period, T, is given by:

$$PE(Z) = 1 - e^{-\vartheta(Z)T}$$

where $\vartheta(Z)$ is the mean annual rate of exceedance of ground motion level Z. Different probabilities of exceedance may be selected, depending on the level of performance required.

The PSHA can be explained through a four-step procedure as follows:

1. The first step involves identification and characterization of seismic sources and probability distribution of potential rupture within the sources. Usually, uniform probability distributions are assigned to each source. The probability distribution of site distance is obtained by combining potential rupture distributions with source geometry.

2. The second step involves characterization of seismicity distribution of earthquake recurrence. An earthquake recurrence relationship such as Gutenberg-Richter recurrence is used to characterize the seismicity of each source.

3. The third step involves the use of GMPEs in assessing the ground motion produced at the site by considering the applicable sources and the distance of the sources to site. The variability of GMPEs is also included in the analysis. The effects of site soil conditions and mechanism of faulting are accounted for in these GMPEs.

4. The fourth and the last step involve combining all of these uncertainties to obtain the probability of ground motion exceedance during a particular time period.

A simplified mathematical expression for these steps is provided below:

$$\nu \ (Sa > z) = \sum_{i=1}^{N_{source}} N_i(M_{min}) \int_{r=0}^{M_{max}} \int_{m=M_{min}}^{M_{max}} f_{s_i}(M) f_{r_i}(r) P(Sa > z | M, r) dr dM$$
Where $\nu(Sa>z)$ is the mean annual rate of a spectral acceleration $(Sa)$ exceeding a test value $(z)$; $N_{\text{source}}$ is the number of seismic sources; $N_i(M_{\text{min}})$ is the rate of earthquakes with magnitude greater than $M_{\text{min}}$ on the $i^{th}$ seismic source; $f_{m,i}(M)$ is the probability distribution of earthquake magnitude $(M)$ of the $i^{th}$ source; $f_{r,i}(r)$ is the probability distribution of the fault rupture location $(r)$; and $P(Sa>z|M,r)$ is the probability that $Sa$ is greater than the test value $(z)$ given the $M$ and $r$.

We have used the computer program EZ-FRISK version 8.00 beta (Risk Engineering, 2015) for our probabilistic analysis. Horizontal response spectral values for the 2 percent in 50-year probability of exceedance were calculated using the probabilistic analysis approach described above. Elastic response spectral values were calculated for a damping factor of 5 percent of critical.

4.9 DETERMINISTIC SEISMIC HAZARD ANALYSIS

The deterministic seismic hazard analysis (DSHA) approach is also based on the characteristics of the earthquake and the causative fault associated with the earthquake. These characteristics include such items as magnitude of the earthquake and distance from the site to the causative fault. The effects of site soil conditions and mechanism of faulting are also accounted for in the GMPE for this site. Per ASCE 7-10, the 84th percentile deterministic site-specific spectral acceleration values at the site were estimated for the Green Valley Connected fault (M6.8), which is the controlling fault for this site. We used a rupture distance of 6.2 km in our analysis.

4.10 DETERMINATION OF SITE-SPECIFIC HORIZONTAL BSE-2N AND BES-1N RESPONSE SPECTRA

To develop the site-specific spectral response accelerations, we first obtained the general seismic design parameters based on the site class, site coordinates and the risk category of the building using the USGS online tool (http://earthquake.usgs.gov/designmaps/us/application.php). These values are summarized in Table 2.
### TABLE 2: GENERAL GROUND MOTION PARAMETERS BASED ON ASCE 7-10

<table>
<thead>
<tr>
<th>PARAMETER</th>
<th>VALUE</th>
<th>ASCE 7-10 REFERENCE</th>
</tr>
</thead>
<tbody>
<tr>
<td>$S_S$</td>
<td>1.558g</td>
<td>Fig 22-1</td>
</tr>
<tr>
<td>$S_1$</td>
<td>0.600g</td>
<td>Fig 22-2</td>
</tr>
<tr>
<td>Site Class</td>
<td>D</td>
<td>Table 20.3-1</td>
</tr>
<tr>
<td>$F_a$</td>
<td>1.00</td>
<td>Table 11.4-1</td>
</tr>
<tr>
<td>$F_v$</td>
<td>1.50</td>
<td>Table 11.4-2</td>
</tr>
<tr>
<td>$C_R$</td>
<td>1.053</td>
<td>Fig 22-3</td>
</tr>
<tr>
<td>$C_R$</td>
<td>1.048</td>
<td>Fig 22-4</td>
</tr>
<tr>
<td>$S_M$</td>
<td>1.558g</td>
<td>Eq. 11.4-1</td>
</tr>
<tr>
<td>$S_M$</td>
<td>0.900g</td>
<td>Eq. 11.4-2</td>
</tr>
<tr>
<td>$S_D$</td>
<td>1.038g</td>
<td>Eq. 11.4-3</td>
</tr>
<tr>
<td>$S_D$</td>
<td>0.600g</td>
<td>Eq. 11.4-3</td>
</tr>
<tr>
<td>PGA</td>
<td>0.592</td>
<td>Fig 22-7</td>
</tr>
<tr>
<td>$F_{PGA}$</td>
<td>1.00</td>
<td>Table 11.8-1</td>
</tr>
<tr>
<td>$PGA_M$</td>
<td>0.592</td>
<td>Eq. 11-8-1</td>
</tr>
</tbody>
</table>

