GEOLOGIC AND SIESMIC HAZARDS
ASSESSMENT AND GOTECHNICAL
INVESTIGATION REPORT
C-608 PE/KINESIOLOGY RENOVATION PROJECT
CONTRA COSTA COMMUNITY COLLEGE
2600 MISSION BELL DRIVE
SAN PABLO, CALIFORNIA
PROJECT #20181293.001A

SEPTEMBER 8, 2017

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THE SPECIFIC PROJECT FOR WHICH THIS REPORT WAS PREPARED.
September 8, 2017  
Project No: 20181293.001A

Mr. P.J. Roach  
Facilities Project Manager  
Contra Costa Community College District  
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Martinez, California 94553  
Email: PRoach@4cd.edu

SUBJECT: Geologic and Seismic Hazards and Geotechnical Investigation Report  
Proposed C-608 PE/Kinesiology Renovation Project  
Contra Costa Community College  
2600 Mission Bell Drive  
San Pablo, California

Dear Mr. Roach:

The enclosed report includes the results of our geologic and seismic hazards assessment and geotechnical engineering investigation for the planned project. It provides a description of the investigation performed and geotechnical recommendations for design and construction of the proposed improvements for the proposed C-608 PE/Kinesiology Renovation Project at Contra Costa Community College in San Pablo, California. The report also includes a site-specific ground motion seismic analysis.

In summary, it is Kleinfelder’s opinion that the site is suitable for the proposed construction provided the recommendations presented in this report are followed. The proposed building and temporary locker room facilities can be supported by shallow spread foundations and the outdoor workout facility can be supported by a mat slab foundation. The site was underlain at shallow depth by potentially expansive clay soils. As a result, surface soils may need to be cement/lime treated and spread footing depths will need to extend to increased depths to reduce the potential for differential vertical movement due to shrinkage and swelling of the clay soils caused by changes in moisture content.

Design plans and specifications should be reviewed by Kleinfelder prior to their issuance for conformance with the general intent of the recommendations presented in the enclosed report.
We appreciate the opportunity to provide our services to you on this project, and we trust this report meets your needs at this time. If you have any questions concerning the information presented in this report, or related project matters, please contact Don Adams at (775) 691-0287.

Sincerely,

KLEINFELDER, INC.

Rebecca L. Money, PE, GE
Senior Geotechnical Engineer

for Don Adams, PE
Project Manager

Byron Anderson PG, CEG #2343
Principal Engineering Geologist
# TABLE OF CONTENTS

<table>
<thead>
<tr>
<th>Section</th>
<th>Page</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>INTRODUCTION</td>
</tr>
<tr>
<td>1.1</td>
<td>PROPOSED CONSTRUCTION</td>
</tr>
<tr>
<td>1.2</td>
<td>SITE LOCATION AND DESCRIPTION</td>
</tr>
<tr>
<td>1.3</td>
<td>PURPOSE AND SCOPE OF SERVICES</td>
</tr>
<tr>
<td>1.4</td>
<td>PREVIOUS INVESTIGATIONS</td>
</tr>
<tr>
<td>1.5</td>
<td>SITE RECONNAISSANCE</td>
</tr>
<tr>
<td>1.6</td>
<td>SUBSURFACE INVESTIGATION</td>
</tr>
<tr>
<td>1.7</td>
<td>LABORATORY TESTING</td>
</tr>
<tr>
<td>2</td>
<td>SITE DESCRIPTION</td>
</tr>
<tr>
<td>3</td>
<td>SUBSURFACE CONDITIONS</td>
</tr>
<tr>
<td>4</td>
<td>GEOLOGIC &amp; SEISMIC FINDINGS</td>
</tr>
<tr>
<td>4.1</td>
<td>REGIONAL GEOLOGY</td>
</tr>
<tr>
<td>4.2</td>
<td>AREA AND SITE GEOLOGY</td>
</tr>
<tr>
<td>4.3</td>
<td>FAULTING AND SEISMICITY</td>
</tr>
<tr>
<td>4.4</td>
<td>2016 CBC SITE CLASS</td>
</tr>
<tr>
<td>4.5</td>
<td>2016 CBC SEISMIC DESIGN PARAMETERS</td>
</tr>
<tr>
<td>4.6</td>
<td>FAULT-RELATED GROUND SURFACE RUPTURE</td>
</tr>
<tr>
<td>4.7</td>
<td>SEISMICALLY-INDUCED GROUND FAILURE</td>
</tr>
<tr>
<td>4.7.1</td>
<td>Liquefaction and Lateral Spreading</td>
</tr>
<tr>
<td>4.7.2</td>
<td>Dynamic (Seismic) Compaction</td>
</tr>
<tr>
<td>4.8</td>
<td>EXPANSIVE SOILS</td>
</tr>
<tr>
<td>4.9</td>
<td>EXISTING FILL</td>
</tr>
<tr>
<td>4.10</td>
<td>LANDSLIDES</td>
</tr>
<tr>
<td>4.11</td>
<td>TSUNAMIS, SEICHES, AND FLOODING</td>
</tr>
<tr>
<td>4.12</td>
<td>NATURALLY-OCCURRING ASBESTOS</td>
</tr>
<tr>
<td>4.13</td>
<td>SOIL CORROSION</td>
</tr>
<tr>
<td>4.14</td>
<td>RADON GAS</td>
</tr>
<tr>
<td>4.15</td>
<td>VOLCANIC ACTIVITY</td>
</tr>
<tr>
<td>5</td>
<td>CONCLUSIONS AND RECOMMENDATIONS</td>
</tr>
<tr>
<td>5.1</td>
<td>GENERAL</td>
</tr>
<tr>
<td>5.2</td>
<td>GEOLOGIC AND SEISMIC HAZARDS</td>
</tr>
<tr>
<td>5.3</td>
<td>EXISTING SITE FILL</td>
</tr>
<tr>
<td>5.4</td>
<td>EARTHWORK</td>
</tr>
<tr>
<td>5.4.1</td>
<td>Expansive Soils</td>
</tr>
<tr>
<td>5.4.2</td>
<td>General</td>
</tr>
<tr>
<td>5.4.3</td>
<td>Site Preparation and Grading</td>
</tr>
<tr>
<td>5.4.4</td>
<td>Engineered Fill Materials</td>
</tr>
<tr>
<td>5.4.5</td>
<td>Fill Compaction Criteria</td>
</tr>
<tr>
<td>5.4.6</td>
<td>Weather/Moisture Considerations</td>
</tr>
<tr>
<td>5.4.7</td>
<td>Excavation and Backfill</td>
</tr>
<tr>
<td>5.5</td>
<td>FOUNDATIONS</td>
</tr>
<tr>
<td>5.5.1</td>
<td>Subgrade Preparation</td>
</tr>
<tr>
<td>5.5.2</td>
<td>Allowable Bearing Pressure</td>
</tr>
</tbody>
</table>
5.5.3 Resistance to Lateral Loads ................................................................. 29
5.6 RETAINING WALLS .................................................................................. 29
   5.6.1 Lateral Earth Pressures ................................................................. 29
5.7 POLE FOUNDATION DESIGN ............................................................ 30
5.8 SLABS ON GRADE .............................................................................. 32
   5.8.1 Subgrade Preparation ................................................................. 32
   5.8.2 Concrete Floor Slabs ................................................................. 32
   5.8.3 Exterior Concrete Flatwork ...................................................... 33
5.9 SITE DRAINAGE ................................................................................... 34
5.10 PAVEMENTS ....................................................................................... 34
   5.10.1 Asphalt Concrete Pavement ................................................... 35
   5.10.2 Portland Cement Concrete Pavement Sections ....................... 36
   5.10.3 Unstable Subgrade ................................................................. 38
   5.10.4 Variations in Subgrade Materials ............................................ 38

6 ADDITIONAL SERVICES AND LIMITATIONS ........................................... 39
   6.1 ADDITIONAL SERVICES .................................................................. 39
   6.2 LIMITATIONS .................................................................................. 39

7 REFERENCES .......................................................................................... 42

FIGURES
Figure 1 – Site Location Map
Figure 2 – Site Plan, Boring Locations, and Site Geology Map
Figure 3 – Regional Geologic Map
Figure 4 – Area Geology Map
Figure 5 – Geologic Cross Section A-A’

APPENDIX A – Boring Logs
   Figure A-1 - Graphics Key
   Figure A-2 - Soil Description Key
   Figures A-3 through A-5 - Boring Logs

APPENDIX B – Laboratory Test Results
   Figure B-1 – Laboratory Test Result Summary
   Figure B-2 – Atterberg Limits
   Figure B-3 – R-Value
   Figure B-4 – Triaxial Compression Test (UU)
   Corrosion Test Results (Sunland Analytical)

APPENDIX C – Logs from Previous Studies
APPENDIX D – Site-Specific Ground Motion Seismic Evaluation
APPENDIX E – GBA Important Info about Your Geotechnical Report
1 INTRODUCTION

This report presents the results of our geologic and seismic hazards assessment a geotechnical investigation performed for the proposed C-608 PE/Kinesiology Renovation at Contra Costa College (CCC) located at 2600 Mission Bell Drive in San Pablo, California. A Site Location Map showing the general location of the school campus is presented on Figure 1. The approximate limits of the project are shown on Figure 2.

This report has been prepared for submittal with supporting design documents to the Division of the State Architect (DSA), as required for new construction of public schools and essential services facilities. This report addresses the potential geologic and seismic hazards that could impact the site as required by the California Geological Survey (CGS) Note 48, which may be incorporated into future projects with appropriate updates of the information presented herein. The updates should include site-specific borings and/or Cone Penetration Tests (CPTs), reconnaissance for individual projects by qualified personnel, and evaluation of the data to confirm that it is consistent with this report.

1.1 PROPOSED CONSTRUCTION

Project plans entitled “C-608 Site-Building-Plans,” created by Lionakis and dated May 11, 2017 were provided by Contra Costa Community College District (District). Based on these drawings and communication with the District, the proposed renovations include construction of an approximately 4,000 square-foot, two-story addition to the southeast side of the existing gymnasium, and construction of an approximately 2,000 square-foot, new outdoor workout area located south of the existing Gym Annex. Based upon review of project plans, we understand the project also includes resurfacing of the tennis courts, interior renovations of the gymnasium seating and existing men’s and women’s locker rooms, addition of a temporary locker room facility, and other hardscape and landscape improvements. Final construction and/or grading plans were not available at the time of this writing. Excavations for utilities are expected to extend to depths of approximately 5 to 8 feet below the ground surface.
1.2 SITE LOCATION AND DESCRIPTION

According to the U.S. Geological Survey (USGS, 1999) 7½-Minute Richmond Topographic Quadrangle map, the existing ground elevation at the site ranges between about 40 and 60 feet above mean sea level. The coordinates at the center of the planned C-608 PE/Kinesiology expansion location are approximately:

Latitude: 37.9689° N
Longitude: -122.3402° W

1.3 PURPOSE AND SCOPE OF SERVICES

The purpose of this investigation was to assess potential geologic and seismic hazards that could impact the proposed project and to explore and evaluate the subsurface soils at the site and provide geotechnical recommendations relating to grading, foundations, and drainage. The scope of work was outlined in our proposal dated May 26, 2017 (File No. LOCALMKT/PLE17P60126). This scope included a geologic and seismic hazard assessment, a site-specific ground motion analysis, site reconnaissance, subsurface investigation, laboratory testing, engineering analyses of the data gathered, and preparation of this report which summarizes our findings, conclusions and recommendations.

Our scope of services for the geologic and seismic hazards evaluation included review of readily available published geotechnical data and unpublished site-specific geologic and seismic evaluations performed by Kleinfelder previously, and the subsurface exploration and laboratory data obtained during our geotechnical engineering investigation. The objectives of this evaluation are the identification and assessment of potential geologic and seismic hazards at the site in accordance with the requirements of the current California Code of Regulations, Title 24, 2016 CBC using guidelines outlined by the CGS. In addition to these requirements, this report has been prepared in accordance with the guidelines established in the following documents:

- California Department of Conservation, Division of Mines and Geology (DMG, currently known as the California Geological Survey [CGS]) Special Publication 117A (Guidelines for Evaluating and Mitigating Seismic Hazards);
- CGS Note 41 (Guidelines for Reviewing Geologic Reports);
• DMG Special Publication 42 (Fault-Rupture Hazard Zones in California);

• DMG Note 42 (Guidelines to Geologic/Seismic Reports);

• DMG Note 44 (Recommended Guidelines for Preparing Engineering Geologic Reports); and

• CGS Note 48 (Checklist for the Review of Engineering Geology and Seismology Reports for California Public Schools, Hospitals, and Essential Services Buildings).

1.4 PREVIOUS INVESTIGATIONS

Kleinfelder has performed numerous subsurface investigations at the CCC campus. These included:

• A liquefaction investigation specifically around the existing gymnasium with results of that evaluation presented in a report titled Geotechnical Investigation to Evaluate Depth of Pleistocene Age Material and Liquefaction in the Vicinity of the Gymnasium at Contra Costa College in San Pablo, California, dated October 15, 2007.

• A campus-wide seismic study titled Master Plan Seismic Study, Contra Costa College Campus, San Pablo, California, dated July 15, 2009 (Project No. 80412/Report/PLE9R266) where potential fault activity was evaluated in the vicinity of 13 campus buildings that the District planned on modifying or replacing at the time.

• Two fault studies were completed to identify additional areas of habitable zones for additional building improvements. The results of these investigations and evaluations were provided in reports titled Subsurface Fault Investigation, Lower Parking Area, Contra Costa Community College, San Pablo, California, dated November 16, 2016 and Subsurface Fault Investigation, Proposal C-4001 Campus Safety Center, Contra Costa Community College, San Pablo, California, dated June 29, 2016.

• A geotechnical and geologic hazards assessment for the proposed Campus Safety Center located in the parking lot to the north east of the gymnasium. Findings from this evaluation are included in two reports titled Geotechnical Investigation Report, Campus Safety Center, Contra Costa Community College, 2600 Mission Bell Drive, San Pablo, California, dated March 17, 2017 and Geologic and Seismic Hazards Assessment Report, Planned
1.5 SITE RECONNAISSANCE

On July 19, 2017, a site reconnaissance was performed of the project area and its immediate vicinity and a District representative was met on site. The purpose of this visit was to observe the existing site conditions, check site accessibility, identify site features that could impact our field investigation and/or the project design and construction, identify exploratory boring locations, and mark for Underground Service Alert (USA) to gain clearance from existing underground utilities. Additionally, a private utility locator reviewed each identified exploratory boring location and marked any utilities encountered. The visit with a District representative also reviewed and identified sensitive vegetation, landscaping, and traffic-flow disturbance concerns involved with our field investigation. It was noted that no significant changes had been made to the project area since Kleinfelder’s March 2017 visit. All issues were discussed and resolved on-site during this meeting.

1.6 SUBSURFACE INVESTIGATION

The subsurface conditions at the site were explored on July 28, 2017 by drilling three borings (B-1, B-2, and B-3) to depths of approximately 31½, 3, and 11½ feet, respectively, below existing site grade. Each location was cleared by USA and marked by a private utility locator on July 19, 2017. Boring B-2 was advanced using only a 3-inch-diameter hand auger due to the close proximity of multiple subsurface utilities to the exploration location. The other borings (B-1 and B-3) were drilled using a truck-mounted, Diedrich D120 drill rig equipped with 6-inch-diameter hollow-stem augers and 4-inch-diameter solid-stem augers. The locations of the borings drilled for this investigation are shown on Figure 2.

The location of the current borings were chosen based on the proximity to previous nearby explorations in order to meet the requirements of the Division of State Architect (DSA), which requires at least two exploration points per building and at least one for every 5,000 square feet of plan area, and based upon geotechnical evaluation needs according to the proposed design. The borings were located in the field by visual sighting and/or pacing from existing site features.
Therefore, the locations of the borings shown on Figure 2 should be considered approximate and may vary from that indicated on the figure. The final locations of the borings were selected in order to avoid subsurface utility conflicts, overhead clearance conflicts, areas with ponded/pooling water, and unapproved restrictions in traffic flow.

A Kleinfelder professional maintained logs of the borings and visually classified the soils encountered according to the Unified Soil Classification System (see Figure A-1). Selected bulk and driven samples were retrieved, placed in bags or sealed in sample tubes, and transported to our laboratory for further evaluation and testing. The number of blows necessary to drive a Standard Penetration Test (SPT) sampler or California-type sampler was recorded. Soil classifications made in the field from samples were in accordance with ASTM Method D2488. These classifications were re-evaluated in the laboratory after further examination and testing in accordance with ASTM D2487. Sample classifications and other related information were recorded on the boring logs. Keys to the soil descriptions and symbols used on the boring logs are presented on Figures A-1 and A-2 in Appendix A. Boring logs are presented on Figures A-3, A-4 and A-5 and are presented as stick logs on the cross sections shown on Figure 5.

After the borings were completed, the Borings B-1 and B-2 were backfilled with cement grout per Contra Costa County requirements, except the upper 5 feet of Boring B-2 was backfilled with drill cuttings. The pavement was patched with concrete that was colored with lamp-black. Boring B-3 was drilled on native soils and was backfilled with cuttings. Excess soil cuttings were contained in drums and removed from the site by the drilling subcontractor.

1.7 LABORATORY TESTING

Kleinfelder performed the following laboratory tests on selected soil samples to evaluate certain physical and engineering characteristics. The laboratory test results are presented in Appendix B.

- Moisture Content (ASTM 2216)
- Dry Density (ASTM 7263)
- Undrained-Unconsolidated Triaxial Shear (ASTM 2850)
- Expansion Index (ASTM 4829)
- R-value (ASTM 2844)
- Sieve Analysis (ASTM D1140)
- Atterberg Limits (ASTM D4318)
A series of chemical tests were performed on a selected sample of the near-surface soils to estimate pH, resistivity, soluble sulfate, and chloride contents for preliminary corrosion assessment. The sample was tested in general accordance with California Test Methods 643, 422, and 417 for pH and minimum resistivity, and soluble chlorides and soluble sulfates, respectively. Test results may be used by a qualified corrosion engineer to evaluate the general corrosion potential with respect to construction materials. The tests were performed by Sunland Analytical of Rancho Cordova, California. The results of the tests are presented in Table 4.2 of Section 4.14 of the report and are included in Appendix B.
2 SITE DESCRIPTION

The Contra Costa College campus is located at 2600 Mission Bell Drive in San Pablo, California. The campus is bound by Mission Bell Drive and Campus Drive to the west, El Portal Drive to the south, Castro Drive and Campus Drive to the east and Campus Drive to the north. Soccer, football and track, baseball and softball fields, tennis courts, the gymnasium, and the pool are located in the southern portion of campus. The majority of the educational classrooms are located in the northern portion of campus. Parking lots are situated in the center of the campus and along the edges to the north and east.

The proposed CE-608 PE/Kinesiology expansion is located at the existing gymnasium located within the southern portion of the campus. The expansion consists of two stories covering approximately 4,000 square feet added to the southeast side of the existing gym. The expansion area is currently an asphalt-concrete paved outdoor patio area, and detached bathroom building.

The proposed outdoor workout area is located within the central-south portion of the campus southwest of the existing gym annex building, and is currently an unoccupied area with minor landscaping and light poles.