As mentioned earlier, BSE-2N is consistent with MCE$_R$ of ASCE 7-10. As discussed earlier, the MCE$_R$ response spectrum is developed by comparing probabilistic, deterministic, DLL, and 80% of the code values. These NGA GMPEs present the spectral accelerations in terms of geometric mean values of the rotated two horizontal ground motions. To estimate both the deterministic and probabilistic the spectral accelerations in the direction of the maximum horizontal response at each period from geometric mean values, we have used the scale factors as used by USGS. To obtain spectral acceleration values in the maximum direction, a factor of 1.1 for periods of 0.2s and less, a factor of 1.3 for period of 1.0s and greater were used. Linear interpolation was used between 1.1 and 1.3 for periods between 0.2s and 1.0s. In addition, the probabilistic spectrum was adjusted for targeted risk using risk coefficients $C_R$ and $C_{R_1}$. $C_R$ and $C_{R_1}$ were estimated from Figures 22-3 and 22-4 of ASCE 7-10 and they are 1.053 and 1.048, respectively. $C_R$ is applied on periods of 0.2s or less and $C_{R_1}$ is applied on periods of 1.0s or greater and linear interpolation in between.

Site-specific deterministic (84th percentile) spectrum for the Green Valley Connected fault is compared with the DLL spectrum per Figure 21.2-1 of ASCE 7-10 on Figure 4. Spectral values are also compared in Table 3 for some specific periods. Figure 4 and Table 3 show that the controlling deterministic values are governed by both the 84th percentile site-specific deterministic
spectrum and the DLL. Therefore, the controlling deterministic spectrum is developed by enveloping the site-specific deterministic spectrum and the DLL.

**TABLE 3: COMPARISON OF SPECTRAL ACCELERATION (G)**

<table>
<thead>
<tr>
<th>Period (s)</th>
<th>Deterministic Max Rot</th>
<th>DLL</th>
<th>Probabilistic Max Rot Risk Adj</th>
<th>DE</th>
<th>80% Code DE</th>
</tr>
</thead>
<tbody>
<tr>
<td>PGA (0.01)</td>
<td>0.623</td>
<td>0.600</td>
<td>0.867</td>
<td>0.415</td>
<td>0.332</td>
</tr>
<tr>
<td>0.2</td>
<td>1.327</td>
<td>1.500</td>
<td>1.875</td>
<td>1.000</td>
<td>0.831</td>
</tr>
<tr>
<td>0.3</td>
<td>1.400</td>
<td>1.500</td>
<td>1.949</td>
<td>1.000</td>
<td>0.831</td>
</tr>
<tr>
<td>0.5</td>
<td>1.388</td>
<td>1.500</td>
<td>1.841</td>
<td>1.000</td>
<td>0.831</td>
</tr>
<tr>
<td>1.0</td>
<td>1.050</td>
<td>0.900</td>
<td>1.422</td>
<td>0.700</td>
<td>0.480</td>
</tr>
<tr>
<td>2.0</td>
<td>0.571</td>
<td>0.450</td>
<td>0.790</td>
<td>0.381</td>
<td>0.240</td>
</tr>
</tbody>
</table>

Site-specific probabilistic spectrum is compared with the controlling deterministic spectrum on Figure 5. Spectral values are also compared in Table 3 for some specific periods. Figure 5 and Table 3 show that the probabilistic values are greater than the controlling deterministic values. Therefore, site-specific MCE<sub>R</sub> spectrum is equal to the controlling deterministic spectrum. The DE spectrum was developed by taking two-thirds of the MCE<sub>R</sub> spectrum. Comparison of the DE spectrum with the 80% of the code spectrum is shown on Figure 6. Spectral values are also compared in Table 3 for some specific periods. Figure 6 and Table 3 show that the DE spectrum is higher than the 80% of the code spectrum for all periods. Therefore, the recommended site-specific horizontal DE spectrum is controlled by two-thirds of the MCE<sub>R</sub> spectrum. Site-specific MCE<sub>R</sub> spectrum is taken as 1.5 times the DE spectrum. Figure 7 shows the site-specific 5% damped BSE-1N and BSE-2N spectra. Site-specific horizontal spectral acceleration values in terms of g for the BSE-1N and BSE-2N are presented in Table 4.
TABLE 4: SITE-SPECIFIC HORIZONTAL BSE-1N AND BSE-2N SPECTRAL ACCELERATIONS (g)

<table>
<thead>
<tr>
<th>Period (sec)</th>
<th>BSE-1N (DE) 5% Damping</th>
<th>BSE-2N (MCE) 5% Damping</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.010</td>
<td>0.415</td>
<td>0.623</td>
</tr>
<tr>
<td>0.120</td>
<td>1.000</td>
<td>1.500</td>
</tr>
<tr>
<td>0.200</td>
<td>1.000</td>
<td>1.500</td>
</tr>
<tr>
<td>0.400</td>
<td>1.000</td>
<td>1.500</td>
</tr>
<tr>
<td>0.600</td>
<td>1.000</td>
<td>1.500</td>
</tr>
<tr>
<td>0.700</td>
<td>0.857</td>
<td>1.286</td>
</tr>
<tr>
<td>0.750</td>
<td>0.807</td>
<td>1.211</td>
</tr>
<tr>
<td>1.000</td>
<td>0.700</td>
<td>1.050</td>
</tr>
<tr>
<td>1.500</td>
<td>0.511</td>
<td>0.766</td>
</tr>
<tr>
<td>2.000</td>
<td>0.381</td>
<td>0.571</td>
</tr>
<tr>
<td>2.500</td>
<td>0.305</td>
<td>0.458</td>
</tr>
<tr>
<td>3.000</td>
<td>0.230</td>
<td>0.345</td>
</tr>
<tr>
<td>4.000</td>
<td>0.160</td>
<td>0.241</td>
</tr>
<tr>
<td>5.000</td>
<td>0.120</td>
<td>0.180</td>
</tr>
</tbody>
</table>

Since the site is not within 5 km of a fault, there is no need to develop fault normal and fault parallel spectra.