The proposed temporary locker rooms are located southwest of the current tennis courts, in an unoccupied area with vegetation and landscaping including mature palm trees. The tennis courts are proposed to be resurfaced as part of this project.

Based on the U.S. Geological Survey (USGS, 2015) 7½-Minute Richmond Topographic Quadrangle Map, the existing ground elevation at the campus ranges from about 40 to 60 feet (North American Vertical Datum of 1988, NAVD88).
3 SUBSURFACE CONDITIONS

The subsurface conditions described herein are based on the soil and groundwater conditions encountered during the current and previous geotechnical investigations in the site area.

According to the observations made during the explorations and borings performed for this investigation, portions of the proposed building expansion, locker rooms, and outdoor workout facility are covered by landscaping and pavement. Boring B-1 encountered an approximately 2-inch-thick layer of asphalt concrete pavement. Subsurface soils encountered beneath the asphalt concrete included fill to a depth of approximately 6 feet deep consisting of very stiff, sandy lean clay. Beneath the fill, alternating layers of medium stiff to stiff, lean and fat clays were generally observed to the termination depth of the boring at about 31½ feet below ground surface. The exception was a 1½ foot loose, poorly-graded sand with clay layer about observed at a depth of approximately 21 feet below ground surface.

Boring B-2, located near the proposed outdoor workout facility, was located near multiple subsurface utilities, and was excavated using a hand auger to prevent damaging the utilities and for staff safety. The boring encountered fill to a depth of approximately 3 feet consisting of stiff, lean and sandy lean clays and poorly-graded gravel with clay. The boring was terminated due to hand auger refusal at a depth of about 3 feet.

Boring B-3, located near the proposed temporary locker rooms, encountered approximately 7 feet of fill consisting of stiff to very stiff, lean and sandy lean clay. Fat clay was encountered below the fill to the maximum depth explored of about 11½ feet.

Previous borings were performed in the area of the gym expansion in 2007 (Borings B-2 and B-3), which were utilized for this investigation. Boring B-2 (2007) encountered about 3 inches of asphalt concrete pavement underlain by about 8 inches of aggregate base material. This was underlain by fat clay, lean clay, and sandy lean clay to a depth of about 51½ feet. The exception was a 6½-foot-thick layer of silty sand located at a depth of about 41 feet. Groundwater was encountered in this boring at a depth of approximately 14 feet. Boring B-3 (2007) encountered about 2 inches of asphalt concrete pavement underlain by about 4 inches of aggregate base material. This was underlain by lean clay and sandy lean clay to a depth of about 51½ feet. The exceptions were a ½-foot-thick layer of well-graded sand located at a depth of about 22 feet and
a 2-foot-thick layer of silty sand located at a depth of about 46 feet. Groundwater was encountered in this boring at a depth of approximately 13 feet.

Geologic fault trenches performed previously in the site area by Kleinfelder (2007) indicate the presence of a 2- to 5-foot-thick layer of undocumented fill that is underlain by Holocene, fine-grained, basin deposits. Near the eastern ends of the geologic trenches, modern stream channel deposits associated with Rheem Creek were encountered. The undocumented fill was generally comprised of layered and mottled soil. The fill was underlain by nearly horizontal, fine-grained basin deposits such as dark gray to black lean clay, light yellowish brown sandy silt, dark brown to black lean clay, dark grayish brown lean clay, gray brown sandy silt, and a light olive lean clay with silt that extended to base of the trenches, 15 to 18 feet in depth. As noted above, the only exception to the basin deposits encountered beneath the fill was located at the northeast end of Trench T-7 and was fine- to coarse-grained stream channel deposits that consisted of relatively “clean” laminated sand with gravel and trace silt. This Trench (T-7) is located approximately 200 feet east from the proposed project and the channel deposits encountered in the trench are not expected to extend beneath the new renovations and additions.

Groundwater was not observed in the current explorations. However, groundwater was observed and encountered in our March 2017 explorations at a depth of 15 feet and in our 2007 investigations at a depth of 13 feet. 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 and groundwater conditions encountered in the borings from this investigation and the borings drilled for previous investigations in the site vicinity. More detailed descriptions of the subsurface conditions encountered are presented in the Boring Logs on Figures A-3, A-4, and A-5 in Appendix A, and on the Boring and Trench Logs from our previous investigations presented in Appendix C. Figure 5, Geologic Cross Section A-A’, presents a typical cross section for the site area. This cross section is based on documented observations of the subsurface conditions encountered in explorations performed within and near the project area from both previous and the current investigation.

Soil and groundwater conditions can deviate from those conditions encountered at the boring locations. 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.
4 GEOLOGIC & SEISMIC FINDINGS

4.1 REGIONAL GEOLOGY

The San Francisco Bay Area lies within the Coast Range geomorphic provinces, a more or less discontinuous series of northwest-southeast trending mountain ranges, ridges, and intervening valleys characterized by complex folding and faulting. The general geologic framework of the San Francisco Bay Area is illustrated in studies by Schlocker (1970), as well as studies by Helley and Lajoie (1979), Wagner et al. (1990), Chin et al. (1993), Ellen and Wentworth (1995), Wentworth et al. (1999), Knudsen et al. (1997 and 2000), and Witter et al. (2006). The regional geologic map covering the site area is presented on Figure 3.

Geologic and geomorphic structures within the San Francisco Bay Area are dominated by the San Andreas fault (SAF), a right-lateral strike-slip fault that extends from the Gulf of California in Mexico to Cape Mendocino on the Coast of Humboldt County in northern California. It forms a portion of the boundary between two independent tectonic plates on the surface of the earth. To the west of the SAF is the Pacific Plate, which moves north relative to the North American Plate, located east of the fault. In the San Francisco Bay Area, movement across this plate boundary is concentrated on the SAF; however, it is also distributed, to a lesser extent across a number of other faults that include the Hayward, Calaveras and Concord among others (Graymer et al., 2002). Together, these faults are referred to as the SAF System. Movement along the SAF system has been ongoing for about the last 25 million years. The northwest trend of the faults within this fault system is largely responsible for the strong northwest structural orientation of geologic and geomorphic features in the San Francisco Bay Area.

Basement rocks west of the SAF are generally granitic, while to the east consist of a chaotic mixture of highly deformed marine sedimentary, submarine volcanic and metamorphic rocks of the Franciscan Complex. Both are typically Jurassic to Cretaceous in age (199-65 million years old). Overlying the basement rocks are Cretaceous (about 145 to 65 million years old) marine, as well as Tertiary (about 65 to 2.6 million years old [USGS, 2010]) marine and non-marine sedimentary rocks with some continental volcanic rock. These Cretaceous and Tertiary rocks have been extensively folded and faulted as a result of late Tertiary and Quaternary regional compressional forces. Regional geologic maps of the area covering the school campus indicate that bedding planes in adjacent hillside areas dip from about 50 to 75 degrees to the southwest.
The inland valleys, as well as the structural depression within which the San Francisco Bay is located, are filled with unconsolidated to semi-consolidated continental deposits of Quaternary age (about the last 2.6 million years). Continental surficial deposits (alluvium, colluvium, and landslide deposits) consist of unconsolidated to semi-consolidated sand, silt, clay, and gravel while the Bay deposits typically consist of very soft organic-rich silt and clay (Bay mud) or sand.

4.2 AREA AND SITE GEOLOGY

Geologic maps emphasizing bedrock formations in the vicinity of the site have been prepared by Weaver (1949), Sheehan (1956), Wagner (1990), Dibblee (1980), Graymer et al. (1994), and Crane (1995) among others. Weaver (1949), Dibblee (1980), and Graymer et al. (1994) mapped the bedrock as Tertiary age (Late Miocene to Pliocene) Orinda Formation. Sheehan (1956), however, mapped the Tertiary strata near Point Pinole as undifferentiated Contra Costa Group following the suggestion of Savage, Ogle, and Creely (1951). Wagner (1978) mapped exposures of the undifferentiated Contra Costa Group in the vicinity of the site as the “Garrity Member.” Graymer et al. (1994) described the Orinda Formation as non-marine, conglomerate, sandstone and siltstone with abundant rock clasts that have been derived from the Franciscan Complex and other Cretaceous age rocks. Wagner (1978) distinguished the “Garrity Member” from the Orinda Formation and other members of the Contra Costa Group by the presence of significant quantities of reworked Monterey formation detritus such as siliceous shale and chert.

Localized studies, which emphasize the Quaternary (younger than approximately 2.6 million years old) geology in the general area of the site, have been prepared by Helley et al. (1979), Knudsen et al. (1997), Helley and Graymer (1997), Graymer (2000) and Witter, et al. (2006). Generally, the unconsolidated alluvial deposits of Pleistocene age are mapped along slightly elevated areas, while the younger Holocene alluvial deposits are mapped blanketing level zones or young creek channels and drainage courses. According to Graymer (2000), the project site is underlain by Holocene alluvial fan and fluvial deposits (map symbol Qhaf), as shown on Figure 4, Area Geology Map. The alluvial fan and fluvial deposits are described by Graymer (2000) as brown or tan, medium dense to dense, gravely sand or sandy gravel that generally grades upward to sandy or silty clay.

Although Graymer (2000) and others have characterized the site as being underlain by unconsolidated alluvial deposits, we mapped the site as being underlain by Holocene basin deposits beneath the surficial undocumented fill layer blanketing the site area. This conclusion is
based on information obtained from the extensive fieldwork performed at the campus during the previously mentioned fault trench studies and current explorations conducted by Kleinfelder. The fill overlies Holocene fine-grained basin deposits except near Rheem Creek where Holocene fine-to coarse-grained channel deposits were encountered. The Holocene deposit soils are presumably underlain by a thicker sequence of older (Pleistocene age) alluvium that is underlain, in turn, by the terrestrial sedimentary bedrock of the Garrity Member of the Contra Costa Group. However, the Holocene stream channel deposits associated with this creek do not likely extend into the project area.

4.3 FAULTING AND SEISMICITY

Much of the campus, including the project site, is located within an Alquist-Priolo Earthquake Fault Zone, associated with the active Hayward fault. Evidence of fault creep across the campus has been documented for several decades (CDMG, 1980) and was observed and mapped during previous site reconnaissance and studies. The secondary western trace of the Hayward fault is mapped trending through the gymnasium and the proposed locker and shower building. An investigation by Kleinfelder (2007) did not find evidence of surface rupture along this fault trace. However, the trenches excavated for this investigation did not extend to the bottom of the Holocene soils, and therefore the existence of the fault trace could not be ruled out. The results of this investigation are discussed further in Section 4.7 below. Because the Hayward fault is known to be active and has been the locus of historic earthquakes with associated ground rupture, the potential for future ground rupture during an earthquake along active traces of this fault within the Contra Costa College (CCC) campus is considered high and should be anticipated.

4.4 2016 CBC SITE CLASS

In developing site-specific ground motions, the characteristics of the soils underlying the site are an important input to evaluate the site response at a given site. Based on the boring logs from our recent and previous geotechnical investigations at the school campus, the site subsurface consists of mainly soft to firm clays and silts with minor with some relatively thin layers of sand them.

Considering the above information, the site can be classified as Site Class D, as presented in Table 1613A.5.2 and Section 1613A.5.5 of the 2016 CBC. Site Class D is defined as stiff soil with shear wave velocities between 600 feet/sec and 1,200 feet/sec, SPT-N = 15 to 50 blows/foot, or Su = 1,000 - 2,000 psf for the upper 100 feet.
Due to the near proximity of the Hayward fault to the project a site specific seismic hazard analysis was required per ASCE 7-10 (ASCE 2010) and Chapter 16A of 2016 California Building Code for the C-608 PE/Kinesiology Expansion project. The seismic hazard analysis was prepared in order 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. Seismic design code values and the site specific parameters developed from this analysis should be considered in the structural design of the proposed facility. Structures with strength discontinuities, soft stories, plan irregularities, discontinuous shear walls and ductile moment frames are particularly vulnerable to these types of motions and should either be avoided or properly evaluated. Seismic design code values are presented below and the full seismic hazard analysis with site-specific seismic design parameters is presented in Appendix D.

For a 2016 California Building Code (CBC) based design, the estimated Maximum Considered Earthquake (MCE) mapped spectral accelerations for 0.2 second and 1 second periods (SS and S1), associated soil amplification factors (Fa and Fv), and mapped peak ground acceleration (PGA) are presented in Table 4.1. Corresponding site modified (SMS and SM1) and design (SDS and SD1) spectral accelerations, PGA modification coefficient (FPGA), PGAM, risk coefficients (CRS and CR1), and long-period transition period (TL) are also presented in Table 4.1. Presented values were estimated using Section 1613.3 of the 2016 California Building Code (CBC), chapters 11 and 22 of ASCE 7-10, and the United States Geological Survey (USGS) U.S. seismic design maps$^1$.

<table>
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<tr>
<td>$C_{R1}$</td>
<td>0.969</td>
<td>Fig 22-4</td>
</tr>
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### 4.6 FAULT-RELATED GROUND SURFACE RUPTURE

Much of the campus, including the project site, is located within an Alquist-Priolo Earthquake Fault Zone, associated with the active Hayward fault. Evidence of fault creep across the campus has been documented for several decades (CDMG, 1980) and was observed and mapped during previous site reconnaissance and studies by one of our CEGs. Kleinfelder completed an investigation of the secondary western trace of the Hayward fault in 2007 that is mapped trending through the gymnasium and the proposed locker and shower building. This investigation did not find evidence of surface rupture along this fault trace, but the trench excavated for the investigation was not deep enough to extend past the Holocene soils and expose Pleistocene soils. The trench log for this investigation (Trench G-1) is provided in Appendix C. Two subsequent fault investigations were performed by Kleinfelder (2016a and 2016b) for the Campus Safety Center, located approximately 90 feet north of the proposed locker and shower building. The investigations did not encounter fault traces in the trenches excavated as part of the studies, but these trenches also were not extended through the Holocene soils to expose Pleistocene soils. The trenches were excavated to depths of approximately 17 feet and exposed soils age-dated to approximately 4,500 years as shown on the Log of Trenches T-1 and T-2, Appendix C. Based on the age of the deposits in these trenches and the recurrence interval of the Hayward fault, it was concluded that up to 32 seismic events may have occurred on the Hayward fault during the 4,500 year age of the deposits explored. Since none of the 32 seismic events resulted in impacting the 4,500 year old deposits in the immediate vicinity of the site, it was concluded that the site area is free of active fault traces. Based on similar findings in Trench G-1 excavated across the mapped secondary west fault trace, Kleinfelder’s 2016 conclusion can be applied to the current project area that the site is free of active fault traces.
4.7 SEISMICALLY-INDUCED GROUND FAILURE

4.7.1 Liquefaction and Lateral Spreading

Soil liquefaction is a condition where saturated, granular soils undergo a substantial loss of strength and deformation due to pore pressure increase resulting from cyclic stress application induced by earthquakes. In the process, the soil acquires mobility sufficient to permit both horizontal and vertical movements if the soil mass is not confined. Soils most susceptible to liquefaction are saturated, loose, clean, uniformly graded, and fine-grained sand deposits. If liquefaction occurs, foundations resting on or within the liquefiable layer may undergo settlements. This will result in reduction of foundation stiffness and capacities.

The campus lies with the Richmond 7.5 Minute Quadrangle, which was partially mapped by CGS during its ongoing effort to map landslide and liquefaction related hazards throughout the San Francisco Bay Area. However, the campus does not lie within the area mapped by CGS. There are no recorded signs of ground failures associated with past earthquakes in Northern California within about 4 km of the project site (Youd and Hoo se, 1978). No historic ground failures were reported within approximately 6½ km of the site in the mapped results of Holzer (1998) as a result of the 1989 M6.9 Loma Prieta earthquake.

Based on the subsurface data obtained from our previous and recent borings, CPTs, and fault trenches at the campus, the site subsurface consists mostly of interbedded layers of firm to hard fine-grained clayey soils within the level areas of the campus. As a result, liquefaction potential at the site is considered minimal due to the soil types encountered and settlement is anticipated to be less than 1-inch.

4.7.2 Dynamic (Seismic) Compaction

Another type of seismically-induced ground failure, which can occur as a result of seismic shaking, is dynamic compaction, or seismic settlement. Such phenomena typically occur in unsaturated, loose granular material or uncompacted fill soils. The subsurface conditions encountered in our borings are not considered conducive to such seismically-induced ground failures. For this reason we conclude that the potential for shaking related random ground cracking to affect the site and surrounding areas is low.
4.8 EXPANSIVE SOILS

Based on the results of our concurrent field investigation and laboratory testing program, near-surface soils located within the building site are considered to have low expansion potential (Expansion Index of 29). However, soil samples from adjacent explorations were considered to be highly expansive. Recommendations to mitigate potentially expansive soils are included in Section 5 of this report.

4.9 EXISTING FILL

Fill measuring between 1 and 6 feet thick was encountered in our nearby fault trenches to the southeast. Our borings drilled in the immediate vicinity of the planned building revealed similar fill thickness (approximately 3 to 7 feet) which was comprised of layered, mottled soil and contained non-continuous 2- to 9-inch-thick layers of aggregate base. Recommendations to mitigate the potentially compressive and expansive effects of existing fill are provided in Section 5.

4.10 LANDSLIDES

The site is relatively flat, with minor topographic relief. Therefore, it is our opinion that the potential for seismically induced (or otherwise) landslides and slope failures to occur at the proposed site is low.

Rheem Creek is located approximately 300 feet north/northeast of the project site. Small, shallow localized creek bank sloughing or slumping may occur during a moderate to major seismic event, especially if the slopes are saturated. We would not expect such failures to extend more than approximately 10 feet from the current tops of banks; therefore such mass wasting episodes are not expected to adversely impact the proposed project.

4.11 TSUNAMIS, SEICHES, AND FLOODING

Flood hazards are generally considered from three sources:

- Seismically-induced waves (tsunami or seiche);
- Dam failure inundation; and
- Long-cycle storm events.
The site is located more than 8,000 feet southeast of the San Pablo Bay at an estimated elevation of about 65 feet above mean sea level. The only historical account of tsunamis impacting the San Francisco Bay area is the “Good Friday” earthquake of 1964 (generated off the coast of Alaska). Run-up at the Golden Gate Bridge was measured at 7.4 feet from the Good Friday earthquake and generally less further to the east. Ritter and Dupre (1972) indicate that the coastal lowland areas, immediately adjacent to San Francisco Bay, are subject to possible inundation from a tsunami with a run up height of 20 feet at the Golden Gate Bridge. Ritter and Dupre’s 1972 map does not show the site area to be within an area that could become inundated by tsunami waves. In addition, the California Emergency Management Agency (CalEMA) in concert with CGS and the University of Southern California have prepared tsunami inundation maps for emergency planning in 2009 and these maps indicate that tsunami generated waves will not reach the site area due to its distance from the Bay and prominent water courses.

Based on the above-noted references, the site’s distance from the Bay, topographical elevation, and the lack of historically damaging tsunamis and seiches, we judge that the potential for a seismically-induced wave to impact the site should be considered negligible.

The Association of Bay Area Governments (ABAG, 1995) prepared maps that show areas that may be inundated by flood water if nearby dams are overtopped or fail catastrophically. According to ABAG, the site could be inundated by 5 different dams. Based on these maps, the potential for flooding to occur at the site due to nearby dam failure should be considered high.