4.11 SITE-SPECIFIC DESIGN ACCELERATION PARAMETERS

Site specific ground motion parameters, $S_{XS}$ and $S_{X1}$ for BSE-1N and BSE-2N were estimated using the site-specific design response spectrum presented in Table 4. According to Section 2.4.2.1.6 of ASCE 41-13, the $S_{XS}$ value should be taken as the value at 0.2 seconds but should not be less than 90 percent of any spectral acceleration after that period. Based on this, the $S_{XS}$ value is governed by the spectral acceleration at 0.2 seconds as shown in Table 4. Additionally, the $S_{X1}$ value should be taken as greater of the value at 1.0 second or two times the value at 2.0 seconds. Based on this, two times the value at 2.0 seconds governs the $S_{X1}$ value as shown in Table 4. Site-specific $S_{XS}$, and $S_{X1}$ values for BSE-1N and BSE-2N are presented in Table 5.

TABLE 5: SITE-SPECIFIC DESIGN ACCELERATION PARAMETERS

<table>
<thead>
<tr>
<th>Parameter</th>
<th>BSE-1N</th>
<th>BSE-2N</th>
</tr>
</thead>
<tbody>
<tr>
<td>$S_{XS}$</td>
<td>1.000 g</td>
<td>1.500</td>
</tr>
<tr>
<td>$S_{X1}$</td>
<td>0.762 g</td>
<td>1.142</td>
</tr>
</tbody>
</table>
Site specific peak ground acceleration ($PGA_m$) for MCE$_0$ was estimated using Section 21.5 of ASCE 7-10. According to Section 21.5 of ASCE 7-10, the site-specific $PGA_m$ shall be taken as the lesser of the probabilistic geometric mean peak ground acceleration of Section 21.5.1 and the deterministic geometric mean peak ground acceleration of Section 21.5.2. Additionally, the site-specific $PGA_m$ shall not be taken as less than 80% of $PGA_m$ determined from Eq. 11.8-1. Based on this procedure, the site-specific $PGA_m$ value is 0.566g and is controlled by the deterministic results. Therefore, the associated earthquake magnitude is 6.8.
5 GEOTECHNICAL AND SEISMIC RECOMMENDATIONS

5.1 SEISMIC SHAKING

We expect the site to be subjected to substantial ground shaking due to a major seismic event on the surrounding faults, especially the active Green Valley Connected fault.

5.2 LIQUEFACTION

Based on the subsurface data obtained from our field investigation, the project site subsurface consists mostly of interbedded layers of soft to stiff fine-grained clayey soils underlain by dense silt overlying bedrock. As a result, liquefaction potential at the site is considered minimal due to the soil types encountered.

5.3 MODULUS FOR SUBGRADE REACTION

Existing building is supported on mat foundation with a plan dimension of 88 feet by 88 feet. Based on a dead load of about 1,500 pound per square foot (psf), we estimated modulus of subgrade reaction of about 30 pounds per cubic inch (pci).

5.4 ULTIMATE BEARING PRESSURE

The building is supported by a mat foundation. An ultimate bearing pressure of 9,000 psf is estimated for this building. For dead plus live loads, a factor of safety of 3 should be used. For BSE-1N and BSE-2N seismic loads, factors of safety of 2 and 1.5, respectively, may be used.

5.5 NEW FOUNDATIONS

At this time, new foundations have not been identified and structural loads are unknown. If new foundations are planned for the seismic retrofit, Kleinfelder should be notified to review the anticipated foundation type and loads. It is anticipated that new foundations can be supported on shallow spread footings, provided the estimated differential settlement between new and existing footings is acceptable.
6 ADDITIONAL SERVICES

The review of final plans and specifications, and field observations and testing during seismic retrofit by Kleinfelder is an integral part of the conclusions and recommendations made in this report. If Kleinfelder is not retained for these services, the client agrees to assume Kleinfelder's responsibility for any potential claims that may arise during the retrofit. The actual tests and observations by Kleinfelder during retrofit will vary depending on methods of retrofit. The tests and observations would be additional services provided by our firm. The costs for these services are not included in our current fee arrangements.
The conclusions and recommendations of this report are provided for the seismic retrofit of the Contra Costa College Headquarters in Martinez, California, as described in the text of this report. The conclusions and recommendations in this report are invalid if:

- The report is used for adjacent or other property
- Any other change is implemented which materially alters the project from that proposed at the time this report was prepared

The scope of services was limited to the drilling of one test boring in an area accessible to our drill rig. It should be recognized that definition and evaluation of subsurface conditions are difficult. Judgments leading to conclusions and recommendations are generally made with incomplete knowledge of the subsurface conditions present due to the limitations of data from field studies. The conclusions of this assessment are based on our subsurface exploration including one test boring drilled to a maximum depth of about 50½ feet; groundwater level measurements in the test boring during our field exploration; and geotechnical engineering analyses.