With respect to the 100-year storm events, the Federal Emergency Management Agency’s (FEMA, 2009) Flood Insurance Rate Map, Community-Panel Number 06013C0227F, effective date June 15, 2009, indicates that the site is located within Zone X, which is defined as areas determined to be outside the 0.2% annual chance flood plain.

4.12 NATURALLY-OCCURRING ASBESTOS

The geologic units that underlie the site (Contra Costa Group, alluvium) are not generally known to contain naturally occurring asbestos (NOA). However, the Contra Costa Group contains many conglomerate beds which received sediment/clasts from Franciscan sources during its time of deposition. Therefore, the presence of occasional clasts made up of rock types which may contain NOA (such as serpentinite) cannot be ruled out. The closest mapped formation, which may contain NOA is ultramafic rock located approximately 1.2 miles (about 2 km) to the south.
according to Graymer et al. (1994) and Churchill and Hill (2000). It is our opinion that the potential for NOA to impact the proposed development at the site is low to moderate.

4.13 SOIL CORROSION

A series of chemical tests were performed by Sunland Analytical of Rancho Cordova, California on a selected sample of the near-surface soils. Laboratory chloride concentration, sulfate concentration, pH, and electrical resistivity tests were performed on a near surface soil sample. The results of the tests are attached in Appendix B and are summarized below in Table 4.2. These tests are generalized indicator of soil corrosivity for the sample tested. Other soils on-site may be more, less, or similarly corrosive in nature. Imported fill materials should be tested to confirm that their corrosion potential is not more severe than those noted.

<table>
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<th>Boring</th>
<th>Depth, feet</th>
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<th>pH</th>
<th>Water-Soluble Ion Concentration, ppm</th>
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<td></td>
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<tr>
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</tr>
<tr>
<td>Bulk A</td>
<td></td>
<td></td>
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<td></td>
</tr>
</tbody>
</table>

Although Kleinfelder’s scope did not include corrosion engineering, resistivity values between 1,000 and 3,000 ohm-cm are normally considered highly corrosive to buried ferrous metals (NACE, 2006). The concentrations of soluble sulfates indicate that the subsurface soils represent a Class S1 exposure to sulfate attack on concrete in contact with the soil based on ACI 318 Table 4.2.1 (ACI, 2011).

Therefore, all buried iron, steel, cast iron, ductile iron, galvanized steel, and dielectric coated steel or iron should be properly protected against corrosion depending upon the critical nature of the structure. All buried metallic pressure piping such as ductile iron firewater pipelines should be protected against corrosion. We recommend that a corrosion specialist be consulted for advice on proper corrosion protection for underground piping which will be in contact with the soils and other design details.

The above are general discussions. A more detailed investigation may include more or fewer concerns, and should be directed by a corrosion expert. Soils actually in contact with concrete...
should be sampled and tested for sulfate content during construction and the concrete mixes used should comply with the requirements of the 2016 CBC based on these results. Consideration should also be given to soils in contact with concrete that will be imported to the site during construction, such as topsoil and landscaping materials. For instance, any imported soil materials should not be any more corrosive than the on-site soils and should not be classified more corrosive than “moderately corrosive.” Also, on-site cutting and filling may result in soils contacting concrete that were not anticipated at the time of this investigation.

4.14 RADON GAS

Radon gas is a naturally-occurring colorless, tasteless, and odorless radioactive gas that forms in soils from the decay of trace amounts of uranium that are naturally present in soils. Radon enters buildings from the surrounding soil through cracks or other openings in foundations, floors over crawlspaces, or basement walls. Once inside a building, radon can become trapped and concentrate to become a health hazard unless the building is properly ventilated to remove radon. Long-term exposure to elevated levels of radon increases one’s risk of developing lung cancer.

The U.S. Environmental Protection Agency (EPA) recommends that all homes (or structures intended for human occupancy) be tested for radon whatever their geographic location. The U.S. EPA recommends that action be taken to reduce radon in structures with an average annual level higher than four picocuries per liter (4.0pCi/l).

The California Department of Health services (2010) performed 28 tests within Zip Code 94806 (last updated on May 4, 2010) where the school campus is located. Of the 20 tests, none reported a minimum of four (4) picocuries per liter.

The noted testing is not intended to represent the entire zip code area for determining which buildings have excessive indoor radon levels. In addition to geology, indoor radon levels can be influenced by local variability in factors such as soil permeability and climatic conditions, and by factors such as building design, construction, condition, and usage. Consequently, building specific radon levels can only be determined by indoor radon testing.

Based on the above information, consideration should be given to consult a radon specialist to provide appropriate tests and recommendations to review this concern.

Additional information about radon gas can be found at the following websites:
California Department of Health Services:

California Department of Public Health, Radon Program:
http://www.cdph.ca.gov/healthinfo/environhealth/Pages/Radon.aspx

California Geological Survey-Mineral Resources Program:
http://www.conservation.ca.gov/cgs/minerals/hazardous_minerals/radon/Pages/Index.aspx

U.S. EPA:
http://www.epa.gov/iaq/radon

4.15 VOLCANIC ACTIVITY

There are no known active volcanic sources within the region, therefore the potential for volcanic hazards to impact this site are considered non-existent.
5 CONCLUSIONS AND RECOMMENDATIONS

5.1 GENERAL

Based on the results of our field investigation, it is Kleinfelder’s opinion that the construction of the proposed C-608 PE/Kinesiology Expansion is geotechnically feasible. This conclusion is based on the assumption that the recommendations presented in this report will be incorporated in the design and construction of this project. According to our current and previous borings, the site is partially covered by an approximate 2-inch-thick layer of asphalt concrete pavement or landscaping underlain by about 3 to 7 feet of fill.

Our laboratory test data and experience at the campus indicates the some site clay soils are moderately to highly expansive. Mitigation of expansive soil behavior is recommended in this report by deepening the footings, blanketing the slab areas with non-expansive soil, and using special earthwork construction procedures.

New building foundation settlements should be primarily elastic with the majority of the settlement occurring relatively soon after application of the loads. We estimate that total settlements should be less than 3/4 inch, and differential settlements over a 50-foot horizontal distance should be less than ½ inch. Differential settlement between the buildings may occur and should be considered during design.

5.2 GEOLOGIC AND SEISMIC HAZARDS

Based on the results of the geologic and seismic findings provided in Section 4, along with subsurface findings from this current study Kleinfelder concludes that the proposed improvements are generally free of geologic and seismic hazards except as noted below:

1. The project site is situated within the limits of the Alquist-Priolo Earthquake Fault Zone (AP Zone) associated with the active Hayward fault and is located approximately 180 feet southwest of the main creeping trace of the Hayward fault. The proposed project is also located within a highly active seismic region with numerous regionally active faults located nearby. Based on the near proximity of the project to active faults, strong to violent ground shaking as a result of future seismic events is likely.
2. The project includes the presence of relatively old undocumented fill soil

3. The project soils have a potential for high expansion and corrosion

Seismic design parameters are provided in Section 4.6 of this report for reference and are discussed in detail in Appendix D.

5.3 EXISTING SITE FILL

Existing fill encountered during site grading may be used as engineered fill if the material meets the requirements contained in this report. Any organic materials, debris or other deleterious materials should be removed from the existing fill before it is reused as engineered fill. Since most of the existing fill is potentially expansive clay, the recommendations for expansive soils presented in this report apply to this material.

5.4 EARTHWORK

5.4.1 Expansive Soils

Based on the results of our field investigation and laboratory testing programs, near-surface, clay soils located within the building site are considered to have low expansive potential. However, nearby soils were considered moderately to highly expansive. Expansive soils are characterized by their ability to undergo significant volume changes (shrink or swell) due to variations in moisture content. Changes in soil moisture content can result from rainfall, landscape irrigation, utility leakage, roof drainage, perched groundwater, drought, or other factors and may cause unacceptable settlement or heave of structures, concrete slabs supported-on-grade, or pavements supported over these materials. Depending on the extent and location below finished subgrade, these soils could have a detrimental effect on the proposed construction.

Since site grading plans were not available at the time this report was prepared, the lateral and vertical extent of potentially expansive soils which may be present subsequent to site grading could not be directly evaluated by Kleinfelder. Therefore, we recommend a representative from Kleinfelder be present during site grading to evaluate proposed building and pavement areas for the presence of near-surface, expansive soils. It is recommended that the upper 30 inches of the building pad be constructed with imported, non-expansive soil that is placed and compacted in
accordance with recommendations provided below for engineered fill. Non-expansive fill should extend a minimum of 5 feet horizontally beyond the building perimeter and beneath adjoining concrete walkway slab sections in order to reduce the potential for differential movement at door thresholds. Dowelling of concrete sidewalk approach slabs to the perimeter foundations at door openings is recommended to help control potential offsets at the joint.

In addition, excavated expansive soil should either be: (1) disposed of off-site; (2) placed in landscape areas of the project; or (3) placed within the lower portions (i.e. at least 24 inches below finished subgrade) of deep fills.

It should be noted these recommendations are consistent with those applied at other projects in the area with similar soil conditions. However, even with proper implementation of these recommendations, minor slab (interior and exterior) and/or pavement movement and/or distress may occur due to swelling and shrinking of the subgrade soils.

5.4.2 General

Earthwork at the site will generally consist of subgrade preparation, fill placement, and placement of aggregate base or crushed rock beneath pavements, concrete slabs-on-grade and flatwork, excavation and backfill of underground utility line trenches, and footing excavations. Although grading plans were not available to us at the time this report was prepared, we anticipate that required grading will consist of minor cuts and fills to create a level building pad. New underground utilities, if planned, are expected to be no deeper than 5 to 8 feet below the ground surface. Kleinfelder should review final grading plans for conformance to our design recommendations prior to construction bidding. In addition, it is important that a representative of Kleinfelder observe and evaluate the competency of existing soils or new fill underlying structures, slabs-on-grade, concrete flatwork, and pavements. In general, soft or unsuitable materials encountered should be over-excavated, removed, and replaced with compacted engineered fill material.

Site preparation and grading for this project should be performed in accordance with the site specific recommendations provided below. Additional earthwork recommendations are presented in related sections of this report, as applicable.
5.4.3 Site Preparation and Grading

Prior to the start of grading and subgrade preparation operations, the site should first be cleared and stripped to remove the existing pavements, landscaped areas, and associated plants located within the footprint of the proposed improvements. Stripping to a minimum depth of approximately 2 to 3 inches is expected to remove a majority of organic-laden surficial soils in landscaped areas. If significant amounts of organics are encountered below this depth, additional stripping may be required. Stripping should extend laterally a minimum of 5 feet beyond the building limits, and 2 feet beyond flatwork and pavement edges. Stripped topsoil may be stockpiled for later use in landscaping areas. However, this material should not be reused for engineered fill.

Any buried tree stumps, roots, or major root systems thicker than approximately 1 inch in diameter, abandoned foundations, septic tanks and leach field lines, and any other deleterious material uncovered during site stripping and/or grading activities should be removed. Unit prices for removal of such materials should be obtained during bidding.

Any existing aggregate base material, asphalt concrete pavement, and concrete (if broken up to within the grading requirements specified below for imported soil) may be re-used as general fill, but should not be used within the footprint of the proposed C-608 PE/Kinesiology expansion without prior approval from the District. All active or inactive utilities within the construction area should be protected, relocated, or abandoned. Any pipelines to be abandoned within the building footprint should either be removed or be filled with a sand-cement slurry. At locations where exiting utility lines will remain in place, the foundations should be extended below the zone of influence affecting such utilities, as discussed in the Foundations and the Excavation and Backfill sections of this report. Active utilities to be reused should be carefully located and protected during construction.

Following stripping and removal of deleterious materials and removals of expansive soils (if any) within the building area, areas of the site to receive fill should be scarified to a minimum depth of 12 inches, moisture-conditioned, and recompacted in accordance with the requirements below for Engineered Fill. Scarification should extend laterally a minimum of 5 feet beyond the limits of structures, and 2 feet beyond flatwork and pavement edges, where achievable. All fills should be compacted in lifts of 8-inch maximum uncompacted thickness. Laboratory maximum dry density and optimum moisture content relationships should be evaluated based on ASTM Test Designation D-1557 (latest edition).
All site preparation and fill placement should be observed by a Kleinfelder representative. It is important that during the stripping and scarification process our representative be present to observe whether any undesirable material is encountered in the construction area and whether exposed soils are similar to those encountered during our field investigation.

5.4.4 Engineered Fill Materials

Except for organic-laden soil or debris, the on-site soil is suitable for use as general engineered fill if it is free of deleterious matter. The maximum particle size for fill material should be limited to 3 inches, with at least 90 percent by weight passing the 1-inch sieve. Where imported “non-expansive” material is required, it is recommended that it be granular in nature, adhere to the above gradation recommendations and conform to the following minimum criteria:

- Plasticity Index: 12 or less
- Liquid Limit: less than 30%
- Percent Soil Passing #200 Sieve: 25% to 60%

Highly pervious materials such as pea gravel, clean crushed rock or clean sands are not recommended because they permit transmission of water into the underlying soils. All on-site or import fill material should be compacted to the recommendations provided for engineered fill below and should be tested prior to being brought on-site to confirm adherence to the above criteria.

5.4.5 Fill Compaction Criteria

Due to the expansive nature of the on-site soils, proper moisture conditioning is important during fill placement and compaction. Where low expansion potential soils or aggregate base materials in paved areas are used, they should be placed immediately over the prepared subgrade to retard drying of the underlying subgrade. The subgrades for exterior concrete flatwork should be moisture conditioned to at least 3 percent above the optimum moisture content prior to compaction, and may require additional moisture conditioning if allowed to dry.

On site clay soils used for engineered fill should be uniformly moisture-conditioned to between 3 and 5 percent above the optimum moisture content, placed in horizontal lifts less than 8 inches in loose thickness, and compacted to between 88 and 92 percent relative compaction. Over-compaction of these clay soils and compaction at lower than recommended moisture content
should not be allowed. Imported non-expansive soils or lime treated soils used for engineered fill should be uniformly moisture-conditioned to between 0 and 5 percent above the optimum moisture content, placed in horizontal lifts less than 8 inches in loose thickness, and compacted to at least 90 percent relative compaction. The upper twelve inches of pavement subgrades should be compacted to at least 95 percent relative compaction. Disking and/or blending will be required to uniformly moisture-condition soils used for engineered fill.

5.4.6 Weather/Moisture Considerations

Based on our experience in the area, grading during the rainy season may be extremely difficult due to the type of soil at the site. If earthwork operations and construction for this project are scheduled to be performed during the rainy season or in areas containing saturated soils, provisions may be required for drying of soil or providing admixtures to the soil prior to compaction. If desired, we can provide recommendations for wet weather earthwork and alternatives for drying the soil prior to compaction. Conversely, additional moisture may be required during dry months. Water trucks should be made available in sufficient numbers to provided adequate water during earthwork operations.

Since portions of the site are currently capped with pavement, the moisture content of the subgrade soils in these areas may be significantly above the optimum moisture content. This occurrence is usually caused by the migration of irrigation water from landscaped areas into the aggregate base material and/or the entrapment of subsurface moisture underneath the pavement. As a result, the subgrade soils may need to be dried prior to undergoing recompaction. It is also recommended that any landscape watering in the area be turned off at least two weeks prior to the start of grading activities at the site. If site grading is performed during the rainy months, the site soils could become very wet and difficult to compact without undergoing significant drying. This may not be feasible without delaying the construction schedule. For this reason, drier import soils could be required or lime treating may be needed to dry back soils if construction takes place during winter months.

5.4.7 Excavation and Backfill

All trenches, regardless of depth, should conform to the current OSHA requirements for work safety. It is the contractor’s responsibility to follow OSHA temporary excavation guidelines and grade the slopes with adequate layback or provide adequate shoring and underpinning of existing structures and improvements, as needed. Slope layback and/or shoring measures should be
adjusted as necessary in the field to suit the actual conditions encountered, in order to protect personnel and equipment within excavations.

We anticipate that excavation for foundations, pavement subgrade, and utility trenches can be made with either a backhoe or trencher. We expect the walls of trenches less than about 4 feet deep to stand near vertical without support. In areas where granular soil are present, some sloughing of soils into trench excavations may occur. Where trenches or other excavations are extended deeper than 4 feet, the excavation may become unstable and should be evaluated to monitor stability prior to personnel entering the trenches. Shoring or sloping of any trench wall may be necessary to protect personnel and to provide stability.

Care should be taken during construction to reduce the impact of trenching on adjacent pavements and structures. Excavations should be located so that no structures, foundations, and slabs, existing or new, are located above a plane projected 2H:1V (horizontal to vertical) upward from any point in an excavation, regardless of whether it is shored or unshored.

At the time of this geotechnical investigation, groundwater was not observed in the borings. However, previous explorations performed in 2007 and 2017 encountered groundwater at depths of about 13 to 15 feet. As described in the Subsurface Conditions section of this report, the actual depth at which groundwater may be encountered in trenches and excavations may vary. As a minimum, provisions should be made to ensure that conventional sump pumps used in typical trenching and excavation projects are available during construction in case groundwater is found to be higher than observed during our investigation, and/or if substantial runoff water accumulates within the excavations as a result of wet weather conditions.

Backfill for trenches and other excavations beneath slabs and within pavement areas should be moisture conditioned, placed and compacted as recommended in the Fill Compaction Criteria section of this report. Special care should be taken in the control of utility trench backfilling under structures, pavements, and flatwork/slab areas. Poor compaction may cause excessive settlements resulting in damage to overlying structures, slabs, and the pavement structural section.

Where utility trenches extend from the exterior to the interior limits of a building or pavement, trench plugs are recommended under the foundation or curb to inhibit the migration of water through the trench backfill materials. This can be accomplished by completely surrounding that portion of the utility in the trench with lean concrete under the footing or curb. Alternatively, native
clayey soils could be used as a trench plug over a distance of 2 feet laterally on each side of the footing/curb limits to reduce the potential for the trench to act as a conduit to exterior surface/subsurface water. Utility trenches located in landscaped areas should also be capped with a minimum of 12 inches of compacted on-site clayey soils.

5.5 FOUNDATIONS

5.5.1 Subgrade Preparation

Based on the information presented herein, the proposed C-608 PE/Kinesiology Expansion should be underlain by newly constructed, non-expansive engineered fill. A connection between the new and existing buildings is needed. That connection should consider that the new building will undergo some post-construction settlement after initial application of the loads and the existing building will not.

5.5.2 Allowable Bearing Pressure

Due to the potential for low to highly expansive soils below the proposed building footprint, foundations for the gym expansion will need to extend deeper than for a non-expansive soil site. In addition, all footings should be continuous and tied together. The recommended allowable soil bearing pressures, depth of embedment, and width of footings are presented below. The allowable bearing values provided have been estimated assuming that all footings uniformly bear on newly constructed engineered fill.

<table>
<thead>
<tr>
<th>FOOTING BEARING CAPACITY RECOMMENDATIONS</th>
</tr>
</thead>
<tbody>
<tr>
<td>Footing Type</td>
</tr>
<tr>
<td>----------------------------------------</td>
</tr>
<tr>
<td>Exterior Continuous Footing***</td>
</tr>
<tr>
<td>Interior Continuous Footing</td>
</tr>
</tbody>
</table>

*  Dead plus live load, pounds per square foot.
** Below lowest adjacent grade defined as bottom of slab on the interior and finish grade at the exterior, in inches.
*** Includes perimeter footing around building.