Kleinfelder offers various levels of investigative and engineering services to suit the varying needs of different clients. Although risk can never be eliminated, more-detailed and extensive studies yield more information, which may help understand and manage the level of risk. Since detailed study and analysis involve greater expense, our clients participate in determining levels of service which provide information for their purposes at acceptable levels of risk. The client and key members of the design team should discuss the issues covered in this report with Kleinfelder so that the issues are understood and applied in a manner consistent with the owner’s budget, tolerance of risk, and expectations for future performance and maintenance.

Recommendations contained in this report are based on our field observations and subsurface explorations, and our present knowledge of the proposed project. It is possible that soil/bedrock or groundwater conditions could vary between or beyond the points explored. If soil/bedrock or groundwater conditions are encountered during retrofit that differ from those described herein,
the client is responsible for ensuring that Kleinfelder is notified immediately so that we may reevaluate the recommendations of this report.

As the geotechnical engineering firm that performed the geotechnical evaluation for this project, Kleinfelder should be retained to evaluate whether the recommendations of this report are properly incorporated in the design of this project and properly implemented during retrofit. This may avoid misinterpretation of the information by other parties and will allow us to review and modify our recommendations if variations in the soil/bedrock conditions are encountered. As a minimum, Kleinfelder should be retained to provide the following continuing services for the project:

- Review the project plans and specifications, including any revisions or modifications
- Observe foundation bearing soils/bedrock for any additional foundations as part of seismic retrofit to evaluate whether conditions are as anticipated

The scope of services for this subsurface exploration and geotechnical report did not include environmental assessments or evaluations regarding the presence or absence of wetlands or hazardous substances in the soil/bedrock, surface water, or groundwater at this site.

Kleinfelder cannot be responsible for interpretation by others of this report or the conditions encountered in the field. Kleinfelder must be retained so that all geotechnical aspects of retrofit will be monitored on a full-time basis by a representative from Kleinfelder. These services provide Kleinfelder the opportunity to observe the actual soil/bedrock and groundwater conditions encountered during construction and to evaluate the applicability of the recommendations presented in this report to the site conditions. If Kleinfelder is not retained to provide these services, we will cease to be the engineer of record for this project and will assume no responsibility for any potential claim during or after construction on this project. If changed site conditions affect the recommendations presented herein, Kleinfelder must also be retained to perform a supplemental evaluation and to issue a revision to our original report.

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

This report was prepared in accordance with the generally accepted standard of practice that existed in Contra Costa County at the time the report was written. No warranty, express or implied, is made.

It is the CLIENT’S responsibility to see that all parties to the project, including the designer, contractor, subcontractors, etc., are made aware of this report in its entirety.

This report may be used only by the client and only for the purposes stated within a reasonable time from its issuance, but in no event later than two years from the date of the report. Land use, site conditions (both on- and off-site), or other factors may change over time, and additional work may be required. Based on the intended use of the report, Kleinfelder may require that additional work be performed and that an updated report be issued. Non-compliance with any of these requirements by the client or anyone else, unless specifically agreed to in advance by Kleinfelder in writing, will release Kleinfelder from any liability resulting from the use of this report by any unauthorized party.
8 REFERENCES


Campbell, K.W. and Bozorgnia, Y. (2008), “NGA Ground Motion Model for the Geometric Mean Horizontal Component of PGA, PGV, PGD and 5% Damped Linear Elastic Response Spectra for Periods Ranging from 0.01 to 10s,” Earthquake Spectra, 24 (1), pp. 139-171.

Cao, T.Q., Petersen, M.D., and Reichle, M.S., (1996), Seismic Hazard Estimate from Background Seismicity, Bulletin of the Seismological Society of America October 1996 vol. 86, no. 5, 1372-1381


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***SITE LOCATION MAP***

**PROJECT NO.: 20181745.001A**

**DRAWN:** 10/17/2017

**DRAWN BY:** D. Ross

**CHECKED BY:** B. Money

**FILE NAME:** 20181745_SLM.mxd

**PROJECT SITE**

**SITE LOCATION MAP**

Contra Costa College Headquarters
500 Court Street
Martinez, California
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LEGEND

Approximate Boring Location

Contra Costa College Headquarters
500 Court Street
Martinez, California

BORING LOCATION MAP

FIGURE 2

APPROXIMATE SCALE: 1 inch = 100 feet

Image courtesy 2017 Microsoft Corporation © 2017 HERE © AND bing
5% Damping

Spectral Acceleration (g)

Period (sec)

Site-Specific Deterministic

DLL

DETERMINISTIC SPECTRA COMPARISON

PROJECT NO. 20181745
DRAWN: 10/04/17
DRAWN BY: Z. Zafr
CHECKED BY: D. Adams
FILE NAME: 4

DISTRICT HEADQUARTERS
CONTRA COSTA COLLEGE
500 COURT STREET
MARTINEZ, CALIFORNIA

www.kleinfelder.com
5% Damping

Spectral Acceleration ($g$)

Period (sec)

- Probabilistic
- Controlling Deterministic
5% Damping

Spectral Acceleration [g]

Period (sec)

DE
80% of Code DE
5% Damping

Spectral Acceleration (g)

Period (sec)

BSE-1N  BSE-2N
The report and graphics key are an integral part of these logs. All data and interpretations in this log are subject to the explanations and limitations stated in the report. Lines separating strata on the logs represent approximate boundaries only. Actual transitions may be gradual or differ from those shown. No warranty is provided as to the continuity of soil or rock conditions between individual sample locations. Logs represent general soil or rock conditions observed at the point of exploration on the date indicated. In general, Unified Soil Classification System designations presented on the logs were based on visual classification in the field and were modified where appropriate based on gradation and index property testing.

### Abbreviations
- **WOH** - Weight of Hammer
- **WOR** - Weight of Rod
- **CL** - Clay
- **ML** - Organic Silts and Very Fine Sands
- **OL** - Organic Clays or Organic Silts of Low Plasticity
- **MH** - Inorganic Silts or Organic Silts of Very Low Plasticity
- **CH** - Inorganic Clays of High Plasticity

### Ground Water Graphics
- **V** - Water Level (level where first observed)
- **V** - Water Level (level after exploration completion)
- **W** - Water Level (additional levels after exploration)
- **O** - Observed Seepage

### Coarse Grained Soils
- **GW** - Well-Graded Gravels, Gravel-Sand Mixtures with Little or No Fines
- **GW-GM** - Well-Graded Gravels, Gravel-Sand Mixtures with Little Clay Fines
- **GW-GC** - Well-Graded Gravels, Gravel-Sand Mixtures with Little Clay Fines
- **GP** - Poorly Graded Gravels, Gravel-Sand Mixtures with Little Fines
- **GP-GM** - Poorly Graded Gravels, Gravel-Sand Mixtures with Little Clay Fines
- **GP-GC** - Poorly Graded Gravels, Gravel-Sand Mixtures with Little Clay Fines
- **GM** - Silty Gravels, Gravel-Silt-Sand Mixtures
- **GC** - Clayey Gravels, Gravel-Sand-Clay Mixtures
- **GC-GM** - Clayey Gravels, Gravel-Sand-Clay Mixtures

### Sands
- **SW** - Well-Graded Sands, Sand-Gravel Mixtures with Little or No Fines
- **SP** - Poorly Graded Sands, Sand-Gravel Mixtures with Little or No Fines
- **SW-SC** - Poorly Graded Sands, Sand-Gravel Mixtures with Little Clay Fines
- **SP-SC** - Poorly Graded Sands, Sand-Gravel Mixtures with Little Clay Fines
- **SM** - Silty Sands, Sand-Silt-Sand Mixtures
- **SC** - Clayey Sands, Sand-Gravel-Clay Mixtures
- **SC-SC** - Clayey Sands, Sand-Gravel-Clay Mixtures

### Silts and Clays
- **ML** - Inorganic Silts and Very Fine Sands, Silty or Clayey Fine Sands, Silts with Slight Plasticity
- **CL** - Inorganic Clays of Low to Medium Plasticity
- **CL-ML** - Inorganic Clays of Low Plasticity, Clayey Clays, Sandy Clays, Silty Clays, Lean Clays
- **OL** - Organic Silts or Organic Silts of Low Plasticity
- **MH** - Inorganic Silts or Organic Silts of Very Low Plasticity
- **CH** - Inorganic Clays of High Plasticity
- **OH** - Organic Clays or Organic Silts of Medium to High Plasticity
CALIFORNIA SAMPLE (blows/ft)  

<table>
<thead>
<tr>
<th>DESCRIPTION</th>
<th>SIEVE SIZE</th>
<th>GRAIN SIZE</th>
<th>APPROXIMATE SIZE</th>
</tr>
</thead>
<tbody>
<tr>
<td>Boulders</td>
<td>&gt;12 in. (304.8 mm.)</td>
<td>&gt;12 in. (304.8 mm.)</td>
<td>Larger than basketball-sized</td>
</tr>
<tr>
<td>Cobbles</td>
<td>3 - 12 in. (76.2 - 304.8 mm.)</td>
<td>3 - 12 in. (76.2 - 304.8 mm.)</td>
<td>Fist-sized to basketball-sized</td>
</tr>
<tr>
<td>Gravel</td>
<td>coarse: 3/4 - 3 in. (19 - 76.2 mm.)</td>
<td>0.19 - 0.75 in. (4.8 - 19 mm.)</td>
<td>Thumb-sized to fist-sized</td>
</tr>
<tr>
<td>Sand</td>
<td>medium: #40 - #10</td>
<td>0.017 - 0.079 in. (0.43 - 2 mm.)</td>
<td>Sugar-sized to rock-sized</td>
</tr>
<tr>
<td>Fines</td>
<td>fine: #200 - #40</td>
<td>0.0029 - 0.017 in. (0.07 - 0.43 mm.)</td>
<td>Flour-sized to sugar-sized</td>
</tr>
</tbody>
</table>