Allowable soil bearing pressures may be increased by one-third for consideration of transient loads such as wind and seismic forces. Where footings are located near and parallel to
underground utilities, the footings should extend below a plane projected at a slope of 2H:1V (horizontal to vertical) upward from the bottom of the underground utility trench to avoid surcharging the utility with building loads.

To help reduce fluctuations in moisture content beneath the buildings, continuous footings should be used around the perimeter of the buildings to provide a barrier against changes in moisture of the soils beneath the interior floor slabs. Where utilities cross perimeter footing lines, the trench backfill should consist of a vertical barrier of impervious type material extending about 2 feet on either side of the perimeter footing or lean concrete or cementitious slurry under the footing.

5.5.3 Resistance to Lateral Loads

Lateral loads may be resisted by a combination of friction between the foundation bottoms and the supporting engineered fill subgrade, and by passive resistance acting against the vertical faces of the foundations. An allowable friction coefficient of 0.3 between the foundation and supporting subgrade may be used. For passive resistance, an allowable equivalent fluid pressure of 270 pounds per cubic foot acting against the footing may be used. The friction coefficient and passive resistance may be used concurrently, and can be increased by one-third for wind and/or seismic loading. We recommend that the first foot of soil cover be neglected in the passive resistance calculations if the ground surface above is not confined by a slab, pavement or in some similar manner. These values include a factor of safety of about 1.5.

Concrete for footings should be placed neat against newly compacted engineered fill. It is important that footing excavations in clayey soils not be allowed to dry before placing concrete. If shrinkage cracks appear in the footing excavations, the excavations should be thoroughly moistened to close all cracks prior to concrete placement. The footing excavations should be monitored by a representative of Kleinfelder for compliance with appropriate moisture control and to confirm the adequacy of the bearing materials. If soft or loose soils are encountered at the bottom of footing excavations, they should be removed and replaced with lean concrete.

5.6 RETAINING WALLS

5.6.1 Lateral Earth Pressures

Subsurface structures and retaining walls should be designed to resist the earth pressure exerted by the retained, compacted backfill plus any additional lateral force that will be applied due to
surface loads placed at or near the wall or below-grade structure. Recommended design criteria for retaining walls and subsurface structures are presented below. These values are based on the on-site soils using a friction angle of 25 degrees and unit weight of 110 pounds per cubic foot (pcf). If imported materials are used for wall backfill, the material should have similar properties, otherwise the lateral earth pressure behind the wall could be different from the values presented in Table 5.1 below.

### Table 5.1
Lateral Earth Pressures for Retaining Walls

<table>
<thead>
<tr>
<th>Backfill Configuration</th>
<th>Earth Pressure</th>
<th>Equivalent Fluid Density (pcf)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Drained</td>
</tr>
<tr>
<td>Level</td>
<td>Active</td>
<td>45</td>
</tr>
<tr>
<td></td>
<td>At Rest</td>
<td>65</td>
</tr>
<tr>
<td>Level</td>
<td>Passive</td>
<td>270</td>
</tr>
</tbody>
</table>

Surcharge factor, $K_a = 0.3 \times$ surcharge pressure for active case  
Surcharge factor, $K_o = 0.5 \times$ surcharge pressure for at-rest case

Passive resistance should be neglected within the upper 1 foot of soil unless the passive pressure area in front of the footing is protected by concrete or pavement and is not disturbed by excavation or other means.

An allowable coefficient of sliding friction of 0.30 may be used between cast-in-place concrete foundations and the engineered fill. Passive pressure available in engineered fill or undisturbed native soil may be taken as equivalent to the pressure exerted by a fluid weighing 270 pounds per cubic foot (pcf).

Lateral resistance parameters provided above are allowable values derived with a factor of safety of at least 1.5. Therefore ultimate values may be calculated by multiplying the above values by the factor of safety. Allowable passive and sliding resistance may be combined without reduction.

### 5.7 POLE FOUNDATION DESIGN

Individual drilled, straight-shafted, cast-in-place concrete piers designed and constructed in accordance with our recommendations may be designed using a unit skin friction of 1,000 psf. The upper 3 feet of all piers should be neglected when evaluating allowable axial capacities.
Total, downward capacities estimated from the parameters provided above may be increased by 33 percent for short-term loads due to wind or seismic forces.

Pole foundation design parameters have not been provided and associated settlements are not able to be estimated at this time. Elastic compression of the pier under design loads will also need to be evaluated. A majority of the settlement should occur shortly after the loads are applied.

Lateral resistance may be evaluated using the "Pole Formula" given in Section 1807.3 of the California Building Code, 2016 edition. For this method, we recommend a lateral soil bearing pressure of 100 pounds per square foot per foot of embedment be used to verify required embedment depth. The 100 percent increase allowed by the Code for isolated poles (which are not adversely affected by a 1/2-inch horizontal deflection at the ground surface due to short-term lateral loads) may be used. The upper one foot of the pier should be neglected in lateral bearing calculations.

Pier excavations should be cleaned such that less than about 1 inch of loose soil or water remains at the bottom of the drilled hole. A representative from Kleinfelder must be present to observe each pier excavation to verify bottom conditions prior to placing steel reinforcement or concrete.

We recommend steel reinforcement and concrete be placed within 4 hours upon completion of each pier excavation. Concrete used for pier construction should be discharged vertically into the drilled holes to minimize aggregate segregation. Under no circumstances during pier construction should concrete be allowed to free-fall against either the steel reinforcement or the sides of the excavation. If water is present during concrete placement, concrete should be placed into the hole using tremie methods. The end of the tremie pipe must remain below the surface of the in-place concrete at all times.

In order to develop the design skin friction value provided above, concrete used for pier construction should have a slump of from 4 to 6 inches if placed in a dry shaft without temporary casing, and from 6 to 8 inches if casing is used. The concrete mix should be designed with appropriate admixtures and/or water/cement ratios to achieve these recommended slumps. Adding water to a conventional mix to achieve the recommended slump should not be allowed.
5.8 SLABS ON GRADE

5.8.1 Subgrade Preparation

Slabs-on-grade for this project will consist of concrete floor slabs and exterior flatwork. As previously discussed, the near-surface soils will be subject to shrink/swell cycles with fluctuations in moisture content. To reduce these potentially adverse effects, we recommend that concrete floor slabs be underlain by 24 inches of imported "non-expansive" engineered fill placed on subgrade prepared as described in the "Earthwork" section of this report. The properties of this "non-expansive" fill should also meet the criteria listed in the "Earthwork" section of this report.

5.8.2 Concrete Floor Slabs

Concrete floors should be supported on at least 6 inches of angular gravel or crushed rock to enhance subgrade support for the slab and serve as a capillary break. The capillary break material should be 3/4-inch maximum size with no more than 10 percent by weight passing the #4 sieve. It is important that placement of this material and concrete be done as soon as possible to reduce drying of the subgrade. The material should be compacted with a vibrating plate compactor.

A Structural Engineer should design the reinforcing and slab thickness. We anticipate the floor slab will be supported on engineered fill, a modulus of subgrade reaction of $K_{V1}=150 \text{ psi per inch}$ (for a 1 square foot bearing plate) may be used for design of floor slabs supported as recommended herein. The modulus used for design should be adjusted for the actual slab size using appropriate formulas or software.

Subsurface moisture and moisture vapor naturally migrate upward through the soil and, where the soil is covered by a building or pavement, this subsurface moisture will collect. The current industry standard is to place a vapor retarder on the compacted crushed rock layer to reduce the impact of the subsurface moisture and potential impact of future introduced moisture (such as landscape irrigation or precipitation). This membrane should meet the requirements of ASTM 1745. It should be noted that although vapor barrier systems are currently the industry standard, this system may not be completely effective in preventing floor slab moisture problems. These systems typically will not necessarily assure that floor slab moisture transmission rates will meet floor-covering manufacturer standards and that indoor humidity levels be appropriate to inhibit mold growth. The design and construction of such systems are totally dependent on the proposed use and design of the proposed building and all elements of building design and function should
be considered in the slab-on-grade floor design. Building design and construction have a greater role in perceived moisture problems since sealed buildings/rooms or inadequate ventilation may produce excessive moisture in a building and affect indoor air quality.

Various factors such as surface grades, adjacent planters, the quality of slab concrete and the permeability of the on-site soils affect slab moisture and can control future performance. In many cases, floor moisture problems are the result of either improper curing of floors slabs or improper application of flooring adhesives. We recommend contacting a flooring consultant experienced in the area of concrete slab-on-grade floors for specific recommendations regarding your proposed flooring applications.

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

It is emphasized that we are not floor moisture proofing experts. We make no guarantee nor provide any assurance that use of capillary break/vapor retarder system will reduce concrete slab-on-grade floor moisture penetration to any specific rate or level, particularly those required by floor covering manufacturers. The builder and designers should consider all available measures for floor slab moisture protection.

Exterior grading will have an impact on potential moisture beneath the floor slab. Recommendations for exterior draining are provided in the “Site Drainage” section of this report.

5.8.3 Exterior Concrete Flatwork

Exterior flatwork exposed to vehicular traffic (buses, garbage trucks, etc.) should be designed as a pavement by the structural engineer according to the actual loadings and frequency of loadings. Concrete exterior flatwork at grade will be constructed on soils subject to swell/shrink cycles. Some of the adverse effects of swelling and shrinking can be reduced with proper moisture treatment. The intent is to reduce the fluctuations in moisture content by moisture conditioning the soils, sealing the moisture in, and controlling it. Near-surface soils should be moisture conditioned according to the recommendations in the Fill Compaction Criteria section of this
report. Even with the moisture conditioning, some movement of exterior slabs may occur and measures to reduce the potential for joint offsets should be considered. Alternatively, external flatwork can be placed on 12 inches of imported non-expansive fill to reduce the potential for movement, as discussed in the Expansive Soil section of this report.

Where concrete flatwork is to be exposed to vehicle traffic, it should be designed as pavement. Exterior flatwork will be subjected to edge effects due to the drying out of subgrade soils. Because of the expansive soils, flatwork should have control joints on no greater than 8 feet on center. Prior to construction of the flatwork, the subgrade soils will need to be checked for appropriate moisture of at least over optimum. If the moisture is found to be below this level, the flatwork areas will need to be moisture conditioned or reworked until the proper moisture content is reached. Where flatwork is adjacent to curbs or building foundations, reinforcing dowels should be placed between the flatwork slabs and the curb or foundation. Expansion joint or mastic material should be used between flatwork and curbs or foundations.

5.9 SITE DRAINAGE

Proper site drainage is important for the long-term performance of the planned structures. The site should be graded so as to carry surface water away from the building foundations, at a minimum of 2 percent to a minimum of 5 feet laterally from the building. In landscaped areas, gradients should be at least 5 percent toward area drains. In addition, all roof gutters should be connected directly into a storm drainage system, or drain on to impervious surfaces that drain away from the buildings, provided that a safety hazard is not created.

We do not recommend having landscape platers adjacent to exterior building foundations due to the potential for excess irrigation water to saturate the foundation soils.

5.10 PAVEMENTS

We anticipate the parking lot and drive aisles will be paved with flexible asphalt concrete (AC). We have assumed Traffic Index (TI) values between 5 and 7 for this project. The appropriate TI should be selected by the pavement designer. Kleinfelder laboratory testing indicates a Resistance Value (R-value) of 15 for the existing clay fill. However, an R-Value of 5 was used in design based on our previous work at the campus for the local clay soils.
5.10.1 Asphalt Concrete Pavement

Based on Caltrans design methods and an R-value of 5 for the onsite clay soils, the recommended pavement sections for TIs ranging between 5 and 7 are provided below. Each TI represents a different level of use. The owner or designer should determine which level of use best reflects the project, and select appropriate pavement sections. A TI of 5 is commonly used for automobile parking spaces. A TI of 6 is commonly used for automobile and light truck access lanes. Higher TIs are typically used for major truck and bus access routes.

Pavement section parameters include AC and Caltrans Class II aggregate base (AB). The recommended pavement section thicknesses are provided in Table 5.2 below:

Table 5.2
Recommended AC Flexible Pavement Sections on Existing Fill Soils
Design R-Value = 5

<table>
<thead>
<tr>
<th>Traffic Index</th>
<th>AC (inches)</th>
<th>AB (inches)</th>
</tr>
</thead>
<tbody>
<tr>
<td>5</td>
<td>3.0</td>
<td>10.0</td>
</tr>
<tr>
<td>6</td>
<td>4.0</td>
<td>11.5</td>
</tr>
<tr>
<td>7</td>
<td>5.0</td>
<td>13.5</td>
</tr>
</tbody>
</table>

The anticipated TIs and the recommended pavement sections presented above should be reviewed by the project civil engineer in consultation with the owner during the development of the final grading and paving plans.

Pavement sections provided above are contingent on the following recommendations being implemented during construction.

- All pavement subgrades should be prepared as recommended in the Site Preparation and Engineered Fill Sections of this report.
- Subgrade soils should be in a stable, non-pumping condition at the time aggregate base materials are placed and compacted.
- Aggregate base materials should be compacted to at least 95 percent relative compaction.
• Adequate drainage (both surface and subsurface) should be provided such that the subgrade soils and aggregate base materials are not allowed to become saturated.

• Aggregate base materials should meet current Caltrans specifications for Class 2 aggregate base material.

• Asphalt paving materials and placement methods should meet current Caltrans specifications for asphalt concrete.

• All concrete curbs separating pavement and landscaped areas should extend into the subgrade and below the bottom of adjacent, aggregate base materials.

Pavement sections provided above are based on the soil conditions encountered during our field investigation, our assumptions regarding final site grades, and limited laboratory testing. In the event actual pavement subgrade materials are significantly different than those tested for this study, we recommend representative subgrade samples be obtained and additional R-value tests performed. Should the results of these tests indicate a significant difference, the design pavement sections provided above may need to be revised.

5.10.2 Portland Cement Concrete Pavement Sections

Portland cement concrete pavements are typically better able to resist the intense stresses induced in pavements by the turning motions of heavy trucks and delivery vehicles. Concrete pavements should be used in areas frequented by such vehicles as well as in driveway and entry aprons. Concrete pavement sections presented in the table below are based on current Portland Cement Association (PCA) design procedures and the assumptions listed below. These assumptions should be reviewed by the project Owner, Architect, and/or Civil Engineer to evaluate their suitability for this project. Changes in the assumptions will affect the corresponding pavement section.

• Modulus of subgrade reaction subgrade soils = 150 psi/in
• Modulus of rupture of concrete = 600 psi
• Aggregate Interlock Joints
• No concrete shoulders
• 15-year design life
• Load Safety Factor = 1.0
Table 5.3
Recommended Portland Cement Concrete Pavement Sections

<table>
<thead>
<tr>
<th>Proposed Use</th>
<th>Assumed Traffic</th>
<th>Portland Cement Concrete</th>
<th>Aggregate Base</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Index</td>
<td>(feet)</td>
<td>(inches)</td>
</tr>
<tr>
<td>Main Drive Aisles/Light Truck Traffic</td>
<td>5.5</td>
<td>0.60</td>
<td>7.0</td>
</tr>
<tr>
<td>Truck Lanes and Access ways</td>
<td>6.5</td>
<td>0.65</td>
<td>7.5</td>
</tr>
</tbody>
</table>

Portland cement concrete pavement sections provided above are contingent on the following recommendations being implemented during construction.

- All pavement subgrades should be prepared as recommended in the Site Preparation and Engineered Fill Sections of this report.

- Adequate drainage (both surface and subsurface) should be provided such that the subgrade soils are not allowed to become saturated.

- Concrete pavement should have a minimum 28-day compressive strength of 4,000 psi. Concrete slumps should be from 3 to 4 inches. The concrete should be properly cured in accordance with PCA recommended procedures and vehicular traffic should not be allowed for 3 days (automobile traffic) or 7 days (truck traffic).

- To help offset plastic shrinkage, concrete pavement may be reinforced with at least No. 3 bars, 24 inches on-center, each way or 6x6-W2.9xW2.9 wire mesh (located 1/3 of the slab thickness from the top of the slab).

- Construction and/or control joint spacing should not exceed 12 feet

- Thickened edges should be used along outside edges of concrete pavements. Edge thickness should be at least 2 inches greater than the concrete pavement thickness and taper to the actual concrete pavement thickness 36 inches inward from the edge. Integral curbs may be used in lieu of thickened edges.

- Over finishing of concrete pavements should be avoided. Typically, a broom or burlap drag finish should be used.
The above pavement recommendations should be incorporated into project plans and specifications by the project architect and/or engineer. These recommendations are not intended to be used as a specification for construction.

5.10.3 Unstable Subgrade

In the event unstable (pumping) subgrades are encountered within planned pavement areas, we recommend a heavy, rubber-tired vehicle (typically a loaded water truck) be used to test the load/deflection characteristics of the finished subgrade materials. We recommend this vehicle have a minimum rear axle load (at the time of testing) of 16,000 pounds with tires inflated to at least 65 pounds per square inch pressure. If the tested surface shows a visible deflection extending more than 6 inches from the wheel track at the time of loading, or a visible crack remains after loading, corrective measures should be implemented. Such measures could include disking to aerate, chemical treatment, replacement with drier material, or other methods. We recommend Kleinfelder be retained to assist in developing which method (or methods) would be applicable for this project.

5.10.4 Variations in Subgrade Materials

Pavement sections provided above are based on the soil conditions encountered during our field investigation, our assumptions regarding final site grades, and limited laboratory testing. In the event actual pavement subgrade materials are significantly different than those tested for this study, we recommend representative subgrade samples be obtained and additional R-value tests performed. Should the results of these tests indicate a significant difference, the design pavement section(s) provided above may need to be revised.
6 ADDITIONAL SERVICES AND LIMITATIONS

6.1 ADDITIONAL SERVICES

As the geotechnical engineering firm that performed the geotechnical evaluation for this project, Kleinfelder should be retained to confirm that the recommendations of this report are properly incorporated in the design of this project, and properly implemented during construction. 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 or rock 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 and evaluate the site earthwork operations to confirm subgrade soils are suitable for construction of foundations, slabs-on-grade, pavements and placement of engineered fill
- Observe foundation bearing soils to confirm conditions are as anticipated
- Confirm engineered fill for the structure and other improvements is placed and compacted per the project specifications

6.2 LIMITATIONS

This report may be used only by the District and the registered design professionals in responsible charge and only for the purposes stated for this specific engagement within a reasonable time from its issuance, but in no event later than two (2) years from the date of the report.

It is possible that soil conditions could vary beyond the points explored. If the scope of the proposed construction, including the proposed location, changes from that described in this report, we should be notified immediately in order that a review may be made and any supplemental recommendations provided.
We have prepared this report in substantial accordance with the generally accepted geotechnical engineering practice as it exists in the site area at the time of our study. No warranty expressed or implied is made.

Land use, site conditions (both on site and off site) or other factors may change over time, and additional subsurface exploration work may be required with the passage of time. Any party other than the client who wishes to use this report shall notify Kleinfelder of such intended use. Based on the intended use of the report, Kleinfelder may require that additional work be performed and that an updated report be issued. Non-compliance with any of these requirements by the client or anyone else will release Kleinfelder from any liability resulting from the use of this report by any unauthorized party.

The scope of services was limited to three borings. 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 current and previous subsurface explorations including borings drilled to a maximum depth of 30 feet; laboratory testing of soil plasticity, gradation, unit weight, and moisture content; and 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 involves 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, limited laboratory tests, and our present knowledge of the proposed construction. It is possible that soil, rock or groundwater conditions could vary between or beyond the points explored. If soil, rock or groundwater conditions are encountered during construction 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. If the scope of the proposed construction, including the estimated building loads, and the design depths or locations
of the foundations, changes from that described in this report, the conclusions and recommendations contained in this report are not considered valid unless the changes are reviewed, and the conclusions of this report are modified or approved in writing, by Kleinfelder.

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, 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 construction will be monitored on a full-time basis by a representative from Kleinfelder, including site preparation, preparation of foundations, and placement of engineered fill and trench backfill. These services provide Kleinfelder the opportunity to observe the actual soil, rock 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, opinion, recommendations, or conclusions contained in the report. Because of the limited nature of any subsurface study, the contractor may encounter conditions during construction which differ from those presented in this report. In such event, the contractor should promptly notify the owner so that Kleinfelder’s geotechnical engineer can be contacted to confirm those conditions. We recommend the contractor describe the nature and extent of the differing conditions in writing and that the construction contract include provisions for dealing with differing conditions. Contingency funds should be reserved for potential problems during earthwork and foundation construction. Furthermore, the contractor should be prepared to handle contamination conditions encountered at this site, which may affect the excavation, removal, or disposal of soil; dewatering of excavations; and health and safety of workers.
7 REFERENCES


California Department of Conservation Division of Mines and Geology, 1986, Guidelines for Geologic/Seismic Reports, DMG Note 42.

California Department of Conservation Division of Mines and Geology, 1987, Guidelines for Preparing Engineering Geologic Reports: DMG Note 44.


Kleinfelder, 2016a, Subsurface Fault Investigation, Proposed C-4001 Campus Safety Center, Contra Costa Community College, San Pablo, California (Project No. 20164720.001A/PLE16R42854),

Kleinfelder, 2016b, Subsurface Fault Investigation, Lower Parking Area, Contra Costa Community College, San Pablo, California, dated November 16, 2016 (Project Number 20164720.001A/PLE16R50013).


LIST OF ATTACHMENTS

The following figures are attached and complete this appendix.

<table>
<thead>
<tr>
<th>Figure</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Site Location Map</td>
</tr>
<tr>
<td>2</td>
<td>Site Plan, Boring Locations, and Site Geology Map</td>
</tr>
<tr>
<td>3</td>
<td>Regional Geologic Map</td>
</tr>
<tr>
<td>4</td>
<td>Area Geology Map</td>
</tr>
<tr>
<td>5</td>
<td>Geologic Cross Section A-A’</td>
</tr>
</tbody>
</table>
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PROJECT NO. 20181293.001A
DRAWN: 8/31/2017
DRAWN BY: D. Ross
CHECKED BY: B. Money
FILE NAME: 20181293_SLM.mxd

SITE LOCATION MAP

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Bright People. Right Solutions.
www.kleinfelder.com

PROJECT SITE

C-608 PE/KINESIOLOGY
CONTRA COSTA COLLEGE
2600 MISSION BELL DRIVE
SAN PABLO, CALIFORNIA
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EXPLANATION of units in SF Bay Area

- Cenozoic nonmarine
- Mesozoic Granitic rocks
- Cenozoic marine
- Mesozoic Ultramafic rocks
- Late Mesozoic shelf and slope
- Late Mesozoic of the Franciscan Formation
- Fault, dotted where concealed, arrows indicate direction of movement

PROJECT NO. 20181293
DRAWN: AUG 2017
DRAWN BY: J.S./D.R.
CHECKED BY: B.M.
FILE NAME: REGIONAL GEOLOGIC MAP.ppt

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LEGEND

SOIL BORING
(Kleinfelder, 2017)

GROUNDWATER LEVEL (INITIAL)
GROUNDWATER LEVEL (FINAL)

Qhb
HOLOCENE BASIN DEPOSITS
(fine-grained)

NOTE: SEE FIGURE 2 FOR LOCATION OF CROSS SECTION.
ALL LOCATIONS ARE APPROXIMATE.

APPROXIMATE VERTICAL SCALE: 1 inch = 10 feet

APPROXIMATE HORIZONTAL SCALE: 1 inch = 60 feet

GEOLOGIC CROSS SECTION A-A'

CONTRA COSTA COLLEGE
2600 MISSION BELL DRIVE
SAN PABLO, CALIFORNIA

PROJECT NO. 20181293.001A
DRAWN BY: D. Ross
CHECKED BY: B. Money
DATE: 08/31/2017

FIGURE 5

FILE NAME: 20164720_2.dwg
DRAWN BY: D. Ross
CHECKED BY: B. Money
DATE: 08/31/2017

This document is not intended for use as a land survey product nor is it designed or intended as a construction design document.

The use or misuse of the information contained on this graphic representation is at the sole risk of the party using or misusing the information.
# APPENDIX A
BORING LOGS

## LIST OF ATTACHMENTS

The following figures are attached and complete this appendix.

<table>
<thead>
<tr>
<th>Figure</th>
<th>Graphics Key</th>
</tr>
</thead>
<tbody>
<tr>
<td>Figure A-1</td>
<td>.......................................................................................... Graphics Key</td>
</tr>
<tr>
<td>Figure A-2</td>
<td>.......................................................................................... Soil Description Key</td>
</tr>
<tr>
<td>Figures A-3 through A-5</td>
<td>.......................................................................................... Boring Logs B-1 through B-3</td>
</tr>
</tbody>
</table>
The report and graphics key are an integral part of these logs. All data and interpretations in this log are subject to the explanations and limitations stated in the report. Lines separating strata on the logs represent approximate boundaries only. Actual transitions may be gradual or differ from those shown. No warranty is provided as to the continuity of soil or rock conditions between individual sample locations. Logs represent general soil or rock conditions observed at the point of exploration on the date indicated. In general, Unified Soil Classification System designations presented on the logs were based on visual classification in the field and were modified where appropriate based on gradation and index property testing. Fine grained soils that plot within the hatched area on the Plasticity Chart, and coarse grained soils with between 5% and 12% passing the No. 200 sieve require dual USCS symbols, i.e., GW-GM, GP-GM, GP-GC, GC-GM, GC-GC, SW-SM, SP-SM, SC-SM. If sampler is not able to be driven at least 6 inches then 50/X indicates number of blows required to drive the identified sampler X inches with a 140 pound hammer falling 30 inches.

### Abbreviations

- **WOH**: Weight of Hammer
- **WOR**: Weight of Rod

### Ground Water Graphics

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

### Notes

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- Logs represent general soil or rock conditions observed at the point of exploration on the date indicated.
- In general, Unified Soil Classification System designations presented on the logs were based on visual classification in the field and were modified where appropriate based on gradation and index property testing.
- Fine grained soils that plot within the hatched area on the Plasticity Chart, and coarse grained soils with between 5% and 12% passing the No. 200 sieve require dual USCS symbols, i.e., GW-GM, GP-GM, GP-GC, GC-GM, SW-SM, SP-SM, SC-SM.
- If sampler is not able to be driven at least 6 inches then 50/X indicates number of blows required to drive the identified sampler X inches with a 140 pound hammer falling 30 inches.

### A-1

**Figure**

**KLEINFELDER**

Bright People. Right Solutions.

C-608 PE/Kinesiology

Contra Costa College

2600 Mission Bell Drive

San Pablo, California

**Graphics Key**

<table>
<thead>
<tr>
<th>Sampler and Drilling Method Graphics</th>
<th>Unified Soil Classification System (ASTM D 2487)</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>BULK / GRAB / BAG SAMPLE</strong></td>
<td>CLEAN GRAVIES WITH &lt;5% FINES</td>
</tr>
<tr>
<td><strong>MODIFIED CALIFORNIA SAMPLER</strong></td>
<td>GW    WELL-GRADED GRAVELS, GRAVEL-SAND MIXTURES WITH LITTLE OR NO FINES</td>
</tr>
<tr>
<td>(2 or 2-1/2 in. (50.8 or 63.5 mm.), outer diameter)</td>
<td>GW-GM WELL-GRADED GRAVELS, GRAVEL-SAND MIXTURES WITH LITTLE FINES</td>
</tr>
<tr>
<td><strong>CALIFORNIA SAMPLER</strong></td>
<td>GP    POORLY GRADED GRAVELS, GRAVEL-SAND MIXTURES WITH LITTLE OR NO FINES</td>
</tr>
<tr>
<td>(3 in. (76.2 mm.), outer diameter)</td>
<td>GM    SILETLY GRAVELS, GRAVEL-SILT-SAND MIXTURES</td>
</tr>
<tr>
<td><strong>STANDARD PENETRATION SPLIT SPOON SAMPLER</strong></td>
<td>GRAVIES WITH 5% TO 12% FINES</td>
</tr>
<tr>
<td>(2 in. (50.8 mm.), outer diameter and 1-3/8 in. (34.9 mm.) inner diameter)</td>
<td>GW-GC WELL-GRADED GRAVELS, GRAVEL-SAND MIXTURES WITH LITTLE CLAY FINES</td>
</tr>
<tr>
<td><strong>HQ CORE SAMPLE</strong></td>
<td>GP-GM POORLY GRADED GRAVELS, GRAVEL-SAND MIXTURES WITH LITTLE CLAY FINES</td>
</tr>
<tr>
<td>(2.500 in. (63.5 mm.), core diameter)</td>
<td>GP-GC POORLY GRADED GRAVELS, GRAVEL-SAND MIXTURES WITH LITTLE CLAY FINES</td>
</tr>
<tr>
<td><strong>SHELBY TUBE SAMPLER</strong></td>
<td>GM    SILETLY GRAVELS, GRAVEL-SILT-SAND MIXTURES</td>
</tr>
<tr>
<td><strong>PUSH TYPE SAMPLER</strong></td>
<td>GC    CLAYEY GRAVELS, GRAVEL-SAND-CLAY MIXTURES</td>
</tr>
<tr>
<td><strong>SONIC CONTINUOUS SAMPER</strong></td>
<td>GC-GM CLAYEY GRAVELS, GRAVEL-SAND-CLAY MIXTURES</td>
</tr>
<tr>
<td><strong>HAND AUGER</strong></td>
<td></td>
</tr>
<tr>
<td><strong>AUGER CUTTINGS</strong></td>
<td></td>
</tr>
</tbody>
</table>

**Coarse Grained Soils**

<table>
<thead>
<tr>
<th>(More than half of material is larger than the #200 sieve)</th>
<th>CLEAN SANDS WITH &lt;5% FINES</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>SW WELL-GRADED SANDS, SAND-SAND MIXTURES WITH LITTLE OR NO FINES</td>
</tr>
<tr>
<td></td>
<td>LW-SM WELL-GRADED SANDS, SAND-SAND MIXTURES WITH LITTLE FINES</td>
</tr>
<tr>
<td></td>
<td>SW-SC WELL-GRADED SANDS, SAND-SAND MIXTURES WITH LITTLE CLAY FINES</td>
</tr>
<tr>
<td></td>
<td>SP-SM POORLY GRADED SANDS, SAND-SAND MIXTURES WITH LITTLE CLAY FINES</td>
</tr>
<tr>
<td></td>
<td>SP-SC POORLY GRADED SANDS, SAND-SAND MIXTURES WITH LITTLE CLAY FINES</td>
</tr>
<tr>
<td></td>
<td>SM    SILETLY SANDS, SAND-SAND MIXTURES WITH LITTLE CLAY FINES</td>
</tr>
<tr>
<td></td>
<td>SC    CLAYEY SANDS, SAND-SAND MIXTURES</td>
</tr>
<tr>
<td></td>
<td>SC-SM CLAYEY SANDS, SAND-SAND MIXTURES</td>
</tr>
</tbody>
</table>

**Fine Grained Soils**

<table>
<thead>
<tr>
<th>(Larger than half of material is smaller than the #200 sieve)</th>
<th>SILTS AND CLAYS (Liquid Limit less than 50)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>ML INORGANIC SILTS AND VERY FINE SANDS, SILT OR CLAYEY FINE SANDS, SILTS WITH SLIGHT PLASTICITY</td>
</tr>
<tr>
<td></td>
<td>CL INORGANIC CLAYS OF LOW TO MEDIUM PLASTICITY, GRAVELLY CLAYS, SANDY CLAYS, SILTY CLAYS, CLAYEY CLAYS</td>
</tr>
<tr>
<td></td>
<td>CL-ML INORGANIC CLAYS OF LOW PLASTICITY, GRAVELLY CLAYS, SANDY CLAYS, SILTY CLAYS, CLAYEY CLAYS</td>
</tr>
<tr>
<td></td>
<td>OL ORGANIC SILTS &amp; ORGANIC SILT CLAYS OF LOW PLASTICITY</td>
</tr>
<tr>
<td></td>
<td>MH ORGANIC SILTS, MICACEOUS OR DIATOMACEOUS FINE SAND OR SILT</td>
</tr>
<tr>
<td></td>
<td>CH INORGANIC CLAYS OF HIGH PLASTICITY, MOTIL CLAYS</td>
</tr>
<tr>
<td></td>
<td>OH ORGANIC CLAYS &amp; ORGANIC SILTS OF MEDIUM-TO-HIGH PLASTICITY</td>
</tr>
</tbody>
</table>

**GENERAL NOTES**

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- If sampler is not able to be driven at least 6 inches then 50/X indicates number of blows required to drive the identified sampler X inches with a 140 pound hammer falling 30 inches.

**Abbreviations**

- **WOR**: Weight of Rod
- **WOH**: Weight of hammer
The thread is easy to roll and not much time is required to reach the plastic limit. The thread cannot be rerolled after reaching the plastic limit. The lump or thread crumbles when drier than the plastic limit.

It takes considerable time rolling and kneading to reach the plastic limit. The thread can be rerolled several times after reaching the plastic limit. The lump or thread can be formed without crumbling when drier than the plastic limit.

**CONSISTENCY - FINE-GRAINED SOIL**

<table>
<thead>
<tr>
<th>CONSISTENCY</th>
<th>SPT-N60 (# blows/ft)</th>
<th>Pocket Pen (tsf)</th>
<th>UNCONFINED COMPRESSIVE STRENGTH (Q/psf)</th>
<th>VISUAL / MANUAL CRITERIA</th>
</tr>
</thead>
<tbody>
<tr>
<td>Very Soft</td>
<td>&lt;2</td>
<td>0.25 &lt; PP &lt; 0.5</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>0.5 &lt; PP &lt; 0.8</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>1 &lt; PP &lt; 2</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>2 &lt; PP &lt; 4</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>4 &lt; PP &lt; 6</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 &lt; PP &lt; 8</td>
<td>&gt;8000</td>
<td>Thumb will not indent soil.</td>
</tr>
</tbody>
</table>

**PLASTICITY / RELATIVE DENSITY - COARSE-GRAINED SOIL**

<table>
<thead>
<tr>
<th>APPARENT DENSITY</th>
<th>SPT-N60 (# blows/ft)</th>
<th>MODIFIED CASAMPLER (# blows/ft)</th>
<th>CALIFORNIA SAMPLER (# blows/ft)</th>
<th>RELATIVE DENSITY (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Very Loose</td>
<td>&lt;4</td>
<td>&lt;4</td>
<td>&lt;5</td>
<td>0 - 15</td>
</tr>
<tr>
<td>Loose</td>
<td>4 - 10</td>
<td>5 - 12</td>
<td>5 - 15</td>
<td>15 - 35</td>
</tr>
<tr>
<td>Medium Dense</td>
<td>10 - 30</td>
<td>12 - 35</td>
<td>15 - 40</td>
<td>35 - 65</td>
</tr>
<tr>
<td>Dense</td>
<td>30 - 50</td>
<td>35 - 60</td>
<td>40 - 70</td>
<td>65 - 85</td>
</tr>
<tr>
<td>Very Dense</td>
<td>&gt;50</td>
<td>&gt;60</td>
<td>&gt;70</td>
<td>85 - 100</td>
</tr>
</tbody>
</table>

**STRUCTURE**

<table>
<thead>
<tr>
<th>DESCRIPTION</th>
<th>CRITERIA</th>
</tr>
</thead>
<tbody>
<tr>
<td>Stratified</td>
<td>Alternating layers of varying material or color with layers at least 1/4-in. thick, note thickness.</td>
</tr>
<tr>
<td>Laminated</td>
<td>Alternating layers of varying material or color with the layer less than 1/4-in. thick, note thickness.</td>
</tr>
<tr>
<td>Fissured</td>
<td>Breaks along definite planes of fracture with little resistance to fracturing.</td>
</tr>
<tr>
<td>Slickensided</td>
<td>Fracture planes appear polished or glossy, sometimes striated.</td>
</tr>
<tr>
<td>Blocky</td>
<td>Cohesive soil that can be broken down into small angular lumps which resist further breakdown.</td>
</tr>
<tr>
<td>Lensed</td>
<td>Inclusion of small pockets of different soils, such as small lenses of sand scattered through a mass of clay; note thickness.</td>
</tr>
</tbody>
</table>

**SOIL DESCRIPTION KEY**

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

**PROJECT NO.: 20181293**

**DRAWN BY:** DR

**CHECKED BY:** BM

**DATE:** 8/1/2017

**REVISED:** 8/31/2017

**KLEINFELDER**

**C-608 PE/Kinesiology**
Contra Costa College
2600 Mission Bell Drive
San Pablo, California
**Lithologic Description**

- **ASPHALT**: about 2 inches
- **Sandy Lean CLAY (CL)**: medium plasticity, reddish tan, moist, very stiff, [FILL]
- **Fat CLAY (CH)**: high plasticity, dark brown, moist, stiff
- **Lean CLAY with Sand (CL)**: medium plasticity, olive, moist
- **Poorly graded SAND with Clay (SP-SC)**: tan, moist, loose, fine sand, medium plasticity, fines
- **Fat CLAY with Sand (CH)**: high plasticity, light brown, moist, medium stiff to stiff, fine sand
- **Fat CLAY (CH)**: high plasticity, olive, moist, medium stiff

**Laboratory Results**

<table>
<thead>
<tr>
<th>Depth (feet)</th>
<th>Sample Number</th>
<th>Sample Type</th>
<th>Soil Group/Grain Size</th>
<th>Water Content (%)</th>
<th>Liquid Limit</th>
<th>Plasticity Index</th>
</tr>
</thead>
<tbody>
<tr>
<td>1-2</td>
<td>1a</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>3-4</td>
<td>2a</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>5-6</td>
<td>3a</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>7-8</td>
<td>4a</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>9-10</td>
<td>5a</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>11-12</td>
<td>6</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

**Groundwater Level Information:**

Groundwater was observed at approximately 30.5 ft. below ground surface during drilling.

**General Notes:**

The boring was terminated at approximately 31.5 ft. below ground surface. The boring was backfilled with neat cement grout and capped with dyed concrete on July 28, 2017.
The boring was terminated because of practical hand auger refusal (↑) at approximately 3 ft. below ground surface. The boring was backfilled with soil cuttings on July 28, 2017.
Lean CLAY (CL): medium plasticity, dark brown, mottled with yellow, moist, stiff. [FILL]

Sandy Fat CLAY (CH): medium plasticity, bluish gray, mottled with brown, moist, very stiff. [FILL]

Fat CLAY (CH): high plasticity, dark brown, moist, very stiff

Fat CLAY with Sand (CH): high plasticity, light brownish gray, moist, very stiff

The boring was terminated at approximately 11.5 ft. below ground surface. The boring was backfilled with neat cement grout on July 28, 2017.