FIELD TEST

<table>
<thead>
<tr>
<th>CONSISTENCY</th>
<th>SPT-N</th>
<th>Pocket Pen (tsf)</th>
<th>UNCONFINED COMPRESSION STRENGTH (Q/psf)</th>
<th>VISUAL / MANUAL CRITERIA</th>
</tr>
</thead>
<tbody>
<tr>
<td>Very Soft</td>
<td>&lt;2</td>
<td>PP &lt; 0.25</td>
<td>&lt;500</td>
<td>Thumb will penetrate more than 1 inch (25 mm). Extrudes between fingers when squeezed.</td>
</tr>
<tr>
<td>Soft</td>
<td>2 - 4</td>
<td>0.25 PP &lt; 0.5</td>
<td>500 - 1000</td>
<td>Thumb will penetrate soil about 1 inch (25 mm). Remolded by light finger pressure.</td>
</tr>
<tr>
<td>Medium Stiff</td>
<td>4 - 8</td>
<td>0.5 PP &lt; 1</td>
<td>1000 - 2000</td>
<td>Thumb will penetrate soil about 1/4 inch (6 mm). Remolded by strong finger pressure.</td>
</tr>
<tr>
<td>Stiff</td>
<td>8 - 15</td>
<td>1 PP &lt; 2</td>
<td>2000 - 4000</td>
<td>Can be imprinted with considerable pressure from thumb.</td>
</tr>
<tr>
<td>Very Stiff</td>
<td>15 - 30</td>
<td>2 PP &lt; 4</td>
<td>4000 - 8000</td>
<td>Thumb will not indent soil but readily indented with thumbnail.</td>
</tr>
<tr>
<td>Hard</td>
<td>&gt;30</td>
<td>4 PP &gt; 8000</td>
<td>Thumb will not indent soil.</td>
<td></td>
</tr>
</tbody>
</table>

PLASTICITY

<table>
<thead>
<tr>
<th>DESCRIPTION</th>
<th>LL</th>
<th>FIELD TEST</th>
</tr>
</thead>
<tbody>
<tr>
<td>Non-plastic</td>
<td>NP</td>
<td>The thread cannot be rolled at any water content.</td>
</tr>
<tr>
<td>Low (L)</td>
<td>&lt; 30</td>
<td>The thread can barely be rolled and the lump or thread cannot be formed when drier than the plastic limit.</td>
</tr>
<tr>
<td>Medium (M)</td>
<td>30 - 50</td>
<td>The thread is easy to roll and not much time is required to reach the plastic limit. The thread cannot be rolled after reaching the plastic limit. The lump or thread crumbles when drier than the plastic limit.</td>
</tr>
<tr>
<td>High (H)</td>
<td>&gt; 50</td>
<td>It takes considerable time rolling and the soil to reach the plastic limit. The lump or thread can be formed without crumbling when drier than the plastic limit.</td>
</tr>
</tbody>
</table>

ANGULARITY

<table>
<thead>
<tr>
<th>DESCRIPTION</th>
<th>CRITERIA</th>
</tr>
</thead>
<tbody>
<tr>
<td>Angular</td>
<td>Particles have sharp edges and relatively plane sides with unpolished surfaces.</td>
</tr>
<tr>
<td>Subangular</td>
<td>Particles are similar to angular description but have rounded edges.</td>
</tr>
<tr>
<td>Subrounded</td>
<td>Particles have nearly plane sides but have well-rounded corners and edges.</td>
</tr>
<tr>
<td>Rounded</td>
<td>Particles have smooth curved sides and no edges.</td>
</tr>
</tbody>
</table>

FROM TERZAGHI AND PECK, 1948; LAMBE AND WHITMAN, 1969; FHWA, 2002; AND ASTM D2488

FIGURE A-2

Contra Costa College Headquarters  
500 Court Street  
Martinez, California
<table>
<thead>
<tr>
<th>Depth (feet)</th>
<th>Lithologic Description</th>
<th>Sample Type</th>
<th>Sample Number</th>
<th>USCSSymbol</th>
<th>Water Content (%)</th>
<th>Liquid Limit</th>
<th>Plasticity Index (NP=NonPlastic)</th>
<th>Dry Unit Wt. (pcf)</th>
<th>Passing #4 (%)</th>
<th>Passing #200 (%)</th>
<th>Passing #200 RPM (%)</th>
<th>Additional Tests/Remarks</th>
</tr>
</thead>
<tbody>
<tr>
<td>0-3</td>
<td>Asphalt</td>
<td></td>
<td>B1</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>HSA to 7.5 feet</td>
</tr>
<tr>
<td>3-6</td>
<td>Aggregate Base</td>
<td></td>
<td>B2</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>5-8</td>
<td>Fill</td>
<td></td>
<td>B3</td>
<td>A5</td>
<td>BC=5</td>
<td>23.1</td>
<td>16°</td>
<td>105.7</td>
<td>80</td>
<td>46</td>
<td>26</td>
<td>Switch to Mud Rotary</td>
</tr>
<tr>
<td>8-12</td>
<td>Silty Sand with Clay (SM)</td>
<td></td>
<td>B4</td>
<td>B5/5</td>
<td>BC=5</td>
<td>8</td>
<td>26°</td>
<td>105.7</td>
<td>37</td>
<td>46</td>
<td>26</td>
<td></td>
</tr>
<tr>
<td>10-13</td>
<td>Fat Clay (CH)</td>
<td></td>
<td>C5</td>
<td>C5/5</td>
<td>BC=5</td>
<td>12</td>
<td>30°</td>
<td>105.7</td>
<td>46</td>
<td>46</td>
<td>30</td>
<td></td>
</tr>
<tr>
<td>12-15</td>
<td>Sandy Lean Clay (CL)</td>
<td></td>
<td>C6</td>
<td>C6</td>
<td>BC=3</td>
<td>23.1</td>
<td>16°</td>
<td>105.7</td>
<td>80</td>
<td>46</td>
<td>26</td>
<td></td>
</tr>
<tr>
<td>14-17</td>
<td>Medium plasticity, brown, moist</td>
<td></td>
<td>C7</td>
<td>C7/7</td>
<td>BC=6</td>
<td>9</td>
<td>6°</td>
<td>105.7</td>
<td>80</td>
<td>46</td>
<td>26</td>
<td></td>
</tr>
<tr>
<td>17-20</td>
<td>Lean Clay with Sand (CL)</td>
<td></td>
<td>C8</td>
<td>C8/8</td>
<td>BC=6</td>
<td>9</td>
<td>10°</td>
<td>105.7</td>
<td>80</td>
<td>46</td>
<td>26</td>
<td></td>
</tr>
<tr>
<td>20-23</td>
<td>Sandy Lean Clay (CL)</td>
<td></td>
<td>C9</td>
<td>C9/9</td>
<td>BC=5</td>
<td>12</td>
<td>10°</td>
<td>105.7</td>
<td>80</td>
<td>46</td>
<td>30</td>
<td></td>
</tr>
<tr>
<td>23-27</td>
<td>Fat Clay with Sand (CH)</td>
<td></td>
<td>C10</td>
<td>A10/10</td>
<td>BC=8</td>
<td>12</td>
<td>10°</td>
<td>105.7</td>
<td>71</td>
<td>52</td>
<td>30</td>
<td></td>
</tr>
<tr>
<td>27-30</td>
<td>Sandy Silt (ML)</td>
<td></td>
<td>B10/B10</td>
<td>C10/C10</td>
<td>BC=8</td>
<td>12</td>
<td>15°</td>
<td>105.7</td>
<td>71</td>
<td>52</td>
<td>30</td>
<td></td>
</tr>
</tbody>
</table>
Sandy SILT (ML): non-plastic, olive yellow, moist, fine sand