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

GENERAL NOTES:
APPENDIX B
LABORATORY RESULTS

LIST OF ATTACHMENTS

The following figures are attached and complete this appendix.

<table>
<thead>
<tr>
<th>Figure</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>Figure B-1</td>
<td>Laboratory Test Result Summary</td>
</tr>
<tr>
<td>Figure B-2</td>
<td>Atterberg Limits</td>
</tr>
<tr>
<td>Figure B-3</td>
<td>R-Value</td>
</tr>
<tr>
<td>Figure B-4</td>
<td>Triaxial Compression Test (UU)</td>
</tr>
<tr>
<td></td>
<td>Corrosion Test Results (Sunland Analytical)</td>
</tr>
<tr>
<td>Exploration ID</td>
<td>Depth (ft.)</td>
</tr>
<tr>
<td>----------------</td>
<td>-------------</td>
</tr>
<tr>
<td>B-1</td>
<td>1.0 - 5.0</td>
</tr>
<tr>
<td>B-1</td>
<td>3.0 - 3.5</td>
</tr>
<tr>
<td>B-2</td>
<td>0.0 - 5.0</td>
</tr>
<tr>
<td>B-2</td>
<td>1.0 - 1.5</td>
</tr>
<tr>
<td>B-3</td>
<td>3.5 - 4.0</td>
</tr>
<tr>
<td>B-3</td>
<td>5.5 - 6.0</td>
</tr>
</tbody>
</table>

Refer to the Geotechnical Evaluation Report or the supplemental plates for the method used for the testing performed above.

NP = NonPlastic
NA = Not Available
<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>1 - 5</td>
<td>Bulk A</td>
<td>SANDY LEAN CLAY (CL)</td>
<td>NM</td>
<td>39</td>
<td>18</td>
<td>21</td>
</tr>
<tr>
<td>B-1</td>
<td>3 - 3.5</td>
<td>1b</td>
<td>SANDY LEAN CLAY (CL)</td>
<td>NM</td>
<td>43</td>
<td>20</td>
<td>23</td>
</tr>
<tr>
<td>B-3</td>
<td>3.5 - 4</td>
<td>1a</td>
<td>LEAN CLAY (CL)</td>
<td>NM</td>
<td>45</td>
<td>19</td>
<td>26</td>
</tr>
</tbody>
</table>

Testing performed in general accordance with ASTM D4318.

NP = Nonplastic
NA = Not Available
NM = Not Measured

For classification of fine-grained soils and fine-grained fraction of coarse-grained soils.

**Chart Reference:** ASTM D2487
Testing performed in general accordance with ASTM D2844.

<table>
<thead>
<tr>
<th>Exploration ID</th>
<th>Depth (ft.)</th>
<th>Sample Number</th>
<th>Sample Description</th>
<th>R-Value @ 300 psi Exudation Pressure</th>
</tr>
</thead>
<tbody>
<tr>
<td>B-2</td>
<td>0 - 3</td>
<td>Bulk A</td>
<td>SANDY LEAN CLAY (CL)</td>
<td>17</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Specimen No.</th>
<th>Moisture at Time of Test (%)</th>
<th>Dry Unit Weight (pcf)</th>
<th>Expansion Pressure (psi)</th>
<th>Exudation Pressure (psi)</th>
<th>Corrected Resistance Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>21.4</td>
<td>106.7</td>
<td>87</td>
<td>784</td>
<td>29</td>
</tr>
<tr>
<td>2</td>
<td>23.4</td>
<td>100.2</td>
<td>48</td>
<td>379</td>
<td>20</td>
</tr>
<tr>
<td>3</td>
<td>25.4</td>
<td>96.3</td>
<td>35</td>
<td>198</td>
<td>14</td>
</tr>
</tbody>
</table>
TRIAXIAL COMPRESSION TEST (UU)

Specimen No. | 1
--- | ---
Diameter, in | D_0 2.432
Height, in | H_0 4.978
Water Content, % | \( \omega_0 \) 15.9
Dry Density, lbs/ft^3 | \( \gamma_d \) 112.1
Saturation, % | S_0 89
Void Ratio | \( \theta_0 \) 0.475
Minor Principal Stress, ksf | \( \sigma_3 \) 0.36
Maximum Deviator Stress, ksf | \( (\sigma_1-\sigma_3)_{\text{max}} \) 6.46
Time to \( (\sigma_1-\sigma_3)_{\text{min}} \), min | \( t_r \) 3.55
Deviator Stress @ 15% Axial Strain, ksf | \( (\sigma_1-\sigma_3)_{0.15} \) 5.71
Ultimate Deviator Stress, ksf | \( (\sigma_1-\sigma_3)_{\text{ult}} \) na
Rate of strain, %/min | \( \varepsilon \) 1.00
Axial Strain at Failure, % | \( \varepsilon_f \) 3.55

Description of Specimen: Olive Sandy Lean Clay

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

LL: nm | PL: nm | PI: nm | G_s: 2.65 Assumed | Specimen Type: Undisturbed | Test Method: ASTM D2850

Membrane correction applied

<table>
<thead>
<tr>
<th>Boring</th>
<th>Sample</th>
<th>Depth, ft</th>
<th>Test Date</th>
</tr>
</thead>
<tbody>
<tr>
<td>B-1</td>
<td>1A</td>
<td>3.5-4.0</td>
<td>8/10/2017</td>
</tr>
</tbody>
</table>

Remarks: nm= not measured, na = not applicable
To: Becky Money  
Kleinfelder-Sacramento  
2882 Prospect Park Dr.Ste 200  
Rancho Cordova CA, 95670

From: Gene Oliphant, Ph.D. \ Randy Horney  
General Manager \ Lab Manager

The reported analysis was requested for the following:  
Location: 20181293.001A  Site ID: B1 BULK A  
Thank you for your business.

* For future reference to this analysis please use SUN # 74949 - 156460

<table>
<thead>
<tr>
<th>EVALUATION FOR SOIL CORROSION</th>
</tr>
</thead>
<tbody>
<tr>
<td>Soil pH</td>
</tr>
<tr>
<td>Minimum Resistivity</td>
</tr>
<tr>
<td>Chloride</td>
</tr>
<tr>
<td>Sulfate-S</td>
</tr>
</tbody>
</table>

**METHODS:**
P.H and Min.Resistivity CA DOT Test #643 Mod.(Sm.Cell)  
Sulfate CA DOT Test #417, Chloride CA DOT Test #422
APPENDIX C
LOGS FROM PREVIOUS STUDIES
The information included on this graphic representation has been compiled from a variety of sources and is subject to change without notice. Kleinfelder makes no representations or warranties, express or implied, as to accuracy, completeness, timeliness, or rights to the use of such information. This document is not intended for use as a land survey product nor is it designed or intended as a construction design document. The use or misuse of the information contained on this graphic representation is at the sole risk of the party using or misusing the information.
## Lithologic Description

<table>
<thead>
<tr>
<th>Depth (feet)</th>
<th>Sample Number</th>
<th>Lithologic Description</th>
<th>Sample Type</th>
<th>Sample Code</th>
<th>Water Content (%)</th>
<th>USCS Symbol</th>
<th>Plasticity Index</th>
<th>Liquid Limit</th>
<th>Passing #200 (%)</th>
<th>Passing #4 (%)</th>
<th>Remarks</th>
</tr>
</thead>
<tbody>
<tr>
<td>0-2</td>
<td>1b</td>
<td>Asphalt, about 2 inches</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>2-5</td>
<td>1a</td>
<td>Sandy SILT (ML): low plasticity, yellowish brown, moist, fine sand</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>3</td>
<td>1c</td>
<td>Lean CLAY (CL): medium plasticity, dark brown to black with olive brown mottling, moist, firm</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>5-10</td>
<td>2a</td>
<td>Fat CLAY (CH): high plasticity, dark brown with olive brown mottling, moist, firm</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>15-20</td>
<td>4b</td>
<td>Sandy Lean CLAY (CL): low plasticity, olive brown, moist, soft</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>20</td>
<td>4b</td>
<td>yellowish brown, wet, fine sand</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>25</td>
<td>5b</td>
<td>Well graded SAND with Gravel (SW): brown, wet, dense, fine to coarse sand, fine subangular gravel</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>30</td>
<td>6b</td>
<td>Lean CLAY (CL): medium plasticity, olive brown, wet, soft</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>30</td>
<td>8b</td>
<td>Poorly graded SAND (SP): brown, wet, medium dense, fine to medium sand</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

The boring was terminated at approximately 30 ft. below ground surface. The boring was backfilled with cement grout using tremie pipe, patched surface with quickset concrete with black dye on February 17, 2017.

**Groundwater Level Information:**
- Groundwater was observed at approximately 15 ft. below ground surface during drilling.
- Groundwater was observed at approximately 15 ft. below ground surface at the end of drilling.

**General Notes:**
- Corrosion Testing
- PP=0.5 tsf
- PP=3.0 tsf
- PP=1.0 tsf
- PP=0.5 tsf
The boring was terminated at approximately 25 ft. below ground surface. The boring was backfilled with cement grout using tremie pipe, patched surface with quickset concrete with black dye on February 17, 2017.
<table>
<thead>
<tr>
<th>Exploration ID</th>
<th>Depth (ft.)</th>
<th>Sample No.</th>
<th>Sample Description</th>
<th>Water Content (%)</th>
<th>Dry Unit Wt. (pcf)</th>
<th>Sieve Analysis (%)</th>
<th>Atterberg Limits</th>
<th>Additional Tests</th>
</tr>
</thead>
<tbody>
<tr>
<td>B-1</td>
<td>0.0 - 2.0</td>
<td>1b</td>
<td>LEAN CLAY (CL)</td>
<td></td>
<td></td>
<td>18.9</td>
<td>40 18 22</td>
<td></td>
</tr>
<tr>
<td>B-1</td>
<td>0.5 - 2.0</td>
<td>1a</td>
<td>FAT CLAY (CH)</td>
<td></td>
<td></td>
<td>104.1</td>
<td>67 22 45</td>
<td></td>
</tr>
<tr>
<td>B-1</td>
<td>2.0 - 4.0</td>
<td>2a</td>
<td>FAT CLAY (CH)</td>
<td></td>
<td></td>
<td>81</td>
<td>29 17 12</td>
<td></td>
</tr>
<tr>
<td>B-1</td>
<td>10.5 - 11.0</td>
<td>2b</td>
<td>LEAN CLAY (CL)</td>
<td></td>
<td></td>
<td></td>
<td>61</td>
<td></td>
</tr>
<tr>
<td>B-1</td>
<td>14.0 - 14.5</td>
<td>5b</td>
<td>SANDY LEAN CLAY (CL)</td>
<td></td>
<td></td>
<td>61</td>
<td>6.5</td>
<td></td>
</tr>
<tr>
<td>B-1</td>
<td>24.5 - 25.0</td>
<td>7a</td>
<td>WELL- GRADED SAND WITH GRAVEL (SW)</td>
<td></td>
<td></td>
<td></td>
<td>6.5</td>
<td></td>
</tr>
<tr>
<td>B-2</td>
<td>1.5 - 2.0</td>
<td>1c</td>
<td>LEAN CLAY (CL)</td>
<td></td>
<td></td>
<td>31.9</td>
<td>46 20 28</td>
<td></td>
</tr>
<tr>
<td>B-2</td>
<td>2.0 - 4.0</td>
<td>2a</td>
<td>FAT CLAY (CH)</td>
<td></td>
<td></td>
<td>86.9</td>
<td>59 11 48</td>
<td></td>
</tr>
<tr>
<td>B-2</td>
<td>10.5 - 11.0</td>
<td>6b</td>
<td>LEAN CLAY (CL)</td>
<td></td>
<td></td>
<td></td>
<td>44 18 26</td>
<td></td>
</tr>
<tr>
<td>Exploration ID</td>
<td>Depth (ft.)</td>
<td>Sample Number</td>
<td>Sample Description</td>
<td>Passing #200</td>
<td>LL</td>
<td>PL</td>
<td>PI</td>
<td></td>
</tr>
<tr>
<td>---------------</td>
<td>-------------</td>
<td>---------------</td>
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<td>--------------</td>
<td>-----</td>
<td>-----</td>
<td>-----</td>
<td></td>
</tr>
<tr>
<td>● B-1</td>
<td>0 - 2</td>
<td>1b</td>
<td>LEAN CLAY (CL)</td>
<td>NM</td>
<td>40</td>
<td>18</td>
<td>22</td>
<td></td>
</tr>
<tr>
<td>▲ B-1</td>
<td>2 - 4</td>
<td>2a</td>
<td>FAT CLAY (CH)</td>
<td>NM</td>
<td>67</td>
<td>22</td>
<td>45</td>
<td></td>
</tr>
<tr>
<td>▲ B-1</td>
<td>14 - 14.5</td>
<td>5b</td>
<td>SANDY LEAN CLAY (CL)</td>
<td>NM</td>
<td>29</td>
<td>17</td>
<td>12</td>
<td></td>
</tr>
<tr>
<td>● B-2</td>
<td>0 - 2</td>
<td>1c</td>
<td>LEAN CLAY (CL)</td>
<td>NM</td>
<td>46</td>
<td>20</td>
<td>26</td>
<td></td>
</tr>
<tr>
<td>○ B-2</td>
<td>2 - 4</td>
<td>2a</td>
<td>FAT CLAY (CH)</td>
<td>NM</td>
<td>59</td>
<td>11</td>
<td>48</td>
<td></td>
</tr>
<tr>
<td>○ B-2</td>
<td>10.5 - 15</td>
<td>6b</td>
<td>LEAN CLAY (CL)</td>
<td>NM</td>
<td>44</td>
<td>18</td>
<td>26</td>
<td></td>
</tr>
</tbody>
</table>

Testing performed in general accordance with ASTM D4318.
NP = Nonplastic
NA = Not Available
NM = Not Measured

For classification of fine-grained soils and fine-grained fraction of coarse-grained soils.
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Approximate potentially Active Fault Location

NOTE: All locations are approximate.
TRENCH T-2 (2016)

N18E  SE FACE

DISTANCE IN FEET

ELEVATION IN FEET

3-INCH DIAMETER GRAY PVC PIPE

2-INCH DIAMETER GRAY PVC PIPE

LAYERED FILL

CARBONATE FILAMENT IMMEDIATELY ABOVE & BELOW MARKER BED

CAVE-INS

LOGGED BY:
Sadek M. Derrega, CEG
James Wetenkamp, CEG

SEE TRENCH T-1 (2016) FOR UNIT DESCRIPTIONS
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File Name: STO16D091.CAD

Drawn by: G.G.

Checked by: J.W.

Drawn: 10/10/2016

Project No.: 20171125

Lean CLAY (CL), dark gray to black, moist, stiff, rootlets finger into lower unit (Holocene basin deposit, "modern" soil)

Sandy SILT (ML), light yellowish brown, moist, stiff (Holocene basin deposit)

Lean CLAY (CL), dark brown to black, moist, stiff, blocky, prismatic structure (Holocene basin deposit, Paleosol)

Sandy Lean CLAY with Silt (CL), yellowish brown to gray brown, moist, stiff (Holocene basin deposit)

FILL: Light olive brown sandy lean clay to clayey sand with gravel, cobbles & debris

Lean CLAY with Silt (CL), light olive, moist, medium stiff (Holocene basin deposit)

Lean CLAY (CL), dark grayish brown, moist, stiff (Holocene basin deposit, Paleosol marker bed)

Lean CLAY with Silt (CL), light olive, moist, medium stiff, resembles layer 6 (Holocene basin deposit)

Lean CLAY with Sand (CL), gray brown, moist, stiff, (Holocene basin deposit, Paleosol)
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NOTE: All locations are approximate.

SITE PLAN AND TRENCH LOCATIONS PHASE I

LOWER CAMPUS FAULT TRENCH INVESTIGATION
CONTRA COSTA COLLEGE
SAN PABLO, CALIFORNIA

PROJECT NO. 2017/125
DRAWN BY: JDS
CHECKED BY: SD
DATE: 10/21/2016
REVISED:

LEGEND

TRENCH LOCATION (This Study)

T-6 (2016)

T-7 (2016)

T-5 (2016)

SCALE: 1" = 80' SCALE IN FEET
SOURCE: Esri, dated 04-12-2014
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Layered FILL

5" Diameter metal pipe

3 1/2" Concrete

4-6" Aggregate base

1/2" Diameter PVC pipe

1" Diameter metal pipe

Two 4" Diameter metal pipe

3" Diameter Gray PVC pipe

Free water at base of trench

Carbonate Filaments right above & below Paleosol marker bed

Dark gray brown Paleosol marker bed

Lean CLAY (CL), dark gray to black, moist, stiff, rootlets finger into lower unit (Holocene basin deposit, "modern" soil)

Lean CLAY with Silt (CL), light olive, moist, medium stiff (Holocene basin deposit)

Lean CLAY (CL), dark grayish brown, moist, stiff (Holocene basin deposit, Paleosol marker bed)

Lean CLAY with Silt (CL), light olive, moist to wet, medium stiff, resembles layer 2 (Holocene basin deposit)
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Lean CLAY (CL), dark gray to black, moist, stiff, rootlets finger into lower unit (Holocene basin deposit, "modern" soil)
Lean CLAY with Silt (CL), light olive, moist, medium stiff (Holocene basin deposit)
Lean CLAY (CL), dark grayish brown, moist, stiff (Holocene basin deposit, Paleosol marker bed)
Lean CLAY with Silt (CL), light olive, moist, medium stiff, resembles layer 2 (Holocene basin deposit)
Lean CLAY with Silt (CL), light olive, moist to wet, medium stiff, resembles layers 2 and 4 (Holocene basin deposit)
Sandy SILT (ML), gray brown, moist, medium stiff, fine to medium grain sand (Holocene basin deposit)
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Lean CLAY (CL), dark gray to black, moist, stiff, rootlets finger into lower unit (Holocene basin deposit, "modern" soil)
Lean CLAY with Silt (CL), light olive, moist, medium stiff (Holocene basin deposit)
Poorly sorted SAND with silt and gravel, brown, moist, medium dense, fine to coarse sand with fine gravel, rounded to angular (Holocene channel deposit)
Lean CLAY with Silt (CL), light olive, moist, medium stiff, resembles layer 2 (Holocene basin deposit)
Lean CLAY with Silt (CL), light olive, moist, resembles layers 2 & 4 (Holocene basin deposit)
Lean CLAY (CL), dark grayish brown, moist, stiff (Holocene basin deposit, Paleosol marker bed)
Lean CLAY with Silt (CL), light olive, moist to wet, medium stiff, resembles layers 2 & 4 (Holocene basin deposit)

Carbonate Filaments
right above & below
Paleosol marker bed

Dark gray brown
Paleosol marker bed

Layered FILL
Fault trench (2007)
A/P creeping fault trace
A/P Fault trace approximately located long dash) inferred (short dash)
Southwest margin of A/P zone
Campus property line

Base aerial photograph provided by Contra Costa Community College

SITE AERIAL PHOTOGRAPH
Contra Costa College
San Pablo, California

KLEINFELDER
7133 Koll Center Parkway, Suite 100
Pleasanton, California 94566
Ph. (925) 484-1700 Fax. (925) 484-5838

PLATE 2

© Kleinfelder, 2007
<table>
<thead>
<tr>
<th>Depth, ft</th>
<th>Sample</th>
<th>Blow/ft</th>
<th>Density (g/cc)</th>
<th>Moisture Content (%)</th>
<th>Compress. Strength (psi)</th>
<th>Other Tests</th>
<th>Pen, tf</th>
<th>Field Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>53</td>
<td>101</td>
<td>19.7</td>
<td>LL=42; Pl=28</td>
<td>2.5</td>
<td></td>
<td></td>
<td></td>
<td>ASPHALTIC CONCRETE - approximately 2 inches thick</td>
</tr>
<tr>
<td>5</td>
<td>2.5</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>SANDY LEAN CLAY (CL) - olive-brown, moist, very hard, low plasticity, fine grained sand, with some gravel (FILL) - approximately 6-inch fat clay lense</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>FAT CLAY (CH) - black, moist, very hard, medium to high plasticity</td>
</tr>
</tbody>
</table>

Boring terminated at approximately 5 feet below ground surface. No groundwater encountered. Boring backfilled with cement grout and capped with cold patch.
STANDING WATER AT APPROXIMATELY 12.5-FOOT DEPTH.

- SILTY CLAY (CL) - mottled light brown, dark gray and black, dry to slightly moist, very stiff to hard, medium to high plasticity, with mottles; contact is uneven and mixed with top of Unit 3 (FILL)
- CLAYEY SAND (SC) - olive-brown, moist, dense, with fine- to medium-grained sand, trace gravel
- CLAY with minor GRAVEL (CH) - black, dry to moist, very stiff, with mottles, fine fractures filled with fine-grained sand from detrital sandstone pebbles, high organic content, upper 3 inches have poorly to moderately developed soil pediments; gradational contact with Unit 4
- SILTY CLAY (CH) - medium gray-brown, moist, stiff to very stiff, minor iron staining, no evidence of soil pediments; gradational contact with Unit 5 based on change in color (weathering front)
- SILTY LEAN CLAY (CL/CH) - dark gray-brown, moist, stiff, minor iron staining, some worm tubes infilled with light gray silt clay, abundant open intrasandy worm and nodular tubes; gradational contact with Unit 6 based on change in texture and color
- SILTY LEAN CLAY (CL) - dark gray, moist, stiff, faint gradational contact with Unit 7
- SILTY LEAN CLAY (CL) - dark gray-brown, moist, stiff, with water soaps
Legend

B-1  Soil Boring (Kleinfelder, July 2007)

Note: Locations are approximate.

## Unified Soil Classification System

<table>
<thead>
<tr>
<th>MAJOR DIVISIONS</th>
<th>LTR</th>
<th>ID</th>
<th>DESCRIPTION</th>
<th>MAJOR DIVISIONS</th>
<th>LTR</th>
<th>ID</th>
<th>DESCRIPTION</th>
</tr>
</thead>
<tbody>
<tr>
<td>GRAVEL AND GRAVELLY</td>
<td>GW</td>
<td></td>
<td>Well-graded gravels or gravel with sand, silt, or clay.</td>
<td>SILTS AND CLAYS</td>
<td>ML</td>
<td></td>
<td>Inorganic silts and very fine sands, rock flour or clayey silts with slight plasticity.</td>
</tr>
<tr>
<td></td>
<td>GP</td>
<td></td>
<td>Poorly-graded gravels or gravel with sand, silt, or clay.</td>
<td></td>
<td>CL</td>
<td></td>
<td>Inorganic fine clays of low to medium plasticity, gravelly clays, sandy clays, silt clays.</td>
</tr>
<tr>
<td></td>
<td>GM</td>
<td></td>
<td>Silty gravels, silty gravel with sand mixture.</td>
<td></td>
<td>OL</td>
<td></td>
<td>Organic silts and organic silty clays of low plasticity.</td>
</tr>
<tr>
<td></td>
<td>GC</td>
<td></td>
<td>Clayey gravels, clayey gravel with sand mixture.</td>
<td></td>
<td>MH</td>
<td></td>
<td>Inorganic plastic silts, micaeous or diatomaceous or silty soils.</td>
</tr>
<tr>
<td>COARSE GRAINED SOILS</td>
<td>SW</td>
<td></td>
<td>Well-graded sands or gravelly sands, silt, or clay.</td>
<td>SILTS AND CLAYS</td>
<td>CH</td>
<td></td>
<td>Inorganic silt clays (high plasticity).</td>
</tr>
<tr>
<td></td>
<td>SP</td>
<td></td>
<td>Poorly-graded sands or gravelly sands, silt, or clay.</td>
<td></td>
<td>OH</td>
<td></td>
<td>Organic clays of medium high to high plasticity.</td>
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<tr>
<td></td>
<td>SM</td>
<td></td>
<td>Silty sand.</td>
<td></td>
<td></td>
<td></td>
<td></td>
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<tr>
<td></td>
<td>SC</td>
<td></td>
<td>Clayey sand.</td>
<td>HIGHLY ORGANIC SOILS</td>
<td>PI</td>
<td></td>
<td>Peat and other highly organic soils.</td>
</tr>
</tbody>
</table>

### Standard Penetration Split Spoon Sampler
- 2.0 inch O.D., 1.4 inch I.D.

### Modified California Sampler
- 2.5 inch O.D., 2.0 inch I.D.

### Bulk Sample
- California Sampler, 3.0 inch O.D., 2.5 inch I.D.

### Shelby Tube
- 3.0 inch O.D.

### Approximate Water Levels
- First observed in boring: Time recorded in reference to a 24 hour clock.
- Observed following drilling: Air logged times.

### Pocket Penetrometers
- Reading, in tsf
- Torvane shear strength, in ksf

<table>
<thead>
<tr>
<th>LL</th>
<th>LIQUID LIMIT</th>
</tr>
</thead>
<tbody>
<tr>
<td>PI</td>
<td>PLASTICITY INDEX</td>
</tr>
<tr>
<td>% #200</td>
<td>SIEVE ANALYSIS (#200 SCREEN)</td>
</tr>
<tr>
<td>DS</td>
<td>DIRECT SHEAR</td>
</tr>
<tr>
<td>C</td>
<td>COHESION (PSF)</td>
</tr>
<tr>
<td>PHI</td>
<td>FRICTION ANGLE</td>
</tr>
<tr>
<td>TX</td>
<td>TRIAXIAL SHEAR</td>
</tr>
<tr>
<td>CONSOL</td>
<td>CONSOLIDATION</td>
</tr>
<tr>
<td>R-Value</td>
<td>RESISTANCE VALUE</td>
</tr>
<tr>
<td>SE</td>
<td>SAND EQUIVALENT</td>
</tr>
<tr>
<td>EI</td>
<td>EXPANSION INDEX</td>
</tr>
<tr>
<td>FS</td>
<td>FREE SWELL (U.S.B.R.)</td>
</tr>
</tbody>
</table>

**Notes:**
- Blow counts represent the number of blows a 140-pound hammer falling 30 inches required to drive a sampler through the last 12 inches of an 18 inch penetration, unless otherwise noted.
- The lines separating strata on the logs represent approximate boundaries only. The actual transition may be gradual.
- No warranty is provided as to the continuity of soil strata between borings. Logs represent the soil section observed at the boring location on the date of drilling only.
<table>
<thead>
<tr>
<th>Depth, ft</th>
<th>Sample</th>
<th>Borehole</th>
<th>Dry Density,pcf</th>
<th>Moisture Content,%</th>
<th>Compress. Strength, tf</th>
<th>Other Tests</th>
<th>Pen, tf</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

**DESCRIPTION**

- **ASPHALTIC CONCRETE**: approximately 3 inches thick
- **GRAVEL**: approximately 8 inches thick, gravelly soil material, possibly aggregate baserock (FILL)
- **LEAN CLAY (CL)**: dark gray-brown, moist, medium plasticity (HOLOCENE)
  - color changes to dark brown
  - dark brown, soft
  - firm, with organic staining
- **SANDY LEAN CLAY (CL)**: brown, moist, soft, low to medium plasticity, fine grained sand
  - iron-oxide staining
- **LEAN CLAY (CL)**: dark brown, moist, firm, medium plasticity, organic and iron-oxide staining

**LOG OF BORING NO. B-1**

**KLEINFELDER**

GYMNASIUM LIQUEFACTION / GEOLOGY ASSESSMENT
CONTRA COSTA COLLEGE
SAN PABLO, CALIFORNIA

**PROJECT NO. 82074 / BORES**
<table>
<thead>
<tr>
<th>Field</th>
<th>Laboratory</th>
<th>Description</th>
</tr>
</thead>
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<tr>
<td>Depth (ft)</td>
<td>Sample</td>
<td>Blows/ft</td>
</tr>
<tr>
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<tr>
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<td>56</td>
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<tr>
<td>45</td>
<td>17</td>
<td></td>
</tr>
<tr>
<td>50</td>
<td>50/5&quot;</td>
<td></td>
</tr>
<tr>
<td>55</td>
<td></td>
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<td></td>
<td></td>
</tr>
<tr>
<td>70</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

*Continued from previous plate*

**LEAN CLAY (CL)- continued**
- hard, iron-oxide staining

**WELL-GRADED GRAVEL with CLAY (GW-GC)**
- brown, firm
- moist, dense, medium plasticity, coarse grained angular gravel

Boring terminated at approximately 51.5 feet below ground surface.
Boring backfilled with cement grout.
**Date Completed:** 7/20/07  
**Logged By:** O. Khan  
**Total Depth:** Approximately 51.5 ft  
**Drilling method:** 8" Hollow Stem Auger  
**Hammer Wt.:** 140 lbs., 30" drop

<table>
<thead>
<tr>
<th>FIELD</th>
<th>LABORATORY</th>
</tr>
</thead>
<tbody>
<tr>
<td>Depth</td>
<td>Sample</td>
</tr>
<tr>
<td>ft</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Sample</td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
</tbody>
</table>

**DESCRIPTION**

Surface Elevation: Estimated 50 feet (MSL)

- **ASPHALTIC CONCRETE** approximately 3 inches thick
- **GRAVEL** - approximately 6 inches thick, gravelly soil material, possibly aggregate baserock (FILL)
- **FAT CLAY (CH)** - brown, moist, medium to high plasticity, iron-oxide staining (HOLOCENE)
  - dark gray, hard
  - dark gray-brown, firm, iron-oxide staining
  - brown, soft, iron-oxide and organic staining
- **SANDY LEAN CLAY (CL)** - brown, wet, low plasticity, firm, fine grained sand, iron-oxide and organic staining
- **LEAN CLAY (CL)** - dark brown, moist, firm, medium plasticity, iron-oxide and organic staining
  - brown, firm, iron-oxide staining
- **SANDY LEAN CLAY (CL)** - brown, moist, soft, fine grained sand

**LOG OF BORING NO. B-2**

KLEINFELDER  
GYMNASIUM LIQUEFACTION / GEOLOGY ASSESSMENT  
CONTRA COSTA COLLEGE  
SAN PABLO, CALIFORNIA  

PROJECT NO. 82074 / BORES  
PLATE 4
(Continued from previous plate)

SANDY LEAN CLAY (CL) continued

PASSING -#200=20% (see Plate 8)

SILTY SAND (SM) - dark gray, wet, medium dense, medium grained sand

- no recovery

SILTY LEAN CLAY (CL) olive-gray, moist, hard, iron-oxide staining

Boring terminated at approximately 51.5 feet below ground surface.
Boring backfilled with cement grout.
Date Completed: 7/20/07  
Drilling method: 8" Hollow Stem Auger  
Logged By: O. Khan  
Total Depth: Approximately 51.5 ft  
Hammer Wt: 140 lbs., 30" drop  
Notes:  

DESCRIPTION  
Surface Elevation: Estimated 50 feet (MSL)  

ASPHALTIC CONCRETE - approximately 2 inches thick  
GRAVEL - approximately 4 inches thick, gravelly soil material, possibly aggregate baserock (FILL)  
LEAN CLAY (CL) - brown, moist, medium plasticity  
(HOLOCENE)  
- dark gray-brown  
- dark brown, firm, medium plasticity, iron-oxide staining  
- soft, iron-oxide staining  
WELL-GRADED SAND with SILT (SW-SM) olive-brown, very moist, fine to medium grained sand, trace clay  
SANDY LEAN CLAY (CL) brown, moist, firm, low plasticity, fine grained sand  
LEAN CLAY (CL) - dark brown, moist, firm, medium plasticity, iron-oxide staining  

KLEINFELDER  
LOG OF BORING NO. B-3  
GYMNASium LIQUEFACTION / GEOLOGY ASSESSMENT  
CONTRA COSTA COLLeGE  
SAN PABLO, CALIFORNIA  
PROJECT NO. 82074 / BORES  
PLATE 5
<table>
<thead>
<tr>
<th>Depth, ft</th>
<th>Sample</th>
<th>Blow/vtt</th>
<th>Dry Density,pcf</th>
<th>Moisture Content,%</th>
<th>Compress. Strength, tf</th>
<th>Other Tests</th>
<th>Pen. tf</th>
</tr>
</thead>
<tbody>
<tr>
<td>13</td>
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<td>1.3-1.5</td>
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<td>70</td>
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<td></td>
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<td></td>
</tr>
</tbody>
</table>

**DESCRIPTION**

(Continued from previous plate)

**LEAN CLAY (CL)- continued**
- iron-oxide and organic staining

- dark gray-brown, firm to hard

- olive-brown, firm

**Silty Sand (SM)- dark gray, moist, medium to coarse grained sand, medium dense, subrounded gravel up to 1/4-inch**

**LEAN CLAY (CL)- olive-brown, moist, firm to hard, medium plasticity**

Boring terminated at approximately 51.5 feet below ground surface.
Boring backfilled with cement grout.
### Unified Soil Classification

#### Fine Grained Soil Groups

<table>
<thead>
<tr>
<th>Symbol</th>
<th>LL &lt; 50</th>
<th>Symbol</th>
<th>LL &gt; 50</th>
</tr>
</thead>
<tbody>
<tr>
<td>ML</td>
<td>Inorganic clayey silts to very fine sands of slight plasticity</td>
<td>MH</td>
<td>Inorganic silts and clayey silts of high plasticity</td>
</tr>
<tr>
<td>CL</td>
<td>Inorganic clays of low to medium plasticity</td>
<td>CH</td>
<td>Inorganic clays of high plasticity</td>
</tr>
<tr>
<td>OL</td>
<td>Organic silts and organic silty clays of low plasticity</td>
<td>OH</td>
<td>Organic clays of medium to high plasticity, organic silts</td>
</tr>
</tbody>
</table>

---

### Sample Descriptions

- **Symbol**: B-2
- **Depth, ft**: 6.0
- **LL**: 65
- **PL**: 18
- **PI**: 47
- **Sample Description**: Dark Gray Fat Clay (CH)

- **Symbol**: B-2
- **Depth, ft**: 21.0
- **LL**: 30
- **PL**: 17
- **PI**: 13
- **Sample Description**: Brown Sandy Lean Clay (CL)
Particle Size Analysis

<table>
<thead>
<tr>
<th>SYMBOL</th>
<th>BORING</th>
<th>DEPTH, ft</th>
<th>SAMPLE DESCRIPTION</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>B-2</td>
<td>41.0</td>
<td>Dark Gray Silty Sand (SM)</td>
</tr>
</tbody>
</table>

*PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 422*
APPENDIX D
SEISMIC HAZARD ANALYSIS

INTRODUCTION

This Appendix presents the results of our site-specific seismic hazard analysis per ASCE 7-10 (ASCE 2010) and Chapter 16A of 2016 California Building Code for the C-608 PE/Kinesiology Renovation project at Contra Costa Community College in San Pablo, California. The subsurface soil conditions used in this study were obtained from our current geotechnical investigations at the project site. Since the mapped $S_1$ value is greater than 0.75g, a site-specific ground motion hazard analysis is required per Section 1616A.1.3 of 2016 CBC.

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 84th percentile deterministic per Chapter 21 of ASCE 7-10.
- Developing site-specific response spectra for the MCE$_R$ and the DE per Chapter 21 of the ASCE 7-10 for damping value of 5%.
- Developing site-specific ground motion parameters ($S_{MS}$, $S_{M1}$, $S_{DS}$, and $S_{D1}$) per Section 21.4 of the ASCE 7-10.
- Estimating site-specific PGA$_M$ per Section 21.5 of ASCE 7-10.
- Report preparation of the results of the site-specific seismic hazard analyses.

It should be noted that a site-specific seismic hazard analysis was performed for the Campus Safety Center, just northeast of this site. Since the subsurface soil conditions are similar and the location of this site is not far from the Campus Safety Center, we did not perform PSHA and DSHA for this site and used the results from previous studies. However, some of the code seismic design values for this site are slightly different than the values for the Campus Safety Center site. Therefore, site-specific MCE$_R$ and DE spectra and seismic design parameters are updated to reflect that.
PROJECT LOCATION

The project site is located in the Contra Costa Community College in San Pablo, California. We have used center of the existing gymnasium as the site location and the approximate site coordinates used for the seismic hazard analysis are:

- **Latitude:** 37.9689° N
- **Longitude:** -122.3402° W

REGIONAL FAULTING

According to Hart and Bryant (1997), the site is located within an Alquist-Priolo Earthquake Fault Zone for the Hayward-Rodgers Creek fault. Other faults located close to the site are, the West Napa fault at about 23 km, the Green Valley Connected fault at about 25 km, the Mount Diablo Thrust at about 29 km, the Calaveras fault at about 34 km, and the Northern San Andreas fault at about 28 km. A seismic event on any of these faults could cause significant ground shaking at the site. Plate D-1 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.