light brownish gray, soft

Clayey SANDSTONE: moist, decomposed

The boring was terminated at approximately 50.5 ft. below ground surface. The boring was backfilled with neat cement grout on August 30, 2017.

GROUNDWATER LEVEL INFORMATION:
Groundwater was observed at approximately 7.5 ft. below ground surface during drilling.

GENERAL NOTES:
<table>
<thead>
<tr>
<th>Exploration ID</th>
<th>Depth (ft.)</th>
<th>Sample No.</th>
<th>Sample Description</th>
<th>Water Content (%)</th>
<th>Sieve Analysis (%)</th>
<th>Atterberg Limits</th>
<th>Additional Tests</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>B-1</td>
<td>3.5 - 5.0</td>
<td>B4</td>
<td>FAT CLAY (CH)</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>B-1</td>
<td>5.5 - 6.0</td>
<td>B5</td>
<td>SANDY LEAN CLAY (CL)</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>B-1</td>
<td>6.0 - 6.5</td>
<td>C5</td>
<td>SANDY LEAN CLAY (CL)</td>
<td>23.1</td>
<td>105.7</td>
<td></td>
<td></td>
</tr>
<tr>
<td>B-1</td>
<td>10.0 - 11.5</td>
<td>C6</td>
<td>SANDY LEAN CLAY (CL)</td>
<td></td>
<td></td>
<td>46</td>
<td>20</td>
</tr>
<tr>
<td>B-1</td>
<td>16.0 - 16.5</td>
<td>C7</td>
<td>LEAN CLAY WITH SAND (CL)</td>
<td></td>
<td></td>
<td>46</td>
<td>17</td>
</tr>
<tr>
<td>B-1</td>
<td>23.0 - 23.5</td>
<td>C9</td>
<td>FAT CLAY WITH SAND (CH)</td>
<td></td>
<td></td>
<td>TXUU: c=0.89 ksf</td>
<td></td>
</tr>
<tr>
<td>B-1</td>
<td>31.0 - 31.5</td>
<td>C10</td>
<td>SANDY SILT (ML)</td>
<td></td>
<td></td>
<td>TXUU: c=1.07 ksf</td>
<td></td>
</tr>
<tr>
<td>B-1</td>
<td>45.0 - 46.5</td>
<td>C13</td>
<td>SANDY SILT</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Refer to the Geotechnical Evaluation Report or the supplemental plates for the method used for the testing performed above.
NP = Non-Plastic
NA = Not Available

**PROJECT NO.:** 20181745
**DRAWN BY:** DR
**CHECKED BY:** BM
**DATE:** 9/5/2017
**REVISED:** 10/17/2017
For classification of fine-grained soils and fine-grained fraction of coarse-grained soils.

<table>
<thead>
<tr>
<th>Exploration ID</th>
<th>Depth (ft.)</th>
<th>Sample Number</th>
<th>Sample Description</th>
<th>Passing #200</th>
<th>LL</th>
<th>PL</th>
<th>PI</th>
</tr>
</thead>
<tbody>
<tr>
<td>B-1</td>
<td>3.5 - 5</td>
<td>B4</td>
<td>FAT CLAY (CH)</td>
<td>NM</td>
<td>57</td>
<td>20</td>
<td>37</td>
</tr>
<tr>
<td>B-1</td>
<td>6 - 6.5</td>
<td>C5</td>
<td>SANDY LEAN CLAY (CL)</td>
<td>NM</td>
<td>46</td>
<td>20</td>
<td>26</td>
</tr>
<tr>
<td>B-1</td>
<td>10 - 11.5</td>
<td>C6</td>
<td>SANDY LEAN CLAY (CL)</td>
<td>NM</td>
<td>46</td>
<td>17</td>
<td>29</td>
</tr>
<tr>
<td>B-1</td>
<td>25 - 26.5</td>
<td>C9</td>
<td>FAT CLAY WITH SAND (CH)</td>
<td>87</td>
<td>52</td>
<td>22</td>
<td>30</td>
</tr>
</tbody>
</table>

Testing performed in general accordance with ASTM D4318.