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 D-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 D-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 D-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 D-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 D-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>Hayward-Rodgers Creek</td>
<td>0</td>
<td>150</td>
<td>7.33</td>
<td>9.0</td>
</tr>
<tr>
<td>West Napa</td>
<td>23</td>
<td>30</td>
<td>6.70</td>
<td>1.0</td>
</tr>
<tr>
<td>Green Valley Connected</td>
<td>25</td>
<td>56</td>
<td>6.80</td>
<td>4.7</td>
</tr>
<tr>
<td>Northern San Andreas</td>
<td>28</td>
<td>473</td>
<td>8.05</td>
<td>17-24</td>
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<tr>
<td>Mount Diablo Thrust</td>
<td>29</td>
<td>25</td>
<td>6.70</td>
<td>2.0</td>
</tr>
<tr>
<td>San Gregorio - Connected</td>
<td>33</td>
<td>176</td>
<td>7.50</td>
<td>5.5</td>
</tr>
<tr>
<td>Calaveras</td>
<td>34</td>
<td>123</td>
<td>7.03</td>
<td>6-15</td>
</tr>
<tr>
<td>Great Valley 4b, Gordon Valley</td>
<td>39</td>
<td>28</td>
<td>6.80</td>
<td>1.3</td>
</tr>
<tr>
<td>Point Reyes</td>
<td>42</td>
<td>47</td>
<td>6.90</td>
<td>0.4</td>
</tr>
<tr>
<td>Great Valley 5, Pittsburg Kirby Hills</td>
<td>44</td>
<td>32</td>
<td>6.70</td>
<td>1.0</td>
</tr>
<tr>
<td>Greenville Connected</td>
<td>46</td>
<td>51</td>
<td>7.00</td>
<td>2.0</td>
</tr>
<tr>
<td>Hunting Creek-Berryessa</td>
<td>55</td>
<td>60</td>
<td>7.10</td>
<td>6.0</td>
</tr>
<tr>
<td>Monte Vista-Shannon</td>
<td>60</td>
<td>45</td>
<td>6.50</td>
<td>0.4</td>
</tr>
<tr>
<td>Great Valley 4a, Trout Creek</td>
<td>61</td>
<td>19</td>
<td>6.60</td>
<td>1.3</td>
</tr>
<tr>
<td>Great Valley 7</td>
<td>74</td>
<td>45</td>
<td>6.90</td>
<td>1.5</td>
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<td>Maacama-Garberville</td>
<td>74</td>
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<td>7.40</td>
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<td>Great Valley 3, Mysterious Ridge</td>
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<tr>
<td>Collayomi</td>
<td>95</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

**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).

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 1989 Loma Prieta earthquake on the San Andreas fault, several California Strong Motion Instrumentation Program (CSMIP) stations in the area recorded free-field horizontal peak ground accelerations ranging from 0.1 to 0.3 g (Thiel Jr., et al., 1990). During the South Napa earthquake, CSMIP stations in the area recorded free-field horizontal peak ground accelerations of less than 0.1g.

Epicenters of significant earthquakes (M>4.0) within the vicinity of the site are shown on Figure D-1.

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).

SEISMIC HAZARD ANALYSIS

Based on the results of the field explorations for this project and the adjacent Campus Safety Center project and using appropriate correlations between penetration resistance and Vs and/or undrained shear strength and Vs, the site is estimated to have an 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 ASCE 7-10, the MCE$_R$ is defined as the most severe earthquake effects determined for the orientation that results in the largest maximum response to horizontal ground motions and with adjustment for targeted risk as defined by ASCE 7-10. In addition, according to ASCE 7-10, the MCE$_R$ is defined as the lesser of: (1) 2 percent probability of being exceeded in 50 years
Both probabilistic and deterministic seismic hazard analyses were used to estimate the spectral accelerations for the MCE\(R\). 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.

**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 (\(V_{S30}\)) of the soil profile in the analysis. Based on the results of our field investigation, a \(V_{S30}\) of 250 m/s was used in the analyses. Some of these GMPEs also require inputs for depth in meters to a layer with \(V_s\) value of 1,000 m/s (\(Z_{1.0}\)) and depth in km to the layer with \(V_s\) value of 2,500 m/s (\(Z_{2.5}\)) 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.

**Probabilistic Seismic Hazard Analysis**

A probabilistic seismic hazards 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

\[
\text{PE}(Z) = 1 - e^{-\theta(Z)T}
\]
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:

\[
\vartheta(Sa > z) = \sum_{i=1}^{N_{source}} N_i(M_{\text{min}}) \int_{r=0}^{M_{\text{max}}} \int f_{m,i}(M) f_{r,i}(r) P(Sa > z | M, r) dr dM
\]

Where \( \vartheta(Sa > z) \) is the mean annual rate of a spectral acceleration \( (Sa) \) exceeding a test value \( (z) \); \( N_{source} \) 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.

**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 Hayward-Rodgers Creek fault (M7.33), which is the controlling fault for this site. Since the site is located within an A-P zone, we used a distance of 0 km in our analysis.
DETERMINATION OF SITE-SPECIFIC HORIZONTAL MCE and de 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://geohazards.usgs.gov/designmaps/us/application.php). These values are summarized in Table D-2.

### TABLE D-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>2.466g</td>
<td>Fig 22-1</td>
</tr>
<tr>
<td>$S_1$</td>
<td>1.025g</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>$C_{RS}$</td>
<td>0.988</td>
<td>Fig 22-3</td>
</tr>
<tr>
<td>$C_{R1}$</td>
<td>0.969</td>
<td>Fig 22-4</td>
</tr>
<tr>
<td>$S_{MS}$</td>
<td>2.466g</td>
<td>Eq. 11.4-1</td>
</tr>
<tr>
<td>$S_{M1}$</td>
<td>1.538g</td>
<td>Eq. 11.4-2</td>
</tr>
<tr>
<td>$S_{DS}$</td>
<td>1.644g</td>
<td>Eq. 11.4-3</td>
</tr>
<tr>
<td>$S_{D1}$</td>
<td>1.025g</td>
<td>Eq. 11.4-4</td>
</tr>
<tr>
<td>PGA</td>
<td>0.955</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>0.955</td>
<td>Eq. 11-8-1</td>
</tr>
</tbody>
</table>

As discussed earlier, the MCE 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 $C_{RS}$ and $C_{R1}$. $C_{RS}$ and $C_{R1}$ were estimated from Figures 22-3 and 22-4 of ASCE 7-10 and they are 0.988 and 0.969, respectively. $C_{RS}$ is applied on periods of 0.2s or less and $C_{R1}$ is applied on periods of 1.0s or greater and linear interpolation in between.

Site-specific deterministic (84th percentile) spectrum for the Hayward-Rodgers Creek fault is compared with the DLL spectrum per Figure 21.2-1 of ASCE 7-10 on Figure D-2. Spectral values are also compared in Table D-3 for some specific periods. Figure D-2 and Table D-3 show that for all practical purposes the controlling deterministic values are governed by the 84th percentile site-specific deterministic spectrum for entire range of periods of up to 5.0 seconds. Therefore, the deterministic values are controlled by the site-specific deterministic spectrum.
TABLE D-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>0.867</td>
<td>0.600</td>
<td>1.105</td>
<td>0.578</td>
<td>0.526</td>
</tr>
<tr>
<td>0.2</td>
<td>1.673</td>
<td>1.500</td>
<td>2.314</td>
<td>1.116</td>
<td>1.315</td>
</tr>
<tr>
<td>0.3</td>
<td>1.893</td>
<td>1.500</td>
<td>2.463</td>
<td>1.262</td>
<td>1.315</td>
</tr>
<tr>
<td>0.5</td>
<td>2.081</td>
<td>1.500</td>
<td>2.523</td>
<td>1.387</td>
<td>1.315</td>
</tr>
<tr>
<td>1.0</td>
<td>1.874</td>
<td>0.900</td>
<td>2.036</td>
<td>1.249</td>
<td>0.820</td>
</tr>
<tr>
<td>2.0</td>
<td>1.210</td>
<td>0.450</td>
<td>1.251</td>
<td>0.807</td>
<td>0.410</td>
</tr>
</tbody>
</table>

Site-specific probabilistic spectrum is compared with the controlling deterministic spectrum on Figure D-3. Spectral values are also compared in Table D-3 for some specific periods. Figure D-3 and Table D-3 show that the probabilistic values are greater than the controlling deterministic for periods of up to 2.0 seconds and then deterministic values are greater beyond that. Therefore, site-specific MCE\textsubscript{R} spectrum is developed by enveloping the controlling deterministic and probabilistic spectra. The DE spectrum was developed by taking two-thirds of the MCE\textsubscript{R} spectrum.

Comparison of the DE spectrum with the 80% of the code spectrum is shown on Figure D-4. Spectral values are also compared in Table D-3 for some specific periods. Figure D-4 and Table D-3 show that the DE spectrum is higher than the 80% of the code spectrum for all periods except the periods between 0.02 and 0.3 seconds where the 80% of the code spectrum is greater. Therefore, the recommended site-specific horizontal DE spectrum is developed by enveloping two-thirds of the MCE\textsubscript{R} spectrum and 80% of the code spectrum. Site-specific MCE\textsubscript{R} spectrum is taken as 1.5 times the DE spectrum. Figure D-5 shows the site-specific 5% damped DE and MCE\textsubscript{R} spectra. Site-specific horizontal spectral acceleration values in terms of g for the DE and MCE\textsubscript{R} are presented in Table D-4.

TABLE D-4: SITE-SPECIFIC HORIZONTAL MCE\textsubscript{R} AND DE SPECTRAL ACCELERATIONS (g)

<table>
<thead>
<tr>
<th>Period (sec)</th>
<th>DE 5% Damping</th>
<th>MCE\textsubscript{R} 5% Damping</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.01</td>
<td>0.578</td>
<td>0.867</td>
</tr>
<tr>
<td>0.125</td>
<td>1.315</td>
<td>1.973</td>
</tr>
<tr>
<td>0.2</td>
<td>1.315</td>
<td>1.973</td>
</tr>
<tr>
<td>0.25</td>
<td>1.315</td>
<td>1.973</td>
</tr>
<tr>
<td>0.3</td>
<td>1.315</td>
<td>1.973</td>
</tr>
<tr>
<td>0.4</td>
<td>1.340</td>
<td>2.010</td>
</tr>
<tr>
<td>0.5</td>
<td>1.387</td>
<td>2.081</td>
</tr>
<tr>
<td>0.75</td>
<td>1.369</td>
<td>2.054</td>
</tr>
<tr>
<td>1</td>
<td>1.249</td>
<td>1.874</td>
</tr>
<tr>
<td>1.5</td>
<td>1.008</td>
<td>1.512</td>
</tr>
<tr>
<td>2</td>
<td>0.807</td>
<td>1.210</td>
</tr>
<tr>
<td>2.5</td>
<td>0.646</td>
<td>0.970</td>
</tr>
<tr>
<td>3</td>
<td>0.529</td>
<td>0.794</td>
</tr>
<tr>
<td>4</td>
<td>0.377</td>
<td>0.565</td>
</tr>
<tr>
<td>5</td>
<td>0.302</td>
<td>0.453</td>
</tr>
</tbody>
</table>
SITE-SPECIFIC DESIGN ACCELERATION PARAMETERS

Site specific ground motion parameters for $S_{DS}$ and $S_{D1}$ were estimated using the site-specific design response spectrum presented in Table D-4. According to Section 21.4 of ASCE 7-10, the $S_{DS}$ 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 $S_{DS}$ value is governed by the spectral acceleration at 0.2 seconds as shown in Table D-4. Additionally, the $S_{D1}$ 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 $S_{D1}$ value as shown in Table D-4. The parameters $S_{MS}$ and $S_{M1}$ shall be taken as 1.5 times $S_{DS}$ and $S_{D1}$. Site-specific $S_{DS}$, $S_{D1}$, $S_{MS}$, $S_{M1}$ values are presented in Table D-5.

**TABLE D-5: SITE-SPECIFIC DESIGN ACCELERATION PARAMETERS**

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value (5% Damping)</th>
</tr>
</thead>
<tbody>
<tr>
<td>$S_{DS}$</td>
<td>1.315 g</td>
</tr>
<tr>
<td>$S_{D1}$</td>
<td>1.614 g</td>
</tr>
<tr>
<td>$S_{MS}$</td>
<td>1.973 g</td>
</tr>
<tr>
<td>$S_{M1}$</td>
<td>2.420 g</td>
</tr>
</tbody>
</table>

It should be noted that $S_{D1}$ and $S_{M1}$ values are greater than $S_{DS}$ and $S_{MS}$ values, respectively.

Site specific peak ground acceleration ($PGA_M$) for MCE$_S$ 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 0.789g and is controlled by the deterministic results. Therefore, the associated earthquake magnitude is 7.3.

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


COMPARISON OF DETERMINISTIC SPECTRA—5% DAMPING

Site-Specific Deterministic

DLL

5% Damping
5% Damping

- Probabilistic
- Controlling Deterministic

Spectral Acceleration (g)

Period (sec)
COMPARISON OF DE SPECTRUM WITH THE CODE SPECTRUM

5% Damping

Period (sec)

Spectral Acceleration [g]

DE
80% of Code DE
5% Damping

Plot of Spectral Acceleration (g) vs. Period (sec) for DE and MCE-R.

- DE
- MCE-R

Site-Specific Response Spectrum—MCE_R and DE

SITE-SPECIFIC RESPONSE SPECTRUM—MCE_R AND DE

C-608 PE/KINESIOLOGY RENOVATION PROJECT
CONTRA COSTA COMMUNITY COLLEGE
SAN PABLO, CALIFORNIA
APPENDIX E
GBA IMPORTANT INFO ABOUT YOUR GEOTECHNICAL REPORT
While you cannot eliminate all such risks, you can manage them. The following information is provided to help.

The Geoprofessional Business Association (GBA) has prepared this advisory to help you – assumedly a client representative – interpret and apply this geotechnical-engineering report as effectively as possible. In that way, clients can benefit from a lowered exposure to the subsurface problems that, for decades, have been a principal cause of construction delays, cost overruns, claims, and disputes. If you have questions or want more information about any of the issues discussed below, contact your GBA-member geotechnical engineer. Active involvement in the Geoprofessional Business Association exposes geotechnical engineers to a wide array of risk-confrontation techniques that can be of genuine benefit for everyone involved with a construction project.

**Geotechnical-Engineering Services Are Performed for Specific Purposes, Persons, and Projects**

Geotechnical engineers structure their services to meet the specific needs of their clients. A geotechnical-engineering study conducted for a given civil engineer will not likely meet the needs of a civil-works constructor or even a different civil engineer. Because each geotechnical-engineering study is unique, each geotechnical-engineering report is unique, prepared solely for the client. Those who rely on a geotechnical-engineering report prepared for a different client can be seriously misled. No one except authorized client representatives should rely on this geotechnical-engineering report without first conferring with the geotechnical engineer who prepared it. And no one – not even you – should apply this report for any purpose or project except the one originally contemplated.

**Read this Report in Full**

Costly problems have occurred because those relying on a geotechnical-engineering report did not read it in its entirety. Do not rely on an executive summary. Do not read selected elements only. Read this report in full.

**You Need to Inform Your Geotechnical Engineer about Change**

Your geotechnical engineer considered unique, project-specific factors when designing the study behind this report and developing the confirmation-dependent recommendations the report conveys. A few typical factors include:
- the client's goals, objectives, budget, schedule, and risk-management preferences;
- the general nature of the structure involved, its size, configuration, and performance criteria;
- the structure's location and orientation on the site; and
- other planned or existing site improvements, such as retaining walls, access roads, parking lots, and underground utilities.

Typical changes that could erode the reliability of this report include those that affect:
- the site's size or shape;
- the function of the proposed structure, as when it's changed from a parking garage to an office building, or from a light-industrial plant to a refrigerated warehouse;
- the elevation, configuration, location, orientation, or weight of the proposed structure;
- the composition of the design team; or
- project ownership.

As a general rule, always inform your geotechnical engineer of project changes – even minor ones – and request an assessment of their impact. The geotechnical engineer who prepared this report cannot accept responsibility or liability for problems that arise because the geotechnical engineer was not informed about developments the engineer otherwise would have considered.

**This Report May Not Be Reliable**

Do not rely on this report if your geotechnical engineer prepared it:
- for a different client;
- for a different project;
- for a different site (that may or may not include all or a portion of the original site); or
- before important events occurred at the site or adjacent to it; e.g., man-made events like construction or environmental remediation, or natural events like floods, droughts, earthquakes, or groundwater fluctuations.

Note, too, that it could be unwise to rely on a geotechnical-engineering report whose reliability may have been affected by the passage of time, because of factors like changed subsurface conditions; new or modified codes, standards, or regulations; or new techniques or tools. If your geotechnical engineer has not indicated an “apply-by” date on the report, ask what it should be, and, in general, if you are the least bit uncertain about the continued reliability of this report, contact your geotechnical engineer before applying it. A minor amount of additional testing or analysis – if any is required at all – could prevent major problems.

**Most of the “Findings” Related in This Report Are Professional Opinions**

Before construction begins, geotechnical engineers explore a site's subsurface through various sampling and testing procedures. Geotechnical engineers can observe actual subsurface conditions only at those specific locations where sampling and testing were performed. The data derived from that sampling and testing were reviewed by your geotechnical engineer, who then applied professional judgment to form opinions about subsurface conditions throughout the site. Actual sitewide-subsurface conditions may differ – maybe significantly – from those indicated in this report. Confront that risk by retaining your geotechnical engineer to serve on the design team from project start to project finish, so the individual can provide informed guidance quickly, whenever needed.
drawings and specifications. Remind constructors that they may learn about specific project requirements, times, locations, and depths/elevations referenced. Be certain that the report, but they may rely on the factual data relative to the specific purposes only. You've included the material for informational or appendices, with your contract documents,

Give Constructors a Complete Report and Guidance

Some owners and design professionals mistakenly believe they can shift unanticipated-subsurface-conditions liability to constructors by limiting the information they provide for bid preparation. To help prevent the costly, contentious problems this practice has caused, include the complete geotechnical-engineering report, along with any attachments or appendices, with your contract documents, but be certain to note conspicuously that you've included the material for informational purposes only. To avoid misunderstanding, you may also want to note that “informational purposes” means constructors have no right to rely on the interpretations, opinions, conclusions, or recommendations in the report, but they may rely on the factual data relative to the specific times, locations, and depths/elevations referenced. Be certain that constructors know they may learn about specific project requirements, including options selected from the report, only from the design drawings and specifications. Remind constructors that they may perform their own studies if they want to, and be sure to allow enough time to permit them to do so. Only then might you be in a position to give constructors the information available to you, while requiring them to at least share some of the financial responsibilities stemming from unanticipated conditions. Conducting prebid and preconstruction conferences can also be valuable in this respect.

Read Responsibility Provisions Closely

Some client representatives, design professionals, and constructors do not realize that geotechnical engineering is far less exact than other engineering disciplines. That lack of understanding has nurtured unrealistic expectations that have resulted in disappointments, delays, cost overruns, claims, and disputes. To confront that risk, geotechnical engineers commonly include explanatory provisions in their reports. Sometimes labeled “limitations,” many of these provisions indicate where geotechnical engineers’ responsibilities begin and end, to help others recognize their own responsibilities and risks. Read these provisions closely. Ask questions. Your geotechnical engineer should respond fully and frankly.

Geoenvironmental Concerns Are Not Covered

The personnel, equipment, and techniques used to perform an environmental study – e.g., a “phase-one” or “phase-two” environmental site assessment – differ significantly from those used to perform a geotechnical-engineering study. For that reason, a geotechnical-engineering report does not usually relate any environmental findings, conclusions, or recommendations; e.g., about the likelihood of encountering underground storage tanks or regulated contaminants. Unanticipated subsurface environmental problems have led to project failures. If you have not yet obtained your own environmental information, ask your geotechnical consultant for risk-management guidance. As a general rule, do not rely on an environmental report prepared for a different client, site, or project, or that is more than six months old.

Obtain Professional Assistance to Deal with Moisture Infiltration and Mold

While your geotechnical engineer may have addressed groundwater, water infiltration, or similar issues in this report, none of the engineer’s services were designed, conducted, or intended to prevent uncontrolled migration of moisture – including water vapor – from the soil through building slabs and walls and into the building interior, where it can cause mold growth and material-performance deficiencies. Accordingly, proper implementation of the geotechnical engineer’s recommendations will not of itself be sufficient to prevent moisture infiltration. Confront the risk of moisture infiltration by including building-envelope or mold specialists on the design team. Geotechnical engineers are not building-envelope or mold specialists.