NP = Nonplastic
NA = Not Available
NM = Not Measured
TRIAXIAL COMPRESSION TEST (UU)

Specimen No. | 1
--- | ---
Diameter, in | D₀ 2.392
Height, in | H₀ 5.011
Water Content, % | ω₀ 21.5
Dry Density, lbs/ft³ | ρ₀ 99.9
Saturation, % | S₀ 98
Void Ratio | e₀ 0.590
Minor Principal Stress, ksf | σ₃ 0.79
Maximum Deviator Stress, ksf | (σ₁−σ₃)max 3.32
Time to (σ₁−σ₃)max min | τₐ 30.00
Deviator Stress @ 15% Axial Strain, ksf | (σ₁−σ₃)₁₅% 3.32
Ultimate Deviator Stress, ksf | (σᵢ)ₘₐₓ na
Rate of strain, %/min | εᵢ 0.50
Axial Strain at Failure, % | ε₁ 15.00

Description of Specimen: Olive Yellow Lean Clay

Amount of Material Finer than the No. 200, %: nm

LL: nm PL: nm PI: nm Gₛᵢ: 2.70 Assumed Specimen Type: Undisturbed Test Method: ASTM D2850

Membrane correction applied

Boring: B-1 Remarks: nm= not measured, na = not applicable
Sample: C7
Depth, ft: 16.0-16.5
Test Date: 9/13/2017

Contra Costa College Headquarters
500 Court Street
Martinez, California
### Specimen Shear Picture

![Specimen Shear Picture](image)

### Specimen Properties

<table>
<thead>
<tr>
<th>Specimen No.</th>
<th>1</th>
</tr>
</thead>
<tbody>
<tr>
<td>Diameter, in</td>
<td>( D_0 ) = 2.845</td>
</tr>
<tr>
<td>Height, in</td>
<td>( H_0 ) = 5.879</td>
</tr>
<tr>
<td>Water Content, %</td>
<td>( \omega_0 ) = 29.1</td>
</tr>
<tr>
<td>Dry Density, lbs/ft(^3)</td>
<td>( \gamma_d ) = 94.5</td>
</tr>
<tr>
<td>Saturation, %</td>
<td>( S_0 ) = 100</td>
</tr>
<tr>
<td>Void Ratio</td>
<td>( e_0 ) = 0.783</td>
</tr>
<tr>
<td>Minor Principal Stress, ksf</td>
<td>( \sigma_3 ) = 1.01</td>
</tr>
<tr>
<td>Maximum Deviator Stress, ksf</td>
<td>( (\sigma_1-\sigma_3)_{\text{max}} ) = 1.78</td>
</tr>
<tr>
<td>Time to ( (\sigma_1-\sigma_3)_{\text{max}} ), min</td>
<td>( t_f ) = 7.25</td>
</tr>
<tr>
<td>Deviator Stress @ 15% Axial Strain, ksf</td>
<td>( (\sigma_1-\sigma_3)_{15%} ) = 1.67</td>
</tr>
<tr>
<td>Ultimate Deviator Stress, ksf</td>
<td>( (\sigma_1-\sigma_3)_{\text{ult}} ) = na</td>
</tr>
<tr>
<td>Rate of strain, %/min</td>
<td>( \dot{\varepsilon} ) = 0.50</td>
</tr>
<tr>
<td>Axial Strain at Failure, %</td>
<td>( \varepsilon_f ) = 3.63</td>
</tr>
</tbody>
</table>

### Description of Specimen

Olive Yellow Lean Clay

### Membrane correction applied

- **Boring:** B-1
- **Sample:** C8
- **Depth, ft:** 23.0
- **Test Date:** 9/13/2017

**Remarks:** nm = not measured, na = not applicable

---

**TRIAXIAL COMPRESSION TEST (UU)**

**Contra Costa College Headquarters**

500 Court Street
Martinez, California

**ENTRY BY:** S. Rader

**CHECKED BY:** C. Pollack

**DATE:** 9/13/2017

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TRIAXIAL COMPRESSION TEST (UU)

Contra Costa College Headquarters
500 Court Street
Martinez, California

S. Rader
C. Pollack

ENTRY BY: S. Rader
CHECKED BY: C. Pollack

DATE: 9/13/2017

PROJECT NO. 20181745.001A

Contra Costa College Headquarters
500 Court Street
Martinez, California

Specimen No.

Diameter, in
Height, in
Water Content, %
Dry Density, lbs/ft³
Saturation, %
Void Ratio
Minor Principal Stress, ksf
Maximum Deviator Stress, ksf
Time to (σ₁-σ₃)max, min
Deviator Stress @ 15% Axial Strain, ksf
Ultimate Deviator Stress, ksf
Rate of strain, %/min
Axial Strain at Failure, %

Initial

2.845
5.879
29.1
94.5
100
0.783
1.01
2.15
18.08
1.67
na
0.50
9.04

Description of Specimen: Olive Yellow Lean Clay

Amount of Material Finer than the No. 200, %: nm

LL: nm PL: nm PL: nm Gφ: 2.70 Assumed

Membrane correction applied

Boring: B-1
Sample: C8
Depth, ft: 23.0
Test Date: 9/13/2017

Remarks: nm = not measured, na = not applicable